Gratitude

In appreciation and gratitude

to The Custodian of the Two Holy Mosques

King Abdullah Bin Abdul Aziz Al Saud

And

H.R.H. Prince Sultan Bin Abdul Aziz Al Saud

Crown Prince, Deputy Premier, Minister of Defence

& Aviation and Inspector General

For their continuous support and gracious consideration,

the Saudi Building Code National Committee (SBCNC)

is honored to present the first issue of

the Saudi Building Code (SBC).

Saudi Building Code Requirements

201	Architectural	
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PREFACE

The Saudi Building Code (SBC) is a set of legal, administrative and technical regulations and requirements that specify the minimum standards of construction for building in order to ensure public safety and health. A Royal Decree dated 11th June 2000 order the formation of a national committee composed of representatives of Saudi universities and governmental and private sectors. In September 2001, the Council of Ministers approved the general plan of the National Committee to develop a national building code for the Kingdom of Saudi Arabia.

To choose a base code for the Saudi Building Code, a number of Codes have been studied. The National Committee has been acquainted with the results of the national researches and the international codes from the U.S.A., Canada and Australia, also, the European Code, and Arab Codes. It has also sought the opinions of specialists in relevant Saudi universities, governmental and private sectors through holding a questionnaire, a symposium and specialized workshops, in the light of which, (ICC) has been chosen to be a base code for the Saudi Building Code.

The International Code Council (ICC) grants permission to the Saudi Building Code National Committee (SBCNC) to include all or any portion of material from the ICC codes, and standards in the SBC and ICC is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Toward expanding the participation of all the specialists in the building and construction industry in the Kingdom through the governmental and private sectors, the universities and research centers, the National Committee took its own decisions related to code content by holding specialized meetings, symposiums and workshops and by the help of experts from inside and outside of Saudi Arabia.

The technical committees and sub-committees started their work in April 2003 to develop the Saudi Building Code that adapts the base code with the social and cultural environment, the natural and climatic conditions, types of soil and properties of materials in the Kingdom

The Saudi Building Code Structural Requirements for Soil and Foundations (SBC 303) were developed based on ICC code in addition to American Concrete Institute (ACI) materials. ACI grants permission to the SBCNC to include ACI materials in the SBC, and ACI is not responsible for any modifications or changes that SBCNC has made to accommodate local conditions.

Throughout the development of the document, several key aspects were considered; among them are the current local practice of geotechnical engineering and the causes related to soil and foundations problems.

The development process of SBC 303 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were made in the IBC and the most important ones were that some sections have been extended to become entire new chapters in the SBC 303, as for the case of retaining walls, design for expansive soil, and design for vibratory loads. Particularly, design for expansive soil has been thoroughly enhanced with the additions of foundation systems that are common in local construction practice, and by emphasizing pre- and post-construction detailing which are usually overlooked and lead to many sequential problems. Sabkha and collapsible soils were not covered in the IBC document, yet these two soil formations are abundantly found on vast areas throughout the Kingdom and historically have created problems for structures. Thus, besides provisions relevant to identification and testing of these soil formations, which have been added to Chapter 2 “Site Investigations”, an entire chapter has been devoted for each soil type, covering all aspects relevant to design and construction of foundations systems on such problematic soil formations.

Although the provisions presume the existence of certain standard conditions, more often than not, every project has a unique combination of variables, and for that reason, all attempts have been made to make these requirements flexible.

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CHAPTER 1
GENERAL

SECTION 1.1
SCOPE

1.1.0 The Saudi Building Code for Soils and foundations referred to as SBC 303, provides minimum requirements for footing and foundation systems in those areas not subject to scour or water pressure by wind and wave action. Buildings and foundations subject to such scour or water pressure loads shall be designed in accordance with SBC 301. This requirement shall govern in all matters pertaining to design, construction, and material properties wherever this requirement is in conflict with requirements contained in other standards referenced in this requirement.

SECTION 1.2
DESIGN

1.2.0 Allowable bearing pressures, allowable stresses and design formulas provided in this code shall be used with the allowable stress design load combinations specified in Section 2.4 SBC 301. The quality and design of materials used structurally in excavations, footings and foundations shall conform to the requirements specified in SBC 301, SBC 304, SBC 305, and SBC 306 of the Saudi Building Code. Excavations and fills shall also comply with SBC 201.

1.2.1 **Foundation design for seismic overturning.** Where the foundation is proportioned using the strength design load combinations of Section 2.3.2 SBC 301, the seismic overturning moment need not exceed 75 percent of the value computed from Section 10.9.6 SBC 301 for the equivalent lateral force method, or Sections 10.10 and 10.14 SBC 301 for the modal analysis method.

SECTION 1.3
DEFINITIONS

1.3.0 The following words and terms shall, for the purposes of this code, have the meanings shown herein.

Acceptance Level. Acceptance level is the vibration level (displacement, velocity, or acceleration) at which a machine can run indefinitely without inducing vibration related maintenance.

Active Zone. Active zone is the upper zone of the soil deposit which is affected by the seasonal moisture content variations.

Alarm Level. Alarm level is the vibration level at which a machine is considered to have developed a defect that will result in related downtime. This level is usually higher than the acceptance level to allow for conservatism and machinery variance and is recommended as 1.5 times the acceptance level but may be varied, depending on specific experience or operational requirements.

Allowable Foundation Pressure. Allowable foundation pressure is a vertical pressure exerted by a foundation on a supporting formation which can be safely tolerated without causing detrimental settlement or shear failure.

Allowable Lateral Pressure. Allowable lateral pressure is a lateral pressure exerted due to a foundation or earth pressure, which can be safely tolerated without causing neither shear failure nor detrimental lateral movement.

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

Backfill. Backfill is earth filling a trench or an excavation under or around a building.

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 and to resist upward heave in expansive soils.

Building Official. Building official means the officer or other designated authority charged with the administration and enforcement of this code, or his duly authorized representatives.

Borehole. Borehole is a hole made by boring into the ground to study stratification, to obtain natural resources, or to release underground pressures.

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.

Cantilever Reinforced Concrete Wall. A cantilever T-type reinforced concrete wall consists of a concrete stem and base slab which form an inverted T.

Cantilever or Strap Footing. Cantilever or strap footing is a setup of a concrete beam placed on two adjacent footings which supports concentrated loads exerted at or close to the edge of the beam. The strap footing is used to connect an eccentrically loaded column footing to an interior column such that the transmitted moment caused from eccentricity to the interior column footing so that a uniform soil pressure is computed beneath both footings.

Cavity. Cavity is an underground opening with widely varying sizes caused mainly by solution of rock materials by water.

Collapse Index. Collapse index is the percentage of vertical relative magnitude of soil collapse determined at 200 kPa as per ASTM D 5333.

Collapse Potential. Collapse potential is the percentage of vertical relative magnitude of soil collapse determined at any stress level as per ASTM D 5333.

Collapsible Soils. Collapsible soils are deposits that are characterized by sudden and large volume decrease at constant stress when inundated with water. These deposits are comprised primarily of silt or fine sand-sized particles with small amounts of clay, and may contain gravel. Collapsible soils have low density, but are relatively stiff and strong in their dry state.

Column. Column is a member with a ratio of height-to-least-lateral dimension exceeding three, used primarily to support axial compressive load.

Combined Footing. Combined footing is a structural unit or assembly of units supporting more than one column load.

Compaction. Compaction is increasing the dry density of soils by means such as impact or by rolling the surface layers.

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.

Contact Pressure. Contact pressure or soil pressure is the pressure acting at and perpendicular to the contact area between footing and soil, produced by the weight of the footing and all forces acting on it.

Continuous or Strip Footing. Continuous or strip footing is a combined footing of prismatic or truncated shape, supporting two or more columns in a row. Continuous or strip footings may be of fixed thickness or upper face can be stepped or inclined with inclination or steepness not exceeding 1 unit vertical in 2 units horizontal.

Distortion Resistance. Distortion resistance corresponds to moment resistance to bending of beams, columns, footings and joints between them.

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.

Effective Depth of Section. Effective depth of section is the distance measured from the extreme compression fiber to centeroid of tension reinforcement.

Enlarged Based 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.

Erosion. Erosion is the wearing away of the ground surface as a result of the movement of wind and water.

Excavation. Excavation is the mechanical or manual removal of earth material.

Expansion Index. Expansion index is the percent volume change determined in accordance with ASTM-D4829 multiplied by fraction passing No. 4 sieve of the soil multiplied by 100.

Expansion Joints. Expansion joints are intentional plane of weakness between parts of a concrete structure designed to prevent the crushing and distortion, including displacement, buckling, warping of abutting concrete structural units that might otherwise be developed by expansion, applied loads, or differential movements arising from the configuration of the structure or its settlement.

Expansive Soil. Expansive soil is a soil or rock material that has a potential for shrinking or swelling under changing moisture conditions. These soils are known to exist in many locations in the Kingdom such as Al-Ghatt, Tabuk, Tyma, Al-Madinah Al-Munuwarah, Al-Hafouf, and Sharora.

Factor of Safety. Factor of safety is the ratio of ultimate bearing capacity to the allowable load-bearing.

Fill. Fill is a deposit of earth material placed by artificial means.

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.

Footing. Footing is that portion of the foundation of a structure which spreads and transmits loads directly to the soil.

Foundation. Foundation is the portion of a structure which transmits the building load to the ground.

Geotechnical Engineer. Geotechnical engineer is an engineer knowledgeable and experienced in soil and rock engineering.

Geotechnical Engineering. Geotechnical engineering is the application of the principles of soils and rock mechanics in the investigation, evaluation and design of civil works involving the use of earth materials and the inspection and/or testing of the construction thereof.

Grade. Grade is the vertical location of the ground surface.

Grade Beam. Grade beam is a continuous beam subject to flexure longitudinally, loaded by the line of columns it supports.

Gravity Concrete Wall. A gravity wall consists of mass concrete, generally without reinforcement. It is proportioned so that the resultant of the forces acting on any internal plane through the wall falls within, or close to, the kern of the section.

Grid Foundation. Grid foundation is a combined footing, formed by intersecting continuous footings, loaded at the intersection points and covering much of the total area within the outer limits of assembly.

Group R Occupancy. See **SBC 201**.

Group U Occupancy. See **SBC 201**.

Heavy Machinery. Heavy machinery is any machinery having rotating or reciprocating masses as the major moving parts (such as compressors, pumps, electric motors, diesel engines and turbines).

High-Tuned System. High-tuned system is a machine support/foundation system in which the operating frequency (range) of the machinery (train) is below all natural frequencies of the system.

Influence Zone. Influence zone is the zone under the foundation lying inside the vertical stress contours of value 0.1 of applied pressure.

Karst Formation. Karst formation is a type of topography that is formed on limestone, dolomite, marble, gypsum, anhydrite, halite or other soluble rocks. Its formation is the result of chemical solution of these rocks by percolating waters that commonly follow the pre-existing joint patterns and enlarge them to caverns. Sinkholes and solution cavities at or near the ground surface are characteristic features of karst, and pose a hazard in the Eastern and Central regions of Saudi Arabia. Collapse features are widespread in these regions and are commonly associated with carbonate and evaporite formations that have been subjected to karst development during Quaternary pluvial epochs.

Lateral Sliding Resistance. Lateral sliding resistance is the resistance of structural walls or foundations to lateral sliding, and it is controlled by interface friction and vertical loads.

Low-Tuned System. Low-tuned system is a machine support/foundation system in which the operating frequency (range) of the machinery (train) is above all natural frequencies of the system.

Machine Support/Foundation System. Machine support/foundation system is a system consisting of the machinery (train) including base plate and the foundation, support structure plus all piers, equipment and process piping supported on the foundation or machinery. The supporting soil, piling or structure shall be considered part of the machine foundation system.

Mat Area. Mat area is the contact area between mat foundation and supporting soil.

Mat Foundation. Mat foundation is a continuous footing supporting an array of columns in several rows in each direction, having a slab like shape with or without depressions or openings, covering an area of at least 75 % of the total area within the outer limits of the assembly.

Mixed System. A mixed system is a machine support/foundation system having one or more of its natural frequencies below and the rest above the operating frequency (range) of the machinery (train).

Modulus of Elasticity. Modulus of elasticity is the ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material.

Modulus of Subgrade Reaction. Modulus of subgrade reaction is the ratio between the vertical pressure against the footing or mat and the deflection at a point of the surface of contact.

Mortar. Mortar is a mixture of cementitious material and aggregate to which sufficient water and approved additives, if any, have been added to achieve a workable, plastic consistency.

Natural Frequency. Natural frequency is the frequency with which an elastic system vibrates under the action of forces inherent in the system and in the absence of any externally applied force.

Net Pressure. Net pressure is the pressure that can be applied to the soil in addition to the overburden due to the lowest adjacent grade.

Overburden. Overburden is the weight of soil or backfill from base of foundation to ground surface.

Overturning. Overturning is the horizontal resultant of any combination of forces acting on the structure tending to rotate as a whole about a horizontal axis.

Pier Foundations. Pier foundations consist of isolated 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 Foundations. Pile foundations consist of concrete 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, through end bearing, or a combination of both.

Pressed Edge. Pressed edge is the edge of footing or mat along which the greatest soil pressure occurs under the condition of overturning.

Rectangular Combined Footing. Rectangular combined footing is a combined footing used if the column which is eccentric with respect to a spread footing carries a smaller load than the interior columns.

Registered Design Professional. Registered design professional is an individual who is registered or licensed to practice the respective design profession as defined by the statutory requirements of the professional registration laws of the state or jurisdiction in which the project is to be constructed.

Reinforced Concrete. Reinforced concrete is structural concrete reinforced with no less than the minimum amounts of nonprestressed reinforcement specified in SBC 304.

Reinforcement. Reinforcement is material that conforms to SBC 304 Section 3.5, excluding prestressing steel unless specifically included.

Retaining Walls. Retaining walls are structures that laterally support and provide stability for soils or other materials, where existing conditions do not provide stability with neither natural nor artificial slope.

Rocks. Rocks are natural aggregate of minerals or mineraloids that are connected together by strong bondings or attractive forces and have some degree of chemical and mineralogical constancy.

Rock Quality Designation. Rock quality designation, RQD, is an index or measure of the quality of a rock mass, and is calculated as summation of length of intact pieces of core greater than100 mm in length divided by the whole length of core advance.

Sabkhas. Sabkhas are salt bearing arid climate sediments covering vast areas of the coasts of Saudi Arabia. These soils either border partially land-locked seas or cover a number of continental depressions. The development of this material is due to low wave energy allowing the settlement of silt and clay particles to take place and then be loosely cemented by soluble material. Varying quantities of calcium carbonate, magnesium carbonate, calcium sulphate and calcium, magnesium, and sodium chlorides are found. The sabkha sediments are highly variable in lateral and vertical extent; various soil types, primarily composed of clays, silts, fine sands, and organic matter are inter-layered at random. In general, sabkha sediments are characterized by high void ratios and low dry densities. Accordingly, upon wetting sabkha soil is renowned for being highly compressible material with low bearing resistance, and hence considered among the poorest of foundation materials. Sabkha terrains are known to exist in many locations in the Kingdom such as Jubail, Rastanura, Abqaiq, Dammam, and Shaibah along the Arabian Gulf coast. They are prevailed in Jeddah, Jizan, Qunfudah, Al-Lith, Rabigh, and Yanbu along the Western coast as well as in Wadi As-Sirhan, around Qasim, and around Riyadh.

Settlement. Settlement is the gradual downward movement of an engineering structure, due to compression of the soil below the foundation.

Shallow Foundations. Shallow foundations are foundations with their depths less or equal to their widths.

Shoring. Shoring is the process of strengthening the side of excavation during construction stage.

Slope. Slope is the inclined surface of any part of the earth's surface.

Soils. Soils are uncemented or weakly cemented accumulation of solid particles that have resulted from the disintegration of rocks.

Soil mechanics. Soil mechanics is the branch of geotechnical engineering that deals with the physical properties of soil and the behavior of soil masses subjected to various types of forces. It applies the basic principles of mechanics including kinematics, dynamics, fluid mechanics, and the mechanics of materials to soils.

Spiral reinforcement. Spiral reinforcement is **c**ontinuously wound reinforcement in the form of a cylindrical helix.

Spread Footing. Spread footing is a concrete pad supporting column load. It can take a rectangular, square or a circular shape and having a uniform or tapered thickness not less than 250 mm.

Spring Constant. Spring constant is the soil resistance in load per unit deflection obtained as the product of the contributory area and coefficient of vertical subgrade reaction.

Steady-State Dynamic Force. Steady-state dynamic force is any dynamic force which is periodic in nature and generated during normal operating conditions, such as centrifugal forces due to unbalances in rotating machinery or piston forces in reciprocating machinery.

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.

Support/Foundation. Support/foundation is the part of the machine support not supplied by the equipment manufacturer as part of the machinery (train). This may include but is not limited to piers, concrete mat or block, pilings, steel structures, anchor bolts and embedded foundation plates.

Surcharge. Surcharge is the load applied to ground surface above a foundation, retaining wall, or slope.

Swell Pressure. Swell pressure is the maximum applied stress required to maintain constant volume of an inundated sample in the oedometer.

Table Top. Table top is a reinforced concrete structure supporting elevated machinery.

Total Core Recovery. Total core recovery, TCR, is the total length of rock pieces recovered divided by the total length of core advance.

Transient Dynamic Force. Transient dynamic force is any dynamic force, which is short term in nature such as starting torques or short circuit moments in electrical machinery, hydraulic forces, resonance forces of low-tuned or mixed systems during start-up or shutdown.

Trapezoidal-Shaped Combined Footing. Trapezoidal-shaped combined footing is a combined footing used when the column which has too limited space for a spread footing carries the larger load.

Underpinning. Underpinning is the process of strengthening and stabilizing the foundation of an existing building or other structure. Underpinning may be necessary for a variety of reasons including, but not limited to, the original foundation is simply not be strong enough or stable enough, the use of the structure has changed, the properties of the soil supporting the foundation may have changed or was mischaracterized during planning, the construction of nearby structures necessitates the excavation of soil supporting existing foundation. Underpinning is accomplished by extending the foundation in depth or in breadth so it either rests on a stronger soil stratum or distributes its load across a greater area.

Wall Footing. Wall footing is strip footing supporting wall such that the centerlines of the footing and the wall coincide.

Water Table. Water table is the planar surface between the zone of saturation and the zone of aeration. Also known as free-water elevation; free water surface; groundwater level; groundwater surface, groundwater table; level of saturation; phreatic surface; plane of saturation; saturated surface; water level; and waterline.

Weep Holes. Weep holes are openings used in retaining walls to permit passage of water from the backfill to the front.

CHAPTER 2
SITE INVESTIGATIONS

SECTION 2.1
GENERAL

2.1.0 Site investigations shall be conducted in conformance with Sections 2.2 through 2.6. Where required by the building official, the classification and investigation of the soil shall be made by a registered design professional.

2.1.1 **Objectives**. Site investigation shall be planned and executed to determine the following:

1. Lateral distribution and thickness of the soil and rock strata within the zone of influence of the proposed construction.
2. Suitability of the site for the proposed work.
3. Proposal of best method for construction on the site.
4. Physical and engineering properties of the soil and rock formations.
5. Groundwater conditions with consideration of seasonal changes and the effects of extraction due to construction.
6. Hazardous conditions including unstable slopes, active or potentially active faults, regional seismicity, floodplains, ground subsidence, collapse, and heave potential.
7. Changes that may arise in the environment and the effects of these changes on the proposed and adjacent buildings.
8. Advice on the suitability of alternative location for the proposed building, if exists.
9. Thorough understanding of all subsurface conditions that may affect the proposed building.

SECTION 2.2
SCOPE

2.2.0 The owner or applicant shall submit a site investigation to the building official where required in Sections 2.2.1 through 2.2.6.

Exception:
The building official need not require a site investigation where satisfactory data from adjacent areas is available that demonstrates an investigation is not necessary for any of the conditions in Sections 2.2.1 through 2.2.5.

No site investigation report is needed if the building meets the following combined criteria:

1. The net applied load on the foundation is less than 50 kPa.
2. There are no dynamic or vibratory loads on the building.
3. Questionable or problematic soil is not suspected underneath the building.
4. Cavities are not suspected underneath the footing of the building.

2.2.1 **Questionable soil.** Where the safe-sustaining power of the soil is in doubt, or where a load-bearing value superior to that specified in this code is claimed, the building official shall require that the necessary investigation be made. Such investigation shall comply with the provisions of Sections 2.4 through 2.6.

2.2.2 **Problematic soils.** In areas likely to have expansive, collapsible, or sabkha soils, the building official shall require site investigation to determine where such soils do exist.

2.2.3 **Ground-water table.** A subsurface soil investigation shall be performed to determine whether the existing ground-water table is within the influence zone underneath the footing of the building.

2.2.4 **Rock strata.** Where subsurface explorations at the project site indicate variations or doubtful characteristics in the structure of the rock upon which foundations are to be constructed, a sufficient number of borings shall be made to a depth of not less than 3 m below the level of the foundations to provide assurance of the soundness of the foundation bed and its load-bearing capacity.

2.2.4.1 **Rock cavities.** In areas of karst formations, the building official shall require site investigation to determine the potential sizes and locations of cavities underneath the building. If cavities are encountered, such investigation shall recommend remedies and construction procedures.

2.2.5 **Seismic Design Category C.** Where a structure is determined to be in Seismic Design Category C in accordance with Chapters 9 through 16 of SBC 301, an investigation shall be conducted, and shall include an evaluation of the following potential hazards resulting from earthquake motions: slope instability, liquefaction and surface rupture due to faulting or lateral spreading.

2.2.6 **Seismic Design Category D.** Where the structure is determined to be in Seismic Design Category D, in accordance with Chapters 9 through 16 of SBC 301, the soils investigation requirements for Seismic Design Category C, given in Section 2.2.5, shall be met, in addition to the following. The investigation shall include:

1. A determination of lateral pressures on basement and retaining walls due to earthquake motions.

2. An assessment of potential consequences of any liquefaction and soil strength loss, including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and shall address mitigation measures. Such measures shall be given consideration in the design of the structure and can include, but are not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements or any combination of these measures. The potential for liquefaction and soil strength loss shall be evaluated for site peak ground acceleration magnitudes and source characteristics consistent with the design earthquake ground motions.

SECTION 2.3
SOIL CLASSIFICATION

2.3.0 Where required, soils shall be classified in accordance with Sections 2.3.1, 2.3.2, 2.3.3, or 2.3.4.

2.3.1 **General.** For the purposes of this section, the definition and classification of soil materials for use in Table 4.1 shall be in accordance with ASTM D 2487.

2.3.2 **Expansive soils.** Soils meeting all four of the following provisions shall be considered expansive. Compliance with Items 1, 2 and 3 shall not be required if the test prescribed in Item 4 is conducted:

1. Plasticity index of 15 or greater, determined in accordance with ASTM D 4318.
2. More than 10 percent of the soil particles pass a No. 200 sieve (75 micrometers), determined in accordance with ASTM D 422.
3. More than 10 percent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D 422.
4. Expansion index greater than 20, determined in accordance with ASTM D 4829.

2.3.3 **Collapsible soils.** Soils meeting all four of the following provisions shall be considered collapsible. Compliance with Items 1, 2 and 3 shall not be required if the test prescribed in Item 4 is conducted:

1. Desiccated Alluvial (Wadi) soils
2. Dry field density less than 17 kN/m^3 determined in accordance with ASTM D1556
3. Clay content 10 to 30 percent, determined in accordance with ASTM D422
4. Collapse index greater than 1 percent, determined in accordance with ASTM D5333.

2.3.4 **Sabkha soils.** Soils meeting the following shall be suspected as sabkha soils:

1. Very soft, with SPT values in the range of 0 to 8, determined in accordance with ASTM D1586.
2. Precipitated salts of different sizes, shape, and composition within the sediments.
3. High soluble salt content.
4. Soil exhibits significant variations in its chemical composition.
5. Soil exhibits high degree of variability of its sediments in both vertical and lateral extent within a considerably short distance.
6. Upon wetting soil becomes impassible.

SECTION 2.4
INVESTIGATION

2.4.0 Soil investigation shall be based on observation and any necessary tests of the materials disclosed by borings, test pits or other subsurface exploration made in appropriate locations. Additional studies shall be made as necessary to evaluate slope stability, soil strength, position and adequacy of load-bearing soils, the effect of moisture variation on soil-bearing capacity, compressibility, liquefaction, expansiveness, and collapsibility.

2.4.1 **Exploratory boring.** The scope of the site investigation including the number and types of borings or soundings, the equipment used to drill and sample, the in-situ testing equipment and the laboratory testing program shall be determined by a registered design professional. In areas likely to have problematic soils, field explorations shall include:

1. Investigations of soils between the ground surface and the bottom of the foundation, as well as materials beneath the proposed depth of foundation.
2. Evaluations and interpretations of the environmental conditions that would contribute to moisture changes and their probable effects on the behavior of such soils.

2.4.2 **Number of boreholes.** The minimum number of boreholes in a given site shall be taken in accordance with Table 2.1 and its provisions. The values included in Table 2.1 shall be considered as minimum guideline.

2.4.3 **Depth of boreholes.** The depth of boreholes shall cover all strata likely to be affected by the loads from the building and adjacent buildings. The minimum depth of boreholes shall be taken from Table 2.1.

SECTION 2.5
SOIL BORING AND SAMPLING

2.5.0 The soil boring and sampling procedure and apparatus shall be in accordance with generally accepted engineering practice. The registered design professional shall have a fully qualified representative on the site during all boring and sampling operations.

2.5.1 **Soil boring and sampling of expansive soils**. In areas likely to have expansive soils the following shall be taken into considerations:

1. Air drilling shall be used to maintain the natural moisture contents of the samples more effectively.
2. The use of lubricant that might react with the soil and change its properties shall be avoided.

3. The depth of sampling shall be at least as deep as the probable depth to which moisture changes will occur (active zone) but shall not be less than 1.5 times the minimum width of slab foundations to a maximum of 30 meters and a minimum of three base diameters beneath the base of shaft foundations.
4. Undisturbed samples shall be obtained at intervals of not greater than 1500 mm of depth. Sampling interval may be increased with depth.
5. A coating of wax shall be brushed on the sample before wrapping.
6. The outer perimeter of the sample shall be trimmed during the preparation of specimens for laboratory tests, leaving the more undisturbed inner core.
7. The sample shall be taken as soon as possible, after advancing the hole to the proper depth and cleaning out the hole, and personnel shall be well trained to expedite proper sampling, sealing, and storage in sample containers.

2.5.2 **Soil boring and sampling of collapsible soils**. In areas likely to have collapsible soils the following shall be taken into considerations:

1. Air drilling shall be used to maintain the natural moisture contents of the samples.
2. The depth of sampling shall be at least as deep as the probable depth to which moisture changes will occur but shall not be less than 2 times the minimum width of footing to a maximum of 30 meters and a minimum of three base diameters beneath the base of shaft foundations.
3. Undisturbed samples shall be obtained at intervals of not greater than 1500 mm of depth.
4. In the event undisturbed samples cannot be obtained from a borehole, test pits shall be excavated to sufficient depth and dry density of the soil shall be measured at various horizons in the pit.
5. Where possible, hand carved undisturbed samples taken in a vertical direction shall be obtained for odometer testing. Alternately, plate load test in unsoaked and soaked conditions shall be performed to determine the most critical collapse potential below foundation level.

2.5.3 **Soil boring and sampling of sabkha soils**. In areas likely to have sabkha soils the following shall be taken into considerations:

1. A full chemical analyses on soil and ground water to determine the average and range of the aggressive compounds and the variation in content with depth.
2. Grading of sabkha shall be determined by using wet sieving with non-polar solvent (sabkha brine, methylene chloride)
3. Basic properties including moisture content and specific gravity shall be determined by using oven drying at 60°C in accordance with ASTM D854 and ASTM D2216.

SECTION 2.6
REPORTS

2.6.0 The soil classification and design load-bearing capacity shall be shown on the construction document. Where required by the building official, a written report of the investigation shall be submitted that includes, but need not be limited to, the following information:

1. Introduction with location map depicting adjacent buildings, existing roads, and utility lines.
2. Climatic conditions such as rain rate, storm water discharge, etc. if relevant effect is suspected on the soil or rock formations.
3. Description of site topography and relevant geological information.
4. A plot showing the location of test borings and/or excavation pits.
5. A complete record of the soil samples.
6. A complete record of the borehole log with the standard penetration test, SPT, values at the corresponding depths for soil samples and RQD and TCR values for rock samples.
7. A record of the soil profile.
8. Elevation of the water table, if encountered and recommended procedures for dewatering, if necessary.
9. Brief description of conducted laboratory and field tests (or its SASO or ASTM standards, or equivalent standard number) and a summary of the results.
10. Recommendations for foundation type and design criteria, including but not limited to: bearing capacity of natural or compacted soil; provisions to mitigate the effects of problematic soils (expansive, collapsible, sabkha, etc.); mitigation of the effects of liquefaction, differential settlement and varying soil strength; and the effects of adjacent loads. The recommendations for foundation design must be based on the facts stated in the report, i.e. on the borehole records and test data. They must not be based on conjecture.
11. Expected total and differential settlements.
12. Pile and pier foundation information in accordance with Section 14.2.
13. Combined footings and mats information in accordance with Section 8.1.
14. Special design and construction provisions for footings or foundations founded on problematic soils in accordance with Chapters 9, 10, and 11, as necessary.
15. Compacted fill material properties and testing in accordance with Section 3.10.
16. Recommended sites for waste material disposal.
17. Suitability of excavated material for reuse as fill material in site.

TABLE 2.1
MINIMUM NUMBER AND MINIMUM DEPTHS OF BOREHOLES FOR BUILDINGS[a,b,c,d,e]

NO. OF STORIES	BUILT AREA (m^2)	NO. OF BOREHOLES	MINIMUM DEPTH[f] OF TWO THIRDS OF THE BOREHOLES (m)	MINIMUM DEPTH[f] OF ONE THIRD OF THE BOREHOLES (m)
2 or less	< 600	3	4	6
	600 – 5000	3 – 10[g]	5	8
	> 5000	Special investigation		
3 - 4	< 600	3	6 - 8	9 - 12
	600 – 5000	3 – 10[g]		
	> 5000	Special investigation		
5 or higher	Special investigation			

a. If possible, standard penetration tests, SPT, shall be performed in all sites.

b. If questionable soils do exist underneath the building, a minimum of one borehole shall penetrate all layers containing this soil.

c. Seasonal changes in groundwater table and the degree of saturation shall be considered.

d. If sufficient data is available, a registered design professional may use number and depth of boreholes that are different from the tabular values.

e. For foundation of pole and towers, a minimum of one boring with sufficient depth shall be located in the center of the foundation.

f. Depth is measured from level of foundation bottom.

g. Number of boreholes shall be selected by a registered design professional based on variations in site conditions, and contractor shall advice if additional or special tests are required.

CHAPTER 3
EXCAVATION, GRADING AND FILL

SECTION 3.1
GENERAL

3.1.0 Proper safety precautions shall be considered at all stages of excavation. Special care, measures, and techniques shall be followed for excavation below groundwater table.

The investigation and report provisions of Chapter 2 shall be expanded to include, but need not be limited to, the following:

1. Property limits and accurate contours of existing ground and details of terrain and area drainage.
2. Limiting dimensions, elevations or finish contours to be achieved by the grading, and proposed drainage channels and related construction.
3. Detail plans of all surface and subsurface drainage devices, walls, cribbing, and other protective devices to be constructed with, or as a part of, the proposed work.
4. Location of any buildings or structures on the property where the work is to be performed and the location of any buildings or structures on adjacent land which are within 5 m of the property or which may be affected by the proposed grading operations.
5. Conclusions and recommendations regarding the effect of geologic conditions on the proposed construction, and the adequacy of sites to be developed by the proposed grading.

SECTION 3.2
COMMENCEMENT

3.2.0 Excavation, grading and fill shall not be commenced without first having obtained a permit from the building official.

Exception: Permit shall not be required for the following:

1. Grading in an isolated, self-contained area if there is no apparent danger to private or public property.
2. Exploratory excavations under the direction of geotechnical engineers.
3. An excavation which (a) is less than 600 mm in depth, or (b) which does not create a cut slope greater than 1500 mm in height and steeper than three units horizontal to two units vertical.
4. A fill less than 300 mm in depth and placed on natural terrain with a slope flatter than five units horizontal to one unit vertical, or less than 1 m in depth, not intended to support structures, does not exceed 40 cubic meters on any one lot and does not obstruct a drainage course.

SECTION 3.3
EXCAVATIONS NEAR FOOTINGS OR FOUNDATIONS

3.3.0 Excavations for buildings shall be carried out as not to endanger life or property. Excavations for any purposes shall not remove lateral support from any footing or foundation without first underpinning or protecting the footing or foundation against settlement or lateral translation. Proper underpinning, sequence of construction, and method of shoring shall be approved by a registered design professional and carried out immediately after start of excavation. Underpinning system shall be periodically checked for safety assurance.

SECTION 3.4
SLOPE LIMITS

3.4.0 Slopes for permanent fill shall not be steeper than one unit vertical in two units horizontal (50-percent slope). Cut slopes for permanent excavations shall not be steeper than one unit vertical in two units horizontal (50-percent slope). Deviation from the foregoing limitations for cut slopes shall be permitted only upon the presentation of a soil investigation report acceptable to the building official and shows that a steeper slope will be stable and not create a hazard to public or private property.

SECTION 3.5
SURCHARGE

3.5.0 No fill or other surcharge loads shall be placed adjacent to any building or structure unless such building or structure is capable of withstanding the additional loads caused by the fill or surcharge. Existing footings or foundations, which can be affected by any excavation shall be underpinned adequately or otherwise protected against settlement and shall be protected against later movement.

SECTION 3.6
PLACEMENT OF BACKFILL

3.6.0 The excavation outside the foundation shall be backfilled with soil that is free of organic material, construction debris, cobbles and boulders or a controlled low-strength material (CLSM). The ground surface shall be prepared to receive fill by removing vegetation, noncomplying fill, topsoil and other unsuitable materials. The backfill shall be placed in lifts and compacted, in a manner that does not damage the foundation or the waterproofing or damp proofing material. Special inspections of compacted fill shall be in accordance with Section 2.7 SBC 302.

Exception: Controlled low-strength material need not be compacted.

SECTION 3.7
SITE GRADING

3.7.0 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 3 m measured perpendicular to the face of the wall or an approved alternate method of diverting water away from the foundation shall be used.

Exception: Where climate or soil conditions warrant, the slope of the ground away from the building foundation is permitted to be reduced to not less than one unit vertical in 50 units horizontal (2 percent slope).

The procedure used to establish the final ground level adjacent to the foundation shall account for additional settlement of the backfill.

SECTION 3.8
GRADING DESIGNATION

3.8.0 The faces of cut and fill slopes shall be prepared and maintained to control against erosion. All grading in excess of 3500 cubic meters shall be performed in accordance with the approved grading plan prepared by a registered design professional, and shall be designated as “engineering grading”. Grading involving less than 3500 cubic meters shall be designated as “regular grading” unless required by the building official to be considered as “engineering grading”.

For engineering grading, grading plan shall be prepared and approved by a registered design professional. For regular grading, the building official may require inspection and testing by an approved agency. Where the building official has cause to believe that geologic factors may be involved, the grading operation shall conform to “engineering grading” requirements.

SECTION 3.9
GRADING AND FILL IN FLOODWAYS

3.9.0 In floodways shown on the flood hazard map established in SBC 301 Section 5.3, grading and/or fill shall not be approved 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 any increase levels during the occurrence of the design flood.

SECTION 3.10
COMPACTED FILL MATERIAL

3.10.0 Where footings will bear on compacted fill material, the compacted fill shall comply with the provisions of an approved report, which shall contain, but need not be limited to, 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 method 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 method for determining the in-place dry density of the compacted fill.
6. Minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.

7. Number and frequency of field tests required to determine compliance with Item 6.

Exception: Compacted fill material less than 300 mm in depth need not comply with an approved report, provided it has been compacted to a minimum of 95 percent Modified Proctor in accordance with ASTM D 1557. The compaction shall be verified by a qualified inspector approved by the building official.

3.10.1 **Oversized materials**. No rock or similar irreducible material with a maximum dimension greater than 300 mm shall be buried or placed in fills within 1.5 m, measured vertically, from the bottom of the footing or lowest finished floor elevation, whichever is lower, within the building pad. Oversized fill material shall be placed so as to assure the filling of all voids with well-graded soil. Specific placement and inspection criteria shall be stated and continuous special inspections shall be carried out during the placement of any oversized fill material.

SECTION 3.11
CONTROLLED LOW-STRENGTH MATERIAL (CLSM)

3.11.0 Where footings will bear on controlled low-strength material (CLSM), the CLSM shall comply with the provisions of an approved report, which shall contain, but need not be limited to, the following:

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.

CHAPTER 4
ALLOWABLE LOAD-BEARING VALUES OF SOILS

SECTION 4.1
DESIGN

4.1.0 The presumptive load-bearing values provided in Table 4.1 shall be used with the allowable stress design load combinations specified in Section 2.4 of SBC 301.

SECTION 4.2
PRESUMPTIVE LOAD-BEARING VALUES

4.2.0 The maximum allowable foundation pressure, lateral pressure or lateral sliding resistance values for supporting soils at or near the surface shall not exceed the values specified in Table 4.1 unless data to substantiate the use of a higher value are submitted and approved by the building official. In case of thin soft layers existing between layers of high bearing values, the foundation shall be designed according to the bearing capacity of the thin soft layers.

Presumptive load-bearing values shall apply to materials with similar physical characteristics and depositional conditions.

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.

TABLE 4.1
ALLOWABLE FOUNDATION AND LATERAL PRESSURE

CLASS OF MATERIALS	ALLOWABLE FOUNDATION PRESSURE (kPa)[a]	LATERAL BEARING (kPa/m below natural grade)[a]	LATERAL SLIDING	
			Coefficient of friction[b]	Resistance (kPa)[c]
1. Crystalline bedrock	600	200	0.70	–
2. Sedimentary and foliated rock	200	60	0.35	–
3. Sandy gravel and/or gravel (GW and GP)	150	30	0.35	–
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	100	25	0.25	–
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	75[d]	15	–	6

a. An increase of one-third is permitted when using the alternate load combinations in SBC 301 Section 2.4 that include wind or earthquake loads.

b. Coefficient to be multiplied by the dead load.

c. Lateral sliding resistance value to be multiplied by the contact area, as limited by Section 4.3.

d. Where the building official determines that in-place soils with an allowable bearing capacity of less than 70 kPa are likely to be present at the site, the allowable bearing capacity shall be determined by a site investigation in accordance with Chapter 2.

SECTION 4.3
LATERAL SLIDING RESISTANCE

4.3.0 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 4.1 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.

4.3.1 **Increases in allowable lateral sliding resistance.** The resistance values derived from Table 4.1 are permitted to be increased by the tabular value for each additional 300 mm of depth to a maximum of 15 times the tabular value.

Isolated poles for uses such as flagpoles or signs and poles used to support buildings that are not adversely affected by a 13 mm motion at the ground surface due to short-term lateral loads are permitted to be designed using lateral-bearing values equal to two times the tabular values.

SECTION 4.4
COMPUTED LOAD-BEARING VALUES

4.4.0 It shall be permitted to obtain the ultimate bearing capacity from appropriate laboratory and/or field tests including, but need not be limited to, standard penetration test conforming to ASTM D1586 and plate load test conforming to ASTM D1194. Where the soil to a deep depth is homogeneous soil, the plate load test shall be conducted at the level of footing bottom. In case the soil medium is made of several layers, the test shall be conducted at each layer to a depth equal to not less than twice the width of footing measured from the bottom of footing. In case there is a large difference between the footing width and plate size, plates of different sizes shall be used to establish the relationship between width and load-bearing.

It shall be permitted to use formulae in the computations of ultimate bearing capacity that are of common use in geotechnical engineering practice or based on a sound engineering judgment and subject to approval to the building official.

4.4.1 **Effect of water table.** The submerged unit weight shall be used as appropriate to determine the actual influence of the groundwater on the bearing capacity of the soil. The foundation design shall consider the buoyant forces when groundwater is above or expected to rise above the foundation level.

CHAPTER 5
SPREAD FOOTINGS

SECTION 5.1
GENERAL

5.1.0 Spread footings shall be designed and constructed in accordance with Sections 5.1 through 5.6. Footings shall be built on undisturbed soil, compacted fill material or CLSM. Compacted fill material shall be placed in accordance with Section 3.10. CLSM shall be placed in accordance with Section 3.11.

The bottom surface of footings is permitted to have a slope not exceeding one unit vertical in 10 units horizontal (10 percent slope). Footings shall be stepped where it is necessary to change the elevation of the top surface of the footing or where the surface of the ground slopes more than one unit vertical in 10 units horizontal (10 percent slope).

SECTION 5.2
DEPTH OF FOOTINGS

5.2.0 The minimum depth of footing below the natural ground level shall not be less than 1.2 m for cohesionless soils, 1.5 m for silty and clay soils and 600 mm to 1200 mm for rocks depending on strength and integrity of the rock formations. Where applicable, the depth of footings shall also conform to Sections 5.2.1 through 5.2.3.

5.2.1 **Adjacent footings.** Footings on granular soil shall be so located that the line drawn between the lower edges of adjoining footings shall not have a slope steeper than 30 degrees (0.52 rad) with the horizontal, unless the material supporting the higher footing is braced or retained or otherwise laterally supported in an approved manner or a greater slope has been properly established by engineering analysis that is accepted by the building official.

5.2.2 **Shifting or moving soils.** Where it is known that the shallow subsoils are of a shifting or moving character, footings shall be carried to a sufficient depth to ensure stability.

5.2.3 **Stepped footings**. Footings for all buildings where the surface of the ground slopes more than one unit vertical in ten units horizontal (10 percent slope) shall be level or shall be stepped so that both top and bottom of such footing are level.

SECTION 5.3
FOOTINGS ON OR ADJACENT TO SLOPES

5.3.0 The placement of buildings and structures on or adjacent to slopes steeper than one unit vertical in three units horizontal shall conform to Sections 5.3.1 through 5.3.5.

5.3.1 **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 5.3.5 and Figure 5.1, 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 (0.79 rad) 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.

5.3.2 **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 5.3.5 and Figure 5.1, the following setback is deemed adequate to meet the criteria. Where the slope is steeper than one unit vertical in one unit horizontal (100 percent slope), the required setback shall be measured from imaginary plane 45 degrees (0.79 rad) to the horizontal, projected upward from the toe of the slope.

5.3.3 **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 2100 mm from the top of the slope shall be capable of supporting the water in the pool without soil support.

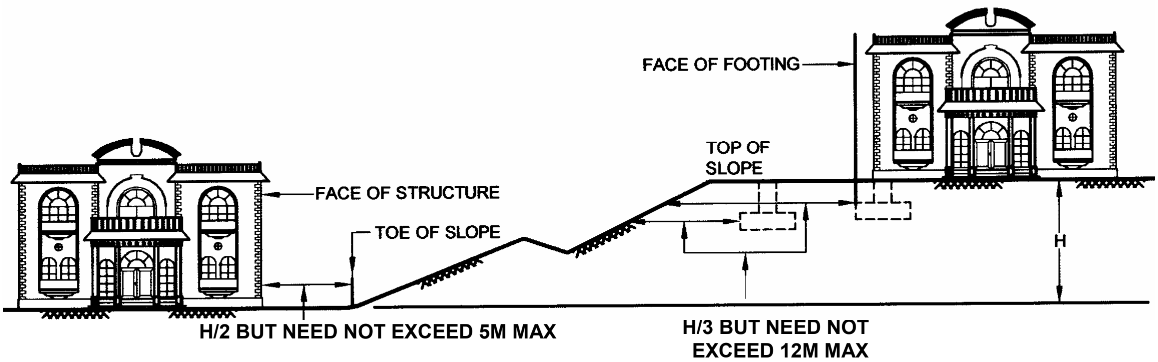


FIGURE 5.1
FOUNDATION CLEARANCES FROM SLOPES

5.3.4 **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 300 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.

5.3.5 **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, slope gradient, load intensity and erosion characteristics of slope material.

SECTION 5.4
DESIGN OF FOOTINGS

5.4.0 Spread footings shall be designed and constructed in accordance with Sections 5.4.1 through 5.4.4.

5.4.1 **General.** Footings shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that total and differential settlements are tolerable. The design of footings shall be under the direct supervision of a registered design professional who shall certify to the building official that the footing satisfies the design criteria. The minimum width of footings shall be 300 mm. Footings in areas with expansive soils shall be designed in accordance with the provisions of Chapter 9. Footings in areas with collapsible soils shall be designed in accordance with the provisions of Chapter 10. Footings in areas with sabkha soils shall be designed in accordance with the provisions of Chapter 11. Footings subject to vibratory loads shall be designed in accordance with the provisions of Chapter 12.

5.4.1.1 **Design loads.** Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in SBC 301 Section 2.4. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in SBC 301 Section 4.8, are permitted to be used in designing footings.

5.4.1.2 **Eccentric loads**. When the footings are subjected to moments or eccentric loads, the maximum stresses shall not exceed the allowable bearing capacity of the soil specified in Chapter 4. The centeroid of the loads exerted on the footings shall coincide with the centeroid of the footing area, and if not possible the eccentricity shall not exceed 1/6 times the dimension of the footing in both sides. For the purpose of estimating the ultimate load-bearing, use shall be made of the effective width taken as the actual width minus twice the eccentricity.

5.4.1.3 **Inclined loads.** For design of footings subjected to inclined loads, it shall be permitted to use the following simplified formula or any method of analysis, subject to the approval of the building official.

$$\frac{V}{P_v} + \frac{H}{P_h} < 1.0 \qquad \textbf{(Equation 5-1)}$$

where:

V = Vertical component of inclined load.

H = Horizontal component of inclined load.

P_v = Allowable vertical load.

P_h = Allowable horizontal load.

Horizontal component shall not exceed soil passive resistance along the footing vertical edge and friction resistance at the footing soil interface taking a factor of safety of 2.

5.4.1.4 **Adjacent loads**. Where footings are placed at varying elevations the effect of adjacent loads shall be included in the footing design.

5.4.1.5 **Design settlements.** Settlements shall be estimated by a registered design professional based on methods of analysis approved by the building official. The least value found from Tables 5.1 and 5.2 shall be taken as the allowable differential settlement.

Exceptions: Structures designed to stand excessive total settlement in coastal areas or heavily loaded structures, like silos and storage tanks, shall be allowed to exceed these limits subject to a recommendation of a registered design professional and approval of a building official.

TABLE 5.1
MAXIMUM ALLOWABLE TOTAL SETTLEMENT

FOOTING TYPE	TOTAL SETTLEMENT (mm)	
	CLAY	SAND
Spread Footings	60	40
Mat Foundations	80	60

TABLE 5.2
MAXIMUM ALLOWABLE ANGULAR DISTORTION[a]

BUILDING TYPE	L/H	δ/l
Multistory reinforced concrete structures founded on mat foundation	---	0.0015
Steel frame structure with side sway	---	0.008
Reinforced concrete or steel structure with interior or exterior glass or panel cladding	---	0.002-0.003
Reinforced concrete or steel structure with interior or exterior glass or panel cladding	$\geq$ 5 $\leq$ 3	0.002 0.001
Slip and high structures as silos and water tanks founded on stiff mat foundations	---	0.002
Cylindrical steel tank with fixed cover and founded on flexible footing	---	0.008
Cylindrical steel tank with portable cover and founded on flexible footing	---	0.002-0.003
Rail for supporting hanged lift	---	0.003

a. L = Building length
l = Span between adjacent footings
H = Overall height of the structure
δ = Differential settlement

5.4.1.6 **Factor of safety.** Factor of safety shall not be less than 3 for permanent structures and 2 for temporary structures. Consideration shall be given to all possible circumstances including, but not limited to, flooding of foundation soil, removal of existing overburden by scour or excavation, and change in groundwater table level.

5.4.2 **Concrete footings.** The design, materials and construction of concrete footings shall comply with Sections 5.4.2.1 through 5.4.2.8 and the provisions of SBC 304 where applicable.

Exception: Where a specific design is not provided, concrete footings supporting walls of light-frame construction are permitted to be designed in accordance with Table 5.3.

5.4.2.1 **Concrete strength.** Concrete in footings shall have a specified compressive strength ($f_c^{'}$) of not less than 20 MPa at 28 days.

5.4.2.2 **Footing seismic ties.** Where a structure is assigned to Seismic Design Category D in accordance with Chapters 9 through 16, SBC 301, individual spread footings founded on soil defined in Section 9.4.2, SBC 301 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 S_{DS} 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.

TABLE 5.3
FOOTINGS SUPPORTING WALLS OF LIGHT-FRAME CONSTRUCTION [a, b, c, d, e]

NUMBER OF FLOORS SUPPORTED BY THE FOOTING[f]	WIDTH OF FOOTING (mm)	THICKNESS OF FOOTING (mm)
1	300	150
2	375	150
3	450	200

a. Depth of footings shall be in accordance with Section 5.2.
b. The ground under the floor is permitted to be excavated to the elevation of the top of the footing.
c. Interior-stud-bearing walls are permitted to be supported by isolated footings. The footing width and length shall be twice the width shown in this table, and footings shall be spaced not more than 1800 mm on center.
d. See SBC 304 Chapter 21 for additional requirements for footings of structures assigned to Seismic Design Category C or D.
e. For thickness of foundation walls, see Chapter 6.
f. Footings are permitted to support a roof in addition to the stipulated number of floors. Footings supporting roof only shall be as required for supporting one floor.

5.4.2.3 **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.

5.4.2.4 **Protection of concrete.** Water shall not be allowed to flow through the deposited concrete.

5.4.2.5 **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 SBC 304.

5.4.2.6 **Minimum concrete cover to reinforcement**. When the concrete of footings is poured directly on the ground or against excavation walls the minimum concrete cover to reinforcement shall not be less than 75 mm. This cover shall also satisfy other requirements with regard to concrete exposure conditions presented in SBC 304.

5.4.2.7 **Dewatering.** Where footings are carried to depths below water level, the footings shall be constructed by a method that will provide the depositing or construction of sound concrete in the dry.

5.4.3 **Steel grillage footings.** Grillage footings of structural steel shapes shall be separated with approved steel spacers and be entirely encased in concrete with at least 150 mm on the bottom and at least 100 mm at all other points. The spaces between the shapes shall be completely filled with concrete or cement grout.

SECTION 5.5
DESIGNS EMPLOYING LATERAL BEARING

5.5.0 Designs to resist both axial and lateral loads employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth shall conform to the requirements of Sections 5.5.1 through 5.5.3.

5.5.1 **Limitations.** The design procedures outlined in this section are subject to the following limitations:

1. The frictional resistance for structural walls and slabs on silts and clays shall be limited to one-half of the normal force imposed on the soil by the weight of the footing or slab.
2. Posts embedded in earth shall not be used to provide lateral support for structural or nonstructural materials such as plaster, masonry or concrete unless bracing is provided that develops the limited deflection required.

5.5.2 **Design criteria.** The depth to resist lateral loads shall be determined by the design criteria established in Sections 5.5.2.1 through 5.5.2.3, or by other methods approved by the building official.

5.5.2.1 **Nonconstrained.** The following formula shall be used in determining the depth of embedment required to resist lateral loads where no constraint is provided at the ground surface, such as rigid floor or rigid ground surface pavement, and where no lateral constraint is provided above the ground surface, such as a structural diaphragm.

$$d = 0.5A\{1 + [1 + (4.36h/A)]^{1/2}\} \qquad \textbf{(Equation 5-2)}$$

where:

d = Depth of embedment in earth in meter but not over 3600 mm for purpose of computing lateral pressure.

h = Distance in meter from ground surface to point of application of "P".

A $= 2.34P/S_1\,b$

P = Applied lateral force in kN.

S_1 = Allowable lateral soil-bearing pressure as set forth in Section 4.3 based on a depth of one-third the depth of embedment in kPa.

b = Diameter of round post or footing or diagonal dimension of square post or footing, meter.

5.5.2.2 **Constrained.** The following formula shall be used to determine the depth of embedment required to resist lateral loads where constraint is provided at the ground surface, such as a rigid floor or pavement.

$$d^2 = 4.25(Ph/S_3\,b) \qquad \textbf{(Equation 5-3)}$$

or alternatively

$$d^2 = 4.25(M_g/S_3\,b) \qquad \textbf{(Equation 5-4)}$$

where:

M_g = Moment in the post at grade, in kN-m.

S_3 = Allowable lateral soil-bearing pressure as set forth in Section 4.3 based on a depth equal to the depth of embedment in kPa.

5.5.2.3 **Vertical load.** The resistance to vertical loads shall be determined by the allowable soil-bearing pressure set forth in Table 4.1.

5.5.3 **Backfill.** The backfill in the space around columns not embedded in poured footings shall be by one of the following methods:

1. Backfill shall be of concrete with compressive strength of 15 MPa at 28 days. The hole shall not be less than 100 mm larger than the diameter of the column at its bottom or 100 mm larger than the diagonal dimension of a square or rectangular column.
2. Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 200 mm in depth.
3. Backfill shall be of controlled low-strength material (CLSM) placed in accordance with Section 3.11.

SECTION 5.6
SEISMIC REQUIREMENTS

5.6.0 For footings of structures assigned to Seismic Design Category C or D, provisions of SBC 301 and SBC 304 shall apply when not in conflict with the provisions of Chapter 5.

CHAPTER 6
FOUNDATION WALLS

SECTION 6.1
GENERAL

6.1.0 Concrete and masonry foundation walls shall be designed in accordance with SBC 304 or SBC 305. Foundation walls that are laterally supported at the top and bottom and within the parameters of Tables 6.1 through 6.3 are permitted to be designed and constructed in accordance with Sections 6.2 through 6.6.

SECTION 6.2
FOUNDATION WALL THICKNESS

6.2.0 The minimum thickness of concrete and masonry foundation walls shall comply with Sections 6.2.1 through 6.2.3.

6.2.1 **Thickness based on walls supported.** The thickness of foundation walls shall not be less than the thickness of the wall supported, except that foundation walls of at least 200 mm nominal width are permitted to support brick-veneered frame walls and 250 mm cavity walls provided the requirements of Section 6.2.2 are met. Corbelling of masonry shall be in accordance with Section 4.2, SBC 305. Where a 200 mm wall is corbelled, the top corbel shall be a full course of headers at least 150 mm in length, extending not higher than the bottom of the floor framing.

TABLE 6.1
200-mm CONCRETE AND MASONRY FOUNDATION WALLS WITH REINFORCING WHERE EFFECTIVE DEPTH $d \geq$ 125 mm[a,b,c]

WALL HEIGHT (mm)	HEIGHT OF UNBALANCED BACKFILL (mm)	VERTICAL REINFORCEMENT		
		Soil classes and lateral soil loads[a] (kPa per meter below natural grade)		
		GW, GP, SW and SP soils (5)	GM, GC, SM, SM-SC and ML soils (7)	SC, MH, ML-CL and Inorganic CL soils (9)
2100	1200 (or less)	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.
	1500	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.	Dia 12 at 1000 o.c.
	1800	Dia 12 at 1200 o.c.	Dia 16 at 1200 o.c.	Dia 16 at 1000 o.c.
	2100	Dia 12 at1000 o.c.	Dia 16 at 1000 o.c.	Dia 18 at 1200 o.c.
2400	1200 (or less)	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.
	1500	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.	Dia 12 at 1000 o.c.
	1800	Dia 12 at 1200 o.c.	Dia 16 at 1200 o.c.	Dia 16 at 1000 o.c.
	2100	Dia 16 at 1200 o.c.	Dia 18 at 1200 o.c.	Dia 18 at 1000 o.c.
	2400	Dia 16 at 1000 o.c.	Dia 18 at 1000 o.c.	Dia 22 at 1000 o.c.
2700	1200 (or less)	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.
	1500	Dia 12 at 1200 o.c.	Dia 12 at 1200 o.c.	Dia 16 at 1200 o.c.
	1800	Dia 12 at 1200 o.c.	Dia 16 at 1200 o.c.	Dia 18 at 1200 o.c.
	2100	Dia 16 at 1200 o.c.	Dia 18 at 1200 o.c.	Dia 22 at 1200 o.c.
	2400	Dia 16 at 1000 o.c.	Dia 22 at 1200 o.c.	Dia 25 at 1200 o.c.
	2700	Dia 18 at 1000 o.c.	Dia 25 at 1200 o.c.	Dia 25 at 800 o.c.

a. For design lateral soil loads, see SBC 301 Section 5.1. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.
b. Provisions for this table are based on construction requirements specified in Section 6.3.
c. For alternative reinforcement, see Section 6.4.

6.2.2 **Thickness based on soil loads, unbalanced backfill height and wall height.** The thickness of foundation walls shall comply with the requirements of Table 6.1, 6.2 or 6.3. When using the tables, masonry shall be laid in running bond and the mortar shall be Type M or S.

TABLE 6.2
250-mm CONCRETE AND MASONRY FOUNDATION WALLS WITH REINFORCING WHERE EFFECTIVE DEPTH $d \geq 175$ mm[a,b,c]

WALL HEIGHT	HEIGHT OF UNBALANCED BACKFILL	VERTICAL REINFORCEMENT Soil classes and lateral soil load[a] (kPa per meter below natural grade)		
		GW, GP, SW and SP soils (5)	GM, GC, SM, SM-SC and ML soils (7)	SC, MH, ML-CL and Inorganic CL soils (9)
2100	1200 (or less)	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.
	1500	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.
	1800	Dia 12 at 1400 o.c.	Dia 12 at 1200 o.c.	Dia 12 at 1000 o.c.
	2100	Dia 12 at 1400 o.c.	Dia 16 at 1400 o.c.	Dia 16 at 1000 o.c.
2400	1200 (or less)	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.
	1500	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.	Dia 12 at 1200 o.c.
	1800	Dia 12 at 1400 o.c.	Dia 12 at 1200 o.c.	Dia 16 at 1400 o.c.
	2100	Dia 12 at 1200 o.c.	Dia 12 at 800 o.c.	Dia 18 at 1400 o.c.
	2400	Dia 16 at 1400 o.c.	Dia 16 at 1000 o.c.	Dia 22 at 1400 o.c.
2700	1200 (or less)	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.
	1500	Dia 12 at 1400 o.c.	Dia 12 at 1400 o.c.	Dia 12 at 1200 o.c.
	1800	Dia12 at 1400 o.c.	Dia 12 at 1000 o.c.	Dia 12 at 800 o.c.
	2100	Dia 12 at 1000 o.c.	Dia 16 at 1200 o.c.	Dia 18 at 1200 o.c.
	2400	Dia 12 at 800 o.c.	Dia 18 at 1200 o.c.	Dia 12 at 400 o.c.
	2700	Dia16 at 1000 o.c.	Dia 18 at 1000 o.c.	Dia 22 at 1000 o.c.

a. For design lateral soil loads, see SBC 301 Section 5.1. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.
b. Provisions for this table are based on construction requirements specified in Section 6.3.
c. For alternative reinforcement, see Section 6.4.

TABLE 6.3
300-mm CONCRETE AND MASONRY FOUNDATION WALLS WITH REINFORCING WHERE EFFECTIVE DEPTH $d \geq 225$ mm[a,b,c]

WALL HEIGHT (mm)	HEIGHT OF UNBALANCED BACKFILL (mm)	VERTICAL REINFORCEMENT Soil classes and lateral soil load[a] (kPa per meter below natural grade)		
		GW, GP, SW and SP soils (5)	GM, GC, SM, SM-SC and ML soils (7)	SC, MH, ML-CL and Inorganic CL soils (9)
2100	1200 (or less)	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.
	1500	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.
	1800	Dia 12 at 1800 o.c.	Dia 12 at 1600 o.c.	Dia 12 at 1200 o.c.
	2100	Dia 12 at 1800 o.c.	Dia 12 at 1200 o.c.	Dia 16 at 1400 o.c.
2400	1200 (or less)	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.
	1500	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.
	1800	Dia 12 at 1800 o.c.	Dia 12 at 1400 o.c.	Dia 16 at 1800 o.c.
	2100	Dia 12 at 1600 o.c.	Dia 16 at 1400 o.c.	Dia 12 at 800 o.c.
	2400	Dia 16 at 1200 o.c.	Dia 12 at 800 o.c.	Dia 16 at 1000 o.c.
2700	1200 (or less)	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.
	1500	Dia 12 at 1800 o.c.	Dia 12 at 1800 o.c.	Dia 12 at 1600 o.c.
	1800	Dia 12 at 1800 o.c.	Dia 12 at 1400 o.c.	Dia 16 at 1600 o.c.
	2100	Dia 12 at 1400 o.c.	Dia 12 at 1000 o.c.	Dia 18 at 1600 o.c.
	2400	Dia 12 at 1600 o.c.	Dia 18 at 1600 o.c.	Dia 18 at 1200 o.c.
	2700	Dia 16 at 1400 o.c.	Dia 22 at 1800 o.c.	Dia 18 at 1000 o.c.

a. For design lateral soil loads, see SBC 301 Section 5.1. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.
b. Provisions for this table are based on construction requirements specified in Section 6.3.
c. For alternative reinforcement, see Section 6.4.

Unbalanced backfill height is the difference in height of the exterior and interior finish ground levels. Where an interior concrete slab is provided, the unbalanced backfill height shall be measured from the exterior finish ground level to the top of the interior concrete slab.

6.2.3 **Rubble stone.** Foundation walls of rough or random rubble stone shall not be less than 400 mm thick. Rubble stone shall not be used for foundations for structures in Seismic Design Category C or D.

SECTION 6.3
FOUNDATION WALL MATERIALS

6.3.0 Foundation walls constructed in accordance with Table 6.1, 6.2, and 6.3 shall comply with the following:

1. Vertical reinforcement shall have minimum yield strength of 420 MPa.
2. The specified location of the reinforcement shall equal or exceed the effective depth distance, *d*, noted in Tables 6.1, 6.2 and 6.3 and shall be measured from the face of the soil side of the wall to the center of vertical reinforcement. The reinforcement shall be placed within the tolerances specified in ACI 530.1/ ASCE 6/TMS 402, Article 3.4 B7 of the specified location.
3. Concrete shall have a specified compressive strength of not less than 20 MPa at 28 days.
4. Grout shall have a specified compressive strength of not less than 15 MPa at 28 days.
5. Hollow masonry units shall comply with ASTM C 90 and be installed with Type M or S mortar.

SECTION 6.4
ALTERNATIVE FOUNDATION WALL REINFORCEMENT

6.4.1 In lieu of the reinforcement provisions in Table 6.1, 6.2 or 6.3, alternative reinforcing bar sizes and spacings having an equivalent cross-sectional area of reinforcement per linear meter of wall are permitted to be used, provided the spacing of reinforcement does not exceed 1.8 m and reinforcing bar sizes do not exceed Dia 36 mm.

SECTION 6.5
HOLLOW MASONRY WALLS

6.5.1 At least 100 mm of solid masonry shall be provided at girder supports at the top of hollow masonry unit foundation walls.

SECTION 6.6
SEISMIC REQUIREMENTS

6.6.0 Tables 6.1 through 6.3 shall be subject to the following limitations in Sections 6.6.1 and 6.6.2 based on the seismic design category assigned to the structure as defined in Chapters 9 through 16 of SBC 301.

6.6.1 **Seismic requirements for concrete foundation walls.** Concrete foundation walls designed using Tables 6.1 through 6.3 shall be subject to the following limitations:

1. Seismic Design Categories A and B. No limitations, except provide not less than two Dia 16 mm bars around window and door openings. Such bars shall extend at least 600 mm beyond the corners of the openings.

6.6.2 **Seismic requirements for masonry foundation walls.** Masonry foundation walls designed using Tables 6.1 through 6.3 shall be subject to the following limitations:

1. Seismic Design Categories A and B. No additional seismic requirements.

2. Seismic Design Category C. A design using Tables 6.1 through 6.3 subject to the seismic requirements of Section 6.1.4 SBC 305.

3. Seismic Design Category D. A design using Tables 6.1 through 6.3 subject to the seismic requirements of Section 6.1.5 SBC 305.

SECTION 6.7
FOUNDATION WALL DRAINAGE

6.7.1 Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 13.4.2 and 13.4.3.

SECTION 6.8
PIER AND CURTAIN WALL FOUNDATIONS

6.8.1 Except in Seismic Design Category D, pier and curtain wall foundations are permitted to be used to support light-frame construction not more than two stories in height, provided the following requirements are met:

1. All load-bearing walls shall be placed on continuous concrete footings bonded integrally with the exterior wall footings.

2. The minimum actual thickness of a load-bearing masonry wall shall not be less than 100 mm nominal or 90 mm actual thickness, and shall be bonded integrally with piers spaced 1800 mm on center (o.c.).

3. Piers shall be constructed in accordance with SBC 305 and the following:

a) The unsupported height of the masonry piers shall not exceed 10 times their least dimension.

b) Where structural clay tile or hollow concrete masonry units are used for piers supporting beams and girders, the cellular spaces shall be filled solidly with concrete or Type M or S mortar.

Exception: Unfilled hollow piers are permitted where the unsupported height of the pier is not more than four times its least dimension.

c) Hollow piers shall be capped with 100 mm of solid masonry or concrete or the cavities of the top course shall be filled with concrete or grout.

4. The unbalanced fill for 100 mm foundation walls shall not exceed 600 mm for solid masonry, and 300 mm for hollow masonry.

SECTION 6.9
SEISMIC REQUIREMENTS

6.9.1 For foundations of structures assigned to Seismic Design Category C or D, provisions of SBC 301 and SBC 304 shall apply when not in conflict with the provisions of Chapter 6.

CHAPTER 7
RETAINING WALLS

SECTION 7.1
GENERAL

7.1.1 **SCOPE.** This Chapter shall apply to all matters pertaining to design and construction of rigid gravity, semi gravity, cantilever, buttressed, and counterfort retaining walls. For special types of retaining walls, provisions of this code shall apply where applicable. General safeguards during construction shall comply with provisions of Chapter 3.

SECTION 7.2
LATERAL EARTH PRESSURES

7.2.0 Computations of lateral earth pressures shall comply with the provisions of Sections 7.2.1 through 7.2.6. Wall movements set forth in Table 7.1 shall be considered the magnitude required for active and passive conditions to exist. Soil permeability characteristics, boundary drainage and loading conditions, and time shall be considered in selection of strength parameters. In soils where partial drainage occurs during the time of construction, analysis shall be performed for short-term and long-term conditions, and the wall shall be designed for the worse conditions.

7.2.1 **Wall friction.** Wall friction and vertical movement, slope of the wall in the backside and sloping backfill shall be considered in determining the lateral pressures applied against the wall. Unless data to substantiate the use of other values are submitted and approved by a registered design professional, the values set forth in Tables 7-2 and 7-3 shall be used in computations that include effects of wall friction.

TABLE 7.1
MAGNITUDE OF ROTATION TO REACH FAILURE

SOIL TYPE AND CONDITION	ROTATION $(\delta/H)^a$	
	ACTIVE	PASSIVE
Dense cohesionless soil	0.0005	0.002
Loose cohesionless soil	0.002	0.006
Stiff cohesive soil	0.01	0.02
Soft cohesive soil	0.02	0.04

a. δ = Horizontal translation at the top of the wall.
H = Height of the wall

TABLE 7.2
ULTIMATE FRICTION FACTORS FOR DISSIMILAR MATERIALS

INTERFACE MATERIALS	FRICTION FACTOR, tan δ [a]
Clean sound rock	0.7
Clean gravel, gravel-sand mixtures, coarse sand	0.55 – 0.60
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 – 0.55
Clean fine sand, silty or clayey fine to medium sand	0.35 – 0.45
Fine sandy silt, nonplastic silt	0.30 – 0.35
Very stiff and hard residual or preconsolidated clay	0.40 – 0.50
Medium stiff and stiff clay and silty clay	0.30 – 0.35

a. Values for δ shall not exceed one-half the angle of internal friction of the backfill soils for steel and precast concrete and two-third the angle of internal friction of the backfill soils for cast-in place concrete.

7.2.2 **Wall movement.** The effect of wall movement on the earth pressure coefficients shall conform to the provisions of Sections 7.2.2.1 and 7.2.2.2.

7.2.2.1 **Rotation.** If the wall is free at the top and there are no other structures associated with, wall tilting shall not exceed 0.1 times the height of the wall. Where the actual estimated wall rotation is less than the value required to fully mobilize active or passive conditions set forth in Table 7-1, the earth pressure coefficient shall be adjusted in accordance with Figure 7.1.

TABLE 7.3
ULTIMATE ADHESION FOR DISSIMILAR MATERIALS

INTERFACE MATERIALS	COHESION (kPa)	ADHESION (kPa)
Very soft cohesive soil	0 - 10	0 - 10
Soft cohesive soil	10 - 25	10 - 25
Medium stiff cohesive soil	25 - 50	25 - 35
Stiff cohesive soil	50 - 100	35 - 45
Very stiff cohesive soil	100 - 200	45 - 60

7.2.2.2 **Translation.** It shall be permitted to consider uniform translation required to mobilize ultimate passive resistance or active pressure equivalent to movement of top of wall based on rotation given in Table 7.1.

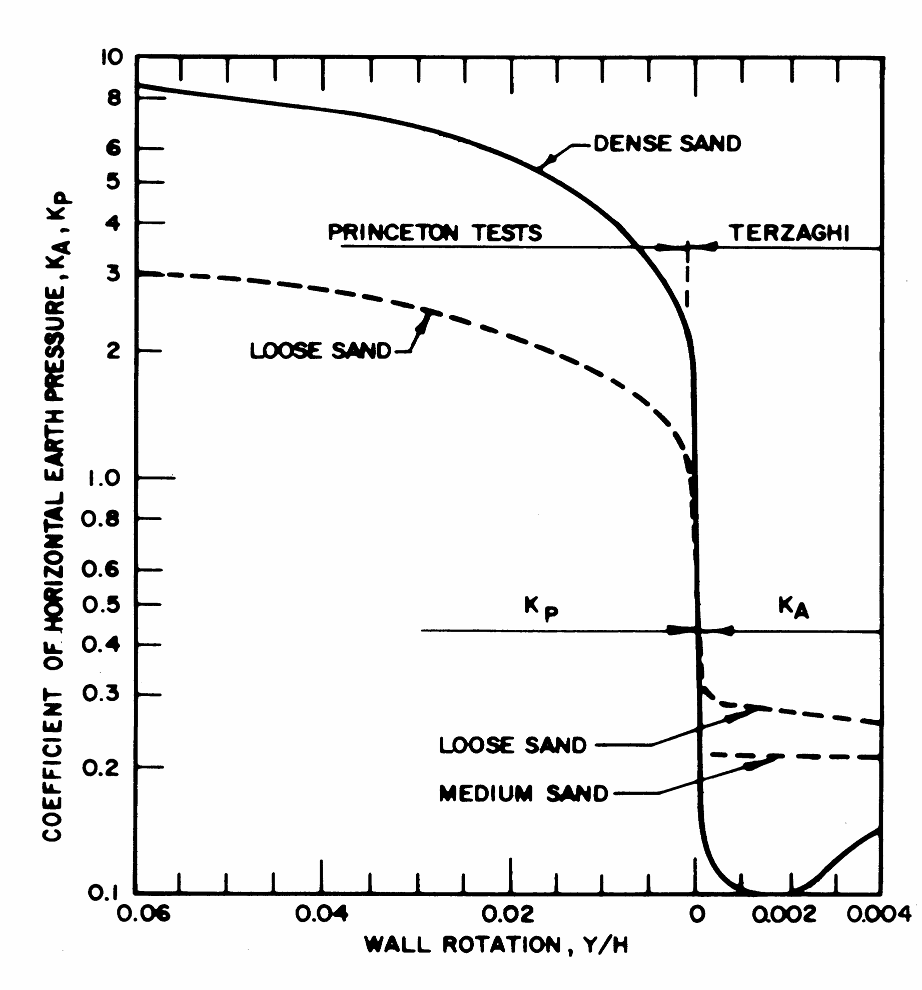


FIGURE 7.1
EFFECT OF WALL MOVEMENT ON WALL PRESSURES (NAFAC, 1986)

7.2.2.3 **Restrained wall**. Where wall is prevented from even slight movement, the earth pressure shall be considered to remain at rest conditions.

7.2.2.4 **Basement and other below grade walls**. Pressures on walls below grade shall be computed based on restrained conditions that prevail, type of backfill, and the amount of compaction. The provisions of Chapter 6 shall apply where applicable.

7.2.2.5 **Wall on rock**. Where the wall is founded on rock, sufficient rotation of the base and wall so that active pressure is developed, shall be accomplished by placing 150 to 300 mm thick earth pad beneath the base and by constructing the stem with sufficient flexibility to yield with the soil pressure.

7.2.3 **Groundwater conditions**. Pressure computations shall include uplift pressures and the effect of the greatest unbalanced water head anticipated to act across the wall. For cohesionless materials, increase in lateral force on wall due to rainfall shall be considered and walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 13.4.2 and 13.4.3.

7.2.4 **Surcharge.** Stability shall be checked with and without surcharge. Lateral pressure on wall due to point and line loads shall be computed based on the assumption of an unyielding rigid wall and the lateral pressures are set equal to double the values obtained by elastic equations. The applicability of the assumption of an unyielding rigid wall shall be evaluated for each specific wall.

For uniform surcharge loading it shall be permitted to compute lateral stress by treating the surcharge as if it were backfill and multiplying the vertical stress at any depth by the appropriate earth pressure coefficient. It shall be permitted for design purposes to considerer a distributed surface load surcharge on the order of 15 kPa to account for construction materials and equipment stored within 5 to 10 meters from the wall. Where construction equipment is anticipated within 2 meters of the wall, it must be accounted for separately.

7.2.5 **Compaction**. For backfill of granular soils compacted in a confined wedge behind the wall, the horizontal pressure beyond those represented by active or at rest values shall be computed in accordance with Figure 7.2.

Compaction-induced pressures shall not be considered in bearing, overturning and sliding analyses and need only be considered for structural design. Backfill shall be brought up equally on both sides until the lower side finished grade is reached and precautions shall be taken to prevent over-compaction which will cause excessive lateral forces to be applied to the wall.

Clays and other fine-grained soils, as well as granular soils, with amount of clay and silt greater than 15 percent shall not be used as a backfill behind retaining wall. Where they must be used, the lateral earth pressure shall be calculated based on at rest conditions, with due consideration to potential poor drainage conditions and swelling. Where loose hydraulic fill is used it shall be placed by procedures which permit runoff of wash water and prevent building up of large hydrostatic pressures.

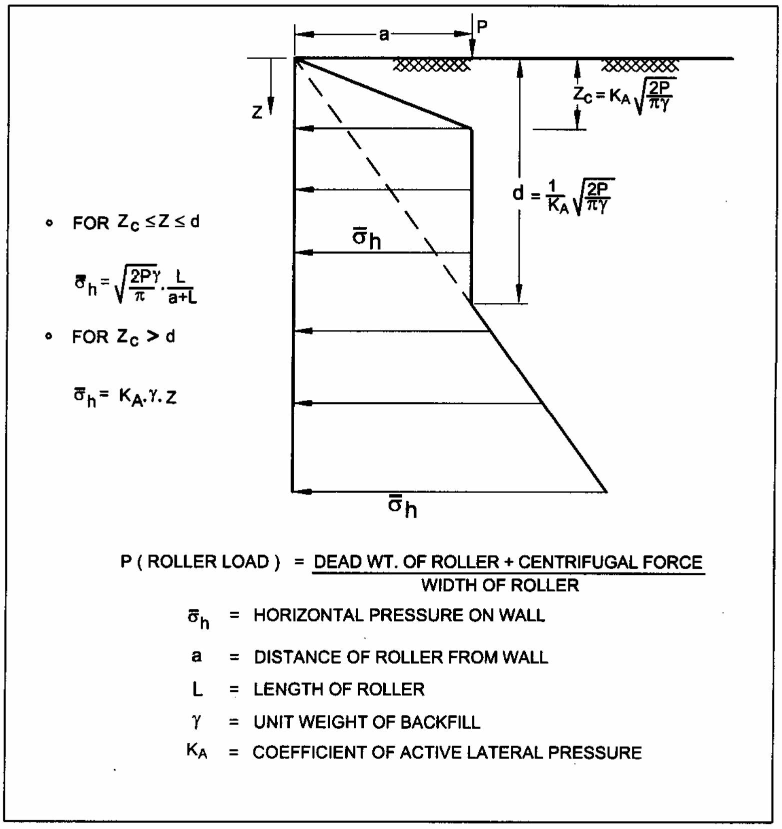


FIGURE 7.2
HORIZONTAL PRESSURE ON WALLS FROM COMPACTION EFFORT (NAFAC, 1986)

7.2.6 **Earthquake loading**. For retaining walls assigned to Seismic Design Category C or D, provisions of SBC 301 and SBC 304 shall apply when not in conflict with the provisions of Chapter 7.

The combined resultant active force due to initial static pressure and increase in pressure from ground motion shall be computed from the following formula

$$P_{AE} = \frac{1}{2}\gamma H^2 k_A(\beta^*, \theta^*)(1-k_v)\frac{\cos^2\theta^*}{\cos\psi\cos^2\theta} \qquad \textbf{(Equation 7-1)}$$

where:

P_{AE} = Combined resultant active force.

H = Wall height.

θ = Slope of wall back with respect to vertical

β = Inclination of soil surface (upward slopes away from the wall are positives).

θ^* = $(\theta + \psi)$ = Modified slope of wall back.

β^* = $(\beta + \psi)$ = Modified inclination of soil surface.

γ = Unit weight of soil.

ψ = Seismic inertia angle given as follows

$$\psi = \tan^{-1}\left(\frac{k_h}{1-k_v}\right) \qquad \textbf{(Equation 7-2)}$$

where:

k_h = Horizontal ground acceleration in g's.

k_v = Vertical ground acceleration in g's.

For modified slope angle β^* and θ^*, the modified coefficient of earth pressures $k_A(\beta^*,\theta^*)$ shall be calculated from the Coulomb theory. Dynamic pressure increment shall be obtained by subtracting static active force (to be determined from Coulomb theory for given β and θ) from combined active force given by Equation 7-1. Location of resultant shall be obtained by considering the earth pressure to be composed of a static and dynamic component with the static component acts at the lowest third point, whereas the dynamic component acts above the base at 0.6 times the height of the wall. Under the combined effect of static and earthquake load the factor of safety shall not be less than 1.2.

Where soil is below water, the hydrodynamic pressure computed from the following formula shall be added

$$p_{wz} = \frac{3}{2} k_h \gamma_w (h_w z)^{1/2} \qquad \textbf{(Equation 7-3)}$$

where:

p_{wz} = Hydrodynamic pressure at depth z below water surface

h_w = Height of water

z = Depth below the water surface.

γ_w = Unit weight of water.

SECTION 7.3
BEARING CAPACITY

7.3.1 The allowable soil pressure shall be determined in accordance with the provisions of Chapter 4. The determination of the allowable bearing pressure shall be made according to the bearing capacity of a foundation subjected to eccentric loads. The bearing capacity shall be checked for the same loading conditions as determined by the overturning analysis for each case analyzed. Where the wall is founded on sloped ground, methods for determination of ultimate bearing capacity that deal with this situation shall be used. The factor of safety with respect to bearing capacity shall not be less than 3.

For walls founded on rocks, high toe pressure that may cause breaking the toe from the remainder of the base shall be avoided by proportioning the footing so that the resultant falls near its center.

SECTION 7.4
STABILITY

7.4.0 Retaining walls shall be designed to ensure stability against overturning, sliding, and stability of supporting ground. Stability analyses shall conform to the provisions of Sections 7.4.1 through 7.4.4.

7.4.1 **Sliding stability**. The base shall be at least 1000 mm below ground surface in front of the wall. Sliding stability shall be adequate without including passive pressure at the toe. Where insufficient sliding resistance is available, one provision shall be taken including, but need not be limited to, increasing the width of the wall base, founding the wall on piles or lowering the base of the wall. If the wall is supported by rock or very stiff clay, it shall be permitted to install a key below the foundation to provide additional resistance to sliding. The key shall conform to the provisions of Section 7.4.4. The factor of safety against sliding shall not be less than 1.5 for cohesionless backfill and 2.0 for cohesive backfill.

7.4.2 **Overturning stability**. For walls on relatively incompressible foundations, overturning check is ignored if the resultant is within the middle third of the base for walls founded on soils and if the resultant is within the middle half for walls founded on rocks. Where foundation is compressible settlement shall be computed based on any method approved by the building official. Tilt of rigid wall shall be obtained from the estimated settlement. Differential settlement shall be limited to the amount of tilting that shall not exceed 5 percent of wall height. If the consequent tilt exceeds acceptable limits, the wall shall be proportioned to keep the resultant force at the middle third of base. The retaining wall shall be proportioned so that the factor of safety against overturning is not less than 1.5. The value of angular distortion (settlement/length of structure) of retaining walls shall not exceed 0.002 radians.

7.4.3 **Deep-seated sliding**. Where retaining walls are underlain by weak soils, the overall stability of the soil mass containing the retaining wall shall be checked with respect to the most critical surface of sliding. The stability analysis shall be made for after construction and for long-term conditions. The factor of safety for the overall stability of the soil mass containing the wall shall not be less than 2.

7.4.4 **Wall with key**. Prior to performing an overturning analysis, the depth of the key and width of the base shall be determined from the sliding stability analysis. For a wall with a horizontal base and a key, it shall be permitted to assume the shearing resistance of the base to be zero and the horizontal resisting force acting on the key is that required for equilibrium. For a wall with a sloping base and a key, the horizontal force required for equilibrium shall be assumed to act on the base and the key In both cases the resisting soil force down to the bottom of the toe shall be computed using at-rest earth pressure if the material on the resisting side will not lose its resistance characteristics with any probable change in water content or environmental conditions and will not be eroded or excavated during the life of the wall.

SECTION 7.5
WALL DIMENSIONS

7.5.1 Thickness of the upper part of the wall shall not be less than 300 mm, where as thickness of the lower part of the wall shall be enough to resist shear without reinforcement. Depth of wall foundation shall be located below line of seasonal changes and shall be deep enough to provide adequate bearing capacity and soil sliding resistance. The wall foundation shall be proportioned such that the wall does not slide or overturn, the allowable bearing capacity of the soil is not exceeded, and that total and differential settlements are tolerable. The base and other dimensions shall be such that the resultant falls within the middle third of the base. Where additional front clearance is needed, it shall be permitted to construct counterfort retaining walls without a toe provided that the sliding and overturning stability requirements stated in Sections 7.4.1 and 7.4.2 are met.

SECTION 7.6
WALL CONSTRUCTION

7.6.0 Concrete 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 and that will provide the depositing or construction of sound concrete in the dry.

7.6.1 **Minimum concrete cover to reinforcement**. When the concrete of retaining walls is poured directly on the ground or against excavation walls the minimum concrete cover to reinforcement shall not be less than 75 mm and not less than 40 mm when concrete is poured against lean concrete or vertical forms. This cover shall also satisfy other requirements with regard to concrete exposure conditions presented in SBC 304.

7.6.2 **Joints**. Construction and expansion joints shall be provided where needed. Construction joints shall be constructed into a retaining wall between successive pours of concrete both horizontally and vertically. Horizontal construction joints shall be kept to a minimum and the top surface of each lift shall be cleaned and roughened before placing the next lift. Long walls shall have expansion joints at intervals of 10 meters. Where vertical-expansion joints are considered, they shall be placed along the wall at spacing of 20 to 30 meters. Reinforcing steel and other fixed metal embedded or bonded to the surface of the concrete shall not extend through the expansion joint. For cantilever concrete walls, it shall be permitted to locate the vertical expansion joints only on the stem, and the footing is a continuous placement.

The thickness of joint filler necessary to provide stress relief at a joint shall be determined from the estimated initial contraction and subsequent expansion from maximum temperature variation.

7.6.3 **Drainage.** Regardless of the drainage system used, the wall must have an adequate factor of safety assuming the drainage system is inoperative. Where drainage measures are considered they shall be designed by a registered design professional and subject to the approval of the building official. As a minimum, there shall be weep holes with pockets of coarse-grained material at the back of the wall, and a gutter shall be provided for collecting runoff. All retaining walls shall have adequate surface drainage to dispose of surface water. A layer of impervious soil shall be placed on top of the soil backfill to reduce surface infiltration of rainfall. It shall be permitted to use inclined and horizontal drains in conjunction with back drain.

The weep holes shall be of sufficient size and be carefully surrounded with a granular filter or by the use of filter fabric on the backfill side and directly surrounding the entrance to the weep holes. The weep holes shall be spaced not more than 3 m apart vertically and horizontally. Where longitudinal drains along the back face are used, a layer of free-draining granular material shall be placed along the back of the wall and surrounding the drain pipes opening. The gradation of the filter shall satisfy the following piping or stability criterion.

$$\frac{d_{15\mathrm{F}}}{d_{85B}} \leq 5 \qquad \textbf{(Equation 7-4)}$$

where:

d_{15F} = size of filter material at 15 percent passing.

d_{85B} = size of protected soil at 85 percent passing.

and

$$\frac{d_{50\mathrm{F}}}{d_{50\mathrm{B}}} \leq 25 \qquad \textbf{(Equation 7-5)}$$

where:

d_{50F} = size of filter material at 50 percent passing.

d_{50B} = size of protected soil at 50 percent passing.

The filter material shall be more permeable than the material being drained and the following condition shall be met

$$4 < \frac{d_{15\mathrm{F}}}{d_{15\mathrm{B}}} < 20 \qquad \textbf{(Equation 7-6)}$$

where:

d_{15B} = size of filter protected soil at 15 percent passing.

Where a blanket of well-graded sand and gravel that is placed along the back of the wall it shall satisfied the requirements of Equations 7-4 through 7-6. Where longitudinal drains are used within drainage blanket, they shall be large enough to carry the discharge and have adequate slope to provide sufficient velocity to remove sediment from the drain. Segregation of sand and gravel during construction shall be avoided. Filter or drain materials contaminated by muddy water, dust, etc. shall be replaced and filter materials subject to cementation shall be rejected.

In lieu of a granular filter, it shall be permitted to use prefabricated geocomposite drains with adequate filter flow capacity and acceptable retention. The size of filter material at 50 percent passing, d_{50F} , shall not be less than the diameter of the hole for circular openings and shall be 1.2 times slot width for slotted openings. The drainage composite manufacture's recommendations for backfilling and compaction near the composite shall be followed.

CHAPTER 8
COMBINED FOOTINGS AND MATS

SECTION 8.1
GENERAL

8.1.0 Analysis and design of combined footings and mats shall conform to all requirements of ACI 336.2R Suggested Analysis and Design Procedures for Combined Footings and Mats except as modified by Chapter 8. All provisions of SBC 303 not specifically excluded, and not in conflict with the provisions of Chapter 8 shall apply to combined footings and mats, where applicable. Design of combined or mat foundations shall be based on the Strength Design Method of SBC 304.

Combined footings and mats shall be designed and constructed on the basis of a site investigation as defined in Chapter 2, unless the building official ascertains that sufficient data upon which to base the design and installation is available. The investigation and report provisions of Chapter 2 shall be expanded to include, but need not be limited to, the following:

1. Values for modulus of subgrade reaction.

2. Recommended combined footings shapes.

8.1.1 **Approval of special systems of design or construction**. Sponsors of any system of design or construction within the scope of Chapter 8, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by Chapter 8, shall have the right to present that data on which their design is based to the building official or to a board of examiners appointed by the building official. This board shall be composed of competent geotechnical and structural engineers and shall have authority to investigate the data so submitted, to require tests, and to formulate rules governing design and construction of such systems to meet the intent of Chapter 8. These rules when approved by the building official and promulgated shall be of the same force as the provisions of Chapter 8.

SECTION 8.2
LOADINGS

8.2.1 Combined footings and mats shall be designed for the most unfavorable effects due to the combinations of loads specified in SBC 301 Section 2.4. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in Section 4.8 SBC 301, are permitted to be used in designing footings. Strength design of reinforced concrete systems and elements shall comply with load combinations specified in SBC 304.

SECTION 8.3
CONCRETE

8.3.1 Material, construction, and placement of concrete shall be in accordance with the provisions of Section 5.4.2. For mats construction joints shall be carefully located at sections of low shear stress or at the center lines between columns. An elapse of at least 24 hours shall be left between pours of adjacent areas. If bar splicing is needed, sufficient overlapping shall be provided. The concrete shall be strong enough to transfer the shear stress across the joint. If necessary, the mat may be thickened to provide sufficient strength in the joints.

SECTION 8.4
CONTACT PRESSURE

8.4.0 Soil contact pressure acting on a combined footing or mat and the internal stresses produced by them shall be determined from one of the load combinations given in Section 2.4 SBC 301**,** whichever produces the maximum value for the element under investigation.

The combinations of unfactored loads which will produce the greatest contact pressure on a base area of given shape and size shall be selected. The allowable soil pressure shall be determined in accordance with the provisions of Chapter 4. Loads shall include the vertical effects of moments caused by horizontal components of these forces and by eccentrically applied vertical loads. Buoyancy of submerged parts where this reduces the factor of safety or increases the contact pressures, as in flood conditions shall be considered.

The maximum unfactored design contact pressures shall not exceed the allowable soil pressure as obtained from Chapter 4 or cause settlements that exceed the values set forth in Table 5.1 and 8.3. Where wind or earthquake forces form a part of the load combination, the allowable soil pressure may be increased as allowed by the Saudi Building Code or approved by the building official.

In determination of the contact pressures and associated subgrade response, the validity of simplifying assumptions and the accuracy of any resulting computations shall be approved by the building official and evaluated on the basis of the following variables:

1. The increased unit pressures developed along the edges of rigid footings on cohesive soils and at the center for rigid footings on cohesionless soils.

2. The effect of embedment of the footing on pressure variation.

3. Consideration in the analysis of the behavior of the foundations immediately after the structure is built as well as the effects of long-term consolidation of deep soil layers.

4. Consideration of size of the footing in determination of the modulus of subgrade reaction of soil.

5. The variation of contact pressures from eccentric loading conditions.

6. Consideration of the influence of the stiffness of the footing and the superstructure on deformations that can occur at the contact surface and the corresponding variation on contact pressure and redistribution of reactions occurring within the superstructure frame.

8.4.1 **Distribution of soil reactions**. Contact pressures at the base of combined footings and mats shall be determined in accordance with Sections 8.4.1.1 through 8.4.1.3.

8.4.1.1 **General.** Except for unusual conditions, the contact pressures at the base of a combined footings and mats may be assumed to follow either a distribution governed by elastic subgrade reaction or a straight-line distribution. At no place shall the calculated contact pressure exceed the allowable bearing capacity as determined from Chapter 4.

8.4.1.2 **Straight-line distribution of contact pressure.** It shall be permitted to assume a linear distribution for soil contact pressure if continuous footings meet the requirement of Section 8.7.1 and mats conform to the requirements of Section 8.9.3.2.

8.4.1.2.1 **Contact pressure over total base area**. If the resultant of all forces is such that all portions of the foundation contact area are in compression, the maximum and minimum soil pressure may then be calculated from the following formula, which applies only to the rectangular base areas and only when eccentricity is located along one of the principal axes of the footing

$$q_{\max,\min} = \frac{\sum P}{BL}\left(1 \pm \frac{6e}{L}\right)$$ **(Equation 8-1)**

where:

$q_{\max}$ = Maximum soil contact pressure.

$q_{\min}$ = Minimum soil contact pressure.

P = Any force acting perpendicular to base area.

B = Foundation width or width of beam column element.

e = Eccentricity of resultant of all vertical forces with center of footing area ($e \leq L/6$).

L = Foundation base length or length of beam column element.

For footings with eccentricity about both axes, soil pressure is obtained from

$$q = \frac{\sum P}{BL}\left(1 \pm \frac{6e_x}{L} \pm \frac{6e_y}{B}\right)$$ **(Equation 8-2)**

where:

q = Soil contact pressure.

e_x = Eccentricity of resultant of all vertical forces with respect to the x-axis.

e_y = Eccentricity of resultant of all vertical forces with respect to the y-axis.

P = Any force acting perpendicular to base area.

8.4.1.2.2 **Contact pressure over part of area.** The soil pressure distribution shall be assumed to be triangular and the resultant has the same magnitude and colinear, but acts in the opposite direction of the resultant of the applied forces.

The maximum soil pressure at the footing edge under this condition shall be calculated from the following expression

$$q_{ult} = \frac{2P}{3B}\left(\frac{L}{2} - e\right)$$ **(Equation 8-3)**

The minimum soil pressure at distance L_1 is set equal to zero, where L_1 is the footing effective length measured from the pressed edge to the position at which the contact pressure is zero and is given by

$$L_1 = 3\left(\frac{L}{2} - e\right)$$ **(Equation 8-4)**

Equations 8-3 and 8-4 are applied based on the assumption that no tensile stresses exist between footing and soil and for cases where the resultant force falls out of the middle third of the base.

8.4.1.3 **Distribution of contact pressure governed by the modulus of subgrade reaction.** It shall be permitted to get the distribution of contact pressure based on modulus of subgrade reaction obtained from Section 8.4.1.3.2. The thickness shall be sized for shear without using reinforcement. The flexural steel is then obtained by assuming a linear soil pressure distribution and using simplified procedures in which the foundation satisfies static equilibrium. The flexural steel may also be obtained by assuming that the foundation is an elastic member interacting with an elastic soil.

8.4.1.3.1 **Beams on elastic foundations.** If the combined footing is assumed to be a flexible slab, it may be analyzed as a beam on elastic foundation. It shall be permitted to analyze a beam on elastic foundation using the discrete element method, the finite element method, or other methods approved by the building official.

8.4.1.3.2 **Estimating the modulus of subgrade reaction.** The value for modulus of subgrade reaction may be obtained from one of the methods in Sections 8.4.1.3.2.1 through 8.4.1.3.2.3. It shall be permitted to use a constant value for the modulus of subgrade reaction except where the rigidity of the footing and superstructure is considered small, the decrease in the value of modulus of subgrade reaction, k_s, with increasing applied load shall be taken into consideration.

8.4.1.3.2.1 **Presumptive modulus of subgrade reaction values**. It shall be permitted to use values for the modulus of subgrade reaction for supporting soils as set forth in Table 8.1 and 8.2 to determine about the correct order of magnitude of the subgrade modulus obtained from Sections 8.4.1.3.2.1 through 8.4.1.3.2.5. The tables' values shall be used only as a representative guide.

8.4.1.3.2.2 **Modulus of subgrade reaction from plate load test**. For mat foundations, this soil property shall not be estimated on the basis of field plate load tests and shall be obtained using subgrade reaction theory, but shall be modified to individually consider dead loading, live loading, size effects, and the associated subgrade response. Zones of different constant subgrade moduli shall be considered to provide a more accurate estimate of the subgrade response as compared to that predicted by a single modulus of subgrade reaction.

TABLE 8.1
PRESUMPTIVE MODULUS OF SUBGRADE REACTION VALUES FOR COHESIONLESS SOILS

RELATIVE DENSITY	UNCORRECTED SPT VALUES	MODULUS OF SUBGRADE REACTION (kN/m^3)	
		DRY AND MOIST SOILS	SUBMERGED SOILS
Loose	Less Than 10	15000	10000
Medium dense	10-30	45000	30000
Dense	> 30	175000	100000

TABLE 8.2
PRESUMPTIVE MODULUS OF SUBGRADE REACTION VALUES FOR COHESIVE SOILS

CONSISTENCY	SHEAR STRENGTH FROM UNCONFINED COMPRESSION TEST (kPa)	MODULUS OF SUBGRADE REACTION (kN/m^3)
Stiff	105-215	25000
Very stiff	215-430	50000
Rigid	> 430	100000

The value for the modulus of subgrade reaction for use in elastic foundation analysis may be estimates from a plate load test carried out in accordance with ASTM D1194. Since plate load tests are of necessity on small plates, great care must be exercised to insure that results are properly extrapolated. The modulus of subgrade reaction from plate load test shall be converted to that of mat using the following formula

$$k_s = k_p \left(\frac{B_p}{B_m} \right)^n \quad \textbf{(Equation 8-5)}$$

where:

k_s = Coefficient (or modulus) of vertical subgrade reaction; generic term dependent on dimensions of loaded area.

k_p = Coefficient of subgrade reaction from a plate load test.

B_m = Mat width.

B_p = Plate width.

n = Factor that ranges from 0.5 to 0.7.

Allowance shall be made for the depth of compressible strata beneath the mat and if it is less than about four times the width of footing, lower values shall be used for n.

8.4.1.3.2.3 **Modulus of subgrade reaction from elastic parameters.** It shall be permitted to estimate the value for the modulus of subgrade reaction based on laboratory or in situ tests to determine the elastic parameters of the foundation material. This shall be done by numerically integrating the strain over the depth of influence to obtain a settlement ΔH and back computing k_s as

$$k_s = \frac{q}{\Delta H} \quad \textbf{(Equation 8-6)}$$

where:

q = Applied pressure.

ΔH = Settlement.

Several values of strain shall be used in the influence depth of approximately four times the largest dimension of the base.

It shall be permitted to estimate the modulus of subgrade reaction based on laboratory measured modulus of elasticity such that

$$k_s = \frac{E_s}{B(1-\nu^2)}$$ **(Equation 8-7)**

where:

ν = Poisons ratio for soil.

E_s = Modulus of Elasticity for soil.

8.4.1.3.2.4 **Modulus of subgrade reaction from load bearing.** In the absence of a more rigorous data, it shall be permitted to consider a value for the modulus of subgrade reaction equal to 120 times the allowable load bearing. The value for k_s shall be verified from in situ tests in case of sensitive and important structures.

8.4.1.3.2.5 **Time-dependent subgrade response**. Consideration shall be given to the time-dependent subgrade response to the loading conditions. An iterative procedure may be necessary to compare the mat deflections with computed soil response. Since the soil response profile is based on contact stresses which are in turn based on mat loads, flexibility, and modulus of subgrade reaction, iterations shall be made until the computed mat deflection and soil response converge within acceptable tolerance.

SECTION 8.5
SETTLEMENT

8.5.0 Settlements of combined footings and mats shall conform to the provisions of Sections 8.5.1 through 8.5.3.

8.5.1 **General**. The combinations of unfactored loads which will produce the greatest settlement or deformation of the foundation, occurring either during and immediately after the load application or at a later date, shall be selected. Loadings at various stages of construction such as dead load or related internal moments and forces, stage dead load consisting of the unfactored dead load of the structure and foundation at a particular time or stage of construction, and stage service live load consisting of the sum of all unfactored live loads at a particular stage of construction, shall be evaluated to determine the initial settlement, long-term settlement due to consolidation, and differential settlement of the foundation.

8.5.2 **Total settlements**. Total settlement of combined footings and mats shall not exceed the value set forth in Table 5.1.

8.5.3 **Differential settlement.** Differential settlements for combined footings shall not exceed the values set forth in Table 5.2. For mats the differential settlement shall be taken as three-fourths the total settlement if this is not more than 50 mm or determined based on relative stiffness, k_r, as shown in Table 8.3.

SECTION 8.6
COMBINED FOOTINGS

8.6.0 Combined footings shall be designed and constructed in accordance with Sections 8.6.1 through 8.6.3.

8.6.1 **Rectangular-shaped footings**. The length and width of rectangular-shaped footings shall be established such that the maximum contact pressure at no place exceeds the allowable soil pressure as obtained from Chapter 4. All moments shall be calculated about the centeroid of the footing area and the bottom of the footing. All footing dimension shall be computed on the assumption that the footing acts as a rigid body.

TABLE 8.3
MAXIMUM ALLOWABLE DIFFERENTIAL SETTLEMENTS OF MATS

k_r	SHAPE	DIFFERENTIAL SETTLEMENT (mm)
0	Rectangular base	$0.5 \times \Delta H$[a]
	Square base	$0.35 \times \Delta H$
0.5		$0.1 \times \Delta H$
> 0.5	Rigid mat: no differential settlement	

a. ΔH = Total settlement estimated based on approved methods of analysis but shall not exceed values in Table 5.1.

When the resultant of the columns load, including consideration of the moments from lateral forces, coincides with centeroid of the footing area, it shall be permitted to assume that the contact pressure is uniform over the entire area of the footing. The resultant of the load of the two columns shall not fall outside the middle third of the footing. In case where this provision cannot be fulfilled the contact pressure may be assumed to follow a linear distribution such that it varies from a maximum at the pressed edge to a minimum either beneath the footing or at the opposite edge to zero at a distance that is equal three times the distance between the point of action of the resultant of loads and the pressed edge..

Consideration shall be given to horizontal forces that can generate vertical components to the foundation due, but need not be limited to, wind, earth pressure, and unbalanced hydrostatic pressure. A careful examination of the free body must be made with the geotechnical engineer to fully define the force systems acting on the foundation before the structural analyses are attempted.

8.6.2 **Trapezoidal or irregularly shaped footings.** For reducing eccentric loading conditions, it shall be permitted to design a trapezoidal or irregularly shaped footing with the footing considered to act as a rigid body and the contact pressure determined in accordance with Section 8.4.

8.6.3 **Strap footings.** The strap shall be rigid enough to avoid rotation of the exterior footing and the footings shall be proportioned for approximately equal soil pressure. A large difference in footing width shall be avoided to reduce differential settlement. It shall be permitted to consider the strap to be rigid if it has a moment of inertia that is not less than four times that of the footing to which it is attached. The width of the strap shall be equal to the smallest column width.

Shear reinforcement in the strap shall not be used to increase rigidity. If the depth of footing is restricted, the depth of the strap may be increased to obtain the necessary rigidity. The strap shall be out of contact with soil. The strap shall be securely attached to the column and footing by dowels so that the system acts as a unit. The footings shall be proportioned so that the least lateral dimensions are within 300 to 600 mm of each other and the soil pressures are approximately equal.

8.6.4 **Overturning calculations.** In analyzing overturning of the footing, the combination of unfactored loading that produces the greatest ratio of overturning moment to the corresponding vertical load shall be used. Where the eccentricity is inside the footing edge, the factor of safety against overturning shall be taken as the ratio of resisting moment to the maximum overturning moment. The maximum overturning moment and the resisting moment caused by the minimum dead weight of the structure; both shall be calculated about the pressed edge of the footing. The factor of safety shall not be less than 1.5.

If overturning is considered to occur by yielding of the subsoil inside and along the pressed edge of the footing, the factor of safety against overturning shall be calculated from

$$FS = \frac{R_{v\,\min}(c - v)}{M_0} \qquad \textbf{(Equation 8-8)}$$

where:

FS = Factor of safety.

c = Distance from resultant of vertical forces to overturning edge.

v = Distance from the pressed edge to $R_{v\,\min}$.

M_0 = Overturning moment.

$R_{v\,\min}$ = Least resultant of all forces acting perpendicular to base area under any condition of loading simultaneous with the overturning moment.

Both cases of rectangular and triangular distribution of the soil pressure along the pressed edge of the footing shall be considered and the value for FS shall not be less than 1.5.

SECTION 8.7
CONTINUOUS FOOTINGS

8.7.0 Continuous foundations shall be designed and constructed in accordance with Sections 8.7.1 through 8.7.3.

8.7.1 **Design for rigid structures.** Continuous strip footings supporting structures which, because of their stiffness, will not allow the individual columns to settle differentially may be designed using the rigid body assumption with a linear distribution of soil pressure determined based on statics.

8.7.1.1 **Rigidity check based on relative stiffness.** If the analysis of the relative stiffness of the footing yields a value greater than 0.5 the footing can be considered rigid and the variation of soil pressure shall be determined on the basis of simple statics. If the relative stiffness factor is found to be equal or less than 0.5, the footing shall be designed as a flexible member using the foundation modulus approach as described under Section 8.7.2. The relative stiffness shall be determined as

$$k_r = \frac{EI_B}{E_s B^3} \qquad \textbf{(Equation 8-9)}$$

where

k_r = Relative stiffness.

E = Modulus of elasticity of the material used in the superstructure.

E_s = Modulus of elasticity of soil.

B = Base width of foundation perpendicular to direction of interest.

I_B = Moment of inertia per one unit width of the superstructure.

An approximate value for the flexural rigidity of structure and footing, EI_B, for one unit width of the structure can be obtained by adding the flexural rigidity for footing, E_fI_f, flexural rigidity for each member in the superstructure, EI_b, and flexural rigidity for shear walls, $Eah^3/12$ as follows

$$EI_B = E_f I_f + \sum EI_{bi} + \sum \frac{Eah^3}{12}$$ **(Equation 8-10)**

where

h = Wall height.

a = Wall thickness.

E_f = Modulus of elasticity for footing.

I_{bi} = Moment of inertia for any member making up the frame resistance perpendicular to B.

I_f = Moment of inertia per one unit width of the foundation.

8.7.1.2 **Rigidity check based on column spacing**. If the average of two adjacent spans in a continuous strip having adjacent loads and column spacings that vary by not more than 20 percent of the greater value, and is less than $1.75/\lambda$, the footing can be considered rigid and the variation of soil pressure shall be determined on the basis of simple statics. The characteristic coefficient λ is given by

$$\lambda = \sqrt[4]{\frac{k_s b}{4E_c I}}$$ **(Equation 8-11)**

where:

b = Width of continuous footing or a strip of mat between centers of adjacent bays.

E_c = Modulus of elasticity of concrete.

I = Moment of inertia of the strip of width b.

k_s = Modulus of subgrade reaction of soil.

If the average length of two adjacent spans as limited above is greater than $1.75/\lambda$, the beam-on-elastic foundation method noted in Section 8.7.2 shall be used. For general cases falling outside the limitations given above, the critical spacing at which the subgrade modulus theory becomes effective shall be determined individually.

8.7.2 **Design for flexible footings.** A flexible continuous footing (either isolated or taken from a mat) shall be analyzed as a beam-on-elastic foundation. Thickness shall be established on the basis of allowable wide beam or punching shear without use of shear reinforcement.

The evaluation of moments and shears can be simplified from the procedure involved in the classical theory of a beam supported by subgrade reactions, if the footing meets the following basic requirements

1. The minimum number of bays is three.

2. The variation in adjacent column loads is not greater than 20 percent.

3. The variation in adjacent spans is not greater than 20 percent.

4. The average length of adjacent spans is between the limits $1.75/\lambda$ and $3.50/\lambda$.

If these limitations are met, the contact pressures can be assumed to vary linearly, with the maximum value under the columns and a minimum value at the center of each bay.

SECTION 8.8
GRID FOUNDATIONS

8.8.1 Grid foundations shall be designed and constructed in accordance with provisions of Sections 8.7. Grid foundations shall be analyzed as independent strips using column loads proportioned in direct ratio to the stiffness of the strips acting in each direction.

SECTION 8.9
MAT FOUNDATIONS

8.9.1 **General**. Mats shall be designed and constructed in accordance with Sections 8.9.1 through 8.9.3. Mats may be designed and analyzed as either rigid bodies or as flexible plates supported by an elastic foundation (the soil). In the analysis and design of mats a number of factors shall be considered that include, but need not be limited to, the following:

1. Reliability of proposed value for the modulus of subgrade reaction obtained in accordance with Section 8.4.1.3.2.

2. Finite soil-strata thickness and variations in soil properties both horizontally and vertically.

3. Mat shape.

4. Variety of superstructure loads and assumptions in their development.

5. Effect of superstructure stiffness on mat and vice versa.

The design and construction of mats shall be under the direct supervision of a registered design professional having sufficient knowledge and experience in foundation slab engineering, who shall certify to the building official that the mats as constructed satisfy the design criteria.

8.9.2 **Excavation heaves**. The influence of heave on subgrade response shall be determined by a geotechnical engineer. Recovery of the heave remaining after placing the mat shall be treated as either a recompression or as an elastic problem. If the problem is analyzed as a recompression problem, the subsurface response related to recompression shall be obtained by a geotechnical engineer. The subsurface response may be in the form of a recompression index or deflections computed by the geotechnical engineer based on elastic and consolidation subsurface behavior.

8.9.3 **Design.** A mat may be designed using the Strength Design Method of SBC 304. Analyses and designs using computer programs shall be permitted provided design assumptions, user input, and computer-generated output are submitted. The mat plan shall be proportioned using unfactored loads and any overturning moments. The pressure diagram is considered linear and computed from Equation 8-2 and shall be less than allowable load bearing. Loads shall include the effect of any column moments and any overturning moment due to wind or other effects. Any moments applied to the mat from columns or overturning, etc., shall be included when computing the eccentricity.

The contact pressure shall not exceed the allowable load bearing determined from Chapter 4. The allowable soil pressure may be furnished as one or more values depending on long-term loading or including transient loads such as wind. The soil pressure furnished by the geotechnical engineer shall be factored to a pseudo "ultimate" value by multiplying the allowable pressure by the ratio of the sum of factored design loads to the sum of the unfactored design loads.

8.9.3.1 **Mat thickness** The minimum mat thickness based on punching shear at critical columns shall be computed based on column load and shear perimeter. The depth of the mat shall be found without using shear reinforcement and determined on the basis of diagonal-tension shear as noted in SBC 304 Chapter 15. Investigation of a two-sided (corner column) or three-sided diagonal tension shear perimeter shall be made for columns adjacent to mat edge. An investigation for wide-beam or diagonal tension shall be made for perimeter load-bearing walls.

8.9.3.2 **Rigid design.** It shall be permitted to design mats as rigid body with linear distribution for contact pressure if the mat, superstructure, or both is rigid enough not to allow differential settlement for columns. The reinforcing steel for bending is designed by treating the mat as a rigid body and considering strips both ways, if the following criteria are met:

1. Column spacing is less than $1.75/\lambda$ or the mat is very thick.
2. Relative stiffness k_r as noted in Equation 8-9 is greater than 0.5.
3. Variation in column loads and spacing is not over 20 percent

These strips are analyzed as combined footings with multiple columns loaded with the soil pressure on the strip, and column reactions equal to the factored (or unfactored) loads obtained from the superstructure analysis. Consideration shall be given to the shear transfer between strips to satisfy a vertical load summation.

8.9.3.3 **Flexible design**. For mats not meeting the criteria of Section 8.9.3.2, it shall be designed as a flexible plate in accordance with Sections 8.9.3.3.1 and 8.9.3.3.2.

8.9.3.3.1 **Uniform loads and spacings.** If variation in adjacent column loads and in adjacent spans is not greater than 20 percent it shall be permitted to analyze mats as continuous footings that can be analyzed according to the provisions of Section 8.7.2. The mat shall be divided into strips the width of each is equal to the distance between adjacent bays. Each strip shall be analyzed independently considering column loads in both directions. The contact pressure is equal to the average contact pressure evaluated for each strip in each direction.

8.9.3.3.2 **Nonuniform loads and spacings.** If columns have irregular spacings or loads, mats may be analyzed based on theory of modulus subgrade reaction, elastic, plate method, finite difference method, finite grid method, finite element method, or any other method approved by the building official.

8.9.4 **Circular mats or plates.** For tall structure, differential settlements shall be carefully controlled to avoid toppling when the line of action of gravity forces falls out of the base. The plate depth shall be designed for wide-beam or diagonal-tension shear as appropriate.

8.9.5 **Ring foundations**. For ring foundations used for water-tower structures, transmission towers, television antennas, and various other possible superstructures, analysis and designed shall be carried out using advanced method of analysis and carried out by a registered design profession knowledgeable in geotechnical and structural engineering.

SECTION 8.10
SEISMIC REQUIREMENTS

8.10.1 For combined footings and mats of structures assigned to Seismic Design Category C or D, provisions of SBC 301 and SBC 304 shall apply when not in conflict with the provisions of Chapter 5. Strips between adjacent columns shall be capable of carrying, in tension or compression, a force equal to the product of the larger column load times the seismic coefficient S_{DS} divided by 10 unless it is demonstrated that equivalent restraint is provided by the strips.

CHAPTER 9
DESIGN FOR EXPANSIVE SOILS

SECTION 9.1
GENERAL

9.1.0 **Scope.** Provisions of this chapter shall apply to building foundation systems in expansive soil areas. Foundation design and construction shall be based on a site investigation as defined in Chapter 2, unless the building official ascertains that sufficient data upon which to base the design and construction of the foundation system is available.

9.1.1 **Approval of special systems of design or construction.** Sponsors of any system of design or construction within the scope of Chapter 9, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by Chapter 9, shall have the right to present that data on which their design is based to the building official or to a board of examiners appointed by the building official. This board shall be composed of competent geotechnical and structural engineers and shall have authority to investigate the data so submitted, to require tests, and to formulate rules governing design and construction of such systems to meet the intent of Chapter 9. These rules when approved by the building official and promulgated shall be of the same force as the provisions of Chapter 9.

SECTION 9.2
LOADINGS

9.2.1 Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in Section 2.4 SBC 301. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in SBC 301 Section 4.8, are permitted to be used in designing footings. Strength design of reinforced concrete systems and elements shall comply with load combinations specified in SBC 304.

SECTION 9.3
DESIGN

9.3.0 **SCOPE.** Design for expansive soils shall be in accordance with the provisions of Sections 9.3.1 through 9.3.5. Provisions of Chapters 5 and 8 not specifically excluded and not in conflict with the provisions of Chapter 9 shall apply, where applicable.

9.3.1 **General requirements.** Footings or foundations placed on or within the active zone of expansive soils shall be designed to resist differential volume changes and to prevent structural damage to the supported structure. Deflection and racking of the supported structure shall be limited to that which will not interfere with the usability and serviceability of the structure. Foundations placed below where volume change occurs or below expansive soil shall comply with the following provisions:

1. Foundations extending into or penetrating expansive soils shall be designed to prevent uplift of the supported structure.

2. Foundations penetrating expansive soils shall be designed to resist forces exerted on the foundation due to soil volume changes or shall be isolated from the expansive soil.

Soil investigation report shall indicate the value or range of heave that might take place for the subject structure. Potential soil movement shall be determined based on the estimated depth of the active zone in combination with either of the following:

1. ASTM-D 4546.
2. Any other method which can be documented and defended as a good engineering practice in accordance with the principles of unsaturated soil mechanics carried out by a Geotechnical Engineer and approved by the building official.

9.3.2 **Foundations.** Footings or foundations for buildings and structures founded on expansive soil areas shall be designed in accordance with Sections 9.3.2.1, 9.3.2.2, or 9.3.2.3. Alternate foundation designs shall be permitted subject to the provisions of Section 9.1.1. Footing or foundation design need not comply with Section 9.3.2.1, 9.3.2.2, or 9.3.2.3 where the soil is removed in accordance with Section 9.3.3, nor where the building official approves stabilization of the soil in accordance with Section 9.3.4, nor where the superstructure is design by a registered design professional to accommodate the potential heave.

9.3.2.1 **Shallow foundations.** Continuous or spread footings shall not be used on expansive soils unless the soil deposit has a low expansion potential, as determined in accordance with Table 9.1 or the superstructure is designed to account for the potential foundation movement. The uplift pressures on the sides of the footing shall be minimized to the extent possible.

For continuous footings, the swell pressure shall be counteracted without exceeding the bearing capacity of the soil deposit by narrowing the width of the strip footing and/or providing void spaces within the supporting beam or wall. The continuous foundation shall be stiffened by increasing the reinforcement around the perimeter and into the floor slab.

For spread footings, a void space shall be provided beneath the grade beams using the same technique as described for pier and grade beam construction in Section 9.3.2.3. The footings shall be designed using as high bearing pressure, as practicable.

TABLE 9.1
CLASSIFICATION OF EXPANSION POTENTIAL

Expansion Index (EI)[a]	Expansion Potential
0 – 20	Very low
21– 50	Low
51 – 90	Medium
91 –130	High
> 130	Very high

a. EI = {1000 × (final height of specimen – initial height of specimen)/initial height of specimen}, as per ASTM D 4829.

9.3.2.2 **Slab-on-grade foundations.** Slab-on-grade (Slab-on-ground) foundations on expansive soils shall be designed and constructed in accordance with WRI/CRSI *Design of Slab-on-Ground Foundations*.

A conventionally reinforced slab-on-grade foundation shall conform to applicable provisions of SBC 304, where applicable. All variables affecting finished-slab performance shall be considered when selecting a slab type and when specifying or executing a slab design. All slab-on-grade foundations, with the exception of conventionally reinforced slabs less than 50 m^2, shall be designed by a registered design professional having sufficient knowledge and experience in structural and foundation engineering. Design of slab shall be conducted for conditions of both center and edge heave. Construction joints shall be placed at intervals not exceeding 4.5 m.

Exception: Slab-on-grade systems that have performed adequately in soil conditions similar to those encountered at the building site are permitted subject to the approval of the building official.

9.3.2.3 **Beam-on-drilled pier.** The design provisions of Chapter 17 shall be expanded to include, but need not be limited to, the requirements of Sections 9.3.2.3.1 and 9.3.2.3.2.

9.3.2.3.1 **General requirements.**

1. A void space shall be maintained beneath the grade beam between the piers. The required void space shall be determined based on the predicted heave of the soil beneath the beam but shall not be less than 150 mm.
2. Care shall be taken in the design to provide for sealing the space between the soil and the pier, such that deep seated heave that may result from water gaining access to soils below active zone along the shaft of the pier, is prevented.
3. Sufficient field penetration resistance tests shall be performed not only to establish the proper friction value but also to ensure that soft soils are not the cause of tensile forces developed in the pier.
4. The upper 1.5 m of soil around the pier shall be excluded when calculating the pier load capacity.
5. Friction piers shall not be used at sites where groundwater table is either high or expected to become high in the future.
6. Uplift skin friction shall be permitted to be assumed constant throughout the active zone.
7. Where the upper soils are highly expansive or if there is a possibility of loss of skin friction along the lower anchorage portion of the shaft due to rise of groundwater table, the bottom of the shaft shall be belled or under-reamed. The vertical side shall be a minimum of 150 mm high and the sloping sides of the bell shall be formed at either 60° or 45°. For piers founded well below the active zone, the shaft may not be under-reamed.
8. Upward movement of the top of the pier and the tensile forces developed in the pier shall be considered in the design of drilled piers.

9. Mushrooming of the pier near the top shall be avoided. Cylindrical cardboard at/or extended above the top of the concrete shall be used to prevent formation of mushroomed piers.

9.3.2.3.2 **Reinforcement**. Reinforcing steel shall extend the entire length of the pier and shall be hooked into the belled bottom, if used, and into the grade beam at the top. The area of the steel shall be designed to resist all tensile loads to which the pier may be subjected but shall not be less than a minimum of 1 percent of the cross-sectional area of the pier.

9.3.3 **Removal of expansive soil**. Where expansive soil is removed in lieu of designing footings or foundations in accordance with Section 9.3.2.1, 9.3.2.2, or 9.3.2.3, the soil shall be removed to a depth sufficient to ensure constant moisture content in the remaining soil. Fill material shall not contain expansive soils and shall comply with Sections 3.6 and 3.10 or 3.11. If the expansive strata are not entirely removed, the fill material shall be impermeable enough not to provide access for water into expansive grades or foundation soils.

Exception: Expansive soil need not be removed to the depth of constant moisture, provided the confining pressure in the expansive soil created by the fill and supported structure exceeds the swell pressure.

9.3.4 **Stabilization.** Where the active zone of expansive soils is stabilized in lieu of designing footings or foundations in accordance with Section 9.3.2.1, 9.3.2.2, or 9.3.2.3, the soil shall be stabilized by chemical, installation of moisture barriers, pre-wetting or other techniques designed by a geotechnical engineer knowledgeable in unsaturated soil mechanics and approved by the building official. In pre-wetting technique, the effect of strength loss shall be evaluated to ensure that strength criteria are met. Limitations and implementation procedures of the contemplated stabilization technique shall receive careful consideration and thorough evaluation.

9.3.5 **Required preventive measures.** Applicable provisions of Chapter 13 shall be expanded to include, but need not be limited to, the following:

1. All water-supply pipes and wastewater pipes shall be watertight and have flexible connections and couplings.
2. All rainwater pipes shall be ducted well away from the foundations. It shall be made sure that all water from downspout is discharged away from the building into storm sewer or suitable ground surface location downhill.
3. The ground surface shall slope away from the structure. Bare or paved areas shall have a slope not less than 2 %, and if possible the ground surface within 3 meters of the structure shall be sloped at a 10 percent grade.
4. Storage tanks and septic tanks shall be reinforced to minimize cracking and have adequate flexible waterproofing as per section 13.5.
5. Plants and irrigation systems shall not be placed immediately adjacent to the structure and spray heads shall be directed away from the structure. Large trees and bushes shall be kept away from the foundations for a distance greater than half their mature height.
6. If horizontal moisture barriers are installed around the building to move edge effects away from the foundation and minimize seasonal fluctuations of water

content directly below the structure, care shall be taken to seal joints, seams, rips, or holes in the barrier. Horizontal moisture barriers may take different forms including, but not necessarily limited to, membranes, rigid paving (concrete aprons, etc.), or flexible paving (asphalt membranes, etc.).

7. If vertical moisture barriers are used around the perimeter of the building they shall be installed at least one meter from the foundation to a depth equal to or greater than the depth of seasonal moisture variation (active zone). Buried vertical barriers may consist of polyvinyl chloride, polyethylene, polymer-modified asphalt or any other approved methods or materials.

8. If the structure has a basement, the backfill shall consist of non-expansive soils and it shall comply with Sections 3.6 and 3.10 or 3.11.

SECTION 9.4
PRE-CONSTRUCTION INSPECTIONS

9.4.1 A pre-construction site inspection shall be performed to verify the following:

1. Vegetation and associated root systems have been removed from the construction site.

2. No beam trench cuttings or scarified material have been placed as fill material.

3. All fill has been placed in accordance with Sections 3.6 and 3.10 or 3.11 in any portions or sections of the foundation supporting grade.

4. Proper soil compaction of the foundation footprint and fill material has been performed to a minimum of 95 percent standard proctor density.

SECTION 9.5
INSPECTION PRIOR TO PLACEMENT OF CONCRETE

9.5.1 Prior to the placement of concrete, an inspection of the beam geometrics, penetrations, cable(s), cable(s) anchorage/steel placements and other details of the design shall be made to verify conformance with the design plans.

SECTION 9.6
CONCRETE

9.6.1 Material, construction, and placement of concrete shall be in accordance with the provisions of Section 5.4.2 and Section 8.3.

CHAPTER 10
DESIGN FOR COLLAPSIBLE SOILS

SECTION 10.1
GENERAL

10.1.0 **Scope**. Provisions of this chapter shall apply to building foundation systems on collapsible soil areas. Foundation design and construction shall be based on a site investigation as defined in Chapter 2, unless the building official ascertains that sufficient data upon which to base the design and construction of the foundation system is available.

10.1.1 **Approval of special systems of design or construction.** Sponsors of any system of design or construction within the scope of Chapter 10, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by Chapter 10, shall have the right to present that data on which their design is based to the building official or to a board of examiners appointed by the building official. This board shall be composed of competent geotechnical and structural engineers and shall have authority to investigate the data so submitted, to require tests, and to formulate rules governing design and construction of such systems to meet the intent of Chapter 10. These rules when approved by the building official and promulgated shall be of the same force as the provisions of Chapter 10.

SECTION 10.2
LOADINGS

10.2.1 Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in Section 2.4 SBC 301. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in SBC 301 Section 4.8, are permitted to be used in designing footings. Strength design of reinforced concrete systems and elements shall comply with load combinations specified in SBC 304.

SECTION 10.3
DESIGN

10.3.0 Design for collapsible soils shall be in accordance with the provisions of Sections 10.3.1 through 10.3.3. Provisions of Chapters 5 and 8 not specifically excluded and not in conflict with the provisions of Chapter 10 shall apply, where applicable.

10.3.1 **Foundations.** Footings or foundations for buildings and structures founded on collapsible soil areas shall be designed in accordance with Sections 10.3.1.1 through 10.3.1.4 Alternate foundation designs shall be permitted subject to the provisions of Section 10.1.1. Footing or foundation design need not comply with Sections 10.3.1.1 and 10.3.1.2 where the soil is removed in accordance with Section 10.3.2, nor where the building official approves stabilization of the soil in accordance with Section 10.3.3, nor where the superstructure is designed by a registered design professional to accommodate the potential settlements.

10.3.1.1 **Classification of collapse potential.** Collapse potential shall be permitted to be classified in accordance with one of the methods prescribed in Sections 10.3.1.1.1, 10.3.1.1.2, or 10.3.1.1.3.

10.3.1.1.1 **Collapse index method.** The collapsibility of a particular soil under specified conditions could be determined in accordance with ASTM D5333. The specimen collapse shall be classified according to the collapse index, I_e, as set forth in Table 10.1.

10.3.1.1.2 **Standard plate load test method**. Where undisturbed soil specimens are irretrievable, collapse potential for specific field conditions could be estimated from standard plate load tests (SPLT), conducted in test pit under unsoaked and soaked conditions in accordance with ASTM D1194.

10.3.1.1.3 **BREA infiltration and plate load test method.** Collapse potential could be determined in accordance with BREA *Building Regulations in Eastern Arriyadh Sensitive Soils* procedures (BPLT). The procedures shall apply to tests performed in test pits or trenches. The infiltration field test shall be performed in accordance with the procedure set forth in Table 10.2 and the field plate load test shall be carried following the procedure outlined in Table 10.3. The stability of the reaction column and side-wall of the test pit shall be considered, particularly for test pits deeper than 4 meters.

10.3.1.1.3.1 **Design curve construction.** A design curve for the site shall be constructed in accordance with the steps outlined in Table 10.4. A data sheet in the form shown in Table 10.4 may be used for the raw data gathered during the test and for the reduced data.

10.3.1.2 **Design procedure**. Based on which method used in estimating the collapse potential of the soil deposit as provided in Sections 10.3.1.1.1, 10.3.1.1.2, or 10.3.1.1.3, design for collapsible soils shall be in accordance with Sections 10.3.1.2.1, 10.3.1.2.2, or 10.3.1.2.3, respectively.

10.3.1.2.1 **Design based on collapse index.** Potential settlement that may occur in a soil layer under the applied vertical stress is obtained as follows

$$\rho = \frac{H}{100} I_c \qquad \textbf{(Equation 10-1)}$$

where:

H = Thickness of the soil layer.

I_c = Collapse potential, determined using a predetermined applied vertical stress applied to a soil specimen taken from the soil layer as follows.

$$I_c = \frac{d_f - d_i}{h_o} \times 100 \qquad \textbf{(Equation 10-2)}$$

where:

d_i = Specimen height at the appropriate stress level before wetting.

d_f = Specimen height at the appropriate stress level after wetting.

h_o = Initial specimen height.

Based on settlement value determined by Equation 10-1, the foundation system shall be designed in accordance with the provisions of Chapters 5 and 8, where applicable.

Limitations. Amount of settlement depends on the extent of wetting front and availability of water, which can rarely be predicted prior to collapse. Prediction of settlement based on collapse potential shall be viewed and interpreted accordingly.

TABLE 10.1
CLASSIFICATION OF COLLAPSE POTENTIAL

Collapse index (I_e)[a] percent	Degree of Specimen Collapse
0	None
0.1-2.0	Slight
2.1-6.0	Moderate
6.1-10.0	Moderately severe
>10	Severe

[a] $I_e = \frac{\Delta e}{1+e_o}100$, where Δe = change in void ratio resulting from wetting, and e_o = initial void ratio.

TABLE 10.2
BREA INFILTRATION FIELD TEST PROCEDURE

1. Excavate a trench or test pit to the desired depth of testing and provide a smooth flat surface for testing. Do not backfill to achieve smoothness.
2. At a distance no less than 3 plate diameters (3D) from the trench or test pit excavated in step 1, excavate a shallow infiltration pit to a depth of 60 to 100 mm and a diameter of 2D. This pit for the preliminary rate-of-infiltration test shall be separated by 3D from the supports of the reference beam. Measure the depth of the dry infiltration pit at the center.
3. Fill the infiltration pit with water and note the time at which wetting was commenced. Add water during infiltration as needed to keep the bottom of the pit covered.
4. After an infiltration time, t_p, of about 10 to 20 minutes, remove the excess water from the test pit, quickly excavate at the center of the pit to locate the depth of wetting, and measure down to the wetting front. The depth of wetting from the preliminary infiltration test (Z_{wp}) is equal to depth of wetting front minus the original depth of the dry pit.
5. The infiltration coefficient for the preliminary test (C_{ip}) is computed as

$C_{ip} = Z_{wp}/(t_p)^{1/2}$	Where: Z_{wp} = depth of infiltration for the preliminary infiltration test, mm. C_{ip} = infiltration coefficient for the preliminary infiltration test, mm/min$^{1/2}$. t_p = time duration of infiltration for the preliminary infiltration test, min.

TABLE 10.3
BREA PLATE LOAD TEST PROCEDURE (BPLT)

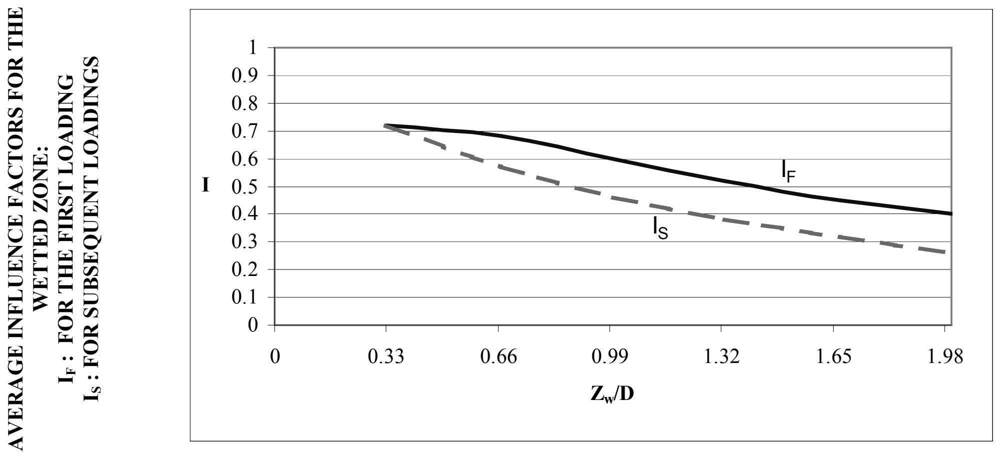
1. For the BPLT, choose the target depth of infiltration (Z_{tar}), equal 0.5D.
2. Compute target time of infiltration (t_{tar}) from: $t_{tar} = (Z_{tar}/C_{ip})^2$
3. Place the loading plate on a smooth flat surface and twist and tap lightly. The bottom of the loading plate may be coated with 5-10 mm of quick setting epoxy before placing it on the soil.
4. Construct a beam to hold water in preparation for ponding. The outside diameter of the ponded water shall be about 2D.
5. Install the reference beam to rest on firm supports located at least 3D from center of loading plate.
6. Attach displacement gauges so that they touch the loading plate on opposite sides and approximately equidistant from the center of the plate.
7. Install the loading jack and reaction column.
8. Apply a seating load of 3 to 8 kPa and zero the displacement gauges. It may be convenient to use the weight of the loading jack and plate as the seating load.
9. Increase load to 15 kPa. Wait one minute and take displacement readings.
10. Commence wetting and note starting time. Maintain water level 10 to 20 mm above the top of the plate.
11. Continue wetting until t_{tar} (computed from step 2) has elapsed. Read displacement gauges, note time and increase load to 40 kPa.
12. Wait Δt minutes, read displacement gauges, note time, increase load to 100 kPa. The time increment Δt may

be chosen as the larger of 2 minutes or $0.1t_{tar}$. For convenience, Δt may be rounded to the nearest minute.

13. Wait Δt minutes, read displacement gauges, note time, increase load to 200 kPa.
14. Wait Δt minutes, read displacement gauges, note time, increase load to 400 kPa.
15. Wait Δt minutes, read displacement gauges, note time, remove load from the plate, remove the plate and quickly excavate to determine the final depth (Z_{wfinal}) and note the corresponding time t_{final}.

TABLE 10.4
DATA REDUCTION FOR DESIGN CURVE CONSTRUCTION

1. In column-1, description of the test stage. This will give meaning to column-2 (time column).
2. In column-2, time shall be recorded.
3. In column-3, elapsed time (t_w) shall be computed. Elapsed time is set equal to zero when ponding is commenced.
4. In column-4, pressure reading on the jack shall be recorded. A load cell could be substituted for the pressure gauge on the jack.
5. In column-5, the added load on the plate shall be recorded.
6. In column-6, the total load on the plate shall be recorded. This is obtained by adding column-5 to the weight (in kN) of the jack and the loading plate.
7. In column-7, the left and right displacement gauges readings shall be recorded.
8. In column-8, the displacement, ΔH, for the left and right gauges shall be recorded. They are obtained by subtracting the initial gauge readings at seating load from each subsequent reading.
9. In column-9, the average displacement (ΔH_{ave}) is obtained by averaging the ΔH values from the left and right reading in column-8.
10. In column-10, the depth of wetting (Z_w) shall be computed by first determining C_{itest} from: $C_{itest} = (Z_{wfinal})/(t_{final})^{1/2}$ and then use it with t_w to get Z_w from: $Z_w = (C_{itest})(t_w)$ ½ .
11. In column-11, Z_w/D shall be computed (column-10 divided by plate diameter).
12. In column-12, the influence factors I_F and I_S obtained from the below figure at Z_w/D shall be recorded.
13. In column-13, the contact pressure (q_{con}) shall be calculated by dividing column-6 by the plate area.
14. In column-14, the average stress within the wetted zone (q_{ave}) shall be calculated by multiplying column-13 (q_{con}) by column-12 (I_F for first loading or I_S for subsequent loadings).
15. In column-15, the average strain (ε_{ave}) shall be computed by dividing column-9 (ΔH_{ave}) by column-10 (Z_w) and multiplying the result by 100.
16. Plot q_{ave} versus ε_{ave} for different tests on the same diagram. Obtain the average of all tests and sketch a DESIGN CURVE.



Data sheet and computations

Date: **Job No.:** **Test Location:** **Test Pit No.:**	**Depth of Test Pit:** **Ground Surface Elevation:** **Z_{wp}:** **t_p:**	**C_{ip}:** **Z_{tar}:** **t_{tar}:** **Δ t:**

(1) Stage	(2) Time (min.)	(3) Elapsed time, t_w (min.)	(4) Pressure reading on Jack	(5) Added load (kN)	(6) Total load on plate (kN)	(7) Dial gauge reading (mm)		(8) Displacement (mm)		(9) ΔH_{ave} (mm)	(10) Z_w (mm)	(11) Z_w/D	(12) I_F or I_S	(13) q_{con} (kPa)	(14) q_{ave} (kPa)	(15) ε_{ave} (percent)
						Left	Right	Left	Right							
Seating load applied																
Dry loading																
Wetting commenced																
etc.																

10.3.1.2.2 **Design based on SPLT**. From the load-deformation curve obtained from standard plate load test under soaked condition in accordance with Section 10.3.1.1.2, the allowable load bearing is taken equal to the value corresponding to settlement of test plate determined from Equation 10-3 as follows

$$\delta_P = \frac{1}{4}\left(1+\frac{B_0}{B_p}\right)^2 \delta_D \qquad \textbf{(Equation 10-3)}$$

where:

δ_p = Settlement of test plate.

δ_D = Design settlement of prototype foundation taken to be equal to half the allowable settlement value given from Section 5.4.1.5.

B_o = Width of prototype footing.

B_p = Width of test plate.

Based on the obtained allowable load bearing, the foundation system shall be designed in accordance with the provisions of Chapters 5 and 8, where applicable.

Limitations. In determining the bearing pressure for the specified tolerable differential settlement, the validity and accuracy of any resulting computations shall be approved by the building official and evaluated on the basis of the following variables:

1. Dependence of the amount of settlement on the extent of the wetting front and availability of water, which can rarely be predicted prior to collapse.

2. The influence depth set to be four times the footing width is significantly different for the model versus prototype footing.

3. Increase soil stiffness due to increased confinement with depth.

10.3.1.2.3 **Design based on BPLT.** Design of spread and strip footings shall conform to the provisions of Section 10.3.1.2.3.1 and mats shall be designed in accordance with the provisions of Section 10.3.1.2.3.2.

10.3.1.2.3.1 **Spread and continuous footings.** Spread and continuous footings are permitted to be used without modifications in areas with low collapse potential, as determined in accordance with Table 10.5. In areas with higher collapse potential, strip footings are permitted, provided that the requirements for additional distortion resistance specified in Table 10.8 are met.

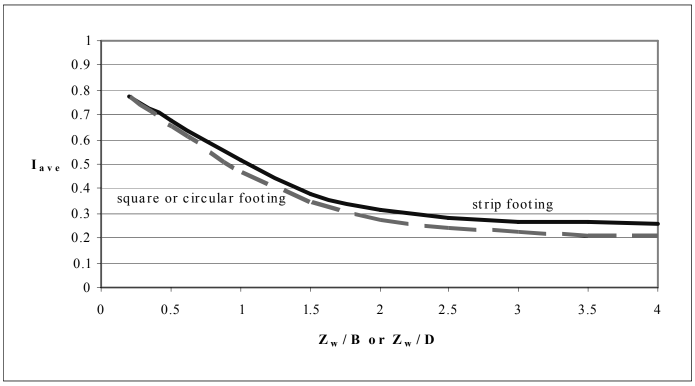
10.3.1.2.3.2 **Stiffened mat foundations.** The design procedure for mat foundations in collapsible soils is summarized in Table 10.9. The mat shall be designed and constructed in accordance with the provisions of Chapter 8, where applicable, and the requirements for additional distortion resistance specified in Table 10.8 shall be met.

10.3.2 **Removal of collapsible soil**. Where collapsible soil is removed in lieu of designing footings or foundations in accordance with Section 10.3.1, the soil shall be removed to a sufficient depth to ensure constant moisture content in the remaining soil. Fill material shall not contain collapsible soils and shall comply with the provisions of Sections 3.6 and 3.10 or 3.11.

TABLE 10.5
DESIGN OF SPREAD AND STRIP FOOTINGS ON COLLAPSIBLE SOILS

1. From Table 10.6, use strain under the plate corresponding to stress of 100 kPa (ε_{100}) to classify the site with respect to collapse potential.
2. Pre-wetting is required for very high collapse potential, and permitted but not required for high collapse potential.
3. When pre-wetting is chosen or required, the DESIGN CURVE constructed in Table 10.4 is replaced with a recompression design curve whose strain values are everywhere 15 percent of those on the original design curve, and the site is reclassified accordingly.
4. Use Tables 10.6 and 10.7 to obtain required design parameters D_f (foundation depth), D_{wdes} (depth of wetted bulb), foundation type, required distortion resistance, ΔH_{diff} /L and $\Delta H_{diff} / \Delta H_{tot}$.
5. From Table 10.7 use column spacing, L, to compute ΔH_{diff} , then compute ΔH_{tot} from the same table.
6. Compute $Z_w = D_{wdes} - D_{f.}$
7. Compute maximum allowable $\varepsilon_{ave} = \Delta H_{tot} / Z_{w.}$
8. Compute the overburden pressure (P_{odes}) at D_{wdes}.
9. From the DESIGN CURVE get allowable ε_{ave} corresponding to P_{odes}.
10. If $\varepsilon_{ave} \geq$ allowable ε_{ave} from step-7, change D_f, foundation type or stiffness level and recalculate.
11. If $\varepsilon_{ave} <$ allowable ε_{ave} , use allowable ε_{ave} in the DESIGN CURVE to get allowable P_{tot}, then find q_{ave} as follows: $q_{ave} =$ allowable $P_{tot} - P_{odes}$
12. Assume a first trial value of B (footing width). Compute Z_w/B, find I_{ave} from the Figure below and compute q_{all} as follows: q_{all} = allowable q_{ave}/I_{ave}
13. Use the trial value of B, footing shape and column load to compute q_{con}. If $q_{con} \approx q_{all}$, then B value is accepted, otherwise change B and iterate until convergence, then proceed with structural design.
14. In case of no convergence or if B or q_{all} are not acceptable, increase D_f, change footing type, change distortion stiffness or a combination of those.

INFLUENCE FACTOR FOR AVERAGE STRESS WITHIN WETTED ZONE VS. DEPTH OF WETTED ZONE



10.3.3 **Stabilization**. Where collapsible soils are stabilized in lieu of designing footings or foundations in accordance with Section 10.3.1, the soil shall be stabilized by compaction, pre-wetting, vibroflotation, chemical, or other techniques designed by a geotechnical engineer knowledgeable in unsaturated soil mechanics and approved by the building official. The provisions of Section 9.3.5 shall also be considered, where applicable. In pre-wetting technique, the effect of strength loss shall be evaluated to ensure that strength criteria are met. Great care must be exercised when using pre-wetting near existing structures that underlain by collapsible soils, particularly if the soil has strong stratification, as in the case of many alluvial soils, and injected water may flow horizontally more than it does vertically. Limitations and implementation procedures of the contemplated stabilization technique shall receive careful consideration and thorough evaluation.

TABLE 10.6
MINIMUM DESIGN PARAMETERS AS A FUNCTION OF COLLAPSE POTENTIAL

ε_{100} (percent)	Collapse potential	Minimum D_f (m)	Minimum design depth of wetting D_{wdes} (m)	Required distortion stiffness	Allowable types of foundation
0 - 0.5	Low	1.0	3.5	Level 0	Spread, Strip, Mat
0.5 - 1.5	Moderate	1.5	3.5	Level I	Strip, Mat
1.5 - 5.0	High	2.0	3.5	Level II	Strip, Mat
>5.0	Very high	2.5	3.5	Level II	Strip, Mat

TABLE 10.7
REQUIRED MINIMUM RATIOS OF DIFFERENTIAL TO TOTAL SETTLEMENT AS A FUNCTION OF FOUNDATION TYPE AND DISTORTION STIFFNESS

Type of foundation	Min. Design $\Delta H_{diff}/L$	$\Delta H_{diff} / \Delta H_{tot}$ as a function of Required Extra Distortion Resistance		
		Level 0	Level I	Level II
Spread	1/500	0.85	0.75	0.65
Strip	1/500	0.65	0.55	0.45
Mat	1/500	0.35	0.30	0.25

TABLE 10.8
REQUIREMENTS FOR EXTRA DISTORTION RESISTANCE

Distortion Stiffness	Type of Foundation	Extra Concrete in Grade Beam	Extra Concrete in Footing	Extra Steel in Footing	Extra Steel in Wall and Floor	Extra Steel in Foundation Column
Level 0	Refers to standard design and requires no extra distortion resistance					
Level I	Spread Footing	10percent higher	10percent thicker	2 bars	–	3 bars
	Strip Footing	10percent higher	10percent thicker	2 bars	–	3 bars
	Mat	–	–	–	15percent, see Figure 10.1	–
Level II	Spread Footing	20percent higher	20percent thicker	3 bars	–	6 bars
	Strip Footing	20percent higher	20percent thicker	3 bars	–	6 bars
	Mat	–	–	–	25percent, see Figure 10.1	–

SECTION 10.4
INSPECTIONS

10.4.1 A pre-construction site inspection shall be conducted to verify that the provisions of Section 9.4 have been met.

SECTION 10.5
CONCRETE

10.5.1 Material, construction, and placement of concrete shall be in accordance with the provisions of Section 5.4.2 and Section 8.3. Prior to the placement of concrete, an inspection of the beam geometrics, reinforcements and other details of the design shall be made to verify conformance with the design plans.

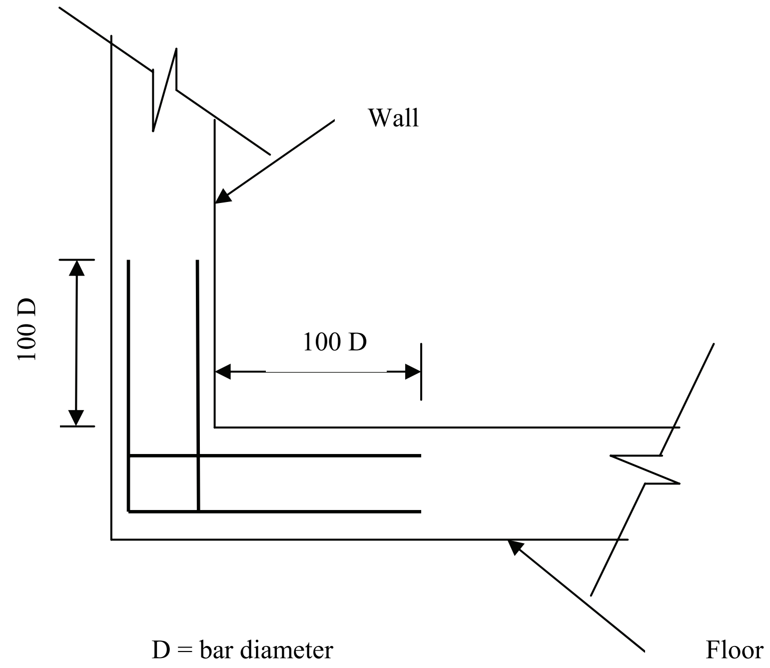


FIGURE 10.1
EXTRA STEEL IN WALL AND FLOOR FOR MAT FOUNDATION

TABLE 10.9
DESIGN OF MAT FOUNDATION ON COLLAPSIBLE SOILS

1. From Table 10.6, use strain under the plate corresponding to stress of 100 kPa (ε_{100}) to classify the site with respect to collapse potential.
2. Pre-wetting is required for very high collapse potential, and permitted but not required for high collapse potential.
3. When pre-wetting is chosen or required, the DESIGN CURVE is replaced with a recompression design curve whose strain values are everywhere 15percent of those on the original design curve, and the site is reclassified accordingly.
4. Use Tables 10.6 and 10.7 to obtain required design parameters D_f (foundation depth), D_{wdes} (depth of wetted bulb), foundation type, required distortion resistance, $\Delta H_{diff}/L$ and $\Delta H_{diff} / \Delta H_{tot}$.
5. Compute ΔH_{tot} .
6. Compute $Z_w = D_{wdes} - D_f$.
7. Compute maximum allowable $\varepsilon_{ave} = \Delta H_{tot} / Z_w$.
8. Compute the overburden pressure (P_{odes}) at 1/3 of the way from the base of the foundation to D_{wdes}. For mat foundations, only a fraction of the P_{odes} acts on the soil during wetting as seen from Table below.
9. The average contact stress under a mat, q_{con}, is governed by the weight of the structure including the mat and the footprint of the structure. Only D_f and distortion stiffness can be changed in pursuit of an acceptable design.
10. Compute q_{con}, Z_w/B and estimate I_{ave} from Table 10.5, using the curve for square footing or interpolate between the curves as a function of the shape of structure in plan (length/width $\geq$ 4 can be interpreted as strip).
11. Compute $q_{ave} = I_{ave} \times q_{con}$
12. Compute $P_{odes} = a \times P_o$, where the factor 'a' represent percentage of overburden stress acting on the wetted soil under mats and is obtained as

Z_w/B	factor a
0 - 0.1	0.1
0.1 - 0.3	0.3
0.3 - 0.6	0.5

13. Compute $P_{tot} = q_{ave} + P_{odes}$
14. Enter the DESIGN CURVE to get ε_{ave} and compute $\Delta H_{tot} = \varepsilon_{ave} \times Z_w$
15. If ΔH_{tot} < allowable ΔH_{tot} from above, the design is acceptable. Proceed with structural design.
16. If ΔH_{tot} > allowable ΔH_{tot} from above, increase D_f and/or increase distortion stiffness and recalculate ΔH_{tot} .

Note: $P_{odes} = aP_o$ = That portion of the overburden stress assumed to produce strain upon wetting.

CHAPTER 11
DESIGN FOR SABKHA SOILS

SECTION 11.1
GENERAL

11.1.0 **SCOPE**. Provisions of this chapter shall apply to building foundation systems in sabkha soil areas. Foundation design and construction shall be based on a site investigation as defined in Chapter 2, unless the building official ascertains that sufficient data upon which to base the design and construction of the foundation system is available.

11.1.1 **Approval of special systems of design or construction.** Sponsors of any system of design or construction within the scope of Chapter 11, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by Chapter 11, shall have the right to present that data on which their design is based to the building official or to a board of examiners appointed by the building official. This board shall be composed of competent geotechnical and structural engineers and shall have authority to investigate the data so submitted, to require tests, and to formulate rules governing design and construction of such systems to meet the intent of Chapter 11. These rules when approved by the building official and promulgated shall be of the same force as the provisions of Chapter 11.

SECTION 11.2
LOADINGS

11.2.1 Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in SBC 301 Section 2.4. The dead load shall include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in SBC 301 Section 4.8, are permitted to be used in designing footings. Strength design of reinforced concrete systems and elements shall comply with load combinations specified in SBC 304.

SECTION 11.3
DESIGN

11.3.0 Design for sabkha soils shall conform to the provisions of Sections 11.3.1 through 11.3.2. Provisions of Chapters 5 and 8 not specifically excluded and not in conflict with the provisions of Chapter 11 shall apply, where applicable.

11.3.1 **General requirements.** Footings or foundations placed on or within sabkha soils shall be designed to prevent structural damage to the supported structure due to detrimental settlement. Deflection and racking of the supported structure shall be limited to that which will not interfere with the usability and serviceability of the structure. Design shall consider, but need not limited to, the following:

1. The decrease in strength of the surface crust of the sabkha as a result of moisture content increase. This crust shall not be used as a foundation layer.

2. The variation of compressibility characteristics of the site resulting from differences in layer thickness, degree of cementation, and relative density of different locations within the site.

3. Differential settlements and foundation instabilities due to volume changes that accompany hydration and dehydration of gypsum rich layers under the hot and humid conditions.

4. High concentrations of chlorides and sulphates in the sabkha sediments and brines, and the subsequent highly corrosive to both concrete and steel.

Soil investigation report shall indicate the value or range of settlement that might take place for the subject structure. Potential settlements shall be estimated by a method of analysis that can be documented and defended as a good engineering practice and approved by the building official. Allowable settlements shall conform to the requirements of Sections 5.4.1.5 and 8.5, where applicable.

11.3.2 **Foundations.** For heavy structures**,** mat or deep foundations shall be considered, and provisions of Chapter 8 and Chapters 14 through 17 shall govern, where applicable.

For lightly loaded buildings and structures founded on sabkha soil areas, and provided that water table is always kept beneath the foundation level, it shall be permitted to design and construct footings or foundations in accordance with Sections 11.3.2.1 through 11.3.2.3, subject to the approval of building official, and under a direct supervision of a geotechnical engineer knowledgeable in sabkha soils.

Alternate foundation designs shall be permitted subject to the provisions of Section 11.1.1. Footing or foundation design need not comply with Section 11.3.1 and 11.3.2 where the soil is removed in accordance with Section 11.6, nor where the building official approves stabilization of the soil in accordance with Section 11.7, nor where the superstructure is design by a registered design professional to accommodate the potential settlements.

11.3.2.1 **Water table below 5 meters depth**

Where groundwater is 5 meters below the ground surface level, external and internal walls have to be supported by a concrete strip foundation. Water infiltration shall be prevented under the floor slabs by installing heavy duty polythene sheeting, or other approved materials, as shown in Figure 11.1(a). Joints in the polythene sheeting shall be lapped and sealed in accordance with the manufacturer's installation instructions. Strip foundation shall be supported by lean mix concrete to prevent contamination of the wet concrete when poured.

11.3.2.2 **Water table between 2.5 and 5 meters depth**

Where groundwater is between 2.5 meters and 5 meters below the ground surface level, provisions of Section 11.3.2.1 shall be satisfied. Slab floors have to be supported by a strip foundation as illustrated in Figure 11.1(b). Coarse, durable gravels shall be placed beneath the floor slab and around the strip foundation.

11.3.2.3 **Water table between ground level and 2.5 meters depth**

Where groundwater is between ground level and 2.5 meters, the provisions of Section 11.3.2.2 shall be fulfilled. Further, the strip foundation and the floor slab shall also be underlain by a rolled coarse gravel capillary cut-off, not less than 150 mm thick, resting on a compacted fill blanket as illustrated in Figure 11.1(c).

SECTION 11.4
REQUIRED PREVENTIVE MEASURES

11.4.1 The applicable provisions of Section 9.3.5 and Chapter 13 shall be expanded to include, but need not be limited to, the following:

1. Domestic and irrigation water shall be strictly controlled, especially where low density sands cemented with sodium chloride. Protection by drainage around major structures shall be considered to reduce the risks associated with rainstorms or burst water mains.

2. There shall be external protection against corrosion for all pipelines, fittings and valves, whether steel, ductile iron, or asbestos-cement. Ductile iron pipe work shall be factory coated with a bituminous coating compatible with a specified pipe wrapping material. Steel pipe work shall be factory coated with either a thermosetting, fusion bonded, dry powder epoxy coating not less than 300 micrometers thick or a catalyst-cured epoxy coating applied in three coats, to a total cured dry film thickness of 240 micrometers. Ductile iron, steel and asbestos-cement pipe work shall then be wrapped with heavy duty self-adhesive, rubber bitumen compound with PVC carrier strip. The pipe work shall be sleeved with 0.2 mm thick polyethylene sleeving.

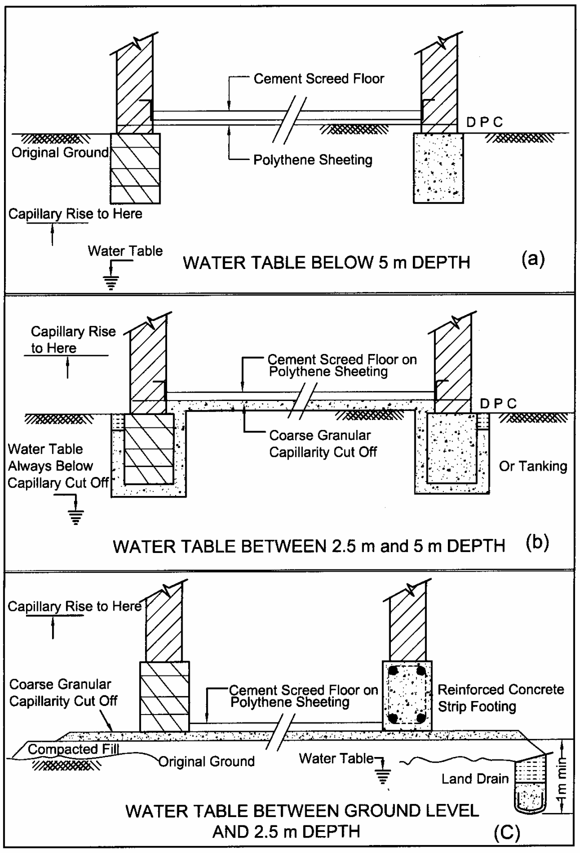


FIGURE 11.1
SHALLOW FOUNDATION DESIGN STRATEGIES
(a) WATER TABLE BELOW 5 m DEPTH;
(b) WATER TABLE BETWEEN 2.5 m AND 5 m DEPTH;
(c) WATER TABLE BETWEEN GROUND LEVEL AND 2.5 m DEPTH.

SECTION 11.5
CONCRETE

11.5.0 Material, construction, and placement of concrete shall be in accordance with the provisions of Section 5.4.2 and Section 8.3, where applicable.

11.5.1 **Concrete protection**. Concrete shall satisfy the durability criteria of SBC 304 Chapter 4. Protection against salt attack on foundation materials, buried pipes, and metal objects shall be provided by using sulphate resistance cement. Concrete used in the construction of foundation on sabkha formations shall be made from Type V Portland cement, with minimum cement content of 370 kg/m^3, and maximum water cement ratio of 0.4 for corrosion protection and 0.45 for sulfate protection. Reinforcement type shall be epoxy coated and a minimum cover to reinforcement of 75 mm shall be stringently enforced.

SECTION 11.6
REMOVAL OF SABKHA SOILS

Where sabkha soil is removed in lieu of designing footings or foundations in accordance with Section 11.3.2, the soil shall be removed to a depth sufficient to ensure adequate load-bearing and tolerable settlement for the remaining soil. Fill material shall not contain sabkha soils and shall comply with the provisions of Sections 3.10 or 3.11.

SECTION 11.7
STABILIZATION

Where the sabkha soil is stabilized in lieu of designing footings or foundations in accordance with Section 11.3.2, the soil shall be stabilized by stone columns, preloading, vibroflotation, or other techniques designed by a geotechnical engineer knowledgeable in sabkha soil and approved by the building official. Where attempts to densify the upper portion of sabkha material by conventional means, in order to improve its bearing capacity and reduce its settlement characteristics, the upper, loose portion of sabkha shall be densified, or treated without adversely affecting the underlying cemented layers. In pre-wetting technique, the effect of strength loss shall be evaluated to ensure that strength criteria are met. Limitations and implementation procedures of the contemplated stabilization technique shall receive careful consideration and thorough evaluation.

CHAPTER 12
DESIGN FOR VIBRATORY LOADS

SECTION 12.1
GENERAL

12.1.1 Where machinery operations or other vibrations are transmitted through the foundation, the foundations and support structures shall be designed according to Sections 12.2 through 12.4.7. Foundations and support structures designed for machinery vibrations must be capable of withstanding dynamic loading due to machinery vibrations and all other loadings to which they may be subjected with stresses not exceeding the allowable-load bearing values specified in Chapter 4.

SECTION 12.2
LOADS AND FORCES

12.2.0 All concrete sections shall be proportioned to resist the sum of the static loads and dynamic forces as described in Sections 12.2.1 through 12.2.3.

12.2.1 **Static loads**. Static loads shall consist of all dead and live loads on the foundation, etc., thermal and fluid forces from process piping, loads due to temperature differentials, wind loads and any other sustained loads.

12.2.2 **Transient dynamic forces.** If not specified by the equipment manufacturer, transient forces consisting of vertical, lateral, and longitudinal forces equal to 25 percent of the total weight of the machine train and acting through the center machine bearing axis shall be used in design. These forces need not be considered to act concurrently. For purposes of strength design, the forces shall be treated as quasi-static loads.

For low-tuned systems, dynamic load effects due to transient resonance during machine start-up or shutdown shall be considered. For transient response calculations, damping effects shall be included to avoid unrealistically high results as the frequency ratio passes through the 0.7 to 1.3 range. Unless foundations or structures or connecting piping are unusual, response due to transient dynamic forces need not be evaluated.

12.2.3 **Steady state dynamic forces.** Information on steady state dynamic forces shall be furnished by the equipment manufacturer(s). For reciprocating machinery the supplied information shall include weights of the machine and all auxiliary equipment with exact location of centers of gravity, number of revolutions per minute (Operating speed or range of operating speeds), diagrams showing all primary and secondary forces and moments, and curves of free forces and moments against crank angle degrees.

For rotating machinery the equipment manufacturer(s) shall supply the weights of the machine, rotor and auxiliary equipment with exact location of centers of gravity, range of operating speeds, possible unbalanced forces and points of application (for operating conditions based on alarm level).

Where there is no manufacturer information available, the steady state dynamic force for rotating machinery can be estimated as follows:

$$F_D = 0.001W(rpm)^{1.5} \qquad \textbf{(Equation 12-1)}$$

where:

F_D = Steady state dynamic force in kN.

W = Total mass of the rotating part in kg.

rpm = Machine speed in revolutions per minute.

SECTION 12.3 SOIL BEARING PRESSURES, PILE CAPACITIES AND SETTLEMENTS

12.3.1 Foundation adequacy for static bearing capacity and settlement considerations shall be checked by a registered design professional. In addition, effect of dynamic loading on foundation soil shall be investigated. In-situ or laboratory testing to establish appropriate dynamic parameters of the foundation soils, whether in-situ treated or untreated, or compacted fill, shall be carried out by an approved agency. If a requirement for piles is established, appropriate dynamic parameters for the piles shall be determined by an approved agency.

The site investigation report shall give insight to the expected dynamic behavior of the soil or piles. As a minimum the report should give the density, Poisson's ratio, dynamic modulus of sub-grade reaction or dynamic pile spring constant and the shear modulus for soils, or the equivalent fixate level of piles.

Unless foundation settlement calculations for dynamic loads show otherwise, the allowable soil bearing pressures shall not exceed 50% of the allowable bearing pressure permitted for static loads, as determined from Chapter 4, for high-tuned foundations and 75% for low-tuned foundations. The allowable soil bearing pressure shall be reduced for heavy machinery foundations to provide a factor of safety against excessive settlement due to vibration.

SECTION 12.4 DESIGN REQUIREMENTS

12.4.0 Foundation and support structures designed for machinery vibrations shall meet the provisions of Sections 12.4.1 through 12.4.7

12.4.1 **General**. The provisions of Chapters 5 and 8 shall be expanded to include the following:

1. Support structures or foundations for centrifugal rotating machinery greater than 500 horsepower shall be designed for the expected dynamic forces using dynamic analysis procedures. For units less than 500 horsepower, in the absence of a detailed dynamic analysis, the foundation weight shall be at least three times the total machinery weight, unless specified otherwise by the Manufacturer.
2. For reciprocating machinery less than 200 horsepower, in the absence of a detailed dynamic analysis, the foundation weight shall be at least five times the total machinery weight, unless specified otherwise by the Manufacturer.
3. All coupled elements of the machinery train shall be mounted on a common foundation or support structure.

4. Foundations for heavy machinery shall be independent of adjacent foundations and buildings. Concrete slabs or paving adjacent to the foundation shall have a minimum 12 mm isolation joint around the foundation using an approved elastic joint filler with sealant on top. Joint filler material shall be an expansion joint material according to ACI 504R Guide for Sealing Joints in Concrete Structures. Preformed expansion joint filler shall be of the full thickness and depth of the joint with splicing only on the length.

5. The clear distance in any direction between adjacent foundations for heavy machinery shall be large enough to avoid transmission of detrimental vibration amplitudes through the surrounding soil or the foundations shall be protected in other ways. Transmissibility of amplitudes shall be limited to 20 percent between adjacent foundations, unless otherwise agreed by the Building official.

6. Where practical and economical, the machine foundation system shall be proportioned to be low-tuned.

7. High-tuned machine foundation systems shall be used only when a low-tuned system is not practical or economical.

8. For elevated machinery, the flexibility of the entire support structure shall be considered in the dynamic analysis.

9. The foundation design shall be capable of resisting all applied dynamic and static loads specified by the machinery manufacturer, loads from thermal movement, dead and live loads, wind or seismic forces as specified in SBC 301, any loads that may be associated with installation or maintenance of the equipment, and fatigue. For fatigue, the dynamic loads shall be increased by a factor of 1.5 and applied as quasi-static loads.

The applied loads shall be combined to produce the most unfavorable effect on the supporting foundations. The effect of both wind and seismic activity need not be considered to act simultaneously. Design load combinations shall be as specified in Section 2.4 SBC 301 except that strength design of reinforced concrete systems and elements shall comply with load combinations specified in SBC 304.

10. Design shall be such that buried cables, pipes etc., will not be incorporated in the foundation, and be protected from the influence of foundation stresses. If incorporation in the foundation cannot be avoided, cables and pipes shall be sleeved.

11. Where practical, operator platforms shall be independent from the main machinery carrying structure(s).

12. Quantifying whole-body vibration in relation to human health and comfort, the probability of vibration perception, and the incidence of motion sickness shall conform to International Organization for Standardization ISO 2631-1 Mechanical Vibration and Shock Evaluation of Human Exposure to Whole-Body Vibration – Part 1: General Requirements and Evaluation of Human Exposure to Whole-Body Vibration – Part 2: Continuous and Shock-induced Vibration in Buildings (1 to 80 Hz) ISO 2631-2.

12.4.2 **Reinforced concrete.** The structural design of all reinforced concrete shall be in accordance with SBC 304 when not in conflict with the provisions of Chapter 12. The following provisions shall be satisfied:

1. The minimum compressive strength of concrete at 28 days shall not be less than 28 MPa.

2. All faces of concrete shall be reinforced bi-axially. For deformed bars, the reinforcement in each direction shall not be less than 0.0018 times the gross area perpendicular to the direction of reinforcement.

Exception: In the event that a foundation size greater than 1200 mm thick is required for stability, rigidity, or damping, the minimum reinforcing steel may be as recommended in ACI 207.2R Effect of Restraint, Volume Change, and Reinforcement on Cracking of Massive Concrete with a suggested minimum reinforcement of Dia 22 mm bars at 300 mm on center.

3. Main reinforcement in piers shall not be less than 1 percent nor more than 8 percent of the cross-sectional area of the piers. Main reinforcement in pedestals shall not be less than 1/2 percent.

4. Minimum tie size in piers shall be 12 mm.

5. Maximum tie spacing in piers shall be the smallest of 8-bar diameters, 24-tie diameters or 1/3 the least dimension of the pier.

6. Slabs with thickness of 500 mm or more shall be provided with shrinkage and temperature reinforcement in accordance with applicable provisions of SBC 304.

7. When foundation thickness is greater than 1200 mm thick, mix and placement of concrete shall conform to the provisions of ACI 207.2R and SBC 304.

12.4.3 **Anchor bolts.** Anchor bolts shall be in accordance with SAES-Q-005. When specified, the diameter, steel quality, projection and installation method shall be as required by the machine manufacturer. Requirements for anchor bolt coating shall be in compliance with Saudi Aramco Materials System Reports 12-SAMSS-007 Fabrication of Structural and Miscellaneous Steel and requirements for double nuts shall be in compliance with Saudi Aramco Engineering Standard SAES-Q-005 Concrete Foundations.

The foundation design engineer shall verify the capacity of any vendor furnished or detailed anchor bolts. Unless otherwise specified by the equipment manufacturer, equipment shall be installed on mounting plate(s), and the direct attachment of equipment feet to the foundation using the anchor bolts shall not be permitted. Mounting plates shall be of sufficient strength and rigidity to transfer the applied forces to the foundation. Grouting shall be in accordance with Saudi Aramco Engineering Standard SAES-Q-011 Epoxy Grout for machinery Support and machine manufacturer’s instructions.

The drawing shall clearly indicate the locations and types of the anchor bolts and sleeves, the anchor bolt diameter, the depth of embedment into the foundation of the anchor bolts, the length of the anchor bolts threads, and the length of the anchor bolt projections.

12.4.4 **Stiffness requirements.** The foundation must be of sufficient width to prevent rocking and adequate depth to permit properly embedded anchor bolts. The width of the foundation shall be at least 1.5 times the vertical distance from the base to the machine centerline, unless analysis carried out by a registered design professional demonstrates that a lesser value will perform adequately. For concrete foundations, the weight of the foundation for reciprocating equipment shall not be less than 5 times and, for rotary equipment, shall not be less than 3 times the weight of the machinery, including its base plate and the piping supported from the foundation, unless analysis carried out by a registered design professional demonstrates that a lesser value will perform adequately.

For foundations and piers constructed with normal weight concrete, the dynamic modulus of elasticity shall be taken as:

$$E_D = 6560(f_c^{'})^{0.5} \qquad \textbf{(Equation 12-2)}$$

where:

E_D = Dynamic modulus of elasticity of concrete in MPa.

$f_c^{'}$ = Compressive strength of concrete at 28 days in MPa.

The minimum thickness of the concrete foundations shall not be less than (0.60+L/30) where L is the length of foundation in meters parallel to the machine bearing axis in meters. Piers shall not be used unless absolutely required by operation or maintenance or if required by machine vendor specification. Block foundations for reciprocating machines shall have a minimum of 50 % of the block thickness embedded in the soil, unless otherwise specified by the equipment manufacturer.

12.4.5 **Allowable eccentricities for concrete foundations with horizontal shaft machinery.** Secondary moments that could significantly influence the natural frequencies of the foundation shall be minimized. The horizontal eccentricity, perpendicular to the machine bearing axis, between the center of gravity of the machine foundation system and the centroid of the soil contact area (or in case of piled foundations, the elastic support point of the pile group) shall not exceed 0.05 times the width of foundation in meters.

The horizontal eccentricity, parallel to the bearing axis between the center of gravity of the machine foundation system and the centroid of the soil contact area (or in the case of piled foundations, the elastic support point of the pile group) shall not exceed 0.05 times length of foundation in meters. The machine bearing axis and the centroid of the support (soil contact area, or pile group) shall lie in a common vertical plane.

Piers and columns shall be proportioned in such a manner that the centroid of their vertical stiffness lies in the same vertical plane as the bearing axis and center of gravity of the machinery.

12.4.6 **Permissible frequency ratios.** The ratio between the operating frequency of the machinery, f, and each natural frequency of the machine foundation system, f_n shall not lie in the range of 0.7 to 1.3. Accordingly, for high-tuned systems, f/f_n, shall be less than 0.7 and for low-tuned systems f/f_n shall be greater than 1.3. A need for exceptions shall be approved by a registered design professional.

12.4.7 **Permissible vibration.** Where Manufacturer's vibration criteria are not available**,** the maximum velocity of movement during steady-state normal operation shall be limited to 3 mm per second for centrifugal machines and 4 mm per second for reciprocating machines. For rocking and torsional mode calculation the vibration velocities shall be computed with the dynamic forces of the machinery train components assumed in phase and 180 degrees out of phase.

CHAPTER 13
DAMPPROOFING AND WATERPROOFING

SECTION 13.1
SCOPE

13.1.0 Walls or portions thereof that retain earth and enclose interior spaces and floors below grade, and underground water-retention structures shall be waterproofed and dampproofed in accordance with provisions of this Chapter, with the exception of those spaces containing groups other than residential and institutional where such omission is not detrimental to the building or occupancy.
Ventilation for crawl spaces shall comply with Section 7.3.4 SBC 201.

13.1.1 **Story above grade.** Where a basement is considered a story above grade and the finished ground level adjacent to the basement wall is below the basement floor elevation for 25 percent or more of the perimeter, the floor and walls shall be damp proofed in accordance with Section 13.2 and a foundation drain shall be installed in accordance with Section 13.4.2. The foundation drain shall be installed around the portion of the perimeter where the basement floor is belowground level. The provisions of Sections 2.2.3, 13.3 and 13.4.1 shall not apply in this case.

13.1.2 **Under-floor space.** The finished ground level of an under-floor space such as a crawl space shall not be located below the bottom of the footings. Where there is evidence that the groundwater table rises to within 150 mm of the ground level at the outside building perimeter, or that the surface water does not readily drain from the building site, the ground level of the under-floor space shall be as high as the outside finished ground level, unless an approved drainage system is provided. The provisions of Sections 2.2.3, 13.2, 13.3 and 13.4.1 shall not apply in this case.

13.1.2.1 **Flood hazard areas.** For buildings and structures in flood hazard areas, as established in SBC 301 Section 5.3, the finished ground level of an under-floor space such as a crawl space shall be equal to or higher than the outside finished ground level.

Exception: Under-floor spaces of Group R-3 buildings that meet the requirements of FEMA/ FIA-TB-11.

13.1.3 **Groundwater control.** Where the ground-water table is lowered and maintained at an elevation not less than 150 mm below the bottom of the lowest floor, the floor and walls shall be dampproofed in accordance with Section 13.2. The design of the system to lower the groundwater table shall be based on accepted principles of engineering that shall consider, but not necessarily be limited to, permeability of the soil, rate at which water enters the drainage system, rated capacity of pumps, head against which pumps are to operate and the rated capacity of the disposal area of the system.

SECTION 13.2
DAMP PROOFING REQUIRED

13.2.0 Where hydrostatic pressure will not occur as determined by Section 2.2.3, floors and walls shall be dampproofed in accordance with this Section.

13.2.1 **Floors.** Dampproofing materials for floors shall be installed between the floor and the base course required by Section 13.4.1, except where a separate floor is provided above a concrete slab. Where installed beneath the slab, dampproofing shall consist of not less than 0.15 mm polyethylene with joints lapped not less than 150 mm, or other approved methods or materials. Where permitted to be installed on top of the slab, dampproofing shall consist of mopped-on bitumen, not less than 0.10 mm polyethylene, or other approved methods or materials. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

13.2.2 **Walls.** Dampproofing 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.

Dampproofing shall consist of a bituminous material, 16 N/m^2 of acrylic modified cement, 3.0 mm coat of surface-bonding mortar complying with ASTM C 887, any of the materials permitted for waterproofing by Section 13.3.2 or other approved methods or materials.

13.2.2.1 **Surface preparation of walls.** Prior to application of dampproofing materials on concrete walls, holes and recesses resulting from the removal of form ties shall be sealed with a bituminous material or other approved methods or materials. Unit masonry walls shall be parged on the exterior surface belowground level with not less than 10 mm of Portland cement mortar. 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.

SECTION 13.3
WATERPROOFING REQUIRED

13.3.0 Where the groundwater investigation required by Section 2.2.3 indicates that a hydrostatic pressure condition exists, and the design does not include a groundwater control system as described in Section 13.1.3, walls and floors shall be waterproofed in accordance with this section.

13.3.1 **Floors.** Floors required to be waterproofed shall be of concrete, designed and constructed to withstand the hydrostatic pressures to which the floors will be subjected.

Waterproofing shall be accomplished by placing a membrane of rubberized asphalt, butyl rubber, or not less than 0.15 mm polyvinyl chloride with joints lapped not less than 150 mm or other approved materials under the slab. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

13.3.2 **Walls.** Walls required to be waterproofed shall be of concrete or masonry and shall be designed and constructed to withstand the hydrostatic pressures and other lateral loads to which the walls will be subjected.

Waterproofing shall be applied from the bottom of the wall to not less than 300 mm above the maximum elevation of the groundwater table. The remainder of the wall shall be dampproofed in accordance with Section 13.2.2. Waterproofing shall consist of two-ply hot-mopped felts, not less than 0.15 mm polyvinyl chloride, 1.0

mm polymer-modified asphalt, 0.150 mm polyethylene or other approved methods or materials capable of bridging nonstructural cracks. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

13.3.2.1 **Surface preparation of walls.** Prior to the application of waterproofing materials on concrete or masonry walls, the walls shall be prepared in accordance with Section 13.2.2.1.

13.3.3 **Joints and penetrations.** Joints in walls and floors, joints between the wall and floor and penetrations of the wall and floor shall be made watertight utilizing approved methods and materials.

SECTION 13.4
SUBSOIL DRAINAGE SYSTEM

13.4.0 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. A subsoil drainage system designed and constructed in accordance with Section 13.1.3 shall be deemed adequate for lowering the ground-water table.

13.4.1 **Floor base course.** Floors of basements, except as provided for in Section 13.1.1, shall be placed over a floor base course not less than 100 mm in thickness that consists of gravel or crushed stone containing not more than 10 percent of material that passes through a No. 4 (4.75 mm) sieve.

Exception: Where a site is located in well-drained gravel or sand/gravel mixture soils, a floor base course is not required.

13.4.2 **Foundation drain.** A drain shall be placed around the perimeter of a foundation. It shall satisfy the requirements of Equations 7-4 through 7-6 or in lieu it shall consist of gravel or crushed stone containing not more than 10-percent material that passes through a No. 4 (4.75 mm) sieve. The drain shall extend a minimum of 300 mm beyond the outside edge of the footing. The thickness shall be such that the bottom of the drain is not higher than the bottom of the base under the floor, and that the top of the drain is not less than 150 mm above the top of the footing. The top of the drain shall be covered with an approved filter membrane material. Where a drain tile or perforated pipe is used, the invert of the pipe or tile shall not be higher than the floor elevation. The top of joints or the top of perforations shall be protected with an approved filter membrane material. The pipe or tile shall be placed on not less than 50 mm of gravel or crushed stone complying with Section 13.4.1, and shall be covered with not less than 150 mm of the same material.

13.4.3 **Drainage discharge.** The floor base and foundation perimeter drain shall discharge by gravity or mechanical means into an approved drainage system that complies with the SBC 701.

Exception: Where a site is located in well-drained gravel or sand/gravel mixture soils, a dedicated drainage system is not required.

SECTION 13.5
UNDERGROUND WATER-RETENTION STRUCTURES

13.5.0 Underground water-retention structures shall meet the provisions of Sections 13.5.1 through 13.5.4.

13.5.1 General requirements. All underground water-retention structures shall meet the following requirements:

1. All internal faces (including the top face) of the water-retention structure shall be waterproofed.
2. Shall not be located under drainage or non-potable water piping.
3. Shall be provided with a waterproof cover to prevent water and foreign matter from entering the tank. The cover shall be large enough to allow access for maintenance.
4. Underground tanks in flood hazard areas shall be anchored to prevent flotation, collapse or lateral movement resulting from hydrostatic loads, including the effects of buoyancy, during conditions of the design flood.

13.5.2 **Design and construction.** For design and constructions of underground water-retention structures, provisions of SBC 304 and ACI 350 Environmental Engineering Concrete Structures shall govern, where applicable.

13.5.3 **Waterproofing**. All internal faces of an underground water-retention structure shall be waterproofed (using approved material such as epoxy films, concrete admixtures, etc.). All such waterproofing materials in contact with water shall neither be toxic nor hazardous to human health. All construction joints shall have proper water-stops. All construction holes, recesses, plumbing sleeves etc. shall be sealed properly.

In cases where the floor slab of the water-retention structure is less than one meter above the anticipated groundwater level, it is required to provide a base layer of compacted granular fill, followed by damp proofing layer as described in Section 13.2.1.

In cases where floor slab is below or close to groundwater level, the floor slab and all exterior faces of the structure shall be water proofed in accordance with Section 13.3. In cases where cover slab of the structure is below or close to groundwater level, all parts of the structure (including the opening of the water-retention structure) shall be waterproofed in accordance with Section 13.3.

13.5.4 **Testing.** Following complete application of waterproofing of the structure, and before backfilling is permitted; underground water-retention structures shall be tested against leakage full of water for a minimum of 48 hours.

CHAPTER 14
GENERAL REQUIREMENTS FOR PIER AND PILE FOUNDATIONS

SECTION 14.1
DESIGN

14.1.1 Piles are permitted to be designed in accordance with provisions for piers in Chapter 14 and Sections 17.3 through 17.10 where either of the following conditions exists, subject to the approval of the building official:

1. Group R-3 and U occupancies not exceeding two stories of light-frame construction, or

2. Where the surrounding foundation materials furnish adequate lateral support for the pile.

SECTION 14.2
GENERAL

14.2.1 Pier and pile foundations shall be designed and installed on the basis of a site investigation as defined in Chapter 2, unless the building official ascertains that sufficient data upon which to base the design and installation is available.

The investigation and report provisions of Chapter 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.

SECTION 14.3
SPECIAL TYPES OF PILES

14.3.1 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.

SECTION 14.4
PILE CAPS

14.4.1 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 75 mm into pile caps and the caps shall extend at least 100 mm beyond the edges of piles. The tops of piles shall be cut back to sound material before capping.

SECTION 14.5
STABILITY

14.5.1 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 centeroid of the group not less than 60 degrees (1 radian) 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 alternatively in lines spaced at least 300 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 10 m in height, provided the centers of the piles are located within the width of the foundation wall.

SECTION 14.6
STRUCTURAL INTEGRITY

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

SECTION 14.7
SPLICES

14.7.1 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% of the least capacity of the pier or pile in bending. In addition, splices occurring in the upper 3 m 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 75 mm, or the pier or pile shall be braced in accordance with Section 14.5 to other piers or piles that do not have splices in the upper 3 m of embedment.

SECTION 14.8
ALLOWABLE PIER OR PILE LOADS

14.8.1 **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.

14.8.2 **Driving criteria.** The allowable compressive load on any pile where determined by the application of an approved driving formula shall not exceed 360 kN. For allowable loads above 360 kN, the wave equation method of analysis shall be used to estimate pile drivability of both driving stresses and net displacement per blow at the ultimate load. Allowable loads shall be verified by load tests in accordance with Section 14.8.3. 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 use of a follower is permitted only with the approval of the building official. The introduction of fresh hammer cushion or pile cushion material just prior to final penetration is not permitted.

14.8.3 **Load tests.** Where design compressive loads per pier or pile are greater than those permitted by Section 14.10, 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 load capacity of the test piers or pile as assessed by one of the published methods listed in Section 14.8.3.1 with consideration for the test type, duration and subsoil. The ultimate load capacity shall be determined by a registered design professional, but shall be no greater than two times the test load that produces a settlement of 7 mm. 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 to 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 through a comparable driving distance.

14.8.3.1 **Load test evaluation.** It shall be permitted to evaluate pile load tests with any of the following methods:

1. Davison Offset Limit.
2. Brinch-Hansen 90% Criterion.
3. Chin-Konder Extrapolation.
4. Other methods approved by the building official.

14.8.4 **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 at minimum depth as set forth in Table 4.1, up to a maximum of 25 kPa, unless a greater value is allowed by the building official after a soil investigation as specified in Chapter 2 is submitted. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless recommended by a soil investigation as specified in Chapter 2.

14.8.5 **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 14.8.3 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.

14.8.6 **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.

14.8.7 **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.

14.8.8 **Overloads on piers or piles.** The maximum compressive load on any pier or pile due to mislocation shall not exceed 110 % of the allowable design load.

SECTION 14.9
LATERAL SUPPORT

14.9.1 **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.

14.9.2 **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 1.5 m below the ground surface and in soft material at 3 m below the ground surface unless otherwise prescribed by the building official after a foundation investigation by an approved agency.

14.9.3 **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 25 mm at the ground surface.

SECTION 14.10
USE OF HIGHER ALLOWABLE PIER OR PILE STRESSES

14.10.1 Allowable stresses greater than those specified for piers or for each pile type in Chapters 14 and 15 are permitted where supporting data justifying such higher stresses is filed with the building official. Such substantiating data shall include:

1. A soil investigation in accordance with Chapter 2.
2. Pier or pile load tests in accordance with Section 14.8.3, 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.

SECTION 14.11
PILES IN SUBSIDING AND CALCAREOUS AREAS

14.11.1 **Piles in subsiding areas**. Where piles are driven 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.

14.11.2 **Piles in calcareous soils.** Where piles are driven through calcareous soils and derive support from frictional forces developed between the pile and the surrounding soil, consideration shall be given to loss of frictional forces due to driving. For bored cast in-situ piles within calcareous soils where support is derived from both friction and tip resistance, consideration shall be given to the possibility of presence of voids/cavities below the tip of the bored pile.

SECTION 14.12
SETTLEMENT ANALYSIS

14.12.1 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.

SECTION 14.13
PREEXCAVATION

14.13.1 The use of jetting, augering or other methods of preexcavation shall be subject to the approval of the building official. Where permitted, preexcavation 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 preexcavated depth until the required resistance or penetration is obtained.

SECTION 14.14
INSTALLATION SEQUENCE

14.14.1 Piles shall be installed in such sequence as to avoid compacting 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.

SECTION 14.15
USE OF VIBRATORY DRIVERS

14.15.1 Vibratory drivers shall only be used to install piles where the pile load capacity is verified by load tests in accordance with Section 14.8.3. 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.

SECTION 14.16
PILE DRIVABILITY

14.16.1 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.

SECTION 14.17
PROTECTION OF PILE MATERIALS

14.17.1 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.

SECTION 14.18
USE OF EXISTING PIERS OR PILES

14.18.1 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 redriving data.

SECTION 14.19
HEAVED PILES

14.19.1 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 14.8.3.

SECTION 14.20
IDENTIFICATION

14.20.1 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.

SECTION 14.21
PIER OR PILE LOCATION PLAN

14.21.1 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.

SECTION 14.22
SPECIAL INSPECTION

14.22.1 Special inspections in accordance with Sections 2.7.6 and 2.7.7, SBC 302 shall be provided for piles and piers, respectively.

SECTION 14.23
SEISMIC DESIGN OF PIERS OR PILES

14.23.1 **Seismic design category C.** Where a structure is assigned to Seismic Design Category C in accordance with SBC 301 Chapters 9 through 16, 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, S_{DS}, divided by 10 unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or 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.

14.23.1.1 **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, as defined in Section 21.1 of SBC 304, 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 Section 2.5 SBC 301.

14.23.1.2 **Design details.** Piers 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.

14.23.2 **Seismic design category D**. Where a structure is assigned to Seismic Design Category D in accordance with SBC 301 Chapters 9 through 16, the requirements for Seismic Design Category C given in Section 14.23.1 shall be met, in addition to the following. Provisions of SBC 304 shall apply when not in conflict with the provisions of Chapter 14 through 17.

14.23.2.1 **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 shall be designed and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of SBC 304 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 Sections 15.2.3.2.1 and 15.2.3.2.2 shall apply.

Grade beams shall be designed as beams in accordance with SBC 304, Chapter 21. When grade beams have the capacity to resist the forces from the load combinations in Section 2.5 SBC 301, they need not conform to Chapter 21 SBC 304.

14.23.2.2 **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% 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 Section 2.5 SBC 301.
2. In the case of rotational restraint, the lesser of the axial and shear forces, and moments resulting from the load combinations of Section 2.5 SBC 301 or development of the full axial, bending and shear nominal strength of the pile.

14.23.2.3 **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 pile 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 Section 2.5 SBC 301.

CHAPTER 15
DRIVEN PILE FOUNDATIONS

SECTION 15.1
PRECAST CONCRETE PILES

15.1.1 **General.** The materials, reinforcement and installation of precast concrete piles shall conform to Sections 15.1.1.1 through 15.1.1.4.

15.1.1.1 **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.

15.1.1.2 **Minimum dimension**. The minimum lateral dimension shall be 200 mm. Corners of square piles shall be chamfered.

15.1.1.3 **Reinforcement**. Longitudinal steel shall be arranged in a symmetrical pattern and be laterally tied with steel ties or wire spiral spaced not more than 100 mm apart, center to center, for a distance of 600 mm from the ends of the pile; and not more than 150 mm elsewhere except that at the ends of each pile, the first five ties or spirals shall be spaced 25 mm center to center. The gage of ties and spirals shall be as follows:

a. For piles having a diameter of 400 mm or less, wire shall not be smaller than 6 mm (No. 5 gage).

b. For piles having a diameter of 400 mm and less than 500 mm, wire shall not be smaller than 6 mm (No. 4 gage).

c. For piles having a diameter of 500 mm and larger, wire shall not be smaller than 7 mm (No. 3 gage).

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

15.1.2 **Precast nonprestressed piles.** Precast nonprestressed concrete piles shall conform to Sections 15.1.2.1 through 15.1.2.5.

15.1.2.1 **Materials.** Concrete shall have a 28-day specified compressive strength ($f_c^{'}$) of not less than 20 MPa.

15.1.2.2 **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.

15.1.2.2.1 **Seismic reinforcement in seismic design category C**. Where a structure is assigned to Seismic Design Category C in accordance with SBC 301 Chapters 9 through 16, the following shall apply. Longitudinal reinforcement with a minimum steel ratio of 0.01 shall 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 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 150 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 200 mm.

15.1.2.2.2 **Seismic reinforcement in seismic design category D**. Where a structure is assigned to Seismic Design Category D in accordance with Chapters 9 through 16

SBC 301, the requirements for Seismic Design Category C in Section 15.1.2.2.1 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 21.4.4.3 of SBC 304 within three pile diameters of the bottom of the pile cap. For other than liquefiable sites and where spirals are used as the transverse reinforcement, it shall be permitted to use a volumetric ratio of spiral reinforcement of not less than one-half that required by Section 21.4.4.1(a) of SBC 304.

15.1.2.3 **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 (f_y) or a maximum of 210 MPa. The allowable tensile stress in the reinforcing steel shall not exceed 50 percent of the yield strength of the steel (f_y) or a maximum of 165 MPa.

15.1.2.4 **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.

15.1.2.5 **Concrete cover**. Reinforcement for piles that are not manufactured under plant conditions shall have a concrete cover of not less than 50 mm.

Reinforcement for piles manufactured under plant control conditions shall have a concrete cover of not less than 30 mm for Dia 16 mm bars and smaller, and not less than 40 mm for Dia 18 mm through Dia 36 mm bars except that longitudinal bars spaced less than 40 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 75 mm.

15.1.3 **Precast prestressed piles**. Precast prestressed concrete piles shall conform to the requirements of Sections 15.1.3.1 through 15.1.3.5.

15.1.3.1 **Materials**. Prestressing steel shall conform to ASTM A416. Concrete shall have a 28-day specified compressive strength ($f_c^{'}$) of not less than 35 MPa.

15.1.3.2 **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 3 MPa for piles up to 9 meters in length, 4 MPa for piles up to 15 meters in length and 5 MPa for piles greater than 15 m in length.

Effective prestress shall be based on an assumed loss of 210 MPa in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in SBC 304.

15.1.3.2.1 **Design in seismic design Category C.** Where a structure is assigned to Seismic Design Category C in accordance with SBC 301 Chapters 9 through 16, 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 6000 mm of the pile.

$$\rho_s = 0.12 f_c^{'} \big/ f_{yh}$$ **(Equation 15-1)**

where:

$f_c^{'}$ = Specified compressive strength of concrete, MPa.

f_{yh} = Yield strength of spiral reinforcement $\leq$ 586 MPa.

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

At least one-half the volumetric ratio required by Equation 15-1 shall be provided below the upper 6000 mm of the pile.

The pile cap connection by means of dowels as indicated in Section 14.23.1 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.

15.1.3.2.2 **Design in seismic design category D**. Where a structure is assigned to Seismic Design Category D, in accordance with Chapters 9 through 16, SBC 301, the requirements for Seismic Design Category C in Section 15.1.3.2.1 shall be met, in addition to the following:

1. Requirements in Chapter 21, SBC 304, need not apply, unless specifically referenced.
2. Where the total pile length in the soil is 10 meters or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 10 m, the ductile pile region shall be taken as the greater of 10 m 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 200 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 Section 12.14.3 of SBC 304.
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:

$$\rho_s = 0.25(f_c^{'}/f_{yh})(A_g/A_{ch} - 1.0)[0.5 + 1.4P/(f_c^{'}A_g)]$$ **(Equation 15-2)**

but not less than:

$$\rho_s = 0.12(f_c^{'}/f_{yh})[0.5 + 1.4P/(f_c^{'}A_g)]$$ **(Equation 15-3)**

and need not exceed:

$$\rho_s = 0.021$$ **(Equation 15-4)**

where:

A_g = Pile cross-sectional area, mm^2.
A_{ch} = Core area defined by spiral outside diameter, mm^2.
$f_c^{'}$ = Specified compressive strength of concrete, MPa.
f_{yh} = Yield strength of spiral reinforcement $\leq$ 580 MPa.
P = Axial load pile, kN, as determined from Section 2.3.2 SBC 301 Combinations (5, 6, and 7).
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 spacing, and perpendicular to dimension, h_c, shall conform to:

$$A_{sh} = 0.3sh_c\,(f_c^{'}/f_{yh})(A_g/A_{ch} - 1.0)[0.5 + 1.4P/(f_c^{'}A_g)] \qquad \textbf{(Equation 15-5)}$$

but not less than:

$$A_{sh} = 0.12sh_c\,(f_c^{'}/f_{yh})[0.5 + 1.4P/(f_c^{'}A_g)] \qquad \textbf{(Equation 15-6)}$$

where:

f_{yh} = $\leq$ 480 MPa.
h_c = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, mm.
s = Spacing of transverse reinforcement measured along length of pile, mm.
A_{sh} = Cross-sectional area of transverse reinforcement, mm^2.
$f_c^{'}$ = Specified compressive strength of concrete~~,~~ MPa.

The hoops and cross ties shall be deformed bars not less than Dia 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.

15.1.3.3 **Allowable stresses**. The maximum allowable design compressive stress, f_c, in concrete shall be determined as follows:

$$f_c = 0.33 f_c^{'} - 0.27 f_{pc} \qquad \textbf{(Equation 15-7)}$$

where:

$f_c^{'}$ = The 28-day specified compressive strength of the concrete.

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

15.1.3.4 **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.

15.1.3.5 **Concrete cover**. Prestressing steel and pile reinforcement shall have a concrete cover of not less than 30mm for square piles of 300 mm or smaller size and 40 mm for larger piles, except that for piles exposed to seawater, the minimum protective concrete cover shall not be less than 70 mm.

SECTION 15.2
STRUCTURAL STEEL PILES

15.2.1 **General**. Structural steel piles shall conform to the requirements of Sections 15.2.2 through 15.2.5.

15.2.2 **Materials**. Structural steel piles, steel pipe and fully welded steel piles fabricated from plates shall conform to ASTM A36, ASTM A252, ASTM A283, ASTM A572, ASTM A588 or ASTM A913.

15.2.3 **Allowable stresses**. The allowable axial stresses shall not exceed 35 percent of the minimum specified yield strength (f_y).

Exception: Where justified in accordance with Section 14.10, the allowable axial stress is permitted to be increased above 0.35 f_y, but shall not exceed 0.5 f_y.

15.2.4 **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 200 mm.
3. Flanges and web shall have a minimum nominal thickness of 10 mm.

15.2.5 **Dimensions of steel pipe piles**. Steel pipe piles driven open ended shall have a nominal outside diameter of not less than 200 mm. The pipe shall have a minimum of 220 mm^2 of steel in cross section to resist each 1360 N-m of pile hammer energy or the equivalent strength for steels having a yield strength greater than 240 MPa. Where pipe wall thickness less than 5 mm is driven open ended, a suitable cutting shoe shall be provided.

CHAPTER 16
CAST-IN-PLACE CONCRETE PILE FOUNDATIONS

SECTION 16.1
GENERAL

16.1.0 The materials, reinforcement and installation of cast-in-place concrete piles shall conform to Sections 16.1.1 through 16.1.3.

16.1.1 **Materials.** Concrete shall have a 28-day specified compressive strength ($f_c^{'}$) of not less than 20 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 100 mm and not more than 150 mm. Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

16.1.2 **Reinforcement.** Except for steel dowels embedded 1.5 m or less in the pile and as provided in Section 16.3.4, 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 semifluid state.

16.1.2.1 **Reinforcement in seismic design Category C.** Where a structure is assigned to Seismic Design Category C in accordance with Chapters 9 through 16, SBC 301, 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 3 m 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 10 mm diameter provided at 16-longitudinal-bar diameter maximum spacing. Transverse confinement reinforcing with a maximum spacing of 150 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.

16.1.2.2 **Reinforcement in seismic design category D**. Where a structure is assigned to Seismic Design Category D in accordance with SBC 301 Chapters 9 through 16, 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 3 m below ground or throughout the flexural length of the pile, whichever length is greater. 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 reinforcing provided in the pile in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of SBC 304 three times the least pile dimension of the bottom of the pile cap. It shall be permitted to use a transverse spiral reinforcing ratio of not less than one-half of that required in Section 21.4.4.1

(a) of SBC 304 for other than liquefiable sites. Tie spacing throughout the remainder of the concrete section shall not exceed 12-longitudinal-bar diameters, one-half the least dimension of the section, nor 300 mm. Ties shall be a minimum of Dia 10 mm bars for piles with a least dimension up to 500 mm, and Dia 12 mm for larger piles.

16.1.3 **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 chutted 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.

SECTION 16.2
ENLARGED BASE PILES

16.2.0 Enlarged base piles shall conform to the requirements of Sections 16.2.1 through 16.2.5.

16.2.1 **Materials.** The maximum size for coarse aggregate for concrete shall be 20 mm. Concrete to be compacted shall have a zero slump.

16.2.2 **Allowable stresses.** The maximum allowable design compressive stress for concrete not placed in a permanent steel casing shall be 25% 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% of the 28-day specified compressive strength ($f_c^{'}$).

16.2.3 **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.

16.2.4 **Load-bearing capacity.** Pile load-bearing capacity shall be verified by load tests in accordance with Section 14.8.3.

16.2.5 **Concrete cover.** The minimum concrete cover shall be 70 mm for uncased shafts and 25 mm for cased shafts.

SECTION 16.3
DRILLED OR AUGERED UNCASED PILES

16.3.0 Drilled or augered uncased piles shall conform to Sections 16.3.1 through 16.3.5.

16.3.1 **Allowable stresses.** The allowable design stress in the concrete of drilled uncased piles shall not exceed 33 percent of the 28-day specified compressive strength

($f_c^{'}$). The allowable design stress in the concrete of augered cast-in-place piles shall not exceed 25 percent of the 28-day specified compressive strength ($f_c^{'}$). The allowable compressive stress of reinforcement shall not exceed 34 percent of the yield strength of the steel or (175 MPa).

16.3.2 **Dimensions.** The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 300 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.

16.3.3 **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 a continuous manner in increments of about 300 mm each. 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 1500 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 during installation of an adjacent pile, the pile shall be replaced.

16.3.4 **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 ducts in the auger prior to filling the pile with concrete. All pile reinforcement shall have a concrete cover of not less than 70 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.

16.3.5 **Reinforcement in seismic design category C** and **D.** Where a structure is assigned to Seismic Design Category C and D in accordance with Chapters 9 through 16, SBC 301, the corresponding requirements of Sections 16.1.2.1 and 16.1.2.2 shall be met.

SECTION 16.4
DRIVEN UNCASED PILES

16.4.0 Driven uncased piles shall conform to Sections 16.4.1 through 16.4.4.

16.4.1 **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.

16.4.2 **Dimensions.** The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 300 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.

16.4.3 **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 less than 48 hours old unless approved by the building official. If the concrete surface in any completed pile rises or drops, the pile shall be replaced. Piles shall not be installed in soils that could cause pile heave.

16.4.4 **Concrete cover.** Pile reinforcement shall have a concrete cover of not less than 70 mm, measured from the inside face of the drive casing or mandrel.

SECTION 16.5
STEEL-CASED PILES

16.5.0 Steel-cased piles shall comply with the requirements of Sections 16.5.1 through 16.5.4.

16.5.1 **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 200 mm.

16.5.2 **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 16.5.2.1 through 16.5.2.4.

16.5.2.1 **Shell thickness.** The thickness of the steel shell shall not be less than manufacturer's standard gage No. 14 gage 1.75 mm minimum.

16.5.2.2 **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.

16.5.2.3 **Strength.** The ratio of steel yield strength (f_y) to 28-day specified compressive strength ($f_c^{'}$) shall not be less than six.

16.5.2.4 **Diameter.** The nominal pile diameter shall not be greater than 400 mm.

16.5.3 **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.

16.5.4 **Reinforcement.** Reinforcement shall not be placed within 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.

16.5.4.1 **Seismic reinforcement.** Where a structure is assigned to Seismic Design Category C and D in accordance with Chapters 9 through 16, SBC 301, the reinforcement requirements for drilled or augered uncased piles in Section 16.3.5 shall be met.

Exception: A spiral-welded metal casing of a thickness not less than manufacturer's standard gage No. 14 gage 1.75 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.

SECTION 16.6
CONCRETE-FILLED STEEL PIPE AND TUBE PILES

16.6.0 Concrete-filled steel pipe and tube piles shall conform to the requirements of Sections 16.6.1 through 16.6.5.

16.6.1 **Materials.** Steel pipe and tube sections used for piles shall conform to ASTM A 252 or ASTM A 283. Concrete shall conform to Section 16.1.1. The maximum coarse aggregate size shall be 20 mm.

16.6.2 **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 (f_y), provided f_y shall not be assumed greater than 250 MPa for computational purposes.

Exception: Where justified in accordance with Section 16.2, the allowable stresses are permitted to be increased to 0.50 f_y .

16.6.3 **Minimum dimensions.** Piles shall have a nominal outside diameter of not less than 200 mm and a minimum wall thickness in accordance with Section 15.2.5. For mandrel-driven pipe piles, the minimum wall thickness shall be 2.5 mm.

16.6.4 **Reinforcement.** Reinforcement steel shall conform to Section 16.1.2. Reinforcement shall not be placed within 25 mm of the steel casing.

16.6.4.1 **Seismic reinforcement.** Where a structure is assigned to Seismic Design Category C and D in accordance with SBC 301 Chapters 9 through 16, 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 5 mm.

16.6.5 **Placing concrete.** The placement of concrete shall conform to Section 16.1.3.

SECTION 16.7
CAISSON PILES

16.7.0 Caisson piles shall conform to the requirements of Sections 16.7.1 through 16.7.6.

16.7.1 **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.

16.7.2 **Materials.** Pipe and steel cores shall conform to the material requirements in Section 15.2. Pipes shall have a minimum wall thickness of 10 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 28 MPa. The concrete mix shall be designed and proportioned so as to produce a cohesive workable mix with a slump of 100 mm to 150 mm.

16.7.3 **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 predicted 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 450 mm, and the diameter of the rock socket shall be approximately equal to the inside diameter of the pile.

16.7.4 **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 50 mm. Where cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

16.7.5 **Allowable stresses.** The allowable design compressive stresses shall not exceed the following: concrete, 0.33 $f_c^{'}$; steel pipe, 0.35 f_y and structural steel core, 0.50 f_y.

16.7.6 **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.

SECTION 16.8
COMPOSITE PILES

16.8.1 Composite piles shall conform to the requirements of Sections 16.8.2 through 16.8.5.

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

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

16.8.4 **Splices.** Splices between concrete and steel 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.

16.8.5 **Seismic reinforcement.** Where a structure is assigned to Seismic Design Category C and D in accordance with SBC 301 Chapters 9 through 16, 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 16.1.2.1 and 16.1.2.2 or the steel section shall comply with Section 15.2.5 or 16.6.4.1.

CHAPTER 17
PIER FOUNDATIONS

SECTION 17.1
GENERAL

17.1.1 Isolated and multiple piers used as foundations shall conform to the requirements of Sections 17.2 through 17.10, as well as the applicable provisions of Chapter 14.

SECTION 17.2
LATERAL DIMENSIONS AND HEIGHT

17.2.1 The minimum dimension of isolated piers used as foundations shall be 600 mm, and the height shall not exceed 12 times the least horizontal dimension.

SECTION 17.3
MATERIALS

17.3.1 Concrete shall have a 28-day specified compressive strength ($f_c^{'}$) of not less than 20 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 100 mm and not more than 150 mm. Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

SECTION 17.4
REINFORCEMENT

17.4.1 Except for steel dowels embedded 1500 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 70 mm concrete cover requirement be reduced to 50 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 Section 16.1.2.1 and 16.1.2.2.

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 Dia 12 mm 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 Dia 12 mm bar, without ties or spirals, when the lateral load, E, to the top of the pier does not exceed 900 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 Dia 12 mm bars, without ties or spirals, when it can be shown the concrete pier will not rupture when designed for the maximum seismic load, E_m, and the soil is determined to be of adequate stiffness.

4. Closed ties or spirals where required by Section 16.1.2.2 are permitted to be limited to the top 1 m of the piers 3 m or less in depth supporting Group R-3 and U occupancies of Seismic Design Category D, not exceeding two stories of light-frame construction.

SECTION 17.5 CONCRETE PLACEMENT

17.5.1 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 chutted 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.

SECTION 17.6 BELLED BOTTOMS

17.6.1 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. Where 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.

SECTION 17.7 MASONRY

17.7.1 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 SBC 305.

SECTION 17.8 CONCRETE

17.8.1 Where adequate lateral support is not provided, piers shall be constructed of reinforced concrete, and shall conform to the requirements for columns in SBC 304. Where adequate lateral support is furnished by the surrounding materials as defined in Section 14.9, piers are permitted to be constructed of reinforced concrete, and the requirements of SBC 304 for bearing on concrete shall apply.

SECTION 17.9 STEEL SHELL

17.9.1 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 14.17. Horizontal joints in the shell shall be spliced to comply with Section 14.7.

SECTION 17.10
DEWATERING

17.10.1 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.

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