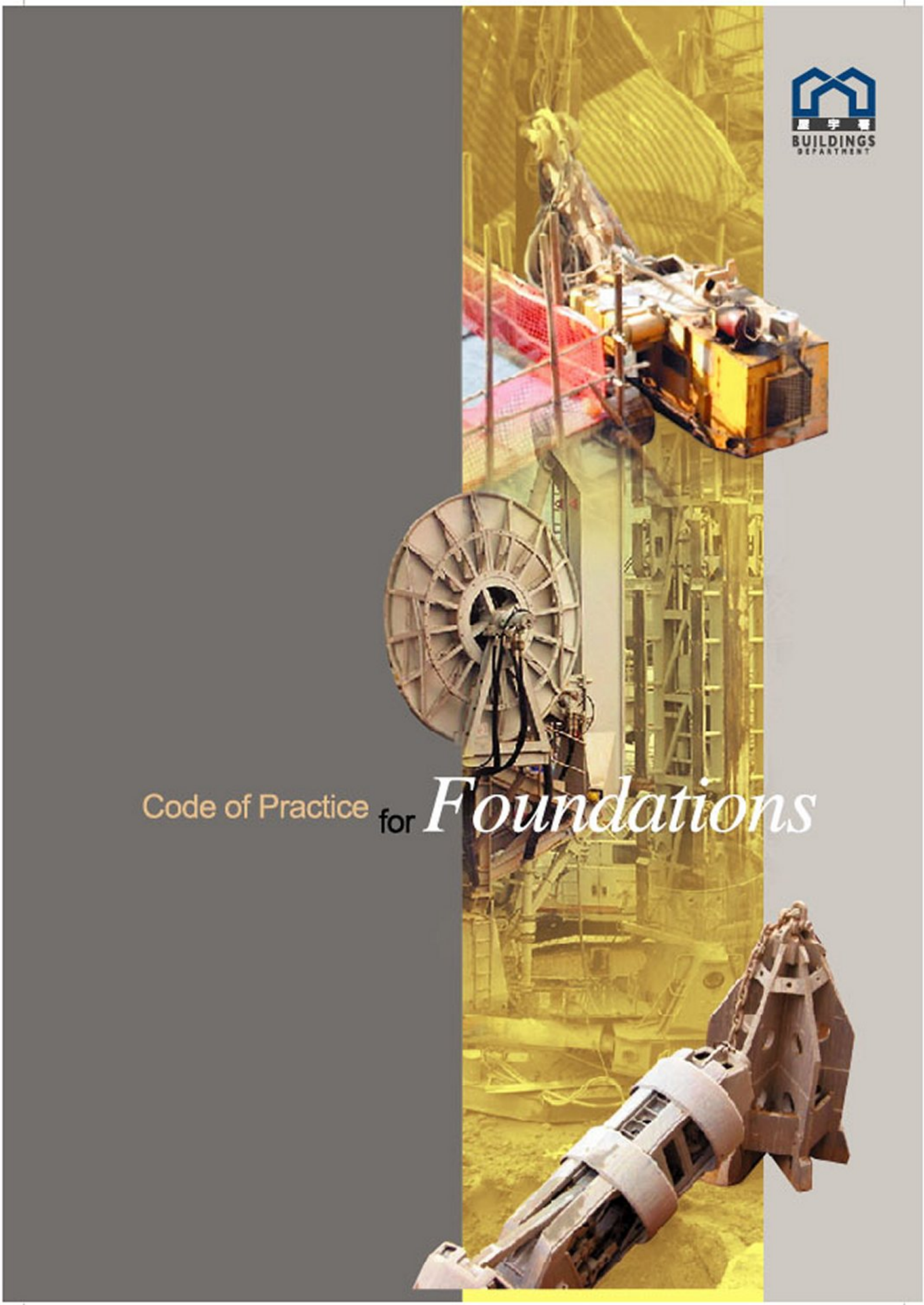

Code of Practice for *Foundations*

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First published: October 2004

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FOREWORD

This Code of Practice provides guidelines for the professionals and practitioners on design, analysis and construction of foundations. It was prepared by the Drafting Committee for the Code of Practice for Foundations convened by the Buildings Department.

This Code of Practice is based on the requirements of the Buildings Ordinance and related regulations, and has taken into account the local conditions, work practice and development of new technologies in analysis, design and construction of foundations.

Although this Code of Practice is not a statutory document, the compliance with the requirements of this Code of Practice is deemed to satisfy the relevant provisions of the Buildings Ordinance and related regulations.

The contributions and efforts given by the invited members of the Drafting Committee in the preparation of this Code of Practice are greatly appreciated.

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1. GENERAL

1.1 SCOPE

This Code of Practice was prepared on the basis of being 'deemed-to-satisfy' the Building (Construction) Regulations as far as the design and construction of foundations are concerned. Departure from the requirements and recommendations of this Code or the use of other standards or codes of practice for design of foundations may require demonstration of the compliance with the provisions of the Building (Construction) Regulations.

This Code of Practice is intended for local use only. Methods of foundation design that are current and commonly used in Hong Kong are included in this Code as far as possible. It should be noted that some methods of foundation design have been developed from practical considerations and experience and have been accepted on the basis that they have been demonstrated to have worked satisfactorily.

In addition to technical aspects, this Code of Practice also includes brief descriptions of local practices that could affect the design and construction of foundations. The descriptions cover mainly the purposes and objectives of the practices. Detailed procedural requirements are not included; reference should be made to the most current practice notes issued by the Building Authority, which can be accessed under "Technical Document" in the homepage of the Buildings Department at http://www.info.gov.hk/bd.

Design for seismic effect is not presently included in this Code. However, where seismic effect is considered in the design of the superstructure, it should also be considered in the design of the foundation.

1.2 DEFINITIONS

For the purpose of this Code of Practice the following definitions apply:

Allowable load. The maximum load that may be applied safely to a foundation after taking into account its ultimate bearing capacity, negative skin friction, pile spacing, overall bearing capacity of the ground below the foundation and allowable settlement.

Allowable bearing pressure. The maximum allowable bearing pressure that may be applied at the base of the foundation, taking into account the ultimate bearing capacity of the soil or rock, the amount and kind of settlement expected and the ability of the structure to accommodate such settlement. [NOTE : The allowable bearing pressure is a combined function of the site conditions, including all construction in the vicinity, and the characteristics of the proposed foundation/structure.]

Authorized Person. A person whose name is on the authorized persons' register kept under Buildings Ordinance section 3(1).

Bell-out. An enlargement of the base area of a pile, formed in situ by undercutting (under-reaming) the soil or rock at the base of a bored pile.

Final set. The penetration per blow of hammer at the founding level of a driven pile.

Foundation. That part of a building, building works, structure or street in direct contact with and transmitting loads to the ground.

Ground investigation. Any exploratory drilling, boring, excavating and probing of land for obtaining any information on ground conditions and includes the installation of instruments, sampling, field testing, any other site operation and laboratory testing of samples obtained from such operations.

Ground investigation field works. All site operations in ground investigation and exclude laboratory testing of samples and field density tests.

Newly reclaimed land. Any land of which the reclamation is completed within 7 years.

Negative skin friction. The downward skin friction resulted from the consolidation of compressible soil strata.

Pile cap. A concrete structure built on the head of a pile or a group of piles for transmission of loads from the structure above to the pile or group of piles.

Pile spacing. The distance measured from centre to centre of adjacent piles.

Pre-boring. Removal of ground or underground obstacles by boring or other means to enable a pile to be installed.

Raking pile. A pile installed at an inclination to the vertical.

Registered Specialist Contractor (Foundation Works). A contractor whose name is on the sub-register of the foundation works category in the register of specialist contractors maintained under section 8A of the Buildings Ordinance.

Registered Specialist Contractor (Ground Investigation Field Works). A contractor whose name is on the sub-register of the ground investigation field works category in the register of specialist contractors maintained under section 8A of the Buildings Ordinance.

Registered Structural Engineer. A person whose name is on the structural engineers' register kept under Buildings Ordinance section 3(3).

Rock socket. The penetration formed in rock for embedding a portion of a pile.

Rock socketed pile or *Socketed pile.* A pile with the toe portion embedded into a rock socket to derive load resistance through bearing, bond or friction with the rock.

Skin friction. The frictional resistance developed at the interface between a foundation member and the surrounding ground.

Site investigation. An investigation of the physical characteristics of the site and includes documentary studies, site survey and ground investigation.

Test driving of pile. Test driving of one or more piles carried out to verify the design assumptions.

Test pile. A pile to which a test is applied, by loading or other suitable method, to determine the load-carrying capacity and/or displacement characteristics of the pile.

Trial pile. A pile installed before the commencement of the main piling works for the purpose of establishing the suitability of the chosen type of pile and confirming the design parameters adopted and load carrying capacity.

Ultimate bearing capacity. The value of the loading intensity for a particular foundation at which the resistance of the bearing stratum becomes fully mobilized or undergoes substantial deformation.

Working load. The service load which the foundation is designed to carry.

1.3 ABBREVIATIONS

For the purpose of this Code of Practice the following abbreviations apply :

AP	Authorized Person
GEO	Geotechnical Engineering Office
GEOGUIDE 2	"Guide to Site Investigation" published by GEO
GEOGUIDE 3	"Guide to Rock and Soil Descriptions" published by GEO
HOKLAS	Hong Kong Laboratory Accreditation Scheme
NSF	Negative Skin Friction
PNAP	Practice Note for Authorized Persons and Registered Structural Engineers
RSE	Registered Structural Engineer
SPT	Standard Penetration Test

Other abbreviations are defined in the text where they occur.

2. GENERAL DESIGN REQUIREMENTS

2.1 GENERAL

2.1.1 BASIC REQUIREMENTS

Foundations of any building or structure shall be designed and constructed to withstand safely all the dead, imposed and wind loads without impairing the stability or inducing excessive movement to the building or of any other building, street, land, slope or services.

The allowable capacity of the soil/rock under working loads where any foundation is founded shall be the lesser of :

(a) the ultimate capacity for bearing, bond or friction with an adequate factor of safety against failure; or
(b) the value in relation to bearing, bond or friction such that the maximum deformation or movement induced to the foundation under working loads can be tolerated by the building, any other building, structure, land, street and services.

The allowable capacity may be increased by 25% when such increase is solely due to wind effects.

In determining the said factor of safety against failure, due consideration shall be given to the form and depth of the foundation, loading characteristics, the general geological conditions of the ground and its surrounding including the presence of dissolution features, jointing conditions and any other relevant characteristics for rock.

2.1.2 COMPATIBILITY OF DESIGN AND CONSTRUCTION

In choosing the method for the determination of the ultimate capacity or for the estimation of settlement, care must be taken to ensure that the site investigation, testing, derivation of parameters, computations, method of construction and standards of acceptance are mutually compatible and consistent with such method.

2.1.3 CLASSIFICATION OF SOILS AND ROCKS

The classification of soils and rocks used in this Code is set out in Table 2.1. Further definition and description can be obtained from GEOGUIDE 3.

2.2 ALLOWABLE BEARING PRESSURE, BOND OR FRICTION OF GROUND

The allowable bearing pressure, bond or friction of soils and rocks should be determined by one of the methods given in Clauses 2.2.1 to 2.2.4.

2.2.1 RATIONAL DESIGN METHOD

Rational design method for calculating the ultimate capacity should be based on sound engineering approach and should include :

(a) the reasonable interpretation of the results of site investigation;
(b) the assessment of test results obtained in situ or from samples in the laboratory; and
(c) an analysis based on the laws of physics and recognized engineering principles taking into account the ground conditions and foundation geometry, or an established empirical method proven with adequate correlation.

Normally, the allowable capacity is estimated by applying a factor of safety of 3 to the calculated ultimate bearing capacity. However, other factors of safety may be adopted having regard to the nature of the soil or rock, its variability over the site and the reliability of the design method.

2.2.2 PRESUMED VALUES

In lieu of a rational design method, the allowable capacity for soils and rocks may also be taken as those presumed values derived from empirical correlation and as stipulated below provided that the following conditions are complied with :

(a) the planning, conducting and supervision of the ground investigation and the interpretation of the results are carried out in accordance with the recommendations given in Chapter 3; and
(b) the structures are not unduly sensitive to settlement or other displacement or movement that may be required to mobilize the allowable capacity.

The presumed values for rock are based on the assumption that slip of the rock will not occur. Therefore, where the rock profile is inclined at such an angle that the bearing capacity of the rock mass may be affected, the rock joints should be checked to ensure that there is no unfavourable joint orientation that could permit slip of the rock to occur.

Allowable Vertical Bearing Values

The allowable vertical bearing pressure for foundations on horizontal ground may be estimated from Table 2.1 on the basis of the material description.

Allowable Lateral Bearing Pressure for Rock

The allowable lateral bearing pressure for rock may be taken as one third of the allowable vertical bearing pressure provided that no adverse rock joints exist.

Allowable Bond or Friction between Rock and Concrete

The allowable bond or friction between rock and concrete for piles may be estimated from Table 2.2.

Table 2.1 Presumed Allowable Vertical Bearing Pressure under Foundations on Horizontal Ground

Category	*Description of rock or soil*	*Presumed allowable bearing pressure (kPa)*	
	Rock (granite and volcanic):		
1(a)	Fresh strong to very strong rock of material weathering grade I, with 100% total core recovery and no weathered joints, and minimum uniaxial compressive strength of rock material (UCS) not less than 75 MPa (equivalent point load index strength PLI_{50} not less than 3 MPa)	10,000	
1(b)	Fresh to slightly decomposed strong rock of material weathering grade II or better, with a total core recovery of more than 95% of the grade and minimum uniaxial compressive strength of rock material (UCS) not less than 50 MPa (equivalent point load index strength PLI_{50} not less than 2 MPa)	7,500	
1(c)	Slightly to moderately decomposed moderately strong rock of material weathering grade III or better, with a total core recovery of more than 85% of the grade and minimum uniaxial compressive strength of rock material (UCS) not less than 25 MPa (equivalent point load index strength PLI_{50} not less than 1 MPa)	5,000	
1(d)	Moderately decomposed, moderately strong to moderately weak rock of material weathering grade better than IV, with a total core recovery of more than 50% of the grade	3,000	
	Intermediate soil (decomposed granite and decomposed volcanic):		
2	Highly to completely decomposed, moderately weak to weak rock of material weathering grade V or better, with SPT N-value ≥ 200	1,000	
	Non-cohesive soil (sands and gravels):	*Dry*	*Submerged*
3(a)	Very dense – SPT N-value >50	500	250
3(b)	Dense – SPT N-value 30-50; requires pick for excavation; 50 mm peg hard to drive	300	150
3(c)	Medium dense – SPT N-value 10-30	100	50
3(d)	Loose – SPT N-value 4-10, can be excavated with spade; 50 mm peg easily driven	<100	<50
		See note (9)	

Table 2.1 Presumed Allowable Vertical Bearing Pressure under Foundations on Horizontal Ground (Continued)

Category	*Description of rock or soil*	*Presumed allowable bearing pressure (kPa)*
	Cohesive soil (clays and silts):	
4(a)	Very stiff or hard – Undrained shear strength >150 kPa; can be indented by thumbnail	300
4(b)	Stiff – Undrained shear strength 75-150 kPa; can be indented by thumb	150
4(c)	Firm – Undrained shear strength 40-75 kPa; can be moulded by strong finger pressure	80

Notes:

(1) The presumed values for allowable bearing pressure given are for foundations with negligible lateral loads at bearing level.

(2) The self weight of the length of pile embedded in soil or rock does not need to be included into the calculation of bearing stresses.

(3) Minimum socket depth along the pile perimeter is 0.5 m for categories 1(a) and 1(b), and 0.3 m for categories 1(c) and 1(d).

(4) Total Core Recovery is the percentage ratio of rock recovered (whether solid intact with no full diameter, or non-intact) to the length of 1.5 m core run and should be proved to a depth at least 5 m into the specified category of rock.

(5) The point load index strength of rock quoted in the table is the equivalent value for 50 mm diameter cores.

(6) The terms used for soils have the following meanings :

dry - soil where the highest anticipated groundwater level is at a depth of not less than 1 m or the width of the foundation, whichever is the greater, below the base of the foundation;

submerged - soil where the highest anticipated groundwater level is at or above base of foundation.

(7) Where the ground is intermediate between dry and submerged, the presumed value may be obtained by linear interpolation.

(8) The use of presumptive values does not preclude the requirement for consideration of settlement of the structure.

(9) Where the footings of minor or temporary structures such as fence walls and hoarding are founded on loose fine sand or soft clay, the presumed allowable bearing pressure may be taken as 100 kPa (if dry) or 50 kPa (if submerged).

Table 2.2 Presumed Allowable Bond or Friction Between Rock and Concrete for Piles

Category of rock as defined in Table 2.1	*Presumed allowable bond or friction between rock and concrete or grout for piles (kPa)*	
	Under compression or transient tension	*Under permanent tension*
1(c) or better	700	350
1(d)	300	150

Notes:

(1) Concrete or grout should have a minimum characteristic compressive strength of 30 MPa at 28 days.

2.2.3 IN SITU TESTING METHOD

The allowable capacity for soils and rocks may also be estimated by appropriate load testing of the foundation on site. The following should be considered when using this method :

(a) the variation at founding conditions between the location of the testing foundation and locations of the actual foundations;
(b) the duration of load application in the test as compared to the working life of the foundation; and
(c) the scale effect of the test relative to the full size of the foundation.

2.2.4 OTHER METHODS

Other methods may be used to estimate the allowable capacity for bearing, bond or friction of soils and rocks provided that the suitability of the method can be demonstrated.

2.3 SETTLEMENT

2.3.1 ESTIMATION OF SETTLEMENT

Prediction of settlement is an important part of foundation design to ensure the future stability and serviceability of the structure supported by the foundation. The prediction of settlement should be :

(a) based on the results of a proper site investigation and appropriate laboratory or field tests identifying the conditions of the groundwater and the ground that contribute to the settlement of the foundation;
(b) based on the principles of mechanics or established empirical methods proven with adequate correlation; and
(c) applicable to Hong Kong soils and in conformity with case histories.

2.3.2 ACCEPTABLE SETTLEMENT

Acceptance of estimated settlements of foundations should be considered on a case-by-case basis, as different structures will have different tolerance in accommodating movements of their foundations. The acceptable settlement for foundations should therefore be determined for each individual case with respect to integrity, stability and functionality of the supported structure.

Where differential settlement is anticipated, it should be assessed accurately or conservatively, and its effect on the supported structure should be checked to ensure that it is acceptable in respect of strength and serviceability.

2.4 STRUCTURES ON NEWLY RECLAIMED LAND

Newly reclaimed lands are liable to significant subsidence due to long-term consolidation of underlying compressible material. All structures and foundations (including floor slabs, partitions, fence walls, ancillary structures, underground utilities and drainage) built on newly reclaimed land must be designed with due consideration of the effect of such significant subsidence.

2.4.1 GENERAL DESIGN RULES

(a) Unless recommended otherwise, the lowest floor slabs of a building should not be designed as on-grade slabs.

(b) Floor slabs directly above a raft-type pile cap may be designed as on-grade.

(c) The following structures may also be designed as on-grade structures provided that they can be readily repaired or replaced if damaged by settlement :
  (i) fence walls, landscaping structures and lightweight covered walkway; and
  (ii) floor slabs used for car parking, loading and unloading, vehicular ramp or pedestrian pavement.

(d) For structures such as transformer rooms and pump houses, the foundations should be carried down through the reclaimed materials to a firm stratum with the lowest floor slabs designed as suspended.

(e) Underground utilities and drainage underneath a building should be supported by suspended floor slabs or pile caps. The pipe connection at the interface between

the structurally supported portion and the on-grade portion of pipes should be designed to accommodate differential settlement due to the subsidence of the latter.

(f) Where significant settlement due to long-term consolidation of the ground is anticipated, measures should be provided in the pile cap design to mitigate the migration of soil into any void that may be formed underneath the pile cap due to consolidation of the ground below.

2.4.2 ALTERNATIVE APPROACH

Where it is intended not to follow the design rules given in Clause 2.4.1 above, the problem of differential and total settlement should be fully considered. The time-related total and differential settlement (including predicted time-settlement curves) should be assessed based on site-specific ground investigation, and measures to overcome or accommodate the problem should be provided.

To ensure the reliability of the time-settlement relationship estimated at the design stage, continuous settlement monitoring (through instrumentation) throughout the construction period should be carried out and the assessment of the settlement should be reviewed from time to time.

In the settlement assessment, reference may be made to the settlement measurements collected during the reclamation period and any previous settlement assessments made for the reclamation. However, such data should only be used as reference to the historical settlement characteristics of the site or as supplementary information to the site-specific assessment unless their accuracy can be guaranteed. The historical settlement record and settlement assessment for government reclamation, if available, can be obtained from the government department who undertook the reclamation projects, usually the Civil Engineering and Development Department.

2.4.3 LONG-TERM MONITORING AND/OR MAINTENANCE

Where the design of structures requires long-term monitoring and/or maintenance, the designer, through the AP, should alert the developer of such requirements and their implications and advise him to inform any prospective buyers who may have to bear the costs for such requirements.

2.5 STRUCTURAL REQUIREMENTS

2.5.1 GENERAL

The structural design of foundations should comply with the provisions of the Building (Construction) Regulations.

2.5.2 DESIGN LOADS

The foundation of a building shall be designed to carry the working load with adequate factor of safety. Dead load, imposed load and wind load should be assessed in accordance with the Building (Construction) Regulations and the relevant codes of practice. The imposed load should include buoyancy force and earth pressure. Buoyancy force should be assessed in accordance with Clause 2.5.4. Earth pressure should be assessed by using recognized geotechnical engineering methods.

Where it is necessary to carry out foundation design based on a set of assumed loads, a detailed schedule of the assumed loads should be prepared and, before the commencement of the construction of the superstructure, it is necessary to demonstrate that the loads from detailed calculations of the superstructure do not exceed the assumed loads used in the foundation design.

2.5.3 RESISTANCE TO SLIDING, UPLIFT AND OVERTURNING

A building or structure shall be so designed and constructed such that :

(a) the sliding resistance shall be at least 1.5 times the sliding force due to any loads;
(b) the uplift resistance shall be at least 1.5 times the uplift force due to any loads, with the uplift resistance taken as the sum of the downward force due to the minimum dead loads plus that due to any permitted anchoring resistance; and
(c) the overturning moment resistance shall be at least 1.5 times the overturning moment due to wind loads, 1.5 (or 1.1 as required by clause 2.5.4) times the overturning moment due to floatation and 2 times the overturning moment due to loads other than wind loads and floatation, with the overturning moment resistance taken as the sum of the stabilizing moment due to the minimum dead loads plus that due to any permitted anchoring resistance.

The foundations shall be so designed and constructed to fulfil the requirements above.

Marine structures should also be designed and constructed such that the resistance to sliding, uplift and overturning satisfies the requirements of acceptable standards or codes of practice for design of maritime structures, such as “Port Works Design Manual” published by Civil Engineering and Development Department.

2.5.4 RESISTANCE TO BUOYANCY

A structure may resist buoyancy by its own weight plus any other suitable anchoring resistance such as tension piles or ground anchors. Any structure satisfying either one of the following criteria, further checking on the stability of the structure against overturning due to buoyancy may not be explicitly required :

(a) a minimum factor of safety of 1.5 against floatation which is due to the highest anticipated groundwater level whereas the resistance is taken as the combined dead loads and permitted anchoring resistance.

(b) a minimum factor of safety of 1.1 against floatation where buoyancy is due to the highest possible groundwater level and the resistance is taken as the minimum dead loads only.

For the purpose of calculating resistance to buoyancy, the dead loads should be taken as the weight of the structural elements plus the weight of any permanent finishes and backfill. In the dead load calculations, conservatively assumed values or the actual thickness and densities of the finishes and the backfill should be used. Finishes and backfill that could be removed should be ignored in the calculations.

Highest Anticipated Groundwater Level

In determining the highest anticipated groundwater level, the following conditions should be taken into consideration :

(a) the current and projected tidal variations;
(b) the design free surface water levels due to storm, wind surge and pounding;
(c) the design groundwater level taken into account the influences of rainfall, surface water run-off and groundwater movement;
(d) the damping of seawater tide influence by intervening ground;
(e) dewatering;
(f) the long term rise in sea level; and
(g) ground permeability.

Highest Possible Groundwater Level

The highest possible groundwater level refers to the level above which the groundwater would not rise under all possible extreme events such as severe rainfall, flooding and bursting of water mains. In the absence of reliable data, the highest possible groundwater level may generally be taken as the ground surface. However, in low-lying areas such as reclamation, it may rise even above the ground surface.

2.5.5 MATERIALS AND STRESSES

(1) General

Materials and stresses shall comply with the requirements of the Building (Construction) Regulations and the relevant codes of practice.

Where the permissible stress method is used in the structural design of foundation members, the working stress may be increased by not more than 25% where such increase is solely due to wind loads.

(2) Concrete

The concrete used for foundation elements shall comply with the Building (Construction) Regulations and the relevant codes of practice. Subject to the provisions of this Code, the design of the reinforced concrete elements of a foundation should be carried out in accordance with the Code of Practice for

Structural Use of Concrete, using either the limit state or the permissible stress method.

For cast-in-place concrete foundations, the concrete strength should be reduced by 20% where groundwater is likely to be encountered during concreting or where concrete is placed underwater.

Cast-in-place concrete foundations of least lateral dimension not exceeding 750 mm should also comply with the following requirements :

(a) For piles subject to axial forces only, where the concrete used is higher than grade 20D, it should only be assumed as grade 20D for design purpose;
(b) For piles subject to lateral forces, the concrete used should not be inferior to grade 25D; where higher concrete grade is specified, it should only be assumed as grade 25D for design purpose; and
(c) contribution to lateral resistance due to flexure of piles with diameter less than 400mm should be ignored.

The axial compressive stress on a driven precast concrete pile under working loads should not exceed 20% of the specified concrete grade strength.

For marine foundations, concrete should not be inferior to grade 45D as required in Clause 2.6.4. Where the concrete is placed under water, the concrete should be assumed as grade 25D for design purpose.

(3) Steel

Subject to the provisions of this Code, the design of the structural steel elements of a foundation should be carried out in accordance with the Code of Practice for Structural Use of Steel.

For driven steel bearing piles with a design safety factor on driving resistance of 2, the stress in the steel at working load should not exceed 30% of the yield stress. The design working stress due to combined axial load and bending may, however, be increased to 50% of the yield stress.

For steel bearing piles installed in pre-bored holes or jacked to the required depth, in which no peak stresses due to impact are set up, higher working stresses up to 50% of the yield stress can be used.

For steel piles subject to compression, the allowable bond stress between steel and grout (with a minimum characteristic strength of 30 MPa at 28 days) may be taken as 600 kPa (or 480 kPa when grouting under water).

For steel piles subject to tension, the allowable bond stress between steel and grout (with a minimum characteristic strength of 30 MPa at 28 days) may only be taken as 600 kPa (or 480 kPa when grouting under water) where nominal

shear studs are provided. As a general rule, nominal shear studs should be adequate to resist not less than 25% of the tension.

For steel piles relying on the bond between steel and grout to resist tension or compression loads, the pile surface should be clean and free from loose mill scale, loose rust or any substance that may reduce the bond.

For corrosion protection of marine foundations, the guidelines as given in Clause 2.6.4 should be followed.

2.6 CORROSION PROTECTION OF FOUNDATIONS

2.6.1 GENERAL

Foundations should be provided with adequate protections against corrosion, or alternatively, they should be suitably designed to allow for the effect of corrosion which may take place during their designed working life.

To ensure effective and economical designs for protection against corrosion, information on the presence of any corrosive material in the ground and the range of fluctuation of ground water table should be obtained.

2.6.2 CONCRETE FOUNDATIONS

Provisions for corrosion protection of concrete foundations should be given in the foundation plans where :

(a) sulphate, chloride, aggressive chemical or other agents causing deterioration is present in the ground;
(b) alkali are present in the concrete and a high moisture content environment exists;
(c) the foundations are constructed on a landfill site; or
(d) damage by abrasion may occur.

To avoid the alkali-aggregate reaction occurring in reinforced concrete structures, the reactive alkali of concrete expressed as the equivalent sodium oxide per cubic metre of concrete should not exceed 3.0 kg.

2.6.3 STEEL PILES

Provisions for corrosion protection of steel piles should be given in the foundation plans where :

(a) sulphate, chloride, aggressive chemical or other agents causing deterioration is present in the ground;
(b) the piles are placed at the splash and tidal zones of the sea;
(c) the piles are in contact with other metals;

(d) stray direct electric current is present; or
(e) damage by abrasion may occur.

2.6.4 MARINE FOUNDATIONS

Corrosion protection of marine foundations should be provided in accordance with acceptable standards or codes of practice for design of maritime structures, such as "Port Works Design Manual Part 1" published by Civil Engineering and Development Department. The following should be considered as general guidelines :

(1) Concrete

Concrete should be of high density and low permeability. It should not be inferior to grade 45D and the water/cement ratio should not exceed 0.38.

Nominal concrete cover to all reinforcement should be :

(a) 75 mm where fully immersed or within the tidal and splash zones, and
(b) 50 mm where above the splash zone.

Crack widths of concrete within tidal and splash zone should not exceed 0.1 mm under typical average long term loading conditions, which may be increased by a factor of 1.25 for flexural crack width design and control purpose.

Correct use of pulverized fuel ash in the concrete mix may increase resistance of concrete against sulphate attack.

(2) Steel

All structural steelwork above seabed level, whether fully immersed, within the tidal or splash zones, or generally above the splash zone, should be fully protected against corrosion for the design working life of the structure. Below seabed level, an allowance for corrosion loss of 0.05 mm per year on the outside face of steel is considered reasonable if no corrosion protection is carried out within this zone.

Stainless steel for use in marine environment should be of a grade which is absolutely free of any chloride. Common grade of stainless steel with the presence of chloride should not be used for marine works.

Steel embedded in concrete and steel in seawater in the same foundation should be isolated since the former is cathodic relative to the latter.

2.7 FOUNDATION PLANS

Foundation plans should consist of adequate and relevant information so as to demonstrate the entire physical and conceptual designs. A typical foundation plan should include the following two parts :

(1) A foundation plan with :

(a) a block plan showing the location of the site;

(b) details showing the characteristic features of the site and environs including locations of ground investigation boreholes, slopes, existing foundations, nullahs, retaining walls and the like;

(c) layout arrangement, identification, expected depths and founding levels, structural details and material specifications of the foundations;

(d) layout arrangement of the pile caps if applicable;

(e) bearing capacity of foundations and method of verification on site;

(f) magnitude of characteristic dead, imposed and wind loads and their critical combinations acting on the foundations (for piled foundations, this should be given for each pile or each group of piles);

(g) installation specifications, such as founding criteria, method of installation, method of overcoming underground obstruction;

(h) details of monitoring requirements for adjacent and nearby buildings, structures, land, street and services;

(i) for piled foundations, method of controlling and monitoring the verticality, inclination and alignment of piles during installation;

(j) where dynamic pile driving formula is used, the parameters for the assessment of the ultimate pile capacity, such as the effective energy per blow, efficiency of blow and penetration of pile for a hammer blow.

(2) Supporting documents:

(k) site investigation report with results of ground investigation, field and laboratory tests and photographs of all the soil samples and rock cores taken;

(l) design calculations based on recognized foundation engineering principles; and

(m) appraisal report on the effects of the foundation works on surrounding land, structures and services, including any proposal of precautionary and protective measures.

For item (m) above, reference should be made to Clause 7.2.

2.8 FOUNDATION DESIGN IN SCHEDULED AREAS

In accordance with the Fifth Schedule of the Buildings Ordinance, there are five Scheduled Areas specified in Hong Kong at present. These areas are:

Area No.	Scheduled Areas
1	Mid-levels
2	North-west New Territories
3	(1) Mass Transit Railway Protection Areas
	(2) Kowloon-Canton Railway Protection Areas
4	Ma On Shan
5	Sewage Tunnel Protection Areas

Particular requirements for foundation design in the Scheduled Areas are given in the relevant PNAP issued by the Building Authority. When planning and designing foundations in the Scheduled Areas, reference should be made to these PNAP.

2.9 FOUNDATION DESIGN IN DESIGNATED AREAS

Designated areas, such as Northshore Lantau, refer to those areas with complex geology, which may impose significant constraints on the foundation design and construction. In some cases, the foundations might prove to be so costly that adjustment of the layout of the development or even site abandonment is warranted. A very comprehensive ground investigation is usually inevitable in order to identify all the geological constraints, and it should be carried out before planning the development. Reference should be made to the relevant PNAP issued by the Building Authority for the locations and other information for the designated areas.

2.10 FOUNDATION DESIGN IN SLOPING GROUND

Where the foundation has imposed additional loads on slopes or retaining walls, or the groundwater regime has been affected, the stability of the affected slopes or retaining walls should be checked as part of the foundation design.

3. SITE INVESTIGATION

3.1 GENERAL

Where foundations are to be constructed, a site investigation should be undertaken to provide all the necessary information for the design and construction of the foundations. For this purpose, a site investigation report should be prepared, giving details of the results of any documentary studies, site surveys and ground investigations, together with the appraisal of the surface and subsurface conditions of the site.

Site investigations should be carried out in accordance with GEOGUIDE 2 and GEOGUIDE 3, and should comply with the provisions of this code of practice.

3.2 DOCUMENTARY STUDIES

Documentary studies should be carried out before planning a site investigation for the design of foundation works. Useful information is available in the Geotechnical Information Unit of the Civil Engineering Library in the Civil Engineering and Development Department. However, care should be taken in using any ground investigation data obtained from previous building projects. The accuracy of such data should be verified before they are used in the design of foundations. Any doubtful data should be discarded.

3.3 SITE SURVEY

The site survey should include a detailed survey of the land, structures and services within and adjacent to the site. All relevant information obtained should be shown on the foundation plan.

(1) Topographical Survey

The topographical survey should identify the topography of the area, including the nature and conditions of the land within and adjacent to the site. The following should be checked to identify any sloping conditions within or nearby the site :

(a) any land with an average gradient measured from boundary to boundary, or across any 50 m strip of the site, greater than 15°; and

(b) any slope with an average gradient greater than 15° within the site or within 50 m from the site boundaries.

(2) Geological Survey

The geological survey should identify all geological information of the site, including any particular geological features, rock outcrops, previous landslides and site formation works, etc.

(3) Survey of structures

The survey of structures should identify the following :

(a) any existing building, structure and foundation in the vicinity of the site; and
(b) any existing retaining wall within and adjacent to the site.

The stability and structural conditions of any nearby existing buildings and structures, in particular pre-war buildings, appendages, party walls etc., which are likely to be affected by the proposed foundation works, should be assessed and reported. Proper records including photographs of any existing building defects should be kept for future reference.

The information on the foundations of such buildings or structures should be obtained for the assessment. Where such information is not available, inspection by trial pit or by drilling vertical and horizontal investigation holes through the foundations may have to be carried out to uncover dimensional and material information of the foundations.

(4) Survey of any disused tunnel, culvert, nullah or stream course

The survey should identify any disused tunnel, culvert, nullah, stream course or ground anchor within and nearby the site.

The presence of any disused tunnel may have an important bearing on the design and construction of the foundations, and may lead to restrictions to be imposed on the foundation design. Records of disused tunnels may be obtained from the Geotechnical Information Unit of the Civil Engineering Library in Civil Engineering and Development Department.

The presence of any culvert, nullah or stream course may seriously prejudice the proposed development. As a general principle, no building should be permitted over a main arterial storm water nullah. Information on nullahs and stream courses, which may not be easily detected in the dry season, may be available from Geotechnical Engineering Office of the Civil Engineering and Development Department or Drainage Services Department.

(5) Survey of underground services

The site investigation should identify all the underground services within and nearby the site. The common types of underground services include water

mains, sewage tunnels, electricity cables, gas mains, drainage pipes, telephone and other communication ducts.

Information regarding underground services, as well as the amount of settlement or movement that they could tolerate, may be obtained from the relevant government departments and utility companies. Such information is often indicative in nature, and should be verified on site.

3.4 GROUND INVESTIGATION

3.4.1 GENERAL

Ground investigation works involve the field works carried out to obtain information on the ground conditions, such as exploratory drilling, excavation and probing of land, installation of instruments, sampling and field testing, and the laboratory works for the testing of samples obtained from such operations.

All ground investigation field works, except field density tests, should be carried out by a Registered Specialist Contractor (Ground Investigation Field Works), whereas all laboratory testing of samples and field density tests should be carried out by laboratories accredited under the Hong Kong Laboratory Accreditation Scheme (HOKLAS).

Sufficient ground investigation works should be carried out to ensure that adequate information on the subsurface conditions within and nearby the site can be obtained prior to the design of the foundation.

3.4.2 SUPERVISION FOR GROUND INVESTIGATION WORKS

Proper supervision should be provided for the carrying out of all ground investigation works. To ensure quality of the works, supervision for the different stages of pre-design ground investigation field works should comply with the requirements set out in the relevant PNAP or code of practice issued by the Buildings Department.

3.4.3 PREPARATION OF GROUND INVESTIGATION REPORTS

To substantiate the design of the foundation, a proper ground investigation report should be compiled. The ground investigation report should include adequate information for the design of the foundation and the subsequent construction works, and/or for other purposes such as site formation works, when required. For detailed requirements of the contents of ground investigation reports, reference should be made to GEOGUIDE 2, and the relevant PNAP and code of practice issued by the Buildings Department.

3.4.4 NUMBER AND DISPOSITION OF BOREHOLES/TRIAL PITS

The disposition and spacing of boreholes/trial pits should be such as to reveal any significant changes in properties, thickness or depth of the strata. The number of

boreholes/trial pits required will vary with the size, type and performance requirements of the structure, the general condition of the site and the completeness or otherwise of available geological records.

It should be noted that the ground conditions in Hong Kong, including bedrock levels, may vary significantly within a short distance. The number of boreholes/trial pits should be sufficient to reveal such variation.

Where it is intended to have the foundation rest on rock, sufficient boreholes should be taken to ensure a reliable prediction of the rockhead level.

3.4.5 DEPTH OF GROUND INVESTIGATION

Boreholes and trial pits should be carried out to sufficient depth such that all strata that are likely to be affected by the foundation loads will be adequately explored. The depth of ground investigation will depend on the type of the structure, the size, shape and disposition of the loaded areas and the nature of the strata. In general, the following should be considered :

(a) Care should be taken to ensure that boulders are not mistaken for bedrock, particularly on sites where borings are made through highly weathered rock. Where bedrock is encountered, it should be proved by coring to a minimum depth of 5 m.

(b) Where compressible cohesive soils are likely to contribute significantly to the settlement of the foundations, the investigation should reach such depth where stress increase would cause insignificant strain or displacement.

(c) For the design of a pile foundation or for the settlement analysis of the foundation, the exploration should be deep enough to cover the characteristics of the underlying strata.

(d) For spread footings, the investigation depth should be :
   (i) 5m into rock; or
   (ii) a depth that can be readily demonstrated that the induced strain and displacement are negligible.

3.4.6 GROUNDWATER

Groundwater may be critical in foundation design and construction. Particularly where dewatering is likely to be required, adequate information on groundwater and geological conditions including permeability, compressibility and consolidation characteristics of the various soil strata, particle size analyses and other test results relevant to the consideration of dewatering activities and preventive measures against settlement should be obtained.

It is generally necessary to install standpipes or piezometers to measure the groundwater levels over an extended period of time for accurate identification of the groundwater conditions and verification of design assumptions.

Where the groundwater or soil may contain constituents in amounts sufficient to damage or affect the foundation structures, chemical analysis of samples of the groundwater and soil should be carried out. Protection of the foundation structures from the effect of such constituents or other appropriate measures should be provided if found necessary.

3.5 GROUND INVESTIGATION IN SCHEDULED AREAS

Ground investigation works in the Scheduled Areas are subject to special control under the Buildings Ordinance and Building Regulations. Approval of the ground investigation plan as prescribed under Building (Administration) Regulation 8(1)(l) and consent from the Building Authority are required before the works can be commenced on site.

Particular requirements for ground investigation in the Scheduled Areas are given in the relevant PNAP issued by the Building Authority. When planning the ground investigation for foundation works in the Scheduled Areas, reference should be made to these PNAP.

4. SHALLOW FOUNDATIONS

4.1 GENERAL REQUIREMENTS

A shallow foundation should be structurally adequate to sustain all the applied loads and transmit them safely to the ground without undue settlement. It should generally be constructed of reinforced concrete, and rest on a rock or soil stratum with adequate bearing capacity at a shallow depth from ground level.

A shallow foundation should neither overload the foundations or structures of adjacent buildings or the ground supporting such foundations or structures, nor render any instability to any hillside or slope, nor interfere with any drain, nullah, sewer or other services in its vicinity.

4.2 ALLOWABLE BEARING PRESSURE AND SETTLEMENT

The allowable bearing pressure of rock or soil supporting a shallow foundation should be determined in accordance with Clause 2.2.

The settlements of the foundation should be estimated and checked in accordance with Clauses 2.3.1 and 2.3.2 respectively.

4.3 STRUCTURAL REQUIREMENTS

Structural design of shallow reinforced concrete foundations should be carried out in accordance with the Code of Practice for Structural Use of Concrete.

The stability of shallow foundations should satisfy the provisions of Clause 2.5.3.

4.4 COMMON TYPES OF SHALLOW FOUNDATIONS

There are three types of shallow foundations commonly used in Hong Kong, namely pad footing, strip footing and raft foundation.

4.4.1 PAD FOOTINGS

In the design of pad footings not founded on rock, the probable total settlements of individual footings and the probable differential settlements between footings should be estimated and checked in accordance with Clauses 2.3.1 and 2.3.2 respectively. In some cases, it may be necessary to enlarge some footings in order to avoid significant differences in bearing pressures and settlements among footings supporting the same building. Where differential settlements may occur between these individual footings,

the elements of the superstructure should be adequately designed for the bending moments and shear forces caused by the differential settlements.

4.4.2 STRIP FOOTINGS

The requirements for pad footings should also apply to strip footings.

The founding conditions for a strip footing should be consistent to avoid differential settlements along its length. However, where differential settlements along its length may occur, the strip footing should be designed with adequate strength to resist the effect of the differential settlement.

4.4.3 RAFT FOUNDATIONS

The requirements for pad and strip footings should also apply to raft foundations.

Where it is necessary to design a raft for differential settlements, the raft should be designed with adequate strength to resist the effect of the differential settlements in both directions.

5. PILE FOUNDATIONS

5.1 GENERAL

All pile foundations should be durable, of adequate load carrying capacity and of a recognized type suitable for the ground conditions. The piles should be able to withstand the expected wear and deterioration throughout the intended design working life of the superstructure that they support.

The allowable load on pile foundations shall be determined by :

(a) acceptable foundation engineering principles; or
(b) tests on the foundations on site,

with an adequate factor of safety appropriate to the type of pile, taking into account the ground conditions, the method of installation, group effects and the allowable displacements of the structures supported by the foundation.

5.1.1 RECOGNIZED TYPES OF PILE FOUNDATIONS

A recognized type of pile foundation is a piling system which has been proved satisfactory to the Building Authority and incorporated into a list which is available from the homepage of the Buildings Department.

The Registered Structural Engineer (RSE) usually in conjunction with the Registered Specialist Contractor experienced in a piling system which is not a recognized type may seek recognition of the system by submitting all technical details of the system to the Building Authority, including material specification, manufacturing process, method of installation, method of assessing pile capacity, applicability relating to ground conditions and selected examples of uses of the system elsewhere, if applicable. A demonstration of the performance of the system is usually required.

Application for a recognized type of pile should be made prior to seeking approval of foundation plans using such type of pile whenever possible.

5.1.2 GROUP EFFECT

A pile group exists when piles are closely placed such that the load carrying capacity and settlement behaviour of a pile may be affected by other piles.

In such case, a group reduction factor determined by recognized foundation engineering principles shall be applied to the total allowable load carrying capacities of any group of piles. A group reduction factor of 0.85 may be considered as generally acceptable for a group of 5 or more vertically loaded piles. Other values of group reduction factor may be used having considered the particular ground condition of the site and justified by recognized engineering principles.

Generally, group reduction factors need not be applied where :

(a) the centre-to-centre spacings are of more than 3 times the perimeter of the piles or the circumscribed rectangles in the case of H-piles; or
(b) the load capacity of the piles is derived from end-bearing; or
(c) the piles are rock-socketed piles; or
(d) the piles are driven to refusal with the toes resting closely onto bedrock (see Clause 5.4.11(2)(c)).

5.1.3 MINIMUM PILE SPACING

The spacing of piles shall be determined with due regard to the nature of the ground, the method of construction, the group effects and shall be sufficient to prevent damage to the piles or any adjacent construction.

For driven piles and other piles, which derive their capacities mainly from frictional resistance, the minimum pile spacing shall be not less than the length of the perimeter of the pile or 1m, whichever is the greater, and the piles shall be placed at not less than half the length of the perimeter of the pile or 500 mm, whichever is the greater, from the site boundary.

For mini-piles, which derive their capacities mainly from bond strength between grout and rock, the minimum pile spacing should be 750mm or 2 times the outer diameter, whichever is the greater.

For bored piles and the like which derive their capacities mainly from end bearing, the minimum clear spacing between the surfaces of adjacent piles should be based on practical considerations of positional and verticality tolerances of piles. It is recommended to provide a nominal minimum clear horizontal spacing of 500 mm between shaft surfaces or edge of bell-outs, as appropriate.

5.1.4 HORIZONTAL RESTRAINTS TO PILES AND PILE CAP

Piles and pile caps shall have adequate lateral stability and be able to cope with any allowed construction tolerance. For driven piles and small diameter piles, adequate horizontal restraints in at least 2 directions shall be provided to individual piles or pile caps.

5.1.5 PILES PROVIDING RESISTANCE AGAINST SLIDING

Where piles are required to provide lateral resistance against sliding, it should be demonstrated that the piles and the supporting ground have adequate lateral capacities to satisfy the requirement specified in item (a) of Clause 2.5.3.

5.1.6 PILES PROVIDING RESISTANCE AGAINST UPLIFT, OVERTURNING AND BUOYANCY

Where piles are required to provide anchorage resistance against uplift, overturning and/or buoyancy, it should be demonstrated that the piles and the supporting ground have adequate anchorage capacities to satisfy the requirements specified in items (b) and (c) of Clause 2.5.3, and/or the requirement specified in Clause 2.5.4.

Where adequate structural continuity is provided to connect two or more pile caps together so that they become an integral unit, the above stability check is only required for the entire integral unit, i.e. only the forces on those critical piles within the integral unit need to be checked.

Alternatively, the above requirements may be deemed to be satisfied by demonstrating that each pile in a pile foundation satisfies the following condition individually :

$$D_{min} + 0.9R_u - 2.0I_a - 1.5U - 1.5W \geq 0$$

where D_{min} = Minimum dead load
R_u = Ultimate anchoring resistance of the pile (see Clause 5.3.3)
I_a = Adverse imposed load including live and soil loads
U = Uplift due to the highest anticipated groundwater table
W = Wind load

5.2 NEGATIVE SKIN FRICTION

5.2.1 DESIGN REQUIREMENT

Where pile foundations are installed through strata which are likely to undergo consolidation after the foundations are in place, the frictional resistance of the strata and the overlying soils shall not be taken into account in the determination of the load carrying capacity of the foundations. The downward frictional force exerted from the strata and the overlying soils, commonly named as negative skin friction (NSF), should be considered in the foundation design by using either the Conventional Approach or the Alternative Approach given in Clause 5.2.2 and 5.2.3 respectively.

NSF on a pile may be reduced by coating the pile surface with bitumen or asphalt. Usually a proprietary system should be used. Extreme care should be taken to avoid damage of the coating. It may be necessary to demonstrate by site trials that the coating will not be damaged during the pile installation. Other details, such as the use of a double skin permanent liner infilled with inert flexible material, may also be considered in reducing or eliminating NSF.

5.2.2 CONVENTIONAL APPROACH

In this approach, the downward frictional forces exerted from the strata and the overlying soils shall be considered as imposed load. That is usually achieved by demonstrating that the following conditions are satisfied:

$$P \geq D + L + NSF; \text{ and}$$
$$P_w \geq D + L + W + NSF$$

where P is the pile capacity without wind,
P_w is the pile capacity with wind,
D is the dead load,
L is the live load,
W is the wind load, and
NSF is the negative skin friction.

Unless more accurate assessment is made, the following equation may be used to estimate the negative skin friction acting on a pile :

$$NSF = \int_0^l K_o \sigma' \tan\delta \, pdl$$

where K_o = the at-rest earth pressure coefficient;
σ' = the effective vertical pressure;
δ = the friction angle between the surface of the pile and soil;
p = the perimeter of the pile, or perimeter of the circumscribed rectangle in the case of H-pile;
l = the depth of the consolidation strata; and
dl = the elemental length of the pile.

Where a pile group exists and a group reduction factor has been applied in accordance with Clause 5.1.2, the same reduction factor may also be applied to the NSF.

5.2.3 ALTERNATIVE APPROACH

In the alternative approach, the ground-bearing capacity, the structural integrity and the settlement behaviour of the pile foundation are considered separately under the following criteria:

(a) The allowable ground-bearing capacity of the piles should be adequate to resist the total loads on the piles excluding NSF, i.e.

$$P_c \geq D + L; \text{ and}$$
$$P_{cw} \geq D + L + W$$

where P_c is the allowable ground-bearing capacity of the piles without wind,

P_{cw} is the allowable ground-bearing capacity of the piles with wind,
D, L and W are as defined in Clause 5.2.2.

NOTE:

This criterion is based on the consideration of the limit state where the ground supporting the pile has reached the ultimate condition. It is considered that the pile settlement under this limit state is larger than the settlement that induced the NSF. Therefore all NSF in the pile will be eliminated and need not be considered in the following equation/condition:

Ultimate ground-bearing capacity of pile ≥ ultimate loads excluding NSF

Assuming the allowable ground-bearing capacity of pile has a factor of safety not less than 2 and the load factors for the ultimate loads are not greater than 2, then the above equation/condition may be simplified as:

Allowable ground-bearing capacity of pile ≥ working loads excluding NSF

which thus becomes the equations/conditions given above.

(b) The structural integrity of the piles should be adequate to resist the total loads on the piles including NSF, i.e.

$P_s \geq D + L + NSF$; and
$P_{sw} \geq D + L + W + NSF$

where P_s is the structural strength of the piles without wind,
P_{sw} is the structural strength of the piles with wind,
D, L, W and NSF are as defined in Clause 5.2.2.

(c) The settlement behaviour of the piles under total loads including NSF should be satisfactory.

The value of NSF may be calculated in accordance with the recommendations given in Clause 5.2.2. For design in accordance with the alternative approach, the test load for any static load testing of the piles should not be less than $2P_c$ + NSF.

5.3 LOAD CAPACITY OF PILES

The allowable load capacity of a pile should be determined from its structural strength and the resistance of ground supporting the pile. The allowable load capacity of a pile group should take into account the group effect described in Clause 5.1.2.

5.3.1 STRUCTURAL STRENGTH

The structural strength of a pile should be determined in accordance with the appropriate limitations of design stresses as given in Clause 2.5.5. For piles embedded in soft strata, the buckling capacity of the piles should be checked.

5.3.2 GROUND RESISTANCE FOR PILES IN COMPRESSION

The ultimate or allowable bearing capacities of driven and non-driven piles may be assessed by the following methods :

(1) Driven piles

For driven piles, the ultimate bearing capacity may be assessed by using any one or more of the following methods:

(a) a dynamic formula based on the data obtained from test driving the pile on site;
(b) a static formula based on design parameters of the supporting soil obtained from suitable tests; or
(c) loading test of the pile on site.

The driving hammer should be large enough to efficiently overcome the inertia of the pile. In the dynamic formula calculation, the final set that can achieve the required ultimate resistance should not be less than 25mm per 10 blows unless rock has been reached.

A suitable factor of safety should be adopted when deriving the allowable bearing capacity of the piles. In general, a factor of safety not less than 3 should be used for a static formula and those formulae for which lower factors of safety have not been established. In no cases should the factor of safety be less than 2.

(2) Non-driven piles

For non-driven piles, the allowable bearing capacity may be determined by :

(a) the founding condition of the pile; and
(b) the allowable bearing pressure and bond or frictional resistance of the ground as stipulated in Clause 2.2.

For piles socketed in category 1(a), 1(b), 1(c) or 1(d) rock as defined in Table 2.1, the load-carrying capacity of the piles may be taken as the sum of the bond or frictional resistance of the rock socket and the end bearing resistance of the piles provided that the socket length used in the calculation of bond or frictional resistance does not exceed 2 pile diameters or 6 m, whichever is the shorter. The contribution from the minimum socket depth stipulated in Note (3) of Table 2.1 should be ignored in the calculation of bond or frictional resistance. The bond or frictional resistance on the inclined faces of bell-out of bored piles should also be ignored.

For all other piles, the load-carrying capacity of the piles should not be derived from a combination of the shaft resistance and end bearing resistance of the piles unless it is justified that the settlements under working load conditions

are acceptable and adequate to mobilize the required shaft resistance and end bearing resistance of the piles simultaneously. Verification by tests on instrumented trial piles and/or monitoring of the settlement may be required.

5.3.3 GROUND RESISTANCE FOR PILES SUBJECTED TO UPLIFT FORCES

Piles subject to uplift forces should satisfy the following requirements:

(a) The ultimate anchorage resistance of the piles satisfy the requirements stipulated in Clause 5.1.6; and

(b) The allowable anchorage resistance, R_a, of the piles can sustain the uplift force in the piles under working load conditions, that is :

$$R_a = I_a + U + W - D_{min}$$

where I_a, U, W and D_{min} are as defined in Clause 5.1.6.

In general, the allowable anchorage resistance of a pile may be taken as:

R_a = allowable uplift resistance of pile shaft + effective self weight of pile

Where the allowable uplift resistance of the pile shaft is based on the ultimate uplift capacity of the pile shaft, the applied factor of safety should not be less than 3 unless the ultimate uplift capacity or the parameters for assessing the ultimate uplift capacity have been verified by tests. In no cases should this factor of safety be less than 2.

Where the ultimate uplift capacity of the pile shaft is based on the allowable uplift resistance of the pile shaft, the applied load factor should not be greater than 2.

The allowable anchorage resistance of a pile group should be the lesser of the following:

(a) the sum of the allowable anchorage resistance of the piles in the group; and
(b) the allowable shear resistance mobilised on the surface perimeter of the group plus the effective weight of soil and piles enclosed in this perimeter.

Verification test is normally required to justify the tension capacity of piles unless such capacity is taken as less than half of the compressive capacity resulting only from shaft friction and bond between the pile and the surrounding soil. In any case, the adequacy of the related soil mass and rock cone supporting the pile should be checked for uplifting effect.

5.3.4 GROUND RESISTANCE FOR PILES SUBJECTED TO LATERAL LOAD

In the design of piles resisting lateral forces, consideration should be given to :

(a) the shear capacity of the soil, taking into account the group effect where appropriate;
(b) the structural capacity of the pile; and
(c) the deflection of the pile that the superstructure may tolerate.

The lateral load capacity of a pile should be determined by recognized foundation engineering methods with due regard on the characteristics of surrounding soil, the characteristics of the pile or pile group, and the interaction between the surrounding soil ad the pile or pile group. When methods based on simplified assumptions and graphical solutions are used in the analysis, the parameters adopted should represent the characteristics of the pile and the soil.

Pile and pile caps should not be used together to resist lateral forces unless the distribution of forces between piles and pile caps can be demonstrated. Friction at the sides and bases of pile caps, basement walls, drag walls or other sub-structures should not be considered in assessing the lateral resistance of the foundations, unless it can be demonstrated that they are compatible and can be mobilized simultaneously without causing unacceptable disturbance to the ground or adjacent structures and services.

Where piles are designed with rock sockets to provide lateral resistance, the stability of rock mass should be checked. For sockets formed in bedrock, the allowable lateral resistance of rock may be taken as one-third of the allowable vertical bearing pressure of the rock provided that no adverse rock joints exist.

Where piles installed on a slope are required to resist lateral load, the effect of the slope on the lateral load capacity of the piles should be considered, and the requirements of Clause 2.10 should be complied with.

5.4 COMMON PILE TYPES

This Clause contains the particular requirements for some specific types of pile commonly used in Hong Kong.

5.4.1 STEEL H-PILES/STEEL TUBULAR PILES

Steel H-piles and tubular piles are usually installed by driving or pre-boring. The design of steel H-piles and tubular piles should follow the recommendations given in Clauses 5.1 to 5.3.

In addition to those stated in Clause 2.7, the foundation plan for such piles should also include :

(a) the physical properties and chemical composition of the piles, especially where the sectional properties are not covered by international standards;
(b) the details of pile splice and pile head;
(c) the details of protection of pile tips where hard driving is anticipated; and
(d) the orientation of steel H-piles designed to resist lateral loads.

Spliced Piles

When the piles are to be lengthened, splices can be made by welding :

(a) welding of the splice should comply with acceptable standards and in accordance with the supplier's instructions and recommendations on weld rods.
(b) preheating may be required to eliminate possible hydrogen-induced cracking in the welded joints.
(c) a sampling rate of not less than 10% of the total number of welded joints should be tested by means of non-destructive tests prior to driving in the spliced sections of the piles. Such test reports, with the joint locations specified, should be included in the piling records.

Pile Head Design

The pile head and the pile cap must be sufficiently designed to transmit the maximum pile loads. Where the pile head is designed as fixed to improve stability of the piles, the pile head must be designed and constructed to achieve the fixed head condition.

5.4.2 SOCKETED STEEL H-PILES

Socketed steel H-piles are installed by inserting steel H-piles into pre-bored holes sunk into bedrock, and subsequently grouting the holes with cementitious materials.

Design Principles

The following design principles should be adopted in general :

(a) The allowable axial working stress or the combined axial and flexural stresses should not be greater than 50% of the yield stress of the steel H-pile;

(b) The rock socket should be formed in category 1(a), 1(b) or 1(c) rock as defined in Table 2.1;

(c) The design bond strength between the rock and grout should not exceed the allowable value given in Clause 2.2.2;

(d) The design bond strength between the grout and the steel H-pile should not exceed the allowable value given in Clause 2.5.5(3);

(e) The grout should be non-shrink and have a minimum characteristic strength of 30 MPa at 28 days; and

(f) Rock sockets subjected to lateral load should be checked for any additional stresses induced.

Construction Considerations

The following should be given due consideration when installing socketed steel H-piles :

(a) The prebored holes should be of adequate size to enable the insertion of the steel H-pile and to allow a minimum grout cover of 40 mm to the pile (except at the base);

(b) A temporary casing passing through the soil layer should be provided in the pre-boring process to prevent collapse of the ground and soil from falling into the pre-bored hole; and

(c) The pre-bored hole should be cleaned thoroughly from debris and soil prior to inserting the pile into it.

(d) See Clause 5.4.1 for pile splices and pile head design.

5.4.3 PRECAST REINFORCED CONCRETE PILES

Precast reinforced concrete piles may be used for low or medium rise buildings on ground without a significant amount of boulders or corestones.

In the design of precast reinforced concrete piles, stresses during lifting, transporting and driving of the pile should also be considered. Hard driving of the pile should be avoided. Special consideration should be given to the very high stresses that may occur at the head and toe of the piles.

5.4.4 PRECAST PRESTRESSED SPUN CONCRETE PILES

These piles should be driven to relatively stiff stratum with sufficient embedded length in residual soil or decomposed rock in order to develop the high bearing capacity and minimize the amount of long-term settlement. The effect of soil movement and percussion during driving on the stability of any adjacent building, structure, land, street and services should be carefully assessed.

Stringent requirements on performance test and quality control are usually required for this type of pile.

Where it is necessary to drive the pile into thick layer of stiff soil, steel conical pile shoes with cross stiffener should be used.

5.4.5 DRIVEN CAST-IN-PLACE CONCRETE PILES

Driven cast-in-place concrete pile should have a maximum size of 750 mm in diameter and is installed by driving a capped steel casing to the required level with concrete placed into the casing subsequently. Joint or splice of the steel casing should be properly designed to avoid breakage during driving. The design of the pile should be in accordance with the provision of Clauses 5.1 to 5.3.

In order not to disturb the stability of the ground or the integrity of newly formed cast-in-place piles, driving of the piles, or any operation that may induce vibration or ground movement such as chiseling and excavation, should not be carried out within a distance less than 5-times the pile diameter from an unfilled pile excavation or a cast-in-place concrete pile with concrete placed in less than 24 hours.

5.4.6 SMALL DIAMETER BORED PILES

Small diameter bored piles should have a diameter not exceeding 750 mm. They are formed by boring casing into the ground and subsequently filling the hole with concrete. The pile resistance is usually derived from shaft friction but could also be combined with end bearing in soil or rock with due consideration of strain compatibility. The design of the pile should be in accordance with the provision of Clauses 5.1 to 5.3.

The contribution of friction from any fill or marine deposit layer, if exists, should usually not be included unless it can be proved that there is no possibility of future consolidation. Negative skin friction should be considered in accordance with the provision of Clause 5.2.

5.4.7 LARGE DIAMETER BORED PILES

Large diameter bored piles should be greater than 750 mm in diameter and is usually installed by machine boring to the required level with concrete filling the bored hole subsequently. The boring operation should be carried out under water or in a suitable fluid such as bentonite and no dewatering for the excavation should be permitted. A steel casing should be used to provide temporary support to the ground during boring operation, unless bentonite slurry is used and with sufficient hydraulic head to maintain the stability of the excavation. The design of the pile should be in accordance with the provision of Clauses 5.1 to 5.3.

Bell-out with a gradient not exceeding 30 degree from vertical, and the size not exceeding 1.5 times the shaft diameter may be permitted at the pile base to increase the end bearing capacity.

Where steep bedrock profile is identified, the founding levels of adjacent piles should not differ by more than the clear distance between the pile bases unless the stability of rock under the piles are checked by recognized engineering principles, taking into account the existence of any adverse joints.

5.4.8 MINI-PILES

A mini-pile usually consists of one or more steel bars encased by grout inside a drill hole not exceeding 400 mm in diameter. It is mainly used to resist compression or tension loads on sites with difficult access.

Design Principles

The following design principles should be adopted in general :

(a) The structural capacity of a mini-pile should be derived solely from the steel bars. Contributions from the grout and steel casing should be ignored because of the relatively high stress in the steel bars and strain incompatibility. The allowable stresses of steel bars should be in accordance with the Code of Practice for the Structural Use of Concrete;

(b) Mini-piles are normally designed to be socketed into rock. The allowable capacity should be derived from the bond strength between the grout and rock. The rock socket should be formed in category 1(a), 1(b) or 1(c) rock as defined in Table 2.1. The bond strength should not exceed the allowable value given in Clause 2.2.2;

(c) Where in the special circumstances that mini-piles are designed to rely on soil friction, testing on trial pile should be carried out to verify the design assumptions;

(d) Mini-piles should not be designed to resist lateral load by bending in view of their limited bending capacity. When lateral loads are to be resisted by the pile cap, the lateral displacement should be restricted to a magnitude which will not induce adverse effect to the strength and integrity of the pile;

(e) Where raking mini-piles are used to resist lateral forces, care should be taken to ensure equilibrium of forces and moments are maintained, taking into consideration the lack of bending stiffness of the piles and the effectively hinged conditions at the pile heads and bases.

(f) The allowable buckling capacity of the mini-piles should be checked because of their relative slenderness. Lateral restraints from the grout, permanent steel casing and the surrounding soil may be allowed in assessing the buckling capacity of the pile.

Construction Considerations

The following should be given due consideration when constructing mini-piles :

(a) Steel casing should be provided to support the pre-drilled hole within the soil and/or fractured rock during drilling operation;

(b) A permanent steel casing should be provided to enhance corrosion protection;

(c) A non-shrink cement grout with a minimum characteristic compressive strength of 30 MPa at 28 days should be used for encasing the steel bars;

(d) Verticality, inclination and alignment of the mini-piles should be checked during installation to verify any design assumption on eccentric moments induced in the piles.

5.4.9 BARRETTES

Barrettes, or barrette piles are installed by machine excavation into a bentonite slurry filled trench down to the founding level, inserting the reinforcement cage and concreting the excavated trench by tremie method. They are usually of rectilinear sections and founded by end bearing on rock. Other founding criteria may also be used provided that their suitability is demonstrated. The design of the pile should be in accordance with the provision of Clauses 5.1 to 5.3.

The bentonite slurry in the excavated trench should be of sufficient hydraulic head to maintain the stability of the trench, including any surcharge from adjacent structures and construction loads. Rigid reinforced concrete guide walls are usually provided to maintain alignment and verticality of the excavation.

5.4.10 HAND-DUG CAISSON

Hand-dug caisson works have been banned for general use because of safety and health reasons. However foundation plans including hand-dug caisson works may still be approved when it can be demonstrated that any of the following circumstances exists :

(a) the depth of the hand-dug caisson does not exceed 3 m and the diameter of the inscribed circle of the hand-dug caisson is not less than 1.5 m;
(b) for the site concerned :
    (i) the use of a hand-dug caisson is the only practical construction method; or
    (ii) there is no other safe engineering alternative.

5.4.11 STEEL H-PILES DRIVEN TO BEDROCK

(1) General Considerations

Where the bedrock is relatively shallow and the soil strata do not have adequate strength to allow the founding of piles, steel H-piles driven to refusal with pile bases terminated on or very close to bedrock are sometimes proposed. However, this type of piles often poses the following problems:

(a) the pile bases are susceptible to damage due to hard driving at or near the bedrock;
(b) the pile bases could easily be deflected at the rock surface particularly where the bedrock profile is sloping or undulating;
(c) the piles are prone to have buckling or stability problem, as the relatively shallow and weak soil strata above bedrock may not provide adequate lateral resistance to the piles.

Where the above problems could not be satisfactorily coped with, other foundation options such as rock-socketed piles should be considered. Where the RSE has confidence in the successful installation of this type of piles, he or she may either use any recognized engineering method to cope with the above

problems, or follow the guidance on design principles and construction requirements given in items (2) and (3) below.

(2) Design Principles

(a) The maximum allowable axial working stress should not exceed 30% of the yield stress of the steel H-pile;

(b) The maximum combined stress due to the axial load and bending moment should not exceed 50% of the yield stress of the steel H-pile;

(c) Piles should be founded on or close to rock not inferior to category 1(d) defined in Table 2.1. Piles may be considered as founded on rock when driven to refusal by using sufficient driving energy (see Clause 5.3.2(1)). Driven to refusal means the actual penetration of a pile is not more than 10mm per 10 blows. To avoid overdriving of piles, monitoring of the peak driving stress by using Pile Driving Analyser or other suitable method should be carried out as appropriate;

(d) The pile bases should be designed for the hard driving on or close to rock; where appropriate, the pile bases should be strengthened by suitable means such as welded-on shoes.

(e) The pile bases should be designed to avoid deflection of piles when the rock surface is encountered; where penetration or key into rock is required, the pile bases should be provided with a rock point or other suitable means;

(f) The stability of the pile foundations should be carefully assessed, particularly where the piles are short and the embedded soils are weak; in this connection, it is recommended that the piles should be designed and properly detailed as fixed head;

(g) The buckling behaviour of the piles should be checked, taking into account the lengths, any lateral load, the embedded conditions and end connections of the piles; and

(h) In the special circumstances that this type of piles are used on site with sloping or undulating rock surface, the stability of individual pile as well as the whole pile group, and that of the rock under the foundations should be assessed, taking into account the joint orientation of the rock.

(3) Construction Requirements

(a) As an accurate estimation of the anticipated founding levels is important to the successful installation of this type of piles, the ground investigation should provide a reasonably accurate estimation of the bedrock profile;

(b) Verticality of the steel H-piles should be checked during installation; and

(c) The RSE must provide adequate supervision of the piling operation so that where any set reading or other sign has indicated a damage or deflection of the pile base, the pile should be abandoned and replaced.

(d) See Clause 5.4.1 for pile splices and pile head design.

(4) Trial Piles

Trial piles may be required to demonstrate that this type of piles is suitable in situations such as the following:

(a) the bedrock profile is sloping or undulating;
(b) the foundations include piles shorter than 10 m (measured from cut-off level); or
(c) the foundations include piles driven through weak strata.

Note: In the absence of better criteria, strata that do not have a 5 m thick soil layer that has an average SPT N-value not less than 10 and no individual SPT N-value less than 5 may be considered as weak strata.

The integrity and capacity of the trial piles should be ascertained by means of:

(a) Pile Driving Analysis (PDA) tests together with modelling by pile wave analysis program; and
(b) static load tests.

(5) Additional Testing Requirements

In addition to the normal testing requirements for piling works, the testing requirements specified below are required for this type of piles.

Prior to the commencement of the piling operation, the RSE should confirm the design assumptions and that this type of piles could be successfully installed by test driving a sufficient number of piles, which should in no case be less than 2.

More stringent proof test requirement than normal piling works may be imposed; PDA tests to verify the integrity and capacity of at least 10% of the working piles are usually required.

6. OTHER FOUNDATION TYPES/ELEMENTS

6.1 BASEMENTS AND HOLLOW BOXES

A basement or hollow box may be used as a foundation to support the superstructures. In the design of basements or hollow boxes, the general design requirements as specified in Chapter 2 should be complied with.

The vertical resistance of ground supporting a basement or hollow box may consist of one or more of the following components :

(a) end bearing at the bases of the side-walls;
(b) end bearing at the bottom slabs; and
(c) bond or frictional resistance on the external surfaces of the side walls.

The horizontal resistance of ground provided for basements or hollow boxes may consist of one or more of the following components :

(a) passive resistance of soil;
(b) bond or frictional resistance on the external surfaces of the side walls; and
(c) bond or frictional resistance at the bases of the side walls and/or the bottom slab.

Where the vertical or horizontal resistance is derived from more than one component, it should be demonstrated that all components of the resistance can be mobilized simultaneously without causing unacceptable disturbance to the ground or adjacent structures and services.

Concrete for the side walls and base slabs of a basement or hollow box should be sufficiently watertight and not inferior to grade 35 designed mix.

Stability against buoyancy should be checked in accordance with the provision of Clause 2.5.4. The stability against buoyancy before the completion of the superstructure should also be checked. The worst groundwater condition and the effect of possible fluctuations in the groundwater table and of possible flooding during and after construction should be considered.

The design of the permanent structures should take into account the stresses that may have developed during the various stages of the construction sequence.

6.2 DIAPHRAGM WALLS

A diaphragm wall may be used as a temporary lateral support wall for deep excavation or the permanent wall of a basement, or it may be designed for both temporary and permanent uses. It may also be used to support vertical loads.

It is usually cast in-situ with tremie concrete inside a bentonite slurry trench formed by excavation with a grab or other machine. The thickness of a diaphragm wall depends on the strength requirement, but it should not be less than 600 mm thick and the length of a diaphragm wall panel may vary from 3 m to 7 m, depending on the excavation tools and subsoil conditions.

The analyses of diaphragm walls should include the following :

(a) seepage analysis for water cut-off;
(b) lateral stability analysis including toe stability;
(c) bending moments, shear forces and deflection due to lateral loads for the proposed construction sequence;
(d) bearing capacity for vertical loads;
(e) slurry trench stability during excavation; and
(f) assessment of settlement induced on adjacent structures, services and ground during construction.

Temporary strutting may be used to reduce the bending moment, shear force or deflection of the diaphragm wall due to lateral loads. The diaphragm wall should have a sufficient toe length to provide stability or cut-off of ground water. Shear pin drilled into bedrock may be used to enhance the toe stability. Grouting may be carried out under the toe level of the wall to enhance the cut-off of ground water.

6.3 RETAINING WALLS

The design and construction of retaining walls shall comply with the requirements of the Building (Construction) Regulations.

Retaining walls may be designed in accordance with “Guide to Retaining Wall Design” (GEOGUIDE 1) (second edition) published by GEO. Reinforced fill retaining walls may be designed in accordance with “Guide to Reinforced Fill Structure and Slope Design” (GEOGUIDE 6) published by GEO.

The design of retaining walls should also comply with the requirements specified in Chapter 2 where :

(a) the retaining walls are used as foundation to support a building or other structure; or
(b) the retaining walls are used to resist external surcharge loads from foundations or other structures.

6.4 GROUND ANCHORS

Ground anchors may be used to resist uplift forces in foundation structures. They may be prestressed or unstressed.

The use of permanent prestressed ground anchors in a project imposes a long-term monitoring commitment on the maintenance parties, which usually involves appreciable recurrent cost and, should deficiencies be revealed, remedial works may be difficult and expensive. The past records show that compliance with this criterion by the owners is not practically viable. For these reasons, permanent ground anchors requiring long-term monitoring are considered as short-lived temporary building works and should not be incorporated into a permanent building.

In exceptional circumstances where permanent ground anchors are to be used, their provision and installation should be in accordance with the requirements and procedures given in "Model Specification for Prestressed Ground Anchors" (GEOSPEC 1) published by GEO.

Monitoring of prestressed ground anchors is essential throughout their service life to ensure their continued satisfactory performance. The parties responsible for subsequent maintenance should be consulted and their agreement should be obtained before prestressed ground anchors are adopted and that they should be provided with a complete set of 'as built' details.

6.5 RE-USE OF EXISTING FOUNDATIONS

Existing foundations that satisfy the current design and construction requirements may be re-used subject to the following conditions:

(a) reliable information of the existing foundations as stipulated in Clause 2.7 is available;
(b) reliable "as-constructed" foundation records and reports as stipulated in Clause 7.3 are available, and if necessary, coring or other integrity tests should be carried out to confirm the as-constructed records; and
(c) the integrity, durability, strength and suitability of the existing foundations for the intended future use are satisfactorily verified, which should include:
    (i) adequate site investigation on environmental factors that could affect the integrity and durability of the foundations, such as tidal or ground water fluctuation, sulphate and chloride content or other aggressive biochemical ingredients in the soil or ground water;
    (ii) adequate ground investigation to verify the conditions of the bearing strata;
    (iii) inspection of the integrity of the existing foundations;
    (iv) any test reports on the load-carrying capacity, integrity or material properties of the existing foundations; and
    (v) any proposed load test, core-drilling test, or other tests on the load-carrying capacity, integrity or material properties of the existing foundations, if considered necessary.

When checking the integrity of existing piles, the piles should be exposed for a depth not less than 2 times the least lateral dimension of the piles or 1500 mm from the cut-off level.

Where foundations are to be reused, care needs to be taken during demolition, particularly when slabs or pile caps are removed above the piles.

Where old and new foundations are to be used together, consideration should be given to possible differential settlements between the old and new foundations and the pre-loaded effect of the old foundations.

There are numerous factors affecting the serviceability of existing foundations. Each case should therefore be considered on its own merits.

7. CONSTRUCTION PRACTICE AND SITE SAFETY FOR FOUNDATION WORKS

7.1 GENERAL

Foundation works should be carried out such that :

(a) the foundation is constructed in accordance with the plans approved by the Building Authority;
(b) suitable methods and sequence are adopted so as not to render inadequate the margin of safety of, or impair the stability of, or cause damage to any building, structure, land, street or services; and
(c) adequate precautionary and protective measures are provided to assure the safety of the workers on site, all persons near the site, and adjacent buildings, structures, lands, streets and services.

7.1.1 GENERAL REQUIREMENTS

All foundation works, except where the penetration depth of the foundation element does not exceed 3 m, are specialized work in the foundation category and shall be carried out by a Registered Specialist Contractor (Foundation Works).

Foundation works should comply with :

(a) the requirements of site safety supervision as stipulated in the Code of Practice for Site Safety Supervision and Technical Memorandum for Supervision Plans; and
(b) the requirements of quality supervision (see Clause 7.1.2).

7.1.2 QUALITY SUPERVISION FOR FOUNDATION WORKS

The foundations are important structural elements of a building or structure. Adequate supervision should be provided to ensure that the quality of the foundation works in respect of materials, general arrangement, installation procedures, workmanship and testing are up to the required standards and conform with all relevant provisions of this Code of Practice. Reference should be made to the relevant practice notes for requirements of quality supervision for foundation works.

7.1.3 CONSTRUCTION MATERIALS

(1) Concrete and grout

Sampling of concrete and compression testing of concrete test cubes should be carried out in accordance with the methods specified in CS1:1990. Testing should be carried out by a HOKLAS accredited laboratory. The test results should be reported on a HOKLAS Endorsed Test Certificate. Concreting for cast-in-situ piles should be in one continuous operation.

(2) Reinforcement

Sampling and testing of steel reinforcement should be carried out in accordance with the methods specified in CS2:1990. Testing should be carried out by a HOKLAS accredited laboratory. The test results should be reported on a HOKLAS Endorsed Test Certificate.

(3) Steel piles

A copy of the mill certificate of the structural steel used should be submitted to Buildings Department together with a statement signed by the RSE to confirm that the requirement of chemical composition and mechanical properties appropriate to the type of steel have been complied with.

7.1.4 EXCAVATION

Foundation works often require excavation, which, even if shallow, can be dangerous if not properly designed. Adequate precautionary and protective measures should be provided to ensure that the excavation works are properly protected and the foundation works are carried out safely.

7.2 EFFECT OF FOUNDATION WORKS ON ADJACENT STRUCTURES AND LAND

7.2.1 ASSESSMENT OF THE EFFECT OF FOUNDATION WORKS

The effect of the foundation works on surrounding land, structures and services should be assessed. The assessment should include :

(a) a detailed report on the structural conditions of all surrounding buildings, structures and existing services and the geotechnical condition of the surrounding land likely to be affected by the foundation works;
(b) estimation of the effect on and the response of these buildings, structures, services and land as a result of foundation works; and
(c) proposals of the preventive measures, monitoring scheme and contingency plans to be undertaken before and during construction.

7.2.2 SHORING AND UNDERPINNING

Details of any shoring or underpinning to any building, structure, land or services required as precautionary measures should be included in the foundation proposal.

7.2.3 MONITORING PLAN

Where the construction of a foundation may affect any building, structure, land, street or services, a monitoring plan should be provided. It should contain :

(a) sufficient monitoring stations for the detailed monitoring of movement and vibration in any building, structure, land, street or services;
(b) sufficient piezometers for the detailed monitoring of the ground water conditions;
(c) the frequency at which the readings will be recorded or taken;
(d) the action levels and the contingency measures to be undertaken.

The construction activities on site should also be properly recorded so that they may be correlated with the monitoring readings if necessary. Such correlation can often explain why some apparently abnormal readings are recorded.

The action level for a particular type of services should be agreed with the relevant government department or utility company and should be based on the amount of movement that services could tolerate.

7.2.4 DEWATERING

Dewatering may cause undue settlement of buildings, structures, streets, land and underground services if not properly designed and carried out. Where dewatering is to be undertaken, an assessment of the effects of dewatering on the adjoining buildings, structures, streets, land and underground services should be made. Recharging should be considered where appropriate. A monitoring scheme in accordance with the provisions of Clause 7.2.3 should be provided.

The groundwater table during construction should be properly controlled such that it would be maintained within the limits permitted in the dewatering design. In case the groundwater table is lowered below the permitted limits, the designer should be informed and appropriate action such as suspension of the dewatering should be taken immediately.

7.2.5 VIBRATION

Vibration caused by foundation works should not induce cracks or other damage to any building, structure, land, street or services.

Where protection of historic buildings or structures is required, stringent requirements on vibration control are usually imposed by the appropriate authorities. Where protection of railway is required, reference should be made to the requirements stipulated in the relevant PNAP.

For protection of buildings in general, the vibration should not cause a peak particle velocity of ground movement exceeding 15 mm/sec.

Where necessary, e.g. the presence of any hospital with sensitive equipment, old masonry building, dilapidated structure or delicate utility sensitive to vibration, a test pile should be carried out to establish the tolerable vibration induced by the proposed method of driving. Measures to reduce vibration such as pre-boring for pile installation, controlling the number of piles being driven at any one time, etc. should be implemented where necessary.

7.2.6 BLASTING

If blasting is to be carried out as part of the foundation works, it must be properly designed and controlled such that it will not adversely affect the stability of any adjacent slope, retaining wall, building, structures and services through ground vibrations or other effects. Adequate measures must also be provided to protect the safety of the workers and the public against possible flyrock.

When blasting is to be adopted for foundation works, reference should be made to the following :

(a) Building (Construction) Regulation 23;
(b) Dangerous Goods (General) Regulation 46;
(c) PNAP 77 - Mass Transit Railway Protection
(d) PNAP 279 – Kowloon-Canton Railway Protection
(e) PNAP 178 - Control of Blasting; and
(f) General Specification for Civil Engineering Works.

7.3 FOUNDATION RECORDS AND REPORTS

The Building Authority requires the submission of record plans, report and a specified form certifying the completion of the foundation works by the relevant personnel upon completion of the works.

Foundation Record Plan

Record plans should include details of the characteristic features of the site and the identification, location, size, depth and level of each foundation unit as constructed.

Foundation Report

The report should include :

(a) the date of construction/installation, the quality and quantity of materials used and any necessary test on the bearing strata for each foundation unit;
(b) the driving performance for driven piles; and
(c) the concrete test cube results, excavation records, predrilling and post-installation proof drilling records for bored piles.

7.4 PILE CONSTRUCTION

7.4.1 DRIVING TEST AND TRIAL PILE

For driven piles, test driving should be conducted to verify the design assumptions made for the piles and founding strata prior to driving any other piles.

Where special ground conditions exist or when a new type of piles is to be used, trial piles may be required to be carried out before the installation of other working piles in order to verify the design assumptions and the performance of the pile. The trial piles should also be tested by the imposition of test loads in accordance with the procedures and criteria specified in Clause 8.4 upon installation.

7.4.2 PRE-DRILLING

For piles founded on rock or rock socket, sufficient pre-drilling should be carried out to identify the depth and quality of the founding rock. Such pre-drilling should be sunk at least 5 m into the category of rock specified for founding or forming of rock socket, or the designed rock socket length of the pile, whichever is the deeper.

For large diameter bored piles, barrettes and the like, pre-drilling should be carried out for each pile.

For mini-piles, socketed steel H-piles, steel H-piles driven to bedrock and similar small diameter bored piles founding on rock or rock socket, pre-drilling should be carried out such that the tip of every pile should be within 5 m distance from a pre-drilling hole. The pre-drilling should be sunk at least 5 m into the category of rock specified for founding or forming of rock socket, or the designed rock socket length of the nearest pile, whichever is the deeper.

7.4.3 POST CONSTRUCTION PROOF DRILLING

Large Diameter Bored Piles, Barrettes and the Like

To ascertain the soundness of the interface, core-drilling should be carried out at the concrete/rock interface for each of the large diameter bored piles, barrettes and the like. The core-drilling should cover at least 1 m above and below the interface.

Socketed H-piles, H-piles driven to bedrock and Mini-piles

To verify the quality of rock for founding or forming of rock socket, additional proof drill holes should be sunk at least 5 m below the as-built top level of the rock socket, or the as-built rock socket length of the nearest pile, whichever is the deeper. The recommended number of such drill holes should be at least 2 for sites with 100 or less piles; or 1% of the number of piles for sites with more than 100 piles (any fraction of a drill hole so calculated should be construed as one additional drill hole). An assessment report with a rockhead contour plan based on the ground investigation, the pre-drilling and the post-installation drilling is required when submitting the piling record plan to the Building Authority.

7.4.4 PROOF TESTS

In accordance with the Building (Construction) Regulation proof tests are required to be conducted on foundation works. The procedures and criteria for proof tests described in Chapter 8 should generally be followed.

Alternative procedures and acceptance criteria other than test loading or core-drilling with sound justification based on recognized foundation engineering principles and relevant to a particular site and building may also be adopted, provided that the following are submitted to the Building Authority to demonstrate the suitability of the proposed method of testing :

(a) relevant recognized engineering principles for the proposed proof test;
(b) detailed procedures of testing and acceptance criteria;
(c) interpretation of the test results; and
(d) any verification tests performed to justify the parameters to be used in the proof test.

7.4.5 FURTHER ON SITE TESTS

Whenever doubt exists as to the design assumption or load carrying capacity of any pile foundation, further on site tests should be carried out.

7.5 GROUND TREATMENT

Where improvement of the load carrying capacity of the ground is to be achieved by ground treatment, adequate proof of the suitability of the method and materials to be used should be given. Where ground treatment has been carried out, adequate tests of the treated ground should be carried out.

Where the ground treatment may affect any building, structure, land, street or services, adequate precautionary measures shall be taken.

7.6 CONTROL OF NUISANCE

Construction site of foundation works may cause environmental nuisance affecting not only the workers on site but also occupants of adjoining buildings and the general public. Appropriate steps should be taken to prevent such nuisance.

(1) Noise

Reference should be made to the Noise Control Ordinance (CAP 400) for detailed requirements in this respect.

In particular, a permit from the Environmental Protection Department is required for carrying out percussive piling operation.

Adequate measures to reduce noise should be taken to meet the requirements of the Environmental Protection Department, if necessary.

(2) Smoke and fume

Reference should be made to the Air Pollution Control Ordinance (CAP 311) for detailed requirements in this respect.

Emission of excessive black smoke or fume from diesel hammer is actionable under the Air Pollution Control Ordinance (CAP 311) which is enforced by Environmental Protection Department.

(3) Waste water and chemical waste

Reference should be made to the Public Health and Municipal Services Ordinance (CAP 132) for detailed requirements in this respect.

In particular, muddy water or chemical waste from a construction site should not be discharged into the drainage system. Any contractor acting against this will be required by the Drainage Services Department to indemnify the Government against costs due to such act.

(4) Vibration

In addition to the provision of Clause 7.2.5, vibration should not create unacceptable discomfort to the occupants of nearby buildings.

7.7 FOUNDATION WORKS IN SCHEDULED AREAS

For foundation works in the Scheduled Areas, there are usually special requirements imposed by the Building Authority. These may include settlement monitoring, control on vibration and performance review of the foundation works. Before commencement of the works, reference should be made to the relevant PNAP issued by the Building Authority.

8. TESTING OF FOUNDATIONS AND GROUND

8.1 GENERAL

Testing of foundations may be required for one or more of the following purposes :

(a) to ascertain the performance of the foundation under load;
(b) to establish or justify the design parameters; and
(c) to verify the structural integrity of the foundation units.

The type of test selected and the number of tests required should be appropriate to the type of foundation and suitable for the purpose required.

8.2 PLATE LOAD TEST

Plate load test may be used to determine the allowable bearing capacity and to estimate the settlement of granular soils. The settlement estimated, however, may not reflect the behaviour of all soil strata affected by the footing. In addition, any extrapolation of the test results to a footing larger than the test plate must be carried out with caution, in particular, for non-homogeneous ground conditions.

Any suitable test procedures and acceptance criteria with full justification may be used. The following may be considered as suitable test procedures and acceptance criteria for plate load tests on cohesionless soils:

(a) The loading plate should not be less than 300 mm, square or circular;

(b) The maximum test load is 3W, where :

W = allowable working pressure % area of loading plate

(c) The test load should be applied in increments of 0.5W up to W, released and reapplied in increments of 0.5W up to 2W, then released and reapplied in increment of 0.5W up to 3W, which should be maintained for at least 72 hours before removal;

(d) The load at each incremental stage should be held for a period of 10 minutes or longer until the rate of settlement is less than 0.05 mm in 10 minutes;

(e) The test is deemed to be unsatisfactory if the maximum settlement of the plate exceeds S_p given by the following equation :

$$S_p = 3 \times S_f \times \left(\frac{B+b}{2B} \right)^2 \times \frac{m+0.5}{1.5m}$$

where S_f = allowable settlement of footing under allowable working load;
B = diameter or least dimension of footing;
b = diameter or least dimension of plate;
m = length to width ratio of footing, where $m \geq 1$.

8.3 STANDARD PENETRATION TEST

Standard Penetration Test is an in situ test to correlate soil strength with compactness or relative density of granular soils. Many empirical formulae make use of SPT to determine stiffness, strength, friction and bearing capacity of granular soils. SPT results for cohesive soils such as marine deposit, however, should be used with extreme caution.

The procedures and equipment needed for standard penetration test are given in GEOGUIDE 2.

8.4 PROOF TESTS BY IMPOSITION OF TEST LOADS

Imposition of test loads to a pile is considered to be most realistic in reflecting the performance of the pile and is widely used for driven piles, small diameter bored piles, socketed piles and mini-piles. When carrying out loading tests :

(a) The pile should be load tested at the cut-off level with no allowance for group effect;

(b) The test load should be applied in 2 equal increments up to the design pile capacity under working load, then released and reapplied in 4 equal increments up to twice the design pile capacity under working load and maintained for at least 72 hours before removal;

(c) The load at each incremental stage should be held for a period of 10 minutes or longer until the rate of settlement is less than 0.05 mm in 10 minutes;

(d) The test load should be measured by a calibrated load measuring device and also by a calibrated pressure gauge in the hydraulic system; and

(e) For piles with a diameter or least lateral dimension not exceeding 750 mm, the test is deemed to be unsatisfactory if any of the following conditions apply :

(i) the maximum settlement at the head of the pile during the test exceeds the value

$$\frac{2WL}{AE} + \frac{D}{120} + 4\text{mm},$$

where W is the design pile capacity under working load in kN;
L is the length of the pile in mm (For piles with rock sockets, L should be measured to the centre of the

rock socket. For piles without rock sockets, L may generally be taken as the entire length of the pile.);

A is the cross-section area of the pile in mm^2;

E is the Young's modulus for the material of the pile in kN/mm^2; and

D is the least lateral dimension of the pile in mm;

(ii) when the rate of recovery after the removal of the maximum test load is less than 0.1 mm/hour observed in a period of not less than 15 minutes, the residual settlement at the head of the pile exceeds the greater of the following :

D/120 + 4 mm; and

25% of the maximum pile head settlement during the test.

(f) In calculating the elastic compression/extension of the test pile, the following should be considered:

(i) for mini-piles, the contribution from steel bars, cement grout and steel casing along the whole length L; and

(ii) for socketed steel H-piles, the contribution from cement grout within the length of the rock socket.

(g) For large diameter bored piles, barrette piles and hand-dug caissons, other suitable acceptance criteria for the loading test with full justification may be used.

8.5 PROOF TESTS BY CORE-DRILLING

Proof core-drilling test is commonly used in large diameter bored piles, barrettes and the like which can reveal the soundness of the founding rock, concrete and the interface between the pile and the rock. When carrying out core-drilling tests :

(a) the core-drilling should be taken through the full depth of the pile and carried down to a distance of at least half a diameter of the pile base, or 600 mm, whichever is larger, into the ground upon which the pile is founded;

(b) the completed core so taken should be properly marked and arranged in proper order for inspection;

(c) the concrete cores should not show evidence of honeycombing or segregation of individual constituent materials;

(d) any rock core obtained should be visually examined to conform with the required rock material specified in the design;

(e) the cores should also be examined to confirm the adequacy of the interface between the concrete and rock; and

(f) where piles are founded on soil, standard penetration tests should be carried out at a maximum interval of 1.5 m from the pile founding level down to a distance of at least 3 times the diameter of the pile base, or 5 m, whichever is larger, to verify the required soil strength.

8.6 SONIC LOGGING

Sonic logging is one of the most commonly used non-destructive pile integrity tests for cast-in-place concrete piles, diaphragm walls and barrettes. It is based on the measurement of the propagation time of a sonic transmission between two piezo-electric probes placed in tubes cast into the pile. The test should be carried out by a HOKLAS accredited laboratory.

It may be used to check the homogeneity of the concrete and to detect defects such as honeycombing, segregation, necking, inclusions and cracks in concrete.

The test should not be used beyond its limitations which include the following :

(a) it cannot identify the nature of the defects;
(b) it may affect concreting or cause defects in the pile due to the cast-in tubes for the test; and
(c) poor bonding between the tubes and concrete may result in anomalous response.

8.7 SONIC ECHO TESTS

This is a rapid test carried out at the pile top. A sonic wave is generated at the pile head, e.g. by striking the pile head with a hand-held hammer. The wave travels down the pile and is reflected back up the pile from the pile toe. Any defect in the length of the pile may cause intermediate echo being recorded. With an assumed propagation velocity for the sonic wave, the length of the pile and the location of the defect can be determined. The test should be carried out by a HOKLAS accredited laboratory.

The test may be used for checking the continuity of concrete and steel bearing piles.

The test should be carried out at least 7 days after casting of concrete so that accurate propagation velocity can be applied

The test should not be used beyond its limitations which include the following :

(a) it is not suitable for piles with joints;
(b) it is sensitive to small bulbs or necks in the pile;
(c) the length to diameter ratio of the pile should not exceed 30 for accurate measurement;

(d) ground vibration due to other construction operation could affect the signals;
(e) response can be highly distorted for anomalies near the pile head; and
(f) diagonal cracks or change in concrete quality may not be easily identified from the test results.

8.8 VIBRATION TEST

This is also a rapid test carried out at the pile top. A force of constant amplitude over a broad frequency band, preferably from 0 to 5000 Hz, is applied, and the velocity in relation to this applied force is measured simultaneously, at the pile head. The applied force may be generated from an electro-dynamic vibrator or a small hand-held hammer fitted with an internal load cell. The velocity at the pile head is measured with a vibration transducer. The mechanical impedance or admittance of the pile, which is obtained by dividing the measured pile head velocity by the applied force, is plotted against the vibration frequency to form the vibration excitation and response curve for determination of the pile characteristics such as pile head stiffness, condition of anchorage at pile toe, resonating length, characteristic mobility and damping factor. Interpretation of the test results and the pile characteristics may identify whether the pile integrity is regular, or defects such as irregularity in pile section, grade of concrete and pile anchorage condition may exist.

The test should be carried out by a HOKLAS accredited laboratory. The test should not be used beyond its limitations which include the following :

(a) the signal is easily damped for pile with a length to diameter ratio of about 20 in stiff and dense soil and 30 in loose soil;
(b) it will not identify small but structurally significant variations in the wave velocity through weak concrete zone;
(c) it is sensitive to abrupt changes, but not gradual changes, in pile cross section;
(d) it is unable to quantify the vertical extent of section changes or the lateral position of defects; and
(e) vertical cracks cannot be detected.

8.9 DYNAMIC LOAD TEST

Dynamic load test, which is more suitable for a driven pile, can be used to detect pile defects or to obtain results for analytical methods which could estimate the pile capacity, the driving stresses and the maximum percussive energy delivered to the pile. The analytical methods should be carried out with accurate test results and correctly assumed values for the parameters required for the analysis. Justification of the assumed values for the parameters may be required. The test should be carried out by a HOKLAS accredited laboratory.

In the test, a hammer with a capacity large enough to cause sufficient pile movement such that the resistance of the pile can be fully mobilized should be used to generate a

stress wave. A minimum pile penetration of about 2 to 3 mm per blow should be achieved where practicable, particularly if it is required to predict the pile capacity.

In view of the limited studies and results on correlation between dynamic load test and static load test in Hong Kong, dynamic load test has not been generally accepted as proof test for foundations under normal circumstances. It may however be used for the following purposes :

(a) to detect pile defects;
(b) to identify defective piles within a large group of piles;
(c) to monitor the driving resistance and driving stresses in the pile; and
(d) to check the consistency of hammer efficiency.

The test should not be used beyond its limitations which include the following:

(a) the accuracy of estimation of pile capacity depends on the proper selection of soil damping factor; and
(b) only major pile defects can be identified as small cracks tend to close up during driving.

8.10 TENSION TEST

If reaction piles are used for tension test, to minimize interaction effects, the reaction piles should be located as far from the test pile as practicable. In any case, the reaction pile should be at least 3 test pile diameters, or 2 m, whichever is larger, from the test pile, measured centre to centre. The test may follow the procedures for static load test given in Clause 8.4. The test should be deemed to be unsatisfactory if any one of the following conditions applies :

(a) the maximum extension at the head of the pile during the test exceeds the elastic extension plus 4 mm;
(b) when the rate of recovery after the removal of the maximum test load is less than 0.1 mm/hour observed in a period of not less than 15 minutes, the residual extension at the head of the pile exceeds the greater of 4 mm and 25% of the maximum pile head extension during the test; or
(c) there is structural failure in the test pile.

The maximum test load should not result in the test pile or anchor being stressed beyond the yield stress. Where the design uplift capacity of the test pile is based on bond and tensile stresses which are taken as 50% of the corresponding values in compression, the test load may be 1.5 times the design uplift capacity of the pile under working load.

8.11 LATERAL LOAD TEST

Lateral load test of piles may be carried out by :

(a) jacking between two piles; or
(b) jacking a test pile against a suitable structure that could provide the reaction for the jacking force.

In the first method, both piles can be considered as test piles. It may be necessary to demonstrate that the spacing of the piles is adequate. In the second method, the suitable structure may be a temporary 'deadman' or weighted platform formed for the test.

Lateral load tests are usually carried out for the purpose of verifying the design parameters and method. In such case, accurate information of the strata along the full depth of the pile is required and analysis should be carried out prior to testing to predict the behaviour of the pile under the test load. The performance of the pile on site should be measured and compared with the predicted behaviour.

The test results should demonstrate that :

(a) the design method can accurately predict the behaviour of the pile under lateral load in the given ground condition; and
(b) the pile in the given ground condition can resist the design lateral load with an adequate factor of safety and without any unacceptable ground deformation or movement.

8.12 ULTRASONIC ECHO SOUNDER TEST

The ultrasonic echo sounder test may be used to measure the profile of the excavation for a pile shaft, such as the dimensions of bell-outs of bored piles. It is also used to check verticality of excavation where accurate construction is required.

The equipment consists of a sensor, a winch and a recorder. In the test, the sensor is lowered into the excavation to measure the distance between the four faces of the sensor and the wall of the excavation. The sensor is prevented from twisting when it is lowered into the excavation, usually by means of guide wires. The recorder will then produce a graphical record of the measured distances against depth.

The sensor measures distances by emitting an ultrasonic pulse, and measuring the time taken for an echo to return to the sensor. The velocity of the pulse is dependent on the density of the fluid used in the excavation. In-situ calibration of the senor under the excavation fluid is normally required. Bubbles and sediment suspended in the excavation fluid can affect the accuracy of the measurement. Therefore, the excavation fluid should be left to stand after excavation to allow bubbles entrained in the fluid to dissipate and any suspended sediment to settle.

The equipment should not be used beyond its limitations, such as the dimensions that it can measure, the range of density of the excavation fluid that it may be used.