

DEP SPECIFICATION

GEOTECHNICAL AND FOUNDATION ENGINEERING - ONSHORE

DEP 34.11.00.12-Gen.

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(DEP Circular 99/11 has been incorporated)

(DEP Circular 68/12 has been incorporated)

DESIGN AND ENGINEERING PRACTICE



PREFACE

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These views are based on the experience acquired during involvement with the design, construction, operation and maintenance of processing units and facilities. Where deemed appropriate DEPs are based on, or reference international, regional, national and industry standards.

The objective is to set the recommended standard for good design and engineering practice to be applied by Shell companies in oil and gas production, oil refining, gas handling, gasification, chemical processing, or any other such facility, and thereby to help achieve maximum technical and economic benefit from standardization.

The information set forth in these publications is provided to Shell companies for their consideration and decision to implement. This is of particular importance where DEPs may not cover every requirement or diversity of condition at each locality. The system of DEPs is expected to be sufficiently flexible to allow individual Operating Units to adapt the information set forth in DEPs to their own environment and requirements.

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1. INTRODUCTION

1.1 SCOPE

This DEP specifies requirements for the geotechnical and foundation engineering of onshore structures and civil engineering works, including jetties and waterfront structures. It does not cover the design of offshore structures.

This DEP specifies design requirements to be used in conjunction with design according to the Eurocodes - EN 1997-1 (Eurocode 7) and EN 1998-5 (Eurocode 8) for geotechnical and foundation engineering. However it is accepted that in some parts of the world or for particular types of design, the use of Eurocodes is not appropriate. The applicability of other codes is detailed in (2).

Specific requirements and recommendations relating to ground investigations, earthworks, geotechnical elements of roads, underground piping and tank pads and bunds are given in the DEPs listed in (11) and thus are not covered by this DEP. Furthermore this DEP does not give recommendations for the design of tailings embankments or fluid retention structures, which is a highly specialised activity requiring expert design, based on local regulatory requirements.

This DEP covers the following main topics:

- a) Design Basis
- b) Spread Foundations
- c) Pile Foundations
- d) Machine Foundations
- e) Retaining Structures and Anchorages
- f) Marine and Waterfront Structures
- g) Embankments and Slopes
- h) Ground Improvement and Reinforcement
- i) Dewatering
- j) Geotechnical Design under Seismic Loading

This DEP shall be used in conjunction with other DEPs covering the specific requirements of various types of civil engineering structures, such as concrete and steel structures, control buildings, stacks, drainage, etc.

This is a revision of the DEP of the same number dated January 2010; see (1.5) regarding the changes.

1.2 DISTRIBUTION, INTENDED USE AND REGULATORY CONSIDERATIONS

Unless otherwise authorised by Shell GSI, the distribution of this DEP is confined to Shell companies and, where necessary, to Contractors and Manufacturers/Suppliers nominated by them. Any authorised access to DEPs does not for that reason constitute an authorization to any documents, data or information to which the DEPs may refer.

This DEP is intended for use in facilities related to oil refineries, chemical plants, gas plants, exploration and production facilities and supply/distribution installations. This DEP may also be applied in other similar facilities.

When DEPs are applied, a Management of Change (MOC) process should be implemented; this is of particular importance when existing facilities are to be modified.

If national and/or local regulations exist in which some of the requirements could be more stringent than in this DEP, the Contractor shall determine by careful scrutiny which of the requirements are the more stringent and which combination of requirements will be acceptable with regards to the safety, environmental, economic and legal aspects. In all cases the Contractor shall inform the Principal of any deviation from the requirements of

this DEP which is considered to be necessary in order to comply with national and/or local regulations. The Principal may then negotiate with the Authorities concerned, the objective being to obtain agreement to follow this DEP as closely as possible.

1.3 DEFINITIONS

1.3.1 General definitions

The **Contractor** is the party that carries out all or part of the design, engineering, procurement, construction, commissioning or management of a project or operation of a facility. The Principal may undertake all or part of the duties of the Contractor.

The **Manufacturer/Supplier** is the party that manufactures or supplies equipment and services to perform the duties specified by the Contractor.

The **Principal** is the party that initiates the project and ultimately pays for it. The Principal may also include an agent or consultant authorised to act for, and on behalf of, the Principal.

The word **shall** indicates a requirement.

The word **should** indicates a recommendation.

1.3.2 Specific definitions

Refer also to Clause 1.5 of EN 1990 and Clause 1.5 of EN 1997.

NON CONFLICTING GUIDANCE is guidance given in codes, standards and other reference documents that does not conflict with EN 1997.

1.4 CROSS-REFERENCES

Where cross-references to other parts of this DEP are made, the referenced section number is shown in brackets. Other documents referenced by this DEP are listed in (11).

1.5 SUMMARY OF MAIN CHANGES

This DEP is a revision of the DEP of the same number dated January 2010. The following are the main, non-editorial changes.

In this revision normative requirements have been summarised.

1.6 COMMENTS ON THIS DEP

Comments on this DEP may be sent to the Administrator at standards@shell.com, using the DEP Feedback Form. The DEP Feedback Form can be found on the main page of "DEPs on the Web", available through the Global Technical Standards web portal <http://swwww.shell.com/standards> and on the main page of the DEPs DVD-ROM.

1.7 DUAL UNITS

In this DEP, the International System of units (SI) shall be understood to prevail over US Customary (USC) units. USC units are provided in brackets following the SI units for information.

2. DESIGN BASIS

2.1 OBJECTIVE

Amended per

Circular 99/11

The objective of the Geotechnical and Foundation Engineering DEP is to provide requirements for all aspects of onshore ground engineering related to oil and gas facilities.

This DEP shall be read in conjunction with and is complementary to the following DEPs:

- a) DEP 34.00.01.10-Gen., Earthquake Design for Onshore Facilities
- b) DEP 34.00.01.30-Gen., Structural design and engineering
- c) DEP 34.11.00.10-Gen., Site Investigations
- d) DEP 34.11.00.11-Gen., Site preparation and earthworks, including tank foundations and tank farms
- e) DEP 34.13.20.31-Gen., Roads, paving, surfacing, slope protection and fencing
- f) DEP 34.19.20.31-Gen., Reinforced concrete foundations and structures
- g) DEP 34.28.00.31-Gen., Onshore Steel Structures
- h) DEP 34.51.01.33-Gen., Above ground vertical storage tanks - Design and construction (based on API 650)
- i) DEP 35.00.10.10-Gen., Design of jetty facilities (amendments/supplements to BS 6349-1/2/4)

2.2 STANDARDS OF PRACTICE

2.2.1 Eurocodes

This DEP specifies design requirements to be used in conjunction with the Eurocodes - EN 1997-1 (Eurocode 7) and EN 1998-5 (Eurocode 8) for geotechnical and foundation engineering.

The partial factors specified in EN 1997-1 are supplemented by National Annexes which set out alternative methods and the Nationally Determined Parameters (NDPs) for individual countries. This Appendix sets out the (UK) NDPs that shall be adopted when design is carried out for projects in countries where EN 1997-1 has not been adopted as the national standard or where there is no National Annex.

Amended per

Circular 68/12

Structures classified as medium or high consequence in accordance with DEP 34.00.01.10-Gen. shall be considered as special structures and shall be designed to meet the specified performance requirements with the additional requirements of this DEP.

Design Approach 1 shall be employed for ULS designs as detailed in EN 1997-1. The design approaches are explained in detail in EN 1997-1 Clause 2.4.7.3.4.

2.2.2 Alternative Codes of Practice

Amended per

Circular 99/11

In some countries, national codes and design standards are mandatory and these may conflict with parts of EN 1997-1. This DEP permits the use of certain codes as an alternative to the Eurocodes. A definitive list of national standards that may be used as an alternative to EN 1997-1 is kept by the Onshore Geotechnical SME. For countries where national codes are not included in this list, the Eurocode shall generally take precedence unless agreed otherwise by the appropriate technical authority.

For projects in the US or countries where American standards and practice are normally followed, the following codes may be employed subject to agreement by the Principal:

- ACI 318 / 318R – Building Code Requirements for Structural Concrete and Commentary.
- ACI 350 / 350R – Code Requirements for Environmental Engineering Concrete Structures and Commentary.
- ACI 543R – Design, Manufacture, and Installation of Concrete Piles.
- AASHTO LRFD Bridge Design Specification.

A consistent set of codes of practice following the same design approach shall be adopted for the design. The codes of practice adopted for each geotechnical and foundation design shall be set out in the Geotechnical Design Report. Geotechnical and foundation design codes shall be compatible with codes used for other elements of Civil Engineering design.

The Contractor shall also determine when national and/or local regulations are more stringent than the applied codes. In all cases, the Contractor shall inform the Principal of any deviation from the requirements of this DEP which is considered to be necessary in order to comply with the national or local regulations.

2.2.3 Hazard consequence levels

Three hazard consequence levels shall be used for Shell projects. The categories depend on the consequence of failure of a structure or facility. For complete definitions refer to DEP 34.00.01.10-Gen.

2.2.4 Geotechnical design categories

Structures shall be classified into Geotechnical Categories, in accordance with EN 1997-1 based on the complexity of the structure, the ground conditions and the loading, and the level of risk that is acceptable to the structure.

2.3 IDENTIFICATION OF DESIGN SITUATIONS

2.3.1 Fundamental Requirements

The detailed specification of design situations shall include consideration of the following as appropriate:

- a) general suitability of the ground on which the structure is located with respect to overall stability and ground movements;
- b) disposition and classification of the various zones of soil, rock and elements of construction which are involved in any calculation model;
- c) the environment within which the design is set;
- d) actions, their combinations and load cases;
- e) in the case of structures resting on or near rock the effect of discontinuities and dipping bedding planes;
- f) potential for deterioration of the ground as a result of the works;
- g) tolerance of the structure and interconnecting pipework to deformations;
- h) effect of the proposed structure on existing structures and services;
- i) behaviour of ground and foundation under earthquake loading
- j) maintenance philosophy for the structure.

Both short term and long term design situations shall be considered.

These considerations are applicable for 'normal' ground conditions where material behaviour is consistent and the risk of unpredictable behaviour is low. Additional design

consideration and expertise may be necessary for 'problem' soils (e.g. calcareous soils), rocks, and geological settings.

2.3.2 Ground Characterisation

A ground model shall be developed by an experienced engineering geologist or geotechnical engineer in accordance with DEP 34.11.00.10-Gen. which describes the disposition and classification of the various zones of soil and rock and elements of construction. This should be based on:

- desk study including maps, photos, satellite imagery, digital elevation models etc.;
- walkover survey;
- results of geophysical surveys;
- results of ground investigations and associated laboratory testing.

Assessment of geohazards shall be performed according to DEP 34.11.00.10-Gen.

2.3.3 Structures on or near rock

Consideration shall, as appropriate, be given to:

- a) irregular rock surface topography;
- b) dipping bedding planes;
- c) interbedded hard and soft strata;
- d) faults, joints and fissures;
- e) possible instability of rock blocks;
- f) solution cavities such as swallow holes or fissures filled with soft material, and continuing solution processes;
- g) durability and soundness.

2.3.4 Environment

Consideration shall, as appropriate, be given to:

- a) variation in ground water levels including the effects of dewatering, perched water tables, regional water abstraction, possible flooding, failure of drainage systems;
- b) effects of freezing and thawing, including permafrost;
- c) effect of seasonal moisture content changes, changes to surface evaporation and the effects of vegetation on drainage and soil strength;
- d) potential for water ingress leading to wetting of soils in arid and semi arid areas;
- e) effects of long duration droughts;
- f) effects of scour and erosion;
- g) effects of biodegradation and gases emerging from the ground;
- h) effects of weathering;
- i) other effects of time and environment, e.g. the effect of holes created by solution and animal activities.

2.3.5 Seismicity

Amended per

Circular 68/12

Seismicity and Seismic Hazard Assessment shall be according to the DEP 34.00.01.10-Gen.

The requirements for seismic design given in this DEP shall be followed unless it has been determined in accordance with DEP 34.00.01.10-Gen. that seismic loading can be neglected.

2.3.6 Actions and Load Cases

The actions to be considered in geotechnical design shall be as given in Clause 2.4.2 of EN 1997-1. Imposed actions from structures shall be determined in accordance with (3.5.6.2) to (3.5.6.6). The design report and calculations shall state clearly whether supplied actions are factored or unfactored and, if factored, which type of safety factors have been applied. Also reference shall be made to the source documents which specify the loads.

2.3.6.1 Compressive and Tensile Actions

The compressive and tensile actions on foundations shall be calculated according to the principles outlined in EN 1997-1 with the load combinations as specified in DEP 34.00.01.30-Gen. Loads during erection, hydrotesting, operation and maintenance shall be considered where appropriate.

2.3.6.2 Lateral actions

Where relevant, lateral loads (e.g. from wind, waves, soil, thermal effects or seismic loading) shall be taken into account in the design of foundations.

Wind and thermal loads shall be calculated in accordance with the principles outlined in DEP 34.00.01.30-Gen.

Amended per

Circular 68/12

Seismic loads shall be calculated in accordance with the principles outlined in DEP 34.00.01.30-Gen. and DEP 34.00.01.10-Gen.

Lateral soil loads shall be calculated in accordance with this DEP (Chapter 8).

Wave and hydrodynamic loading shall be calculated in accordance with DEP 34.00.01.30-Gen.

2.3.6.3 Dynamic and cyclic actions

Where relevant, dynamic actions on foundations (e.g. from machines or earthquakes) shall be taken into account for the design of the foundations. The natural frequency of the foundation-ground system shall be evaluated - see DEP 34.00.01.30-Gen and this DEP (Chapter 5).

Consideration shall be given to possible deterioration of bearing capacity and/or stiffness under cyclic and dynamic loading. Soil parameters shall be determined accordingly.

2.3.6.4 Abnormal actions

Abnormal actions arising from explosions, collisions, etc. shall be considered as outlined in DEP 34.00.01.30-Gen.

2.3.6.5 Load combinations

The load combinations set out in DEP 34.00.01.30-Gen. shall be considered.

2.3.7 Tolerance to deformations

Limiting values of displacement or strain for the serviceability limit state shall be realistically assessed as values representing an unacceptable condition.

Limiting values of displacement shall usually be determined by consideration of the maximum differential settlement and distortion that a structure/process equipment/piping can tolerate.

Requirements for settlement criteria for the design of storage tanks are detailed in (3.5) and DEP 34.51.01.33-Gen. Consideration shall be given to the influence of construction sequence and time dependency and whether total, post construction or post hook-up displacements are critical.

2.3.8 Interaction with adjacent structures

Implementation of a new structure in the vicinity of an existing structure may have adverse effects on the existing structure and/or on the new structure. In the design and during construction possible interactions shall be analysed.

Such checks shall include but not be limited to:

- a) excavations or fills which may cause horizontal and vertical soil deformations that may affect adjacent piles, shallow foundations or surface/buried pipelines and services;
- b) dewatering, which may cause settlements of foundations or surface/buried pipelines and services or cause down drag loads on piled foundations;
- c) loading of a new foundation adjacent to an existing foundation which may cause tilting of either one or both foundations or settlement of surface/buried pipelines and services;
- d) vibrations, heave and horizontal soil displacement due to pile driving that may cause settlements of adjacent structures or damage to sensitive equipment in adjacent structures, or to drainage pipes, services, etc.;
- e) creation or interruption of natural drainage paths;
- f) Integrity of all existing temporary works structures.

2.4 REQUIREMENTS FOR SPECIFIC GEOTECHNICAL DATA

2.4.1 Fundamental Requirements

Geotechnical data acquisition shall be according to DEP 34.11.00.10-Gen.

2.4.2 Evaluation of geotechnical parameters

Properties of soil and rock masses, as quantified for design calculations by geotechnical parameters shall be determined from the test results, either directly or through correlation, theory or empiricism, and from other relevant data in accordance with Clause 2.4.3 of EN 1997-1.

Evaluation of the results of ground investigations shall be in accordance with Clause 3.4.3 of EN 1997-1, and any limitations in the data commented upon in the Geotechnical Design Report.

The characteristic value of a soil or rock parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state. The selection of values shall be in accordance with Clause 2.4.5.2 of EN 1997-1.

2.5 SELECTION OF DESIGN METHOD

2.5.1 Fundamental Requirements

Verification that limit states are not exceeded shall involve one or more of the following approaches:

- use of calculations ;
- adoption of prescriptive measures;
- experimental models and load tests;
- an observational method.

The method(s) to be used for demonstrating that limit states will not be exceeded shall be identified by a Geotechnical Contractor.

2.5.1.1 Geotechnical design by calculation

Geotechnical design by calculation requires consideration of the following elements:

- a) Actions (either as imposed loads or imposed displacements);
- b) Properties of soil or rock materials;
- c) Geometrical data describing the problem under consideration;
- d) Limiting values of deformations, stresses etc.;
- e) A calculation model.

The calculation model should accurately describe the problem under consideration and may consist of:

- an analytical model;
- a semi-empirical model;
- a numerical model.

Design by calculation shall follow the procedures given in Clause 2.4 of EN 1997-1 in which design values of actions, material parameters or resistances are obtained by applying partial factors to characteristic values.

Unless agreed otherwise with the Principal, where National Annex is not available or when working outside Europe, Design Approach 1, defined in EN 1997 shall be adopted together with the UK Nationally Determined Parameters (NDP) given in Appendix 1 of this DEP.

More severe values of partial factors shall be considered in cases of abnormally great risk or unusual or exceptionally difficult ground or loading conditions. Where it can be justified on the basis of possible consequences, less severe values may be used for temporary structures or transient situations. A reduced return period for earthquake and/ or environmental loadings may be applied, subject to the agreement of the Principal. All changes shall be justified in the design report for review and comment by the Principal/Technical Authority.

2.5.1.2 Geotechnical design by prescriptive measures

Design by prescriptive measures shall only be used where comparable experience of similar situations makes design calculations unnecessary. It may also be used to ensure durability against frost action and chemical or biological attack for which calculations are not required.

2.5.1.3 Load tests and tests on experimental models

When the results of load tests or tests on large or small-scale models are used to justify or validate a design, or in order to compliment another design approach, the following factors shall be given consideration:

- Difference in ground conditions between the test location and the construction location;
- Time effects, especially if the test or model duration is significantly different to the duration of loading during construction;
- Scale effects (especially for small models).

2.5.1.4 Observational method

Where prediction of geotechnical behaviour is difficult, the adoption of the observational approach, in which the design is approved within pre-determined constraints during construction, may be applied.

The following requirements shall be met before construction commences:

- a) Acceptable limits of behaviour shall be established;
- b) The range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits;

- c) A system of monitoring shall be established that provides timely and accurate data to ensure that performance is within acceptable limits;
- d) The response time of the monitoring system and the speed with which results are analysed shall be sufficiently rapid to allow rapid evolution of the design if behaviour diverges from that expected;
- e) A plan of contingency actions shall be devised, which will be adopted if the monitoring reveals behaviour outside of acceptable limits.

A maintenance plan shall be established to ensure that any failures of the monitoring system are rectified without detriment to the performance of the system.

The observational method is acceptable for ground improvement and temporary dewatering, but for the construction of permanent works may only be used with the approval of the Principal.

2.6 SELECTION OF GEOTECHNICAL DESIGN CONCEPTS

Foundation types with proven track records in similar ground conditions and/or in the area shall always be considered. Innovative designs that result in substantial cost or schedule benefits may also be considered but shall be referred to the Principal based on the recommendations of an experienced geotechnical engineer for approval.

All designs shall be made with due allowance for safety, constructability, costs, programme, reliability, risks, performance and practicality of the structures in mind.

2.7 LIMIT STATES

The limit state design procedure shall be undertaken in two stages;

- set up design situations; and
- show that limit states will not be exceeded in the design situations.

The selected design situations shall be sufficiently severe and so varied as to encompass all conditions which can reasonably be foreseen to occur during the construction and use of the structure.

Various methods may be used to demonstrate that limit states will not be exceeded in the design situations. Methods that may be used are given in (2.5).

2.8 GEOTECHNICAL DESIGN REPORT

To allow concept and detailed design of a facility, a geotechnical design report shall be produced. The level of detail within the geotechnical design report shall be appropriate to the project phase and the availability of ground investigation data. For some projects, where staged ground investigation is undertaken across several design stages, more than one geotechnical design report may be created, with the level of detail increasing as the project progresses.

The geotechnical design report shall demonstrate safe and efficient design. As a minimum this shall include the following items, where relevant:

- 1) a description of the site and surroundings;
- 2) a description of the ground conditions, based on the ground investigations and other available sources of information, and with reference to the ground investigation report ;
- 3) a description of the proposed construction, including loads and limit states (allowable deflections) and assessed geotechnical category;
- 4) the codes and standards applied for the geotechnical design;
- 5) the design approach adopted and corresponding factors of safety with a justification of the values used;

- 6) characteristic values of soil and rock properties;
- 7) foundation design recommendations together with any risks identified and justification regarding their acceptability;
- 8) the geotechnical design calculation results, including drawings;
- 9) a listing of items to be reviewed at a later stage (e.g. by a specialist contractor), those to be checked during construction and those requiring maintenance or monitoring;
- 10) the seismic hazard assessment and design criteria.

NOTE: The Seismic Hazard Assessment and design criteria shall be developed by a specialist seismic expert according to the DEP 34.11.00.10-Gen. and shall be referred to or included in the geotechnical design report.

- 11) the requirements for supervision of the construction and monitoring (including items to be checked) and the actions to be taken dependant on monitoring results (including all the information listed in Clause 2.8 of EN 1997-1)
- 12) requirements for maintenance.

If significant changes are made to the geotechnical design at a later stage, an addendum to the geotechnical design report shall be issued to reflect the changes.

2.9 REVIEW AND ACCEPTANCE OF GEOTECHNICAL DESIGN

Review of the geotechnical design report shall consider the specified design requirements. If available, other data such as site test results and the performance of similar designs in similar ground conditions shall be taken into account. Items to be considered shall include, but not be limited to:

- a) fitness of the geotechnical design for its purpose ;
- b) weaknesses, sensitivity to variations in site conditions and workmanship, potential savings ;
- c) constructability;
- d) operability and maintenance;
- e) Safety during construction, operation and decommissioning.

2.10 SUPERVISION OF CONSTRUCTION

Supervision of construction of geotechnical structures by qualified staff is of major importance, as these structures are usually buried and are often impossible to inspect and difficult to repair following construction.

A plan of supervision shall be set out in the Geotechnical Design Report and carried out in accordance with Clauses 4.1 to 4.4 of EN 1997-1.

An Inspection and Test Plan shall be developed in parallel with the geotechnical design. This shall include all relevant QA/QC and integrity testing associated with construction and safety issues.

2.11 MONITORING

When required, monitoring shall be carried out in accordance with the plan set out in the Geotechnical Design Report and Clauses 4.1 and 4.5 of EN 1997-1.

Settlements during load tests and hydrotests, and of structures where the performance is sensitive to total or differential settlement shall be monitored.

2.12 MAINTENANCE

Any maintenance required to ensure the safety and serviceability of the structure for the geotechnical design (e.g. painting of sheet pile walls, anchors, drainage system permissible settlement, retaining wall deflection) shall be specified in the Geotechnical Design Report.

2.13 UNITS OF MEASUREMENT

SI units according to DEP 00.00.20.10-Gen. shall be used in the geotechnical and foundation engineering activities.

2.14 MATERIALS

All materials shall be of approved types conforming to the relevant international and national standards.

All materials shall be accompanied by the appropriate testing, calibration and certification documents.

3. SPREAD FOUNDATIONS

3.1 SCOPE

This section specifies requirements for:

- pad and strip foundations;
- raft foundations;
- storage tank foundations.

Design requirements for machine foundations are specified in (6).

3.2 GENERAL REQUIREMENTS

3.2.1 Standards of Practice - Eurocodes

In accordance with Figure 2.1, design shall be according to Eurocodes - EN 1997-1 (Eurocode 7) and EN 1998-5 (Eurocode 8) for shallow foundations. Design Approach 1 shall be employed for ULS designs as detailed in EN 1997-1. For geographical locations where a National Annex does not apply the (UK) Nationally Determined Parameters (NDPs) (Appendix A) shall be adopted.

3.2.2 Standards of Practice - US

See Section 2.2.2.

3.2.3 Foundation depth

The depth of foundations shall be selected in accordance with EN 1997-1 Section 6.4.

3.2.4 Settlement Criteria for Foundations

Settlement limits for total or differential settlement shall be defined for the foundation by the CONTRACTOR responsible for design of the topsides for review and comment by PRINCIPAL.

Calculations of settlement shall follow requirements of EN 1997-1 Section 6.6.2.

The method used to define differential settlement shall be clearly identified.

3.2.5 Durability

In the geotechnical design report, an assessment shall be made on the aggressivity of the environment to allow requirements for concrete mix design in accordance with Appendix 1 of DEP 34.19.20.31-Gen.

3.2.6 Construction issues

Consideration shall be given to the effect of groundwater on the characteristics of the bearing stratum. In cases where the bearing layer may be susceptible to rapid softening the time between excavation and construction shall be as short as possible. Groundwater control methods and schemes shall be selected that do not adversely affect the characteristics of the bearing stratum (for example by causing heave or settlement) or the performance of adjacent foundations.

3.3 PAD AND STRIP FOUNDATIONS

The design of foundations shall be in accordance with Clause 6 of EN 1997-1 using either:

- a direct method; or
- an indirect method; or
- a prescriptive method.

The direct method shall be used for Geotechnical Category 3 structures.

The indirect method may be used for Geotechnical Category 2 structures subject to the following conditions:

- a) well established and documented successful experience shall exist;
- b) there is no explicit settlement limit specified ;
- c) exceptional loading conditions do not prevail, such as for highly inclined or eccentric loads, highly variable or cyclic loads;
- d) the method is not applicable for soft clays and highly organic soils for which settlement calculations are always required;
- e) the method is not applicable for collapsible soils.

Presumed bearing values given in relevant local building codes or derived using other methods shall only be used with the agreement of the Principal.

The effect of groundwater, and changes in groundwater level, on the performance of foundations shall always be carefully considered.

3.3.1 ULS calculations for shallow foundations by the direct method

3.3.1.1 Design Approach

Unless required otherwise by a National Annex or agreed by the Principal, shallow foundations shall be designed in accordance with EN 1997-1 using Design Approach DA-1.

3.3.1.2 Calculation method

Bearing resistance calculations shall generally be in accordance with Annex D of EN 1997-1.

Depth correction factors shall not be applied when calculating the bearing resistance of conventional shallow foundations. However, where spread foundations are constructed at substantial depth and the ground above founding level is at least as competent as the ground below founding level depth correction factors may be applied as follows:

For undrained conditions calculated in accordance with Annex D of EN 1997-1 the bearing resistance formula (equation D.1) becomes,

$$R/A' = (\pi + 2)c_u b_c s_c i_c d_c + q$$

and d_c may be taken as,

$$d_c = 1 + 0.27\sqrt{(D/B)} \text{ for } D/B \leq 3$$

where D is the foundation depth and B the foundation width.

For drained conditions calculated in accordance with Annex D of EN 1997-1 the bearing resistance formula (equation D.2) becomes,

$$R/A' = c' N_c b_c s_c i_c d_c + q' N_q b_q s_q i_q d_q + 0.5 \gamma' B' N_\gamma b_\gamma s_\gamma i_\gamma d_\gamma$$

$$d_\gamma = 1$$

Values of d_c and d_q given in Figure 3.1 may be used but N_γ should then be taken as,

$$N_\gamma = 1.5(N_q - 1)\tan\phi$$

NOTE: These values are from Brinch Hansen (1968) but see also Lyamin et al (2007)

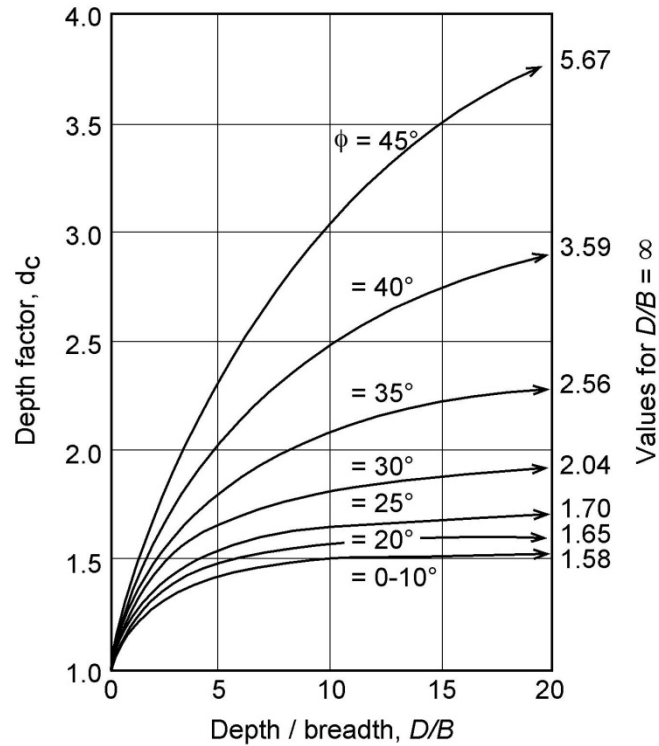


Figure 3.1 Depth correction factor d_c

Where a competent soil layer overlies a weaker layer the following failure mechanisms shall be considered:

- punching failure, see Figure 3.2(a);
- extrusion/squeezing, see Figure 3.2(b);
- bearing failure, see Figure 3.2(c).

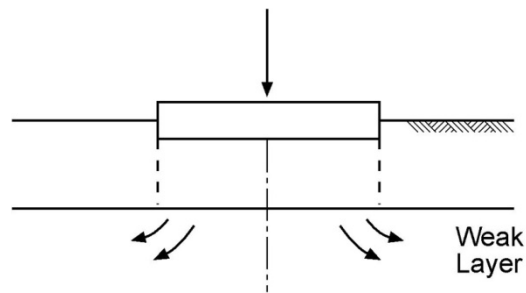


Figure 3.2(a) Punching Failure

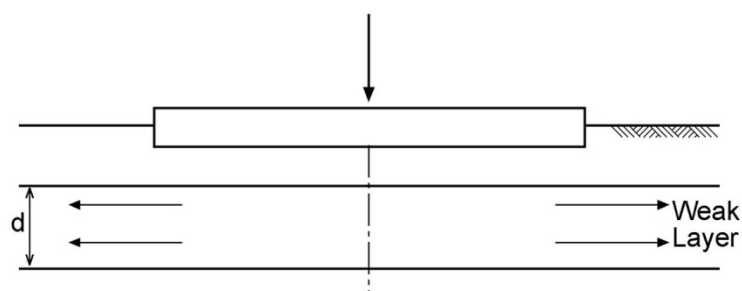


Figure 3.2(b) Extrusion Failure

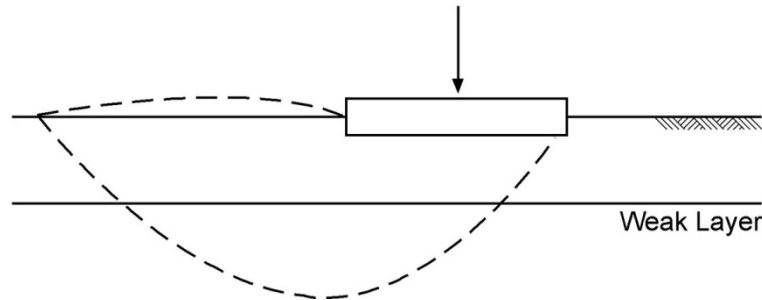


Figure 3.2(c) Bearing Failure

Methods that may be used for calculating the bearing resistance (Figure 3.2(c)) includes:

- Limit equilibrium calculations in which the layered soil profile is modelled and allowance is made for bulging of the competent layer ;
- Non linear finite element analysis using the strength reduction technique.

The net bearing resistance, q_r , for extrusion within a thin weak layer (Figure 3.2(b)) may be taken as,

$$q_r = N_c s_u$$

where s_u is the undrained shear strength of the clay and,

$$N_c = (B/2d + \pi + 1) \text{ for a strip foundation with width } B \text{ and } B/d \geq 2$$

$$N_c = (B/3d + \pi + 1) \text{ for a circular foundation with diameter } B \text{ and } B/d \geq 6$$

3.3.1.3 Overall stability

Where the foundation is on or close to sloping ground, the design shall be checked for the overall stability of the ground mass in accordance with EN 1997-1.

3.3.1.4 Foundations on rock

For design of foundations on highly tectonised or disturbed rock masses, a direct or indirect design method should be used.

The charts given in EN 1997-1 Annex G should not be applied to:

- shallow foundations on weak carbonate rocks (e.g. chalk with porosity > 35%);
- highly tectonised or disturbed rock masses;
- when there are bedding planes or discontinuities that could adversely affect stability;
- on sloping ground.

For design of foundations on chalk, reference shall be made to CIRIA C574.

3.3.1.5 Eccentric loading

The reaction determined from SLS loads should generally be within the middle third. Where large eccentricities could occur, the design value of the actions should be carefully reviewed including construction tolerances.

3.3.2 SLS calculations for shallow foundations by the direct method

Where shallow foundations are designed by the direct method, the total vertical settlement, S , shall be taken as,

$$S = S_0 + S_1 + S_2$$

Where S_0 = immediate settlement

S_1 = consolidation settlement

S_2 = creep settlement

Where site formation works or the construction of structures result in raising or lowering of ground level or changes in groundwater level, consideration should be given to the settlement resulting from these works as well as the settlement due to foundation loading.

Lateral displacement shall be considered where:

- Horizontal or inclined loads are present;
- The foundation is placed on a slope;
- The bearing strata are significantly inclined.

The uncertainties relating to settlement predictions shall be recognised, and where appropriate ranges given to reflect this.

3.4 RAFT FOUNDATIONS

Raft foundations should be used where:

- lightly loaded structures are constructed on soft or variable ground where it is necessary to spread the load or bridge across weaker zones;
- differential settlement needs to be reduced and loads spread more evenly on the ground;
- pad foundations would occupy a large part of the available area;
- compensated foundations are used to reduce net loading.

The performance of a raft foundation depends on the relative flexibility of the raft and ground. An assessment of soil-structure interaction shall generally be required for Geotechnical Category 2 and 3 structures.

3.5 STORAGE TANK FOUNDATIONS

3.5.1 Steel storage tanks

Requirements for tank pads and bunds are given in DEP 34.11.00.11-Gen Section 5.

Design of foundations for conventional storage tanks shall comply with the requirements of DEP 34.51.01.31-Gen. and the additional non-conflicting guidance in EN 14015:2004 Annex I.

Where the tank is underlain by soft compressible ground, particular consideration shall be given to tank edge stability and the tank bottom settlement profile.

3.5.1.1 Tank Stability

Base shear failure

For tanks founded on weak soils both base shear failure and edge shear failure shall be considered.

Edge shear failure

For tanks with a flexible bottom edge failure mechanisms shall be considered.

3.5.1.2 Tank Settlement

Predictions of tank settlement shall be made for all loading cases including:

- Under full hydrotest;
- After long term service.

Tank distortion shall be considered resulting from:

- edge – centre differential settlement;
- perimeter differential settlement;
- overall tilt.

The calculated design settlements shall be less than the maintenance settlement criteria for conventional storage tanks given in EEMUA159 Volume 1. The choice of stiffness parameters (most probable, cautious estimate, worst credible) shall take in account the cost and impact on production of re-levelling the tanks during their construction and lifetime.

If re-levelling of the tanks is not accepted as part of the design, the worst credible combination of soil stiffness parameters shall be used in the calculations (e.g. for tank dishing calculations softest credible stiffness at tank and stiffest credible stiffness at tank edge).

If re-levelling of the tanks is possible during the lifetime of the facility but not desirable, a cautious estimate of soil stiffness parameters shall be used in the calculations.

Where re-levelling of the tanks is possible and favoured, the most probable soil stiffness shall be used, together with a sensitivity analysis. The design report shall assess the probability of the acceptable tank settlements being exceeded. Where it is anticipated that design settlements shall be exceeded, this shall be reflected in the monitoring program.

Differential settlement between tanks and pipe connections shall also be considered.

The tank settlement behaviour shall be monitored during hydrotesting, which in some cases will be the most onerous load case. Requirements for testing and inspection are given in DEP 64.51.01.31-Gen.

3.5.2 Cryogenic storage tanks

Soil-structure interaction analyses shall be carried out for static (short term hydrotest and long term loading) and seismic load cases. The analyses shall incorporate non linear soil properties unless it is demonstrated from the results of the geotechnical investigation that a linear response may reasonably be assumed. Where the ground conditions and imposed loads do not vary significantly in the circumferential direction, an axisymmetric model may be used. Where there is significant variation in the circumferential direction, a 3 dimensional analysis shall be undertaken

Analyses for seismic loading shall consider a lower and upper bound range of soil properties according to the recommendations given in ASCE 4.

3.6 FOUNDATIONS IN ARID REGIONS

In arid regions the following shall be considered:

- Potential influence of water on ground behaviour;
- Potential for collapse settlement;
- Potential for expansive clays;
- Possible reduction in bearing capacity with depth due to presence of a soil crust.

3.7 FOUNDATIONS IN COLD REGIONS

This DEP does not cover the design of foundations in permafrost regions for which specialist advice shall be obtained.

3.8 FOUNDATIONS ON EXPANSIVE SOILS

Where shallow foundations are constructed on expansive soils, possible changes in water content that may occur independently from those occurring as a consequence of foundation loading shall be considered. These include:

- changes to evaporation or infiltration at the ground surface due to covering the surface;
- changes to the evapotranspiration regime due to the planting or removal of vegetation and in particular trees;
- infiltration of water due to irrigation or leakage from service pipes.

3.9 SEISMIC DESIGN

3.9.1 Philosophy of Seismic Design

Amended per

Circular 68/12

Structures shall be classified as high, medium or normal consequence in accordance with DEP 34.00.01.10-Gen. They shall be designed using one or two levels of earthquake using partial factors as defined in Appendix 2 to achieve the appropriate performance requirements determined from Section 4 of that DEP. Either one or two performance criteria shall be satisfied for all structures. The structure and foundation shall remain operational (elastic) under moderate, frequent events with tolerable deformation allowed under larger, less frequent events. These are termed the Ultimate Limit State (ULS) and Accident Limit State (ALS) respectively.

EN 1998-5 gives the requirements for seismic design of foundations, retaining walls and slopes. The corresponding values of partial factors to be used in conjunction with EN 1997-1 Design Approach 1 and serviceability requirements are given in Appendix 2.

3.9.2 Foundation Performance

3.9.2.1 Potential Modes of Deformation

All potential modes of deformations shall be considered.

3.9.2.2 Required Performance

Compliance with the requirements of EN 1998-5 may be particularly onerous for some structures when subject to moderate to high seismic loading (e.g. slopes whose failure is of minor consequence). Subject to the approval of the Principal, permanent displacements under the ULS and ALS may be permitted and a performance based assessment of the structure may be undertaken covering both ULS and ALS load cases in order to achieve a more economic design. The designer shall ensure in this case that ongoing serviceability following the ULS event, and safe shutdown/life safety requirements following the ALS event are not compromised.

The selection of the seismic force for design (E_{Fd}) depends on whether the structural design assumes ductile (i.e. dissipative) or elastic design (see EN 1998-5, clause 5.3.1).

- For structures that are designed to remain elastic, E_{Fd} shall be obtained from an analysis without capacity design considerations;
- For structures that are designed to be ductile, E_{Fd} shall be based on capacity design principles accounting for the development of possible overstrength. As a simplified rule for common foundations, the action effects can be calculated according to the following equation (see EN 1998-1, clause 4.4.2.6):

$$E_{Fd} = E_{F,G} + \gamma_{Rd} E_{F,E}$$

where γ_{Rd} is the overstrength factor taken equal to 1.4

$E_{F,G}$ is the action effect due to non-seismic actions

$E_{F,E}$ is the action effect from the analysis of the seismic design situation.

3.9.2.3 Seismic Displacements

EN 1998-5 requires that foundations of structures remain elastic under both the ULS and ALS design earthquakes. However, for some structures, particularly in high seismicity regions, these design requirements may be particularly onerous and designing for allowable displacements may result in significant economies by comparison to alternative 'elastic' design approaches.

In cases where the transient seismic loadings exceed the available foundation resistance, permanent displacements will occur. The accelerations at which displacement commences are termed threshold accelerations. In many cases the peak earthquake accelerations can exceed the threshold or "critical" values by a substantial margin with minimal foundation displacement occurring. Although EN 1998-5 requires that foundations remain elastic, for foundations above the water table, where the soil properties remain unaltered and the sliding will not affect the performance of any lifelines connected to the structure, a limited amount of sliding may be tolerated.

3.9.2.4 Performance Based Design

When a performance based assessment of the structure is carried out, results of analyses shall be presented in terms of ranges of predicted plastic deformations, and incorporate uncertainty in both load and material properties. The requirement in ASCE 4 shall be adopted to define the upper and lower bounds for soil stiffness. Justification for the adoption of interpreted parameters shall be presented. Analyses should focus on justification of adequate performance rather than satisfying a code-based safety factor.

Amended per

Circular 68/12

To assess serviceability following an earthquake, the soil stiffness, appropriately degraded due to cyclic loading, shall be used to assess permanent foundation displacements, and ensure tolerances of the structure are not exceeded (see 3.5.7). For hazardous or sensitive structures, structures with irregular loading, or subject to moderate to large seismic loading, a dynamic soil structure interaction (DSSI) analysis should be carried out. Reference should be made to the DEP 34.00.01.10-Gen and Appendix 4 of this DEP for guidance on DSSI modelling. DSSI analyses shall conform to ASCE 4.

3.9.3 Seismic Design Approach

3.9.3.1 General Requirements

The requirements in EN 1998-5 for the seismic design of foundations shall be followed for normal structures using the partial factors given in Appendix 2.

Where, with the agreement of the Principal, AASHTO (2007) has been adopted for design, the load and resistance factors given in Appendix 3 may be used.

The same methodology as used in static design for assessment of particular modes of failure (limit states) may be adopted, but with additional pseudo-static inertial loads from the structure applied to the foundation. In addition, reduced ground resistance due to seismic load shall also be considered, both in terms of inertia loading, and cyclic degradation of strength and stiffness.

3.9.3.2 Sliding Capacity

Load combinations shall be selected such that the maximum horizontal load coincides with minimum vertical loading. Conventional Mohr Coulomb soil strengths (ϕ') for dry or free-draining non-liquefiable soil materials, or cyclically degraded undrained shear strength ($S_{u(cyc)}$) for cohesive materials should be used for the soil-foundation interface. Cyclic undrained strength ($\tau_{cy,u}$) shall be considered for loose and saturated cohesionless deposits.

The design horizontal shear force on the foundation (V_{Ed}) shall be resisted by the sum of the contributions from the following mechanisms (see EN 1998-5, clause 5.4.1), namely:

The shear resistance between the base of the foundation and the ground (F_{Rd}) for footings above the water table F_{Rd} can be calculated as,

$$F_{Rd} = N_{Ed} \frac{\tan \delta}{\gamma_M}$$

where N_{Ed} is the design normal force on the foundation base

δ is the interface friction angle on the base of the footing
(EN 1997-1, clause 6.5.3)

γ_M is the material partial factor, with the same value as for $\tan \phi'$

For foundations under the water table, the design shearing resistance shall be evaluated on the basis of the interface undrained shear strength.

- The shear resistance between the vertical sides of the foundation and the ground parallel to the direction of seismic load (E_{fd}). Where this mechanism is to be relied upon, appropriate measures shall be taken on site to ensure proper compaction of the backfill against the sides of the footing or pouring of concrete directly against a vertical soil face.
- The passive resistance against the foundation faces perpendicular to the direction of the seismic load (E_{pd}). This value shall not be greater than 30%, as mobilising the entire passive resistance requires significant displacements, which may exceed the maximum allowable values for the structure.

The design horizontal shear force on the foundation should then satisfy the inequality:

$$V_{Ed} \leq F_{Rd} + E_{fd} + 0.3 E_{pd}$$

For foundations above the water table, provided the soil properties remain unaltered during the earthquake and sliding does not affect the performance of lifelines connected to the structure, a limited amount of sliding may be tolerated because it is an efficient way of dissipating energy.

3.9.3.3 Bearing Capacity

Amended per

Circular 68/12

A normalised bearing capacity function for seismic loading is given in EN 1998-5. Alternatively the static methods given in (3.3.1) with appropriate inclination factors to account for the inertia loading on the structure may be used together with published bearing capacity coefficients for seismic loading of strip footings (e.g. Kumar and Mohan Rao 2002). Rigorous numerical analytical methods adopted for the analysis of slope movements (e.g. limit equilibrium method of slices) as well as generalised finite element analysis can also be applied to pad footing design. If this latter approach is adopted, validation against published closed form solutions shall be completed for a sample problem to demonstrate applicability. Further requirements on the use of numerical analysis are given in Appendix 4 and DEP 34.00.01.10-Gen on Earthquake Design for Onshore Facilities.

EN 1998-5 requires the bearing capacity of the foundation shall be verified under a combination of applied seismic loads, normal (N_{Ed}), horizontal (V_{Ed}) and moment (M_{Ed}), and possibly the effect of the soil inertia forces in the supporting soil.

In assessing bearing capacity the following issues shall also be considered:

- Strength and stiffness degradation mechanisms which can start even at relatively low strain levels;
- Sensitive clays might suffer a shear strength degradation;
- Cohesionless materials are susceptible to dynamic pore pressure build-up under cyclic loading as well as to the upwards dissipation of the pore pressure from underlying layers after an earthquake.

If these phenomena are taken into account, EN 1998-5 permits the reduction of partial factors for material properties.

3.9.3.4 Overturning

Overturning shall be checked if the bearing resistance is sufficiently high that a bearing failure does not initiate prior to overturning.

3.9.4 Foundation connections

In accordance with EN 1998-5 clause 5.4.2.1, tie beams shall be provided between all foundations, except

- For ground type A (rock) in areas of high seismicity;
- For ground type A and B (very dense or very stiff soil) in areas of low seismicity.

EN 1998-1 clause 3.2.1 considers an area of low seismicity to have the surface design ground acceleration ($a_g S$) not greater than 0.1g

The tie beams shall be designed to withstand an axial force, considered in both tension and compression, not less than:

- $\pm 0.3(a_g/9.81)SN_{Ed}$ for ground type B (very dense or very stiff soil);
- $\pm 0.4(a_g/9.81)SN_{Ed}$ for ground type C (dense or stiff soil);
- $\pm 0.6(a_g/9.81)SN_{Ed}$ for ground type D (loose to medium dense or soft to firm soil).

where, N_{Ed} is the mean value of the design axial forces of the connected vertical elements

S is the soil factor (as per EN 1998-1 Table 3.2 or 3.3)

4. PILE FOUNDATIONS

4.1 SCOPE

This section details additional requirements, which applies to vertically and laterally loaded pile foundations. The principles set out shall normally also be applied to other deep foundations not specifically covered in this DEP, such as barrettes or load bearing diaphragm walls or sheet pile walls.

The design of piles forming retaining walls is covered in (6). The design of piles for jetties is covered in DEP 35.00.10.10-Gen.

4.2 GENERAL REQUIREMENTS

4.2.1 Standards of Practice - Eurocodes

In accordance with Figure 2.1, design shall be according to Eurocodes - EN 1997-1 (Eurocode 7) and EN 1998-5 (Eurocode 8) for shallow foundations. Unless agreed in advance with the Principal, Design Approach 1 shall be employed for ULS designs as detailed in EN 1997-1. For geographical locations where a National Annex does not apply the (UK) Nationally Determined Parameters (NDPs) (Appendix 1) shall be adopted.

4.2.2 Standards of Practice - Other

See Section 2.3.

4.2.3 Standards of Practice – Global Factors

Where agreed by the Principal, and if the structural design of piles and supported structures is not being carried out to Eurocodes or the AASHTO LRFD specification, the geotechnical design of piles may be based on overall (lumped) factors of safety. The factors of safety to be adopted shall be selected by the Geotechnical Contractor and are subject to the agreement of the Principal. They should depend on previous local experience and the verification and testing regime to be adopted.

4.3 SELECTION OF PILE TYPE

The geotechnical design report shall state how the selection of piling system for a particular structure or project has taken account of:

- a) Cost;
- b) Schedule;
- c) the anticipated ground and groundwater conditions;
- d) local experience and expertise;
- e) working space available;
- f) availability of suitable construction plant;
- g) local availability of materials;
- h) possible effect of pile construction on adjacent structures/equipment;
- i) environmental issues such as the potential for vertical contamination migration during and after pile installation or removal of polluted soil;
- j) any other factors that have influenced pile type selection.

4.4 DESIGN OF SINGLE PILES

4.4.1 Static design for vertical loading

The characteristic ultimate resistance shall be determined by calculation from derived soil parameters or directly from field test results (such as CPT or pressure meter tests) and verified by full-scale testing. In special circumstances, and subject to agreement of the Principal, the ultimate resistance may be obtained directly from pile tests.

The characteristic ultimate resistance may be evaluated by calculation using either of the following methods:

1. Using the procedure set out in clauses 7.6.2.3(5) and 7.6.3.3(4) of EN 1997-1 in which the resistance is calculated for the profile at each ground investigation point and then the calibration factors given in EN 1997-1 (which depend on the number of profiles considered) are applied to determine a characteristic resistance.
2. Using the procedure set out in clauses 7.6.2.3(8) and 7.6.3.3(6) of EN 1997-1 in which a design profile is first established from the results of the ground investigation and the resistance then calculated for this profile. This follows conventional practice, but a model factor has to be applied to the values of resistance given by conventional calculation methods. Model factors are not given in EN 1997-1 but may be given in National Annexes. Where values are not given in the relevant National Annex, the values given in Appendix 1 of this DEP shall be used.
3. Unless required otherwise by a National Annex, the model factor shall be as given in Appendix 2, and shall depend on whether or not a maintained load test is carried out to a maximum load of at least R_k on a preliminary trial pile.

For each project involving the construction of piled foundations, the Geotechnical Contractor shall determine the pile testing regime to be adopted, which shall be subject to the approval of the Principal.

The design of driven piles shall take account of drivability as discussed in (4.8.1) below.

Where piles are bearing in interbedded strata, where hard or dense layers overlie softer strata, checks shall be made that 'punching' will not occur.

The design of bored cast in situ piles shall take into account the methods of construction and inspection that will be adopted. This shall include consideration of:

- a) The effect of construction duration and possible softening on shaft friction;
- b) The stability of the pile bore during construction;
- c) The possible effect of drilling fluid on shaft friction;
- d) The techniques to be used for cleaning the pile base and feasibility of verifying base cleanliness prior to concreting;
- e) The possibility of remolded material being smeared on the sides of the shaft (particularly when drilling in hard clays or weak rocks in dry conditions).

The ultimate resistance and settlement characteristics of bored piles may be improved by shaft or base grouting. These techniques require a high level of supervision and control and shall therefore only be used with the approval of the Principal. When these techniques are used the design assumptions shall be verified by a trial pile test(s).

The design of piles for end bearing on rock shall consider the likelihood of cavities or mine workings within the rock; and for bored piles the base cleanliness that can be achieved. Where base cleaning cannot be verified during construction the design shall be based on shaft friction only. For piles in rock which are designed using both shaft friction and end bearing, the design considers the displacements required to mobilise both components.

The design and testing of piles shall take account of potential downdrag (negative skin friction) and heave and the design considerations set out in Clause 7.4.2 of EN 1997-1.

Reference shall be made to DEP 35.00.10.10-Gen. Section 3 for specific requirements for marine structures.

4.4.2 Static design for lateral loading

Design for lateral loading shall be in accordance with Clause 7.7 of EN 1997-1. Specific requirements for marine structures are given in DEP 35.00.10.10-Gen Section 3.

4.4.3 Structural design

The structural design of piles shall comply with DEP 34.00.01.30-Gen and Clause 7.8 of EN 1997-1, the relevant structural Eurocodes, EN 1536 Clause 7 and EN 12699 Clause 7.8.

The structural design of precast concrete piles shall take account of handling stresses.

4.5 DESIGN OF PILE GROUPS

4.5.1 Static design for vertical loading

The static design of pile groups shall be in accordance with Clauses 7.6.2.1 and 7.6.3.1 of EN 1997-1 and shall consider both the serviceability and ultimate limit states. For the ultimate limit state failure of individual piles and block failure of the group shall be considered.

The calculation of pile group settlement requires interaction between piles to be considered.

In addition, the design of the pile cap shall take into account the possibility of individual piles within a group forming hard or soft areas due to varying ground conditions.

4.5.1.1 Ultimate resistance

The ultimate resistance of a pile group subject to compressive loading can be taken as the lesser of:

- the combined resistance of the individual piles;
- the resistance of the group of piles and enclosed soil acting as a block.

The ultimate resistance of a pile group subject to tension loading shall be determined for both the UPL and GEO limit states and can be taken as the lesser of:

- the combined resistance of the individual piles;
- the resistance of the group of piles and enclosed soil acting as a block.

The factor of 1.1 for redistribution given in EN 1997-1 shall not be applied to the correlation factors ξ when assessing uplift resistance of pile groups or when the non elastic load distribution is explicitly taken into account (see below).

Where the pile group is subject to eccentric or inclined loads, the loads in each pile will vary with the edge piles being more heavily loaded than the other piles. The edge piles will also be stiffer than the central piles (due to less interaction with adjacent piles) and tend to attract more loads for this reason. This effect may particularly be evident when an elastic pile group analysis with pile interaction is carried out (see 4.5.2).

When assessing pile group behaviour, it is necessary to ensure compatibility between geotechnical and structural behaviour.

A ULS is not necessarily reached when the calculated load in one or a small number of piles in the group exceeds the design resistance because some redistribution may be possible. It is then necessary to check that:

- there are sufficient piles to support the structure with redistribution and that the structure is strong enough to redistribute the loads;
- the structure is also strong enough to resist pile loads which could occur from an elastic distribution.

For the first case, axial pile loads are limited to the design resistance. For the second case axial pile loads should be limited to $R_k \gamma_R$ (the inverse of the partial resistance factor is applied to check that the structure will perform adequately if the ground is stronger than expected), and the structural design of the piles carried out for the maximum loads obtained.

Where lateral load tests are carried out, consideration shall be given to the differences between the response of a single pile and piles within a pile group.

4.5.1.2 Displacement

Approaches for estimating the settlement and lateral displacement of pile groups that may be adopted include:

- elastic analyses based on assumed load distributions in the piles and load transfer distributions between the pile and ground (e.g. equivalent raft). These methods are only appropriate for simple pile group geometries and ground conditions.
- elastic analyses in which pile interaction is explicitly taken into account (e.g. using the interaction factors of Randolph & Wroth given in Fleming et al, 1992).
- Non linear analyses in which the load transfer is modelled with non linear springs (e.g. p-y, t-z and Q-z curves) and pile interaction is modelled using elastic interaction factors.
- Non linear numerical models where the ground is modelled as a continuum with appropriate soil models for the ground and soil-pile interface.

4.5.2 Static design for lateral loading

Evaluation of the behavior of pile groups subject to lateral loading shall consider interaction between piles.

4.5.3 Cryogenic storage tanks

For design of piled foundations of cryogenic storage tanks, a soil structure interaction analysis shall be carried out. The analysis shall incorporate non linear pile stiffnesses unless it is demonstrated from the results of the geotechnical investigation that a linear response may reasonably be assumed.

Where pile groups are simplified from 3D orthogonal/radial behaviour to axisymmetric behaviour, verification shall be carried out to demonstrate that the analysis assumptions adequately capture and bound the actual behaviour.

Analyses for seismic loading shall consider a lower and upper bound range of soil properties according to the recommendations given in ASCE4.

4.6 PILES IN DEFORMING GROUND

4.6.1 Negative skin friction

If piles are installed in a ground that is subject to settlement, the resulting downdrag force shall be evaluated and considered in the design of the piles.

4.6.2 Heave

If piles are installed in a ground that is subject to heave, the resulting tensile forces shall be evaluated and included in the structural design of the pile.

4.6.3 Lateral displacement

If piles are installed in a ground that is subject to lateral displacement, the resulting load on the pile should be evaluated by a soil structure interaction analysis.

4.7 CYCLIC LOADING

The effect of cyclic loading on the tension capacity of piles shall be considered, and it will often be appropriate to increase the partial resistance factor for this case.

Where cyclic loads are large in relation to the permanent loads, increased settlement of piles loaded in compression may occur and shall also be considered (see for example O'Riordan et al, 2003).

4.8 PILE CONSTRUCTION

4.8.1 Driven prefabricated piles

These changes in resistance with time after driving shall be taken into account when considering the results of dynamic tests, see (4.9.4). A check whether occurrence of relaxation has occurred may be made by a restrike test. The pile is redriven after some time has elapsed since the end of initial driving and the PDA measurements compared with those at the end of initial driving.

The driving stresses and feasibility of installing the piles with the proposed hammer shall be checked in all cases by a wave equation analysis. Particular consideration shall be given to driving stresses and the possibility of pile damage where hard driving is anticipated, or concrete piles are to be driven through hard layers into softer layers (which can lead to tensile stresses).

Consideration shall be given to noise and vibration associated with pile driving and in particular to the potential for tripping machinery. Monitoring shall be carried out when piling close to sensitive structures or equipment and when piling near to potentially unstable slopes. This shall as a minimum include vibration monitoring but where appropriate shall also include settlement and/or pore pressure monitoring and crack monitoring.

Prediction of attenuation of vibrations during pile driving shall be based on experience using similar equipment in similar ground conditions.

For acceptable vibration limits for machinery refer to (5).

4.8.2 Driven cast in situ piles

The execution of driven cast in situ piles shall be in accordance with EN 12699 Clause 8.5 and BS 8004 Clause 7.4.4. Consideration shall be given to noise and vibration associated with pile driving as in (4.8.1).

4.8.3 Bored cast in situ piles

The execution of bored piles shall be in accordance with EN 1536 and BS 8004 Clause 7.4.5.

Where drilling fluid is used to support the bore during construction, there shall be proper control of the fluid properties including de-sanding and replacement with fresh fluid as required.

4.8.4 Continuous Flight Auger piles

Satisfactory construction of cfa piles requires the supply of concrete to match the rate of withdrawal of the auger and thus cfa piles shall only be used where construction is with instrumented rigs with real time displays and records of:

- auger depth;
- concrete pressure;
- amount of concrete placed relative to the nominal hole volume;
- nominal level of concrete above the auger tip.

If there is a high penetration resistance such that penetration of the auger is not compatible with the speed of rotation flighting of spoil can occur leading to overbreak and ground loss. This shall be avoided.

The execution of cfa piles shall be in accordance with EN 1536 Clause 8.1.5 and 8.3.6.

4.9 PILE TESTING

4.9.1 General Requirements

The testing strategy and specification of pile tests shall be an integral part of pile design. Adequate time shall be allowed in the construction programme for testing. The static load testing programme shall be planned by taking into account the level of risk associated with

the pile design and construction based on the guidance given in EN 1997-1. The programme shall be agreed by the Principal based on the recommendations of an experienced geotechnical engineer. Table 5.1 gives the minimum requirements for the numbers of pile tests to be carried out.

Trial pile test(s) shall be carried out for all Geotechnical Category 3 structures unless agreed otherwise with the Principal based on the recommendations of an experienced geotechnical engineer.

Table 4.1 below details the requirements for pile testing for geotechnical design categories 2 and 3.

Table 4.1 Pile load testing strategy

Characteristics of the piling works	Risk level	Testing strategy
Complex ground conditions No previous pile test data New piling technique or limited local experience of piling method	High	Both preliminary trial and working pile tests required 1 preliminary pile test per 250 piles (minimum 2) 1 working pile test per 100 piles
Consistent ground conditions No previous pile test data Limited experience of piling in similar ground	Medium	Either preliminary trial and/or working pile tests required 1 preliminary pile test per 500 piles (minimum 2) 1 working pile test per 100 piles
Consistent ground conditions Previous pile test data is available Extensive experience of piling in similar ground	Low	Pile tests not essential but may result in cost savings by allowing lower partial resistance factors and/or model factor to be used 1 preliminary pile test per 500 piles 1 working pile test per 100 piles

In high density clusters of piles, for example large tank foundations, the number of pile load tests may be reduced subject to the Principal approval.

Where piles are designed to carry both tension and compression loads, tests shall be carried out on tension and compression. Lateral pile tests are not normally required unless lateral loading is shown to be governing the design.

Rapid load tests and dynamic tests may be considered as alternative or complimentary methods to static load testing as set out below.

Rapid load testing may be carried out instead of static pile load testing for works pile tests in low risk conditions as defined by Table 4.1 subject to approval of the Principal. Rapid load testing shall take due regard of the limitations given in (4.9.3).

Dynamic tests on driven piles may be carried out subject to the approval of the Principal instead of static load testing on works piles. It is recommended that tests are performed on not less than 10% of working piles with signal matching. Restrike tests shall be carried out unless it has been demonstrated that relaxation (see 4.9.4) is not an issue. Dynamic load testing shall take due regard of the limitations given in (4.9.4).

4.9.2 Static load tests

Preliminary trial pile tests

The maximum test load shall not be less than the calculated ultimate resistance.

Preliminary trial piles should be constructed sufficiently in advance of the installation of the working piles to allow time for the test, the evaluation of the results and the adoption of modifications if these prove necessary.

If one pile load test is carried out, it shall be located where the most adverse ground conditions are believed to occur. If load tests are carried out on 2 or more piles the test locations shall be representative of the site with one of the test piles located where the most adverse conditions are believed to occur.

The criteria to determine ultimate resistance shall be defined. For a compression pile, this should be taken as the most stringent of the resistance mobilised at a pile head displacement of 10% of the pile diameter or the maximum allowable settlement criterion at working load.

Satisfactory construction of cfa piles requires the supply of concrete to match the rate of withdrawal of the auger and thus cfa piles shall only be used where construction is with instrumented rigs with real time displays and records of:

- auger depth;
- concrete pressure;
- amount of concrete placed relative to the nominal hole volume;
- nominal level of concrete above the auger tip.

If there is a high penetration resistance such that penetration of the auger is not compatible with the speed of rotation, flighting of spoil can occur leading to overbreak and ground loss. This shall be avoided.

The execution of cfa piles shall be in accordance with EN 1536 Clause 8.1.5 and 8.3.6.

The ultimate resistance of bored piles (drilled piers) according to the AASHTO LRFD Bridge Design Specification is limited to a lower settlement of 5% of the pile diameter.

Pile test results shall not be extrapolated beyond the maximum test load to determine the ultimate resistance.

Maintained load (ML) tests in accordance with the ICE Specification for piling and embedded walls (2007) or procedure C in ASTM D 1143 shall be specified.

Where resistance over part of the pile shaft or base is not relied upon in the design due to the loading conditions or uncertainty over construction effects, the test shall be designed to allow the resistance mobilised over these sections to be separately measured. This will normally require the pile to be instrumented. In some cases special measures may be required to reduce resistance over sections of the test pile (for example by sleeving or forming a 'soft toe') in order that the ultimate resistance over the working sections of the pile can be determined.

Bi-directional tests may, subject to approval of the Principal, be considered for tests on bored piles. They are performed by placing a load cell (e.g. Osterberg cell) at the base prior to concreting or part way up a pile bore during concreting. This will require some variation

in construction procedure compared to works piles, the possible significance of which shall be considered when selecting the test method and evaluating the test results. If a bi-directional test is performed without additional reaction at the pile head, the maximum test load that can be applied will be the lower of the resistance of the sections of pile above or below the load cell. Interpretation of the results is needed to derive the equivalent load/settlement characteristic of the complete pile and this interpretation shall be carried out by persons experienced in this type of testing. The applicability of bi-directional testing to assess the ultimate resistance and load-settlement behaviour shall be reviewed by an experienced geotechnical engineer.

Works pile tests

The purpose of proof load tests on works piles is to check that the pile performance meets the required load settlement criteria. The load transfer along a single pile subject to test may not replicate the pile design assumptions and this shall be taken into account when specifying the test loads for works pile tests. The maximum test load shall not be less than:

- 1.5 times the representative design load; or
- 1.5 times the representative design load plus 2 times the characteristic negative skin friction for piles subject to negative skin friction.

Maintained load (ML) tests in accordance with the ICE Specification for piling and embedded walls (2007) or procedure C in ASTM D 1143 shall be specified

The settlement of a working pile shall be compared with specified performance criteria or against the performance of trial piles constructed at the same site. Piles shall be selected for testing on the basis of the results of the ground investigations and any observations during pile construction that give cause for concern.

4.9.3 Rapid load testing (Quasi static)

The results of a quasi static test shall be corrected for dynamic effects.

A test with several loading steps should be undertaken so that the ultimate resistance can be reliably determined.

Where quasi static testing is used to verify static pile resistance it shall be calibrated against static load tests carried out on comparable piles (i.e. of the same type and similar size constructed in the same ground conditions using the same installation criteria).

The method for performing quasi static testing shall be according to the ISO 22477.

4.9.4 Dynamic testing

Unless agreed otherwise by the Principal, dynamic testing shall only be used to test driven piles.

Restrike tests at intervals after the end of driving are required to evaluate the set up.

The pile resistance shall be estimated by numerical simulation in which a synthesized force-time relationship is matched to the measured force-time relationship. The testing shall be carried out by a PDA accredited organization or an equivalent organization approved by the Principal.

Where dynamic testing is used to verify static pile resistance it shall be calibrated against static load tests carried out on comparable piles (i.e. of the same type and similar size constructed in similar ground conditions using the same installation criteria).

4.9.5 Integrity Testing

Low strain integrity tests shall be undertaken on all concrete piles.

It is recommended that cross hole sonic logging be specified on large diameter bored foundation piles (>1000 mm diameter). Normally all piles should be tested, but where large numbers of piles are required, and subject to the agreement of the Principal, the rate of testing may be reviewed after 25% of piles have been tested depending on:

- consistency of test results;

- previous local experience and practice;
- the degree of redundancy in the foundation design.

4.10 DURABILITY

Assessment of the aggressivity of the environment and requirements for concrete mix design shall be in accordance with Appendix 1 of DEP 34.19.20.31-Gen.

In arid regions the design shall take into account highly saline conditions which can give rise to potential for both chemical and physical attack of concrete.

The recommendations given in BS 8004 Clause 10 and BS 8002 Clause 4.4.4.3 shall be followed for the protection of steel piles or for evaluating rates of corrosion in onshore environments. For marine environments, the recommendations given in DEP 35.00.10.10-Gen shall be followed.

4.11 SEISMIC DESIGN OF PILES

4.11.1 Philosophy of Seismic Design

This is defined in (3.9.1).

4.11.2 General Requirements

Unless required otherwise by the National requirements, pile foundations shall be designed for seismic actions in accordance with clause 5.4.2 of EN 1998-5, clause 5.8.4 of EN 1998-1, and EN 1997-1 using Design Approach DA-1. The seismic design of piled foundations shall include consideration of vertical and horizontal stresses and the structural integrity of the foundation.

The requirements in EN 1998-5 for the seismic design of foundations shall be followed with the revised partial factors given in Appendix 2 to account for the performance requirements of medium and high consequence facilities.

Where, with the agreement of the Principal, AASHTO (2007) has been adopted, the load and resistance factors given in Appendix B may be used.

4.11.3 Key Design Considerations

The design of piles for earthquake loading shall include consideration of vertical and horizontal loads.

The performance of piled foundations is influenced by the method of construction and workmanship; thus the design of piles shall take account of anticipated construction methods, local experience and the scope of verification testing that will be carried out.

4.11.4 Vertical Loading

The axial resistance of piles subject to seismic loading shall be taken to be the same as under static loading and may be determined in accordance with Section (4.4.1). The effect of soil strength reduction due to pore water pressure build up shall be taken into account. The partial factors and serviceability requirements shall be as given in Appendix 2.

Occurrence of liquefaction shall be considered in the pile design. Pile buckling within liquefied zones shall be checked.

4.11.5 Lateral Loading

All piles shall be designed to remain elastic under lateral seismic loading under OBE conditions, as it is very difficult to inspect and repair piles following an earthquake. Under certain circumstances (high seismic loading, very poor soil conditions etc), this may not be possible. In these cases, distinct plastic hinge zones shall be identified and appropriately designed.

- For concrete piles, transverse and confinement reinforcement appropriate to the critical region should be provided.

- For steel tubular piles, the buckling capacity of the pile shall be checked and if necessary the critical section filled with concrete.

Piles shall be designed to resist the inertia loads transmitted from the superstructure onto the piles. In certain circumstances the kinematic forces, arising from the free-field soil displacement due to the seismic wave passage, should also be considered (see Figure 4.1).

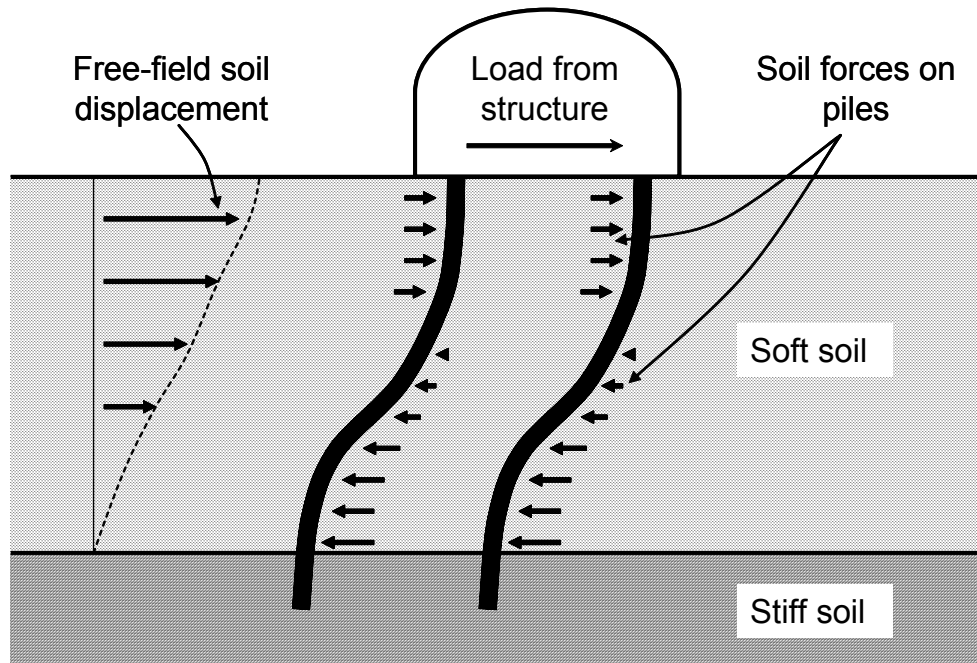


Figure 4.1 Soil-pile interaction

In addition to the requirements of EN 1998-5, kinematic interaction shall be considered for all facilities covered by this DEP, where at least conditions (1) and (3) or (2) and (3) are met:

1. The average shear wave velocity of the soil profile (V_{S30}) is less than 180m/s, or susceptible to liquefaction;
2. The soil profile contains consecutive layers of varying stiffness (EN 1998-1 clause 5.8.4 indicates that, in general, this is where there is a ratio of shear moduli greater than 6);
3. The zone is of moderate or high seismicity (i.e. the 475 year surface peak acceleration is greater than 0.1g).

4.11.6 Pile Group Effects

The group effect of pile groups shall be considered.

4.11.7 Raking Piles

When designing loading platform and trestle structures for moderate to large seismic loads, as a minimum a space-frame analysis shall be carried out that takes into account the varying stiffness of the piles. Design of the piles shall account for displacement demand and appropriate ductility.

4.11.8 Liquefaction induced lateral loading

When liquefaction occurs, the side resistance of susceptible soil layers shall be ignored and the load imposed by these layers on the piles considered

Buckling is an alternative mechanism that may govern liquefaction-induced failure; and shall be included in the pile design checks (Bhattacharya *et al.* 2003). This is in addition to

assessing moment and shear capacity. Piles are to be detailed for ductility to EN 1998-1 clause 5.8.4.

4.11.9 Performance Based Design

Amended per

Circular 68/12

The seismic design of critical structures, such as LNG tanks subject to moderate and high seismic loading should follow a performance based design approach in accordance with DEP 34.00.01.10-Gen. For other less critical cases it is also permissible, subject to approval by the Principal, to undertake a performance based assessment of the structure.

4.11.10 Additional Requirements for Dynamic Soil-Structure Interaction (DSSI)

The requirements in ASCE 4 for defining the ranges of upper and lower bounds soil shear modulus shall be adopted. Justification for the interpreted parameters shall be presented.

Where soft and/or weak soils are present, development of a dynamic soil-structure interaction (DSSI) model to optimise the design should be considered to gain a more accurate appraisal of the system performance. As many piling applications include interaction with soft soils, DSSI should be considered for all piled structures of high consequence except in areas of low seismicity.

The following levels of analysis are accepted:

- Complete: Full numerical analysis of pile, soil and structure in the time domain. The structure may be simplified to a single degree of freedom (SDOF) oscillator in order to apply loading to the foundation, whilst ground acceleration modelled using an input time history (accelerogram).
- Simplified: A simplified conceptual model is analysed with soil behaviour modelled as Winkler springs attached to an elastic pile/ foundation system. Pseudo-static loads may be applied based on separate analyses (e.g. 1-D site response analysis).
- Hybrid models incorporating reduce modelling complexity and computation time, whilst retaining some of the advantages of time-domain numerical analysis.

The approach adopted shall reflect the complexity of the ground conditions and the structural elements. In general, the adoption of more complex methods will yield a higher degree of accuracy of field behaviour, leading to efficient design. Complete analyses should include material non-linearity (strength and stiffness) of both soil and structure.

The experience and competency of the design analyst and software should be verified by the Principal in advance of the analysis being undertaken.

Amended per

Circular 68/12

Further details on requirements for DSSI are given in DEP 34.00.01.10-Gen Earthquake Design for Onshore Facilities.

4.12 DESIGN OF PILES IN COLD REGIONS

This DEP does not cover the design of piles in permafrost regions, for which specialist advice should be obtained.

5. MACHINE FOUNDATIONS

5.1 SCOPE

This section specifies the requirements for the design of machine foundations.

5.2 GENERAL REQUIREMENTS

Foundation design for all machines shall follow the requirements of the following codes:

- CP 2012 Part 1;
- DIN 4024-1;
- ISO 1940-1;
- ISO 10816-1.

In North America design shall be according to the following standard:

- ACI 351.3R-04.

5.2.1 Design Practice

The design of foundations for reciprocating machines shall be in accordance with CP 2012 Part 1. The design of foundations for rotating machines shall be in accordance with DIN 4024 Part 1.

The design of machine foundations shall be governed by performance requirements under static and dynamic loading. The analysis shall include the following:

- Shear Modulus – The shear modulus (G) shall be defined at the appropriate level of strain produced by the dynamic loading, The variability in shear modulus against depth shall also be addressed;
- Geometric and Material Damping;
- Soil Stratigraphy – The method of analysis shall account for the differences between a homogeneous soil, a layered soil and one that has bedrock or a stiff layer at shallow depth;
- Machine Loading.

5.2.2 Design Criteria

Static and dynamic deformation criteria shall be met.

Static deformations shall be within the operating limits defined by the machine manufacturer and shall be calculated to account for the modes discussed in DEP 34.00.01.30-Gen.

Dynamic deformations shall be either defined by the machine manufacturer or by code requirements. The dynamic amplitudes shall not exceed the lower of the following values:

- The maximum allowable values stated by the machine manufacturer.
- If there is no specific manufacturer's guidance, for reciprocating machines the limit on the foundation block shall be defined by line ABB' in Figure 3 of CP 2012 Part 1. This is reproduced in Figures A6.1 and A6.2 as the "limit to ensure reasonable comfort to people".
- For rotating machines the effective velocity ($v_{r.m.s}$) limits shall be as defined by ISO 10816.

5.2.2.1 Machine Data

The following machine data shall be supplied by the machine manufacturer prior to design of the foundations:

- a) general arrangement drawing of the machine assembly showing all dimensions and location, type, and size of anchor bolts and support plates;
- b) mass of the machine and mass and eccentricity of its rotating components;
- c) location of the centre of gravity for the combined machine assembly and for each component;
- d) machine speed (or speed range for variable speed machines) for rotor components of rotary machinery, consisting of maximum continuous operating speed, maximum and minimum speed for trip or alarm setting, or both;
- e) frequency of unbalanced primary and secondary loads of reciprocating machinery;
- f) harmonic, impulsive and other non-stationary loads generated by the machinery;
- g) allowable amplitudes of acceleration, velocity or displacement for machinery and foundation;
- h) space restrictions and maximum maintenance lift load;
- i) magnitude, direction, and point of application of unbalanced forces or couples, or both, for vertical and horizontal directions.

5.2.2.2 Geotechnical Data

For static and dynamic analysis, a geotechnical investigation shall be carried out to determine the soil formation, its representative properties and groundwater conditions, in accordance with this DEP.

The soil properties for static analysis for either shallow or deep foundations shall be in accordance with Sections 3 and 4 respectively of this DEP.

The required soil properties for dynamic analyses shall include soil density (γ), Poisson's ratio (ν), and dynamic shear modulus (G). The value of G shall be determined taking into consideration the following points:

- The G shall be derived from a low strain shear modulus (G_0) and appropriate G/G_0 degradation curves.
- The G_0 value should be determined from direct measurement of shear wave velocity (v_s). Where v_s data are not available, empirical relationships based on SPT N values, CPT Q_c values or Menard pressure meter E_m values may be used.
- The G value should be determined which matches the likely strain amplitude of the machine vibration (typically between 0.001% and 0.0032 %.)

Assuming an analysis for the serviceability limit state; the material partial factors required by EC7 shall be set to unity. Therefore the characteristic soil property shall be used. Both upper and lower bounds values of G shall be considered, with the best estimate G varied by a factor $(1 \pm C_v)$, where C_v shall not be less than 0.5.

5.2.3 Static Analysis

The static analysis of either a shallow or deep foundations shall follow the requirements of Chapters 3 and 4 respectively of this DEP.

5.2.4 Dynamic Analysis

The exciting forces used for design shall be the maximum value of either:

- The manufacturer's recommended value occurring during the lifetime of the equipment;
- The value computed from the Codes of Practice listed above.

The following material damping levels should be taken unless substantiated elsewhere:

- For concrete structures, material damping shall not be greater than 1%;
- For soil, material damping shall not be greater than 3%.

Since the allowed induced strains to the foundation system are restricted to extremely small values (Davies, 1991), linear elastic analyses are accepted for design purposes.

5.2.5 Frequency and Amplitude Requirements

For design of simple high tuned rotating machines, the foundation design shall avoid resonance as described in DIN 4024.

NOTE: Small rotating pumps and generators

For reciprocating or more important rotating machines, or if it is difficult to meet the frequency criteria in DIN 4024, the amplitude of vibration of the foundation shall be assessed and checked against the acceptable criteria.

NOTE: Turbines, Compressors and machines on tables

Unless there is a specific requirement by the machine manufacturer to limit the maximum amplitude of vibration, the following limits shall be used:

- Reciprocating machines should lie below the CP2012 “reasonable comfort to people” level;
- Rotating machines should not exceed velocities of 2 mm/s, or those defined in ISO 10816-1.

5.2.6 Static Deformations

The static deformations of the foundations for rotary equipment shall be within the limits defined by the machine manufacturer. Where applicable, the effects of deformations caused by the following shall also be investigated:

- Shrinkage and creep of the concrete;
- Temperature effects caused by radiation or convection of heat or cold generated by machinery, piping and ducting;
- Elastic deformations caused by changing vapour pressure in the condensers.

5.2.7 Proprietary Mountings

If proprietary mountings are employed to reduce vibration effects on the equipment, the benefits should be demonstrated by suitable dynamic analysis. The type and position of the mountings shall be agreed with the machine manufacturer.

5.3 DESIGN RULES FOR MACHINES AT GRADE

For shallow foundations Arya et al. (1979) suggests a series of rules that may be used to obtain a reasonable initial trial design for which further analysis shall be carried out. These are:

- a) The foundation mass should be at least twice the mass of centrifugal machines and at least three times the mass of reciprocal machines.
- b) The thickness of the foundation should be sufficient to ensure rigidity and, in any case, the thickness shall not be less than 0.6 m or one-fifth of the width or one-tenth of the length.
- c) The foundation plan dimension normal to the axis of rocking should exceed the vertical distance from the foundation base to the machine centre-line.
- d) The plan dimensions should be adjusted to eliminate vertical load eccentricities with respect to the foundation’s centre of resistance.
- e) The embedded depth should be estimated based on operational requirements and construction costs.

For pile foundations, the pile caps should weigh three-quarters the mass of the corresponding shallow foundations. The same general principles of rigidity (pile head fixity) and load centricity apply.

5.4 TABLE FOUNDATIONS

Table foundations shall be in accordance to the norms set in (5.3) for machines at grade, in addition to the following guidelines:

- the structural dynamics of the superstructure shall be considered;
- Structures which are not symmetrically rigid shall require complex finite element analyses.

6. RETAINING STRUCTURES

6.1 TYPES OF RETAINING WALL

The geotechnical design report shall state how the selection of the type earth retaining structure to be adopted in any particular situation is based on:

- a) the location of the wall, its position relative to other structures and the amount of space available, including the necessity or otherwise to confine the support system within the site boundaries;
- b) the proposed height of the wall and the topography of the ground, both before and after construction;
- c) the ground conditions;
- d) the ground water and tidal conditions;
- e) the extent of ground movement acceptable during construction and in service and the effect of movement of the earth retaining structure on existing or supported structures and services;
- f) external live loading;
- g) the availability of materials;
- h) required life and maintenance;
- i) Construction access;
- j) Environmental limitations;
- k) Cost.

6.2 GENERAL REQUIREMENTS

6.2.1 Standards of Practice - Eurocodes

In accordance with Figure 2.1, design shall be according to Eurocodes - EN 1997-1 (Eurocode 7) and EN 1998-5 (Eurocode 8) for retaining walls. Design Approach 1 shall be employed for ULS designs as detailed in EN 1997-1. For geographical locations where a National Annex does not apply the (UK) Nationally Determined Parameters (NDPs) (Appendix 1) shall be adopted.

6.2.2 Standards of Practice - US

See Section (2.3).

6.3 DESIGN

The design of retaining walls in general requires consideration to be given to:

- Method of excavation ;
- Temporary and permanent excavation support;
- Temporary and/or permanent groundwater control;
- Lateral and vertical loading from the ground and groundwater in permanent condition.

Where open excavations are made with battered slopes, the slope design shall be in accordance with section (8) of this DEP.

Where temporary or permanent support is required, the recommendations given in section (6) should be followed:

- Retaining walls shall be designed in accordance with EN 1997-1;
- Quay walls shall be designed in accordance with section 7 of this DEP.

Careful consideration shall be given to design groundwater levels and water pressures which may vary in response to tides, seasonally or in response to the construction works. Where reliance is placed on controlling water levels, the design assumptions shall be verified by appropriate monitoring of pore pressures. The base stability of excavations below ground water level shall be verified in accordance with section 10 of this DEP.

6.3.1 Gravity walls

The design of gravity walls shall comply with Clause 9 of EN 1997-1. Structural design shall be in accordance with DEP 34.00.01.30-Gen.

The passive earth pressure force acting on the resisting side shall take account of planned and unforeseen excavations in front of the wall, for example due to service trench excavations. The passive earth pressure force shall be considered as an earth resistance when designing against base sliding and as a favourable geotechnical action when designing against bearing failure of the foundation.

6.3.2 Spread footing reinforced concrete walls

The design of spread footing reinforced concrete walls shall comply with Clauses 6 and 9 of EN 1997-1.

Structural design shall be in accordance with DEP 34.00.01.30-Gen.

6.3.3 Embedded walls

The design of embedded walls shall comply with Clause 9 of EN 1997-1 and non conflicting guidance given in CIRIA Report C580.

The execution of sheet pile walls shall be in accordance with EN 12063 and the non conflicting guidance in BS 8002 Clauses 4.4.4.3 and 4.4.4.4 shall also be considered

The execution of bored pile walls and diaphragm walls shall be in accordance with EN 1536 and EN 1538.

Structural design shall be in accordance with DEP 34.00.01.30-Gen.

6.3.4 Composite retaining structures

The design shall comply with Clause 9 of EN 1997-1 and non conflicting guidance for cellular cofferdams given in BS 8002 Clause 4.5.3 and for double walled cofferdams given in BS 8002 Clause 4.5.1.4.5.

The design of nailed and reinforced soil structures shall be in accordance with (9).

6.4 ANCHORAGES

The design and testing of anchorages shall be in accordance with EN 1537 and shall consider the additional non-conflicting guidance given in BS 8081.

6.5 GROUND MOVEMENTS

Where excavations are required near to existing facilities the ground movements resulting from the excavation and their effects on adjacent structures shall be evaluated. These may include:

- Ground movements due to retaining wall installation;
- Ground movements due to deflection of the retaining wall as the excavation is formed;
- Ground movements due to dewatering (see Section 10);
- Ground movements due to application of loads near walls.

The methods used to predict ground movements, acceptable limits on movement and requirements for monitoring, shall be agreed with the Principal.

6.6 SEISMIC DESIGN OF RETAINING WALLS

6.6.1 Philosophy of Seismic Design

This is defined in (3.9.1).

6.6.2 Wall Performance

6.6.2.1 Wall Behaviour

In seismically active areas, seismic loading during retaining wall design shall be considered.

6.6.2.2 Required Performance

Walls shall be checked against failure by sliding, overturning and bearing capacity failure, in addition to structural design and global stability checks. The definition of failure is determined on the level of acceptable displacement, which shall be defined using a qualitative risk assessment of the impact of these displacements on the operability and safety of the facility/structure.

For walls whose movement or collapse would not impinge on the performance of critical structures or the operation of the facility, larger tolerable displacements may be permitted with the permission of the Principal. A performance based analysis approach shall be followed. However, if the ongoing serviceability cannot be ensured, the wall should be designed according to EN 1998-5.

For moderate to high consequence structures, performance requirements shall be agreed with the Principal. For normal structures, adherence to code recommendations for allowing controlled translation of the wall to occur in an earthquake shall be followed.

6.6.3 Design Approach

6.6.3.1 General Requirements

The requirements in EN 1998-5 for the seismic design of retaining structures shall be followed using partial factors given in Appendix 2. Where, with the approval of the Principal, AASHTO (2007) has been adopted for design; the load and resistance factors given in Appendix 3 may be used.

For medium and high consequence facilities the design shall be checked using more rigorous performance-based methods.

6.6.3.2 Dynamic Earth Pressures

For yielding walls the Mononobe-Okabe (M-O) design approach should be used (EN 1998-5 Annex E4), and for non-yielding wall types the Wood (1973) elastic method is applicable (EN 1998-5 Annex E9). Acceptable variations to these methodologies may be used with agreement of PRINCIPAL.

NOTE: The same methods are given in AASHTO LRFD Bridge Design Specification. The AASHTO guidance for non-yielding walls suggests a factor of 1.5 may be applied for use in the M-O method for these walls where deflections are uncertain. It is instead recommended that for embedded or fixed base walls of varying flexibility, that the relative stiffness of the wall-soil and wall-foundation system be checked after the method of Veletsos & Younan (1997), and an appropriate factor from those tabulated in that reference be adopted for application to the Wood method of deriving dynamic earth pressures. This enables a more rigorous, yet simplified, appraisal of the dynamic loading on walls of varying flexibility between fully rigid and yielding.

6.6.3.3 Performance Based Design

Performance based design shall follow the relevant guidance in EN 1998-5 clause 7.3.2., noting that this approach does not apply to all wall forms.

NOTE: Or AASHTO (2007) clause 11.6.5 where agreed by the Principal.

For high consequence facilities, a rigorous performance analysis incorporating dynamic soil-structure interaction analyses shall be conducted in addition to meeting code requirements. This is to ensure that the predicted permanent displacements are tolerable for ongoing serviceable use of the facility (ULS) and for the safe-shutdown/ life safety requirements (ALS).

6.6.3.4 Practical Design Recommendations

- a) Choose a carefully graded and compacted backfill to give as much homogeneity as possible with the existing soil mass.
- b) The drainage systems shall be capable of absorbing transient and permanent movements without impairment.
- c) Drainage in cohesionless submerged soils should be well below the potential failure surface and assurance that the supported (or supporting) soil is not liquefiable under the design earthquake. Significantly higher seismic loading occurs when a water table is present in the retained backfill – typically of the order of 1.6 to 2.2 times that for dry backfills.
- d) Concrete walls should be designed either to remain elastic throughout the design earthquake, or to allow limited permanent outward movement. In both scenarios the wall structural elements should be designed not to yield wherever practicable. Where limited permanent displacements are catered for, provision should be made to accommodate the calculated displacement with minimal damage and without encroaching on clearances. Walls other than supported on piles should be designed to slide rather than rotate. Due account shall be taken on the probable upper and lower bounds of the critical acceleration to cause wall displacement. The resistance to overturning shall be greater than 1.25 times the overturning moment derived from upper-bound combination of forces to cause sliding.
- e) Anchored walls shall be designed to remain elastic under the ULS event. Consideration shall be made to the consequence of tie and wall flexibilities under design conditions. Walls shall be designed and detailed so that under the ALS, controlled displacement of the wall will occur through yielding of the anchor material, and sudden failure will be avoided. Particular attention shall be made to post-ULS effectiveness of the tie corrosion protection.
- f) The seismic design of reinforced earth walls shall follow the recommendations of FHWA document No. FHWA-NHI-00-043 *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes. Design and Construction Guidelines*. (Elias et al. 2001). Alternative design approaches such as BS 8006:1995 modified to account for additional seismic inertia loading may be acceptable subject to approval of the Principal. Further specific requirements as follows (after Transit New Zealand 2003):
 - The connection strengths between reinforcement and the facing shall be such that the failure under the ALS is always ductile, that is, by either pull out of the reinforcement through granular materials without loss of pullout capacity with displacement, or by yielding or deformation of the reinforcement, and not by failure of the connections. The strength margin over connection failure shall be at least 1.3
 - A wall intended to undergo permanent displacement shall be designed so that outward movement results from sliding of the reinforced block as a whole and not due to internal instability and pullout or rupture of the reinforcement.
 - Using strip reinforcement, under the ALS, deformation shall be preferably by pullout of the reinforcement strips, or, where this is impractical, by ductile extension of the reinforcement strips.
 - Using grid reinforcement, particularly geogrids with closely spaced transverse members, under the ALS, any internal deformation shall be by ductile elongation of the reinforcement rather than pullout of the reinforcement through the soil.

Where design is for pull out, the nominal strength of the connection between the reinforcement and the wall facing shall be at least twice the pull out force calculated from the probable apparent coefficient of friction. Upper and lower bounds of the critical

acceleration k_c , required to produce incipient failure, shall be calculated by considering the reinforcement acting both horizontally and along the failure surface and allowing for probable variations in the pull out resistance and yield strength of the reinforcement. Stability shall be checked under the upper bound acceleration. Design displacements shall not encroach on required clearances.

6.7 DURABILITY

The guidance given in DEP 34.19.20.31-Gen shall be followed for the protection of concrete in contact with the ground. In arid regions, highly saline conditions giving rise to potential for both chemical and physical attack of concrete may be encountered.

The guidance given in BS 8002 Clause 4.4.4.4.3 shall be followed for the protection of steel piles or evaluating rates of corrosion in onshore environments. For marine environments, the guidance given in DEP 35.00.10.10-gen shall be followed.

Corrosion protection of anchorages shall be in accordance with EN 1537 and shall take account of the non conflicting guidance given in BS 8081.

6.8 EXCAVATIONS IN CLOSE PROXIMITY TO EXISTING FACILITIES

The methods to be used to predict and, where required, limit ground movements shall be agreed with the Principal.

7. MARINE AND WATERFRONT STRUCTURES

7.1 SCOPE

This section applies to the geotechnical design of marine and waterfront structures including:

- Quay walls;
- Jetties and dolphins;
- Breakwaters and revetments.

7.2 QUAY WALLS

7.2.1 General requirements

The design of quay walls shall be in accordance with Sections 4 and 6 of this DEP. Design actions to be considered shall include:

- Earth pressures due to retained soils;
- Tidal range;
- Tidal lag;
- Groundwater levels range (including possible storm surges);
- Hydrodynamic loading;
- Wave loading;
- Scour;
- Over dredging;
- Berthing and mooring loads;
- Imposed loads from operation of the facility (e.g. cranes, containers etc).

Design actions shall be in accordance with BS 6349 Parts 1, 2 and 4.

For relieving platforms, such as crane platforms, interaction between the piles and deck shall be modelled to ensure load - displacement compatibility.

7.3 JETTIES

The design of piles for jetty structures shall be in accordance with DEP 35.00.10.10-Gen.

Interaction between the piles and deck shall be modelled to ensure load - displacement compatibility. Guidance on the use of computer models for soil-structure interaction analyses is given in Appendix 4.

7.4 BREAKWATERS AND REVETMENTS

The design of slopes shall be in accordance with Section 8 of this DEP with additional non-conflicting detail provided in BS 6349 Part 7. The selection of rockfill and assessment of durability shall be in accordance with CIRIA SP83/CUR 154.

7.5 SEISMIC DESIGN OF MARINE AND WATERFRONT STRUCTURES

7.5.1 Philosophy of Seismic Design

This is defined in (3.9.1).

7.5.2 Structure Performance

7.5.2.1 Behaviour of Marine and Waterfront Structures

The avoidance of liquefiable materials is a key requirement for these structures.

7.5.2.2 Required performance

Walls and slopes shall be checked against failure by sliding, overturning and bearing capacity failure; in addition to structural design and global stability checks. The definition of failure is determined on the level of acceptable displacement, which is dependent on the nature of the retaining walls (i.e. a rigid or flexible wall).

For medium and high consequence structures, where seismic loading is moderate to high, additional performance checks should be considered to ensure that serviceability after the ULS event, and safe shutdown/life safety requirements after the ALS event are not compromised. See (7.5.3.2) for further discussion on performance based design.

7.5.3 Design Approach

7.5.3.1 General Requirements

One of two alternative approaches may be used in seismic design. Either the structure can be designed against failure in accordance with EN 1998, or alternatively a performance-based design approach may be employed in accordance with PIANC (2001) where some level of deformation (and allowable level of damage) may be acceptable after an earthquake event subject to agreement with the Principal.

The design approach should depend on the form of the structure being considered:

- For pile supported wharf structures, refer to Section 4 of this DEP;
- For gravity wall, anchored bulkhead and gravity caisson structures, refer to Section 6 of this DEP;
- For wharves incorporating slopes, or subject to a potential "global" failure mechanism, refer to Section 8 of this DEP.

For high consequence facilities and where uncertainty exists in the mode of deformation for medium consequence facilities, dynamic soil-structure interaction (DSSI) analysis using numerical methods (e.g. time domain Finite Element/Finite Difference) should accompany simplified analyses for assessing the likely performance (e.g. Newmark sliding block for gravity and cellular quay walls).

7.5.3.2 Performance Based Design

Subject to approval by the Principal, the seismic design of port and harbour structures may conform to the approach set out in PIANC (2001). However, the design levels in DEP 34.00.01.30-Gen and international code requirements for hazardous facilities include longer return period events than the PIANC guidelines and the amended Performance Matrix given in Table 7.1 with the respective damage criteria expanded in 7.2 shall be applied.

Table 7.1: Modified Performance Matrix for waterfront structures (after PIANC 2001)

Performance Grade	Design Earthquake	
	Level 1 (L1 = ULS, 475 year RP)	Level 2 (L2 = ALS, 2475/ 4975 year RP)
High Consequence	Degree I: Serviceable	Degree III: Near Collapse
Medium Consequence	Degree I: Serviceable	N/A
Low Consequence	Degree III: Near Collapse	N/A

**Table 7.2: Acceptable level of damage in performance-based design¹
(After PIANC 2001)**

Level of Damage	Structural	Operational
Degree I: Serviceable	Minor or no structural damage	Little or no loss of serviceability
Degree II: Repairable	Controlled damage ² .	Short-term loss of serviceability ³ .
Degree III: Near Collapse	Extensive damage in near collapse	Long-term or complete loss of serviceability

- Notes:
1. Considerations: Protection of human life and property, functions as an emergency base for transportation, and protection from spilling hazardous materials, if applicable, should be considered in defining the damage criteria in addition to those shown in this table
 2. With limited inelastic response and/or residual deformation.
 3. Structure out of service for short to moderate time for repairs.

8. EMBANKMENTS AND SLOPES

8.1 SCOPE

This section applies to embankments and slopes. It does not include dikes or dams or bunds retaining free water, tailings or other liquids, which shall be designed by specialists.

Earthworks shall be in accordance with DEP 34.11.00.11-Gen., EN 1997-1 and non-conflicting clauses of BS 6031.

Compaction of fill in embankments shall be end-product specified unless agreed otherwise by the Principal.

8.2 GENERAL REQUIREMENTS

8.2.1 Standards of Practice - Eurocodes

The design of embankments and cutting slopes shall comply with sections 11 and 12 of EN 1997-1. Unless agreed in advance with the Principal, Design Approach 1 shall be employed for ULS designs as detailed in EN 1997-1. For geographical locations where a National Annex does not apply the (UK) Nationally Determined Parameters (NDPs) (Appendix 1) shall be adopted.

8.2.2 Standards of Practice - US

See Section (2.3).

8.3 DESIGN

The geotechnical design report shall address both short-term and long-term stability considering all possible temporary and permanent load conditions and the impact of failure.

Factors to be considered include:

- a) the presence of pre-existing slips;
- b) the orientation and characteristics of discontinuities in rock slopes;
- c) groundwater conditions and pore pressure distributions (both short and long-term);
- d) performance requirements under seismic loading;
- e) suitability of excavated materials for re-use as fill or construction materials and cut-fill balance;
- f) deterioration of slope condition with time and its impact on stability;
- g) requirements for slope protection (berms, gabion mattresses, rock-traps, vegetative cover, rock-mesh etc.);
- h) the use of anchorages, rock bolts, soil-nails or other reinforcement;
- i) the impact of other site development activities adjacent to a slope (for example building loads or the effect of surface water drainage);
- j) ground movements of embankments and areas adjacent to slopes and cuttings;
- k) design of temporary works including surface water control ;
- l) the influence of vegetation;
- m) access and safety for construction and maintenance;
- n) use of staged construction or ground improvement for embankments constructed on soft ground (discussed in Section 9 of this DEP;

NOTE: Additional guidance is given in TRL Report CR341

- o) potential impact of slopes and cuts on air-circulation. The layout of cuttings, slopes, terracing and site development shall be designed by a combination of geotechnical, plant-layout and process specialists;

p) requirements for slope monitoring and maintenance programs.

8.3.1 Cut Slopes in Soil

8.3.1.1 Methods of analysis

The design of embankments and cutting slopes shall comply with sections 11 and 12 of EN 1997-1.

When selecting characteristic strength values, consideration shall be given to the possibility of pre-existing slips and potential for strain softening of the soil.

Since distinction between favourable and unfavourable gravity loads is not possible in assessing the most adverse slip surface, any uncertainty about ground unit weight shall be considered by applying upper and lower characteristic values in separate calculations.

8.3.1.2 Pore pressures and drainage

The stability of cut slopes shall be verified by an effective stress analysis with pore pressures corresponding to steady state seepage. These will depend on the local groundwater levels and drainage measures incorporated in the slope design.

8.3.1.3 Nailed slopes

Where soil nails are used to improve the stability of cut slopes, the design shall consider both overall stability and internal stability. Stability shall be considered for intermediate construction stages as well as at the end of construction.

When assessing overall stability of steep slopes specific consideration shall be given to sliding at the base of the nailed soil block and local bearing capacity at the toe of the slope.

Internal stability checks shall consider:

- pullout of nails through failure at the soil/nail interface;
- rupture of the soil nails;
- bearing capacity or structural failure of the head plates or bearing plates;
- failure of soil between the nails or of any structural facings.

Testing of nails shall be carried out in accordance with the recommendations given in CIRIA C637 and prEN 14490. CIRIA C637 gives frequencies of testing based on the Geotechnical Category.

Durability assessment and corrosion protection of nails shall follow the recommendations of CIRIA C637.

8.3.2 Cut Slopes in Rock

8.3.2.1 Failure Mechanisms

The orientation, nature and spacing of discontinuities shall be established either by field mapping of exposures where practical or by appropriate ground investigation.

The following failure mechanisms shall be considered:

- a) Planar failure;
- b) Wedge failure;
- c) Circular slip failure ;
- d) Toppling failure.

8.3.2.2 Pore pressures and drainage

Full account of water pressures in discontinuities shall be included in rock slope stability assessments. It shall not be assumed that if no seepage is observed from a rock face that groundwater is not present.

8.3.3 Fill Slopes

8.3.3.1 Methods of analysis

Both the stability of the fill slope and overall stability of the fill and underlying ground shall be verified. Checking of ULS stability may be carried out using:

- 'assumed failure surface' methods; or
- numerical analysis and strength reduction technique.

8.3.3.2 Pore Pressures and Drainage

Long term stability shall normally be verified by an effective stress analysis with pore pressures corresponding to long term equilibrium values. These shall take account of possible changes in groundwater level due to placing the fill and changes in infiltration.

Where fills are placed on soft clays short term stability shall be verified assuming undrained conditions in the clay. Staged construction with pre-defined waiting periods may be required to allow an adequate factor of safety against failure to develop.

Stability for each stage shall be verified based on the undrained shear strength at the start of each stage for a stage construction.

8.3.3.3 Reinforced fill

Where soil reinforcement is used to improve stability, the design shall consider both overall stability and internal stability. Specific consideration shall be given to sliding at the base of the reinforced soil block and local bearing capacity at the toe of the slope. Where reinforced soil is placed on soft foundation, soils extrusion stability shall also be considered. See Figure 9.1.

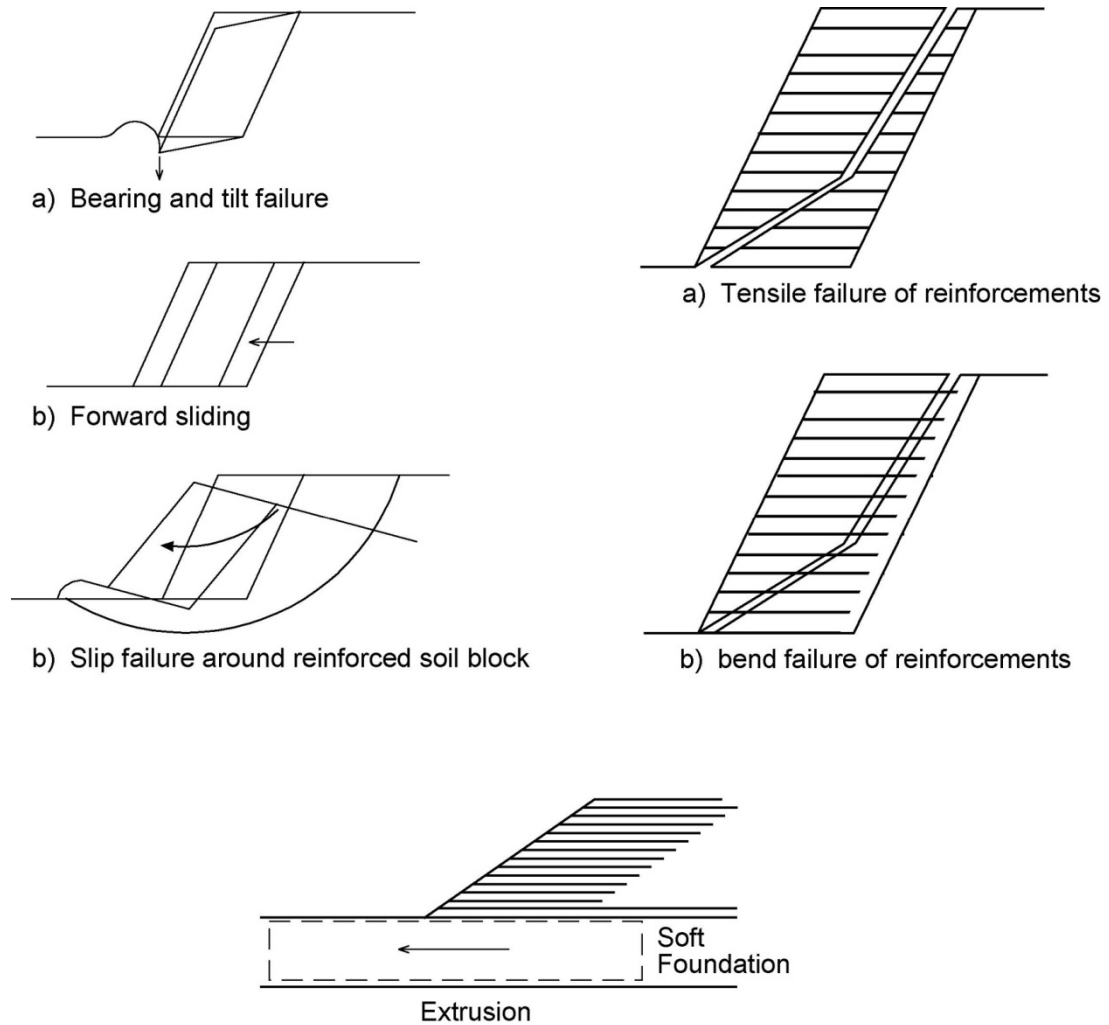


Figure 9.1 Ultimate limit states for reinforced fill

Reinforced fill slopes shall be designed in accordance with BS 8006 adopting the partial factors given in BS 8006 in place of those in EN 1997-1.

8.4 SEISMIC DESIGN OF SLOPES

8.4.1 Philosophy of Seismic Design

This is defined in Section (3.9.1) of this DEP.

8.4.2 Slope Performance

8.4.2.1 Slope Failure Mechanisms

All slopes shall be checked against inertial instability and weakening instability.

8.4.2.2 Required Performance

EN 1998-5 requires that slopes be checked for global stability failure when structures are to be erected on or near natural or artificial slopes, in order that safety and/or serviceability of the structures is preserved under the design earthquake.

The limit state is defined as being unacceptably large permanent displacements of the ground mass, within a depth significant for the structure considered.

8.4.3 Design Approach

8.4.3.1 General Requirements

The requirements in section 4 of EN 1998-5 for the seismic design of embankments, cutting slopes, and assessment of natural slopes shall be followed using the partial factors given in Appendix 2 (A2.3.1).

NOTES: Where, with the agreement of the Principal, AASHTO (2007) has been adopted for design, load and resistance factors given in Appendix 2 (A2.4.1) shall be used.

Seismic design procedures shall follow the same methodology as used for static design in the assessment of slope stability, with additional pseudostatic inertial loads applied to the slope mass considered. Degraded strength and stiffness of the soil due to cyclic loading shall also be considered.

8.4.3.2 Slope Dynamic Effects

The amplification of ground motions for slopes with a slope angle of $\geq 15^\circ$ shall be addressed using the methods set out in EN 1998-5 Clause 4.1.3.2.

If high consequence facilities are supported by, or would be adversely affected by the failure of a slope in excess of this angle, a 2-D site response analysis shall be conducted to better assess ground motions affecting both slope and structure.

8.4.3.3 Additional Considerations

The criteria by which a particular slope is designed should be determined depending on the consequences of slope movement and failure.

Where important structures are located on or close to a slope, an explicit check of slope movement shall be made.

9. GROUND IMPROVEMENT AND REINFORCEMENT

9.1 SCOPE

This section applies to ground improvement and reinforcement using the following ranges of techniques:

- a) Vibration/Compaction;
- b) Preloading;
- c) Structural reinforcement;
- d) Structural fill;
- e) Admixtures;
- f) Grouting;
- g) Drainage/Pre-Consolidation.

9.2 GENERAL REQUIREMENTS

An appropriate procedure for selection and implementation of Ground Improvement shall be followed such as that presented in CIRIA Report C573.

The design of ground improvement and reinforcement shall comply with the requirements of Clause 5 of EN 1997-1.

NOTE: Additional detailed guidance is given in CIRIA Report C573

The main purposes of ground improvement are:

- a) Improving the bearing capacity of the ground;
- b) Reducing the potential total and differential settlement;
- c) Reducing the time during which settlement takes place;
- d) Reducing the potential for liquefaction;
- e) Reducing the permeability of the ground;
- f) Removing or excluding water from the ground;
- g) Increasing the shear strength of the ground and improving slope stability;
- h) Increasing erosion resistance.

Ground improvement frequently involves the use of proprietary techniques. The responsibilities of the Contractor and specialized ground improvement contractor shall be clearly defined. The Principal or Contractor shall:

- a) determine the required purpose(s) of ground improvement;
- b) determine the performance requirements and measurement methods;
- c) select appropriate ground treatment method(s);
- d) determine requirements for trials;
- e) determine testing strategy and acceptance criteria;
- f) assess risks of the scheme with regard to noise, vibration and impact damage;
- g) assess the compatibility of the design of the proposed structure(s) with the expected behaviour of the improved ground.

NOTE: The Geotechnical Contractor will usually undertake these duties on behalf of the Contractor

The selection of improvement method shall take account of:

- a) the area, depth and total volume of soil to be treated;
- b) soil type and its initial properties;
- c) materials availability;
- d) environmental factors, such as waste disposal, erosion, water pollution, effects on adjacent structures and facilities;
- e) local experience and preferences;
- f) time available;
- g) cost.

Specification should be based on end-product performance specifications that can be validated effectively using basic field or laboratory tests.

The party responsible for the design shall:

- carry out the detailed design of the ground improvement works to meet the performance criteria set by the Principal or the Contractor;
- prepare method statements;
- Ensure the execution of the ground improvement works to meet the performance criteria.

Thorough understanding of the treatment process and its effect on the behaviour of the ground shall be acquired where ground improvement is employed. Methods of predicting performance should be based on a realistic assessment of past experience and monitored performance of similar types of treated ground.

9.3 GROUND IMPROVEMENT BY VIBRATION/COMPACTION

Generally accepted methods are listed below:

- a) Vibro-compaction;
- b) Vibro stone columns;
- c) Vibro concrete columns;
- d) Dynamic replacement;
- e) Dynamic compaction;
- f) Rapid impact compaction and high energy impact compaction;
- g) Vibratory probing compaction piles;
- h) Blasting.

Other methods that are not listed above may also be considered subject to approval by the Principal.

Ground improvement shall be undertaken in accordance with the recommendations presented in BS 8004 and CIRIA Report C573.

The execution of ground treatment by deep vibration shall be in accordance with EN 14731.

9.4 GROUND IMPROVEMENT BY PRELOADING/PRE-CONSOLIDATION

The design and installation of vertical drains shall be in accordance with EN 15237.

Analysis shall include an assessment of whether preloading should be staged to prevent instability caused by undrained bearing capacity failure of soft layers.

All preloading schemes shall include appropriate instrumentation and monitoring to determine when the required improvement has been achieved and the preload can be removed.

9.5 GROUND IMPROVEMENT BY STRUCTURAL REINFORCEMENT

The selection of ground improvement by structural reinforcement shall consider their limitations as detailed in CIRIA Report C573 Clause 6 and the guidance on the design of reinforced soil and on the strength and durability of reinforcement is given in BS 8006 and CIRIA Report C637.

9.6 GROUND IMPROVEMENT BY STRUCTURAL FILL

The application of ground improvement by structural fill shall be undertaken in accordance with guidance in CIRIA Report C573 Clause 7.

The use of displacement will not be accepted and shall only be considered with the agreement of the Principal.

The use of lightweight fill is only permitted with the agreement of the Principal and shall only be considered in areas where there is no possibility of contact with hydrocarbons.

9.7 GROUND IMPROVEMENT WITH ADMIXTURES

The application of ground improvement with admixtures shall be undertaken in accordance with guidance in CIRIA Report C573 Clause 8. The execution of deep soil mixing shall be in accordance with EN 14679.

9.8 GROUND IMPROVEMENT BY GROUTING

The application and limitations of ground improvement by grouting are described in CIRIA Report C573 Clause 9 and BS 8004 Clause 6. Execution of grouting works is covered by EN 12715 and of jet grouting by EN 12716.

9.9 TREATMENT OF COLLAPSIBLE SOILS

In situ compaction, chemical stabilization or grouting, removal and replacement and pre-wetting are techniques that may be considered for treating collapsible soils.

Robust acceptance criteria shall be developed for all these techniques based on thorough field or laboratory studies to ensure that the potential for residual settlement or collapse after treatment is within acceptable limits.

9.10 SEISMIC DESIGN

The project design, performance requirements and the site constraints (i.e. site location and its accessibility, possible adjacent structures sensitive to vibration, construction program, budget and availability of specialty contractors in the area) shall be considered when determining the type of improvement which is most appropriate to achieve satisfactory performance in terms of stability and acceptable deformation of the treated ground under seismic loading.

The execution of the ground improvement shall be in accordance with section (9.2) with appropriate field trials and testing.

The treatment depth shall extend either to the depth of influence of the structure or to the bottom of the liquefiable layer. The treatment zone shall extend outside the perimeter of the structure at least a distance equal to half the thickness of the treated layer.

The behaviour of a soil deposit subject to seismic loading is not always positively affected by ground improvement, therefore aspects of behaviour that shall be considered include:

- the strength of treated deposits may be reduced by migration of the pore pressure from neighbouring untreated zones into the improved soil;
- improved ground may amplify the earthquake motion on the overlying structure;
- additional forces can be imposed by an earthquake on treated ground located in sloping areas if adjacent unimproved soil experiences lateral spreading.

10. DEWATERING AND UPLIFT

10.1 SCOPE

This section applies to the design of:

- temporary or permanent dewatering schemes;
- submerged structures (including basins and basements);
- uplift of impermeable layers in excavations.

10.2 GENERAL REQUIREMENTS FOR DEWATERING SCHEMES

The design and construction of dewatering schemes shall comply with the requirements of EN 1997-1 Clauses 5 and 10.

The issues that shall be considered include:

- a) Effects of uncertainties in stratigraphy and permeability and potential value of pumping tests;
- b) Changes in permeability or hydraulic conductivity caused by pumping;
- c) Capacity and allowance for modification to system depending on observed performance;
- d) Requirements for monitoring ground water levels;
- e) Requirements for redundancy to allow for clogging, breakdown, maintenance etc;
- f) Settlement resulting from drawdown;
- g) The effects of short-term extreme events such as localized high groundwater levels due to storms, floods or damaged water pipes, etc;
- h) Discharge of groundwater and possibility of groundwater being contaminated;
- i) Potential impact on contaminant migration;
- j) Environmental regulations and requirements for consents/permits;
- k) Potential for saline intrusion;
- l) Potential for contamination of aquifers;
- m) Hydraulic stability.

10.3 UPLIFT

Design situations involving uplift, where there is no seepage of water, shall be analysed using the partial factors for the UPL limit state in EN 1997-1. This shall apply to submerged structures and uplift of impermeable layers.

Failure by uplift is checked by comparing the sum of the water pressures under the structure and any other upwards forces with the sum of the stabilising permanent vertical actions and the design value of any additional resistance to uplift provided, for example by tension piles or anchors. If water pressure acts on top as well as beneath a structure, the single source principle in EN 1997-1 shall be applied and a single partial factor applied to the difference between these actions.

For design against uplift of an impermeable layer, the destabilising total water pressure shall be less than or equal to the stabilising total vertical stress due to the weight of soil above the interface.

Design situations where there is seepage due to hydraulic gradients should be analysed using the partial factors for the HYD limit state in EN 1997-1.

11. REFERENCES

In this DEP reference is made to the following publications:

- NOTES:
1. Unless specifically designated by date, the latest edition of each publication shall be used, together with any amendments/supplements/revisions thereto.
 2. The DEPs and most referenced external standards are available to Shell staff on the SWW (Shell Wide Web) at <http://sww.shell.com/standards/>.

Amended per

Circular 99/11

SHELL STANDARDS

The use of SI quantities and units (endorsement of ISO 31, ISO 1000 and ISO 80000)	DEP 00.00.20.10-Gen
Earthquake design for onshore facilities - Seismic hazard assessment	DEP 34.00.01.10-Gen
Structural design and engineering	DEP 34.00.01.30-Gen
Site Investigations	DEP 34.11.00.10-Gen
Site preparation and earthworks, including tank foundations and tank farms	DEP 34.11.00.11-Gen.
Roads, paving, surfacing, slope protection and fencing	DEP 34.13.20.31-Gen.
Reinforced concrete foundations and structures	DEP 34.19.20.31-Gen
Onshore Steel structures	DEP 34.28.00.31-Gen
Vertical steel storage tanks - selection design and construction (amendments/supplements to EN14015)	DEP 34.51.01.31-Gen.
Aboveground vertical storage tanks – Design and construction (based on API 650)	DEP 34.51.01.33-Gen
Design of jetty facilities	DEP 35.00.10.10-Gen
Vertical tanks - Erection and testing (based on EN 14015)	DEP 64.51.01.31-Gen

AMERICAN STANDARDS

Building Code Requirements for Structural Concrete and Commentary	ACI 318 / 318R
Code Requirements for Environmental Engineering Concrete Structures and Commentary	ACI 350 / 350R
Design, Manufacture, and Installation of Concrete Piles	ACI 543R

Issued by:

*American Concrete Institute (ACI)
38800 Country Club Drive
Farmington Hills, MI 48331, USA*

LRFD Bridge Design Specification	AASHTO
Guide specifications for seismic isolation design	AASHTO

Issued by:

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APPENDIX 1 EN1997-1 – REQUIREMENTS OF UK NATIONAL ANNEX

A1.1 APPLICATION OF EN 1997-1

For geographical locations where a National Annex does not apply, the following (UK) Nationally Determined Parameters (NDPs) shall be adopted. Where no NDP are given the recommendations of EN 1997-1 shall be followed.

Clause 2.4.7.1(4)

The values of partial factors to be used in cases of abnormal risk or unusual or

Clause 2.4.7.1(5)

The values of partial factors for temporary structures or transient design situations shall generally be as given for permanent structures.

Note: Only where it can be justified on the basis of possible consequence, less severe values may be used for temporary structures or transient loadings.

Where the likely consequences justify less severe values, the values to be used shall be agreed with the Principal.

Clause 2.4.7.3.4.1 (1)

Design Approach 1 shall be used for the STR and GEO limit states.

Annex A

A2.2 Partial factors for soil parameters (γ_m) for the EQU limit state²

Soil Parameter	Symbol	Value
Angle of shearing resistance ¹	$\gamma_{\phi'}$	1.1
Effective cohesion	$\gamma_{c'}$	1.1
Undrained shear strength	γ_{cu}	1.2
Unconfined strength	γ_{qu}	1.2

A3.2 Partial factors for soil parameters (γ_m) for the STR and GEO limit state²

Soil Parameter	Symbol	Set M1	Set M2
Angle of shearing resistance ¹	$\gamma_{\phi'}$	1.0	1.25
Effective cohesion	$\gamma_{c'}$	1.0	1.25
Undrained shear strength	γ_{cu}	1.0	1.4
Unconfined strength	γ_{qu}	1.0	1.4

- NOTES
1. Applied to $\tan\phi'$ and $\tan\phi'_{cv}$ although it might be more appropriate to determine the design value of ϕ'_{cv} directly.
 2. The value of the partial factor for soil parameters should be taken as the reciprocal of the specified value if such a reciprocal value produces a more onerous effect than the specified value.

A1.2 PARTIAL RESISTANCE FACTORS FOR PILE FOUNDATIONS

For verification of the structural (STR) and Geotechnical (GEO) limit states of pile foundations the partial factors on resistance given in tables A6 to A8 below shall be used. These values are used to convert characteristic resistances to design values for ultimate limit state calculations. They apply irrespective of the process by which the characteristic resistances are derived.

Table A6 - Partial resistance factors (γ_R) for driven piles for the STR and GEO limit

Resistance	Symbol	Set		
		R1	R4 without explicit verification of SLS*	R4 with explicit verification of SLS*
Base	γ_b	1.0	1.7	1.5
Shaft (compression)	γ_s	1.0	1.5	1.3
Total/combined (compression)	γ_t	1.0	1.7	1.5
Shaft in tension	$\gamma_{s,t}$	1.0	2.0	1.7

Table A7 - Partial resistance factors (γ_R) for bored piles for the STR and GEO limit

Resistance	Symbol	Set		
		R1	R4 without explicit verification of SLS*	R4 with explicit verification of SLS*
Base	γ_b	1.0	2.0	1.7
Shaft (compression)	γ_s	1.0	1.6	1.4
Total/combined (compression)	γ_t	1.0	2.0	1.7
Shaft in tension	$\gamma_{s,t}$	1.0	2.0	1.7

Table A8 - Partial resistance factors (γ_R) for continuous flight auger piles for the STR and GEO limit

Resistance	Symbol	Set		
		R1	R4 without explicit verification of SLS*	R4 with explicit verification of SLS*
Base	γ_b	1.0	2.0	1.7
Shaft (compression)	γ_s	1.0	1.6	1.4
Total/combined (compression)	γ_t	1.0	2.0	1.7
Shaft in tension	$\gamma_{s,t}$	1.0	2.0	1.7

* The lower γ values in R4 may be adopted in Tables A6 to A8 if

- (a) serviceability is verified by load tests (preliminary and/or working) carried out on more than 1% of the constructed piles to loads not less than 1.5 times the representative load for which they are designed or
- (b) if settlement is explicitly predicted by a means no less reliable than in (a) or
- (c) if settlement at the serviceability limit state is of no concern

The values of partial factors given in Tables A6 to A8 are considered to be generally applicable for pile foundations. If variation of these factors is justified in particular circumstances by thorough consideration and documented experience then any such variation shall be agreed by the Principal.

A1.3 CORRELATION FACTORS FOR PILE FOUNDATIONS

For the verification of Structural (STR) and Geotechnical (GEO) limit states, the correlation factors ξ given in Tables A9 to A11 below shall be applied to derive the characteristic resistance of axially loaded piles.

- ξ_1 on the mean values of the measured resistances in static load tests
- ξ_2 on the minimum value of the measured resistances in static load tests
- ξ_3 on the mean values of the calculated resistances from ground test results
- ξ_4 on the minimum values of the calculated resistances from ground test results
- ξ_5 on the mean values of the measured resistances in dynamic load tests
- ξ_6 on the minimum value of the measured resistances in dynamic load tests

Table A9 - Correlation factor (ξ) to derive characteristic values of the resistance of axially loaded piles from the static load tests (n - number of tested piles)

ξ for n =	1	2	3	4	≥ 5
ξ_1	1.55	1.47	1.42	1.38	1.35
ξ_2	1.55	1.35	1.23	1.15	1.08

NOTE: For structures having sufficient stiffness and strength to transfer loads from 'weak' to 'strong' piles values of ξ_1 and ξ_2 may be divided by 1.1.

Table A10 - Correlation factor (ξ) to derive characteristic values of the resistance of axially loaded piles from ground test results (n - number of profiles of tests)

ξ for n =	1	2	3	4	5	7	10
ξ_3	1.55	1.47	1.42	1.38	1.36	1.33	1.30
ξ_4	1.55	1.39	1.33	1.29	1.26	1.20	1.15

NOTE: For structures having sufficient stiffness and strength to transfer loads from 'weak' to 'strong' piles values of ξ_3 and ξ_4 may be divided by 1.1.

Table A11 - Correlation factor (ξ) to derive characteristic values of the resistance of axially loaded piles from dynamic impact tests (n - number of tested piles)

ξ for n =	≥ 2	≥ 5	≥ 10	≥ 15	≥ 20
ξ_5	1.94	1.85	1.83	1.82	1.81
ξ_6	1.90	1.76	1.70	1.67	1.66

NOTE 1 The ξ values may be multiplied by a model factor of 0.85 when using dynamic impact tests with signal matching

2. If different piles exist in the foundation groups of similar piles should be considered separately when selecting the number n of test piles

The CUR committee on rapid load testing are developing a table of correlation factors for rapid load testing. Prior to release of these factors, Table A11 above shall be used for rapid load testing with a model factor of 0.85.

NOTE: The model factor has been taken to be the same as for dynamic tests with signal matching

A1.4 PARTIAL FACTORS FOR THE UPLIFT LIMIT STATE (UPL) VERIFICATION

A1.4.1 Partial factors on actions (γ_F)

The partial factor given in Annex A of EN 1997-1 for permanent unfavourable actions does not account for uncertainty in the level of the groundwater or free water. In cases where the verification of the UPL is sensitive to the level of ground water or free water, the design value of uplift due to water pressure may be directly assessed in accordance with 2.4.6.1(2) and 2.4.6.1(6) of EN 1997-1:2004.

Alternatively, a safety margin may be applied to the characteristic water level, see 2.4.6.1(8) of EN 1997-1.

A1.4.2 Partial factors on soil parameters (γ_M) and resistances (γ_R)

For the verification of the uplift limit state (UPL) the partial factors on soil parameters and resistances should be as given in Table A16 below

Table A16 Partial factors for soil parameters (γ_M) and resistances (γ_R) at the uplift (UPL) limit state

Soil parameter	Symbol	Value ²
Angle of shearing resistance ¹	$\gamma_{\phi'}$	1.25
Effective cohesion	$\gamma_{c'}$	1.25
Undrained shear strength	γ_{cu}	1.4
Resistance	Symbol	Value
Tensile pile resistance	$\gamma_{s,t}$	See note 3
Anchorage	γ_R	1.4 ⁴

- NOTES
1. Applied to $\tan\phi'$ and $\tan\phi'_{cv}$ although it might be more appropriate to determine the design value of ϕ'_{cv} directly.
 2. The value of the partial factor for soil parameters should be taken as the reciprocal of the specified value if such a reciprocal value produces a more onerous effect than the specified value.
 3. Pile design should comply with A3.3.2 and A3.3.3 above.
 4. Larger values of γ_R should be used for non-prestressed anchorages to make their designs consistent with those of tension piles.

A.1.5 MODEL FACTORS FOR PILE DESIGN

When the approach of EN 1997-1 clause 7.6.2.3(8) Equation 7.9 or clause 7.6.3.3(6) Equation 7.18 is used to derive the characteristic resistance, a model factor shall be applied to the shaft and base resistance calculated using characteristic values of soil properties to give values of characteristic resistance. The value of the model factor should be 1.4, except that it may be reduced to 1.2 if the resistance is verified by a maintained load test taken to the calculated, unfactored ultimate load.

APPENDIX 2 APPLICATION OF EN1998-5 TO HAZARDOUS FACILITIES

A2.1 OVERVIEW

EN1998-5 gives the requirements for seismic design of foundations, retaining walls and slopes. The corresponding values of partial factors to be used in conjunction with EN1997-1 Design Approach 1 and serviceability requirements are given in A2.3.1 below.

A2.2 APPLICATION OF EN1998-5 WITH EN1997-1

Partial factors and serviceability requirements

The partial factors and serviceability requirements given in Tables A2.1 and A2.2 shall be used.

Table A2.1 Partial load and strength factors and serviceability requirements (not including piles)

Performance Level	Load Factors			Strength Factors			Serviceability Requirements
	Static		Seismic	Soil Properties			
	Perm. γ_G	variable γ_Q	Importance γ_I	Drained Shear γ_ϕ	Undrained Shear / Unconfined Rock γ_{cu}, γ_{qu}	cyclic undrained shear strength $\gamma_{\tau cy}$	
Normal Consequence Facilities							
OBE / Life Safety (ALS)	1.0	1.0	1.4	1.25	1.4	1.25	N/A
Medium and High Consequence Facilities							
OBE /Operational (ULS)	1.0	1.0	1.0	1.25	1.4	1.25	Elastic Response
SSE/Safe Shutdown (ALS)	1.0	1.0	1.0	1.0	1.0	1.0	Damage Limitation Check

Table A2.2 Partial load and strength factors and serviceability requirements for pile foundations

Performance Level	Load Factors		Strength Factors				Resistance Factor R	Serviceability Requirements
	Static		Seismic	Soil Properties				
	Perm. γ_G	variable γ_Q	Importance γ_I	Drained Shear γ_ϕ	Undrained Shear / Unconfined Rock γ_{cu}, γ_{qu}	cyclic undrained shear strength $\gamma_{\tau cy}$		
Normal Consequence Facilities								
OBE / Life Safety	1.0	1.0	1.4	1.25 ^{1.}	1.4 ^{1.}	1.25 ¹	See Tables A6 to A8 above	N/A
				1.0 ^{2.}	1.0 ^{2.}	1.0 ²		
Medium and High Consequence Facilities								
OBE / Operational	1.0	1.0	1.0	1.25 ^{1.}	1.4 ^{1.}	1.25 ¹	See Tables A6 to A8 above	Elastic Response
				1.0 ^{2.}	1.0 ^{2.}	1.0 ²		
SSE/Safe Shutdown	1.0	1.0	1.0	1.0 ^{1.}	1.0 ^{1.}	1.0 ¹	See Tables A6 to A8 above	Damage Limitation Check

NOTES: 1. Applies to lateral loading and negative skin friction effects
2. Applies to axial loading

APPENDIX 3 AASHTO LRFD BRIDGE DESIGN SPECIFICATION

A3.1 OVERVIEW

The fundamental inequality that shall be satisfied for each limit state is

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n$$

Where Q_i is the force effect caused by applied loads, imposed deformations or volumetric changes

η_i is a load modifier relating to ductility, redundancy and operational importance

γ_i is a load factor

R_n is the nominal resistance determined on the basis of the specified dimensions, permissible stresses, deformations or strength of materials

ϕ is a resistance factor

A3.2 APPLICATION OF LRFD BRIDGE DESIGN SPECIFICATION

Foundations

Clause 10 of the AASHTO LRFD specification covers the design of foundations. Three sets of limit states are identified in Section 10.5.1

- service limit states
- strength limit state
- extreme event limit states

The load factors, γ_i , to be adopted depend on the load combination. In general load combinations SERVICE I and STRENGTH I given in section 3 should be considered for foundation design. Where appropriate, load combinations EXTREME I and/or EXTREME II shall also be considered.

The value of η_i should generally be taken as 1.0.

The values of resistance factor for the strength limit state ϕ are set out in AASHTO LRFD Clause 10.5.5. They depend on the method of determining the resistance and in the case of driven piles the method of field verification of pile capacity.

If a type of foundation or method of evaluating resistance is to be used that is not specifically covered by the AASHTO LRFD specification, the resistance factors to be adopted shall be subject to agreement by the Principal. They shall be justified by thorough consideration and documented experience and take account of the method(s) of field verification to be undertaken.

Presumptive values for bearing pressure (AASHTO LRFD Clause 10.6.2.) may only be adopted for Category I structures (see Appendix 1 of this DEP) and if agreed by the Principal. All values used shall be justified by comparable local experience and shall not be taken directly from Table C10.6.2.3.1-1 or Table C10.6.2.3.2-1 (AASHTO LRFD) without further substantiation.

Overall stability shall be verified using the load factors for load combination SERVICE I and resistance factors given in AASHTO LRFD Clause 10.5.2.

For the seismic design of spread foundations and pile foundations, the load and resistance factors given in Tables A2.3 and A2.4 (AASHTO LRFD) may be used

Table A3.1 Partial load and resistance factors and serviceability requirements for spread foundations

Performance Level	Load factor γ_i			Resistance factor ϕ_i			Serviceability Requirements
	Permanent γ_P	Temporary, including Earthquake γ_{EQ}	Importance γ_I	Sliding		Bearing ϕ_b	
				Shear ϕ_t	Passive ϕ_p		
Normal Consequence Facilities							
OBE / Life Safety	Various – refer Table 3.4.1-2 AASHTO 2007)	1.0	1.4	1.0		N/A	
Medium and High Consequence Facilities							
OBE / Operational (ULS)	Various – refer Table 3.4.1-2 AASHTO 2007)	1.0	1.0	1.0		Elastic Response	
SSE/Safe Shutdown (ALS)	Various – refer Table 3.4.1-2 AASHTO 2007)	1.0	1.0	1.0		Damage Limitation Check ¹ .	

Note: 1. Damage Limitation Check required for medium and high consequence facilities only

Table A3.2 Partial load and resistance factors and serviceability requirements for pile foundations

Performance Level	Load factor γ_i			Resistance factor ϕ_i		Serviceability Requirements
	Permanent γ_P	Temporary, including Earthquake γ_{EQ}	Importance γ_I	Condition/Resistance Determination Method	Site Variability/ Test Frequency	
Normal Consequence Facilities						
OBE / Life Safety	Various - refer Table 3.4.1-2 AASHTO 2007	1.0	1.4	1.0 (compression) 0.8 (tension/ uplift) (refer10.5.5.3.3 AASHTO 2007)		N/A
Medium and High Consequence Facilities						
OBE / Operational (ULS)	Various - refer Table 3.4.1-2 AASHTO 2007	1.0	1.0	1.0 (compression) 0.8 (tension/ uplift) (refer10.5.5.3.3 AASHTO 2007)		Elastic Response
SSE/Safe Shutdown (ALS)	Various - refer Table 3.4.1-2 AASHTO 2007	1.0	1.0	1.0 (compression) 0.8 (tension/ uplift) (refer 10.5.5.3.3 AASHTO 2007)		Damage Limitation Check ¹ .

Note: 1. Damage Limitation Check required for medium and high consequence facilities only.

Retaining structures

Clause 11 of the AASHTO LRFD specification covers the design of retaining structures. Three sets of limit states are identified in Clause 11.5 (AASHTO LRFD).

- service limit states
- strength limit state
- extreme limit states

The load factors, γ_i , to be adopted depend on the load combination. In general, load combinations SERVICE I and STRENGTH I given in section 3 should be considered. Where appropriate, load combinations EXTREME I and/or EXTREME II shall also be considered.

The value of η_i should generally be taken as 1.0.

The values of resistance factor for the strength limit state, ϕ are set out in AASHTO LRFD Clause 11.5.6.

APPENDIX 4 NUMERICAL ANALYSIS

A4.1 INTRODUCTION

No specific guidance is given in EN 1997-1 on the use of numerical analysis for design calculations. This Annex gives recommendations for:

- calculations using computer software;
- finite element analysis (FEA) for limit state design calculations;
- dynamic numerical analysis.

Requirements are given for presentation and reporting of numerical analyses together with a recommended approach to checking.

For further guidance on the use of FEA, reference should be made to Baudin et al (2000) and Simpson & Yazdchi (2003).

A4.2 PRESENTATION AND REPORTING

A4.2.1 General Requirements

Where calculations are undertaken using computer software, the calculation report shall include

- A description of the software describing the basis of the calculations carried out and specific options selected
- The version of the software used
- A sketch and/or graphics and tabulated data showing the model analysed, assumptions made and input parameters
- Key results including graphical output where appropriate

The recommendations for stiffness parameters in ULS calculations given in A3.4.3 below should generally be followed.

A4.2.2 Finite Element Analyses

Where a numerical analysis forms part of the design calculations, the calculation report shall include

- a) A sketch or drawing of the model analysed showing boundary conditions, material zones, pore water pressures and applied loads. For a staged analysis, a sketch or drawing should be given for each stage.
- b) A description of the constitutive model for each material and input parameters for the material
- c) A statement describing how stresses have been initialised
- d) A statement on the tolerance limits specified and convergence achieved
- e) Key results for forces and displacements together with sufficient additional plots of output to describe the behaviour of the model

A4.3 CHECKING AND VERIFICATION

The checking of finite element analyses should always include inspection of plotted output including overall patterns of stress, displacement and pore pressure as well as checking of input data. Checking should be recorded and the use of a proforma is recommended.

A4.4 USE OF FEA FOR ULS ANALYSES

Where FEA is used for ULS analyses the method by which factors of safety are applied requires careful consideration. The key issues that shall be addressed are:

- How to apply factors
- When to apply factors in a staged analysis
- Whether values of stiffness should be factored

A4.4.1 Application of partial factors

Where design is carried out to EN 1997-1 using Design Approach DA1, the partial factors may be applied to the material strengths and loads as described in A3.4.2.

NOTE: If design is carried out to EN1997-1 using Design Approaches DA2 or DA3, or to AASHTO LRFD, where partial factors are applied to resistances, FEA is less readily used for ULS analyses.

Where complex soil models are used with several input parameters and the partial factors given in EN 1997-1 cannot be applied, a procedure in which Mohr-Coulomb limits, based on factored strengths, are applied to the stresses computed at critical stages should be used.

A4.4.2 Factors in staged analyses

In computations for stage construction, the computation should generally be carried out in terms of characteristic values, with factors for ULS design applied at specific stages.

The results from stages in which factors have been applied correspond to EN 1997-1 DA1 Combination 2.

Results from analyses in which characteristic values have been used throughout should as a check be multiplied by a load factor (which acts as a model factor). This is equivalent to EN 1997-1 DA1 Combination 1.

A4.4.3 Stiffness parameters

Characteristic values of deformation parameters should generally be used in ULS analyses except where the failure mechanism or structural forces may be significantly influenced by deformation parameters. This includes

- situations involving laterally loaded piles or retaining walls. In these cases, p-y curves should be derived using factored soil strengths or reduced soil modulus values consistent with the factored values of strengths adopted.
- problems with complex ground conditions where there are large variations in stiffness between adjacent soil layers. In these cases analyses should be performed using upper and lower bound deformation parameters.

A4.4.4 Use of elastic-plastic models for structural elements

Where elastic-plastic models are used for structural elements, including improved ground such as grouted soil, consideration shall be given to the likelihood of brittle behaviour.

A4.5 MODELLING UNDRAINED BEHAVIOUR IN FINITE ELEMENT ANALYSES

Several approaches are available for modelling undrained behaviour in finite element analysis, using combinations of effective stress and total stress parameters. Both stiffness and strength shall be considered.

If effective stress parameters (c' , ϕ') are used for strength, the model shall compute undrained pore pressure in order to determine effective stresses, and hence to find the undrained strength

A4.6 MODELLING ROCK IN NUMERICAL ANALYSES

Careful consideration shall be given to the type of model used for modelling rock in numerical analyses.

A4.7 DYNAMIC NUMERICAL ANALYSIS

Key considerations in the selection of appropriate software and development of analysis models are:

a) Dynamic Boundary Conditions:

Correct performance of dynamic boundaries depends on lateral and vertical extents of the problem being modelled.

b) Integration Method and Numerical Noise:

High frequency noise is often a by-product of the numerical integration process. Methods to remove this noise exist including modification of the integration procedure, or addition via viscous damping (e.g. Rayleigh damping).

c) Spatial-Time Discretisation:

The spatial discretisation of the mesh affects the wave propagation in the model, and if not correctly considered, may filter important frequencies, or induce numerical noise. The temporal discretisation may influence the stability of the integration procedure.

d) Modelling Large Displacements:

Specific consideration needs to be given to the capabilities of the software to handle large displacements if non-linear analysis is to be carried out.

e) Soil-Structure Interfaces:

Consideration shall to be given as to how the interfaces between soil and structure elements are modelled, including effects such as “gapping” of piles, and sliding interfaces.

f) Soil Dynamics:

Soil models have a significant impact on how soil dynamic behaviour is modelled. Consideration should be made to correct modelling of soil damping, regardless of soil model used. Simple models for static analysis are often unsuitable for dynamic analysis.

g) Input Loading and Time

History Selection: The seismic loading is modelled using earthquake time history recordings, appropriate selection of which requires detailed consideration.

h) In-situ stress state:

The in-situ stress state should be modelled for performance based design. It may be ignored for a purely inertial analysis (i.e. site response analysis).

i) Site Response and Validation:

Careful consideration of the site response should be made in order to aid in validation of the model. Use of simplified 1-D models is advised to assist this process.