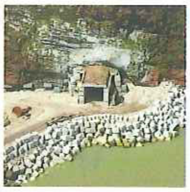
TRANSLATED VERSION OF THE ORIGINAL SPANISH TEXT

Description and Characterisation of Design Factors

RECOMMENDATIONS FOR MARITIME WORKS



ROM 0.5-05

Geotechnical recommendations for the Design of Maritime and Harbour works

GOBIERNO DE ESPAÑA
MINISTERIO DE FOMENTO
Puertos del Estado

RECOMMENDATIONS FOR MARITIME WORKS **SERIES 0**

Description and Characterisation of Design Factors

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January 2008

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Foreword

The current **ROM Programme** comprises a variety of studies and debate activities for developing the set of reflections released as the "Recommendations for Maritime Works". This series of publications set up a permanent framework of technical regulations in harbour and coastal engineering that is dependant on the Public Administrations that have competence over ports of general interest for the State.

The ROM programme is aimed at consolidating the extensive Spanish experience in this field and to establish the various standards of Reliability, Serviceability and Operability to which each Project should conform. In summary, it will foster a higher quality in the performance of all the stages of Harbour Works from their planning, design, construction and first operation to their maintenance and potential repairs or dismantlement.

This regulatory Programme of Recommendations was started in 1987, by the then– called Ministry of Public Works, through the General Directorate of Coasts and Ports. The programme's activities have continued ever since, in a constant evolution that has materialised in a set of publications of wide applicability. They have received general praise –in Spain and abroad– as excellent technical tools for the design and construction of harbour infrastructures.

These documents are the outcome of continuing processes of discussion and subsequent agreements between the different technical viewpoints existing in this field. They go through various Working Groups –made up of experts, collaborating with the main document proponent- and several extended Committees for revising the drafts from the future user's perspective. This procedure is –and will be– the best guarantee of the publications' technical rigour and applicability, as they incorporate a critical revision of the latest techniques employed and valuable experience gained by Spanish port engineers, both from practical applications and theoretical developments.

The publications now included in the ROM Programme can be grouped according to several subjects:

- **Series 0:** Description and characterization of the factors of projects of maritime and port works.
- **Series 1:** Works providing shelter against sea oscillations.
- **Series 2:** Works inside the sheltered port areas.
- **Series 3:** Planning, management and operation of port areas.
- **Series 4:** Superstructures and land facilities in port areas.
- **Series 5:** Maritime and port works in the surroundings of the coastal area.
- **Series 6:** Technical, administrative and legal provisions.
- **E/ROM discussion forum:** Scientific and technical studies and analyses on the ROM Guidelines.

The first one, geared toward characterising the works site and design factors in maritime and harbour works, includes some Recommendations already approved and released: on 'Use and Operation Factors: Actions in Design' (ROM 0.2-90), 'Climate Agents I: Sea Level Oscillations' (ROM 0.3-91), 'Weather Agents. Atmospheric Processes: wind' (ROM 0.4-95) and 'Geotechnical Recommendations for Maritime Structures' (ROM 0.5-94) and the last one on 'General Procedure and Requirements in the Design of Harbour and Maritime Structures, Part I' (ROM 0.0). Some documents are still being prepared, like those on 'Construction Materials' (ROM 0.1), 'Seismic agents' (ROM 0.6) and 'Measurement and Monitoring Systems' (ROM 0.7).

So, back in 1994, *Series 0* included a fist publication on ***Geotechnical Recommendations*** *for the Design of Maritime and Harbour Works (ROM 0.5-94)*. After more than a decade in force, they have proved to be very useful and influential in improving harbour designs in the geotechnical field. Today, these recommendations have been revised and updated taking into account the results of their application, the new developments and the latest experience in order to keep this text permanently in line with current needs and best practise.

This new ROM 0.5-05 therefore is the first finished revision or update of one of the Recommendations for Maritime Works previously released. Some revisions of current documents will follow suit.

The procedure for preparing the present Recommendations has been the one adopted for the regulatory ROM Programme. It has been drawn up under the responsibility –and the supervision– of *Puertos del Estado*, with the participation of the following people:

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The Expert Groups work finished on July 2005 and the Document was approved unanimously by the extended Technical Committee. The President of Puertos del Estado ratified it on November that year.

This new version of the **Geotechnical Recommendations for Maritime and Harbour Works (ROM 0.5-05)** distinguishes itself from the previous one in the adoption of the new General and Calculation Procedure that the ROM 0.0 (2001) established for all Ports Works –specially its new requirements on Safety and works capability for Service and Use or Operation. Moreover, this version includes new contents in the light of new technological developments and knowledge gathered in recent years. In this respect, it is worth noting a preliminary approach to the influence that dynamic agents –earthquakes, wave action or other sea-level oscillations– may have on saturated soil behaviour and, consequently, on the verification for various geotechnical failure modes affecting harbour works.

Nevertheless, these new updated Recommendations keep the essential goal of the preceding version, which is no other than combining methodological, regulatory and technological aspects in a true Guide –modern, wide-ranging and comprehensive– for providing technical assistance in everything affecting, conditioning or involved in ports infrastructure from the geotechnical point of view. For that reason, ROM 0.5-05 includes the following:

- Definitions of programmes and techniques for **Geotechnical Investigation.**
- Setting up of **Design Criteria** for choosing geotechnical parameter values, verification procedures and safety requirements against failure modes.
- Coverage of **specific geotechnical problems** in different types of maritime and ports works, with recommended analysis methods and solutions for each one.

Any comments, suggestions, requests for clarification or other contributions that may arise about the contents of this new ROM 0.5-05, as well as on the practical results of its application, will be welcome. They shall be taken into account in future revisions of this document.

- The entire ROM 0.5-05 can be downloaded from *http://www.puertos.es*
- Please send comments to the ROM Programme Coordinators, at *programarom@puertos.es*

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General

GENERAL

Part I

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1.1 SCOPE AND FIELD OF APPLICATION

These Geotechnical Recommendations for the Design of Maritime and Harbour Works (referred to as ROM 0.5) apply to all maritime and harbour works of whatever type or purpose and to the materials and elements employed in their construction.

For these purposes, the expression Maritime and Harbour Works refers to structures, structural elements and earthworks located in maritime or river ports or in any other State-owned surface in the shoreline of Spain, whether fixed or floating, provided that they remain in a stationary situation during the Service Stage.

These Recommendations also cover the auxiliary construction stages or elements that could affect the safety or correct operation in the Service Stage of existing structures meeting the requirements set out in the previous paragraph.

1.2 CONTENTS

These Recommendations include the necessary criteria for defining and carrying out geotechnical investigation studies, processing the information obtained from such studies, the methods for analysing the different geotechnical aspects and the manner of evaluating the parameters in each case and, finally, for studying the geotechnical problems involved in each of the most common types of structure in maritime and harbour works.

This ROM 0.5 Set of Recommendations has been arranged into four parts to cover the geotechnical aspects related to maritime and harbour works.

- Part 1. General. This covers the general aspects required to apply and understand these Recommendations properly: their field of application, general summary of their contents, definitions, the units, notations and symbols used and reference documentation.

- Part 2. Geotechnical Investigation. This lays down the different stages needed for choosing the values of the geotechnical parameters to be used in design analysis and calculations: compilation of preliminary data, planning and implementation of the investigation programme (fieldwork and laboratory tests) and preparation of the Geotechnical Report.

- Part 3. Geotechnical Criteria for Design. This gives general recommendations referring to the analysis methods to be used in evaluating different typical situations and the safety margins to be adopted in each case.

- Part 4. Geotechnical Problems in Different Types of Maritime and Harbour Works. This deals with the situations that must be analysed for each of the most common types of maritime and harbour works and gives specific recommendations concerning the analysis methods for each of them.

Comment: The different Parts of this ROM 0.5 are interrelated. Although the order chosen to present them follows the logical sequence of geotechnical design, engineers need to be familiar with the Recommendations as a whole before they apply any of their parts. Particularly, in order to programme the appropriate investigation (Part 2), it will first be necessary to know the type of problem that the geotechnical investigation will contribute to solving (Part 4) and the analysis methods to be used (Part 3).

1.3 SAFETY CRITERIA

The ground is responsible for a large number of failures in maritime and harbour works. Geotechnical design should guarantee against ground failure and, in general, any abnormal behaviour of the works originated by the ground.

The general concept of *safety* needs to be broken down into the following three parts:

a. Reliability. It refers to safety against serious failures, those that are called *Ultimate Limit States* in the ROM 0.0 publication and this ROM 0.5. It is defined as the complementary value of the joint probability of failure for all the principal modes assigned to Ultimate Limit States.

b. Serviceability. To be taken as the level of safety existing against failure causing the works to lose serviceability –either reversibly or irreversibly– as a result of structural, aesthetic, environmental failures or legal constraints which, in the ROM Publications, are called *Limit States of Serviceability*. The concept is defined as the complementary value of the combined probability of failure for all the principal modes assigned to Serviceability Limit States.

c. Operability. This concept corresponds to the level of safety against potential breaks in operation, that is, of guaranteed usage of the works. Such interruptions mainly differ from serviceability failures in the type of cause giving rise to the loss of usage guarantees. Serviceability failures are supposed to persist until the relevant engineer decides to correct them by repairing or reconstructing the works, whereas operational failures end as soon as the originating agent (generally related to the weather) ceases to exist. Operability is defined as the complementary value of the probability of stoppage due to all the principal modes of interruption assigned to Limit States of Operational Stoppage.

The level or degree of safety that should be guaranteed in the design of the different maritime and harbour works is indicated in Section 3.2 and depends on the *nature* of the works.

1.4. RELATION TO STRUCTURAL EUROCODES

The Commission of the European Community has made every effort to lay down a set of technical rules known as Eurocodes and designed to be systematically applied to the design and construction of building and civil engineering works in EU member states. Amongst other aspects, the codes attempt to regulate *geotechnical designs*.

In relation to this ROM 0.5, some Eurocodes are particularly relevant, namely *EN1990 Eurocode 0: Basis of structural design* and *EN1997 Eurocode 7: Geotechnical design - Part I: General rules*. Both of these codes were taken into consideration when drawing up this version of ROM 0.5.

Eurocode EC0 indicates that the reliability verifications related to safety, utilisation and durability of structures should be performed following the procedures indicated in each specific Eurocode. Similarly, in the ROM Programme of publications, ROM 0.0 states that reliability, serviceability and operability -entailing similar concepts to those defined in the Eurocodes- must be checked according to the different specific publications in the ROM Programme.

Specifically, EC0 and subsequent Eurocodes (from EC1 to EC9) indicate that, if checking calculations are done, they must be organised according to the concept of Limit States and solved by applying calculation methods based on partial factors. An alternative method is to base designs directly on probabilistic methods in the specific cases that remain unsatisfactorily resolved using partial coefficients. This requires the help of experts and goes beyond the scope of the Eurocodes. In a manner similar to and compatible with EC0, ROM 0.0 states that whenever calculations are done, the concept of Limit States should be used and partial factors applied and, in addition, for certain cases, the analysis completed with probabilistic calculation procedures.

An Annex to the EC0 states that the partial factors appearing in the Eurocodes have been deduced with the intention of guaranteeing the minimum levels of reliability required.

In order to fix the minimum level of reliability required for Ultimate Limit States, three types of works (or structures or structural components) are laid down in the Eurocodes, as a function of the consequences of their failure. Each one is assigned a minimum required reliability index β, as shown in Table 1.4.1.

Table 1.4.1. Eurocodes. Minimum Values Recommended for the Reliability Index β (Ultimate Limit States)

	Reliability Class	Minimum β Values	
		1 Year Reference Period	50 Year Reference Period
Ultimate Limit States	RC3	5,2	4,3
	RC2	4,7	3,8
	RC1	4,2	3,3

In a similar manner, ROM 0.0 indicates a target reliability level for the Ultimate Limit States that depends on the nature of the works.

For irreversible Ultimate States of Serviceability, EC0 indicates the following target reliability indices: $\beta = 2.9$ for a one-year reference period and $\beta = 2.3$ for a 50-year reference period. Similarly, ROM 0.0 indicates several minimum reliability indices for Ultimate States of Serviceability.

Though the application fields for the Eurocodes and the ROM publications are different, one common area exists (harbour works) where it would be desirable for them to be compatible. Current information points to the fact that application of Eurocode EC0 to fields in common with the ROM Programme will be compatible with publications following the general lines indicated in ROM 0.0. In the end, the different publications applying the ROM Programme will be the ones to conclude whether the application of the Eurocodes is acceptable for the ROM Programme and compatible with it in specific cases of the set of works where the fields of application overlap.

ROM 0.5 indicates target values for reliability indices for geotechnical failure modes associated with Ultimate Limit States. Namely, Section 3.2 recommends values $\beta_1 = 2.3$; $\beta_2 = 3.1$ and $\beta_3 = 3.7$; depending on the nature of the works and their design working life. Maritime and harbour works tend to have working lives ranging from 15 to 50 years. The target reliability given in ROM 0.5 could be similar to that of the Eurocodes when the design life is 50 years and when low SERI rated works are comparable to the RC1 of the Eurocodes and high or very high SERI rated works can be compared to Eurocode RC2. Eurocode RC3 does not have a counterpart in ROM 0.5. Works with a minor SERI rating in this ROM 0.5 would not have any equivalent representation either in the Eurocodes. For cases such as these and for working lives other than 50 years, the partial coefficient values would need to be adapted somewhat. This is certainly possible as both the Eurocodes and ROM 0.5 provide a simple procedure for adjusting the partial coefficient values slightly, thereby allowing the effect of the intended target reliability to be taken into consideration, when the differences are not substantial.

The partial safety factors (or *partial factors*, for simplicity) defined in Eurocode EC7 are for guidance purposes only - the different EU member states are free to adapt them to their specific conditions. Even more, each country can choose several verification paths within the EC7, which vary in the manner of considering safety, be it in the loads or in their effects, or else in the strength parameters and their values. The same is true for calculation models. EC7 indicates several possible routes; each member state is free to choose one of them or permit several alternatives. The procedures currently indicated in Code EC7 and the safety factors now included for guidance purposes concur with the philosophy in this ROM 0.5 publication. Eurocode EC7 admits three possible checking procedures that can be symbolically represented as follows:

- **Design Approach 1**

  Checking the two possible combination sets of partial factors:

  - Combination 1: A1 '+' M1 '+' R1
  - Combination 2: A2 '+' M2 '+' R1

  For piles and anchors, the partial coefficient combinations are:

  - Combination 1: A1 '+' M1 '+' R1
  - Combination 2: A2 '+' (M1 or M2) '+' R4

- **Design Approach 2**

  Checking with a single set of partial factors:

  - Combination: A1 '+' M1 '+' R2

- **Design Approach 3**

  Checking with a single set of partial factors:

  - Combination: (A1 or A2) '+' M2 '+' R3.

  In these symbolic expressions, the terms imply the following meanings:

  - A1: partial factors for actions, equal to those for the structural calculation;
  - A2: partial factors for actions, with somewhat lower values, specific for the geotechnical calculation;
  - M1: partial factors for material strength parameters equal to 1;
  - M2: partial factors for material strength parameters;
  - R1, R2, R3 and R4: partial factors for resistances;
  - '+': means combined with;
  - (A1 or A2): means that A1 is the set of factors for structural actions and A2 for geotechnical actions.

The partial factors for actions can be applied to the load values or to their effects.

Design Approach 1 includes two checks as a way of covering both the purely geotechnical design and the structural design of foundation elements.

It is obvious that understanding all the details of these three design procedures requires studying the whole of Code EC7.

This ROM 0.5 chooses a design procedure -solely for geotechnical checks- consisting of the set of partial factors that can be represented symbolically as follows:

- **ROM 0.5 Approach**

  Geotechnical calculation: partial factors set: A2 '+' M1 '+' F.

In this manner, the design calculations are always done with the soil parameters believed to be the most representative and the load values increased by a small factor. This method ties in, with minor changes, with the previous practice described in ROM 0.5-94, only now the calculation is done increasing the value of the loads somewhat.

This manner of proceeding, although similar to those appearing in Eurocode 7, does not match any of them. The values for *F* indicated in this ROM 0.5 would be formally comparable to the values for *R* indicated in EC7, although they are generally greater, because most of the geotechnical uncertainty is included in them.

This ROM 0.5, which does not take into account the structural design for foundation or retaining elements (walls), states that a different set of partial factors must be considered for the structural design, the definition of which lies beyond the scope of this publication.

As long as the National Application Documents for EC7 are being drawn up and while their application remains non-mandatory, this ROM 0.5 indicates provisional ways for making calculations considered to be acceptable and compatible with what the Spanish national application document is currently sensed to be. This point should be confirmed in the future and some aspects of this ROM 0.5 may inevitably need to be changed.

When EC7 compliance becomes mandatory, it is advisable that designs for any maritime and harbour works that could be implemented under its regulations be also analysed with the criteria of the ROM Programme. This will provide experience for weighing up the possible differences.

1.5 DEFINITIONS

Only definitions of the terms most commonly used in dealing specifically with geotechnical problems have been included in this section. Readers are referred to the remaining ROM Programme publications for other terms not included in the following list.

- ACCIDENTAL LOAD: fortuitous or abnormal load occurring as a result of an accident, incorrect use, human error, defective functioning of an element, exceptional working or environmental conditions, etc.
- ACTION (or LOAD): any acting agent or force capable of generating changes in stress or strain either in structures or in the ground.
- ACTIVE ANCHOR: anchor which is prestressed after its installation.
- ACTIVE EARTH PRESSURE: pressure on a retaining structure when it moves away from the load with a sufficiently extensive displacement.
- ANCHOR: a tensile element enabling a point load to be applied to a retaining structure.
- APPARENT DENSITY: real mass of a sample (solid particles + water) divided by its total volume.
- APPARENT SPECIFIC WEIGHT: real weight of a sample (solid particles + water) divided by its total volume.
- AT-REST EARTH PRESSURE: pressure on a retaining structure in the ideal situation of null displacement.
- BACKFILL FACE: unexposed surface of a retaining structure, in contact with the backfill it retains.
- BOREHOLE: deep, small-diameter shaft, drilled with extraction of the core.
- CLAQUAGE/FRACTURE GROUTING: treatment consisting of injecting a cement grout under high pressure into the ground, which causes the formation of thin grout lenses as a result of hydraulic fracturing in the soil.
- CLAY: soil fraction with an apparent particle size of less than 0.002 mm (approx.) in grading tests by sedimentation.
- COEFFICIENT OF UNIFORMITY: D_{60}/D_{10} ratio of a soil sample.
- COLLAPSE: reduction in volume of some soils when their water content is increased without any variation in the pressure applied. This term is sometimes used in a general sense referring to overall failure of works.
- COMPACTION GROUTING: treatment consisting of grouting a thick sand and cement mortar into the bottom of a borehole thereby causing displacement in the ground.
- CONSOLIDATION: transient period when saturated soil expels water until a stable state is reached.
- COUNTERPRESSURE: the earth pressure that needs to be mobilised in the bottom of a diaphragm wall to obtain a balance between the forces acting on it.
- DEGREE OF CONSOLIDATION: dissipated percentage of the excess porewater pressure generated as a result of applying a load to saturated soil with a high proportion of fines.

- CREEP: slide of soil over a rock substrate along a surface lying largely parallel to the slope.
- DEGREE OF SATURATION: the percentage of voids filled with water.
- DESIGN SITUATION: Simplified model of an actual problem that includes a definition of the geometry and characteristics of the materials and loads, for the associated timeframe (duration). It is the basis for carrying out the corresponding calculations. Sometimes it is also called a *Design State*.
- DOLPHIN: freestanding structure designed to support horizontal mooring or berthing loads or to protect other structures (preventing direct vessel impact against them).
- DRY DENSITY: mass of the solid particles divided by the total volume of a sample.
- DRY SPECIFIC WEIGHT: weight of the solid particles divided by the total volume of a sample.
- DYNAMIC COMPACTION: ground treatment consisting of applying energy to its surface by means of the impact caused by a large mass falling from a great height.
- DYNAMIC LOAD: a load causing significant acceleration in the ground or structures.
- EFFECTIVE PRESSURE: intergranular pressure transmitted through the contacts between soil particles. Also called *effective stress*.
- EMBEDMENT: the part of a structure remaining below the surface of the ground.
- FILL: artificial deposit composed of natural materials or artificial or waste products.
- FINES: particles with size less than ASTM 200 sieve (0.076 mm) or UNE 0.080 sieve (0.08 mm). In rock-fills, the term is usually applied to fragments of less than 1" (25 mm).
- FOUNDATION: Part of a structure that transmits loads to the ground. The term may also refer to the ground itself that supports the structure.
- FREE SWELLING: percentage volume change experienced by a soil when saturated under low pressures.
- FRONT FACE: external surface of a retaining structure.
- GENERAL FILL: fill in areas away from structures and/or without a structural function.
- GRANULAR FILL: fill of materials with a low content of fines (i.e., of silts and clays).
- GRAVEL: soil fraction with a particle size ranging from 2 mm to 60 mm (approx.), classified as *fine gravel* up to 6 mm, *medium gravel* up to 20 mm and *coarse gravel* over 20 mm.
- GROUND RESISTANCE FACTOR: coefficient used in dividing the estimated value of ground resistance to obtain the (reduced) design value for use in calculations.
- HYDRAULIC FILL: fill deposited by a process of sedimentation of the solid particles contained in effluents (generally from dredging).
- HYDRAULIC GRADIENT: the difference in piezometric level between two points divided by the distance between them.
- JET-GROUTING: soil treatment whereby a mixture is produced consisting of the cement grout and the soil itself (broken up as a result of applying high speed jets of fluid through small diameter nozzles).

- JOINT: surface of discontinuity in a rock mass caused by stresses experienced over the course of its geological history.

- LIMIT STATES: design states in which the works as a whole or sections or elements thereof remain out of use or service as a result of failure to comply with the safety, service or operating requirements specified in the design.

- LIQUEFACTION: loss of the shear strength of saturated fine granular soil with low relative density, as a result of porewater pressure increases originated by vibration.

- LOAD FACTOR: coefficient by which the representative values of loads are multiplied to obtain the (increased) values used in calculations.

- MAXIMUM SIZE: the smallest sieve opening which allows the whole of a particular soil sample to pass through.

- METASTABLE SOIL: soil susceptible of undergoing substantial volumetric change on variation of its water content.

- MICROPILE: pile drilled in situ with a small diameter (normally less than 300 mm), generally reinforced (with bars or tubular elements) and filled with mortar grouted by pressure applied at the mouth.

- MOISTURE (or WATER) CONTENT: quotient between the weight of water contained in a particular sample and the weight of the dry ground.

- MUD: fine-particle soil with low density and consistency deposited in a watery medium.

- NEGATIVE FRICTION: the increase in the load on a pile due to settlement in the surrounding ground. Also, *negative skin friction*.

- NORMALLY CONSOLIDATED SOIL: soil whose current effective pressure is equal to its preconsolidation pressure.

- OVERCONSOLIDATION RATIO (OCR): quotient between the preconsolidation pressure and the current effective pressure.

- PASSIVE ANCHOR: anchor not prestressed (or only very slightly) during the installation process, which will later take up load over the course of the successive Stages of the works.

- PASSIVE EARTH PRESSURE: pressure on a retaining structure when it undergoes sufficiently extensive displacement against the ground.

- PEAK STRENGTH: the maximum shear strength of a particular soil.

- PENETRATION TEST: test consisting of driving rods with a standardised tip and size into the ground and measuring the resistance to the driving process. The driving can be done by applying blows (*dynamic penetration test*) or pressure (*static penetration test*).

- PENETROMETER: apparatus for carrying out penetration tests.

- PERMANENT LOAD: load acting constantly during the design stage under analysis.

- PERMEATION GROUTING: treatment consisting of injecting cement and water grout (with or without additives) or other products through the soil voids. The similarity of the operation procedures means that the term can also be applied to grouting for filling fractures or small voids in rock.

- PIEZOCONE: apparatus consisting of a static penetrometer with a piezometer fitted to its tip.
- PIEZOMETRIC LEVEL: the height reached by the water level in a piezometric tube placed at a particular spot.
- PILE: a slender structural member used as deep foundation.
- PIPING: particle entrainment producing internal erosion in natural ground or artificial fills caused by water seepage.
- POREWATER PRESSURE: the pressure transmitted though the water filling the interstitial spaces in the ground.
- POROSITY: ratio between the volume taken up by voids and the total volume of the sample (solid particles + voids).
- PRECONSOLIDATED SOIL: soil whose current effective pressure is less than its preconsolidation pressure.
- PRECONSOLIDATION PRESSURE: the maximum effective pressure a soil has borne in the past.
- PRELOADING: ground treatment whereby a temporary load (usually an earthfill) is applied to the ground before the final loads come into action, in order to accelerate consolidation and improve the strength and deformability of the ground.
- PRESSURE: Force or stress applied against a surface, normally due to earth or water actions. When given in units of stress, it is usually called *unitary pressure*.
- PROP: see *strut*.
- ROCK: natural aggregate of one or more minerals not undergoing any substantial change in the presence of water.
- RUBBLE FILL: fill using human waste products (refuse, rubble, etc.) of a non-uniform nature. Also known as *anthropic fill*.
- RESIDUAL STRENGTH: the shear strength of a particular soil subjected to much greater deformations than that corresponding to peak strength.
- RESISTANCE: capacity of an element or system to withstand actions without reaching failure. Force that opposes to movement or failure. *Unitary resistance* (per unit area) may coincide with *strength*.
- SAFETY FACTOR: quotient of two uniform magnitudes - one representing ground resistance (numerator) and the other representing the effect of loads (denominator).
- SAND: soil fraction with a particle size ranging from 0.08 mm to 2 mm (approx.), classified as *fine* up to 0.2 mm; *medium* up to 0.6 mm and *coarse* over 0.6 mm.
- SATURATED DENSITY: density of a saturated sample with all its voids full of water.
- SATURATED SPECIFIC WEIGHT: the specific weight corresponding to a saturated sample with all its voids full of water.
- SATURATION MOISTURE CONTENT: the moisture content of a saturated sample.
- SCOUR: ground surface erosion caused by water movement.

- SERVICEABILITY LIMIT STATES: situations where a particular works, structure or element ceases to comply with the design quality requirements laid down (for functional, aesthetic, durability reasons, etc.), although this may not mean it will be immediately unserviceable or useless.

- SHORING: structure to retain an excavation, installed at the same time as excavation work proceeds.

- SHRINKAGE: reduction in volume experienced by some soils when their moisture content decreases.

- SILO EFFECT: physical phenomenon of reducing the vertical compression on a material stored in a silo as a result of friction with the silo walls.

- SILT: soil fraction with a particle size of between 0.002 mm and 0.08 mm (approx.).

- SOIL: part of the earth's crust formed by fragmented or loose material that can be easily separated into individual particles.

- STABILITY NUMBER: dimensionless parameter (unit weight of the soil multiplied by the height and divided by the cohesion) used in the analysis of stability, earth pressure on shoring, etc.

- STATIC LOAD: a load not causing significant acceleration in the ground or structures.

- STRENGTH: property of a material which represents its capacity of resisting mechanical actions. Normally expressed in units of stress. Different from *resistance*.

- STRUCTURAL CAPACITY: resistant capacity of a foundation or retaining element regardless of the ground.

- STRUCTURAL FILL: fill near to a structure and whose characteristics play an important role in the stability or deformation of the works as a whole.

- STRUT: member working under compression enabling a load to be applied to a retaining structure. Also known as *brace* or *prop*.

- SUBGRADE REACTION MODULUS: the result of dividing the compression applied to a ground surface by the displacement produced in the direction of the compression. Also known as the *Winkler modulus*.

- SUBMERGED DENSITY: the virtual density that saturated material would acquire when immersed in water. Also known as *buoyant density*.

- SUBMERGED SPECIFIC WEIGHT: the virtual specific weight that the saturated material would acquire on being immersed in water. Also known as *buoyant unit weight*.

- SUBSIDENCE: generalised settlement on the ground surface owing to a substantial change in internal stress - a typical example is caused by lowering the groundwater table in soft soils.

- SWELLING PRESSURE: pressure preventing the expansion of soil during its saturation process.

- SWELLING: increase in volume experienced by some soils when their moisture content is increased.

- THIXOTROPY (THIXOTROPIC SENSITIVITY): ratio between the shear strength of a clayey material in its natural state and that of the same material after being remoulded with identical density and moisture content.

- TOTAL PRESSURE: combination of the pressure transmitted through contacts in the soil skeleton and the porewater pressure. Also called *total stress*.

- TRIAL PIT: shallow excavation carried out manually or with machinery to explore the soil.

- ULTIMATE LIMIT STATES: situations where a particular works, structure or element becomes unserviceable as a result of fracture, bearing failure, loss of stability or any other type of failure.

- UPLIFT: buoyant force produced by water on a structure, retaining element or underwater foundation.

- VARIABLE (or LIVE) LOAD: a load whose size or position may vary during the design stage under analysis.

- VIBROCOMPACTION: densification treatment of granular soil by introducing a vibrating probe or 'vibroflot'.

- VIBROFLOTATION: vibrocompaction treatment without the addition of backfill.

- VIBROREPLACEMENT: vibrocompaction treatment with the addition of coarse granular material.

- VIRTUAL BACKFILL FACE: equivalent backfill face on which earth pressure is considered to act when specific problems related to retaining structures are under analysis.

- VOID RATIO: ratio between the volume taken up by voids and the volume occupied by solid particles. Sometimes known as *pore index*.

- WAVE HEIGHT: the vertical distance between a crest and the preceding trough.

1.6 SYSTEM OF UNITS

This ROM 0.5 uses the International System of Units (SI), which is the legally mandatory system of measurement units in Spain.

The most commonly used basic SI units applied in the geotechnical field are:

- Length : metre (m).

- Mass : kilogram (kg) or its multiple, the metric tonne (1 t = 1000 kg).

- Time : second (s).

- Force : newton (N) or its multiple, the kilonewton (1 kN = 1000 N).

  The pascal (1 Pa = 1 N/m^2) is the unit of pressure or stress (force per unit area) along with its multiples, the kilopascal (1 kPa = 1 kN/m^2) and the megapascal (1 MPa = 1 MN/m^2 = 10^3 kN/m^2 = 10^6 N/m^2). A further multiple, the bar, is also used, equivalent to 10^5 Pa = 100 kPa.

Comment: The correspondence between the international system of SI units and the metro-kilopond-second system is as follows:
Newton – kilopond: 1 N = 0.102 kp and inversely 1 kp = 9.81 N
Newton per square millimetre – kilopond per square centimetre: 1 N/mm^2 = 10.2 kp/cm^2 and conversely 1 kp/cm^2 = 0.0981 N/mm^2.

The units preferred for defining the principal geotechnical parameters are indicated in Table 2.14.1.

1.7 NOTATIONS

Table 1.7.1 gives the conventional notations, abbreviations and symbols relating to the geotechnical aspects specifically used in these ROM 0.5 Recommendations.

Table 1.7.1. Basic Conventional Notations, Abbreviations and Symbols Used in this ROM 05

Symbol	Definition	Units
I. UPPER CASE LETTERS		
A	Area	m^2
A,B	Porewater pressure coefficient	*
A_f	Pile shaft area	m^2
A_p	Pile tip area	m^2
B	Breadth or lesser plan dimension of a foundation	m
B	Breadth of an excavated site	m
B*, L*	Equivalent dimensions of a shallow foundation	m
C	Perimeter of a pile cross-section	m
C_c	Compression index	*
CI	Consistency index	*
C_s	Swelling index	*
C_u	Coefficient of uniformity	*
D	Depth (of a shallow foundation, thickness of a layer, etc.), width, diameter	m
D	Relative damping	%
$D_{máx}$	Maximum particle size of a sample	mm
D_r	Relative density	%
D_x	Size of sieve allowing x% of a particular soil sample to pass through	mm
E	Modulus of elasticity	MN/m^2
E	Resultant of earth pressure on a retaining structure	kN or kN/m
E_a	Active earth pressure resultant	kN or kN/m
E_{ap}	Apparent elasticity modulus (in total stresses)	MN/m^2
E_h, E_v	Earth pressure components acting on a wall	kN or kN/m
E_m	Oedometer modulus	MN/m^2
E_o	At-rest earth pressure resultant	kN or kN/m
E_p	Passive earth pressure resultant	kN or kN/m
E_w	Resultant of water pressure	kN or kN/m
F	General designation for safety factors	*
F_A	Horizontal force due to active earth pressure on the piles of a particular alignment (A) located near a wall back face	kN
F_d	Safety factor against sliding along a plane surface	*
F_h	Safety factor against bearing failure	*
F_v	Safety factor against overturning	*
G	Specific weight of solid particles relative to water (also known as *specific gravity*)	*
G	Shear modulus	MN/m^2
G_o	Shear modulus for small deformations	MN/m^2
H	Horizontal load	MN
H	Height of a wall	M
H	Drainage length in consolidation problems	m
H	Height in general	m
H	Depth in general	m
H_{fail}	Horizontal load on a pile producing ground failure	kN
H_d	Design wave height	m
I	Hydraulic gradient	*
I_v	Vertical gradient of a water flow	*

(Continued)

Table 1.7.1. Basic Conventional Notations, Abbreviations and Symbols Used in this ROM 05

Symbol	Definition	Units
I. UPPER CASE LETTERS		
J	Coefficient of viscous resistance used in the dynamic study of pile driving	*
K	Winkler modulus	kN/m^3
K	Wedge factor	*
K_a	Coefficient of active earth pressure	*
K_{ac}	Active earth pressure coefficient for calculating the effect of cohesion	*
K_{ah}	Horizontal active earth pressure coefficient	*
K_{av}	Vertical active earth pressure coefficient	*
K_o	At-rest earth pressure coefficient	*
K_p	Passive earth pressure coefficient	*
K_{ph}	Horizontal passive earth pressure coefficient	*
K_{pv}	Vertical passive earth pressure coefficient	*
K_R	Spring constant in the analysis of diaphragm walls	kN/m^2
K_v, K_h	Spring constants in the study of foundation displacement	kN/m
K_θ	Spring constant in the analysis of foundation rotation	kN·m
L	Length of a pile	m
L	Thickness of a compressible layer	m
LI	Liquidity index	*
LL	Liquid limit	*
LP	Plastic limit	*
LU	Lugeon Unit	*
M	Bending moment	kN·m
M	Total load per linear metre acting on an elongated foundation less the weight of the earth excavated for its construction	kN/m
M	Mass of the hammer in dynamic compaction treatment and in pile driving studies	kg
N	SPT index: the number of blows for driving the central 30 cm	*
N	Total load acting on a foundation less the weight of the earth excavated for its construction (net load)	kN
N'	Effective compression normal to a plane	kN
N_B	Borro test index: required blowcount for advancing 20 cm in the penetration test	*
N_c, N_q, N_γ	Bearing capacity factors for estimating the ultimate bearing capacity of shallow foundations	*
N_f	Total number of flow tubes in a flownet	*
N_k	Dimensionless factor for interpreting the results of static penetrometer tests	*
N_o	Stability number (specific weight times height divided by cohesion)	*
N_p	Total number of head increments in a flownet	*
P	Perimeter	m
PI	Plasticity index	*
PL	Plastic limit	*
Q	Water flow rate	m^3/s
Q	Vertical load acting on a pile	kN
Q	Shear force	kN
Q_f	Maximum load a pile can transfer along its shaft	kN
Q_h	Ultimate bearing capacity of a pile	kN
Q_L	Linear overburden	kN/m

Table 1.7.1. Basic Conventional Notations, Abbreviations and Symbols Used in this ROM 05

Symbol	Definition	Units
I. UPPER CASE LETTERS		
Q_p	Point resistance of a pile	kN
Q_p	Point load	kN
R	Radius of influence of a well	m
R	Radius of a circular foundation	m
R_h	Resultant horizontal force	kN
R_p	Point resistance of a pile	kN
S	Shear force	kN
S, S_w	Uplift	kN
T	Pull-out resistance of a pile	kN
T	Elastic length of a pile	m
T	External horizontal force acting on a retaining structure	kN
T	Force to which an anchorage element is subjected	kN
T	Total shear force acting on a surface	kN
T	Tensile force in an element	kN
T_a, T_p	Tangential components of the earth pressure resultant on a retaining structure (active and passive pressure zones respectively)	kN or kN/m
T_θ	Fundamental period of vibration for rotation	s
T_h, T_v	Fundamental period of vibration in horizontal and vertical modes	S
T_v	Time factor	*
U	Degree of consolidation	*
V	Vertical load on a foundation	kN
V	Vertical resultant force	kN
W	Weight	kN
X, Y	Coordinates in a plan view	m
X_d	Design value of a design factor	*
X_k	Characteristic value of a design factor	*
X_m	Average value or best estimate of a design factor	*
Z	Elevation	m
Z	Impedance of a pile, used in dynamic driving studies	kN·s/m
II. LOWER CASE LETTERS		
a	Ground-structure adhesion	N/m^2
a	Seismic acceleration	m/s^2
a_h	Horizontal seismic acceleration	m/s^2
a_v	Vertical seismic acceleration	m/s^2
c	Cohesion	N/m^2
c	Velocity of compression waves in piles	m/s
c_{ap}	Cohesion in unsaturated conditions	N/m^2
c_u	Cohesion in undrained conditions	N/m^2
c_v	Coefficient of consolidation	m^2/s
d	Distance	m
d_x	Opening of the sieve allowing x% of a particular soil sample to pass through	mm
e	Void ratio	*

(Continued)

Table 1.7.1. Basic Conventional Notations, Abbreviations and Symbols Used in this ROM 05

Symbol	Definition	Units
II. LOWER CASE LETTERS		
e	Eccentricity of the load applied to a foundation	m
e	Unitary earth pressure	kN/m^2
e_a	Active earth pressure (unitary)	kN/m^2
e_p	Passive earth pressure (unitary)	kN/m^2
e_o	At-rest earth pressure (unitary)	kN/m^2
e_o	Initial void ratio of the ground	*
f_g	General correction factor	*
g	Acceleration of gravity	m/s^2
h	Difference in water level between the two sides of an impermeable element	m
h	Thickness of a layer, height of an element, etc.	m
i_c, i_q, i_γ	Inclination coefficients for estimating the bearing capacity of a shallow foundation	*
k	Coefficient of permeability	m/s
k_h	Coefficient of horizontal permeability	m/s
k_v	Coefficient of vertical permeability	m/s
m, n	Geometric factors in elastic settlement or stress distribution analyses	*
n	Porosity	*
n	Shape factor used in studying permeability problems	m or *
n_h	Constant expressing increase rate with depth in Winkler modulus	kN/m^3
p	Pressure transmitted by a shallow foundation	N/m^2
p_a	Increase in unitary active earth pressure produced by the existence of an overburden	kN/m^2
p_{ah}	Increase in unitary horizontal active earth pressure produced by the existence of an overburden	kN/m^2
$p_{v,\,adm}$	Allowable bearing capacity of the ground	kN/m^2
$p_{v,\,eq}$	Equivalent vertical pressure	kN/m^2
q	Effective pressure at the support depth of a shallow foundation	N/m^2
q	Surface overburden	N/m^2
q_c	Static penetration resistance	N/m^2
q_p	Unitary point resistance of a pile	N/m^2
q_u	Unconfined compressive strength	N/m^2
s	Settlement	m
s	Spacing between pile axes in a group of piles	m
s	Advance by a blow at the end of pile driving	mm
s_c, s_q, s_γ	Shape factors for estimating the bearing capacity of shallow foundations	*
s_f	Settlement at the end of a load test	m
s_t	Thixotropic sensitivity	*
s_u	Undrained shear strength in saturated clayey soils	N/m^2
s_∞	Theoretical settlement at infinite time	m
t	Embedment depth of sheetpiles and diaphragm walls below the excavation bottom	m
t	Time required for a particular degree of consolidation	s
t_{min}	Minimum embedment of a diaphragm wall deduced after consideration of Ultimate Limit States governed by ground strength	m
u	Porewater pressure	N/m^2

Table 1.7.1. Basic Conventional Notations, Abbreviations and Symbols Used in this ROM 05

Symbol	Definition	Units
II. LOWER CASE LETTERS		
v	Velocity	m/s
v_p	Compression wave velocity	m/s
v_s	Shear wave velocity	m/s
w	Moisture content	%
w_{sat}	Saturation moisture content	%
z	Depth below a reference level (ground surface, foundation level, etc.)	m
z_{min}	Minimum depth value	M
III. GREEK LETTERS		
σ	Dimensionless parameter used in different formulas	*
α	Angle formed by the virtual backfill face of a wall to the vertical	deg or rad
α	Angle formed by the base of a slice to the horizontal in stability calculations	deg or rad
α	Semi-angle between faces of a rock wedge	deg or rad
β	Dimensionless parameter used in different formulae (length of boreholes, etc.)	*
β	Correction factor for the allowable vertical pressure on a shallow foundation on granular soil when there is an upward water seepage	*
β	Angle formed by the surface on the backfill of a wall to the horizontal	deg or rad
β	Slope of the cant edge of a rock wedge	deg or rad
β	Reliability index	*
γ	Specific (or *unit*) weight	kN/m³
γ	Angular deformation	%
γ'	Submerged specific weight	kN/m³
γ_A	Load factor for accidental loads	*
γ_{ap}	Apparent specific weight	kN/m³
γ_d	Dry specific weight	kN/m³
γ_G	Load factor for permanent loads	*
γ_Q	Load factor for variable loads	*
γ_s	Specific weight of solid particles	kN/m³
γ_{sat}	Saturated specific weight	kN/m³
γ_w	Specific weight of water	kN/m³
Δ	Drop in the piezometric level at a point outside a pumping well	m
Δ	Increased length of a diaphragm wall to mobilise counterpressure	m
Δ_m	Maximum lowering of the piezometric level produced by a pumping well	m
δ	Inclination of the resultant force applied on a foundation	deg or rad
δ	Horizontal displacement	m
δ	Inclination of (active or passive) earth pressure to the normal of the real or virtual backfill face of a wall ("angle of friction between earth and wall")	deg or rad
ε	Angle formed by the plane bisecting a rock wedge and the vertical plane crossing the cant edge	deg or rad
ζ	Inclination of the ground failure line in studying earth pressure on walls - also used for other definitions	deg or rad
θ	Inclination of resultant total pressure (earth + water) in relation to the normal of the backfill face of a wall	deg or rad

(Continued)

Table 1.7.1. Basic Conventional Notations, Abbreviations and Symbols Used in this ROM 05

Symbol	Definition	Units
III. GREEK LETTERS		
λ	Correction factor for the allowable vertical pressure on a shallow foundation on granular soil to consider the depth of a permanent groundwater table - also used for other definitions	*
μ	Coefficient for describing the distribution of artificial drains	*
ν	Poisson's ratio	*
ν_{ap}	Apparent Poisson's ratio (in total stress)	*
ξ	Dispersion coefficient for obtaining the characteristic value of a parameter based on its average value	*
ρ	Density	kg/m^3
ρ	Polar radius of a logarithmic spiral (in analysing soil failure problems)	m
σ	Normal stress on a plane	kN/m^2
σ'	Normal effective stress on a plane	kN/m^2
σ_v	Normal vertical stress (acting on a horizontal plane)	kN/m^2
σ'_v	Vertical effective stress	kN/m^2
σ_{vo}, σ'_{vo}	Vertical (total or effective) stress prior to carrying out works	kN/m^2
σ'_{vp}	Vertical effective pressure at the level of a pile tip	kN/m^2
τ	Shear stress on a plane	kN/m^2
τ_f	Unitary resistance along the shaft of a pile	kN/m^2
ϕ	Diameter	m
ϕ	Internal angle of friction	deg or rad
ϕ	Hydraulic head or potential	m
ϕ_{ap}	Internal angle of friction in unsaturated conditions	deg or rad
ϕ_c	Angle of friction at ground-structure interface	deg or rad
ϕ_d	Equivalent diameter of a drain	m
ϕ_{equiv}	Fictitious equivalent internal angle of friction for calculations on heterogeneous ground	deg or rad
ϕ_u	Internal angle of friction in undrained conditions	deg or rad
ϕ_1, ϕ_2	Steinbrenner functions for calculating settlement	*
Ψ, Ω	Auxiliary angles used in formulas for calculating earth pressures	deg or rad

N.B.: * indicates a dimensionless value.

Abbreviation	Meaning
IV. ABBREVIATIONS	
AES	Limit State of Serviceability for aesthetic reasons
AIPCN / PIANC	Permanent International Association of Navigation Congresses (International Navigation Association)
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BSI	British Standards Institute
CBR	California Bearing Ratio
CEDEX	Centro de Estudios y Experimentación de Obras Públicas (Spanish Centre for Research and Testing in Public Works)
CIRIA	Construction Industry Research and Information Association (UK)

Table 1.7.1. Basic Conventional Notations, Abbreviations and Symbols Used in this ROM 05

Abbreviation	Meaning
IV. ABBREVIATIONS	
CPT	Cone Penetration Test
CPTU	Cone Penetration Test with pore pressure (U) measurement
DIN	Deutsche Institute für Normung (German Institute for Standardization)
DPH	Dynamic Probing Heavy
DPSH	Dynamic Probing Super-Heavy
EGD	Serviceability Limit State caused by excessive ground deformation
EHE	Instrucción de Hormigón Estructural (Spanish code for structural concrete)
EQU	Ultimate Limit State owing to loss of static equilibrium
ERI	Economic Repercussion Index
GEO	Ultimate Limit State owing to ground failure
HYD	Ultimate Limit State owing to hydraulic gradients inside the ground
ICSMFE	International Conference on Soil Mechanics and Foundation Engineering
ISOPT	International Symposium on Penetration Testing
ISRM	International Society for Rock Mechanics
ITGE	Instituto Tecnológico y Geominero de España (Technology and Geo-Mining Spanish Institute)
MSK	Scale of Seismic Intensity (Medvedev, Sponheuer & Karnik)
NAVFAC	Naval Facilities Engineering Command (USA)
NLT	Normas de Laboratorio del Transporte (Spanish Transport Laboratory Norms)
OCR	Overconsolidation Ratio
OSLS	Operational Stoppage Limit State
PSA	Property Services Agency (UK)
ROM	Recomendaciones para Obras Marítimas (Spanish Recommendations for Maritime and Harbour Works)
RQD	Rock Quality Designation
SEP	Serviceability Limit State owing to excessive seepage
SERI	Social and Environmental Repercussion Index
SGA	Serviceability Limit State caused by significant geometrical alterations
SLS	Serviceability Limit State
SPT	Standard Penetration Test
STR	Ultimate Limit State owing to structural failure
ULS	Ultimate Limit State
UNE	Una Norma Española (Spanish Technical Codes)
UPL	Ultimate Limit State owing to excess uplift

1.8 DOCUMENTARY REFERENCES

In preparing this ROM 0.5, several technical publications have been consulted, mainly the ones prepared for similar purposes (standards, documents published by specialised committees, etc.). These reference documents are detailed below and engineers may find additional clarifications in them.

From the large number of books and published articles related to this subject, the titles listed here were those deemed to be of most interest because of their wide distribution in Spain or their direct application to the geotechnical aspects of maritime works.

This ROM may recommend, for each specific topic , analytical procedures or solutions attributable to other authors. In these cases, reference is made to the source of the idea for further research.

- **Acciones en el Proyecto de Obras Marítimas y Portuarias. Recomendaciones para Obras Marítimas. ROM 0.2-90**. Dirección General de Puertos y Costas. Ministerio de Obras Públicas y Urbanismo (April, 1990).
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Part II

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION

Part II

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2.1 INTRODUCTION

Geotechnical design springs from knowledge of the ground, which is why this part of this ROM 0.5, devoted to geotechnical investigation, is the key to and basis of any project supported on the ground and/or utilising this element as construction material.

Part 2 begins by describing the geotechnical parameters, which are the numbers (or sometimes qualitative data) that current techniques mainly use to characterise the behaviour of the ground. In view of the enormous number of parameters existing (there are many behaviour models with their own geotechnical parameters), the only ones to be mentioned here will be those used most frequently in regular practice. This does not rule out the need to use specific models, parameters and procedures in particular cases that cannot be solved with the simple methods described in this ROM 0.5.

Part 2 continues describing the most common exploration techniques used both for field investigations and laboratory tests necessary for characterising the ground.

Part 2 ends with the recommendations deemed appropriate for helping to draw up the key document in relation to ground investigation, namely the Geotechnical Report.

2.2 GROUND PROPERTIES

Geotechnical investigation and particularly field and laboratory tests are designed to obtain a set of ground parameters that will later play an important role in the analyses and calculations necessary for studying different problems. The most commonly geotechnical properties and parameters and their determination by tests are described and commented on below.

2.2.1 Grading

Certain aspects of the behaviour of soils can be ascertained in the laboratory by separating their particles by size and determining their relative proportion of the total sample. In the analysis of different geotechnical problems, it is relatively frequent to use the following sizes and proportions deduced from the grading curve.

$D_{máx}$ = maximum particle size.

D_x = size of sieve allowing x% of a soil sample to pass through - D_{85}, D_{60}, D_{50}, D_{15} and D_{10}. are of particular interest.

Effective size (or diameter) = the name usually given to D_{10}.

Percentage of fines = percentage of soil passing an 0.08 UNE sieve, with an 0.08-mm mesh. When rockfill or riprap is involved, the definition *percentage of fines* can also refer to that passing a 1-inch (25-mm) sieve.

Clay content = defines the percentage of soil in grading tests by sedimentation techniques with an apparent diameter of less than 2 microns. In some studies this size is fixed at five microns. The first criterion is recommended in this ROM 0.5.

Coefficient of uniformity = is the D_{60}/D_{10}. quotient.

As a function of the particle size, D, soils are classified as:

a. gravels $D \geq 2$ mm
b. sands 2 mm $> D >$ 0.08 mm
c. silts 0.08 mm $\geq D >$ 0.002 mm
d. clays $D \leq$ 0.002 mm.

Natural soils tend to be aggregations, in varying proportions, of the different types mentioned. To define these materials qualitatively, the usual practice is to add a qualifying term (*silty*, for instance) to the name of the essential component (*clay*, for instance). These qualifying terms should be interpreted according the following rough scale:

Qualifying Term	Proportion (% by weight)	Name (example)
With traces of	5% – 10%	Clay with traces of silt
Slightly + y/ey suffix	10% – 20%	Slightly silty clay
Rather + y/ey suffix	20% – 35%	Rather silty clay
y/ey suffix	35% – 50%	Silty clay

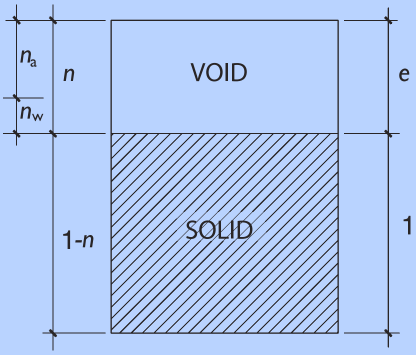
2.2.2 Void Ratio and Porosity

The conceptual model shown in Figure 2.2.1 is widely spread. It considers the solid matter of soil or rock formations as segregated from the voids in such a way that when the volume of solids is equal to 1, the sum of all the voids is worth e, which is the letter used to define the *void ratio* concept. The *pore ratio* (or *pore index*) synonyms are used less frequently.

There is an alternative method for comparing the volumes taken up by solid and hollow spaces (whether filled with liquids or otherwise) inside soil and rock formations. This is the *porosity*, usually designated by the letter *n*, and clearly correlated with the void ratio, as indicated in the same Figure 2.2.1.

In rocks, two parts of the porosity value are frequently distinguished, one formed by the voids accessible in laboratory samples and the other inaccessible, disconnected from the outer surface.

Figure 2.2.1. Most Common State Variables



e = Void ratio $= n / (1-n)$

n = Porosity $= e / (1-n) = n_a + n_w$

γ_s = Unit weigh of the solid particles

γ_w = Unit weigh of the water

G = Unit weigh of the solid particles relative to water γ_s / γ_w

Dry specific weight	γ_d	=	$\gamma_s / (1+e)$ or $\gamma_d = \gamma_s (1-n)$
Moisture content	w	=	water weight / dry soil weight $= \dfrac{n_w \cdot \gamma_w}{(1-n) \cdot \gamma_s}$
Saturation moisture content	γ_{sat}	=	e / G
Degree of saturation	S_r	=	w / w_{sat}
Wet (or apparent) specific weight	γ_{ap}	=	$\gamma_d (1+w)$
Saturated specific weight	γ_{sat}	=	$(G+e)\, \gamma_w / (1+e)$
Submerged specific weight	γ'	=	$\gamma_{sat} - \gamma_w$

*The term *density* is commonly –and incorrectly– used as a synonym of *specific weight*.

The void ratio or porosity is always determined in the laboratory and in an indirect way, since the specific weight of the solid particles is obtained on the one hand and the dry specific weight (or dry density) on the other. The relation between these two parameters is illustrated in Figure 2.2.1.

2.2.3 Moisture Content and Saturation Level

The *moisture content* –or *water content*– of a particular ground is defined as the quotient between the weight of water and the weight of dry ground. This ratio can be expressed as a percentage or as a fraction of one.

When all the voids in the ground are full of water, the corresponding moisture content is known as the *saturation moisture content*.

Below the groundwater table, the ground tends to be saturated, whereas close to or above this it tends to be partially saturated. On a scale of 1 to 100, the saturation level S_r measures the percentage of voids full of water and this coincides with the quotient between the *moisture content* and the *saturation moisture content* multiplied by 100. The saturation level is sometimes expressed as a fraction of one instead of a percentage.

2.2.4 Densities and Specific Weights

The term *density* is popularly also used to designate the concept of *specific weight* and this is the case in common practice.

In this ROM 0.5, the term *density* will be used with its real meaning of mass per unit volume, represented by the Greek letter ρ, instead of γ, which will be reserved for *specific weight*, also known as *unit weight*.

The most common specific weights for geotechnical calculations are:

γ_d = *dry specific weight*, corresponding to zero saturation level.
γ_{ap} = *wet* or *apparent specific weight*, corresponding to the level of saturation existing in the ground.
γ_{sat} = *saturated specific weight*, corresponding to a state of total saturation.
γ' = *submerged specific weight*, corresponding to the virtual specific weight that the saturated ground would have when immersed in water, after subtracting the buoyancy from the weight of the sample.

The relation between these four variables and the basic variables mentioned above is specified in Figure 2.2.1.

The concept of the *relative density* of sand is used to compare its real density to the maximum and minimum densities that could be obtained in the laboratory with the same sand. It is defined as follows:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

where:

D_r = relative density expressed as a percentage.
e = void ratio.
e_{max} = void ratio corresponding to ρ_{dmin} (UNE 103105:1993 Standard).
e_{min} = void ratio corresponding to ρ_{dmax} (UNE 103106:1993 Standard).

Comment: Sometimes, the term density index, *I_D, is used as a parameter for measuring the relative compactness of granular soils. In some texts (Geotecnia y Cimientos I, for instance), its definition coincides with what is given here for relative density ($D_r = I_D$). There are other texts (Manual of Estimating Soil Properties for Foundation Design. EPRI EL-6800. Cornell University. 1990, for instance) defining $I_D = Dr \cdot \rho_d/\rho_{dmax}$*

This ROM 0.5 does not explicitly use the term density index *for any specific application. If engineers need to use this concept for any particular reason, they are recommended to use it as a synonym for the concept* of relative density.

2.2.5 Plasticity

The property changes that clayey soils experience as their moisture content increases are usually characterised by two limit states called *Atterberg limits*:

- Plastic Limit, PL: minimum moisture content of a semi-saturated soil allowing it to be moulded without the appearance of cracks.
- Liquid Limit, LL: moisture content of a saturated soil causing it to behave like a viscous fluid.

The moisture content corresponding to these two limit states is determined by routine tests in all soil mechanics laboratories.

The range of moisture content between these two behaviour patterns is measured by the:

$$\textit{plasticity index, PI:} = LL - LP$$

A particular clayey soil may have a moisture content varying between these two limits or even beyond any one of them. The relative moisture content, *w*, in relation to these limit states is measured with the following indices:

$$\text{Liquidity index} = LI = \frac{w - PL}{PI}$$

$$\text{Consistency index} = CI = \frac{LL - w}{PI}$$

These five indicators associated with clayey soil are not usually design parameters. They are however used to correlate properties and identify different soil types.

2.2.6 Permeability

Permeability denotes the ground property permitting water or another liquid to flow through it. Generally speaking, it is complicated to measure this property, but in some specific cases it can be done simply.

This ROM 0.5 considers it acceptable to admit Darcy's Law for studying only the laminar flow problems of water in saturated soils. The water flow rate through a unit area section is called the discharge velocity or simply velocity, *v*. This velocity (always less than the actual average water velocity in the ground) is taken to be proportional to the hydraulic gradient *I* as defined in the paragraph below. The opposite of this coefficient of proportionality is known as the *coefficient of permeability* and is usually represented by the letter *k*.

$$v = -k \cdot I$$

Gradient *I* is the spatial derivative of the scalar *potential* (or *head*) ϕ, defined thus in geotechnical problems:

$$\phi = \frac{u}{\gamma_w} + z$$

where:

γ_w = specific weight of the water.
u = pore water pressure.
z = elevation of the water over a reference horizontal plane.

In soils with anisotropic permeability, the relationship between the velocity and gradient vectors is a square matrix (two-dimensional problems) or a cubic matrix (three- dimensional problems) rather than a number.

Permeability can be determined using tests in boreholes (Lefranc, for instance), pumping tests and laboratory tests (using different types of permeameter and oedometer tests).

Typical permeability rates for different types of ground are given in Table 2.4.3.

2.2.7 Drained and Undrained Behaviour

Whenever the state of stress of a totally or partially saturated ground is modified, its particles undergo some displacement and the water can temporarily vary its pore pressure and subsequently reach a balance in a hydrostatic state (with uniform potential) or a steady-state seepage. This change may occur rapidly or slowly in comparison to the study timeframe and this is a particularly important aspect in analysing different geotechnical problems.

Two limit situations are frequently considered in connection with this subject:

- undrained: so-called *undrained conditions* occur when the water in the ground cannot move and adapt to the new state of loading. Consequently pore pressures exist that attempt to displace the water to positions of equilibrium;

- drained: so-called *drained conditions* occur when the hydraulic potential of pore water reaches a permanent distribution instantaneously and in each moment of time.

The possibility that any of the situations may occur and the study of the consequences of each of them is a must in the analysis of any geotechnical problem.

When the load conditions vary in permeable soils, the water flows rapidly and, as a result, there is no consolidation period. Pore pressures are generated and dissipated in very short periods of time. Except in the special case of cyclic or impulsive loads (earthquakes, wave action, etc.), ground with a coefficient of permeability greater than 10^{-2} cm/s is considered infinitely permeable to all other effects. It is therefore not necessary to take into consideration the undrained situation in such circumstances.

In clayey soils with a permeability of less than 10^{-4} cm/s, it is always advisable to consider that an *undrained* situation could occur, i.e., zero water movement in respect of the soil.

In ground with an intermediate permeability between 10^{-2} and 10^{-4} cm/s, engineers must decide on the need to analyse the extreme *undrained* situation, taking into account the geometry of the ground and the type and timeframe of the loads imposed. If in doubt, it may be advisable to make a prior estimate of the extent of the consolidation period involved, as indicated in 3.4.8.

Recommendations for considering the drained or undrained behaviour of soil affected by the action of waves or other cyclic or impulsive loads are given in Subsections 3.4.11 and 3.10 of this ROM 0.5.

2.2.8 Soil Strength

The shear strength of a soil depends, among other factors, on the load characteristics. The following sections deal with the most common case of quasi-static loads. The strength of soil under cyclic and impulsive loads is dealt with in Section 3.10.

2.2.8.1 Strength Parameters in Effective Pressures

For the vast majority of geotechnical calculations on saturated soils, the common practice is to assume the validity of Coulomb's law, whereby

$$\tau = \sigma' \tan \phi + c$$

where:

τ = shear stress producing failure.
σ' = effective pressure normal to the plane.
ϕ = internal angle of friction.
c = cohesion.

Furthermore, the frequent practice recommended in this ROM 0.5 is to admit Terzaghi's effective pressure principle for calculating the effective pressure in saturated soils:

$$\sigma' = \sigma - u$$

where:

σ = total pressure.
u = porewater pressure.

The friction angle ϕ and cohesion *c* are obtained from slow tests permitting total drainage (CD triaxial tests with prior consolidation and drained failure, for instance) or tests monitoring porewater pressures, which are subtracted during the test interpretation, (CU triaxial, with prior consolidation and undrained failure but measuring pore pressures, for instance).

In the majority of soils and for practical purposes, the angle of friction can be assumed to be constant. In actual fact, this parameter depends on the intensity of the load.

This effect is particularly significant in cemented sands or sands formed by organic carbonates (conchiferous, foraminiferous, coralline sands, etc.) and also in rockfills. The internal angle of friction decreases when the average effective pressure *p'* increases. This unfavourable effect, which engineers need to analyse in each case, is usually represented by the expression:

$$\phi = \phi_0 - b \cdot \log_{10} \frac{p'}{p'_0}$$

where:

ϕ = angle of friction corresponding to the average effective pressure *p'*.
ϕ_o = angle of friction corresponding to the reference average effective pressure p'_0.
b = decrease in the angle of friction produced when the average effective pressure is multiplied by ten.

It is worth noting that this effect can be significant, as parameter *b* measured in some carbonated soils has proved to be greater than 15°.

This decrease is generally attributed to a fracture of the soil particles, occurring in carbonated sands at low pressures but in siliceous sands at pressures higher than 20 MPa.

2.2.8.2 Unsaturated Soils

For unsaturated soils, this ROM 0.5 recommends the use of a similar expression:

$$\tau = \sigma \cdot \tan \phi_{ap} + c_{ap}$$

where:

ϕ_{ap}, c_{ap} = apparent friction and cohesion in unsaturated conditions.
σ = total pressure on the failure plane.

In practice, the angle of friction ϕ_{ap} and cohesion c_{ap} are used less with unsaturated soils and would in fact be obtained from field and laboratory tests run on samples or ground with the same degree of partial saturation object of interest.

2.2.8.3 Undrained Shear Strength

For a better knowledge of the behaviour of clayey or silty soils, and all low-permeability soils in general, it is usually necessary to investigate the shear strength in situations where drainage is prevented. This attempts to simulate the *undrained* conditions described in Subsection 2.2.7.

In these conditions, soil strength can be approximated using an expression similar to Coulomb's law:

$$s_u = c_u + \sigma \cdot tg\ \phi_u$$

where:

ϕ_u = apparent angle of friction in the undrained failure.
c_u = apparent cohesion for undrained failure.
s_u = undrained shear strength.
σ = total stress on the failure plane.

In analysing short termed situations, compared to the soil consolidation time, and without the possibility of partial water drainage, it is frequently assumed that ϕ_u = 0. Such is the result of triaxial tests run on saturated clays - the shear strength is independent of the cell pressure applied.

Undrained shear strength can be measured in the field with *in situ* tests (vane and static penetration tests, for instance) or in the laboratory (UU triaxial tests on undisturbed samples, for instance).

It is sometimes interesting to know what this strength would have been if the soil were more consolidated (i.e., with greater densities) as could occur deeper in the ground. The density can also increase, thereby increasing the undrained shear strength, after a certain time in areas under the additional load of the works being studied.

To investigate these cases, the samples can be artificially consolidated in the laboratory up to the state which is going to be simulated and the undrained test subsequently performed until failure.

On many occasions, the undrained shear strength of the cohesive soils forming marine or coastal beds constitutes a critical design parameter. It must be determined with the greatest precaution, since the results obtained can have crucial effects on the type of works to be carried out.

The undrained shear strength of normally consolidated clayey or muddy soils increases with depth as the vertical effective pressure grows. Engineers need to seek this correlation in their geotechnical explorations. The law given by the following expression can be used as a reference:

$$s_u = \left(0.2 + 0.3 \cdot \frac{PI - 30}{100}\right) \cdot \sigma'_v \geq 0{,}15\sigma'_v$$

where:

s_u = undrained shear strength of a normally consolidated cohesive soil.
PI = plasticity index expressed in %.
σ'_v = vertical effective pressure acting on the horizontal plane where the shear strength is determined.

It is possible to measure even lower shear strength values than the one indicated by the above equation. This may indicate that the ground is still undergoing a consolidation process.

In coastal soils or seabeds, overconsolidation may occur. Such situation is measured by the OCR parameter (see 2.2.10.2).

Overconsolidation may result from the subsequent removal of a previous load. It may also be the result of certain ageing or cementation processes or even of densification caused by the shear loads acting on seabeds due to water movement. The effect of these processes can be simulated in calculations by assuming an equivalent preconsolidation load.

Overconsolidation increases the undrained shear strength of clayey or muddy soils. A theoretical relation exists indicating that this increase is:

$$s_u^* = s_u \cdot OCR^{0.8}$$

where:

s_u^* = undrained shear strength of an overconsolidated clayey soil.
OCR = overconsolidation ratio (see 2.2.10.2).
s_u = shear strength of the same soil if it were normally consolidated.

Engineers should investigate whether this law is valid, if soils of this nature are involved in their projects. The expression indicated can serve as a reference for them.

For granular soils, Coulomb's law in effective pressures can represent their strength in undrained conditions, which can sometimes occur with rapid load application, by entering the value of the pore pressures thus generated into the calculations.

2.2.9 Rock Strength

Rocks appear in nature with certain degrees of fracture and weathering. Normally, the strength it is advisable to know is the one representing the volume of rock affected by the works. This strength depends on several parameters in addition to those already mentioned (density and porosity). The main characteristics determining the behaviour of rock formations are:

- the nature of the rock.
- the degree of jointing and the characteristics of the joints.
- the degree of weathering.

The *nature of a rock* refers not only to its mineralogical composition but also to its geological origin, as the latter can permit the use of detailed data available from rocks of a similar provenance.

To characterise fractures in rocks, it is advisable to define each of the joint sets affecting them. This characterisation can be done with the help of the parameters given in Table 2.2.1.

Table 2.2.1. ***Joint or Discontinuities Characterisation. (Adapted from the ISRM* Recommendations)***

Aspect	Significant Parameter	Remarks
Number of sets	Number	Each set should have a similar genesis and some like characteristics.
Orientation	Orientation (strike). Dip	Grouping by similarly orientated sets.
Spacing	Average distance between joints, s.	One value per set. See Subsection 2.2.9.1.
Fracture index	Number of joints per unit volume, J_v.	A global value representative of the area explored. See Subsection 2.2.9.2.
Continuity	Extension of a joint set within a rock mass, P.	Measured in unit of length. Also denominated *persistence*. See Subsection 2.2.9.3.
Aperture	Distance between the two blocks separated by a joint, *a*.	See Subsection 2.2.9.4.
Roughness	Qualitative measurements of the deviations of a joint face in respect of a plane.	See Subsection 2.2.9.5.
Fill type	Description of a material filling a joint, if any.	Soil is described according to the soil indications given in Subsection 2.2.
Rock strength	Unconfined comprenssive strength in the fresh faces on the two sides of the crack.	See Subsection 2.2.9.6.
Hidraulic state	Description of the possible presence of water in the joint.	Usually described by one of the following terms: *dry, damp, dripping, with flowing water,* etc.

(*) ISRM (1981). "Rock Characterization, Testing and Monitoring: ISRM Suggested Methods". Ed. Brown E.T. Pergamon Press

2.2.9.1 Average Distance between Joints

The bearing capacity of rock formations for both shallow and deep foundations depends, among other factors, on the average joint separation.

Each joint set usually has a similar orientation and a similar spacing pattern, represented by the average value for the distance between two consecutive discontinuities.

It is relatively frequent to define the spacing in qualitative terms. These definitions must be compatible with the average spacing indicated in the following list:

Qualifying Term	Spacing
Extremely close	< 2 cm
Very close	2 a 6 cm
Close	6 a 20 cm
Moderate	20 a 60 cm
Wide	60 a 200 cm
Very wide	200 a 600 cm
Extremely wide	> 600 cm

2.2.9.2 Number of Joints per Unit Volume

In normal circumstances, rock masses are affected by several sets of discontinuities, meaning that a particular volume of rock mass is affected by several joints. The average value of the number of joints crossing a one-cubic-metre volume of rock is known as the index J_v.

The equivalence between the qualitative values used in geological descriptions and the physical reality should abide by the following list:

Qualifying Term	Fracture Index, J_v (Number of Joints per m^3)
Massive	< 1
Lightly jointed	1 – 3
Averagely jointed	3 – 10
Fairly highly jointed	10 – 30
Highly jointed	30 – 60
Crushed	> 60

2.2.9.3. Discontinuity Persistence

A particular joint will always be limited in extent. *Faults* several kilometres long may divide the rock, whereas some joints will split the rock mass just a few metres along.

The greatest length or maximum extension in any direction of a given joint is known as its *persistence*. As a rule, joints belonging to the same set show similar persistence.

In qualitative terms and as a function of persistence, the adjectives usually used to describe this parameter are the ones listed below:

Qualifying Term	Persistence
Very small	< 1 m
Slight	1 – 3 m
Average	3 – 10 m
High	10 – 20
Very high	> 20 m

2.2.9.4 Joint Apertures

The distance between the opposite faces of a particular discontinuity is known as its *aperture*.

The usual values for apertures and the qualifying terms it is advisable to use to describe them are indicated in the following list:

Qualifying General	Qualifying Detailed	Aperture
Closed joints	Very tight Tight Partly open	< 0 – 1 mm 0,10 – 0,25 mm 0,25 – 0,50 mm
Fractured rock mass	Open Moderately open Wide	0,50 – 2,50 mm 2,50 – 10 mm > 1 cm
Open joints	Very wide Extremely wide Cavernous structure	1 – 10 cm 10 – 100 cm > 1 m

2.2.9.5 Roughness

The joints affecting rocks tend to be approximately plane surfaces. Consequently, they are described using the geometrical parameters of *orientation* (or *strike*) and *dip*. However, the physical reality may be more complex.

Faults and joints of high or very high persistence should be described by zoning their full length into stretches with similar roughness.

Joints with lower persistence (average, slight or very small) and comparable areas of more persistent joints should be characterised at least qualitatively using one of the following three groups:

a. *stepped*, when in conjunction with other joints the discontinuity face shows steps;

b. *undulating*, when the shape of the joint face looks like a wave;

c. *planar*, when the direction and dip of the rock separation surface varies only slightly.

d. In all cases, in addition, and in a smaller scale (lengths in the order of a centimetre), joints should be classified as *rough*, *smooth* or *slickensided*, depending on their degree of roughness.

2.2.9.6 Unconfined Compressive Strength

The strength of laboratory-cut samples subjected to uniaxial compression is one of the parameters of greatest interest for characterising rocks.

Sometimes this parameter is far from being determinant and, as a result, it can suffice to characterise it using indirect procedures. The relation that should exist between the approximate procedures for determining unconfined compressive strength, the qualifying terms used and the actual strength are indicated below:

Indirect Procedure	Qualifying Term	Estimated Value (MPa)
Scratched with finger-nail.	Especially weak	< 1
Broken with moderate hammer blows. Scratched by penknife.	Very low	1 – 5
Difficult to scratch by penknife.	Low	5 – 25
Not scratched by penknife. Broken with one hammer blow.	Average	25 – 50
Broken with several hammer blows.	High	50 – 100
Difficult to break with a geological hammer.	Very high	100 – 250
Geological hammer blows only produce a few splinters.	Extremely high	> 250

2.2.9.7 Weathering Degree

Rock normally weathers as a natural consequence of ongoing physical and chemical processes. Depending on how advanced this process is, the technical community has agreed to classify rock weathering according to the nomenclature given in Table 2.2.2.

Table 2.2.2. Scale of Rock Mass Weathering (ISRM)

Degree	Name	Identification Criterion
I	Unweathered (fresh) rock	Rock showing no visible signs of weathering, slight discoloration may exist or small oxide stains in discontinuity planes.

N.B.: Table 4.9.2 indicates some additional assessments of weathering degrees (Section 4.9, Dredging and Fills)

Table 2.2.2. **Scale of Rock Mass Weathering (ISRM) (Continuación)**

Degree	Name	Identification Criterion
II	Slightly weathered rock	Rock and discontinuity planes show signs of discolouration. The entire rock may have lost its original colour owing to weathering and be weaker on the surface than the fresh rock.
III	Moderately weathered rock	Less than half the material has decomposed into soil. Fresh or slightly weathered rock appears either continuously or in isolated zones.
IV	Highly weathered rock	Over half the material has decomposed into soil. Fresh or slightly weathered rock appears discontinuously.
V	Completely weathered rock	The entire material has decomposed into soil. The original rock structure remains intact.
VI	Suelo residual	The rock has totally decomposed into soil without the original structure and texture being recognisable. The material remains *in situ* with a significant change in volume.

N.B.: Table 4.9.2 indicates some additional assessments of weathering degrees (Section 4.9, Dredging and Fills)

2.2.10 Ground Deformability

2.2.10.1 The Elastic Model

On a large number of occasions, ground deformability can be characterised by some equivalent elastic constants

E = modulus of elasticity (Young's modulus).
ν = Poisson's ratio.

These constants apply when the stresses used for calculating are effective stresses.

When it is not possible to know the effective pressures, some apparent elastic parameters can be defined and used in total-pressure calculations:

E_{ap} = apparent elasticity modulus for calculating deformation using total stress.
ν_{ap} = apparent Poisson's ratio for these calculations.

It is relatively frequent practice to study *undrained* problems in saturated soils where the water is assumed to be incompressible and unable to move and consequently the soil volume cannot change. The elastic solids that do not change volume have a Poisson's ratio equal to 0.5 and, for this reason, it is frequently assumed in this type of calculation that:

ν_{ap} = 0.5 *undrained* strain calculations in saturated soils using total pressures.

According to the theory of elasticity, the shear modulus *G*, resulting from dividing the shear stresses by the angular deformations they produce, is:

$$G = \frac{E}{2 \cdot (1+\nu)}$$

In theory, this modulus is independent of whether it is established in terms of total pressure or in terms of effective pressure. For this reason and in the absence of better information, for *undrained* calculations it can be assumed that:

$$E_{ap} = \frac{1.5}{(1+\nu)} \cdot E$$

where E and ν are the elastic parameters of the soil skeleton.

Rocks are treated in a similar way, even though the assumption about the non-compressibility of water is more dubious and its application not always to be recommended.

2.2.10.2 The Oedometric Model

The compressibility of clayey soils subjected to lower loads than the ground had previously suffered is clearly different from that corresponding to greater loads. This is why deformability is usually characterised in two different ways depending on whether the load is smaller or greater than a critical value known as the *preconsolidation pressure*, which is the maximum effective pressure the soil has undergone in the past.

Clayey soils subjected, in the ground, to natural pressures equalling the preconsolidation pressure are termed *normally consolidated*. Otherwise, they are called *overconsolidated*. The OCR or *overconsolidation ratio* is defined as the quotient between the preconsolidation pressure p_c and the current effective pressure p.

In soft cohesive soils, the deformability in loading processes is usually characterised using a expression like:

$$\Delta e = C_s \cdot \log_{10} \frac{p_2}{p_1} \qquad (\text{If } \; p_2 < p_c)$$

$$\Delta e = C_s \cdot \log_{10} \frac{p_c}{p_1} + C_c \cdot \log_{10} \frac{p_2}{p_c} \qquad (\text{If } \; p_2 > p_c)$$

where:

Δe = reduction in the void ratio when pressure increases from p_1 to p_2.
C_c = compression index (dimensionless)
C_s = swelling index (dimensionless)
p_c = preconsolidation pressure.

In unloading processes, the increase in void ratio when pressure drops from p_2. to p_3. would be:

The three parameters defining deformability (C_c, C_s and p_c) are obtained from oedometer tests and apply to problems in which the loading process is similar to that of the test, i.e., when the ground deformation is prevented in any direction perpendicular to the load.

The following value is defined as the oedometric modulus E_m:

$$E_m = \frac{\Delta p}{\Delta e} \cdot (1 + e_o)$$

where:

Δ_p = variation in the absolute value of the effective pressure.
Δ_e = variation in the absolute value of the void ratio.
e_o = original void ratio of the ground.

For very small load variations around the effective pressure *p*, a relation exists between the oedometric modulus and the magnitudes indicated above:

$$E_m = 2.3 \cdot p \cdot \frac{1+e_o}{C_c} \qquad \text{(Virgin compression processes)}$$

$$E_m = 2.3 \cdot p \cdot \frac{1+e_o}{C_s} \qquad \text{(Recompression or unloading processes)}$$

The elastic constants *E* and ν, which produce the same deformation as the oedometric model, are related with the oedometric modulus:

$$E = E_m \frac{(1+\nu)\cdot(1-2\nu)}{(1-\nu)}$$

Marine and coastal sediments usually contain highly compressible soft silty or clayey soils that can give rise to large settlements.

The compression index of these soils generally increases with their plasticity. A typical expression engineers can use as a reference is:

$$C_c = 0.10 + \frac{LL - 20}{100} \geq 0.10$$

where:

C_c = compression index of a normally consolidated clayey soil.
LL = liquid limit expressed as a percentage.

The swelling index C_s is usually several times lower than C_c (in the order of 5 to 10 times less) except in soils with a metastable structure.

For each specific case, engineers should study the values of C_c and their possible correlation to other identifying parameters (the liquid limit has been indicated as an example) in order to be able to extend their application to other similar soils in the vicinity.

Reference should be made to the cases incorrectly designated in theory as underconsolidation (when the existing effective pressure is lower than the theoretical effective pressure in the plane under study assuming normal consolidation). This can occur in sediments that are still pending part of their natural consolidation process caused by their self-weight, or in areas with an artesian pore water regime or in those affected by the generation of natural gases due to certain processes of organic matter decomposition. This type of situation can also happen as the result of partial liquefaction caused by the cyclic stresses induced by water movement.

In these cases, porewater pressures are higher than hydrostatic pressures, effective pressures can be very small and consequently compressibility is much greater.

2.2.10.3 The Winkler Model

On other occasions and for more particular problems, ground deformation is represented by the so called *Winkler modulus* or *subgrade reaction modulus*, usually represented by the letter *K*, which is the quotient between the pressure applied, *p*, and the displacement produced, *s*.

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

Given that, for the same ground, this ratio depends on the extension, intensity and orientation of the load (among other factors), it should not be interpreted as a soil property, but rather as a design parameter applicable solely to deformation problems similar to those of the test in which it was determined.

This parameter is typically used in the study of foundation slabs and of horizontal deflection of piles and is normally obtained from plate bearing tests and from horizontal pull tests on piles, among others.

2.2.10.4 Other Ways of Considering Ground Deformation

In geotechnical engineering, there are other ways of taking soil deformation into consideration. To complement to those already covered, a further two will be mentioned here. One is a simple method usually employed in studying wall displacements and the other, more common one, is used in certain computer programmes.

In the analysis of earth pressure on retaining walls, it is a relatively frequent practice to specify the limit deformations giving rise to the extreme active or passive states. These deformation parameters are usually obtained from previous experience or from tests on models.

There exist non-linear elasticity models (hyperbolic model, for example) or elasto-plastic models or other complex models of coupled flow-deformation that are frequently used in computations with finite elements, but they are deemed to be beyond the scope of this ROM 0.5.

2.2.11 Other Characteristics

Many other soil characteristics and parameters exist that are used in geotechnics -yet more seldom in practice- but not for this reason are any less significant. The following list covers some of them.

- *Peak* and *residual strength*. In dense sandy soils, overconsolidated clays, carbonated silts and in many other soil types, it can happen that the shear strength reaches a maximum level for a particular deformation (*peak strength*) and subsequently tends to drop to a lower value (*residual strength*). In the study of certain geotechnical problems, it will be necessary to know both strength values.

- *Thixotropy* and *thixotropic susceptibility*. Some clayey soils exhibit much higher shear strength in their natural state than after being remoulded, even though this remoulding process is done preserving the same density and water content. The quotient of both strengths is known as the *thixotropic sensitivity factor*, or simply referred to as the *susceptibility* of the clay and denominated s_t.

- *Degree of consolidation*. This uses a scale of 1 to 100 to express the progress made in the consolidation process, either referred to the dissipation of the excess porewater pressure of a specific level (degree of consolidation U_z) or else in terms of the progress in surface settlement (degree of consolidation U).

- *Coefficient of consolidation*. It is frequent practice to use a combined parameter of permeability, k, and deformation, E_m (oedometric modulus), known as the coefficient of consolidation:

$$c_v = \frac{k \cdot E_m}{\gamma_w}$$

where γ_w is the specific weight of the water.

This parameter is usually expressed in cm^2/s and is of special interest in consolidation time calculations.

- *Porewater pressure coefficients*. In triaxial tests, variations occur in pore water pressure, Δu, when drainage is not permitted and the cell pressure σ_3 and vertical load σ_1 are modified. It is common practice to use Skempton's dimensionless coefficients *A* and *B* to estimate porewater pressure variation via the expression:

$$\Delta u = B\left(\Delta\sigma_3 + A\left(\Delta\sigma_1 - \Delta\sigma_3\right)\right)$$

- *Expansiveness* and *collapse*. Some soils undergo substantial alteration in their structure when they are saturated and as a result can swell or collapse (both phenomena have been observed in some soils; one or the other predominates depending on the applied load). The magnitude of the swelling or collapse depends on several factors, including effective pressure. In this respect, the term *swelling pressure* is frequently used to denote the effective pressure that prevents expansion during the saturation process and *free swelling* for the percentage change in volume occurring when the soil is saturated with low effective pressures (in the order of 1 kN/m_2). The collapse is usually measured with the parameter γ_c, or vertical unitary deformation caused by the saturation. This parameter depends, among other factors, on the pressures applied at the time of the saturation.

2.3 OBJECTIVE AND STAGES OF GROUND INVESTIGATION

Any study or design for maritime or harbour works must be preceded by a specific Geotechnical Investigation, adapted to the circumstances prevailing in each specific case.

All studies, site explorations, laboratory tests, data analysis, etc. should be collated into a single document, which will constitute the Geotechnical Report that must serve as a basis for subsequent work.

The scope of the Geotechnical Report will depend on the objective of the work for which it is to be used and to this end the following classification can be drawn up:

a. *Choice of Site*

A comparative assessment of the different sites considered viable or a selection of the most suitable areas within a single site.

b. *Feasibility*

Obtaining the information required to verify whether the conditions at a site are suitable for the work planned on it.

c. *Design*

Ascertaining the soil conditions (stratigraphy, geotechnical parameters, piezometric levels, etc.) which will enable the works to be accurately detailed, including works of a temporary nature.

d. *Construction*

The complementary study of specific aspects to enable the most suitable construction procedures to be adopted, to anticipate problems that might appear during construction as a result of the ground geotechnical characteristics, to identify possible alternative material borrow-pits, to select plant and stockpiling areas, etc.

e. *Impact of the Works*

Study of the changes that can take place in the ground either naturally or as a result of the works themselves, which may affect either these new works or other existing or future works and, in a general way, the area surrounding the zone under study.

f. *Other Objectives*

These include studies into the safety conditions of existing structures, incident investigation, etc.

The Geotechnical Report must expressly include a statement of its purpose setting out the type of works or structures for which the information is to be used. Any further use of the information for a different purpose must be expressly justified.

The extent and thoroughness required in a geotechnical investigation for maritime and harbour works will depend on the nature of the work to be carried out on the site and the geotechnical characteristics of the ground. In the light of these factors, the technician responsible for the geotechnical investigation needs to draw up a detailed programme for it, making allowance for the adaptations required in view of the results obtained as the actual exploration proceeds.

The exploration should take place in stages which can generally be summarised as described below.

a. ***Preliminary Study***

This includes an initial compilation and analysis of existing information. This stage will generally consist of deskwork although it should include site inspection visits –with data collection–, simple investigation work (e.g. trial pits, surface material mapping, etc.) and the initial boreholes required to ascertain the basic ground structure if this is not already known. This stage ends with preparation of the Preliminary Geotechnical Report.

b. ***Investigation***

This stage includes all field exploration work subsequent to the preliminary study as well as the corresponding laboratory tests.

c. ***Geotechnical Report***

The Report must include a summary of existing data, a description and analysis of the investigations carried out, the identification of existing subsoil materials and a determination of the geotechnical parameters required for the work for which the study is undertaken.

In certain cases, the existence of peculiar geotechnical problems may require that a specific study or investigation is carried out, which should be adapted to the case in question.

Comment: The stages set forth in this ROM 0.5 for carrying out a geotechnical investigation constitute a basic methodology which will ensure that the information required for carrying out the subsequent work (feasibility studies, construction design, etc.) is obtained. The intensity and duration of these stages will be adapted to the particular circumstances of each case.

Provided that the overall timescale permits, the three stages referred to should take place consecutively. An appropriate Preliminary Study with comprehensive compilation and interpretation of existing documentation will enable investigations to be properly programmed, with a consequent cost saving and optimisation of information that can thereby be achieved.

Anyway, the different stages may overlap (the more general investigations may begin before completing the Preliminary Study or drafts of the Geotechnical Report may be issued before fully completing exploration work), in order to adapt the geotechnical investigation to the time schedule and overall requirements of the works for which it is needed.

The contents of any initial draft of the Geotechnical Report must be included in the final version of the Report, so that this is comprehensive and serves as a single reference when carrying out the work for which it is intended.

2.4 PRELIMINARY STUDY

The Preliminary Study must cover the following three basic aspects:

I) Compilation and analysis of existing information.

II) Preliminary determination of the stratigraphy and geotechnical characteristics of the materials involved, along with any other conditioning factors, so that preliminary aspects of the work for which the investigation is made can be started: solutions examined, alternatives compared, pre-dimensioning carried out, etc.

III) Selecting and programming the investigation work (fieldwork and laboratory tests) considered necessary to complete the Geotechnical Investigation.

2.4.1 Existing Information

Potential existing sources of information can be grouped as follows:

a. *Published Information*

- Codes and Standards applicable.

- General geological and geotechnical documentation published by the *Instituto Tecnológico y Geominero de España* (ITGE) [*Technology and Geo-Mining Spanish Institute*]. It is particularly recommended to consult the following publications:
  - 1:200,000 scale Geological Map
  - 1:50,000 scale Geological Map
  - 1:200,000 scale Geotechnical Map
  - 1:200,000 scale industrial rock map.

- The geotechnical atlases existing in some ports.

- Aerial photographs, where applicable.

- Marine charts.

- Old maps (if they can be found).

- Published studies and/or articles relating to project designs and works in an area near the site of the Geotechnical Investigation.

b. *Unpublished Information*

- Review of local experience relating to geotechnical aspects.

- Geotechnical studies and investigations for projects and works in the vicinity.

- Information on past and present land use (in order to locate possible foundations or buried works, areas of fill or dredging, etc.).

- Supply sources of construction material.

- Miscellaneous information that could have geotechnical implications: nearby existing structures, industrial plants and properties that could be affected, previous excavation works, old constructions, etc.

c. *Site Inspection and Initial Investigation*

All the above information must be supplemented with:

- Site inspection visits.
- Verification of the geological and geotechnical maps and their adaptation to a scale that will provide sufficient detail for subsequent work.
- Performing simple exploration work (trial pits, *vibrocorer* sampling, penetrometer tests, geophysical exploration, identification tests, etc.) and the boreholes required to ascertain the basic subsoil structure, if unknown.
- General comparison with the other documentation obtained.

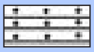
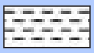
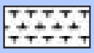
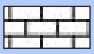
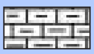
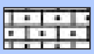
2.4.2 Preliminary Geotechnical Report

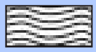
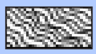
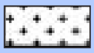
The Preliminary Geotechnical Report should be drawn up based on this information. It will describe the geological and geotechnical conditions in the area examined in sufficient detail to enable initial work and calculations to proceed, until full information is available. Where possible, this description should include the following steps:

- The geological background and geomorphological evolution of the site.
- Geotechnical profiles and/or typical lithographic columns.
- Identification of materials. In order to provide a uniform criterion, a system of graphic symbols to designate materials is given in Table 2.4.1.

  This identification should be complemented by any comments adequate for a better understanding of the ground structure - geological age, the origin of materials (alluvial, piedmont, colluvial, Aeolian formation, etc.), colouring, etc.

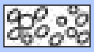
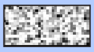
Table 2.4.1. Common Terms and Symbols Used for Preliminary Ground Description

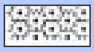
Rock Type	Symbol
SEDIMENTARY ROCKS	
Conglomerate	
Sandstone	
Siltstone	
Argillite	
Marl	
Limestone	
Marly limestone	
Calcarenite	

Rock Type	Symbol
SEDIMENTARY ROCKS	
Dolomite	
Gypsum	
METAMORPHIC ROCKS	
Slate	
Schist	
Gneiss	
PLUTONIC ROCKS	
	
VOLCANIC ROCKS	
	

(Continued)

Table 2.4.1. ***Common Terms and Symbols Used for Preliminary Ground Description***

Soil Type	Symbol
GRANULAR SOILS	
Gravel	
Sand	
COHESIVE SOILS	
Silt	
Clay	

Soil Type	Symbol
ORGANIC SOILS AND FILLS	
Topsoil	
Mud and peat	
ARTIFICIAL FILLS	
Random fills	
Pit-run berms	
Rockfill	

- Assignment of basic characteristics that can be used in initial estimations (density, shear strength, deformation moduli and permeability). For reference purposes and when no specific data for the area are available, the approximate average values indicated in Subsection 2.4.3 should be used.

- The position of the piezometric level.

- Any other particular aspects that could condition the design from the geotechnical point of view.

Based on the initial information, the Preliminary Geotechnical Report must conclude by identifying the important but unknown aspects needing to be the subject of further investigations. To this end, it is necessary to have used a description as accurate as possible of the works to be carried out. Experience has shown that the more detailed the design is at the time of planning investigation programmes, the more effective the latter will be and the lower the need for subsequent supplementary geotechnical exploration work.

The geotechnical investigation should be programmed in the light of the geotechnical aspects requiring exploration, confirmation or greater detail. Special attention must be paid to this programming; otherwise, comprehensive information may be obtained as to some aspects while big gaps may still exist in others.

It may be that no further geotechnical investigation is required beyond the extent carried out for the Preliminary Study. This can occur in the case of minor works on sites with good ground for which the existing geotechnical information is abundant and properly verified. In this case, and provided the technician in charge duly states and justifies the same, the Preliminary Geotechnical Report will be complete and then become the definitive Geotechnical Report.

Comment: *In most situations –projects of high cost, or involving a risk of damage to persons or property, or to be constructed on poor quality or heterogeneous ground or where there is little previous information– the Preliminary Geotechnical Report will conclude by identifying problems requiring further analysis and by stating the need to obtain further geotechnical data. In these cases, the Preliminary Geotechnical Report should end by identifying the additional information requirements.*

As a result of the prior knowledge of the works to be carried out and the nature of the ground involved, all the potential geotechnical problems requiring analysis must be clearly identified -stating, for each one of them, which geometrical variables and geotechnical parameters are of interest for analysing them correctly. Furthermore, the degree of precision of the data required for subsequent analysis should be assessed for each of the anticipated problems. This necessary degree of precision will determine the scope of the geotechnical investigation needed.

The Preliminary Geotechnical Report is extremely important and maximum effort must be put into it to avoid subsequent geotechnical investigation being carried out blindly with only a vague idea of what may be found. Geotechnical exploration should never be carried out without an approximate idea of its results.

2.4.3 Preliminary Geotechnical Parameters

Any fact related to the ground that could prove to be critical to a project design must be specifically determined. On the other hand, some geotechnical parameters of secondary importance in the design can be defined approximately as a function of duly verified experience.

The values for the geotechnical parameters included in this subsection are purely for illustration purposes. The real value may even fall outside the range of values indicated.

Table 2.4.2 gives approximate average values for specific weight, unconfined compressive strength and elastic model parameters of different fresh rocks.

Similarly, Table 2.4.3 gives typical average values for different soil formations and fills.

Table 2.4.2. Some Basic Characteristics of Fresh Rocks* for Use in Initial Estimations

	Rocks	Specific weight (kN/m^3)	Unconfined Comprensive Strength of Fresh Fragments (MPa)	Deformation Modulus (MPa)	
Hard	IGNEOUS METAMORPHIC Gneiss, quartzites SEDIMENTARY Well-cemented sandstones, some more compact limestones and dolomites	26	100	MASSIVE	50.000
				JOINTED	20.000
				HIGHLY JOINTED	10.000
Medium	METAMORPHIC Schists and slates SEDIMENTARY Excepting poorly cemented sandstones, marls and conglomerates	24	50	MASSIVE	20.000
				JOINTED	10.000
				HIGHLY JOINTED	5.000
Soft	SEDIMENTARY Excepting poorly cemented sandstones, marls and conglomerates	22	20	MASSIVE	5.000
				JOINTED	2.000
				HIGHLY JOINTED	1.000

Specific weight: The value indicated can vary by ± 2 kN/m^3 or even more in some rocks, particularly if they contain heavy minerals (pyrite, for instance).
Strength: The value may range from less than half to over double the figure indicated.
Deformation modulus: This refers to the equivalent deformation modulus of the rock mass when an area larger than 1 m^2 is loaded. The modulus value can vary across a wide range – values three times greater or smaller than those indicated can be found. Poisson's ratio may be taken to equal 0.2 for the hardest rocks, 0.25 for medium and 0.3 for soft rocks.

(*) Moderate rock weathering may reduce deformation moduli by one order of magnitude. Heavy weathering makes the rock mass behave like the soil resulting from weathering.

Table 2.4.3. Some Basic Soil Characteristics for Use in Initial Estimations

	Soil Type	Compacity	Void Ratio (2)	Cohesion (kPa)	Friction Angle (ϕ^0)	Drained deformation Modulus (5) (MPa)	Permeability Coefficient (3) (cm/s)
Granular Soils (1)	Clean sands and gravels (> 10% sand)	Dense	0,25	0	45	100	10^{-2}
		Medium	0,35	0	40	50	
		Loose	0,45	0	35	20	
		Very losse	0,60	0	30	10	

(Continued)

Tabla 2.4.3. Some Basic Soil Characteristics for Use in Initial Estimations

	Soil Type	Compacity	Void Ratio (2)	Cohesion (kPa)	Friction Angle (φ°)	Drained deformation Modulus (5) (MPa)	Permeability Coefficient (3) (cm/s)
Granular Soils (1)	Gravel and sands with low silt and/ or clay contents (5-10%) (4)	Dense	0,20	10	40	50	10^{-3}
		Medium	0,30	5	35	20	
		Loose	0,40	2	30	10	
		Very loose	0,60	0	27	5	
	Gravel and sands with high fine soil contents (10-20%) (4)	Dense	0,15	20	35	50	10^{-4}
		Medium	0,25	10	30	20	
		Losse	0,35	5	27	10	
		Very loose	0,50	0	25	5	
Artificial Fills	Dumped pit-run berms and continuously graded (dirty) rockfills	Loose	0,50	0	40	10	1
		Very loose	0,70	0	35	5	

	Soil Type	Consistency	Void Ratio (2)	Undrained Shear (6) Strength (kPa)	Drained Strength C(kPa)	Drained Strength (φ°)	Drained deformation (5) Modulus (MPa)	Permeability Coefficient (3) (cm/s)
Cohesive Soils	Uniformly graded silts with some sand and clay content	Hard or firm	0,40	100	50	30	40	10^{-6}
		Medium	0,60	60	20	25	15	
		Soft	0,80	20	10	20	7	
		Very soft	1	10	0	18	2	
	Clays and clayey silts. May contain less than 70% gravels and/or sands	Hard or firm	0,35	>100	50	28	50	10^{-8}
		Medium	0,50	80	20	23	20	
		Soft	0,70	40	10	19	5	
		Very soft	1	20	0	15	1	

Soils with noticeable content of organic material have much poorer mechanical characteristics than those shown in this table.

(1) Calcareous soils, particularly conchiferous or coralline types, may have clearly lower friction angles, especially at high pressures (ultimate bearing loads for end-bearing piles).
(2) A particle relative specific weight of 2.7 can be used to calculate specific weight.
(3) The permeability coefficients shown are typical figures only. Soils with the same preliminary description may have a permeability differing by two or even three orders of magnitude from those shown.
(4) The fines content refers to the percentage by weight passing through a 0.080 UNE sieve.
(5) Poisson's ratio can be taken as between 0.30 for dense soils and 0.40 for the loosest or softest soils.
(6) The undrained shear strength of normally consolidated clayey soils depends on the vertical effective pressure (see Subsection 2.2.8.3).

2.5 INVESTIGATION PROGRAMME

In view of the wide variety of problems that can arise from both the initially anticipated characteristics of the project works and the ground geotechnical conditions, it is not feasible to state detailed recommendations covering each individual scenario that could be studied.

Based on the needs that should have been expressly identified in the Preliminary Geotechnical Report, the most suitable form of meeting such requirements must be studied, taking account of the different investigation techniques available.

Independently of any assistance provided by specialists, choice of the most suitable investigation method in each case, the location of points to carry out fieldwork, the depth of exploration, sample tests, special tests, etc. will be the responsibility of the engineers in charge of the project or works for which the investigation is done.

It is recommended that the investigation programme is contained in a document which, apart from defining the work, describes the specific objectives to be achieved by all the different items of work to carry out, provides for possible variations in the work (in terms of depth, complementary investigations, etc.) based on the results obtained and any other information or criteria enabling the necessary adaptations to be made during the investigation stage in order to ensure that the required data are obtained.

Once the necessary geotechnical exploration work has been decided on, the corresponding document should be drawn up, which will serve as a basis for carrying this out. This document should follow the same basic structure as civil engineering design documents, i.e., it should include a written section justifying the geotechnical investigation decided on, as a function of the need for additional information shown by the Preliminary Geotechnical Report, to which it should refer. It should also include drawings that enable the investigation points to be correctly laid out. It should contain a specification of technical requirements on the details of the exploration work and, finally, it must include a cost estimate.

The following sections contain basic recommendations to be taken into account at the time of programming and selecting the different investigation techniques.

2.6 GEOPHYSICAL EXPLORATION

Geophysical methods are most suitable when extensive areas or long alignments need to be explored rapidly and economically.

Fieldwork and the interpretation of measurements must be carried out by highly qualified personnel, after detailed examination of the ground information it has been possible to obtain by other procedures.

In any event, geophysical exploration must be complemented by boreholes made at strategic points to confirm the stratigraphy and ground characteristics deduced from interpreting the geophysics.

Geophysical exploration can be used as a tool for interpolating data between points where a detailed investigation is carried out by boreholes. In this respect, it is advisable to perform the geophysical exploration along the same alignments used for laying out the boreholes.

Table 2.6.1 sets out the most usual geophysical exploration methods, gives a brief description of them and comments on their application.

There are techniques based on the propagation of surface deformation waves in the ground or the measurement of wave dispersion (variation in propagation velocity with frequency) that prove particularly useful for other more specific purposes (especially pavement studies or soil dynamic analyses) than those of geotechnical investigations for maritime works in general.

Tabla 2.6.1. Summary of Different Geophysical Methods Used in Geotechnical Investigation

Name	Parameters obtained	Application	Limitations
SEISMIC REFRACTION Determining the arrival time of waves generated by impact or small detonations at points located at different distances.	Arrival time-distance curves	• Approximate determination of bedrock position in soil deposits. • Determination of the depth of weathering in rocks. • Estimation of ground properties based on propagation velocities.	Possible areas of shade caused by loose strata. Maximum exploration depth of around 20 m.

(Continued)

Tabla 2.6.1. Summary of Different Geophysical Methods Used in Geotechnical Investigation

Name	Parameters obtained	Application	Limitations
WAVE PROPAGATION IN BOREHOLES *Cross-hole*, between two nearby boreholes. *Down-hole* and *up-hole*, along the borehole length. *Seismic tomography*, several boreholes and multiple signals.	Compression and shear wave propagation velocities.	• Dynamic ground deformability studies.	Possible inter-strata interferences, hard to interpret.
SEISMIC REFLECTION Recording echoes of surface-generated pressure waves reflected by the ground.	Image of the geological structure of the ground.	• Approximate location of bedrocks. Generally used to explore depths of over 500 m.	Reflected and refracted waves can interfere and complicate interpretation.
SONAR Recording the arrival times of sound waves generated in water and reflected off the bottom and the rock substrate.	Echo arrival times.	• Determination of sea depth and thickness of soft soil over bedrock.	Prior determination of the velocity of wave propagation in the soil covering the bedrock is necessary
ELECTRICAL METHODS Measuring the electrode-induced current and voltage drop between different points in the ground.	Current intensities and voltage differences for different system configurations.	• Estimation of ground type based on resistivities calculated when interpreting the data. • Used down to depths of 20 m approximately.	Little correlation between the resistivity and the mechanical behaviour of the ground.
GRAVIMETRY Precise measurement of the acceleration of gravity at different points.	Variations in gravity acceleration.	• Detection of large variations in density (voids, faults, salt domes).	Not very accurate for geotechnical work.

There are also techniques for emitting and receiving electromagnetic waves (*georadar*) that vary in usefulness and have limited application in present practise.

2.7 BOREHOLES

Geotechnical investigation with boreholes is generally the most direct method of gathering knowledge of the ground at depth, since it enables samples to be taken and specimens prepared for laboratory testing, *in situ* tests to be carried out and monitoring instruments to be installed, such as piezometers, inclinometers, settlement tubes, etc.

2.7.1 Drilling Boreholes

Drilling boreholes in maritime and harbour zones generally requires the use of specific rigs. These can take the form of fixed or properly anchored floating platforms. Poorly sheltered areas need specially equipped vessels to prevent sea movement having a negative effect on the drilling work.

Boreholes must be drilled at selected points where the information obtainable is of most value and they should be put to good use to obtain the most data. In this respect, it is recommended to specify that borehole drilling should provide the following minimum information:

- Drilling date and identification data of the borehole and the driller performing it.

- Top of hole coordinates and elevation. It is vital that the location of boreholes and especially their elevation are accurately surveyed.

- A drilling log which states the equipment used, the procedure and rates of advance, the depths where samples were taken or tests made, the encased sections and the type and size of the casing and any incident of interest, such as losses in drilling water, water levels or material falling away from the perimeter of the borehole, etc. It is worth mentioning that drilling equipment is available with automatic recording of some of these parameters and its use is recommended.

- A sufficiently robust core box, properly marked with both the corresponding borehole identification and clearly visible figures for the initial and final depths of the core for each run.

- Colour photographs of the core boxes, taken from the front so that clear details of the ground can be seen in each case. It could sometimes be desirable to take detailed photographs of certain individual cores.

- A lithological profile carried out by a graduate-level geotechnical specialist and including a graphical description of the ground at each level, the drilling advance parameters, the location of samples and tests, the elevation of the groundwater table and the percentage of core recovered (1). Depending on the type of ground and exploration carried out, further details of particular interest could be specified for inclusion in these lithological columns.

Table 2.7.1 summarizes some common methods of carrying out boreholes and also provides a short description of them with comments on their use.

TablE 2.7.1. Some ways of drilling Boreholes

Type	Description	Application field
AUGER BORINGS	Continuous drilling by turning a flight auger.	Soils of soft and medium consistency. Undisturbed samples cannot be taken except in cases where the flight auger has a hollow shaft.
DRIVE BORINGS	Driving a tube by impact or vibration and extraction of the detritus with a small spoon.	Soils of soft and medium consistency. Enables tests to be carried out in the borehole and undisturbed samples taken.
ROTARY DRILLING WITH A NON-CORING BIT	Advance by rotary drilling or rotary percussion with hammer at the head or base.	Hard soil and rock. Samples cannot be taken. Can be used to advance drilling between two points where there is interest in taking samples by another procedure.
ROTARY DRILLING WITH CORING BIT	Advance by rotary drilling with a hollow bit.	Firm soils and rocks. Enables samples to be taken and continuous recovery of drilling core.

N.B.: The usual minimum external diameter of the boreholes is 76 mm (3"), so that SPT tests can be carried out inside, with a 2" (51 mm) diameter spoon. When boring unstable soils, they must be cased and drilled to a slightly greater diameter, decreasing with depth (telescopic casing).

Boreholes can be drilled at sea with equipment working on the bed, but usually call for the use of pontoons on which to install the drill rig. Offshore drilling requires special vessels that also include other additional in situ test apparatuses and soil mechanics laboratory where the full geotechnical investigation can be carried out.

(1) In the case of rock, it is suggested that the RQD (Rock Quality Designation) paarameter should be used, which measures –for each meter of advence– the percentage of the core length resulting in fragments with an individual length of over 10 cm.

2.7.2 Investigation Depth

The factors most affecting advisable borehole depths are:

- type of problem under analysis.
- configuration of the subsoil.
- intensity of load applied.

The wide variety of likely scenarios makes it impossible to provide detailed recommendations for all cases, so general recommendations follow which are always applicable, along with a series of specific recommendations for certain typical cases.

In general, explorations must be carried out to sufficient depth to investigate all the levels whose behaviour could significantly affect the works performance, either in relation to bearing capacity or foundation and/or fill settlement, or to problems of seepage or stability of fill or excavation slopes.

It is frequent practice to extend boreholes down until "fresh rock" is found and, unless the purpose of the investigation is specifically to explore the quality of the rock (for point-bearing pile foundations), it should suffice to drill through it just a few metres to confirm its continuity.

If the boreholes reach the rock substrate, they must penetrate at least 2 m into the fresh rock in cases where the nature of the rock is known from prior information and it is only slightly weathered.

If the rock is highly weathered or there is no precise prior information on its nature or if cemented layers appear (sandstone, conglomerate, etc.) interbedded with other poorly cemented layers, boreholes must penetrate into at least 6 m of the substrate rock.

1. *SHALLOW FOUNDATIONS*

The exploration depth necessary for studying shallow foundations must be determined by two methods as shown below. On the one hand, the boreholes must extend below the ground zone that could be involved in a potential foundation bearing failure. To cover this aspect, it is recommended that the exploration depth z below foundation level should be at least:

$$z_{min} = 1.5\ B$$

where *B* is the foundation transversal dimension to be used in studying the bearing capacity.

In shallow foundations with very wide rafts, and when the bearing failure load is not critical, the exploration depth should be fixed for settlement purposes.

On the other hand, the boreholes must be sufficiently deep to investigate the ground that could contribute to foundation settlement. In normal circumstances, when the deformability of the ground decreases with depth, it is assumed that the necessary exploration depth is such that, at the deepest level explored, the vertical load induced by the foundations is a small proportion, around 10%, of the vertical effective pressure existing prior to the works.

Comment: This criterion is fulfilled for small-sized footings compared to the thickness of compressible soils (B << z_{min}) when the investigation depth z below foundation level is:

$$z_{min} = 0.8\ \text{m}\ \sqrt[3]{\frac{N}{N_o}} \quad ; \quad N_o = 1\ \text{kN}$$

where N is the total load on the foundation footing less the earth excavated for its construction and z_{min} is the investigation depth necessary beneath the foundation support level.

For elongated foundations where the ratio between the greater and lesser dimensions is over 2, the following expression can be used:

$$z_{min} = 0.8 \text{ m} \sqrt{\frac{M}{M_o}} \quad ; \quad M_o = 1 \text{ KN/m}$$

where M is the net foundation load per unit length and z_{min} is the investigation depth necessary beneath the foundation level.

For appurtenant harbour works with extensive foundation slabs or when studying settlements in stockpiling areas where the net load is p (total weight less excavated earth), the investigation depth, in metres, below the support level fulfilling this criterion is:

$$z_{min} = 1 \text{ m} \cdot \frac{p}{p_o} \quad ; \quad p_o = 1 \text{ kPa}$$

unless fresh rock or firm soil of negligible compressibility appears first.

For intermediate-sized loaded areas, it may be necessary to carry out a prior study of the stresses induced in the ground to determine depth z_{min}.

When the groundwater table lies below the foundations, the minimum investigation depth for settlement purposes can be decreased by applying the reduction factor:

$$\alpha = 1 - 0.2 \frac{z}{z_{min}} \not< 0.8 \qquad \text{non – elongated foundations}$$

$$\alpha = 1 - 0.3 \frac{z}{z_{min}} \not< 0.7 \qquad \text{elongated foundations}$$

where z is the depth of groundwater table below the foundation plane and z_{min} is the value obtained above for settlement reasons.

2. *DEEP FOUNDATIONS*

The exploration depth required to study deep foundations must be estimated after considering three typical problems - individual pile bearing failure, bearing failure of a group of several piles and foundation settlement.

To cover the first aspect, the recommended practice is to extend the investigation to a depth of five times the pile diameter below the level intended for its tip, i.e., the minimum exploration depth below the surface of the pile cap must be:

$$z_{min} = L + 5\,\phi$$

where *L* and ϕ are the pile length and diameter respectively.

To cover the bearing failure of a group of piles, the following minimum borehole depths below their cap level are recommended:

$$z_{min} = L + 1.5 \text{ B} \qquad \text{end - bearing piles}$$

$$z_{min} = \frac{5}{3} L + 1.5 \text{ B} \qquad \text{shaft - bearing piles}$$

where *L* is the length of the pile and *B* is the breadth of the group.

To cover possible settlement problems, the above criteria for shallow foundations should be used, taking the equivalent foundation plane to be the level of the pile tips if they work basically as end-bearing piles and 1/3 of the pile length above the point plane if they basically work as shaft-bearing piles.

Comment: In these cases and in order to take the main difference between deep and equivalent shallow foundations into account, the corresponding investigation depths deduced from the recommended formulae for shallow foundations for settlement purposes should be reduced by applying the factor:

$$\beta = 1 - 0.2\frac{L}{z_{min}} \nless 0,6$$

where:

L = *length of pile to be used.*
z_{min} = *minimum exploration depth corresponding to the equivalent shallow foundation.*

The minimum depths obtained should be measured from the virtual foundation level assumed.

3. *STABILITY OF CUT OR FILL SLOPES*

To define the depth of boreholes intended for studying stability problems in cut or embankment slopes, the maximum depth of potential slides will need to be estimated beforehand.

The investigation depth must reach the level of the deepest predicted slide.

4. *DREDGING*

The minimum borehole depth for dredging studies should be at least approximately 2 m greater than the thickness of the anticipated dredging. It may sometimes be necessary to investigate even greater depths. The recommendations from the above paragraph apply near the side slopes of dredged areas.

These investigation depths correspond to normal, homogeneous ground conditions where compacity increases regularly with depth.

If fresh rock appears at lesser depths than those indicated, the borehole depth criterion recommended at the beginning of this section should be applied.

If soft areas of organic soil or normally consolidated clay or silt appear at the recommended investigation depth, this should be increased to pass completely through them. Even in these cases, it is not deemed necessary to extend the exploration beyond three times the minimum depths for normal situations.

It is advisable to make one or more of the investigation boreholes substantially deeper in order to confirm the assumptions made about the general ground structure.

2.8 WELLS, TRIAL PITS AND TRENCHES

The quickest, most direct and inexpensive form of exploring the topmost ground region in emerged areas is the open excavation of trenches or trial pits. For greater depths, wells can be dug, but they only present advantages over boreholes in special cases.

These excavations are particularly appropriate in exploring borrow materials in which compaction tests will be carried out, as these tests require bulky samples difficult to obtain by other procedures.

It is possible to take undisturbed samples from the bottom and walls of such excavations. Nevertheless, it is recommended that the document prepared for planning the geotechnical investigation and serving as a guide to this work expressly forbids manual sampling with personnel working at a depth of more than one metre, unless the excavation is properly shored up. Accidents in this type of exploration are frequent and dangerous.

To document these excavations, it is recommended to draw a plan view diagram showing their position and coordinates, to describe the excavated materials and those appearing at the bottom of the excavation -with appropriate drawings and photographs- and to specify the position of the groundwater table plus a list of the samples taken and the *in situ* tests it was possible to carry out.

2.9 IN SITU TESTING

Determining ground characteristics by *in situ* tests has a clear advantage over obtaining properties in the laboratory. In the first case, the ground is tested under conditions similar to the ones relevant for subsequent studies. This is not always the case, as situations do exist (borrow material tests, for example) where maintaining natural conditions is not of interest. There are also special cases when *in situ* tests have to be carried out under conditions that are further removed from those of interest than the ones that can be simulated in the laboratory.

It is generally recommended that as many geotechnical parameters as possible should be obtained by *in situ* tests, particularly those related to shear strength, compressibility and permeability. Laboratory tests should later enable these characteristics to be extended to different pressure and environment ranges from those of *in situ* testing and that could be of interest in the context of the investigation objectives.

2.9.1 Standard Penetration Test, SPT

The Standard Penetration Test, SPT, is the most common *in situ* test. Virtually all firms involved in geotechnical investigation have the necessary equipment to carry this out and in current geotechnical practice it is also the best test for exploring the level of compacity of deep sand deposits.

The test is carried out in a borehole whose walls are either stable or supported by an adequate casing. Once the intended testing depth has been reached and the bottom of the borehole is clear of detritus, a pipe with a bevelled cutting tip (SPT spoon) is driven into the bottom. This has an external diameter of 2" (51 mm) and an internal diameter of 11/8" (35 mm). The drilling rods are driven by hammer blows to the head. The hammer weighs 63.5 kg and is allowed to drop freely from a height of 76 cm over the head of the rods.

In gravel deposits, solid cone tips with the same external diameter are used to avoid damaging the spoon.

During the driving process, the number of blows required to penetrate 15-cm sections are counted. Driving ceases when total progress amounts to 60 cm.

The blow-count required to penetrate the middle 30 cm constitutes the N-index of the SPT. When the spoon is removed once the test is completed, a sample of the ground cut through and disturbed by the driving process can be obtained.

The test has been standardised (Spanish UNE 103800 or ASTM D-1586-67) and it must be carefully carried out, so that the results can be interpreted in the light of the considerable experience accumulated. In particular, it must be noted that the following factors –among others– can give rise to substantial errors:

- Heave of the bottom of the borehole due to an imbalance between the ground piezometric level and the water level in the borehole.
- Inadequate cleaning of the bottom prior to start of the test.
- Hammer weight other than 63.5 kg.

- Mistaken measurement of the hammer drop height.
- Friction during hammer drop (not a free fall).
- Eccentric blows on the rods.
- Poor spoon condition (damaged or blunted bevels).
- Loosely connected rods.
- Borehole diameter too large.
- Casing considerably above or below the test level.
- Heavier than standard rods.

Dynamic analysis of the driving process of the SPT spoon allows the energy conveyed through the rods to be evaluated. This energy can be measured during the course of the tests with special monitoring devices (accelerometers and strain gauges). In SPT tests carried out under normal conditions, this energy is approximately equal to 60% of the theoretical potential energy of the hammer. It is sometimes possible to know this energy (by calibrating equipment) and so reliable information may exist on the percentage of energy transmitted, η. When this is the case, the value of the SPT N-index can be transformed into the value that would correspond to a standardised driving process of 60% energy, by applying the following equation:

$$N_{60} = N\ (SPT) \cdot \frac{\eta}{60}$$

where η is the corresponding energy fraction expressed as a percentage. In the absence of specific information, it should be assumed that η = 60%.

SPTs enable ground parameters to be obtained that are particularly relevant for estimating the ease of pile and sheet pile driving.

The SPT N-index is related to the compacity of sands. Terzaghi & Peck (2) (1948) proposed the following relation:

N (SPT)	Compacity
0-4	Very loose
5-10	Loose
11-30	Medium
31-50	Dense
Over 50	Very dense

It is sometimes desirable to quantify the concept of compacity. For this purpose, using the concept of relative density, D_r, is recommended (see 2.2.4).

The relationship between the relative density, D_r, and the N-index of the SPT varies depending on the vertical effective pressure at test level. Among existing correlations and in the absence of better data, the Gibbs & Holtz (3) (1957) correlation as shown in Figure 2.9.1 is considered admissible.

(2) Terzaghi & Peck (1948): *Soil Mechanics in Engineeting Practice*. John Willey & Sons.
(3) Gibbs, H. J.; Holtz, N. G. (1957): *Research on Determining the Density of Sands by Spoon Penetration Testing*. 4th. ICSMFE. Londres.

An obvious correlation exists between the angle of friction of granular soils and the N-index (SPT). The most widely applied is probably the correlation defined by Schmertmann [(4)], which can be approximated by the following analytical expression:

$$\tan\phi = \left(\frac{N}{12.2 + 20.3\,\dfrac{\sigma'_{vo}}{p_a}} \right)^{0,34}$$

where:

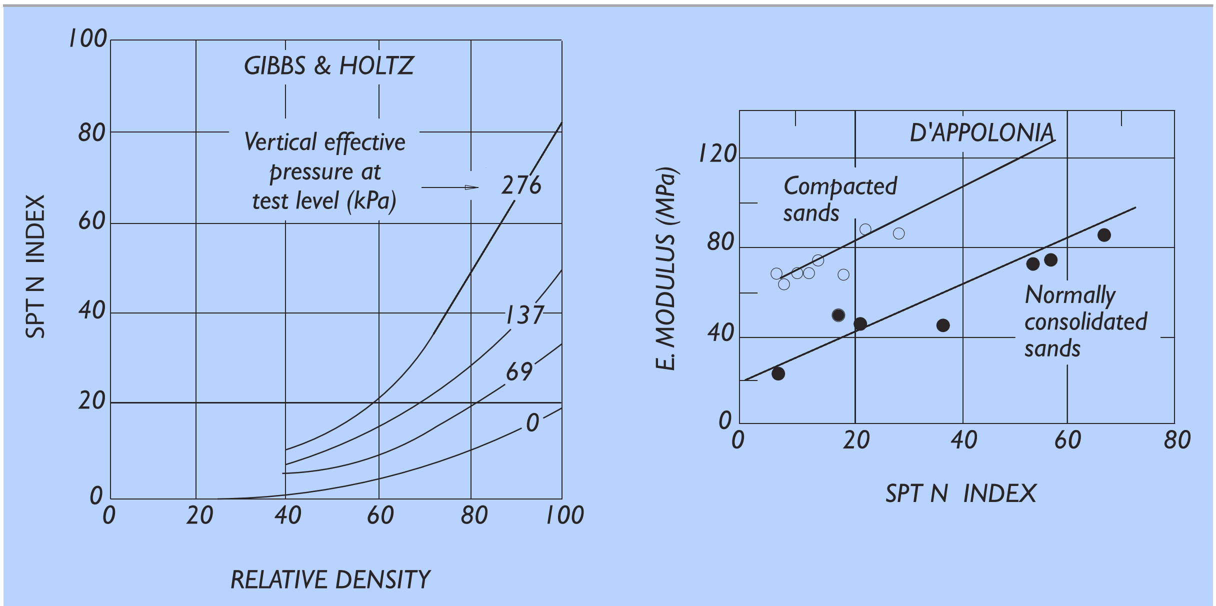
ϕ = angle of friction
N = SPT index
σ'_{vo} = vertical effective pressure at test level
p_a = reference pressure (1 bar = 100 kPa).

For the purpose of calculating shallow foundation settlement and also in the absence of more specific information, the deformability of sands can be estimated according to the D'Appolonia [(5)] (1970) correlation also shown in Figure 2.9.1.

Criteria on bearing capacity for shallow or deep foundations can be established, and also settlement estimated, on the basis of SPT results when done in formations of normal sands and for which verified local experience exists. When the SPTs are carried out in cohesive soils or soft rock, their results can only serve as a guidance in this respect.

In calcareous sandy formations, particularly of organic origin (conchiferous, coralline, etc.), it is recommended not to use the usual criteria based on SPT experience, as the bearing capacity could be overoptimistic.

Figure 2.9.1. ***SPT Correlations (Sands)***

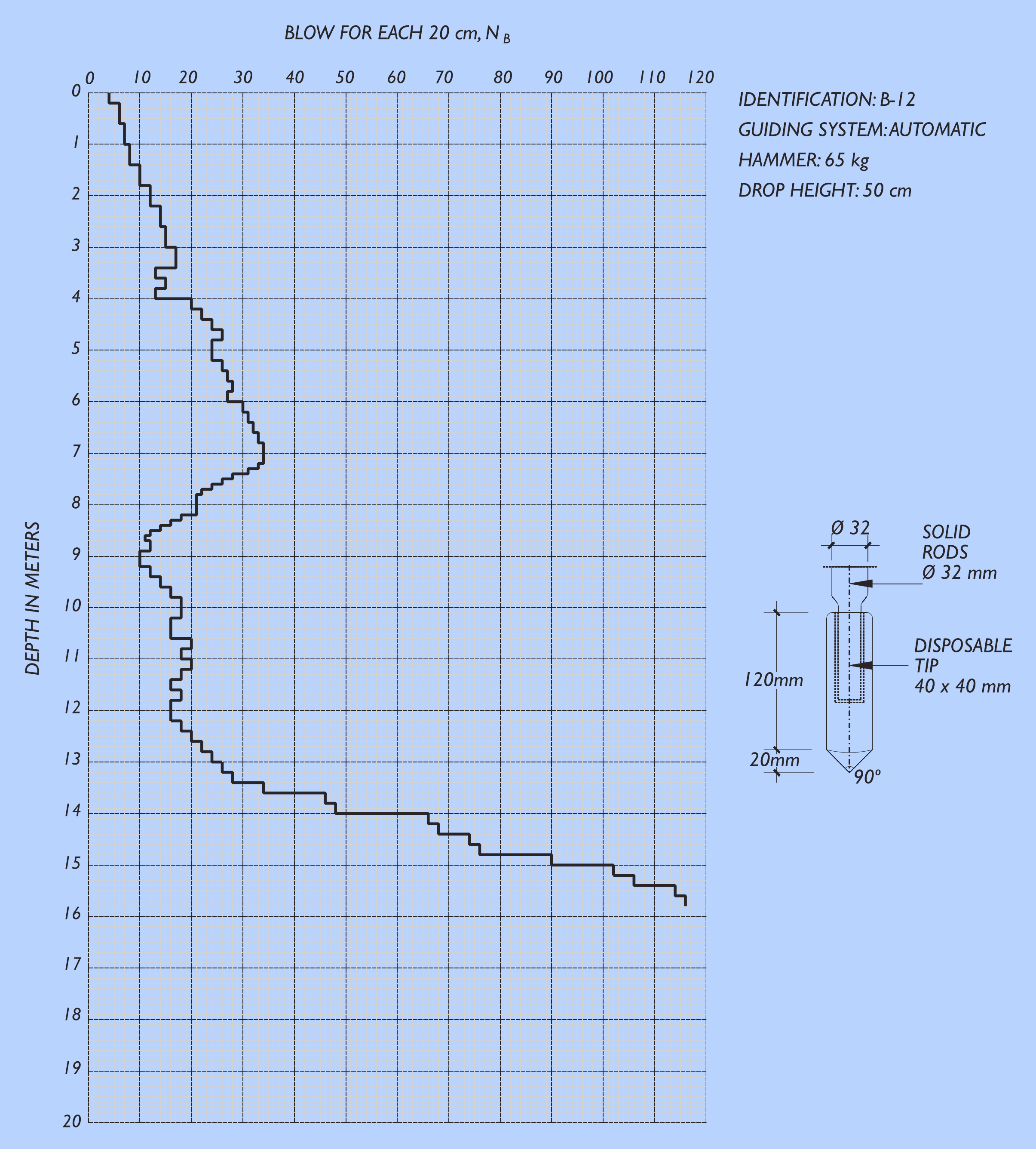


(4) Schmertmann, J. H. (1975): *Measurement of In-Situ Shear Strength.* Proc. ASCE Specialty Conference on In-Situ Measurement of Soil Properties. Raleigh. USA.
(5) D'Appolonia, D.J.; D'Appolinia, E. and Brisette, R.F. (1970). *Discussion on "Settlement of Spread Footing on Sand".* Proc. ASCE, J. Soil Mech. and Found. Enging. Div., 96 (S12), pp. 754-761.

2.9.2 Dynamic Penetrometers

The cheapest and simplest way of testing soil at depth consists in driving in a set of rods with a suitably shaped metal tip. The most widespread penetrometer test used in Spain (and in the rest of Europe) is known as the Borro test (NLT 261). This apparatus consists of a set of solid metal rods with an external diameter of 32 mm that pushes a thick metal tip with the shape and dimensions shown in Figure 2.9.2. It is driven by a 65-kg hammer (or a SPT 63,5-kg one), dropped freely from a height of 50 cm. The blows needed to advance 20 cm are counted as the driving proceeds. The result is usually shown in the form of a diagram with the number of blows, N_B, obtained at each depth.

Figure 2.9.2. Borro Dynamic Penetration Test



As the size of the tip exceeds the diameter of the rods, little friction occurs between the latter and the ground and the result of the test should be related to the ground strength around the tip.

The driving continues down to the desired depth as previously determined or until a highly resistant ground is reached. The metal tip is left buried in the ground when the rods are recovered.

The rods are usually rotated, even if only by hand, to avoid friction between them and the ground. In some types of drill rigs, this rotation is made regularly and they also have a hammer guiding mechanism preventing any possible friction in the hoist cable.

The Borro apparatus can also have a different tip that is smaller and conical in shape.

In addition to the Borro equipment, there are dynamic penetrometers that are used less often, such as the light DIN and the Stump, which were initially designed for manual driving (with no motor nor winch for hammer lifting).

Two continuous dynamic penetration tests are standardised in Spain:

- DPSH. UNE 103.801 Standard
  (Dynamic Penetrometer Super Heavy).

- DPH. UNE 103.802 Standard
  (Dynamic Penetrometer Heavy).

The possible correlations between the results of the different types of penetrometer tests can be explored as indicated in Subsection 2.9.5.

The best field of application for dynamic penetrometers is to determine the depth of soft or medium consistency soils lying on top of much stronger formations in which the driving is stopped.

The penetration test is very useful for detecting changes in ground compacity (softer areas in fills, cavities, defective compaction in embankments, etc.). The test is also useful for estimating the ease of pile driving.

It is recommended to carry out dynamic penetration tests along the same alignments as the investigation boreholes and/or along the same profiles used for geophysical exploration. The results can confirm the homogeneity of the ground between points explored by boreholes and detect any local heterogeneity indicating the need for a denser borehole grid.

In any event and given the variations possible in execution details, logs of the results from these tests should expressly include the following data:

- weight of the hammer and drop height,

- type of hammer drop and guiding mechanism (manual or automatic),

- tip shape, supplying a small diagram.

The resistance to advance in the borehole casings that have been driven and the number of blows required to drive the sampler in the ground can also serve for estimating its consistency. To this purpose, knowing the details of such driving is necessary.

It is recommended not to use data from continuous dynamic penetrometers for quantifying bearing capacity or settlement in any type of foundation except where local experience exists that has been clearly verified by other methods.

2.9.3 Static Penetrometers

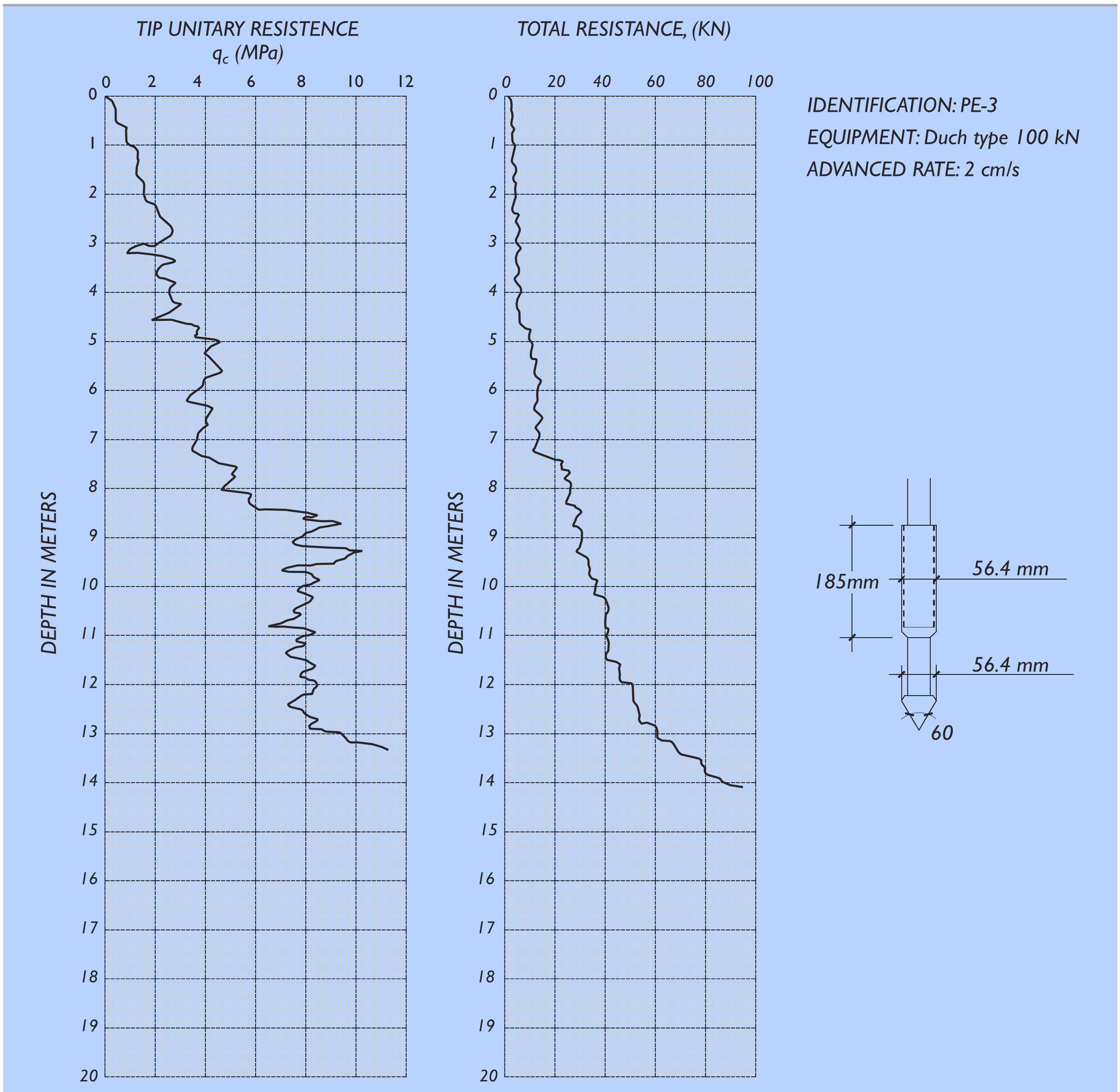
The static penetration test (or CPT, *Cone Penetration Test*) consists in applying pressure to force a rod with a suitable tip into the ground at a low speed (1-3 cm/s). The most common apparatus used in Spain and the rest of Europe is the Dutch cone (UNE 103804).

The penetrometer advances in discontinuous stages, which enables the penetration resistance to be measured at the tip alone or along the complete device. Automatic apparatuses are available that advance continuously and measure penetration resistance at the tip and through the friction on the side sleeve.

Equipment is available with different thrust capacity and tip shapes. Other countries have standards for carrying out the test (DIN 4094, ASTM D-3441) that could be useful to apply in Spain.

In the graphic test logs, it is advisable to include a sketch of the tip used, as this factor is not always the same (differing regulations) – see Figure 2.9.3.

Figure 2.9.3. ***Static Penetration Test***

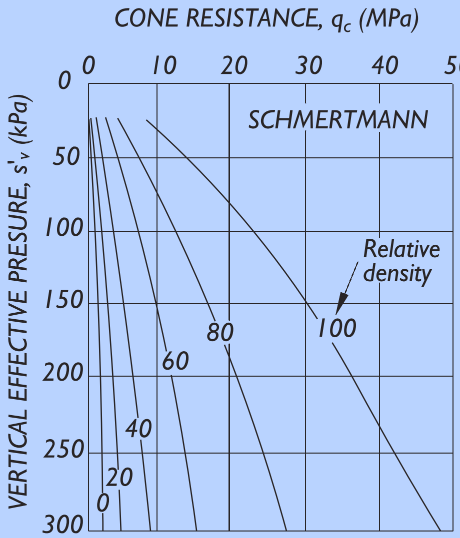


Interpretation of these tests enables the shear strength of the ground to be determined, along with an indirect description of the type of soil penetrated and its compressibility. In this respect, complementary exploration by other methods will be necessary to obtain an accurate description of the nature of the ground and even a more exact determination of its deformability.

The ground shear strength resulting from these tests is particularly suitable for calculating the bearing capacity of deep foundations.

A relationship exists between the tip resistance in the static penetration test, q_c, and the relative density of sands. There is also a relationship between this sand strength and the modulus of deformation that should be used in shallow foundation settlement calculations. In the absence of more data, the Schmertmann [6] (1978) correlations shown in Figure 2.9.4 are considered acceptable.

Figure 2.9.4. ***Static Penetration Test Correlations***



Normally consolidated, recent and uncemented fine sands

MODULUS OF DEFORMATION FOR SHALLOW FOUNDATION SETTLEMENTS CALCULATION

$E_s = 2{,}5\ q_c \quad \text{for}\ \frac{L}{B} < 2$

$E_s = 3{,}5\ q_c \quad \text{for}\ \frac{L}{B} > 10$

E_s = Constrained modulus of deformation. This is theoretically related to the modulus of elasticity and Poisson's ratio by the equation:

$$E_s = E\,\frac{1-\nu}{(1+\nu)(1-2\nu)}$$

L,B = Length and breadth of the shallow foundation

A clear correlation exists between the tip resistance in the static penetration test and the angle of friction in granular soils. Even though this correlation depends on several factors, the following expression given by Robertson & Campanella [7], can be used as a guidance value:

$$\tan\phi = 0.10 + 0.38 \cdot \lg_{10} \frac{q_c - \sigma_{vo}}{\sigma'_{vo}}$$

where:

ϕ = angle of friction.
q_c = tip resistance.
σ_{vo} = total vertical pressure at test level.
σ'_{vo} = vertical effective pressure at test level.

(6) Schmertmann, J.H. (1978): *Guidelines for Cone Penetration Test Performance and Design.* Federal Highway Administration. Report FHW-TS-78-209, USA.
(7) Robertson P.K. y Campanella R.G. (1983) «Interpretation of Cone Penetration Test». Canadian Geotechnical Journal. Vol. 20.

The static penetration test is particularly suitable for measuring the undrained shear strength of soft cohesive soils. The relationship usually established for cohesive soil on the seabed is:

$$s_u = \frac{1}{N_k}\left(q_c - \sigma_v\right)$$

where:

s_u = Undrained shear strength.
q_c = Unitary tip resistance to cone advance.
σ_v = Total vertical pressure at test level.
N_K = dimensionless scale factor.

The factor N_K is close to 15. Engineers can use this value as a basic reference but must be aware that N_K varies depending on the type of ground, the depth and other possible factors that are not well known as yet.

2.9.4 The Piezocone. CPTU

The piezocone or CPTU consists of a continuous static penetrometer with a piezometer fitted to the tip. This apparatus provides continuous logging of the tip and shaft resistance as well as control of the pore water pressures generated during the driving process and also of their dissipation when the driving stops.

This type of penetrometer is very suitable for investigating soft soils, basically in determining parameters for studying consolidation and stability problems implying deep failure surfaces through soft soils.

The logs obtained during the driving of a piezocone allow the different permeability levels to be identified even though these may not be very thick. This aspect is of particular interest when it comes to evaluating the drainage conditions of consolidation processes.

There are no clearly established standards for this test, although abundant technical literature exists on interpreting its results. To interpret the data measured by a piezocone, the usual practice is to calculate the auxiliary parameters listed below, at each depth involved.

- Dimensionless penetration resistance, Q_t, according to the following expression:

$$Q_t = \frac{q_c - \sigma_v}{\sigma'_v}$$

where:

q_c = unitary resistance to advance of the cone tip in the area under study.
σ_v = total natural vertical pressure (prior to running the test) in the area under study.
σ'_v = natural vertical effective pressure (prior to running the test) in the area under study.

- Relative increase in porewater pressure, B_t, defined by:

$$B_t = \frac{u_c - u_o}{q_c - \sigma_v}$$

where:

u_c = porewater pressure measured by the piezocone at the desired level.
u_o = previously existing porewater pressure at the desired level.
q_c, σ_v = same meaning as in the preceding case.

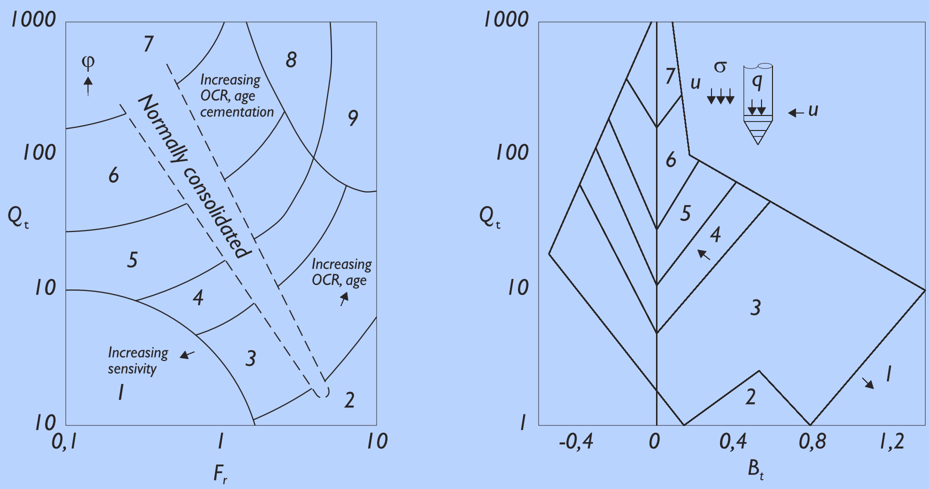
- Relative lateral friction, F_r, calculated as follows:

$$F_r = \frac{f_s}{q_c - \sigma_v}$$

where f_r is the lateral friction measured by the piezocone at the desired level and q_c and σ_v have the same significance as given above.

Experience-based correlations exist enabling the ground type to be classified as a function of these parameters. One of them [8] –in widespread use– is illustrated in Figure 2.9.5.

Figure 2.9.5. Soil Classification Based on CPTU Results



$$Q_t = \frac{q_c - \sigma_v}{\sigma'_v} \qquad B_t = \frac{u_c - u_o}{q_c - \sigma_v} \qquad F_r = \frac{f_s}{q_c - \sigma_v} \times 100\ \%$$

Typical soil behaviour	Typical soil behaviour	Typical soil behaviour
1. Sensitive, fine grained	4. Silty mixtures - Clayey silt to silty clay	7. Gravelly sand to sand
2. Organic soils - peats	5. Sand mixtures - silty sand to sandy silt	8. Very stiff sand to clayey sand
3. Clays - clay to silty clay	6. Sands - clean sand to silty sand	9. Very stiff, fine grained

In order to interpret the results of porewater pressure dissipation tests (pressure evolution as measured by the piezometer when the cone stops advancing), it is necessary to calculate the time value corresponding to 50% of the dissipation, from the test graph. This time can be obtained by applying the same techniques used to interpret consolidation times in oedometric tests (logarithmic or Casagrande method and square root of time method, for example).

The rigidity index I_r must also be known, as defined by the following equation:

$$I_r = \frac{G}{s_u}$$

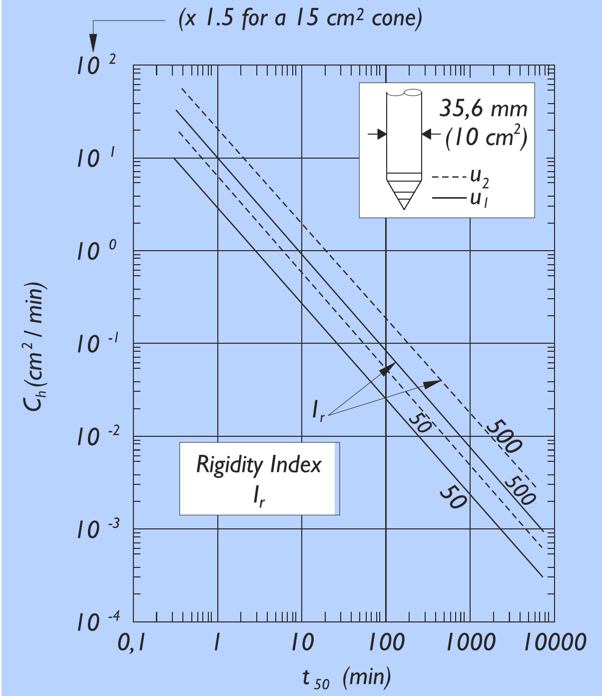
where:

G = shear modulus of the soil.
s_u = undrained shear strength.

(8) Robertson, P.K. (1990). "Soil Classification Using the Cone Penetration Test". Canadian Geotechnical Journal, 27 (1), p. 151-158.

With these data (t_{50} and I_r), the value of the radial consolidation coefficient, c_h, can be estimated at the level where cone driving is stopped and the dissipation test carried out. This estimation is based on previous experience [9], as shown in Figure 2.9.6.

Figure 2.9.6. Graph for Determining C_h Based on t_{50} and a Rigidity Index (I_r) between 50 y 500



2.9.5 Correlation Between Penetration Tests

An equivalence can be established between different continuous dynamic penetrometers so that the specific driving energy becomes similar.

The N_B index for the Borro test (with automatic mechanism for dropping the 65-kg hammer from 50 cm, a 4x4 cm² square tip and counting the number of blows needed to penetrate 20 cm) is usually higher than the N (SPT) at greater depths and less than N during the initial metres. The correlation is a very weak one, however, and if it needs to be defined, it must be analysed for each soil formation and at each depth. Otherwise, previously established –or based on verified local experience– correlations should be used.

2.9.6 Pressuremeters and Dilatometers

These tests consist in applying a certain pressure within the ground and measuring the deformation caused. They are usually carried out inside a previously-drilled borehole (PBP or *pre-bored pressuremeter*) into which the pressuremeter device is introduced. The *self-boring pressuremeter* or SBP has recently begun to be used. This is housed in the actual drilling equipment and consequently the test does not have to be separated into two stages. This new apparatus prevents the stress relaxation and deformation (even possible failure) that may occur prior to positioning conventional pressuremeters. *Push-in pressuremeters* or PIPs may also be used which, as their name implies, are pushed into soft soils.

These tests are not standardised in Spain but are usually carried out under the French standard (NFP 94-110).

Pressuremeters use a cell sealed by a membrane holding a controllable volume of fluid. The increase in this fluid volume compresses the membrane against the walls of the borehole. These two quantities (volume and pressure) allow the deformation-pressure diagram to be drawn, as illustrated in Figure 2.9.7.

The results of pressuremeter tests are usually presented in graph form with the x-axis corresponding to the radial deformation, ε_r, defined by the expression:

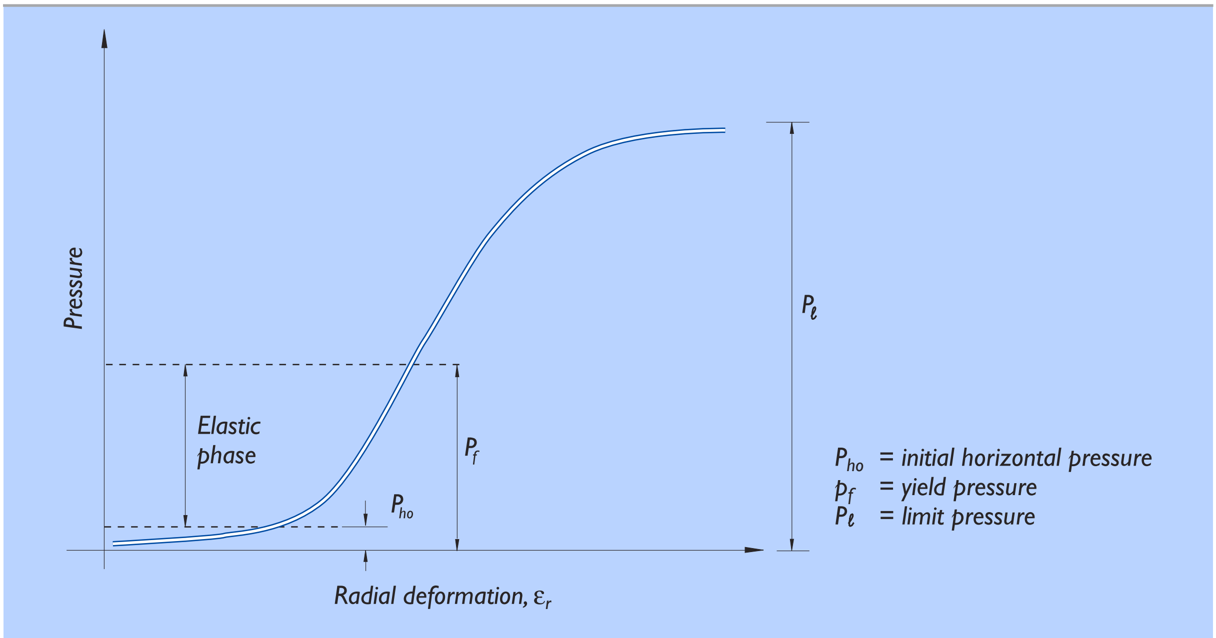
$$\varepsilon_r = \frac{r - r_o}{r_o}$$

where:

r = average cavity radius at the time of the test.
r_o = initial reference radius.

(9) Robertson, P.K. et al (1992). "Estimating Coefficient of Consolidation from Piezocone Test". Canadian Geotechnical Journal, Vol. 29, n° 4, pp. 539-550.

Figure 2.9.7. **Schematic Result for a Pressuremeter Test**



To perform this test on very firm soils and rocks, more robust devices can be used, known as dilatometers, which measure the deformation using extensometers and provide the radial strain directly and more accurately.

The apparatus calibration, prior to its use, identifies which part of the pressure being applied is necessary to deform the membrane. This value must be subtracted from the pressure applied in order to obtain the corrected pressure, which is the one that should be used in the results graph.

Interpretation of the pressuremeter test allows three interesting pressure values to be obtained.

a. *Initial horizontal pressure*, p_{ho}. This is the pressure that must be applied to establish a contact between the membrane and the ground and displace it up to its original position (before drilling the borehole). In conventional pressuremeters, this pressure corresponds to the sudden change of slope in the pressure-deformation curve (point of maximum curvature). Detailed procedures have been published to determine this [(10)].

b. *Yield pressure*, p_f. This is the pressure at the end of a straight section that usually appears in these diagrams. From there onwards, strains are clearly non linear. There are specific techniques for determining this point in a detailed way although it is routine practice to define it almost just by looking at the curve.

c. *Limit pressure*, p_l. This is the pressure that originates a 41% radial strain (100% volumetric deformation). If the test has not reached this level of deformation, it must be obtained through an extrapolation.

The data mentioned (p_{ho}, p_f and p_l) can be used to design both shallow and deep foundations as indicated later, in Part 3 of this ROM 0.5.

In addition, interpretation of the pressuremeter curve enables certain parameters of ground behaviour to be obtained, although very imprecisely, as indicated below.

(10) See, for instance, the doctoral thesis by H. Cano Linares "Técnicas 'in situ' para la caracterización del comportamiento elástico no lineal de suelos duros". E.T.S. Ing. Caminos, Canales y Puertos. U.P.M. Madrid 2004.

The at-rest pressure coefficient of the ground, K_o, can be obtained by the following expression:

$$K_o = \frac{p_{ho} - u}{\sigma'_{vo}}$$

where:

p_{ho} = initial horizontal pressure deduced from the test.
u = porewater pressure at test level.
σ'_{vo} = vertical effective pressure at test level.

The ground shear modulus, G, for the strain level of the test and for the corresponding load direction (perpendicular to the borehole axis), can be obtained with the expression:

$$G = V_o \cdot \frac{\Delta p}{\Delta V}$$

where:

Δp = increase in pressure between the two points delimiting a linear response.
ΔV = increase in volume between these same two points.
V_o = reference volume.

In normal circumstances, the reference volume to be used is the one corresponding to the start of the elastic phase (straight stretch). The value for the pressuremeter modulus, E_p, is defined using the following expression:

$$E_p = \frac{G}{2(1+\nu)}$$

where:

G = shear modulus, as defined above.
ν = Poisson's ratio.

Poisson's ratio cannot be obtained with this test. If the pressuremeter modulus is to be known, a suitable value for ν will have to be assumed.

Pressuremeter tests allow the nature of the ground to be ascertained, since their execution requires prior extraction of the ground where the test apparatus is to be installed. These sample cores should be laboratory analysed - at least the most elementary identification tests must be run on them.

In saturated clayey soils, it is possible to obtain a recommended value for the undrained shear strength, s_u, from a specific analysis on the non-linear zone at the end of the pressuremeter curve (test pressure ranging between p_f and p_l). The corresponding expression is:

$$s_u = \frac{p_2 - p_1}{\ln\left(\frac{V_2 - V_o}{V_1 - V_o}\right)}$$

where:

p_1, p_2 = any pressures in the $p_f - p_l$ section.
V_1, V_2 = volumes of fluid in the pressuremeter for these same pressures.
V_o = reference volume. The one corresponding to p_{ho} is to be taken.

In theory, the value of s_u should be independent of Points 1 and 2 selected. But different values will need to be tried in order to obtain one that is reasonable.

In permeable sandy soils, provided it can be guaranteed that substantial porewater pressures are not generated during the test, an approximate idea of the internal angle of friction of the ground can be obtained from this curve branch at the end of the test, using the following expression [11]:

$$\phi = 7^\circ\ (1 + 10\ s) > 30^\circ$$

where:

$$s = \frac{\ln\left(\dfrac{p_2 - u_o}{p_1 - u_o}\right)}{\ln\left(\dfrac{r_2 - r_o}{r_1 - r_o}\right)}$$

where:

p_1, p_2 = any pressures in the $p_f - p_l$ section.
r_2, r_1 = pressuremeter radii for these same pressures.
r_o = reference radius, the one corresponding to p_{ho} should be taken.
u_o = hydrostatic porewater pressure at test level.

2.9.7 Vane Tests

Vane tests are carried out at the bottom of boreholes or by directly driving the vane to test levels. They prove particularly appropriate for investigating the undrained shear strength of soft clayey soils.

Foreign codes exist for the vane test (ASTM D 2573, DIN 4096) and it is in the process of being standardised in Spain.

Interpretation of the results also enables soil deformability to be indirectly estimated.

The strength parameters obtained are also adequate for determining the bearing capacity of both shallow and deep foundations in undrained conditions and also for studying the stability of side slopes in dredging or fills, equally in undrained conditions.

2.9.8 Plate-bearing Tests

Plate-bearing tests are particularly adequate for studying the bearing capacity of compacted fills and also of natural ground.

Interpreting the results also provides values for the deformation moduli that can be applied to settlement predictions and an approximate estimation of the ultimate bearing capacity of shallow foundations.

As these tests affect a small area of ground with the regular plate sizes (see NLT 357, ϕ 30, 60 or 76.2 cm), they will not allow soil deformability to be ascertained beyond the zone adjacent to the tested surface.

(11) This simplified expression was deduced from a graph in the publication by Mair, R.J. & Word, D.M. (1987). "Pressuremeter Testing: Methods and Interpretation". Butterworths. London. For this puerpose, a conventional granular soil was assumed with a critical angle of friction of 30°.

2.9.9 Permeability Tests in Boreholes and Trial Pits

The ground permeability can approximately be obtained by monitoring water losses in trial pits whose surroundings have been previously saturated, in boreholes filled with water to a height above the groundwater table of the nearby area (Lefranc test) or in plugged boreholes under forced pressure (Lugeon test).

The permeability resulting from these tests can be used in the qualitative analysis of drainage conditions for a particular problem. If the permeability of a certain ground level proves to be a critical design parameter, then it should be determined by pumping tests specifically designed to analyse the particular problem involved.

Useful formulae for interpreting the results of *in situ* permeability tests are given in Appendix 1.

2.9.10 Field Tests and Other *in Situ* Tests

For studying geotechnical problems that could have important repercussions, it may well be advisable to carry out field tests specially aimed at analysing the problem in question.

The following stand out:

- Analysing settlement and consolidation in soft zones by means of instrumented test embankments is relatively frequent.
- Pile driving tests are particularly recommended, as they can nowadays be instrumented and analysed in detail.
- Tests on pile bearing capacity and on the strength of anchors or elements in horizontal tension are frequently carried out and are advisable in large-scale works.
- Tests of horizontal push or pull between piles are of particular interest because they are easier to carry out. They can be used for accurately obtaining ground deformability parameters to be employed in analysing piles under horizontal loads in different configurations.
- Pumping tests are particularly useful for studying seepage problems and are considered essential when investigations are carried out to study works that will subsequently require forced drainage involving permanent dewatering.

Other *in situ* testing equipment is worth pointing out, like:

- Combined (static/dynamic) penetration devices, which can be dynamically driven through hard zones, making it possible to run static penetration tests at greater depth. Although they could be useful in soft formations with harder cemented or encrusted levels, these combination devices are not widespread.
- *In situ* permeability test devices are available (self-drilling permeameters) plus a wide variety of other borehole test apparatuses (bottom plate load) tending to be useful in particular cases but still not widely used.

If some special *in situ* exploration equipment is required, technicians planning geotechnical investigations are recommended to consult specialist firms, as there are an increasing number of procedures available, designed for specific purposes, which can be particularly useful in some geotechnical investigation programmes.

A wide variety of other *in situ* tests and field trials exist (riprap and rockfill shear strength, bearing capacity of shallow foundations, instrumented excavation, observation nets for pore pressure evolution, etc.) which are more related with instrumentation and monitoring of works. They are extremely interesting but lie beyond the scope of the general recommendations in this ROM 0.5.

2.10 SAMPLING

Sampling is one of the most important parts of geotechnical investigation programmes and for this reason must be planned prior to the start of fieldwork.

Samples can be taken from boreholes, trial pits or specified sites where no prior excavation or drilling has taken place.

Samples can either be disturbed, i.e., after they are taken they have a different density or water content from the original, or they can be undisturbed, in other words, the state in which the moisture content and density (and therefore strength, deformability and permeability) remain as close as possible to the original. In any event, samples must be representative of the soil which will be tested. In this respect, the samples should never be washed or segregated unless, for some particular reason, this is not important in the problem under study.

Disturbed samples can be taken manually using pick and shovel, mechanically by excavator or come from borehole cores. They can be transported in sacks or bags.

Undisturbed or slightly disturbed samples can be taken with specific *samplers*, (by pushing short bevelled tubes in the ground) from the sides of previously shored trial pits, wells or trenches. They should be packed, transported and stored in the laboratory -until tested- in such a way that disturbance is minimized.

In geotechnical investigations, sampling is most usually carried out in boreholes by specific samplers adapted to the type of ground. Table 2.10.1 summarizes information about the most common sampling devices.

Undisturbed or only slightly disturbed samples of clean granular soils cannot be taken by conventional procedures. Bishop or Osterberg piston-tube samplers can prove effective for some sands.

Sampling must be supervised by the specialist in charge of fieldwork. It is extremely important that an expert estimates the degree of disturbance in the samples taken.

Table 2.10.1. ***Common Procedures for Deep Sampling***

	Ground Type	Sample quality
PUSH-TUBE SAMPLERS At the bottom of boreholes thin-walled Shelby tubes can be driven, which will slightly disturb the soil contained within them. Different procedures are used depending on whether the soil is soft (samplers forced in by pressure or by vibration, *vibrocorer*, with different core-retaining systems) or by percussion when the soil has a medium and even high compacity.	Very loose, loose and medium cohesive soils and some granular soils, with enough fines and not very dense	Slightly disturbed
CORE BARREL SAMPLERS In boreholes made by rotary drilling, a tube is housed inside the drilling bit that partially protects the sample from the rotary drilling effects.	Cohesive soils with a firm to very firm consistency and rocks.	Somewhat disturbed
DRILLING CORES In boreholes drilled with hollow coring bits into firm soils, it is possible to obtain a core of the soil that has not been destroyed during drilling.	Cohesive soils with a firm or very firm consistency and rocks.	Fairly to highly disturbed
SPT SPOON The tube forming the SPT spoon makes it possible to obtain a sample of the ground in which the spoon is driven, for a large number of soil types.	Not very firm cohesive soils and not very dense granular soils without gravel but containing some fines.	Highly disturbed

The sampling procedure used should be documented, indicating for each individual sample or group of samples the provenance (borehole, trial pit or other point with known coordinates), the lithological column corresponding to the sampling location, express indication of its depth, position of the groundwater table at the sampling location and any other observation deemed appropriate by the geotechnical specialist in charge.

As samples are taken to run laboratory tests, their number and location should be planned only after deciding on which tests will be necessary for analysing the problems the geotechnical report addresses.

2.11 LABORATORY TESTS

Laboratory tests are the main tool used nowadays for studying the ground geotechnical characteristics. It will rarely be possible to do correct geotechnical studies with no laboratory tests.

Some tests are specifically designed to define the nature of the soil, i.e., its particle size grading and mineralogical composition, index properties, etc. There are also special laboratory tests for studying soil strength, deformability and permeability.

Whenever laboratory tests are carried out, details of each specimen used must be clearly recorded together with a statement of its condition in respect of being totally or partially disturbed or undisturbed. The manner in which the sample was packed, transported and stored in the laboratory until the test must also be recorded along with the actual test procedures used in the laboratory.

2.11.1 Soil Identification Tests

This group of laboratory tests includes the following:

- Determination of the *grading curve*, using sieves and sedimentation (UNE 103101:1995 and UNE 103102:1995),
- *Atterberg limits* tests (UNE 103103:1994 and UNE 103104:1993),
- *Maximum and minimum sand densities* (UNE 103105:1995 and UNE 103106:1993),
- *Unit weight of soil particles* (UNE 103302:1994),
- *Soil chemical analyses*: sulphate, carbonate and organic matter content being the most useful (UNE 103201:1996 -quantitative- or UNE 103202:1995 -qualitative-, UNE 103200:1993 and UNE 103204:1993),
- *Chemical analysis of pore water*.

These tests can be carried out on disturbed or undisturbed samples. In any event, the sample must be previously disaggregated.

The first two tests (grading and Atterberg limits) enable soil types to be classified according to similar geotechnical characteristics. In this respect, use of the widespread Unified Soil Classification System is recommended.

Also considered to belong to this group are *dry density* and *natural moisture content* tests enabling the two most important soil condition variables to be determined. They should be run, however, on undisturbed or slightly disturbed samples.

2.11.2 Unconfined Compression Tests in Soils

These are suitable for testing cohesive soil samples with a medium, firm or very firm consistency that are undisturbed or slightly disturbed and can also be run on recompacted cohesive soils (UNE 103400:1993).

The results provide an accurate idea of the soil's shear strength under similar saturation conditions to those in the test.

The results may not be too accurate in clays showing signs of cracking.

Whenever this test is carried out, it is recommended that the moisture content and dry density values should be specifically determined for each specimen prior to testing.

The unconfined compressive strength of clayey soils can be qualified according to the following scale:

Consistency of clayey soils	Manual tests	Unconfined comprenssive Strength (kN/m^2)
Very soft	Squeezes between fingers when fist is closed	0-25
Soft	Easily moulded by fingers	25-50
Medium	Moulded by strong pressure of the thumb	50-100
Firm	Deformed by strong pressure of the thumb	100-200
Very firm	Indented by thumbnail	200-500
Hard	Indented with difficulty by thumbnail	>500

2.11.3 Direct-shear Tests on Soils, Gravels and Fine Rockfills

This is suitable for any type of cohesive or granular soil sample, undisturbed or otherwise. Preparing test specimens from undisturbed sandy samples is obviously complicated and requires special techniques making it inadvisable.

The direct-shear test can be carried out on semi-saturated specimens, in the same condition of the original sample material, or with additional saturation provoked in the test apparatus.

The test provides an approximate estimation of the shear strength. Strain conditions vary so much in the shear box that little accuracy can be expected in terms of strength parameters. For this reason, it is only advisable to use this test when there is no opportunity to carry out triaxial tests.

Spain has a standardised version of the direct-shear test (UNE 103401:1998).

Large-scale apparatuses are needed to test coarse-grained soils and especially for gravels and fine rockfills (materials for levelling berms). Shear boxes measuring up to 1 m × 1 m are available in Spain.

2.11.4 Soil Triaxial Test

This test is particularly suitable for ascertaining the strength and deformability of soils in different degrees of confinement. It can be carried out on samples of any soil type, disturbed or undisturbed. It is, however, difficult to prepare undisturbed specimens of granular soils.

The test can be run on different specimen sizes . Cylindrical specimens are usually tested, whose height is twice their diameter (UNE 103402:1998). The usual minimum diameter is 1-1/2" (38 mm) and it is relatively normal in Spain to test specimens of up to 9" (229 mm) in diameter if the soil contains gravel with up to 2" (51 mm) in size.

The test is normally carried out with or without prior consolidation and specimens sheared under drained or undrained conditions. The following tests are typical:

- UU Unconsolidated and Undrained failure.
- CU Consolidated and Undrained failure.
- CD Consolidated and Drained failure.

The CU test can be run with or without measurement of pore pressures in the specimen.

The test is normally carried out on previously saturated samples at a 6-bar initial chamber pressure, although the UU test can be carried out on unsaturated specimens.

Three specimens are normally sheared in each triaxial test, each of them subjected to a cell pressure at 0.5, 1 and 3 bars higher than the saturation initial cell pressure. It is possible and advisable to indicate other test pressures that could be more suitable to the problem being investigated.

Deformability is monitored during the vertical loading stage of the test -until failure- and a record made of the load required to reduce the specimen height by each additional 0.5%. It is essential to know these deformation data to be able to deduce the soil deformability.

The interpretation of triaxial tests will provide the soil strength and deformation parameters under undrained conditions (UU tests) or drained conditions (CD tests or else CU tests with pore pressure measurement).

The soil strength and deformation results from triaxial tests can be applied to the study of all geotechnical problems.

2.11.5 Oedometer Tests

Oedometer tests are particularly suitable for studying settlement in saturated soft clayey soils and can be carried out on undisturbed samples of cohesive soils or on recompacted samples of any material.

These tests are normally run by increasing the vertical load in steps in such a way that each new load doubles the vertical compression existing at the preceding step. The test normally reaches a maximum vertical load of 1 MN/m^2, although it is possible to specify greater loads if the problem under analysis should require these. Deformation during unloading is also monitored in the test.

Each load step for the oedometer test lasts for one day. It is recommended that this minimum waiting time should be maintained and it is therefore inadvisable to specify shorter periods when ordering these tests.

Oedometer tests are normally carried out on saturated specimens although it is possible in special cases to run them with less moisture content or to saturate them after having applied a certain pressure. These variations can be useful in studying the collapse or expansion of metastable soils.

The interpretation of oedometer tests makes it possible to deduce the geotechnical parameters relative to soil deformability and permeability that are particularly applicable to the study of consolidation problems.

The Spanish standard UNE 103405:1994 regulates the way these tests are performed.

2.11.6 Compaction Tests

Compaction tests are appropriate for studying the effect of the moisture content on the maximum density that can be reached when soil is compacted. They are carried out on samples of any soil type, even on gravels with a maximum size of about 1" (25 mm).

The most traditional tests are the Standard Proctor (UNE 103500:1994) and the Modified Proctor (UNE 103501:1994). The latter is carried out by compacting the soil in larger moulds using higher energies and therefore tends to achieve significantly greater densities (5 to 15% more than the values corresponding to the Standard Proctor).

The result of these tests is particularly applicable to the quality control of fill compaction.

2.11.7 Permeability

Permeability can be determined in the laboratory by the constant-head method (UNE 103403:1999) or the falling-head method.

Test conditions such as sample size, manner of preparation, hydraulic gradients, etc., must be properly specified, as there are no clearly established standards for all the variants of this type of testing.

The permeability of cohesive soils can be deduced from oedometer tests.

2.11.8 Dynamic Tests

Dynamic tests equipment is available to study the effects of cyclic dynamic loads, chiefly those due to strong earthquakes. This equipment enables appropriate geotechnical parameters to be obtained.

The simple shear test is the most appropriate for studying problems of resistance to cyclic loads (liquefaction).

The resonance column test is the most appropriate for problems of dynamic deformation.

These and other dynamic tests must be specified and interpreted by specialists, as they are still far from being standardised routine tests.

2.11.9 Tests in Rocks

The most common tests specific to rock mechanics include:

- determination of unit weight and absorption,
- unconfined compressive strength (UNE 22950-1:1990), with strain gauges,
- shear strength tests on joints,
- point-load test (UNE 22950-5:1996),
- determination of wave propagation velocity in rock samples,
- triaxial tests in rock (UNE 22950-4:1990),
- permeability tests in hollow cylindrical specimens,

- petrographic examination of thin sections
- the Slake Durability Test for rocks,
- the Los Angeles rock abrasion test (UNE EN 1097-2:1994),
- test of resistance to wetting and drying cycles (NLT 260),
- tests of resistance to sodium or magnesium sulphate solutions,
- Deval abrasion test (UNE EN 1097-1:1997).

2.11.10 Other Laboratory Tests

The tests mentioned in the foregoing sections are far from being a full list of all those possible. A wide range of less common tests exists that can be of great value in studying specific individual geotechnical problems.

Other soil tests worthy of mention include:

- the CBR - determination of the bearing capacity of subgrades and pavement layers,
- determination of the relation between suction and moisture content in soil wetting and drying processes,
- determination of the swelling pressure and the free swelling of expansive soils,
- dispersivity tests using chemical analysis of the cations in the adsorption water of clays: pinhole dispersivity tests - dispersion test by double densitometer,
- Brazilian compression tests (indirect measurement of tensile strength),
- laboratory-run vane and penetrometer tests,
- Rowe cell compression tests (10"/25-cm diameter oedometer).

Special laboratory tests are being carried out with increasing frequency (large specimens, prototypes tested in centrifuges, scale models, etc.), which are of great interest but lie beyond the scope of the problems covered in this ROM 0.5.

2.12 INTENSITY OF GEOTECHNICAL INVESTIGATION

The number of boreholes it is advisable to carry out, their location and length, the extent of the geophysical exploration, the number of *in situ* tests (penetrometers and tests in boreholes), the number of samples to be taken and the tests to be run on them, plus any element defining the geotechnical investigation necessary, must all be decided as a function of the factors listed below.

- *Geotechnical Conditions*

  The ground at the construction site may have more or less favourable conditions for the planned works. Conditions are understood to be *favourable* when there is no concern over ground failure problems (lack of stability or insufficient bearing capacity). Furthermore, whatever problems do arise are commonplace and have been easily solved on earlier occasions and duly verified experience exists for defining the appropriate foundation methods for the type of ground involved.

Geotechnical conditions should be considered *unfavourable* whenever the ground characteristics are expected to prove clearly critical to the project design, considering the structural solution adopted, or whenever the works to be carried out require the use of unusual techniques or procedures on which there is little experience.

Heterogeneity of the ground is one of the aspects that can lead to unfavourable geotechnical conditions. Examples of this would be the ground with distinctly different characteristics at some points from others or presenting localised defects where strength characteristics can be clearly deficient.

Before deciding on the investigation intensity, the expected conditions must be explicitly estimated and qualified as *favourable*, *normal* or *unfavourable*.

- *Nature of the Works*

The intensity of the geotechnical investigation must be chosen after having established the nature of the works. This is defined in the ROM 0.0, which describes the right procedure for obtaining the ERI and SERI ratings.

The ERI (Economic Repercussion Index) attempts to quantify the economic consequences brought about if the works were to undergo ultimate failure.

The SERI (Social and Environmental Repercussion Index) attempts to quantify in non-economic terms the effect on human lives and the environment and the general social impact that total destruction of the works would imply.

As a function of these two indices and for the purpose of deciding on the investigation intensity and other geotechnical design aspects, three categories can be defined: A, B and C, as given in the following table.

Table 2.12.1. **Works Category as a Function of the ERI and SERI Ratings for Defining the Intensity of Geotechnical Investigation**

SERI \ ERI	Low ≤ 5	Medium 5 – 20	High > 20
Minor < 5	C	B	A
Low 5 - 19	B	B	A
High and very high ≥ 20	A	A	A

Comment: Table 4.6 in the ROM 0.0 classifies the nature of the works, which for geotechnical investigations can be grouped into three categories, as per the following correlation:

Category	Nature
C	$r_1\ s_1$
B	$r_2\ s_1$, $r_2\ s_2$ *and* $r_1\ s_2$
A	*all others*

This same grouping will also be valid for deciding on the types of calculation or checking operation suitable for the design.

The procedures for analysing the geotechnical problems can be grouped under two types or classes:

Class 1. Procedures based on general statistical data enabling safety requirements to be set up by using overall or partial safety factors - the Level I methods listed in ROM 0.0.

Class 2. Procedures requiring the explicit use of local and specific statistical data that should be defined in the geotechnical report - these are the Level II and Level III methods described in ROM 0.0.

In all works, safety checks based on the Class-I procedure will subsequently have to be made, but for Category A works, additional Class-II safety verifying methods will also have to be used. Consequently, when the A Category is involved, it will not only be necessary to describe the ground's geotechnical characteristics with proper values for the corresponding data, but also with additional parameters capable of representing their variability. In these cases, in addition to taking into consideration the aspects indicated in the different sections of this Part 2, the indications given in Section 2.14 should specifically be considered.

2.12.1 Types of Investigation

In order to establish criteria to help engineers in deciding on the intensity of the most suitable explorations, three basic types are defined, namely, *detailed*, *smallscale* and *minimum* investigations.

Although the scope of a minimum geotechnical investigation will be specified later on, suffice it to anticipate here that its purpose is to lay down the general ground structure without going into any detail.

At the other extreme, a detailed geotechnical exploration investigates the ground structure and its characteristics in a way that will provide sufficiently accurate knowledge of the geotechnical data necessary for any point of interest affected by the works.

Table 2.12.2 indicates the type of investigation recommended in each case when it comes to drawing up the geotechnical report for a Construction Design whose solution has already been studied.

Tabla 2.12.2. ***Type of Investigation Recommended for a Construction Design***

Works Category	Geotechnical Conditions		
	Unfavourable	Normal	Favourable
A	Detailed	Detailed	Detailed
B	Detailed	Small-scale	Small-scale
C	Detailed	Small-scale	Minimum

The Preliminary Geotechnical Report should have previously analysed the corresponding situation and determined the investigation objectives in such a way that the number of boreholes and their location are already reasonably well determined by the planned geometry of the works to be carried out and by the information existing beforehand.

2.12.2 Number of Exploration Points in Detailed Investigations

The ground should normally be explored with boreholes allowing samples and cores to be extracted, on which appropriate tests will later be run. Continuous penetration tests can also be carried out at specific spots, which allow the resistance to driving to be ascertained and, knowing the type of ground involved, the results to be correctly interpreted.

Execution of the boreholes must be monitored in detail, and the initial results (lithological columns) known as soon as possible, in order to confirm the depths and locations of subsequent boreholes. Generally speaking, it is a good strategy to use well-spaced boreholes to explore the different zones of the works, and to intersperse other boreholes later. The order for executing the explorations should be expressly stated in the investigation plans.

Carrying out geophysical explorations in maritime or port areas is virtually unavoidable as it provides a large volume of data at a reasonable cost. Geophysical programmes provide the opportunity for a very useful ground appraisal for planning borehole locations. It may be advisable to complement the geophysical explorations after having carried out the boreholes in order to extend knowledge of certain details.

Seismic geophysical tests make it possible to be more assured about possible interpolations of the nature of the ground existing between different boreholes -in the orientation of the exploration. Explorations should normally be carried out along the alignments where exploration points (boreholes or penetration tests) are located.

In ground where continuous penetration tests can be carried out, the number of boreholes can be substantially less, provided the nature of the ground passed through is clearly determined as a result of interpolation between nearby boreholes or by some other procedure.

For guidance purposes, some recommendations are given below for some typical cases where it is assumed that the geotechnical investigation is aimed at the Construction Design for a solution with a previously defined geometry and that the ground is not well known but expected to be uniform, i.e., without any localised variations capable of having a significant effect on the works.

If the purpose of the work is to carry out a preliminary study of possible solutions or a feasibility study for a particular works project, it may happen –as stated in the comment in Section 2.4– that no more boreholes are necessary for the geotechnical investigation.

If the ground is well known as a result of preliminary borings and the corresponding tests, the number of boreholes needed can be reduced, provided the site where they were located coincides with the points of interest to the programme required.

No specific recommendations are given on the minimum number of investigation boreholes required when the ground is not homogeneous. The extra boreholes needed in these cases will depend on the particular feature making the ground less homogeneous (faults with a large surface print, irregular weathering zones, riverbeds in the vicinity, karst caverns, erratic soil deposits whose properties have a pronounced effect on the works design, etc.). Generally speaking, if the ground encountered when drilling a particular borehole is substantially different from what would be expected from interpolation between nearby boreholes, so that these differences could induce unacceptable changes in the works planned, then the conclusion to be drawn is that the boring investigation is not sufficient yet. In major works, it may be advisable to space boreholes even less than 5 m apart if, in addition to not being uniform, the conditions are unfavourable.

When the degree of knowledge required for the ground is so great that it calls for boreholes to be spaced as closely as the above example, the type of planned solution should be reconsidered so that it is less sensitive to the local variations in the nature of the ground.

1. *EXTENSIVE WORKS*

Generally speaking, exploration points should be placed along alignments deliberately arranged to make it easier to carry out subsequent geotechnical profiles in the orientations of greatest interest.

The recommended method for general planning is to set up these points on a roughly square plan grid, spaced a maximum of some 30 m apart. If the preliminary data indicate that the variation in the ground characteristics is more pronounced in a specific direction (perpendicular to the coast, for instance) and milder in another substantially orthographic direction, the most suitable grid pattern for the exploration points would be rectangular, with investigation points closer together in the direction of the greatest variation and farther apart in the orthogonal direction.

If ground conditions are clearly unfavourable, it may prove necessary to reduce the spacing to roughly 20 m.

In any event, a minimum of three unaligned boreholes –in plan view– need to be made.

In addition to these investigation aimed at providing general geotechnical knowledge of extensive areas, other specific boreholes will be necessary to obtain concrete information on the subsoil conditions in each part of the works.

2. *CONCENTRATED STRUCTURES*

These structures, the typical case of which could be a graving dry dock, need to undergo a specific investigation with points spaced around 25 m apart.

It is recommended to carry out additional boreholes in places of particular interest such as dewatering wells, major load supports, deep-set galleries, etc.

3. *LINEAR STRUCTURES*

These structures, a typical example of which could be a deep-draught quay, need to have the geotechnical investigation points arranged in three alignments parallel to the coastline, separated by a distance similar to the height of the works and with the centre line running roughly along the most heavily loaded area. For lesser draughts, just one or two alignments may be sufficient (see Fig. 2.12.1).

These points should be spaced around 40 m apart, the separation depending on how favourable or unfavourable the ground conditions are.

4. *LIGHTWEIGHT BUILDINGS OR INSTALLATIONS*

These can be defined as taking up extensions greater than 200 m^2 and not requiring application of major concentrated loads.

The recommendation here is to drill one borehole for at least every 400 m^2 of occupied area.

The smallest number of boreholes will be four, except when the area to be occupied is less than 400 m^2, in which case there could be three boreholes.

With more than three boreholes, some of them could be replaced by an alternative exploration procedure (continuous penetration test, for example).

5. *CUT OR FILL SLOPES*

Prior to drilling boreholes, the location of the analysis profiles -in the direction of possible slides- must be decided on. These profiles should be set about 50 m apart.

At least one boring should be made in every profile in small-scale works and up to three in larger-scale works (see Fig. 2.12.1).

6. *DREDGING*

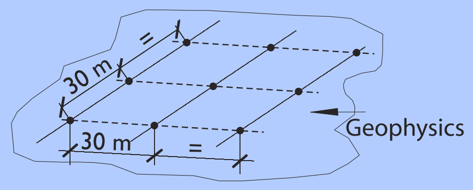
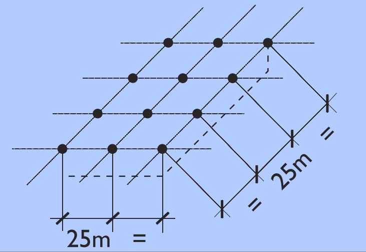
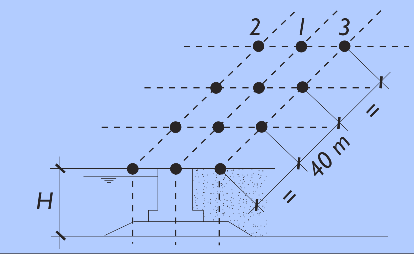
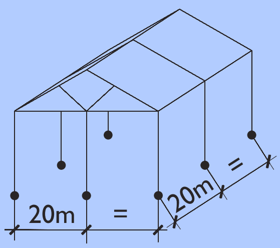
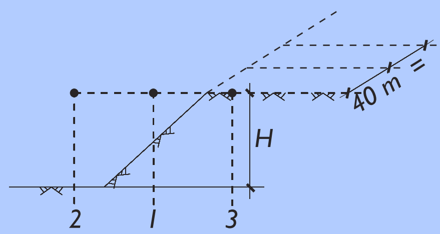
For the design of dredging projects, the investigations should be carried out in vertical alignments or exploration points with the plan arrangement indicated in Point 1.- *Extensive Works*, and with a greater intensity at the edges of the dredging area, as indicated in Point 5.- *Cut or fill slopes*.

In the case of dredging works, in addition to the geotechnical parameters that can be needed for the excavation work (see Section 4.9), it may also be necessary to obtain other parameters associated with environmental impact. This aspect may require a specific investigation programme.

The number of boreholes initially defined in an exploration programme needs to be confirmed, extended or reduced if, as the ground structure becomes known, this turns out to be more or less homogeneous or favourable compared to the earlier estimation.

The number of boreholes can be reduced if, on the other hand, an extensive programme of static penetration tests exists (see footnote to Fig. 2.12.1).

Figure 2.12.1. **Recommended Number of Points for Detailed Investigations (See Text for Clarifications and Complementary Information)**

Works Type	Profile Spacing	Spacing between profiles or points per profile	Diagrams
Extensive areas. Example: fill supporting a pavement	30 m	30 m	
Concentrated structures. Example: graving dry dock	25 m	25 m	
Linear structures. Example: quay	40 m	H (m): ≤ 10 → 1 10 – 15 → 2 > 15 → 3	
Lightweight buildings or installations. Example: warehouse	20 m	20 m	
Cut or fill slopes	40 m	H (m): ≤ 10 → 1 10 a 15 → 2 > 15 → 3	

N.B.: in soft soils, static penetration tests can be used to investigate exploration points even though it is recommended that at least half the points indicated are investigated with boreholes.

2.12.3 Small-scale and Minimum Investigations

When there is reliable and convincing evidence that foundation conditions are not unfavourable and when, in addition, the works are of limited importance (Category B or C as mentioned above), the number of geotechnical exploration points to be recommended is lower than the one indicated in preceding sections.

Small-scale geotechnical investigation can be carried out exploring the ground by means of (seismic) geophysical profiles and boreholes arranged in alignments and/or profiles set farther apart than those indicated in the preceding section, but never spaced farther apart than the distances proposed in Table 2.12.3.

In exceptional circumstances, in minor (Category C) works and in favourable ground conditions, it is recommended to carry out minimum geotechnical investigations with the minimum number of points given in Table 2.12.3.

Table 2.12.3. **Number of Points in Small-scale and Minumum Investigations**

Type of works	Investigations Type	
	Small-scale	Minimum
Extensive areas	50 × 50 m grig	75 × 75 m grid
Concentrated structures	50 × 50 m grig	75 × 75 m grid
Linear structures	One point every 50 m	One point every 100 m
Lightweight building or installations	25 × 25 m grig	40 × 40 m grid
Cut or fill slopes	One point every 50 m	One point every 100 m

2.12.4 The Number of Laboratory Tests

In determining the number of laboratory tests, the first step necessary is to classify the types of subsoil existing. In general, the minimum acceptable definition for particular formation (sand deposit, clay layer, rock bottom, etc.) and a particular aspect requires that at least two laboratory determinations are made of the property governing the aspect in question.

As a general rule, it is recommended that at least two samples should be taken in each different formation through which a particular borehole passes. Spacing between samples in any one borehole should be no more than 5 m, even though the formation appears to be highly homogeneous.

All samples taken in exploratory work must be subjected to the simplest identification tests (grading and plasticity, where appropriate), with just a few of them (one in five and a minimum of two in each formation) being subjected to more comprehensive identification tests (specific weight of particles, chemical analysis and minimum and maximum sand densities).

All undisturbed samples must be tested to determine their dry density and natural moisture content.

In one of the boreholes, it is advisable that sampling be done with a greater intensity and a larger number of tests be performed, so that the ground structure is defined in greater detail in one vertical at least.

Once the results of the identification tests are known, or beforehand if sufficient information exists, the ground passed through should be classified into groups or levels for which the expected strength, deformability and permeability characteristics are similar.

When substantial similarity of characteristics is required, it may well be that too many formations or levels are defined, leading to ground models that are difficult to manage with the calculation methods dealt with below. Classifying the different types of ground into uniform groups is a crucial and difficult task. Engineers must always look for an optimum point in the complexity of the ground layout it is advisable to prepare in order to decide on the strength and permeability tests that will subsequently be run on samples coming from a particular formation.

Depending on the type of problem to be analysed, the predominant tests should be strength tests (triaxial and unconfined compression tests for bearing failure or stability problems), deformation tests (oedometer tests in cohesive soils for estimating settlements and consolidation times) or permeability tests (using permeameters or pumping tests for dewatering problems).

Strength and deformability tests (unconfined compression, direct shear, triaxial and oedometer) must only be carried out on undisturbed or slightly disturbed samples.

The number of strength tests (both drained and undrained, in clays) it is advisable to run on samples from each formation should at least be five units in detailed investigations and three in small-scale investigations. If this aspect is of interest in a particular minimum investigation, two units of these tests could be carried out. These same quantities apply to oedometer tests in soft soils and to permeability tests where these are of interest.

In any event, it is advisable that the precise details of the laboratory test programme to be carried out are fixed only after the results of the first boreholes are known and to complete this detailed definition once the results of all fieldwork are available.

2.13 GEOTECHNICAL REPORT

Two prior documents must exist when a Geotechnical Report is drawn up for a particular purpose (preliminary study, project design, etc.), namely:

- the Preliminary Geotechnical Report
- the Geotechnical Investigation Programme.

In minor works, where geotechnical problems are not anticipated and abundant prior information exists, verified with the appropriate boreholes carried out during the preliminary study stage, the second of these documents may not exist, as additional geotechnical investigation did not prove necessary. In this case, no new documents will need to be prepared and the Preliminary Geotechnical Report will then form the only section of the final Geotechnical Report.

In normal circumstances, when additional exploration work has been carried out, both documents should exist. They form an integral part of the Geotechnical Report and can be issued separately or included as appendices to this final Geotechnical Report.

The Geotechnical Report should contain all the available geotechnical information, arranged in a Main Report plus a set of Annexes to facilitate subsequent consultation.

The detailed information to be included in the Annexes will depend on the amount of data existing. Most typically, the detailed information should be arranged in at least two annexes, as indicated in Subsections 2.13.1 and 2.13.2.

2.13.1 Fieldwork Annex

This Annex should contain information relating to:

- Detailed location of the exploration points and date of execution.
- Lithological columns and photographs of the borehole cores.
- A description of the trenches, trial pits and wells, along with their photographs.
- A description of the location of samples taken and observations on the procedures used in taking and transporting them.

- Detailed results of each of the *in situ* tests.
- Data from underground water table observations.
- Detailed remarks by the specialist in charge of the fieldwork.

2.13.2 Laboratory Testing Annex

This annex should contain detailed information relating to:

- A list of samples with their origin and nature (disturbed, undisturbed, packed in sacks, core barrels, waxed cores, block samples, etc.).
- A report on the opening of the samples and a description of their condition. It is recommended to include photographs taken at the time of their opening.
- A detailed list of the tests carried out.
- Properly laid out test results.

These and any other possible Annexes (geological mapping, geophysical exploration, etc. may be the subject of other Annexes) should contain all the basic information, so that the main section of the Report may be drawn up and read clearly.

2.13.3 Main Section of the Report

The main section of the Geotechnical Report should at least cover the following aspects:

- Purpose of the Geotechnical Report.
- A description of the works or project (or the characteristics of the problem) motivating the Report. If it refers to works to be carried out in the future, its description section should include the location and geometry of the works, the types of structure planned, the materials to be used and an estimate of the main loads involved.
- Fieldwork. This should include chronological references to the different fieldwork operations and describe the equipment used and the personnel who actually carried them out.
- Laboratory work, describing the test procedures used, the types of sample analysed and the quantity of tests of each type run.
- A description of the geological and geotechnical site conditions. This section should include the geological and geomorphological history of the site (backing this up, where applicable, by aerial photographs is recommended), geological mapping, subsoil structure, the soil or rock types which could be involved in the problem under study, the local geotechnical experience, groundwater table observations, etc. This description needs to be supported by enough maps, drawings, geotechnical profiles, photographs and diagrams or drawings to provide a clear explanation of all the details of interest.
- The geotechnical characteristics of the ground. After classifying the different materials explored, the geotechnical characteristics deduced from the laboratory and *in situ* tests should be described for each of them. In general terms, an attempt should be made to determine the causes of variation of the different parameters using graphs, figures and correlations. Diagrams showing the variation of different parameters with depth are of particular interest.

- Comments by the specialist responsible for the Geotechnical Report about the degree to which its objectives have been met, and expressly stating the points where in his/her opinion it will be necessary to extend the investigation further.

If, after due consideration of all available information, doubtful aspects still remain that are inadmissible, the investigation must be extended and the Geotechnical Report accordingly revised or any necessary supplements issued.

2.14 GEOTECHNICAL INVESTIGATION FOR CATEGORY-A WORKS OR PROJECTS

For Category-A works or projects (see Table 2.12.1 and its comments), it is advisable to carry out geotechnical verifications or checks by applying Level II and Level III probabilistic procedures, in addition to the methods based on safety factors. This requires specific local information on the variability of the ground parameters. To obtain this type of information, engineers need to plan ground investigations and analyse their results in an appropriate manner.

2.14.1 Identifying Different Types of Ground

The first step necessary for arranging and analysing geotechnical information is to identify the different types of intervening ground.

In the first place, existing artificial fills should be identified separately and a distinction made between the Quaternary soils and the older materials.

Any rocks found must be classified according to their nature and levels or stretches will be differentiated in each group, according to their degree of fracturing and weathering.

Existing soils will be classified in levels of sufficiently uniform characteristics. As a general rule, two soils (or two stretches or areas of ground) must be considered to differ from each other when any of the following circumstances apply:

a. When the estimated difference in the average value for the tangent of the angle of friction, tan ϕ, exceeds a specific threshold. Generally, differences in the order of 10% in the value of tan ϕ should justify identifying different types of ground.

b. In cohesive soils, two formations should be considered different when the average value of the ratio existing between the undrained shear strength and the vertical effective pressure (s_u/σ'_v) corresponding to each of them differs by more than 10% approximately.

The study of these two key values (ϕ and s_u) requires using, in addition to the specific data obtained from the investigation programme, local correlations (obtained with data from ground types similar to those in the zone under study) with the following state and identification parameters:

- Grading: the correlation existing between the fines content (0.08 UNE sieve) and the angle of friction is especially interesting.

- Plasticity: the correlation of the quotient s_u/σ'_v with the plasticity index, PI, and the overconsolidation ratio, OCR, are of particular interest.

- Dry density, natural moisture content and liquidity index: a clear correlation should exist between these three parameters and the value of the undrained shear strength in cohesive soils.

2.14.2 Common Geotechnical Parameters

For each ground type identified, the geotechnical parameters of interest must be described. These will vary depending on the problem under study and the verification procedure to be used.

Generally, regardless of which ground, problem under study and analysis procedure are involved, the following data must always be stated.

Basic data (identification and state):

- Dry density and natural moisture content
- Grading, in soils.
- Plasticity of the fines fraction
- Petrographic identification of rocks.

For rocks whose behaviour is capable of affecting any limit state, it will also be compulsory to define the following parameters.

Rock characterisation:

- Description of the joints and degree of weathering.
- Unconfined compressive strength of fresh rock.
- Some of the following values related to its deformability:
  - *pressure and shear wave propagation velocity (v_p and v_s)*
  - *pressuremeter (or dilatometer) modulus.*

For soils which can play a significant role in the behaviour of works in respect of Ultimate Limit States, it is advisable to know:

Soil characterisation:

- Mineralogical and chemical identifications of the soil particles.
- Undrained shear strength, s_u, in cohesive soils.
- Drained shear strength - parameters of the Mohr-Coulomb model c' ϕ'.
- Deformation parameters in accordance with one of the following models:
  - Elastic: elasticity modulus and Poisson's ratio, which may vary during the loading process.
  - Oedometric: specifying at least the following five data:

e_o = initial void ratio
p_c = preconsolidation pressure
C_c, C_s = compression and swelling indices
c_v = consolidation coefficient.

Strength data can be replaced, with due justification, by indirect data such as:

- N-index (SPT)
- resistance in static penetration tests (q_c and q_f)
- limit pressure in pressuremeter tests
- continuous dynamic penetration tests.

Deformability data can be obtained indirectly, equally with due reasoning, by means of known correlations applicable to the particular case in question.

2.14.3 Variability of Geotechnical Parameters

For each geotechnical parameter X, the most representative value must be defined, which will generally be an estimation of the mean value X_m.

For each level or stratum, or soil or rock zone differentiated, it must be indicated not only this representative value, but also the expected variation range. The limits of the possible variation range for the parameter in question must be established in such a way that the probability of the parameter value falling outside this interval will be very low (nominally in the order of one in a thousand).

In order to establish certain probabilistic models for the variability of the ground data, it is vital that at least a sufficiently accurate value for the variation coefficient is included (12).

The variation coefficient of the ground parameters depends on the extent of the zone in question. In the area surrounding the tip of a pile, the variation coefficient will not result the same as that for the whole of the ground involved in the overall stability of the works. The variation coefficients given in the geotechnical report must be assigned to specific zones of the ground.

The variation coefficients must only be assigned to the geotechnical parameters that are to be used directly in the calculations. This is not necessary for the auxiliary parameters such as grading, plasticity, mineralogical data, etc.

The variation coefficient should be determined via statistical analysis of the results. As a general indication, the following approximate value is given for the variation coefficient:

$$v = \frac{\text{Maximum estimated value - Minimum estimated value}}{n \times \text{Mean value}}$$

where n is an dimensionless number that can vary, depending on the cases, from 4 to 6 (see further details in Subsection 3.3.10).

(12) For a statistical sample containing «n» data X_i, the two most basic determinations thaat can be made are the ones that lead to a mean value X_m and to a standard deviation σ_x defined by the following expressions:

$$X_m = \frac{1}{n}\sum_1^n X_i$$

$$\sigma_x = \left\{\frac{1}{n-1}\sum_1^n \left(X_i - X_m\right)^2\right\}^{1/2}$$

The variation coefficient «v» is defined as the following quotient:

$$v = \frac{\sigma_x}{X_m}$$

2.14.4 Recommended Probability Functions

To do calculations using Class 2 methods (i.e., those of Levels II or III - see Section 2.12), the geotechnical parameters entered in the calculations need to be defined through a probabilistic function.

This function may vary depending on the type of parameter involved. Engineers should investigate, for each specific case, which distribution type best represents the variability observed.

In general, although normal distributions are easy to use, they should be avoided, as their tail shape is far from being even approximately representative of the natural variability of most geotechnical parameters. And the tail shape is essential in probabilistic calculations, particularly those corresponding to works with a high SERI rating.

One simple law used relatively often is the log-normal. It is defined by the average value λ and the standard deviation ζ of the logarithm of the parameter X in question, according to the following expressions:

Mean value of logarithm $$\lambda = \ln X_m - \frac{1}{2}\zeta^2$$

Standard deviation of logarithm $$\zeta = \sqrt{\ln\left(1 + v^2\right)}$$

where X_m and v are the mean value and variation coefficient of X, as defined in 2.14.3.

Whenever specific information exists, engineers should propose the variation laws most in line with the variability of each parameter.

Table 2.14.1 gives typical values for some variation coefficients for guidance purposes. It also shows the usual unit of measurement for each.

Table 2.14.1. Recommended Reference Units and Approximate Values for the Variation Coefficient of Geotechnical Parameters for Normally Homogeneous Ground

Parameter	Symbol	Recommended reference unit	Typical coefficient of variation
Unit weight	γ_d	kN/m³	0.05
Moisture content	w	%	0.10
Angle of friction (tangent)	tg ϕ	Dimensionless	0.07
Cohesion	c	kPa	0.10
Undrained shear strength	c_u, s_u	kPa	0.15
Unconfined compressive strength, soils	q_u	MPa	0.15
Unconfined compressive strength, rocks	R_c	MPa	0.20
Tip resistance in static penetration tests	q_c	MPa	0.15
Limit pressure, pressuremeter tests	p_l	MPa	0.15
N-index (SPT)	N	Dimensionless	0.15
Elasticity modulus	E	MPa	0.30
Poisson's ratio	ν	Dimensionless	0.05
Preconsolidation pressure	p_c	MPa	0.15
Compression and swelling indices	C_c, C_s	Dimensionless	0.10
Consolidation coefficient	c_v	cm²/s	0.50
Permeability coefficient (Darcy)	k	cm/s	*

2.15 COST OF GROUND INVESTIGATIONS

The negative effects that may arise during the construction of works as a consequence of poor knowledge of the ground are substantial in some instances. Some situations may involve increased costs and/or longer completion date that are difficult to take on and may even signify the total failure of the works if the problem is not detected in time.

The fact that the ROM Programme admits explicit failure probabilities with rather high values should not condone, far from it in fact, any failures brought about as a result of lack of knowledge of the ground. These failure probabilities will always be controlled and due to any other cause but never the result of a knowledge of the ground of poorer quality than the one indicated in this Part 2 of the ROM 0.5 publication.

The cost of investigations should be compared against the benefits they provide. The usual costs of geotechnical investigation programmes account for a noticeable fraction of the total investment (construction of the works for which the investigation is done). This proportion varies and can range from 0.5% for investigations in berthing works on good quality ground up to over 3% in extremely costly programmes for breakwaters, requiring the aid of floating facilities. These costs should be considered as part of the investment; in fact as one more item assigned to execution of the works in question.

APPENDIX I. FORMULAE FOR INTERPRETING PERMEABILITY TESTS IN WELLS AND BOREHOLES

A1.1 Interpreting Permeability Tests in Boreholes

A1.1.1 Lefranc Tests

The most common permeability tests in boreholes consist in monitoring the water flow injected into the ground under certain geometrical conditions of the area of contact between the free water and the surrounding ground and certain hydraulic head conditions.

In all these tests, which are three-dimensional, the flow rate will be given by an expression of the type:

$$Q = \Delta\phi \cdot K \cdot n$$

where:

- Q = water flow rate needed to maintain a constant water level in the borehole.
- $\Delta\phi$ = difference in potential between the inside and outside of the borehole.
- K = unknown permeability.
- n = shape factor.

The *n* coefficient has dimensions of length.

Values for the *n* coefficient in some typical cases are given in Figure A1.1.

A1.1.2 Lugeon Tests

Lugeon tests are exclusively designed for hard rocks in which permeability is measured in Lugeon units, LU. A Lugeon unit is one that allows the passage of a one-litre-per-minute flow rate for each linear metre of borehole when the test pressure is 10 bar. Only certain types of very resistant rocks allow such high pressures without fracturing.

The following theoretical correlation can be used to estimate the equivalent permeability in porous media:

$$1 \text{ LU} \cong 1.3 \times 10^{-5} \text{ cm/s}$$

However, this correlation can be weak and must be verified if the specific application in which it will be used is affected by this dispersion.

A1.1.3 Falling-Head Tests

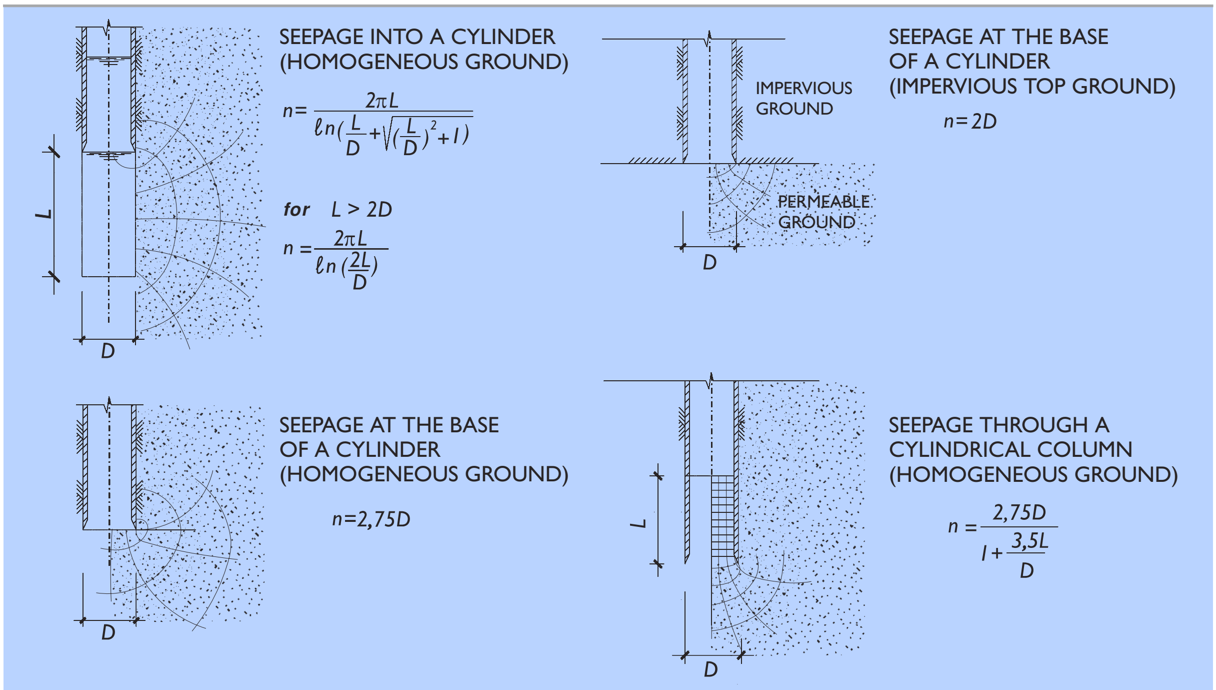
Falling-head tests, in which the head difference between the water level in a borehole and the groundwater table is monitored, can be interpreted using the expression:

$$K = \frac{\pi\ D_o^2}{4(t_2 - t_1)} \frac{1}{n} \ln\left(\frac{\Delta H_1}{\Delta H_2}\right)$$

where:

D_o	=	internal diameter of the casing in the area where the water level varies.
t_1, t_2	=	the time at which measurements ΔH_1 and ΔH_2 are taken.
n	=	the shape factor of the contact area between the free water inside the borehole and the ground (see Figure A1.1)
$\Delta H_1, \Delta H_2$	=	the elevation of the free water surface inside the borehole relative to the groundwater table at times t_1 and t_2.

Figure A1.1. **Shape Factors in the Lefranc Test**



Details on carrying out the tests, such as cleaning out the bottom of boreholes, the stability of their walls, plugging the cased part of the borehole, etc. have or can have a serious effect on the results. Permeability determination therefore always provides imprecise results.

A1.2 Interpreting Pumping Tests

Specialist technical assistance in the field of well hydraulics is desirable for studying soil permeability by pumping tests or to design a set of wells for lowering the water table.

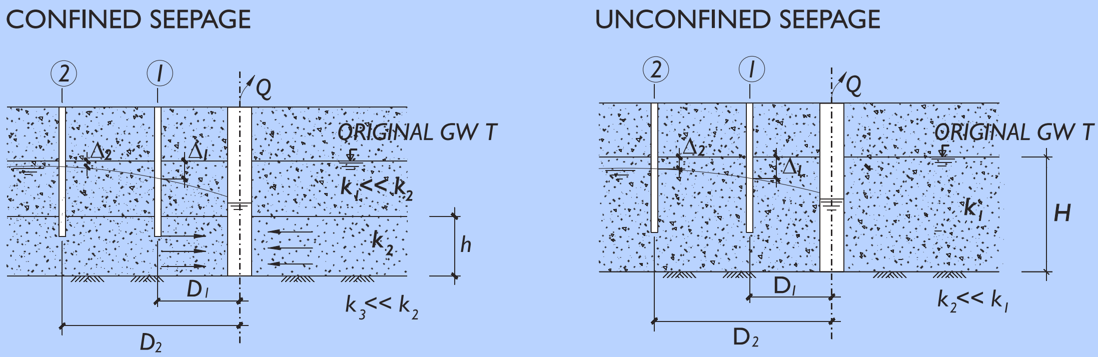
For solving simple matters of limited importance, this ROM 0.5 includes the most widely used basic formulae.

A1.2.1 Steady State Tests

It is neither common nor recommended practice to extend pumping tests until a steady state is achieved, as they are then difficult to interpret. Their results are affected by distant boundary conditions that are often largely unknown and they can also require long testing times.

In any event, if drops in the groundwater table are measured at least two points away from the well and the permanent discharge causing them is known, permeability can be estimated using the expressions indicated in Figure A1.2, depending on whether the aquifer is confined or unconfined and whether the well reaches a deep impervious stratum or not. The specialist technical literature gives solutions for other more complex configurations.

Figure A1.2. Steady State Pumping Tests



$$k_2 = \frac{Q \ \ell n\left(\frac{D_2}{D_1}\right)}{2\pi h\,(\Delta_1 - \Delta_2)}$$

$$k_1 = \frac{Q \ \ell n\left(\frac{D_2}{D_1}\right)}{2\pi H^*(\Delta_1 - \Delta_2)} \qquad H^* = H - \frac{1}{2}(\Delta_1 - \Delta_2)$$

A1.2.2. Transient State Tests

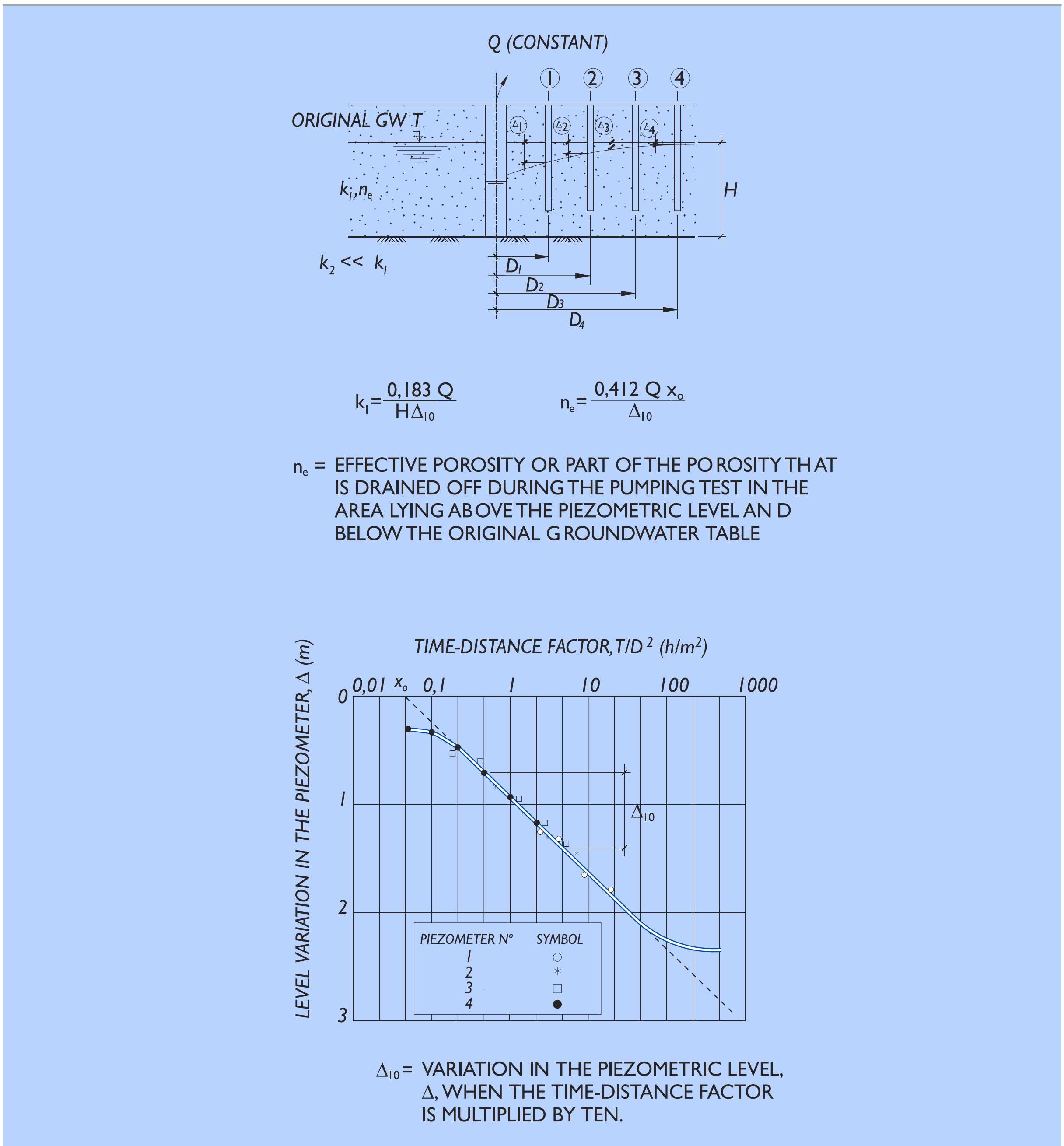
It is normal and recommendable practice to carry out transient pumping tests in such a way that the evolution of drops in the underground water level away from the well is monitored as a function of time, when a constant discharge *Q* is pumped from the well.

Practice shows that in many cases there is one way of representing the drops in underground water level that subsequently permits a simple test interpretation. This method is shown in Figure A1.3.

The following factor is shown on the x-axis on a logarithmic scale:

$$x = \frac{T}{D^2}$$

Figure A1.3. **Diagram of the Transient State Pumping Test**



where:

T = time elapsed since start of the test.
D = distance from the point to the well axis.

The drops recorded are shown downwards on the y-axis on a 1:1 scale.

The straight line best fitting the different time and piezometer data enables a value for the theoretical abscissa x_o and the slope Δ_{10} to be defined, which are related to the effective porosity of the medium and to the permeability as shown in the figure.

A1.2.3 Inverting the Pumping Test

It is common practice, and also one to be recommended, to monitor how water rises in wells once pumping has stopped. Interpreting these level rises enables new data to be estimated in each piezometer *i* and at each moment of time *T'*, taken from the time when pumping at discharge rate *Q* ceases. This estimation can be made by subtracting the drop that would have occurred if pumping had continued from the actual drop measured. The latter can be obtained by prolonging by guesswork the records of drops observed during pumping or by using the correlation from the logarithmic graph (see Figure A1.3) in order to extrapolate data over longer periods of time.

The new set of data enables *K* and n_e to be evaluated again using the same formulae mentioned above, as the end of pumping can be interpreted as the superimposition of a flow rate -*Q* onto the earlier state of pumping discharge Q.

A1.2.4 Water Level inside Wells

In any pumping test, it is not considered appropriate to use the water level inside the well as an input for the «piezometric level», as there is generally a seepage face (hence the difference in elevation between the groundwater table and the level of water inside the well) which is difficult to monitor.

Part III

Geotechnical Criteria

GEOTECHNICAL CRITERIA

Part III

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3.1 GENERAL ASPECTS

The release of the ROM 0.0 Publication, which lays down the general procedure and the calculation bases for the design of maritime and port works, prompted the adaptation of the geotechnical criteria in this ROM 0.5 in order to tailor them to the general procedure and adapt them to the calculation bases recommended in ROM 0.0.

Before the solution to a particular problem is defined, it is essential, as stated in ROM 0.0, to determine the most appropriate failure probabilities as a function of the nature of the works (or a section thereof). In any event, *Puertos del Estado* has established some maximum limits for failure probabilities in ROM 0.0.

Part 3 of this ROM 0.5 first takes into consideration the criteria for assigning maximum individual failure probabilities during the Design Stage under study for each of the geotechnical failure modes that may occur in maritime and harbour works, which correspond to Ultimate, Serviceability and Operational-Stoppage Limit States, so as not to surpass the maximum combined probabilities required in ROM 0.0.

The probability of geotechnical failure is one more to be combined within the overall set of possible failures and, as a result, must be clearly lower than the limit stated in ROM 0.0. As a general rule and except for the occasional specific case where probabilities of geotechnical failure can be established more precisely, this ROM 0.5 has opted for establishing clearly lower failure probabilities than those indicated in ROM 0.0 so that their contribution to the failure probability as a whole can therefore be neglected, thereby simplifying the failure diagram and calculations for the overall failure probability of the works.

Subsequently and as a consequence of the foregoing, ROM 0.5 describes and recommends procedures for verifying the reliability, serviceability and operability demanded of maritime and harbour works and their sections in the various design states against the different modes of geotechnical failures that can arise. Among others, procedures based on calculations are dealt with here, differentiating between Levels I, II and III, but fundamentally developing Level I procedures.

This Part 3, in addition, includes well known and easy-to-use solutions for common calculation problems in soil mechanics and which –to a certain extent– apply to all types of works. The majority of these calculation methods take into account the sea-level oscillations by considering static equivalences of water levels as acceptable. They are not applicable when the cyclic or impulsive nature of these sea-level oscillations (waves, tides, etc.) has a substantial effect on the behaviour of the ground, and also when they play a predominant role in triggering the failure mode.

While the specific ROM publications are being prepared, the end section of Part 3 includes some general recommendations on ground behaviour under dynamic, cyclic or impulsive loading agents. Consequently, this comprises a summary preview of the matter of ground under seismic loads. Other publications in the ROM Programme, related to the different types of maritime and harbour works, develop these aspects in greater depth.

3.2 SAFETY, SERVICEABILITY AND OPERABILITY CRITERIA

The stress-strain behaviour of the ground under the effects of acting agents can constitute the predominant factor in some failure modes affecting maritime and harbour works and therefore have a significant effect on their safety and serviceability. The geotechnical design should be capable, with certain guarantees, of preventing the failure of the ground and, in general, any anomalous behaviour of the works caused by this.

The basic elements of the geotechnical design are:

- geotechnical investigation.
- selection of types of solution.

- definition of the construction process.
- study of the solution details.
- design calculations.
- behaviour monitoring.

All these elements are interrelated, as the decisions made on any one of them will affect all the others.

Failures in maritime and port works may be the result of a partial defect in one of these activities. There are obvious negative consequences to an incorrect general conception of the works (inappropriate typology) or of a defective geotechnical investigation.

Engineers should prevent failures, not only by thoroughly investigating the ground and selecting appropriate construction solutions but also by taking care over details, choosing appropriate calculation or experimental models and observing the behaviour of the works during their construction and exploitation.

It is difficult to standardise the conceptual process necessary to guide engineers through each of these tasks. Nevertheless, this ROM 0.5 attempts to do it and to this end it includes a large number of recommendations which should lead to sufficiently safe and serviceable works against so-called geotechnical failure modes, in which the ground is the agent bearing a predominant influence.

The conceptual path directing the revision of the ROM 0.5-94 is stated in ROM 0.0. This is the source of the basic safety, serviceability and operability criteria that must be followed in geotechnical design. For specific works as a whole or a section of them, these concepts can be defined as:

Reliability: complementary value of the joint failure probability during the Design Stage for all the principal modes assigned to Ultimate Limit States.

Serviceability: complementary value of the joint failure probability in the Design Stage for all the principal modes assigned to Limit States of Serviceability or utilisation.

Operability: the complementary value of the probability of stoppage in the Design Stage for all the principal stoppage modes assigned to the Limit State of Operational Stoppage. The operational failure mode differs from the serviceability failure mode in that, once the agent (generally climate-related) causing the operational stoppage ceases to exist, the works and their installations go back to operation, meeting the requirements specified in the design. On the contrary, a serviceability failure persists and service is only recovered when the works are repaired or rebuilt.

3.2.1 Reliability against Ultimate Limit States

It is possible to measure the safety of works (or sections thereof, as defined in ROM 0.0) during a Design Stage, if only on a conceptual level. This can be done with the failure probability for the full set of possible principal failure modes associated with the Ultimate Limit States that could arise throughout the Design Stage being analysed. With this measuring parameter, the reliability of the works would be the complement to 1 of the failure probability.

Reliability is measured with the *reliability index*, β, also known as the *confidence index*, and which is biunivocally related with the failure probability p:

$$\beta = -\phi^{-1}(p) \qquad \text{or} \qquad p = \phi(-\beta)$$

where ϕ is the normalised standard cumulative probability function.

The minimum safety required for maritime and harbour works in each of their Design Stages is a function of their importance, defined through the works general nature, which is established as a function of the economic repercussion index (ERI) and the social and environmental repercussion index (SERI) ratings. These indexes are indicators of the economic, social and environmental repercussions generated in the event of destruction of the works (see ROM 0.0). In this respect, safety should be greater when the social or environmental consequences of failure become serious (high or very high SERI rating). In these cases, engineers must take all the precautions necessary to prevent possible damage. The failure probability indicated here (10^{-4} over design working life) is consistent with the specifications given in other design codes. This probability is purely a formal reference necessary for arranging the design in an orderly manner. As stated above, failure should not happen. In many maritime works, the damage tends to be strictly economic (minor or low SERI rating), in which case thought should be given to optimising the design by applying a minimum generalised cost theory. In this type of works (involving strictly material damage), a specific reliability needs to be set up, calculated for each particular case, although it should not be lower than the value recommended in ROM 0.0 as a function of the applicable SERI value. The method for defining the corresponding optimisation process lies beyond the scope of this ROM 0.5.

Having made whatever economic optimisation calculations prove necessary and having obtained the optimum reliability to be assigned to a particular design, it may happen that these reliability values are much lower (or the failure probabilities much higher) than the indices normally accepted in other branches of civil engineering.

It is also clear that the opposite could happen - the optimisation studies for the works may make it advisable to design safer and more serviceable works than the minimum thresholds recommended, as this increased works reliability is achieved at moderate economic costs. For this reason, geotechnical engineers are advised to make an estimate for each individual case. It may prove of interest to design safer and more serviceable works, with much lower failure probabilities than the recommended ones.

Finally, it must be stated that there exist codes and standards clearly of higher range than the recommendations given in the ROM Programme. Compliance with them is mandatory in order to verify the safety of certain failure modes, except where explicit and irrefutable justification to the contrary is given. Among these, special mention must be made of the EHE code for the design and construction of concrete works and the European standards once they have become mandatory (Eurocodes).

In any event and following the indications given in ROM 0.0, the joint failure probability during the Design Stage analysed for maritime and harbour works should be lower than the threshold values laid down as a function of the social and environmental impact (SERI rating) of the failure.

ROM 0.0 should be consulted for the definition of the SERI rating, which increases with the social and environmental consequences of the failure, as also the threshold values recommended for reliability and failure probability as a function of this index. That publication also explains the concept of *design working life* and the procedure required to define it quantitatively through the ERI rating.

ROM 0.0 has provisions regarding the sensitivity of the joint failure probability to failure modes whose occurrence probability is drastically reduced by small modifications to the geometry of the works or of some of its elements and whose economic repercussions are not significant. In line with them, as a general simplified rule, this ROM 0.5 considers that, in the majority of maritime and harbour works where the sea-level oscillations are not the predominant agent triggering the geotechnical failure modes, these failures (which only make up one part of the several contributing to overall failure probability) admit substantially lower individual occurrence probabilities (even a different order of magnitude) than those indicated in ROM 0.0 for the combined set of failures, without this having a significant effect on the economic optimisation of the works. In such cases and whenever this criterion is adopted, it is possible not to consider these geotechnical failure modes as principal failures and, consequently, to neglect their contribution in calculating the overall failure probability, thereby enormously simplifying the failure diagrams and the calculation of the works joint failure probability.

For these purposes, the individual failure probabilities corresponding to geotechnical principal failure modes associated with Level-I verification methods generally adopted in this ROM 0.5 are in the order of the following:

Table 3.2.1. Individual Failure Probability Associated with Level I Verification methods Adopted in this ROM 0.5 (Ultimate Limit States. Geotechnical Failure Modes)

SERI	Minor < 5	Low 5 - 19	High or very high ≥ 20
Maximum probability over the design working life	10^{-2}	10^{-3}	10^{-4}
Reliability or confidence index	2.33	3.09	3.72

In cases where it is very difficult to achieve these increases in works reliability associated with geotechnical failure modes or this approach is far from proving to be economically viable and therefore the individual failure probabilities considered cannot be low enough or the sea-level oscillations are the predominant agent triggering the failure (e.g., vertical face or reflecting breakwaters, short-duration design states or situations, such as construction stages, etc.), the geotechnical failure modes must be considered as principal failure modes and it will not be possible to neglect their contribution to the overall failure probability. These cases are pointed out in Part 4 of this ROM 0.5, as also in other ROM publications, along with an indication of the value of the individual failure probability associated with the Level I verification method adopted (see Subsection 4.7.3.).

3.2.2 Serviceability. Limit States of Serviceability

The ground, due to excessive displacements or deformations induced in the works or any movements in its own mass, may cause a reduction or loss of serviceability, capable of affecting the possibilities of normal utilisation or operation of the works as a whole or a section of them, thereby causing them to fail to fulfil the service requirements (Limit States of Serviceability). One example is excessive settlement preventing the correct operation of a gantry crane.

The serviceability of particular works (or sections thereof, as defined in ROM 0.0) in a Design Stage can be measured using the parameter of the failure probability for the set of all the possible principal failure modes associated with Limit States of Serviceability (ground-provoked or not) that may occur throughout the Design Stage under analysis. In this manner, the serviceability of the works would be the complement to 1 of this failure probability.

The minimum serviceability required for maritime and harbour works in each of the Design Stages is a function of their importance as defined by their general nature, established as a function of the ERI and the SERI ratings, in the same way as is for reliability. In this respect, greater serviceability is required when important social and environmental consequences arise from serviceability failures (high SERI rating). Nevertheless, in many maritime works the consequences of serviceability failure tend to be strictly economic (medium or low SERI) and to carry out an economic optimisation study is then recommended in order to obtain the optimum serviceability corresponding to each project.

A specific serviceability level should be established in these works, calculated for each individual case. However, it is not advisable to adopt a value lower than the level set in ROM 0.0 as a function of the applicable SERI rating. The method for defining the corresponding optimisation process lies beyond the scope of this ROM 0.5.

In the absence of more specific studies, it is recommended to limit the individual probability associated with principal serviceability failures caused by the ground in the Design Stage under analysis to the following values (provided more than one failure mode is contemplated in the failure diagram and therefore in the joint probability of serviceability failure):

Table 3.2.2. Maximum Recommended Individual Failure Probability Associated with Level I Verification methods (Limit States of Serviceability. Geotechnical Failure Modes)

SERI	Minor < 5	Low 5 - 19	High or very high ≥ 20
Maximum failure probability over the working life	0.10	0.07	0.05
Reliability or confidence index β	1.28	1.48	1.65

3.2.3 Operability. Limit States of Operational Stoppage

The operability of particular works (or a section of them) during a Design Stage can be measured by means of the stoppage probability in a Design Stage for the set of all the possible principal stoppage failure modes assigned to Limit States of Operational Stoppage. The term *operational stoppage mode* is defined as the reason causing the works or a section thereof to cease to be operational or their service level to be reduced, and which return to their normal operation once the reason for the stoppage ceases to exist.

Consequently, since operational stoppage modes cannot be directly caused by the ground, as it does not behave as an agent in these cases, ground-caused operational stoppage modes should not be taken into account.

In this respect and to illustrate the matter further, an operational failure mode can consist in exceeding the drainage capacity (flooding) of an installation. This operational failure is caused by rain as climatic agent. Even though a ground parameter such as permeability could have an influence, the probability of occurrence of this operational failure mode is directly linked to the exceedance probability of the climatic agent causing it (rainfall).

In any event, design engineers should attempt, wherever possible and necessary, to make the solutions utilised in the geotechnical design contribute to reducing the effects caused by the presence of the agent originating the operational stoppage and, consequently, increasing the level of the agent causing the operational failure under analysis, thereby generating an increased operability.

3.3 DESIGN CALCULATIONS

In order to justify that particular works or sections thereof are sufficiently reliable and serviceable from the geotechnical point of view, the following paths can be used:

- **a.** comparison with similar experiences.
- **b.** well-proven construction measures.
- **c.** large-scale field tests and/or laboratory models.
- **d.** observational method.
- **e.** design calculations.

Design measures and details are normally justified by calculations, except where some other sufficiently reliable procedure can ensure that the behaviour of the works will be correct.

The calculations used to verify designs included in the scope of the ROM Programme should also follow, whenever possible, the general analysis procedure known as the *Limit State method*. This method is laid down in ROM 0.0 and knowledge of it is recommended for all engineers in charge of verifying the reliability or serviceability of works.

Works are considered safe and serviceable when there is sufficiently low probability during any Design Stage that any of the sets of ultimate or serviceability limit states composing the failure diagrams will occur.

3.3.1 Ultimate Limit States (ULS)

Ultimate Limit States (ULS) render the works useless as a result of structural failure or collapse of the works as a whole or a section of them. For the purpose of arranging the geotechnical calculations in an orderly fashion, it is a good idea to classify the possible failure modes assigned to Ultimate Limit States into the groups listed below.

EQU. *Loss of Static Equilibrium*. The structure loses its stability conditions as a result of an excessive action without the strength of the structure materials or the ground playing a substantial role in the process. A typical failure mode leading to this state is the so-called "rigid-body overturning" covered in this ROM 0.5.

STR. *Failure of a Structural Foundation Element*. *Structural foundation elements* are defined as the parts of the structure in direct contact with the ground. These elements may fail in different ways (punching shear, flexural bending, etc.) in which the strength of the material plays a predominant role and the ground strength a secondary role.

GEO. *Ground Failure*. This is a failure in which ground strength plays a crucial role. The bearing failure of a shallow or deep foundation is an example of the failure mode assigned to this type of ULS.

UPL. *Failure Caused by Excessive Uplift*. This is a type of failure caused by excessive hydrostatic or hydrodynamic pressure in which the mechanical strength of the ground and the structure play a secondary role. An example of a failure mode assigned to this type of ULS would be the buoyancy of the base of a dry dock with a drained base slab as a result of failure in the dewatering system.

HYD. *Failure Caused by Hydraulic Gradients in the Ground*. Water seepage can give rise to particle transport (internal erosion) leading to the collapse of maritime works. An example of the failure mode assigned to this type of ULS would be the loss of fines from the core of a rubble-mound breakwater during a storm.

Classifying each failure mode corresponding to an Ultimate Limit State that could occur in works or sections thereof into one of the five groups is a relevant exercise because this ROM 0.5 recommends specific calculation procedures for each of them.

These five groups are a further development of the groups included in a general way in ROM 0.0, so as to placing greater emphasis on geotechnical failure modes. Thus, groups EQU and UPL correspond to an Ultimate Limit State of loss of static equilibrium, STR and GEO to a strength Limit State and HYD to a Ultimate Limit State of deformation.

3.3.2 Serviceability Limit States (SLS)

Serviceability Limit States (SLS) produce reversible or irreversible loss of serviceability in the works or a section of them owing to structural, aesthetic, environmental failures or legal compliance issues. All the failure modes reducing or conditioning the use and operation of the works or capable of shortening its working life are considered Limit States of Serviceability.

The serviceability failures capable of causing works to stop meeting the service requirements laid down in the design provisions –even though they may not immediately render the works useless– must be identified for each specific case.

For the purpose of arranging the geotechnical calculations, the possible serviceability failure modes assigned to SLS can be classified into one of the following groups:

EGD. *Excessive Ground Deformation*. These serviceability failures are produced by very large ground displacements or deformations that either directly –or induced in the facilities or structural elements resting on the ground or located in the vicinity– prevent the works as a whole or a section of them from operating properly. Settlement in the rolling path of a crane is an example of this failure mode.

SGA. *Significant Geometrical Alterations*. This refers to situations experienced by works or their elements where alterations, deposits or erosion produced in the ground generate cumulative geometrical changes which prevent the service requirements being met. An example of this failure mode could be the progressive loss of draught of a navigation channel owing to the instability of its banks or excessive deformation in a quay due to erosion or scour of the ground.

AES. *Aesthetic*. These are failures produced by ground actions affecting the aesthetic appearance of works. Loss of alignment of the capping beam or loss of verticality in a dock constitute examples of this failure mode.

SEP. *Failures Caused by Water Seepage*. Water seepage or the effect of sea-level oscillations on the ground can produce effects on works serviceability. Settlement induced on platforms as a result of fines washout in fills and excessive seepage discharge in dewatering are examples of this serviceability failure mode.

These groups go beyond the classification done in ROM 0.0 and are intended to highlight geotechnical failure modes. Consequently, Groups EGD and SEP would correspond to a Serviceability Limit State of excessive deformations, SGA to a Limit State of Serviceability due to cumulative geometrical alterations and AES to a Limit State of Serviceability related to aesthetic, environmental or legal issues.

3.3.3 Operational Stoppage Limit States (OSLS)

Operational Stoppage Limit States (OSLS) are situations in which operation is temporarily reduced or suspended as a result of causes independent of the works or their installations, without any structural damage in them. Once the cause ceases to exist, the works and their installations totally recover the operating requirements laid down in the design.

The potential contribution of the ground to the occurrence of any operational stoppage failure mode can be considered irrelevant, as the ground does not normally behave like an agent causing a temporary and self-rectifying operational stoppage.

3.3.4 Calculation Methods

The Limit States analysis methods are used to check whether, during each Design Stage, works or sections thereof meet the minimum safety, serviceability and utilisation and exploitation requirements stipulated in this ROM 0.5, ROM 0.0 and in any other applicable regulations. To this end, it is necessary to verify the design against all the failure modes capable of arising in each Limit State, and to assess the probability of occurrence for each of them during the Stage under analysis and the joint probability of presentation of all the principal modes, so that they do not exceed the values recommended.

Having identified a particular limit state and failure mode, there are several types or classes of calculation procedures that can be used to quantify the reliability and serviceability of the works under such an event. Part 3 of this ROM 0.5 describes some of the most common approaches, whose application is recommended, based on establishing a verifying equation, as also the criteria for passing the verification. These are the most common calculation procedures.

It should not be forgotten that there are failure modes that are not susceptible to quantitative analysis (design by calculation) and in this cases only the engineer's good judgement, duly endorsed by experience, will indicate whether sufficient arrangements have been adopted to prevent them. Failure modes will also exist that do not need to be analysed in quantitative terms, as they can be avoided by adopting solutions widely approved in practice. In these cases, the occurrence probability of this failure mode is very low, so it is classed as a non-principal failure mode, and therefore its intervention is not taken into account for calculating the joint failure probability.

The design approach based on monitoring the works during their construction (known as the Observational Method) may prove useful in circumstances in which the abovementioned methods (calculations or recommendations) fail to offer sufficient guarantees.

Finally, it is worth pointing out the methods based on nearly full-scale tests or on the study of physical models of the works. In these cases and in order to accept the verifications and to calculate the probability of presentation of one or all of the failure modes, the experimental results should be analysed using probability models.

All the calculations for justifying safety, serviceability and operability of works or sections thereof share a common philosophy. These ideas are described below so that all studies related to the same works are carried out on a uniform basis.

For the failure modes capable of being verified using a particular calculation method, this ROM 0.5 provides a calculation procedure allowing the failure condition corresponding to each case to be determined.

A check equation is established for each failure mode assigned to a limit state, be it ultimate or serviceability. The check equation is an equation of state, consequently the design factors intervening in it may take on nominal values or be statistical variables depending on the level of the verification method used.

The check equation, $g(x_1, x_2, x_3, \dots x_n) = 0$, defines the failure condition as a function of all the design factors intervening in the calculation. In the case of geotechnical failure modes, these variables are: the works and the ground geometry, the ground parameters and the loading agents and loads.

Comment: The verifying equations for the commonest geotechnical problems are usually expressed in the form of safety factors, F, *as follows:*

$$F = \frac{T_f(x_1, x_2 \dots\dots x_n)}{T_d(x_1, x_2 \dots\dots x_n)}$$

where T_f *and* T_d *are expressions grouping together the resistances (or favourable terms) and the loads (or unfavourable terms) respectively. Sometimes (overall stability problems, for instance), the factors* x_i *are a function of* F *and the check equation is then non-explicit. Its solution requires an iterative procedure for calculating* F.

It may sometimes be advisable to express the check equation in terms of the safety margin SM *as follows:*

$$SM = T_f\,(x_1, x_2 \dots\dots x_n) - T_d\,(x_1, x_2 \dots\dots x_n)$$

This equation for verifying the failure condition can be used in the context of three calculation types or levels.

Level I. Overall Factor of Safety Method or Partial Factors Method

Safety or serviceability is introduced by modifying the particular representative values of the design factors intervening in the check equation with coefficients according to the reliability or serviceability fixed as the design target, which weigh the possibility of the actions being simultaneous and compatible, as also their influence (favourable or unfavourable) on the failure mode, and by the safety factor, or minimum safety margin called for.

The design calculation is concluded when the safety factor or the safety margin obtained is equal to or greater than the minimum required, which generally tends to be 1 and 0 respectively in the partial factors method.

Level II. Confidence Index Method (β).

The reliability and serviceability levels set as design target, directly related to the failure probability, should be translated into an equivalent confidence index. The reliability or confidence index is usually represented by the Greek letter β.

The design factor values x_i intervening in the check equation are represented not by specific values but by random variables, through their distribution and covariance functions.

By transforming the distribution and covariance functions into normalised and independent Gaussian variables, the confidence index β is roughly equal to the minimum distance from the origin of the coordinates to the failure hypersurface G (x_i) = 0.

The verification is done when the confidence index obtained is greater than or equal to the value set as design target.

Level III. Calculation of Failure Probability.

The design factors intervening in the check equation are represented as random variables by means of joint distribution functions.

In most cases, it is extremely complicated to reach an analytical solution allowing the failure probability to be calculated. The calculation normally requires carrying out numerous simulations, each of which generates random values for the design factors x_i according to their respective probabilistic distributions. Some failure cases will be obtained as a result of all these simulations, which will be a fraction of the total simulations carried out. This fraction constitutes the failure probability.

The calculation is concluded when the failure probability obtained is lower than the value set as design target.

ROM 0.0 covers these calculation methods in more detail.

As for the other failure modes, the verification method recommended for geotechnical failure modes depends on the general nature of the works established in ROM 0.0, as a function of the ERI and SERI ratings. Consequently, to verify the safety and serviceability of the works included in the scope of application of the ROM Programme, Level I methods should always be used. Level II or III methods will have to be used in addition for works with high or very high ERI or SERI, corresponding to those defined as Category A in Table 2.12.1 of this ROM 0.5 for the purpose of planning geotechnical investigations.

In designs for Category A works, where multiple verifications should be carried out (at the least a double check) on the safety and serviceability parameters (Level I method and one or more higher-level methods), the verification is fulfilled when at least two of the verifying procedures used indicate sufficient reliability or serviceability. In any event, it is up to engineers to decide on this matter and always with due justification.

It is worth repeating that the verifying equation is independent of whatever calculation approach is followed. A particular failure criterion may be used with all three calculation levels indicated. The only differences lie in the manner of determining the parameters, agents and loads intervening in the check equation and in the criterion for accepting the result obtained. For this reason and because it is normally the simplest procedure, it is recommended to take the Level I calculation as a reference, since it has always to be done. This obviously does not exclude the additional relevant calculations from being carried out, nor whichever Levels II and III calculations are recommended in the ROM corresponding to each works type (or Part 4 of this ROM 0.5).

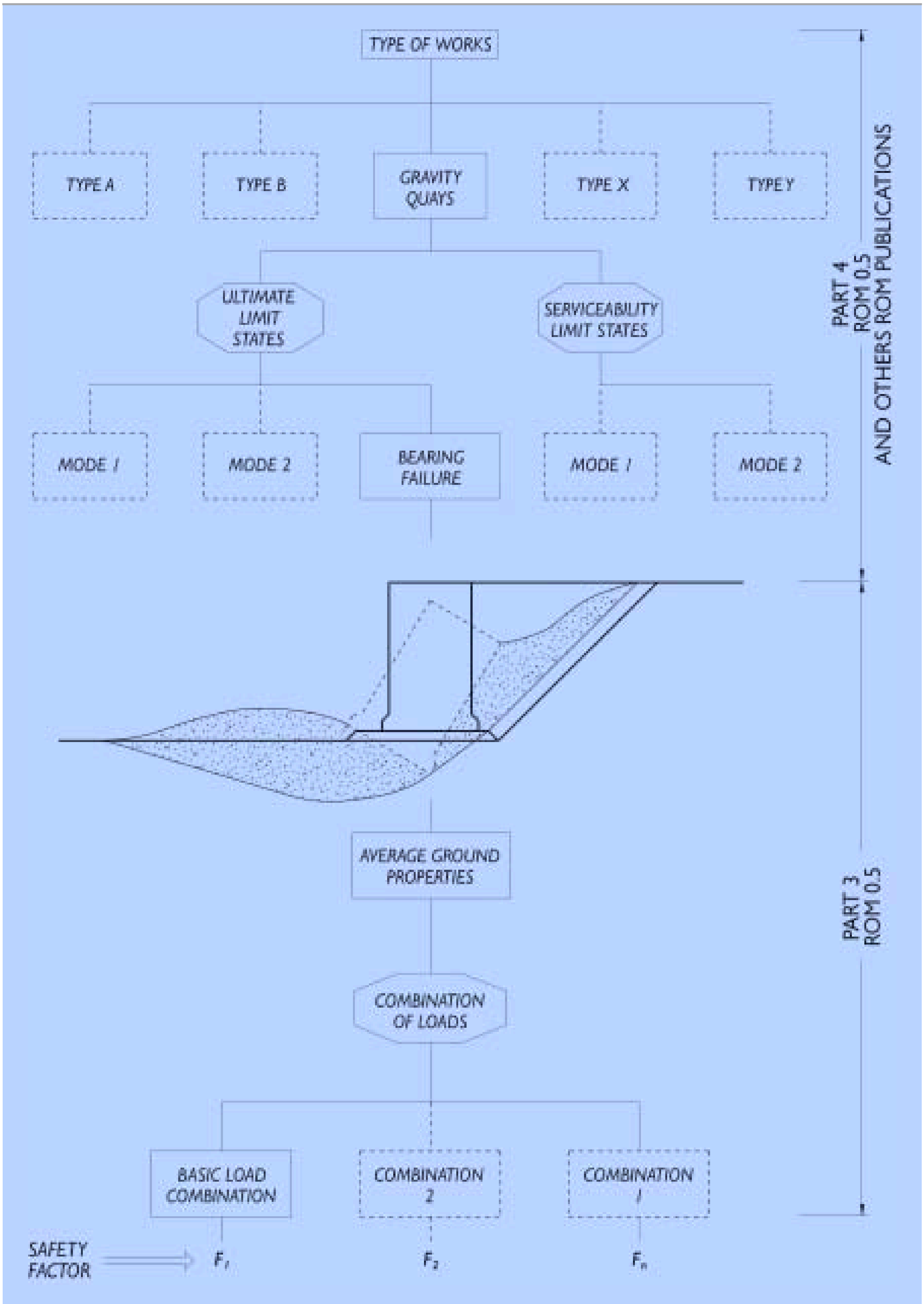
Part 3 of this ROM 0.5 includes details of a verification method appropriate for Level I calculations. Subsection 3.3.10 indicates a procedure for estimating the failure probabilities based on the results of these calculations.

The general procedure for verifying safety and serviceability in line with the Level I calculation method is explained diagrammatically in Figure 3.3.1.

3.3.5 Defining Design Situations or States

In order to analyse a particular Limit State, for the works or a section of them under study, it is necessary to consider a geometry for them and the ground, properties for the materials, the environment and the ground and a set of agents and actions. All that will generally amount to a simplification of the actual situation, valid for a particular time span during which the design factors and the structural and functional response of the works can be assumed to be statistically stationary, i.e., the variability of the design factors can be statistically described by probability functions and their corresponding statistical descriptors. These simplifications are known as the *design states* or *situations*.

Figure 3.3.1. **Process for Assessing Safety and Serviceability**



The more complex the problem and the less powerful the tools of analysis, the rougher this simplification will be. The greater the uncertainty surrounding the phenomenon under analysis and the available data and the greater the simplifications made to model the problem, the higher factors of safety or safety margins should be adopted.

In this way, the design states or situations to be considered in the calculations should be established by analysing all the possible conditions in which the works will be during each Project Stage (Construction, Operation, Conservation, Repair and Maintenance). Whenever possible, statistically stationary values will be adopted for the different design factors: geometry, properties and agents and actions. The design states or situations are grouped into working conditions as a function of the simultaneous action and compatibility of the predominant agents.

The duration of a design situation or state depends on the temporal variability of the design factors, including the response of the works. In terms of duration, the design situations or states are classified as:

- *Persistent*: corresponding to normal operating conditions of the works and which can occur over long periods of time similar to their design working life.
- *Transient*: short-lived in relation to the working life of the works, for several reasons including the geometry of the works (Construction Stage), the ground characteristics (consolidation or transient behaviour) or the acting loads (different service loads during the Repair or Maintenance Stages).
- *Exceptional*: situations where a design factor appears in an unexpected, accidental, or extraordinary way and lasts for a very short time in relation to the Project Stage under consideration (the effect of an earthquake, for instance).

In Level I calculations, for analysing all the different modes of geotechnical failures associated with each design situation or state in a particular site, it is recommended to consider the same geometrical configuration for the Project and the works and some permanent characteristics for the materials, the environment and the ground. In this way, the different design calculations will be compatible to a certain extent and therefore enable engineers to have an overall view of the project.

The following sections present the basic criteria recommended for defining design states and situations and the factors acting in them for verifying the geotechnical failure modes with Level I calculation methods.

3.3.5.1 Defining Geometrical Parameters

In Level I calculations, the geometrical parameters in each of the design situations or states should be considered permanent.

The geometrical configuration of the subsoil will be that deduced from the geological and geotechnical investigations, which should have clearly determined the different types of ground involved and their contacts. It is especially important to locate the layers of the soft soils capable of causing settlements, with enough precision in their thickness, and to represent weak zones capable of causing failures in structures or the ground.

The safety of works depends largely on the ground configuration chosen for future calculations. It is not possible to automate the ground modelling process using partial safety factors (equivalent increases in thickness of soft strata, fictitious increases in the depth of rock, etc.). Only the engineer's good judgement and experience will provide a simplified ground representation that is sufficiently exact and safe for the different calculations subsequently made.

When this proves unfavourable, geometrical dimensions of the works should be represented by their nominal value, increased or decreased in the construction tolerance indicated in the design.

Definition of the free water and groundwater levels, the state of the water in the ground, the observation of piezometric levels and their representation in different analyses are of particular importance in maritime and port works. For the purpose of the geometrical configuration of the works and the ground, the levels of free external water and the saturation levels in natural ground and fills to be adopted should be the levels that can be assimilated to a stationary situation to the effects of the response of the works and the ground – that is, without taking into account rapid variations, that can be generated by the effect of waves among other causes. In this respect, the corresponding levels associated with measuring periods of about five minutes can be used.

The representative water-level values should be defined in each design situation or state. For each design situation a couple of representative levels will normally be considered –corresponding to a high level (high tide and flood level, where appropriate) and to a low level (low tide and low-water mark, where appropriate)– for the free external water, the saturation lines in the ground and the levels inside the fixed or floating structures

Table 3.3.1. Representative Levels to Be Adopted for External Waters as a Function of the Combination Type Under Consideration

		Type of Load Combination			
		Quasi-permanent and Seismic	Fundamental and Infrequent, when Water Levels Are non-Predominant	Fundamental and Infrequent, when Water Levels Are Predominant	Frequent and Accidental
Representative Level of External Waters	High Level	Overall level associated with a 50% non-exceedance probability taken from the mean regime of maximum levels (1)	Overall maximum level with a 20-year return period (3)	Overall maximum level with a 50-year return period (5)	Overall level associated with an 85% non-exceedance probability taken from the mean regime of maximum levels (7)
	Low Level	Overall level associated with a 50% non-exceedance probability taken from the mean regime of minimum levels (2)	Overall minimum level with a 20-year return period (4)	Overall minimum level with a 50-year return period (6)	Overall level associated with an 85% non-exceedance probability taken from the mean regime of minimum levels (8)

(1) In the absence of relevant statistical data, the following simplified levels can be adopted:
- MHW (mean high water, roughly corresponding to a coefficient 70 tide), in seas with a significant astronomical tide (U.A. > 0.5 m)
- MSL (mean sea level), in seas without a significant astronomical tide
- MHW (mean high water) and MFL (mean flood level) (annual maxima), in areas with a significant astronomical tide and subjected to fluvial currents
- MFL (mean flood level, annual maxima), in fluvial currents unaffected by tides.

(2) In the absence of relevant statistical data, the following simplified levels can be adopted:
- MLW (mean low water, roughly corresponding to a coefficient 70 tide), in seas with a significant astronomical tide
- MSL (mean sea level), in seas without a significant astronomical tide
- MLW (mean low water) and MeanLW (mean river low-water), in areas with a significant astronomical tide subjected to fluvial currents
- MeanLW (mean river low-water), in fluvial currents not affected by tides.

(3) In the absence of relevant statistical data, the following simplified levels can be adopted:
- HAT (highest astronomical tide) +0.5 m, in seas with a significant astronomical tide
- MSL (mean sea water level) + 0.8 m, in seas without a significant astronomical tide.

(4) In the absence of relevant statistical data, the following simplified levels can be adopted:
- LAT (lowest astronomical tide) – 0.3 m, in seas with a significant astronomical tide
- MSL (mean sea water level) – 0.6 m, in seas without a significant astronomical tide.

(5) In the absence of relevant statistical data, the following simplified levels can be adopted:
- HAT (highest astronomical tide) + 0.6 m, in seas with a significant astronomical tide
- MSL (mean sea water level) + 1.00 m, in seas without a significant astronomical tide
- HWM (high water mark), the highest water level observed locally in fluvial currents affected or unaffected by tides.

(6) In the absence of relevant statistical data, the following simplified levels can be adopted:
- LAT (lowest astronomical tide) – 0.4 m, in seas with a significant astronomical tide
- MSL (mean sea water level) – 0.8 m, in seas without a significant astronomical tide
- LWM (low water mark), the lowest water level observed locally in fluvial currents affected or unaffected by tides.

(7) In the absence of relevant statistical data, the following levels simplified can be adopted:
- MHWS (mean high water springs, roughly corresponding to a coefficient 90 tide), in seas with a significant astronomical tide
- MSL (mean sea water level), in seas without a significant astronomical tide.

(8) In the absence of relevant statistical data, the following simplified levels can be adopted:
- MLWS (mean low water springs, roughly corresponding to a coefficient 90 tide), in seas with a significant astronomical tide
- MSL (mean sea water level), in seas without a significant astronomical tide.

and their compatible and simultaneous values respectively in the ground, external free water or structures. The corresponding combination values will be adopted as a function of the combination type and simultaneous action of the design factors intervening in the check equation (see Subsection 3.3.5.3). In cases where the water level is not considered jointly with other climatic loads (waves, currents, wind), thus forming a group of dependent, variable loads, and the water level is not considered an accidental load either, the representative values in Table 3.3.1 should be taken for the water levels, as a function of the combination type under consideration. The way these exceptions should be treated is specifically covered in the ROM 0.3 publication as also will be in other Recommendations corresponding to different structural types.

The representative values of the levels of free external water in the different Spanish port areas and the saturation lines compatible with them in natural ground and fills are also covered in ROM 0.3 and in Section 4.2 of this ROM 0.5.

3.3.5.2 Defining Ground Properties

Ground properties will be represented in each design situation or state by a set of parameters that will normally have been deduced from the field or laboratory tests carried out during the geotechnical investigation. As a general rule, these parameters should be the same in each individual problem analysed or at least be mutually compatible.

In Level I calculations, the ground properties should be considered permanent in each of the design states or situations.

In general, it should be assumed that the representative value of these ground properties is a cautious estimate of the average value of the results obtained from the geotechnical exploration corresponding to the area affected by the failure mode under consideration, unless for some duly justified reason engineers should choose a more conservative nominal value. Such exceptions can arise when the information available is deemed insufficient for statistical processing or the results are seen to be widely disparate. Other exceptions worth recommending are expressly indicated in this ROM 0.5 or elsewhere in the same ROM Programme publications.

It is rare to obtain all the data related to a particular geotechnical property or parameter by a single procedure. The usual and recommended practice is to obtain the most important geotechnical parameters by two or more independent and complementary ways. With all the data available, drawing a conclusion on the design value for a particular parameter will therefore be somewhat subjective. The engineer's experience and good judgement will thus be indispensable. Part of the final safety against the problem under study is involved in this process.

When there is a statistical base allowing the distribution function of a particular geotechnical parameter to be determined in a bounded spatial region, the representative value of this parameter will be the one corresponding to the quantile of the non-exceedance probability of 5% or of 95%, whichever proves the least favourable for the calculations.

3.3.5.3 Defining Actions

The actions that must be considered in the design for particular maritime and harbour works are specified in other ROM publications, other applicable codes or specific standards or by the conditions imposed by operation. The only time when the statements included in this subsection are to be taken into consideration is when insufficient specific information is available about the actions to be considered in a particular case and when, in addition, a geotechnical failure mode is to be verified.

Actions can be classified as a function of their temporal variation in the design situation or state under consideration as:

- Permanent actions (G).
- Non permanent or variable actions (Q).
- Extraordinary actions (A).

Direct actions caused by the ground such as gravitational loads, earth pressure, loads caused by ground movements and other loads should be considered permanent loads [1]. Similarly, quasi-static actions caused by water levels, such as the gravitational load of water, hydrostatic water pressure and uplift, etc. should also be considered permanent loads, regardless of the representative value for the water levels adopted in the load combination considered in the check equation. Loads transmitted by the ground as a result of the action of external loads should be considered variable loads.

For Level I calculations, the agents causing the actions –and even the actions, where applicable– should be defined using a nominal value or through a representative value linked to a particular quantile of their distribution function whenever sufficient statistical base allows the distribution function to be determined.

Gravitational agents, some agents related to utilisation and operation, construction agents and some accidental agents are generally quantified by means of a nominal value.

In the case of permanent agents and actions, whenever a statistical base is available, the characteristic value will be the one corresponding to the quantile of 5% or of 95%, whichever proves the most unfavourable for the calculations.

With variable agents and loads, wherever sufficient statistical base exists, the characteristic value should be the one corresponding to the quantile of 2% or of 98% –whichever proves the most unfavourable for the calculations– of the distribution function, considering a reference period representative of the design state or situation. This period will generally equal one year.

In the case of unfavourable variable loads in persistent situations, considering a one-year reference period is the equivalent of taking the value corresponding to a 50-year return period. In many cases and for the sake of simplicity, the representative value of favourable variable loads can be taken as equal to zero (non occurrence).

In the case of extraordinary agents or loads, including earthquakes, whenever a statistical base exists, the characteristic value to be adopted shall correspond to a very high annual non-exceedance probability, about 99.8%, the equivalent of 500-year return periods.

The general safety factors recommended in this ROM 0.5 for Level I calculations are established taking into consideration the representative load values pointed out in this subsection. If, in a particular application, the nominal or characteristic load values had been determined by clearly different criteria, the necessary modifications will have to be introduced in order to obtain values meeting similar criteria to those indicated here, or else a study will need to be done on the weighting coefficients of the loads and the safety factor appropriate for each case.

Comment: The case mentioned at the end of the preceding paragraph can arise in works where, for economic optimisation purposes and taking into account the corresponding SERI rating, occurrence probabilities for geotechnical failure modes similar to those covered generally in this ROM 0.5 are not allowable. In these cases, the geotechnical failure modes should be considered principal failure modes. Some of them will be statistically related, as they share the same cause and because they constitute different forms of approaching a single failure (ground failure). The allowable failure probability for the works and for each of the geotechnical failure modes should be adequately defined and tailored to each individual case rather than in the general way used for other types of works. Consequently, the representative value for the load and

(1) To check structural failure modes using Level I calculations, loads caused by the ground may be considered differently.

the corresponding partial factors should be specific to each project. This ROM 0.5 highlights the types of maritime and harbour works in which this load definition must be specific, as also the minimum associated safety factors.

3.3.5.4 Defining Combinations of Actions and Other Design Factors

The design factors –along with their values– intervening in the check equation will be specified for several combinations of actions, in order to consider the possible simultaneity and compatibility of the different design factors. The relevant combination criteria or procedures are given in ROM 0.0. These criteria should be used to define the calculation details corresponding to each maritime and harbour works.

For each specific type of works, the ROM publications will define the combinations of actions to be used to verify the reliability, serviceability and operability of maritime and port works. The indications below should only be applied to the study of the geotechnical failure modes and always provided that the corresponding combination criterion has not been laid down in any other more specific ROM publication.

In recent decades, there has been a gradual approximation in Level I methods between the geotechnical and the structural fields with the result that today it seems adequate to modify the recommendations given in ROM 0.5-94 on how to obtain the effects of the loads in the check equation for calculating the corresponding safety factor.

Ultimate Limit States (ULS)

In order to verify safety against Ultimate Limit States, in the structural field it suffices to consider the following three types of combinations of actions:

- *Fundamental* or *characteristic* combinations for persistent or transient design situations.
- *Accidental* combinations for exceptional design situations.
- *Seismic* combinations for exceptional design situations with an earthquake.

In the geotechnical field and owing to the weight of the past practice of calculations based on allowable stresses, the following load combination is also used:

- *Quasi-permanent* combination.

This new ROM 0.5 version recommends retaining the same ideas, albeit some details are refined later. Consequently, in order to verify the geotechnical failure modes assigned to Ultimate Limit States, it is advisable to consider, in addition to the types of combination corresponding to structural calculations, another combination, known as the *quasi-permanent* combination and which is used in the structural domain only to verify Limit States of Serviceability.

The reason for retaining this combination basically lies in conserving a feature of the former geotechnical calculation practice which, with an ample safety factor, forced failure to remain "far enough" from the average situation of the largest duration (quasi-permanent combination).

Consequently, in Level I calculations, the geotechnical failure modes assigned to an Ultimate Limit State should be verified for the following combinations of actions, even though the ROM publication corresponding to each works type may well establish a different criterion.

1. *Quasi-permanent Combination*

   This combination consists of all the permanent actions acting on the works and the ground plus the quasi-permanent values of the simultaneous and compatible variable loads, which are obtained by multiplying their nominal or characteristic values by a compatibility factor Ψ_2. This combination can be symbolically represented by the following formula:

$$G + \Sigma\ \psi_{2,i} \cdot Q_i \qquad \text{for } i \text{ between 1 and } n$$

where:

G = permanent actions.
Q_i = simultaneously acting variable loads.
$\psi_{2,i}$ = quasi-permanent compatibility coefficient..

In general terms, this combination attempts to represent the average value of the loads during the time interval associated with the design situation or state under consideration.

2. *Fundamental or Characteristic Combinations*

This kind of combination takes into consideration several variable loads with compatible values acting simultaneously when the failure mode occurs. In this way, the principal or predominant variable load in the occurrence of the failure mode –and the loads directly dependent on this– intervene with their characteristic values and the other simultaneous and compatible variable loads intervene with their fundamental combination values, which are obtained by multiplying their nominal or characteristic values by a compatibility factor ψ_0. This combination can be symbolically represented by the following formula:

$$\gamma_g \cdot G + \gamma_{q,1} \cdot Q_1 + \Sigma\ \psi_{0,i} \cdot \gamma_{q,i} \cdot Q_i \qquad \text{for } i \text{ between 2 and } n$$

where:

G = permanent loads.
Q_1 = principal or predominant variable load in the failure mode and simultaneously acting variable loads directly dependent on the predominant load.
Q_i = other simultaneously acting variable loads compatible with the predominant load and statistically independent of it.
$\psi_{0,i}$ = fundamental or characteristic compatibility coefficient.
γ_g, γ_q = partial weighting coefficients.

From all the fundamental or characteristic combinations, i.e., for each variable action that can become predominant, engineers may eliminate those combinations that, on a duly justified basis, are capable of originating less dangerous stresses in the ground than other combinations that consider the same action.

3. *Accidental Combinations*

Whenever the failure mode verification takes into account the action of an extraordinary load –accidental or otherwise– with a very low probability of occurring during the time interval under consideration and, at the same time, acting for a short period, the compatibility value of the variable loads acting simultaneously should be clearly lower.

For each exceptional action that can occur, there will be an accidental combination that can be formulated in the following terms:

$$G + A + \psi_1 \cdot Q_1 + \Sigma\ \psi_{2,i} \cdot Q_i \qquad \text{for } i \text{ between 2 and } n$$

where:

G = permanent actions.
A = extraordinary action.

Q_1 = principal or predominant variable action in the failure mode and simultaneously acting variable loads which depend directly on the predominant load.
Q_i = other simultaneously acting variable actions, compatible with the predominant load and statistically independent of it.
ψ_1 = frequent compatibility coefficient.
$\psi_{2,i}$ = quasi-permanent compatibility coefficients.

The action of exceptional loads should not be taken into account –nor therefore the accidental combinations– in short-lived states or situations. The same applies to provisional works, provided their working life is less than one year.

4. *Seismic Combinations*

Whenever a failure mode verification takes into account the action of a seismic load, which has a very low probability and a very short period of action compared to the design situation or state under study, the compatibility value of the variable loads acting simultaneously should be clearly lower, and no distinction made between the compatibility value of the principal (predominant) variable action and the other variable actions. This combination can be symbolically represented by the following formula:

$$G + S + \Sigma\, \psi_{2,i} \cdot Q_i \qquad \text{for } i \text{ between 1 and } n$$

where:

G = permanent loads.
S = seismic action.
Q_i = simultaneously acting variable loads.
$\psi_{2,i}$ = quasi-permanent compatibility coefficient.

In general terms, this combination tries to take into account the fact that the compatibility value of the variable loads to be considered when an earthquake acts is roughly the average value of the actions during the time interval associated with the design state or situation under study.

Compatibility coefficients have the meanings given below:

- $\psi_0 Q$: fundamental combination value of the action or compatibility value of a variable load when acting simultaneously with another predominant variable action in the failure mode occurrence and is statistically independent of it. This is obtained in such a way that the values of the effect of the load combined with the predominant load have a probability of being exceeded approximately similar to the probability of the effect of the predominant load being exceeded. For two incompatible loads, the value of ψ_0 is equal to 0.00. For two completely dependent loads, the value of ψ_0 is 1.00.

- $\psi_1 Q$: frequent combination value of the load. This value is not exceeded during a long time period compared to the duration of the design state or situation under consideration.

  Comment: When a sufficient statistical base is available, the frequent combination value that can be adopted for a variable action of climatic origin is the value associated with a non-exceedance probability of 85%, taken from the corresponding average regime.

- $\psi_2 Q$: the quasi-permanent combination value of the load. This value is not exceeded during a long time period compared to the duration of the design situation under study. When a sufficient statistical base is available, the quasi-permanent combination value that can be adopted for a variable load is the value associated with a non-exceedance probability of 50%, taken from the corresponding average regime. This value represents the mean value of the load during the time interval associated with the design state or situation under study.

Consequently, with this definition, the values for the ψ coefficients depend on the particular agent causing the variable load and should be found in the corresponding load codes or in Series 0 of the ROM recommendations. When no specific data are available, the values defined in ROM 0.0 can be taken.

LIMIT STATES OF SERVICEABILITY (LSS)

In Level I calculations, the following load combinations should be used to verify the failure modes assigned to Limit States of Serviceability:

- *Infrequent combination:*

  $G + Q_1 + \Sigma\, \psi_{0,i} \cdot Q_i$ for *i* between 2 and *n*

- *Frequent combination:*

  $G + \psi_1 \cdot Q_1 + \Sigma\, \psi_{2,i} \cdot Q_i$ for *i* between 2 and *n*

- *Quasi-permanent combination:*

  $G + \Sigma\, \psi_{2,i} \cdot Q_i$ for *i* between 1 and *n*

  where:

  G = permanent actions
  Q_1 = principal or predominant action in the failure mode and directly dependent variable loads simultaneously acting
  Q_i = other simultaneously acting variable loads compatible with the predominant action and statistically independent of it
  $\psi_{0,i}$ = fundamental compatibility coefficients
  ψ_1 = frequent compatibility coefficient
  $\psi_{2,i}$ = quasi-permanent compatibility coefficients.

The different ROM publications and the standards, codes and specific recommendations applicable will fix the conditions for verifying each failure mode assigned to a Limit State of Serviceability, specifying which load combination type applies.

As a general rule, in Level I calculations, the quasi-permanent load combination should be used for checking long-term settlements and for verifying other geotechnical failure modes assigned to Limit States Of Serviceability.

3.3.5.5 Considering Short-Duration States or Situations

As mentioned in earlier sections, the design values for variable and accidental loads are formally dependent on the duration of the design situation under study. In the case of geotechnical design, it is advisable to study explicitly a specific situation related to ground consolidation.

The application of loads and, more generally, any action on saturated ground produces certain variations in porewater pressure. On certain occasions, which will be specified throughout this ROM 0.5, it is advisable to assume that the ground does not drain and that, for a certain period of time, the porewater pressure generated by the different agents under consideration remains constant.

The time frame corresponding to these *undrained* situations should be estimated by the engineer and, where necessary, also the representative values of the actions corresponding to the estimated duration of the period in which no noticeable consolidation is produced.

This situation normally lasts a short time compared to the design life of the works and can therefore be considered as transient when defining the design situation in which to verify the different failure modes.

During the construction process, actual short-term situations may also occur temporarily and they also require the design values for the loads to be correspondingly adapted.

Engineers will have to decide on the nominal or characteristic values of the variable loads that can be taken as compatible and simultaneously acting in this type of situation. As a general rule, when no reduction in reliability is intended, the values for variable loads should be chosen so that they have an exceedance probability, while the transient situation lasts, roughly equal to the exceedance probability of the corresponding –and different– values adopted for the design life.

Comment: *The relation between the exceedance probability of a variable action* n *during a Design Stage with a duration* L *and a return period,* T_R*, is given by the expression:*

$$P_{n,L} = 1 - \left(1 - \frac{1}{T_R}\right)^L$$

As it has been adopted a characteristic value of the variable loads for the Service Stage corresponding to a 50-year return period, the exceedance probability of the action ranges from 0.63 for Service-Stage durations (design life, V) of 50 years to 0.26 for 15-year Service Stages. Consequently, the characteristic value of the loads in transient situations with duration L *can be calculated based on the above expression, by associating it with the return period which, for the duration of these situations, gives rise to a similar exceedance probability to those mentioned above for the Service Stage. As a result, for practical purposes the value corresponding to a return period of the same order of magnitude as that of the duration of the stage can be adopted as characteristic value for a variable load in transient Stages, ranging from an asymptotic maximum of* T_R=50L/V *(for the longest possible transient situations in relation to the Service Stage) to twice that ratio (for transient cases with less duration) with a minimum value of two years.*

Furthermore, in some of these transient Stages, the nature of the works can vary from their general nature and, as a result, it may prove advisable to run a specific study of the reliability required in these circumstances.

The fundamental load combination should be adopted to verify the safety against Ultimate Limit States. As a general rule, it is not considered necessary to assume that exceptional or accidental loads will act during short-term states or situations.

3.3.6 Partial Factors for Actions

In order to use the Level I calculations provided in this ROM 0.5 to verify the failure modes corresponding to Ultimate Limit States, partial factors are applied to the actions when they are combined, in the manner indicated in Subsection 3.3.5.4.

As explained in that subsection, the factors weighting the actions should only be applied in fundamental or characteristic combinations. In quasi-permanent accidental and seismic combinations, partial factors for the actions are not considered.

For the case of fundamental combinations, actions should be increased with different factors, depending on the type of agent causing the actions, its temporal classification (permanent or variable), the way in which it participates in the occurrence of the mode (favourable or unfavourable) and the type of failure mode studied.

Table 3.3.2 gives the partial coefficients for the actions recommended in this ROM 0.5 for verifying the failure modes assigned to Ultimate Limit States.

Table 3.3.2. Partial Factors for Actions* for Verifying the Failure Modes Assigned to Ultimate Limit States (ULS). Fundamental Combinations

Action	Symbol	Type of Failure Mode				
		EQU	STR	GEO	UPL	HYD
Permanent						
Unfavourable	γ_g	1.10	1.35	1.00	1.00	1.35
Favourable		0.90	1.00	1.00	0.90	0.90
Variable						
Unfavourable	γ_q	1.50	1.50	1.30	1.50	1.50
Favourable		0.00	0.00	0.00	0.00	0.00

(*) These coefficients do not apply to works where, for economic optimisation criteria, occurrence probabilities for geotechnical failure modes similar to those generally considered in this ROM 0.5 (e.g., in breakwaters) cannot be admitted (see comment in Subsection 3.3.5.3).

3.3.7 Partial Factors for Ground Resistance

Current versions of some regulations and several research studies in the geotechnical engineering field point to the systematic use of reduced ground strength parameters as the future calculation procedure. Nevertheless, this ROM 0.5 adopts the criterion of not factoring them on a point of caution. Sufficient experience has still not been acquired at the present time concerning the results derived from such a reduction and its effect on the definition of the minimum safety factors or margins required for verifying the different geotechnical failure modes. Consequently, this ROM will not incorporate partial factors reducing ground strength values (see justification in Section 1.4).

However, it is always possible, with a degree of scientific rigour, to reduce the final resistance or, what comes to be an equivalent exercise, to require an appropriate minimum safety factor when the effect of the loads are compared to the ground resistance.

3.3.8 Safety Factors

This ROM 0.5 will generally formulate the check equation for the safety of particular works or a section of them for a geotechnical failure mode in terms of the safety factor, considering increased loads and non-reduced resistances:

$$F = R/E_d > F_m$$

and specifying for each circumstance the minimum safety factor, F_m, to be achieved.

Safety can be said to be verified when the safety factor calculated exceeds the threshold F_m specifically defined for each failure mode analysed. In turn, for each failure mode analysed, this safety factor additionally depends on the nature of the works and the type of design situation involved, according to its duration and the type of load combination studied.

3.3.8.1 Minimum Safety Factors

To verify each specific geotechnical failure mode, this ROM 0.5 will generally establish the corresponding check equation specifically, as also three associated overall safety factors, clearly different, depending on the type of design situation or state and the type of load combination studied, as indicated in Table 3.3.3.

In a persistent state or situation and for the quasi-permanent combination, the safety factor should be high in order to stay away from failure and try to have the works foundation behave normally in an elastic regime.

Table 3.3.3. ***Minimum Safety Factors Recommended for Verifying Geotechnical Failure Modes Assigned to Ultimate Limit States 5 ≤ SERI < 19.***

Design Situation	Load Combination	Required Safety Factor, F
Persistent	Quasi-permanent Fundamental	F_1 F_2
Transient (including short-term geotechnical situations)	Quasi-permanent Fundamental	F_1 or F_2 F_2 or F_3 (see text)
Excepctional	Accidental with no earthquake Seismic	F_3 F_3

For these same states or situations but for fundamental combinations, when variable loads with high values (increased) are already acting, the safety factor adopted should logically be lower ($F_2 < F_1$).

For exceptional situations, the usual practice is to consider that the works almost reach failure when the nominal or characteristic value of the load is acting. Normally, F_3 will have a value close to 1. The value recommended will be lower than the two preceding ones, $F_3 < F_2$.

For the transient or short-lived situations defined in Subsection 3.3.5.5, values for F_1 and F_2 equal to that for persistent states or situations can be used, taking into account the duration of this situation when determining the value of the loads. Nevertheless, if it should happen that the same representative values for the loads are used as those corresponding to persistent situations, the safety factors can be lowered somewhat, replacing value F_1 by F_2 for quasi-permanent combinations and value F_2 by F_3 for fundamental combinations, in order to take this fact into account.

As a general rule and provided no indications to the contrary are given, the values for these coefficients included in this ROM 0.5 can be taken to be associated with very low occurrence probabilities (in the order of 10^{-3}) during the Design Stage under consideration, which are the ones assigned in this ROM 0.5 to geotechnical failures in works with a low SERI rating (between 5 and 19). This means that, by adopting these safety factors in the calculation, in many cases it is possible not to consider the geotechnical failure modes as principal failure modes and, therefore, their contribution can be disregarded when calculating the overall failure probability, in line with the indications given in Subsection 3.2.1.

When the safety factor of a specific application of this ROM 0.5 is associated with a different failure probability, it will be expressly indicated in the relevant section, as will the criteria adopted in this case for defining the design factors and particularly for defining the characteristic and design values of the actions intervening in the check equation.

3.3.8.2 Minimum Safety Factors Associated with Other Failure Probabilities

In order to check a failure mode, the minimum safety factor required can be related to its occurrence probability by applying Level II and III calculation methods.

In a simplified manner, for certain ranges of the reliability index and when the duration of the Design Stage and other uncertainty factors, such as the intensity of the geotechnical investigation, are equal, a normal distribution of the safety factor logarithm is an acceptable assumption. This assumption leads to the conclusion that the minimum safety factor that should be required for any type of combination F_i (i = 1, 2, 3), and here called F for short, depends exponentially on the reliability index required and can therefore be written as:

$$F = F_0 \cdot \exp\left[\,\zeta\,(\beta - \beta_0)\right] \geq 1$$

where:

F_0 = safety factor associated with a reliability index β_0.
F = safety factor associated with a reliability index β.
ζ = standard deviation of ln F.

Use of this expression should be limited to differences of $|\beta - \beta_0| \leq 0.7$ as its error increases with this value.

Details of the ζ, value, which depends amongst other factors on the failure mode studied, can be seen in Subsection 3.3.10.

Comment: For guidance purposes, the following value can be used:

$$\zeta = 0.15\, F_1 - 0.10$$

where F_1 is the minimum target safety factor in quasi-permanent combinations for the failure mode in question in low SERI works (5-19) and can be looked up in the corresponding tables throughout this ROM 0.5.

Therefore, if the minimum required *F* values are determined for a particular reliability, it is possible, within a certain margin, to calculate the values corresponding to any other reliability or failure probability.

3.3.9 Using Numerical Models

This ROM 0.5 describes analytical solutions for the majority of the problems encountered in maritime and harbour geotechnical engineering. This is possible thanks to certain simplifications.

For every solution (or calculation method), ROM 0.5 defines not only the operating procedure but also the value or values of the safety factors associated with the method.

It is currently possible to utilise more complicated mathematical models than the ones explained here. The time is near when such models will make us forget the formulas so painstakingly drawn up by geotechnical engineers preceding us and that currently constitute the core of this ROM 0.5. Furthermore, practically all designs for current projects already use, to a greater or lesser extent, numerical models that replace, sometimes incorrectly, the simple formulae and methods laid out here.

It is good practice to use complex numerical models. Here, in the pages of this ROM 0.5, these models are recommended for solving complex calculations that cannot be tackled accurately by another method. It is not a bad idea, either, to use complex models even in simple problems that could be tackled without them. Experience is gained in this way and more information obtained concerning the particular problem under study.

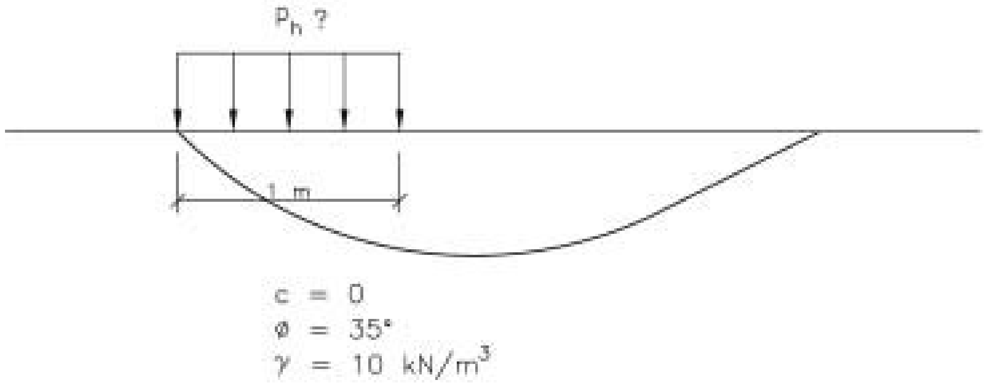
The ROM 0.5-94 edition did not contain any specific recommendation about the precautions necessary to use numerical models. Now, when the use of these new methods is beginning to be widespread, it is expedient to indicate some minimum instructions.

Programs for Slope Stability Analysis

These are the best developed of the computational software used in geotechnical engineering. They are a far cry from the primitive versions presenting convergence errors in solving the basic mathematical equation for stability.

Substantial problems do not exist when these programs are used to solve overall equilibrium problems but can lead to confusing errors when an attempt is made to apply them for calculating bearing capacities. In this particular aspect, the error in these mathematical models can be excessive. Small changes in the trial failure line and/or in the directions of the earth pressure between slices lead to results that can err on the safe or unsafe sides, and this cannot be known *a priori* by some simple, discriminatory procedure. Even the number of slices, that the user has to choose, is capable of modifying the result in a direction that is not easy to predict.

Engineers, faced with this problem when they are to use a commercial program, before crediting the safety factors resulting from the calculation, are recommended to become really familiar with the program and to test it by solving simple cases with well-known solutions, such as the example given in the following diagram.



For this problem, whose solution is p_h = 226 KN/m^2 according to the simple approximated formula given in this ROM 0.5, a safety factor of F = 1 should be obtained. Engineers must tackle this type of preliminary problem in order to gain confidence (and share it) in these applications.

Programs for slope stability analysis always supply a result that is the safety factor against *slope sliding*, which is the number by which each individual *c* and *tan* ϕ parameters appearing in the input data has to be divided by to obtain the strict equilibrium.

For bearing capacity problems, it is however a good idea to do another type of calculation. The load causing the bearing failure should be increased so that its two components grow homothetically until F = 1 is obtained.

When this has occurred (strict equilibrium), the safety factor against bearing failure is the factor by which the loads have been multiplied to achieve this.

The plastic overturning mechanism can equally be solved with slope stability analysis programs. All that is required is to define the geometry of the problem and its loads and to obtain the corresponding safety factor given by the computer program for this particular situation. Then the horizontal component of the resultant of the loads should be gradually increased –and accordingly increasing the eccentricity by reducing the effective foundation width– until the strict equilibrium (F = 1) is obtained. The value by which it proved necessary to multiply the horizontal component of the resultant of the loads is the safety factor against plastic overturning.

For verifying purposes, engineers should tackle simple problems like the one illustrated in the diagram below to check whether the program's solution coincides with or approximates the one known by other procedures.

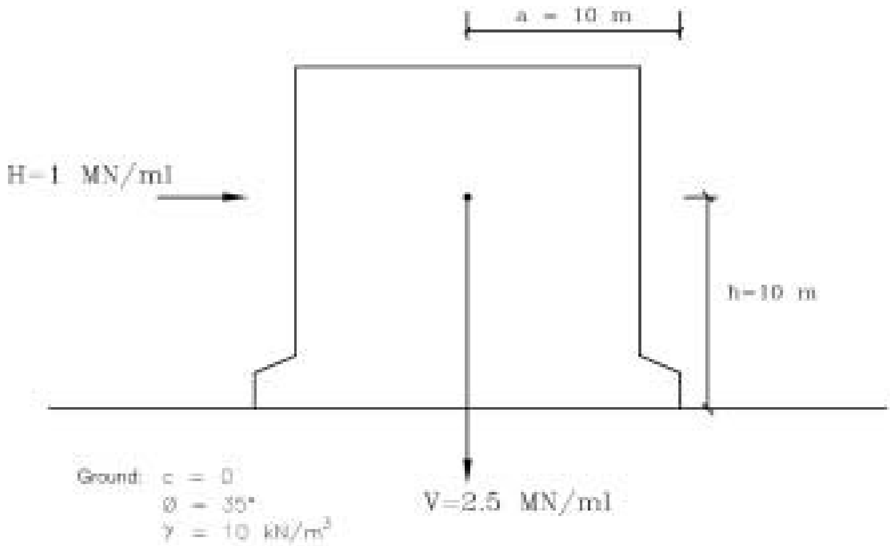
Analytical solution according to this ROM 0.5: N_γ = 45.2 for ϕ = 35°
Safety factor against bearing failure

$$F_h = 2 \cdot \gamma \cdot N_\gamma \frac{\left(V \cdot a - H \cdot h\right)^2 \left(V - H\right)^3}{V^6} = 2.81$$

Safety factor against plastic overturning

$$F_v = \frac{V}{H}\left[1 - \sqrt[5]{\frac{V}{2 \cdot \gamma \cdot N_\gamma \cdot a^2}}\right] = 1.28$$

(for a = h).



Numerical Models in Total Pressures

General-purpose numerical models (stress-strain problems) usually formulate the mechanics (or the dynamics) of the problem by using equations written in terms of total pressures (or stresses). This is a serious disadvantage substantially limiting the application to maritime and harbour problems in which the presence of water cannot be omitted.

In spite of this, it is possible to solve problems in which the water is in a hydrostatic state (calm, motionless) and even the situations in which the water is in a steady-state regime, with a spatial distribution of pressures constant in time.

In these circumstances (steady-state flow), it is possible to fix an arbitrary reference plane, normally the free water plane (mean sea level) and to assume that the weight of the ground and of the structures located beneath this level is the submerged weight, whereas above this it is the apparent or saturated weight (whichever applies).

The water pressure differences in relation to this level must later be explicitly considered as loads. This same procedure is detailed rather extensively in the calculation of the earth pressure on walls that can be seen in Section 3.7 of this ROM 0.5.

From the experience gained from the application of ROM 0.5-94 over the past decade, the most significant problems encountered -in the designs for projects in which *Puertos del Estado* was involved- have precisely been in the manner of applying these excess pressures in the numerical models formulated in total pressures.

With numerical models, it is not necessary to resort to the "method of failure modes" in order to analyse safety, as the actual model itself is capable of indicating which is the most critical failure mode. The fact that this is not necessary does not mean it is not advisable to do so in any case.

Whenever a complex numerical method is employed, it is advisable to use it to calculate safety factors against the most common failure modes mentioned in this ROM 0.5, as explained below. This calculation should be twofold – following the simple procedures indicated in the ROM on the one hand and using the numerical model on the other.

To use a numerical model to calculate the safety factor against bearing failure for shallow foundations, it must be assumed that the foundation is reduced to its effective breadth B^*. This is a conservative assumption but constitutes the usual practice on which the safety factors recommended in this ROM 0.5 are based. Calculations using foundation-structure interface elements with limited strength can also be employed, but special precautions are needed to confirm that effective tensile stresses are not generated.

For using numerical models to calculate the safety factor against plastic overturning, the horizontal component of the loads must be increased and the effective breadth decreased in a compatible way until failure is reached.

Computing with numerical models the safety factor corresponding to overall stability is a simpler procedure, as this generally only calls for a simultaneous reduction of c and $\tan\ \phi$ in all the materials until failure occurs. Some commercial programs perform this operation automatically.

With numerical models it is therefore possible to obtain the four basic safety factors in shallow foundation problems; namely, overall equilibrium, bearing failure, plastic overturning and also sliding along the support plane (even though this has not been described in these paragraphs because it is obvious).

Engineers need to check that their software is appropriate to this type of problem by using it first to solve similar verification problems with a well-known solution. The problems suggested in preceding paragraphs are an example of this.

The soil behaviour model, with regard to failure, can be the one deemed most fitting by the engineers. It is however advisable not to leave the Mohr-Coulomb model out of the calculations made, owing to the vast experience acquired with this.

Port works involve soft soils giving rise to substantial settlement (displacements). When the model takes into account the geometrical changes for updating the coordinates of the points defining the grid, local convergence problems tend to arise; current second-order methods (large strain) have still not developed sufficiently well and, as a result, the recommendation is not to use them to simulate Ultimate Limit States unless engineers have had positive experience with the procedure.

Using the Mohr-Coulomb model to formulate plasticity requires defining a dilatancy angle, Ψ, which is usually clearly lower than the angle of internal friction ϕ.

For cases where information is not available, is it recommended to assume that $\Psi = 0$.

The response of different commercial programs to an increase in the angle of dilatancy varies and engineers will therefore need to check this aspect, as the above assumption ($\Psi = 0$) may lead to results erring unnecessarily on the safe side.

Complex numerical models still do not solve failure states satisfactorily. From the time back in the 1960s when the first finite element method was formulated as applied to civil engineering (precisely to solve a geotechnical problem), no adequate formulation has been found to simulate failure well enough. Limit equilibrium methods (slope stability programs) still appear to produce more reliable solutions at the present time than the complex stress-strain models do.

Numerical Models in Effective Pressures

There is no doubt about the future of geotechnical calculations: numerical models will have to solve the coupled flow-deformation problem. This requires formulating the geomechanical problems in terms of effective pressures. The porewater pressure will appear as a dominant variable in many maritime and harbour problems, particularly where waves and other sea-level oscillations constitute the predominant agent in the failure mode.

The seepage process is a complex one as two principal mechanisms coexist in it – the generation of water pressures as a result of the volumetric changes in the soil and in the water itself and the dissipation or attenuation of pressures as a result of water flow inside the soil from the areas of high potential to those of low potential.

Numerical models in effective pressures are still a long way from being validated against reality, at least in a broad and general way. For this reason, they should be used with a reasonable backup from uncoupled calculations (total stresses).

DYNAMIC PROBLEMS

Adding inertia and damping into static problems substantially complicates the issue. The information needed to solve these numerical problems generally involves difficult mathematical manipulation (see Section 3.10).

In these cases, to focus the problem, it is advisable to use models in total pressures, to solve approximately the effect of the water on the strength and deformation parameters and to repeat the computations iteratively until a solution is found that is compatible with the information available.

Even though complex numerical models may mean the future of geotechnical calculations in maritime and harbour works, it is still advisable now to use the least complex models –among the available options– and to work them with convenient parameters that should be chosen as a function of the actual result, and this requires correctly guided iterative calculations.

If a geotechnical problem cannot be accurately solved by one of the procedures laid out in this ROM 0.5, which can be tackled with spreadsheets or simple software programs, then there is no possibility of specifying the solution much farther. Engineers should not seek this solution by using more complicated methods but instead by improving the input data and the necessary understanding of the problem in order to be able to simplify it.

3.3.10 Reliability in Geotechnical Engineering

The safety factor against a geotechnical failure mode *F*, calculated as indicated in this ROM 0.5, constitutes a good index of the works safety. Prestigious geotechnical experts have declared something that could be seen to contradict the former statement. According to them, a safety factor, per se, is not representative of safety if it does not entail some measure of the uncertainty of the design factors that have intervened in its determination. Both statements are true because the safety factor calculated in line with the ROM 0.5 indications implies a certain control of the possible lack of knowledge concerning the ground and the actions. ROM 0.5 intends that the uncertainties in the geotechnical parameters, the design geometry (including the representation of the actual ground) and the loads are limited and adequate to each case. Wherever it is difficult to know the ground, more intensive investigations will have to be carried out; whenever variable loads (always more imprecise) are present, their design value will be higher, etc. Furthermore, not all the geotechnical problems dealt with in this ROM 0.5 will be analysed with equally exact methods, which is why the corresponding target safety factors take this fact into consideration somewhat, albeit partially or indirectly. The minimum threshold values recommended for the safety factors are higher when the calculation method is more imprecise. In short, the safety factors calculated by the Level I methods described in this ROM 0.5 provide relevant information related to the safety of the works.

Nevertheless, it remains true that the uncertainty existing in the safety factor value is not expressly reflected in the results of the calculations made using Level I procedures. For this reason it is advisable to apply additional Level II and III methods in major works with high or very high ERI or SERI, in line with the recommendations in ROM 0.0. This will enable the reliability of the works to be specified more precisely and therefore its failure probability also.

However, this section introduces a simplified method enabling engineers to rate the uncertainty of the safety factor obtained with Level I procedures in each specific case and consequently to obtain the failure probability associated with it more precisely. This procedure, which basically amounts to a sensitivity analysis, is based on obtaining the coefficient of variation for *F* and is detailed below.

3.3.10.1 Sources of Uncertainty and Their Measurement

These geotechnical recommendations will not delve into the causes limiting knowledge. This branch of gnoseology must be reserved for other publications. To put it very briefly and only in respect of geotechnical problems, uncertainties can stem from the geometrical data, actions and geotechnical parameters involved. There also exists some degree of uncertainty associated with the calculation method (or the check equation, as this mathematical tool is sometimes called).

The simplest way of measuring the uncertainty of quantity x is its standard deviation, σ. When there is a possibility of measuring x, if the values obtained in N determinations are $x_1, x_2 \ldots\ldots x_n$, the standard deviation of x is said to be:

$$\sigma = \sqrt{\frac{\sum\left(x_m - x_i\right)^2}{N-1}}$$

where x_m is the average value of all the determinations.

In mathematical terms, each design factor intervening in the calculation of the safety factor F can be represented by two values. In traditional calculations (Level I methods), the data are only represented by a single value. However, calculating the reliability requires another quantity for each factor, which is the corresponding standard deviation.

When the uncertainty needs to be quantified even better, more information will have to be input. It will not be enough to indicate a representative value and the confidence in it, providing a measurement for its standard deviation, but other values will also be needed. It might even be necessary to provide a mathematical definition of a probability density function for the variable in question. However, this is not the case to be dealt with here. The following paragraphs assume for every relevant magnitude in the problem that, in addition to the conventional definition that has always been done and should still be used, there exists an estimated value for its standard deviation, as a measurement of its dispersion or uncertainty.

Geotechnical engineers are sometimes at a loss to tackle the problem of estimating the standard deviation. If no specific comments were made concerning the best way to go about this, different engineers would most probably come up with very different values for the standard deviation. It is therefore convenient to give some general ideas to serve as guidelines on this procedure.

The standard deviation value for a geotechnical parameter can be estimated in three complementary ways – based on experience, on test results and on the rule of the two standard deviations.

Usual σ Values, Based on Experience

Prestigious geotechnical experts, fully conversant not only with soil mechanics but also with the issues related to reliability calculations, have many times already performed the task of estimating the standard deviations for the commonest geotechnical parameters in different locations and have been so kind as to publishing them [(2)]. Table 2.14.1 shows some approximate average values found to be reasonable in the opinion of this ROM 0.5 editing committee on knowing the data published by different authors. But engineers are encouraged, where appropriate, to consult suitable and updated bibliographical sources in order to gain a fuller impression of the corresponding coefficient of variation. Following the customary practice of the technical publications on the subject, this table indicates the so-called *coefficient of variation*, v, which is the value of the standard deviation divided by the average value.

$$v = \frac{\sigma}{x_m}$$

(2) See for instance J. M. Duncan (2000). "Factors of safety and reliatibility in geotechnical engineering", ASCE, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 4.

OBTAINING THE STANDARD DEVIATION FROM TESTS

From a theoretical point of view, the best way of obtaining the standard deviation is undoubtedly by having a good statistical data bank available and calculating it directly. But some practical issues reduce the validity of this procedure somewhat.

The tests analysed should correspond to the same type of ground. Data from tests corresponding to different ground types should not be mixed. In this respect, this ROM 0.5 gives some guidelines for identifying two zones or two strata as different grounds. This separation is not easy and unfortunately it is not possible to give precise recommendations on the procedure to make such discrimination.

The tests indicate the response of the ground to situations similar to the one of interest, but differing from it in many aspects, including the size of the area tested. There is no easy way of knowing whether the variations in the test results stem from ground-related aspects (spatial variability) or are due to the test itself.

Without going into any further details, it may be concluded that it is not advisable to use the results of the statistical studies from data banks as the only source of information for obtaining the standard deviation of a design factor. It is advisable, as always, to introduce an additional criterion in order to determine a suitable value for σ.

RULE OF THE TWO DEVIATIONS

Engineers normally represent geotechnical data in graph form. They usually draw up resistance diagrams, graphs on the variation with depth of the most relevant data, etc. The aim of these plots is to provide a visual appraisal of the results in order to judge their uniformity or possible dispersion.

Even confidence bands or limit lines encompassing the possible results are frequently drawn. This procedure, albeit non standardised, is almost universal and this ROM 0.5 recommends that engineers should keep doing this and complete the procedure in order to select not only the design value, as used to be the practice, but also in order to obtain a measure for the standard deviation.

The rule of the two deviations, covered and discussed by the researchers on this topic, indicates that the data for a population tend to be grouped between limits which the geotechnical specialists, without a deliberate attempt, usually place two standard deviations away from the mean value. If this is true, then:

$$\sigma = \frac{x_{max} - x_{min}}{4}$$

where x_{max} and x_{min} are the maximum and minimum values engineers expect to find as the variation range for variable x.

There is a fragile but significant justification for the preceding statement. It can be statistically demonstrated that, in certain conditions (normal distribution of the variable in question), the amplitude of the variation range to be expected from the results obtained from some 30 determinations of a random variable is equal to twice the standard deviation, as indicated above. When fewer determinations are involved, about ten, this range is somewhat narrower, in the order of one and a half times the standard deviation on each side of the average. It can therefore be said that what geotechnical engineers usually do to fix the extreme values of their data appears to coincide roughly with the indications in the above rule.

It is always advisable for engineers to try out the three procedures indicated for obtaining the value of σ which will be used for each design factor playing a relevant role in determining the safety factor. With the three results, they must then use their own judgement to decide which is the best in measuring the uncertainty of the factor in question.

In addition, geotechnical engineers must also characterize the uncertainty of the actions. No conceptual problem exists with the action of the ground's self weight, as the specific weights tend to be the sort of parameters on which geotechnical engineers can establish adequate criteria. The problem arises when it is a matter of determining a standard deviation associated to the nominal or characteristic value of a variable load, either originating from operation of the works or from a climatic agent. In these cases, geotechnical engineers will need some specific assistance, because the problem is a complicated one and the approximation of the σ value they may obtain could be far from the actual reality. Later on in this publication, however, some values will be indicated for σ that could be used if no better information can be found.

3.3.10.2 The Reference Calculation

Estimating the failure probability by the simple procedure commented here requires the preliminary step of calculating the corresponding safety factors by the Level I method as recommended in this ROM 0.5. These results will form the basis of subsequent calculations. Level I calculations should always be done and will constitute the basic reference for any subsequent sensitivity study.

3.3.10.3 One-dimensional Sensitivity Analyses

Geotechnical engineers are accustomed to running sensitivity analyses (sometimes also known as parametric studies). These consist in repeating calculations for obtaining new safety factor values, changing the data of the problem one by one and keeping the other parameters fixed.

The procedure recommended here for estimating reliability in major works and described in this subsection is just an orderly sensitivity study.

To measure the sensitivity of a particular design factor, it is advisable to change its design value while maintaining the other factor values fixed and to obtain the corresponding *F* value again. It is advisable to do a value shift equalling the standard deviation, in the unfavourable sense.

The safety factor value obtained from this new calculation will be F^-. With these data, the previously existing *F* value and its new value, the sensitivity index, *v*, is defined as the value:

$$\nu = \frac{F - F^-}{F^*}$$

where F^* is a centred estimation of *F*.

Comment: *In actual fact, the sensitivity index of* F *to the design value* x *of a specific design factor would be the one defined by the following mathematical expression:*

$$\nu = \frac{\sigma}{F_m} \cdot \frac{\partial F}{\partial x}$$

where: σ *= standard deviation of* x*;* F_m *= average value of* F.

The expression proposed earlier for evaluating v *is a simplification that can be used to ease calculations.*

Alternatively, if greater precision is required, the sensitivity could be determined using smaller increments and reductions of the design factor value ($\pm \alpha\, \sigma$ *with* $\alpha<1$*) and then amplifying the effect adequately (dividing the difference in the* F *values by the* α *factor).*

It would also be possible and even advisable, as it does not involve any great additional effort in these calculations, to run two rather than one sensitivity analyses, by changing the design factor in both directions - favourable and unfavourable - in order to obtain F^+ and F^- and to calculate:

$$\nu = \frac{F^+ - F^-}{2F_m}$$

It must be up to engineers to decide on the required precision, according to the accuracy of the data and the importance to be assigned to these sensitivity analyses.

3.3.10.4 Combining One-dimensional Sensitivities

Having done the preceding calculations, engineers must combine the results obtained.

The one-dimensional sensitivities to be combined are indicated further below, because they depend on the type of the corresponding load combination. In any case, after having defined the partial results to be combined, the simplest and recommended way of obtaining the joint value representative of all of them is by using the expression:

$$\nu_F = (\Sigma\, \nu_i^2)^{1/2}$$

Comment: *The above expression does not presuppose any probabilistic structure in the corresponding data and can be used in a general way. When it is assumed that the terms in which the design factors intervene for defining* F *exhibit a lognormal distribution, as some authors suggest, it may be more convenient to compound the sensitivities by the following rule:*

$$\ln (1 + \nu_F^2) = \Sigma \ln (1 + \nu_i^2)$$

3.3.10.5 Centred Value of the Safety Factor

The safety factor value that used to be calculated in conventional geotechnical practice -when factors for increasing loads and reducing resistances were not used- is the one usually known as the most probable value, F^*, in texts on probability and reliability methods in geotechnical engineering. But this attribution should not be considered strict, as there is still no scientific rigour in these matters.

Perhaps when the probabilistic methods have been developed further and gained more general consensus, this aspect will be refined and it will then be possible to define more accurately which central reference value should be used.

Unless specific guidelines are available, sensitivity studies should use a centred value, F^*, as previously estimated by Level I calculation methods.

The centred value F^* for the safety factor depends on the load combination and is defined below in each of the following subsections.

3.3.10.6 Calculation for the Quasi-permanent Combination

The Level I calculation for a quasi-permanent situation and for a specific failure mode leads to the safety factor F that should be obtained as indicated in this ROM 0.5. This value is also considered as a centred value $F = F^*$.

In major works, whenever it is advisable to run the orderly sensitivity analyses indicated in this subsection, the most relevant design factors should be changed one by one. They will normally be the geotechnical parameters defining the strength and a few others linked to potential uplift or to certain actions.

Once the different sensitivities, v_i, have been calculated and all compounded as indicated in Subsection 3.3.10.4, the coefficient of variation of *F* will be obtained and, with it, the reliability index and the corresponding failure probability, as indicated in 3.3.10.9.

3.3.10.7 Calculating Each One of the Fundamental Combinations

The Level I calculation performed previously, prior to commencing the sensitivity analyses, will serve as the basic reference for studying the failure probability corresponding to each individual combination of this type.

For each variable action to have been considered as principal or dominant, a calculated value will exist for *F*. To run sensitivity studies, the procedure needs to be repeated, increasing by a standard deviation the value of the principal variable load defining the combination under study. When the standard deviation of the load value is not known, the procedure described below can be adopted.

For geotechnical failure modes, known as type GEO, the load factor normally used for the Level I calculation will be 1.3. At this stage and in order to repeat the calculation, the load factor will be increased to 1.5 and a lower safety factor obtained. The difference from the earlier value will be ΔF_a.

Comment: In regular civil engineering works, including those within the EHE field of application, a factor for variable loads γ_q = 1.5 is normally used. This factor depends on the target reliability index, which is normally β = 3.8, during the works design life. The partial reliability usually introduced in loads, β_A, is in the order of 70% of the total intended, i.e., β_A = 2.7. To introduce this safety, the load value needs to be multiplied by the partial coefficient:

$$\gamma_q = 1 + \beta_A \cdot v$$

where v is the coefficient of variation of the load value. Equalling this γ_q value to the one previously indicated (γ_q = 1.5) provides a rough estimate of the coefficient of variation of the load, implicit in this procedure. This coefficient is in the order of v = 0.5 / 2.7≅ 0.2.

As the intention, in line with the simplified method being described, is to increase the action value by a standard deviation, this increase would be achieved when the load factor is increased by two tenths; from the 1.3 value that served as basic reference for calculating F *it would go to 1.5 -the value suggested for calculating the desired coefficient of sensitivity.*

The *F* value, calculated by the Level I procedure described in this ROM 0.5 is not the centred or most probable value when fundamental combinations are concerned, since factored loads have already been considered in their calculation. To take this into account, the value needs to be risen up to the value of F^*, which would be a more useful centred value, namely:

$$F^* = F + 1{,}5\ |\Delta F_a|$$

where $|\Delta F_a|$ is the absolute value of the variation in *F* brought about by increasing the fundamental load by a standard deviation, as calculated earlier.

Comment: The 1.5 coefficient used in this expression is the partial reliability that is introduced into the actions, when they are factored as stated in this ROM 0.5, and corresponds to the normal uncertainty indicated above in the previous comment. For other cases, an adequate factor will have to be used.

This first step will produce the coefficient of sensitivity of *F* to the design value of the action, the first of the sensitivity coefficients needing to be calculated. This fundamental coefficient is denominated:

$$\nu_a = \frac{|\Delta F_a|}{F^*}$$

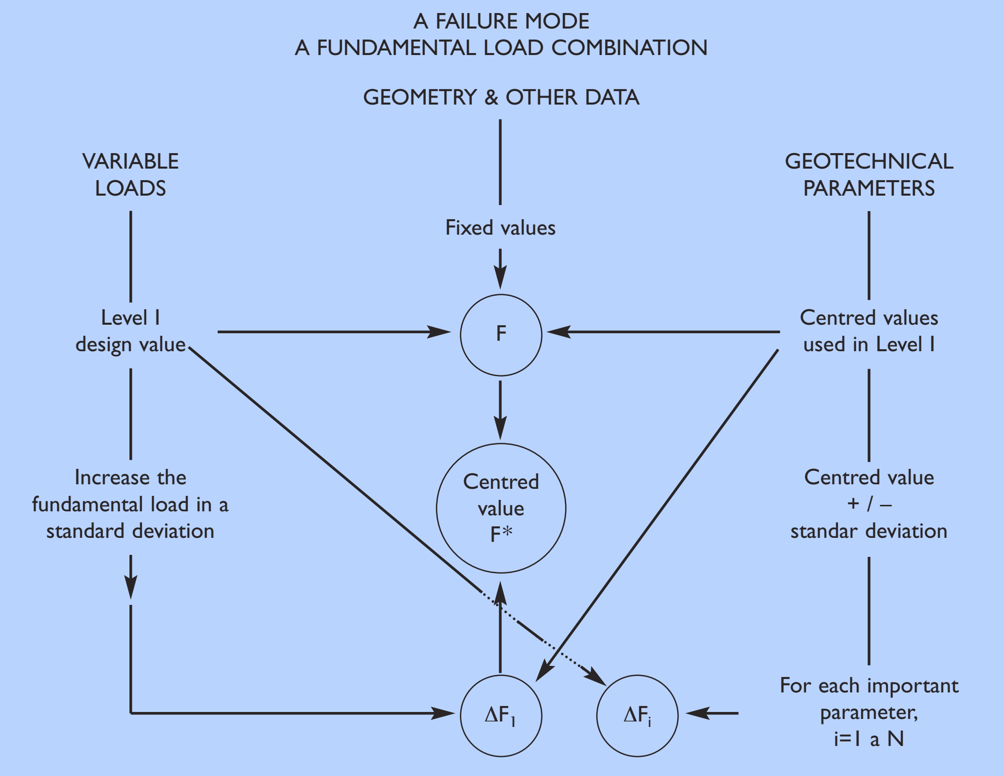
The next step should be to calculate the sensitivity of F to the variability of the geotechnical parameters of interest, which would normally be those corresponding to the strength definition and the weight. Each parameter of interest should be varied, one by one, in these sensitivity analyses. Each of them must keep the variable loads multiplied by the nominal load factor (1.3 in the normal cases covered in this ROM 0.5). For each factor i that is changed, a variation in the safety factor ΔF_i will be obtained. When this difference is divided by the safety factor F^* (already known prior to making the change), the corresponding value for the sensitivity, $v_i = \Delta F_i / F^*$, is obtained.

The sensitivities to the load and to the other parameters must be combined quadratically (square root of the sum of the squares) in order to obtain the desired sensitivity, v_F, which is the coefficient of variation of F when the considered predominant load is acting.

Figure 3.3.2 is a schematic representation of this procedure.

This whole procedure must be repeated for each fundamental load combination, each one dominated by a principal agent. Few predominant agents will normally prove to be critical and consequently need to be taken into account in the calculations. A prior selection of the ones that could be critical would save analysis time.

Figure 3.3.2. ***Possible Order of Parametric Studies (Sensitivity Analyses)***



Sensitivity to the load $\qquad \nu_a = \dfrac{\Delta F_a}{F^*}$

Sensitivity to the parameter $\qquad \nu_i = \dfrac{\Delta F_i}{F^*}$

Joint Sensitivity $\qquad \nu_F = \left(\nu_a^2 + \sum \nu_i^2\right)^{1/2} \quad (i = 1, 2, \ldots N)$

3.3.10.8 Calculating Accidental or Seismic Combinations

Exceptional combinations, whether they be accidental, seismic or of any other type, will have been analysed and their safety assessed through the corresponding safety factor *F*.

To estimate the failure probability of the works caused by one of these load combinations, a calculation should be carried out of the sensitivity to the value of the actual action characterising the situation. To this end, the corresponding action should be increased by a value equal to the standard deviation. Engineers will need to investigate this value. If no specific information is available, the standard deviation value can be assumed to be equal to 13% of the nominal value of the extraordinary load.

Comment: *The nominal value, A_N, of an exceptional action is usually associated with a low probability of occurrence during the design life of the works. Its definition already contains a certain implicit reliability, so it could be expressed as:*

$$A_N = A\,(1 + \beta_A \cdot v_A)$$

where β_A is the partial reliability mentioned above, v_A the corresponding coefficient of variation and A *a centred value of the load.*

This partial reliability is the approximate equivalent of the one introduced when the variable actions are increased in the structural codes (γ_q = 1.5). This would give:

$$1 + \beta_A \cdot v_A = 1.5$$

The uncertainty in the value of the exceptional action is assumed here, in the absence of other criteria, to be v_A = 0.20 as in a typical case of uncertainty in the variable loads. Making these assumptions, the standard deviation of the exceptional action would be:

$$\sigma_A = v_A \cdot A = \frac{v_A}{1 + \beta_A \cdot v_A} \cdot A_N = \frac{0.2}{1.5} \cdot A_N = 0.13\ A_N$$

which is the value recommended in this sensitivity calculation.

Increasing the value of the extreme load in a standard deviation, the absolute value of the change in the coefficient, ΔF_a, will be calculated and, with this, the coefficient of sensitivity corresponding to the load:

$$v_a = \frac{\Delta F_a}{F^*}$$

The centred value of the safety factor F^* for this type of combination will be:

$$F^* = F + 2.7\ |\Delta F_a|$$

where $|\Delta F_a|$ is the absolute value of the difference in the safety factor obtained when the load value is changed and *F* is the value of the safety factor obtained prior to this change.

The 2.7 coefficient of the previous expression is based on certain assumptions (see the above comments where β_A = 2.7). Engineers should adapt this coefficient when other assumptions apply. Next, by modifying in a standard deviation, on the unfavourable side (and also on the favourable side if more precision is required), each individual ground parameter (mainly the strength parameters) and other possible factors of interest, the respective modifications of the safety factor, ΔF_i, will be obtained, which will enable the corresponding individual sensitivity coefficients, v_i, to be calculated. The sensitivities should be compounded as indicated above (Subsection 3.3.10.7).

3.3.10.9 Confidence Index and Failure Probability

Having calculated the approximate value for v_F and ascertained the centred value, F^*, of the safety factor, the confidence index β and the corresponding failure probability for the combination under study will be obtained.

The reliability index, β, in respect of a failure mode and for a particular type of combination of actions, is defined as the number of standard deviations that the centred value, F^*, needs to me displaced to reach theoretical failure (F = 1).

Calculating the reliability index requires assuming a particular structure for the probabilistic distribution of F^*. The recommendation made here is to assume a lognormal distribution. This produces a β reliability index given by the following expression:

$$\beta = \frac{\ln F^*}{\zeta} - \frac{1}{2}\zeta$$

where ζ is the standard deviation of *lnF,* given by the expression:

$$\zeta = \sqrt{\ln\left(1 + \nu_F^2\right)}$$

and where ν_F is the value defined in the preceding subsections.

Having carried out the sensitivity study indicated for each load combination, the confidence index corresponding to the failure mode in question should be chosen to be the lowest of the indices found, irrespective of its being the one for the quasi-permanent combination or that for some of the fundamental or accidental or seismic combinations.

There exists a one-to-one correspondence between the failure probability, p, and the reliability index, β, which can be mathematically expressed by the following equation:

$$p = \phi(-\beta) = 1 - \phi(\beta)$$

where φ is the normalized form of the cumulative normal probability function.

The φ function can be found in many texts and calculating routines (including EXCEL). Table 3.3.4 gives some typical values for this function.

Table 3.3.4. β and p values

β	p	β	p	β	p	β	p
0.0	0.5	2.0	2.3×10^{-2}	3.0	1.3×10^{-3}	0	0.5
0.5	0.31	2.1	1.8×10^{-2}	3.1	0.97×10^{-3}	1.30	0.10
1.0	0.16	2.1	1.4×10^{-2}	3.2	0.67×10^{-3}	1.64	0.05
1.2	0.13	2.3	1.1×10^{-2}	3.3	0.48×10^{-3}	2.32	10^{-2}
1.4	0.081	2.4	0.82×10^{-2}	3.4	0.33×10^{-3}	3.09	10^{-3}
1.5	0.067	2.5	0.62×10^{-2}	3.5	0.23×10^{-3}	3.72	10^{-4}
1.6	0.055	2.6	0.47×10^{-2}	3.6	0.16×10^{-3}	4.27	10^{-5}
1.7	0.045	2.7	0.35×10^{-2}	3.7	0.11×10^{-3}	4.75	10^{-6}
1.8	0.036	2.8	0.26×10^{-2}	3.8	0.072×10^{-3}	5.20	10^{-7}
1.9	0.029	2.9	0.19×10^{-2}	3.9	0.048×10^{-3}	5.61	10^{-8}

3.3.10.10 Verifying Safety

Having completed the calculations as indicated, in addition to the safety factors F_1, F_2 and F_3, the supplementary data of the corresponding failure probabilities will also be obtained. A failure probability value will have been associated with each individual safety factor calculated by the Level I method, after finishing these simplified reliability calculations.

In the most general cases and for low SERI works (between 5 and 19) in which the increases in reliability associated with geotechnical failure modes are not too difficult to achieve or are not too far off the economically interesting ones, the safety against these failure modes can be taken as verified when the F_1, F_2 and F_3 coefficients obtained in line with the criteria set in this ROM 0.5 are greater than the minimum thresholds it recommends. Under normal conditions, in addition, the failure probabilities associated with these safety factors will remain below the value of 10^{-3}, so that these failures can be considered non-principal and therefore their contribution to the calculation of the works overall failure probability can be disregarded, thereby enormously simplifying the failure diagrams.

If any of the failure probabilities should prove to be higher than the maximum recommendable, engineers must analyse the situation and either raise the safety of the works or else assess its effect on the calculation of the works overall failure probability.

The contrary strategy is not to be recommended: lowering the safety of works without meeting the minimum requirements for F_1, F_2 and F_3, based on a simplified calculation of the failure probability like the one described here. The probabilistic calculation that may justify failing to meet the safety thresholds fixed in terms of the calculation using Level I procedures should be more exact and more detailed than the simplified calculation described in this subsection.

The indications given in this section are also valid for works with a minor SERI rating (<5) or a high SERI rating (≥20), considering in these cases the minimum safety factors associated with ULS failure probabilities of 10^{-2} and 10^{-4} respectively, obtained from the F_1, F_2 and F_3 coefficients defined in this ROM 0.5 in line with the criteria given in Subsection 3.3.8.2.

These failure probability values have been considered here with a particular value to serve as reference for arranging the calculations in order. They should not be taken to imply any real possibility of wreckage of the works.

In any case, engineers are advised that the current state of the art of probabilistic calculations for the subjects covered in this ROM 0.5 is still not fully developed for the cases of low failure probability under study, since the results are far too oversensitive to the shapes of the statistical distribution tails assigned to the design factors. And these shapes are not well known at the present time. Engineers must bear this fact in mind when checking the different aspects involved in safety.

3.4 SEEPAGE AND CONSOLIDATION

3.4.1 Most Common Geotechnical Problems

The most common geotechnical problems associated with water flow inside the ground (seepage) involve studying:

- seepage discharge rate
- water pressures on structures
- lowering of the groundwater table
- safety against bottom heave
- potential internal erosion.

These topics also include:

- soil mass consolidation
- generation and dissipation of porewater pressures

as, although they are mixed fluid flow-deformation problems, they are basically governed by the permeability of the ground, a crucial aspect of the problems covered in this section.

These topics are of particular interest in the design of dry docks, where permanent dewatering has to be performed, and also in the case of hydraulic fills, which are usually employed as backfill for gravity quay walls.

Problems of bottom heave and particle migration are typical in the dewatering works normally carried out -on a temporary basis- to enable dry construction beneath the groundwater table. The basic principles that should guide the study of this type of situation are covered in the following sections

3.4.2 Drawing Flownets

3.4.2.1 Homogeneous and Isotropic Ground. Two-dimensional Problems

Seepage problems in porous media not deformable by water flow (i.e., soils or fills in drained conditions) are usually solved by admitting the validity of Darcy's law. This assumes a laminar regime, as defined in Subsection 2.2.6 (Permeability). A particular problem is solved analytically when the potential ϕ is known at any point in the seepage domain. The *potential* (or *head*) value is given by the expression:

$$\phi = \frac{u}{\gamma_w} + z$$

where:

u = pore water pressure.
γ_w = unit weight of water.
z = elevation over a reference plane.

Where the ground is homogenous and isotropic and both the fluid and the ground are incompressible, the theoretical approach to the problem leads to Laplace's equation:

$$\Delta \phi = 0$$

The lines connecting points of equal potential are known as *equipotential lines* (or ground water contours). In isotropic media, any line that, throughout its length, intersects them orthogonally will be a streamline. The combination of both line sets, usually plotted in a way to delimit elements of similar dimensions in both directions, is known as a flownet.

A large number of analytical or semi-analytical solutions exist for solving seepage problems in homogenous and isotropic media which can help to solve real cases with simple geometry, where it would not be necessary to draw flownets.

Drawing exact flownets can prove difficult and computer programs are needed for creating an accurate net in complex situations. However, many seepage problems can and should be solved using simple approximations. The additional use of more detailed procedures, other than the simple methods recommended here, would only be justified when the importance of the problem requires this.

3.4.2.2 Anisotropic Ground

When permeability is clearly anisotropic, the problem can be solved by applying the Samisöe transformation, which can be found in the specialised literature on the subject. It essentially consists of plotting the net with a different geometry, in which the horizontal dimensions (assuming that the greatest permeability occurs in this direction, which is normally the case) are reduced by a proportion equal to the square root of the quotient of permeabilities, $(k_h/k_v)^{1/2}$.

Having plotted the squared flownet in this more slender figure where the heights are more pronounced (or the breadths more reduced), the change in scale needs to be reverted in order to obtain the real flow lines and equipotential lines. These will compose a system of curvilinear, non-orthogonal quadrilaterals with sides of different length along equipotentials and streamlines.

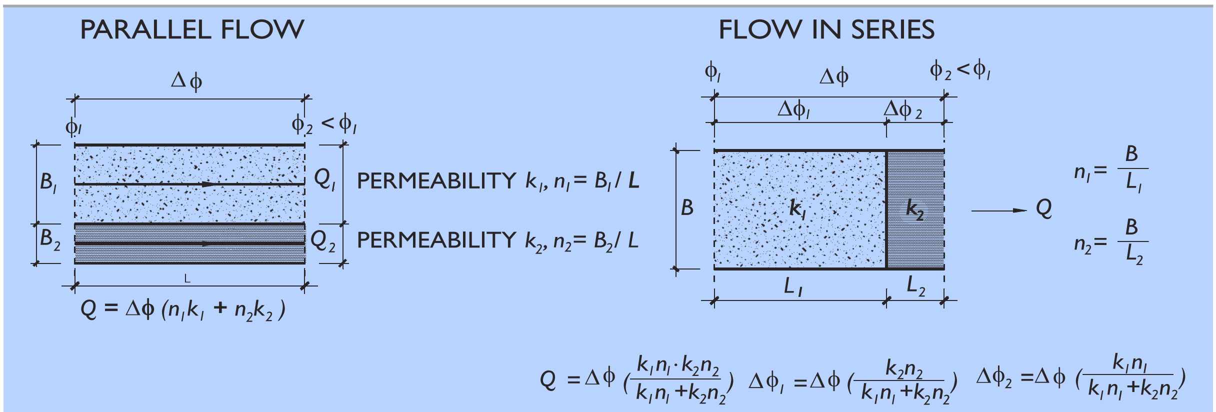
3.4.2.3 Heterogeneous Ground

The most frequent case consists of heterogeneous ground formed by strata or zones with different permeability. In these cases it is possible to carry out a approximate plot of the flownet. This is particularly easy when the boundary separating two media with different permeability coincides either with a streamline or with an equipotential line, since two independent parallel flows then occur (first case) or else a flow in series with an immediate solution, as illustrated in Figure 3.4.1.

Real cases, particularly those involving materials with very different permeability, can be approximated by composing the flows through each of the materials with dissimilar permeability, either by independent parallel flows or by serial flows.

It is always possible, and indeed advisable, to try out a simple solution using this procedure, making sure to choose the simplifications so that a conservative solution results. Figure 3.4.2 illustrates a simplified solution for the typical case of seepage through the permeable berm of a gravity quay.

Figure 3.4.1. Simple Seepage in non-Homogenous Media



3.4.2.4 Three-dimensional Problems

When the three-dimensional nature of the problem is very pronounced and the two-dimensional solution applied to different plane sections of the works is not considered very appropriate by engineers, they will need to resort to a three-dimensional study of the flownet, which will generally require the use of a computer and a suitable seepage analysis program.

Given that relatively little effort is required for plotting flat flownets compared to that needed for three-dimensional calculations, use of the latter should not exclude the analysis of some flat sections, albeit for guidance purposes at least, which will serve to demarcate the solution to the problem.

3.4.3 Seepage Discharge Rates

3.4.3.1 Problems in Two-dimensional, Isotropic and Homogenous Ground

For two-dimensional seepage problems in homogenous and isotropic soils and when the water flows between two extreme equipotential lines (groundwater table in the backfill and in the front face of a wall, for example), the seepage discharge per unit of length in the direction perpendicular to the study plane can be expressed as:

$$Q = \Delta\phi \cdot k \cdot n$$

where:

$\Delta\phi$ = head loss between intake and discharge.
k = permeability.
n = dimensionless number depending on the geometry of the medium.

Bearing in mind that permeability normally is not known accurately, it is rare that the calculation of a discharge rate, or other variables associated with seepage, requires a great deal of effort for determining *n*.

The dimensionless number *n* is the quotient between the total number of *flow tubes*, N_f, and the total number of head drops, N_p, in squared flownets, i.e.:

$$n = \frac{N_f}{N_p}$$

The examples included in the figures from this Section 3 can serve as examples for determining the dimensionless number *n*.

3.4.3.2 Anisotropic Ground

In anisotropic problems, the seepage rate is given by the expression:

$$Q = \Delta\phi\ (k_h \cdot k_v)^{1/2} \cdot n$$

where *n*, as in isotropic cases, is the quotient between the number of flow tubes and the number of head drops in flownets plotted as indicated in 3.4.2.2 and where:

$\Delta\phi$ = head loss between intake and discharge.
k_h and k_v = permeability in the two principal anisotropic directions.

3.4.3.3 Heterogeneous Ground

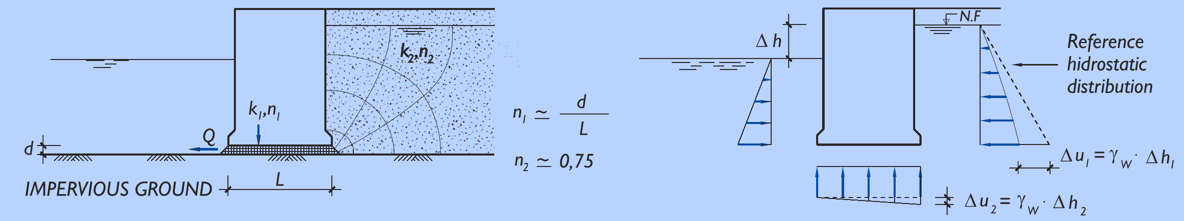
In the case of heterogeneous ground but with extensive homogenous zones, the flow rate passing through each of these zones can be estimated using the expressions shown above (3.4.3.1 or 3.4.3.2, depending on whether the medium is isotropic or not). In these cases, the head loss between intake and discharge will be taken to refer to the beginning and to the end of the homogenous zone under study, with the *flow tubes* and head drops being the ones contained within this homogenous zone.

Examples of flow rate calculations for non-homogenous situations are given in Figures 3.4.1 and 3.4.2.

3.4.3.4 Three-dimensional Problems

The calculations for three-dimensional problems are more complex, although the type of expression relating the total discharge rate to the permeability and head loss continues to be the same.

Figure 3.4.2. ***Simple Solution for Seepage under a Quay***



$$Q = \frac{1}{\gamma_w} n_1 \cdot k_1 \cdot \Delta u_1 = \frac{1}{\gamma_w} n_2 \cdot k_2 \cdot \Delta u_2$$

$$\Delta u_1 + \Delta u_2 = \Delta u = \gamma_w \cdot \Delta h$$

$$\text{Para} \quad k_1 = 1 \frac{cm}{s}, \quad k_2 = 10^{-2} \frac{cm}{s}, \quad \frac{d}{L} = 0{,}1, \quad \Delta h = 3 \text{ m}$$

$$\Delta h_1 = 2{,}8 \text{ m} \qquad \Delta h_2 = 0{,}2 \text{ m} \qquad Q = 0{,}2 \text{ l/s m}$$

In these cases, the number *n* has dimensions of length and a computer or the use of published analytical solutions will generally be required to determine it.

The Appendix to Part 2 of this ROM 0.5 gives solutions for some three-dimensional cases used in interpreting *in situ* permeability tests which could also be useful for studying other three-dimensional seepage problems.

3.4.4 Porewater Pressures on Structures

The walls of structures affected by water seepage are subjected to pressures with a value differing from the hydrostatic value (at-rest water).

The determination of these pressures will generally require a previous drawing of the flownet. Structural materials are normally much less permeable than the ground and therefore the corresponding flownets can usually be plotted by assuming that the contact between the structure and the ground is an impervious boundary of the problem and hence coincides with a seepage line.

On the occasions where the permeability of the structure is similar to that of the ground, the flownet should be plotted considering the problem as heterogeneous and entering the structure as one more material.

Having plotted the flownet, the water pressures at the interface between the soil and the structure (or the pressure due to water) are easy to estimate, as:

$$u = (\phi - z)\, \gamma_w$$

where z is the elevation of each point above the reference plane used to define the potential ϕ.

One example of estimating water pressure distributions on a caisson of a gravity quay is illustrated in Figure 3.4.2. The example from Figure 3.4.3 corresponds to the pressure acting on the sheet piles of a shored excavation with dewatering.

The water pressure shown in these figures acts directly on the contact face between the retaining structure and the ground. Sometimes it will be necessary to calculate the water pressures at other points. This is the case when calculating water pressure in order to apply the calculation methods described in Subsections 3.7.5 and 3.7.7 for estimating active and passive earth pressures on retaining structures, respectively. In such cases, it is necessary to know the porewater pressures along some reference planes close to the slide planes of the active or passive wedges.

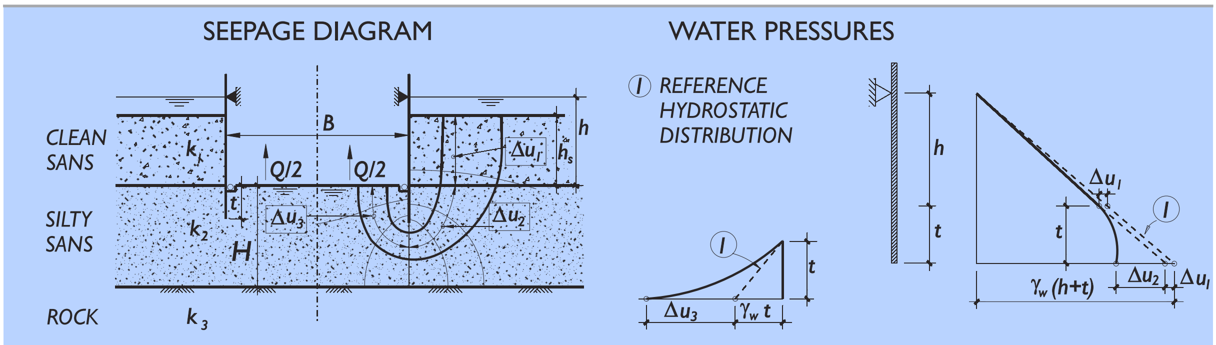
These water pressures, which will generally be somewhat greater than those on the wall face, can easily be calculated once the flownet is known.

3.4.4.1 External Water Levels and Saturation Lines in Natural Ground and Fills

The water levels to take into account for obtaining the water pressure on structures, which can be assigned a stationary nature -in terms of the response of the works and of the ground-, will be determined by the long-period oscillations of free water, the existence of incoming underground water, the structural type of the works, the permeability of the foundations, natural ground, fills and works and, where applicable, by the type and capacity of the drainage systems and other ways of artificially varying water levels planned in the design, as also the tolerances admitted for these cases in the Particular Technical Specifications.

Engineers should fix the representative water levels in each design state or situation and for each type of load combination considered, whenever possible, on a statistical basis or through experiments, especially when there is significant underground water flow coming from land, artesian pressure or constant exposure to wave action.

Figure 3.4.3. **Solution for a Shored Excavation with Dewatering**



Level of External Free Water

The high and low levels of external water in the coastal areas that can be considered in relation to the works response and are stationary in nature are essentially caused by the combination of astronomical tides, meteorological tides, seiches (long waves), coastal undercurrents (wave-setup) and hydraulic pattern of the fluvial currents in estuaries, river mouths and ports.

In cases where the water level is not considered jointly with other climatic loads (waves, currents, wind, etc.) forming a group of dependent variable loads, nor is the water level considered as an accidental load, the values given in Table 3.3.1 should be taken as representative external water levels as a function of the type of load combination considered. Specific representative levels in the different Spanish port areas are given in ROM 0.3.

Additionally, in sheltered areas, either natural (bays or estuaries) or artificial (harbour basins), engineers need to be particularly careful to check the possibility and frequency of resonance phenomena owing to long-wave penetration. In these cases, levels may alter by as much as 3 m and will need to be made compatible with the representative levels generally defined for external waters.

Level of Saturation Line in Fills and Natural Ground

As a general rule, in seas with a significant astronomical tide, the level of the saturation line in fills and natural ground can be assumed to remain constant at the mean sea level + 0.3 m starting from a distance of some 20 m off the coastline. For seas without a significant astronomical tide, this level will coincide with the mean sea level. For fluvial currents, whether affected or unaffected by tides, or tidal zones subjected to fluvial currents, the saturation line will coincide with their mean low water or flood levels depending the seasonal period. These levels can rise considerable in the presence of:

- Underground water incoming from land or water coming from direct rainfall, whose drainage is hampered or prevented by vast coastal structures.
- Artesian pressures.
- Low-permeability soils or rocks in the backfill of retaining structures or in the foundation ground.
- Hydraulically created fills.
- Continuous wave action on the resistant structure, fill or natural ground.
- Artificial systems supplying additional water or drainage.

In the absence of relevant statistical or experimental data, water levels in a fill or natural ground in the ground-structure interface and compatible with the representative levels adopted for external water as a function of the load combination considered (see Table 3.3.1), are shown in a simplified manner for the most usual cases in Table 3.4.1, as a function of the long-period oscillations of the free water and of the permeability of foundations, fills and works. A factor not taken into consideration is the action of groundwater flow, wave action or other natural or artificial forms of modifying the levels. For the cases not covered in the table, the levels must be determined on a statistical or experimental basis.

In the particular case of a fill created hydraulically and as a precaution against the possibility that the drainage capacity is exceeded during dumping, a permanent saturation level that is reasonably higher than the lowest level above which water can flow freely could be taken in unfavourable cases for the Construction Stage.

Incoming water originated exclusively from rain falling directly on the external surface of the ground should be taken into account in unfavourable cases by assuming that it produces additional rises in the groundwater tables of fills and natural ground. In the absence of other data, the only factor considered relevant for the calculations should be level rises owing to rain falling directly on highly permeable backfills of non-permeable structures founded in ground with low permeability or with poorly permeable intermediate strata. This situation should only be taken into consideration for fundamental or infrequent load combinations, adopting a level rise equal to the one required to accommodate the volume corresponding to the maximum 24-hour precipitation rate for a 5-year return period.

Artificial Variation in Levels of External Water and Saturation Lines in Fills or Natural Ground

In the case of locked basins or other areas subjected to artificial variations of the external water level, the maximum and minimum nominal water levels should be fixed according to the operating criteria laid down in the design. Similarly, the design must state the maximum rises and drops anticipated in a 24-hour period.

When incoming water of artificial sources enters the natural ground or the fill, and in the case where artificial drainage systems are adopted, the maximum and minimum groundwater oscillation levels should be determined according to the specific characteristics of each case. If the causes of the external oscillations are artificial water sources and these are the only ones wetting a fill or natural ground, in the absence of other data it could be considered that the groundwater table will coincide with that of the external water in the long term, following its oscillations with the maximum delays given below as a function of the works permeability:

Table 3.4.1. Maximum Differences to Be Adopted between Representative Levels of Free External Water and Saturation Lines in Natural Ground or Fills

				Type of Load Combination: Quasi-Permanent ans Seismic (S)	Fundamental and infrecuent, when water level are non-predominant	Fundamental and infrecuent, when water level are predominant	Frecuent and Accidental (A)
Differences between Representative External Water Levels and Saturation Lines in Ground or Fills	Sea with Significant Astronomical Tide	Works Permeability	Low	EWL – MSL			
			Medium	0.3 (MHW – MLW)	0.3 (HAT – LAT)		0.3 (MHWS – MLWS)
			High	0.15 (MHW – MLW)	0.15 (HAT – LAT)		0.15 (MHWS – MLWS)
	Sea without Significant Astronomical Tide	Works Permeability	Low	0	EWL – MSL		0
			Medium	0	0.30 m		0
			High	0	0.15 m		0
Differences between Representative External Water Levels and Saturation Lines in Natural Ground or Fills	Areas With Astronomically Significant Tide Subjected To Fluvial Currents	Works Permeability	Low	EWL – (MeanLW or MFL)			
			Medium	0.3 (mean tidal range with MeanLW or mean tidal range with MFL, respectively)	0.3 (24-hr rise or drop in river level corresponding to a 20-yr return period + mean tidal range for spring tides in low-water or flood situation respectively)	0.3 (24-hr rise or drop in river level corresponding to a 50-yr return period + mean tidal range for astronomical spring tides in low-water or flood situation respectively)	0.3 (24-hr rise or drop in river level associated with a 50% non- exceedance probability + mean tidal range in low-water or flood situation respectively)
			High	0.15 (mean tidal range with MeanLW or mean tidal range with MFL)	0.15 (24-hr rise or drop in river level corresponding to a 20-yr return period + mean tidal range for spring tides in low-water or flood situation respectively)	0.15 (24-hr rise or drop in river level corresponding to a 50-yr return period + mean tidal range for astronomical spring tides in low-water or flood situation respectively)	0.15 (24-hr rise or drop in river level associated with a 50% non- exceedance probability + mean tidal range in low-water or flood situation respectively)
	Corrientes fluviales no afectadas por mareas	Works Permeability	Low	EWL – (MHWS or MFL)			
			Medium	0	0.3 (24-hr rise or drop in river level corresponding to a 20-yr return period in low-water or flood situation respectively)	0.3 (24-hr rise or drop in river level corresponding to a 50-yr return period in low-water or flood situation respectively)	0.3 (24-hr rise or drop in river level associated with a 50% non- exceedance probability in low-water or flood situation respectively)
			High	0	0.15 (24-hr rise or drop in river level corresponding to a 20-yr return period in low-water or flood situation respectively)	0.15 (24-hr rise or drop in river level corresponding to a 50-yr return period in low-water or flood situation respectively)	0.15 (24-hr rise or drop in river level associated with a 50% non- exceedance probability in low-water or flood situation respectively)

KEY: EWL: representative external water level; MSL: mean sea level; MHW: mean high water; MLW: mean low water; HAT: highest astronomical tide; LAT: lowest astronomical tide; MHWS: mean high water springs; MLWS: mean low water springs; MeanLW: mean low-level in fluvial currents; MFL: mean flood level in fluvial currents.

N.B.:

For the purposes of this table, the works permeability should be qualified as follows:

- Low – when the structure permeability is low, and the works and/or fill also have low permeability.
- High – when the fill, structure and foundation have high permeability.
- Medium – in all other cases.

Furthermore, the following ideas are used for qualifying the permeability:

- Fills or natural ground of **high permeability** have a permeability coefficient of $k > 10^{-3}$ cm/s;
- Fills or natural ground of **low permeability** have a permeability coefficient of $k < 10^{-5}$ cm/s;
- **Permeable structures** are the ones whose permeability does not present physical interruptions cutting off the flow of water on attaining certain elevations (e.g., in retaining structures with weep holes)

- **High permeability:**

  delay equalling 0.15 times the maximum foreseeable variation in the external level over a 24 hour period.

- **Medium permeability:**

  delay equalling 0.30 times the foreseeable maximum variation in the external level over a 24 hour period.

- **Low permeability:**

  the groundwater table of the fill or natural ground is not considered to undergo any variation when the level of the external water is temporarily altered.

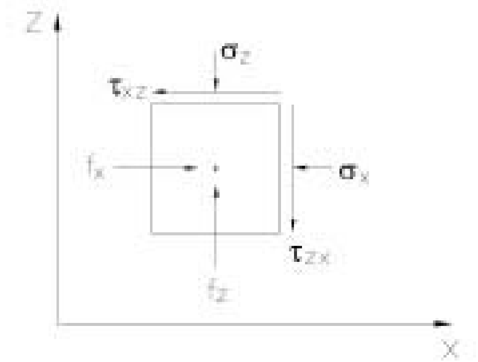
The potential drop in levels owing to drainage systems being implemented in fills or natural ground should only be taken into account if the particular drainage system adopted allows its working operation to be checked and cleaning and correction measures adopted at any time. When drainage is committed to small-scale elements such as weep holes or clapper valves, the cracking elevation should be taken to be 0.30 m above their actual position (this value corresponds to the minimum water head required for them to work). In these cases, a minimum drop of 1 m between the external water and the groundwater table of the fill or natural ground should be considered.

3.4.5. Seepage Forces

The stress equilibrium inside the ground, as formulated by two-dimensional continuum mechanics disregarding acceleration, leads to the following two basic expressions:

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xz}}{\partial z} = f_x$$

$$\frac{\partial \sigma_z}{\partial z} + \frac{\partial \tau_{zx}}{\partial x} = f_z$$



where τ_{xz} is the shear stress in horizontal and vertical faces and σ_x, σ_z are the normal stresses –positive in compression– along the axes. It is assumed that body forces f_x, f_z exist per unit of volume. In normal circumstances, $f_x = 0$; $f_z = \gamma$ = unit weight of the ground.

When the ground is saturated, to analyse the effective pressures and the water pressures separately, Terzaghi's principle can be applied ($\sigma' = \sigma - u$) which, substituted in the equations above, leads to:

$$\frac{\partial \sigma'_x}{\partial x} + \frac{\partial \tau_{xz}}{\partial z} = f_x - \frac{\partial u}{\partial x}$$

$$\frac{\partial \sigma'_z}{\partial z} + \frac{\partial \tau_{zx}}{\partial x} = f_z - \frac{\partial u}{\partial z}$$

In the problem of water seepage in porous media, the basic variable normally used is the potential ϕ (also denominated *h* –from *head*- in many texts) as defined by the expression:

$$\phi = \frac{u}{\gamma_w} + z$$

With this definition, the following results:

$$\frac{\partial u}{\partial x} = \frac{\partial \phi}{\partial x}\gamma_w \qquad \frac{\partial u}{\partial z} = \frac{\partial \phi}{\partial z}\gamma_w - \gamma_w$$

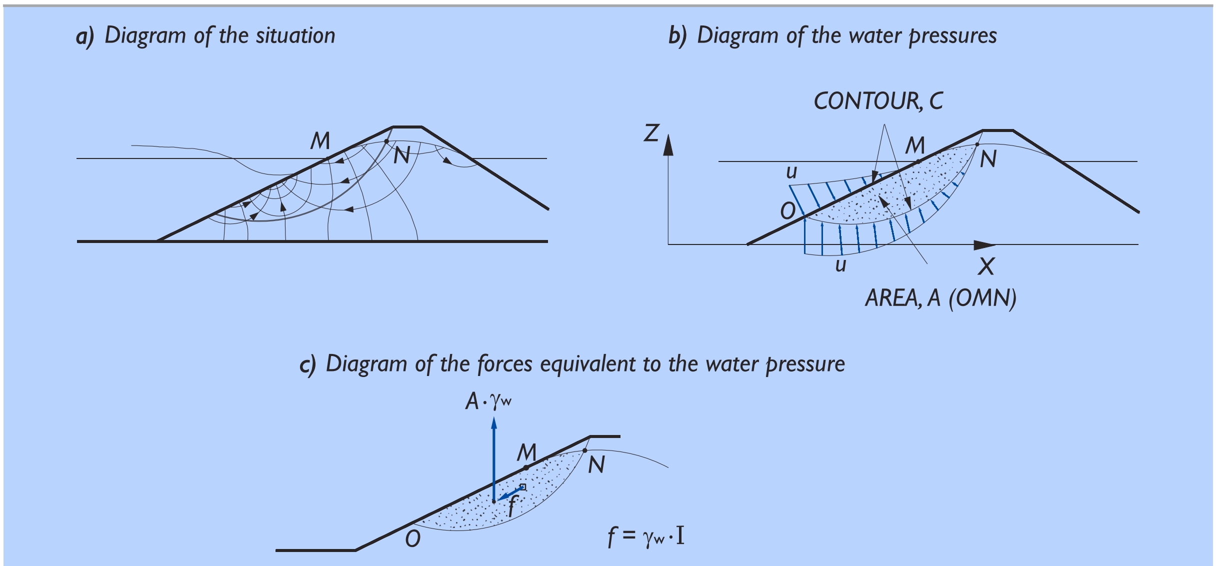
The presence of water makes the ground undergo buoyancy with an intensity of γ_w and its movement, when acceleration is negligible, also induces a body force.

$$\bar{f} = \left(f_x, f_z\right) = -\gamma_w \cdot \overline{\text{grad}}\ \phi$$

The product of the water unit weight and the opposite of the potential gradient (sometimes denominated *I*) is normally known as seepage force ($\gamma_W \cdot I$). It is perpendicular to the equipotential lines and in the direction of decreasing potentials.

The practical application for the concept of seepage forces lies in the stress analysis of submerged soil masses, like the example shown in Figure 3.4.4.

Figure 3.4.4. Schematic Flownet and Corresponding Seepage Forces (Laminar Regime)



N.B.: The vertical buoyancy force A γ_w can be entered by calculating the weights of area *A* using the value γ' (submerged unit weight).

The effect of water pressures (free and porewater) on the mass contour is a load *F*, whose value can be calculated using the integral:

$$\bar{F} = \int_c u \cdot \overline{dl}$$

where, logically, is a vector normal to the contour delimiting the mass in question and whose modulus is the differential of its length. This is called the contour integral of *u*.

Assuming that *u* has a harmonic solution, as assumed all along these ROM 0.5 sections, its contour integral equals the integral of its divergence

$$\left(\frac{\partial u}{\partial x}, \frac{\partial u}{\partial z}\right)$$

within the surface area enclosed by the contour:

$$\overline{F} = \iint_A \left(\frac{\partial u}{\partial x}, \frac{\partial u}{\partial z}\right) \cdot dx \ \ dz$$

which, bearing in mind the above expressions, leads to:

$$\overline{F} = -\gamma_w \iint_A \overline{grad} \ \ \phi \ \ dx \ \ dz + A\gamma_w \cdot \overline{k}$$

where is the unit vector along the z axis.

The water pressures acting on the entire (external and internal) contour, therefore, are equivalent to the sum of the forces:

a. Buoyancy equal to the weight of the fluid displaced by the mass - this effect can be represented by subtracting the value of $A\gamma_w$ from the total weight $A\gamma_{sat}$,i.e., by calculating the weights with the submerged unit weight $\gamma' = \gamma_{sat} - \gamma_w$

b. Distribution of seepage forces, , equal to $I \cdot \gamma_w$, as indicated above per unit area (per unit volume in three-dimensional problems).

It may sometimes be advisable to sidestep the problem of contour pressures and instead use submerged specific weights (in the submerged zone) and the seepage forces indicated. This may simplify certain analyses, as shown farther down in the different cases to which this ROM 0.5 applies.

3.4.6 The Problem of Bottom Heave

Calculations associated with dewatering, either in excavations or for permanent or transient dewatering installations, are usually aimed at analysing the possible discharge rate or water pressures on structures, as already mentioned. These calculations could also be aimed at studying the heave at the bottom of excavations - their most common Ultimate Limit State.

Comment: Ground failure at the bottom of excavations subjected to intense vertical head gradients is sometimes known in Spanish practice as "sifonamiento" *(piping). In this ROM 0.5 this term is reserved for another problem, the one caused by particle entrainment or internal erosion.*

The solution to the problem should be obtained by consulting the technical references available on solved cases with a similar geometry or, if the geometry of the problem is not easy to find, by specifically drawing the corresponding flownet. Figure 3.4.3 includes a simplified solution for a typical case of an excavation with dewatering inside an enclosure supported by sheetpile walls. The corresponding analytical formulae are given in Figure 3.4.5.

It is recommended to calculate safety against bottom heave in this type of shored excavation by comparing the submerged density and the vertical component of the discharge gradient.

$$F = \frac{\gamma'}{I_v \cdot \gamma_w}$$

It is advisable to calculate the discharge gradient, I_v, as the average of the unit head loss in the zone adjacent to the sheetpile at the bottom of the excavation. When the soil is homogenous, it is sufficiently conservative to assume that:

$$I_v = \frac{\Delta u_3}{\gamma_w \cdot t}$$

where:

Δu_3 = excess porewater pressure at the tip of the sheetpile.

t = depth of sheet pile embedment.

Figure 3.4.5. ***Solution for Seepage inside a Sheetpile Enclosure***

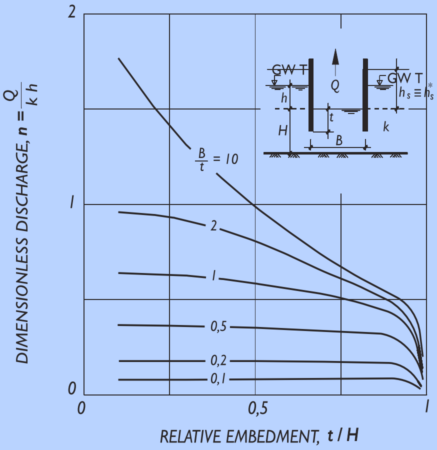
CALCULATION OF THE EXCESS PRESSURE AT THE TIP OF THE SHEETPILE, Δu_3

$$\Delta u_3 = \frac{\gamma_w \ h}{1 + \left(1 + \frac{h_s^*}{t}\right)^{1/3}} \quad \text{if } B > t \text{ or } \Delta u_3 = \gamma_w h \text{ if } B < 0.2\, t$$

where:

$$h_s^* = h_s \cdot \frac{k_2}{k_1} \quad \text{if } h > h_s \quad \text{or} \quad h_s^* = h \cdot \frac{k_2}{k_1} \quad \text{if } h > h_s$$

A linear interpolation between the values shown for Δu_3 can be made for intermediate values of B.



CALCULATION OF THE DEWATERING DISCHARGE

Enclosure with parallel walls	$Q = k_2 \cdot h \cdot n$ (per unit of length)
Square enclosure	$Q = 1{,}4 \cdot k_2 \cdot h \cdot n \cdot B$
Circular enclosure	$Q = 1{,}25 \cdot k_2 \cdot h \cdot n \cdot B$

N.B.: Graph data correspond to the case $h_s = 0$. Para $h_s > 0$, the discharges are slightly lower.

A warning should be given concerning the risk involved in cases where there is a less permeable zone lying under the excavation bottom. Much higher water pressures may be generated at the bottom of the impermeable stratum than those corresponding to a homogenous ground. In such cases this gradient can reach:

$$I_v = \frac{h}{d}$$

where:

h = total head drop.

d = vertical distance from the excavation bottom to the bottom of the less permeable stratum through which the sheetpiles pass.

From the point of view of safety rating, the problem of heave in the bottom of shored excavations should be treated as an Ultimate Limit State, ULS, of geotechnical nature, GEO, since the configuration of the ground and its characteristics are determining factors in the problem.

Consequently, the possible "loads" should be used with their nominal values when they are permanent and adequately factored (γ_q = 1.3 in fundamental combinations) when they are variable. One fundamental design factor in these cases is the total head drop *h*, or difference in level between the groundwater table on the exterior and the interior of the enclosure. As pointed out in Subsection 3.3.5, both for the purpose of defining the geometrical factors and the loads, the water levels should be considered permanent design factors and, as a result, should not be affected by load factors, regardless of whatever representative value is adopted for them as a function of the combination under consideration (see Table 3.3.1).

The minimum safety factors recommended for the problem of bottom heave are indicated in Table 3.4.2.

Table 3.4.2. Minimum Safety Factors against Bottom Heave, Low SERI Rating (5 - 19)

Load Combination Type	Safety Factor, F
Quasi-permanent, F1	1.5
Fundamental, F2	1.3
Accidental or Seismic, F3	1.1

For works with a minor or high SERI rating or for other allowable failure probabilities, the F values can be modified as indicated in Subsection 3.3.8.2.

These minimum safety factors should be taken into account provided they are obtained by this simplified calculation procedure or other similar ones deducible for other configurations or systems of excavation with dewatering.

3.4.7 Using Wells to Lower the Groundwater Table

In order to design wells for lowering the groundwater table, it is relatively common to apply analytical expressions. These solutions, which estimate the drop in the original groundwater table caused by permanent pumping, are as follows:

$$\Delta = \frac{Q}{2\pi kh} \ln\left(\frac{R}{D}\right) \qquad \text{(Confined flow)}$$

or

$$\Delta = \frac{Q}{2\pi kH^*} \ln\left(\frac{R}{D}\right) \qquad \text{(Unconfined flow)}$$

where:

Δ = drop in the groundwater level at a point outside the well.
Q = pumping discharge.
k = ground permeability.
h = thickness of the permeable zone (in cases of confined flow).
H^* = $H - 1/2\ \Delta$.
H = elevation of the original groundwater table over the impervious bottom (in the case of an unconfined aquifer).
D = distance from the point in question to the well axis.
R = radius of influence, or distance from the well axis to the vertical contour where no drop in the groundwater table occurs.

Since R is difficult to estimate, the following empirical expression is widely used:

$$R = 300\Delta_m \sqrt{k}$$

where:

Δ_m = maximum lowering of the groundwater level.
k = permeability coefficient in cm/s.

The drop caused by a set of several wells at a particular point can be estimated by the expression:

$$\Delta = \Delta_1 + \Delta_2 + \dots\dots\dots + \Delta_n \qquad \text{(Confined flow)}$$

or

$$\Delta^2 = (\Delta_1)^2 + (\Delta_2)^2 + \dots\dots\dots + (\Delta_n)^2 \qquad \text{(Unconfined aquifer)}$$

where:

Δ = lowering of the groundwater table achieved by a set of *n* wells when a discharge, *Q*, is extracted from each of them in steady state operation.

Δ_i = drop of the groundwater table achieved at the location of well *i*, when it continuously discharges a flow rate *Q* and the other wells are not operating.

Engineers should be warned about the possible inaccuracies of this expression and to the fact that more precise determinations will be needed with appropriate field tests whenever a reliable estimate of some dewatering aspect is required.

3.4.8 Particle Entrainment and Internal Erosion

Water flow through natural ground or artificial earth fills may originate particle migration. As a result, the ground (or the fill) undergoes internal erosion, changes its structure and can go so far as to collapse, giving rise to the failure of the corresponding part of the works. This problem is also known as *piping* because the erosion is frequently concentrated along certain preferential seepage routes ("pipes") that may even act like siphons.

The problem should be treated as a particular failure mode leading to an Ultimate Limit State, ULS, of the hydraulic type, HYD (see Subsection 3.3.1).

This ROM 0.5 does not lay out any procedure for calculating the internal erosion process. All it does is to indicate a set of preventive measures for avoiding it.

The problem can occur in any type of ground, but is particularly significant in artificial fill materials having to withstand water flow through the intergranular pores of their solid skeleton.

To prevent fines migration and the consequent internal erosion, appropriate protective filters should be installed, either made of artificial products (geotextiles) or of granular soil.

The primary quality for a soil to be used as a filter is uniformity. Poorly uniform soils subjected to a seepage flow can segregate in such a way that fine particles are entrained by the water through the voids in the larger particles (3). Unless specific laboratory tests are available, segregable soils are taken to be those with a coefficient of uniformity greater than 20:

$$C_u > 20$$

(3) This segregability condition can be made mathematically expressed by the following equation:

$$p_{crit} = \frac{15}{\log_{10} n} \cong 20$$

where:

p_{crit} = critical value of the slope of the granding curve.

n = critical value of the D_{15}/d_{85} quotient for which the filter condition is strictly obeyed - normally assumed to be n = 5.

The grading curve slope in the area between two sieves can be calculated with the expression:

$$p = \frac{p_2 - p_1}{\log \dfrac{D_2}{D_1}}$$

where p_1, p_2 are the percentages passing sieves with openings D_1 y D_2 respectively.

When the slope of the grading curve in any part is lower than the one indicated as critical, the soil should be considered to be especially susceptible to internal erosion. The corresponding fine fraction could move across the coarse fraction. These are the types of soil that can be segregated by the water seepage.

Granular soils (cohesionless) with a lower coefficient of uniformity than the one indicated can be used as filters for other granular soils provided that the following expression is obeyed:

$$\frac{D_{15}}{d_{85}} < 5$$

where:

D_{15} = sieve size corresponding to the filter.
d_{85} = sieve size corresponding to the soil to be protected.

If the fines content of the soil to be protected is higher than 10%, the corresponding filter size should be:

$$0.3 \text{ mm} \leq D_{15} \leq 0.5 \text{ mm}$$

The use of finer filters will only be justified in particular cases where highly dispersive soils (able to be dispersed into very fine particles) are to be protected from entrainment. In any event, it is not recommended to use finer filters than

$$D_{15} < 0.20 \text{ mm}$$

since they are difficult to obtain without any cohesion appearing.

To avoid entrainment, it may be necessary to install more than one filter in series until the average grain size of the final filter is stable with the estimated water velocity. This stability can be increased by installing a suitable support device (metal grids, concrete elements, etc.) on the outer face of the thickest material.

3.4.9 Entrainment, Scour and Other Types of External Erosion

The problem of the external erosion of natural ground or fills caused by water movement on their surface is one of the most investigated in the context of maritime and port works engineering as it is highly critical to their stability. Relevant examples are the scour that can be produced in the foundations of berthing or breakwater works and the entrainment in the armour layer of rubble-mound berthing or protective works and in the berms of vertical breakwaters.

Erosion occurs when the velocity of the water movement near the surface of the soil exceeds a certain threshold fixed by the soil's external erosion resistance.

The problem should be treated as a particular failure mode leading to an Ultimate Limit State, ULS, of the geotechnical type, GEO (see Subsection 3.3.1).

Calculating the profile for the water velocities in each case is a hydrodynamic problem that lies beyond the scope of this ROM 0.5.

The erosion resistance of soils can be studied in several ways. In a large number of situations, this resistance is expressed in terms of critical velocity. This velocity represents the minimum value from which the erosive process would commence.

Many authors have investigated the determination of velocities triggering erosion in different soil types, mainly using tests in laboratory models.

Rocks can withstand high velocities (> 6 m/s) before the breaking process starts. Good quality concrete can withstand similar velocities without any problems. But cohesive soils are barely capable of admitting velocities in the order of 1 to 3 m/s, even if they are firm.

Granular soils and rockfills base their resistance to erosion on their self weight. That is why their average size and specific weight are normally used as the representative resistance parameter with respect to erosion. The most common formulae for obtaining the critical velocity are of the following type:

$$v_{crit} = [A \cdot g\ (G-1)\ D_{50}]^{1/2}$$

where:

v_{crit} = critical velocity initiating the erosive process.
A = dimensionless parameter depending on the type of water movement and the shape of the particles.
g = gravity acceleration.
G = specific gravity of the soil particles (γ_s/γ_w).
D_{50} = size of the sieve allowing 50% of the weight of the soil to pass through.

It is advisable to consult the technical literature for ascertaining the values for the *A* parameter. For guidance purposes only, its value tends to be between 0.5 and 1.

When the erosive process is triggered, the soil is transported by the water as a bed load. The particles roll or slide, remaining permanently in contact with other particles that may detach from the moving mass.

When erosion occurs in the vicinity of an obstacle that causes a local rise in the velocity of the movement, substantially deep scour may occur locally, until a situation is reached in which the geometrical change itself caused by the erosion reduces the velocities and attenuates the erosive process.

It is possible to calculate erosion depths but the results are not reliable. As a general rule, when the problem can be critical, some protection system is arranged to stop the process from starting. In any event, the problem of defining erosion depth clearly depends on the type of works involved and more references to this will be found in the specific ROM publications and in Part 4 of this ROM 0.5.

3.4.10 Soil Consolidation

3.4.10.1 Basic Formulation

Soils with appreciable fines content tend to have sufficiently low permeability for the water flow caused by any imbalance to take a considerable time to reach a steady-state seepage or the corresponding hydrostatic regime. This delay can give rise to geotechnical problems, since the increase in shear strength that could be required for other purposes is also delayed, and will produce long-term deferred settlement.

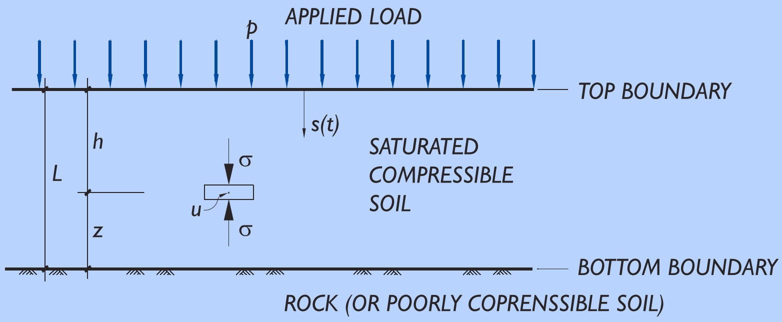
The problem that can be caused by dewatering or the lowering of the groundwater table is similar, since the change in the state of groundwater can cause settlement which, in turn, may affect nearby structures.

These problems are normally analysed by a simplified behaviour model. It implies the validity of Darcy's Law (defined in Subsection 2.2.6), Terzaghi's effective pressure principle (defined in Subsection 2.2.8) and the validity of a linear stress-strain relationship.

With these assumptions, it is possible to solve a series of situations with the semi-analytical expressions published in the technical literature. Only the broad principles of these solutions are mentioned in this publication.

The generic problem consists of ascertaining the state of the porewater pressures in the ground and the surface settlement, at any given time, *t*. The geometry of the soil deposits and their geotechnical characteristics should be known, as well as the external action causing the consolidation process. Figure 3.4.6 presents an approximate analytical solution to the theory of one-dimensional consolidation. It is also possible to use the semi-analytical solution usually included, with the help of tables and graphs, in elementary soil mechanics texts.

Figure 3.4.6. Diagram of the One-dimensional Consolidation Problem



PROBLEM DATA

- Thickness of the soil deposit, L
- Applied load p, and moment of application, t = 0
- Permeability of the rock compared wwith the soil: Case A: impermeable bottom. Case B: permeable bottom
- Oedometer modulus of the soil, E_m
- Coefficient of consolidation, C_v

INTERMEDIATE PARAMETERS

- Long term settlement, $s_\infty = \dfrac{pL}{E_m}$
- Degree of consolidation, $U(t) = \dfrac{s(t)}{s_\infty}$
- Time factor, $T_v = \dfrac{c_v}{H^2}\, t$

(Case A: $H = L$, Case B: $H = \frac{1}{2} L$)

BASIC SOLUTION

$$U = \sqrt{\frac{4 . T_v}{\pi}} \qquad (T_v < 0{,}286)$$

$$U = 1 - 0{,}811 . e^{-2{,}467\, T_v} \qquad (T_v > 0{,}286)$$

3.4.10.2 *Excess Porewater Pressure*

At the start of consolidation, the fact that a disturbance has occurred -in this case, a load *p* on the top boundary of the soft soil under study- means that the porewater pressure increases at all points by the same amount.

$$(\Delta u)_{t=0} = p$$

This pressure increase dissipates as time elapses, more rapidly close to the draining edges and more slowly in the centre of the layer (if it is drained through both faces) or the bottom edge (if this is impermeable).

The maximum pore water pressure inside the soft layer after a certain time has elapsed can be approximated by the expression:

$$\Delta u_{máx} = 1.5\ p\ (1 - U) \qquad U > 0.4$$

where:

p = load applied to the upper edge.
U = degree of consolidation as a fraction of 1.

Using this approximated expression it can be seen that, when 50% consolidation occurs (U = 0.5), the excess porewater pressure can be as high as 75% of the initial excess. This way of dissipating the pressure leads to a slower increase in the strength of a layer (governed by its weakest or less consolidated level) than the rate indicated by the average degree of consolidation, U.

The time required to achieve 50% consolidation (i.e., for settlement to be half of the long-term value expected) is:

$$t = 0.196 \frac{H^2}{c_v}$$

where:

H = the longest distance to the drainage.
c_v = the soil's consolidation coefficient.

It can be deduced from this expression that the distance to the drainage is the key geometrical element governing the rate of the consolidation process. The artificial introduction of sand drains to allow horizontal flow or the natural or artificial presence of more permeable sand layers substantially reduces the value of *H* (the solution in such cases is different but similar to the one shown, since it is governed by the same time factor) and therefore significantly speeds up the consolidation process.

The solution to the theory of consolidation with more complex geometries (including horizontal flow or three-dimensional problems or the use of non-linear stress-strain laws, etc.) must be sought in the specialised literature and may require the use of appropriate computer programs.

The particular case of consolidation with vertical artificial drains is considered in 3.9.2.4.

3.4.10.3 Subsidence Created by Dewatering

Lowering the groundwater table makes the effective pressure in the soil layers below the original groundwater table increase by:

$$p = \Delta \cdot [\gamma_w - (\gamma_{sat} - \gamma_{ap})]$$

where:

Δ = drop in the groundwater table.
γ_{sat} = saturated specific weight of the ground in the area where the groundwater table oscillates.
γ_{ap} = apparent specific weight of the ground in the area where the groundwater table oscillates after its lowering.
γ_w = unit weight of the water.

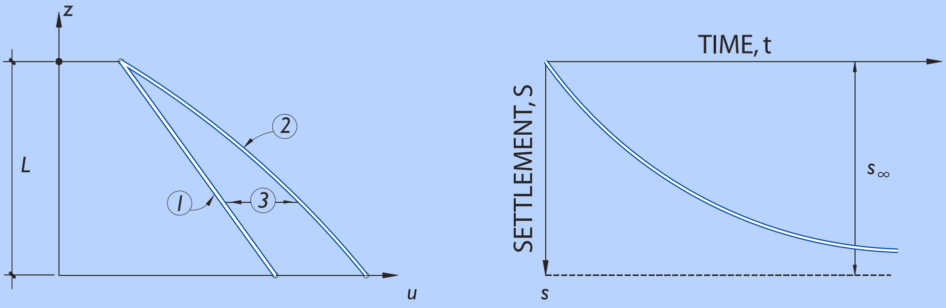
This load *p* compresses the soils below the groundwater table and, if the permeability and also the deformation modulus are low, long settlement periods may result.

Independently of whether the settlement is caused by lowering the groundwater table or by the application of a load, the consolidation process of the affected soil layer below is governed by the same conditions and therefore the same solutions can be applied.

It must be pointed out that if the solution described in Figures 3.4.6 and 3.4.7 is applied to analyse this groundwater lowering, the reference hydrostatic distribution to be considered should correspond to the lowered groundwater table.

Settlement caused by dewatering can be estimated using the same expressions as those corresponding to consolidation under a load *p*. Several observations follow with respect to such settlement.

Figure 3.4.7. ***Diagram of the One-dimensional Consolidation Problem (Contd.)***



PORE WATER PRESSURE DISTRIBUTION

SETTLEMENT PROFILE

1 Reference hydrostatic profile
2 Distribution at a moment in time, "t"
3 Excess pore water pressure, Δu

PORE WATER PRESSURE DISTRIBUTION

Case A: $\Delta u = 1{,}5\, p\, (1 - U)\, (1 - (\frac{z}{L})^2)$ $U < 40\ \%$

Case B: $\Delta u = 1{,}5\, p\, (1 - U)\, (1 - (\frac{2z - L}{L})^2)$ $U > 40\ \%$

EVOLUTION OF SURFACE SETTLEMENT

$s = s_\infty \,.\, U$

N.B.: In these expressions, *U* is the average degree of consolidation, depending on time as shown in Figure 3.4.6.

3.4.10.4 Consolidation Settlement

The long-term reduction in a layer thickness, *L*, owing to the increase in the pressure *p* on its top surface is:

$$s_\infty = \frac{p \cdot L}{E_m}$$

For this expression to be sufficiently exact, the load must be extensive enough for the problem to be considered one-dimensional.

The oedometer modulus, E_m, must be obtained from field or laboratory tests (oedometer tests) over the same range of pressures actually existing, i.e., between:

$$p_{initial}\ y\ (p_{initial} + p)$$

where:

$p_{initial}$ = the vertical effective stress prior to applying surcharge *p*.

As the vertical effective stress varies with depth, a representative average value can be considered, corresponding to the central point of the layer, or else it can be divided into several sections and the same principle subsequently applied to each one.

The simple solution referred to above is valid for homogenous soils with a single coefficient of consolidation, c_v.

$$c_v = \frac{k \cdot E_m}{\gamma_w}$$

The initial vertical pressure at each depth will differ and the oedometer modulus will therefore also vary with depth. Even so, it is acceptable to assume that c_v is constant. The variation in E_m can be offset by an opposite variation in permeability, *k*, and the assumption that the coefficient of consolidation is constant can thus be maintained (in fact the soil moduli can also be expected to increase and their permeability decrease as the consolidation process progresses).

Using these simple or simplified theories and with good laboratory data, it is difficult to reduce the margin of possible error in settlement calculations to less than ± 30%. The estimation of consolidation times is less accurate. Even with good laboratory tests, in the absence of previous similar experiences, it will not be easy to estimate times. Actual consolidation times in well-analysed situations can vary between half and double the value estimated. The predictions of consolidation calculations must be verified by subsequent observation or even with specific field tests, if this is an essential aspect of the works.

Field observation of the initial evolution of the surface settlement will not enable the degree of consolidation reached to be accurately ascertained nor therefore the total settlement that can occur, unless a long time is allowed to elapse and a clear trend towards stabilisation of the process can be seen. This occurs with high degrees of consolidation of around 80%.

More information will need to be obtained to interpret consolidation processes in less time and more accurately. Monitoring the evolution of porewater pressure at different depths is a practice to be recommended for these purposes.

Further recommendations are included in Subsections 3.9.2.5 and 4.9.7 on the observation and interpretation of the data obtained from monitoring consolidation processes.

3.4.11 Porewater Pressures Generated by Waves and Other Sea Level Oscillations

The action of waves and other sea-level oscillations, either direct or transmitted to the foundation ground through the resistant structure, generates variations in total and porewater pressures in the soil or fills and, consequently, also flow in some of the water saturating it.

Knowing these variations is particularly important for verifying the stability of many harbour structures (rubble-mound breakwaters, foundation berms, etc.). In turn, theoretical and experimental studies carried out reveal that waves during a storm can produce substantial increases in pore water pressures, in some particularly susceptible soils (e.g., fine sands in an undrained situation), which can lead to liquefaction, bottom heave or sliding in the ground. In these cases, engineers are advised to take special care to prevent these effects occurring.

This type of problem is difficult to solve, depending on the nature and the behaviour of the soil under the dynamic action of waves and other sea-level oscillations, and will generally call for the use of complex analytical or numerical models, whose study lies beyond the scope of this ROM 0.5 (see ROM 1.1). However, for some specific cases (uncoupled problems), approximate solutions are admissible, some of which are covered in this ROM 0.5.

3.4.11.1 Porewater Pressure Distribution in Seabeds

SIMPLIFIED ANALYTICAL SOLUTION. UNCOUPLED FLOW AND DEFORMATION

There are simple, analytical solutions to the problem of flow in a porous medium when it can be uncoupled from the deformation of the soil and also when the rate of water flowing into and out of the soil is small. In these simple situations, it can be assumed that the boundary condition on the surface of the seabed is the one corresponding to rigid and impervious ground.

This approximation can be considered to be sufficiently valid for the following two alternative assumptions:

a. The ground behaves as impervious, water is incompressible and consequently volume changes do not occur inside the ground. This assumption is normally described in geotechnical terms as an *undrained situation*.

b. The ground is permeable but non-deformable, so that volume changes do not occur either with water flow, even though changes do occur in effective pressures. This assumption was also made in the preceding sections for obtaining flownets based on harmonic solutions for the porewater pressure field.

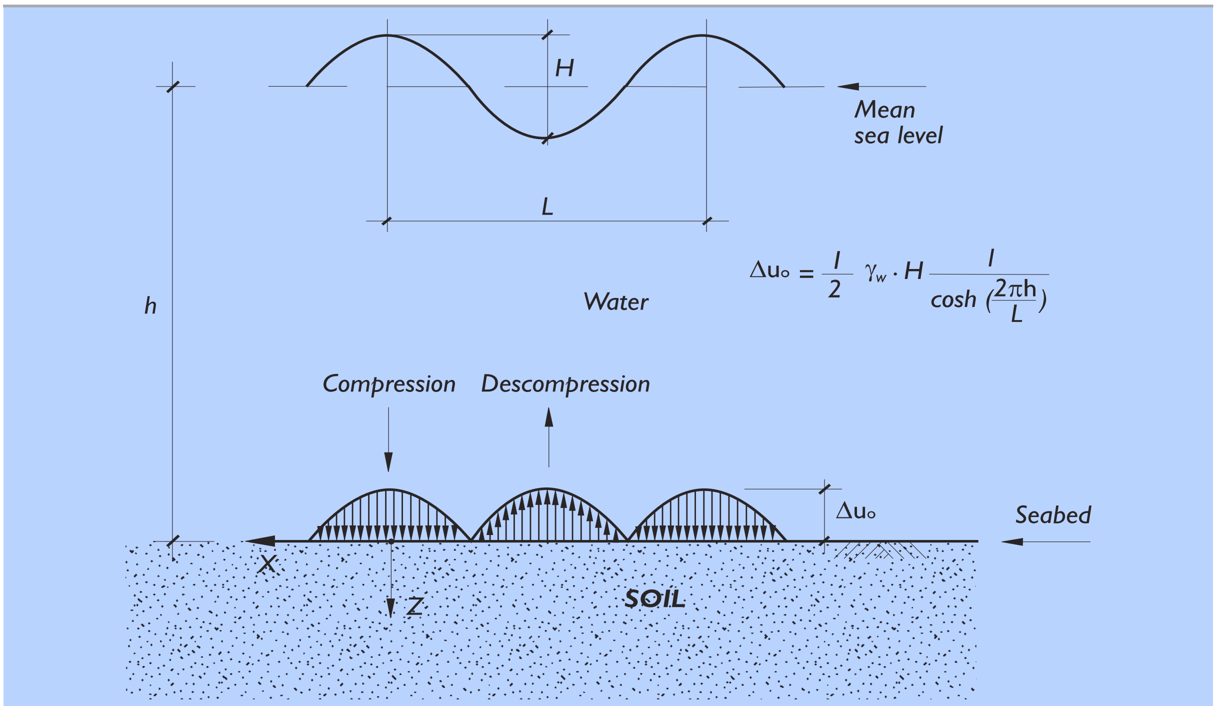
In either of these two extreme hypotheses, the pressure exerted by the water on the seabed can be supposed to be the one shown in Figure 3.4.8. That is, considering the linear theory of waves and a train of travelling waves, the increment of water pressure on the bottom at any particular location can be taken as:

$$\Delta u = \frac{1}{2}\gamma_w \cdot H \frac{1}{\cosh\left(\frac{2\pi h}{L}\right)} \cos\left(Kx - \omega t\right)$$

where:

H	=	wave height.
L	=	wave length (impermeable bottom).
T	=	wave period
h	=	depth at the site.
K	=	wave number ($2\pi/L$).
ω	=	angular frequency ($2\pi/T$).
γ_w	=	specific weight of the water.
x	=	abscissa of the point in question, measured from the vertical of a wave crest.
t	=	time.

Figure 3.4.8. **Increment in Pressure on the Seabed Due to a Train of Travelling Waves (Rigid and Impervious Ground)**



With these assumptions, the approximate solution, in terms of total, neutral and consequently also of effective stresses, is independent of the mechanical characteristics of the soil and the water. This solution is described below.

If the accelerations are neglected, the solution for the mechanical problem has an analytical expression in total pressures. This solution for a seabed of indefinite depth is as follows:

$$\Delta\sigma_x = \Delta u_0\,(1 - Kz)\,e^{-Kz} \cdot \cos\,(Kx + \omega t)$$
$$\Delta\sigma_z = \Delta u_0\,(1 + Kz)\,e^{-Kz} \cdot \cos\,(Kx + \omega t)$$
$$\Delta\tau_{xz} = \Delta u_0 \cdot Kz \cdot e^{-Kz} \cdot \mathrm{sen}\,(Kx + \omega t)$$

The Mohr's circle corresponding to the increase in total pressure induced by this wave train has the following values for its centre and radius (average stress *p* and deviatoric stress *q*):

$$p = \Delta u_0 \cdot e^{-Kz} \cdot \cos\,(Kx + \omega t)$$
$$q = \Delta u_0 \cdot Kz \cdot e^{-Kz}$$

As can be seen, the size of the Mohr's circle is independent of time and of the abscissa of the centre of the circle, depending only on the depth. In addition, for $K_z = 1$, it reaches a maximum value of $q_{max} = \Delta u_0/e$, where *e* is the base of the natural logarithms (*e* = 2.718).

Profile of Porewater Pressures in Very Thick Homogenous Beds

The above solution in total pressures is independent of the characteristics of the ground. All that is needed for the formulae indicated to be valid is that the ground be homogenous and isotropic, one of the two above conditions a) or b) are fulfilled and the accelerations are negligible.

In the solution of interest corresponding to the undrained condition (constant volume of the soil skeleton), the following expression is obeyed:

$$\Delta u = p = \Delta u_0 \cdot e^{-Kz} \cdot \cos\,(Kx + \omega t)$$

The wave-induced changes in effective stresses would be:

$$\Delta\sigma'_x = -\Delta u_0\,Kz \cdot e^{-Kz} \cdot \cos\,(Kx + \omega t)$$
$$\Delta\sigma'_z = \Delta u_0\,Kz \cdot e^{-Kz} \cdot \cos\,(Kx + \omega t)$$
$$\Delta\tau_{xz} = \Delta u_0\,Kz \cdot e^{-Kz} \cdot \mathrm{sen}\,(Kx + \omega t)$$

As can be seen, the Mohr's circle corresponding to this variation in effective pressure is always centred on the origin and its radius varies solely with depth, as stated previously.

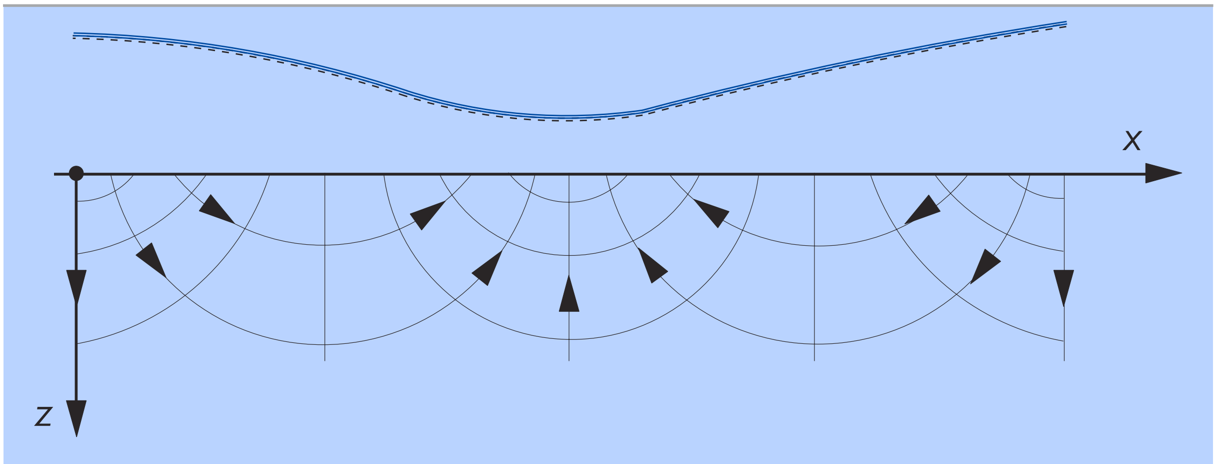
On the ground surface (z = 0), the vertical effective pressure and the shear stress are zero. The vertical gradients are $(\partial q/\partial z) = K \cdot \Delta u_0$ for the shear stress induced beneath the trough of the wave and γ' for the vertical effective pressure. This means that the maximum wave induced value of τ/σ'_v can be deduced as:

$$\frac{\tau}{\sigma'_v} \leq \frac{K\,\Delta u_0}{\gamma'} = \frac{2\pi\ \ \Delta u_0}{L \cdot \gamma'}$$

This analytical solution is applicable when analysing the failure of sandy seabeds subjected to cyclic loads (Section 3.10), whose failure criterion is formulated in terms of τ/σ'_v.

The flownet set up in the ground is shaped as shown in Figure 3.4.9.

Figure 3.4.9. **Instantaneous Flownet Corresponding to Wave Action on a Homogenous and Isotropic Seabed**



The equations for the isobars with equal excess pore water pressure and the orthogonal streamlines are:

$$e^{-Kz}\cos Kx = \frac{\Delta u}{\Delta u_0} \qquad \text{(Isobars)}$$

$$e^{-Kz}\text{sen}Kx = \frac{\Delta Q}{k\,\Delta u_0\ \ \gamma_w} \qquad \text{(Streamlines)}$$

where Δu is the pressure increase of the corresponding isobar and ΔQ is the value of the flow rate increase corresponding to the streamlines.

The flow rate in a stretch as long as the wave length is:

$$Q = \frac{2k\ \ \Delta u_0}{\gamma_w}$$

where *k* is the coefficient of permeability.

The seepage gradient has a constant modulus at each depth.

$$I = \frac{2\pi\ \ \Delta u_0}{\gamma_w \cdot L} e^{-Kz}$$

and its orientation makes one complete revolution with each time increment T.

The hydraulic gradient at the surface that could cause bottom heave is $I = \gamma'/\gamma_w$ where γ' is the unit weight of the submerged soil. Comparing this critical gradient with the maximum gradient obtained from the preceding expression (which occurs for z = 0 under the trough of the wave) leads to the critical bottom heave condition:

$$\Delta u_0 = \frac{\gamma' L}{2\pi}$$

Using the relationship existing between Δu_0 and the wave data gives:

$$\frac{\pi H}{L} \leq \cosh\left(\frac{2\pi h}{L}\right) \cdot \frac{\gamma'}{\gamma_w}$$

as the necessary condition for preventing this.

EFFECT OF THE SOIL'S COMPRESSIBILITY AND PERMEABILITY

The real seabed state does not generally obey the simplifying assumptions mentioned above and therefore the ground's deformability and permeability enter the picture, as also its drainage capacity in relation to the period of the acting oscillatory load. In these more general conditions, the dimensionless parameter governing the process is:

$$C = \frac{c_v T}{D^2}$$

where

c_v = consolidation coefficient.
D = characteristic dimension.
T = sea-level oscillation period.

The characteristic dimension, *D*, required to obtain the dimensionless parameter *C* is theoretically the distance from the point in question to the drainage. This distance could be one of the following:

- Depth of the point in question under the seabed.
- Thickness of the soil layer if this has a bottom boundary (rock for instance) and this boundary is impermeable.
- Half of the soil thickness of the item above if both faces of the layer in question are drained.

When the value of *C* is clearly greater than 1, it can be assumed that the evolution of the pore water pressures on the bottom is entirely reproduced and without any lag inside the ground. This will generally occur with long-period or very long-period oscillations in highly permeable and very stiff ground. In this case, the porewater pressures generated can be obtained by drawing the corresponding flownet in steady-sate flow conditions.

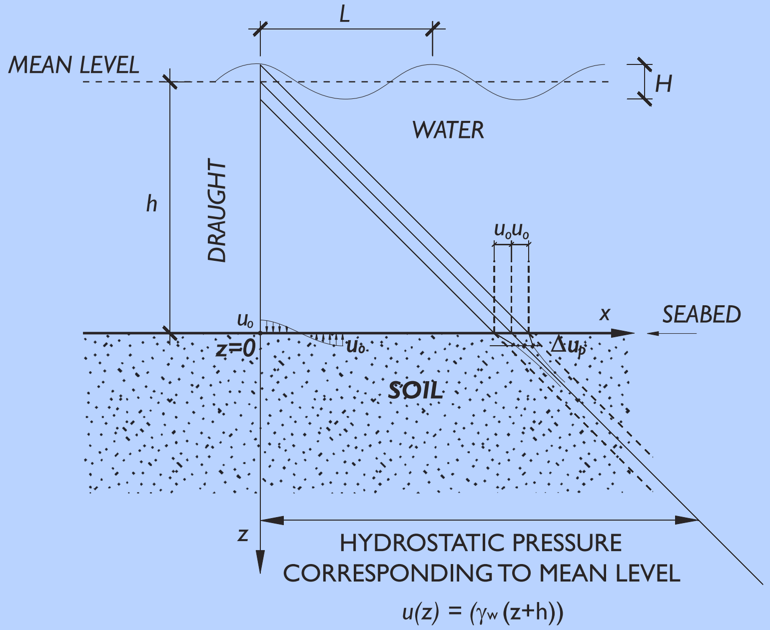
When the value of *C* is in the order of 0.01 or below, it can then be assumed for practical purposes that the soil does not drain and that the porewater pressures generated are those indicated in the preceding subsection for zero-permeability ground (undrained situation).

For intermediate situations, the coupled flow-deformation problem in a porous medium will need to be analysed with an analytical or numerical model or else by prototype observation using monitoring techniques (see ROM 1.1). Nevertheless, given the complexity of these computations, in these cases it may be advisable to carry out the geotechnical calculations in total stresses. This has the advantage of not requiring the estimation of the porewater pressure distributions.

One simplified solution to this problem is given by de Rouck (1991) and its use is somewhat widespread. It is summed up here in Figure 3.4.10. With this formulation the porewater pressures depend on the relative depth (h/L), and they stabilise in the value of the hydrostatic pressure corresponding to the mean level. This happens from a certain depth, which depends of and increases with permeability, soil compacity and the period of sea-level oscillation. Thus, the variations in porewater pressures induced by tides decrease very slowly with depth in permeable ground while, on the other hand, they decrease more rapidly in less permeable ground. On the contrary, the porewater pressures caused by waves decrease much more rapidly in all types of ground and consequently affect a more limited soil thickness.

According to the indications in this section, the soil behaviour under the action of travelling waves (period in the range of 5 to 20 s) will generally be in undrained conditions for saturated fine-granular soils. Medium sands can be considered in partially drained conditions and coarse sands, gravels and rockfills in totally drained conditions. For longer-period sea-level oscillations (tides for example), soil behaviour will generally be in drained conditions for granular soils and rockfills and in undrained conditions for cohesive soils.

Figure 3.4.10. Estimated Variation in the Porewater Pressure Profile in a Drained Seabed under the Action of Waves or Other Sea-level Oscillations (de Rouck, 1991)*



KEY

$$u(z) = \gamma_w (z + h) + \Delta u_p$$

$$\Delta u_p (z) = u_0 e^{-Az} \cos\left(\frac{2\pi x}{L} + \frac{2\pi t}{T} - Az\right)$$

where:

Δu_p = porewater pressure deviation in relation to the hydrostatic pressure corresponding to the mean level.

u_0 = amplitude of the variation of the pore water pressure in the seabed.

$$u_0 = \pm \gamma_w \frac{H}{2}, \text{ for tides}$$

$$u_0 = \pm \frac{\gamma_w \cdot H}{2 \cosh\left(\frac{2\pi h}{L}\right)}, \text{ for waves admitting the linear wave theory}$$

γ_w = specific weight of the water.
h = seabed depth in relation to mean water level.
L = wave length of sea-level oscillation.
T = period of sea-level oscillation.

$$A = \left(\frac{\gamma_w \cdot \pi}{k \cdot E_m \cdot T}\right)^{0,5}$$

k = coefficient of permeability.
E_m = soil's oedometer modulus.
x = horizontal coordinate.
z = vertical coordinate.
t = time.

*: de Rouck, J. 1991. "De Stabilitet Van Stortsteengolfbrekers. Algemeen Glijdingsevenwicht - Een Nieuw Deklaagelement", Hydraulic Laboratory, University of Leuven, Belgium.

3.5 SHALLOW FOUNDATIONS

3.5.1 Foundation Types

Shallow foundations are those where the contact plane between the structure and the ground lies below the ground surrounding the structure at a shallow depth compared with the breadth of the foundation. In fact, when the foundation depth is of the same order as the foundation breadth, the formulae and procedures given here may already be too conservative.

The ground surrounding the foundations will normally be at a similar elevation on both of their sides. The foundations of retaining structures with a large difference in ground elevation from one side of the foundations to the other can also be analysed by the procedures described here. In any event, the ground elevation around foundations to consider in the calculations specified here should be the lowest possible next to any of their sides.

There are several basic types of shallow foundations, including those described below.

a. Isolated or Tied Footing Foundations

These are typical in the foundations of buildings or structures supported by columns. The tie-beams connecting the different elements do not usually have any substantial effect on reducing the vertical loads acting in each footing and for the purposes of bearing failure and settlement, therefore, the footings can be considered individually.

b. Rigid Strip Foundations

These are typical in the foundations of walls or gravity retaining structures (blockwork or caisson quay walls, for example).

The stiffness of the structure means that the deformation of the structure itself can be neglected in settlement calculations.

c. Flexible Strip Foundations

These are typical in the foundations of structures supported by pillars on ground with a low bearing capacity not permitting isolated foundations. They can also be interesting from a constructional or even economical point of view. They are equivalent to the foundations described in a) with tie beams and footings integrated into a single element, which would become a continuous beam or footing. Strip beams can extend in a single direction, tied by beams or otherwise, or in two or more directions, each set crossing and tying the others.

It can generally be expected that the effect of soil-structure interaction will play an important role in the foundation load distribution and this effect must therefore be taken into account in this type of foundation.

d. Slab (or Raft) Foundations

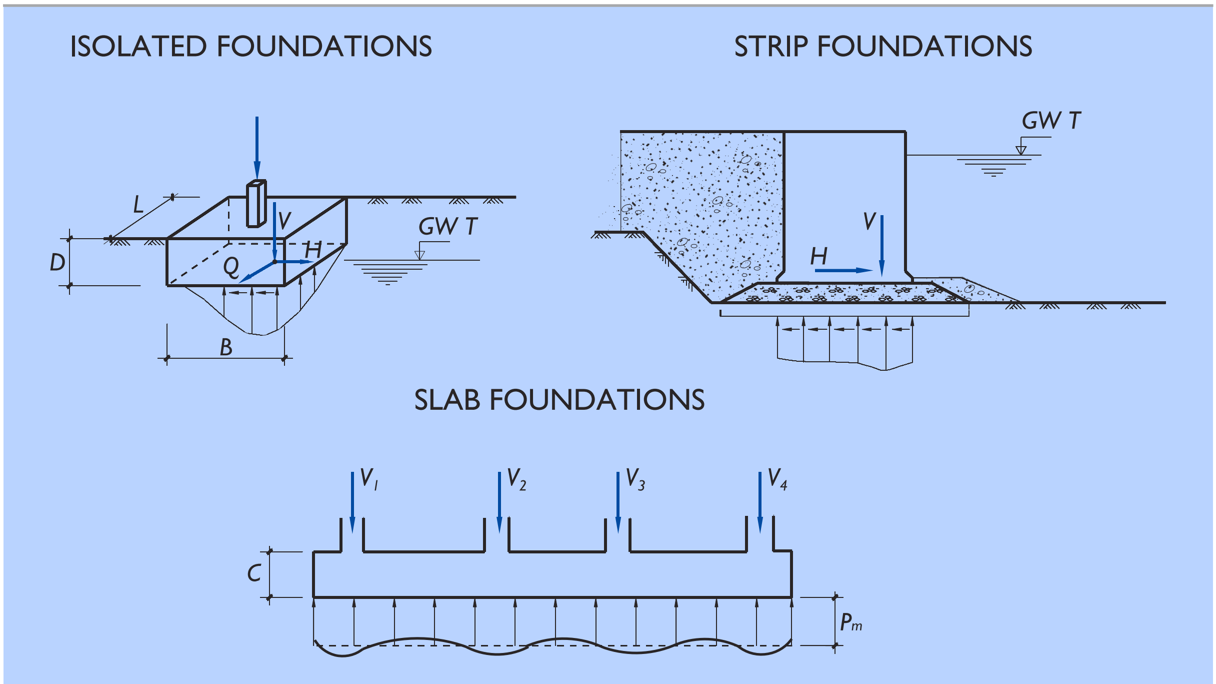
This is a common solution in the case of soils with too low a bearing capacity for using isolated foundations or strip footings. Raft solutions can also be adopted for other, very varied reasons.

Slabs can be of constant or variable thickness and even ribbed. The thickness of concrete slabs normally used requires the soil-structure interaction to be taken into account in the foundation structural design.

These foundation types are illustrated in Figure 3.5.1.

Semi-deep foundations, where the minimum depth is several times the breadth, can be studied using the procedures indicated here and disregarding part of the embedment in the ground. Special procedures for assessing the improvement due to additional depth can be found in the existing technical literature that may be worthwhile consulting in some cases. The limit of such improvement can be estimated by assimilating the semi-deep foundation to a deep foundation and following the procedures explained in Section 3.6.

Figure 3.5.1. ***Typical Shallow Foundation Shapes***



3.5.2. Failure Modes

3.5.2.1 Ultimate Limit States

The Ultimate Limit States covered in this part of ROM 0.5, which is still general, are of the geotechnical type (GEO), which are essentially governed by the strength of the ground.

The following failure modes should in any event be considered for analysing the reliability of shallow foundations.

a. Overall Instability

The structure and its foundations may fail as a whole without any other localised failures occurring previously. This type of failure is typical of foundations in slopes of breakwater or berms.
The problem of overall stability should be analysed using the procedures given in Section 3.8.

b. Bearing Failure

This type of ground failure can occur when the load acting on the ground below any foundation element exceeds its bearing capacity. Recommendations for estimating this are included later on.

c. Sliding

The contact between foundations and ground can be subjected to shear stresses. If these exceed the strength of this contact, sliding can occur between the two elements, foundations and ground.

d. Plastic Overturning

Overturning typically occurs in structures with foundations on ground with a much greater bearing capacity than the one required to support the foundation. If this were not the case, the foundation would undergo bearing failure before overturning occurred. This mechanism (involving local plastification of the foundation ground close to an edge) is known as plastic overturning.

In addition to these geotechnical failure modes, this ROM 0.5 will briefly cover some other similar failure modes corresponding to other Ultimate Limit States that are not strictly geotechnical. These include the structural capacity of the foundations. Stresses in the structural elements making up the foundations can exceed their structural strength, as can happen with any other structural element. The Ultimate Limit States to be taken into account in this respect are the same as for any other structural elements.

3.5.2.2 Limit States of Serviceability

The Limit States of Serviceability requiring geotechnical analysis are the ones associated with ground deformability or with its excessive vibration. Displacement limitations or maximum allowable movements must be specified in each case. This ROM 0.5 only includes several general criteria in Subsection 3.5.7.4.

3.5.2.3 Other Problems of Shallow Foundations

This part of ROM 0.5 presents criteria covering analysis procedures for the most common failure modes (overall stability, bearing failure, sliding, plastic overturning, structural capacity of the foundations themselves and movements causing damage to the structure) but it must be pointed out that this is not an exhaustive list of possible problems.

Problems can also exist with the stability of excavations while being carried out and problems can occur related to dewatering and seepage and even of erosion or chemical attack on the concrete used.

Settlement problems due to poor-quality execution (failure to clean out the bottom of excavations, for example) may arise, as also problems related to poor dynamic behaviour (foundations for vibrating machinery or structures subjected to wind, wave or seismic action) and problems of watertightness in basement foundation slabs or also owing to the growth of vegetation or shrubs that displace foundations, etc.

Problems may also arise involving cracking or heave associated with expansive clays, problems with karstic dissolution or with the erosion of clay fills in foundation rock joints, potential future excavations undermining the foundation under study and even seismic effects on the foundation ground itself.

All these and other effects that could be considered as belonging to geotechnical engineering or on the borderline with other disciplines must be anticipated before shallow foundations are designed and constructed, even though there is no specific section in this ROM 0.5 providing guidance on the corresponding analysis methods for each of these aspects.

3.5.3 Foundation Characteristics

3.5.3.1 Geometrical Configuration

The general principles set out in 3.3.5.1 should be taken into account for defining the geometrical configuration of the subsoil in shallow foundation studies.

Foundations should be defined by their nominal [4] dimensions, such as breadth, *B*, length, *L*, etc.

Irregularly shaped foundations can be assimilated to equivalent rectangles so that the formulae shown here for rectangular foundations can be applied. This equivalence must preserve the characteristics of most interest in each case, as will be indicated for each analysis method, be it the area, the relationship of the moments of inertia about orthogonal axes, etc.

The foundation depth, *D*, should be an estimate of the minimum value that can reasonably be expected in any design situation on any side of the foundations.

3.5.3.2 Actions

The recommendations given in Subsection 3.3.5.3 should be taken into account when evaluating the design actions to be used to study the different Ultimate Limit States involved.

As a general rule, Ultimate Limit States should be analysed in persistent and transient situations taking into account at least three fundamental (or characteristic) load combinations:

a. the one producing the greatest vertical load
b. the one producing the greatest inclination of the load
c. the one producing the greatest eccentricity of the load.

In exceptional situations, all the combinations must be studied that, in the engineer's judgement, could be most unfavourable in the particular project involved. The seismic situation must be considered, when appropriate, according to the provisions of applicable regulations.

In vertical loads and when studying Ultimate Limit States of bearing failure, sliding and overturning, the weight of the foundation and the uplifts should be included as actions. Possible excess porewater pressure caused by application of the loads within impermeable clayey soils should generally not be considered an integral part of this design uplift.

In analysing bearing failure Limit States and for the cases where the resultant of the loads is eccentric in relation to the foundation, the following geometrical parameters should be calculated for each load combination:

Equivalent breadth, $B^* = B - 2e_B$
Equivalent length, $L^* = L - 2e_L$

Where e_B and e_L are the eccentricities in the two orthogonal directions shown in Figure 3.5.2.

With other foundation shapes, the assimilation to rectangles should retain various common characteristics. Figure 3.5.3 gives a procedure for converting circles to rectangles, which is recommended.

Having calculated these equivalent dimensions, the value of the average vertical effective pressure should also be obtained for each load combination, defined by:

$$P_v = \frac{V}{B^* \cdot L^*}$$

where

V = fuerza vertical efectiva.

(4) When this effect can be important, the value of the construction tolerance appearing in the design drawings should be subtracted or added (depending on which is the least favourable) to the geometrical dimensions.

Figure 3.5.2. **Equivalent Foundation Geometry**

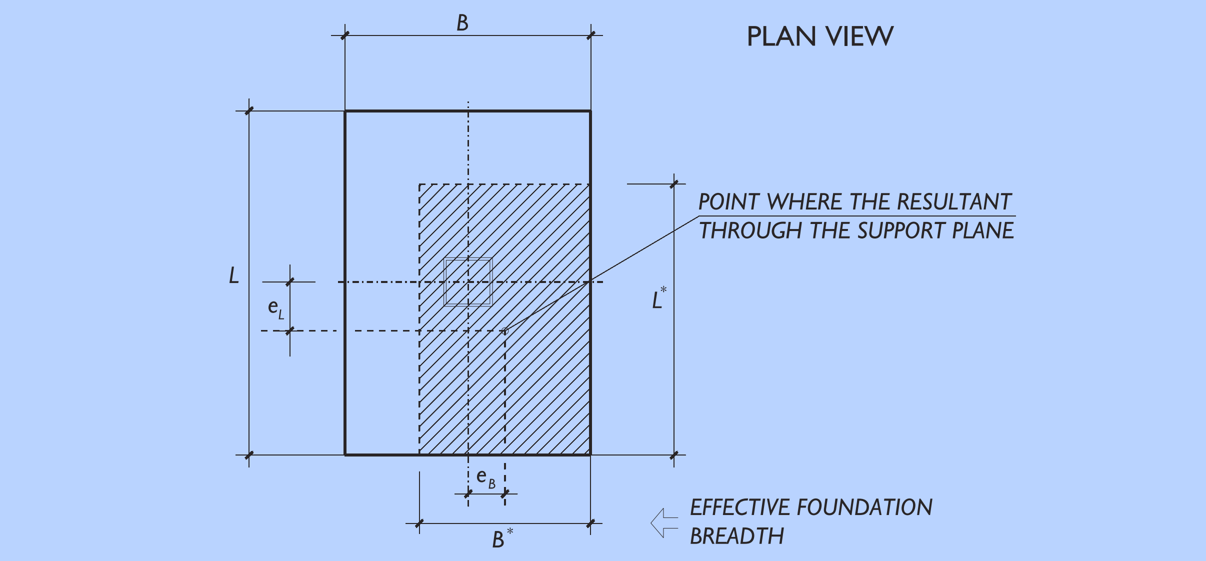
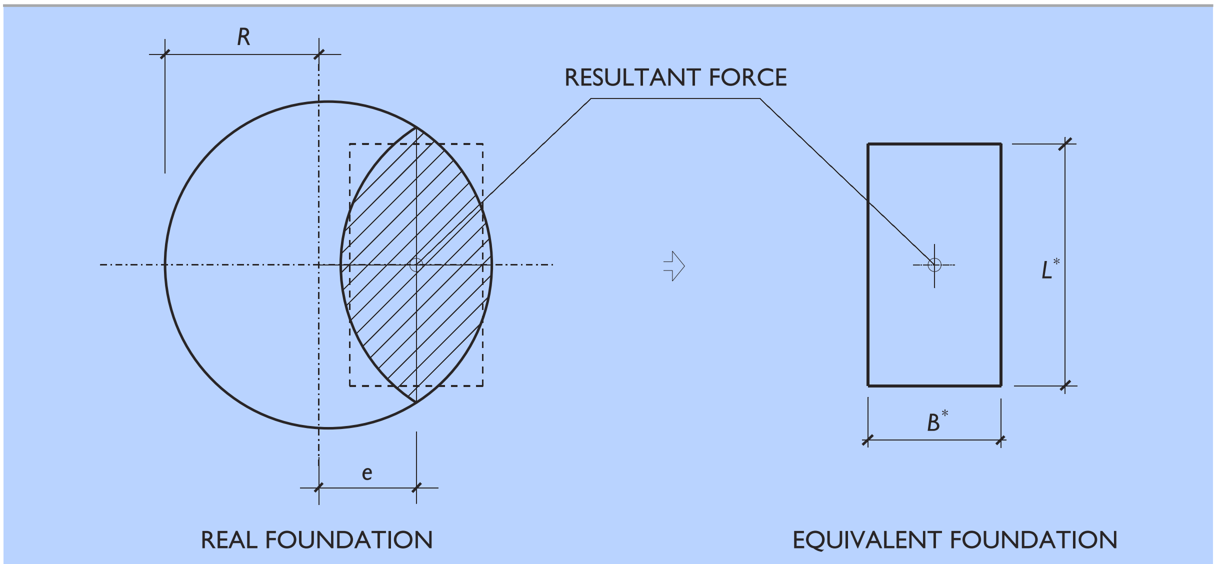


Figure 3.5.3. **Rectangular Foundation Equivalent to a Circular Foundation with an Eccentric Load**



Eccentricity e/R	Equivalent Breadth B*/R	Equivalent Length L*/R
0	1.73	1.81
0.10	1.55	1.77
0.20	1.37	1.71
0.30	1.19	1.64
0.40	1.02	1.56
0.50	0.84	1.46
0.60	0.67	1.33
0.70	0.50	1.18
0.80	0.33	0.98
0.90	0.17	0.71

This ROM 0.5 admits some methods based on so-called "duly verified comparable experience" for safety verification procedures. For the purpose of comparing foundation pressures, this ROM 0.5 defines *equivalent vertical pressure*, $p_{v,eq}$, as the highest of the following three p_v values:

a. the one corresponding to the quasi-permanent combination
b. 90% of the one that would correspond to the fundamental (5) combination leading to a higher value for p_v.
c. 80% of the maximum p_v value corresponding to accidental combinations.

The values for the geometrical dimensions of the foundations (D, B^*, L^*) and of the corresponding inclination of the load should be considered associated to the resulting $p_{v,eq.}$ value.

The foundation plane will normally be horizontal and this assumption has been made so far here. If this plane is slightly inclined, the vertical and horizontal concepts can be changed for normal and tangential to the foundation plane and the rules presented here continue to be applied. Inclinations greater than 10% require specific analytical techniques beyond the scope of this ROM 0.5.

3.5.3.3 Ground Characteristics

In order to fix the characteristics of the ground, suitable geotechnical parameters should be used as indicated in Subsection 3.3.5.2. Different parameters will be necessary, depending on the type of analysis involved.

To study Ultimate Limit States, the unit weight and moisture content parameters defined in Subsections 2.2.3 and 2.2.4 will normally be necessary, as also the strength parameters defined in 2.2.8 for soils and 2.2.9 for rocks. For problems that can be solved by experience-based methods (to be dealt with later on), ground strength can be represented by other indirect parameters such as:

- N-index of the SPT.
- resistance in continuous penetrometer tests (static or dynamic).
- limit pressure in pressuremeter tests.

In some clear-cut cases where plenty of previous experience has been gained, it may even be sufficient to compare the basic ground properties in order to assess safety against Ultimate Limit States.

In such cases, the unit weight and moisture content parameters should not be omitted, nor should other elementary index properties endorsing the resemblance of the case under study with others where experience exists.

To study Limit States of Serviceability, the ground deformation parameters defined in Subsection 2.2.10 may also be necessary plus other specific parameters, depending on the circumstances, as set out in Subsection 2.2.11.

In low-risk situations and cases where plenty of experience exists, it is possible that verifying Serviceability Limit States does not require any more ground-related information other than the basic index properties needed to ensure the similarity of the case in question to cases on which experience already exists.

3.5.4 Verifying Safety against Bearing Failure

3.5.4.1 Verification Methods

Bearing failure in a foundation constitutes a principal failure mode that must always be taken into consideration. The ways of checking whether shallow foundations are safe or not are extremely varied. Engineers

(5) For this particular calculation, the load values shall not be factored ($\gamma_g = \gamma_q = 1$).

will have to choose the one most suited to each individual case. This choice should be guided by several criteria, the most important of which are ground type and importance of the works.

For the purpose of choosing the most suitable verifying procedure, three general ground types can be distinguished:

a. Granular ground – that essentially composed of sands and gravels even though it may contain a fraction of fine soils (clays and silts), but always less than 15% and, in addition, with a permeability greater than 10^{-4} cm/s.
b. Cohesive ground – the one with a mechanical behaviour chiefly governed by the presence of fine soils (clays and silts). It has an appreciable unconfined compressive strength (yet less than 1 MPa) and is reasonably impermeable, with permeability of less than 10^{-4} cm/s.
c. Firm, cohesive ground and rock – that with a unconfined compressive strength normally exceeding the threshold value q_u = 1 MPa.

Complex ground may exist that cannot be properly classed in any of these three groups. Faced with such a situation, engineers should classify them in more than one group, follow the recommended procedure for each classification and finally, once the end results have been obtained, choose the one leading to the more conservative result from the two alternatives used.

The verifying procedures given below and their possible application are summarised in Table 3.5.1.

Table 3.5.1. Methods for Verifying Safety against Bearing Failure

Method Name	Ground Soil	Works Category (6)
Verified local experience	Firm soils and rocks	C
SPT-based method	Granular soils	B and C
Method based on pressuremeter tests	Any	B and C
Method based on static penetration tests	Any except rock	B and C
Method based on other field tests	Any	C
Bearing capacity in firm soils and rocks	Firm soils and rocks	A, B and C
Analytical calculation of the bearing capacity	Any	A, B and C

N.B.: In special cases, the verification method may include carrying out properly monitored *in situ* load tests.

3.5.4.2 Verification Based on Dependable Local Experience

Local experience gained from observing the adequate behaviour of similar foundations and similar ground to those under study can be all that is needed to justify the dimensions of foundations.

In these cases, by summarising previous experience, engineers should proceed to define the appropriate foundation depth and service pressures judged to be valid for different foundation breadths.

In addition, the representative vertical pressure (or equivalent pressure) transmitted by the foundation member to the ground should be calculated, considering the load combinations mentioned in Subsection 3.5.3.2.

Finally, it will be necessary to investigate the ground in order to ascertain at least its structure, the location of the groundwater table and the state parameters (dry unit weight and moisture content).

Shallow foundations are verified in respect of bearing failure when the service pressure does not exceed the value endorsed by experience for foundation conditions and dimensions similar to the case under study.

(6) The works category is defined in Section 2.12.

3.5.4.3 Verification Based on SPT Results

From the different correlations existing between the N-index of the standard penetration test (SPT) and the equivalent vertical pressure, the one described below has a format similar to the one initially proposed by Meyerhof [7] (1956).

The allowable vertical pressure in sands guaranteeing safety against bearing failure and, additionally, a settlement less than 1" (2.54 cm), is:

$$p_{v_{adm}} = 6N\left(1+\frac{D}{3B^*}\right) \text{ kPa} \qquad \text{for } B \le 1.3 \text{ m}$$

or

$$p_{v_{adm}} = 4N\left(1+\frac{D}{3B^*}\right)\left(1+\frac{0.3m}{B^*}\right)^2 \text{ kPa} \qquad \text{for } B \ge 1.3 \text{ m}$$

where

D = foundation depth, defined in 3.5.3.1
B^* = equivalent breadth of the foundation, defined in 3.5.3.2.

The *N* value to be used is the one corresponding to a driving efficiency in the order of 60%, the most frequently reached in well-performed tests in the past. If this aspect is adequately monitored, the relevant corrections can be made.

The N-index of the SPT to be used in this expression must be the average value obtained in the area between the foundation plane and a depth $1.5\ B^*$ below it.

As the values for the N-index depend on the effective earth overburden at the test level, these values should refer to a standardized pressure of 100 kPa. The correction factors to be used are shown in Table 3.5.2.

Table 3.5.2. Correction Factor of the SPT N-index to Account for the Effective Earth Overburden. N (Corrected) = f . N

Vertical Effective Pressure at Test Level (kPa)	Correction Factor, f
0	2
25	1.5
50	1.2
100	1
200	0.8
400 or over	0.5

Intermediate values can be obtained by linear interpolation between the data shown.

In any event, the value for the SPT N-index used in the above expressions after correction should never be above 50.

The foundation depth *D* to be used in calculations should never be greater than the equivalent foundation breadth B^*.

(7) «Penetration Tests and Bearing Capacity of Cohesionless Soils». Journal of Soil Mechanics and Foundation. Eng. ASCE.

The above formulae are considered applicable to shallow foundations with a maximum breadth of about 5 m.

The above expressions are valid for essentially vertical loads. If horizontal loads exist causing more than a 10% inclination in the resultant, then this procedure should not be used.

The above expressions are designed for situations where the groundwater table is near to or above the foundation plane and the water is at-rest or under low gradients.

If for any reason there is upward water flow in the vicinity of the shallow foundations with a head gradient I_v, the values of the allowable vertical pressure should be multiplied by the following reduction factor:

$$p_{v_{adm}} \text{ (corrected by the upward flow)} = \beta \cdot p_{v_{adm}}$$

$$\beta = 1 - I_v \cdot \frac{\gamma_w}{\gamma'}$$

where:

γ' = submerged unit weight of the soil.
γ_w = unit weight of the water.

If, on the contrary, in the corresponding design situation it is assumed that the groundwater table will always stay deeper than the foundation plane, then the allowable pressure could be corrected using the following expression:

$$p_{v_{adm}} \text{ (corrected by the groundwater table depth)} = \lambda \cdot p_{v_{adm}}$$

$$\lambda = 1 + 0.6\left(\frac{h}{B^*}\right) \leq 2$$

where:

h = the groundwater table depth below the foundation plane.

Anyway, it is necessary to check that excessive settlement does not occur as a result of the presence of looser soils at a greater depth.

Shallow foundations can be taken to be verified when the equivalent pressure defined in 3.5.3.2 is less than or equal to the $p_{v,adm}$ value defined in this subsection.

3.5.4.4 Verification Based on Pressuremeter Tests

Pressuremeter tests carried out in the ground near to the foundations under study make it possible to know an essential parameter with which to estimate the bearing capacity p_{vh} of shallow foundations. This is the limit pressure p_l resulting from properly interpreted tests.

To be able to apply this method, in addition to the limit pressure, the location of the groundwater table, the nature of the support ground and its unit weight must also be known. It is also necessary to know the foundation dimensions and the depth at which it will rest.

The bearing capacity can be estimated using the following expression:

$$p_{vh} = p_o + K \cdot \Delta p \cdot f_\delta \cdot f_D$$

where:

p_{vh} = vertical effective pressure at which bearing failure occurs.

p_o = effective pressure at the level of the support plane.
K = dimensionless coefficient
K = 0.8 for cohesive soils [8].
K = 1.0 for granular ground and soft rocks [8].
Δp = net limit pressure, once the possible pore water pressure and the horizontal effective pressure that exist at test level are subtracted from the gross limit pressure. The average value corresponding to the ground up to a depth B^* beneath the support plane should be used.
f_δ = correction factor for the effect of load inclination. The following value should be taken:

$$f_\delta = (1{,}1 - \text{tg}\,\delta)^3 \leq 1$$

f_D = correction factor due to the foundation embedment. This should be calculated as indicated below.

$$f_D = 1 + \left(a + \frac{\Delta p}{b}\right) \cdot \eta \frac{D}{B^*}\left(0.6 + 0.4\frac{B^*}{L*}\right)$$

where:

a = dimensionless parameter, where the following values should be taken:
a = 0.2 cohesive soils
a = 0.3 granular soils
a = 0.25 soft rocks
Δp = net limit pressure, as defined above in this same subsection.
b = reference pressure, where the following values should be taken:
b = 10 MPa cohesive soils and soft rocks
b = 5 MPa granular soils
η = dimensionless factor measuring the relationship existing between the values of the net limit pressure in the area above the support plane, of thickness *D*, and the *Δp* value, used to represent the resistance of the ground below the support plane.

Dimensions *D*, B^* and L^* are defined in Subsections 3.5.3.1 and 3.5.3.2. In any event, the value of the design depth cannot be larger than the width: $D \leq B^*$.

To verify the safety of shallow foundations against bearing failure using this method, the safety factors given in Subsection 3.5.4.9 apply.

3.5.4.5 Method Based on Static Penetration Tests

Static penetration tests are usually carried out on soft soils where shallow foundations are infrequent. In spite of this, in certain circumstances (small loads, structures not very sensitive to settlements, etc.), it may be necessary to estimate the bearing capacity of shallow foundations using the results of this test [9]. The following relationship enables the vertical effective bearing failure pressure to be evaluated:

$$p_{vh} = p_0 + q_c\left\{K_1 + K_2\frac{D}{B^*}\eta\left(0.6 + 0.4\frac{B^*}{L^*}\right)\right\}f_\delta$$

where:

p_{vh} = vertical effective bearing pressure.
p_o = vertical effective pressure in the vicinity of the foundations and at support plane level.
q_c = average value of the penetration resistance in the area of depth B^* below the support plane.

(8) The definition for ground typs can be seen in Subsection 3.5.4.1.
(9) Also, in certain circumstances, this test can be carried out in soils of firm consistency even though the penetration is limited.

K_1 = dimensionless coefficient, which should be:
K_1 = 0.32 for cohesive soils
K_1 = 0.17 for soft rocks

$$K_1 = \left(7 + \frac{q_c(\text{MPa})}{3\text{MPa}}\right)^{-1} \quad \text{for sands}$$

K_2 = dimensionless parameter, for which the following values should be taken:
K_2 = 0.10 for cohesive soils
K_2 = 0.05 for granular soils and rocks
η = ratio between the average value for the static penetration resistance in the area of ground above the support plane and the q_c value used to represent penetration resistance below the foundation plane.

The meanings of f_δ and the dimensions D, B^* and L^* are given in 3.5.4.4. Depth D used in calculations should be restricted to breadth B^*; i.e., $D \leq B^*$.

To verify the safety against bearing failure of shallow foundations using this method will require the safety factors given in Subsection 3.5.4.9.

3.5.4.6 Methods Based on Other Field Tests

The most common field tests for checking safety against bearing failure include dynamic penetrometer tests. They are described in Part 2 of this ROM 0.5.

There are no direct procedures to correlate the ultimate bearing pressure of shallow foundations with the results of this type of exploration. Obtaining the bearing capacity based on these tests has to be done indirectly, converting the results of this type of test into another (limit pressure, static penetration resistance, N-index of the SPT, etc.).

Local correlations existing between dynamic penetration tests and the other tests covered here may allow bearing capacity to be estimated in certain circumstances. It must not be forgotten, however, that the accumulation of inaccuracies in the field data and in subsequent correlations means that they must be very conservatively used. Any safety factors adopted must be in accordance with the inaccuracies of the analysis procedures.

Although the direct field tests for bearing capacity are not frequently carried out, this procedure should be mentioned here, as it is the most precise, particularly when the field model dimensions come close to those of the real foundations under study. It may sometimes be interesting to carry them out.

3.5.4.7 Bearing Capacity in Firm Cohesive Soils and Rocks

Bearing capacity for shallow foundations on firm cohesive soils or rocks can be estimated using the simplified procedure described in this subsection.

The following data are needed to evaluate bearing capacity:

- Foundation geometrical data - lengths D, B, L, B^*, L^* defined in 3.5.3.1 and 3.5.3.2 and the inclination of the load on the foundations (δ angle).

- General structure of the rocky formation – it is particularly interesting to know about the rock located in the close vicinity of the foundations, within a plan area $4B \times L$ wide and up to a depth $2B$ under the support plane.

- The rock characteristics to be used in the calculations should be those of the rock in the poorest conditions appearing in the said area. Making any other simplifying assumption in this respect calls for specific justification.

- Unconfined compressive strength of the rock – it can be determined using specific tests (unconfined compression tests in rock specimens properly cut from cores taken from fresh rock) or else by correctly interpreted indirect techniques (point-load tests).

- Degree of weathering in the rock, as defined in 2.2.9.7. The one corresponding to the support surface should be taken - provided, as is usually the case, that the weathering does not increase with depth.

- Distance between joints – the value corresponding to the set of joints appearing most closely spaced in the area near the support.

- The average RQD value in the area of depth *B* below the support plane and with $B \times L$ plan dimensions.

Whenever the degree of weathering is IV or higher or when joints are spaced less than 10 cm apart or when the RQD is less than 10%, this procedure should not be used. The ground will have to be taken as a soil and a different calculation procedure used.

The vertical effective bearing pressure in foundations on rocky formations can be estimated as follows:

$$p_{vh} = 3\,(p_r \cdot q_u)^{1/2} \cdot f_D \cdot f_A \cdot f_\delta < 15\ MPa$$

where:

p_{vh} = vertical effective pressure producing bearing failure.
p_r = 1 MPa reference pressure.
q_u = unconfined compressive strength of the fresh rock.
f_D = reduction factor owing to jointing. It shall be the minimum of the following two:

$$f_D = 2 \cdot \left(\frac{s}{B^*}\right)^{1/2} \leq 1 \quad ; \quad f_D = 0.2 \cdot \left(\frac{B_0 \times RQD(\%)}{B^*}\right)^{1/2} < 1$$

s = joint spacing (see text) - this procedure should not be used when s < 0.10 m.
B^* = equivalent foundation breadth.
B_o = reference breadth; B_o = 1 m should be taken
RQD = Rock Quality Designation (see 2.7.1.)
f_A = reduction factor due to the degree of weathering in the rock, taking the following value:

Weathering Degree (See 2.2.9.7)	f_A Factor
I	1
II	0.7
III	0.5

f_δ = factor taking into account the inclination of the load, to be taken as:
$f_\delta = (1{,}1 - \text{tg}\ \delta)^3 < 1$

This calculating procedure is not adequate when any of the circumstances indicated above in this subsection arises nor in any of the following situations:

- Foundations in ground sloping more than 10% in the area surrounding the support.

- Foundations executed close to a slope – in this case, the conditions of overall stability will have to be checked by a different procedure.

- For large support areas (> 100 m^2), the bearing failure problem may be governed by singular accidents (faults, exceptional joints, etc.) the effect of which needs to be investigated.

The bearing capacity of foundations resting on rock depends in theory on the nature of the rock. In equal conditions of unconfined compressive strength and the rest of the parameters (jointing, weathering and size of the supporting area), some rock types are capable of withstanding greater loads than others. The influence of the type of rock has still not been well established in regular practice, which is why this ROM 0.5 does not lay out an explicit procedure for taking it into account.

Verification for limit states other than bearing capacity (overall instability, sliding, overturning, etc.) is not guaranteed with this bearing pressure verification; consequently, other complementary checks will be necessary.

Using this method to check on the safety against bearing failure of shallow foundations requires the safety factors set out in Subsection 3.5.4.9.

3.5.4.8 Analytical Calculation of Bearing Capacity

3.5.4.8.1 Polynomial Formula

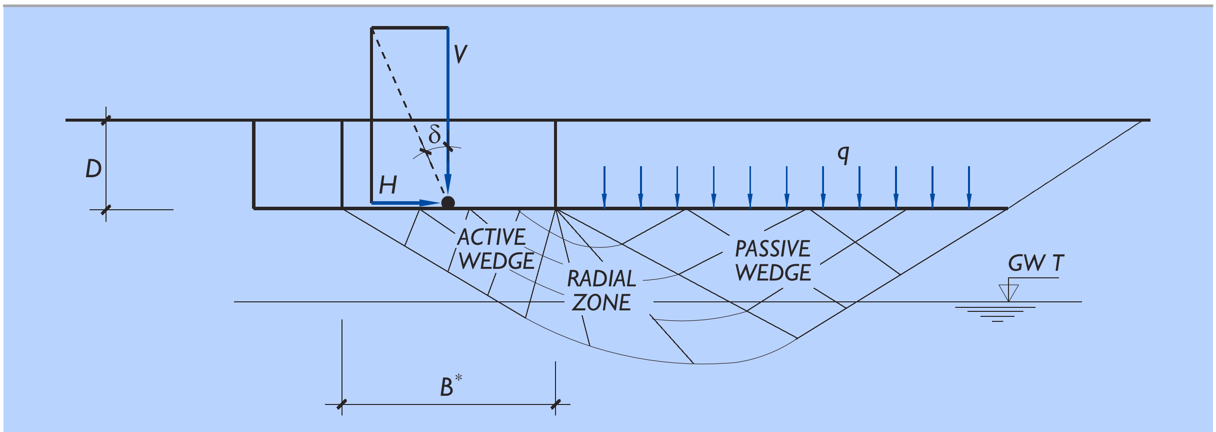
The most frequently used formula for estimating the safety against bearing failure of shallow foundations, which is recommended here, is known as the Brinch Hansen formula, but it has several versions which differ in several details concerning the procedure for obtaining certain parameters. Figure 3.5.4 shows a diagram of the problem under study, indicating the failure surface adopted. According to the version recommended in this ROM 0.5, the vertical component of the ultimate bearing pressure is:

$$p_{vh} = q \cdot N_q \cdot f_q + c \cdot N_c \cdot f_c + \frac{1}{2}\gamma \cdot B^* \cdot N_\gamma \cdot f_\gamma$$

where:

q	=	overburden owing to the weight of earth at the foundation depth in the area surrounding the foundations.
c	=	cohesion.
γ	=	unit weight of the soil (the design value is given later).
N_q, N_c, N_γ	=	bearing capacity factors.
f_q, f_c, f_γ	=	correction factors.

Figure 3.5.4. **Possible Bearing Failure Geometry**



The correction factors are a function of, among other variables, the angle of friction. They can be obtained as shown in Subsection 3.5.4.8.2.

The bearing capacity coefficients are an exclusive function of the angle of friction of the ground and can be obtained from the following analytical expressions:

$$N_q = \frac{1+\sin\phi}{1-\sin\phi}\cdot e^{\pi tg\phi}$$

$$N_c = \frac{N_q - 1}{\tan\phi} \quad \text{(for } \phi = 0 \text{ it result } N_c = \pi + 2\text{)}$$

$$N_\gamma = 2(N_q - 1)\cdot\tan\phi \quad \text{(Rough base)}$$

To facilitate the calculation, Table 3.5.3 includes values for the bearing capacity coefficients obtained for several values of the angle of friction.

For values of ϕ > 45°, the bearing capacity factor N_γ can prove overoptimistic. To take this into account, a design value for the angle of friction can be used exceeding 45° by only half the actual excess. That is:

$$\phi_{design} = \frac{1}{2}(\phi_{real} + 45°), \quad \text{when } \phi_{real} > 45°$$

The N_γ value indicated is adequate for studying structural foundations with a rough base. In exceptional cases of supports on "smooth bases" (due to inserted sheets or low-friction surfaces), the N_γ value to be used should be half the value above indicated.

In soft clayey soils and when high porewater pressures are expected to be generated in the foundation ground that will not be attenuated during the loading process, it will be necessary to consider the *undrained* condition.

Table 3.5.3. Bearing Capacity Factors for the Brinch Hansen Formula

ϕ (Degrees)	N_q	N_c	N_γ
20	6.4	14.8	3.9
21	7.1	15.8	4.7
22	7.8	16.9	5.5
23	8.7	18.1	6.5
24	9.6	19.3	7.7
25	10.7	20.7	9.0
26	11.8	22.3	10.6
27	13.2	23.9	12.4
28	14.7	25.8	14.6
29	16.4	27.9	17.1
30	18.4	30.1	20.1
31	20.6	32.7	23.6
32	23.2	35.5	27.7
33	26.1	38.6	32.6
34	29.4	42.2	38.4
35	33.3	46.1	45.2
36	37.8	50.6	53.4
37	42.9	55.6	63.2
38	48.9	61.4	74.9
39	56.0	67.9	89.0
40	64.2	75.3	106.1
41	73.9	83.9	126.7
42	85.4	93.7	152.0
43	99.0	105.1	182.8
44	115.3	118.4	220.8
45	134.9	133.9	267.8

Some indications follow about the way of characterising the ground weight and strength in both types of calculation.

a. Undrained Bearing Failure

Calculations of the vertical bearing pressure in foundations on a poorly permeable ground should be made on the assumption that no consolidation will occur in the ground.

"Poorly permeable" refers here to the capacity of dissipating the porewater pressures generated by application of the load precisely while it is being applied. The more impermeable the soil and the faster the load application, the nearer to the theoretical case of undrained failure the real situation will be.

As a general rule, the theoretical *undrained case*, undoubtedly an extreme case, must be assumed to be possible when the conditions indicated in Subsection 2.2.7 arise, unless specific arrangements are taken at the Design Stage which allow some consolidation (or porewater pressure dissipation) to take place during construction and provisions are made to check this fact.

In the extreme *undrained* situation, ground strength can be simulated by a zero friction angle and a cohesion equal to the shear strength obtained from undrained shear tests, either field (vane test, for example) or laboratory (UU triaxial tests, for example) or else through indirect estimation using correlations (static penetrometer, for example) or other tests covered in Part 2 of this ROM 0.5.

The strength parameters to be used in the polynomial formula will therefore be:

$$\phi = 0$$
$$c = s_u$$

where:

s_u = average undrained shear strength of the area of depth B^* below the foundation plane.

The *q* parameter in the polynomial formula should be calculated by the expression:

$$q = \gamma_{ap}\, D$$

where:

γ_{ap} = apparent unit weight of the ground in the area between the surface of the ground and the foundation plane.
D = foundation depth, defined in 3.5.3.1.

When the foundation ground is totally submerged, the submerged unit weight should be used in place of the apparent value for evaluating the overburden *q*.

b. Drained Bearing Failure

Safety against bearing failure in *drained* conditions should always be verified, since this situation tries to represent the behaviour of shallow foundations in dry or saturated sandy soils and foundations on any other type of ground once the foundation ground is fully adapted to the loads imposed.

The data of interest should represent the anticipated average behaviour of the ground along the failure surface. The strength data should comprise the effective cohesion and angle of friction, which can be obtained from laboratory tests carried out on undisturbed samples:

- Triaxial shear tests run with prior consolidation and undrained failure (CU), measuring and subtracting the pore pressures.
- Triaxial shear tests carried out with prior consolidation to drained failure (CD).

They can also come from other laboratory tests (direct shear, simple unconfined shear) or from correlations previously established by field tests (penetration, bearing plate tests, etc.).

The *q* overburden to be used in the first term of the polynomial formula should be equal to the effective pressure at the level of the foundation plane around the foundations. If this value proves to be variable, the average value along whichever side has less overburden should be used.

The unit weight for calculating the third term of the polynomial formula should be:

- the apparent unit weight, γ_{ap}, if the groundwater table is at a depth greater than 1.5 B^* below the foundation plane
- the submerged unit weight, γ', if the groundwater table is at or above the foundation plane
- an intermediate unit weight, linearly interpolated, if the groundwater table lies between the two preceding cases.

If there is an upward water flow with a gradient I_v affecting the foundation plane, the unit weight for calculating the third term in question should be:

$$\gamma_{design} = \gamma' - I_v \cdot \gamma_w$$

where:

γ' = submerged unit weight of the ground.
γ_w = unit weight of the water.
I_v = average vertical gradient in the area with a thickness of $1.5B^*$ below the foundation plane.

The polynomial formula applied in drained conditons is particularly inadequate when the ground is not horizontal. Shallow foundations in the vicinity of slopes can experience bearing failure with much lower loads. In such cases, the calculation methods shown in 3.8 should be used to check safety against bearing failure.

3.5.4.8.2 Correction Factors

Each of the three terms in the polynomial formula must be affected by a correction factor, *f*, whose subscript will indicate the corresponding term.

In turn, each of the three correction factors should be obtained as the product of several coefficients taking into account each individual partial effect intervening in the problem.

The following effects are covered below:

Effect	Coeficient
Foundation shape	s
Load inclination	i
Strength of the ground above the support plane	d
Support plane inclination	r
Slope of the ground in the vicinity of the foundations	t

Thus, any of the three correction factors will in turn be the product of five coefficients:

$$f = s \cdot i \cdot d \cdot r \cdot t$$

The way for obtaining each of them is given below.

a. Shape Factors

Shape factors take into account the proportions of the plan dimensions of the equivalent foundations. These dimensions are the lengths B^* and L^* defined in Subsection 3.5.3.2 for each load combination.

The following formulae should be used to obtain these coefficients:

$$s_q = 1 + \frac{B^*}{L^*} \cdot \frac{N_q}{N_c}$$

$$s_c = s_q$$

$$s_\gamma = 1 - 0.4\frac{B^*}{L^*}$$

b. Inclination Factors

Inclination factors take into account the deviation from the vertical in the direction of the resultant load acting on the support plane. This inclination is represented by the absolute value of the angle formed between the two directions (see Fig. 3.5.4).

For two-dimensional problems (shallow foundations of infinite length), the values of the inclination factors should be calculated using the following expressions:

$$i_q = \left(1 - 0{,}7\tan\delta\right)^3$$

$$i_u = \frac{i_q N_q - 1}{N_q - 1} \geq 0 \ ; \ \text{for } \phi = 0,\ i_c = 0.5\left(1 + \sqrt{1 - \frac{H}{B^* \cdot L^* \cdot c}}\right)$$

$$i_\gamma = \left(1 - \tan\delta\right)^3$$

δ = angle of deviation of the load from the vertical.

When a certain cohesion value, *c*, can be guaranteed in the foundation-ground interface, the lesser δ^* angle can be taken, given by the expression:

$$\tan\delta^* = \frac{\tan\delta}{1 + \dfrac{B^* \cdot L^* \cdot c}{V\tan\phi}}$$

In these expressions:

V = vertical resultant of the actions on the foundations.
H = horizontal resultant of the actions on the foundations.

When the problem is three-dimensional (foundation with finite dimensions), the bearing failure should be analysed as a plane problem in the least favourable direction. The load inclination in the other direction, δ_T, can be taken into account by reducing each of the three factors in the following way:

$$i\ (\textit{three-dimensional}) = i\ (\textit{two-dimensional}) \times (1 - \text{tg}\ \delta_T)$$

c. Effect of the Resistance of the Ground over the Foundation Plane

In the above formulae, the ground above the foundation plane was assumed to act exclusively as an overburden.

On the occasions when the permanent integrity of this ground can be guaranteed in an extensive area around the foundations, free of any natural or artificial discontinuities (trenches, localised dredging, etc.), the bearing capacity can be increased by multiplying each of the three terms in the polynomial formula by the following coefficients:

$$d_q = 1 + 2\tan\phi(1 - \sin\phi)^2 \arctan\frac{D}{B^*}$$

$$d_c = 1 + 2\frac{N_q}{N_c}(1 - \sin\phi)^2 \arctan\frac{D}{B^*}$$

$$d_\gamma = 1$$

The arc appearing in these formulae should be expressed in radians.

d. Effect of the Foundation Plane Inclination

The above formulae assume that the foundation plane is horizontal. If it is slightly inclined, the bearing capacity will be somewhat lower and this effect can be taken into account by making two modifications to the polynomial formula.

On the one hand, each term in the polynomial formula should be multiplied by the following reducing factors:

$$r_q = e^{-2\eta \tanh \phi}$$
$$r_c = 1 - 0{,}4\,\eta$$
$$r_\gamma = r_q$$

where η is the angle of deviation from the horizontal of the foundation plane, measured in radians.

On the other hand, the angle δ defined in the polynomial formulae as deviation from the vertical of the load on the foundations should be interpreted in this case as a deviation from the normal to the foundation plane. All that will be needed to take into account is to add the angle δ to the angle η before calculating the inclination factors.

This approximate way of considering the inclination effect should not be used for foundations inclined more than 10%, as considerable errors could arise.

e. Foundations on Sloping Areas

The above formulae are intended for cases when the ground in the passive zone is plane and horizontal (see Fig. 3.5.4).

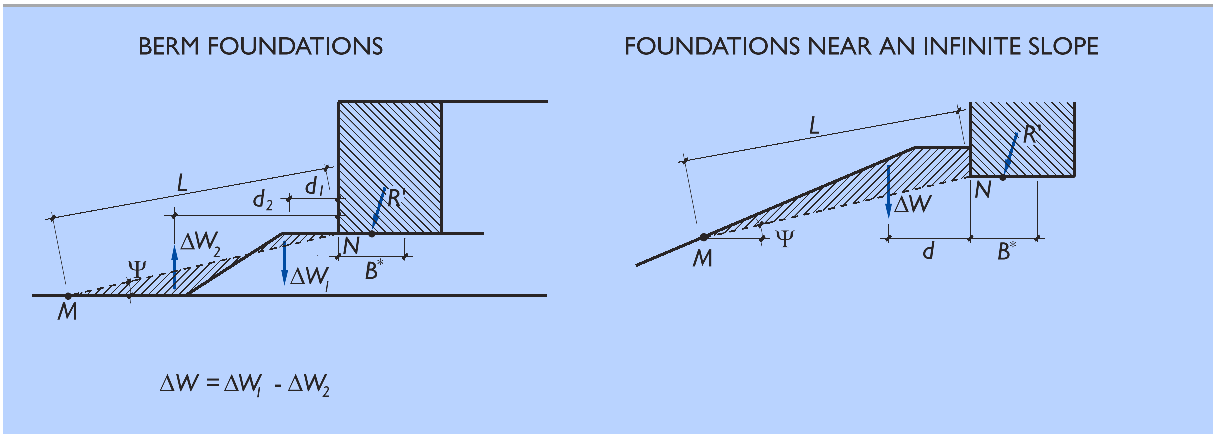
If the ground slopes downwards in this area, it is difficult to estimate the value of the overburden *q* to be taken into account in the calculations. For the purposes of the simplified calculation described here, the plane *MN* shown in Figure 3.5.5 can be assumed to be the reference plane where the design overburden would act.

This plane passes through the forward toe of the foundations –point *N*– and intersects with the free ground surface at point *M*, at a distance *L* from *N* as shown in the figure.

The equivalent surcharge, *q*, acting on this plane is calculated in such a way that acting perpendicularly on *MN* (in the normal direction), it gives a normal force equal to 60% of the resultant of the effective weights of the ground situated above this line (see Fig. 3.5.5).

It is worth noting that the value for *q* can be negative in some foundations on narrow berms.

Figure 3.5.5. **Foundations in Sloping Areas**



Having estimated the adequate design overburden, the polynomial formula can be used, affecting its different terms by the following reducing factors:

$$t_q = t_\gamma = (1 - 0.5\ \mathrm{tg}\ \Psi)^5$$
$$t_c = 1 - 0.4\ \Psi$$

where Ψ is the angle of inclination to the horizontal of *MN* , expressed in radians.

The greater the Ψ angle, the larger the errors from this approximate method may be. For cases where this effect is critical to the design, more complicated procedures need to be used and the technical literature consulted for this.

3.5.4.8.3 Shallow Foundations on Heterogeneous Ground

In shallow foundations for maritime and harbour works, similar situations to the one shown in Figure 3.5.6 are fairly common. The foundation rest directly on competent material, which lays in turn on poorer-strength soil.

It is complicated to calculate the bearing capacity in these cases; hence, if this is a critical issue, specific numerical computations should be used, as there are no general analytical solutions for this problem.

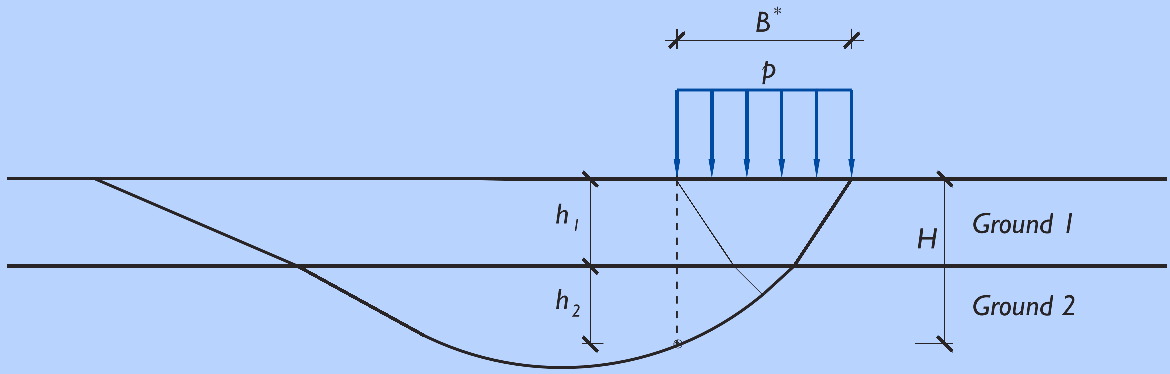
In any event, the problem should never be simplified by "projecting down" load *p* to the contact between the two materials and assuming that Ground I only collaborates with its self-weight. In this load transmission, the equivalent foundation breadth B^* can decrease considerably and it is not easy to ascertain its right design value. To assume that B^* is constant in this operation of virtual load projection can lead to overestimating the bearing capacity. To assume that B^* increases in the load transmission is an inappropriate assumption leading to far too dangerous results (it may lead to a computed bearing capacity as much as ten times greater than the real value).

For cases not requiring a high degree of precision, heterogeneous ground can be assimilated to another homogeneous one whose bearing capacity factor N_γ is that shown in Figure 3.5.6.

This way of interpolating is adequate provided that the angle of friction of the upper ground is higher than that of the lower ground.

To average the cohesion and density along the failure line and, provided these parameters have no great influence on the result, the following approximate expressions can be used:

Figure 3.5.6. **Approximate Solution in Heterogeneous Ground**



$$N_\gamma\,(\text{equiv}) = \frac{N_{\gamma 1} \cdot h_1^2 + N_{\gamma 2}\left(2h_1 + h_2\right) \cdot h_2}{H^2} \qquad (1)$$

$$H = \tfrac{1}{2} B^* \, \frac{\exp\left[\left(\frac{\pi}{4} - \frac{\phi}{2}\right) \cdot \tan\phi\right]}{\cos\left(\frac{\pi}{4} + \frac{\phi}{2}\right)} \qquad (2)$$

N.B.: The Angle ϕ to be used in Expression (2) is the corresponding to the value N_γ obtained in Expression (1). An iterative calculation is required to determine it.

$$c_{equiv} = \frac{c_1 h_1 + c_2 h_2}{H}$$

$$\gamma_{equiv} = \frac{\gamma_1 h_1 + \gamma_2 h_2}{H}$$

When analysing granular berms on soft soils, the foregoing recommendations in this subsection can be used to evaluate the bearing capacity in drained conditions. The indications in the following subsection should be taken into account to calculate undrained bearing capacity.

3.5.4.8.4 Berm Foundations on Cohesive Soils

It is not infrequent to encounter the situation covered here in port works. A soft soil is covered by a granular fill (berm) on whose surface the foundation load is applied. It is normally necessary to calculate the undrained bearing capacity, assuming that the load applied does not produce any consolidation at all. In these circumstances, the vertical component of the bearing capacity can be estimated using the following expression:

$$p_{vh} = p_s + i_s K \frac{H}{B^*} (\gamma \cdot H + 2q)$$

where:

p_{vh} = vertical effective pressure that would produce bearing failure when applied to the berm.

i_s = inclination factor that can be estimated by the following expression:

$$i_s = (1 - 0.5 \text{ tg } \delta)^3$$

K = coefficient depending on the angle of friction corresponding to large strain in the berm material.

$$K = 6 \text{ tg}^3 \phi$$

γ = unit weight of the berm (submerged, in applicable cases).

q = indefinite-length surcharge on the berm.
p_s = pressure depending on the shear strength of the cohesive soil, which can be estimated by the following expression:

$$p_s = (\pi + 2)\, s_u\, i_c + q\, i_q - \gamma\, H\, (1 - i_q)$$

i_q = inclination factor for surcharges that can be taken as:

$$i_q = (1 - \alpha\ \mathrm{tg}\ \delta)^3$$

where:

$$\alpha = 0.7 - \frac{\gamma \cdot H}{10 \cdot s_u} > 0.5$$

i_c = inclination factor for the shear strength. The following value should be taken.

$$i_c = \frac{1}{2}\left(1 + \sqrt{1 - x}\right)$$

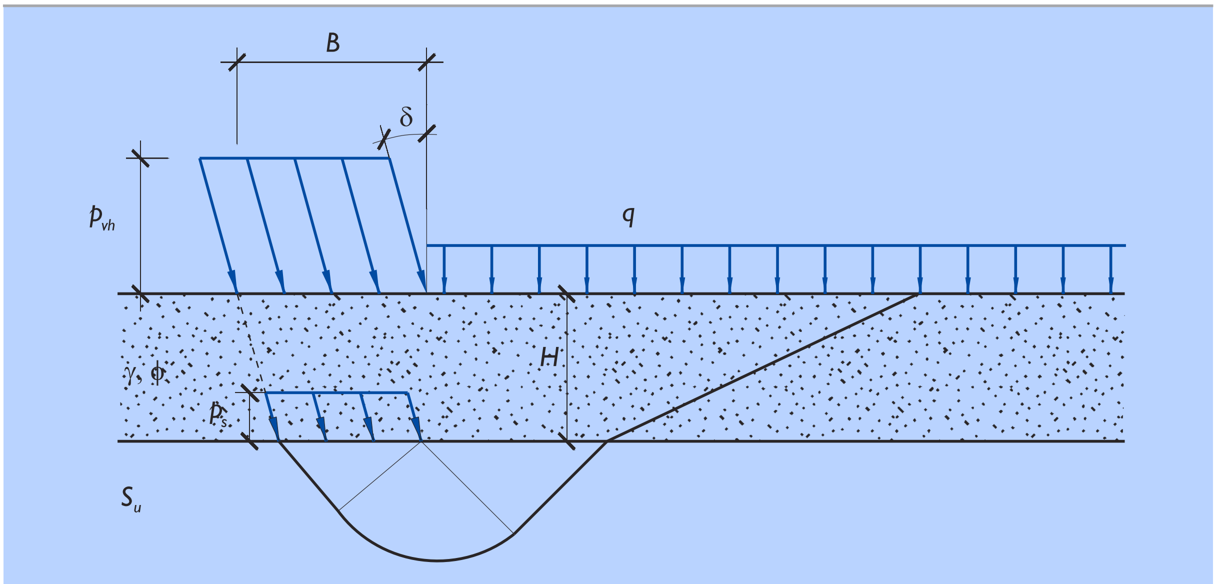
where: $x = 10\ \mathrm{tg}\ \delta < 1$

The geometrical variables B^*, H and δ are the ones indicated in Figure 3.5.7. The s_u value represents the undrained shear strength of the cohesive soil. On many occasions, this s_u term varies with the vertical effective pressure σ'_{vo}, existing prior to application of the load, according to a linear relationship:

$$s_u = \eta \cdot \sigma'_{vo}$$

where η is an dimensionless number depending on the nature of the soil and on its state parameters (see Subsection 2.2.8.3).

Figure 3.5.7. **Diagram of Failure in a Granular Berm Resting on Undrained Soft Soil**



To include this type of variation with depth in the problem covered in this subsection, the following can be taken as the representative value for s_u:

$$s_u(\mathrm{equiv}) = 3 s_{uo} \cdot \frac{2 s_{uo} + \gamma' B^* \eta}{6 s_{uo} + \gamma' B^* \eta}$$

where:

s_{uo} = undrained shear strength in the contact plane between berm and cohesive soil.
γ' = specific weight (submerged in appropriate cases) of the cohesive soil.

3.5.4.9 Required Safety against Bearing Failure

After having investigated the ground, analysed the design and defined the foundations, engineers will have utilised some procedure for verifying safety against bearing failure, from those described in Section 3.5, appropriate for their specific case in terms of ground type and importance of the works.

The safety against bearing failure can be considered verified by checking the simple inequality:

$$p_{v,eq} \leq p_{v,adm}$$

only for the methods described in Subsections 3.5.4.2 and 3.5.4.3 under the titles *Local Experience* and *SPT Results*. In these cases, the safety for Limit States of Serviceability relative to foundation settlement can also be taken as checked, subject to the limitations stated there.

Contrariwise, in each of the procedures described in Subsections 3.5.4.4 to 3.5.4.8, it is possible to calculate, for each load combination, an acting vertical pressure and a vertical bearing pressure.

The safety factor against bearing failure, F, is defined by:

$$F = \frac{p_{vh}}{p_v}$$

where:

$p_{v,h}$ = vertical bearing pressure, obtained by applying the corresponding calculation method.
p_v = acting vertical pressure for the corresponding load hypothesis, calculated as indicated in Subsection 3.5.3.2.

For works with low SERI (5 - 19), the minimum values recommended for the safety factor are shown in Table 3.5.4.

For works with a minor or high SERI rating or for other allowable failure probabilities, the F values can be adjusted as shown in Subsections 3.3.8.2 and 3.3.10.

Nevertheless, for works with minor socio-environmental repercussions (SERI $<$ 5), and when bearing failure probabilities greater than 10^{-3} are considered admissible, smaller safety factors can be used than those shown in Table 3.5.4. In these cases, engineers should judge whether the consequent saving is economically advantageous when compared to the increased possibilities of bad behaviour implied in reducing the safety against this failure mode. In any event, it is recommended that safety factors against bearing failure should always be greater than 80% of the value assigned in Table 3.5.4.

Even though in the majority of cases the safety factors in Table 3.5.4 are considered to be associated with failure probabilities in the order of 10^{-3}, it is always advisable to have more exact knowledge of the reliability of the foundations in respect of the problem of its bearing failure. This is the reason for recommending that the confidence (or reliability) index β should always be specifically calculated for works of a certain importance (Category A defined in Part 2 of this ROM 0.5, corresponding to works with high or very high ERI or SERI). To do this calculation, it is worthwhile taking into account the recommendations given in Subsection 3.3.10.

Table 3.5.4. ***Safety Factors against Bearing Failure in Shallow Foundations. Minimum Recommended Values for Works with a Low SERI (5 - 19)***

Calculation Method	Load Combination and Factor, F		
	Quasi-Permanent, F_1	Fundamental, F_2	Accidental or Seismic, F_3
Pressuremeter tests	2.6	2.2	2.0
Static penetration tests	2.6	2.2	2.0
Other field tests	3.0	2.5	2.2
"Firm soil and rocks" method	2.8	2.3	2.1
Analytical calculation	2.5	2.0	1.8

For each type of works, the reliability associated with the failure probabilities indicated in Subsection 3.2.1 is the minimum required for a failure mode in order not to have to consider it a principal failure and therefore to be able to disregard its contribution in calculating the overall failure probability of the works.

If these calculations result in a lower reliability index than the one required in that table, engineers should proceed to increase it (with more data, greater dimensions or depth of the foundations, etc.) until the minima are exceeded. Otherwise, the effects it will have in the calculation of the overall failure probability of the works should be assessed.

3.5.5 Verifying Safety against Sliding

3.5.5.1 Preliminary Remarks

This failure mode is specially important in the foundations of earth or wave retaining structures, which is why certain supplementary comments are made when referring to them in Section 3.7 and in Part 4 of this ROM 0.5.

Foundations with isolated or continuous footings are usually stable with respect to sliding since, when they have to support loads with substantial inclination or eccentricity, they are usually inter-tied in order to redistribute the loads acting on the structure over several foundation elements.

In any event, each untied shallow foundation and each set of inter-tied foundations must comply with the following sliding stability criteria.

3.5.5.2 Calculating Procedure

This section covers only the most common case of foundations laid on horizontal planes.

In such cases, the horizontal force capable of inducing the foundation to slide along its plane of contact with the ground can be estimated by the expression:

$$H_{(failure)} = V \tan \phi_c + a \cdot S + (E_p - E_a) + R_c$$

where:

- V = vertical effective load.
- ϕ_c = angle of friction of the contact between foundation element and ground.
- a = foundation-ground adhesion.
- S = support surface area.
- E_p = passive earth pressure resultant over a depth D (front face, opposing the slide).
- E_a = active earth pressure resultant over a depth D (rear face)
- R_c = other possible resistances along the contour of the foundation lateral faces.

It is common –and advisable– to adopt a conservative assumption when estimating this resistant force; namely, to eliminate the components of the resistance due to the ground above the foundation level, (E_p - E_a) and R_c, since:

a. considerable displacement is needed to mobilise them, which could involve damage to the structure;
b. the continuity in such lateral contacts is not always guaranteed (possible ground shrinkage alongside the foundations).

A contribution from the resistance of the foundation lateral faces can only be justified if, owing to the particular circumstances of the case in question, construction precautions can be taken as required to ensure the permanence of these contacts with the ground and when the structure is not sensitive to the displacement required to mobilise these resistances.

It is difficult to avoid some remoulding of the ground in the area close to the contact of foundation and ground, particularly when the water table is near the excavation. For this reason, it is also frequent practice, and one to be recommended, to assume that the contact zone between ground and foundation behaves similarly to a remoulded ground.

In saturated clayey soils, the friction term takes a certain time to develop and it is advisable to make a check for this failure mode on the assumption that $\phi_c = 0$ (see 2.2.7, undrained calculation). In this case, the only strength term should be the one corresponding to adhesion.

The value of the adhesion between the foundation and the ground in the case of clayey soils can be assumed to be equal to the value indicated in Subsection 3.6.4.7.2 for adhesion in the shaft of piles (a = c').

The contact area, *S*, must be taken as the effective area, $B^* \times L^*$. These dimensions are defined in Subsection 3.5.3.2.

When safety against sliding in undrained conditions is critical, the ground-foundations contact can be drained (by porous concrete in the contact zone, for example) in such a way that high pore pressures are avoided precisely on the contact plane. The improvement it is possible to achieve in such cases requires a specific study.

Finally, in long-term situations, it is recommended to assume that all the sliding resistance is due to the friction component. The friction angle to be used in calculating the horizontal force producing the slide will depend on some case-specific conditions and on the angle of friction of the ground. The recommended values are as follows:

- Foundations on saturated clays without any drainage measures in the contact zone:

$$\phi_c = 0$$

- Foundations on saturated clays with drainage measures in the contact zone

$$\tan\phi_c \le \frac{1}{2}\tan\phi$$

(The more effective the drainage is deemed, the higher the value of ϕ_c will be).

- Precast foundations, in the long term.

$$\phi_c = \frac{2}{3}\phi$$

- Foundations with concrete cast against the ground and long-term calculations.

$$\phi_c = \phi$$

In gravity quays made of concrete blocks, potential slides along the concrete-concrete contact planes must be checked. If the friction angle in question is critical, it will be necessary to carry out special tests to determine this or to consult the technical literature for similar cases. Otherwise, the precast concrete contacts should be assumed to have a friction angle, ϕ_c = 3° (μ = tan ϕ_c = 0.7).

For precast concrete foundations on rockfills, the above comments apply; namely, that knowing the specific value of the friction between concrete and the levelling berms in each structure is a fundamental design aspect. For minor works and when the problem of sliding is not critical, in the absence of specific information, it can be assumed that the angle of friction for calculating the sliding safety will be ϕ_c = 32° (μ = tan ϕ_c = 0.625).

The same comments as to the value and advisability of specific studies also apply to cast-in-situ concrete foundations on rockfills where, in the absence of data, it can be assumed that ϕ_c = 40° (μ = tan ϕ_c = 0.84).

In the three cases just mentioned, the adhesion or cohesion term shall be considered null.

An exception to this calculation procedure are the foundations where sliding safety is critical and for which special design provisions are made to increase safety. In such cases, the calculation procedure for the checks should be specifically established following the principles and philosophy set out in this section.

3.5.5.3 Safety against Sliding

Sliding safety can be considered to suffice when the following expression has been verified:

$$F \leq \frac{H_{(failure)}}{H}$$

where:

H = acting horizontal load
$H_{(failure)}$ = horizontal load producing failure
F = safety factor shown in Table 3.5.5.

Table 3.5.5. **Minimum Values for the Safety Factors against Horizontal Sliding. Works with Low SERI (5 - 19)**

Load Combination	Sliding Safety Factor, F
Quasi-Permanent, F_1	1.5
Fundamental, F_2	1.3
Accidental or seismic, F_3	1.1

In inclined foundations, the concepts of vertical and horizontal should be taken as the normal and tangential to the foundation plane.

When the stabilising contribution of the ground existing above the foundation plane is considered (friction on lateral faces or passive earth pressure), it should also be checked that these resistance terms do not account for more than 15% of the total resistance to sliding.

For works with a minor or high SERI rating or for other allowable failure probabilities, the minimum *F* values stated in Table 3.5.5 can be adjusted as given in Subsections 3.3.8.2 and 3.3.10. They can equally be adapted for transient situations (including short-term geotechnical situations) in accordance with the provisions of Subsection 3.3.8.1.

3.5.6 Verifying Safety against Overturning

Shallow foundations with tied footings or slabs do not tend to overturn, since the eccentricity of the loads is normally offset by the tying system. Overturning is more typically found in strip foundations, particularly in retaining structures (retaining walls, gravity quays, etc.).

Apart from the specific considerations included later on in other sections of this ROM 0.5, safety against overturning must be checked in all cases in which overturning is more likely as a result of the type of foundations involved (untied strip footings, for example) or the nature of the ground (highly eccentric foundations on rock, for example).

Two different failure modes need to be considered to verify safety against overturning. The first failure mode, from here onwards termed *rigid overturning*, corresponds to an Ultimate Limit State, ULS, of loss of equilibrium and in this ROM 0.5 is classed as a type EQU, as defined in Subsection 3.3.1. The second failure mode is called *plastic overturning* in this ROM 0.5 and is essentially governed by the ground characteristics. For this reason, it is an Ultimate Limit State of geotechnical type (GEO in the definition given in 3.3.1). The analytical procedures for both failure modes are described below.

3.5.6.1 Rigid Overturning

Rigid overturning, like any other failure mode, is a simplified theoretical conception attempting to represent a potential failure mechanism. In this case and to simplify the problem, the ground is assumed to be infinitely resistant and the foundation structure too, in such a way that the foundations could rotate like a rigid body about an edge of the support area (in rectangular foundations).

The analysis should start by defining the design loads and their combinations. In the absence of other specific guidelines appearing in mandatory regulations or in other publications in this ROM Programme, the recommendations given in Subsection 3.3.6 should be followed. The following values were indicated there as load factors:

- Permanent Actionss

Unfavourable	γ_g = 1.0
Favourable	γ_g = 0.9

- Variable Actions

Unfavourable	γ_q = 1.5
Favourable	γ_q = 0.0

To analyse rigid overturning only the fundamental combinations and the extraordinary ones (accidental or seismic) are to be considered. It is not necessary to consider the quasi-permanent combination, which is only recommended in this ROM 0.5 for analysing Ultimate Limit States when they are of type GEO. Rigid overturning is not a geotechnical case.

Classifying actions (or their effect) as favourable or unfavourable calls for a certain judgement on the part of design engineers. This subsection and 3.7.11.3 give some criteria to serve as guidelines on this topic.

After knowing the actions and combining them adequately, what needs to be calculated is the location of the point at which their resultant passes through the contact plane between footing and ground. The check should be taken as successful when this point remains inside the support area.

In any event and unless special justification is given, for all permanent or quasi-permanent actions, it is recommended that the effective resultant of the loads lies within the core of the foundation base (central third in a rectangle). Otherwise, there will be a permanent (or frequent) gap under the foundations and this may prove problematical in long-term behaviour.

The load factors indicated are those recommended in most texts on the subject and can be interpreted as corresponding to the high SERI works defined in the ROM Programme. Consequently, its recommended to modify these coefficients when verifying overturning in works with different nature. In the absence of better information, it is advisable to carry out this modification in the same manner as suggested for the geotechnical safety factor indicated in Subsection 3.3.8.2.

In any event, as this is not a geotechnical failure mode, the analysis method for rigid overturning for each particular structure should be sought in the relevant specific ROM publication.

3.5.6.2 *Plastic Overturning*

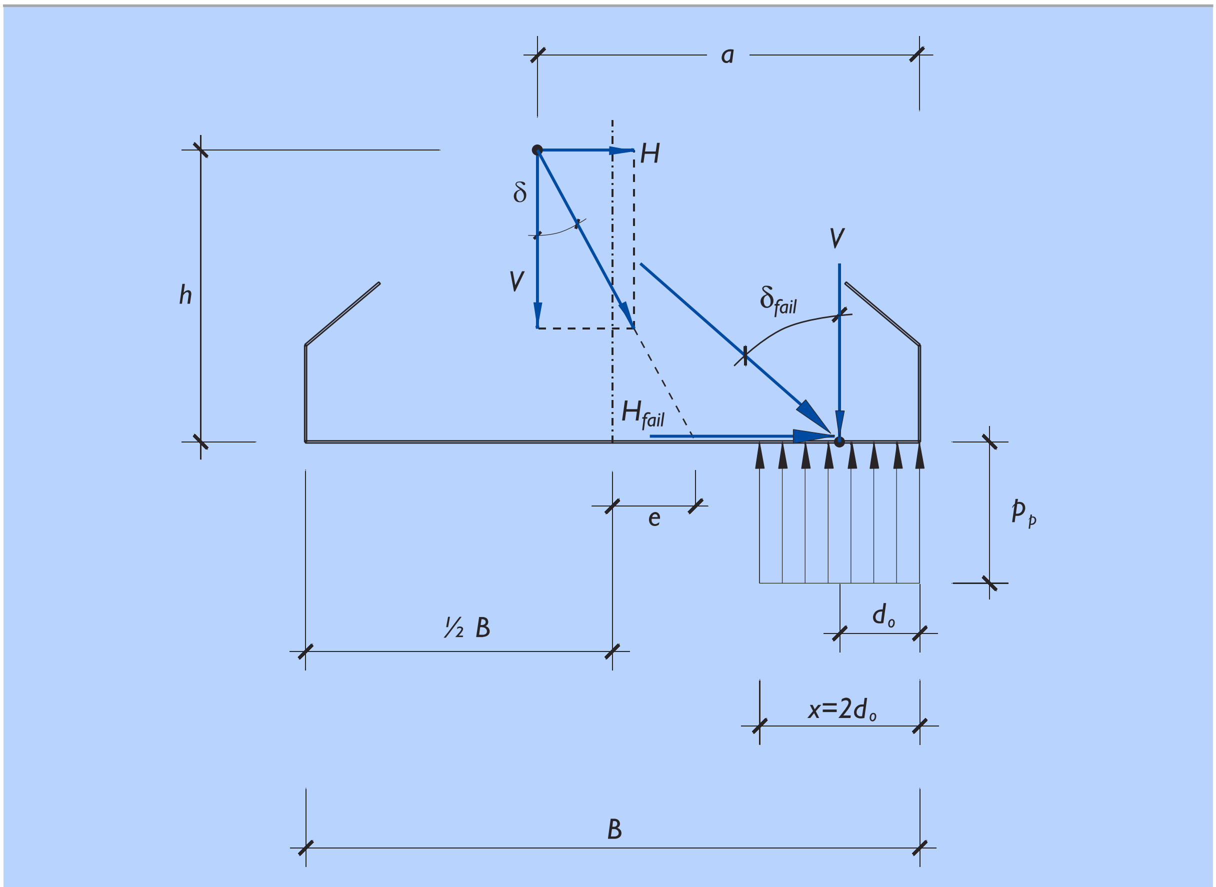
Harbour structures can undergo a type of failure similar to rigid overturning. When the resultant of the loads on the ground approaches the border of the support area, the stress concentration can be high enough to result in local failure (plastification) of the area. The ground would subside, the structure lean and, in the absence of any other support elements capable of restraining the movement, overturning could even take place and the works would eventually fail.

Plastic overturning is so called precisely because of the local plastification that takes place on the edge of the support area when this failure mechanism occurs. As will be seen, ground strength plays a role that can be significant in the analysis procedure indicated in this ROM 0.5, which is why it has been classed here as a GEO type failure mode, as defined in Subsection 3.3.1.

The safety factor against plastic overturning is defined as the number by which the horizontal component of the resultant of actions, *H*, should be multiplied for local plastification to take place in the ground. With the notation given in Figure 3.5.8, several equivalent definitions can result for F:

$$F = \frac{H_{fail}}{H} = \frac{\tan\delta_{fail}}{\tan\delta} = \tan\delta_{fail} \cdot \frac{V}{H}$$

Figure 3.5.8. *Checking Plastic Overturning*



In order to obtain the necessary local plastification in the ground, H should be increased gradually while the other factors defining the design situation remain constant. Apart from the geotechnical parameters, these factors are the vertical resultant of the loads and the distances a and h corresponding to the arms of forces H and V.

Comment: The same safety factor definition is obtained if the following expression is used:

$$F = \frac{M_{resist}}{M_{overturn}}$$

where:

M_{resist} = *moment of the horizontal force causing failure = $H_{fail} \cdot h$. It is the maximum resistant moment.*

$M_{overturn}$ = *overturning moment of the horizontal force = $H \cdot h$.*

The failure condition is reached when the average value of the vertical component of the pressure acting on the compressed area equals the pressure producing the local plastification of the ground. This pressure can generally be assumed to be the vertical bearing pressure p_{vh} defined in Subsection 3.5.4.

The general way of calculating F involves an iterative procedure, increasing H and keeping the other factors constant as stated above, until the failure condition is reached, thereby obtaining the H_{fail} value.

In each of the calculation iterations, a different equivalent foundation breadth, B^*, should be obtained and a different inclination for the loads, δ. This will require repeatedly calculating p_{vh} in each iteration, as its value depends, among other factors, on these two parameters.

If a routine is used for computing the bearing capacity by any of the procedures described in Subsection 3.5.4, if would suffice to change the horizontal component of the load, H, keeping its arm, h, constant until bearing failure is reached (safety factor against bearing failure equal to 1).

This ROM 0.5 describes several methods for studying bearing failure. Some of them, depending on the ground type and kind of information available, will have been used in the check on the foundation bearing failure. This same method can and should be used again at this stage to analyse plastic overturning. To this end and given the specific characteristics of the overturning, it is advisable to take into account the following remarks.

a. Empirical Methods for Calculating Safety Factors

All procedures for calculating bearing capacity have a certain empirical component. However, the *empirical procedures* for obtaining the bearing capacity of shallow foundations are understood here to be all those mentioned and described in 3.5.4, with the exception of the one known as the Brinch Hansen analytical method described in Subsection 3.5.4.8.

For the comparable experience method described in 3.5.4.2, the value to be used for the pressure originating local plastification, p_p, is equal to three times the allowable pressure that experience has sanctioned for narrow foundations, multiplied by the factor f_δ. That is,

$$p_p = 3 \cdot p_{vadm} \cdot f_\delta$$

where:

$$f_\delta = (1{,}1 - \mathrm{tg}\ \delta)^3$$

Throughout this subsection, the angle δ refers to the inclination of the load to the vertical.

For the method described in 3.5.4.3 (foundations on sand, based on SPT results), the value to be used for the plastification pressure, p_p, should be three times the $p_{v,adm}$ defined in that subsection, calculated for the case of a small foundation width, and factored by f_δ , as just defined. It is worth noting that the maximum value that could be obtained for somewhat buried foundations is:

$$p_p = 48 \cdot N \cdot f_d \; kPa \qquad \text{(N = SPT-index corresponding to the support area),}$$

when the ground is dry, and a maximum of half that value when it is submerged (see the details indicated in that subsection for a more precise calculation).

For the calculation method based on pressuremeter tests (Subsection 3.5.4.4) and the method based on static penetrometer tests (Subsection 3.5.4.5), the following value should be adopted:

$$p_p = p_{vh}$$

when p_{vh} is calculated as defined there for foundations with small breadth B^* (when B^* is small, the D/B^* factor can be assumed to be equal to 1, provided the foundations are buried to a certain extent).

The method based on other field tests (Subsection 3.4.5.6) requires converting the results, by correlation with the preceding methods. Once the corresponding equivalence is made, p_p can also be obtained.

When using the method described in Subsection 3.5.4.7, for firm cohesive soils and rocks, it can be assumed that:

$$p_p = p_{vh}$$

and calculating p_{vh} for the case of a small equivalent foundation breadth, B^*. As a general rule, the value for f_D defined there is considered to have a unit value, $f_D = 1$.

With all these methods, it turns out that p_p is a function of the angle δ of deviation from the vertical of the loads, i.e., p_p can be expressed as follows:

$$p_p = p\,(1.1 - \tan\delta)^3$$

After obtaining the value of p by one of the procedures indicated, the condition of plastic overturning can be set out as follows:

$$\left(a - h\ \tan\delta_{fail}\right)\left(1.1 - \tan\delta_{fail}\right)^3 = \frac{V}{2p}$$

The meaning of a, h and V is shown in Figure 3.5.8 and the manner of obtaining their values was described in the preceding subsection. The current subsection recommends procedures for obtaining p , which –in turn– allows the δ_{fail} value and, eventually, the safety factor to be calculated:

$$F = \tan\delta_{fail} \cdot \frac{V}{H}$$

b. Calculating the Safety Factor with the Brinch Hansen Polynomial Formula

The use of this calculation method requires running a series of trial iterations, assuming each time a predetermined result for the desired safety factor against overturning.

For a particular trial, the safety factor against overturning will be suposed to be F_i. This assumption will give the two components of the resultant of the loads:

vertical component = V,
horizontal component = $F_i \cdot H$,

and the corresponding moment about the border of the foundation will be calculated as:

$$M = V \cdot a - F_i \cdot H \cdot h$$

which should turn out to be positive. If not, the F_i will need to be reduced. The plastic overturning safety factor is always less than the limit value $F_{máx} \leq (V \cdot a) / (H \cdot h)$.

Accordingly, the distance from the point where the resultant intersects the base of the foundation to its edge (see Fig. 3.5.8) is:

$$d_0 = \frac{M}{V}$$

Hence, the design parameters needed for the polynomial formula can be obtained:

$$\tan\delta = F_i \cdot \frac{H}{V} \quad \text{and} \quad B^* = 2 \cdot d_0$$

The other intervening factors (ground strength, earth overburden, etc.), which remain constant in each iteration, were already known.

The polynomial formula will supply a value for p_{vh} that should be compared with the value of the vertical pressure corresponding to the F_i value assumed, which is:

$$p_p = \frac{V}{B^*}$$

The computations will conclude if $p_p = p_{vh}$, otherwise the process will need to be repeated until a sufficiently accurate convergence is achieved.

3.5.6.3 Safety against Plastic Overturning

As a general rule and unless a specific indication to the contrary is given in Part 4 of this ROM 0.5, in any other mandatory regulation or in a later publication than this one within the ROM Programme, safety against plastic overturning in low SERI works (5 - 19) is considered acceptable when the safety factor, as just defined, exceeds the minimum thresholds of Table 3.5.6.

Table 3.5.6. Minimum Safety Factors against Plastic Overturning, Works with Low SERI (5 - 19)

Load Combination	Minimum Value, F
Quasi-Permanent, F_1	1.5
Fundamental, F_2	1.3
Accidental or Seismic, F_3	1.1

For works with a minor or high SERI rating or for other allowable failure probabilities, the minimum *F* values given in Table 3.5.5 can be adjusted as indicated in Subsections 3.3.8.2 and 3.3.10. Similarly, they can be adapted for transient situations (including short-term geotechnical situations) in accordance with the provisions in Subsection 3.3.8.1.

3.5.7 Settlement and Other Foundation Movements

3.5.7.1 Isolated Footings

In cases where foundations with isolated footings are designed on the basis of previous experience (Subsection 3.5.4.2) or of the SPT results in granular ground (Subsection 3.5.4.3), settlement of over 1” (2.5 cm) is not to be expected.

In foundations on granular ground whose compacity has been determined with SPTs, the settlement calculation procedure indicated by J.B. Burland & M.C. Burbidge (10) (1985) can be followed.

Settlement calculations for isolated foundations can be carried out by applying solutions from the theory of elasticity contained in texts on foundation engineering. Figure 3.5.9 shows some of the formulae deriving from this theory that could prove of most interest.

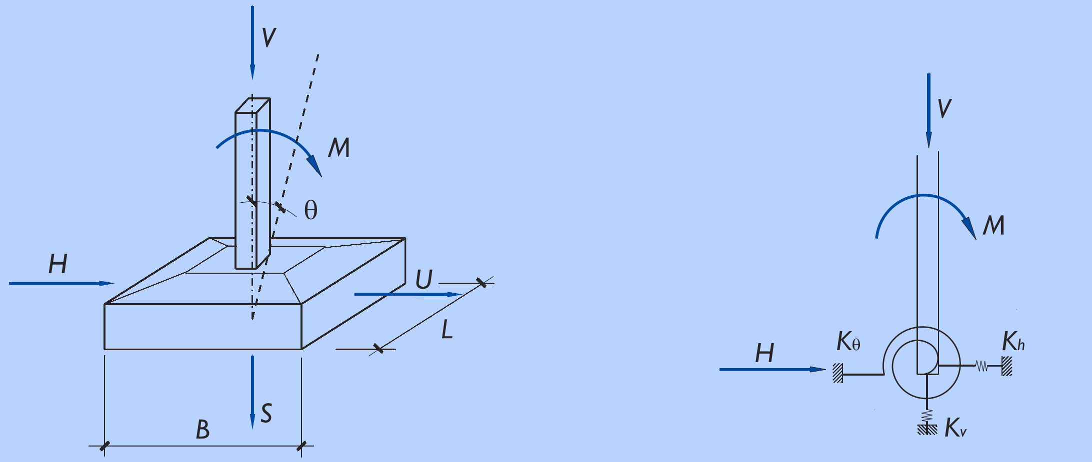
Horizontal and rotational movements can be of interest in some cases and Figure 3.5.5 therefore also includes some typical solutions to estimate these movements.

In shallow foundations with isolated footings in saturated clays, the instantaneous movement can be calculated using apparent elastic parameters related to those corresponding to the soil skeleton by the following expressions:

$$\nu_{apparent} = 0.5$$

$$E_{apparent} = \frac{1.5}{1+\nu} \cdot E$$

Figure 3.5.9. Movements in Isolated Footings and Spring Constants for Soil-Structure Interaction Calculations



REAL FOUNDATIONS

EQUIVALENT FOUNDATIONS

MOVEMENTS

$$s = \frac{V}{K_v} \qquad u = \frac{H}{K_h} \qquad \theta = \frac{M}{K_\theta}$$

SPRING CONSTANT

RECTANGULAR FOUNDATION

$$K_v = \frac{E}{1-\nu^2}\, 1.25\sqrt{B\ L}$$

$$K_h = E\sqrt{B\ L}$$

$$K_\theta = \frac{E}{1-\nu^2}\, 0.25\ B^2\ L$$

CIRCULAR FOUNDATION (Radius R)

$$K_v = \frac{E}{1-\nu^2}\ 2\ R$$

$$K_h = \frac{(1-\nu)E}{(1-\nu)(7-8\nu)} 16\ R$$

$$K_\theta = \frac{E}{1-\nu^2}\frac{4}{3} R^3$$

(10) "Settlements of foundations on sand and gravel". Proc. Inst. of C.E. London.

In the long term, saturated clays will have the settlement corresponding to the elastic parameters of the ground, E and ν.

Furthermore, in both sands and clays, there is a secondary creep settlement which could even take years and can amount to a substantial proportion of the elastic settlement calculated with the effective deformation parameters. The magnitude of this settlement is difficult to estimate and only experience and observation will enable it to be accurately ascertained.

For this reason, unless specific verified information exist, it should be assumed that the loads corresponding to the quasi-permanent combination can in the long term produce an additional settlement of up to 20% of the corresponding elastic settlement.

The flexibility of isolated foundations has an effect on the forces on the structure. Therefore the same solutions from the theory of elasticity, relating loads and settlements, can be used to represent foundation deformability, by means of springs, in the structural calculations where it is advisable to introduce the effect of soilstructure interaction.

The formulae shown in Figure 3.5.9 do not take into account the contribution from the foundation depth or embedment, which can reduce the movements to much lower values. To include this and other possible favourable effects, engineers should consult technical books on the subject or carry out numerical computations that lie outside the scope of this ROM 0.5.

3.5.7.2 Strip Foundations

b. Rigid Strip Foundations

Movements in rigid foundations with elongated shapes can be estimated using the formulae shown in Figure 3.5.9.

With this formulation, for very long foundations, both settlement and horizontal displacement increase indefinitely with depth. This does not occur, however, with foundation rotation. This defect in the theory of elasticity used, which assumes a uniform modulus of elasticity with depth, is difficult to mitigate except in situations where there is a bedrock which can be assumed to be non-deformable. In such cases, the Steinbrenner method described in 3.5.7.3 can be used for calculating settlement. As there is a relationship

$$K_h = 0.8\,(1 - \nu^2)\,K_v$$

independent of length for elongated foundations, this expression can be used to estimate horizontal displacement once the settlement has been calculated.

b. Flexible Strip Foundations

On average, settlement in flexible foundations is the same as in rigid foundations of similar plan dimensions. The ground movement however will not be uniform and therefore, in eventual soil-structure interaction analyses, the equivalent springs used to represent ground deformation should not be concentrated at a single point.

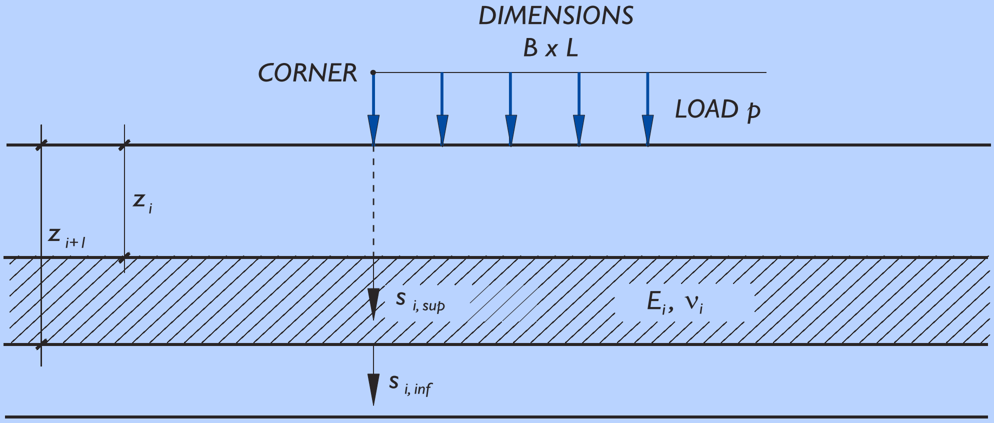
The following subsection provides various indications for the case of slabs. These ideas are also applicable to flexible strip foundations.

3.5.7.3 Settlement for Slabs and Spread Loads

The main movements in foundation slabs and other spread surface loads are settlements.

The most suitable procedure for calculating settlement in the case of spread loads is known as the Steinbrenner method, the formulae for which are shown in Figure 3.5.10.

Figure 3.5.10. **Steinbrenner Method Formulae**



SHOTERING OF LAYER $i = s_i(z_i) - s_i(z_{i+1})$

FORMULA FOR SETTLEMENT AT EACH DEPTH

$$s(z) = \frac{p\,B}{2\,E}\,(A\,\phi_1 - B\,\phi_2)$$

where:

$$A = 1 - \nu^2$$

$$B = 1 - \nu - 2\nu^2$$

$$\phi_1 = \frac{1}{\pi}\left(\ell n\,\frac{t+n}{t-n} + n\,.\,\ell n\,\frac{t+1}{t-1}\right)$$

$$\phi_2 = \frac{m}{\pi}\,\arctan\frac{n}{t\,m}$$

$$m = \frac{z}{B}$$

$$n = \frac{L}{B}$$

$$t = (1 + n^2 + m^2)^{\frac{1}{2}}$$

To calculate the deformation of a particular ground stratum, *i*, it must be assumed that all of the subsoil is homogeneous and consists of that ground. The formulae make it possible to calculate settlements at the two depths demarcating the layer and their difference will therefore be the desired shortening.

The sum of the shortening in each of the strata enables surface settlement to be evaluated.

The formulae shown correspond to settlement under the corner of a rectangular loaded area. Superimposing several loads of this type can represent the case under analysis.

With settlement at different points having been calculated, spring constants can be prepared and distributed at the computational nodes of the foundation slab structure. This set of springs will produce settlement distributions similar to the ones obtained from the analytical calculations when they are used in subsequent soilstructure analyses for different load combinations.

Somewhat simpler, although less to be recommended, are interaction analyses carried out by applying Winkler's moduli. The spring constants in these cases are uniform throughout the slab extension and directly proportional to the area allocated to each node. The curvature induced by ground deformation under uniform loads would not then be reproduced.

3.5.7.4 Allowable Settlement

Foundation movements begin to occur as soon as construction starts. As loads are being applied, more displacements take place; therefore, only part of the total movements can have negative effects on the structure.

Furthermore, when construction is completed, displacements can be produced not only through variations in service loads or due to the creep referred to in 3.5.7.1, but also through other external causes, such as variations in the groundwater table, nearby construction activities (excavations, fills, stockpiling) and even vibration, either due to construction activity, exploitation or even seismic effects. In this particular case, the engineer’s judgement is important in estimating the movements that could affect the structure.

The elastic formulae mentioned in the previous sections apply to studying the movements brought about by applying certain loads to certain types of foundation geometry. Engineers must investigate which loads cause movements that could affect the structure or, in more general terms, the design solution under examination. They also have to investigate the extent of allowable movements.

This subsection is designed to provide some help in evaluating limit or maximum allowable movements.

Allowable movements will depend on the type of structure involved and can vary from the very small, in the order of millimetres (foundations for sensitive equipment) to the very large, of several centimetres or even decimetres in the case of settlement affecting blockwork quay walls, for example.

Differential movement between two parts of a structure is the one that can induce loads that may damage it and this differential movement will represent a fraction of the total movement. Angular distortion is defined as the quotient between the differential movement and the distance between the points at which is it produced. Common concrete and steel structures can withstand distortions of a maximum of around 1/150 and this tolerance may only be half that amount (1/300) if they support rigid closing walls that may crack.

In common building structures, it is very usual to limit angular distortion to 1/500.

With respect to foundation rotation, allowable values can vary greatly, although the following are considered limits:

- walls and retaining structures; a maximum allowable inclination of 0.6%
- isostatic concrete or steel structures and steel storage tanks, 0.4%
- hyperstatic concrete or steel structures and ordinary buildings with reinforced concrete structure, 0.2%
- structures supporting movement-sensitive machinery, 0.1%.

With respect to maximum settlement, the most common limitation (apart from the one relating to possible angular distortion) is 2.5 cm with isolated foundations and up to 5 cm with slab foundations.

The limit movement for each project design must be defined by a reasoned procedure in each specific case. The allowable limits given here should only be considered for guidance purposes.

3.5.8 Comments on Structural Design

The part of a structure in contact with the ground (the foundation structure) must be designed as one more structural element.

In calculations for verifying this structure, the ground reactions obtained from the geotechnical calculations described in the preceding points must be taken as earth pressures acting against the structure.

These ground actions will have the same nature as the external action causing them.

Since the procedures for verifying safety in structural calculations call for the loads to be considered differently (with load factors corresponding to Ultimate Limit States of type STR), when verifying the Ultimate and Serviceability Limit States of the foundation, supplementary geotechnical calculations are recommended to estimate the ground reaction to the other design situations set out in the structural calculations.

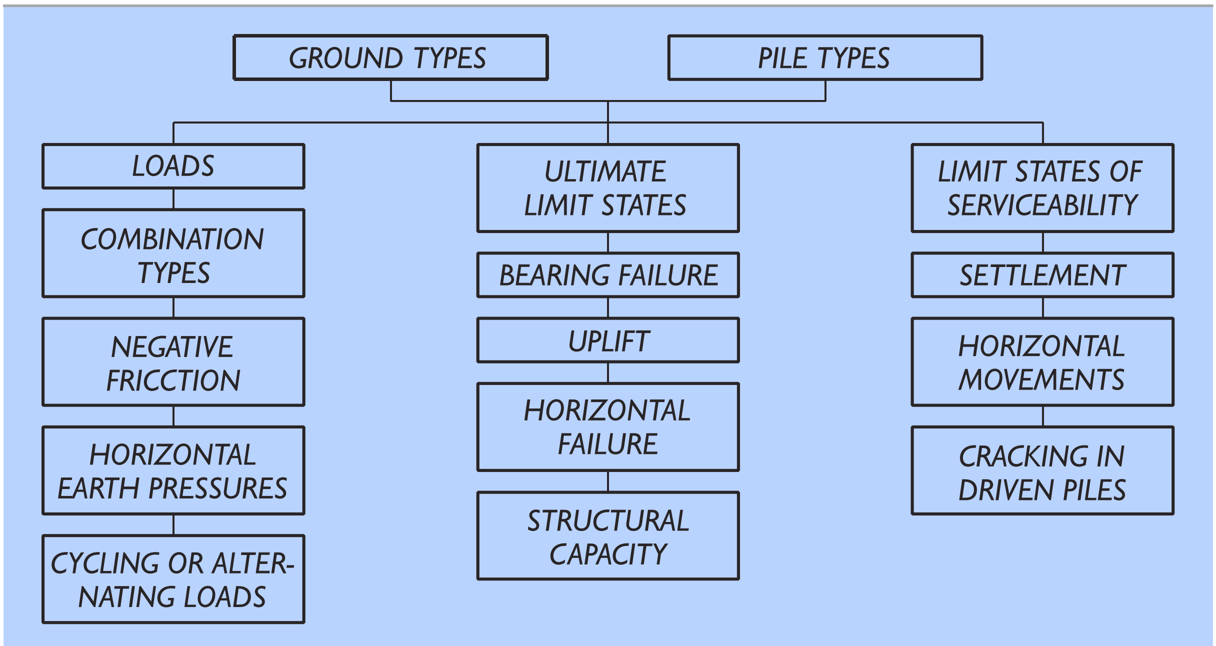
3.6 DEEP FOUNDATIONS

3.6.1 General Aspects

The presence of low-strength soil near to the surface of the ground is one reason, among others, making it necessary to transfer loads down to a certain depth. One of the most common procedures for deep foundations is piling and this is dealt with below.

The matters on which recommendations will be made are shown in Figure 3.6.1. Some relate to the way safety is assessed with respect to various Ultimate Limit States and others relate to recommendations about the procedure for evaluating various Limit States of Serviceability.

Figure 3.6.1. **Main Topics to Be Considered in Deep Foundations**



These Recommendations do not cover all the matters that could arise and need to be dealt with in a design for deep foundations. In an attempt to reduce this shortcoming, certain general considerations have been included when discussing Ultimate and Serviceability Limit States.

Since specific analysis methods are used in this section depending on the type of ground or pile involved, some subsections on these two aspects are included first to define the concepts needed for subsequent development of the Recommendations.

3.6.1.1 Types of Ground

For the purpose of checks and calculations to be carried out in deep pile foundations, three basic types of ground can be identified - rock, granular soil and cohesive soil.

At a particular site, the ground will often be heterogeneous and therefore require zoning according to the three classes mentioned above.

Intermediate ground that is difficult to classify in one category or another is also commonplace. In such cases, engineers must repeat the checks shown here assuming different categories of ground and choose the most conservative results -from reasonable hypothesis- as the outcome of their analysis.

The definitions of these three ground classes and the data necessary for analysing piles are given below.

a. Rocks

For the purpose of piling studies, the ground is considered a rock when:

- In boreholes carefully drilled by rotation, over 50% of the core can be recovered.
- The unconfined compressive strength of intact core samples systematically exceeds 1 MPa.
- The mineral aggregations composing them are stable over time and therefore do not undergo substantial changes during the works design life.

As can be seen, this class can include some over-consolidated clays, while highly weathered or jointed rocks could be excluded

The analysis of rock foundations requires knowing the topography of the bedrock along with its nature, degree of jointing and weathering and the unconfined compressive strength of the freshest fragments.

Furthermore, if the nature of the project calls for foundation deformation analysis, it will be necessary to know the deformation moduli of the rock.

Part 2 of this ROM 0.5 describes investigations appropriate for these purposes.

b. Granular Soils

Granular soils are defined as cohesionless soils. It should also be assumed that they are considerably permeable, so that the processes of variation in porewater pressure inside them occur over negligible time periods compared with those of application of the foundation loads. The minimum permeability for this classification is 10^{-4} to 10^{-5} cm/s.

This group of granular soil includes sands and gravels, even with a noticeable silty fines content, about 15% maximum, as well as some highly fractured or weathered rocks, provided that the weathering product is not clayey (thus conferring appreciable cohesion and impermeability on the mass).

The data required for analysing Ultimate Limit States of piles in granular soils include, in addition to a description of their nature (identification tests) and density, some of the following, related to soil strength:

- angle of friction.
- resistance in the SPT.
- resistance in continuous dynamic or static penetrometer tests.
- other tests for obtaining shear strength.

In major projects, it is advisable to obtain this ground strength information by more than one procedure.

Furthermore, in order to analyse displacements, normally associated with Limit States of Serviceability, the deformability of the granular soil must be known. The modulus of deformation of granular soils tends to increase with depth and therefore is not usually defined by a single number but by a graph showing how the modulus varies with depth.

In studying horizontal pile deflection, it is traditional to define granular soil deformability by a modulus of subgrade, normally named n_h, and which is discussed in Subsection 3.6.9.2.

c. Cohesive Soils

In pile foundations analysis, the term *cohesive soil* refers to ground with cohesion, provided that its unconfined compressive strength is less than 1 MPa and that it is sufficiently impervious for not being classified as granular.

In the study of pile foundations involving cohesive soils, in addition to their nature (identification tests) and density, it is necessary to ascertain some strength parameters, both short-term (the undrained shear strength) and long-term (drained shear strength) plus some data on their deformability.

The exploration and characterisation procedures for this type of soil are described in Part 2 of this ROM 0.5.

3.6.1.2 Types of Piles

Piles can be classified by a variety of criteria depending on the aspect of most interest. For the purpose of the following recommendations, it is advisable to classify piles into two types depending on their construction procedure.

a. *Driven piles*, also known as *displacement piles*. Their basic characteristic lies in the compaction their installation can induce in the surrounding soils, since the pile is inserted into the ground without any pre-boring which may facilitate the driving.
b. *Cast-in-situ* excavated or drilled piles, sometimes known as *piers*, which are installed in shafts previously excavated in the ground.

Intermediate types of pile can exist between the two above, such as piles driven into partial pre-shafts that are smaller in size than the pile. Piles driven with the aid of water jets (to go through harder layers) should also be included in this intermediate group.

This ROM 0.5 will also draw a distinction between concrete, steel and timber piles as the pile shaft resistance in each case can vary.

Distinctions with respect to the internal structure of piles, reinforced as opposed to pre-stressed concrete, or composite structures with concrete and steel sections or concrete-filled tubular steel piles, are of secondary importance from the geotechnical point of view even though they are of decisive importance when assessing their structural strength.

Pile shapes (circular, ring-shaped, tubular, square, H-shaped, etc.) are of some importance in terms of the procedure for evaluating their bearing capacity, particularly the tip shape of driven piles when the driving continues until penetrating rock or a resistant stratum.

3.6.2 Most Common Failure Modes

3.6.2.1 Ultimate Limit States

3.6.2.1.1 Ultimate Limit States of Geotechnical Type (GEO)

From the geotechnical Ultimate Limit States (GEO) that can occur, the failure modes indicated below should be considered for piles.

a. Bearing Failure

This is the classic failure situation. The vertical load on the pile head exceeds the ground resistance and disproportionate settlement occurs.

Section 3.6.6 gives recommendations on procedures for checking safety against bearing failure.

b. Uplift Failure

Piles can be used to withstand tensile loads at their head. If this load exceeds the *uplift* (or *pullout*) resistance, the pile will break its connection to the ground, thus producing the consequent failure.

This type of mechanism and the appropriate evaluation procedure is dealt with in Subsection 3.6.7.

c. Ground Failure owing to Lateral Loads

When horizontal loads applied to piles produce stresses in the ground that it is unable to withstand, excessive deformation occurs, or even overturning if the pile has sufficient structural capacity.

The limit horizontal load that isolated piles or pile groups can withstand is dealt with in Subsection 3.6.8.

d. Overall Instability

The structure and its pile foundation can undergo a joint collapse as a result of failure mechanisms developing deeper than the foundation or shallower ones shearing the shaft of the piles.

This type of failure should be analysed using the limit equilibrium methods described for slope stability in 3.8.

3.6.2.1.2 Ultimate Limit States of Structural Type (STR)

Among the Ultimate Limit States of structural type (STR), this ROM 0.5 provides suitable procedures for carrying out part of the study of the following failure mode:

a. Pile Structural Failure

Loads transmitted to piles through their heads induce forces and moments in the piles that can damage their structure.

Criteria on procedures for computing stress resultants in piles are given in Subsection 3.6.10 of this ROM 0.5.

The verification criteria for the structural capacity of piles should be the same being applied to other structural elements, bearing in mind the different pile materials and execution methods that may be used in each particular project.

b. Other Structural Failure Modes

Though not mentioned in this ROM 0.5, as the topic is considered to lie beyond the scope of this publication, it must be stated that the structural analysis of piles is fundamental. The loads generated in the handling, transport and driving of precast piles must be taken into account. It is also crucial to analyse buckling. And an essential task of structural design consists of studying the caps and decks to which piles are attached and especially the behaviour of pile-structure connections.

3.6.2.2 *Limit States of Serviceability*

Serviceability Limit States in deep foundations are normally associated with movements. In this respect, the ideas contained in 3.5.2.2 relating to shallow foundations apply here.

The checks that must be carried out in the design of both isolated piles and pile groups take into account not only the ground strength but also its deformability.

There is an additional problem in the case of pile groups relating to the load distribution between individual piles. Since this aspect is intimately linked with soil deformability and even though the topic is more general in nature, the problem is included under the set of items grouped under the heading of Limit States of Serviceability.

Finally, this set of topics carries recommendations for estimating the forces and moments acting on the buried part of piles as a function of the loads acting at ground level and of ground deformability.

This ROM 0.5 does not cover the study of concrete cracking in piles, as this is judged to be decidedly structural in nature and specific to individual types of works. Nor are specific recommendations made concerning the dynamic behaviour (dynamic soil-structure interaction) of piled structures.

3.6.2.2.1 Other Failure Modes in Deep Foundations

Other failure modes corresponding to Limit States of Serviceability may occur which, although less common, need to be taken into account properly in design or construction work where problems of this type are predictable. The list that follows is not exhaustive.

Geotechnical failure or excessive deformations produced by changes in the geometry of the ground or the works, such as erosion or scour.

Damage to foundations and structures caused by excessive vibration as a result of pile driving. Pile driving in soft clayey soils has and can cause considerable damage even in areas far away from the pile-driving location (over 100 metres).

Environmental attack on the pile material with a subsequent loss of durability and diminished capacity. Even though concrete and timber also deteriorate, the effect of steel corrosion in areas swept by waves, tidal runs or variations in the groundwater table are particularly worthy of mention in this respect.

3.6.3 Defining Design Factors

3.6.3.1 Geometrical Configuration

The most interesting geometrical data (11) for analysing the behaviour of isolated piles are their embedded length and their diameter (or diameters, if not uniform).

In groups of piles it will be necessary to know their number and distribution. The spacing between piles, *s*, should be mentioned as the most indicative parameter.

The geometrical configuration of the subsoil must be known along the pile full length and also deeper than the pile tip levels. With a single pile, the effects of loads will scarcely extend deeper than five times its diameter but in the case of pile groups these effects can be considerable, reaching down under the tips as much as over one and a half times the breadth of the group in plan view.

The descriptions in the following sections refer to pile diameter in general. In non-circular piles, this dimension must be taken as the diameter of a circular pile of equal external perimeter or cross section, depending on the purpose for which the equivalence is necessary.

Piles with elongated rectangular sections should be considered as a special case. They are usually executed with machinery for installing reinforced-concrete diaphragm walls, which are of relatively frequent use.

In these cases and to check the safety against bearing failure, for elongated straight sections, $B \times L$,, the tip can be assumed to have an equivalent area given by the following expression:

Equivalent tip area

$$A_p = 0.6\ BL + 0.4\ B^2$$

where *L* is the bigger dimension of the cross-section.

Bearing elements composed of steel sheetpiles can be treated similarly. For the purpose of calculating safety against bearing failure, the cross-section of the tip can be considered rectangular and equal in breadth to the horizontal distance existing between the vertical planes circumscribing the sheetpiles.

For non-circular shapes, the lateral contour to be considered is the one corresponding to the polygon of the shortest contour length capable of encircling the cross-section of the pile.

Some of the terms used in subsequent sections related to geometrical configuration are shown in Figure 3.6.2.

This figure shows two zones, known as active and passive, close to the pile point. It is in these zones that the ground fails, where the plastic yield condition is reached, during the process of vertical bearing failure. The amplitude of these zones chiefly depends on the ground's angle of friction. For calculating purposes, when necessary, the following assumptions should be made:

- The lower active zone affects a depth under the pile tip equal to:

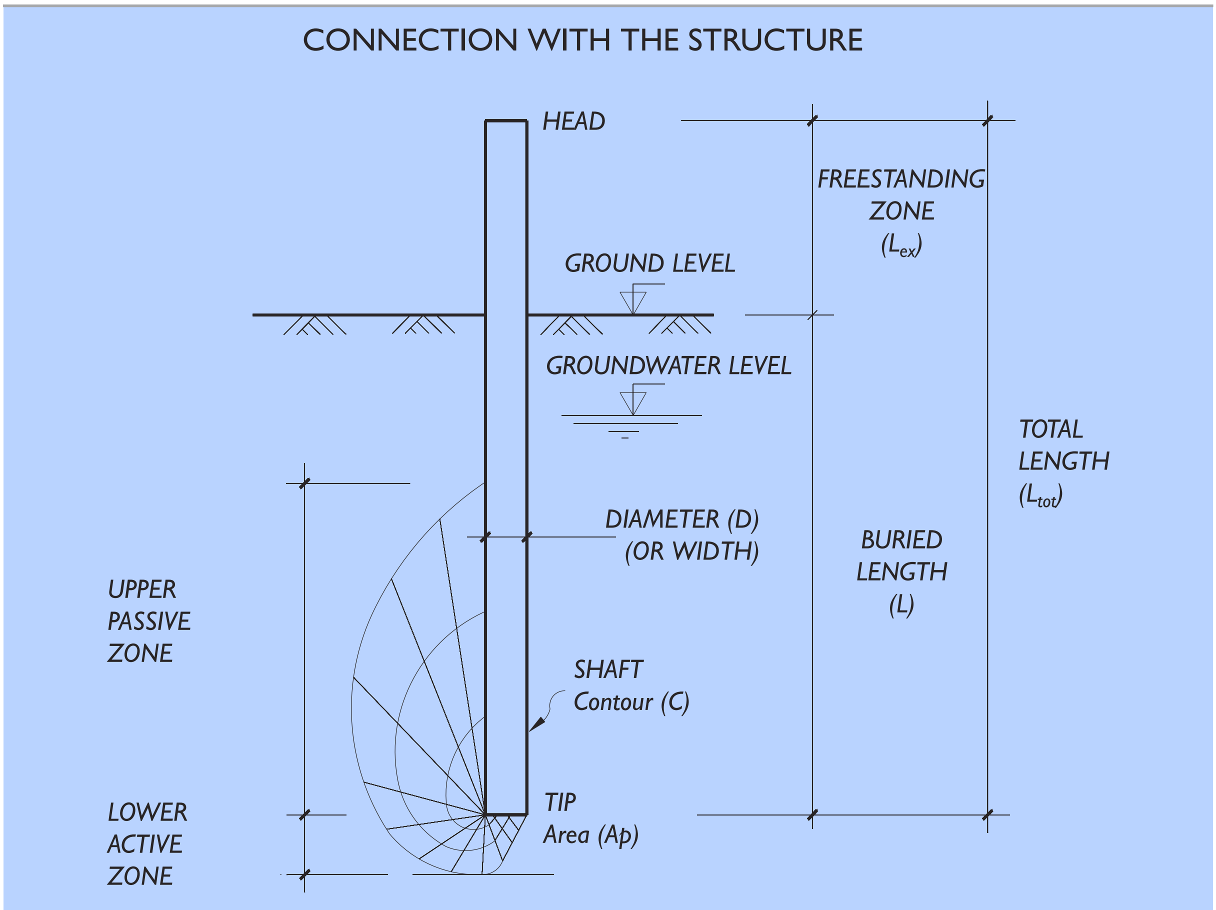
  2D – cohesive ground.
  3D – granular ground and rocks.

(11) Given the conditions of installation, geometrically significant deviations can occur that will have to be limited (tolerances) and taken into consideration in the design calculations.

- The upper passive zone affects a height over the pile tip equal to:

  4D – cohesive ground.
  6D – granular ground and rocks.

Figure 3.6.2. **Pictorial Glossary for Isolated Piles**



3.6.3.2 Ground Characteristics

Ground characteristics should be described using design parameters deduced from the corresponding tests. Depending on the type of ground, different characteristics will be required, as indicated in each case below.

The values of strength parameters must be the best estimates for their average values. To use the Level I calculating method described in this ROM 0.5, partial safety factors should not be used to increase or reduce them. In any event, the ideas set out in Subsection 3.3.5.2 should be taken into account.

3.6.3.3 Actions

Each design situation is intended for verifying safety against an undesirable failure mode. The situation will be defined by various geometrical data and ground characteristics, referred to in earlier sections, and by some load combinations covered below.

The recommendations in Subsection 3.3.5.3 should be taken into account in evaluating the actions.

In order to analyse the limit state of bearing failure, the load combinations producing the greatest vertical component should be considered (or axial component, in a more general case).

For analysing the failure mode consisting of ground failure owing to horizontal loads, the load combinations producing the greatest horizontal component and/or the greatest moment should be considered.

If there are piles in tension, the combination of loads producing the maximum tensile forces should be used to assess safety against uplift.

To study forces and moments acting on piles and to analyse the foundation-structure interaction, as many load combinations as necessary to check the structural capacity of the foundation and the structure itself should be used.

3.6.3.4 Parasitic Effects

Owing to their interaction with the ground, piles can be subjected to loads that will add to those caused by the structure itself they are supporting. These actions are called here *parasitic effects*.

This section refers to the parasitic effects that occur most often. Engineers need to judge their potential occurrence in each works under study and, where applicable, assess their importance following the recommendations given here.

3.6.3.4.1 Negative Skin Friction

Problem Identification

Negative skin friction occurs when the general settlement in the ground surface is greater than the settlement of the pile head. In this situation, the pile supports part of the weight of the ground in addition to the load transmitted by the structure. As a result, the negative friction increases the total compressive load the pile has to bear.

Ground settlement can be caused by the self-weight of the ground (undergoing consolidation) or more frequently by the surface loads (fills) placed after pile construction or before that, but while the consolidation process is still ongoing. It may also occur as a result of lowering the groundwater table. Vibrations and earthquakes can induce settlement giving rise to negative friction.

The problem can be identified by previously computing ground and pile settlement and comparing the two. A small difference in settlement is normally sufficient for negative skin friction to arise. Based on published experiences, a differential settlement of around 1 cm is enough for considerable negative friction to develop.

Calculating Negative Skin Friction

Unitary negative skin friction (force per unit of surface area) should be assumed to equal shaft resistance, as indicated in Subsection 3.6.4.

Technical solutions are available (bitumen treatment for driven pile shafts) that substantially lessen negative skin friction. If any of these techniques are used, the corresponding unitary negative friction value needs to be estimated.

Tangential force, represented here with the variable R^-, is the result of integrating the unitary negative skin friction value, denoted τ^-, over the pile contour and a shaft length going from the head of the pile down to a certain depth *x*, which will have to be estimated. The mathematical expression for this idea is as follows:

$$R^- = \int_0^x \pi D \tau^- dz$$

where *D* is the equivalent pile diameter (equal contour).

For piles going through soft soils but resting on rock, *x* should be assumed to equal the depth of the bedrock measured from the pile head.

For floating piles capable of undergoing relatively large settlement, and only rarely used in ground that can cause negative skin friction, a value for *x* lower than the length of the pile can be estimated, by applying the simplified procedure described below.

Firstly, the general settlement in the area should be calculated, assuming that no piles exist. This calculation should include not only a determination of the settlement occurring at the surface but also at any depth. It is a good idea to obtain points of the pile settlement distribution at several depths. This is Curve 1 from Figure 3.6.3.

Secondly, settlement of the pile (or group of piles, if applicable) will have to be examined. To this end, it can be assumed that the piles are rigid and that their settlement is given by a value, s_p, which depends on the negative friction value. The greater the depth at which negative skin friction is still assumed to act, the greater the load transmitted to the ground below ($Q + W + R^-$) and the smaller the extent of the support zone. As a result, the settlement of the pile (or the pile group where applicable) is a function that clearly increases with the depth *x* up to which the negative skin friction is assumed to act. This is Curve 2 shown in Figure 3.6.3.

Having made these calculations, depth *x* should be chosen so that it leads to a pile settlement similar to the ground settlement at the same depth when no piles are present. This is the ordinate of the intersection of Curves 1 and 2 drawn in Figure 3.6.3.

Settlement should be calculated with a suitable procedure from those presented in this ROM 0.5 or from those endorsed by experience for similar situations.

A negative skin friction exists for every service load *Q*. It is always on the safe side to estimate this for a low service load and subsequently apply it to any other situation.

Group Effect on Negative Skin Friction

The outermost piles in pile groups should be considered to be subjected to the same negative friction as if they were isolated, particularly the corner piles.

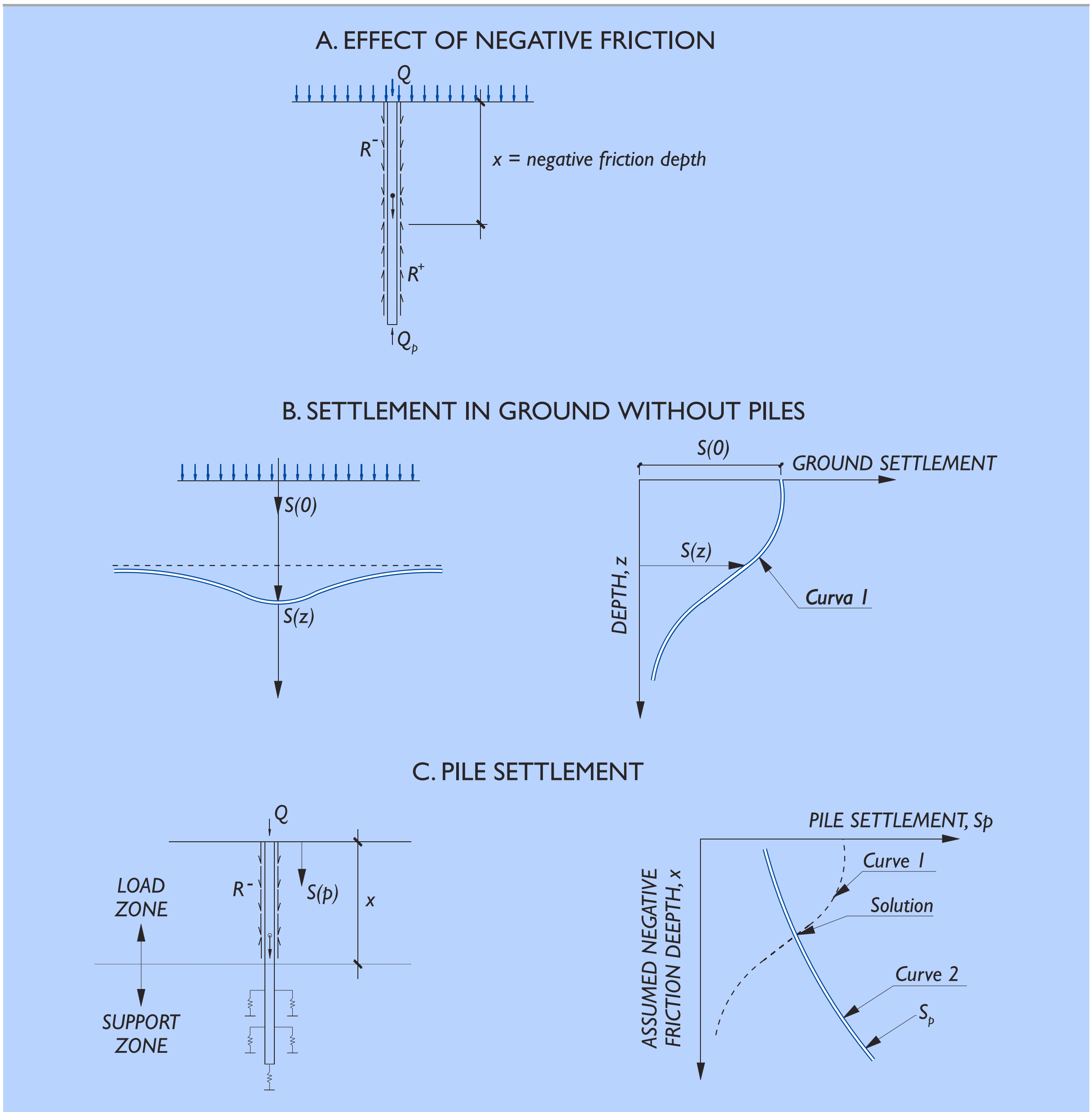
For interior piles, it is possible that the negative skin friction calculated as individual piles and following the above recommendations exceeds the weight of the surrounding soil. For this reason, in the case of dense pile groups or closely spaced piles, it must be checked that the accumulated total negative friction of the inner piles does not exceed the weight of the earth inside the group and above the level where the friction sign changes over. Otherwise, to estimate total negative skin friction, a value should be taken that is less than or equal to the weight of this earth mass.

3.6.3.4.2 Horizontal Earth Pressure Caused by Vertical Surface Overburdens

Surface-placed loads cause horizontal displacements in the ground that can adversely affect nearby pile foundations when deep soft soils are present. A typical situation is illustrated in Figure 3.6.4.

Piles installed in slopes can also be subjected to substantial horizontal loads. This second problem is covered in the following subsection.

Figure 3.6.3. **Procedure for Calculating Negative Skin Friction**



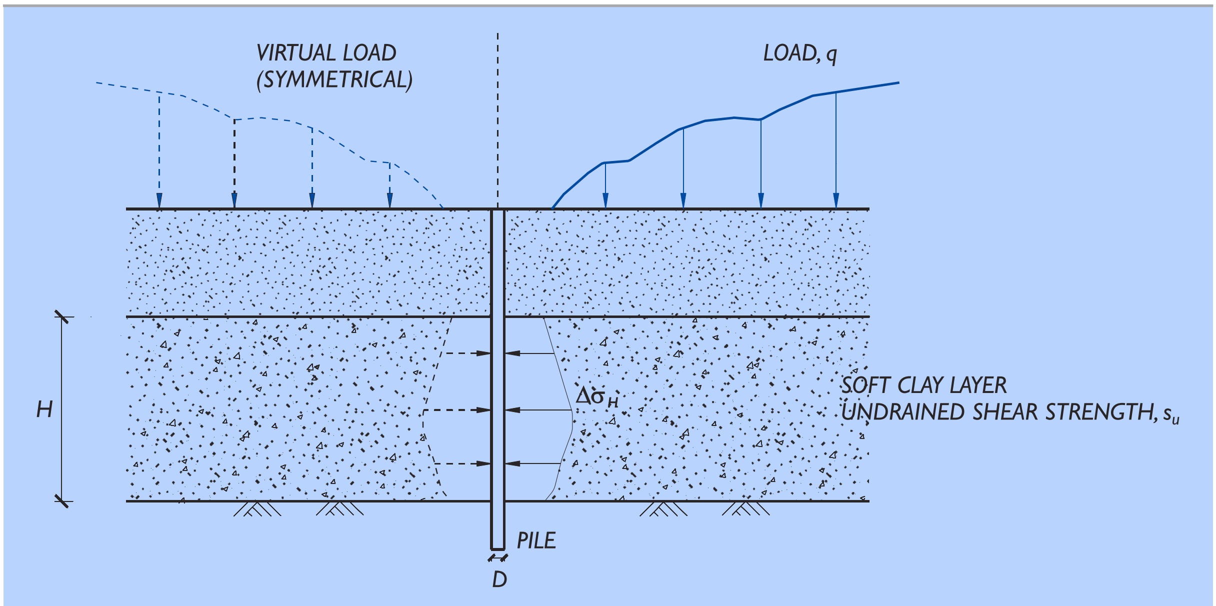
It will not normally be necessary to study this effect when the undrained shear strength of the ground is greater than the following value:

$$s_u \geq q \cdot \frac{D \cdot H}{a}$$

where:

q = vertical pressure applied at the surface.
D = pile diameter.
H = soft soil thickness.
a = 5 m^2 (approximately).

Figure 3.6.4. **Horizontal Stresses. Overburden on Soft Ground**



A study of these problems calls for a ground-pile interaction analysis, which should be more detailed for more critical problems. In the absence of such studies, the simplified method shown below can be followed.

The maximum horizontal earth pressure that a surface load can originate may be approximately estimated when the subsoil is highly deformable and the piles are close to each other, forming a "barrier" totally preventing horizontal ground movement.

When no great precision is required in estimating the forces acting on the piles, the calculation method from Figure 3.6.4 can be adopted. The method is based on the simplified assumptions given below, which generally lead to an overestimation of the horizontal earth pressure.

- Acting as a rigid horizontal barrier, the piling withstands all the horizontal earth pressure generated by the load at each depth z and in a breadth B equal to the lesser of the following:

  a) the width of the loaded area
  b) the width of the pile group plus three times the pile diameter
  c) three times the pile diameter multiplied by the number of piles in the first row of the group, the closest to the load.

- The horizontal pressure generated at a depth z in the ground can be calculated using the theory of elasticity and assuming that the ground is homogeneous and isotropic.

In these conditions, it is possible to calculate the unbalanced horizontal earth pressure corresponding to zero deformation, which would occur when the piling is far less deformable than the ground.

To simulate the condition of rigid horizontal barrier, in addition to the real load, a virtual load is assumed to act, the symmetrical to the real one about the axis of the first row of the pile group under study. With this double load, the plane of symmetry would not undergo any deformation and would be subjected, on one and the other side, to the same increase in horizontal pressure, $\Delta\sigma_H$. The earth pressure on the hypothetical barrier would precisely be this horizontal pressure (necessary to prevent deformation), as the ground on the other side of the barrier, assumed to be highly deformable, would not collaborate in retaining the horizontal earth pressure.

The total earth pressure can be obtained by integrating the horizontal pressures over the height of the soft stratum and over the breadth *B*.

There is an upper bound for the unitary earth pressure, $\Delta\sigma_H$, which is precisely the value of the ground ultimate capacity, and can be estimated using the following expression:

$$\Delta\sigma_H = 1/2\ \sigma'_V + 2\ s_u$$

where:

σ'_V = long-term vertical effective pressure after placement of overburden.
s_u = undrained shear strength of the soft soil.

Comment: *The initial stress state, before placing the earth overburden on one side of the foundation, can be represented by the following stresses:*

σ'_V = *vertical effective pressure.*
σ'_H = *horizontal effective pressure* = $K_o\ \sigma'_V$

With the load applied, one side of the pile row will have an increase in horizontal pressure, $\Delta\sigma_H$, *whereas the increase in vertical pressure may prove to be small and the assumption made that* $\Delta\sigma_v = 0$. *The deviatoric stress (difference in principal stresses) consequently has the following value:*

$$\sigma_H - \sigma_V = \Delta\sigma_H - (1 - K_o)\ \sigma'_V$$

imposing the failure condition

$$\sigma_H - \sigma_V = 2\ s_u$$

gives:

$$\Delta\sigma_H = (1 - K_o)\ \sigma'_V + 2\ s_u$$

and finally, assuming an approximate value of $K_o = 1/2$ *gives the also approximate expression shown in the text.*

3.6.3.4.3 Earth Pressure on Piles Installed in Slopes

Construction of open wharfs, in which a deck is suspended on piles, constitutes a clear-cut example of this type of problem. The horizontal displacement of the earth slope always involves a parasitic horizontal earth pressure that should always be taken into account.

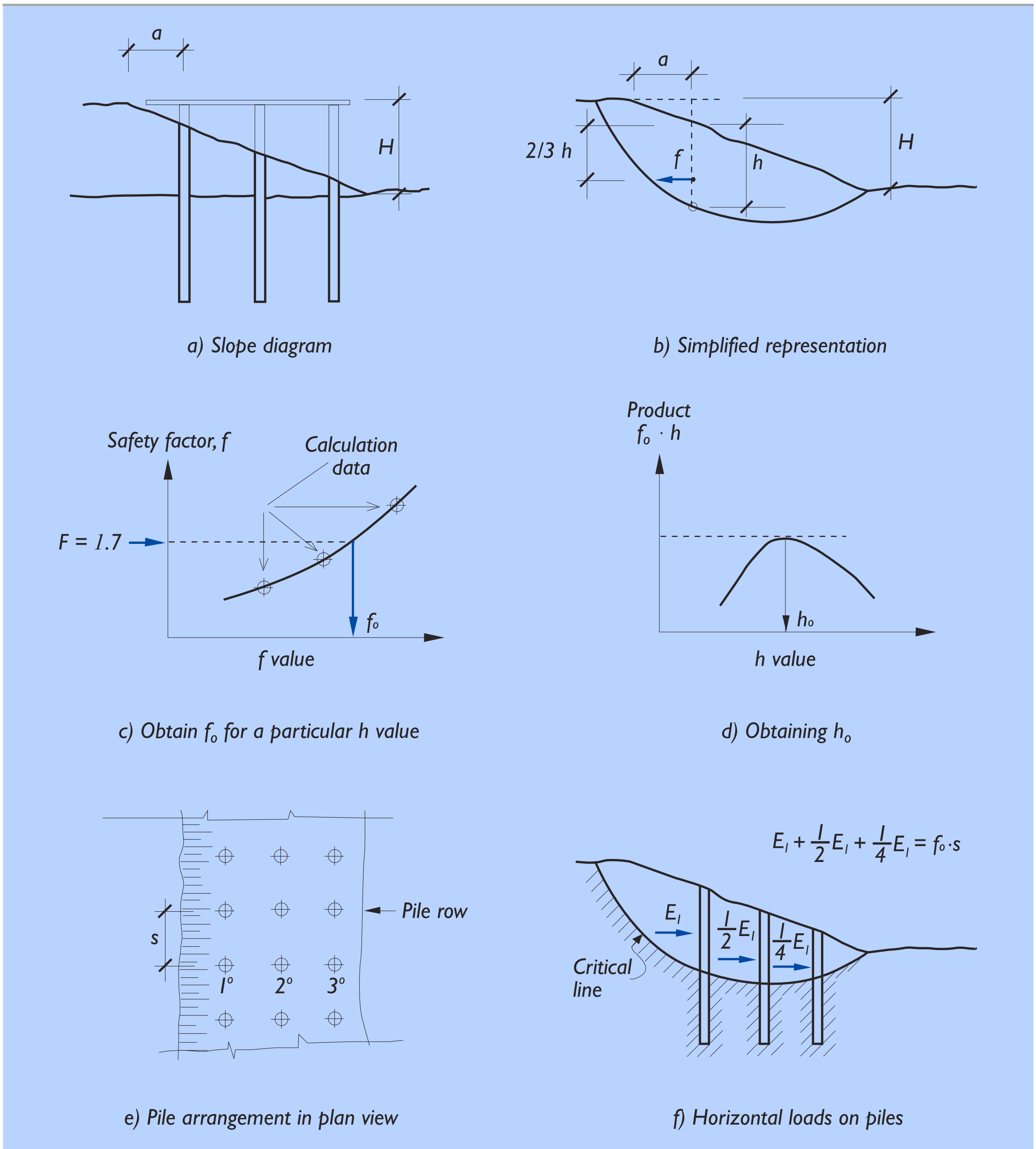
The problem can be substantially alleviated when piles are executed "a posteriori" once the earth slope is created, but this technique may be excessively costly.

When driven piles are executed and a earthfill subsequently placed between them to obtain the definitive configuration, it is always advisable to place the fill in horizontal lifts.

As a general rule, the problem is of such concern that it is advisable to analyse it specifically using a numerical model for the case in question and, in addition, to consult the technical publications describing similar cases where the behaviour has been observed.

In order to obtain just an initial approximation of the pressure acting on the piles, the simple method described below can be followed (see Fig. 3.6.5).

Figure 3.6.5. **Procedures for Estimating Earth Pressures on Piles in Sloping Ground**



The simplified calculation requires the use of a software program for analysing slope stability problems that allows an internal horizontal force to be entered as a load inside the sliding mass.

The joint effect of all piles should be represented by a single internal horizontal force, *f*, to be applied in the vertical of the axis of the piles closest to the head of the slope and at a depth of 2/3*h* from the slope surface, where *h* is the depth of the failure surface in this same vertical. This will require specifying in the calculation that the failure lines pass through a particular fixed point.

The computation should be repeated, keeping *h* constant but changing the value of *f*, in order to find the *f* value that would correspond to a safety factor equal to 1.7. This value will be called f_o.

The process should be repeated for different *h* values with a view to estimating the maximum value of the product $f_o \cdot h$. The corresponding failure line and the associated f_0 and h_0 values should then be used to determine the earth pressure on the full group of piles and its point of application.

The total thrust on all the piles in a *row* (set of piles appearing in the same cross section, see Fig. 3.6.5) will be:

$$E_{total} = s \cdot f_o$$

where *s* is the separation between pile rows. If this spacing is larger than the height of slope, H, this value *H* should be taken as the design separation in calculations.

The total earth pressure on the piles in a row should be distributed among them in a weighted manner. The first pile, the one nearest to the head of the slope, should be assigned double the load of the second and this, double that of the third, and so on.

This load should be assumed to be applied at 2/3 the buried depth of the pile inside the sliding mass of the critical failure line, the one that led to the determination of f_o and h_o.

Some available commercial software make it possible to represent the presence of piles in a slope by other procedures. The approximate procedure just described can be adapted to the program. In any event, it is necessary to obtain the force that would act on the piles if the ground strength parameters (*c* and tanϕ) were 1.7 times lower than the real values and strict equilibrium were reached in these conditions. The forces that would act on the piles in these circumstances are the ones recommended here to estimate the slope pressures acting on the piles.

For the purpose of structural design, the earth pressure acting on each pile can be assumed to be distributed linearly and increasing with depth from the slope surface to the pile intersection with the critical failure line.

3.6.3.5 Effects of Cyclic or Alternating Loads

Shaft Resistance Degradation

The limited experience available on the behaviour of piles subjected to cyclic compression on their head indicates that when the cyclic load amplitude is large enough to produce alternating compression and tension, a significant change can occur in the pile bearing capacity, whereas cyclic loads that keep a pile permanently in compression appear to have a less significant effect.

Experience also shows that in piles working permanently in traction, the maximum shaft resistance that can be mobilised is lower than the corresponding compressive one. This fact was taken into account in Subsection 3.6.7 when referring to the uplift or pull capacity of a pile. In these cases, the shaft resistance to be considered is half that of compression.

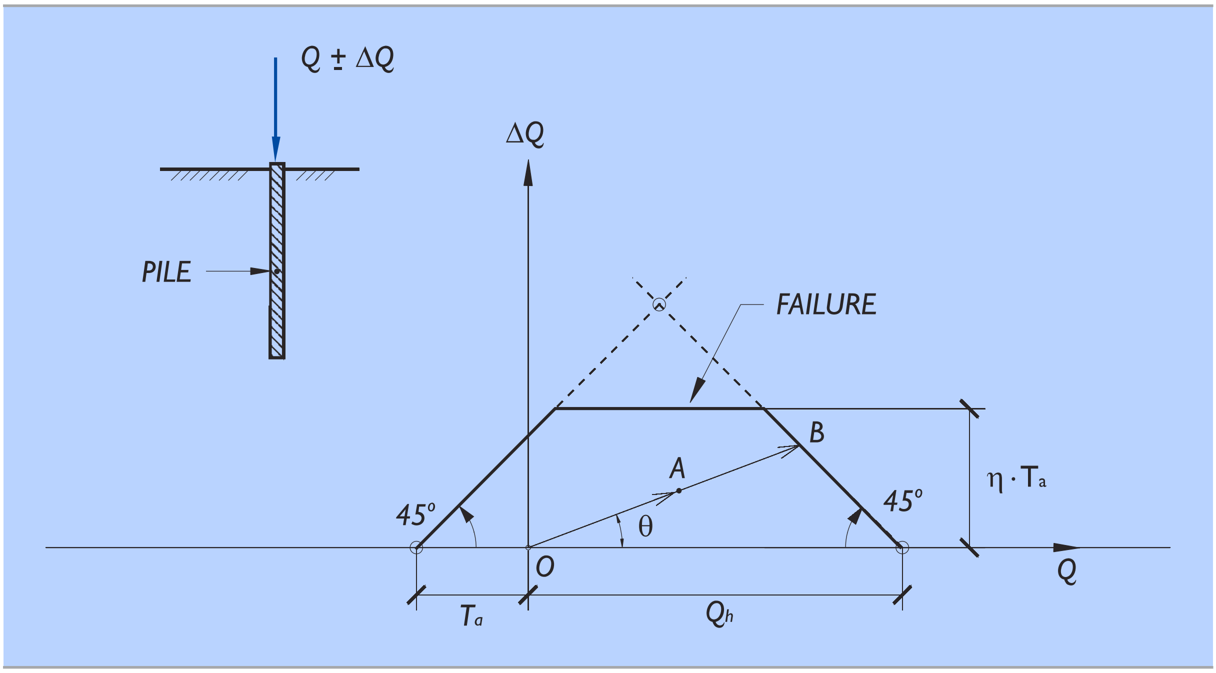
In order to take into account the unfavourable effect of alternating compression and tension, it can be assumed that whenever some tensile load results at the pile head, the shaft resistance should be reduced in one half.

In piles working permanently in tension, it appears that the cyclic effect does not require further reductions, once it has been considered that the direction of pull (uplift) is unfavourable and has already been taken into account when reducing the corresponding resistance caused by this effect.

The verification graph in Figure 3.6.6 was drawn up on this basis [12], enabling the safety factor against bearing failure and/or uplift to be calculated in the case of a cyclic load.

(12) Further detail can be found in H.G. Poulos "Marine Geotechnics". Chapman & Hall. 1988.

Figure 3.6.6. ***Evaluating the Safety of a Pile with a Cyclic Axial Load***



A = representative point of the loads on a pile in a ground-level section

$$\text{Safety factor, } F = \frac{\overline{OB}}{\overline{OA}}$$

The line marked *failure* separates the safe region from the unsafe one. The safety factor corresponding to a particular action (point *A* on the diagram) can be calculated as shown in the figure.

The failure boundary can be constructed with the values for Q_h (static bearing capacity, see 3.6.4 and 3.6.5) and T_a (uplift resistance, also in static conditions, Subsection 3.6.7). The advisable η value to use for calculating the ordinate of the horizontal section of the failure boundary would be given by:

$$\eta = \frac{1}{2} + \alpha - \beta\left(1 + \alpha\right) \geq 1 - \beta$$

where:

α = ratio between the point resistance and the shaft resistance for bearing failure in static conditions;

$$\alpha = \frac{R_p}{R_f}$$

The values for R_p and R_f will be calculated in accordance with the recommendations made in 3.6.4

β = the quotient between the effective weight of the buried pile and the uplift resistance

$$\beta = \frac{W'}{T_a}$$

T_a should be calculated as indicated in 3.6.7.

When the pile axis is not vertical and deviates from this by a certain angle, the effective weight to be used in the calculations should be the real value multiplied by the cosine of that angle.

The η value has been obtained in such a way that, when a the sign of the load acting at the head is reversed in any cycle, the shaft resistance is also reduced by half, even in the case of a failure under compression. Some interaction diagrams (lines composing the failure boundary) have been obtained with numerical models and indicate that these boundaries have a curved "hump", but there is an ample central zone where the condition ΔQ = *constant* comes fairly close to the results.

This diagram shows three regions (marked by abrupt changes of slope in the failure boundary) where the safety verification can be approached differently, as indicated below.

1. High Compression

This situation occurs when:

$$\Delta Q < \eta \frac{T_a}{Q_h - \eta T_a} \cdot Q$$

In this case, it is not necessary to take the cyclic nature of the load into account. All that is needed is to check that sufficient safety against bearing failure exists with the maximum compression $Q + \Delta Q$.

2. Low Compression or Tension

This situation occurs when:

$$Q < (\eta - 1)\, T_a$$

It is not necessary in this case either to take the cyclic or alternating nature of the load into account. It suffices with checking that the uplift safety is adequate for the maximum traction $Q - \Delta Q$.

3. Intermediate Situation

When neither of the above two conditions are fulfilled, the alternating nature of the load will be dominant and then the safety factor will be:

$$F = \frac{|\Delta Q|}{\eta \cdot T_a}$$

The load will normally change irregularly and ΔQ will not have a constant value. To do the calculations in these cases, it is believed to be sufficient to take the ΔQ value as the greatest deviation from the corresponding static value and to do the same calculations as for a cyclic load. Notwithstanding, engineers should consider the possibility of using lower values for ΔQ, as indicated in the paragraphs below when dealing with the duration and the potential impulsive nature of the load.

The previous paragraphs refer to a very high number of load cycles, when a steady-state response is attained (after several hundred cycles). When the number of cycles involved is clearly less, the degradation will also be less. To take this effect into account, engineers will need specific information.

Increase in Resistance with Loading Velocity

Varying the loading velocity in laboratory failure tests means that the greater the velocity, the higher resulting strength will be obtained. Standardised tests are normally carried out with much lower loading velocities than those actually occurring when the piles bear cyclic or alternating loads due to waves. To take this effect into account, a higher resistance could be considered or, alternatively, an adequate reduction of the cyclic component of the load, instead of the peak value mentioned above.

The increases in resistance with load velocity are usually logarithmic. A similar capacity increase occurs each time the velocity is multiplied by the same factor. Floating piles executed in non-permeable (cohesive) ground can increase their axial bearing capacity from some 10% to 20% each time the loading velocity is multiplied by ten. As a general rule, this increase seems more moderate in granular soils.

This effect of resistance increase may compensate the above mentioned effect of degradation and consequently engineers could ignore the oscillating or alternating nature of the load. To this end, they should have specific information relative to the problem concerned. Otherwise, they are advised to exercise certain caution and only take into account the effect of the degradation of the shaft resistance indicated at the beginning of this subsection.

Increase of Deformability

The load-displacement relation in the pile head during a cyclic or alternating load process can be determined as given in Subsection 3.6.9 for the static load case but making certain adaptations.

The ground parameters to be considered in this case are those indicated in Subsection 3.10.2 including the corresponding damping.

Degradation of the ground in the vicinity of the pile shaft makes its deformability increase as it undergoes new load cycles.

The increase in deformability is not important when the axial load remains below certain thresholds, as is the case when applying the safety factors indicated in this ROM 0.5. It is therefore advisable, when specific information is not available and when a higher deformability is unfavourable, to reduce the dynamic deformation parameters somewhat (for instance, by one half, in the dynamic modulus G) in order to take into account the degradation in soil stiffness when estimating movements.

Horizontal Loads

Horizontal loads on the heads of vertical piles and, more generally, actions transverse to pile axes are transmitted to the ground through the pile structure. This generates alternating compressions on the ground that, in certain zones (the closest to the surface) can go so far as to decompress it. The pile may even lose contact with the ground in this area.

To take this into account, back in Subsection 3.6.8 it was recommended to disregard the collaboration from the ground in the topmost area, down to a depth generally equal to one and a half times the pile diameter.

Ground degradation and strength gain owing to the cyclic nature of the load are -in the case of horizontal, lateral load- less important than in the case of vertical load and can consequently be left out of the calculations.

Ground deformability increases with the amplitude of the horizontal load. This increase proves difficult to estimate, as it depends on several factors, including the relative stiffness of the pile with respect to that of the soil itself and the amplitude of the loads.

Under normal service loads, deformability after a large number of load cycles can be in the order of 10% to 100% higher than during the first cycle.

To take this into account, engineers can assume two alternative and complementary design hypothesis – one with the initial soil stiffness and the other with half that value. They would subsequently use the least favourable.

When the topic is of interest, engineers can and should consult the technical literature. In these cases, furthermore, they are highly recommended to run horizontal pull tests (generally transverse) that are relatively simple to perform.

3.6.4 Calculating Bearing capacity with Static Formulae

3.6.4.1 Basic Formulation

The *bearing capacity* of an isolated pile can be divided into two parts for the sake of simplification - the contribution of the point and the contribution of the shaft. Thus the following expression can be written:

$$Q_h + W' = Q_p + Q_f$$

where:

Q_h = cultimate vertical load, producing bearing failure when applied to the pile head: when part of the pile stands free (out of the ground), the "head" should be taken to be the horizontal cross-section at ground level for the purpose of calculating bearing capacity
W' = effective weight of the pile: the submerged weight should be considered beneath the groundwater table.
Q_p = part of the load taken to be supported by the tip: *tip* (or *point*) *resistance*.
Q_f = part of the load assumed be supported by the pile-ground contact area in the shaft: *shaft resistance* (also *skin* or *side resistance*).

The tip and the shaft resistances can be calculated by the following expressions:

$$Q_p = q_p \cdot A_p$$

$$Q_f = \int_0^L \tau_f \cdot C \cdot dz$$

where:

q_p = unitary tip resistance.
A_p = tip area.
τ_f = unitary shaft resistance.
L = length of pile inside the ground.
C = perimeter of pile cross-section.
z = depth measured from the surface of the ground.

As the unitary shaft resistance generally varies with depth, calculating it will normally require integration, as shown in the preceding expression. In cases where the shaft resistance is constant in each layer and the pile contour is also constant in any horizontal cross-section, the expression for shaft resistance is simpler and will be the sum of one term for each layer, i.e.:

$$Q_f = \Sigma\, \tau_f \cdot A_f$$

where:

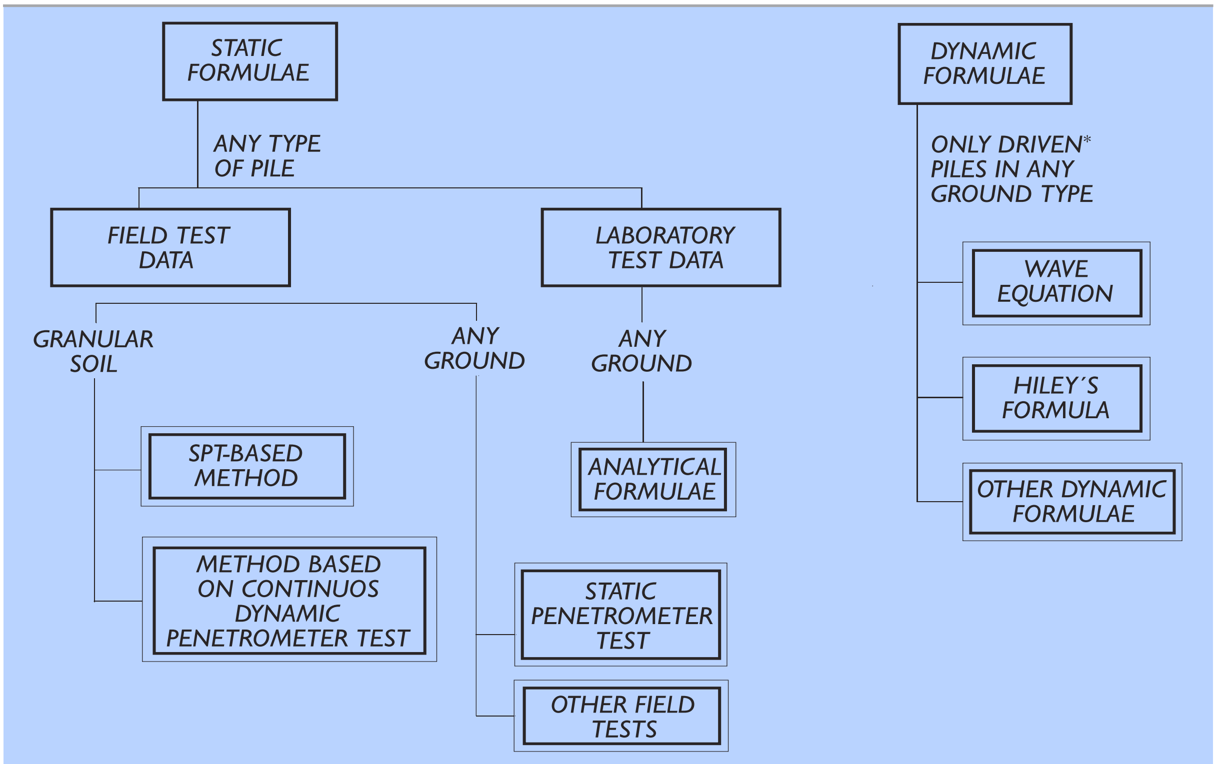
A_f = area of contact between pile shaft and ground in each layer.
τ_f = unitary shaft resistance in each layer.

The values for the tip areas and for the shaft perimeters depend on the geometrical shape of the pile (see Subsection 3.6.3.1).

In open-ended piles, the earth plug that is normally formed at the tip could have a lower unitary tip resistance than the one deduced from this expression. In such cases, it is necessary to investigate a failure mechanism due to movement of the plug at the tip, retained solely by its lateral friction against the inner face of the pile. It is relatively easy to check the length of this earth plug and its development as the driving advances in the case of hollow (not capped) driven piles.

The ultimate load bearing capacity of a vertical pile therefore depends on a crucial parameter, q_p, which defines its unitary tip resistance, and a function describing the variation with depth of another key parameter, τ_f, quantifying the shaft resistance. These two parameters can be assessed by very different methods depending on the type of ground and pile. The most common, and recommended here, are shown in Figure 3.6.7.

Figure 3.6.7. Procedures for Calculating the Bearing Capacity of Piles



* And dynamic checking of cast in situ concrete piles.

It is for engineers to decide on the use of one or several procedures for analysing a specific project, taking into account that some methods are specific to some soil types and can only be applied in such situations.

There are, on the other hand, procedures based on dynamic formulae that can be used at the same time for comparison purposes and which are described in Subsection 3.6.5.

Any one of the procedures indicated can be applied to any works, independently of their importance, although *in situ* load tests are recommended in Category-A works, as defined in Part 2 of this ROM 0.5, to confirm the results of the bearing capacity estimation.

As a general rule, the only reasonable procedure for accurately estimating the bearing capacity is the load test. The other methods are inaccurate or not very reliable and, when using only one of them, will require ample safety factors such as the ones given here after commenting each method.

The formulae and procedures shown below are intended for uniform ground. There will logically be situations in which the pile goes through different ground types.

In such cases of heterogeneous ground, the tip bearing capacity should be assumed to be governed by a ground with the average characteristics of the area between twice the diameter below the tip (lower active zone) and five times the diameter above the tip (upper passive zone), approximately.

The extent of the active and passive zones will depend, at least theoretically, on the angle of friction of the ground and will increase with this angle value (see Subsection 3.6.3.1). Engineers will have to determine the dimensions of such zones in each specific case if this aspect is of particular importance.

When passing through several types of ground, the shaft resistance can be calculated individually for each ground type and then added up. It should be noted that this is not so for column piles resting on rock, where this shaft resistance should not be taken into account in soils with a clearly higher deformability than the one corresponding to the tip area.

After these general remarks, applicable to any method using static formulae for estimating the bearing capacity of an individual pile, the particular details belonging to each calculation method are described below.

3.6.4.2 Bearing Capacity from SPT Values

The SPT-based method for evaluating safety against bearing failure in piles is suitable for granular ground with a low proportion of gravel and can be applied to both driven and bored piles.

Current experience shows that the tip resistance can be evaluated for *driven piles* by the expression:

$$q_p = \alpha \cdot N \qquad \text{(MPa)}$$

where:

N = average value for the SPT N-index - for these purposes, the averages for the lower active zone and the upper passive zone should be obtained and the average of these two values used as N (limiting the N value to 50)

α = dimensionless number depending on ground type and pile size.

The α value should be obtained from specific local data. In the absence of verified comparable experience, it can be assumed that:

$$\alpha = \left(0.1 + \frac{D_{50}}{D_r}\right) \cdot f_D \leq 0.4$$

where:

D_{50} = average size in the grading curve of the sands (mm).

D_r = reference size = 2 mm.

f_D = correction factor by pile size, the value to be taken is:

$$f_D = 1 - \frac{D}{D_0} > 0.7$$

D = pile diameter.

D_o = reference diameter, to be taken as D_o = 3 m.

In the same way, the shaft resistance at a particular level inside the ground for a timber or concrete driven pile can be taken to be:

$$\tau_f = 2.5\ N \qquad \text{(kPa)}$$

For driven steel piles, the τ_f value obtained from this expression should be reduced by 10%.

In each case, N is the SPT value at the level studied. The half-sum of the average value obtained in the active zone and of the average value in the passive zone should be used as the design value for evaluating the tip resistance. The N-index values should be those corresponding to a nominal energy of 60%. If this energy is known for the test in question, the N-indices can be appropriately adjusted.

Engineers are warned that in any event the SPT N-indices should be obtained for effective pressures similar to the service pressure of the foundations in the future. Excavations or fills carried out post-execution of the SPTs can induce changes in the corresponding index, since the test result depends, amongst other factors, on the vertical effective pressure at test level. The correlation indicated in Subsection 3.5.4-.3 is one way to estimate the effect on the N value of the possible difference in pressures caused by an excavation or a fill.

In any event, N-indices of over 50 should not be used for the purpose of these calculations.

Excavated piles can have clearly lower tip and shaft resistances. Ground expansion occurs during excavation in sands and this can lower their strength. In addition, it must be borne in mind that the horizontal compression of the ground against pile shafts is greater in driven piles (displacing and compressing the ground) than in in situ excavated piles. It is important to ensure the proper cleaning of the bottom when boring in-situ piles with tips in sands, as their collaboration in the bearing capacity is counted on. To take these facts into account, calculations of bearing capacity in sands with the SPT method will use the q_p values indicated for driven piles, reduced by factor equal to 0.5, and will apply a reducing factor of 0.75 to the τ_f values. Special procedures (grouting) are available that can substantially increase the resistance of excavated piles.

For *intermediate type piles,* between driven and bored (driven with the help of pre-boring techniques, water jets, etc.), reduction factors should be applied that come between the above values and 1, depending on how much the driving aid is estimated to be capable of reducing the effect of compaction of the granular soil.

An exception to this SPT-based procedure is provided by organic calcareous sand (shell or coralline) that can give high SPT-indices and low bearing capacity in piles, as a result of their cementation being broken by piles, particularly by driven tubular steel piles. In these cases, load tests are the only possible way of estimating the bearing capacity with any degree of confidence. Dynamic monitoring of the pile driving can also give some guidance with this estimation.

The SPT method can only be used on cohesive soil as a general indication. Using an alternative method is strongly recommended.

In the case of gravels or rocks (with the definition of rock given in this ROM 0.5), performing SPTs should lead to systematic refusal, so that the bearing capacity cannot be assessed by this procedure. An alternative method needs to be used in such cases.

3.6.4.3 Bearing Capacity Based on Continuous Dynamic Penetration Tests

If the results of continuous dynamic penetrometer tests are available for granular soil, these results can be converted into SPT-indices using the correlation shown in Part 2 of this ROM 0.5 (Subsection 2.9.5) and the SPT method can then be used.

Given the poor correlation existing between different penetration tests, engineers are advised to turn to local experience or to verification tests carried out at the works site to reinforce the correlation. In any event, they need to exercise reasonable caution when converting dynamic penetrometer tests into the corresponding SPT N-indices for calculating bearing capacity.

It should be especially pointed out that continuous penetrometer tests can lead to optimistic results (high blow counts) at large depths. The results of these tests should not be used, without additional confirmation, to evaluate the bearing capacity of piles with an embedded length over some 15 m, unless a special justification is provided.

3.6.4.4 Bearing Capacity Based on Static Penetrometer Tests

The use of static penetrometers will provide continuous measurement of the unitary resistance of the cone tip, q_c, and shaft resistance, τ_ϕ, in any soil type, depending on the mechanical power of the test equipment.

The values measured can be used in calculating the bearing capacity of vertical driven piles, taking into account the following considerations:

- For the purpose of these calculations, values for q_c greater than 20 MPa should not be used.
- The q_c value to be used to assess the tip resistance should be the average of the mean value of q_c corresponding to the lower active zone and the mean value of q_c for the upper passive zone.
- The mean value should be multiplied by the correction factor f_D indicated in Subsection 3.6.4.2.
- If the shaft resistance has been measured in the test, the corresponding mean value should be directly applied for calculating the pile shaft resistance.
- If the unitary shaft resistance has not been measured by the penetrometer test, its value can be taken to equal 1/100 of the tip resistance at the same level if the soil is granular and around 1/50 if it is cohesive. In any event, the shaft resistance obtained in this indirect manner should not be greater than 0.12 MPa.

In the case of *excavated piles or those driven with some aid* (pre-boring, water jets) installed in granular soil, the reduction factors given in 3.6.4.2 should be applied.

3.6.4.5 Bearing Capacity Based on Pressuremeter Tests

The pressuremeter test gives the *limit pressure* value by applying horizontal pressure against borehole walls. The point resistance of piles is related to this value - a linear correlation exists between the two variables. According to existing experience, the point resistance of driven piles can be estimated as:

$$q_p = K\,(p_l - K_o\,\sigma'_{vo}) \cdot f_D$$

where:

K_o = at-rest earth pressure coefficient.
p_l = effective limit pressure, after subtracting the water pressure measured in the borehole.
σ'_{vo} = vertical effective pressure at test level at the time it was carried out.
q_p = unitary point resistance of driven piles.
f_D = reduction factor owing to the effect of the pile size as indicated in 3.6.4.2.
K = dimensionless correlation factor. If no local studies exist allowing an adequate value to be justified, the following value can be assumed:
K = 1.5 for cohesive soils.
K = 3.2 for granular soils.

For weathered rock, it is prudent to adopt the *K* value corresponding to cohesive soils. If the rock weathering does not lead to the presence of clays, the parameter *K* can be higher, but never greater than the one corresponding to granular soils.

The p_l value to be used in the calculations should be the mean value corresponding to the tip area, as described in Subsection 3.6.4.2.

The shaft resistance can be assumed to be equal to the following value:

$$\tau_f = \frac{1}{30} p_l$$

where:

τ_f = unitary shaft resistance for driven piles.
p_l = mean value of the effective limit pressure at the level under consideration

The maximum τ_f value should be limited to the following values:

τ_f < 125 kPa, granular soils and rocks.
τ_f < 90 kPa, clayey soils.

For driven steel piles, the τ_f value obtained by the above procedure should be reduced by 10%.

For *piles excavated in granular soils*, the reductions to be made are indicated in Subsection 3.6.4.2 - the tip resistance, unless specific construction processes are adopted, should be reduced by 50% and the shaft resistance by 25% with respect to the values indicated for driven concrete piles.

3.6.4.6 Pile Foundations on Rock

In rocky ground covered by soil, it is common to reach the bedrock with the pile tip, which is embedded in the rock. With driven piles it is not easy in practice to ascertain the degree of embedment and an alternative, complementary procedure must therefore be used to estimate the bearing capacity.

The point resistance of a pile embedded in rock can be estimated using the following expression:

$$q_p = \frac{2}{3} p_{vh} \left(1 + 0.4 \frac{L_R}{D}\right)$$

where:

q_p = unitary tip resistance.
p_{vh} = vertical ultimate pressure in rock, obtained as indicated in 3.5.4.7.
When using the expressions indicated in 3.5.4.7, the equivalent foundation width (called there B^*) should be taken as the value of the equivalent diameter of the pile tip. In addition, f_δ = 1 should be taken.
L_R = embedment length in the rock, provided that within that length the rock has the same quality as at the tip. When the embedment is longer than 2.5D, the value to be taken is L_R = 2.5D.

The shaft resistance of piles in rock should only be taken into account in areas where the degree of weathering is equal to III or lower (see 2.2.9.7 for a definition of weathering degrees).

The value that can be used is:

$$\tau_f = \frac{1}{10} p_{vh} < 2\text{MPa}$$

where p_{vh} will be the vertical bearing pressure corresponding to the rock along the shaft, obtained as indicated in 3.5.4.7, and assuming that B^* = D and f_δ = 1.

3.6.4.7 Methods Based on Analytical Solutions

This general procedure is partially based on the theory of plasticity and enables approximate tip and shaft resistances to be obtained for the three types of ground and two types of pile (driven and bored) considered in this ROM 0.5.

3.6.4.7.1 Granular Soils

By analogy with the formulae derived from the theory of plasticity when studying bearing failure problems, it will be assumed that the tip bearing capacity of piles driven into granular soils is:

$$q_p = 3 \cdot \sigma'_{vp} \cdot N_q \cdot f_D \not> 20 \text{ MPa}$$

where:

σ'_{vp} = vertical effective pressure at tip level prior to installing the pile.
N_q = the surcharge bearing capacity factor given by the expression,

$$\frac{1+\sin\phi}{1-\sin\phi} \cdot e^{\pi \cdot tg\phi}$$

where ϕ is the internal friction angle of the soil (this factor is given for each value of ϕ, degree by degree, in Table 3.5.3)
f_D = dimensionless factor defined in 3.6.4.2.

The angle of friction ϕ should be obtained from laboratory tests carried out on undisturbed or only slightly disturbed samples subjected to the high pressures tending to exist in the pile tip area. In the absence of tests, the friction angle can be indirectly deduced from correlations based on the data available.

If the length of the pile through sands L_s is larger than the value:

$$L_a = D \cdot \sqrt{N_q}$$

where:

D = the pile diameter.
N_q = the average bearing capacity factor in the tip area (defined above)

which can occur in long piles, then the value to use should be the one for σ'_{vp} at a depth L_s below the surface of the sands, thus partially taking into account the effect of the critical depth, below which the tip bearing capacity will cease to increase.

Values for q_p of over 20 MPa should not be used in bearing capacity calculations for driven piles, unless they can be specially justified.

Tip resistance in the case of excavated piles should be calculated as shown and a reduction factor of 0.5 applied to the result obtained, unless the settlement will not have important consequences or unless special precautions are taken to improve the contact of the pile tip with the ground.

Shaft resistance in granular soils should be estimated by the expression:

$$\tau_f = \sigma'_v \cdot K \cdot f \cdot \tan\phi$$

where:

σ'_v = vertical effective pressure at the level in question.
K = empirical earth pressure coefficient.
f = reduction factor of the shaft friction.
ϕ = internal angle of friction of the granular soil.

For driven piles, a value of K = 0.75 should be taken and for bored piles, K = 0.5. For hybrid piles, installed with measures reducing ground displacement, an intermediate value should be taken based on the magnitude of the aid employed.

For timber or cast-in-situ concrete piles, a value of f = 1 should be used. For precast concrete piles, a value of f = 1 should be taken and for piles with steel shaft, f = 0.9.

In addition, the τ_f value should not exceed the following limits:

- Driven piles $\tau_f < 125$ kPa
- Excavated piles $\tau_f < 90$ kPa

3.6.4.7.2 Cohesive Soils

The bearing capacity of vertical piles in cohesive soils, evaluated by static formulae, should be obtained for two situations corresponding to undrained or short-term bearing failure and drained or long-term bearing failure.

a. Undrained Bearing Capacity

If the load capable of causing pile bearing failure is applied rapidly compared with the ground drainage capacity, as shown in Subsection 2.2.7, the unitary tip bearing capacity, q_p, both for driven and bores piles will be:

$$q_p = (9 - 3\ D)\ s_u > 6\ s_u \qquad \text{(D in metres)}$$

where:

D = the real or virtual (equal tip area) pile diameter, expressed in metres
s_u = the undrained shear strength of the cohesive soil at tip level (in the area of ± 2D around that level).

In these same conditions, the unitary shaft resistance will be:

$$\tau_f = \frac{100 s_u}{100 + s_u} \qquad \left(\tau_f \text{ and } s_u \text{ in kPa}\right)$$

The undrained shear strength of the soil, s_u, can be deduced from the field and laboratory tests detailed in Part 2 of this ROM 0.5.

For piles with a steel shaft in contact with cohesive soil, the τ_f value should be multiplied by a factor of 0.8.

b. Drained Bearing Capacity

The long-term bearing capacity in cohesive soil should be estimated by using the effective values of the angle of friction and of cohesion deduced from laboratory tests. The expressions given in 3.6.4.7.1 corresponding to granular soils should be used for this purpose and the corresponding cohesion terms added, as indicated below.

- Tip Resistance

  This will be given by the following expression:

  $$q_p = (3\ \sigma'_{vp}\ N_q + 3\ c'\ N_c) \cdot f_D \qquad \text{(driven piles)}$$

  where:

  $N_c = \dfrac{N_q - 1}{\tan\phi}$

  c' = effective cohesion

and where the rest of the parameters have the same meaning as indicated in Subsection 3.6.4.7.1.

For bored piles, q_p will be half the value obtained by the preceding equation, unless specific measures are adopted in its construction to increase tip resistance.

- Shaft Resistance

This will be evaluated using the following expression:

$$\tau_f = \sigma'_v \, K \, f \tan \phi + c'$$

donde:

c' = effective cohesion of the ground.

and where the rest of the parameters have the same meaning as indicated in Subsection 3.6.4.7.1.

3.6.4.7.3 Cohesive Layers Underlying Pile Tips

Sometimes, point-bearing piles are designed and constructed with tips in resistant granular strata resting, in turn, on strata or formations of softer cohesive soils. In these cases, *punching shear* may occur in the resistant stratum, with the piles subsiding into the soft soil underneath.

In some cases, lower-strength zones can also exist under the tip that reduce the tip bearing capacity q_p. If this weak zone is clay with an undrained shear strength of s_u, the point resistance of the piles will be limited by the following value:

$$q_p \leq 9 s_u + \sigma'_v \left(1 + \frac{4H}{D} \tan^3 \phi\right)$$

where:

H = distance from pile tip to the softer cohesive soil stratum underneath.
D = real or equivalent (equal area) pile diameter
s_u = undrained shear strength of the softer cohesive soil.
ϕ = internal angle of friction of the granular material in the tip zone.
σ'_v = natural vertical effective pressure (before installing the pile) at tip level.

Moreover and as explained in Subsection 3.6.9, this situation can lead to a severe limitation of the working load on the grounds of pile settlement.

3.6.4.8 *Ultimate Load Tests*

This procedure provides the most accurate estimates of bearing capacities.

The elements listed below should be taken into account in order to plan a load test.

- A load test must be planned after the general geotechnical investigation of the site is completed and the deep foundations intended for construction have been studied at preliminary design level.

- The site selected as test zone should be representative of the area of interest, where the piles will be installed.

- The preliminary geotechnical investigation of the ground in the test zone should be particularly intensive in order to make an accurate classification of the different soils and/or rocks appearing at depth.

- It is always advisable to test more than one pile in order to obtain some information about to the potential dispersion of the results.

- The procedure for installing the piles to be tested must be similar in every way to the technique intended to be used for constructing the deep foundations. It may be of interest, for the sake of economic saving, to test piles with a slightly smaller diameter than the one being considered in the design.

- It is advisable to provide adequate monitoring for the piles to be tested, so as to be able to discriminate the loads supported by the tip from those by the shaft.

- It is advisable to plan ultimate load tests so that they clearly reach the bearing capacity. This implies settlements in the pile head equal to or greater than 10% of its diameter. Tested piles will never be used to support the project structures.

- Tests should be interpreted by a specialist, as specific details will exist that are not easy to standardise. In the absence of better criteria, the pile bearing capacity should be taken as the value of the maximum load applied to the pile head plus the weight of the pile, provided this load does not exceed three times the one that caused a settlement equalling 1% of the diameter. That is:

$$Q_h = Q\ (10\%) + W < 3\ Q\ (1\%) + W$$

where:

Q_h = bearing capacity to be considered in the calculations.
$Q\ (x\%)$ = load applied at the head producing a settlement equal to x% of the diameter.
W = pile weight.

Load tests are generally costly, as large vertical loads need to be applied, which require major reaction elements (anchors, tension piles, dead weights, etc.). They can however be useful in large-scale projects and it is always worth considering carrying them out.

Special bearing capacity tests are recently being run that can prove more cost-effective and even be performed on piles that will subsequently be used as part of the works. One of these test methods consists in introducing a load cell with the same diameter as the pile near to the tip (or in the actual pile tip itself). Once the pile has been installed, the expansion of the load cell allows the shaft to be loaded upwards and the tip, with an equal load, downwards, thus providing extremely valuable information about bearing capacity.

Furthermore, recent developments in the dynamic analysis of pile driving make it possible to perform dynamic load tests, less violent than the actual driving process. These tests, when correctly interpreted, provide valuable information about the bearing capacity.

The different ways in which load tests can be carried out and interpreted are considered specialist topics outside the scope of this ROM 0.5.

When load tests are carried out to help evaluate bearing capacities, the corresponding safety factors can be reduced. The more extensive the load tests and the greater the similarity of the conditions of the piles tested to real conditions, the larger this reduction can be. With adequate load tests, safety factors as low as those from Subsection 3.6.6 can be applied.

3.6.5 Bearing Capacity from Dynamic Formulae

The bearing capacity of driven piles can be estimated using *dynamic formulae* as described below.

The approach in this case is different from that of the previous static formulae. The aim of static formulae is to find a relationship between bearing capacity and ground and pile data, virtually irrespective of the equipment used for installing the pile. With dynamic formulae, the same load is sought as a function of the pile and the driving equipment data and, to this end, the ground resistance is essentially represented by the driving penetration produced by one hammer blow.

Dynamic formulae do not provide the bearing capacity value, Q_h, but rather the relationship existing between the ground resistance to driving, R_u, and the penetration with each hammer blow, s, once the characteristics of the driving equipment and of the pile are known.

Bearing capacity, Q_h, tends to be similar to the resistance the ground exerts against the pile driving, R_u. However, relaxation and consolidation processes exist that can make the bearing capacity smaller or larger than the resistance to pile driving.

In clayey soils, the resistance to pile driving can be substantial, but transient, and will subsequently relax. The static bearing capacity may be much lower than the ground resistance to driving.

In clayey soils, as time elapses, the ground around the shaft may consolidate and the bearing capacity increase. This will produce the opposite effect of the above case.

In granular soils, a consolidation process can also occur (resistance increasing with time) or a relaxation process (resistance decreasing with time).

For these reasons, the design specifications should require that the penetration per hammer blow s is monitored not only during the main driving, but also some time afterwards, in a subsequent re-driving.

Resistance to pile driving includes a certain viscous component. This is difficult to estimate *a priori* and can only be correctly ascertained by carrying out static bearing capacity tests on piles where the driving has been properly monitored.

The viscous component is small in granular soil and it is generally sufficient to consider its effect indirectly. In cohesive soil, this effect of viscous strength can be greater and require more detailed analysis (even with static load tests).

3.6.5.1 Dynamic Formulae for Pile Driving

Initial attempts to relate the ground's resistance to pile driving, R_u, and the penetration produced by one hammer blow, s, were based on energy considerations. The total energy of the blow is used up in making the pile advance (energy $R_u \cdot s$) plus in any other energy required to deform the pile, the ground and the elements through which the blow is transmitted.

Hiley's formula generally has the most widespread acceptance and is recommended in preliminary studies for selecting driving equipment, pre-dimensioning piles, drivability (the depths that can be reached with different equipment and piles), etc.

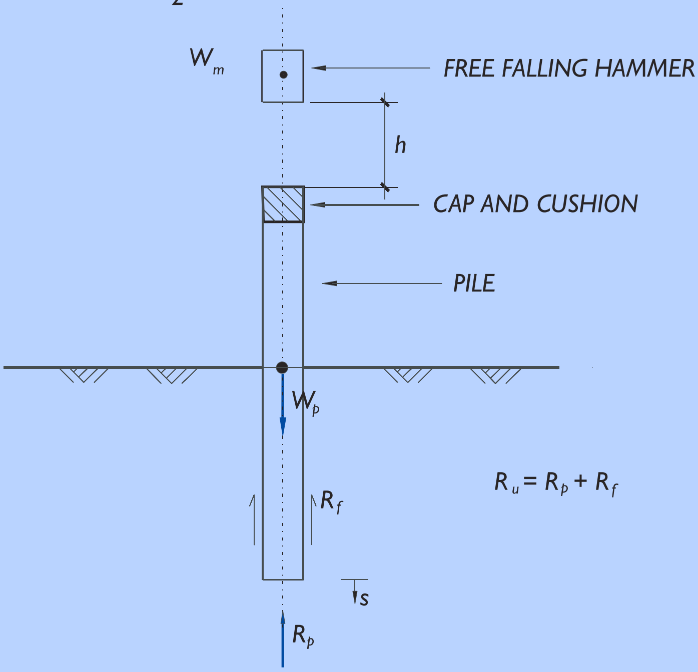
Pile foundations of some importance should be designed with more appropriate methods.

Hiley's formula is shown in Figure 3.6.8 as applied to a driving rig with a free-falling hammer. Its expression is similar in the case of other pile driving equipment with the energy $\eta \cdot W_m \cdot h$ being replaced by the expression corresponding to the net energy of the pile driving equipment in question.

Some concepts introduced into this formula are difficult to quantify and engineers should investigate, in each particular case, before deciding on an adequate value.

Figure 3.6.8. **Pile Driving, Hiley's Formula**

$$R_u = \frac{\eta \, W_m \, h}{s + \frac{1}{2}(C_1 + C_2 + C_3)} \; \frac{W_m + e^2 W_p}{W_m + W_p}$$



η	=	efficiency of driving equipment.
W_m	=	hammer weight.
h	=	fall height.
e	=	coeficient of restitution.
W_p	=	pile weight (including cap and cushion).
C_1, C_2, C_3	=	shortening of the cap and cusshion, of the pile and of the ground (see text).
s	=	pile penetration as a result of one hammer blow.
R_u	=	ground resistance to pile driving.

The efficiency of the driving equipment, η, tends to be supplied by the manufacturer. From this information it can be concluded that performance usually varies between 75% (free-falling hammers with clutch and winch) and 100% (free-falling hammers released by automatic mechanisms). Experience has shown that the performance can be much lower unless special precautions are taken on site.

The coefficient of restitution, e, measures the "elasticity" of the blow. Very soft or worn cushions can lead to null e values. For cushions in good condition, e values of between 0.2 and 0.4 can be taken. When steel strikes steel (metallic piles) without any inserted cushions, it can be assumed that e = 0.55.

Elastic shortening in the blow transmission system (parameter C_1 of the formula) varies considerably (from less than 1 mm to close to 1 cm) and does not only depend on the elements inserted between the pile and the hammer but also on their degree of wear. Wear increases as driving progresses. At the design stage, it is advisable to calculate driving penetration assuming that these elements are not worn.

Consequently, before measuring pile-driving penetration on-site, any worn element should be replaced whenever this measurement is to be used for calculating ground resistance.

The elastic shortening of the pile (parameter C_2 of the formula) can be evaluated using the expression:

$$C_2 = \frac{R_u}{AE} \cdot l$$

wwhere:

A = net area of the pile cross-section.
E = modulus of elasticity.
l = equivalent pile length.

The equivalent pile length can be taken as equal to the total length of the pile (a conservative option) or as somewhat shorter, considering that part of the buried length is subjected to lower compressive stresses as a result of partial transmission of the load along the shaft.

The transient ground compression parameter (C_3 in Hiley's formula) is obtained from practical experience and is normally taken as 0.1" (2.54 mm). Curiously enough, other more advanced calculation methods have confirmed the adequacy of this apparently whimsical calculation assumption.

There are other dynamic formulae, whose use is not advisable, since their only advantage could be simpler analytical expressions.

By using Hiley's formula, it is possible to assume a set of values for R_u, to calculate the corresponding *s* values and draw a curve relating them.

It is common practice to use, instead of *s*, an equivalent variable, which is the blow count required to advance driving by a certain penetration.

In practice, there is a certain tendency to use the term "refusal" for the penetration obtained with a hammer blow at the end of the driving process, so that the penetration *s* is sometimes referred to as *refusal*.

Curves relating the blowcount required for a certain penetration with the driving resistance are known as *pile capacity curves*. In each design, engineers must include the pile capacity curves corresponding to the equipment that can be used in pile driving and also specify the "refusal" (advance per blow or the blowcount for a certain penetration) that must be achieved.

When works are in progress, it should be checked that the pile penetrates the ground during the final stage of pile driving at lower rates than those stipulated. This should be checked, as already mentioned, with the pile driving equipment in the same conditions as used to draw up the curve, working at full performance and with a blow transmitting system in good order; its stiffness should be similar to that used when drawing up the pile capacity curves.

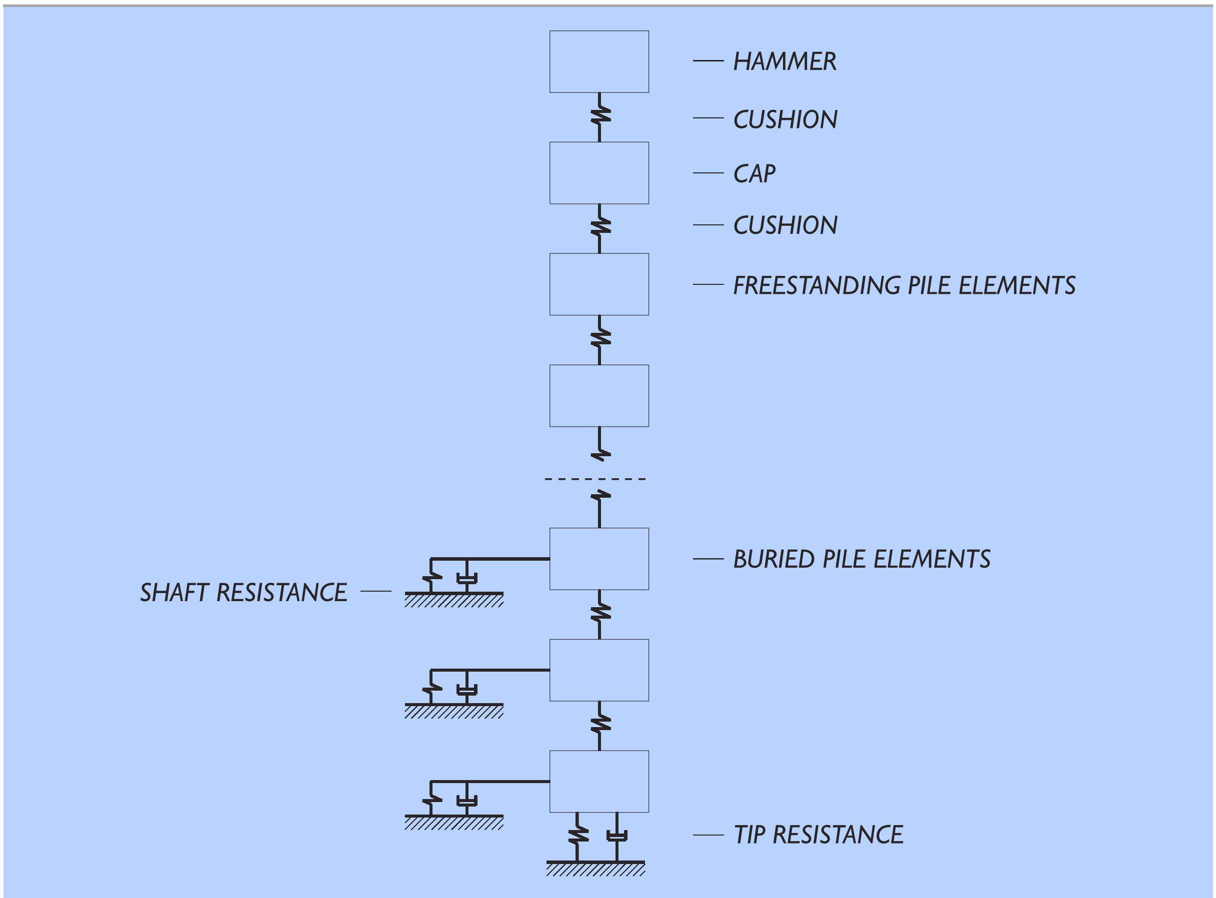
3.6.5.2 The Wave Equation

The first computers made it possible to calculate the relationship between R_u and *s* (which was the objective of dynamic formulae) in what is now believed to be a more accurate manner. At least, some effects not previously taken into account can now be considered.

The method that has come to be known as the *wave equation* is therefore an alternative procedure to dynamic formulae for pile driving. Using this method, *pile capacity curves* can be obtained which can subsequently be used during design and construction in the same way as those from the earlier dynamic formulae.

The pile driving penetration *s* due to one hammer blow is, in this case, calculated with a discrete model, as shown in Figure 3.6.9.

Figure 3.6.9. **Discrete Model of the Wave Equation**



Each pile element is represented by a rigid block, where the whole of the mass is assumed to be lumped, and a spring whose shortening is equivalent to that of the pile length represented. The hammer and the blow transmitting elements are likewise simulated. For these purposes, the experience accumulated in applying Hiley's formula (the value of the C_1 constant) applies to this case.

The ground is represented by coupled springs and dashpots. It is assumed that when they reach a force equal to ΔR_u, the contact is broken and then a constant force is maintained.

The value of ΔR_u must be predicted by distributing the total resistance, R_u (which is an input item), over the different elements. The result will depend (without changing radically) on the way in which the resistances are distributed. Engineers must make an adequate distribution, similar to what is expected in reality.

The constants of the springs representing the ground are:

$$K_{(ground)} = \frac{\Delta R_u}{C_3}$$

where ΔR_u has the same meaning given above and C_3 is the shortening of each spring at the moment of failure. Experience with the use of Hiley's formula has led to assume in most applications that C_3 = 0.1" (2.54 mm).

The viscous dampers provide an additional resistance, which is taken to be proportional to the elastic strength:

$$(\Delta\ R_u)_{viscous} = \Delta\ R_u \cdot J_s \cdot V$$

where:

J_s = viscous resistance coefficient (in s/m).
V = pile movement velocity (in m/s).

Experience has shown that the typical values for the viscous resistance coefficient will be between 0.15 and 0.20 s/m for sands and between 0.50 and 0.70 s/m for clays. In the pile tip zone, it appears that the viscous effect is greater in sands (J_s of up to 0.50 s/m in sands) and can be lower in clays (even less than 0.1 s/m).

The effect of this latter factor can prove decisive in some cases and engineers must therefore examine it in detail and carry out the appropriate sensitivity analyses.

The response of this discrete system to a hammer blow can easily be obtained by a finite difference integration with a simple computer program (some academic programs do this in less than 50 sentences).

The result will be the penetration, *s*, caused by one hammer blow for the configuration defined when distributing the total assumed resistance, R_u.

The computation will also provide stresses at each point of the pile during the hammer blow, which could be of value in the structural design.

The computation can be further complicated by assuming initial conditions different from the null-stress state - the new conditions would therefore be the residual stresses from earlier blows. It can also be completed by simulating joints (common in long piles) or by introducing more complex constitutive laws.

The same recommendations given for other dynamic pile-driving formulae apply to subsequent use of the pile capacity curves generated with the wave equation.

3.6.5.3 Monitoring Pile Driving

The bearing capacity of a driven pile can be estimated at the design stage either by the wave equation or by more simplified pile-driving formulae.

In these cases, it is essential that the driving is monitored, since the pile bearing capacity is not linked to a particular depth but to other design specifications (driving equipment and refusal criteria).

Pile driving must be monitored in accordance with the procedure used for estimating bearing capacity at the design stage. It may consist of only a check on the pile-driving equipment characteristics and the monitoring of penetration.

It is now common practice in important works to routinely monitor the driving of each pile and to instrument one or all of the piles driven, with a view to obtaining supplementary field data confirming the design assumptions.

Additional instrumentation consists of placing two piezoelectric accelerometers close to the head of the pile (whose readings will be averaged) along with two strain gauges (or extensometers) to measure the shortening of two segments of opposite pile generatrices, also in the area close to the head (also to be averaged).

Other equipment can provide a record of velocity (by integrating acceleration over time) and of force. In order to obtain the second record, the net pile cross-section area, *A*, and its modulus of elasticity, *E*, must be previously known and supplied as input data for the computer (or field microprocessor). The field measurement consists of the unitary vertical shortening, ε, so that the product of the three data leads to the value of the compressive force on the instrumented section:

$$F = A \cdot E \cdot \varepsilon$$

The most elementary theory of dynamics (linear elasticity) shows that the unitary shortening at any point, as a result of a travelling impact wave, is proportional to the pile movement velocity, *V*, in other words:

$$\varepsilon = \pm \frac{V}{c}$$

$$\text{where: } c = \sqrt{\frac{E}{\rho}}$$

and ρ is the density of the pile material.

The positive sign occurs when the impact wave and velocity (positive only downwards) are in the same direction (direct compression wave produced initially by the pile-driving). The negative sign applies when the two directions are opposite (upward compression wave produced as a reflection of the ground response during pile driving).

The wave propagation speed in the pile, *c*, which is a characteristic of the pile material, appears as a proportionality constant in this equation.

The proportionality between shortenings and velocities enables a similar equation to be established between forces and velocities:

$$F = \pm \frac{AE}{c} \cdot V$$

The *AE/c* quotient is known as the *pile impedance* in these dynamic pile-driving analyses.

$$\text{Impedance, } Z = \frac{AE}{c}$$

It also holds that:

$$Z = A \cdot \rho \cdot c$$

Hence, the simplest way to express the differential equation governing the piledriving process is:

$$F = \pm ZV$$

For the dynamic monitoring of pile driving, the value of the pile impedance is normally entered as a supplementary item so that the processor will always multiply the velocity records by this value.

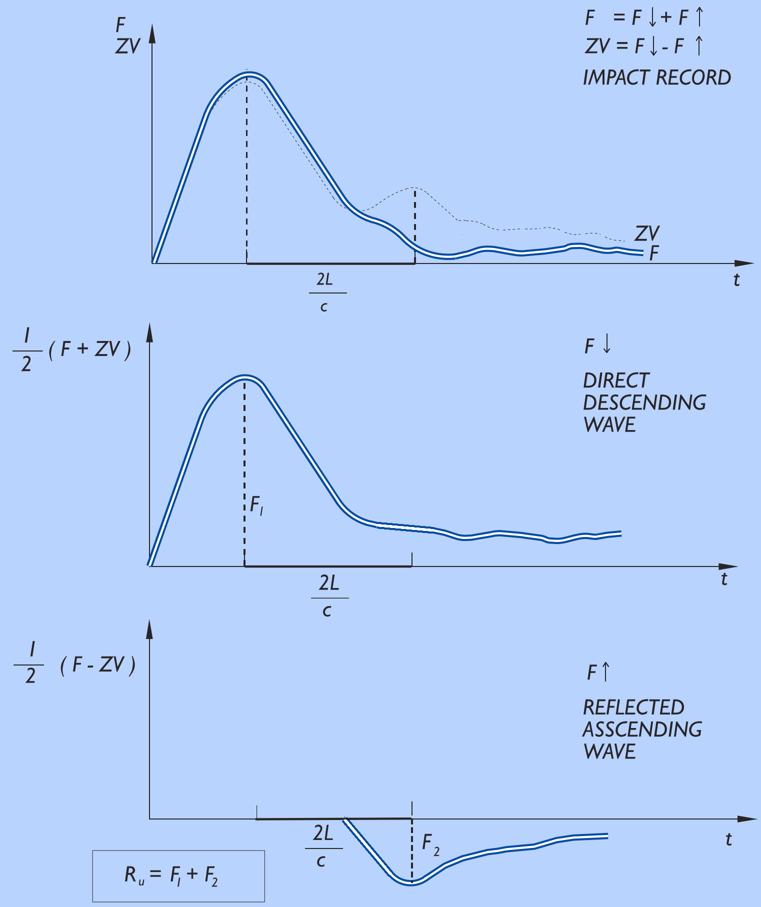
With this procedure, the result of monitoring pile driving will be temporal records of forces, *F*, and the products of impedance and velocity, *ZV*, in a section close to the pile head. It is possible to obtain a record for further sections but this is not advisable, since the additional data will not always be relevant and would be more difficult to obtain, as it would have to be done in the buried part. Several sections from the freestanding area of the pile would not provide any significant additional information.

These logs serve many useful purposes. Conclusions can be drawn from them as to driving resistance, pile integrity and, more importantly, the mechanical efficiency of the pile-driving equipment. Other aspects can also be investigated by detailed analysis of the records.

a. Penetration Resistance

Figure 3.6.10 shows a typical log of *F* and *ZV* caused by a hammer blow on the head of a pile, whose driving is monitored as explained in the previous paragraphs. The values for *F* and *ZV* are identical while the dynamic effect is solely due to the descending wave.

Figure 3.6.10. Penetration Resistance of a Pile Obtained from Dynamic Monitoring of the Driving



Some time later, when there are reflections from the ground, both *F* and *ZV* are the result of both downward and upward waves, whose effects must be differentiated, as shown in the figure. In this manner, the ground response can be isolated. It shows a peak that is displaced with respect to the impact peak by a time of *2L/c*, where *L* is the pile length, measured from the instrumentation level to the pile tip.

The simplest interpretation consists of assuming that the ground resistance to driving has been the sum of the two forces, the one of the direct wave F_1 and its reflection F_2. The figure illustrates a situation where the reflected wave produces a tensile force, with the result that the driving resistance, R_u, is less than F_1. This is the usual case at the start, when driving is far from making the most of the pile, the driving equipment and the resistant capacity of the ground.

In theory, the maximum resistance that can be mobilised by one hammer blow is equal to $2\ F_1$. When this occurs, a compression wave is reflected that has the same amplitude as the one initially originated by the hammer in the head area.

The maximum value of F_1 is limited by the pile-driving equipment and the structural strength of the pile itself. It has a maximum value of ZV_0, where V_0 is the velocity of the falling hammer. This maximum is only reached with sufficiently heavy hammers and adequate blow transmitting elements.

The maximum velocity usually induced in piles is limited by potential structural damage to them and rarely exceeds 1.5 m/s. In this way, driven piles normally have a bearing capacity in the order of 3 m/s multiplied by their impedance, or less.

It is possible to include the effect of the viscous component of the driving resistance in this formulation. The pile penetration velocity, at the time when R_u is calculated, is such that:

$$ZV = F_1 - F_2$$

and therefore it can be thought that there is a viscous component proportional to the velocity:

$$(R_u)_{viscous} = J \cdot ZV$$

In this case, J will be an dimensionless coefficient that should not be confused with the value J_s normally used in the wave equation, where it has dimensions and a different meaning.

Taking this into account, the ground's resistance to pile driving, after subtracting the viscous component, would be:

$$R_u^* = F_1\,(1 - J) + F_2\,(1 + J)$$

Accumulated experience is not yet sufficient to know usual values of *J* for all types of ground. To date, either zero values or values of around 0.1 are taken as a maximum in granular soil and higher values, even close to 1, in cohesive soil.

Engineers can investigate the effect of this viscous component by driving the same pile with hammer drop heights slightly larger or smaller than the ones specified. The driving resistance value will vary but its non-viscous component should remain constant.

b. Pile Integrity

The dynamic monitoring of pile driving or monitored re-driving of piles of any type will provide *F* and *ZV* logs similar to the ones shown in the first diagram in Figure 3.6.10.

If there are structural defects (i.e., a reduction of the pile impedance, Z), a prematurely reflected wave will be recorded, whose analysis will enable the damage to be located and quantified.

Although interpretation in such cases is simple, it is not possible to include the necessary recommendations here. Engineers who have detected defects of this type must consult technical references or an expert in order to investigate the matter further.

c. Performance of the Pile-driving Equipment

The biggest recent contribution to the study of pile-driving dynamics, achieved through electronic instrumentation, has probably been the capacity to measure the energy passing through a particular pile cross-section during a hammer blow.

It is clear that the useful energy for driving is equal to the following integral:

$$\text{Energy} = \int_{t_0}^{t_1} F \cdot V \cdot dt$$

This integral, taken on a pile section between the beginning and end of the shock wave passage, is relatively easy to calculate with field microprocessors. The results are known before another hammer blow takes place.

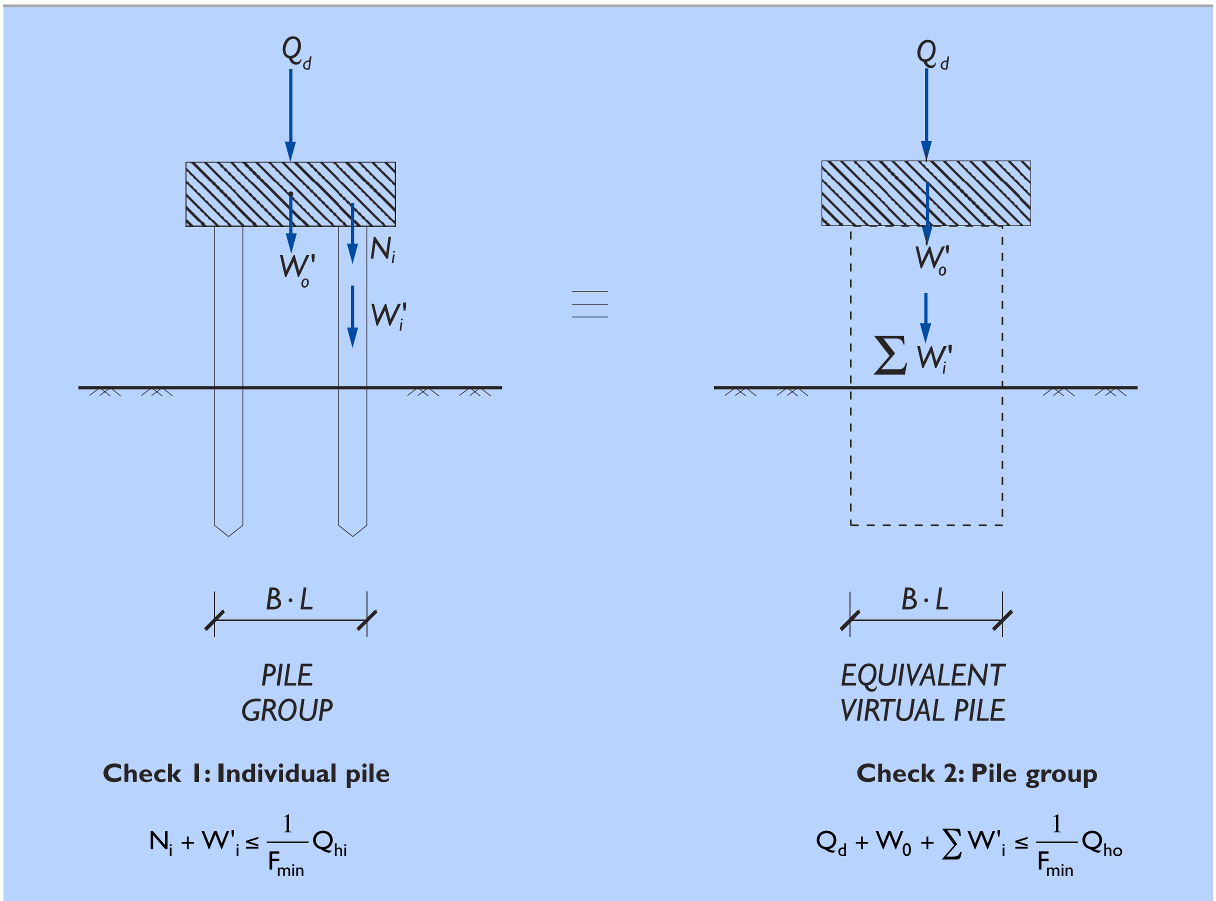
Experience has shown that this energy, compared with the nominal energy of the hammer, corresponds to efficiencies that can be very poor. Mechanical efficiency is highly sensitive to details such as the condition of the pads or cushions transmitting the blow, the correct hammer-pile alignment, etc.

This is the main reason for recommending electronic monitoring of pile driving in major works. This will reduce the uncertainty that there would otherwise be about the performance of the driving equipment, so that lower safety factors could be used as shown in 3.6.6.

3.6.6 Evaluating Safety against Bearing Failure

In order to verify safety against bearing failure of deep foundations, it should be checked that the safety factor for bearing failure is high enough for each individual pile in a group and also for the group as a whole, considered as an equivalent virtual pile (see Fig. 3.6.11).

Figure 3.6.11. ***Verifying Safety against Bearing Failure***



3.6.6.1 Bearing Failure in Individual Piles

The safety of a pile is verified when the following expression holds:

$$F = \frac{Q_{hi}}{N_i + W'_i} \geq F_{min}$$

where:

F = safety factor against bearing failure
N_i = vertical load acting on the pile. To obtain this, the load acting on the set of piles should have been distributed over each individual pile, taking into account the weight of the eventual cap and, if applicable, that of the ground that could gravitate onto it. The loads should have been factored as shown in Table 3.3.2.
W'_i = effective weight of the freestanding portion of the pile (out of the ground) that could possibly exist.
Q_{hi} = bearing capacity of the isolated pile.
$F_{mín}$ = the safety factors shown in Table 3.6.1.

Table 3.6.1. *Minimum Safety Factors, F, against Bearing Failure in Piles. Works with a Low SERI (5 - 19)*

Analysis Procedure Used in Estimating Bearing Capacity	Safety Factor (For each Combination)		
	Quasi-Permanent, F_1	Fundamental, F_2	Accidental or Seismic, F_3
Any Type of Piling (1)			
SPT method in granular soils	2.5	2.2	2.0
Method based on static penetrometer tests	2.0	1.8	1.7
Methods based on other continuous penetrometer, pressuremeter tests and other field tests	2.6	2.3	2.1
Method based on the unconfined compressive strength of the rock (only for piles embedded in rock)	2.5	2.2	2.0
Method based on analytical formulae and laboratory tests to measure the friction angle (or on laboratory or field tests to measure the undrained shear strength of clay)	2.5	2.2	2.0
Pilotes hincados			
a) With penetration monitoring and applying Hiley's formula (2)	2.5	2.2	2.0
b) With penetration monitoring and applying the wave equation (2)	2	1.8	1.7
c) With electronically monitored driving	1.6	1.5	1.4
d) With electronically monitored driving and confirmed with load tests	1.4	1.3	1.3

(1) When the calculation is contrasted with *in situ* bearing capacity tests, the safety factors shown may be reduced by between 10% and 15%, depending on the number of piles tested and the dispersion of results obtained in the tests (see Subsection 3.6.4.8).
(2) The factors indicated correspond to soft or moderately hard pile driving, that is, for driving operations in which the final refusal (penetration with a hammer blow) is greater than 2 mm (two millimetres). For harder pile driving, larger safety factors should be used, or alternatively, the load should be limited in the driving curves so that it does not increase for lower penetrations than the one indicated (2 mm).

The bearing capacities, Q_h, can be estimated using the procedures shown in previous sections, depending on the type of ground and pile involved and the type of information available.

3.6.6.2 Ultimate Bearing Capacity of Pile Groups

In a group of closely spaced piles, it is possible that the bearing capacity of a pile equivalent to the group is less than the sum of the bearing capacities of each individual pile, hence the need to check both values.

The virtual pile equivalent to the group is defined by a length equal to the average length of the different piles in the group, by a cross-section area equal to the internal area of the simple geometrical line (circle, quadrilateral, etc.) capable of circumscribing the group on a plan view. The length of this line should be adopted as the equivalent pile contour.

The buried weight of the equivalent pile, which is important in these calculations, is the sum of the buried portion of the weights of all the individual piles plus the ground within the equivalent contour line and taken from the top ground level to the average level of the tips. These weights should be taken as submerged in the zones where they are in fact under water. This weight should be used to estimate the adequate bearing capacity, Q_{ho}, by subtracting it from the sum of the tip and shaft resistances (see Subsection 3.6.4.1).

The safety against bearing failure of a set of piles is verified when the following inequality holds:

$$F = \frac{Q_{ho}}{Q_d + W'_o + \sum W'_i} \geq F_{min}$$

where:

F = safety factor against bearing failure.
Q_d = vertical component of the loads, properly factored, acting on the cap's top face.
W'_o = effective cap weight, submerged where appropriate.
$\sum W'_i$ = sum of the effective weights of the freestanding portion (above the ground) of all the piles in the group.
Q_{ho} = bearing capacity of the virtual pile equivalent to the group, calculated as indicated in this Section 3.6
F_{min} = minimum safety factor shown in Table 3.6.1.

3.6.6.3 *Minimum Safety Factors against Bearing Failure*

In the most general cases of works with a low SERI (5 - 19), the minimum safety factor values recommended are those shown in Table 3.6.1 depending on the different methods utilised in assessing bearing capacity.

For transient or short-term situations, the indications given in 3.3.8.1 should be taken into account.

For works with a minor or high SERI or for other allowable failure probabilities, the F values can be adjusted as indicated in Subsections 3.3.8.2 and 3.3.10.

3.6.7 Verifying Safety against Uplift

3.6.7.1 *Individual Piles*

The uplift resistance of an individual pile is equal to its shaft resistance plus the component corresponding to the self-weight of the pile. Experience has shown that the shaft pull-out resistance is lower than the shaft resistance in compression.

The most accurate way to determine the uplift resistance of a pile is to carry out load tests.

In the absence of tests or experience to indicate a more precise value, the following should be taken as the uplift or extraction resistance, T, of a pile:

$$T = W_\alpha + 1/2\ R_f$$

where:

W_α = component of the pile weight in the direction of pull.
R_f = shaft resistance obtained as indicated in 3.6.4.

It is assumed that the pull or extraction force is applied along the axis of the pile.

3.6.7.2 *Considering the Group Effect*

The uplift resistance of a group of piles will be the lower of the following two values:

a. The sum of the individual uplift resistance values for each of the piles in the group.

b. The resistance of a virtual pile formed by the piles and the ground around them where the shaft is formed by the external envelope of the pile group.

3.6.7.3 *Safety against Uplift*

Piles working under traction are not a common situation in civil engineering works. Anyway, particular circumstances may arise in which some of the following situations occur.

a. A pile is individually subjected to tensile forces (negative N_i value).

In these cases, safety is verified when the following expression is true:

$$F = \frac{T}{|N_i|} \geq F_{min}$$

where:

F = safety factor against uplift.
T = uplift resistance (Subsection 3.6.7.1).
$|N_i|$ = absolute value of the factored load, as indicated in 3.6.3.3, acting on the pile head, assumed to be at ground level.
F_{min} = the safety factor shown in Table 3.6.1 for works with low SERI ratings (5 - 19). For works with minor or high SERI ratings, or for other allowable failure probabilities, the minimum F values given in Table 3.6.1 can be adjusted as indicated in Subsections 3.3.8.2 and 3.3.10. They can equally be adapted for transient situations (including short-term geotechnical situations) as per the provisions in Subsection 3.3.8.1.

b. The group of piles, or several of them, are working under traction.

In this circumstance, in addition to the above individual check, the check for to a virtual pile equivalent to the group of piles under tension should be done according to the following expression:

$$F = \frac{T}{|Q_d|} \geq F_{min}$$

where:

$|Q_d|$ = absolute value of the factored tensile load acting on the group, at pile head level, including the effective weight of the cap and of the freestanding portion of the piles
T = uplift resistance of the group of piles under tension obtained according to the provisions in Subsection 3.6.7.2

F_{min} = the safety factor given in Table 3.6.1 for works with a low SERI rating (5 - 19). For works with a minor or high SERI rating, or for other allowable failure probabilities, the minimum F values laid down in Table 3.6.1 can be adjusted as indicated in Subsections 3.3.8.2 and 3.3.10. They can equally be adapted for transient situations (including short-term geotechnical situations) as per the provisions in Subsection 3.3.8.1.

3.6.8 Verifying Safety against Ground Failure Owing to Horizontal Pull or Pressure

3.6.8.1 Individual Piles

Pile groups subjected to horizontal loads can be specially arranged so that these loads are mainly withstood by compression on the heads of some battered piles.

In any event, many situations will exist where the individual piles have to withstand loads perpendicular to their axis – i.e., horizontal loads for vertical piles.

When an excessive transverse force acts on the head of a pile, considerable bending normally results, which is why failure happens in the pile itself. The corresponding Limit State is normally governed by the pile structural capacity. This type of failure is analysed in Subsection 3.6.10. In regular practice, this situation is known as "long pile".

However, if the pile can withstand this bending, the ultimate load depends on the resistance of the ground. This tends to happen in piles that are only slightly embedded or in piles with a large structural capacity in relation to the ground's lateral resistance against the pressures transmitted by the pile. In regular practice, this situation is known as "short pile". The corresponding Ultimate Limit State is geotechnical in nature (GEO) and the recommended analysis procedure is described in this subsection.

The horizontal load applied to the pile head, H_{fail}, that will provoke the passive failure of the ground under the pile pressure can be estimated with the calculation diagram shown in Figure 3.6.12. The *x* value –necessary for calculating H_{fail}– should be obtained using the moment equilibrium equation

$$(e + z_{CR}) \cdot CR = (e + z_R) \cdot R$$

The height at which the force *H* is applied (distance *e* in the figure), is an important variable in these calculations and is sometimes not well known. The point where the load *H* is applied is one which has zero bending moment on the pile axis, which engineers must decide as a function of other structural calculations.

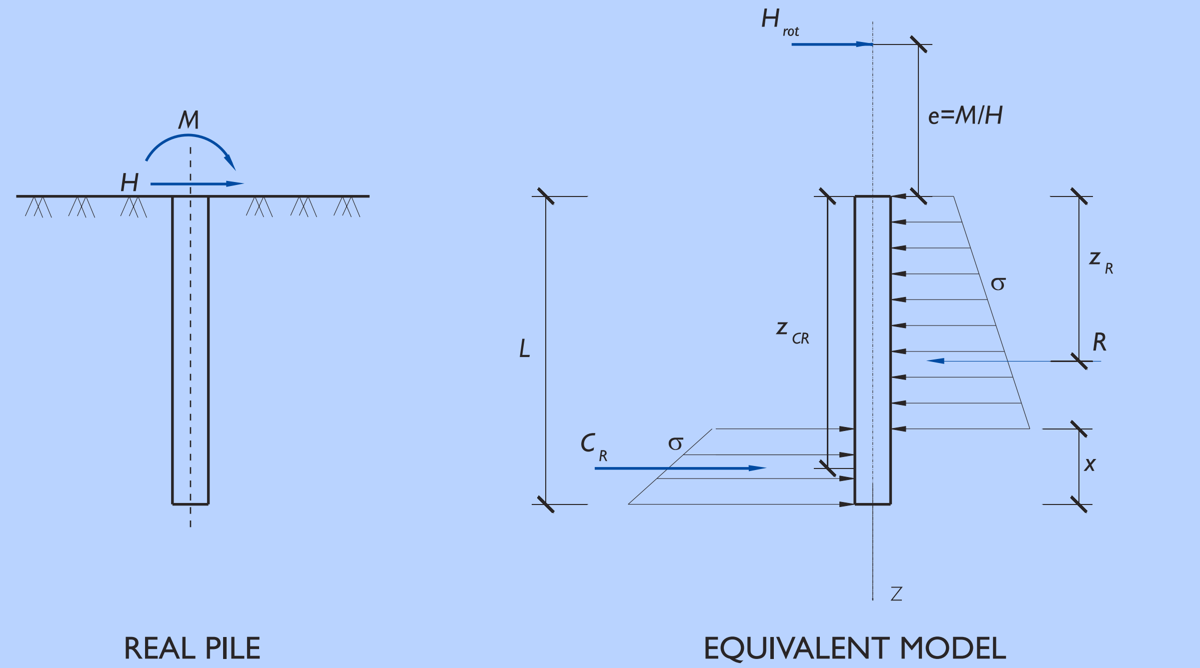
The particular cases of c = 0 (purely frictional ground) and ϕ = 0 (purely cohesive ground) have been solved and tabulated as illustrated in Figures 3.6.13 and 3.6.14 taken from Broms [(13)] (1964).

A singular situation is shown in Figures 3.6.12 and 3.6.13 of a particular case of horizontal rigid displacement. This would correspond to a hypothetical situation in which an external cause (common rigid cap, deeper pull, etc.) makes the lines of action of the lateral force and ground resistance to coincide. This situation involves a simpler problem, as no counter-reaction is produced and the moment equilibrium is satisfied by the assumption of horizontal rigid pile displacement.

It is worth mentioning that these figures make the additional assumption of not relying on the resistant contribution from the surface ground, down to a depth equal to one and a half times the pile diameter. Only the contribution from its weight is taken into account. Engineers will have to make a similar assumption to take any potential scour, erosion or excavations into account, as they have such a considerable effect on horizontal resistance.

(13) Broms, 1964. "Lateral Resistance of Piles in Cohesive Soils" and "Lateral Resistance of Piles in Cohesionless Soils". Journal of Soil Mechanics and Foundation Engineering. ASCE.

Figure 3.6.12. **Diagram for Calculating the Ultimate Horizontal Force**



Ultimate Load $H_{rfail} = R - CR$

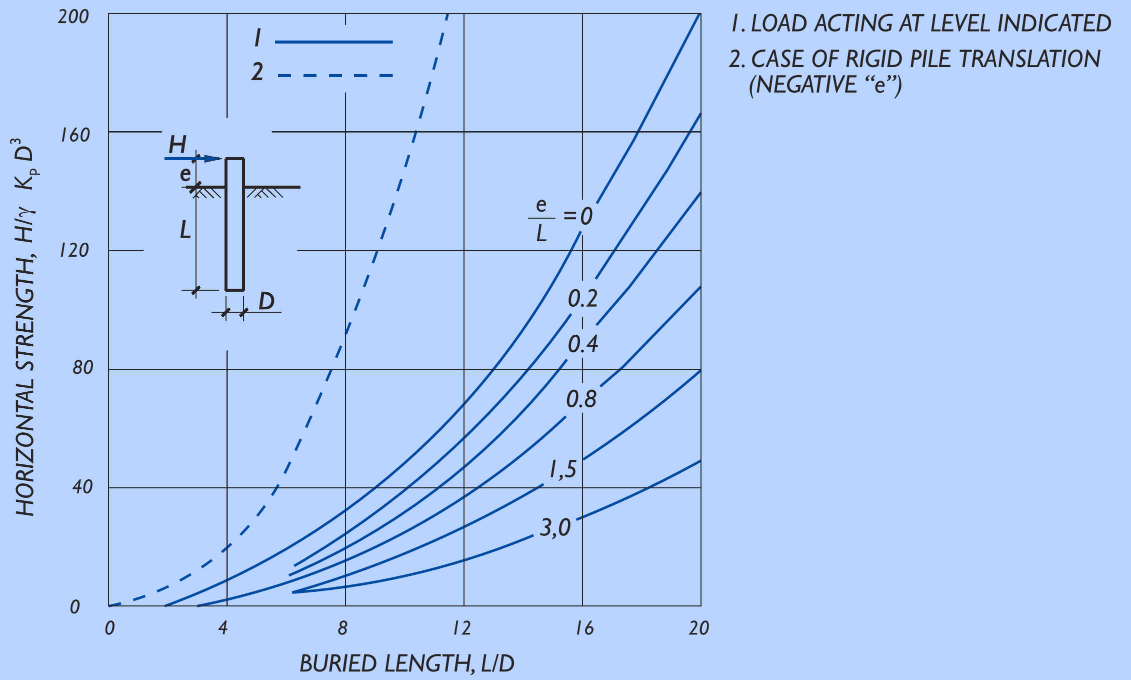
Reaction $R = \int_0^{L-x} \sigma \; D \; dz$

Counter - Reaction $CR = \int_{L-x}^{L} \sigma \; D \; dz$

Ultimate Pressure $\sigma = 9c + 3\sigma'_v \tan^2(45 + \phi / 2)$

σ'_v = vertical effective pressure at depth z

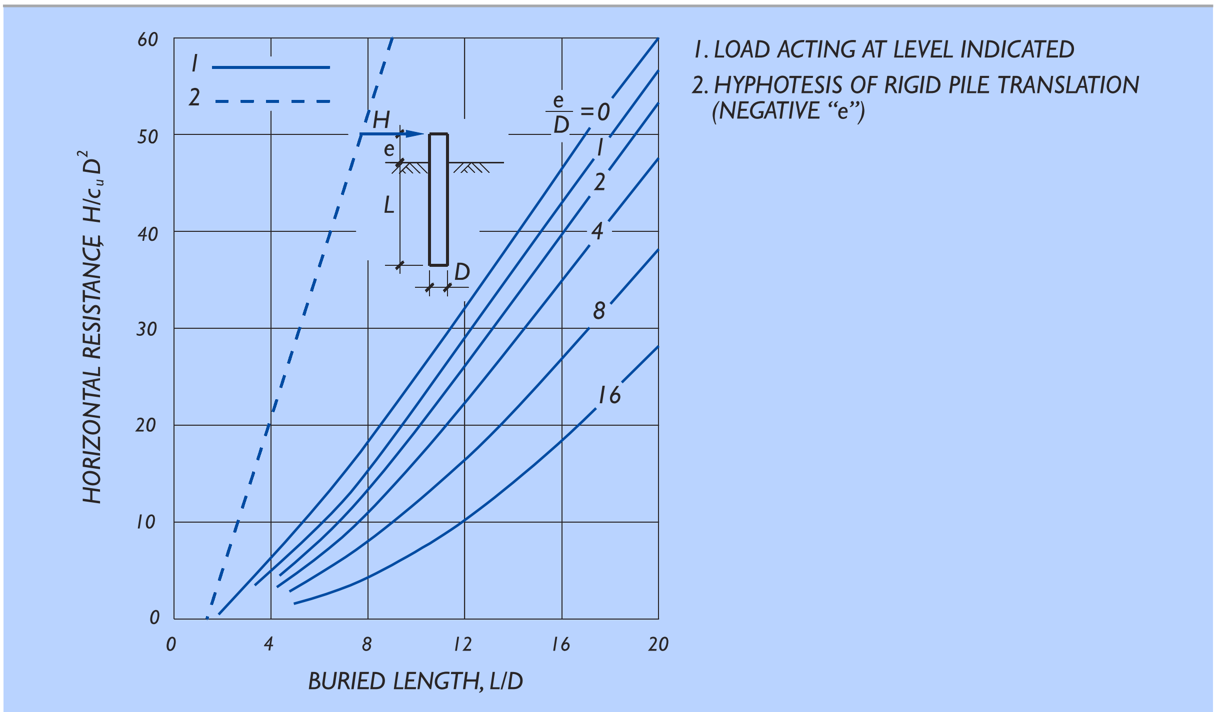
Figure 3.6.13. **Horizontal Ground Failure, Granular Soils (Broms)**



In addition, in the Broms graph corresponding to purely cohesive ground (ϕ = 0), the unitary ultimate earth pressure –in failure– is assumed to be equal to $8\ c_u$ (only for the case of horizontal rigid displacement) instead of the value, believed to be more adequate, of $9\ c_u$ that would result from the general calculation procedure shown in Figure 3.6.12.

It should be pointed out that the graph in Figure 3.6.13 defines the ultimate load for different e values, which are expressed in a dimensionless form by dividing them by the pile length (parameter e/L). Figure 3.6.14 however uses the pile diameter to make the eccentricity dimensionless (parameter e/D). This was considered advisable for the sake of greater clarity in the drawing of the corresponding curves.

Figure 3.6.14. Horizontal Ground Failure, Purely Cohesive Soils (Broms)



The H_{fail} value may be lower when the applied horizontal force is oscillating. Load alternation is capable of causing a situation in which the upper part of the ground does not collaborate in its horizontal resistance. This effect needs to be analysed by a procedure whose description lies beyond the scope of this ROM 0.5.

3.6.8.2 Consideration of the Group Effect

The resistance against ground failure of a pile group under horizontal loads should be estimated as the lesser of the following two values:

a. The sum of the horizontal resistance of each pile, calculated individually.

b. The horizontal resistance of an equivalent pile with a diameter equal to the width of the group and a depth equal to the average depth of the piles in the group.

In cases where this design aspect is critical, more detailed calculation procedures should be used.

3.6.8.3 Safety Factor against Horizontal Ground Failure

Each safety verification can be taken as satisfied when:

$$F = \frac{H_{fail}}{H} \geq F_{min}$$

The safety factor *F* should be calculated for individual piles (Subsection 3.6.8.1) and for the pile group (3.6.8.2).

The safety factors F_{min} that should be used for this ground failure mode are shown in Table 3.6.2 for works with a low social and environmental repercussion rating (SERI).

Table 3.6.2. Minimum Safety Factors Relative to Horizontal Ground Failure, Low SERI Works (5 - 19)

Load Combination	Safety Factors, F
Quasi-Permanent, F_1	1.8
Fundamental, F_2	1.6
Accidental or Seismic, F_3	1.5

For works with minor or high SERI ratings or for other allowable failure probabilities due to economic optimisation criteria, the minimum F values given in Table 3.6.2 can be adjusted as indicated in Subsections 3.3.8.2 and 3.3.10. They can also be adapted for transient situations (including short-term geotechnical situations) in accordance with the provisions in Subsection 3.3.8.1.

In projects where this aspect is critical for dimensioning the pile foundations, load tests for estimating the ultimate load more accurately are recommended.

If load tests are used to estimate the horizontal ultimate load, the minimum safety factors can be reduced as a function of the importance of the tests.

Horizontal cyclic loads can increase deformations or even create permanent gaps in cohesive soils. This effect should also be taken into account when considering deformational aspects (Subsection 3.6.9).

3.6.9 Verifying Limit States of Serviceability

3.6.9.1 Settlement

If the problem of settlement in piled foundations is a critical aspect of the project, load tests are recommended, as this is the only accurate way of ascertaining the loadsettlement relationship.

In cases where this aspect is not critical, settlement can be estimated as described below, making sure that the settlement estimated this way is at least three times less than the one that causing the structure to be unserviceable.

Analytical or semi-analytical solutions are available and numerical models can be formulated to solve the problem of calculating settlement in isolated piles or pile groups. The problem however depends so much on the local conditions in the contact zone between pile and ground that these methods can be inaccurate.

In the absence of specific load tests, the following procedure to estimate settlement is recommended.

3.6.9.1.1 Settlement of Isolated Piles

When an isolated vertical pile is subjected to a vertical service load on its head equal to the maximum recommended for bearing failure reasons –estimated as described in Subsection 3.6.4–, it undergoes a typical settlement close to 2.5% of its diameter plus the elastic shortening of the pile itself. On this basis, the load-settlement relationship can be expressed by the following approximate formula:

$$s_i = \left(\frac{D}{40\ Q_h} + \frac{l_1 + \alpha\, l_2}{AE} \right) \cdot P$$

where:

s_i = settlement of the individual isolated pile
D = pile diameter (for non-circular shapes, an equivalent diameter should be obtained)
P = load acting on the pile head
Q_h = bearing capacity
l_1 = length of pile out of the ground
l_2 = length of pile in the ground
A = cross-sectional area of the pile
E = modulus of elasticity of the pile
α = a variable parameter depending on the type of load transmission to the ground, $\alpha = 1$ for mainly end-bearing piles and $\alpha = 0.5$ for floating piles. For situations in-between, the value of α will also be between the two values given, interpolating linearly as a function of the estimated tip and shaft loads.

3.6.9.1.2 Considering the Group Effect

In pile groups and owing to load interference, group settlement can be greater than the settlement of an isolated pile subjected to the corresponding individual load. The following estimation can be made to take this into account:

a. For end-bearing column piles in rock, spaced more than three diameters apart, the group effect can be considered negligible.

b. In other situations, it can be assumed that the whole of the group load is evenly distributed on a plane situated at depth z below the surface of the ground

$$z = \alpha \cdot l_2$$

with α and l_2 having the meaning given in Subsection 3.6.9.1.1 and with transverse dimensions, $B_1 \times L_1$, given by:

$$B_1 = B_{group} + (1 - \alpha)\, l_2$$
$$L_1 = L_{group} + (1 - \alpha)\, l_2$$

The settlement at each depth owing to this distributed vertical load can be estimated using general settlement calculation procedures, a few of which are described in Subsection 3.5.7.

3.6.9.2 Horizontal Movements and Structural Analysis

If the horizontal movement of pile foundations is a critical aspect of the problem under study, field tests will be necessary to estimate it.

When this aspect is not critical, an approximate calculation can be carried out as shown in the following subsections.

The horizontal deflections capable of rendering the structure resting on the piles unserviceable should be at least three times the values estimated by the simplified manner recommended below. Otherwise, it will be necessary to run load tests or to modify the design to withstand greater movements.

A numerical procedure should be used to compute horizontal movements in piles under lateral loads. Computational methods are available in which a pile is represented by an elastic beam and in which the ground is introduced as a series of springs (with p-y relationships between pressure and displacement) deduced from previous experiences in different ground types.

Unitary load-displacement relationships can be non-linear to simulate the behaviour of the ground close to failure. In this case, this procedure also makes it possible to know the value of the ultimate horizontal load. This would therefore constitute an alternative to the method shown in 3.6.8 for calculating this load.

3.6.9.2.1 Simplified Method

The numerical model referred to in 3.6.9.2 can be complex when trying to analyse several piles or several pile foundations at the same time. The number of springs required to represent the ground could be excessive.

A simplified method can be used for representing the ground by a single equivalent spring and is described in this subsection. It is considered sufficiently accurate for situations where the deformational aspect is not a critical or determining factor.

To apply this method, the type of ground involved should be known, discriminating granular from cohesive ground.

For granular ground, the *elastic length, T,* should be calculated using the following expression:

$$T = \left(\frac{EI}{n_h}\right)^{1/5}$$

where:

E = elasticity modulus of the pile material.
I = moment of inertia of the pile cross-section about an axis orthogonal to the direction of the load. $I = 1/64\ \pi\ D^4$ in solid circular piles with a diameter *D*.
n_h = ground parameter shown in Table 3.6.3.

In cohesive soils, it can be assumed that:

$$T = \left(\frac{EI}{100c}\right)^{1/4}$$

where variables *E* and *I* have the above meaning and:

c = the average value of the shear strength of the soil in the area of depth 3D, starting from the ground surface considered in the calculation.

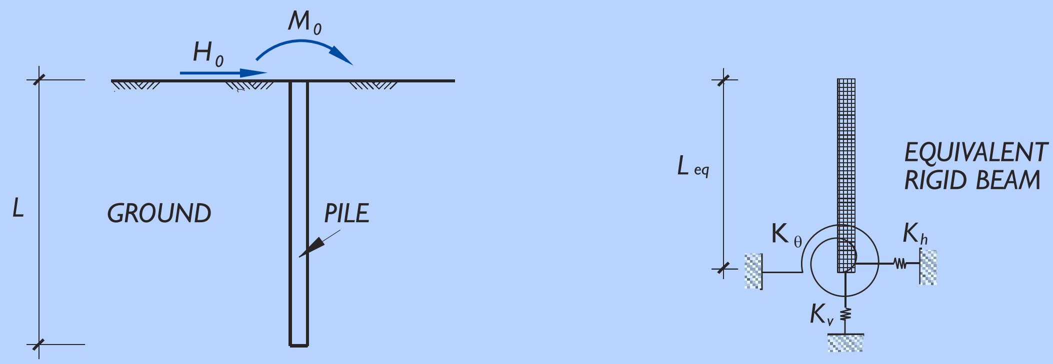
It is advisable to disregard the contribution from loose top soils in these calculations and to take into account, in addition, that the pile may not be in permanent contact with the ground in a topmost zone. This zone may be more extensive when the horizontal load is cyclic and alternating.

Table 3.6.3. Values for Parameter n_h (MPa/m)

Sand Compacity	Position in Respect of the Groundwater Table	
	Above	Below
Loose	2	1.2
Medium	5	3
Compact	10	6
Dense	20	12

Knowing the value of *T*, the ground and the buried portion of the pile can be represented by a rigid beam (undeformable) supported at its tip by springs with an elastic constant as given in Figure 3.6.15.

Figure 3.6.15. Simplified Modelling of the Buried Portion of a Pile



VALUES OF THE EQUIVALENT PILE PARAMETERS

$$L_{eq} = (1{,}10 - 0{,}15 \ln \frac{L}{T}) \cdot T \geq 0{,}8T$$

$$K_h = (0{,}68 + 0{,}20 \ln \frac{L}{T}) \cdot \frac{EI}{T^3} \leq \frac{EI}{T^3}$$

$$K_\theta = (0{,}3 + 0{,}20 \ln \frac{L}{T}) \cdot \frac{EI}{T} \leq 0{,}6 \frac{EI}{T}$$

N.B.: The meaning of the variables is indicated in the text. See Subsection 3.6.10.1 for the K_v value.

To obtain the stress resultant diagrams in the buried portions of piles, the specialised texts giving semi-analytical solutions from the elastic beam theory can be consulted. The following simplified solution is accepted in this ROM 0.5:

Bending moment	$M = H_o \cdot z_o \cdot \alpha + M_o \, \beta$
Shear force	$Q = H_o \, \beta$

where α and β are dimensionless numbers that vary with depth *z* according to the relationship given in Table 3.6.4.

The value z_o is a reference depth depending on the length of the pile inside the ground, *L*, and its elastic length, *T*. It can be estimated by the equation:

$$z_o = T \cdot \left(0.25 + 0.8 \ln \frac{L}{T}\right) \leq 1.3\ T$$

The model indicated in this subsection applies when the pile is sufficiently buried inside the ground – at least two and a half times its elastic length.

Table 3.6.4. Reduction Factors for Forces and Moments in the Buried Portion of Piles

Depth	Reuction Factors	
z/z_o	α	β
0	0.00	1.00
0.5	0.52	0.95
0.8	0.62	0.88
1.0	0.64	0.75
1.2	0.62	0.62
1.5	0.54	0.42
2.0	0.32	0.15
2.5	0.13	0.05
3.0	0.00	0.00

To estimate the horizontal movement of a group of piles, the technical publications illustrating the specific problem under study should be consulted. To analyse the situations in which the horizontal movements estimated are about three times less than the movement that would damage the structure (taking it beyond its serviceability limit), the simplified procedure described here can be adopted, but modifying somewhat the equivalent lengths of the rigid beam and the spring constants representing the buried portion of the piles.

Each pile in the group can be taken as having its buried section replaced by a virtual rigid rod supported by the springs shown in Figure 3.6.15, but affecting the elastic length estimated in the *isolated pile* hypothesis by a increasing factor.

$$T \text{ (pile in group)} = \alpha \cdot T \text{ (isolated pile)}$$

In the absence of more reliable specific data, the following values for α are recommended:

$$\alpha = 1 + \frac{0.5}{m^2} < 1.25$$

where *m* is the ratio between the separation of the piles in the group and their diameter. For groups of piles spaced more than three times the diameter apart, this effect does not need to be taken into account.

3.6.10 Considerations on Structural Failure Modes

As structural elements, piles must comply with the same conditions of safety and serviceability as the other structural elements of the works.

Determining the actions on piles is, however, a complex matter of soil-structure interaction in respect of which some recommendations are given below.

3.6.10.1 Load Distribution in Pile Groups

The individual load acting on each pile in a group under near-service conditions can be obtained by an elastic calculation. Each pile is represented by its freestanding portion connected to a rigid rod. This is supported at its bottom end by the elastic springs shown in Figure 3.6.14, but taking into account the modifications resulting from the group effect defined in Subsection 3.6.9.2.1.

The vertical spring constant in isolated piles will be the quotient between the vertical load, *P*, and the settlement, *s*, it produces in the pile at ground level.

This quotient, deduced from the formula shown in 3.6.9.1.1, gives the expression:

$$K_v = \frac{1}{\left(\frac{D}{40Q_h} + \frac{\alpha\, l_2}{AE}\right)}$$

with the variables having the same meanings as indicated in that subsection.

In the case of piles in a group, the expected settlement must be calculated beforehand as shown in 3.6.9.1.2.

With these calculations, the average settlement of the group s_{group}, corresponding to the total vertical load N_{group} can be estimated. Hence, the load-settlement ratio $K_{v,group}$ can be approximated as:

$$K_{v,group} = \frac{N_{group}}{s_{group}}$$

This factor can be taken as the sum of the contributions from all the piles in the group.

$$K_{v,group} = \sum K_{v,i}$$

The distribution of the total stiffness $K_{v,group}$ over the different piles in the group $K_{v,i}$ is a problem that can only be solved by consulting specialized technical literature.

In projects where this aspect is not critical and provided that the cap is sufficiently rigid, it is advisable to distribute the total estimated stiffness proportionally over the different piles in a weighted manner. The following weighting factor should be assigned to each pile:

$$f_i = A_i\, E_i\, p_i$$

where p_i is an dimensionless number that will depend on the relative position of the pile in the group.

The factor p_i varies from 1 for outer piles in the group to a higher value for inner piles, and can be as much as 1.5 in the case of inner piles in dense groups. For pile groups where the spacing is more than seven times the diameter, the p_i weighting factors can be taken as equal to 1.

Taking the above into account, the load-settlement relationship of a pile in a group is:

$$K_{v,i} = \frac{N_{group}}{s_{group}} \cdot \frac{A_i E_i p_i}{\sum A_i E_i p_i}$$

3.6.10.2 Structural Strength

Loads acting on a pile can exceed the structural strength of its resistant section.

Stress resultants on piles will generally be defined by diagrams of the values corresponding to the axial and shear forces and bending moments occurring along their axis.

As with any other structural element, it should be verified -along the whole length of the pile- that its resistant capacity compared with the estimated stress resultants will lead to the same safety levels required for the structural elements in the project.

Buckling of piles with a freestanding portion should be made with restraint models simulating embedment in the ground by springs, as indicated in Subsections 3.6.9 and 3.6.10.

Buckling in totally buried piles does not generally need to be taken into consideration, except where piles cross layers of soft or very soft cohesive soils. In these cases, it should also be checked that axial the load, *N*, is clearly lower than the theoretical buckling load given by the expression:

$$K_{buckling} = 8 \cdot \left(s_u EI\right)^{\frac{1}{2}}$$

where:

s_u = average undrained shear strength in the soft layer.
EI = product of inertia of the pile section.

The safety factor against buckling, $F = N_{buckling}/N$, deduced from this expression, should in any event be higher than the minimum shown for bearing failure in Table 3.6.1 (analytical formulae).

The structural calculation of the pile lies beyond the scope of this ROM 0.5 devoted to geotechnical engineering. Structural design requires different checks and use of other methods. The considerations indicated in this Subsection 3.6.10 should only be followed as a guide and in the absence of other more specific recommendations. In any event, in service conditions (unfactored loads), it is advisable that the average normal compression acting on the pile section does not exceed the values shown in Table 3.6.5 (14):

Table 3.6.5. Maximum Average Compression in Piles

Type of Material	Execution Method	Type of Support	
		Floating	End-bearing
Reinforced concrete ($f_{ck} \geq 2.5$ MPa)	Driven	$0.3\ f_{ck}$	
	Excavated with casing	5 MPa	6 MPa (15)
	Excavated (other procedures)	4 MPa	5 MPa
Prestressed concrete	Driven	$0.3\ (f_{ck} - 0{,}9\ f_p)$	
Steel	Driven	$0.33\ f_{yk}$	
Timber	Driven	5 MPa	

f_{ck} = characteristic compressive strength of concrete.
f_p = nominal compression due to active reinforcements.
f_{yk} = elastic limit of the steel.

(14) These stresses define the concept of "structural capacity", commonly used in the design of deep foundations.
(15) Intensive control of the execution may justify the use of greater *structural capacities* for piles embedded in rock, but never by more than 20% of the value indicated.

3.7 EARTH RETAINING STRUCTURES

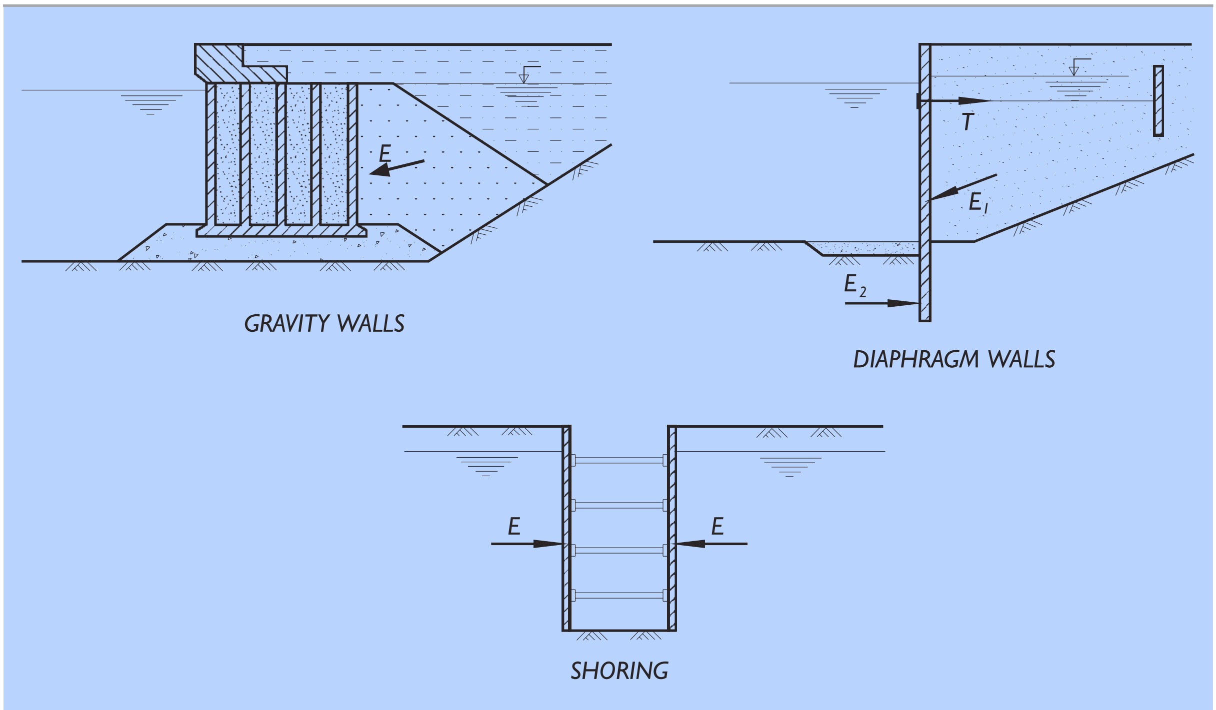
3.7.1 Types

Retaining structures are artificial elements used to create differences in ground level that would otherwise lack stability.

A wide variety of retaining structures can be used as aids to keep slopes stable. They range from simple bolts for rock slopes to massive concrete structures to retain major differences in soil levels.

For these Recommendations on the design of maritime and harbour works, retaining structures have been classified in three basic types. They are described below and illustrated in Figure 3.7.1.

Figure 3.7.1. ***Basic Types of Retaining Structures***



3.7.1.1 Gravity Walls

These are retaining structures basically using self-weight to withstand earth pressures.

Gravity quays formed by concrete blocks or precast caissons are typical examples.

Other examples of this group are reinforced concrete walls which, even with a light structure, withstand the earth pressure with the help of the weight of the ground acting on their heel slab or on shelves at mid-height or other structural arrangements.

Gravity walls can rest on the ground supported by shallow foundations in the form of strip footings or by deep foundations. In any event, the general principles recommended in Sections 3.5 and 3.6 of this ROM 0.5 relating to foundations apply to the study of these retaining structures.

3.7.1.2 Diaphragm Walls

These are retaining structures that receive earth pressure directly and withstand it by embedment of their toe and by possible anchors close to their head.

Driven sheet-pile quay walls and retaining walls made from continuous reinforced concrete diaphragms or closely spaced piles provide typical examples of these structures.

The quays (or other walls) formed by sheet-pile cells are a special case, since they have characteristics in common with flexible diaphragm walls in some details although, as a whole, they also have aspects in common with gravity walls.

3.7.1.3 Shoring

Retaining structures should be classified in this group when, taking advantage of two nearby excavation planes, loads are transferred from one to the other through structural elements working basically in compression (struts, braces, etc.).

The most characteristic example is a trench excavation. As excavation advances, the shoring of each wall is braced against the other, thus maintaining stability of the whole system.

The essential feature of this type of structure is the small amount of displacement they allow, since the struts are generally rigid compared with the ground, either because of their large cross-section or short length.

3.7.2 Limit Values of Earth Pressure

Pressure between the ground and a retaining structure will depend, first and foremost, on the relative movement between ground and structure.

When the retaining structure moves away from the backfill, earth pressure decreases and, for a sufficiently large relative displacement, the ground will eventually exceed its resistant capacity and fail along certain sliding surfaces. At the limit state of failure in the backfill material, the pressure on the wall reaches a minimum value known as *active earth pressure*.

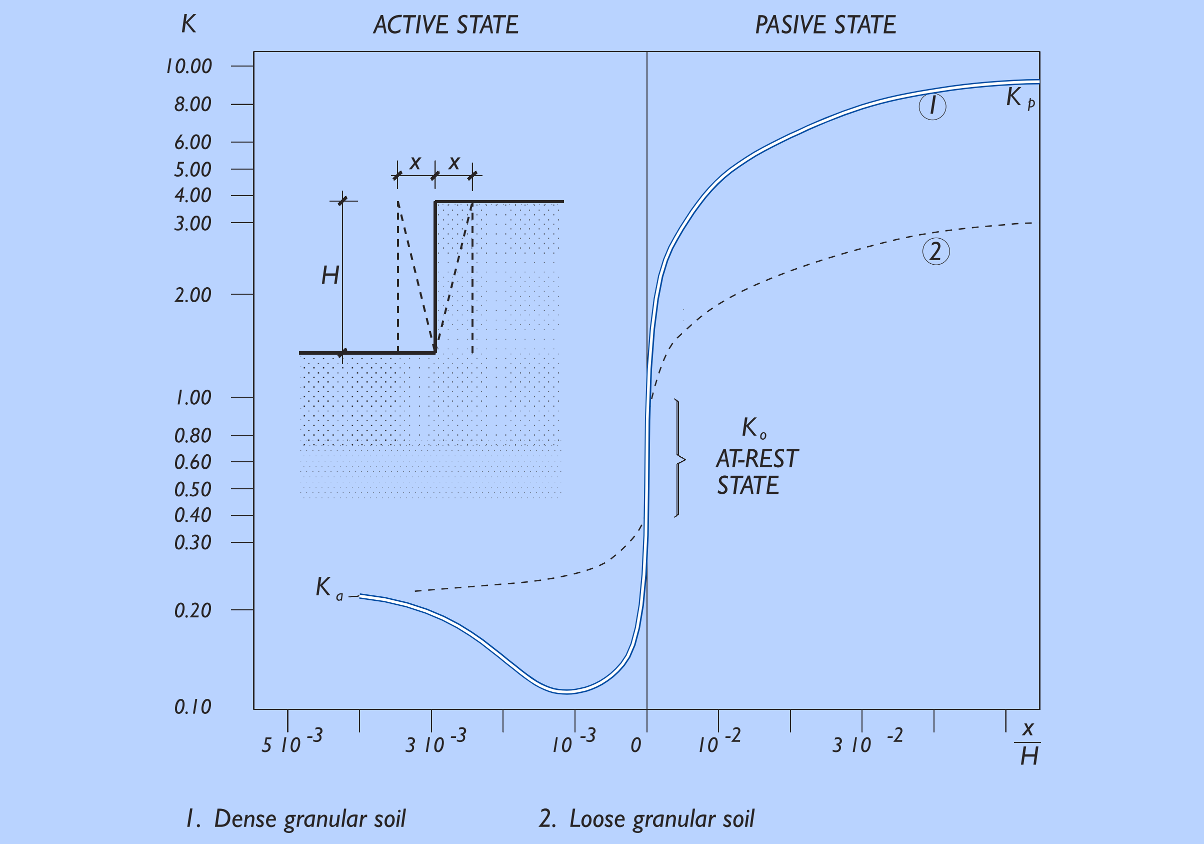
Similarly, when the wall moves against the ground in such a way that the latter helps it to withstand external loads (berthing pressures, anchor pulls, etc.), earth pressure increases with deformation. There is also a limit to the pressure that can be mobilised. This limit is mainly governed by ground strength and known as *passive earth pressure*.

Some soils exhibit peak strength and therefore experience a drop in shear strength with large strains. It is then possible to find intermediate states of deformation where an even lower pressure than the active pressure is reached - or a higher pressure than the passive pressure. In this ROM 0.5, limit values (active and passive) are taken to mean the values corresponding to large strain, beyond those necessary to exceed the possible peak strength.

For small relative deformations, such as the ones that can occur in walls with rigid structures at their heads preventing movement (the basement walls of buildings, for example) or which cannot move relative to the ground because of some other structural arrangement, the earth pressure will be close to the so-called *earth pressure at rest*. This theoretical concept would correspond to the hypothetical case of null relative movement between the retaining structure and the backfill.

Figure 3.7.2 gives an example illustrating the variation in the earth-pressure coefficient (a concept defined later on) as a function of the rotation of a hypothetical wall retaining a granular fill.

Figure 3.7.2. **Diagram of the Relationship between Earth Pressure and Displacement**



Ground Type	Rotation x/H	
	Active Pressure	Passive Pressure
Dense Granular	10^{-3}	2.10^{-2}
Loose Granular	4.10^{-3}	6.10^{-2}
Hard Cohesive	10^{-2}	2.10^{-2}
Soft Cohesive	2.10^{-2}	4.10^{-2}

This figure also shows typical values for the rotation needed to mobilise limit pressures in other types of ground.

Other experiences have shown that the movements necessary to bring about the limit state of active pressure in medium-density granular ground is in the following order of magnitude:

Rotation about the head = 0.002 H
Rotation about the toe = 0.005 H
Horizontal displacement = 0.001 H

where *H* is the height of the wall.

Most of the calculations mentioned in this ROM 0.5 refer to checks on Ultimate Limit States in which it is assumed that the ground fails and in most cases it will therefore be necessary to calculate the earth pressure limit values.

In calculations for the verification of Limit States of Serviceability and in other calculations relating to structural failure modes, intermediate earth pressure situations may need to be taken into consideration.

3.7.3 Simplifications Required for Earth Pressure Calculations

The simplified procedures shown in this ROM 0.5 enable earth pressures to be calculated in relatively simple geometrical cases, which is why certain simplifications of the real problem need to be made in order to facilitate subsequent calculations.

Firstly and except for some considerations dealt with later (Subsection 3.7.9), the problem of earth pressure calculation should be two-dimensional. To this end, engineers should consider as many vertical plane sections as necessary to evaluate earth pressures in different parts of the works.

After this simplification, engineers will need to make further simplifications on the back face geometry, the ground geometry and the inclination of the earth pressures. The following Subsections give guidelines on adequate procedures for some specific cases. These simplifications may be interconnected. The way in which the back face is simplified will affect the assumption about the inclination angle of earth pressure, for example.

Finally, engineers should make some additional simplifications that will be referred to for each particular case.

3.7.3.1 Geometry of the Wall Back Face

The solutions existing for calculating earth pressures have generally been deduced from certain theories in which the contact surface between the wall and the backfill is a plane. The broken surfaces occurring in real cases must be converted, even if this is done in sections, into equivalent plane surfaces to simplify the subsequent calculation.

It is considered admissible to substitute the real back face by a virtual one that is a vertical plane passing through the back heel of the wall. The ground between this plane and the real back face should be taken as part of the wall itself. This situation is illustrated in Figure 3.7.3.

If this simplification is made, it should be assumed, when subsequently calculating the earth pressures, that the effective earth pressure component runs parallel to the surface of the ground (horizontal in the case of the diagram referred to). To assume greater inclinations could lead to optimistic results in calculations of active pressures [(16)].

Similarly, this assumption ($\delta = 0$) may prove to be too severe in cases where the virtual and the real face are very close (rightmost diagram in Fig. 3.7.3). In these cases, it is advisable that the design back face coincide with the real contact between the wall and the ground in the greatest possible portion of the height. Thus, it will be possible to utilise there the values for the δ angle recommended in Subsection 3.7.3.3.

The above simplification may not apply to certain shapes of back faces such as those illustrated in Figure 3.7.4 and, furthermore, may be somewhat gross for application in cases where the distribution of unitary pressures needs to be known closer to the real contact of the ground with the back face of the retaining structure.

An L-shaped wall, a frequent type, can be assimilated to a wall with a plane virtual back face, as shown in Figure 3.7.5.

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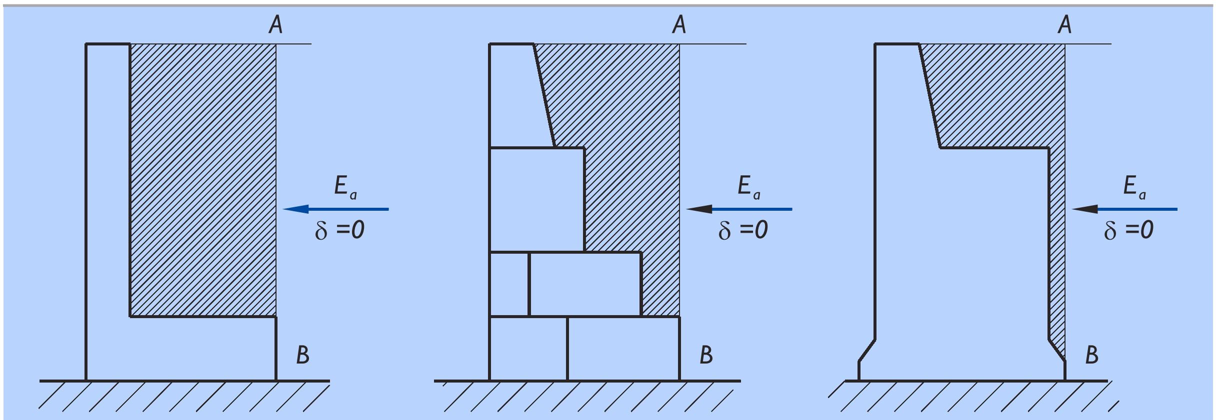
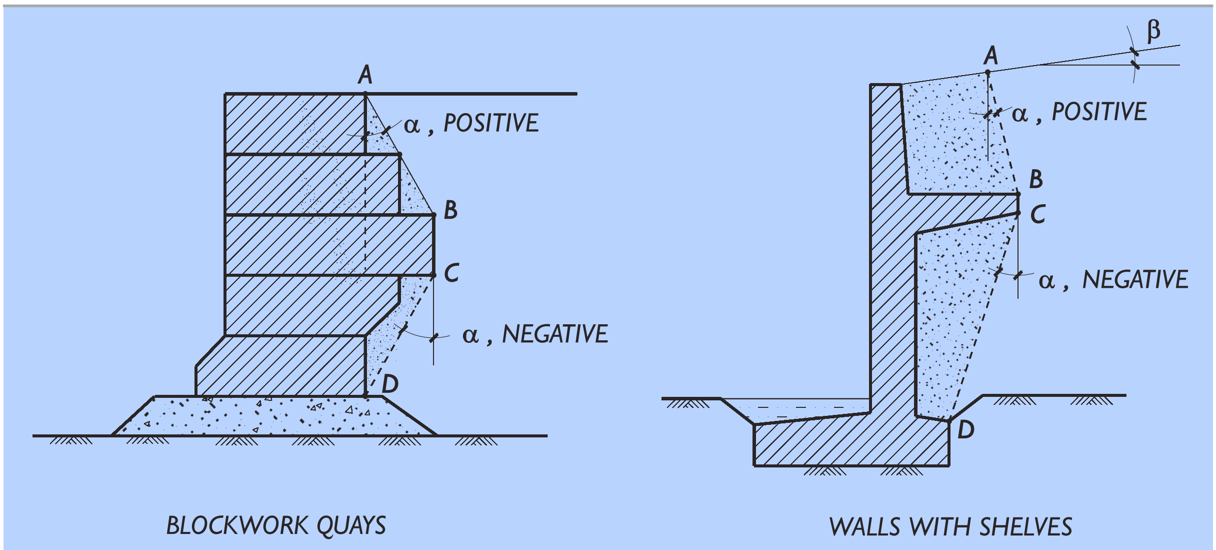
(16) Consider, by way of example, the L-shaped wall drawn in the diagram in Figure 3.7.3. In this case, with the vertical virtual back face, a large equivalent wall weight, W_o, would be obtained, including the weight of the ground in the shaded zone, and a horizontal earth pressure E_a. When a broken line is considered for the equivalent back face, as shown in Figure 3.7.4, and additionally the angle of inclination of the earth pressure on the sloping portion of the back face is $\delta_1 = \phi$, what is obtained is a lower weight $W_1 < W_o$, an identical value for the horizontal active earth pressure (Rankine's method) and a vertical earth pressure component that exactly offsets the weight deficit, i.e., $E_V = W_1 < W_o$. The loads acting against the foundation would be identical in both cases.
This means that assuming $\delta = 0$ in the vertical virtual back face leads to the minimum horizontal earth pressure that could occur and, consequently, any value of $\delta > 0$ assumed in the vertical virtual face will lead to dangerously optimistic results.

Figure 3.7.3. Substituting an Uneven Back Face by an Equivalent Vertical Plane



N.B.: The simplification shown may prove to be excessively conservative in the case of the wall (caisson) in the rightmost diagram (see text).

Figure 3.7.4. Equivalent Virtual Face in Walls with Broken Back Face



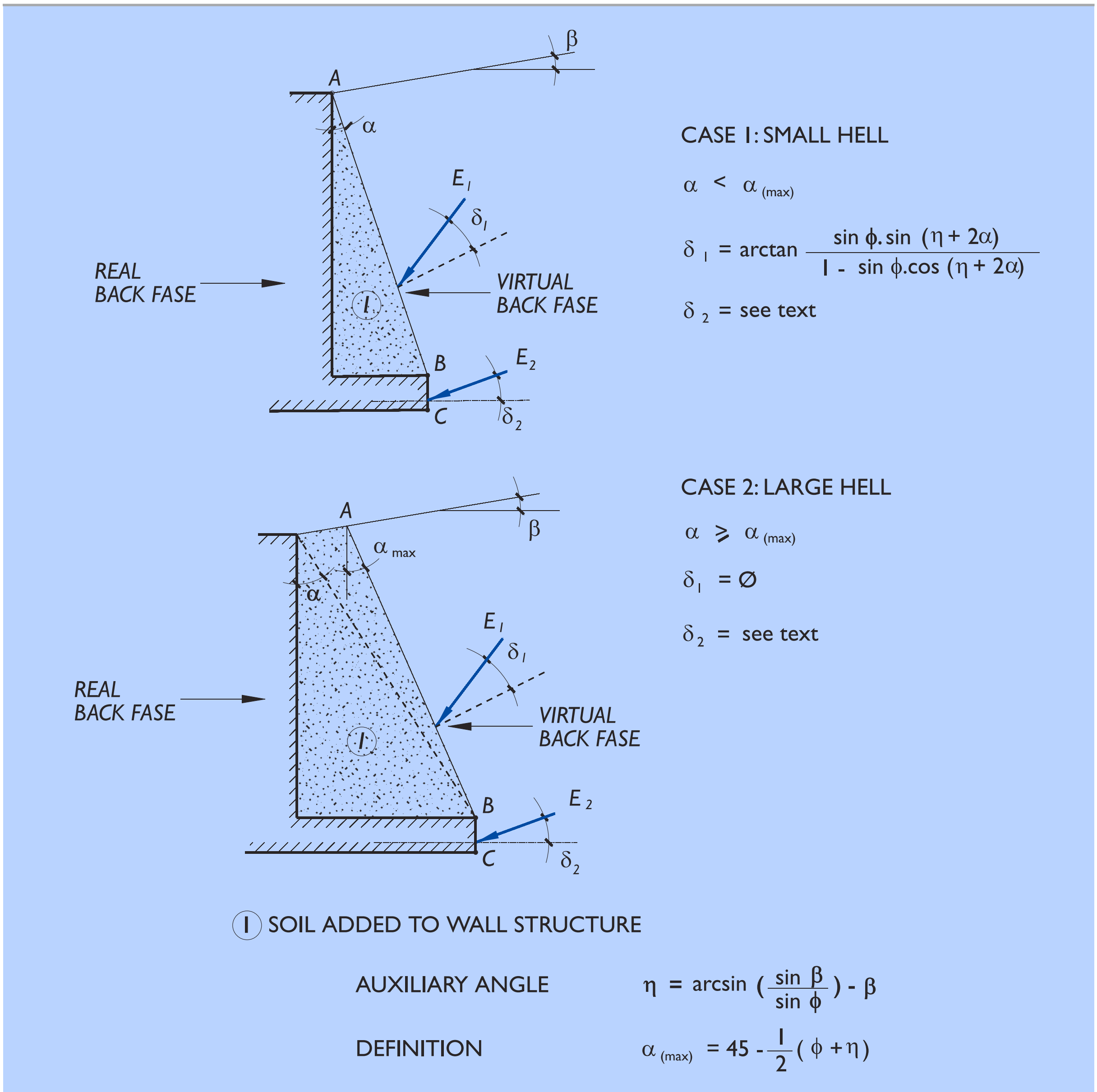
If the simplification from this figure is adopted, the inclinations shown should be also assumed when calculating the effective earth pressures corresponding to the active earth pressure state. These inclinations are compatible with the earth pressure theory derived from Rankine's hypothesis of active plastification.

The relationship existing between the angle of inclination of the effective earth pressure with respect to the normal to the virtual face (angle δ) and the other characteristic angles of the problem is a known analytical function, which is shown in Figure 3.7.6.

For the sections of the wall where the real face is not substituted by an equivalent virtual plane (sections BC in the foregoing figures), the angle of inclination of the earth pressure must be determined in accordance with the recommendations given in the following section.

In L-shaped walls, it is possible to determine equivalent virtual faces even closer to the real face than the ones given here. In any event, it is recommended not to use faces with angles of deviation from the vertical (α angles) outside the range of α_{max} - α_{min} shown in Figure 3.7.6. It has not been proven that α angles outside this range always lead to realistic estimates of earth pressures.

Figure 3.7.5. ***Equivalent Back Face in L-Shaped Walls***



3.7.3.2 Heterogeneous Ground

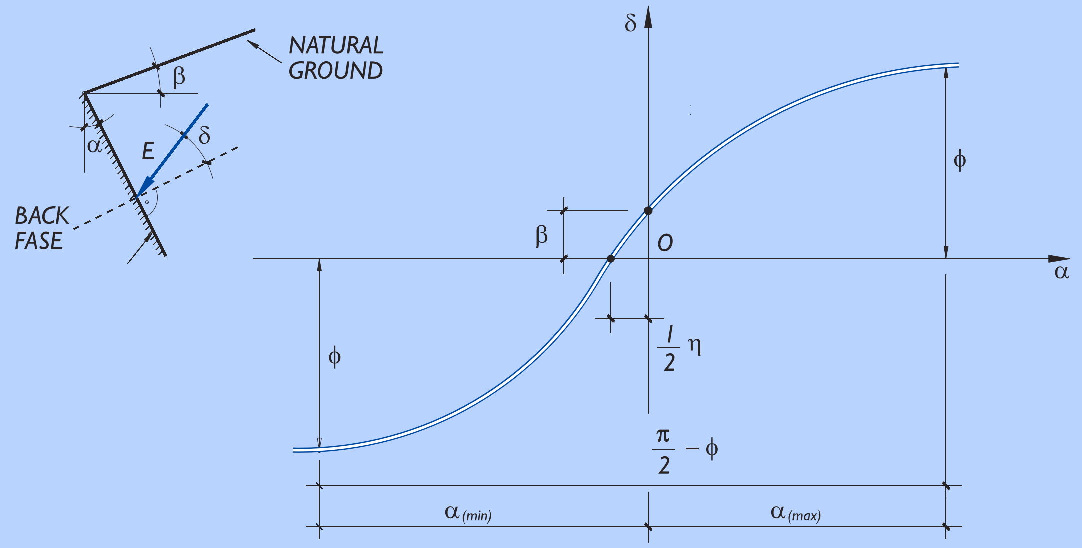
The ground around the retaining wall will generally be heterogeneous and this will make simplifications necessary so that the calculation is viable in a sufficiently simple manner.

The backfill of retaining walls should be represented by zones of uniform strength and density. For this purpose, the groundwater table can be a cause of discontinuity.

When making the necessary simplifications to represent the real ground in plane sections for calculations, engineers should exercise caution, assigning the largest thickness than could reasonably be expected to the weaker levels or strata.

In cases where geometrical variations in the ground could be considerable and are not well controlled, it is advisable to carry out a sensitivity analysis, repeating calculations for different ground geometries.

Figure 3.7.6. Graph of the Variation of Earth Pressure Inclination



α = deviation of the wall virtual back face from the vertical

δ = earth pressure inclination

β = ground surface inclination

η = auxiliary angle defined by:

$$\eta = \arcsin\left(\frac{\sin\beta}{\sin\phi}\right) - \beta$$

RANKINE´S SOLUTION TO THE RELATIONSHIP BETWEEN “δ” AND “α”

$$\tan\delta = \frac{\sin\phi.\sin(\eta + 2\alpha)}{1 - \sin\phi.\cos(\eta + 2\alpha)}$$

3.7.3.2.1 Fills on Slopes

In the general procedure for calculating earth pressure given later on, this type of heterogeneity does not need to be simplified. In the less laborious calculation procedures based on earth pressure coefficients (Subsection 3.7.5), it is however necessary to simplify the problem.

As shown in the same figure, the problem can be simplified by assuming a single equivalent ground with a friction angle obtained as the weighted average of the ones corresponding to the materials forming the backfill. Effective weights should be used for weighting, i.e., calculated with the apparent density above the groundwater table and the submerged density below it.

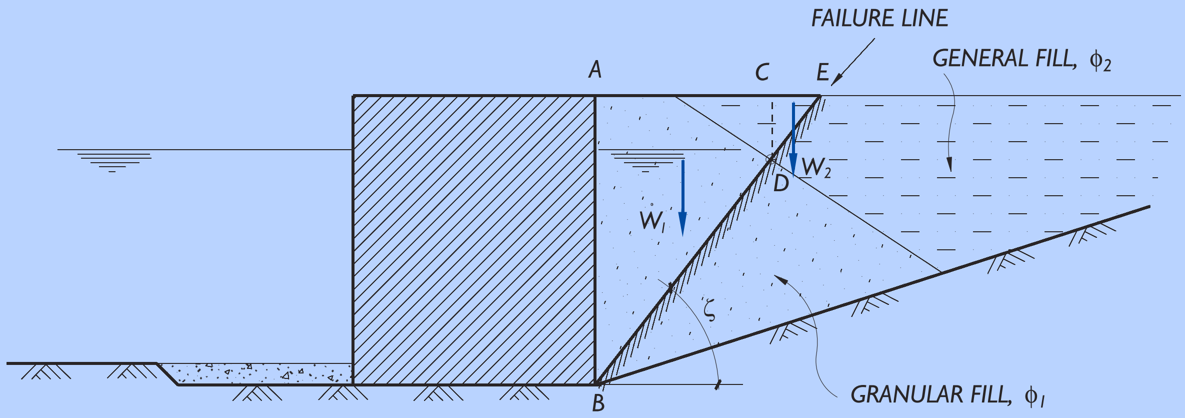
If cohesion is present, the weighted average should be obtained in respect of the different lengths of the failure line sections in each type of ground (lengths BD and DE in the figure).

The base line, BE, used for these weighting, attempts to be a first approximation to the ground failure plane in the case of active earth pressure. Having obtained the value of $\phi_{equivalent}$, a somewhat better value can be approximated by using the following angle as the inclination of this base line:

$$\zeta = 45° + \frac{1}{2}\phi_{equivalent}$$

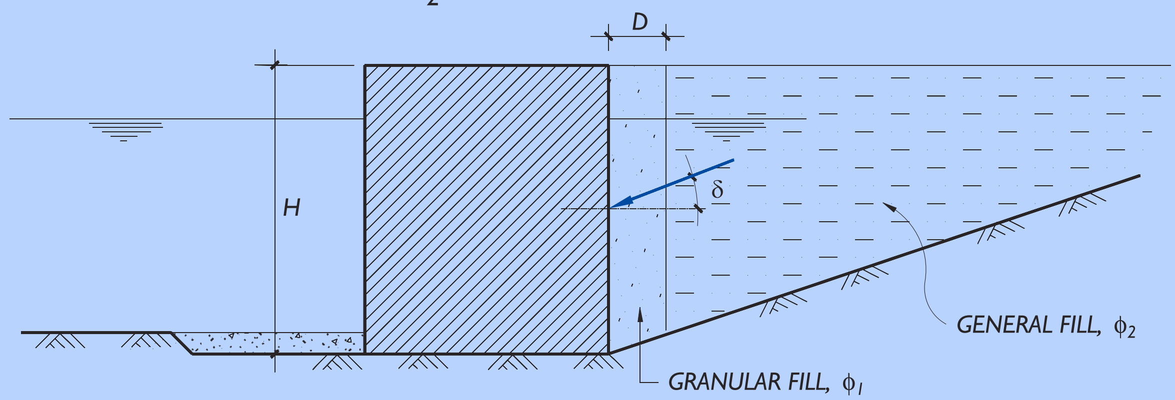
This iteration can improve the estimation of desired the angle of friction.

Figure 3.7.7. **Sketch of Typical Heterogeneities**



$$\tan\phi_{(equivalent)} = \frac{W_1 \tan\phi_1 + W_2 \tan\phi_2}{W_1 + W_2}$$

$$\zeta = 45 + \frac{\phi_2}{2}$$



$$\tan\delta < \tan\phi_2 + \frac{2D}{K_{ah} \cdot H}$$

K_{ah} = the horizontal active earth pressure coefficient corrsponding to the general fill on the assumption that $\delta_2 = \phi_2$ (See Subsection 3.7.5.)

This simplification will give rise to errors that will generally be on the safe side. The active earth pressure calculated using the equivalent ground will generally be greater and will be located somewhat higher than the one corresponding to a more precise calculation.

A similar procedure can be followed in the case of more than two types of ground, using as base line the straight line inclined at an angle of 45° + ϕ/2, where ϕ is the angle of friction of the weakest soil. By means of an iterative calculation, this initial ϕ angle can then be approximated to the equivalent angle of friction resulting from the weighting process.

If this simplification is used for calculating passive earth pressures, a similar procedure can be followed, although in this case the inclination of the base line for interpolation purposes should be:

$$\zeta = 45° - \frac{\phi_1}{2}$$

where ϕ_1 is the angle of friction of the most resistant ground which, in successive iterations, can then be approximated to the $\phi_{equivalent}$ resulting from the weighting process.

In any event, the angle δ to be used in calculations should be the one corresponding to the material of the contact between wall and backfill.

3.7.3.2.2 Fills in Narrow Bands

One rather common situation is illustrated in the bottom section of Figure 3.7.7. A more resistant granular material is used only in a narrow band in the contact zone of the wall against the ground.

In such cases, it is possible to use a similar procedure to the one described in the above section to calculate the equivalent angle of friction. If the strip of granular material is narrow (D << H), the equivalent ground strength, for practical purposes, will be the one corresponding to the general fill.

The angle of friction between the soil and the wall, δ, can however be assumed to be governed by the granular material, even though the limitation shown in the figure should be obeyed as also the general limitations indicated in Subsection 3.7.3.3 below.

If this last simplification is used in passive earth pressure calculations, the K_{ah} coefficient mentioned in the figure should be substituted by the K_{ph} coefficient corresponding to the passive earth pressure.

3.7.3.3 Earth Pressure Inclination

The inclination of the effective earth pressure is defined in this ROM 0.5 by the angle δ formed between the earth pressure and the normal to the face. This angle is considered positive when the action of the ground against the wall pushes this lower down than the normal indicates. Positive values of δ are shown in several figures in this Section 3.7.

The δ value is generally a parameter that engineers must decide on following the recommendations given below.

In calculating active earth pressures, the greater the angle δ assumed, the more optimistic will generally be the result (smaller horizontal earth pressure and a more favourable inclination with respect to the different Ultimate Limit States). For this reason, in active earth pressure calculations engineers should use moderate δ values, which in any event must not exceed the angle of friction of the backfill material with the wall face).

In passive earth pressure calculations, the δ value is less clear, as it may be conditioned by the direction of the external forces causing the passive earth pressure to be mobilised. In anchor walls, for example, the pull of the anchor conditions the direction of the earth pressure.

As a general rule, passive earth pressure will increase as the angle δ decreases. The value of ? will normally be negative, but it should be checked in each works that the relative displacement necessary for this earth pressure inclination to occur is compatible with the external restraints.

In any event and regardless of whether the limit earth pressure is active or passive, the absolute value of ? should not exceed the shearing resistance angle of the ground-wall contact. Table 3.7.3 gives indicative values for this friction angle, which engineers can use in the absence of better information. If the structure or ground are subjected to substantial vibrations, $\delta = 0$ should be considered.

In calculations where the wall face is virtual (resulting from simplifying a real broken back face) and is therefore represented by a line within the ground, the limitations given in Subsection 3.7.3.1 should be taken into account. Limitations on the value of δ indicated in 3.7.3.2 resulting from earlier simplifications of possible heterogeneities in the ground should also be taken into account.

In calculations of earth pressure at rest, the direction of the pressure should be taken to be parallel to the surface of the ground. There may be exceptions to this general rule, duly justified by engineers.

Tabla 3.7.1. ***Maximum Values for the Angle of Friction between Wall Face and Ground, Depending on the Ground's Angle of Friction, ϕ***

	Cohesive and Granular Soils, Long Term	Cohesive Soils, Short Term
Perfectly smooth faces (*)	0	0
Steel	$(2/3) \cdot \phi$	0
Precast concrete/other types of concrete/masonry/rockfill/timber	$(2/3) \cdot \phi$	0
Concrete cast against the ground	ϕ	0

* Treated with asphalt, tar, bitumen, etc.

3.7.4 General Method for Calculating Active Earth Pressure

The state of active earth pressure is a situation of ground failure and can therefore be analysed by usual methods of statics, once the different data characterising the problem are known and after having made any supplementary assumptions required to complete its determination.

3.7.4.1 Geometry of the Problem

The two-dimensional section under study will be defined by a wall back face formed by a smoothed continuous line coinciding with the real back face or coming close to it as indicated in Subsection 3.7.3.1. The other line bounding the problem will be the external surface of the backfill, which can be any shape.

The backfill should be represented by several areas of homogeneous material that will have been defined applying the criteria given in Subsection 3.7.3.2.

The angles of inclination of the earth pressures to the normal of the wall face in each section (angle δ) should follow the indications from Subsection 3.7.3.3.

Another essential element for calculating earth pressures will be a definition of the condition of the water in the backfill of the wall. To define this, it will be necessary to have previously analysed the flownet that may exist. The recommendations given in the next section should be followed for this.

3.7.4.2 Flownet

Seepage in the backfill of the wall can be the result of different causes. In port zones one of the predominant causes can be variation in the sea level as a result of tides and wave action.

The range of the water level differences (total head loss between the two boundaries of the flownet) should be estimated following the recommendations shown in Table 3.4.1 of this ROM 0.5.

The corresponding flownet must be obtained following the general principles and recommendations given in Section 3.4 of this ROM 0.5.

To apply the general calculation method described below, the pore water pressure existing in all points of the backfill should be known.

3.7.4.3 External Loads

The external loads capable of acting on the backfill of the wall must be taken with their design values, i.e., their characteristic values affected by the combination factors corresponding to the design situation under study and increased by the corresponding load factor.

For the active earth pressure calculations whose results are to be used in analysing non-geotechnical Ultimate Limit States (those where the ground strength plays a secondary role), other design values can be used for loads, applying whichever increasing or reducing factors correspond.

Any point loads that may exist can be converted into equivalent linear loads by dividing the total load by a length equal to the distance from the load to the wall back face.

Linear loads acting along a line of length *L*, running parallel to the crest of the wall and separated from it by a distance, *D*, must be taken to act along an indefinite length. In this case, the loads can be reduced by multiplying them by the following factor:

$$f = \frac{L}{L + D}$$

3.7.4.4 Ground Data

Each type of ground forming the wall backfill should be characterised in order to know its unit weight, both in the submerged and non-submerged zone, along with its shear strength.

These parameters may change with time. The parameters to be used in calculations must match the time frame of the design situation under analysis.

The backfill of major retaining structures will generally be granular and undergo fast consolidation. Therefore the active earth pressure must be calculated using the effective pressure strength parameters that simulate long-term resistance.

When the backfill of a retaining wall is a pre-existing clayey ground (trench-excavated diaphragm walls, for example), the active earth pressure on the wall will generally increase with time and for this reason it will not normally be necessary to check any situation other than the one corresponding to the long-term.

Retaining works with a backfill that can take a considerable time to consolidate need a special calculation corresponding to the short-term situation, since this will generally lead to higher values for the active earth pressure.

The extreme case of null consolidation of a submerged fill can be simulated by taking a zero angle of friction and cohesion equal to the undrained shear strength. Section 4.9 of this ROM 0.5 gives some typical values for the undrained shear strength of submerged fills shortly after being deposited. These values can provide guidance for engineers in preliminary calculations. When this aspect conditions the design of the structure in question, it is advisable to carry out specific tests to determine the strength of the fill.

3.7.4.5 Calculation Sections

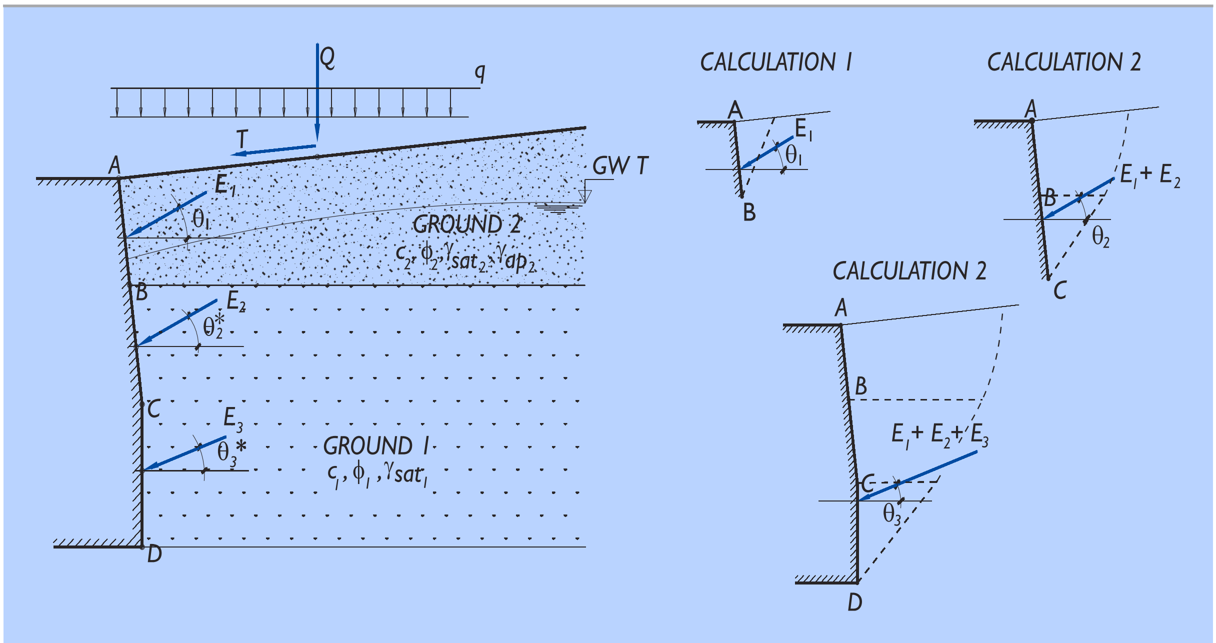
The geometry of the problem, the corresponding flownet, the external loads and the ground properties should allow the corresponding problem to be configured as illustrated in Figure 3.7.8.

The wall back face will be defined by a series of points (A, B, C and D in that figure), which separate either contacts between different types of ground or changes of inclination in the face of the wall. The intersection between the wall back face and the groundwater table can be taken as an additional point in this division, although this is not strictly necessary nor will it provide any more accuracy for the value of the earth pressure calculated.

The accuracy about the position of the line of action of the earth pressure will, however, increase with the number of calculation sections fixed on the back face of the wall. To this end, it may be useful to insert more points than otherwise strictly necessary.

The earth pressure should be calculated in sections beginning with the topmost zone. Each section must be calculated as shown in Subsection 3.7.4.6 below. The calculation indicated there provides the total earth pressure and therefore includes the pressure due to the water. This is why variable ? is used for the angles of deviation of the total earth pressure from the normal to the back face shown in Figure 3.7.8, to distinguish those angles from the angles of inclination δ corresponding to effective earth pressures. The relationship between these two angles is explained in the following Subsection.

Figure 3.7.8. Diagram of the General Method for Calculating Active Earth Pressure



N.B.: The θ angles are marked in relation to the horizontal.

The first calculation will enable the earth pressure, E_1, to be obtained. The second will provide the vector sum of E_1 and E_2, and the value of E_2 can therefore be obtained by subtraction. This procedure, repeated in each successive calculation, will enable each of the earth pressures to be ascertained.

The position of each earth pressure (the intersection point of its line of action with the back face of the wall) is left undefined by these calculations and it will therefore be necessary to make a supplementary assumption.

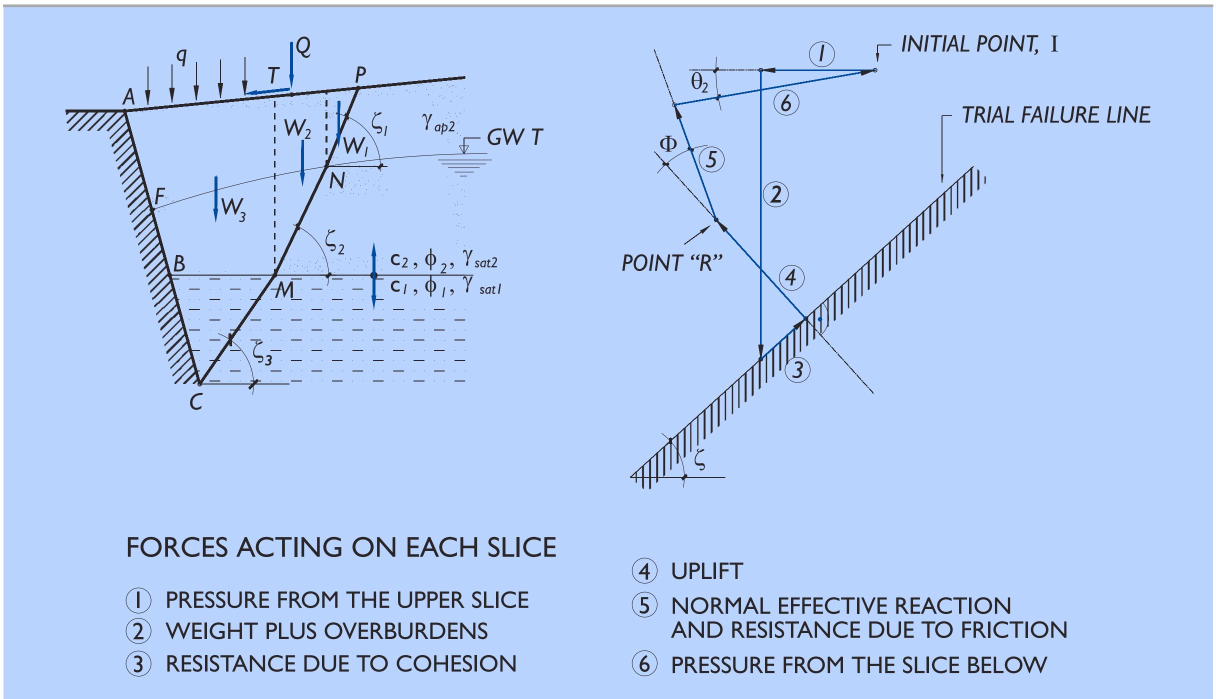
The earth pressure in the first section must be taken to be situated at a distance between 1/2 the length of the section (if considerable surcharges exist) and 2/3 of that length (if no nearby overburdens affect E_1).

The earth pressure for each other section can be assumed to be applied at its centre. The error involved in this will generally be on the safe side and the shorter the calculation sections, the smaller it will be.

3.7.4.6 Calculating the Earth Pressure in Each Section

The calculation of the active earth pressure on a wall is illustrated in Figure 3.7.9. The back face of this wall would correspond to section AC of the top part of the wall shown in Figure 3.7.8. It would therefore represent Calculation 2 of those needed to obtain the earth pressures on this wall.

Figure 3.7.9. Calculating the Active Earth Pressure on Section AC



To calculate the total earth pressure on the *AC* section, different polygonal failure lines should be tried, whose slope ζ must be constant or increasing as the line rises from the base of the wall *C* to the ground surface *P*. After trying out several combinations, the one leading to a higher earth pressure should be used.

Utilising the intersections between this failure line and the lines marking discontinuities in the ground (separation of materials, intersection with the groundwater table), the vertexes of these polygonal lines (Points *M* and *N* in the figure) should be identified.

The verticals passing through the vertexes will define a set of slices. The desired earth pressure can be obtained by studying the equilibrium of each of these slices.

The six forces shown in Figure 3.7.9 should be considered in studying the equilibrium of each slice. Calculation should start by analysing the top slice.

Force 1 is the earth pressure of the previous slice and will therefore be null for the first slice.

Force 2 will be the sum of the total weight of the slice plus any external loads acting on its head or in its interior. To calculate the total weight of each slice, the apparent densities should be used in the zone above the groundwater table - and the saturated densities below it.

Force 3 is the product of the cohesion (if not null) and the length of the slice base and Force 4 is the water uplift acting on this same base.

Point *R* is reached after composing Forces 1 to 4, starting from the initial point, *I*.

The forces closing off the polygon are the normal effective reaction in the base of the failure line, whose direction is known (Force 5 in the diagram), and the earth pressure on the following slice, about whose inclination some sort of an assumption will have to be made.

The diagram in Figure 3.7.9 shows that the load on the preceding slice is horizontal. This should be assumed for all pressures between slices, since experience has shown that this assumption is a sufficiently conservative one.

Only when calculating the last slice, in order to obtain the pressure on the wall, it should be assumed that the earth pressure deviates from the horizontal by an angle θ, which may be different from zero.

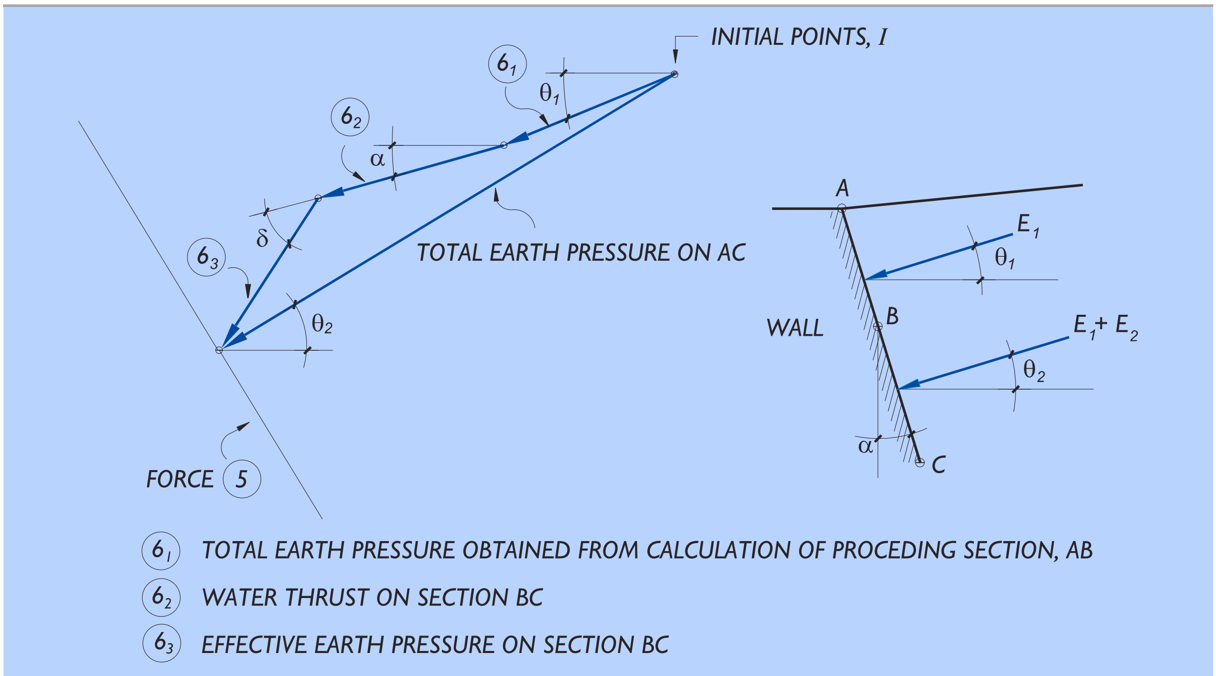
Angle of Inclination of the Total Earth Pressure

The angle of inclination from the horizontal of the total earth pressure on the wall *AC* was shown in Figure 3.7.9 by the symbol θ_2. The subscript indicates that this is Calculation 2 required to obtain the earth pressure on the wall in Figure 3.7.8.

It is therefore assumed that the total earth pressure on the preceding section, *AB*, and its orientation θ_1. was obtained from a previous calculation.

Figure 3.7.10 illustrates a procedure for determining the desired angle θ_2 of the total earth pressure on wall section AC.

Figure 3.7.10. ***Determining the Earth Pressure Inclination***



The total earth pressure on the preceding section, *AB*, is known from the preceding calculation. The pressure of the water on *BC* must be determined with the porewater pressure data on the wall face. The third component of the total pressure is the effective pressure on section *BC*, which must form the corresponding angle δ with the normal to this section of the wall face. This angle must be defined in accordance with the recommendations given in Subsection 3.7.3.3.

Using this graphic construction or its equivalent analytical expression, the orientation of the total earth pressure on the section in question can therefore be obtained.

Supplementary Calculation for Water Pressure

The general calculation procedure described in this section provides the total earth pressure value, i.e., the part corresponding to the effective ground pressure plus the part corresponding to the porewater.

A supplementary calculation of the pressure due to the porewater in each section of the wall is however necessary purely for obtaining the angle of orientation of the total pressure. The water thrust can be obtained by integrating the value of the porewater pressure at the wall-ground contact over its entire length. As a general rule, the backfill of walls, at least in the immediate vicinity of the ground-wall contact, consists of highly permeable granular material where the pressure distribution is hydrostatic. In such cases, the water pressure on any section of the wall-ground contact will be simple (the porewater pressure varying linearly) and will therefore be easy to integrate.

Simplification in Cases of a Hydrostatic Regime Throughout the Backfill

If the whole of the backfill is under a hydrostatic regime, i.e., if the backfill water is at rest, the general method given above can be simplified by calculating the effective pressures on the one hand and then calculating the water pressures separately.

The effective pressures should be calculated as laid out in the general method given above, but eliminating uplift and using the submerged density for weight calculations.

The orientation of pressures between slices will, as before, continue to be horizontal. Only the pressure of the last slice on the wall face will have an inclination ? with respect to the wall back face.

When this earth pressure calculation is carried out using submerged densities below the groundwater table, the hydrostatic pressure acting on the wall face has to be calculated independently and should be added to the resulting effective pressures.

The calculation with saturated densities and uplifts leads to identical results to the calculation with submerged densities without uplifts. For this reason engineers could choose whichever proves more convenient (usually the second).

Simplification for Seepage with a Uniform Gradient

Seepage in the backfill of the wall gives rise to a vertical gradient that can sometimes be assumed to be constant in the zone affected by the earth pressure calculation, i.e., between the potential failure surfaces and the wall's back face.

These situations can allow a simplified calculation where the total earth pressure is split into two parts that are calculated independently – one is the effective earth pressure and the other is the water pressure.

The effective earth pressure should be calculated as shown in the general procedure described above but using a design unit weight for calculating below the groundwater table:

$$\gamma_{calculation} = \gamma' + I_v \cdot \gamma_w$$

where γ' and γ_w are the submerged unit weight of the ground and the specific weight of the water respectively and I_v is the vertical gradient of the water flow.

In this simplified calculation, the earth pressures between slices must still be taken as horizontal except for the last one, nearest to the wall, where the force against the wall will form the corresponding angle ? with the normal to the face.

This calculation procedure requires the porewater pressure to be obtained separately. This pressure should be determined by integrating the pore water pressure distribution throughout the length of the wall face.

3.7.5 Methods Based on the Coefficient of Active Earth Pressure

The general method described in the preceding section can be laborious, as it requires to calculate several sections for each wall and to try out different failure lines for each section until a maximum earth pressure is obtained.

To facilitate the calculations, the procedure described below can be used, based on the concept of earth pressure coefficients.

3.7.5.1 Concept of the Active Earth Pressure Coefficient

The analytical solution of various simple problems has shown that when the backfill is homogeneous, dry and granular and, furthermore, there are no surface overburdens, the active earth pressure on the wall E_{at} (which would solely be due to the weight of the ground) is proportional to the ground's unit weight, γ, and to the square of its height, *h*, that is:

$$E_{at} = \frac{1}{2} \gamma \, K_a \, h^2$$

The coefficient of proportionality, K_a, in this expression is known as the *coefficient of active earth pressure*.

The face of the wall on which the earth pressure acts has a length:

$$l = \frac{h}{\cos\alpha}$$

where α is the deviation from the vertical of the back face plane.

The derivative of the earth pressure resultant in the direction of the wall face enables the pressure on the earth-wall contact to be calculated, which will be referred to as *unitary earth pressure*:

$$e = \frac{\partial E_{at}}{\partial l} = \gamma \, h \, K_a \cdot \cos\alpha$$

The product γh coincides with the vertical stress that would act on a horizontal plane at a depth *h* if the ground were horizontal. For this reason, it will be called here σ_v, that is::

$$\sigma_v = \gamma \, h$$

Using this convention, the *coefficient of active earth pressure* can be defined as the value given by:

$$K_a = \frac{e}{\sigma_v \cos\alpha}$$

It is worthwhile repeating here that whereas e is a stress with a clear physical meaning (effective earth compression against the wall in the back face plane), this is not the case with σ_v. In walls retaining a ground with a sloping surface, the vertical stress on a horizontal plane passing through the point where e acts is not constant, but varies with the distance from the vertical passing through the crest of the wall and, therefore, its value in the wall backfill will depend on the angle α. The vertical pressure σ_v is merely a convenient reference pressure because it is easy to handle.

As explained later, the coefficient of active earth pressure depends on the strength of the ground, on the strength of the ground-wall contact and on the angles of inclination of the wall back face and of the surface of the ground. The coefficient of active earth pressure is not therefore a ground characteristic, but a design parameter that is basically governed by the ground strength.

The concepts of the *coefficient of horizontal active earth pressure* and the *coefficient of vertical active earth pressure* will also be used from now onwards, defined by the expressions:

$$K_{ah} = K_a \cdot \cos(\sigma + \delta)$$
$$K_{av} = K_a \cdot \sin(\sigma + \delta)$$

where α and δ measure the deviations of the wall back face from the vertical and of the earth pressure with respect to the normal to the wall back face, as shown in several figures in this section.

By extension of this concept of active earth pressure, it will be assumed in this ROM 0.5 that for problems of water partially saturating the backfill of a wall, a similar relationship to the one defined in the concept of active earth pressure also holds. This relationship is shown in Figure 3.7.11.

It is still assumed, in this case, that the ground is uniform and granular and that it does not bear any surface surcharge, hence the active earth pressure is due to two causes - the effective weight of the ground and the pore water pressure.

It is also assumed that the horizontal water pressure is the one resulting from integrating the pore water pressures along a line, *BC*, inclined at an angle of 45° + ϕ/2 degrees from the horizontal, that is:

$$E_{hw} = \frac{1}{2} \cdot u_B \cdot (H - h_2) \qquad \text{(see Fig. 3.7.11)}$$

and hence the water pressure on the BF section would be:

$$E_w = \frac{1}{\cos\alpha} \cdot E_{hw}$$

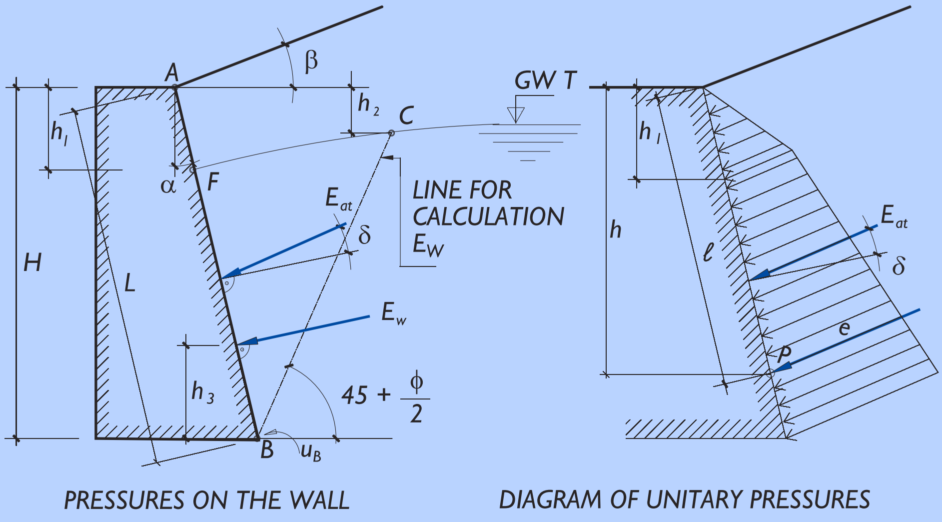
This integration assumes a linear pressure variation along the *BC* line, although in certain cases with significant seepage gradients, it could be advisable to calculate this integral more accurately.

It is also assumed that the water pressure acts at a height h_3 from the base of the wall, which is equal to a third of the difference in elevation between B and C, that is:

$$h_3 = \frac{1}{3} \cdot (H - h_2) \qquad \text{(see Fig. 3.7.11)}$$

Figure 3.7.11. ***Extended Concept of Coefficient of Active Earth Pressure in Granular Soils***

$$K_a = \frac{e}{\sigma'_v \cos\alpha}$$



e = *effective unitary pressure due to the weight of the earth at a point P on the wall black face in the active situation.*

σ'_v = *reference vertical effective pressure on P (see text).*

α = *angle formed by the wall back face and the vertical, which is positive when the earth overhangs the wall, as in this figure.*

ϕ = *internal angle of friction of the ground.*

E_{at} = *active pressure due to the effective weight of the earth.*

E_w = *pore water pressure resultant (see text).*

u_B = *pore pressure on the heel of the wall.*

BC = reference line for calculating pore water pressure.

The reference vertical effective pressure, at a point on the wall situated at a depth *h* below the wall head, should be calculated by the expression:

$$\sigma'_v = \gamma_{ap} \cdot h_1 + (\gamma' + I_v \cdot \gamma_w) \cdot (h - h_1)$$

where:

γ_{ap} = apparent unit weight.
γ' = submerged unit weight.
I_v = downward hydraulic gradient.

This gradient is defined as the quotient between the drop in piezometric level of the backfill water between *F* and *B* and the height separating these two points, that is:

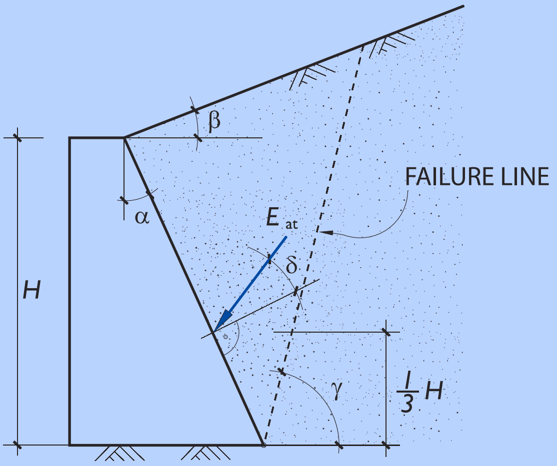
$$I_v = 1 - \frac{u_B}{\gamma_w (H - h_1)}$$

When the water pressure is calculated as shown and when the reference vertical effective pressure has the meaning defined above, in this ROM 0.5 the *coefficient of active earth pressure* is understood to be the quotient shown in the top part of Figure 3.7.11.

3.7.5.2 Value of the Coefficient of Active Earth Pressure

The coefficient of active earth pressure can be estimated using Coulomb's theory, the solution to which is shown in Figure 3.7.12 for the case of dry, uniform granular ground.

Figure 3.7.12. ***Problem Geometry and Coulomb's Analytical Solution***



SOIL CHARACTERISTICS

Unit weight, γ

Cohesion, $c = 0$

Friction, ϕ

SOLUTION:

$$E_{at} = \frac{1}{2}\gamma K_a H^2$$

where:

$$K_a = \left[\frac{\sec\alpha \cdot \cos(\phi-\alpha)}{\sqrt{\cos(\alpha-\delta)} + \sqrt{\dfrac{\sin(\phi-\alpha)\ \sin(\phi-\beta)}{\cos(\beta-\alpha)}}}\right]^2$$

Particular case

$$\alpha = \beta = \delta = 0$$

$$K_a = \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right)$$

$$\delta = \frac{\pi}{4} - \frac{\phi}{2}$$

The angle δ is an input parameter, therefore engineers will have to decide on its value for calculating the corresponding pressure coefficient. Subsection 3.7.3 gives criteria for evaluating this angle.

To help in pressure calculations with Coulomb's method, Table 3.7.2 is included, which shows the value of K_{ah}:

$$K_{ah} = K_a \cdot \cos(\sigma + \delta)$$

for different values for angles ϕ, α and β.

Coulomb's active earth pressure coefficient can always be applied for active pressure calculations.

The inclination of the failure line in the backfill resulting from Coulomb's theory for dry, homogeneous granular soil is given by the expression:

$$\tan(\zeta - \beta) = \frac{\cos\rho\sqrt{\cos(\alpha-\beta)\ \sin(\phi-\beta)}}{\sqrt{\cos(\alpha+\delta)\ \sin(\phi+\delta)} - \sin\rho\sqrt{\cos(\alpha-\beta)\ \sin(\phi-\beta)}}$$

where ρ is the auxiliary angle.

$$\rho = \phi + \alpha + \delta - \beta$$

Table 3.7.2. Values of K_{ah} according to Blum

φ / α	δ / β	20°					30°					40°				
		–20°	–10°	0°	+10°	+20°	–20°	–10°	0°	+10°	+20°	–20°	–10°	0°	+10°	+20°
+20°	+20°	1.132	1.132	1.132	1.132	1.132	0.798	0.708	0.646	0.595	0.547	0.555	0.488	0.454	0.416	0.380
	+10°	1.132	0.818	0.721	0.651	0.590	0.708	0.605	0.537	0.483	0.435	0.498	0.440	0.396	0.359	0.324
	0°	1.132	0.721	0.609	0.532	0.470	0.646	0.537	0.468	0.414	0.367	0.454	0.396	0.353	0.316	0.283
	-10°	1.132	0.651	0.532	0.455	0.394	0.595	0.483	0.414	0.362	0.317	0.416	0.359	0.316	0.281	0.250
	-20°	1.132	0.590	0.470	0.394	0.335	0.547	0.435	0.367	0.317	0.275	0.380	0.324	0.283	0.250	0.220
+10°	+20°	1.000	1.000	1.000	1.000	1.000	0.661	0.558	0.539	0.502	0.468	0.426	0.385	0.355	0.329	0.306
	+10°	1.000	0.726	0.644	0.588	0.541	0.588	0.505	0.454	0.413	0.378	0.385	0.344	0.313	0.288	0.265
	0°	1.000	0.644	0.551	0.489	0.440	0.539	0.454	0.402	0.361	0.326	0.355	0.313	0.283	0.259	0.237
	-10°	1.000	0.588	0.489	0.426	0.377	0.502	0.413	0.361	0.321	0.288	0.329	0.288	0.259	0.235	0.213
	-20°	1.000	0.541	0.440	0.377	0.300	0.468	0.378	0.326	0.288	0.256	0.306	0.265	0.237	0.213	0.193
0°	+20°	0.883	0.883	0.883	0.883	0.883	0.532	0.477	0.441	0.413	0.389	0.315	0.287	0.267	0.250	0.235
	+10°	0.883	0.638	0.569	0.523	0.486	0.477	0.413	0.374	0.344	0.321	0.287	0.258	0.238	0.221	0.207
	0°	0.883	0.569	0.490	0.440	0.401	0.441	0.374	0.333	0.304	0.279	0.267	0.238	0.217	0.201	0.187
	-10°	0.883	0.523	0.440	0.388	0.350	0.413	0.344	0.304	0.275	0.251	0.250	0.221	0.201	0.186	0.172
	-20°	0.883	0.486	0.401	0.350	0.311	0.389	0.321	0.279	0.251	0.227	0.235	0.207	0.187	0.172	0.159
–10°	+20°	0.773	0.773	0.773	0.773	0.773	0.421	0.375	0.348	0.327	0.311	0.219	0.200	0.187	0.177	0.169
	+10°	0.773	0.551	0.492	0.455	0.426	0.375	0.325	0.296	0.275	0.258	0.200	0.181	0.169	0.159	0.150
	0°	0.773	0.492	0.426	0.385	0.355	0.348	0.296	0.266	0.245	0.228	0.187	0.169	0.156	0.146	0.138
	-10°	0.773	0.455	0.385	0.344	0.313	0.327	0.275	0.245	0.224	0.208	0.177	0.159	0.146	0.137	0.128
	-20°	0.773	0.426	0.355	0.313	0.283	0.311	0.258	0.228	0.208	0.191	0.169	0.150	0.138	0.128	0.121
–20°	+20°	0.665	0.665	0.665	0.665	0.665	0.312	0.277	0.258	0.244	0.233	0.135	0.124	0.117	0.112	0.108
	+10°	0.665	0.456	0.412	0.382	0.360	0.277	0.240	0.220	0.206	0.195	0.124	0.114	0.107	0.101	0.097
	0°	0.665	0.412	0.357	0.325	0.302	0.258	0.220	0.199	0.185	0.174	0.117	0.107	0.100	0.095	0.090
	-10°	0.665	0.382	0.325	0.292	0.269	0.244	0.206	0.185	0.717	0.160	0.112	0.101	0.096	0.090	0.085
	-20°	0.665	0.360	0.302	0.269	0.246	0.233	0.195	0.174	0.160	0.150	0.108	0.097	0.090	0.085	0.081

In the particular case of vertical walls with a horizontal backfill ($\alpha = \beta = 0$), the above expression is simplified:

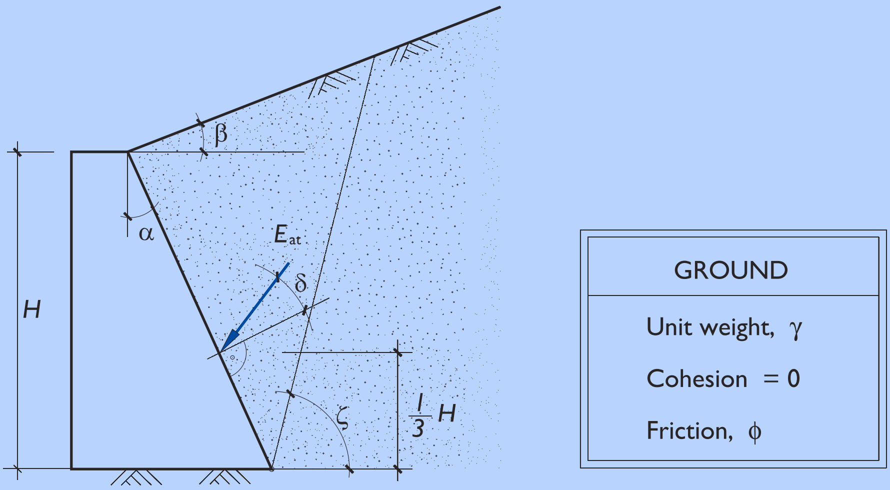
$$\tan\zeta = \tan\phi + \frac{1}{\cos\phi}\sqrt{\frac{\sin\phi\cos\delta}{\sin(\phi+\delta)}}$$

If, in addition, the direction of the earth pressure coincides with the normal to the back face ($\delta = 0$), the result would be:

$$\zeta = \frac{\pi}{4} + \frac{\phi}{2}$$

In cases where the design back face of the wall has been obtained by some simplification, so that the virtual line representing it lies substantially away from the real wall back face, it may be more advisable to evaluate the coefficient of earth pressure using Rankine's method, which is shown in Figure 3.7.13.

Figure 3.7.13. ***Problem Geometry and Rankine's Solution***



$$E_{at} = \frac{1}{2}\gamma K_a H^2 \qquad K_a = \frac{\cos(\alpha-\beta)}{\cos^2\alpha} \cdot \frac{\sqrt{1+\sin^2\phi - 2\sin\phi\cos\rho}}{\cos\beta\sqrt{\sin^2\phi-\sin^2\beta}} \qquad \tan\delta = \frac{\sin\phi\sin\rho}{1-\sin\phi\cos\rho}$$

Auxiliary angle,

$$\rho = 2\alpha - \beta + \arcsin\left(\frac{\sin\beta}{\sin\phi}\right)$$

PARTICULAR CASE OF WALLS WITH A VERTICAL BACK FACE $\alpha = 0$

$$K_a = \cos\beta\,\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

$$\delta = \beta$$

Rankine's method assumes a complete plastification by extension of the ground in the wall backfill and this implies a particular earth pressure inclination. This method is considered applicable provided that the direction of the earth pressure resulting from its application is compatible with the relative earth-wall movements that could occur in the design situation under analysis.

The inclination of the backfill failure line in a dry, uniform granular medium as deduced from Rankine's theory is:

$$\zeta = \frac{\pi}{4} + \frac{\phi}{2} - \frac{1}{2}\left[\arcsin\left(\frac{\sin\beta}{\sin\phi}\right) - \beta\right]$$

which, as can be seen, does not depend on the inclination of the wall back face.

For the particular case of horizontal ground, Rankine's method shows that:

$$\zeta = \frac{\pi}{4} + \frac{\phi}{2}$$

With these simple theories, not only can the coefficient of active earth pressure be estimated but also the approximate position of the failure line. In practical applications, it should not be forgotten that the inclination of this line is highly sensitive to other secondary effects. The existence of extensive overburdens will not affect the location of the failure line, but its inclination will however be affected by the presence of surcharges of limited extent, concentrated loads, the position of the groundwater table, the presence of cohesion, etc.

There are more complex calculation procedures that lead to slightly different values for the coefficient of active earth pressure. These include the method based on the theory of plasticity. One of these methods is described in Subsection 3.7.7 for calculating passive earth pressure, which shows how it could be used for calculating active earth pressure.

The differences between this latter method and Coulomb's method for calculating the coefficient of active earth pressure are generally minor ones and experience has not shown which of the two most closely matches reality. For this reason, applying the method based on the theory of plasticity for a weightless material is only considered to be useful in calculations for some special cases.

3.7.5.3 Calculating the Effective Earth Pressure Due to Ground Weight

When the back face is plane and the backfill is homogeneous, extending the concept of active earth pressure leads directly to a determination of the pressures. This is illustrated in Subsection 3.7.5.1.

The most general case of a broken back face and different ground types can be solved by generalising the application of the concept of of earth pressure coefficient to a somewhat greater extent.

On a particular wall with a back face *ABCD* as shown in Figure 3.7.14, the horizontal earth pressure due to the effective weight of the earth can be calculated by the procedure shown in the same figure.

The input data for the problem will be the strengths of the different types of ground, the inclinations of the different sections of the wall, the design specific weights and the vertical hydraulic gradient (the latter obtained as shown in the following section).

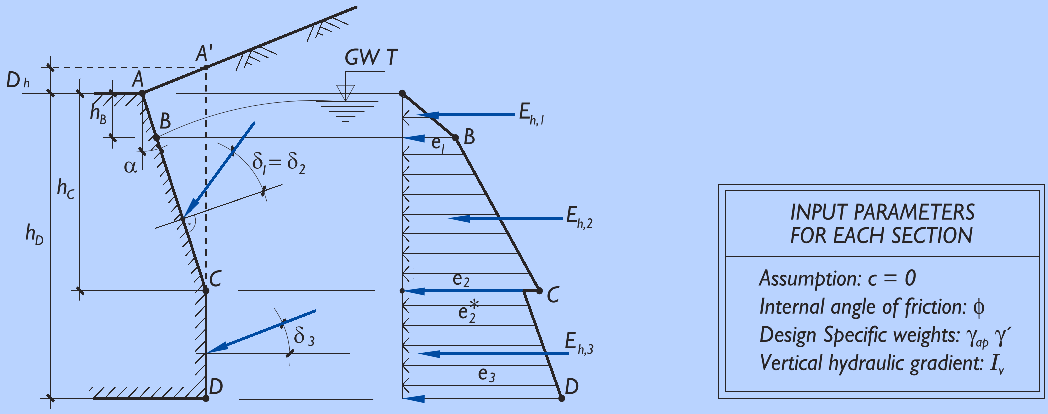
The first calculation step determines the values for the coefficient of horizontal earth pressure corresponding to the different sections of the wall as a function of the internal angle of friction of the ground in each section and of the corresponding α and δ angles.

The second calculation step determines the values for the reference vertical effective pressures. In this calculation, the apparent densities should be used above the groundwater table (between *A* and *B*) and the submerged densities below this level. The submerged specific weights should be increased by $\gamma_w\ I_v$ to take into account the potential vertical gradient of the water movement, as shown in the following section.

To measure the design depths, the reference level will be the point where the extension of the wall back face cuts across the surface of the ground. This is Point *A* for Section *AC* and Point *A'* for Section *CD* in Figure 3.7.14.

The third step provides the horizontal components of the unitary earth pressures at each point of the wall. At the break points where the wall face changes its direction or at the points where the ground strength changes, two values for the pressure should be obtained, one from the calculation as the lower point in the top section and the other as the higher point in the next section. The two values may be different.

Figure 3.7.14. ***Calculating Horizontal Active Earth Pressures Due to Effective Ground Weight***



1. CALCULATION OF HORIZONTAL EARTH PRESSURE COEFFICIENTS, $(K_{ah})_i^*$ and $(K_{ah})_3$

2. CALCULATION OF VERTICAL EFFECTIVE PRESSURES

$\sigma'_B = \gamma_{ap}\, h_B$ $\qquad$ $\sigma'_C = \sigma'_B + (\gamma' + \gamma_w I_{v1})\,(h_C - h_B)$

$\sigma'^{*}_B = \sigma'_c + \gamma_{ap}\, \Delta h$ $\qquad$ $\sigma'_D = \sigma'_C + (\gamma' + \gamma_w I_{v2})\,(h_D - h_C)$

3. CALCULATION OF HORIZONTAL UNITARY EARTH PRESSURES

$e_1 = (K_{ah})_1\ \sigma'_B \cos\alpha$ $\qquad$ $e_2 = (K_{ah})_1\ \sigma'_C \cos\alpha$

$e^*_2 = (K_{ah})_3\ \sigma'^{*}_C$ $\qquad$ $e_3 = (K_{ah})_3\ \sigma'_D$

4. CALCULATION OF HORIZONTAL EARTH PRESSURES

$$E_{h,1} = \frac{1}{2}\, e_1 \frac{h_B}{\cos\alpha} \qquad E_{h,2} = \frac{1}{2}\,(e_1 + e_2)\,\frac{h_C - h_B}{\cos\alpha} \qquad E_{h,3} = \frac{1}{2}\,(e^*_2 + e_3)(h_D - h_C)$$

N.B.: In this example, K_{ah} is assumed to be constant on the *AC* section because the ground and the inclination of the wall in this section are uniform.

The fourth step consists in integrating the horizontal unitary pressure distributions over each section in order to obtain the corresponding horizontal earth pressures and their lines of action.

When the vertical component of the earth pressure on each section is also needed, it suffices with multiplying the value of the horizontal earth pressure by the tangent of the corresponding angle of inclination for that section, that is:

$$E_{v,i} = E_{h,i}\, \tan(\alpha + \delta)_i$$

This vertical earth pressure will point downwards if it has a positive sign and will intersect the back face of the wall at the same point as the associated horizontal earth pressure.

Calculating the water pressure, considering the effect of potential backfill cohesion and estimating the effect of overburdens will enable a complete calculation of earth pressures on the wall to be made. The following sections give some recommendations designed to help engineers in these calculations.

3.7.5.4 Water Pressures

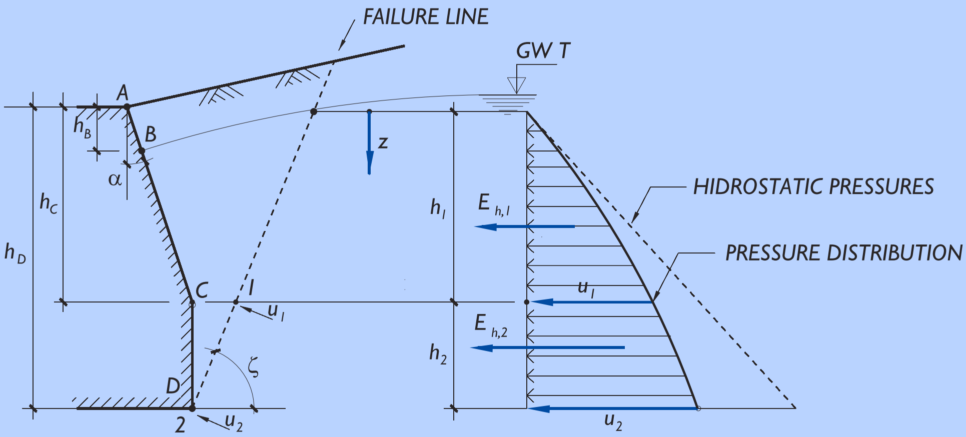
The simplified method based on earth pressure coefficients requires the water effect to be calculated separately. The method chosen to calculate pressure due to the effective earth weight should be consistent with the method for estimating the complementary water pressure. The procedure described below should be applied when the one described in the previous section is used simultaneously to evaluate the part of the pressure caused by the effective earth weight.

The state of the water in the backfill must be defined by the distribution of porewater pressure along a line passing through the heel of the wall and forming an angle of 45° + ϕ/2 from the horizontal. For this purpose, the angle of friction for the weakest material in the backfill should be used.

This pressure distribution should be obtained after calculating the corresponding flownet, to which end the recommendations given in Section 3.4 should be followed.

Horizontal porewater pressure on the wall should be calculated following the procedure illustrated in Figure 3.7.15.

Figure 3.7.15. Calculating Horizontal Water Pressure



INPUT DATA

Porewater pressures on the failure line

CALCULATION OF THE DIRECT HORIZONTAL PRESSURE

$$E_{h,1} = \frac{1}{2} u_1 h_1 \qquad E_{h,2} = \frac{1}{2} (u_1 + u_2)\, h_2$$

CALCULATION OF THE INDIRECT HORIZONTAL PRESSURE

Vertical gradient

Section BC $$(I_v)_1 = \frac{\gamma_w\ h_1 - u_1}{\gamma_w\ (h_C - h_B)}$$

Section CD $$(I_v)_2 = \frac{u_1 - (u_2 - \gamma_w h_2)}{\gamma_w\ h_2}$$

N.B.: For the purpose of this calculation, the failure surface can be assumed to be plane and defined by the following angle: ζ = 45° + ϕ/2 where ϕ is the angle of friction of the weakest ground in the backfill.

This figure attempts to show that the horizontal pressure due to the water on each calculation section is obtained by direct integration of the pressure distribution over Sections h_1 and h_2. In the figure, this integration is done assuming a linear variation of pressure in each section.

The vertical component of the water pressure is deduced by multiplying these horizontal pressures by the tangent of the angle of the wall back face to the vertical in the corresponding section, that is:

$$E_{v,i} = (\tan\alpha)_i\, E_{h,i}$$

This calculation not only provides the value for the horizontal water pressure but also defines its line of action.

As indicated, the resultant water pressure (the sum of its horizontal and vertical components) will be normal to the wall face. Its line of action will intersect it at the same point as the corresponding horizontal component.

Furthermore, the indirect effect produced by the vertical gradient due to water flow must be taken into account. To this end, these gradients should be calculated by sections as shown in the figure referred to. These gradients are needed for obtaining the active earth pressure due to the effective weight of the ground as described in the previous section.

3.7.5.5 The Effect of Cohesion

Backfills for earth retaining structures will normally be granular and therefore the effect of cohesion should be taken into account for only a few active earth pressure calculations.

In cases where there is cohesive soil in the backfill of the retaining works, the effect of cohesion should be taken into account with certain limitations.

Cohesion has always a beneficial effect, since it reduces active earth pressure. Cohesion, however, is one component of ground strength that can be lost in the long run, either through a loosening of the ground structure (swelling caused by wetting or saturation), or by cracking that can occur as a result of displacements in the wall or climatic changes (cracking due to desiccation).

Having investigated the value for the cohesion that could reasonably exist in the material of the wall backfill at the time corresponding to the design situation under analysis, its effect on the active earth pressure can be estimated as indicated below.

The effect of cohesion on the strength of a material that also possesses friction ($\phi \neq 0$) is equivalent to the improvement obtained when compressing the ground mass with an isotropic stress of the following intensity:

$$q = \frac{c}{\tan\phi}$$

In fact, a capillary suction of this intensity, *q*, provides unsaturated moist sand with an apparent cohesion governed by the above expression.

Based on this basic principle and using the simplified theory of plasticity as a calculation model (described in more detail in Subsection 3.7.7.2), the value of the coefficient of active earth pressure, K_{ac}, due to cohesion can be derived. Its horizontal component is:

$$K_{ach} = K_{ac} \cdot \cos(\alpha + \delta)$$

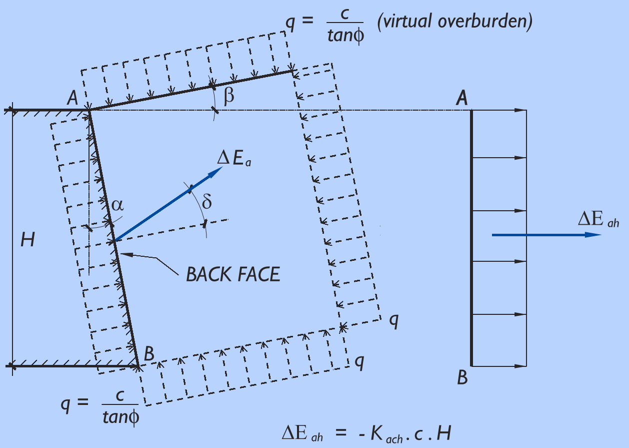
The value of K_{ach} can be estimated using the formulae shown in Figure 3.7.16.

The angle δ corresponding to the reduction in earth pressure as a result of cohesion must be the same as the one defined in Subsection 3.7.3.3 and which is used to calculate pressure owing to the effective earth weight.

These expressions use the auxiliary angle Ψ, whose physical meaning can be seen in the theoretical explanation given in Subsection 3.7.7.2. This angle must be positive, since the geometry of the failure assumed in order to deduce the formulae will no longer be valid for negative values of this angle.

However, when Ψ is negative, a sufficiently close approximation of K_{ach} can be obtained by using the alternative expression shown in the same Figure 3.7.16, deduced from calculating earth pressures on the assumption that the failure surface is a plane.

Figure 3.7.16. **Effect of Cohesion on Horizontal Active Earth Pressure**



$$\Delta E_{ah} = -K_{ach} \cdot c \cdot H$$

GENERAL CASE

Auxiliary angles: $\eta = \arcsin\left(\dfrac{\sin\delta}{\sin\phi}\right) - \delta$

$$\psi = \beta - \alpha + \frac{1}{2}\eta$$

$$K_{ach} = \frac{\cos(\delta+\alpha)}{\cos\alpha}\left[1 - \frac{1-\sin\phi\cos\eta}{\cos\delta\,(1+\sin\phi)}\,e^{-2\psi\tan\phi}\right]\frac{1}{\tan\phi} \qquad (\text{for } \psi > 0)$$

$$K_{ach} = 2 \cdot \frac{\cos(\delta+\alpha)}{\cos\alpha} \cdot \frac{\cos(\beta-\alpha) \cdot \cos\phi}{1+\sin(\phi+\alpha+\delta+\beta)} \qquad (\text{for } \psi < 0)$$

PARTICULAR CASE $\alpha = \beta = \delta = 0$

$$K_{ach} = 2 \cdot \tan\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$$

CASE OF GROUND WITH $\phi = 0$

$$K_{ach} = 2\,(1 + \beta - \alpha) \text{ if } \beta > \alpha$$

$$K_{ach} = 2\tan\left(\frac{\pi}{4} - \frac{\alpha-\beta}{2}\right) \text{ if } \beta < \alpha$$

N.B.: The angles are expressed in radians.

In cases in which the strength of the backfill material is wholly due to cohesion, i.e., when the design angle of friction is $\phi = 0$, a few of the formulae referred to will lead to an undetermined situation. By studying their limit values, however, the formulae to be applied in this particular case can be deduced. These analytical expressions are shown in the bottom section of Figure 3.7.16.

When considering the effect of cohesion in unitary earth pressure calculations, negative values in the area close to the head of the wall can result, after they have been composed with the unitary earth pressures due to other effects. Although these values may be theoretically correct, they should be replaced by null values. It is not recommended to admit such tensile stresses.

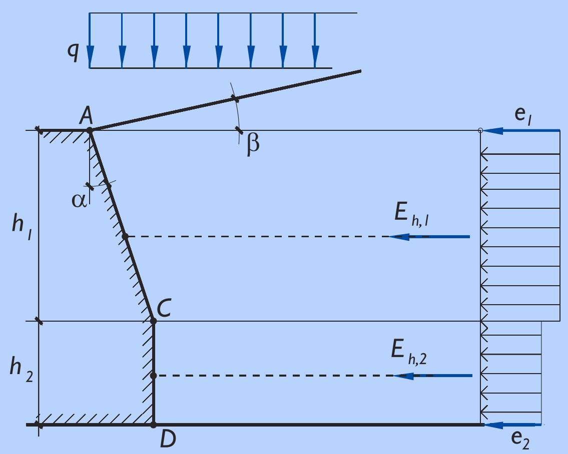
3.7.5.6 Effect of Overburdens

Surcharges acting on the surface of a wall backfill increase the active earth pressure. Guidelines on the procedure for evaluating the additional pressures caused by overburdens are found below.

3.7.5.6.1 Extensive Vertical Uniform Overburdens

The simplest overburden from the point of view of calculation and also fairly common in port works is a uniform surcharge acting over an extensive area. The intensity of this overburden is defined as the total load (in kN) acting over each square metre in plan projection. The effect of this surcharge on the active earth pressure can be considered equivalent to the one producing an additional unitary earth pressure on the back face of the wall face as that illustrated in Figure 3.7.17.

Figure 3.7.17. ***Effect of a Uniform Overburden on Horizontal Active Earth Pressure***



UNITARY EARTH PRESSURES

$$e_1 = (K_{ah})_1 \cdot \frac{\cos^2\alpha \cos\beta}{\cos(\beta-\alpha)} \cdot q$$

$$e_2 = (K_{ah})_2 \cdot q$$

EARTH PRESSURES PER SECTION

$$E_{h,1} = e_1 \cdot \frac{h_1}{\cos\alpha}$$

$$E_{h,2} = e_2 \cdot h_2$$

Nota: See text for definition of K_{ah}.
Overburden *q* is vertical and defined as acting on the area of the plan projection.

This figure shows only the value for the horizontal component of the unitary earth pressure how to obtain the horizontal resultant of the pressure on each section of the wall.

The vertical components of the earth pressures due to this type of surcharge can be obtained by multiplying the values of the horizontal earth pressures by the tangents of the corresponding angles of inclination in each section, that is,

$$E_{v_i} = E_{h_i} \cdot \tan(\alpha + \delta)_i$$

3.7.5.6.2 Uniform Vertical Overburdens of Limited Extent

The vertical uniform overburdens referred to in the previous section may have limited dimensions, either perpendicular to the head of the wall or parallel to the wall.

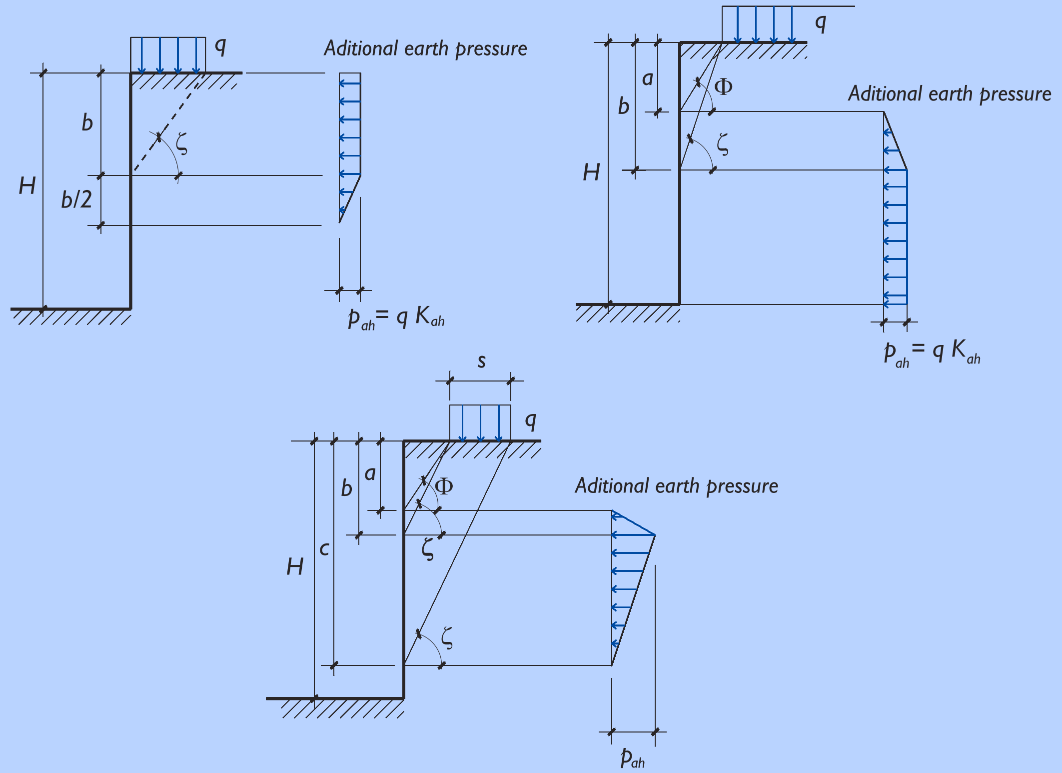
When these loads extend indefinitely along the wall but are limited in a direction normal to it, the problem of calculating the earth pressure continues to be two-dimensional and can be solved using the simplifying assumptions illustrated in Figure 3.7.18.

This figure shows only the value of the horizontal component of the earth pressure. Previous knowledge of the corresponding angle δ will enable the associated vertical component to be calculated.

It is worth noting that, because this is a problem of limit equilibrium and considering the simplifications implicit in Figure 3.7.18, the effects of surcharges cannot be strictly overlaid. In other words, the earth pressure caused by the sum of two overburdens is different from (normally less than) the sum of their partial effects, when considered individually.

For this reason, using the diagrams in the figure is recommended only for adding up the effects of different surcharges, since this will generally lead to conservative results. These solutions must not, however, be used to subtract the effect of overburdens.

Figure 3.7.18. Effect of Partial Overburdens on Horizontal Active Earth Pressure



$$p_{ah} = \frac{2.q.s.\theta}{c - a} \qquad \text{for } \theta = \frac{\sin(\zeta - \phi)\cos\delta}{\cos(\zeta - \phi - \delta)}$$

ϕ : *Internal angle of friction.*

ζ : *Angle of the surface corresponding to the active earth pressure (see 3.7.5.2)*

This latter situation could arise, for instance, in cases where a particular zone is subjected to a lower overburden and the engineer decides to tackle the calculation in two stages; firstly assuming an uniform and constant intensity over the whole of the zone and afterwards subtracting the local overburden deficit as a negative overburden. This procedure is not recommended, as it will lead to optimistic results, i.e., erring on the unsafe side.

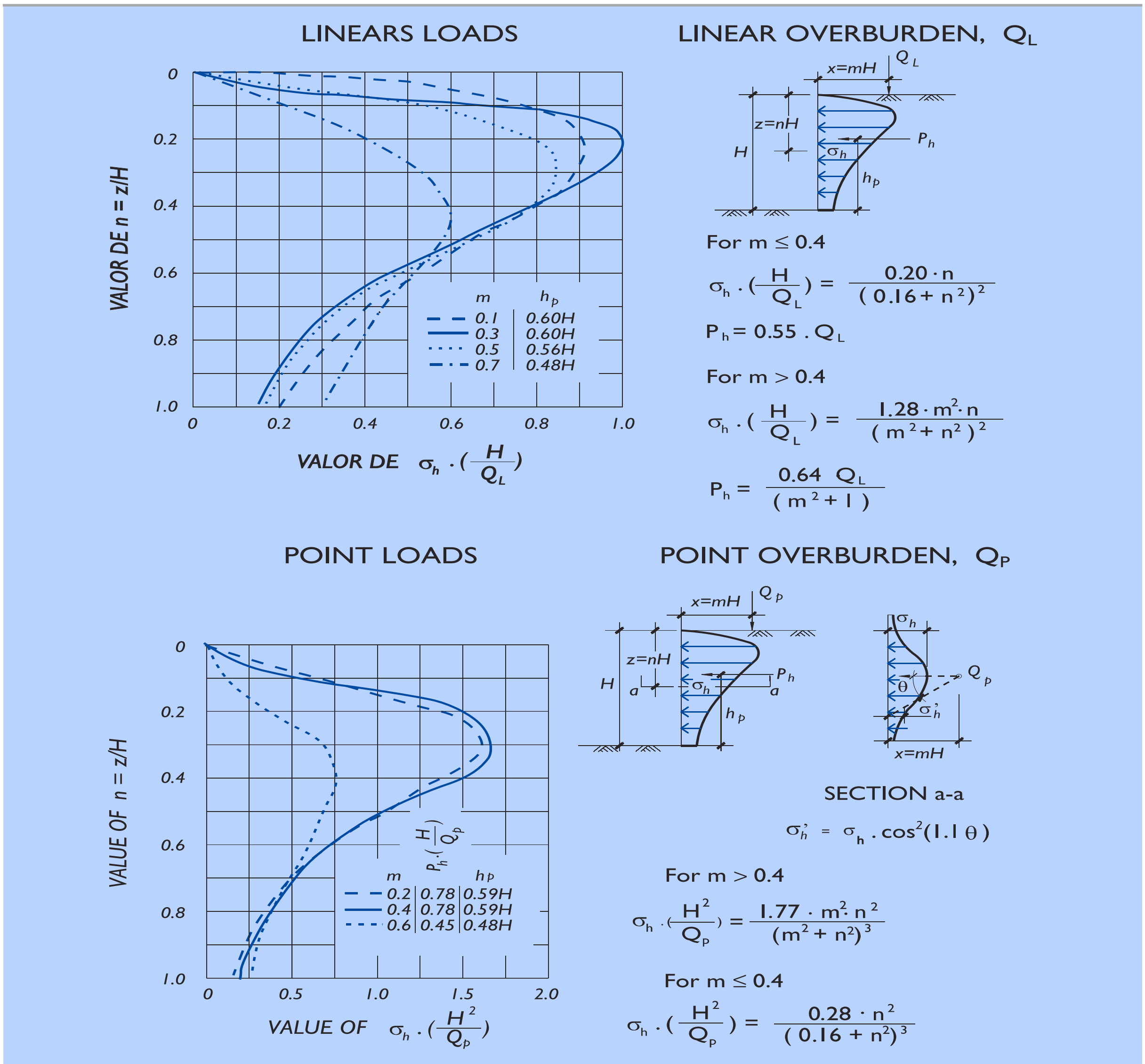
3.7.5.6.3 Linear or Concentrated Vertical Overburdens

Surcharges acting along lines parallel to the head of the wall increase the active earth pressure on it. The additional horizontal unitary pressure they bring about and its resultant can be estimated using the procedure shown in Figure 3.7.19.

The bottom section of this figure illustrates the increase in horizontal earth pressure caused by a concentrated vertical overburden.

In both cases, the solution is obtained from certain considerations based on the theory of elasticity. The solutions shown do not depend on the strength of the backfill material nor on the possible location of the failure line. As a result, superimposing the effects is fully consistent and engineers can compound the effect of the real overburdens existing by adding and subtracting these elementary cases.

Figure 3.7.19. **Effect of Linear or Concentrated Loads on Horizontal Active Earth Pressure**



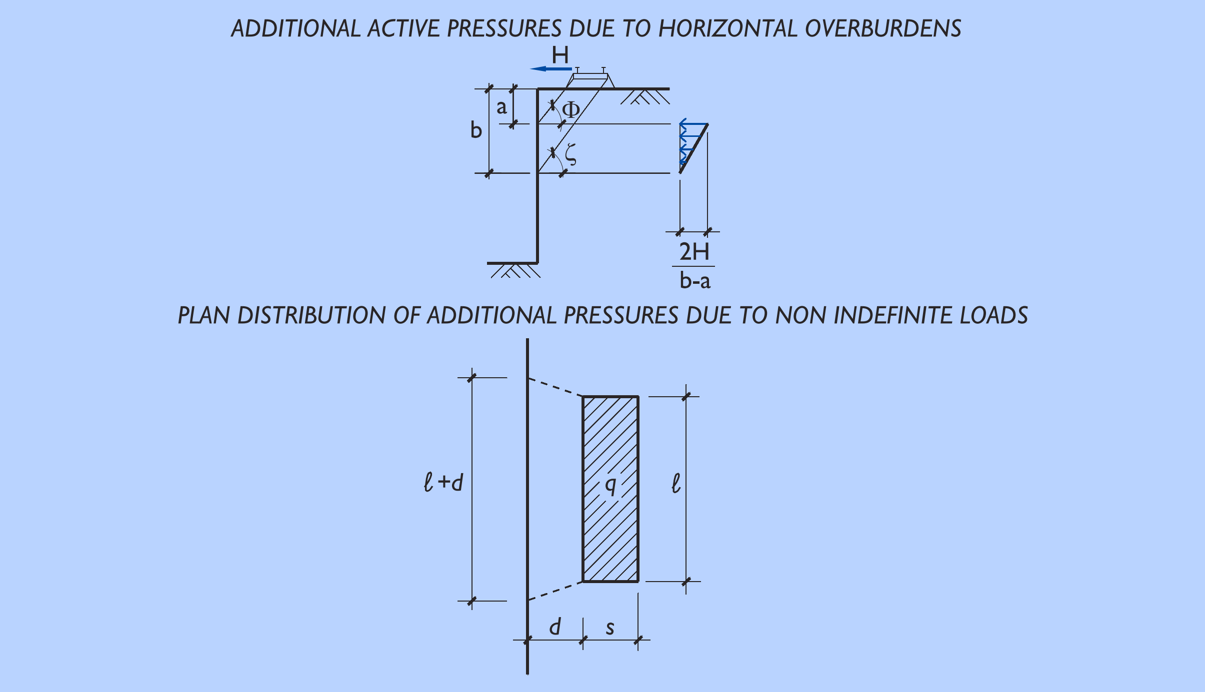
3.7.5.6.4 Other Overburdens

Horizontal surcharges, whether concentrated or distributed linearly, are common in the vicinity of gravity quays. It is fundamental that they are taken into account when calculating active earth pressure, as their action is wholly transmitted to the wall when they lie within the failure wedge.

A simplified procedure for taking their effect into account is shown in Figure 3.7.20.

Surcharges with a limited extent in the direction of the wall can be considered as if they had an indefinite length in calculations, since this simplification leads to results erring on the safe side. In cases where the length of the overburdened area is relatively short compared with the wall dimensions (less than the distance between two structural joints, for example), it is admissible to assume that the load is distributed on a section of the wall face that is longer than the length of the surcharge itself. This increase in length must in any event be equal to or less than the shortest distance from the overburden to the wall face. The figure exemplifies this for a vertical uniform surcharge that is bounded in both directions. The same idea is considered applicable to any other type of overburden limited longitudinally.

Figure 3.7.20. ***Effects of Other Overburdens***



Nota: ϕ and ζ as defined in Figure. 3.7.18.

3.7.6 General Method for Calculating Passive Earth Pressure

Passive earth pressure of the ground against the retaining structure occurs when their relative movement brings them nearer together.

Passive earth pressure occurs in walls having to withstand substantial external loads such as berthing pressures on piled open quays (back wall on the land side). It also occurs in buried structures built to support the pull of anchors. In a diaphragm wall, passive earth pressure is partly mobilised in the embedded zone at its toe.

Calculation of passive earth pressure is in many respects similar to calculation of active earth pressure. In fact, it would suffice to change the sign of the angle of friction and of the cohesion of the ground for the formulae associated with the calculation of active earth pressure to be valid in this other situation.

Calculation of passive earth pressure does however have certain particular features, which should be highlighted and are shown in the following sections.

3.7.6.1 Data Required

The data required for calculating passive earth pressure between a structure and its backfill are the same as the data shown in Section 3.7.4 for calculating active earth pressure. The essential difference between the two calculations lies in the fact that the sensitivity of the results to the input data can have the opposite sign in the two cases.

In calculating active earth pressure it is advisable to make a reasonable and conservative estimate of the ground strength (cohesion and friction). The same holds true for the case of passive earth pressure, since the greater the strength assumed, the higher will be the passive earth pressure.

This does not hold for the weight of the ground or the intensity of the overburdens. The greater they are, the more optimistic the evaluation of passive earth pressure will prove to be (higher values). For this reason, weight needs to be estimated cautiously and only the loads that are really present in the design situation under analysis should be taken into account.

Something similar occurs with the height of the wall. If in calculating active earth pressure, the tallest sections should be considered, the opposite must be done in the case of passive earth pressure.

In works where both active and passive earth pressures (diaphragm walls, for instance) are involved in the analysis at the same time, different vertical sections along the works will have to be chosen. The tallest section may not be the most critical.

The inclination of the earth pressure with respect to the normal to the face, measured by the angle δ, will be conditioned by the external restraints that will force a certain type of relative wall-ground sliding to occur.

This angle will generally be negative, which implies a considerable increase in the pressure value. The absolute value of δ should be limited as shown in 3.7.3.3.

3.7.6.2 Failure Line

In the limit equilibrium, when the wall causes the ground failure, a slip surface is developed (a slip line in plane problems), which forms a much smaller angle with the horizontal than in the case of active earth pressure.

The failure line is therefore much longer than the one corresponding to active earth pressure and, furthermore, its slope angle varies more along its length.

To calculate passive earth pressures, failure lines should be assumed to have a variable and increasing inclination from the heel of the wall to the point where they emerge at ground surface.

Different trial lines should be calculated, and they can be polygonal. These lines can change direction at their points of intersection with surfaces separating different ground types, with the unconfined water table and at some intermediate auxiliary points allowing accuracy to be improved.

3.7.6.3 Calculating Passive Earth Pressure

Passive earth pressure should be calculated by a similar procedure to the one described in 3.7.4 for calculating active earth pressure, simply by changing the sign of the forces due to cohesion and friction.

Figure 3.7.21 shows the diagram of the forces for calculating equilibrium of a slice in the failure zone corresponding to passive earth pressure. The homologous diagram for active earth pressure is the one shown in Figure 3.7.9.

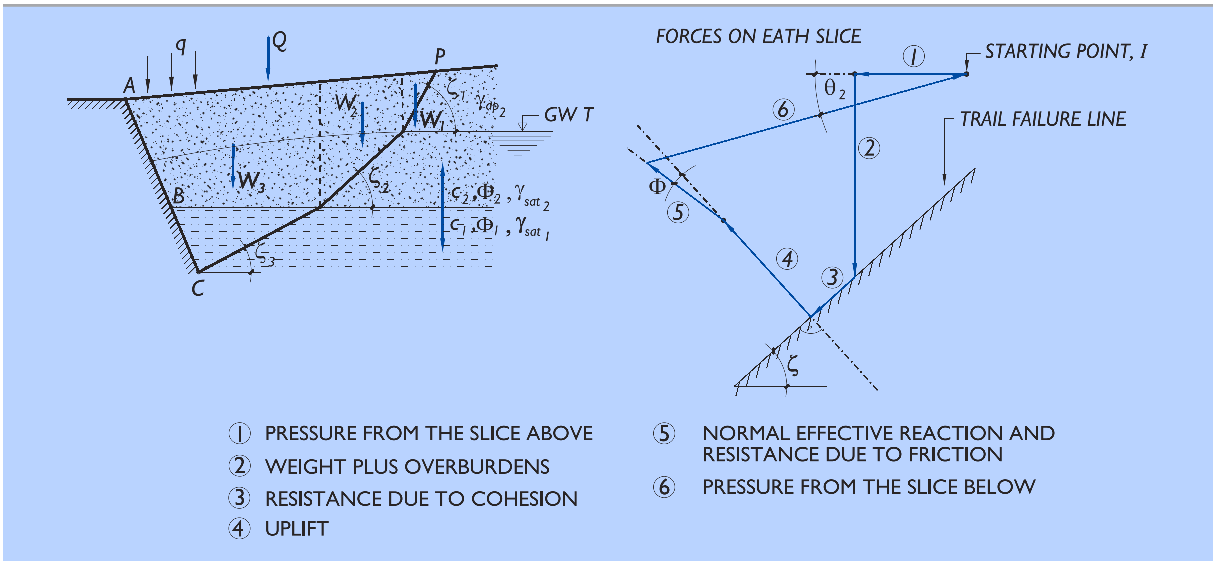
The passive earth pressure for a particular failure line must be calculated beginning with the upper wedge, assuming that the pressure between this wedge and the next one is horizontal.

This hypothesis of horizontal pressure between slices is advisable, except in the case of the slice next to the wall, where an inclination corresponding to the angle δ, as previously defined, must be assumed.

Passive earth pressure can be calculated using saturated densities and considering uplifts on the failure line, or using submerged densities as shown in 3.7.4.6 and then adding the pressure of the water, calculated separately.

In calculating passive earth pressures, the downward water gradient is favourable (it produces higher pressures). An upward gradient can be extremely unfavourable, as it can substantially reduce the passive earth pressure. This situation, which can occur in the embedded area of a diaphragm wall, requires the engineer's full attention.

Figure 3.7.21. **General Method for Calculating Passive Earth Pressure**



3.7.7 Methods Based on the Coefficient of Passive Earth Pressure

Calculating passive earth pressure by the general method described above can be a tedious process, as several failure lines passing through the heel of the wall have to be checked. If, in addition, the point of application of the pressure needs to be known, it will be necessary to repeat this process considering several virtual wall heels placed at different points along the back face.

For this reason, a simple alternative procedure is shown below that is generally sufficiently precise, based on the concept of the passive earth pressure coefficient.

3.7.7.1 Concept of the Passive Earth Pressure Coefficient

The concept of the passive earth pressure coefficient is defined in sketch form in Figure 3.7.22.

Passive earth pressure coefficient refers to the following quotient in this ROM 0.5:

$$K_p = \frac{e}{\sigma'_v \cos\alpha}$$

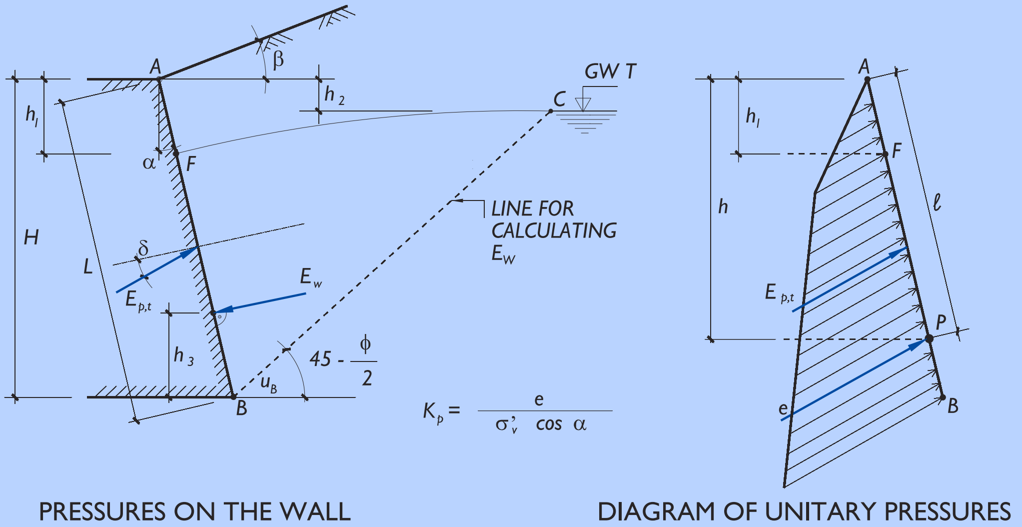
where:

e = effective earth pressure against the wall when failure occurs.
σ'_v = the reference vertical effective pressure.
α = deviation from the vertical of the wall face. This is positive when the backfill overhangs the wall.

The reference vertical pressure used here is the same as the one defined in Subsection 3.7.5.1.

As defining the reference vertical pressure involves a particular way of taking the effect of the water into account, this very same procedure should be followed when subsequently evaluating the pressure due to porewater.

Figure 3.7.22. ***Definition of the Passive Earth Pressure Coefficient in Granular Soils***



e = unitary effective pressure due to weight of the earth at a point P on the wall back face in the passive situation

σ'_v = the reference vertical effective pressure on P (see text)

α = angle formed by the wall back face to the vertical, which is positive when the backfill overhangs teh wall, as in the figure

ϕ = internal angle of friction of the earth

$E_{t,p}$ = passive pressure due to the effective earth weight

E_w = pore water pressure (see text)

u_B = pore water pressure on the heel of the wall

BC = the reference line for calculating pore water pressure

3.7.7.2 Value of the Coefficient of Passive Earth Pressure

The coefficient of passive earth pressure is not a ground property but a design parameter mainly governed by the shear strength of the ground and also depending on the characteristic angles defining the orientation of the wall face, the direction of the pressure and the inclination of the ground.

The basic idea behind the method for evaluating the passive earth pressure coefficient is the theory of plasticity, the simplest expression of which is illustrated in Figure 3.7.23.

This same theory is used for analysing the bearing capacity of shallow foundations and for deducing the bearing capacity factors referred to in Section 3.5. With this theoretical basis and extrapolating from theory to reality in a way similar to the one used in the case of bearing capacities, the *coefficient of horizontal passive earth pressure* is defined in Figure 3.7.24 as:

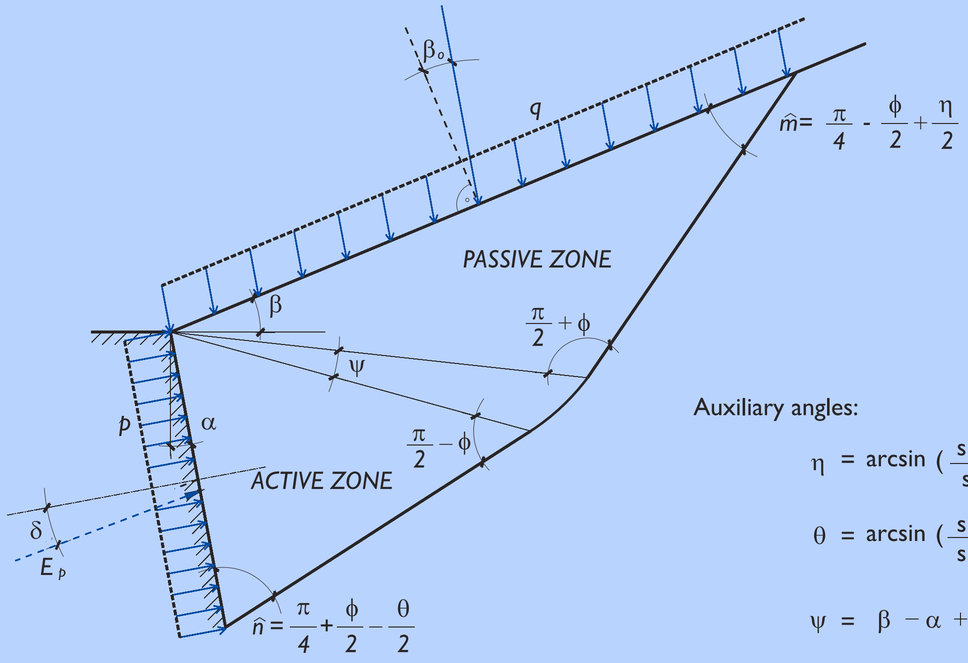
$$K_{ph} = K_p \cdot \cos(\alpha + \delta)$$

Its recommended values are shown in the same figure.

The Rankine and Coulomb methods, used for calculating the coefficient of active earth pressure, also enable the coefficient of passive earth pressure to be obtained by simply changing the sign of the internal angle of friction.

There are reasons, however, for not applying in a generalized manner the theory of plasticity referred to in the calculation of active earth pressures (already given in Subsection 3.7.5.2). There are also reasons for not systematically using either Coulomb's method or Rankine's method for calculating the coefficient of passive earth pressure. These reasons are set out below.

Figure 3.7.23. **Simplified Theory of Plasticity Applied to Passive Earth Pressure Calculations**



Auxiliary angles:

$$\eta = \arcsin\left(\frac{\sin\beta_o}{\sin\phi}\right) - \beta_o$$

$$\theta = \arcsin\left(\frac{\sin\delta}{\sin\phi}\right) + \delta$$

$$\psi = \beta - \alpha + \frac{1}{2}(\eta - \theta)$$

Basic coefficient:

$$R = \frac{p\cos\delta}{q\cos\beta_o} = \frac{p_n}{q_n} = \frac{1 + \sin\phi\cos\theta}{1 - \sin\phi\cos\eta} \cdot e^{2\psi\tan\phi} \quad (\text{for } \psi > 0)$$

As mentioned, the slip surface observed in passive failures is far from being plane, yet both the Coulomb and Rankine methods imply plane failure.

In the same way, the theory of plasticity could be used to calculate active earth pressures. The same analytical expression that gives the passive earth pressure coefficient will be valid by simply changing the sign of the internal angle of friction to obtain the coefficient of active earth pressure.

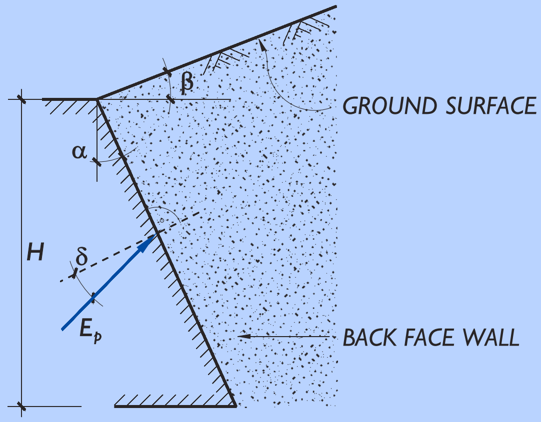
Passive earth pressure is usually associated with negative δ angles and this results in failure lines that are distinctly curved. In cases where passive earth pressures should be calculated with small δ inclinations, either of these two methods (Rankine or Coulomb) can be applied. Furthermore, when the central angle of the plastic radial wedge (angle ψ in Fig. 3.7.23) is negative, it is advisable to use one of these methods, since the geometrical model for plastic failure ceases to be valid.

In general terms, Coulomb's method is considered to be applicable provided that the angle of inclination of the pressure with respect to the wall normal is zero and the backfill surface is practically horizontal. Coulomb's method (or Rankine's) should be applied when the angle δ is positive and the wall back face and the ground form a right angle or less, i.e., when $\alpha > \beta$ and at the same time $\delta > 0$.

A series of diagrams are given here to make it easier to calculate passive earth pressure, showing values for the horizontal coefficient of passive earth pressure, K_{ph}, deduced from the theory of plasticity.

Figure 3.7.25 represents values for K_{ph} on the horizontal axis versus the value of the friction angle ϕ on the vertical axis, for several values of the angle δ of deviation of the pressure from the normal to the wall. The data in this figure correspond to walls with a vertical back face ($\alpha = 0$) and backfills with a horizontal surface ($\beta = 0$).

Figure 3.7.24. ***Coefficients of Passive Earth Pressure***



$$E_{ph} = E_p \cdot \cos(\alpha+\delta) = \frac{1}{2}\,\gamma\, K_{ph} H^2$$

SIMPLIFIED THEORY OF PLASTICITY

$K_{ph} = R \,.\, \cos\beta \,.\, \cos(\beta+\delta)$ **(R definido en Fig. 3.7.23)**

RANKINE´S THEORY

Auxiliary angles:

$$\rho = \arcsin\frac{\sin\beta}{\sin\phi} + \beta - 2\alpha$$

$$\cos\omega = \frac{\cos\phi}{\cos\beta}$$

$$K_{ph} = \frac{\cos(\beta-\alpha)\cos(\alpha+\delta)}{\cos^2\alpha\cos\beta}\;\frac{\sqrt{1+\sin^2\phi + 2\sin\phi\cos\rho}}{1-\sin\omega}$$

$$\tan\delta = \frac{\sin\phi\,\sin\rho}{1+\sin\phi\cos\rho}$$

COULOM´S THEORY

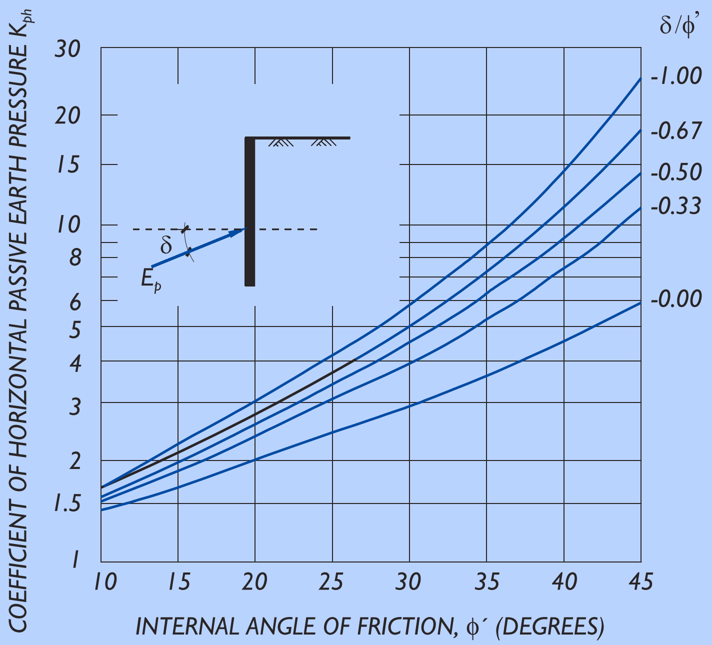
$$K_{ph} = \frac{1}{\cos^2\alpha}\left[\frac{\cos(\alpha+\phi)}{1-\sqrt{\dfrac{\sin(\phi-\delta)\;\sin(\phi+\beta)}{\cos(\alpha+\delta)\;\cos(\beta-\alpha)}}}\right]^2$$

Como puede observarse, únicamente se consideran ángulos "δ" negativos (como caso extremo se incluye δ = 0). Para valores positivos del ángulo "δ" esta teoría no es aplicable, y sería necesario aplicar la fórmula del método de Coulomb o de Rankine. De entre estas últimas se debe elegir una u otra con criterio semejante al que se cita en 3.7.5.2 al comparar estos métodos.

As can be seen, only negative values of the angle δ have been considered (including δ = 0 as a limit case). This theory is not applicable for positive values of angle δ. For these cases, it will be necessary to apply the formulae from Coulomb's or Rankine's methods. One or the other should be chosen upon similar criteria to the ones referred to in 3.7.5.2 where these methods are compared.

Figure 3.7.26 shows values of the horizontal passive earth pressure coefficient, K_{ph}, for vertical walls pushing against sloping ground. Positive and negative ground inclinations are included in these figures. This method will no longer be applicable for smaller values of the angle β than those shown in the figure. In that case, the Coulomb or Rankine solution would have to be used.

Figure 3.7.25. **Coefficient of Horizontal Passive Earth Pressure, K_{ph}, for Vertical Walls and Horizontal Backfills. Theory of Plasticity**



N.B.: Only negative values of angle δ are included. See text for positive values.

3.7.7.3 Calculating Passive Earth Pressure

Calculating passive earth pressure by the procedure based on the concept of passive earth pressure coefficient described in this Subsection 3.7.7 requires that several components are considered separately. These are the effective pressure due to the earth weight, the water pressure, the increase in earth pressure due to cohesion and the increase in earth pressure due to surcharges.

The effective pressure due to the weight of the ground can be calculated by integrating the distribution of horizontal components of unitary earth pressure along the face of the wall. The calculation is in all respects similar to the one described in Subsection 3.7.5.3 for horizontal active earth pressures but simply using K_{ph} instead of K_{ah}.

Once the horizontal component of the earth pressure is known, its vertical component can be determined, since the orientation of the pressure is known.

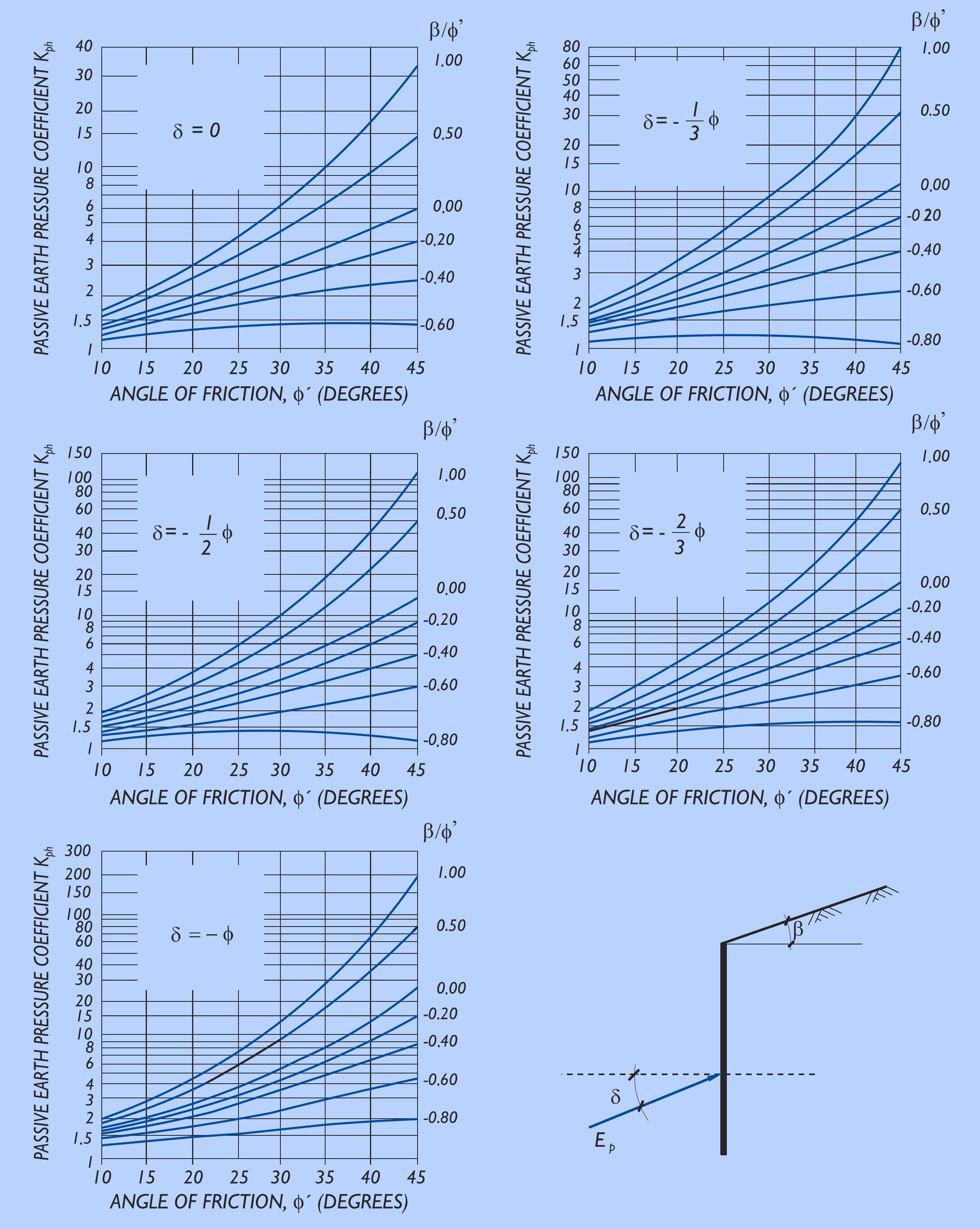
The pressure due to porewater is obtained by integrating the distribution of porewater pressure on the reference line, *BC*, which passes through the heel of the wall *B* and intersects the groundwater table at point *C* (see Fig. 3.7.22). It should be assumed that this line is inclined at an angle of 45° - (φ/2) to the horizontal. Where several types of ground are involved, it would be conservative in these calculations to take the highest value of φ.

Except for the inclination of this line *AC*, the calculation of the water pressure is in all respects identical to the one given in Subsection 3.7.5.4.

The increase in earth pressure due to cohesion can be estimated as shown in the diagram in Figure 3.7.27.

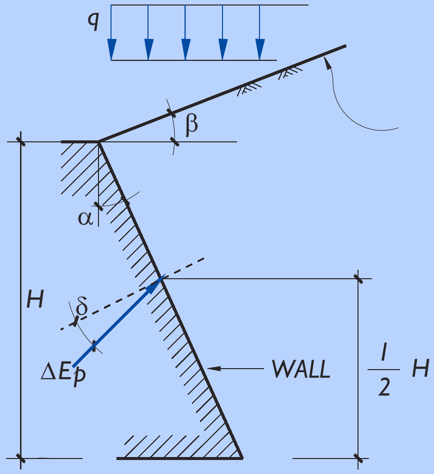
The increase in pressure due to uniform overburdens acting on the backfill surface can be estimated as shown in the same figure.

Figure 3.7.26. **Coefficient of Horizontal Passive Earth Pressure, K_{ph}, for Vertical Walls. Theory of Plasticity**



These increases in earth pressure, as shown in the figure, are calculated by determining their horizontal component first. As the direction of the pressure is known (deviated from the normal to the wall by an angle ?) the corresponding vertical component can be calculated.

Figure 3.7.27. **Effect of Cohesion and Overburdens on Passive Earth Pressure**



$$\Delta E_p = \frac{\Delta E_{ph}}{\cos(\alpha+\delta)}$$

Effect of cohesion, E_p

$$\Delta E_{ph} = c \cdot K_{phc} \cdot H$$

$$K_{phc} = \left(\frac{K_{ph}}{\cos\beta \cdot \cos(\beta+\delta)} - 1 \right) \cot an\,\phi \qquad (\text{for } \phi > 0)^*$$

Effect of overburden, q

$$\Delta E_{ph} = q \cdot K_{phq} \cdot H$$

$$K_{phq} = \frac{\cos\beta}{\cos(\beta+\delta)} \cdot K_{ph}$$

q = vertical uniform load per m² of horizontal plan projection

* *For f = 0 the passive earth pressure coefficient for the cohesive term is the same as the active earth pressure coefficient for the cohesion term given in Figure 3.7.16*

3.7.8 Calculating Earth Pressure at Rest

It may prove necessary to calculate the earth pressure at rest in order to check various Limit States of Serviceability and some more associated to the structural behaviour of the wall.

The theoretical concept of earth pressure at rest corresponds to the ideal situation of null displacement of the wall with respect to the ground. This situation can occur in cases where the external restraints prevent wall movement. Walls that are well founded on rock can be subject to similar pressures to the ones in this ideal situation.

The method recommended here for calculating earth pressures at rest is based on the concept of the coefficient of earth pressure at rest, K_0, which is defined as the quotient between the effective unitary pressure caused by the earth weight and the reference vertical effective stress defined in Subsection 3.7.5.1. That is,

$$K_o = \frac{e}{\sigma'_v \cos\alpha}$$

A reference line, *BC*, is needed for this definition to take into account the effect of the water (see Figure 3.7.11). In this case, the reference line should be inclined at an angle of 45° from the horizontal.

If the soil skeleton behaves elastically, with the same Poisson's ratio, ν, throughout the entire process of backfilling the wall, the value of K_0 would theoretically be:

$$K_{0(elastic)} = \frac{\nu}{1-\nu}$$

where ν is Poisson's ratio.

Other non-elastic processes may occur however in the generation of horizontal pressures. Therefore, it is recommended to calculate the value of K_0 using the following analytical expression:

$$K_o = (1 - \sin\phi)\,OCR^{\frac{1}{2}}$$

where:

OCR = overconsolidation ratio, as defined in 2.2.10.2.
ϕ = angle of friction.

In granular soils, the overconsolidation ratio should be taken as the quotient between the equivalent compaction pressure and the vertical effective pressure in the vicinity of the wall back face.

In the compaction of granular soils, stresses are produced that are equivalent to static pressures of between 1 and 3 bar, depending on the compaction energy applied. This pressure range leads to an approximate value of the overconsolidation ratio to be used in the above formula.

Theoretically, the parameter OCR reaches infinite values on the surface of a compacted granular fill. The recommended practical limit for this parameter is around 2 (light compaction) to 4 (intense compaction).

The same evaluation procedure can be followed in clayey soils, but considering higher possible values for the OCR parameter close to the surface, of up to 4 for light compaction and 9 for heavy compaction.

Earth pressure at rest on walls with an broken-line back face, or with non compacted and heterogeneous backfills ($K_0 < 1$), or with non-horizontal ground surfaces and with several types of overburden, can be calculated applying the procedures described in earlier sections as if it were the case of active earth pressure, but with a lower virtual angle of friction than the real value. In these cases, it is considered admissible to assume the fictitious angle of friction, $\phi_{equivalent}$, deduced from this expression:

$$\tan^2\left(45° - \frac{\phi_{equiv}}{2}\right) = K_o$$

The external surface of the fill will normally have no or moderate inclination (β angle). If it should happen that $\phi_{equivalent} < \beta$, the situation would be incompatible. In these cases, in addition to revising the overall stability conditions of the works, it is advisable to calculate the at-rest earth pressure assuming that $\phi_{equivalent} = \beta$.

It should generally be assumed that the at-rest earth pressure acts on the back face plane at an angle of deviation from the normal, δ, equal to the angle that would result from applying Rankine's method (see Fig. 3.7.6). Doing otherwise requires an express justification.

After determining the value of the coefficient of at-rest earth pressure and the orientation of its line of action, the effective pressures due to the weight of the ground can be calculated using the same procedures as in the case of active pressure, which is explained in Subsection 3.7.5.3.

Water pressure should be calculated as shown in Subsection 3.7.5.4, simply changing the inclination of the reference line *AC*, which in this case should be at an angle of 45° from the horizontal.

The effect of overburdens on walls that cannot move must be calculated by admitting the theory of elasticity. In such cases, the non-deformation can be simulated by placing an additional virtual load symmetrical to the real load with respect to the plane of the wall back face. The solutions published in geotechnical texts enable a large number of load cases to be solved.

These solutions from the elasticity theory involve vertical deformations between the wall and the ground that will generally not be real. Even so, they are considered to apply to the case.

3.7.9 Comments on Some Special Cases

Checking Ultimate Limit States of a geotechnical nature will require analysis of particular situations not expressly covered in the preceding sections. Engineers should define the specific computation procedure applicable to each individual case following the foregoing recommendations as far as possible.

This section covers four special situations that can occur fairly frequently. The analysis procedures recommended can serve as a guide not only to help engineers in these specific circumstances, but also to show some simplifications with which other similar situations can be tackled.

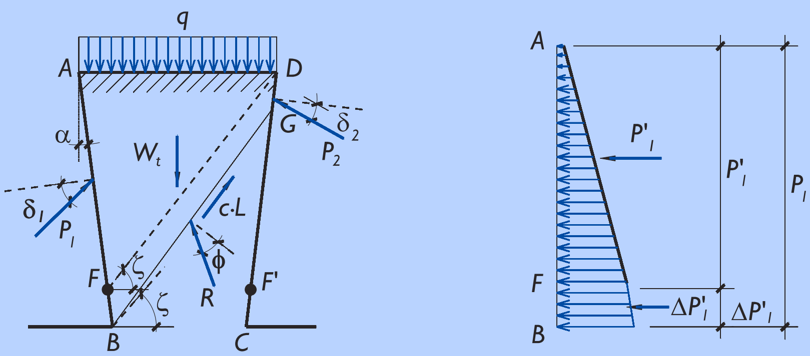
3.7.9.1 Active Earth Pressure on Parallel Walls

At some specific locations in harbour works two parallel walls can exist where an active earth pressure acts simultaneously on both as a result of one backfill shared by the two. It also occurs on the lateral retaining walls of narrow embankments constructed to provide access to very tall bridges.

The calculation of active earth pressure on two parallel walls is illustrated in Figure 3.7.28, which also describes the recommended analysis process.

Figure 3.7.28. *Active Earth Pressure on Adjacent Parallel Walls*

FOR ACTIVE EARTH PRESSURES ON AB



Wedges ABDG should be tried out applying the general theory with the earth pressures AF (P'_1) being those calculated without taking into account the existence of the opposite structure. The distance BF should be obtained by drawing a parallel line from D to the theorical sliding plane on the assumption that there is the only one structure (ζ).
Having obtained P_1 the pressure on BF will be equal to the increment with respect to P'_1 ($\Delta P'_1$) distributed linearly over FB

The pressure on each of the walls is calculated independently of the presence of the other, down to a certain depth (Point *F* in the figure) where the interference process begins.

Earth pressure on the bottom area (Stretch *FB* in the figure) should be calculated using an approximate method. The one described in Figure 3.7.28 is sufficiently accurate although, in order to apply it, there must be a minimum separation between the walls, so that Point *G* is situated within the area where there is no interference in the pressure between walls; i.e., above Point *F'* in the right-hand wall, which is the counterpart to Point *F* shown in the figure.

When the walls are so close together that this effect actually occurs, the pressure on the bottom part below the interference area (ΔP' in the figure) can be calculated by the expression:

$$\Delta P'_1 = \frac{\left[\frac{1}{2}W - \frac{1}{8}d^2 \tan\phi\right]}{\tan(\alpha + \delta)} - P'_1$$

where:

W = total effective weight of the fill plus the vertical component of possible loads and overburdens
d = horizontal distance between the heel of the walls (distance *BC* in the figure).

This simplified procedure means that the entire weight of the backfill plus any overburdens is borne by the vertical component of the earth pressures, except for the weight corresponding to a small bottom wedge with a flat base, *BC*, and an angle ϕ at *B* and at *C*.

When one of the two angles α and δ, or both simultaneously, are small, the above expression can lead to very high pressures. In such cases, a different failure mechanism occurs and the earth pressure on each wall has an upper limit, which is the one obtained by the general calculation, assuming that the other wall does not exist.

The silo effect, corresponding to collapse of the stored material while the walls remain fixed, is to a certain extent similar to the situation created by two very close parallel walls and the theoretical solutions for the silo effect can help in evaluating the earth pressures. The silo effect is covered in some detail in Subsection 3.7.9.4.

3.7.9.2 Buried Anchor Walls

Diaphragm walls generally need to be supported by anchors. These anchors can transfer the load to different types of buried structures.

A relatively frequent solution is that of discontinuous diaphragm walls. Their discontinuity can be in the vertical direction, because they do not reach the ground surface, or horizontal because the separation between anchors is considerable.

Figure 3.7.29 sketches approximate solutions that can be adopted when calculating the corresponding pulls.

When the vertical discontinuity is such that the height of the buried element is less than 2/3 of the total height, the simplification shown could lead to overoptimistic results. Other more recommendable approaches are indicated in Subsection 4.4.5.3.

3.7.9.3 Piles in the Backfill of Walls

Supports for crane rail tracks normally existing in quays lead to similar situations to the one sketched in Figure 3.7.30. Other circumstances could also lead to similar arrangements.

Figure 3.7.29. Calculating Pull in Some Discontinuous Structures

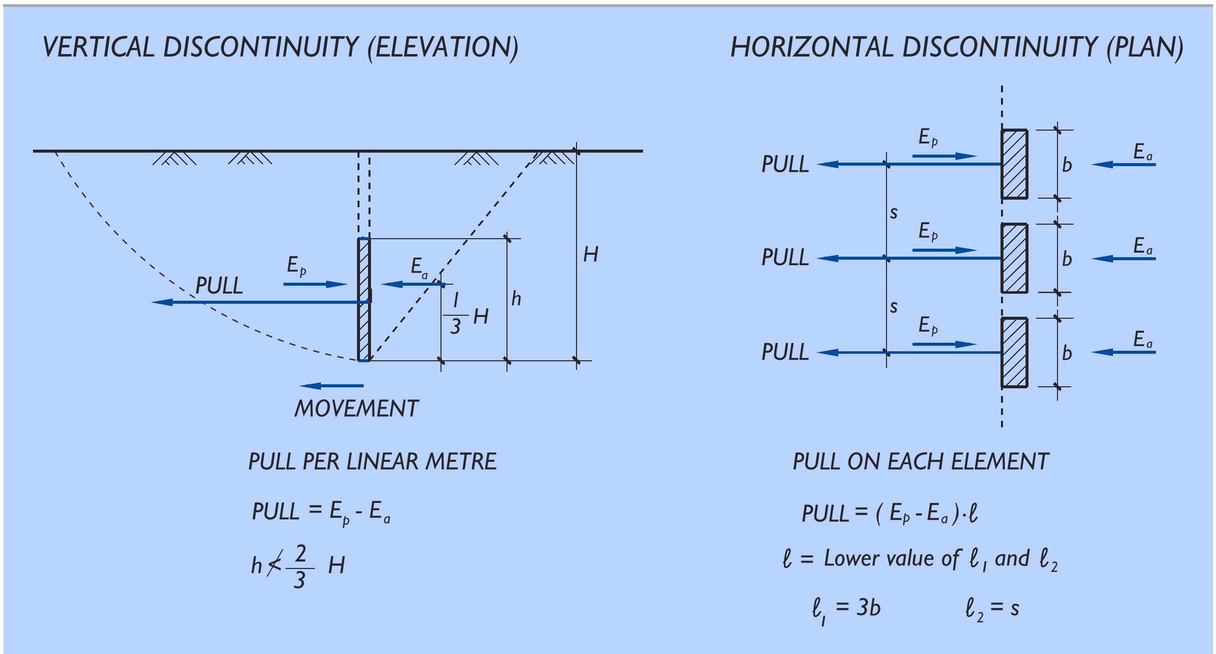
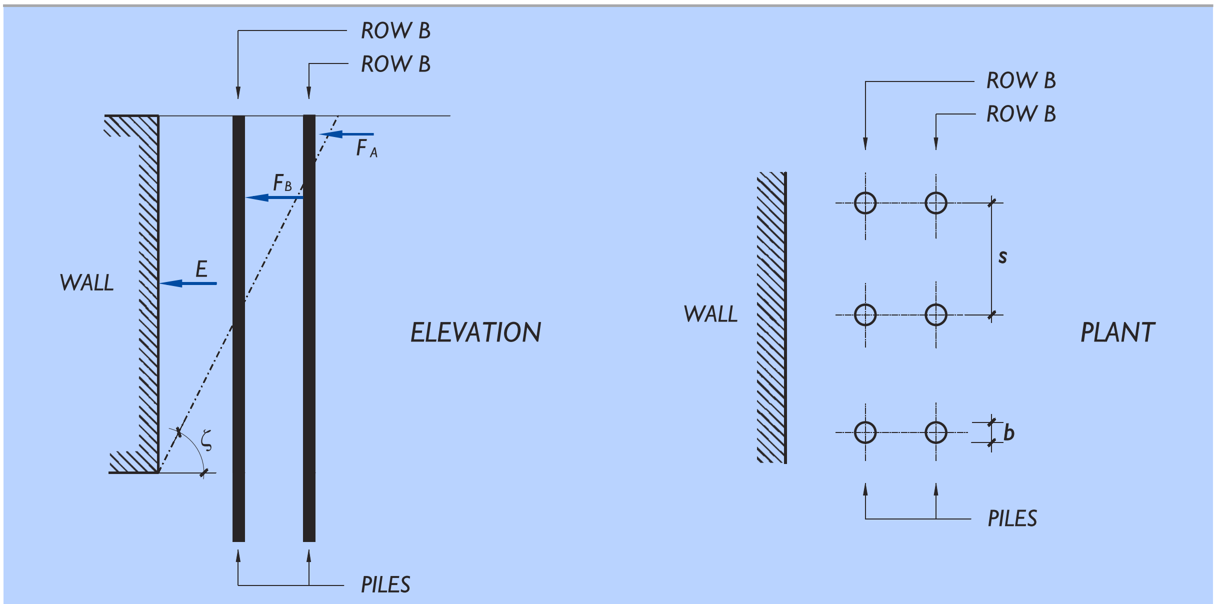


Figure 3.7.30. Earth Pressures on Piles in the Backfill of a Wall



Active earth pressures should be estimated in such cases by some simplified procedure, as there is no simple analytical solution that will allow the pressures to be calculated directly.

Active earth pressure can be calculated with several trial failure lines, which in the figure are plane for the sake of simplicity. When there is a heterogeneous backfill, the most unfavourable failure line may be a broken one.

Each failure line will give rise to calculating several vertical slices (see Subsection 3.7.4). In this case, some planes separating these slices should coincide with the piles.

The calculation of each slice will lead to an earth pressure on the contact between slices, one of which should correspond to the first plane of piles (Row A).

The active earth pressure on the inner row should be taken as equal to the higher of the following two values:

$$F_A = E_A \cdot s$$
or
$$F_A = E_A \cdot 3b$$

To calculate the next slice, it should be assumed that the pressure of the previous slice, E_A changes to $E_A{}^*$, given by:

$$E_A^* = E_A - \frac{F_A}{s}$$

Calculation of subsequent slices should proceed in the general way up to the slice whose vertical left-hand face (nearest to the wall) coincides with the next row of piles.

From the earth pressure on this face, the pressure acting on this row of piles can be calculated as also the pressure transmitted to the following slice.

The earth pressure against the piles will be the lesser of the following two values:

$$F_B = E_B \cdot s$$

$$F_B = E_B \cdot 3\ b$$

The earth pressure transmitted to the following slice will be:

$$E_B^* = E_B - \frac{F_B}{s}$$

At the end of the process, the earth pressure on the wall will be obtained.

Several potential failure lines must be tried out in this process, always at a greater angle to the horizontal than the angle of friction of the ground.

As a simplification, it is acceptable to define the active earth pressure on each pile and on the wall as the maximum value, for each element, resulting from all the trial lines. This will generally occur with a different failure line for each element.

3.7.9.4 Silo Effect

Materials stored in silos exert pressure on the walls that can be estimated by similar procedures to those used for active earth pressure cases. When a silo is emptied from below, the stored material collapses, thereby mobilising its full strength on the contact with the walls.

Another typical harbour situation that could be similar to the silo effect occurs with the fill of the tall narrow cells in prefabricated foundered caissons. As the fill itself gradually settles, it mobilises friction between soil and wall and unloads vertical stresses from the bottom.

For this reason it was deemed appropriate to include in this ROM 0.5 the classic solution for the silo effect that is described below.

The theory of silo effect assumes that the stored material is compressed against the silo walls with a stress with components σ_n and τ, so that:

$$\tan\delta = \frac{\tau}{\sigma_n}$$

where δ is the angle of friction between the silo contents and its walls.

The vertical pressure on horizontal planes, referred to here as p_v, will be less than the pressure corresponding to the weight of the total height of the stored material, since part of the vertical load is transmitted by friction onto the silo walls.

In addition, a relationship between the horizontal component of the earth pressure against the walls and the vertical pressure should be assumed, namely:

$$\sigma_n = K_{oh} \cdot p_v$$

where K_{oh} is a coefficient of horizontal earth pressure acting under conditions similar to those at rest, since the walls do not displace. This coefficient will be similar to the coefficient corresponding to a vertical wall ($\alpha = 0$), with horizontal backfill ($\beta = 0$) consisting of a purely frictional material with the same angle of friction of the stored material and which acts at a positive (downward friction) angle, δ, to the normal to the silo walls. This angle is equal to the friction angle between the stored material and the silo walls.

The solution to the vertical equilibrium equation leads to the following expression for the vertical pressure:

$$p_v = \gamma \cdot z_o \cdot (1 - e^{-\frac{z}{z_o}}) + q \cdot e^{-\frac{z}{z_o}}$$

where:

γ = unit weight of the stored material.
z = depth measured from the top surface of the stored material.
z_o = reference depth.
q = overburden that could eventually act on the surface.

The reference depth, z_o, will depend on the dimensions of the silo, on the strength of the material and on the shearing strength of the contact between the stored contents and the silo wall. It is given by the following expression:

$$z_o = \frac{A}{K_{oh}\, P \tan\delta}$$

where:

A = cross-section area of the silo compartment.
P = inside perimeter of the silo cell.

and where K_{oh} and δ have the same meaning as previously.

This solution not only enables p_v to be calculated at any depth, z, but also other relevant stresses such as σ_n and τ that are directly and proportionally related to it, as indicated in the previous paragraphs.

This section on the silo effect should be complemented by further reflection on adequate values for the design parameters.

For granular fills, except for coarse gravel or rockfill, it is advisable to use:

K_{oh} = 0.5 When filling the silo.
K_{oh} = 1.0 During silo discharge.

and as values of the δ angle

δ = 0.75 ϕ In filling.
δ = 0.60 ϕ In discharge.

where ϕ is the internal angle of friction of the fill.

For powdery materials, $\delta = \phi$ should be adopted in both filling and discharge processes.

To take into account the dynamic effects occurring during discharge, it will be necessary to multiply the pressures obtained from the above formulation by a factor of 1.50.

The theoretical earth pressure diagrams can be simplified in the calculations by adopting enveloping values erring on the safe side.

3.7.10 Earth Pressure in Braced Excavations

In trench excavations, earth pressures on the shoring system scarcely resemble the typical trapezoidal diagrams for active and passive earth pressures used in the calculations described in the preceding sections.

In the case of works of some importance, it may be necessary to resort to a complex computation simulating the construction process by finite element models or spring models as indicated in Subsection 3.7.11.2.

In most cases, however, a complex calculation will not be necessary and a simple approximation will suffice.

The earth pressure distributions that should be assumed in these cases are the ones deduced from observing earth pressures on this type of works. The pressure diagrams recommended are shown in Figure 3.7.31.

3.7.11 Most Common Failure Modes in Retaining Structures

3.7.11.1 Ultimate Limit States

The failure modes [17] capable of causing a retaining structure to become totally unserviceable and where ground strength plays the dominant role are described below.

Geotechnical Ultimate Limit States, GEO

a. Loss of Overall Stability

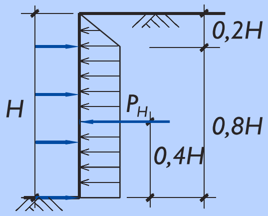
The retaining structure and surrounding ground can fail as a whole, globally, as a result of a deep slide caused by areas of the ground that may not be in contact or even close to the retaining structure.

This failure mechanism should be analysed using the general procedures explained in Section 3.8.

(17) Diagrams are included (see Fig. 3.7.32) to help to describe different failure modes.

Figure 3.7.31. **Earth Pressure against Shoring Walls**

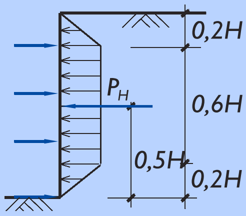
LOOSE GRANULAR SOILS



$0{,}8 \cdot K_a \cdot \gamma \cdot H \cdot \cos\delta$

$P_H = 0{,}72\ K_a \cdot \gamma \cdot H^2 \cdot \cos\delta$

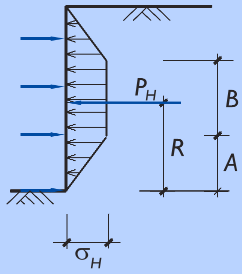
DENSE GRANULAR SOILS



$0{,}8 \cdot K_a \cdot \gamma \cdot H \cdot \cos\delta$

$P_H = 0{,}64\ K_a \cdot \gamma \cdot H^2 \cdot \cos\delta$

COHESIVE SOILS



$2 < N_0 < 5$

$P_H = 0{,}78 \cdot H \cdot \sigma_H$

$\sigma_H = \gamma \cdot H - 1{,}5\ (1 + N_0) \cdot c$

$A = 0{,}15H;\ B = 0{,}55H;\ R = 0{,}46H$

$5 < N_0 < 10$

$P_H = 0{,}78 \cdot H \cdot \sigma_H$

$\sigma_H = \gamma \cdot H - 4c$

$A = 0{,}15H;\ B = 0{,}55H;\ R = 0{,}46H$

$10 < N_0 < 20$

$P_H = (21 - 0{,}055N_0) \cdot H \cdot \sigma_H$

$\sigma_H = \gamma \cdot H - (8 - 4N_0) \cdot c$

$A = (3 - 0{,}015N_0) \cdot c$

$B = (1{,}1 - 0{,}055N_0) \cdot H$

$R = 0{,}38H$

$N_0 > 20$

$P_H = 0{,}5 \cdot H \cdot \sigma_H$

$\sigma_H = \gamma \cdot H$

$A = B = 0$

$R = 0{,}33H$

KEY:

N_0 (stabilty number) $= \dfrac{\gamma \cdot H}{c}$

γ : apparent unit weight of the soil

H : elevation of the retainig element above the excavation bottom

c : cohesion

K_a : coefficient of active earth pressure

δ : angle of friction ground-wall

b. Sliding

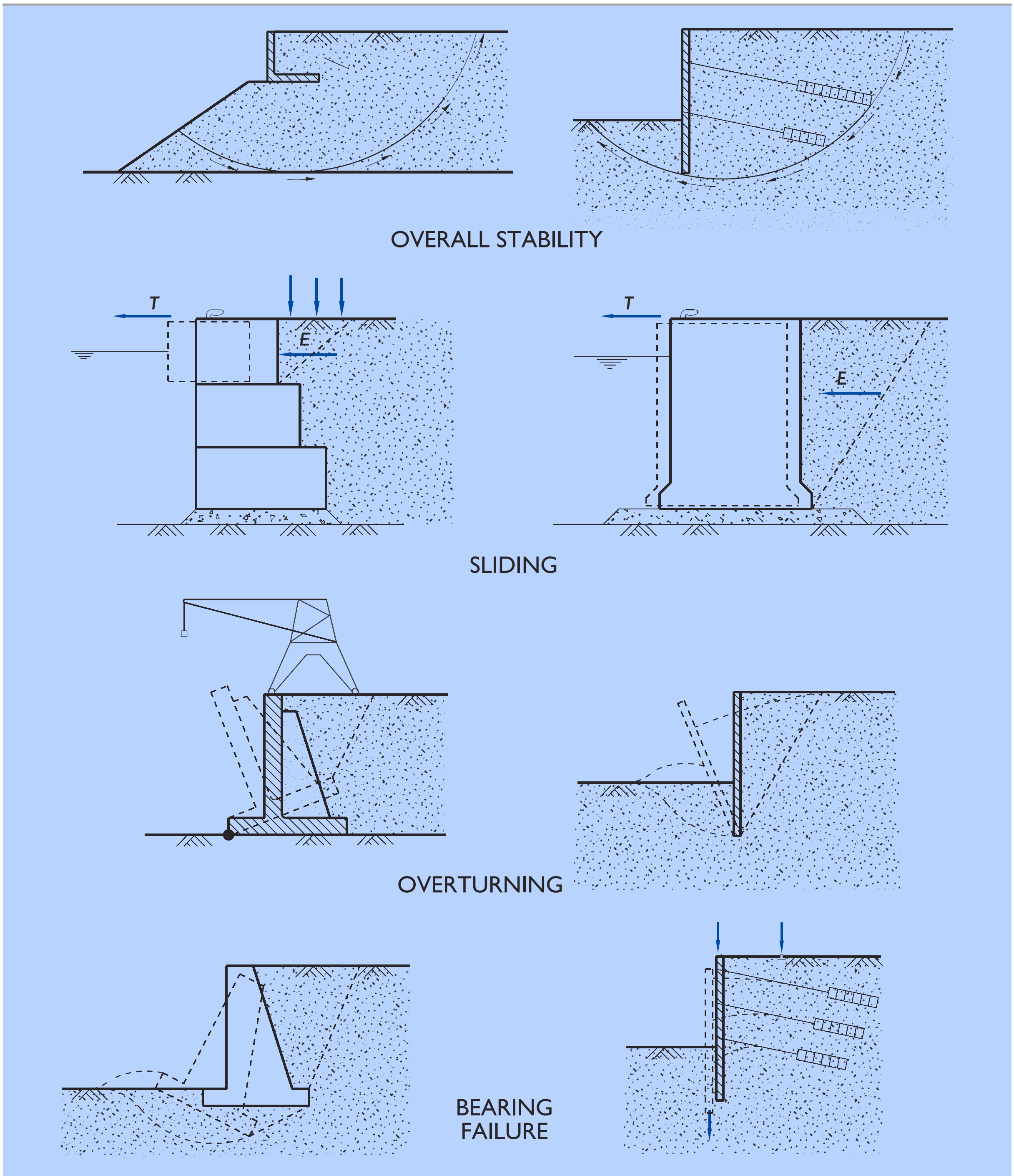
The most common failure mechanism in gravity retaining structures is sliding of the structure along its foundation plane when shallow foundations are involved.

This failure mechanism is described in Section 3.5 dealing with shallow foundations.

c. Overturning

Overturning is a failure mechanism affecting gravity retaining structures with shallow foundations on firm ground. In soft soils, bearing failure will occur before overturning.

Figure 3.7.32. **Diagram of Various Failure Modes**



In diaphragm walls, different failure situations may occur that can be assimilated to overturning when the anchors or the embedment zone fails.

d. Bearing Failure

When the loads acting on the foundations of a gravity retaining structure exceed the ultimate bearing capacity of the ground, then a bearing failure occurs, which generally translates into settlement, displacement and rotation.

Since the loads acting on the foundations of walls are generally inclined and often eccentric, the bearing capacity of the ground in the case of shallow foundations or the resistant capacity of the piling in the case of deep foundations is an matter requiring special attention.

The bearing capacities for different types of foundation are analysed in Sections 3.5 and 3.6.

Situations similar to bearing failure can occur in diaphragm walls when the vertical loads are large. This can occur when anchors are inclined downwards, when angle δ is large or when vertical loads are placed on the head of the wall.

e. Bottom Heave

In addition to the general mechanisms described, some special mechanisms affect retaining structures used to shore excavations.

When the soil has low strength and the shoring, as is normally the case, does not extend beyond the bottom of the excavation, bottom heave can occur with simultaneous large settlements in the backfill.

A similar problem can occur as a result of the effect of upward vertical seepage at the toe of diaphragm walls (front side). The problem is sometimes known as "blowout".

STRUCTURAL ULTIMATE LIMIT STATES, STR

Structural failure in retaining elements is particularly important. Some are shown in Figure 3.7.33. Analysing this type of failure, which results when the structural capacity is exceed, lies outside the scope of this ROM 0.5.

Tensile or shear forces can occur in plain concrete walls, causing them to break and one part of the wall to slide over another.

In reinforced concrete walls, structural calculations are closely linked to earth pressures. Subsection 3.7.8 gives recommendations for the method of estimating earth pressures at rest that will make it possible to evaluate the structural loads under near-service conditions.

Flexible diaphragm walls, and particularly their anchors, must have sufficient structural capacity to withstand direct earth pressures and to transmit them to zones away from the risk of overall instability.

Structural failure in shoring should always be considered, either occurring in the elements in contact with the ground -as a result of bending or shear –or in the struts– that could fail as a result of eccentric compression.

The foundation elements of gravity structures, whether shallow strip footings or pile caps connecting the piles of possible deep foundations, are essential parts of the structure whose capacity must be verified.

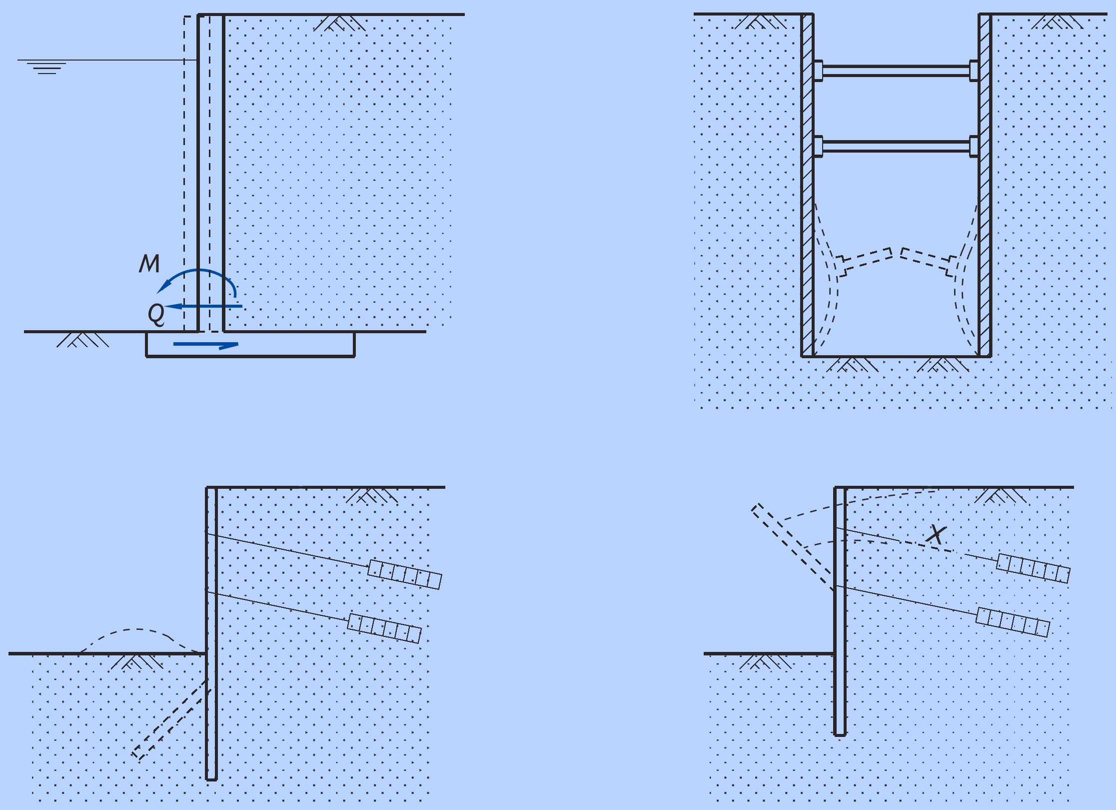
Each specific works can present individual types of structural failure, whose detailed study lies outside the scope of this ROM 0.5.

Special mention however should be made of sheet-pile enclosures, which present specific types of failure as a result of internal shear, overturning or localised structural failure owing to tensile forces on connections.

Other failure modes that can occur in earth pressure retaining structures include the following, without this list being exhaustive:

- Excavations or scour in their front face zone.

Figure 3.7.33. ***Diagram of Various Types of Structural Failure***



DIFFERENT TYPES OF STRUCTURAL FAILURE

- Erosion caused by seepage through or around the works and/or excessive seepage rates in works where a certain degree of water-tightness is intended.

- Variations in the hydraulic regime in the surrounding area induced by the retaining works and that may originate undesired rises or drops in the groundwater table that could damage structures or other adjacent works.

- Changes in the local erosion regime affecting the surrounding area or the works themselves in coastal zones.

- Movements caused by anchors that transmit loads close to the foundations of other structures.

- Potential harmful effects of vibrations during sheet-pile driving or fill compaction.

Furthermore, when designing retaining structures, the comments relating to shallow and deep foundations should be taken into account, as appropriate, according to Subsections 3.5.3 and 3.6.3.

All these failure modes must be duly analysed following the general procedures given in this ROM 0.5. The following sections contain various supplementary guidelines for three specific problems only: overturning of gravity walls , ground failure in diaphragm walls and bottom heave in shored excavations. The other failure modes should be analysed by the general procedures already explained.

3.7.11.1.1 Characterising Design Situations

In analysing Ultimate Limit States of retaining structures, design situations capable of representing the reality of the actual works as faithfully as possible should be considered. Some recommendations in this respect are given below.

a. Defining Geometrical Parameters

The geometrical parameters of retaining works should be clearly defined in each situation to be analysed and must be the same for all analyses of a geotechnical nature. Although studies of sensitivity to the variation of certain geometrical parameters can be carried out, a common basic geometry should always be maintained.

Representation of the contact between different types of ground under foundations, and of the subsoil in general, should follow the corresponding recommendations given in Subsections 3.3.5.1, 3.5.3.1 and 3.6.3.1, as appropriate.

In retaining structures, as a general rule, it is of particular interest to define accurately the ground level in backfills, and –of the utmost importance– the groundwater levels in the vicinity of the works and their possible fluctuation, according to the definitions given in Subsections 3.3.5.1 and 3.4.4.1.

In the front face areas, on which diaphragm-wall retaining structures rely to withstand the loads acting on their back face, any potential scour or excavations should be considered.

b. Defining Ground Properties

The ground properties of most interest in the design of retaining structures are:

- The geotechnical characteristics of the foundation, set out in Subsections 3.5.3.2 and 3.6.3.2 for shallow and deep foundations respectively.
- The geotechnical characteristics of the backfill material at the back face, which on many occasions will consist of artificial backfills. The data most commonly required relate to its unit weight and shear strength.

To define backfill characteristics, representative values of the corresponding parameters should be taken, without applying any reducing or increasing factors, as explained in Subsections 3.3.5.2 and 3.3.7.

The angle of friction of the backfill material to be used in calculations should generally be the residual angle of friction, i.e., the one corresponding to large strains. The potential unfavourable effect of progressive failure is thus taken into account. In special circumstances, the use of another criterion may be justifiable.

c. Defining Actions

In the geotechnical calculations necessary for verifying safety in relation to Limit States, the earth pressure can be taken as a load (input data for the problem under study) or as a reaction to be calculated, depending on the failure mode under analysis.

Tensile forces on anchors and compression on shoring struts can also be considered as loads in some calculations (calculating anchor length, structural dimensioning of struts), or as part of the solution in other problems, that is, as reactions to be calculated as the response to loads.

In accordance with the indications given in Subsection 3.3.5.3, earth pressures should be taken as permanent loads regardless of the failure mode to be verified. Therefore their unfactored representative values, i.e., their characteristic values, should be used in the equations for checking geotechnical (GEO) or owing to excess water pressure (UPL) failure modes. Nevertheless, additional earth pressures owing to overburdens should be considered as variable loads. In order to check structural failure modes (STR), earth pressures could be considered otherwise.

3.7.11.1.2 Verifying Safety against Overturning in Gravity Walls

Earth pressure is the predominant load exerted on gravity walls and they must be designed to resist this so that they meet all the requirements of safety against the failure modes set out in Subsection 3.7.11.1 as well as any others that engineers might conceive.

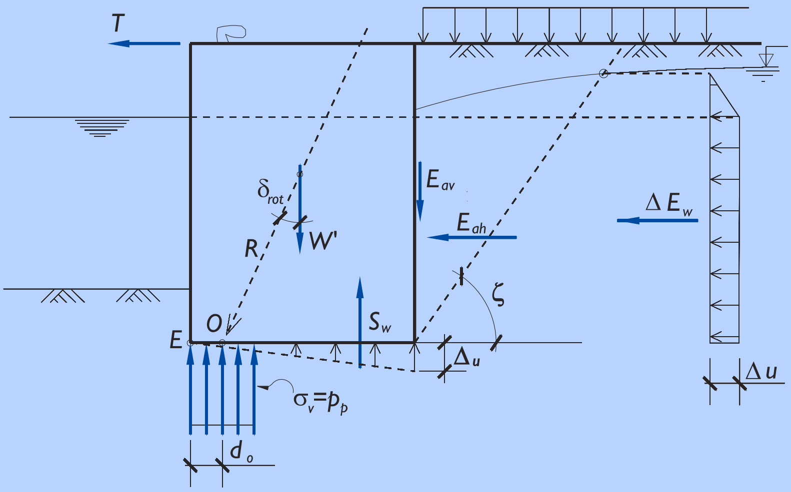
The right analysis procedure will depend on the failure mode under analysis. This ROM 0.5 provides criteria for analysing failures due to a loss of overall stability (Section 3.8), sliding, overturning or bearing failure when the wall foundations are shallow (Section 3.5) and failures due to bearing capacity or horizontal failure for piled foundations (Section 3.6).

In addition, criteria are given throughout this ROM 0.5 for analysing other failure modes such as scour, erosion, bottom heave, etc.

This section includes some details of the specific analysis procedure for plastic overturning in gravity walls with shallow foundations. This type of failure was also considered, although in a more general manner, when discussing shallow foundations in Subsection 3.5.6.

Figure 3.7.34 illustrates the forces involved in the equilibrium of the wall and shows a recommended way of classifying them as stabilising or overturning forces.

Figure 3.7.34. Checking Safety against Overturning



R = resultant force of the effective loads on the foundations (horizontal loads factored by F)

SAFETY FACTOR AGAINST OVERTURNING:

$$F_v = \frac{M_{(stabilising)}}{M_{(overturning)}} \quad \text{(Moments about 0)}$$

Stabilising forces (as in the sketch):

- W' = effective weights and potential vertical loads on the wall
- $-S_w$ = differential uplift (negative stabilising moment)
- E_{av} = vertical component of active earth pressure

Overturning forces (as in the sketch):

- E_{ah} = horizontal component of active earth pressure
- ΔE_w = differential water pressure
- T = pull and other external horizontal forces

For plastic overturning to occur, the overturning forces need to grow while the stabilising forces remain stable. It has been agreed to define the *safety factor against plastic overturning* as the number, *F*, by which all overturning forces should be multiplied -preserving their lines of action- to reach a state of failure in the supporting ground.

In the overturning limit state, the wall will rest on a narrow strip, centred at Point *O*, and plastification of the ground will occur in the zone close to the toe of the wall. The plastification pressure is referred to here as p_p.

At the instant of failure, with the effect of the overturning forces multiplied by *F*, the balance of moments about Point O leads to the equation:

$$F \cdot M_{overturning} = M_{stabilising} \qquad \text{(moments about O)}$$

The location of Point *O*, however, depends on *F* and consequently the only way to calculate *F* is by an iterative procedure in which *F* should be taken as a variable and incremented gradually.

The calculating procedure can entail the steps listed below.

a. Order and classify the values of the actions, obtaining the data shown in Figure 3.7.34 for each component of the actions.

b. Take moments from these loads about Point *E* on the outer edge of the support area to obtain the values of the stabilising moment M_1 and overturning moment M_2. Please note that if the ground were infinitely resistant, the quotient of these two moments would be precisely the desired safety factor. The plastic overturning safety factor will always be lower than this quotient:

$$F < \frac{M_1}{M_2}$$

M_1 = stabilising moment about Point E.
M_2 = overturning moment about Point E.

c. Take a reasonable *F* value to commence the iterative process.

d. Calculate the point where the resultant goes through the foundation plane for this *F* value. This point can be defined by the distance d_o to the edge of the foundation, which is defined by the equation:

$$d_o = \frac{M_1 - F \cdot M_2}{V'}$$

where *V'* is the vertical component of the resultant of the loads. For the case illustrated in Figure 3.7.34, it holds that:

$$V' = W' + E_{av} - S_w$$

e. Calculate the inclination of the loads corresponding to the assumed *F* value. For the case illustrated in Figure 3.7.34, it holds that:

$$\tan\delta_{fail} = F \cdot \frac{E_{ah} + \Delta E_w + T}{W' + E_{av} - S_w}$$

f. Calculate the vertical component of the ultimate bearing pressure of the ground resulting from a foundation width $B^* = 2\, d_o$ and the value for $\delta_{failure}$ obtained in the preceding epigraph. This value will be designated p_p.

Subsection 3.5.6.2 gives calculation procedures relating the vertical pressure producing bearing failure or plastification in the ground (p_p) with the effective foundation width ($B^* = 2\ d_o$) and with the inclination of the load causing plastification of the ground (angle δ_{fail}).

The appropriate procedure for calculating the ultimate pressure p_p should be chosen depending on the nature of the ground and the importance of the works (see Subsection 3.5.4.1).

g. Calculate the vertical component of the pressure that the wall transmits to the ground for the assumed safety factor. This pressure is:

$$p = \frac{V'}{2d_o}$$

h. Compare p and p_p and proceed as indicated:

If $p > p_p$, run another iteration with a smaller value for *F*.
If $p < p_p$, increase the value of *F* and repeat the process.

With the new *F* value, the process should be restarted from Point d) in the list of calculation steps.

The calculation should be taken as complete when the *F* value is sufficiently bounded, for example, with a maximum error in the order of 1%.

It is advisable that the safety factor against plastic overturning thus defined be greater than the safety factors indicated in Subsection 3.5.6 for works with a low SERI rating (5 - 19).

For works with a minor or high SERI, or for other allowable failure probabilities, the minimum *F* values laid down in Table 3.5.8 can be adjusted as indicated in Subsections 3.3.8.2 and 3.3.10. They can similarly be adapted for transient situations (including short-term geotechnical situations) in accordance with the recommendations in Subsection 3.3.8.1.

3.7.11.1.3 Verifying Safety against Ground Failure in Diaphragm Walls

Diaphragm walls can cause an active state of ground failure in the backfill and, at the same time, a passive state in their embedment zone when they undergo the large deformations associated with failure Ultimate Limit States.

Failure modes in diaphragm walls are varied, and involve the structural collapse of the wall itself or its anchors in most cases. Recommendations will be given elsewhere in the ROM Programme, when dealing specifically with the study of these structures, as to the analysis procedures applicable to such cases. Section 4.3 in this ROM 0.5 provides some additional guidelines on quays with diaphragm walls.

This section here only covers the failure modes exclusively governed by ground strength.

The overall equilibrium of the works, due to failure of the surrounding ground, should be analysed with different trial failure lines, whose stability should be verified as laid out in Section 3.8.

The bottom heave at the toe of a diaphragm wall, as a result of potential upward seepage on its front side, is examined in Subsection 3.4.6.

The problem of internal erosion is also examined in Subsections 3.4.8, 4.3 and 4.4.

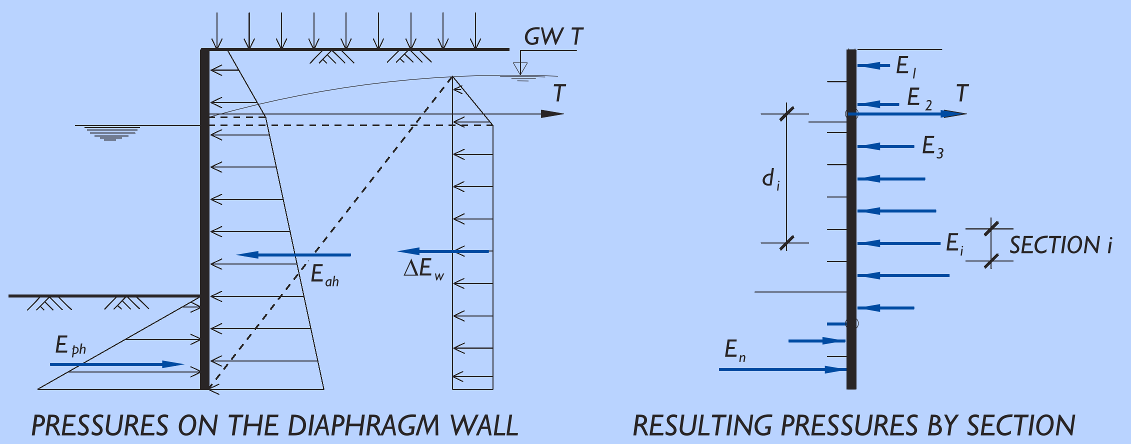
The vertical equilibrium of a diaphragm wall can be studied following the principles set out in Subsection 4.4.5.

Any other possible Ultimate Limit States engineers could conceive should be analysed in accordance with the general criteria indicated throughout this ROM 0.5.

Minimum Embedded Depth

There is a simple and general Ultimate Limit State common to all anchored diaphragm walls and which should always be analysed, independently of any other complementary analyses that may be carried out for other purposes. Figure 3.7.35 tries to represent this Ultimate Limit State.

Figure 3.7.35. Simplified Analysis of an Anchored Diaphragm Wall



MINIMUM EMBEDDED DEPTH,

$$\Sigma E_i d_i = 0$$

TENSILE FORCE ON THE ANCHOR

$$T = \Sigma E_i$$

In this Ultimate Limit State it is assumed that the diaphragm wall has undergone sufficient displacement, so that there is a state of active earth pressure on its back face and one of passive earth pressure in the front embedment.

The causes leading to this limit situation can vary considerably. The ground may have less strength than predicted, the overburdens may be greater than anticipated, etc. In the following example it could be considered that the cause of the limit state is a deficit in the embedment length.

Having assumed that extreme earth pressures are mobilised, it will be possible to calculate the diagrams for the unitary earth pressure acting on both faces of the diaphragm wall by following the recommendations given in earlier sections.

It will also be possible to calculate the resulting pressures by sections in order to obtain the values of E_1, E_2, etc. as shown in Figure 3.7.35.

By taking moments about the anchorage point, new terms can be added from top to bottom to the summatory $\Sigma E_i d_i$. The point at which this summatory is null for the second time (after having entered the diaphragm wall embedded zone) marks the minimum embedment depth.

If the diaphragm wall has a shorter embedded length, equilibrium will not be possible. If it is embedded deeper down, there will be a degree of safety that the Ultimate Limit State under discussion cannot occur.

One measure of safety with respect to this Ultimate Limit State could be the ratio between the real embedment length and the necessary embedment length. These lengths can either be measured from ground level on the front side or else taken from a point lower down (the point where the active earth pressures equal the passive pressures, for example).

When verifying safety against ground failure for diaphragm walls in works with a low SERI rating, this ROM 0.5 considers acceptable that the real embedment length be 1.3 times greater than the minimum embedment length necessary for quasi-permanent load combinations, 1.25 times for fundamental combinations and 1.2 times for accidental or seismic combinations. All these lengths should be measured from the ground level on the wall front face. Consideration of other Ultimate Limit States may lead to greater embedment depths.

Anchor Tension

Studying this Ultimate Limit State will also provide certain useful information about other aspects of the diaphragm wall behaviour. As shown in Figure 3.7.35, if the diaphragm wall had exactly the minimum embedded length, the tension on the anchor would be exactly equal to the algebraic sum of all the earth pressures. Deeper embedment will generally lead to lower values for *T;* consequently, the *T* value obtained as indicated in this section is an initial estimate of the anchor capacity needed to support the diaphragm wall.

The Case of Several Anchor Levels

When diaphragm walls are anchored at different levels (onshore constructions), a similar procedure can be followed to analyse the Ultimate Limit State described above. To this end, it should be assumed that the ground at the wall front is being progressively excavated and that the anchors are inserted as this excavation advances.

At the moment that the first anchor level has already been inserted and the second one is about to be put in place, the limit state for a single anchor can be analysed and the tensile force on the upper anchor obtained.

This tension should thereafter be considered as a constant force. This assumption will make it possible to calculate the force acting on the second anchor level by setting out the limit equilibrium just before the third level is put in place.

Completing the process will produce a set of minimum anchor forces and a minimum embedment depth that can be used to verify the safety of the diaphragm wall as already indicated.

Cantilever Walls

This Ultimate Limit State can now be analysed in a similar manner. As there are no anchors, the moments must be taken with respect to the lowest point of the diaphragm wall. This is equivalent to assuming that there is a virtual anchor at the wall toe.

The calculation should be carried out as in the earlier case, progressively lowering the toe of the diaphragm wall, until the equation of the moment equilibrium about it is satisfied. At this moment, the minimum embedment depth required to guarantee equilibrium will have been reached.

The sum of all the earth pressures will in this case be directed towards the backfill and would amount to a compression on the hypothetical virtual anchor at the wall toe. This reaction is usually known as *counter-pressure*.

In order to mobilise counter-pressure, a certain additional embedded length is required. This length can be estimated by assuming that, in the counter-pressure area, a pressure is acting similar to the passive pressure corresponding to the vertical pressure in the backfill. In other words:

$$\Delta = \frac{\sum E_i}{K_{ph}\sigma'_v}$$

Δ = additional driven length required to mobilise counter-pressure.
K_{ph} = coefficient of passive earth pressure in the embedded zone.
σ'_v = vertical effective pressure at the counter-pressure depth on the back side.

Cantilever walls in low SERI works should also be embedded in the ground at least 1.25 times the minimum depth necessary for quasi-permanent load combinations, 1.2 times for fundamental combinations and 1.15 times for accidental or seismic loads in order to prevent this failure mode (including the length required to mobilise counter-pressure).

Engineers may in fact decide on greater embedment for other reasons. As a general rule, cantilever diaphragm walls can undergo very large deflections, not only because of deformation in the ground itself but also due to deformation of the actual diaphragm wall structure. These displacements tend to be restrictive or critical in certain projects. This is why they should be the subject of detailed calculation and, in addition, the different origins of the displacements should be taken into account before validating the results.

3.7.11.1.4 Verifying Safety against Bottom Heave in Braced Excavations

Shored excavations executed in soft soil can fail because of bottom heave. The vertical pressure on a horizontal plane at the bottom level differs on each side of the excavation and this pressure difference can lead to ground plastification at the bottom of the open cut and cause it to heave following a similar mechanism to the one illustrated in Figure 3.7.36.

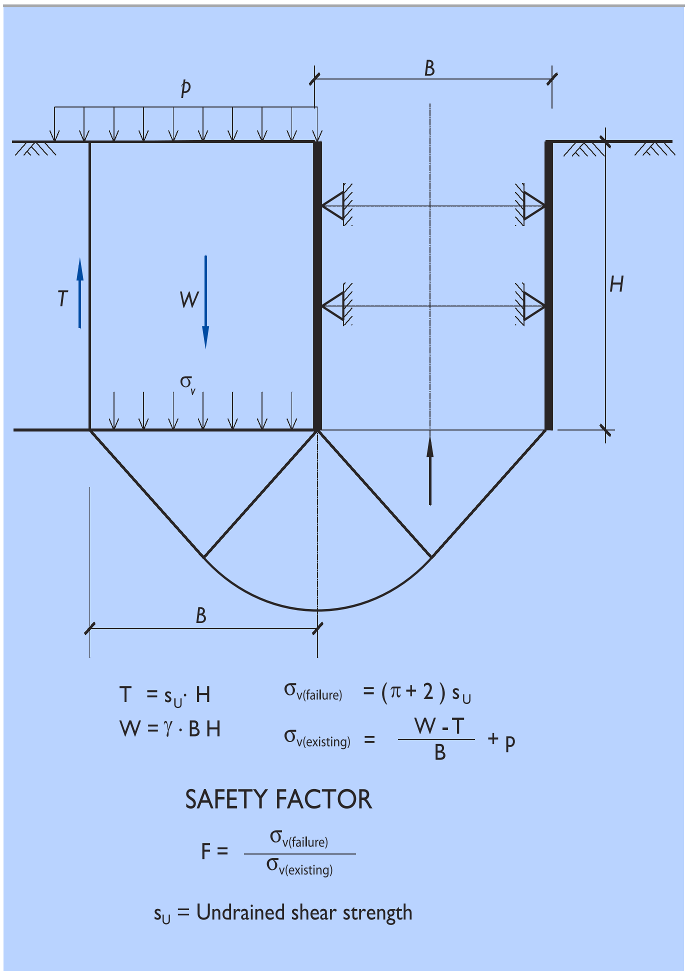
This or other similar mechanisms are typical in soft saturated soil with low permeability and in short-term situations. This is why the shear strength is represented by the parameter s_u in the figure.

In granular soils or in cohesive soils under long-term conditions, i.e., in cases where a substantial angle of friction can be relied on, this type of failure does not normally occur and this check will not therefore be necessary.

The formula given here applies to trench excavations of indefinite length. If this length is relatively short, this effect can be taken into account by reducing the real width to another equivalent width given by the expression:

$$B_{(equiv)} = B \cdot \frac{L}{L + 2B}$$

Figure 3.7.36. **Bottom Heave in Shored Excavations**



and then carrying out the check with this virtual width, which is smaller than the real one.

In the case of homogeneous ground and with surface overburden, the safety factor against this type of failure is shown in Figure 3.7.36.

The safety factors for this failure mode should be greater than the minima shown in Table 3.4.2 for low SERI works (5 - 19). When failure as a result of bottom heave could involve potential loss of human life, engineers should carry out shored excavations incorporating higher safety factors, as stipulated in Subsections 3.3.8.2 and 3.3.10 in this ROM 0.5.

3.7.11.2 Serviceability Limit States

3.7.11.2.1 Displacements in Gravity Walls

Areas close to retaining structures generally undergo large displacements and it is therefore essential to check that they will not damage any adjacent structures.

Gravity retaining structures can be assumed to behave as a rigid body and their movements estimated as being only governed by deformation in the foundations.

To this effect, foundation movements can be calculated by the procedures indicated in Sections 3.5 and 3.6 for shallow and deep foundations respectively.

The loads to be taken into account in these calculations will be the ones corresponding to service situations that engineers need to check. Earth pressure should normally be considered as one of the actions and should have been evaluated in the extreme condition of active earth pressure. These active earth pressures will have been calculated to check the different Ultimate Limit States described in the foregoing sections.

The displacement resulting from these calculations must be checked against the movement necessary to mobilise the active earth pressure. The displacement values given for guidance purposes in 3.7.2 can be used to this effect.

If the movement calculated is greater than that needed to mobilise active earth pressure, the calculation can be considered correct - the resulting situation is compatible. Otherwise, it is necessary to assume a larger earth pressure value than the minimum, which corresponds to the extreme situation of active pressure.

If the displacement is less than the value needed to mobilise active earth pressure, engineers must modify their calculations to take into account the fact that, in the service situation under study, the pressure on the retaining structure may be greater than the active earth pressure.

In such cases, it is considered admissible to calculate movements using earth pressures on the retaining structure that have been calculated under the theoretical condition of earth pressure at rest.

In any event, given the inaccuracy of the methods for calculating deformations, the verification of service conditions should not be considered satisfactory unless the estimated displacements are merely a fraction (in the order of 1/3) of the ones that take the structure beyond the corresponding limit.

3.7.11.2.2 Displacements in Diaphragm Walls

Diaphragm walls retain the earth pressure with the aid of their buried part, which partially mobilises passive earth pressure, and with the help of anchors or other support elements.

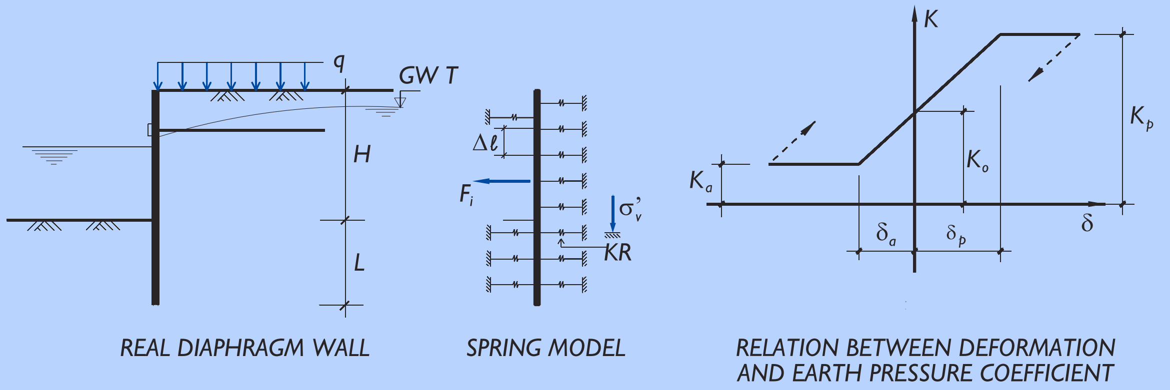
Calculating the movements, earth pressures and loads acting on support elements under near-service conditions is a complex problem of soil-structure interaction that needs adequate procedures to be solved.

All soil-structure interaction problems can be solved, at least in an initial approximation, by representing the structure by appropriately connected beam elements and the ground by springs having a similar load-deformation response to the one expected in the ground.

In addition, in these problems of pronounced and non-linear interaction, the construction process itself also bears a certain significance. The forces and moments acting on the diaphragm walls and their supporting elements will not only depend on the final geometry of the works but also the sequence of the work executed. For this reason, pressures must be calculated in steps simulating the different construction stages.

The structural model chosen could be like the one shown in Figure 3.7.37.

Figure 3.7.37. **Diagram for Calculating Diaphragm Walls by Equivalent Springs**



$$KR = \frac{K_p - K_a}{\delta_p - \delta_a} \cdot \sigma'_v \cdot \Delta\ell$$

The basic stages in the calculation procedure can include the following:

- Excavation on the front side.
- Backfilling.
- Variation of the groundwater table.
- Installation or removal of anchors.
- Tensioning of anchors.
- Placement or removal of overburdens.

Each of these basic operations must be adequately simulated in the calculations.

a. Excavation and Fill

Excavation on the front side, for example, can be simulated using the computation operations described below.

- Eliminating the springs representing the excavated zone. In these calculations it is advisable to simulate excavation in small steps so that no more than one spring is removed in each step.
- Calculating the unbalanced loads which, for each spring on the front side, will be:

$$\Delta F = \frac{\Delta\sigma'_v}{\sigma'_v} \cdot F$$

where:

$\Delta\sigma'_v$ = variation in the vertical effective stress due to the excavation.
σ'_v = prior vertical effective stress.
ΔF = the unbalanced load to be put into the calculation.
F = prior load on the spring.

For the springs removed, this rule will clearly lead to:

$$\Delta F = F$$

The state of the deformations, earth pressures and stress resultants at the end of the simulated stage can be obtained by entering these loads as pulls on the nodes.

- Updating the spring constants.

  The new state of vertical effective stresses make the spring constants change, as shown in the figure. Their values need to be updated at each calculation stage. Operations in the backfill can be simulated by taking similar steps.

b. Variation in Groundwater Table

Water pressures should always be considered as additional external forces. The calculation should normally begin with balanced hydrostatic pressures transmitting a zero net force on the diaphragm wall. Any change in the groundwater table on either side of the diaphragm wall should be represented as a set of external horizontal forces on each node under the original groundwater table. The sum of all these forces will represent the unbalanced net pressure caused by the drop in this groundwater table.

The groundwater table variation makes the vertical effective pressures change and the spring constants will therefore change. These should be updated as indicated earlier.

c. Anchors

The insertion of anchors can be modelled in a simple way by adding a spring with a constant that must coincide with the one estimated for the anchor.

The tensioning of an anchor can be simulated by applying a force equal to that of the anchor to the corresponding node. It is a good idea to perform this operation in several calculation stages to facilitate modelling non-linear behaviour.

The removal of an anchor can be simulated by removing the spring representing it and applying the anchor's previous load –in the opposite direction– at the corresponding node. It is advisable to simulate this in small increments.

d. Overburdens

The effect of a uniform surface surcharge, *q*, can be simulated in the same way by assuming the following increase in the vertical pressure:

$$\Delta \sigma'_v = q$$

on the side affected by the overburden.

The earth pressure on the diaphragm wall should be represented by a horizontal force on each node, as follows:

$$F = q \cdot K_{0q} \cdot \Delta l$$

where K_{0q} is the coefficient of earth pressure at rest corresponding to the overburdens, calculated as set out in Subsection 3.7.8.

For heavy overburdens, it is also advisable to simulate their effect in several stages. Furthermore, the spring constants must be updated at each calculation stage with the new vertical effective pressures.

e. Non-Linear Process

If, during simulation of the construction process, the limit deformation is reached in any spring, the corresponding spring must be represented with $KR = 0$ for the following load state. When the deformation sign changes again (as can occur during anchor tensioning stages), the spring stiffness must be restored. The hysteresis implied in Figure 3.7.37 can thus be simulated.

f. Spring Constants

The main problem in models of this type lies in determining the coefficients of earth pressure and the limit deformations required to reach them. The former can be obtained from the analytical solutions given in this Section, 3.7. The latter are usually estimated by experience.

In the absence of other data, it could be estimated that:

$$|\delta_p| - |\delta_a| = 2 \text{ to } 5\% \text{ de } H \qquad \text{(difference in absolute values for } \delta_p \text{ and } \delta_a\text{)}$$

depending on whether the ground is more or less dense.

Given the inaccuracy that will normally exist in determining the deformability parameters, analyses of sensitivity to variation in deformability should be run.

These computation models do not only provide movements in the diaphragm wall but also the diagrams of stress resultants. These diagrams can be useful in the structural design.

The structural design of diaphragm walls and their anchors will require applying procedures not included in this ROM 0.5 recommendations, which deal only with geotechnical aspects. The ideas included, however, may help engineers in these other analyses.

3.8 SLOPE STABILITY

3.8.1 Introduction

This part of ROM 0.5 covers the general analysis principles and includes some details of the most usual computations associated with the study of overall ground stability.

Ground instability is revealed by major displacements that cause total or partial collapse of the works. Some types of instability have already been looked at in other parts of this ROM 0.5, including:

- Bearing failure or sliding in shallow foundations.
- Bearing failure or ground failure owing to horizontal forces in deep foundations.
- Bearing failure or sliding in retaining walls.

Other general ground failure mechanisms are possible along surfaces of various shapes, depending on the geometry of their weakest zones. These failure mechanisms and the recommended methodology for analysing them are considered below under the generic heading of "slope stability". The reason is that, apart from the three cases mentioned (shallow foundations, deep foundations and retaining walls), overall instability usually occurs when there is a major difference in ground level, either natural or artificially created by excavation (open cuts), fill (embankment slopes) or by mixed procedures.

Slopes may simply consist of rockwork or earthworks with no structural elements, but may also consist of composite structures formed by the ground itself and adjoining structural elements. Examples of the latter include shallow or deep foundations on slopes or in zones close to slopes. The same type corresponds to the retaining elements usually employed to ensure overall stability, such as gravity or flexible diaphragm retaining walls (cantilever or anchored), bolts for retaining wedges in rock, etc.

In all these cases where there is a difference in ground level, with or without associated structures, it will be necessary to consider the limit states mentioned below and follow the general principles shown in each case when analysing them. The calculation methods suggested apply to the specific cases referred to when describing each method.

3.8.2 Most Common Failure Modes of Slopes Associated with Ultimate Limit States

3.8.2.1 Loss of Overall Stability

The overall stability of particular works is lost when the shear stress needed to maintain it exceeds the shear strength of the ground. This, furthermore, occurs along a failure surface that divides the works into two parts, so that the region between the external surface and the failure surface will slide over the rest.

This slide can be abrupt (a displacement of several metres in a few minutes) or slow (creep), mainly depending on the ground characteristics.

Loss of overall stability in earthworks (cut or fill slopes) and, especially, in composite works with earthworks and adjacent or nearby structures, is a serious accident implying total collapse of the part of the works that has moved. An essential task of works design is to study this Ultimate Limit State and to explicitly demonstrate that the degree of safety reached meets the minimum requirements laid out in Subsection 3.8.6.

Figure 3.8.1 includes various diagrams of the failure modes associated with slope stability that are covered in this part of ROM 0.5.

3.8.2.2 Deformations

Ground deformations in zones close to differences in ground level or slopes can be substantial and give rise to Ultimate Limit States in structures located in the vicinity, before reaching the Ultimate Limit State of loss of overall stability, while the slope actually is still in good serviceable condition.

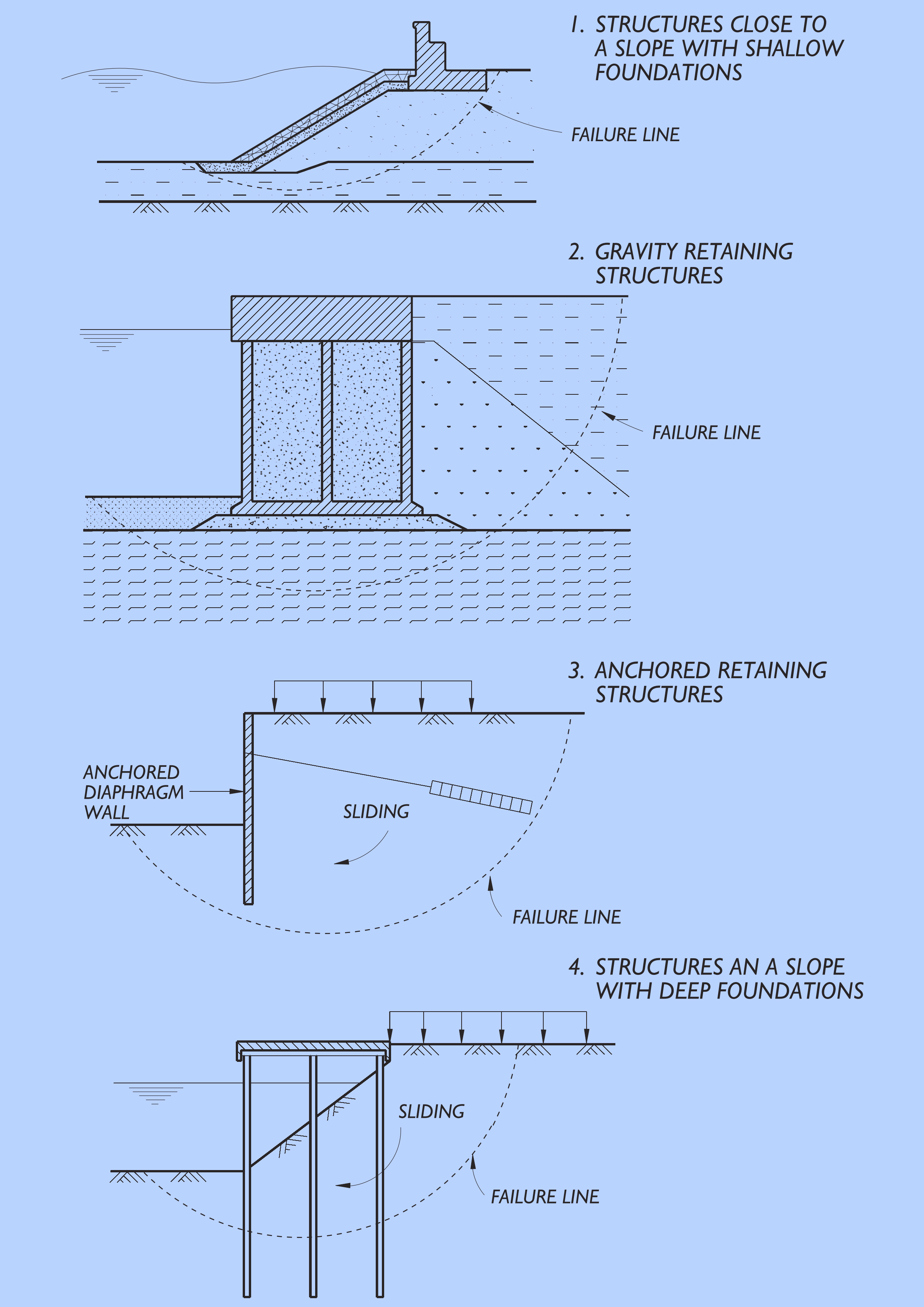
Studying these deformations or placing the structures sensitive to them away from the area affected by the slopes is one aspect to be taken into account in design work. The basic criteria set out in Subsection 3.8.7 should be followed to analyse this deformation problem.

3.8.2.3 Erosion

Water movement can lead to the destruction of slopes and the structures depending on them, either by external erosion degrading their geometry or by internal erosion entraining materials and causing local bearing failure or settlement.

Both processes can be considered as causing an Ultimate Limit State of progressive collapse, since they can cause loss of equilibrium of the slope itself or of the adjacent structures.

Figure 3.8.1. **Diagram of Various Ultimate Limit States Caused by Loss of Overall Stability**



Erosion processes must be analysed following the general principles set out in Section 3.4 of this ROM 0.5 and, if necessary, adequate preventive measures taken.

3.8.3 Most Common Failure Modes of Slopes Associated with Limit States of Serviceability

Both movements and erosion could be compatible with the serviceability requirements of the works if they are moderate in extent.

Limits for allowable displacements should be laid down in the design specifications of each particular project. Section 3.8.7 sets out the general principles for calculating deformation that could be used to estimate movements near slopes for the purpose of these checks. Allowable limits for deterioration caused by erosion, which should be stipulated in the design criteria, must be guaranteed by preventive measures that will not generally be amenable to quantitative analysis.

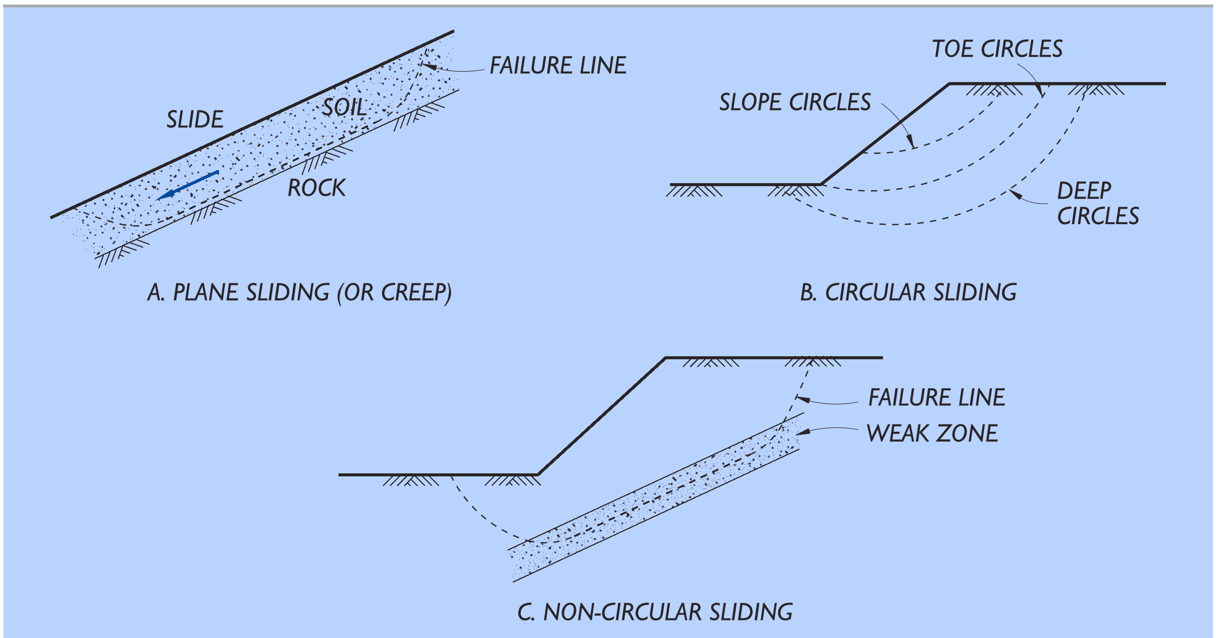
3.8.4 Overall Stability Analysis in Soils

3.8.4.1 Types of Slide

Different quantitative analysis techniques have been developed on the basis of observations of the shapes of failure surfaces when the Limit State of loss overall stability has been reached in soils.

For calculation purposes, the most common types of sliding surface can be assimilated to one of the three kinds described below (see Fig. 3.8.2).

Figure 3.8.2. *Types of Slides in Soils*



a. Sliding along Plane Failure Lines Running Parallel to the External Ground Surface

This type of failure is frequent on natural hillsides where the soil covering the underlying rock or firm soil slides along a surface that, for the most part, runs parallel to the external edge of the natural ground.

Sliding can be caused by excavation works (sliding of the area above the excavation) or by works that increase the load (structures or fills), thus causing the added load and the lower part of the bank to slide. The upper part may also slide as a result.

Slides can also occur naturally (owing to periods of rainfall, for example).

b. Circular Slides

Circular sliding is typical of homogeneous soil formations.

The upper area sliding over the lower one along a circular line is the only kinematically possible movement for a rigid body, which logically includes plane slides as a limit case.

Depending on the depth of the failure surface, circular sliding can occur along *slope circles*, which partially affect it, along *toe circles* passing through the toe of the slope, or along *deep circles* that exit the slope beyond its toe (see Fig. 3.8.2).

Failure surfaces in heterogeneous soils, either in cut or fill slopes, are normally similar to slides with circular sections and so the hypothesis of circular failure is adopted in most situations. This does not mean that other types of trial failure lines do not need to be analysed in cases where the ground structure suggests other possible non-circular shapes, which could be more critical.

c. Non-Circular Slides

On some occasions, the ground configuration, particularly the arrangement of various weaker zones, makes it necessary to consider failure surfaces with a plane section that cannot be approximated to a circumference.

3.8.4.2 Characterising Design Situations

In order to analyse an overall stability problem, it is necessary to know its geometrical configuration, the external actions that could affect it and the strength characteristics of the ground.

a. Defining the Geometrical Parameters

Apart from exceptional cases, the geometry of the problem should be defined by a representative plane section. If there is doubt as to which plane section could be the most critical, several sections should be analysed to select the most critical one, which must fulfil the requirements of minimum safety factors indicated in Subsection 3.8.6.

The elements defining the geometry are, among others, the distribution of soils of different nature and strength and an adequate representation of the structures, if they intervene in the overall stability analysis.

The contacts between different ground types should be identified with caution. Weaker layers should be represented as having the greatest thickness they could reasonably be expected to have. In this respect, the possible beneficial effect of the penetration of rockfill dumped over soft soils should not be taken into account unless this aspect is duly monitored and justified by the engineer.

A key element in the geometrical configuration of the subsoil is the description of groundwater, i.e., the phreatic surface in the case of a hydrostatic state or the configuration of the flownet if the water is moving.

It is advisable to carry out analyses of sensitivity to the geometrical conditions in cases where they cannot be defined with enough precision.

b. Defining Actions

The main actions in the study of overall stability problems are gravitational. These loads should be calculated, in the part corresponding to the ground, as the product of the corresponding areas by the specific weights of each one of them. These unit weights should be the average value of the results obtained from the geotechnical investigation, as was stated in Subsection 3.3.5.2.

The loads transmitted by the structure and the overburdens that may affect stability should be evaluated as the product of their nominal or characteristic values by the compatibility factors, ψ, corresponding to the combination of loads under study and by the load factors corresponding to them as a function of the action type and failure mode analysed (see Subsections 3.3.5.3, 3.3.5.4 and 3.3.6).

The study of overall stability should be carried out for the quasi-permanent load combination and for all relevant fundamental, accidental and seismic combinations.

c. Defining Ground Properties

Construction works in the area surrounding the slope or the action of the loads will modify the stress state of the ground, which must deform to adapt itself to the new conditions. This adaptation takes a certain time, which will increase with the deformability and impermeability of the ground. The ground strength will change during this adjusting process and this introduces an element of uncertainty when deciding on the strength characteristics to adopt in the stability analysis.

In this situation, engineers must consider two extreme hypothesis known as *undrained* and *drained* situations. The real situation will be more or less near one of these extremes depending on the circumstances (see Subsection 2.2.7).

Undrained Analysis

This extreme hypothesis corresponds to the theoretical situation in which the ground strength has not accommodated to a new stress state and retains the same resistance it had initially. Construction processes (filling, excavation, erecting structures, applying loads, etc.) and load application are assumed to take place over a period in which the ground does not have time to adapt itself to the new situation.

The undrained situation can be close to the real behaviour of saturated ground if it has a low consolidation coefficient (or low permeability).

Subsections 2.2.7, 3.4.11 and Section 3.10 provide guidelines on the types of ground and design situations in which it is recommended to assume this undrained condition.

Ground strength in undrained situations can be represented by the following cohesion and friction parameters:

$$c_u = s_u$$
$$\phi_u = 0$$

where:

s_u = undrained shear strength.

Part 2 of this ROM 0.5 deals with some procedures (field and laboratory tests) that engineers can use to determine adequate values for undrained shear strength.

When it comes to fixing the loads and strengths to be included in stability analyses, the recommendations in Section 3.3 apply.

Drained Analysis

Calculating stability in the drained situation corresponds to the equally extreme hypothesis that the ground strength accommodates completely to the stress state of the subsoil. The transient excess pore pressure generated by the construction operations or by the loads applied will have dissipated.

This type of calculation will always be indispensable, irrespective of the nature of the subsoil.

The ground strength parameters for these calculations should be obtained using the testing techniques shown in Part 2 of this ROM 0.5.

As a general rule, the shear strength in drained situations should be represented by two parameters:

c = cohesion.
ϕ = angle of friction.

These parameters may vary at each different level of the ground.

In cases where this aspect is particularly important, the strength parameters can be considered as variables depending on the stress state of the ground (by simulating non-linear failure criteria with angles of friction that decrease with pressure).

Subsection 2.2.8.1 provides some supplementary explanations on these parameters.

The recommendations given in Section 3.3 need to be taken into account when fixing the most adequate values of the strength parameters for verifying safety.

These parameters should, in any event, be obtained by laboratory tests carried out on fully saturated specimens. If indirectly obtained, they should correspond to this condition of total saturation.

In slopes above the water table, it could be assumed that the soil will not be saturated in the long term and, consequently, a greater resistance could be relied on (c_{ap} and ϕ_{ap} as defined in 2.2.8.2). This assumption is not to be recommended because in most maritime and port works full saturation can occur sometime during their working life.

d. Contribution from Structural Elements

Failure surfaces will, for the most part, develop through the ground but could affect elements of the structure itself in some cases.

In cases where the potential failure surface analysed involves the shear failure of some structural element, its resistance can be considered as an external load. It should be taken as the lower of the following two values:

1. The force that the structural element is capable of withstanding, divided by the distance separating this structural element from adjacent counterparts in a direction perpendicular to the calculation plane.

2. The resistance that the structural element opposes to the plastic flow of the surrounding ground. This resistance should also be divided by the spacing indicated in the previous paragraph.

If the structural element in question is not repeated at regularly spaced intervals, the design spacing can be taken as equalling the height of the slope, which, for these purposes, is defined as the difference in level between the two points where the failure line intersects the external surface of the ground.

When engineers come to evaluate the contribution of possible structural elements on the overall stability of the works, they should exercise caution and estimate these resistance values with distinctly conservative criteria.

In this respect, it is reasonable to assume that the safety level of the structural elements will be at least equal to –and preferably much greater than– the degree of safety of the slope.

3.8.4.3 General Principles of Calculation

a. Definition of Safety

The safety factor of a slope against sliding is defined as the number *F* by which the strength parameter values should be divided to reach the failure condition.

The values of the strength parameters strictly necessary to maintain equilibrium will be a fraction of the actual values. When the Mohr-Coulomb strength criterion is used, for each of the possible ground materials involved in the stability, this means:

$$c_{necessary} \leq 1/F \cdot c$$
$$\tan \phi_{necessary} \leq 1/F \cdot \tan \phi$$

b. Considering the Effect of Water

The effect of the presence of water in slopes can be represented in several ways in calculations. All of them do not always give the same results and it is therefore a good idea to take the following recommendations into account.

In any event, studying the effect of water requires a previous analysis of the distribution of porewater pressure in the ground by means of the relevant flownet or other methods for analysing water flow in porous media (see Sections 3.4 and 3.10). If no gradients of hydraulic potential exist, the pressure regime will be hydrostatic and it will suffice with determining the position of the groundwater table.

Except in some extreme cases (completely submerged slopes and some computations where $\phi = 0$), the presence of water implies calculating the problem of a heterogeneous slope. The Taylor charts, shown later on for homogeneous ground, will not apply.

Slopes under a Hydrostatic Regime

When a slope is subjected to a hydrostatic regime there are two calculation alternatives. Total weights can be used, introducing uplifts in the calculation afterwards (Alternative 1); or else effective weights can be calculated using the submerged specific weights below the water level, without taking uplifts into account (Alternative 2).

If free water is present, it must also be considered in Alternative 1 as one more material with weight and with no shear strength.

Both alternatives are, in principle, correct, and lead to the same results, provided that the pressure between slices is assumed to be horizontal. The path of the failure line through the free water does not affect the results, which can in fact vary under other calculation assumptions.

In any event, Alternative 2 is recommended as a general procedure for the sake of its simplicity. If Alternative 1 is used, it is advisable to take the pressure between slices as horizontal or at least severely restrict its inclination.

Slopes Subject to Seepage

When hydraulic gradients are present, calculations can be done with total weights, taking into account the external pressures and internal uplifts due to water (this will be equivalent to Alternative 1 in the preceding section). Any free water that may be present must be considered as one more material with weight but with no shear strength.

If there is free water (totally or partially submerged slopes but with water gradients), another calculation procedure can be used. The free water level, which may not be well defined, could be at any elevation and, in addition, may not necessarily be horizontal. As a general rule, the free water level should be chosen to coincide with the unconfined surface of the flownet corresponding to the design situation under study. The effective weights should be calculated, i.e., using submerged unit weights below the unconfined water level and apparent specific weights above it. Any possible uplift and water pressure existing above the unconfined water level must be computed. Below this level, only the difference between the actual water pressure and the hydrostatic pressure corresponding to the unconfined water level should be computed as uplift and as external pressures. To carry out the calculations, either the pressure differences just mentioned should be used directly or else, if it seems more convenient, these pressure differences should be replaced by the corresponding seepage forces (see Subsection 3.4.5). This method, sometimes known as calculation in effective pressures, done either with pressure differences or with equivalent seepage forces, would be the equivalent of Alternative 2 mentioned above.

Alternative 2 is always to be recommended. When using Alternative 1, the above comments for the hydrostatic regime also apply.

When calculations with $\phi = 0$ are made, to know the flownet is not necessary. The excess pore water pressure will not affect the results.

Slopes in Breakwaters

In these cases, the water action is complex and inertia forces cannot be disregarded. See Section 4.7 and the ROM 1.1 publication for these situations.

c. Simplifying Hypothesis

Calculating the safety factor will generally require knowledge of the stress state of the ground. This is difficult to obtain and some simplifying hypothesis are needed.

Every calculation method makes some simplifications in this respect. Engineers must know them, particularly when using computer programs as design tools. Not all the usual simplifying assumptions are acceptable.

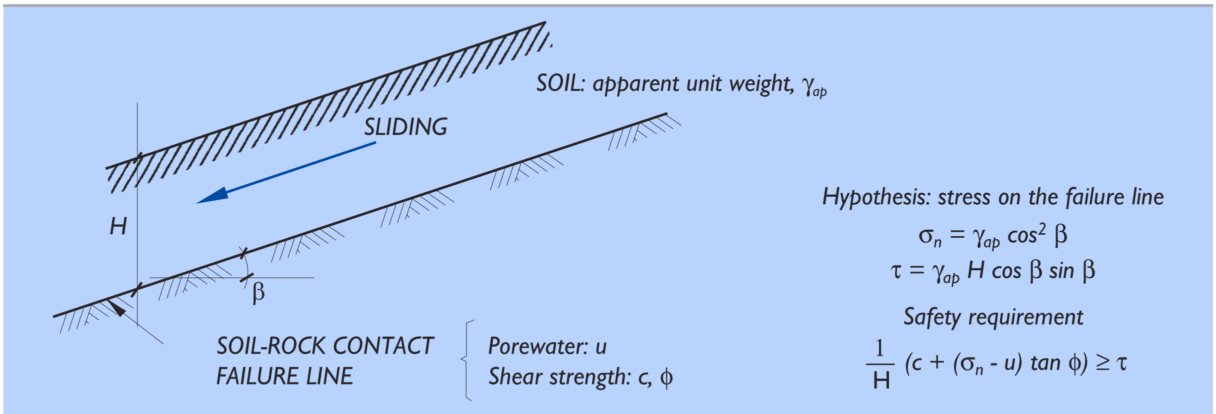
Some calculation methods are described below which are recommended for the different situations that can arise in analysing stability under different design situations. Subsection 3.3.9 contains some remarks on the application of more complex numerical models.

3.8.4.4 Indefinite Plane Sliding

This name is given to the sliding of soil over rock or other firmer soils along surfaces roughly parallel to the external surface of the ground.

When the failure line is sufficiently elongated, the conditions on the upper and lower boundaries can be disregarded and safety verified as shown in Figure 3.8.3.

Figure 3.8.3. ***Checking Safety against Indefinite Plane Sliding***



The data required for the calculation are the average apparent unit weight of the soil and the strength characteristics of the ground in the area close to the failure line, which tends to be the contact between weak and firmer ground.

The method can be applied to studying the stability of natural hillsides and can also be used to analyse artificial slopes where a weaker soil layer covers a more resistant formation (either artificial or natural).

Generally, the method will be applicable when the sliding mass length is much larger that its thickness.

3.8.4.5 Study of Circular Sliding

Circular failure lines, besides being close to the shapes actually observed in practice, have the additional advantage of considerably simplifying calculations compared to other potential failure shapes.

Among the calculation methods based on the circular failure assumption, the methods referred to below can be recommended.

3.8.4.5.1 Friction Circle Method

When the ground is homogeneous or slightly heterogeneous so that its shear strength can be represented with sufficient accuracy by average values of the strength parameters, the friction circle method can be applied.

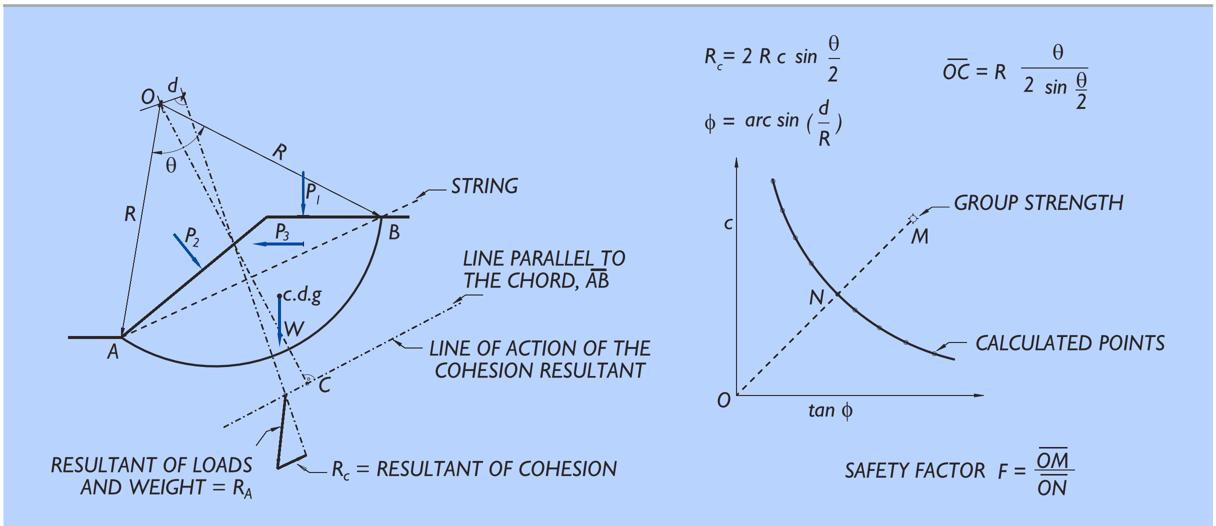
This method requires analysing several trial failure lines and checking that the corresponding safety criterion is fulfilled for each one of them.

Several calculations should be done for each failure line, with a view to obtaining pairs of c, ϕ values leading to a situation of strict equilibrium. With these pairs of values the stability diagram can be drawn (bottom part of Fig. 3.8.4). This diagram is the geometric locus of the values for c and $\tan \phi$ that lead to a safety factor for the slope equal to one.

Each of the trial calculations should be done assuming an arbitrary value for the cohesion, c_i, in order to obtain the corresponding ϕ_i value.

Figure 3.8.4 shows the procedure to be used for calculating a particular failure line with a particular cohesion. For each failure line, the resultant of the actions, R_A, owing to self weight and possible external loads should be calculated first and, then, the resultant of the resistances due to the cohesion that has been assumed.

Figure 3.8.4. **Calculation by the Friction Circle Method**



The distance *d*, from the centre of the circle to the line of action of the force resulting from compounding the resultant of the loads and the resistance due to cohesion, makes it possible to calculate the angle of friction required for strict equilibrium corresponding to the assumed cohesion.

This angle is calculated by the following expression:

$$\phi_{necessary} = \arcsin\left(\frac{d}{R}\right)$$

By repeating the process for a series of cohesion values, a set of values for the necessary angles of friction can be obtained, one for each cohesion assumed.

With these pairs of values (*c* and *tan* ϕ from successive calculations), the curve of necessary strength shown in Figure 3.8.4 can be constructed.

A comparison between the actual strength (Point *M* of the diagram in Fig. 3.8.4) and the necessary strength, as shown in this figure, will provide the safety factor *F* corresponding to this particular failure line.

Once the calculation of the particular safety factor for a given failure line is complete, the next line is calculated and so on until a sufficient number of trial circles have been analysed to ensure that the location of the most critical failure line is known with enough precision.

The friction circle method suffers from one basic inconsistency. The stresses on the failure line, after subtracting the part due to cohesion, deviate by an angle ϕ from the normal and therefore pass at a distance of $d = R \cdot \sin\ \phi$ from the centre of the circle. This leads to assuming that their resultant will also meet this condition. The calculation of ϕ mentioned above is precisely based on this fact. But that fact is not true.

As a result of this inconsistency, the friction circle method can give rise to errors, particularly when the central angles, θ, shown in Figure 3.8.4 are large.

In spite of this, the friction circle method is the most adequate for studying homogeneous slopes subjected to different loads on their contour and interior, apart from the principal load of self-weight.

3.8.4.5.2 Taylor's Stability Charts

For homogeneous slopes with only gravitational loads, the verification of safety by the friction circle method has been solved and arranged in the Taylor charts, after calculating a sufficiently large number of trial failure lines.

This solution also takes into account a potential bedrock at some depth. This is of little significance in cases where the angle of friction is appreciable (>10°), but proves to be decisive when calculating the extreme case of undrained situations, where $\phi = 0$ is assumed.

In the particular case of $\phi = 0$, the chart shown in Figure 3.8.5 can be applied, from which the cohesion necessary for strict equilibrium can be obtained as a function of the geometrical slope data stated in the figure and the unit weight of the soil.

Taylor's chart also shows the type of circular line whose stability requires a greater cohesion (critical failure line). This line can be a toe, slope or deep circle. The generic shape of these three types is shown in Figure 3.8.2 and also in Figure 3.8.5.

Once the type of failure is known, the bottom part of this figure will enable the position of the critical failure line to be located.

When failure occurs through deep circles, their centre lies along the vertical line passing through the mid point of the slope and they are tangential to the bedrock.

By a dotted line, the same Taylor's chart shows the cohesion required for a slope to be stable along toe circles in situations where the most critical line proves in fact to be a deep circle. This information could be useful in some cases.

In trench excavations with slopes on each side, deep sliding cannot occur, particularly if the trench has a narrow bottom. The geometry of the trench itself will determine the kinematics of the failure. On such occasions or in similar circumstances, it is necessary to know the cohesion necessary to maintain equilibrium along lines that are not so deep-seated.

For the general case, where cohesion and friction do exist, stability can be analysed by the Taylor's chart shown in Figure 3.8.6.

Pairs of values for the strength parameters (c and tan ϕ) leading to strict equilibrium can be obtained from this chart. A diagram similar to the one shown in Figure 3.8.4 can be drawn with these pairs of values. This facilitates comparison with the actual strength parameters of the ground and therefore enables the safety factor to be calculated.

This chart makes it possible to know also the type of circle that led to the most unfavourable failure line in Taylor's calculations.

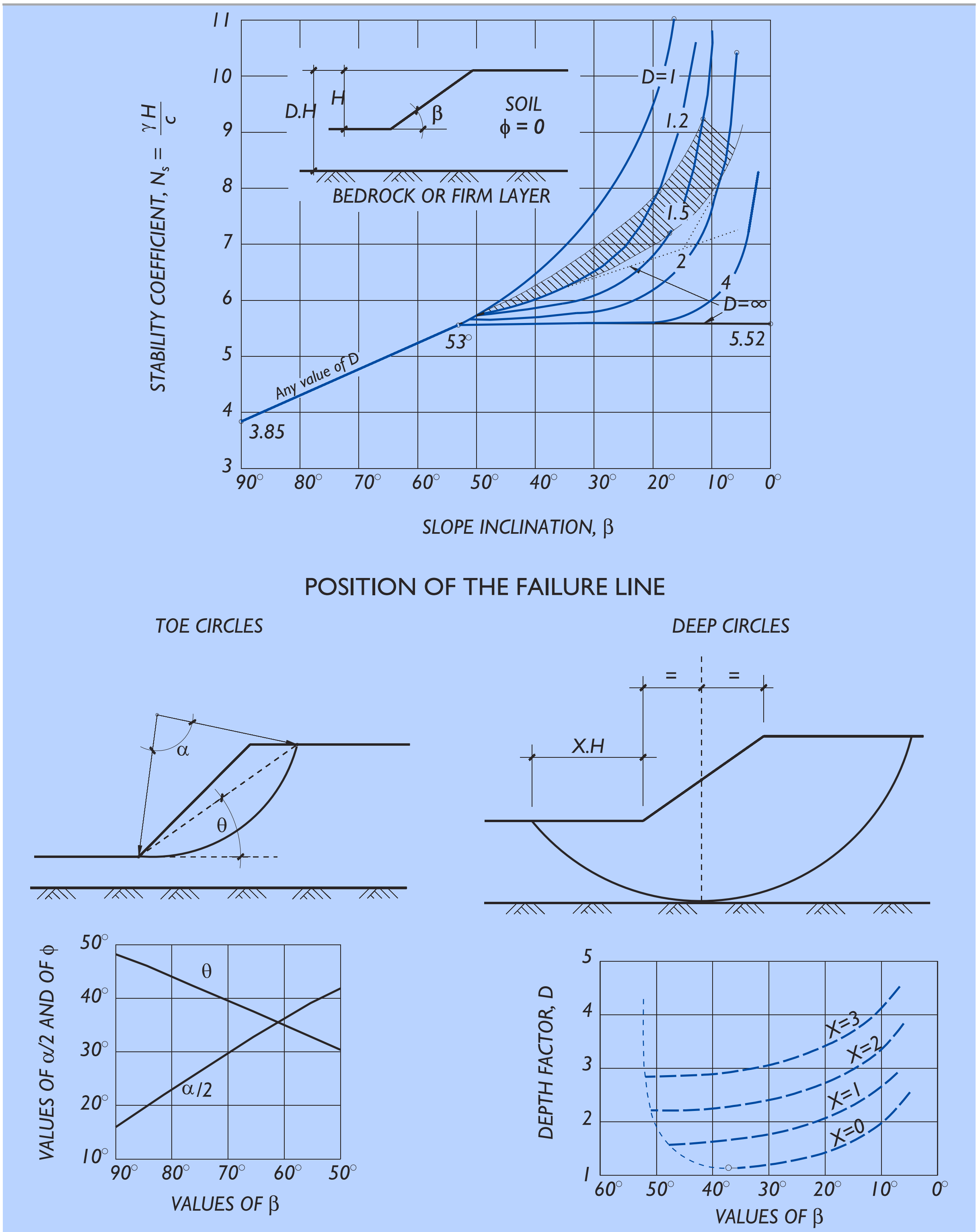
3.8.4.5.3 Method of Slices. Circular Lines

When the ground is far from being homogeneous and there is also water seepage within it, the most adequate calculation method is the slice method sketched in Figure 3.8.7.

Evaluating the safety for a particular circular line requires dividing the sliding mass into several vertical slices in such a way that a homogeneous ground can be assumed at their bottom and, in addition, their curved base can be represented by a straight line. For the degree of precision normally required, the sliding mass is frequently divided into at least ten slices.

Considering the vertical equilibrium in each slice and the overall equilibrium of moments about the centre of the circle leads to a definition of the safety factor as shown in Figure 3.8.7.

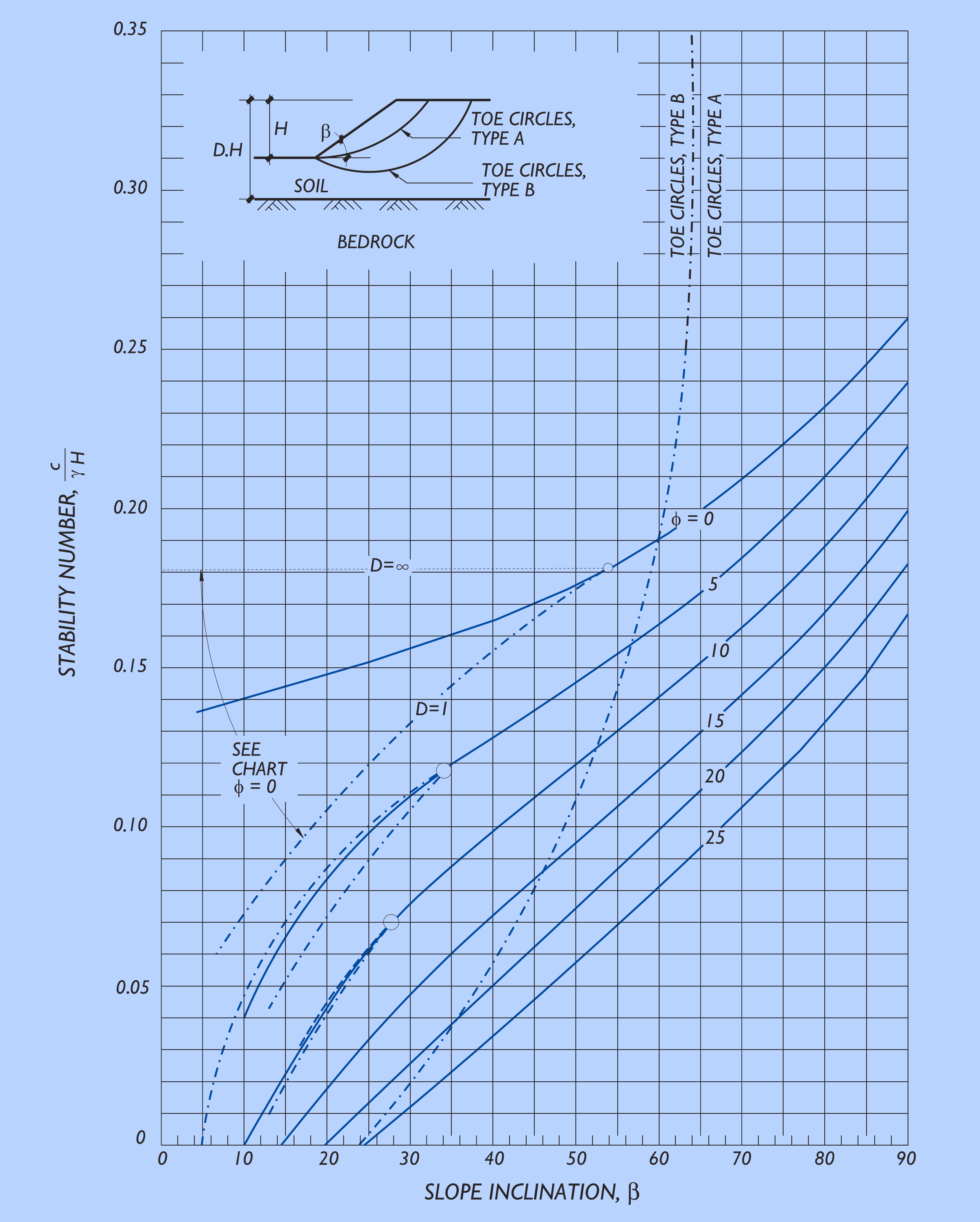
Figure 3.8.5. **Taylor's Chart for Puerely Cohesive Soil, $\phi = 0$**



N.B.: Failure will occur along toe circles in the shaded zone and the straight line extending it, for slope angles greater than 53°. Above this zone, failure will be due to slope circles and below it, to deep circles.
The dotted line indicates the stability of toe circles in cases where deep failure is more critical. This data can be useful for certain purposes (see text).
If and when failure occurs in the form of circles intersecting the slope itself (*slope circles*), they will be tangential to the bedrock.

As can be seen, the safety factor is still not entirely defined, as the forces ΔT are not known. Depending on the hypotheses used in calculating them, different computational procedures will result. Among the most

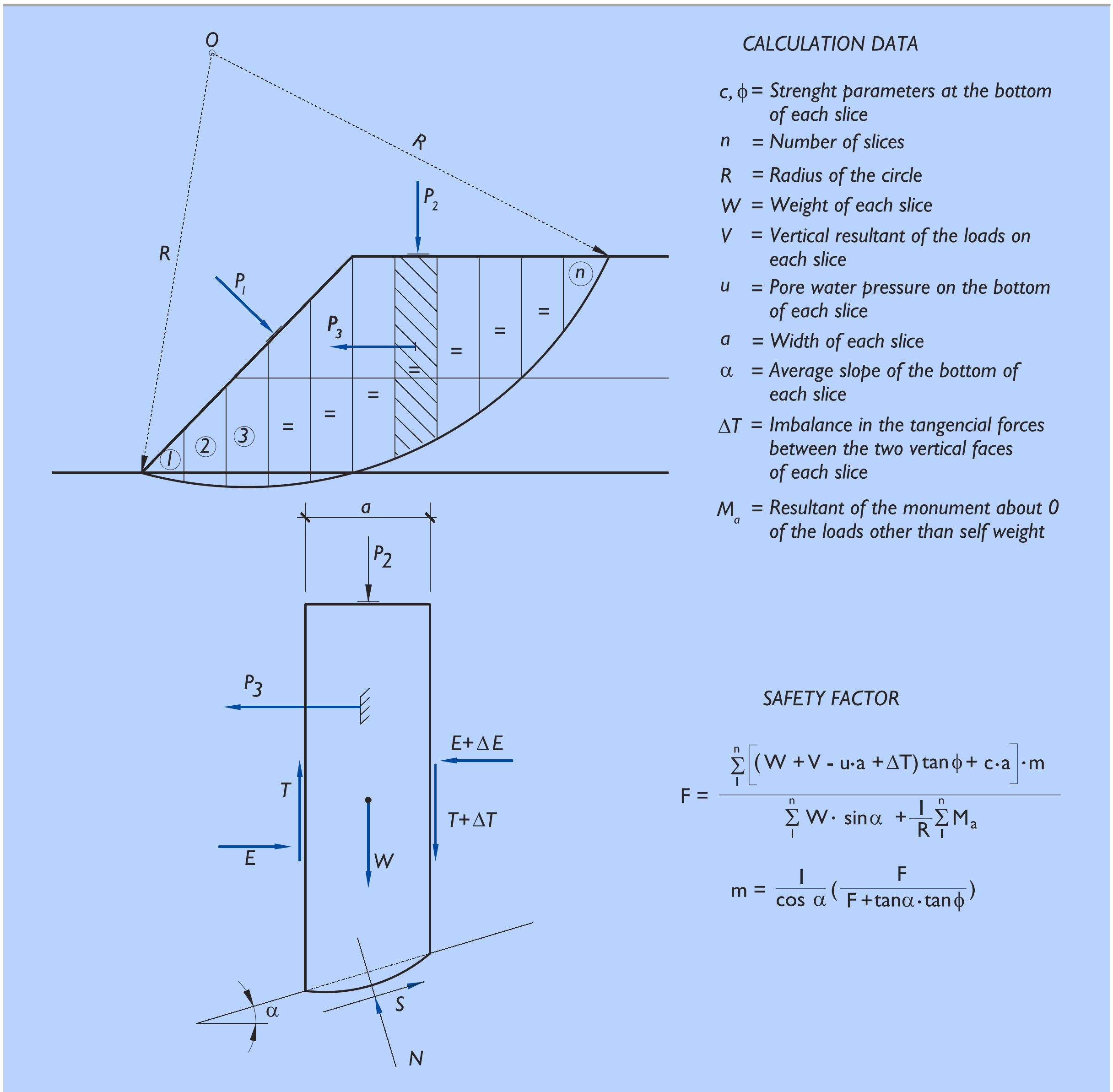
Figure 3.8.6. **Taylor's Chart for a General Case, $\phi \neq 0$**



N.B.: The broken lines shown for small angles of friction correspond to deep-circle failures (long dashes) and slope-circle failures (short dashes). For ϕ > 10° it can be assumed that failure lines are always toe circles.

reasonable hypothesis, it is worth mentioning that from the simplified Bishop's method, where $\Delta T = 0$ is assumed.

Figure 3.8.7. **Diagram of the Method of Slices for Circular Sliding**



Since the safety condition has not an explicit expression, the computation must proceed by trial and error. The iterative calculation will usually converge rapidly but care should be taken with the procedure when using a computer, since theoretically there are as many solutions to the problem (*F* values) as there are slices dividing the sliding mass. Commercial computer programs usually obtain, for each trial failure line, the largest *F* value meeting the safety condition. Computational problems may arise when slices appear with negative α values. In these cases, the factor *F* can converge to abnormally high values.

This effect can be particularly important in failure lines with a steep exit slope at the toe, when the ground is granular in this zone. In such cases, it is not advisable to assume that $\Delta T = 0$, as it could lead to optimistic results (on the unsafe side). It is therefore necessary to assume reasonable values for ΔT or to modify the shape of the failure line, at least locally.

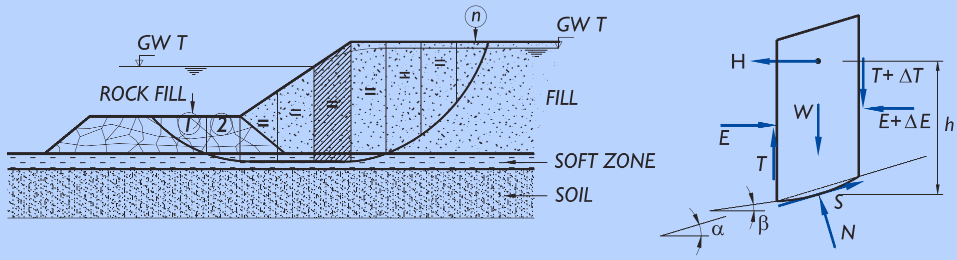
Subsection 4.7.4.5.1 provides an adequate procedure for overcoming this drawback in the study of breakwater stability. This procedure could be useful for other similar problems.

3.8.4.6 *Analysis of Non-Circular Sliding*

3.8.4.6.1 Method of Slices, Non-Circular Lines

In cases where the geometrical configuration of the ground suggests failure lines that cannot be easily assimilated to circles, the corresponding equations need to be modified, since the equation of overall moment equilibrium will take a different form in these cases, as shown in Figure 3.8.8.

Figure 3.8.8. Method of Slices for Non-Circular Failure Surfaces



OVERALL EQUILIBRIUM OF MOMENTS:

$$\sum\left[\left(E+\frac{1}{2}\Delta E\right)\tan\alpha - \left(T+\frac{1}{2}\Delta T\right)\right]\cdot a + \sum H\cdot h = 0$$

SAFETY CONDITIONS:

$$F = \frac{\sum\left[(W + \Delta T - u\cdot a + V)\tan\phi + c\cdot a\right]\cdot m}{\sum (W + V + \Delta T)\cdot \tan\alpha + R_h}$$

$$m = \frac{1}{\cos^2\alpha}\left(\frac{\gamma_R}{\gamma_R + \tan\alpha\cdot\tan\phi}\right)$$

DATA:

R_h = Resultant of the hoorizontal forces accting on the slope
For other data see Figure 3.8.7.

The solution to the problem is similar to the one described in 3.8.4.5.3. As in that case, some assumptions have to be made regarding the ΔT values. An iterative procedure is also needed to solve the problem and parasitic solutions also exist.

In such cases it is recommended either to assume $\Delta T = 0$ or else to try out some other reasonable assumption about them, but in that case the overall equation of moment equilibrium should be fulfilled.

The methods assuming a certain inclination of the pressures between slices:

$$\tan\delta = \frac{T}{E}$$

will only fulfil the moment equilibrium when [18]:

$$\int_L R\cdot\sin(\delta-\alpha)\,dl = \sum H\cdot h$$

where R is the resultant of *E* and *T*, i.e.:

(18) For moment equilibrium to exist, the contour integral of $\overline{R}$ along the slip line must be equal to the moment of the horizontal forces applied to the sliding mass.

$$R = \sqrt{E^2 + T^2}$$

Spencer's method assumes that δ is a constant value. This assumption is acceptable in general terms but may prove to be optimistic in certain circumstances. It is possible that the angle δ necessary to meet the equilibrium conditions is excessively high, particularly if horizontal forces are applied to the sliding mass in the unfavourable direction ($\sum H \cdot h > 0$). High δ values can greatly increase the resistance attributed to the toe of the failure line, especially when there is a high friction angle in this area. Spencer's method can be dangerous when ϕ is high and α is negative at the toe of the slope.

The method of Morgenstern-Price assumes that:

$$\tan \delta = \lambda \cdot f(x)$$

where λ is a constant to be determined and *f(x)* a function to be specified by the user. The variable *x* is the abscissa along a horizontal axis. The particular case of *f(x)* = 1 is Spencer's method commented above.

It is clear that the Morgenstern-Price method is a step towards a better solution. The λ value fulfilling the equilibrium equations is:

$$\lambda = \frac{\sum H \cdot h + \int_L E \cdot \tan\alpha \cdot dx}{\int_L f(x) \cdot E \cdot dx}$$

To avoid convergence problems, some commercial programs prevent the user from specifying negative *f(x)* values that could lead to a null value for the denominator of the preceding expression. And yet in some special cases (negative α values in areas with high ϕ, for example), the adequate solution demands that $f(x) < 0$.

There is an elementary alternative commonly used, consisting of not utilising failure surfaces with strongly negative values for α (limitation of the exit slope of failure lines in areas with high ϕ). Although this alleviates the problem somewhat, it would still not be fully solved.

Finally, it should be mentioned that the trial failure lines analysed must be smooth, without sudden changes of slope, with good kinematics. Fairly unrealistic surfaces together with certain hypotheses about ΔT values can lead to extraordinarily high safety factors but also to very low safety factors.

Since the method of slices does not have a single solution, it is advisable to use some reasonable simplification. If the aspect is important, at least the critical slip line found in the computations must be recalculated (by hand or with an auxiliary computer program) under different reasonable hypothesis about the direction of inter-slice forces. The overall moment equilibrium equation should be fulfilled in these trials, as experience has shown to be of maximum importance in these problems.

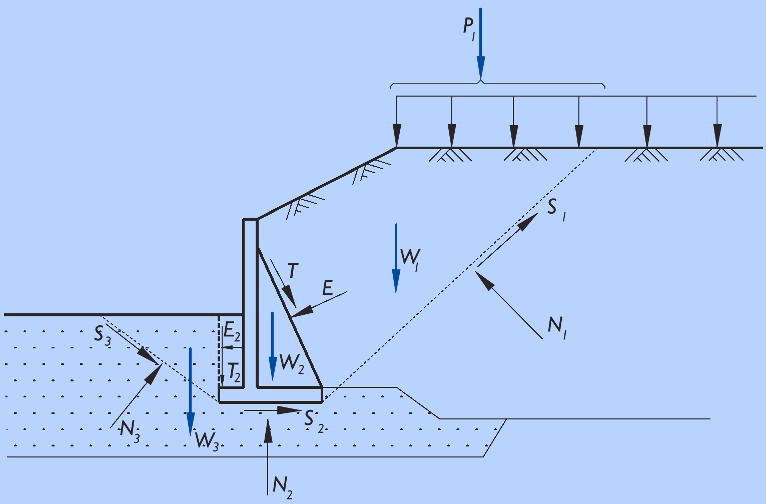
3.8.4.6.2 The Wedge Method

The sliding mass can be divided by non-vertical lines giving rise to wedges instead of slices, as shown in the diagram in Figure 3.8.9.

In such cases, a certain relative movement between wedges can be assumed and a criterion specified with respect to the ratios between the tangential component, *T*, and the normal component, *E*, on the division lines, as shown in that same figure.

This calculation procedure has generally an easier solution, as it does not require dividing the whole area into a large number of wedges, and this consequently leads to a limited number of equations.

Figure 3.8.9. **Diagram of the Wedge Method**



EQUATIONS FOR "n" WEDGES

EQUILIBRIUM OF EACH WEDGE	2n	3n
FAILURE CONDITION	n	

UNKNOWNS FOR "n" WEDGES

N,S	= REACTIONS AT THE BASES	2n	4n-1
E,T	= REACTIONS BETWEEN WEDGES	2n-2	
F	= SAFETY FACTOR	1	

ADDITIONAL HYPOTHESIS NECESSARY n-1

$$F\ T = E \tan\phi + c\ L$$

Trials for potential critical failure shapes should cover not only the location of the failure line but also to the inclination of the failure planes between wedges.

This method can have the advantage over slice methods of a better representation of the kinematics of the movement –in some specific cases– as well as being easier to calculate. Nevertheless, as the overall moment equilibrium cannot be set out, at least in a realistic manner, lower accuracy should be expected than in the case of the method of slices. Therefore, its use must be more restricted.

3.8.5 Calculating Overall Stability in Rocks

The instability problems in rock masses in the vicinity of substantial differences in ground elevation (slopes) are a special case, since rock masses generally show a failure behaviour that is highly conditioned by the existence of planes of natural weakness in their stratification joints (sedimentary rocks) and in the joints existing in all types of rock.

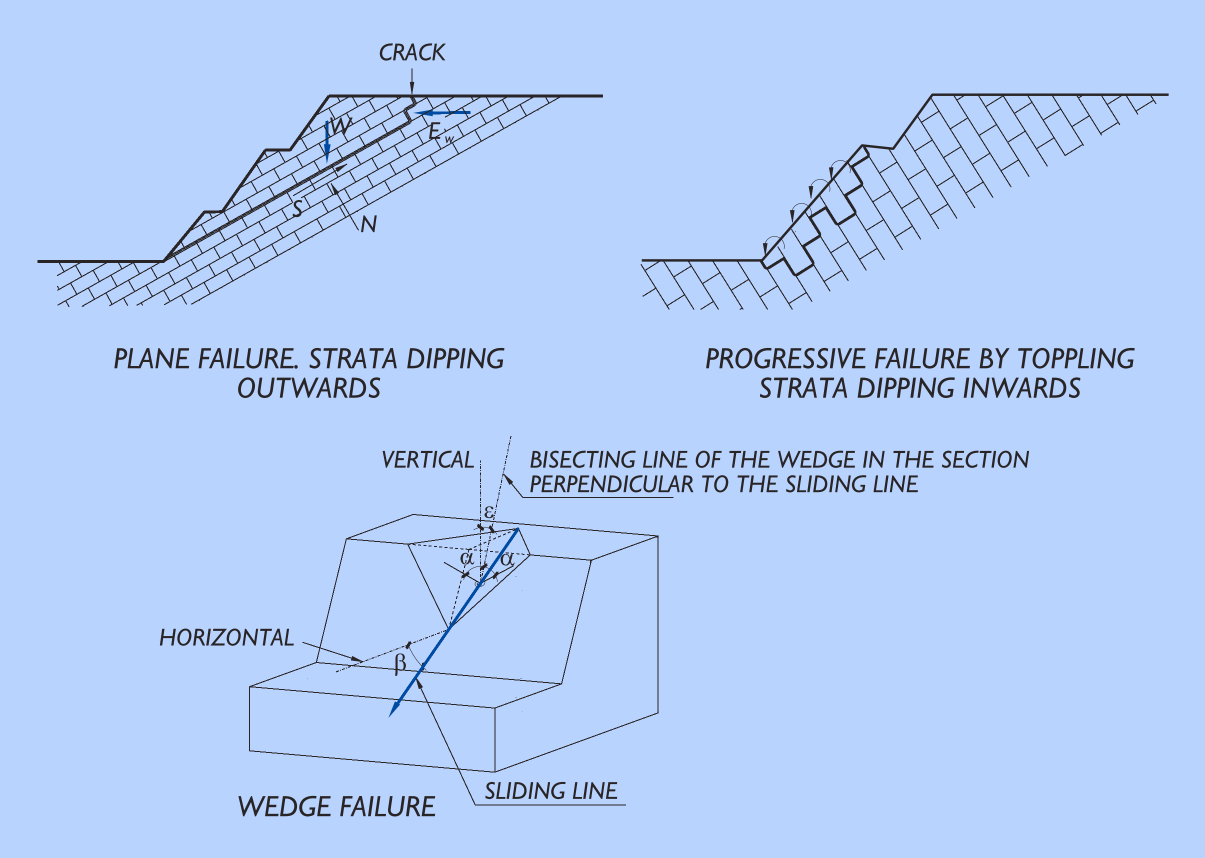
In highly jointed rock, i.e. with small joint spacing –compared with the dimensions of the works being carried out– and affected by joint sets with different orientations, and in highly weathered rock, it is sufficiently conservative to use the procedures given in Section 3.8.4 for analysing overall stability. To this end, an equivalent homogeneous ground should be assumed, with a shear strength equivalent to that of the weakest joints in the rock mass.

In fresh rock, particularly if it is lightly jointed, analysis procedures specific to rock mechanics should be used to evaluate safety against overall instability for works carried out in the surrounding area. The following sections provide some basic ideas on these analysis procedures.

3.8.5.1 Types of Instability

Depending on the relative orientation of the main discontinuity plane of the rock mass (stratification, schistosity, etc.) and for the purpose of defining the most adequate analysis procedures, three typical cases can be found, which are illustrated in Figure 3.8.10.

Figure 3.8.10. Basic Types of Instability in Rock Slopes



When the direction of the plane of the slope runs substantially parallel to the stratification (or to the main discontinuity) and, in addition, the stratification dips at an angle less than that of the slope plane and of the same sign, so that the stratification dips outwards, the most critical failure mechanism is plane.

Where the direction of stratification (or main discontinuity) runs roughly parallel to the slope but dips inwards, the most critical failure mechanism will usually be *toppling* or overturning of the blocks.

In the most typical case of slide, however, the directions of the main discontinuity planes and the direction of the slope plane are far from parallel. In this case, the instability can occur in the shape of a wedge sliding along two discontinuity planes with their line of intersection dipping outwards.

Apart from these three most basic types of failure, there are others with various shapes, such as:

- Block fall as a result of erosion which undermines softer underlying rock layers. This is typical in a sub-horizontal sedimentary series of alternating strata with different strength.
- Buckling of quasi-vertical strata on slopes excavated roughly parallel to their stratification.

- Complex slides in the area surrounding faulted zones.
- Sliding along particularly weak layers that may exist within the rock mass and lie close to the excavation slope.

Each type of instability should be examined with the same calculation principles, as shown below. These principles should consider the potential effects of external actions that might be caused by works carried out close to the rock slope or constructed on the slope itself, as well as the stabilising effects caused by possible retaining elements.

The most traditional stabilisation procedures in the case of fresh rock consist of anchors adequately oriented, whose design requires sliding stability analyses that must be carried out in accordance with the basic principles set out below.

3.8.5.2 Characterising the Design Situation

The data required for studying the stability of jointed rock masses relate to their geometry, actions and the shear strength along joints.

a. Defining Geometrical Parameters

The most important geometrical parameters, besides those necessary to describe the works, relate to the orientation of discontinuities in the rock.

Given the three-dimensional nature of the problem, its geometrical description tends to be complex. Special procedures are required for representing the distribution of joints in an intelligible way. In this case, it is advisable to use a stereographic projection of equal area to represent the joints identified in the ground. Each joint will be represented in the diagram by a dot and a study of the field-observed dot clusters will provide the regular orientation of the different joint sets.

In addition, it will be necessary to know the continuity, spacing between consecutive joints, etc. for each set of joints. In complex cases or those of some importance, it is advisable to use probabilistic descriptions of the jointing of the rock mass.

An essential data in analysing the stability of jointed rock masses is the description of groundwater, since it will be necessary to know the water pressure in the most critical joints, which will have a very important effect on the safety of the works.

The geometrical data to be adopted in a particular design situation should be a reasonably conservative simplification of reality. A large portion of the works safety will be determined by this geometrical modelling and, therefore, it is recommended to carry out analyses of sensitivity to the least known geometrical variables in important cases.

b. Defining Actions

Other actions can be present in the design situation under analysis apart from self-weight, such as overburdens, partial weights from the structure, etc. These loads should be taken into account following the basic guidelines indicated in 3.8.4.2.b.

Slope stabilising or retaining elements will generally be of the following two types:

a) *active* elements, which are usually formed by long flexible anchors, installed with a substantial initial load compared to their ultimate load;

b) *passive* elements, which are usually formed by short rigid bolts installed with a low initial load compared to their failure load.

In calculations of overall safety along failure lines intersecting active elements between their head and their bonded length in deep rock, the anchor can be represented by a force equal to its installation load, applied at the position of its head on the rock slope. For failure lines encompassing the anchor bonding length, the presence of the element must be taken to have no effect.

The presence of passive elements in overall stability calculations should be represented as a force equal to the pullout resistance of the part of the element outside the area that can slide.

The two types of elements are considered in a different way due to the deformation required to mobilise the resistant force. This would theoretically be infinite for passive elements and null for active elements. If accurate calculations are required or in situations where there are elements of an intermediate type between the two mentioned, the load due to their presence can be represented by the forces corresponding to reasonable deformations that engineers will have to determine.

c. Defining Ground Properties

Except in very exceptional circumstances, the strength of the fresh rock matrix will not play a part in overall stability calculations. It is, however, a useful index for indirect assessment of the shear strength of the joints.

The shear strength of joints should be determined either directly by laboratory or field tests or indirectly following correlations previously determined by experience.

The engineer's judgement in this case is fundamental. The choice of one correlation or another or the decision to carry out specific tests will depend on the importance of the case and on the local experience on similar issues.

In any event, it will be necessary to define the shear strength parameters for each of the different sets of rock joints appearing in field explorations in order to make an explicit evaluation of the overall safety factor of the rock mass.

d. Contribution of Structural Elements

If the trial failure surface analysed cuts across other structural elements, their resistance should be estimated as if it were a case of a passive retaining element and this resistance should be considered as an external load.

3.8.5.3 General Principles of Calculation

In order to calculate the safety factor against overall instability of a rock slope, as many failure mechanisms as can be conceived with feasible kinematics must be tried out.

Each of these mechanisms will be defined by a failure surface separating the zone that would move from the zone that would remain at rest. Within the sliding mass, internal failures could also occur due to the slide of some areas over others.

The safety factor for a specific failure mechanism is defined as the factor F by which the shear strength on the trial failure surface should be divided to reach strict equilibrium in the design situation under consideration.

The safety factor in terms of overall stability of the works will be the lowest figure resulting from analysing several forms of failure and, with each form, considering different variants for the details of the specific location of failure planes.

The specific calculation for each failure line should be carried out in accordance with the basic principles of rational mechanics. To this effect, some special recommendations are given in the following sections for each of the three basic types of instability in jointed sound rock masses.

a. Plane Sliding

In this type of situation, it is advisable to project all actions on the directions normal and tangent to the plane of sliding.

At the head of the potential slide there could be a pre-existing tension crack full of water, with the corresponding hydrostatic pressure, which must be included in the analysis.

The normal component, *N*, and the possible uplift on the slide plane, S_w, will allow the shear strength to be evaluated:

$$\text{Strength} = (N - S_w) \tan \phi + c \cdot A$$

where:

c, tan ϕ = strength parameters on the slide plane.
A = area where N and S_w are acting.

The safety factor is obtained by comparing this strength with the tangential component of the resultant of the loads, *T*:

$$F = \frac{\text{Strength}}{T}$$

b. Toppling failure

If the problem is important, studying these cases will require some hypotheses on the details of the actual blocks of rock. There are specific calculation procedures that will enable these detailed analyses to be carried out.

As a general rule, it is considered sufficiently close to reality and safe enough to calculate the stress state in the area surrounding the critical point where it is expected that toppling may begin as if it were a case of homogeneous ground. To this end, different trial locations for this point should be analysed.

Defining the stress state may require using solutions published in the technical literature or a previous numerical calculation.

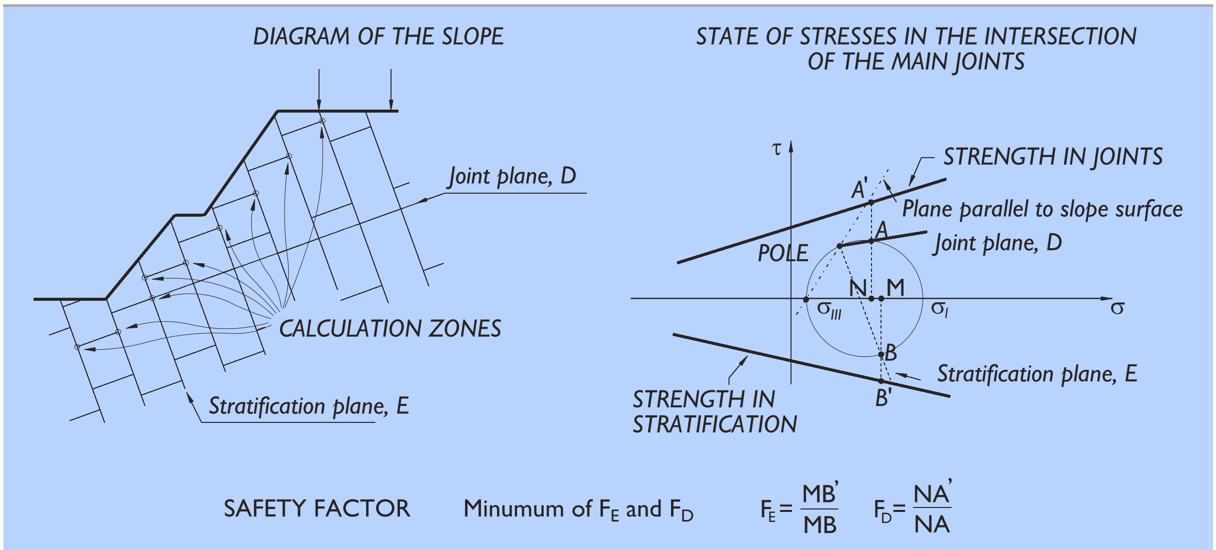
Having defined the state of stress at the point in question by its Mohr's circle as shown in Figure 3.8.11, the shear stress can be estimated on the joint planes and the corresponding safety factor evaluated with these data.

The minimum safety factor obtained in different evaluations (with different rock joints and in different zones) should be taken as the stability coefficient of the corresponding slope.

c. Wedge Sliding

Having identified a particular wedge formed by two planes, *A* and *B*, as kinematically possible, the mechanical problem will be statically determined when a reasonable assumption is adopted to distribute the loads on the wedge between the reactions over *A* and *B*. To this effect, it is reasonable to assume that the shear forces in the joints run parallel to the intersection of planes *A* and *B* (sliding line) and this is the recommended procedure.

Figure 3.8.11. **Simplified Procedure for Evaluating Safety in Jointed Rock Slopes**



It should be mentioned that in cases where neither cohesion nor uplift exists in the joints and where, in addition, both joints have equal friction, the natural safety factor (without external loads or retaining elements) is given by:

$$F = K \cdot \frac{\tan\phi}{\tan\beta}$$

where:

ϕ = friction in the joints.
β = slope of the line of sliding.
K = wedge factor.

The wedge factor is given by:

$$K = \frac{\cos\varepsilon}{\sin\alpha} \quad \nless 1$$

where:

ε = angle formed by the plane bisecting the wedge and the vertical plane through the slide line.
α = semi-aperture of the wedge dihedron..

These angles are explained in Figure 3.8.10.

3.8.6 Minimum Safety Factors against Loss of Overall Stability

The minimum safety factors required for Ultimate Limit States of overall instability should be appropriately established in the design bases for individual projects as a function of their importance and the consequences of a failure.

In works with a low SERI rating, for verifying safety against the failure mode of loss of overall stability, analysed in accordance with the foregoing criteria, the values included in Table 3.8.1 are generally acceptable as the minimum safety factors.

Table 3.8.1. Minimum Safety Factors Recommended against Loss of Overall Stability. Low SERI Works (5 - 19)

Load Combination	Safety Factors, F
Quasi-Permanent, F_1	1.4
Fundamental, F_2	1.3
Accidental or Seismic, F_3	1.1

For works with a minor or high SERI rating, or for other allowable failure probabilities, the minimum *F* values given in Table 3.8.1 can be adjusted as indicated in Subsections 3.3.8.2 and 3.3.10. They can equally be adapted for transient situations (including short-term geotechnical situations) in line with Subsection 3.3.8.1.

3.8.7 Comments on Deformations

As a general rule, rigid structures should not be installed in the vicinity of slopes, as the surrounding ground usually undergoes considerable deformation. In spite of this, in some cases it will be unavoidable to install structures in the area affected by slopes.

Movements in the area near slopes must be calculated by numerical procedures. Using methods based on the theory of linear or non-linear elasticity is recommended. The higher the overall safety factor, the more applicable these procedures are considered.

The deformations will be obtained, in any event, with little precision. Therefore a situation will be considered acceptable when the deformations resulting from the best estimate are at least three times lower than the ones that would damage the structure near the slope.

For the purpose of gaining knowledge and in cases where the deformational aspect is important, it is advisable to specify a careful monitoring of movements during construction as a preventive measure and to include the appropriate contractual provisions allowing the works to be adapted as a function of the results of the observations made.

3.9 GROUND IMPROVEMENT

3.9.1 General Aspects

The ground quality can be improved by different types of treatment, some of which are described in this part of ROM 0.5.

Under certain circumstances, treating the ground to improve one of its characteristics may be more advisable than adapting the design solution to the natural ground conditions.

A clear-cut example of this situation occurs in the construction of pavements over recent fills. In these cases, treating the fill and its foundation ground (if it can also be improved) could be more advisable than constructing a provisional pavement to be replaced later on. The two alternatives should be looked into and compared.

Similar comments apply to constructing lightweight installations on soft soils. The problem of founding them can be solved by deep foundations or, alternatively, by shallow foundations resting directly on the ground after some treatment.

"Post-event" ground treatment is virtually inevitable in some pathological cases.

Reference needs to be made to the exceptional case of anti-liquefaction treatment of soil deposits potentially unstable during an eventual earthquake or other similar vibrations. Treatment of the ground is nearly unavoidable here.

There are circumstances where soil improvement is out of the question. It will be easier and more cost-effective to adapt the design solution to the ground quality rather than improve this.

It is worth stating that the best solution for treating problematical ground is to excavate it and replace it with something better. This solution, adopted on many occasions, should always be considered as the first alternative.

Current techniques for ground treatment are aimed at increasing its strength, reducing deformability or lowering permeability, either throughout the whole of its mass or in local areas, be it in a permanent or transient manner.

The improvement procedures have been classified into several types for the purpose of covering them in this ROM 0.5. This classification is merely aimed at facilitating the comments and recommendations intended in relation to them. Classification is not easy, as there are mixed techniques sharing the characteristics of two or more of the procedures described.

It must also be said that there is no clear-cut separation between ground improvement and foundation work. This is the case with the rigid inclusions described in Subsection 3.9.6 and even some deep compaction procedures, described in 3.9.3, and jet grouting, described in 3.9.7.4. These procedures can also be considered as special cases of deep foundations with non-conventional piles. In this respect, the procedures set out in Section 3.6 of this ROM 0.5 could apply to the study of this type of ground treatment.

It must also be added that surface compaction is always adequate as standalone treatment or as a complement to other techniques.

3.9.2 Preloading

3.9.2.1 Improvements Resulting from Preloading

Ground subjected to a certain load will experience greater deformation the first time this load is applied than on subsequent occasions. The first loading will make the soil more resistant and less deformable even if this load is later removed. Clayey ground in particular is highly sensitive to this beneficial effect of preloads.

On the other hand, the beneficial effect of preloading may require considerable time in saturated ground. This time will increase with the thickness and compressibility of soft soil and will decrease with its permeability. For this reason, accelerated preloading with artificial drainage measures can be much more effective.

Preloads increases the ground density, increments its strength and reduces deformability, thus improving its conditions against potential stability problems and future deformation problems.

Preloading is such an effective procedure that it is always advisable to consider it.

In each particular case, depending on the specific site conditions, different beneficial effects will be achieved. A preloading study will enable a reasonable estimate to be made of such effects.

Table 3.9.1 shows approximate improvements that could be brought about by preloading on two types of soil, for guidance purposes only.

Table 3.9.1. ***Example of Possible Improvements Resulting from Preloading***

	Prior to Preloading	After Preloading
Soft Cohesive Soils		
Dry unit weight, γ_d (kN/m^3)	12	15
Oedometer deformation modulus, E_m (MPa)	2	10
Undrained shear strength, s_u (kPa)	10	20
Loose Sandy Soils		
Relative density, D_r (%)	40	70
Modulus of elasticity, E (MPa)	10	30
Internal angle of friction, ϕ	28	32

3.9.2.2 *Types of Preloading*

The most traditional type of preloading consists of placing an embankment on top of the soft natural soil or artificial fill that needs to be improved, in such a way that settlement occurs as the soil consolidates under the weight of the earth-fill. The earth-fill is then totally or partially removed.

Preloading can be carried out by other procedures, including:

- piling up concrete blocks or heavy goods.
- water loads in watertight enclosures (deposits).
- temporarily lowering the groundwater table by artificial means.
- using jacks and anchors in localised areas.

Accelerating the consolidation process by artificial drains is adequate in cases involving long consolidation times.

The drainage system commonly used consists of prefabricated *strip drains* (*wick drains*). Cylindrical sand drains were used in the past but are now considered less effective, although they could have a beneficial secondary effect as a reinforcing element.

3.9.2.3 *Prior Investigation*

Before designing any preloading, the ground must be investigated to determine its stratigraphy and geotechnical characteristics. Particularly recommended in this respect is a prior geotechnical investigation with continuous static penetration tests, especially measuring porewater pressures (piezocone or CPTU tests). Specifically, the following parameters should be known for each stratum or level:

- density and natural moisture content
- moduli of deformation
- coefficient of consolidation.

These data can be directly obtained by oedometer tests carried out on undisturbed samples. They can also be obtained indirectly by correlation with other tests.

It is also necessary to investigate the shear strength of the natural ground, not just to compare it with the corresponding strength after treatment, but also to analyse the stability problems that preloading could give rise to. High earth loads on areas of soft soil can lead to short-term ground failure. This problem must be studied because if it occurs it will mean bearing failure in the preloaded area, particularly at its edges.

Vane tests or static penetration tests are particularly useful for determining strength in these cases. Triaxial laboratory tests could also be required if the strength problem is critical.

In cases where there is a need to accelerate consolidation by including artificial drains, it is particularly important to consider the anisotropy of the soil in terms of its coefficient of consolidation c_v, since in many natural soils this is usually several times greater in the horizontal direction than in the vertical direction, which is the one commonly tested in the laboratory. This can be determined empirically by comparison with similar experiences or by special laboratory tests with horizontal flow.

Particularly recommended in this respect is the static penetration test with measurement of porewater pressures (piezocone, CPTU), with stops at different levels to analyse the consolidation of the excess porewater pressures due to the cone penetration.

3.9.2.4 Design of Preloads

The higher the intended efficiency, the higher the preloading intensity should be. The upper limit of preloading intensity is due to the stability of the ground itself, apart from economic considerations.

When deciding on the intensity of the surcharge (height of embankment, in the usual case of preloading with earth-fill), a check should be made that the embankment created is stable and that the movements it produces do not have harmful effects on nearby structures.

The degree of consolidation achieved by a particular preloading can be estimated by applying the theory of one-dimensional consolidation, when no artificial drainage is provided (see Subsection 3.4.8 of this ROM 0.5).

In the particular case of vertical preloading on horizontal soil strata in which artificial drainage has been installed, the degree of consolidation achieved can be calculated using the formula shown in Figure 3.9.1.

This analytical expression, or other more complex numerical computations, can make it possible to estimate the degree of consolidation achieved when the preloading is maintained over a given period of time *t* and therefore the density and the strength of the ground.

Sometimes, the vertical flow in the ground has a certain importance even though artificial drains are installed to accelerate consolidation. Its beneficial effect can be taken into account by separately calculating the degree of consolidation that would have been reached if no drains existed. This calculation can be made using the theory of one-dimensional consolidation for vertical flow described in Subsection 3.4.8.

The degree of overall consolidation *U*, as a result of radial horizontal drainage and of vertical flow, can be calculated by the expression:

$$1 - U = (1 - U_1)\,(1 - U_2)$$

where U_1 and U_2 are the degrees of consolidation that each of these two flows would separately produce.

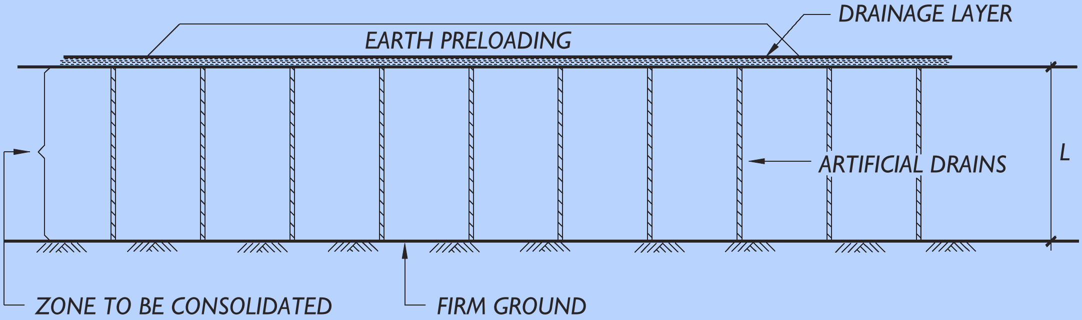
It is common practice to leave the surcharges in place until the degree of consolidation achieved is around 75% or more and this is to be recommended. In these circumstances, the total or partial removal of the preloading tends to cause some swelling, which is only a small fraction of the previous settlement.

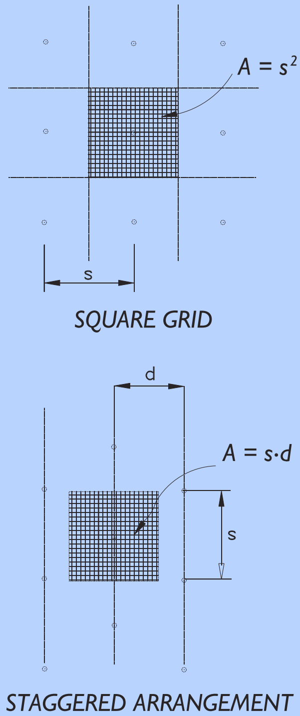
In any event, determining the coefficient of consolidation for horizontal flow, c_h, is usually very imprecise and only detailed monitoring of the consolidation process will make it possible to evaluate the benefits obtained from preloading on poorly permeable soils.

3.9.2.5 Monitoring the Treatment

It is always necessary to monitor the preloading treatment and this should be aimed at three fundamental aspects.

Figure 3.9.1. Estimating the Degree of Consolidation of Preloaded Strata with Accelerated Drainage





DEGREE OF CONSOLIDATION

$$U = 1 - \exp\left(-\frac{8\, c_h\, t}{\mu\, d^2}\right)$$

where:

U = Degree of consolidation.

c_h = Coefficient of consolidation (for horizontal flow).

d = Equivalent separation between drains = $\sqrt{\frac{4}{\pi} A}$

A = Area allocated to each drain or total area divided by the numbre of drains.

μ = Dimensionless coefficient given by the expression:

$$\mu = \ln\left(\frac{d}{\phi_d}\right) - 0{,}75$$

ϕ_d = Equivalent diameter of the drain. For flat drains:

$$\phi_d = \frac{2\,(b + e)}{\pi}$$

b = Drain breadth.
e = Drain thickness.

- The embankment height or intensity of the loads, which can be done by simple procedures – in the case of preloading with earthfill, by monitoring its apparent density and topographical levelling.

- Ground settlement, which can be monitored by settlement plates placed on the surface of the natural ground. It is also advisable to control settlement at different depths using either soil extensometers installed vertically or by telescopic tubes for measuring settlement.

- Pore water pressures, by piezometers (pneumatic or vibrating-wire) inserted in the ground at different depths and interspersed between the artificial drains as far away as possible from them. Their exact position should be carefully laid out.

These or other parameters are tools for observing the progress of the consolidation and deciding the time when a particular improvement should be considered completed.

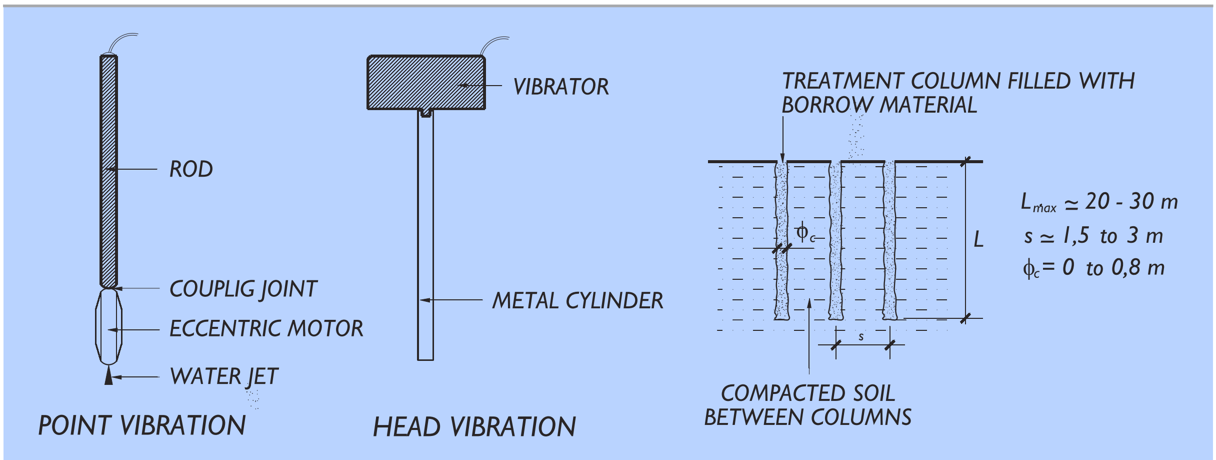
Subsequent geotechnical investigation, with the same or similar procedures to those used for investigating the ground prior to treatment, will indicate the improvement achieved and make it possible to design adequately the structures and/or pavements to be constructed on the treated ground.

3.9.3 Deep Compaction

Vibration of loose granular soil induces a densification that improves its strength and reduces its deformability. On this principle, a wide variety of techniques have been used to compact loose sand deposits, which can be collectively referred to as ***vibro-compaction***.

Figure 3.9.2 illustrates two types of soil vibrators that have been successfully used in Spain to treat sandy soils down to depths of about 15 m. This treatment depth, or a few further metres, appears to be the effective limit of current practice.

Figure 3.9.2. ***Diagram of Vibro-compaction Treatment***



The vibration transmitted to the ground causes instability in its structure, inducing partial liquefaction and originating surface settlement.

The effective range of vibration will depend on the power of the equipment and on the type of ground. In common practice, the treatment usually involves one vibration column every 3 to 5 m^2. Substantial average improvements in density can be obtained with this spacing.

3.9.3.1 Treatment Types

For a ground to liquefy under vibration, it must have virtually zero fines content (silts plus clays), otherwise, liquefaction will not occur. With almost zero fines content (<5% approx.), densification can be achieved by simply vibrating the probes introduced into the ground. This treatment is usually known as ***vibro-flotation***.

When the fine particle content is higher (ranging roughly between 5 and 20%), the probe vibration partially densifies the soil around the vertical in which it is introduced and leaves a hollow cylinder that is gradually refilled with granular borrow material. This combination technique of vibration and inclusion of a more rigid material is normally known as **vibro-replacement**.

When the fines content exceeds 20 or 25%, vibration will not assist in compacting the surrounding soil and if this procedure were to be used, it would constitute a rigid inclusion treatment (like stone columns, for example), which will be dealt with later on.

In any event, these treatments will leave the top part of the ground with little compaction and a final surface compaction treatment is therefore necessary using vibrating rollers.

It is not easy to recompact the surface of the ground in a submerged fill after the vibration treatment. This means that this technique is not recommended in these circumstances, particularly if the soft soil to be treated is not very thick. If this treatment is however used, the procedure should be completed by some system to solve this problem.

3.9.3.2 Potential Improvements

High relative densities can be obtained with deep compaction (D_r = 75%) and resistance to dynamic penetration (the SPT N-index) in the order of N = 25 or over, or resistance to static penetration (q_c for the Dutch cone) greater than 10 MPa.

The degree of densification obtained may be fairly homogeneous and consequently these density measurements may not differ greatly when taken at different distances from the treatment verticals. Within the column of refill material (in the case of vibro-replacement), the relative density and penetration resistance (static or dynamic) may be higher.

In loose sand deposits, the soil can be so improved that deep foundations, which would otherwise have been used, are no longer needed. Whereas this generally holds true for lightweight structures, a special check is required where large loads are concerned.

Table 3.9.2 summarizes the general applicability of these methods to different types of ground.

Table 3.9.2. ***Applicability of Deep Compaction Treatments to Different Ground Types***

Ground Type	Vibro-Flotation	Vibro-Replacement
Clean sand (*)	Excellent	Nota applicable
Silty sand	Average	Excellent
Mud	Poor	Good or average
Clay	Not applicable	Good
Dumped fill	Depends on the type of material	Good
Tailings	Not applicable	Not applicable

* Less than 5% fines.

3.9.3.3 Control Procedures

It is always necessary to control ground improvement by deep compaction.

Before designing the treatment, the ground must be known in detail. The treatment design must include a detailed prediction of the desired improvements. This prediction should be expressed in terms of the anticipated variation in the most representative parameters (relative density, SPT N-index, static penetration resistance, etc).

The improvement that can be obtained may be known from previous experience. Various typical values for sands improved by deep compaction were given in the preceding section.

Treatment by deep compaction, with or without the use of refill material, requires geometrical surveying (treatment points and surface levelling) as well as controlling the amounts of borrow material used.

The tests carried out in the geotechnical investigation prior to treatment must be repeated in order to assess its effectiveness. SPT, continuous dynamic penetration and static penetration tests are particularly appropriate in these cases.

Geophysical tests and, in particular, seismic refraction tests carried out in boreholes (cross-hole or its variants) may also be useful in investigating the ground improvement.

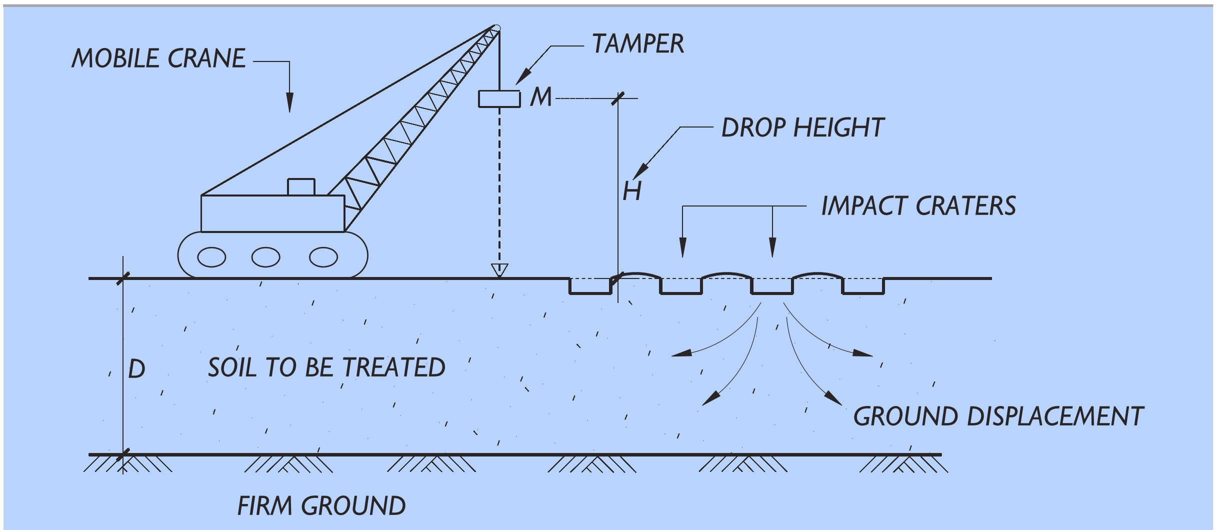
In situ load tests may be required to estimate the bearing capacity and the modulus of deformation, either using plate load tests in trial pits at varying depths or by pressuremeter tests in boreholes.

Experience has shown that ground strength generally increases and its deformability is reduced as time elapses after completion of the treatment. Thus, geotechnical investigations for evaluating the improvement, performed immediately after the vibration treatment, can result in pessimistic conclusions. Investigations carried out some months after completing the treatment may give a more accurate idea of the benefits obtained.

3.9.4 Dynamic Compaction

Large weights falling from a great height cause soils to fail and displace towards the sides of the impact zone. At greater depth, an improvement can also be obtained as a result of the pressure wave that affects the soil particles and modifies their contacts, making them denser (see illustration in Fig. 3.9.3).

Figure 3.9.3. ***Diagram of Dynamic Compaction***



This type of surface soil treatment has been carried out with tamping weights ranging from 1 ton to over 100 t and from drop heights of as much as 40 m.

The usual spacing between impact points is between 2 or 3 m in the case of small hammers and over 10 m in the case of heavy hammers.

The treatment is usually carried out in several stages, alternating the drop points in the final treatment grid designed.

The depth of the densified zone is related to the energy of the tamping. Experience has shown that this depth is given by the following empirical formula:

$$D = \alpha \cdot \sqrt{M \cdot H}$$

where:

M = tamping weight, in tons.
H = drop height, in metres.
D = effective treatment depth, in metres.
α = factor depending on the type of ground and treatment characteristics concerned. The usual value is close to 0.5 (m/t) $(m/t)^{1/2}$. The presence of a bedrock may increase the value of α.

If a rigid layer exists at a shallower depth than D, this lesser depth should be taken as the D value.

Having selected the hammer weight and drop height to achieve a treatment depth according to the problem to be solved, the treatment intensity should be decided, i.e., the number of hammer blows and their spatial and temporal arrangement. To this end, it is advisable to define a reference parameter known as the *specific energy* or energy applied per unit volume of treated soil, which is the following:

$$E_s = \frac{1}{2} \cdot \sum_{i=1}^{N} \cdot \frac{M\, V_i^2}{A\, D}$$

where:

E_s = specific energy.
N = total number of blows.
M = tamping weight.
V_i = impact velocity.
A = total plan area of the treated surface.
D = effective treatment depth.

A correlation exists between the specific energy applied and the average increase in dry density of the treated ground. However, this relation depends a great deal on the type of ground. For saturated sandy soils, the average dry density increase can be assumed to be given by the following expression:

$$\Delta\gamma_d(\%) = \eta \cdot E_s^{1/2}$$

where:

$\Delta\gamma_d$ (%) = average increase in dry density, expressed as a percentage of the dry density existing prior to treatment.
E_s = specific energy.
η = constant, whose typical value is in the order of 0.2 $kPa^{-1/2}$.

One way of checking on site that the desired increase in dry density is being achieved is to monitor surface settlement *s* as, according to the definitions given, this fulfils the expression:

$$\Delta\gamma_d(\%) = 100\frac{s}{D}$$

where:

s = average settlement in the ground surface caused by the treatment.
D = effective treatment depth.

The specific energy of the treatments usually carried out range from a few hundred kPa in fairly shallow soft soils up to a specific energy value of several thousand kPa in treatments of very thick soils deposits.

The total number of blows *N* is usually distributed in several runs (usually two or three passes). Drop points are arranged in a rectangular or triangular plan grid and roughly with one impact point per 10 m^2. During each pass, the same point is struck several times consecutively (four blows is the usual number) and the next stage is done on different points, to intersperse them between the previous points used. It is advisable to set up a certain waiting period between consecutive stages to help the ground to consolidate more easily.

After dynamic compaction, the ground surface will be very uneven. Subsequent grading and surface compaction work may be required, using other procedures.

This technique has been successfully used in both granular and soft cohesive ground (with less reliable results in the latter case) and seems particularly indicated for the treatment of heterogeneous artificial fills that are difficult to improve using other procedures.

This treatment can also be carried out under water (with tampers of special hydrodynamic shapes).

Dynamic compaction can be carried out locally to improve the area supporting individual foundations.

The potential harmful effect on adjacent structures (and the noise) should be monitored, since significant vibration can be produced in the surrounding ground, as also happens with large pile driving .

As with other techniques, geotechnical investigation after the treatment will be required to evaluate its effectiveness.

3.9.5 Explosive Compaction

Blasting explosive charges inside the ground causes failure around the charge and transient displacement of the adjacent ground, which returns to a subsequent situation that is generally more stable and denser.

This technique has been used in harbour works to displace muds underlying better-quality deposits and to compact dumped fills.

In deposits of fine sand, liquefaction can be triggered and considerable increases in density thereby achieved. Over half a metre of surface settlement has been measured in some treatments.

The explosives are normally drilled in at depths close to the centre of the zone to be densified or somewhat farther down (see Fig. 3.9.4).

This figure shows some typical parameters for explosive charges and their spacing as applicable to granular soils. Higher energies may be required in treating other soil types.

The vibrations induced in nearby structures need to be controlled, as this type of treatment can cause damage at a considerable distance away from the explosion area.

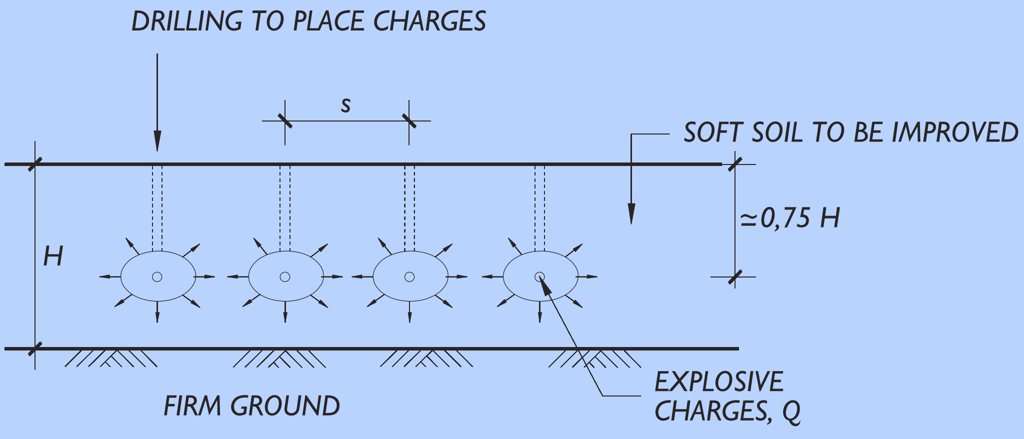
The result obtained can and should be evaluated by measuring the settlement achieved and by repeating the geotechnical investigation carried out prior to treatment with a view to determining the compacity of the treated ground.

3.9.6 Rigid Inclusions

Ground improvement by rigid inclusions can be considered a technique for executing foundation elements rather than a soil treatment.

This type of solution, however, is treated in practise in a way that does normally have a certain similarity to other ground improvement techniques, either because the ground with inclusions is considered as a homogeneous medium with improved equivalent properties or because the soil itself forms part of the rigid inclusion. From this point of view, these solutions can be considered to be improvement procedures.

Figure 3.9.4. **Diagram of Treatment Using Explosives**



TYPICAL PARAMETERS

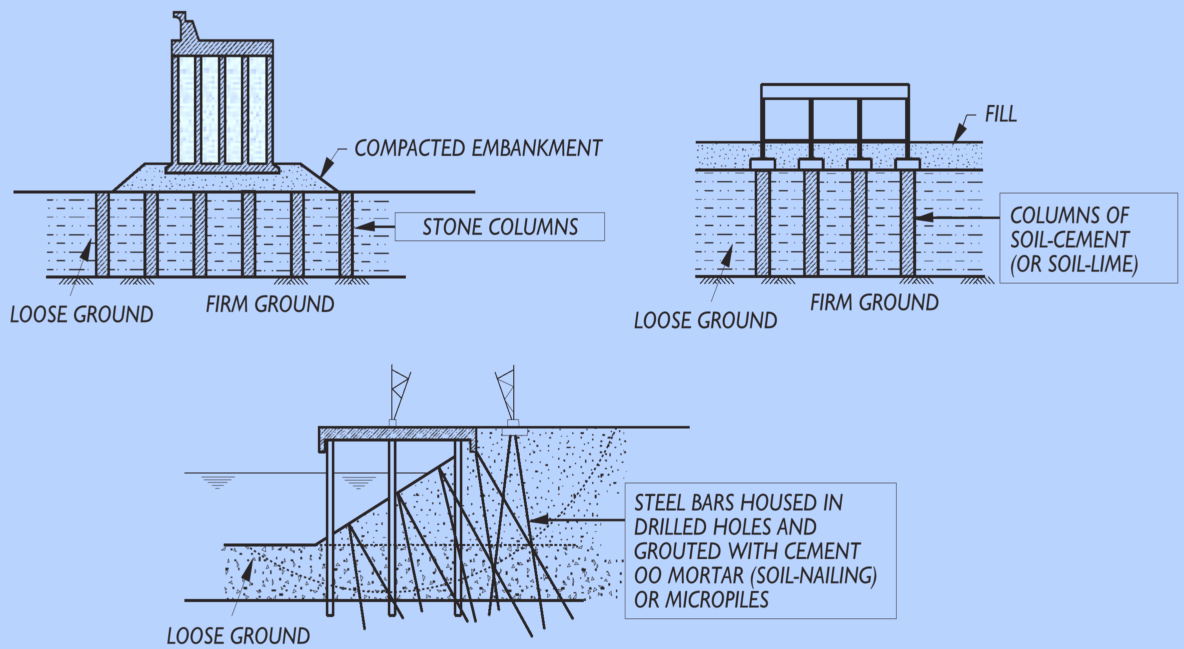
Charges, Q = 1 to 10 kg of dynamite, TNT or ammonite

Spacing, s = 5 to 15 m.

Consumption = 10 to 30 g per m^3 of treated soil.

The most common rigid inclusions are illustrated in Figure 3.9.5 and are the following:

Figure 3.9.5. **Diagrams of Various Ground Treatments with Rigid Inclusions**



- Stone columns executed with vibro-replacement or other specific techniques.

- Soil-cement or soil-lime columns, usually executed with augers which are driven in the natural soil and mix it with the lime or cement – other binder materials can also be used.

- Soil nailing (cement-grouted steel bars inside drilled holes) or micropiles (groups of several steel bars or tubes housed in boreholes later filled with cement grout or mortar). Reinforcing the ground with timber piles (mainly eucalyptus) is common practice in muddy or swampy areas.

This type of ground improvement can also include reinforced-earth structures, either with metal strips, geogrids or other artificial products.

Considerable experience has already been acquired with each kind of treatment and there are specific design procedures for dimensioning the reinforcements required in each case.

Compared with the improvement procedures mentioned above (preloading, vibro-compaction, dynamic compaction and explosive compaction), methods of rigid inclusions generally tend to be more costly but may still be more cost-effective than other equivalent deep foundation systems.

3.9.6.1 Dimensioning Inclusions

Rigid inclusions composed of soil-cement or soil-lime columns or of elements similar to conventional piles (steel bars or tubes, timber, etc.) should be analysed as if they were piles, since their stiffness will generally be much greater than that of the untreated surrounding soil.

Stone columns, however, mainly owe their stiffness to the lateral confinement produced by the soil to be reinforced. Consequently, they may prove to be more flexible than other types of inclusion. Loads can be distributed over the soil and the columns following the steps described below.

a. First, it is necessary to estimate the settlement that the external surface of the natural ground would undergo without any treatment and owing to the ground layer of thickness H, where the columns will be inserted. This settlement is designated s_o.

b. Second, some parameters should be assumed for the treatment. The values needing to be defined are:

- column diameter and depth, D and H.
- angle of friction for the gravel, ϕ.
- stone column spacing, assigning a plan area of influence, A, to each column.

With these data, the dimensionless number ρ can be defined, which measures the volume of soil taken up by the columns in relation to the total soil treated, with a depth H.

$$\rho = \frac{\pi \cdot D^2}{4A}$$

c. Calculating settlement with stone columns.

The gravel columns reduce the previously calculated settlement s_o to the estimated settlement:

$$s = \alpha\ s_o$$

Based on measurements made in actual treatments of this type, the following empiric relation has been found between α and ρ:

Approximate Correlation Between α and ρ					
Values of ρ	0.1	0.2	0.3	0.4	0.5
Approximate values of α	0.8	0.65	0.47	0.35	0.25

d. Stone columns are normally constructed to increase the bearing capacity of the ground. Otherwise, if they were only used to reduce settlement, they would not hold any clear advantage over other simpler procedures.

In order to estimate the ground strength after executing stone columns, the vertical stress σ_v acting on each column should be known, which is greater than the vertical stress p acting on the natural ground without the existence of any gravel columns. As a stone column is somewhat more rigid than the natural ground, the σ_v/p ratio is greater than 1 and has been designated here as β.

The values (19) of β essentially depend on ρ. The following are typical:

Approximate Correlation Between β and ρ					
Values of ρ	0.1	0.2	0.3	0.4	0.5
Values of β	2.8	2.4	2.2	2.0	1.7

e. For undrained stability analyses in natural soil (if necessary), it can be assumed that the whole of the ground (natural soil and columns) is equivalent to a homogeneous ground with the following parameters:

Weight $\quad \gamma = \gamma_1\,(1\text{-}\rho) + \gamma_2 \cdot \rho$
Cohesion $\quad c = s_u\,(1\text{-}\rho)$
Friction $\quad \tan\phi = \beta \cdot \rho \cdot \tan\phi_g$

When stone columns are executed outside the loaded area –in plan view–, the combined equivalent angle of friction is clearly smaller. In this case, it should be assumed that:

$$\tan\phi = \rho \tan\phi_g \qquad \text{(outside the loaded area)}$$

The undrained shear strength of the natural soil is s_u. The value to be used is the one corresponding to the relevant design situation. Loads are sometimes applied after certain waiting periods, which are set up as a means of achieving an increase in s_u.

f. For calculations in undrained conditions, the following equivalent geotechnical parameters should be used for the improved ground:

Weight $\quad \gamma = \gamma_1\,(1\text{-}\rho) + \gamma_2 \cdot \rho$
Cohesion $\quad c = c'\,(1\text{-}\rho)$
Friction $\quad \tan\phi = \alpha\,(1\text{-}\rho)\tan\phi'_s + [\beta \cdot \rho]\tan\phi_g$

(19) When the presence of columns is assumed to reduce the settlement s_o to the value $\alpha \cdot s_o$, it is admissible to consider that the load acting on the soil is reduced from the value p to the value $\alpha \cdot p$.
Vertical equilibrium can be established by adding the load acting per unit area over the columns, $\beta{\cdot}p{\cdot}\rho$, to that acting over the soil, $\alpha{\cdot}p$ $(1-\rho)$, and equalling this sum to the load p. This gives:

$$\beta \cdot \rho + \alpha\,(1-\rho) = 1$$

Hence, the value of β can be estimated once α and ρ are known.

For the treated areas outside the vertical of the loaded area, the friction will be:

$$\text{Friction (outside the load area): } \tan \phi = (1-\rho) \tan \phi'_s + \rho \tan \phi_g$$

The equivalent densities, γ, those of the soil, γ_1, and the gravels, γ_2, in Steps e) and f), will be the ones corresponding to the apparent values above the groundwater table. For points below the groundwater table, these values are the ones corresponding to the submerged state ($\gamma' = \gamma_{sat} - \gamma_w$) of each material.

The strength parameters of the natural soil in effective pressures have been designated *c'* (effective cohesion, which will normally be taken to be equal to zero in preconsolidated soft soils) and ϕ'_s, which is the effective angle of friction.

The angle of friction ϕ_g is the one corresponding to the material composing the columns.

3.9.7 Grouting

The technique of grouting is particularly useful for reducing the permeability of the soil, although it can also be used for other purposes, including the increase of strength and reduction of deformability.

Grouting is particularly useful in treating seepage towards dewatering systems in dry docks with drained bases, since it can substantially reduce the pumping discharge required to keep these docks dry.

The most common grouting techniques are included in the diagram in Figure 3.9.6. The effects achieved by them are indicated below.

3.9.7.1 Permeation Grouting

The most adequate technique for reducing the permeability of the ground consists of forcing cement and water grouts (with bentonite and other additives) through the pores of the soil.

These grouts are made to circulate under moderate pressure, without fracturing the soil, through highly permeable granular soils ($k > 10^{-1}$ cm/s, approx.).

Using ultra-fine cement (microcement), it has been possible to grout the pores in finer sands ($k = 10^{-1}$ to 10^{-2} cm/s).

For even finer soils ($k = 10^{-2}$ to 10^{-4} cm/s), other products with higher penetrating power (silicates, resins, etc.) can be used for grouting without fracturing the soil.

Grouting is normally carried out at closely spaced points (from 1 to 3 m) if a certain degree of watertightness needs to be achieved. The resulting permeability of the treated ground can be fairly low, in the order of 10^{-5} cm/s or less.

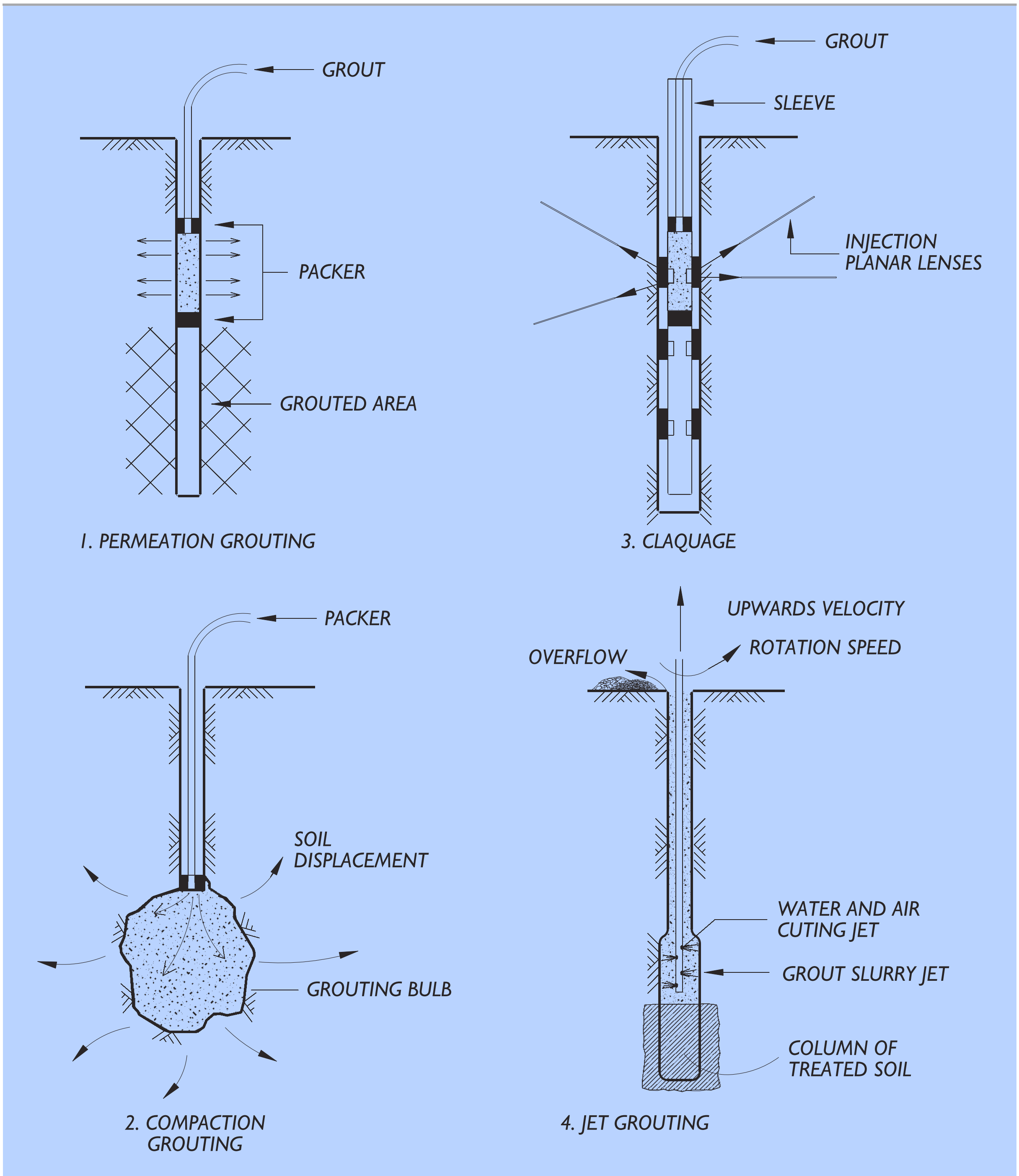
Ground strength and deformability are also improved by this permeation grouting treatment. Its effectiveness should be investigated in each specific application.

3.9.7.2 Compaction Grouting

This technique consists in pressing a cement and thick sand mortar through the bottom of a vertical borehole, which forces the soil to displace and reduce its volume.

This treatment has been successfully used to make ground heave compensate the settlement produced by other causes.

Figure 3.9.6. ***Common Types of Grouting***



3.9.7.3 **Fracture/*Claquage* Grouting**

In this technique the cement grout is applied through a tube with regularly spaced openings protected on the outside by flexible sleeves.

Grouting is executed in sections, pumping grout that is forced out through one or several sleeves. When a section of the tube is isolated and the grout forced into it under pressure, the sleeve yields and the grout slurry is forced out, fracturing the soil in planes that can have different orientations as a function of their state of stress.

The pressures used to cause fracture (*claquage*) are usually several tens of bars. More moderate pressures of around 10 bars or less are used to maintain the grout discharge after fracturing.

Under these pressures, the grouting flows into the open fractures and compresses the soil. As a result, the ground is reinforced by a series of grout lenses filling the fractures.

This technique has been successfully applied in underpinning buildings with slab foundations. Controlled uplift can be achieved in this type of foundation.

3.9.7.4 Jet Grouting

Jet grouting is a high-pressure technique aimed at treating soils at some depth in order to form areas of improved soil or to replace the soil by other products.

The main element in jet grouting is a tube with small orifices (nozzles) allowing jets to be formed with very high speeds and discharge rates of several litres per second.

The ground cutting jet can consist of a cement grout alone (Jet I, or single-fluid) or a cement grout enclosed in a jet of air (Jet II, or double-fluid) or by a water jet enclosed in a jet of air. In the latter case, the grout is injected through additional nozzles (Jet III, or triple-fluid).

Especially powerful jets are recently being used to execute large-diameter columns. The corresponding technique is known as Superjet.

The ground is treated from the bottom upwards. First, the grouting device is inserted in a previously bored hole down to the desired depth and is then extracted at an appropriate speed, fracturing the soil and mixing it with the grout. The amount of grout to be mixed with the soil can be controlled by the rate at which the jet-grouting device is extracted.

Columns can be obtained using this procedure (by rotating the cutting jet) or plane walls made of grout or mortar or of these materials mixed with the soil.

Jet grouting is used in a wide variety of problems owing to its versatility and the speed with which the treatment can be applied.

Treated soil columns of up to 3 m in diameter can be achieved using this technique (the largest, in granular soil) with strength as high as that of concrete (the highest, again, in granular soil).

It is difficult to control the resulting product and this calls for complicated investigations, laboratory tests, preliminary trial treatments and even load tests on the end columns.

The procedure is applicable to any type of ground except those excessively permeable (rockfills or very clean gravels) in which a sealing treatment is needed prior to jet grouting. Difficulties may arise in the setting of the grout-ground mix in soils with organic matter.

3.9.8 Other Procedures

The following techniques should be mentioned for the purpose of covering a comprehensive list of the different soil treatment techniques available.

Electroosmosis

Introducing steel or aluminium bars as anodes in the ground and well-point tubes as cathodes, with an arrangement and spacing similar to the one described for preloading drains (Subsection 3.9.2.), causes a flow of water in the ground towards the cathodes when a direct current is set up between the anodes and cathodes.

The procedure can be completed by adding chemicals (silicates, for example) that are entrained by the water and retained by the ground in the course of their movement towards the cathodes.

This technique accelerates the consolidation of soil masses and reinforces the soil with chemical grouting. Although it is a very appealing idea and has been well developed in theory, its practical application is limited by its high energy consumption.

Heat Treatments

Soft, moist ground can be desiccated to increase its strength by combusting diesel oil or other fuels in wells or boreholes previously drilled in the ground.

This technique is not known to have been applied in Spain yet. Its energy cost is extremely high.

The ground can be frozen by generating low temperatures with dry ice (CO_2) or expanding liquid nitrogen in closed circuits through tubes in the ground or in open circuits in previously drilled boreholes. A high transient ground strength is developed allowing certain retaining tasks to be carried out during excavations. This very costly technique has been successfully used in several construction projects in Spain.

3.10 DYNAMIC EFFECTS

3.10.1 General Considerations

Some climatic loads such as the action of sea oscillations or wind, earthquakes and actions owing to vessel operations, amongst other causes, may cause the ground to be subjected to cyclic or impulsive stresses. Their values depend on complex phenomena of interaction or combined dynamic response of the structures and the supporting ground under these actions.

Dynamic effects can be especially important in maritime and harbour works. For example, wave loads on vertical breakwaters can be amplified, depending on the dynamic characteristics of the breakwater and its foundation.

Ground behaviour under dynamic loads exhibits some typical features that engineers must be aware of. The testing procedures employed to determine the ground parameters to be used in dynamic analyses and the methods for ground characterisation differ in some respects from those corresponding to static conditions.

The main difference observed in soil behaviour under cyclic or alternating actions is the generation of porewater pressures in some saturated soils. In monotonous loading processes, pore water pressure grows in accordance with certain laws. Porewater pressure in a load-unload cycle may be different in the loading and the unloading stages and consequently, even though the total pressure added is null at the end of the cycle, a residual porewater pressure may remain. Additional load-unload cycles can cause successive increases in porewater pressure which, if not drained, could decrease the bearing capacity or resistant capacity of the ground.

Another essential aspect observed in ground behaviour under alternating loads is its energy dissipation capacity. This quality, known as damping, is hysteretic within a broad range of excitation frequencies (1 to 10 Hz, for instance).

Besides, the resistant capacity of soils subjected to cyclic loads depends not only on the level and degree of variation in the stresses produced but also on the ground's drainage capacity. This can cause transient increases in porewater pressures in some soils and, as a consequence, reductions in the soil's resistant capacity. In dilatant granular soils, the opposite phenomenon can occur, giving rise to a reduction of porewater pressures and, consequently, to increased resistant capacity of the soil.

A saturated soil under cyclic loads can be taken to behave as in drained, partially drained or undrained conditions depending on the degree of similarity between two time scales: the necessary for the soil to reach complete consolidation –i.e., to dissipate the excess porewater pressure– under the load action, $t_{U(100\%)}$, and the separation between load cycles, t_c, which is equivalent to a semi-period ($t_c = T/2$) or to the duration of an impulsive load.

Thus, for soil behaviour, if:

- $t_{U(100\%)} << t_c$, the load can be considered to be static and the soil to be in totally drained conditions;
- $t_{U(100\%)} \approx t_c$, the load can be considered to be cyclic and the soil to be in partially drained conditions;
- $t_{U(100\%)} >> t_c$, the load can be considered to be cyclic and the soil to be in undrained conditions.

To this end and in line with the indications given in Subsection 3.4.8 of this ROM 0.5, the value of $t_{U(100\%)}$ can be approximated by:

$$t_{U(100\%)} = H^2/c_v$$

where c_v is the soil's consolidation coefficient and *H* is the longest distance to drainage. *H* is therefore the thickness of the least permeable layer, if it is drained on one face, or half the thickness, if it is drained on both, or half the width of the foundation when the load is transmitted through a monolithic structure and this value is lower than the drainage layer thickness .

Thus, soil behaviour under the action of non-breaking waves (period *T* between 5 and 20 s) will generally be in undrained conditions for saturated soils, both for cohesive and fine granular ground. Medium sands can be taken to be in partially drained conditions and coarse sands, gravels and rockfills in drained conditions.

In addition, soil behaviour under breaking waves, under impulsive loads produced by waves on monolithic structures (t_c in the range of 0.01 to 0.05 s) or under seismic loads (t_c between 0.05 and 0.5 s) will generally be in undrained conditions for the majority of soils and some random fills.

In order to analyse ground behaviour under alternating or cyclic loads or problems of soil-structure interaction, engineers first need to examine the type of drainage expected. The most appropriate analytical procedure to be followed next depends on the drainage conditions.

3.10.2 Undrained Dynamic Behaviour

In the laboratory, and presumably in the field, the behaviour of soils subjected to uniform load cycles –with no drainage– can be represented by an equivalent, viscous linear elastic model where, in addition, a certain increase in pore water pressures is imposed depending on the number and intensity of the cycles. The parameters used most frequently and the laboratory tests usually carried out to determine them are indicated below.

3.10.2.1 Generation of Pore Water Pressures

When drainage does not exist, load cycles generate porewater pressures that generally conform to an expression of the following type:

$$\frac{n}{N} = \left[\frac{1}{2}\left(1 - \cos\left(\pi \frac{u_n}{\sigma'_{vo}}\right)\right)\right]^{\theta}$$

where

n = number of load cycles applied.
N = number of load cycles before failure is reached.
u_n = porewater pressure accumulated at the end of cycle number *n*.
σ'_{vo} = vertical effective pressure at the start of the test.
θ = model parameter – a typical value is $\theta = 0.7$.

For each soil and type of cyclic load, laboratory tests make it possible to obtain values for *N* and θ, which are the parameters of this model.

In addition to this law of generation of residual pressure at the end of the cycle, a variation in pore water pressure occurs during each cycle:

$$\Delta u = \pm B\left(\left|\Delta\sigma_{oct}\right| + a\left|\Delta\tau_{oct}\right|\right)$$

where $\Delta\ \sigma_{oct}$ and $\Delta\ \tau_{oct}$ are the cyclic variations in octahedral stresses, and *a* and *B* are parameters to be determined. For saturated elastic soils, $B = 1$ and $a = 0$. As a general rule, *B* and *a* have to be measured in the laboratory and may prove to depend on the number of cycles previously applied.

There are many other numerical expressions capable of representing porewater pressure variation during these loading-unloading processes. That is why engineers need to consult the specialised technical literature.

Knowledge of the porewater pressures generated in undrained conditions by wave action or other sea oscillations, as also by earthquakes, is particularly important for verifying the stability of many harbour structures (rubble-mound breakwaters, foundation berms of vertical breakwaters, etc.). There are no general tabulated solutions or simple analytical formulae for solving this problem, so engineers must refer to the specialised technical literature or resort to the use of models for each individual case. Subsection 3.4.11 of this ROM 0.5 provides approximate solutions for some specific cases, in relation to the pore water pressures generated by waves and other sea oscillations. Subsection 3.10.4 does the same in relation to earthquakes.

On many occasions, the dynamic geotechnical calculation of undrained situations is done in total pressures. In such cases, it is not necessary to define the process of generating porewater pressures.

3.10.2.2 Undrained Dynamic Strength

The dynamic strength of soils can generally be characterised using shear strength parameters defined in static conditions and in terms of effective pressures, adding to this definition the law governing the rise in the porewater pressures generated by the alternating loads. As a general rule, all types of ground can be taken to have the same effective shear strength parameters in dynamic as in static conditions.

Alternating shear stresses do not generally generate high porewater pressures, except in saturated fine sands with moderate or low relative densities, as also in limes and in clays, and specially in other very loose soils or soils with a metastable structure (quick clays, that are not found in Spanish ports).

Alternating loads produce increased porewater pressure in low-density saturated sands that may even grow in a monotonous manner. Rising pore water pressure weakens the soil's strength and may eventually lead to its failure.

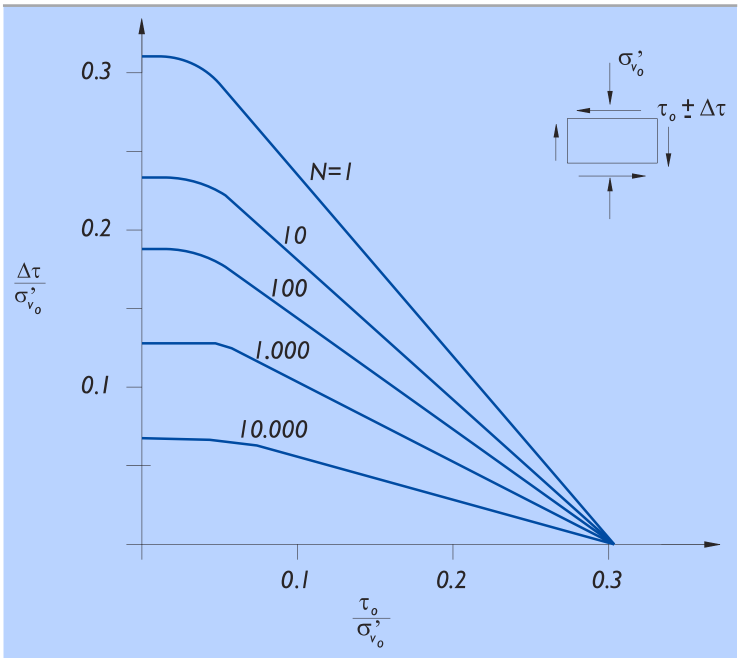
Failure is shown (at least in laboratory tests) by a generalised softening. The soil appears to behave like a fluid. The name *liquefaction* is usually used to define this type of ground failure.

It is normally considered that failure (liquefaction) has been reached when the accumulated shear strain equals or is greater than 5%.

It is also feasible to characterise the undrained shear strength of a soil by defining the number of load cycles causing failure in laboratory tests. This number of load cycles will be a function of the ground characteristics, its previous state of stress and the intensity of the dynamic load.

One or other method could be used for characterising dynamic strength in each dynamic analysis procedure. The analysis procedures are not well established and for this reason general recommendations cannot be given here as to the manner of defining strength.

Figure 3.10.1. ***Number of Cycles, N, Necessary to Cause a 5% Shear Strain***



The dynamic strength of a soil can be investigated in the laboratory by undrained cyclic simple shear tests. Their results can be expressed as shown in the graph in Figure 3.10.1, which could correspond to a normally consolidated silty-clayey fine sand.

Cyclic simple shear tests are particularly appropriate, as they represent soil behaviour under the type of loads that proves to be the most critical: alternating shear stresses with constant vertical pressure.

Cyclic triaxial tests can also be used, with the appropriate precautions, to measure strength in foundations for structures being subjected to dynamic loads.

When initial shear stresses can be assumed to be zero (anisotropic consolidation), the diagrams for soil strength under the action of dynamic loads can be simplified as shown in Figure 3.10.2. This figure corresponds to an undrained cyclic triaxial test in which the tangential stresses prior to dynamic loading are null.

Experience has shown that undrained cyclic shear strength is virtually independent of the excitation frequency (in the 1 to 10 Hz range).

3.10.2.3 Strains under Cyclic Loads in Undrained Conditions

When calculated in undrained conditions, volumetric strains in soils are usually assumed to be null, as the compressibility of the water is generally very small compared to that of the ground itself.

The geotechnical calculation of constant-volume strains can be done in terms of total pressures or of effective stresses. As a general rule, the first method is used for undrained conditions, i.e., a stress-strain calculation with total stresses as the unknowns.

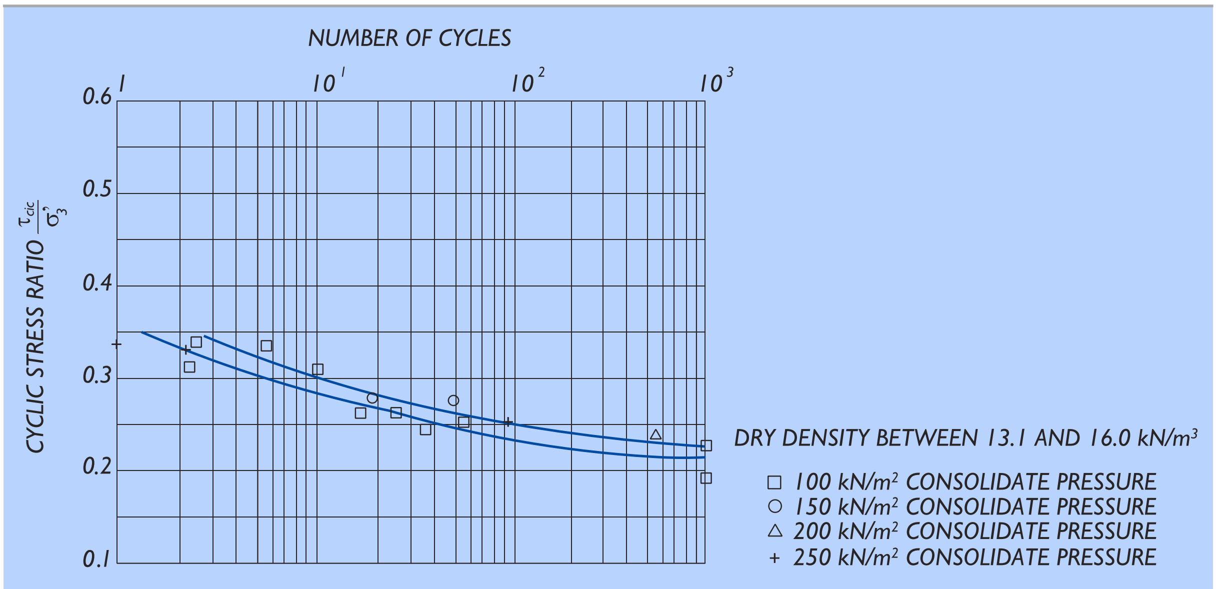
When calculating in total pressures, the stress-strain relation to be used should lead to a shear modulus, *G*, that is identical to the one used in effective pressures and referred to in the following subsection.

The condition of constant volume can be obtained with equivalent or apparent values of the Poisson's ratio that are close to 0.50. The real Poisson's ratio of the soil skeleton does not intervene in these calculations in total pressures.

Para realizar cálculos sin drenaje en presiones efectivas pueden utilizarse las leyes de tensión deformación que se indican más adelante y leyes de generación de presiones intersticiales acordes de forma que el volumen permanezca constante.

El valor del amortiguamiento viscoso equivalente que puede usarse en los cálculos no depende de las condiciones de drenaje, es sólo función de las deformaciones resultantes y no del tipo de cálculo que se realice. Los valores del amortiguamiento se indican en el apartado que sigue.

Figure 3.10.2. Typical Example of the Strength of a Saturated Sand Under Dynamic Load (Sandy Soils from the Port of Motril, Spain)



Note: Figure taken from the doctoral thesis of C. Olalla, "Problemas Geotécnicos y Comportamiento Dinámico de los Fondos Costeros Arenosos". Madrid Polytechnic University, June, 1992.

For undrained calculations in effective pressures, the stress-strain laws shown below and the appropriate laws of porewater pressure generation can be used so that the volume remains constant.

The equivalent viscous damping value that can be used in the calculations does not depend on the drainage conditions; it is only a function of the resulting strains and not of the type of calculation done. Damping values are given in the following subsection.

3.10.3 Drained Dynamic Behaviour

As an initial approximation, the deformation behaviour of soils under dynamic loads can be assimilated to the behaviour of a linear elastic solid with viscous damping. The parameters that would define the stress-strain behaviour in this case will be:

G = shear modulus.
ν = Poisson's ratio.
D = relative damping.

These parameters depend on the magnitude of the angular distortion. For very small angular strains (in the 10^{-5} to 10^{-6} range), the corresponding G_0 (also called G_{max} in many texts) and ν_0 parameters can be obtained by geophysical exploration (see Section 2.6 of this ROM 0.5). In these tests, the propagation velocity of compression waves, v_p, and of shear waves, v_s, can be measured and from these two data the following can be deduced:

$$G_o = \rho_{ap} \cdot v_s^2$$

$$\nu_o = \frac{1-2\alpha}{2(1-\alpha)}$$

where:

$$\alpha = \frac{v_s^2}{v_p^2}$$

ρ_{ap} = apparent density of the ground.

For larger strains, the shear modulus G will be smaller, whereas the value of Poisson's ratio ν will undergo little change or will increase.

Specific laboratory tests will be required to obtain values for these parameters for a different range of strain. The most adequate test is the resonant column. Some typical results from these tests carried out on silty sands are reproduced in Figure 3.10.3.

In addition, the shear modulus G_0 –for small strain– depends on the cell pressure. This relation must then be established by testing specimens under different pressures.

The relative damping D, which generally increases with the amplitude of the displacement, can also be obtained from this test and typical results are shown in Figure 3.10.3.

For the majority of cases, it will be sufficiently precise to represent the ground by its shear strength and by the parameters G, D and ν (dependent on the amplitude of the deformation). In special dynamic problems (offshore platforms, for example), more complex soil behaviour models may be necessary.

Alternating dynamic loads and vibrations can induce densification in unsaturated soils, specially in sands. This aspect can be explored in the laboratory. Several of the ground improvement techniques defined in Section 3.9 are based on this phenomenon.

When the soils are saturated, the densification process requires part of the pore water pressure to be expelled. This consolidation process delays the eventual densification.

Substantial settlement can occur as a result of the densification. The settlement will increase with the number of load cycles and their amplitude.

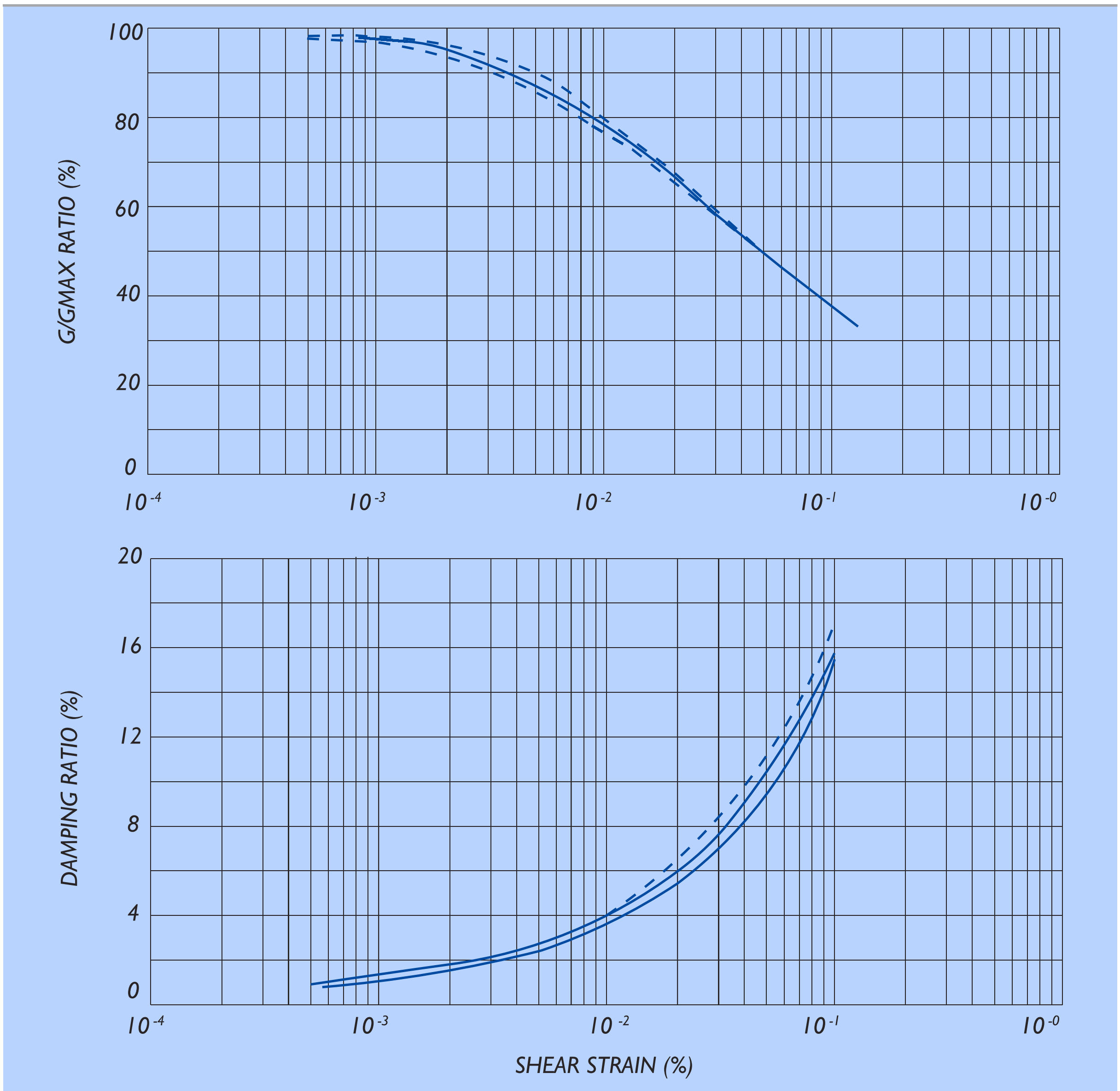
If the dynamic load is cyclic, with a definite average value and an oscillation around this average value of constant amplitude, the permanent volumetric strain, ε_p, can be approximated using an expression of the following type:

$$\varepsilon_p = \varepsilon_0 \cdot N^{\alpha} \cdot \exp\frac{\beta}{F}$$

where:

ε_o = volumetric strain caused by a static load equal to the maximum value of the dynamic load.
N = number of cycles
F = ratio between the oscillation amplitude of the load that would cause failure in a single cycle and the amplitude really acting – it is equivalent to a local safety factor.
α, β = dimensionless parameters obtainable from laboratory tests.

Figure 3.10.3. **Typical Example of Deformation Behaviour Parameters of a Soil under Cyclic Load (Sandy Soils in the Port of Motril)**



Note: Figures taken from the doctoral thesis of C. Olalla, "Problemas Geotécnicos y Comportamiento Dinámico de los Fondos Costeros Arenosos". Madrid Polytechnic University, June, 1992.

Typical values of α range from less than 0.05 to over 0.15, depending on how sensitive the soil is to the alternation of load. Typical values for β, for the same reason, range from 1 to 10, approximately.

If the dynamic action is not cyclic, but irregular, it is always possible, in an initial approximation, to represent it by an equivalent cyclic load, capable of producing similar permanent deformation.

The strength of drained soils in dynamic conditions can generally be assumed to be equal to the strength corresponding to static conditions. The strength parameters obtained in conventional drained static tests are also applicable to the case of dynamic loads.

When drainage is incomplete, some sort of consideration should be made about the porewater pressures that may be generated.

Knowing the variations in porewater pressures generated in the ground as a result of vibrations, seismic action or wave or wind action –either directly applied or transmitted through resistant structures– on marine beds in partially drained situations is extremely complicated and associated with substantial uncertainties. They could be estimated by integrating the coupled equations of hydraulics and elastodynamics, without omitting the compressibility of the water and of the soil skeleton, taking as boundary conditions the variation in total stress caused by the load in question. These computations are complex and lie outside the scope of this ROM 0.5 (see ROM 1.1 in relation to the porewater pressure variations due to wave action). The best way of knowing them in a particular case is to observe them in a model with monitoring techniques.

In some soils (e.g., dilatant granular soils), a substantial variation in porewater pressures occurs when shear stresses change. This significant non-elastic behaviour can considerably alter the results corresponding to calculations based on the simplification of elastic behaviour with respect to the laws of the generated porewater pressure.

Given the complexity of the problem, it is especially recommended for these cases to arrange effective observation systems on site, making it possible to compare monitoring data with results of theoretical calculations and, where applicable, to take any decisions necessary for the stability of the works.

3.10.4 Soil Behaviour under Seismic Action

Earthquakes are one of the most important actions capable of producing significant dynamic effects on the soil-structure system and of modifying soil behaviour, both in respect of resistant capacity and deformational behaviour.

As explained in Subsection 3.10.1 of this ROM 0.5, soil behaviour under seismic action will generally be in partially drained or undrained conditions for all types of soil and fill (it could even be so in rockfill random fills). Important increases in porewater pressure and loss of ground resistance against shear forces will arise, especially in saturated loose granular soils. This phenomenon, known as liquefaction, is one of the chief causes of failure to be taken into account in the design of maritime and harbour works in seismic zones.

Consequently and as a general rule, when seismic action is under study the soil strength and deformation parameters to be considered in calculations will be the parameters corresponding to undrained behaviour for soils below the groundwater table, and taking into account the degradation of the soil's resistant parameters owing to the action of dynamic loads.

As part of the ROM Programme, a specific Recommendation should be devoted to seismic effects on maritime and harbour works. On a provisional basis, this section will advance some basic ideas to help engineers analyse these effects.

3.10.4.1 Characterising Seismic Events

The main factors characterising a particular earthquake are its magnitude, intensity, ground acceleration and response spectrum. Some basic ideas follow on these four factors.

3.10.4.1.1 MAGNITUDE

The total energy released by a seismic event can be measured, at least in conceptual terms, by recording the movements of the earth's surface taking place during an earthquake. It can also be estimated by energy considerations applied to the failure zone (seismic moment).

Different definitions for magnitude simultaneously exist, with the following names:

M_L = local magnitude or Richter scale.
M_s = magnitude of surface waves.
m_b = magnitude of short-period internal waves.
m_β = magnitude of long-period internal waves.
M_J = magnitude as defined by the Japanese Meteorological Agency.
M_w = magnitude based on seismic moment.

At the present time, the M_w values tend to be used most.

In each particular case where this concept has to be used, it is advisable to make clear which definition is involved, as some differences exist, which can be as much as 1 unit in major earthquakes.

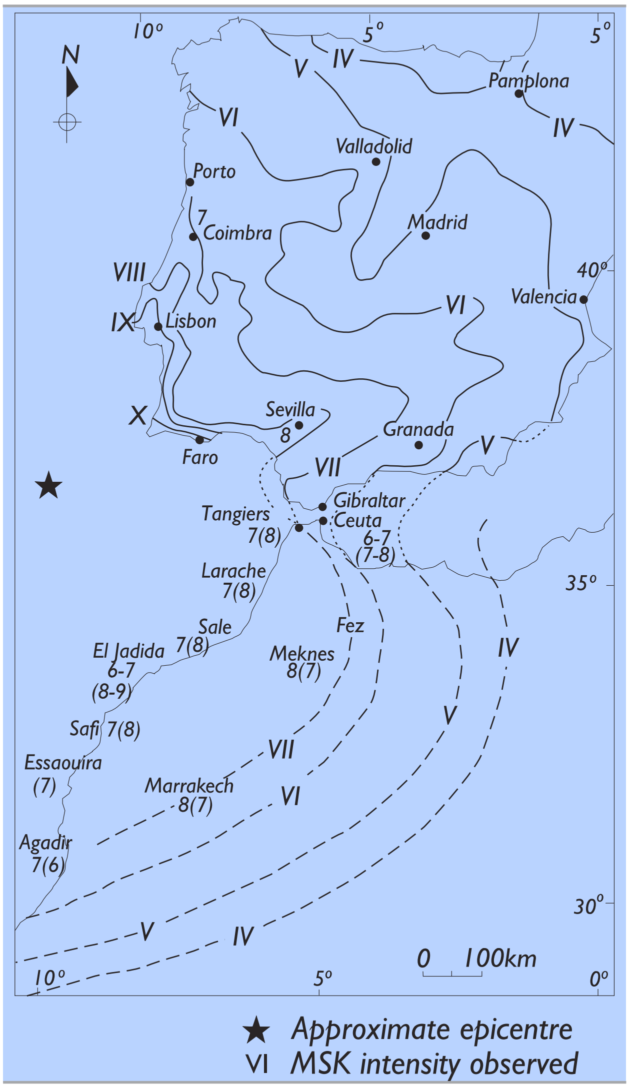
3.10.4.1.2 Intensity

The intensity of an earthquake can be valued somewhat objectively using previously established damage scales. In Europe, it has been agreed to draw up the European Macroseismic Scale, EMS, which is very similar to the MSK scale by Medvedev, Sponheuer & Karnik and that basically coincides (12-point scale) with the Modified Mercalli (or Rossi-Forel) Scale usually applied in the United States. Japan uses its own 10-point scale.

Each earthquake produces damage with intensities generally decreasing with their distance to the epicentre. Having assessed the distribution of damage caused by a particular earthquake, the corresponding *attenuation map* can be drawn up. It is also known as an *isosist map*. Figure 3.10.4 gives an isosist map of the 1755 Lisbon Earthquake, the largest known to have affected the Iberian Peninsula.

The epicentral intensity, I_o, is somehow related to the magnitude of the earthquake. Factors such as focal depth, geological structure, focal mechanism, soil cover, etc. intervene in this relationship.

Figure 3.10.4. Isosists of the 1755 Lisbon Earthquake



3.10.4.1.3 Ground Acceleration

Ground acceleration during a seismic event can be defined, at each point (three coordinates), by a vector (three components). The continuous record of a component in a specific place makes it possible to define an accelerogram for the earthquake.

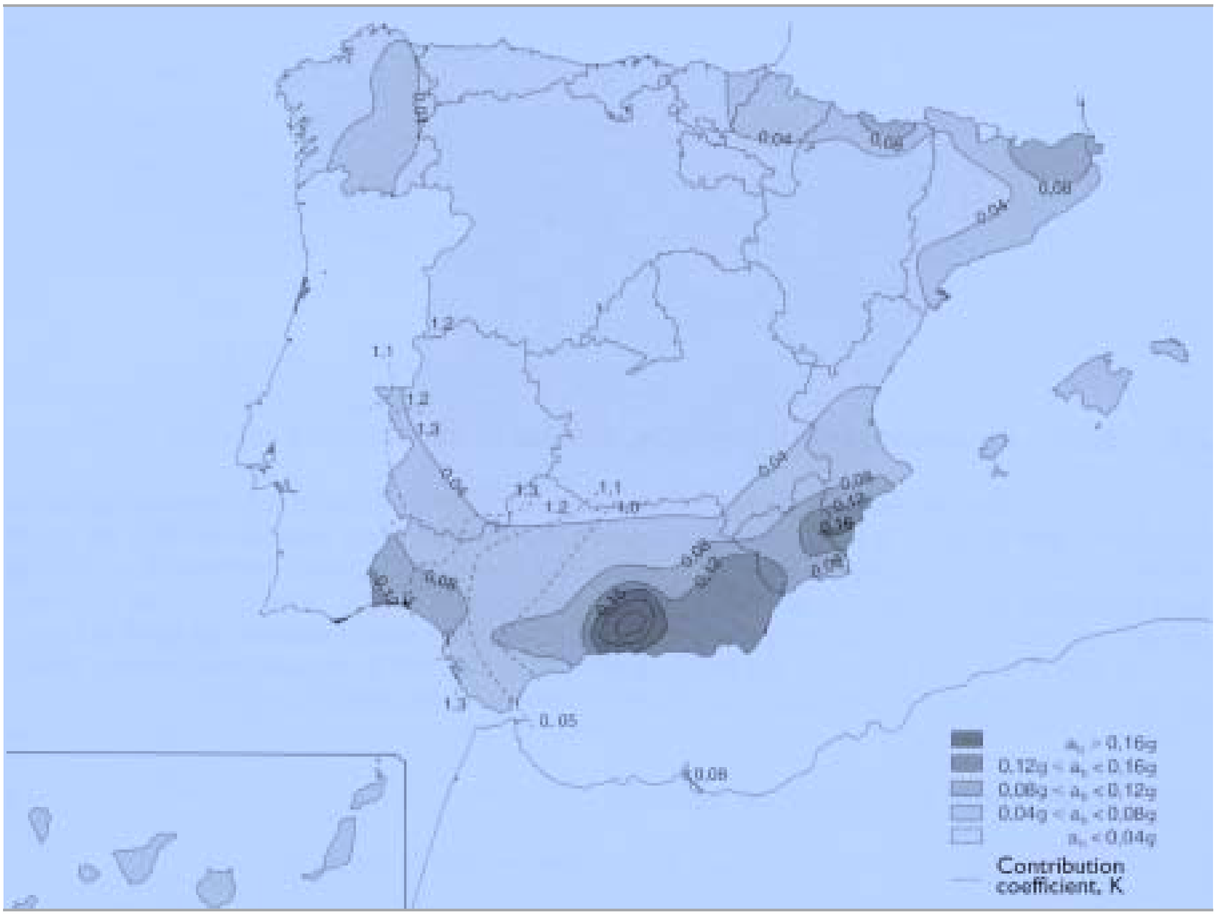
The accelerograms corresponding to points located on the ground surface vary depending on the nature of the soil covering the bedrock at the point in question.

For a particular area, it is common practise to define the value of the maximum acceleration a_{max}, also known as peak ground acceleration, PGA, which represents the greatest value of the horizontal acceleration component and which also usually refers to conditions of "firm soil" or rock.

The a_{max} value corresponding to certain specific return periods is included in most of the seismic codes or standards as a parameter defining the seismic action.

In Spain, the regulation currently in force is the *Norma de Construcción Sismoresistente: parte general y edificación, NCSE-02*, (Design Code for Earthquake-Resistant Construction: General Section and Buildings). This code, whose general provisions are of compulsory application to harbour works, includes, under the name of basic seismic acceleration, a_b, a characteristic value for the maximum horizontal acceleration associated with a return period in the order of 500 years, for each Spanish municipality. Figure 3.10.5 shows the seismic hazard map from this Code, which provides that parameter value in a simplified way.

Figure 3.10.5. Spanish Map of Seismic Hazard (NCSE-02)



3.10.4.1.4 Response Spectrum

The action of an earthquake on a structural system with a degree of freedom, a natural vibration frequency

$$\omega_n = \sqrt{\frac{K}{M}} \qquad (K = \text{stiffness constant},\ M = \text{moving mass})$$

and viscous damping with a constant *C*, can immediately be calculated by the expression:

$$x(t) = \frac{1}{\omega_d} \int_0^t e^{-\omega_n \cdot D(t-\tau)} \cdot \cos\ \omega_d (t-\tau) \cdot a_g(\tau) \cdot d\tau$$

where:

$x(t)$ = relative displacement of the moving mass in relation to the ground.

$a_g(\tau)$ = ground acceleration (accelerogram).

ω_n = natural frequency of the moving mass.

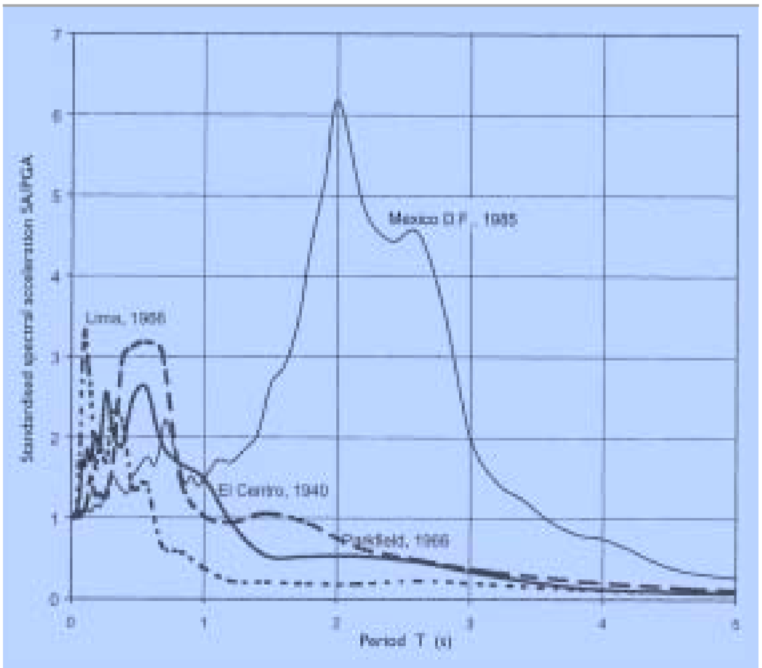
ω_d = dampened frequency of the moving mass; $\omega_d = \omega_n \sqrt{1 - D^2}$

D = relative damping, $D = \dfrac{C}{2\sqrt{KM}}$

From this expression it is possible to obtain the maximum acceleration of the moving mass or the spectral acceleration, *SA*. The approximate expression $SA = \omega_n^2 x_{max}$ is commonly used. This value is known as the absolute pseudo-acceleration and is a function of the natural frequency ω_n (or corresponding T = period) and of the relative damping, *D*.

The response spectrum of the absolute accelerations of an earthquake is defined as the function relating the *SA* value to *T* and *D*. Figure 3.10.6 shows a typical diagram of some response spectra.

Figure 3.10.6. Examples of Response Spectra (Damping D = 5%)



For a specific area, it is possible to define standard response spectra or envelopes of real spectra to be used in designs, corresponding both to horizontal and vertical movements. The Spanish code in force (NCSE-02) defines standard spectra in Spain as a function of the local characteristics of the ground, location of the zone (essentially the distance to the Azores-Gibraltar Fault governing seismic activity in the Iberian Peninsula) and the relative damping of the resistant structure.

3.10.4.2 Soil Liquefaction Due to Seismic Action

Much of the earthquake damage in ports has been due to soil liquefaction phenomena. A clear example is the virtually total destruction of Port Valdez during the 1964 Alaskan Earthquake. Other types of documented earthquake damage in ports are given in Table 3.10.1.

In order to predict the possibility of soil liquefaction occurring as a result of seismic action, at the present time it is practically compulsory to follow the empirical procedure described below. In addition to this method, engineers can and should use whichever other complementary procedures or methods they judge advisable, without this meaning that the method described here should be omitted.

Geotechnical Investigation

In the first place, to evaluate safety against the phenomenon of liquefaction, a detailed geotechnical investigation of the ground under study must be carried out. This investigation should allow a precise determination of the following parameters:

- Dry density, moisture content and unit weight of the soil particles.
- Grading and, if a clayey component is present, plasticity of the fine fraction of the soil.

Table 3.10.1. ***Examples of Seismic Damage to Ports in the Past Two Decades***

Year	Country	Port	Damage
1983	Japan	Akita	Two sheetpile quay walls failed. Cranes collapsed
1984	Chile	San Antonio	Blockwork quay wall failed. Cranes collapsed
1985	Greece	Patras	Rubble-mound breakwater collapsed
1986	Greece	Kalamota	Blockwork quay wall failed
1989	Algeria	Argel	Blockwork quay wall failed
1989	U.S.A.	Oakland	Piled quay
1990	Philippines	San Fernando	Piled quay
1993	Japan	Kushiro	Caisson and sheetpile quay walls
1993	Japan	Hakodate	Sheetpile quay wall
1993	U.S.A.	Guam	Sheetpile quay wall
1993	Japan	Okushuri	Breakwater
1994	U.S.A.	Los Angeles	Piled quay
1995	Japan	Kobe	Piled quay, dolphins, sheetpile cellular quay wall, breakwater. Cranes collapsed
1999	Turkey	Devince	Cranes collapsed
1999	Taiwan	Taichung	Caisson quay wall

List based on the information appearing in the "Seismic Guidelines for Port Structures" PIANC, Ed. Balkema, 2001.

- SPT N-index. Alternatively, continuous penetration tests, CPTU, or dynamic penetration tests can be carried out. In any event, SPTs should not be omitted and the results of other possible tests should be converted into equivalent SPT values using duly verified local correlations.

Additionally, the quality of the study will improve if the results of geophysical tests (cross-hole or similar) are available providing the shear wave propagation velocity v_s at each depth.

Knowledge of the possibility that liquefaction can occur will also be more precise if laboratory-run dynamic soil test results are available, particularly resonant column tests or cyclic simple shear tests that make it possible to know the vales for *G*, *D* and ν referred to earlier (3.10.2.3).

Design Value for the SPT N-Index. Value $(\bar{N})_{60}$

Knowing the abovementioned data, the liquefaction resistance of a specific soil level is defined, for the depth z of the layer, as a function of the number $(\bar{N})_{60}$, which is the average value for the SPT N-Index after corrections due to the effect of overburden (1) and the test energy (2). These two corrections can be done following the indications in Subsections 3.5.4.3 (Correction 1) and 2.9.1 (Correction 2).

Value of the Resistance to Liquefaction. Parameter CRR

For each $(\bar{N})_{60}$ blowcount, a value is obtained for the dimensionless CRR number (Cyclic Resistance Ratio), which measures resistance to liquefaction. This number varies depending on the fines content of the ground.

The values currently considered to be the most adequate for the CRR parameter are given in Table 3.10.2.

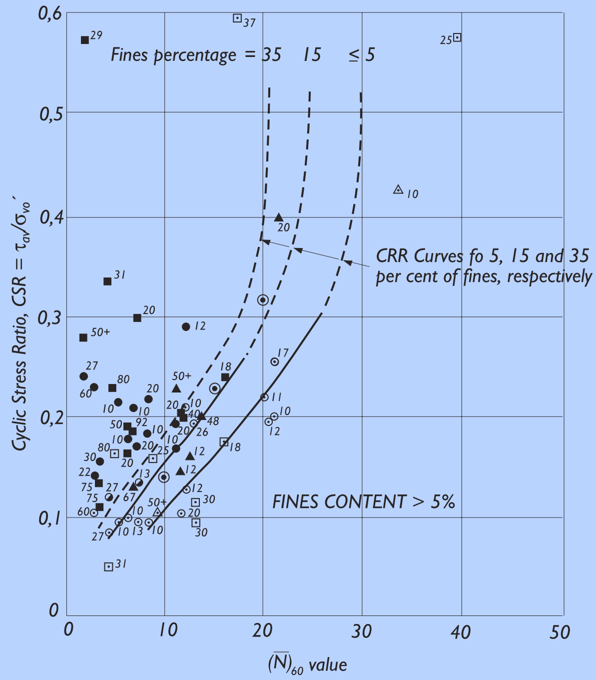
This same information is summed up graphically in Figure 3.10.7. These values are empirical and are based on the field data gradually accumulating. It is possible that they vary somewhat as time passes.

The CRR value indicated only corresponds to earthquakes with a magnitude of M_w = 7.5. When the earthquake magnitude is different, the CRR values should be multiplied by the correction factors given below.

Table 3.10.2. Values of the CRR Parameter

	Fines Percentage		
$(\bar{N})_{60}$	≤ 5%	15%	30%
5	0.06	0.10	0.13
10	0.11	0.16	0.18
15	0.16	0.23	0.26
20	0.22	0.29	0.40
25	0.29	>0.4	>0.5
30	>0.4		

Figure 3.10.7. Liquefaction Resistance (20)



The value obtained for the dimensionless resistance, CRR, after considering the $(\bar{N})_{60}$ value, fines percentage and earthquake magnitude, should be compared to the action described in the following subsection.

TablE 3.10.3. Correction Factor for the CRR Parameter as a Function of Earthquake Magnitude (Youd & Idriss, 1997)

Magnitude M_w	Coreection Factor
5.5	2.20 a 2.80
6	1.76 a 2.10
6.5	1.44 a 1.60
7	1.19 a 1.25
7.5	1
8	0.84
8.5	0.72

(20) Figure drawn up based on information published by Youd, T.L. & Idriss I.M. (2001). "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils", Journal of Geotechnical and Geoenvironmental Engineering. ASCE. Apr. 2001. Vol. 127. N° 4.

Seismic Action. CSR Parameter

At a specific depth inside the deposit of soils and with the geotechnical information available, the following dimensionless number can be calculated:

$$CSR = 0.65 \frac{\sigma_{v_o}}{\sigma'_{v_o}} \cdot \frac{a_{max}}{g} \cdot r_d$$

where:

- CSR = dimensionless Cyclic shear Stress Ratio.
- σ_{vo} = total vertical stress prior to earthquake.
- σ'_{vo} = vertical effective stress prior to earthquake.
- a_{max} = design value of the maximum horizontal acceleration of the ground at the site under study. It is equivalent to the design seismic acceleration obtained from the basic seismic acceleration, a_b, in accordance with the provisions in the NCSR-02 Code (see Figure 3.10.5).
- g = gravity acceleration.
- r_d = reduction factor mainly depth-dependent.

The last factor (r_d) is always less than 1 and measures the reduction of the a_{max} value with depth. A specific analysis of amplification (deconvolution) is required to determine it. It is also possible to assume the following approximate value:

$$r_d = 1 - 0.001 \cdot z^2$$

where z is the depth of the area whose liquefaction is being studied, expressed in metres.

Verifying Safety against Liquefaction

A comparison between the resistance, CRR, and the action, CSR, allows the safety factor against liquefaction to be defined as:

$$F = \frac{CRR}{CSR}$$

When this factor is close to 1 ($0.9 < F < 1.1$), it should be understood that the possibilities of soil liquefaction are high. Even though there are publications attempting to quantify the liquefaction probability based on the value of F, at present it is still not possible rely on this procedure.

When the liquefaction safety factor is not acceptable, it is necessary to change the typology of the solution or, in very specific cases, to go ahead with a ground treatment or substitution or with installing drains to facilitate dissipation of the porewater pressures generated. In Spain, for this reason, a massive densification treatment was executed using vibro-replacement (see Subsection 3.9.3) in the sands of the Bay of Algeciras in order to construct installations for the Los Barrios Thermal Power Station.

3.10.4.3 Seismic Loads Transmitted to Foundation Ground through a Resistant Structure

The combined dynamic response of the soil and structure or their joint displacement under seismic action will be all the more pronounced, the closer the natural vibration period of the soil-structure is to the predominant period of the earthquake.

A large number of harbour structures, especially gravity structures, have natural oscillation periods similar to the predominant periods in earthquakes (0.2 s < T < 2 s), which is why, in the majority of cases, significant dynamic behaviour will be produced in the soil-structure system during the seismic event, and this will give rise to the transmission of substantial loads to the foundation ground.

The loads transmitted to foundations are not obvious in this case. Analyses of soil-structure interaction must necessarily be carried out before they can be determined.

Soil-structure interaction under seismic loads can be calculated by very different methodologies or by approximate pseudostatic empirical procedures.

Subsection 3.10.5 of this ROM 0.5 defines the most common procedures that have been endorsed by some experience of their use.

3.10.5 Dynamic Analysis of Soil–Structure Interaction

The combined dynamic analysis of soil and structure can be done using a wide range of methodologies, the most common of which are described below.

Computing with Numerical Models Fully Representing Foundation Ground

a. The computation involves the following steps

1° Definition of the discrete geometrical model (finite elements or finite differences) that represents a plane section of the problem under study including both the structure and a significant fraction of the foundation ground. Three-dimensional dynamic calculations are also possible, but require considerable work in preparing the geometrical model. The model boundaries must be fitted with springs and dampers representing the radiation damping with a certain degree of precision.

2° Definition of the parameters of the basic behaviour model. To facilitate calculations, the ground is normally represented by a viscous, linear elastic model, formulated in total stresses. The parameters governing the stress-strain relationship, G, ν and D, (see Subsections 3.10.2 and 3.10.3) should be adapted to the resultant strain level, which requires an iterative computation until the non-linear behaviour is reproduced with sufficient accuracy.

Instead of the preceding model, it is also possible, although more complex, to do dynamic calculations in effective pressures, in this case including a double coupled formulation: deformation (1), generation-dissipation of porewater pressure (2). These analyses are highly sensitive to small details and at the present time their use is generally restricted to research work.

3° Load definition. Dynamic analysis attempts to represent the joint behaviour of the structure and ground under a situation of rapidly varying loads. Its dynamic nature is due to the fact that, unlike in static cases, significant accelerations are produced on the structure and/or on the ground.

Dynamic action can be represented by a time history of the values of the corresponding representative parameters:

- In the case of waves, it would be the temporal history of pressures caused by the series of wave heights.

- In the case of earthquakes, the way to represent seismic loads and the way to represent the problem (geometry and characteristics of the materials) should be related in some way, i.e., the seismic load depends on the geometrical model utilised to analyse the problem.

One of the ways of representing them is by a set of inertia forces equal to the products of the masses in the calculation model by accelerations assumed to affect the base of the model. Once these forces are defined, seismic analyses will in all respects be similar to dynamic computations carried out to analyse the effects of other external dynamic loads (wind, waves, impacts, etc.). As seismic displacements are normally specified for firm ground, to obtain them in the base of the model requires carrying out prior amplification calculations (increase in accelerations due to the presence of soft soils over firm grounds) and deconvolution analyses (reduction in accelerations with depth) starting from the movements for firm ground.

4° Integration process. To carry out coupled dynamic calculations (effective pressures), it is essential to integrate the problem using an explicit finite difference scheme for the time variable.

For calculations in total pressures, the process for integrating (solving) the dynamic problem can be done with an explicit finite difference scheme in the time domain (Alternative 1) or by applying a Fourier transform to solve the problem for each frequency and by integrating later in this domain (Alternative 2) or by a modal spectral analysis (Alternative 3). Although these procedures are theoretically equivalent, the first one appears to provide a certain advantage for treating non-linear problems.

b. Computing with a Simplified Representation of the Ground

The most widespread dynamic analysis is the one in which the foundation ground is represented by a series of equivalent dampers and springs. The structural model can consist of a discretization in finite elements, or a model of masses and concentrated springs that represent the geometry, stiffness and inertia of the structure under study with sufficient accuracy.

Both the ground and the structure must be assumed to be linearly elastic. An iterative calculation allows the parameters to be adjusted for the overall strain level.

The damping of the system should be estimated separately, based on the results of the dynamic tests that need to be carried out to this effect. The dynamic load is normally defined by an adequate system of nodal forces. The integration procedure used can be any one of those indicated above (Point. a)).

The *spring constants* are a function of the deformation amplitude and of the load frequencies.

When the integration process is done in the frequency domain, there is no problem with specifying spring constants (stress-strain ratio) that depend on this factor. Otherwise, it is necessary to make some kind of simplification.

In an approximate way, the spring constants can be deduced with the static formulae shown in foregoing sections of this ROM 0.5 for shallow foundations (Section 3.5.) or deep foundations (Section 3.6.). The dynamic coefficients deduced as indicated in Subsection 3.10.3 should be used in these formulae.

For preliminary seismic calculations or for seismic analyses not needing to be very precise, the following ratio can be assumed between the average value of the shear modulus, G, and the shear modulus for small strain, G_o:

Local MSK Intensity	G/G_0 Ratio
VI	1
VII	0.80
VIII	0.50
IX	0.35

In these preliminary or approximate calculations, these modulus reductions should not be applied to the soils whose stress level is far from the one that would cause failure. In the absence of other estimates, it can be assumed that $G = G_o$ for:

a. all soils with a shear wave propagation velocity, measured by *in situ* geophysical tests, of:

$$v_S > 300 \text{ m/s}$$

b. all soils, regardless of their deformability, located at a depth of more than 20 metres.

A relative damping value in line with the corresponding shear modulus should be adopted in these calculations.

When a large part of the structure is buried and the ground is represented by springs and dampers, the definition of the seismic load in the base of the model may require some preliminary calculations. The movement at the level of the connection between the structure and the foundation ground is different from ground movement in the open field (without any structure) even when the structure had no inertia whatsoever. It will be necessary to take this additional interaction effect -known as kinematic interaction- into account, either by a prior dynamic calculation or by some other duly justified consideration. Notwithstanding, it is relatively frequent practice to disregard this effect (validly, in some theoretical cases) and assume that the action on the model is exclusively a set of inertia forces acting on the structure, whose values are estimated based on the seismic accelerations at the supporting ground level.

3.10.5.1 Dynamic Analysis of Foundations

The foundation dynamic calculations can be done jointly with the structural analysis when the approach chosen is Option a) of those mentioned above (see Subsection 3.10.5). Otherwise -Option b)-, it is necessary to do a specific model of the foundation ground and subject it to the dynamic loads obtained from the structural calculation.

In both cases (joint calculation or independent dynamic calculation), the results of the foundation analysis will give the following information in each foundation area:

a. Evolution (or time history) of displacements and total and effective stresses for coupled stress-strain-flow analyses.

b. Evolution of the displacements and total stresses, when the calculation is done in total pressures without taking directly into account the consequences of the water flow inside the ground.

With this information, the behaviour of the system can be appraised. Directly, in the first case, as the calculation will give the displacements and effective stresses, which makes it possible to evaluate the safety of the works and the eventual serviceability during and after the event (storm or any other cause).

In the second case (calculation in total pressures), a subsequent interpretation process is needed in order to obtain the porewater pressures generated by the dynamic load. The starting basis for this interpretation will be the information on the dynamic strength of the soil obtained in laboratory tests in total stresses with similar stress histories to those resulting from the calculation.

If, after this evaluation process, it turns out that the plastification or failure thresholds (liquefaction) are not exceeded in any significant foundation area at any time, then the response of the foundation ground could be considered to be virtually elastic. It should then be concluded that the situation is acceptable.

For significant dynamic loads, however, the ground may undergo transient partial failures that could be admissible, as they need not necessarily imply failure of the works but only a noticeable residual displacement.

The dynamic calculation of displacements has been practiced for decades in geotechnical engineering and its application to failure problems is adequate and recommended. Its conceptual basis is to assume an instantaneous failure with the mechanism shown by the calculation and to posit some distribution of residual strength at that moment.

The equations for the movement of the moving part in this failure mechanism can be written based on the laws governing the dynamics of rigid blocks. Their integration will then provide the displacement occurring during this partial phase of movement. This is a well-established analysis procedure for calculating movements in walls and slopes and can likewise be applied to foundations of maritime and harbour works subjected to wave action.

3.10.5.2 Pseudostatic Analysis of Foundations

The dynamic calculation of foundations, or in general of any works resting on the ground (including excavations with or without support), can be tackled with the usual procedures of statics, but adding additional forces to represent the dynamic load.

The equivalent static loads to be introduced essentially depend on the type of problem under study, as commented below.

3.10.5.2.1 Static Loads Equivalent to Wave or Wind Action

In the problems of dynamic loads due to waves, wind on structures, etc., static calculation methods can be used in which the dynamic actions are represented by some equivalent static loads.

The effects of wave actions, when transmitted to the foundation ground through a resistant structure, depend on the period and the magnitude of the load and especially on the combined response of the soil and structure, that is, their displacements under this load.

Harbour structures such as caisson gravity walls, breakwater superstructures and other monolithic works have natural oscillation periods in the range of 0.2 to 2 seconds (far away from wave periods). This is why the loads transmitted to the ground due to the action of non-breaking waves or tidal sea-level oscillations are not expected to undergo dynamic amplification. It can be assumed that these actions are transmitted to the ground with the same period as the acting load. In these cases, the load transmitted to the foundation ground can be obtained by static analyses in which, a priori, the stiffness of the soil–structure system does not need to be considered.

On the contrary, substantial dynamic amplifications are certainly to be expected in the loads transmitted to the foundation ground when these structures are under the action of breaking waves, as these loads have oscillation periods in the same order of magnitude as those of the soil-structure system. In these cases, the loads transmitted to the foundations are difficult to evaluate and a specific dynamic calculation or an alternative approximate empirical procedure is needed (e.g., multiplication of the static load by a dynamic load factor), so that the dynamic load transmitted to the foundation ground can be defined for the structure in question.

Very different methodologies can be used for calculating the joint dynamic response of soil and structure. Subsection 3.10.5 defines the most common procedures, which have been somewhat endorsed by experience.

The process of obtaining static loads equivalent to the action of waves or wind may include considering certain “dynamic load factors” or directly considering equivalent forces obtained from experimentally-based empirical formulations, which lie outside the scope of this ROM 0.5.

In the ROM publications devoted to wind and wave action and to specific works, the equivalent static forces will be defined that are associated with the representative value adopted in each load combination for the wind or wave storm.

For Serviceability Limit States, behaviour can be evaluated with a similar assumption, but corresponding to a less intense storm (more frequent) as stated in Section 3.3 of this ROM 0.5.

The equivalent static forces acting on the foundation ground can be obtained from a static structural calculation in which static loads equivalent to the dynamic load are acting on the structure or else by adopting the maximum force transmitted to the foundation as deduced from a dynamic calculation.

To evaluate safety for the failure modes associated with Ultimate Limit States in shallow and deep foundations, when they are analysed using the static formulae given in Sections 3.4 and 3.5 of this ROM 0.5, it is advisable not to include any additional force to represent the dynamic load due to soil inertia.

3.10.5.2.2. Static Loads Equivalent to Seismic Action

When failure modes are verified using approximate static formulations, an earthquake can be considered through equivalent static loads. These equivalent loads depend on the typology of the structure, the characteristics of the soil-structure interaction and the failure mode under analysis.

The ROM 0.6 publication devoted to seismic action and the ROM publications devoted to specific works will define in detail the static forces equivalent to the seismic load.

Nevertheless, it seems convenient to include in this ROM 0.5 the static forces equivalent to seismic load needed to verify the most usual geotechnical modes occurring in maritime and port works.

3.10.5.2.2.1 Equivalent Static Forces Acting on Foundations

The equivalent static forces acting on foundations can be obtained from a structural static calculation by considering the action on the structure of static loads equivalent to the seismic action or by adopting a percentage of the maximum force transmitted to the ground as deduced from a dynamic calculation. In the absence of further data, 65% of that maximum value can be taken as the equivalent static force.

If the dynamic calculation does not include the self-weight of the foundation element (footing or pile cap), then an additional force will have to be added. This, in the absence more precise indications to be found in more specific places (other ROM publications), will be the product of its total weight (with emerged density) by the horizontal acceleration a_h, defined in Subsection 3.10.5.2.2.2.

3.10.5.2.2.2 Equivalent Static Forces for Verifying Failure Modes of Loss of Overall Stability

To check the effect of seismic load on the analysis of overall stability problems governed by ground strength, and in the absence of a better procedure for doing a dynamic calculation, it can be assumed that the seismic load is equivalent to some inertial body forces defined as the product of the mass by the following accelerations:

$$\text{Horizontal: } a_h = \alpha \cdot a_c$$

$$\text{Vertical: } a_v = \frac{1}{2} a_h$$

where a_c is the design seismic acceleration at the site, defined in the *Norma de Construcción Sismorresistente* now in force (NCSE-02).

The factor α attempts to evaluate the flexibility of the works under earthquakes. The more restricted the movement, the higher its value should be.

Isolated slopes, unrestrained gravity quays, etc.	$\alpha = 0.5$
Slopes with rigid inclusions (e.g. piles) or walls with head restraints, anchored sheetpile quay walls, etc.	$\alpha = 0.75$ to 1

3.10.5.2.3 Equivalent Earth Pressures on Retaining Structures

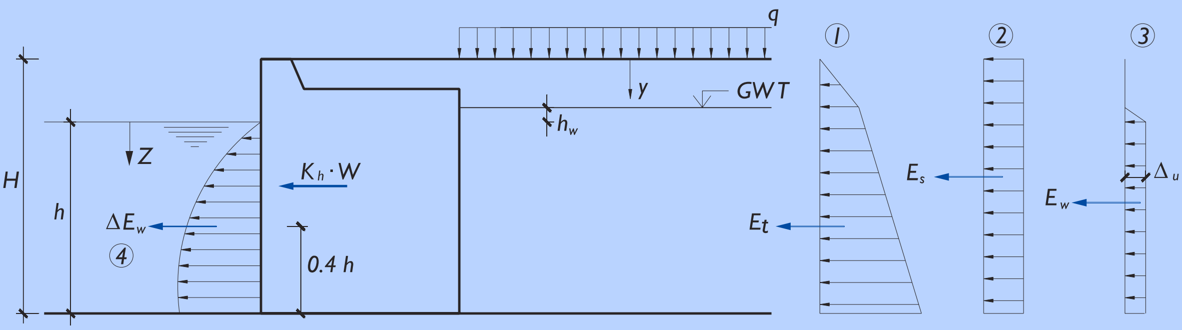
Seismic action causes a transient increase in earth pressure on walls. The active earth pressure in seismic conditions is greater than the one corresponding to the static situation.

Similarly, the passive earth pressure that can be transmitted by retaining structures supported against the ground can be significantly reduced during earthquakes. The passive earth pressure in seismic conditions is less than the one corresponding to the static situation.

The ROM 0.6 on "Seismic Action" will include detailed recommendations on the methodology that should be followed to take the seismic effect into consideration in the design of retaining structures.

A simple method can be provisionally suggested here for calculating earth pressures, whose application is advisable in cases of moderate seismic action, i.e., where it does not substantially affect the design. In other circumstances, engineers must use more appropriate methods.

Figure 3.10.8. Diagram of Equivalent Earth Pressures on a Retaining Structure in Seismic Conditions



1 Effective earth pressure, E_t
2 Earth pressure due to overburdens, E_s
3 Excess hydrostatic pressure, E_w
4 Hydrodynamic depression.

$E_t = \int_0^H K_{ad}\, \sigma'_v\, dy$ As with static conditions but changing K_a by K_{ad}

$E_s = \int_0^H K_{ad} \cdot q\, dy$ As with static conditions but changing K_a by K_{ad}

$E_w = \int_0^H \Delta_u \cdot dy$ As with static conditions

$$\Delta E_w = \int_0^h \frac{7}{6} \cdot \frac{a_h}{g} \gamma_w \cdot \sqrt{z \cdot h} \cdot dz = \frac{7}{12} \cdot \frac{a_h}{g} \gamma_w \cdot h^2$$

W = total weight of the caisson or wall and any earth covering it.

See text for other details.

3.10.5.2.3.1 *Active Earth Pressure*

a. Coefficient of Active Earth Pressure in Dynamic Conditions

The coefficient of active earth pressure in dynamic conditions can be taken as equal to the result of the following expression:

$$K_{ad} = \frac{\cos(\alpha + \theta)}{\cos\theta \cdot \cos\alpha} \cdot K_a^*$$

The coefficient K_a^* will be obtained by the formulae defining the coefficient of active earth pressure in static conditions (Subsection 3.7.5), but entering angle $(\alpha + \theta)$ in place of α and $(\beta + \theta)$ in place of β.

The θ angle should be as defined by the following expressions:

$$\theta = \arctan\left(\frac{a_h}{g - a_v}\right) \qquad \text{Case 1}$$

$$\theta = \arctan\left(\frac{a_h}{g - a_v} \cdot \frac{\gamma_d}{\gamma'}\right) \qquad \text{Case 2}$$

$$\theta = \arctan\left(\frac{a_h}{g - a_v} \cdot \frac{\gamma_{sat}}{\gamma'}\right) \qquad \text{Case 3}$$

where:

g = acceleration of gravity.
a_h, a_v = horizontal and vertical equivalent seismic accelerations. The values for these parameters are defined in Subsection 3.10.5.2.2.2.

Case 1 corresponds to dry or partially saturated backfill, always located above the groundwater table.

Case 2 corresponds to fills below the groundwater table and that are free-draining even under seismic conditions (rockfill, for example).

Case 3 corresponds to fills located below the groundwater table that are not free-draining.

When Coulomb's formulae are used in this process for calculating the coefficient of active earth pressure, the result is the Mononobe-Okabe formulae shown in the top section of Figure 3.10.9.

Once K_{ad} is calculated, the procedure for estimating earth pressures (unitary pressures, effect of water, effect of cohesion, effect of overburdens, etc.) will in all respects be similar to the procedure described in Subsection 3.7.5 for calculating active earth pressures, although the following considerations must be taken into account.

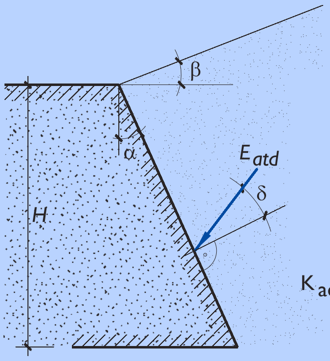
b. Angle of Friction between Earth and Wall

The angle of friction between the earth and the wall can be substantially reduced during earthquakes. This would mean an additional increase in active earth pressure.

This reduction of δ will be greater the more sensitive the backfill material is to the effect of seismic vibrations.

Engineers should take this into account and cautiously reduce angle δ for use in the calculations. The assumption that $\delta = 0$ is always conservative.

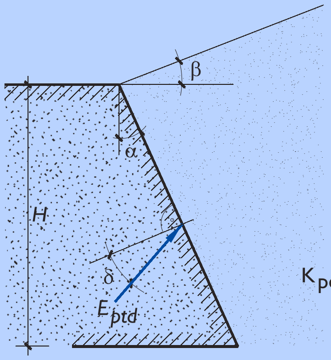
Figure 3.10.9. **Mononobe-Okabe Formulae**



ACTIVE EARTH PRESSURE

$$E_{atd} = \frac{1}{2}\,\gamma\,K_{ad}\,H^2$$

$$K_{ad} = \frac{1}{\cos\theta}\left[\frac{\sec\alpha \cdot \cos(\phi - \alpha - \theta)}{\sqrt{\cos(\alpha + \delta + \theta)} + \sqrt{\dfrac{\sin(\phi + \delta)\quad \sin(\phi - \beta - \theta)}{\cos(\alpha - \beta)}}}\right]^2$$



PASSIVE EARTH PRESSURE

$$E_{ptd} = \frac{1}{2}\,\gamma\,K_{pd}\,H^2$$

$$K_{pd} = \frac{1}{\cos\theta}\left[\frac{\sec\alpha \cdot \cos(\phi + \alpha - \theta)}{\sqrt{\cos(\alpha + \delta - \theta)} - \sqrt{\dfrac{\sin(\theta - \delta)\quad \sin(\phi - \beta - \theta)}{\cos(\alpha - \beta)}}}\right]^2$$

c. Specific Weight To Be Used in the Calculations

The effective earth pressure due to the weight of the earthfill should be calculated using higher unit weights for the ground than the real ones.

The unit weight of the ground should be increased both in the area situated above the groundwater table, γ_{ap}, and in the area below it, γ'. In both cases, the amplification factor for these specific weights will be:

$$f = 1 + \frac{a_v}{g}$$

d. Pressure Due To Pore Water

The pressure of the porewater in the backfill under seismic conditions should be taken to be equal to the one calculated under static conditions.

e. Effect of Cohesion

The possible beneficial effect of cohesion can be taken to be equal to that in the static situation, i.e., independent of the seismic action.

f. Effect of Loads and Overburdens on the Backfill

The effect of any loads and surcharges existing in the backfill area can be evaluated using the same statics procedures as indicated in Subsection 3.7.5.

The intensity of the overburdens should be multiplied by the factor *f* defined in Point c).

The value of the coefficient of active earth pressure to be used in formulae should be the one corresponding to dynamic situations, as previously defined.

g. Pressure due to Free Water

When there is free water on the front face (the usual case in harbour quays), its dynamic pressure must be taken into account by an adequate procedure. To this effect and as initial approximation, Westergaard's theory is admitted:

$$\Delta E_w = \pm \frac{7}{12} \frac{a_h}{g} \cdot \gamma_w h^2$$

where:

ΔE_w = increase in free water pressure caused by the earthquake.
h = depth of free water.
g = gravity acceleration.
γ_w = unit weight of the water.

The value of the horizontal equivalent seismic acceleration, a_h, was defined in Subsection 3.10.5.2.2.2.

This increase in pressure can be taken as applied at a depth of 0.6 h, measured from the free water level.

3.10.5.2.3.2 Passive Earth Pressure

The passive earth pressure on walls can be reduced during earthquakes. This reduction can be studied by a procedure similar to the one indicated in the previous paragraphs.

The coefficient of passive earth pressure under dynamic conditions can be obtained by the expression:

$$K_{pd} = \frac{\cos(\alpha - \theta)}{\cos\alpha \cdot \cos\theta} \cdot K_p^*$$

The angle θ will be the same as the one defined for the case of active earth pressure but should be subtracted from angles α and β in the formulae providing the coefficient of passive earth pressure under static conditions defined in Subsection 3.7.7. The corresponding value for K_p^* will thus be obtained.

When this simplified procedure is applied to Coulomb's method, the Mononobe-Okabe formula for passive earth pressure results, as shown in the bottom section of Figure 3.10.9. This formula may be optimistic for large angles of δ or β.

The correction factor f used to increase the specific weights will be obtained by changing the sign of the vertical acceleration, thus obtaining values for f that are less than 1.

The angle of friction between the soil and the wall must be cautiously reduced, applying the same criteria indicated for the active case.

3.10.5.2.4 Equivalent Static Forces for Checking Slope Stability

The effect of earthquakes on slopes can be analysed in the simplified manner set out below only in cases where the seismic action does not significantly condition the design. Otherwise, a more detailed study will be necessary, which lies outside the scope of this ROM 0.5.

The seismic effect should be considered as equivalent to body forces equal to:

$$\text{Horizontal force} = \pm \frac{a_h}{g} \cdot \gamma \qquad \text{Vertical force} = \pm \frac{a_v}{g} \cdot \gamma$$

where γ is the unit weight of the ground and a_h and a_v are the equivalent seismic accelerations defined in Subsection 3.10.5.2.2.2.

Irrespective of whether the stability calculation is done with total weights –and subsequently subtracting the effect of uplift– or in terms of effective weights without uplift (see Subsection 3.8.4), the value of γ to be used in the above expression should be the apparent unit weight of the ground.

When there is free water totally submerging the slope, the calculation should be done using the submerged weights below the free water level (see Subsection 3.8.4), as otherwise excessively pessimistic results would be obtained.

The hydrodynamic effect of the free water should not be taken into account in this simplified procedure.

The most harmful dynamic effect on overall stability of the works is, however, the weakening of the ground strength that alternating loads can cause. The sensitivity of the soils to this reduction in strength must be explored, be it the result of increase in the pore water pressures or for other reasons. The strength parameters to be used in the calculations should take this effect into consideration.

Part IV

Particular Geotechnical Aspects of different types of Maritime and Harbour Works

Particular Geotechnical Aspects of different types of Maritime and Harbour Works

Part IV

4.1. INTRODUCTION 399

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4.1 INTRODUCTION

From the geotechnical point of view, each type of maritime and harbour works has certain particular features. Part 4 of this ROM 0.5 highlights the most significant aspects of some of the most frequent types of works.

If the entire ROM Programme has been conceived as a dynamic and ongoing activity, allowing for possible revisions and extensions, this particularly applies to Part 4, which has been published in what is considered to be a condensed form with the intention of completing it as the full ROM Programme progresses.

But the intention has certainly been to provide some initial guidelines, applicable to different types of maritime and port works that can help engineers in their design work.

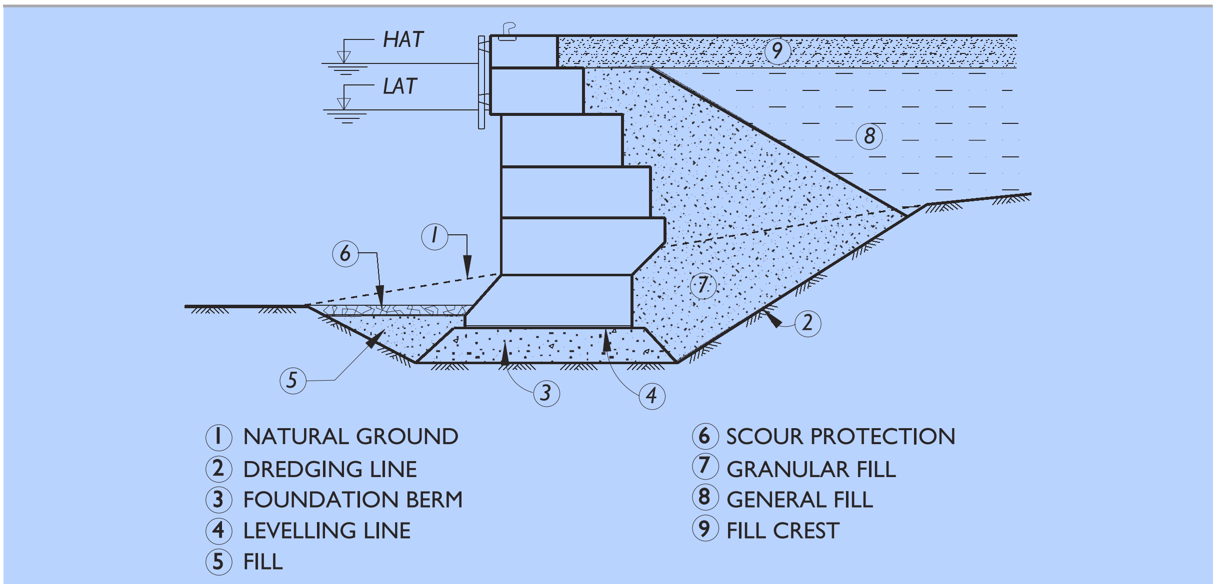
4.2 GRAVITY QUAYS WALLS

Gravity quays are berthing facilities fundamentally using the self-weight of the wall structure to retain the difference in ground levels from the backfill to the front face.

The most usual types in Spain are referred to as *blockwork*, *caisson* and *submerged concrete* gravity quay walls. Other special configurations, such as sheetpile enclosures, also retaining earth pressures by gravity, are dealt with in Section 4.5, since the use of sheetpiles converts them into special structures.

Figure 4.2.1 illustrates the general structure of blockwork quay walls, indicating the most characteristic elements from a geotechnical point of view.

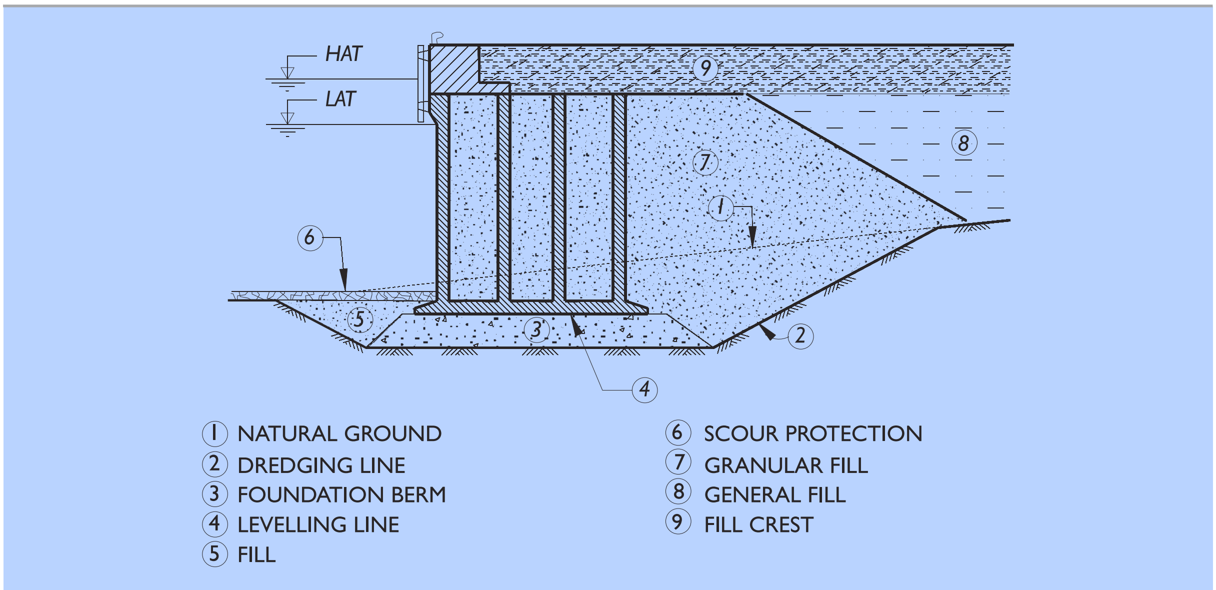
Figure 4.2.1. **Blockwork Quay Wall**



These quay walls can also be constructed using cavity blocks or with initially hollow cylinder blocks that are subsequently filled solid.

Similarly, Figure 4.2.2 shows the general structure of caisson quay walls and the most characteristic geotechnical elements.

Figure 4.2.2. ***Caisson Quay Wall***



The caissons used to construct quays usually have a closed plane bottom as shown in the figure, but can have a different type of base to suit the foundation ground. In foundations to be excavated after founding the caisson, it is possible to leave a chamber in the base, formed by side walls and a ceiling that would them be the raised bottom for the caisson.

4.2.1 Natural Ground. Site Investigation

The main factor that will determine the feasibility of this type of solution is the natural ground. Given their nature, gravity quay walls need to transmit high pressures to the ground. Soft soil is not generally apt for supporting this type of quay.

Site investigation is fundamental to the design of this type of work. The strength and deformability of the ground in the area affected by the works must be ascertained. Part 2 of this ROM 0.5 gives recommendations on this issue.

Ground investigations must also aim at studying eventual dredging. This may be necessary to achieve the required draught or to prepare the foundation ground, removing the softer top soils.

4.2.2 Source and Quality of Materials

In addition to the natural ground, other materials are involved in the design of a quay, which have a considerable effect on its future behaviour.

The backfill can be made of several materials: one in direct contact with the quay wall structure, which will generally be granular material, and another general fill, farther away from the quay structure, which can be of different nature (but may also have to fulfil certain conditions). A third type of fill material may exist above the water level, overlying the other two.

These three materials (or possibly more in some projects) must be characterised from the geotechnical point of view in order to know their grading, dry density, void ratio, shear strength, permeability and, sometimes, deformability.

It is advisable to obtain the strength parameters of the fills close to the quay wall, which intervene in its stability, by specific shear tests in density conditions similar to the ones expected once the quay begins operation. An indirect determination of their properties using tables of approximate values (such as the one included in Section 4.9) or by indirect correlations will generally be conservative and it may therefore be beneficial to carry out specific tests.

The materials used in foundation berms and any possible armour layers for erosion protection will generally consist of rock fragments (quarry run or rockfill). For these materials, it is expedient to know the nature of the rock, its unconfined compressive strength and its weatherability, plus the grading curve to be used. The shear strength, permeability and compressibility of the support berms can be indirectly evaluated as a function of these properties and its durability also estimated.

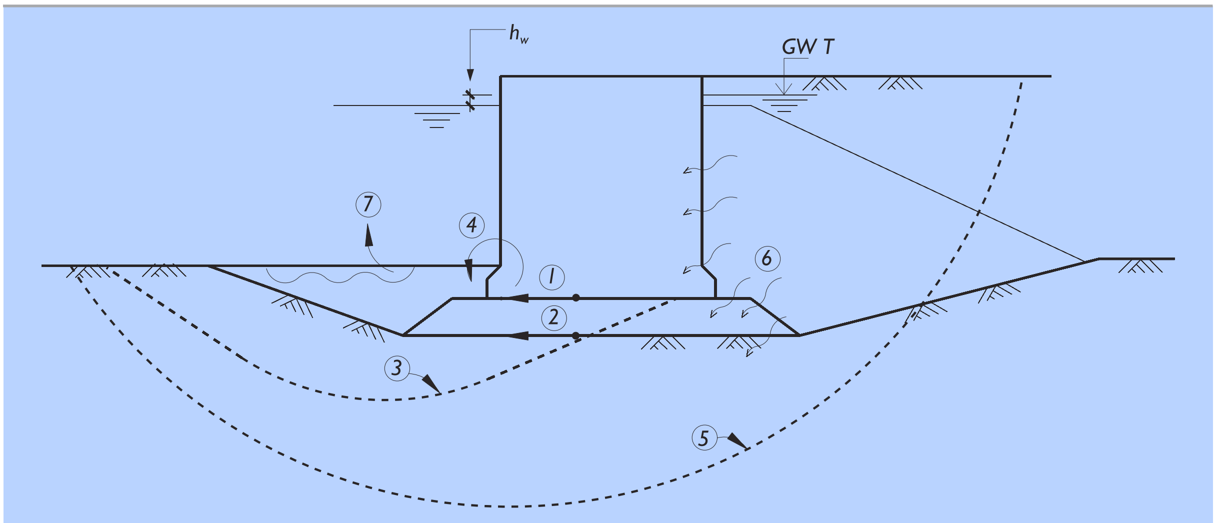
In cases where the strength of the support berm conditions the design, special tests may be desirable to determine this strength, such as shear tests on large-sized specimens.

Some further comments are included in Subsection 4.7.2.2 on the shear strength of rockfills that engineers need to know.

4.2.3 Ultimate Limit States

The theoretical checks that should be made at the design stage should include all those aimed at verifying that no Ultimate Limit State is exceeded, including the ones illustrated in Figure 4.2.3 and commented below.

Figure 4.2.3. ***Geotechnical Failure Modes in Gravity Quay Walls***



1. SLIDE ALONG THE BERM-CONCRETE CONTACT
2. SLIDE ALONG THE CONTACT BETWEEN BERM AND NATURAL GROUND
3. BEARING FAILURE
4. PLASTIC OVERTURNING
5. OVERALL STABILITY
6. INTERNAL EROSION IN BACKFILL OR FOUNDATION
7. SCOUR

N.B.: Only geotechnical failure modes are listed, i.e., those mainly governed by ground characteristics.

To verify safety against any of these failure modes, the corresponding design factors must be defined, namely geometry of the problem, actions and their combinations and ground properties. This definition process should be guided by the recommendations given in Part 3, particularly Subsection 3.3.5.

As a significant aspect of the loads and the geometry of the problem, the design water levels in the backfill and front face of the quay should be defined, as these levels are important when it comes to verifying the safety of the works. To this end, the indications in Subsection 3.4.4.1 should be taken into account.

As a general rule, the most unfavourable condition will be low tide with a water level in the backfill with the maximum possible difference to the external water. In these cases, the representative levels to be adopted for the external water will be a function of the type of load combination studied, as specified in Table 3.3.1 in this ROM 0.5.

The water level differences between the free water on the front side and the groundwater table in the backfill (h_W value in Fig. 4.2.3) should be fixed after a detailed study of the corresponding transient flownet (see Section 3.4) or based on the existence of relevant statistical or experimental data. The most usual cases are summarised in Table 3.4.1 as a function of the long-period oscillations of free water and of the permeability of the foundation ground, fills and works involved.

The groundwater table will not generally have a plane surface in the backfill and the design engineer can consider it so. Section 3.7 indicates the manner for calculating earth pressures when the line of zero porewater pressure is not straight.

In most cases, it will be possible to assimilate the position of the groundwater table in the backfill to a plane and assume a hydrostatic pressure of the water acting on both sides of the quay wall and a linear uplift on the base of the structure. This simplification normally errs on the safe side. Anyway, engineers should decide on this matter.

Waves inside harbours and capable of affecting the quays should be taken into account in the calculations. They will affect the free water level and could modify its pressure on the quay. Except in special circumstances, it should be assumed that potential waves will not modify the location of the groundwater table in the backfill.

Sliding between blocks in gravity quays is an Ultimate Limit State essentially governed by the structural strength of the contact between concrete elements. It is not therefore considered to be a geotechnical issue and consequently lies beyond the scope of this ROM 0.5.

Similarly, in caisson quays, the superstructure will normally be connected to the caisson itself by structural elements. If this is not the case, sliding along this connection should be verified not to occur. This also constitutes a structural problem and as such also lies beyond the scope of this ROM 0.5.

4.2.3.1 Verifying Safety against Sliding between Concrete and Support Berm

The support berm will normally have an upper granular layer with a high shear strength (it will generally be a so-called "levelling gravel") but the friction in the contact between the berm and the precast concrete should be taken as less than that of the granular materials forming the levelling layer, as indicated in Subsection 3.5.5.2. In any event, the shear strength values of the materials constituting the levelling layer and the berm will be necessary for verifying safety in this type of failure.

The criteria recommended for carrying out these verifications and the minimum values for safety factors are given in Subsection 3.5.5 of this ROM 0.5.

The recommendations included in Section 3.7 of this ROM 0.5 should be followed when calculating active earth pressures on the wall back face.

In order to improve the conditions with respect to this Ultimate Limit State, sand and cement grouting can be carried out in this contact zone or other special measures taken (high-friction artificial bases, pouring *in situ* concrete or mortar in this contact zone, etc.).

4.2.3.2 Verifying Safety against Sliding in the Contact between Berm and Natural Ground

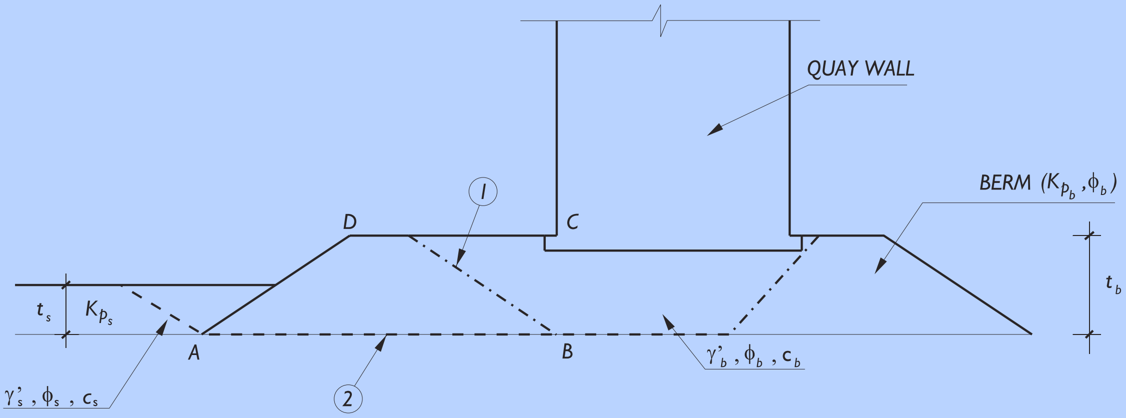
When the natural ground has lower shear strength than the one corresponding to the contact between the quay wall structure and the levelling berm, it will be necessary to check that there is sufficient safety against a deeper slip occurring along the contact between the natural ground and the berm.

This check must assume that the active earth pressure of the backfill acts on the quay wall and the berm, in addition to the other actions (or their combinations) corresponding to each design situation.

The recommendations given in Section 3.7 of this ROM 0.5 should be followed when calculating the active earth pressures in the backfill.

To calculate the passive resistance on the front side, it is necessary to choose one of the following two alternatives (see Fig. 4.2.4).

Figure 4.2.4. *Sliding along the Berm Base*



CALCULATION OF PASSIVE RESISTANCE ON THE VERTICAL PLANE $\overline{CB}$

Case 1

Passive resistance $R_p = \frac{1}{2} \cdot K_{Pb} \cdot \gamma'_b \cdot t_b^2$

Case 2

Passive resistance $R_p = W'_{ABCD} \cdot \tan \phi_c + \overline{AB} \cdot c_c + \frac{1}{2} \cdot \gamma'_s \cdot K_{P_s} \cdot t_s^2$

where:

c_c, ϕ_c	=	Shear trength parameters of the AB contact.
γ'_b	=	Submerged unit weight of the berm material.
γ'_s	=	Submerged unit weight of the natural ground.
K_{P_s}, K_{P_b}	=	Passive earth pressure coefficients of the soil and the berm, respectively.

1. Failure line going up through the berm. In this case, the resistance that should be added is the passive pressure of the berm.
2. Failure line running all along the contact between natural ground and berm. The soil's passive pressure should be added.

From these two options, the one leading to a lower safety factor should be selected.

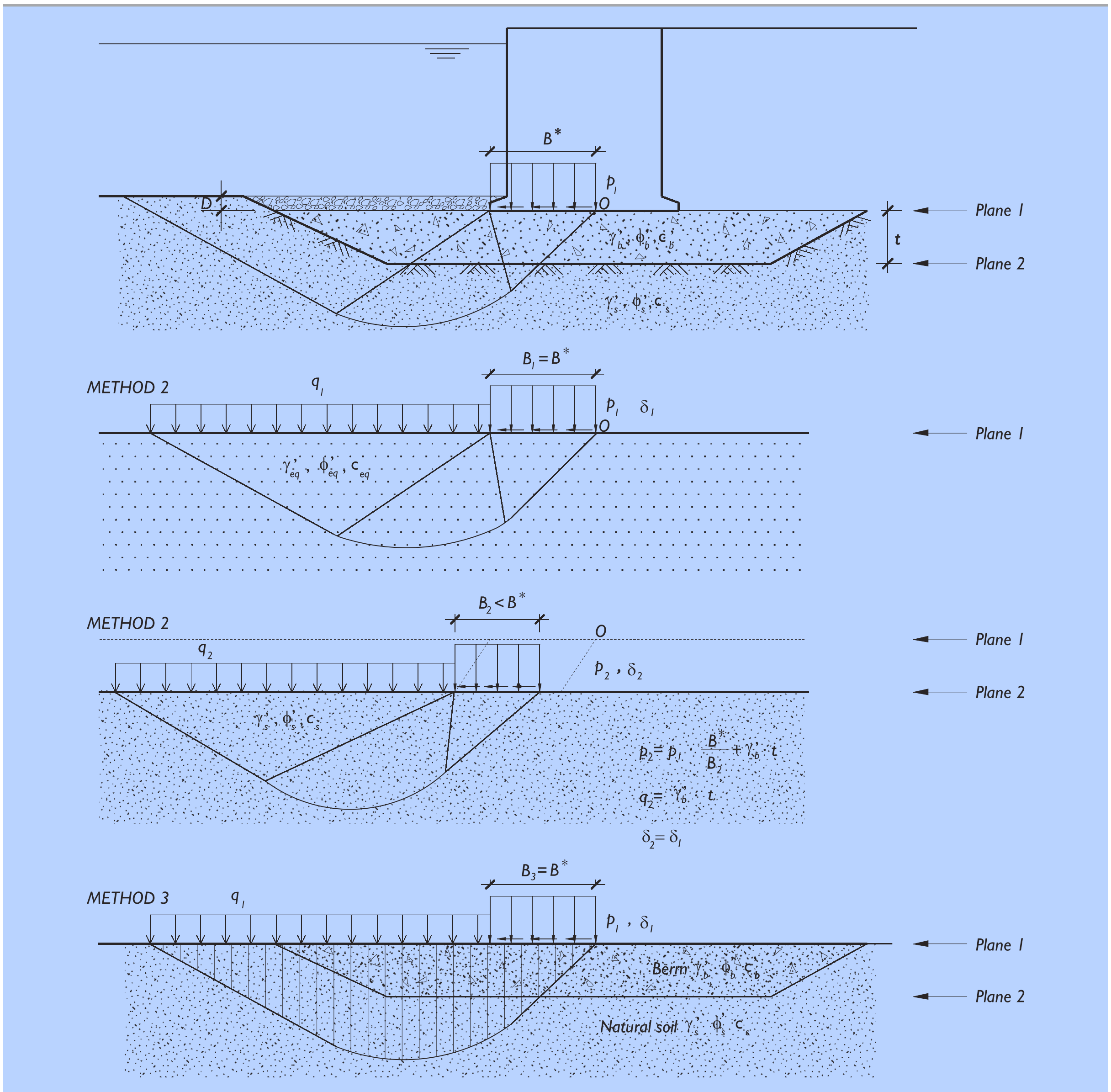
The minimum factors of safety required against this failure mode are given in Table 4.2.1.

4.2.3.3 Verifying Safety against Bearing Failure

Bearing failure should be checked in accordance with the recommendations given in Section 3.5 for shallow foundations.

Owing to the presence of the berm, calculation of the bearing capacity will correspond to a ground with two levels of clearly different strength. Detailed calculation of the ultimate bearing load using the theory of plasticity is complex and it is therefore considered admissible to use one of the simplified procedures illustrated in Figure 4.2.5.

Figure 4.2.5. Simplified Methods for Calculating the Bearing Capacity of a Gravity Quay Wall on a Berm



Method 1. Equivalent Homogeneous Ground

In this simplified procedure, the berm and the natural ground are represented by an equivalent homogeneous ground with strength characteristics (γ_{eq}, c_{eq} and tan ϕ_{eq}) which are the weighted average of the corresponding values for both materials, as indicated in Subsection 3.5.4.8.4.

This method is adequate for calculations in which the natural ground has significant internal friction (foundation ground in drained conditions). For undrained calculations where the ground's angle of friction is assumed to be $\phi = 0$, it is not correct to obtain an equivalent angle of friction. Method 2 or 3 should be used to calculate bearing capacity in these cases or else they should be analysed with adequate numerical models.

Method 2. Transferring Loads down to the Plane of the Berm Base

To use this method, not only the equivalent active loads and the corresponding passive loads have to be calculated but it is also necessary to define an adequate foundation breadth in Plane 2.

In the absence of other information, it can be assumed that $B_2 = B^* - 1/2\ t \geq 0.5\ B^*$.

For calculations in undrained conditions, assuming that the soil has no friction ($\phi = 0$), the choice of the B_2 value is also important (in potential cases of cohesion growing with depth) but not as decisive as for the case of drained analyses ($\phi \neq 0$).

The value of the inclination angle of the surface load, δ, should be considered constant as the load is transmitted downwards.

The method of transferring loads down to a virtual footing is not very adequate for calculating capacities in drained conditions. Method 1 is preferable in these circumstances.

For the case of undrained failure ($\phi = 0$), the rather more detailed procedure given in 3.5.4.8.5 can be applied, making it possible to estimate more accurately the p_2 values recommended for use.

Method 3. Using Methods of Slices

The bearing capacity can be obtained by using methods of slices on several trial failure lines passing through Point 0.

Details on the procedure to use in this case and the expected accuracy of this method are commented on in Subsection 3.8.4.

Although Method 3 should always be used, this does not rule out the use, in addition, of Method 1 (drained foundations) or Method 2 (undrained foundations). A comparison of their results will lead to more accurate final values.

In any event, unless the situation is clear, a numerical model is advisable to check this failure mode.

4.2.3.4 Verifying Safety against Plastic Overturning

Detailed calculation of blockwork quay walls will make it possible to obtain the position of the point at which the resultant of the action of the upper part of the quay passes through the top face of each of the blocks.

Some wall blocks overturning over the resting blocks is a structural problem considered to lie outside the scope of this ROM 0.5.

Overturning about a point near the quay wall toe on the front face (seaward side) should be analysed as described in 3.7.11.1.2 and the resulting safety factor should not be less than the one recommended in Table 4.2.1.

In this ROM 0.5, the failure mechanism or mode referred to as overturning is governed by the same equations that describe bearing failure. The difference between bearing failure and overturning lies in the fact that in the first case failure will happen when increasing simultaneously the horizontal and vertical loads. In the second case, only the horizontal loads will be increased. Consequently, the load that gives rise to overturning will be more inclined and more eccentric and, therefore, breadth B^* should be much smaller and the failure line for overturning would be not so deep-seated as that for bearing failure.

Bearing this in mind, the same calculation procedures as indicated for the simplified study of bearing failure in quays are applicable to the case of overturning (see Fig. 4.2.5).

4.2.3.5 Verifying Safety against Overall Instability

Earlier points refer to failure mechanisms occurring near certain planes, the weakest and closest to the structure of gravity quay walls. It may be, however, that failure lines further apart from the structure itself are more critical.

Failure along surfaces enveloping the whole of the quay wall should be analysed in accordance with the recommendations given in Section 3.8, and the minimum safety requirements set out in that section should be complied with.

Among these mechanisms for loss of overall equilibrium in gravity quays, express mention must be made of the case of deep sliding along a potential weak stratum in the ground. It is possible that the presence of a weak zone of small thickness is compatible with reasonable safety against the bearing failure, overturning and sliding mechanisms set out in earlier sections. Nevertheless, a failure mechanism with a slip line developing mostly inside such potential weak zones could make it necessary to analyse overall stability along non-circular lines, with a flat base in the weakest level. This analysis should be carried out following the recommendations given in Section 3.8.

4.2.3.6 Verifying Safety against Internal Erosion of the Backfill

Water movement due to the action of waves or tides around the quay can cause entrainment of the materials from the backfill, the berm or even the foundation ground. If not held in check, this process can cause damage to the platform and even to the quay wall.

Safety against this type of failure cannot be quantified by simple or well-established calculation procedures and consequently such safety must be achieved by preventive measures in design.

Potential paths of water flow should be identified at the design stage and the grading of the material arranged in such a way that the filter conditions set out in Subsection 3.4.7 are always complied with in the contact between different materials.

Vertical joints between caissons are particularly dangerous and must be fitted with special watertight elements (or devices for containing the migrated particles).

It is not possible to guarantee that inter-block joints in quays of this type are impermeable. Specific anti-entrainment measures must therefore be arranged at the contact zone between the quay and the backfill.

4.2.3.7 Verifying Safety against Scour at the Wall Toe

The phenomenon of scour at the front toe of gravity quays is probably the main cause of their long-term deterioration.

Scour at the toe can be the result of several causes, including:

- Rip currents.
- Artificial dredging in the vicinity.
- The effect of propellers, especially transverse ones, in berthing zones.
- The effect of waves.

Erosive processes are particularly important at the ends of quays. Entrainment velocities can increase at these edges.

Scour may even affect the forward portion of the support plane of the foundations, thus reducing their effective width.

Scour reduces the vertical pressure in the passive zone that makes the structure safe against bearing failure (reducing the design value of surcharge, *q*, in the formula recommended for evaluating bearing capacity in Subsection 3.5.4.8).

As a result of all this, scour can induce overall failure in the works.

Preventive measures must be taken to achieve sufficient safety with respect to this problem. For example:

- Arranging layers of rockfill or other non erodible materials in potentially erosive zones.
- Prohibiting and controlling any dredging that could affect the front toe of the quay.
- Regular bathymetrical surveys in front of the quay shelf.

When the problem is important and unless sufficient justification to the contrary is given, erosion armours made of rockfill will have a grading of adequate size and weight for the case concerned and should be placed with a minimum thickness in the order of $2D_{50}$ –no less than 1.50 m– and in two layers, covering a width no less than the depth of the quay.

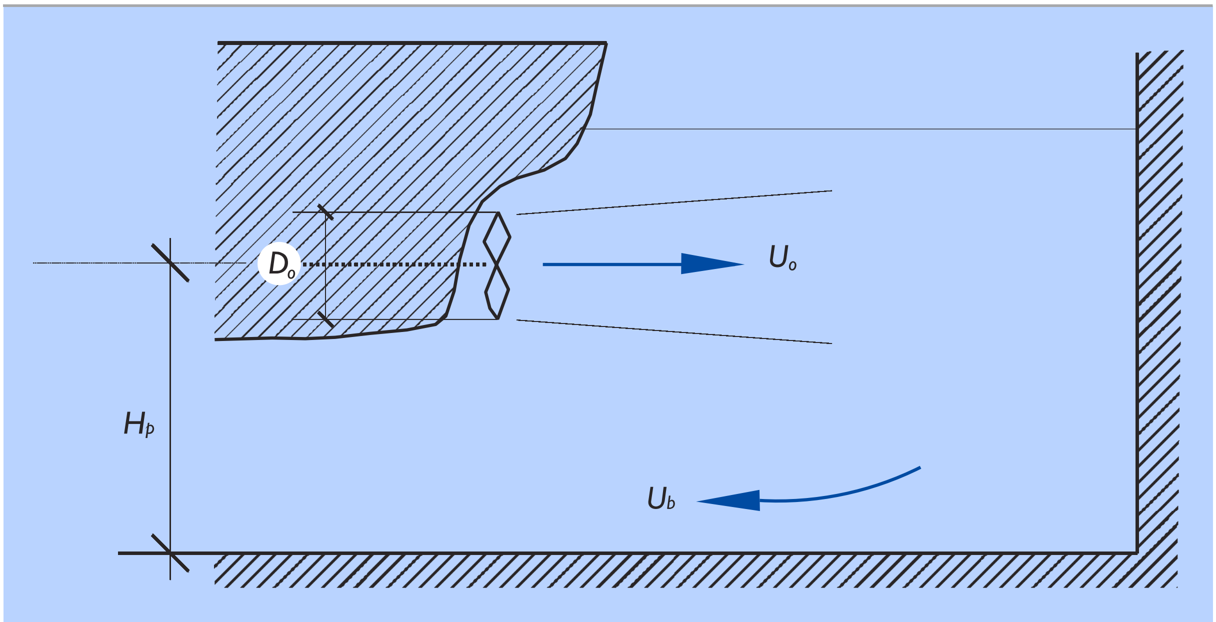
The erosive action of ship propellers, especially transverse ones during the course of berthing manoeuvres, can be extremely important. Water movement close to propellers can reach velocities of some 10 to 15 m/s. In the vicinity of structures these values may still be high. The joints of gravity quay walls (either between blocks or caissons) can suffer damage. Special attention must be paid to this aspect.

In harbour bottoms, water velocities tend to be only a fraction of the maximum indicated but even so, erosion will be produced when the soils have low resistance to it. Subsection 3.4.9 makes some comments on the erosion resistance of certain types of ground.

It is possible to calculate water velocities in relevant places caused by different agents (effect of propellers, waves, currents, etc.) but it lies beyond the scope of this ROM 0.5. It is however to be found in ROM 3.1 and in specialised publications. Knowing the water velocity on the harbour bottom is however necessary in order to be able to design a solution that prevents the scour process from developing.

A large number of formulae exist for dimensioning the armour systems needed to prevent erosion. Engineers should look for the most adequate analytical procedures for each case in specific publications of the ROM Programme. In this respect, Section 4.7 of this ROM 0.5 also provides recommendations that will be useful in solving this problem when the agents causing erosion are waves or currents.

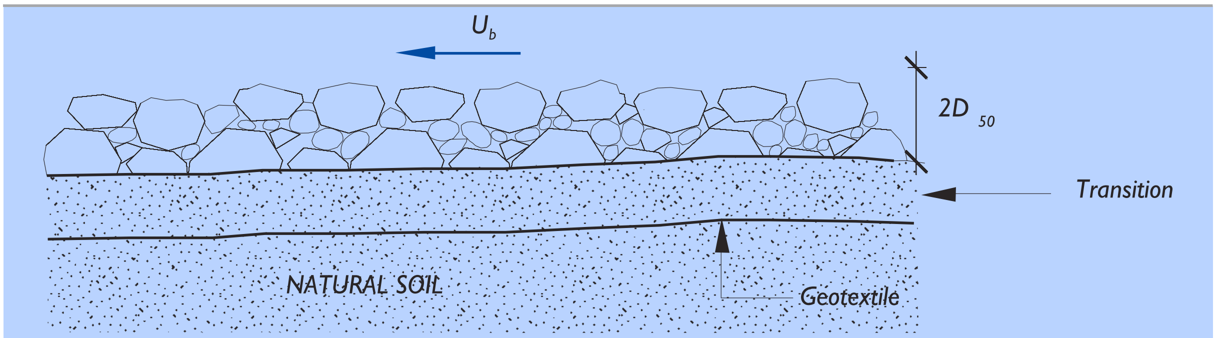
Figure 4.2.6. **Diagram of the Erosive Action of Propellers**



In addition to the armour shown in Figure 4.2.7, mention should be made of other possible scour protection systems that are used with some frequency, such as the following:

- Large rockfill blocks.
- Concrete slabs (usually articulated, dowel-jointed).
- Mats of synthetic material filled with concrete.
- Gabions.

Figure 4.2.7. **Diagram of Typical Erosion Protection**



Size Recommended as a Function of Velocity

U_b (m/s)	D_{50} (m)
1	0.05
2	0.20
3	0.40
4	0.70
5	1.10
6	1.60

On occasions, active solutions are used, which modify water flow (deflectors).

On coarse granular beds or medium to large-sized granular fills, engineers could contemplate not providing any anti-erosion measures. In this case, they are advised to estimate the potential erosion depth and to this end should rely on local experience or bibliographical sources.

Comment: In the event of not providing any armour, the maximum scour depth when the bed is composed of coarse granular soils can be estimated using the following approximate expression:

$$Z_{max} = \frac{1}{250} \cdot \left(\frac{F_o}{H_p / D_o} \right)^{2,9} \cdot H_p$$

where:

Z_{max}	=	*maximum erosion depth.*
F_o	=	*Froude number, given by:*

$$F_o = \frac{U_o}{\sqrt{g \cdot D_s (G-1)}}$$

U_o	=	*water velocity at the propellers.*
G	=	*specific gravity of ground particles.*
g	=	*gravity acceleration.*
D_S	=	*representative diameter of soil grains - usually D_{50}.*
H_p, D_o	=	*dimensions indicated in Figure 4.2.6.*

The formula is not very accurate when used outside the range of $0.1\ m < D_S < 0.3\ m$

See G.P.Tsinker "Marine Structures Engineering", Chapman & Hall, 1995.

Engineers should explicitly consider the problem of scour at the toe of gravity quays and justify that the measures adopted provide reasonable safety against this problem.

4.2.3.8 Summary of Minimum Safety Factors

The analytical procedures and safety factors engineers should adopt in designing gravity quay walls to ensure that each of the Ultimate Limit States is not exceeded are defined in Part 3 of this ROM 0.5.

When describing each of the Ultimate Limit States in earlier sections, reference was made to the sections in this ROM 0.5 where the calculation procedure and the safety factor to be adopted are defined. Both aspects (calculation method and safety factor) are interconnected and should not be dissociated. Table 4.2.1 gives a summary of the minimum safety factors recommended for each failure mode for works with a low SERI rating. Engineers must be aware of the associated analysis method before using them.

4.2.4 Limit States of Serviceability

Gravity quay walls designed under the safety criteria given in this ROM 0.5 may experience substantial deformations that engineers should estimate and then check that they are compatible with the service requirements of the quay.

Subsections 3.5.7 and 3.7.11.2 contain recommendations on the procedure engineers should follow to estimate displacements in gravity retaining structures. These recommendations are directly applicable to the types of quay being studied here.

Where small displacements are required for quay operation, special design measures and more detailed deformation analysis procedures may be needed.

Table 4.2.1. Minimum Recommended Safety Factors for Designing Gravity Quays, Works with low SERI (5 - 19)

Section on Defining the Associated Calculation Method	Geotechnical* Ultimate Limit States (GEO)	Type of Load Combination		
		Quasi-Permanent F_1	Fundamental or Chracteristics F_2	Accidental or Seismic F_3
3.5.5	Sliding along the contact between concrete and berm	1.5	1.3	1.1
3.5.5	Sliding along the contact between berm and natural ground	1.5	1.3	1.1
3.5.4	Bearing failure	2.5	2	1.8
3.5.6 y 3.7.11.3	Plastic overturning	1.5	1.3	1.1
3.8	Overall instability	1.4	1.3	1.1
–	Internal backfill erosion	PM	–	–
–	Scour at the front toe	PM	–	–

* These are the modes mainly governed by ground resistance.
PM In these cases, safety is not normally quantified. The problem can be avoided by taking adequate preventive measures (PM).
Note 1: Before using these safety factors, it is necessary to master the associated calculation methods defined in this ROM 0.5, as described in Section 4.2 and in the sections/subsections appearing in the table first column.
Note 2: Depending on the nature of the works and the duration of the design situation, the adjustments recommended in Subsections 3.3.8 and 3.3.10 should be carried out for the purpose of adapting the recommended safety factors.
Note 3: The safety factors against bearing failure shown correspond to the use of the polynomial formula (Subsection 3.5.4.8) or to the use of slice methods. For other methods, the minimum safety factors indicated in Table 3.5.6 apply.

4.2.5 Other Recommendations

Apart from recommendations for the design of gravity quays that may be given elsewhere in the ROM Programme, some general recommendations on the geotechnical aspects of these works are anticipated in this ROM 0.5.

Levelling Berms

The quality of the rock to be used for the rockfill in levelling berms affects several aspects such as the durability and resistance to crushing of the contacts between different fragments. With fresh and resistant rock (q_u >100 MN/m^2), higher pressures can be used in the structure-berm contact (σ_{peak} of up to 1 or 2 MN/m^2) than for rock with low or moderate strength (q_u < 50 MN/m^2), where it is advisable to limit such contact pressures (σ_{peak} < 0.5 MN/m^2) so as to reduce the crushing of fragment contacts. In any event, the use of rockfill with a compressive strength clearly below 50 MN/m^2 is not advisable.

Levelling berms are generally constructed on soil that could have been previously dredged. This contact zone will be irregular and may require a greater amount of berm material than strictly required in theory. In this respect, it is recommended that the theoretical design thickness of the berm be at least 1 m in order to cover any unevenness.

There will generally be a flow of water around this contact zone and this could lead to an scour process undermining the berm. For this reason, it is recommended that the berm base be a closed-graded layer, so it may act as a filter to the natural seabed.

Great care must be taken to ensure the evenness of the top face of the levelling berm, so that the support for the wall blocks or caissons is as plane and uniform as possible. Otherwise differential settlement and higher parasitic stresses will be produced on the structure of the quay.

In the top of the berm, in the strip on which the quay wall rests, the maximum size of the rocks should be restricted (to a weight of around 50 kg) so that a plane surface can subsequently be prepared with gravel for the final levelling.

Cleaning operations of the supporting base should be carried out very carefully. It is possible that sedimentation occurs between completing construction of the berm and installing the wall blocks or caissons, which would leave a layer of mud on the top face of the berm. In this respect, as little time as possible should elapse between levelling the berm and installing the structure. The foundations should be inspected –and cleaned, if necessary– immediately prior to installing the structure on the berm.

Comments on Movements

In the design of gravity quays on soils, it is important to take into account the displacements that could occur as a result of deformation of the foundation ground.

This can lead to the construction of quay walls on foundations at higher than theoretical elevations to compensate for possible settlement and even with a certain theoretical inclination to offset future overhang resulting from rotation. Otherwise, the subsequent corrections, which can always be made when building the superstructure, can prove to be too large.

Quay wall displacements should be monitored during construction so that when the superstructure is constructed, a reliable prediction of the residual movements is available, enabling action to be taken accordingly.

Comments on Joints

Joints between caissons –or wall blocks– will constitute preferential water paths in tidal oscillations and during wave action.

The gap between caissons must be sufficiently large to absorb any construction inaccuracies, but they should subsequently be protected by some method of proven effectiveness to avoid possible material entrainment caused by the water flow.

Joints between blocks are difficult to control and therefore, in zones that could be affected by wave action, substantial uplift will occur which can destabilise the wall blocks (the water pressure inside the joints takes time to dissipate and may be greater than that of the water on the front face when the wave runs down). This is why blockwork quay walls in areas affected by waves are problem-prone.

The superstructure must be designed in such a way that allowance is made for the residual movements due to construction of the quay as well as the displacements due to future loads and overburdens. In this respect, appropriate joints must be provided.

Construction Sequence

As a general rule, the worst conditions with respect to Ultimate Limit States will occur when the works are completed and under certain surcharge positions. It is possible, however, that an adverse situation occurs temporarily during certain Construction Stages.

This situation can arise, for example, when backfilling starts before the wall structure is completed.

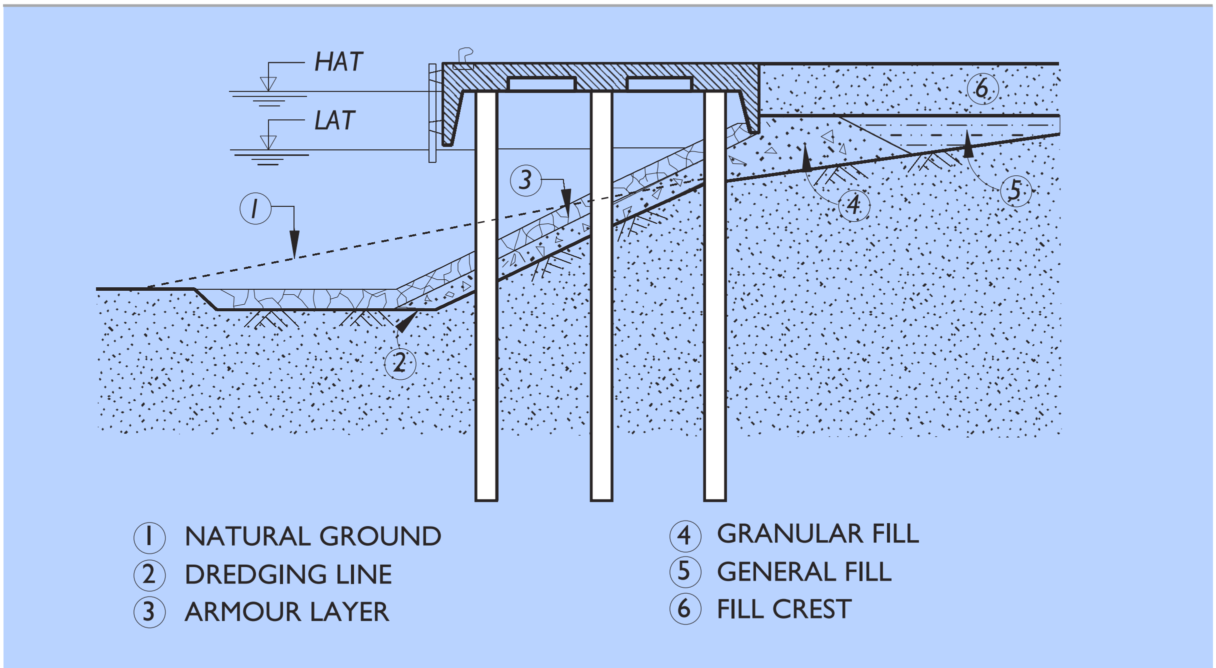
For this reason, the safety checks detailed in previous sections must be extended to all stages in the quay construction sequence.

4.3 PILED QUAYS AND JETTIES

The construction of quays on deep foundations is unavoidable in ground in which the firm substratum is too deep for constructing gravity walls. They may also be of interest in ground with medium compacity as an alternative to other possible types.

The typical diagram of a piled quay is represented in Figure 4.3.1 showing the most characteristic geotechnical elements.

Figure 4.3.1. ***Piled Quay***



Although the platform of the quay is represented in the diagram as supported solely by vertical piles, other arrangements are possible including some battered piles, which help to withstand the horizontal loads with their axial resistance.

Piled jetties generally have a simpler structure. The backfill and front slope complicating quays with an adjacent embankment do not exist here. The recommendations given here for quays should, however, be followed where applicable to the design of jetties.

4.3.1 Natural Ground. Site Investigation

The ground around the works must be studied with a view to evaluating safety against the limit states mentioned in this Section 4.3 and in order to study possible execution problems in advance, particularly in pile driving, when this type of solution is considered.

For these purposes, it will be necessary to ascertain the stratigraphy of the ground down to a level clearly below the tips of the deepest piles that could subsequently be designed.

It will be necessary to know the nature and grading curve of each different level or type of ground involved (identification tests) and their natural density, moisture content and shear strength. As a general rule, the deformability of the ground should also be known, either by running specific field tests (pressuremeters, for example) or by laboratory tests.

Among the most interesting geotechnical investigation methods for studying the bearing capacity of piles, mention must be made of continuous static penetration tests in soft soils and SPTs in granular soils.

When investigating deep rock formations where pile tips may be embedded or rest on, it will be necessary to know their nature and state of density and moisture content, as well as their degree of jointing and weathering and the unconfined compressive strength of their matrix.

It is also important to study the soil-rock transition zone carefully. The pile tips could stop there if specified in the design.

When planning geotechnical investigations, the area surrounding the works must also be taken into account, since it will generally be necessary to analyse problems associated with overall stability of the quay as well as aspects relating to local sliding of the quay slope between the piles supporting the platform.

The recommendations given in Part 2 of this ROM 0.5 need to be taken into account in order to plan and carry out the ground investigation. It should also be borne in mind that the information provided by this investigation will have to be used subsequently following the recommendations given in Section 3.6, "Deep Foundations", which should be known and taken into account.

4.3.2 Choice of Pile Type

A wide variety of pile types can be used in the construction of quays and jetties. Two major groups should be distinguished from the geotechnical point of view.

a. Cast in Situ Concrete Piles

These require previous drilling of the ground and subsequent concreting. A wide variety of types exist depending on the construction procedure and no attempt will be made here to describe them.

b. Precast Driven Piles

The structural element of the pile is driven into the ground by percussion (occasionally by vibration) applied at its head.

There are also a large number of types of driven piles, depending on the type of material used (reinforced or prestressed concrete, steel, timber, etc.) and on the driving technique employed.

Some piles are also used that be classified as intermediate between the two basic types, such as the ones described in Subsection 3.6.1.2 of this ROM 0.5.

There are no general geotechnical criteria indicating the suitability of a specific type of pile for constructing quays or jetties and engineers must therefore consider several types of pile in each case and choose the most adequate after a comparative study.

This comparative study needs to take into account the following factors:

- **Installation Possibilities**

  Virtually any type of pile can be constructed in any type of ground. The difficulties found when passing through certain hard levels, or when embedding the piles in rock, are usually best solved using cast in situ concrete piles, even though it is possible to use driven piles (assisted by water jets, for example).

  The use of cast in situ piles in quay construction may necessitate a special casing on the top zone (the freestanding part plus the initial metres of natural ground) or the construction of temporary fills. The two alternatives must be studied and compared.

It is difficult to drive large precast concrete piles at a steep angle. The horizontal loads, in such cases, must be borne by bending, taking advantage of the structural capacity of the pile.

Piles with a large individual bearing capacity are generally more suitable for quays with a large draught.

- **Making the Most of Structural Capacity**

  The structural capacity of the piles must be consistent with the resistance of the ground. Using piles with a large structural ultimate load in ground providing a low bearing capacity would mean structural over-dimensioning. This may prove necessary when large horizontal loads are withstood by vertical piles, which need high bending resistance.

  As a general rule, when the majority of the ground resistance is mobilised by skin friction, smaller piles are more appropriate. With column piles transmitting loads mainly through their tips, large piles may lead to a better utilisation of their structural capacity.

- **Long Term Behaviour**

  The aspect of durability under service conditions should be taken into account when choosing the pile materials. Poor quality concrete, untreated timber and unprotected steel elements may lead to problems of loss of capacity, particularly in areas under tide run.

The choice of pile type, and specially its size, has considerable repercussions on the design of the quay superstructure. A comparative study of different pile alternatives must include consideration of the type of associated superstructure.

A comparison during the design stage of construction costs and timeframes for different alternatives with the same degree of safety should result in selecting the most adequate type of pile.

4.3.3 Source and Quality of Materials

Fill materials to be used in piled quays can be very different in nature and purpose. There will generally be:

- A material to protect the forward toe of the quay against scour
- An armour layer to cover the slope.
- A general fill.

Also as a general rule, the first two protections will be of rockfill, although they may consist of blocks, slabs or other elements, generally made of concrete.

In the case of rockfills, to know the nature of the rock, its density, weatherability and unconfined compressive strength is of interest. From these properties rock durability can be evaluated (there are also specific durability tests to carry out in cases of doubt). The grading curve of the material is a basic information when evaluating its erosion resistance.

Under the slope's armour layer, it may be necessary to place transition layers that fulfil the filter condition with respect of the natural ground or general fill. The grading characteristics of these materials should be known as well as the durability of the minerals making up their grains.

Regarding the other fill materials that may be used for a particular project, their identification characteristics (grading and Atterberg limits) will generally be of interest as well as their possible state variables (dry density and moisture content). On occasions, their strength and/or deformability may be important and in such cases additional tests will be necessary.

With respect to the earthfills to be placed above the water level by spreading and compaction, it will be necessary to know the data specified in the preceding paragraph and also the compaction characteristics from laboratory tests (Proctor and CBR, for example) as well as results from other specific tests in order to qualify the platform that can be obtained with these materials, as indicated in ROM 4.1, "Harbour Pavements".

The natural beds where piled quays are constructed tend to consist of poor strength soils. The weight of the fills will cause settlement that will influence the use of the platform. A study of the consolidation of natural beds (oedometer tests, geotechnical investigation using piezocones, etc.) will prove of great importance in such cases.

4.3.4 Actions on Piles

For the purpose of checking the limit states indicated later on, it is first necessary to calculate the loads on the pile heads or points where the pile and superstructure are connected.

Loads on quays may be distinctly impulsive in nature (due to berthing and mooring). It may be necessary to consider the effect of waves on quays and especially on jetties.

The design values for the loads must be determined after analysing the interaction of the loading agent with the quay structure and ground taken as a whole. This may require a dynamic interaction analysis, the bases of which are given in Section 3.10.

To calculate these loads, each pile can be represented in the corresponding calculation model by a beam with similar characteristics to those of the real pile in the freestanding portion. The buried portion can be represented by a rigid beam supported at the lower end by a set of springs whose constants depend on the characteristics of the ground and the pile itself. Figure 3.6.15 in Part 3 of this ROM 0.5 gives some approximate formulae for estimating the length of this rigid beam and for calculating the constants of the springs supporting it.

Wave actions on piles can be evaluated using Morison's formulae, use of which is widespread. They have not been included here, as this is mainly a hydraulic problem with a well-known solution.

The cyclic and alternating or impulsive nature of the loads needs to be accounted for in the calculation. Subsection 4.3.5.3 gives some recommendations in this respect.

The load combinations to be taken into account when studying the limit states considered here are those indicated in Subsection 3.3.5.4 of this ROM 0.5.

The earth pressure on the back of the quay structure should be consistent with the expected displacement of the quay. In analysing the Ultimate Limit States considered here, it is generally admitted that the pressure can be calculated in the active condition when the quay structure, under the corresponding load case, moves away from the earth producing the pressure.

When the quay movement is in the opposite direction, the earth pressure to be considered should fall between the earth pressure at rest and the passive earth pressure, depending on the relative displacement between the structure and the earthfill. The recommendations given in Section 3.7 should be followed for the purpose of evaluating this situation.

In the case of structural arrangements with battered piles, it may be that some piles experience tensile forces in some design situations. In such cases, it may be desirable to install heavier structural platforms capable of restricting these tensile forces.

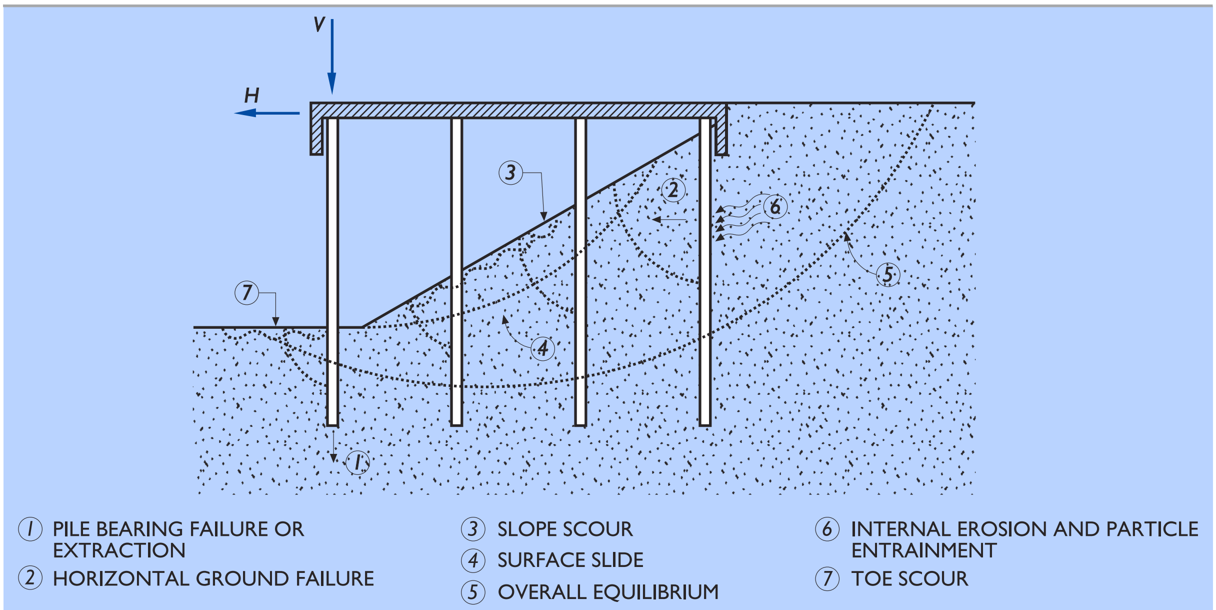
For the purpose of analysing other limit states governed by the pile structural capacity, other calculations may be necessary, for which recommendations are given in Section 3.6 of this ROM 0.5 publication.

4.3.5 Ultimate Limit States

The Ultimate Limit States that should be considered when designing piled quays and jetties governed by ground resistance are included in the following sections. Figure 4.3.2 shows a diagram of the corresponding failure modes.

In addition to these failure modes of a geotechnical nature, other modes principally governed by the structural strength of the piles or the quay superstructure should be analysed, which are not the subject of this ROM 0.5.

Figure 4.3.2. ***Geotechnical Failure Modes in Piled Quays***



4.3.5.1 Verifying Safety against Bearing Failure or Extraction of Piles

The load acting individually on the head of each pile (or the section where the superstructure is connected) should be obtained for each combination of actions specified, following the recommendations given in Subsection 4.3.4 above.

The axial component of the load acting on each individual pile will enable the safety factor against bearing failure (or against extraction, if the load is tensile) to be calculated in accordance with the procedure given in Section 3.6 of this ROM 0.5 and to check that the safety factors obtained exceed the minimum indicated in Table 3.6, as appropriate.

The potential effect of negative skin friction should be taken into account following the recommendations given in Subsection 3.6.3.4.1.

The group effect should be considered as indicated in Subsections 3.6.6 and 3.6.7.

4.3.5.2 Verifying Safety against Horizontal Ground Failure

The horizontal loads on quays and jetties can be very high, due to berthing thrust, mooring pull and, sometimes, the effect of wind on the superstructure, of waves and of earth pressure.

Wind and wave loads on a quay should be determined as specified in the corresponding ROM publications and following the general criteria given in Subsection 3.3.5.

For the purpose of checking this Ultimate Limit State, the earth pressures on the rear heel of the quay should be calculated assuming the active earth pressure condition and treating them as an external load.

In the special cases where the horizontal ground failure mechanism being checked involves a movement of the quay structure towards the earthfill, the coefficient of passive earth pressure will be used to evaluate the earth pressure on the back of the quay. The passive earth pressure should be treated as a resistance to be added to that of the piles and not as an external load. In order to make these checks, it should be assumed that the pile is buried from its intersection with the surface of the embankment slope and that the only loads acting on the pile are the ones transmitted by the structure and the waves that can affect it directly. The earth pressures that usually act on piles in a slope, calculated as indicated in Subsection 3.6.3.4.3, should not be taken into account in this case of the quay structure pushing against the earthfill.

When the opposite situation occurs, with a pull on the quay, which moves seawards, or when quay movement is not significant, the piles should be considered to be buried at a certain depth *h* under the point where the quay slope intersects the pile axis. On this pile length, *h*, a permanent load corresponding to the earth pressure of the quay slope should be assumed. Calculation of *h* and the corresponding lateral load should follow the indications in Subsection 3.6 3.4.3.

The safety factor against horizontal ground failure is defined as the quotient between the sum of the resistance of each pile (and the passive earth pressure of the backfill, where applicable) and the total horizontal load, that is:

$$F_{(\text{horizontal failure})} = \frac{\sum_{1}^{n} H_{(\text{failure})}}{H}$$

where:

$H_{(\text{failure})}$ = individual resistance to horizontal ground failure of each of the *n* piles in each cross-section of the quay. The passive earth pressure should be considered as an additional resistance when the failure under examination occurs landwards.

H = total force acting on the length of the quay separating consecutive cross-sections with piles. If the failure under study is seawards, the active earth pressure on the back wall of the quay should be considered as one more load.

The individual horizontal ultimate load in each pile should be evaluated following the recommendations given in Subsection 3.6.8, which also gives recommendations for considering the group effect and the minimum safety factors that should be obtained.

4.3.5.3 *Verifying Safety against Slope Erosion*

Seawater movement in the vicinity of piled quays and jetties can be significant and engineers will need to determine a reasonable value for the parameters and factors intervening in this problem.

This movement is caused by waves, potential currents and vessels themselves, especially by those with powerful lateral propellers.

Water flow in the vicinity of a quay can bring about slope surface erosion. This is why the slopes of piled quays must have armour layers made of rockfill or other similar elements.

The stability of the armour layer for slopes can be analysed using empirical/experimental formulae of the type initially put forward by Iribarren. To this end, the Hudson formula could be used:

$$W = \frac{\gamma_s H_d^3}{K_D \left(\frac{\gamma_s}{\gamma_w} - 1 \right)^3 \mathrm{cotan}\alpha}$$

where:

W = nominal weight of the armour blocks.
H_d = design wave height.
γ_s = specific weight of the armour units.
γ_w = specific weight of the water.
α = angle of inclination of the slope from the horizontal.
K_D = stability coefficient of the armour units.

Engineers should know the design wave height that could affect the works. In the absence of other more specific data, for a common harbour case (sheltered quays), a design wave height at least equal to 1 m ($H_d \geq 1$ m) should be taken, which could represent the wave caused by the passage of a vessel. When the berthing of vessels with transverse propellers is predictable, this design height should at least be equal to 2 m ($H_d \geq 2$ m).

The stability coefficient of the rough and angular rockfill blocks normally used should be taken as:

K_D = 4 for the front quay zone.
K_D = 2.3 for the end zones where surface erosion may be more pronounced.

If artificial armour elements are used, engineers should use adequate stability coefficients for them.

The thickness of the armour layer should correspond to two layers of the material chosen and not be less than 1 to 1.5 metres.

Engineers may opt for other design procedures for the armour layer, providing they give due justification for the choice.

4.3.5.4 Verifying Safety against Slope Surface Slide

Sliding in a quay slope along failure lines that do not penetrate deeper than approximately half of the spacing between pile rows (i.e., two consecutive cross-sections with piles) should be considered as a surface slide. Stability for such failure lines will not be affected by the presence of the piles. The slope could slide between two pile rows without their making any contribution to the sliding resistance.

The sliding stability of the slope along these slip lines should be studied following the recommendations given in Section 3.8. Supplementary recommendations are given in Subsection 4.7.3.2 that could be of interest to engineers.

Deeper slides will involve the piles in the slope. The problem will then be different and must be classed as an Ultimate Limit State of overall stability of the works. Combined failure of the slope and of the piles is considered next.

4.3.5.5 Verifying Safety against Overall Instability

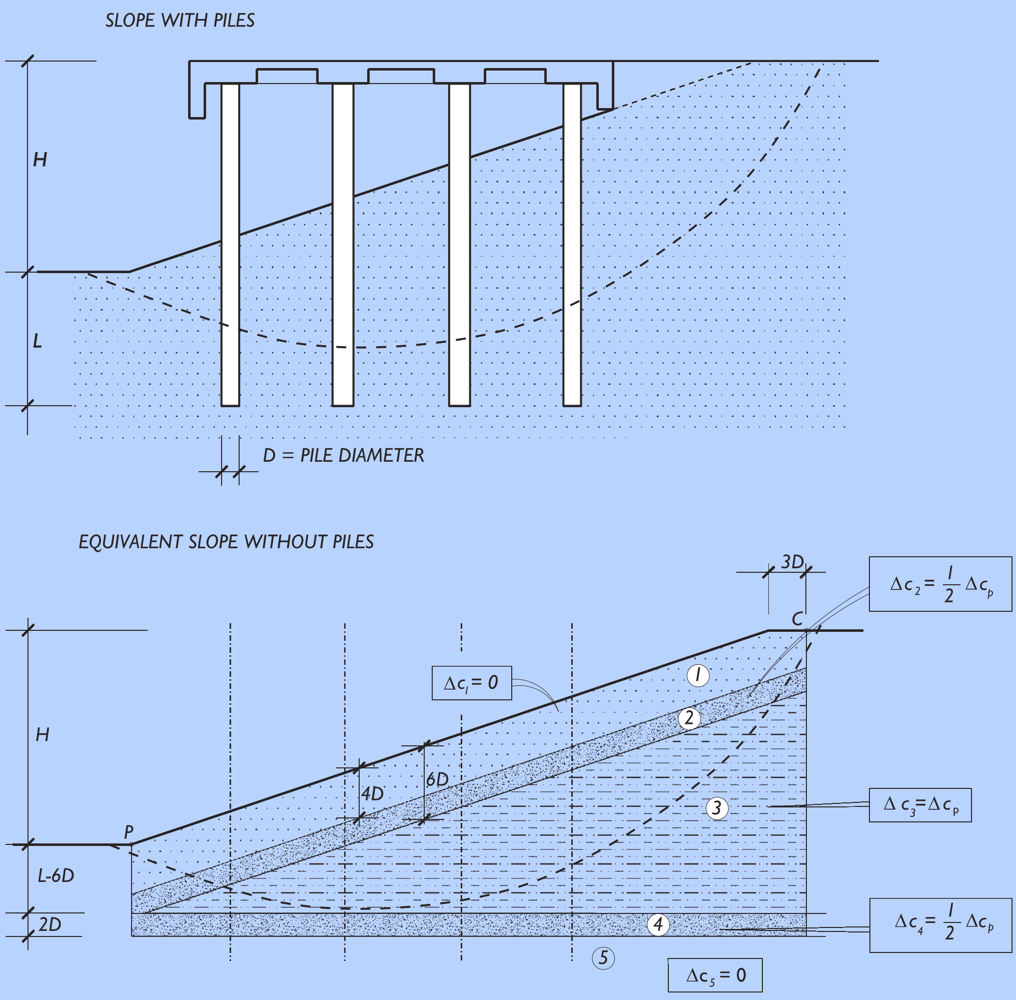
Ground failure affecting the whole or part of a quay is a limit state mainly governed by ground resistance.

Surface sliding in the slope along shallow failure lines -compared to the distance separating the pile rows, i.e., when the sliding mass has a maximum thickness of less than half of such spacing, approximately- should be considered just as surface slides and not as an overall stability problem, since its occurrence would not necessarily involve the simultaneous failure of the quay piles. An analysis of the Ultimate Limit State caused by surface sliding was dealt with in the previous section.

Safety against the Ultimate Limit State of overall instability should be verified following the recommendations given in Section 3.8 of this ROM 0.5 and the levels of safety achieved should exceed the minima set out there.

The presence of piles in the slope means that calculation of the corresponding safety factor will be complicated, if the general recommendations of Subsection 3.8.4 are followed. For this reason, the simplified calculation method illustrated in Figure 4.3.3 is considered acceptable.

Figure 4.3.3. Simplified Method for Calculating Stability of Piled Quays



Δc_p = the least of the following

$$\Delta c_a = n \frac{M_u}{H^2 S}$$

$$\Delta c_b = \alpha \frac{D}{S}\left(c + \gamma' D \frac{1 + \sin\phi}{1 - \sin\phi}\right); \quad \alpha = n \frac{18D}{H} \leq 6$$

where:

- n = number of piles per row (4 in the drawing)
- S = distance separating pile rows. If S>H then S=H should be taken
- M_{rot} = ultimate bending moment of the pile section for the contaminant axial load
- γ' = submerged specific weight of the ground

In accordance with this procedure, the ground and the piles can be represented by an equivalent ground with no piles, but with a virtual increase in cohesion.

The virtual increase in cohesion to be assumed in order to represent the presence of the piles varies for each of the five zones shown in the figure referred to.

Zona 1. Delimited by the vertical passing through the toe *P* and by the vertical passing through the head *C* and with a vertical thickness equal to 4*D*.
Zona 2. Confined by the same two verticals, lying below Zone 1 one and with a vertical thickness of 2*D*.
Zona 3. Lying between Zones 2 and 4.
Zona 4. Horizontal strip with thickness 2*D* and its top face located at a depth of *L*-6*D* below the toe *P*..
Zona 5. The rest of the half-space.

It is in Zone 3 where an increase in cohesion equal to the value Δc_p shown in Figure 4.3.3 should be assumed. In the adjacent Zones 2 and 4, the value to be taken for the increase in cohesion is half the previous amount. In Zones 1 and 5, no increase in cohesion should be assumed.

The increase in cohesion, Δc_p, corresponding to Zone 3 is calculated in this simplified procedure in a twofold way. On the one hand, the maximum load that the structure of each pile can withstand is considered, characterised by the ultimate bending moment M_u concomitant with the axial load corresponding to the design situation analysed. On the other hand, what is considered is the maximum load that the sliding ground mass is capable of transmitting to each pile before flowing around it. The lower value of the two increases in cohesion should be used in calculations.

When calculating the virtual increase in cohesion produced by the second effect (Δc_b in the figure), the strength of the slope must be used as input value. This will generally be formed of materials with different strengths and it will therefore be necessary to take an adequate average. It will always be conservative to assume that the ground is homogeneous and to use the strength of the weakest ground.

When the analysis of overall equilibrium is a critical issue, more precise calculations will have to be made.

It is worth warning about the danger of deep slides that may affect thin, weak strata. This type of failure, where the risk cannot be assessed with circular failure surfaces, should be analysed following the general method set out in Subsection 3.8.4.6 and assuming potential failure lines with their bottom substantially running in the weak stratum.

4.3.5.6 Verifying Safety against Internal Erosion and Entrainment

Water movement can cause entrainment of soil particles and lead to internal erosion.

Internal erosion must be avoided by analysing the water flow and choosing appropriate grading curves for the different materials, so that they fulfil filter conditions in each contact zone. It should also be guarantied that the external materials, in direct contact with free water, are self-stabilising with respect to seepage. More detailed recommendations are given in Subsection 3.4.7 of this ROM 0.5 on this subject.

4.3.5.7 Verifying Safety against Scour

Scour at the toe of the slope of a piled quay can cause an Ultimate Limit State of progressive deterioration that could lead to total failure of the works.

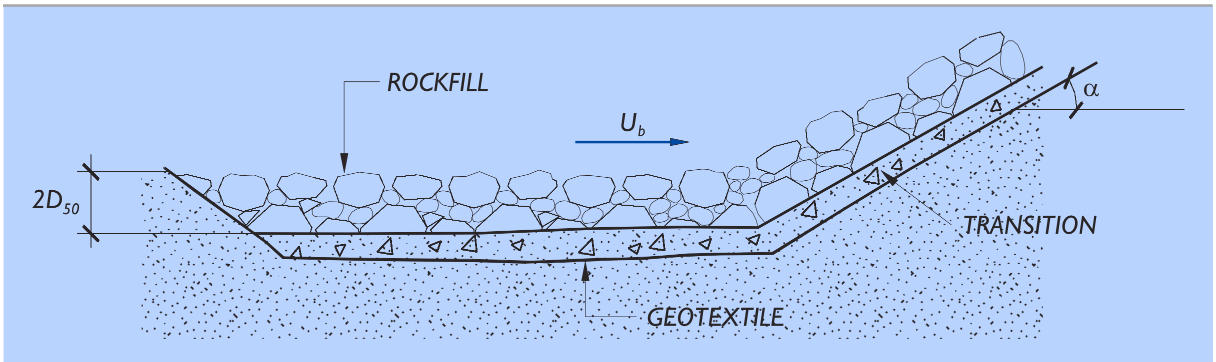
Scour can be originated by the propellers of vessels, by currents, waves and by other less predictable artificial causes.

Engineers must take this failure mode into account and adopt adequate preventive measures.

A possible form of protection is shown in Figure 4.3.4. Its dimensioning is based on the estimated water velocity, U_b, which should have been previously determined.

Subsection 4.2.3.7 gives some recommendations relating to scour around the toe of gravity quay walls and which also apply in this case.

Figure 4.3.4. ***Method for Calculating the Protection for a Slope Toe Using non-Uniformly Graded Rockfills***



SIZE OF ROCKFILL

$$D_{50} \geq h \cdot \left[\frac{U_b}{B\sqrt{K \cdot \Psi \cdot g \cdot (G-1) \cdot h}} \right]^{2.5}$$

D_{50} = average size of armour rockfill.
B = dimensionless number depending on flow type:
- B = 6 (turbulent flow).
- B = 7 to 8 (normal flow).
- B = 8 to 10 (low-turbulence flow).

K = reduction coefficient due to the effect of slope inclination $K = 1 - \sin^2 \alpha / \sin^2 \phi$ where α = slope inclination angle and ϕ = internal friction angle of the armour's supporting ground
ψ = dimensionless parameter depending on the desired behaviour:
- ψ = 0.03 (without movement of units).
- ψ = 0.04 (start of movement of units).
- ψ = 0.06 (failure).

g = gravity acceleration.
G = relative specific weight of the rockfill.
h = draught.

4.3.5.8 Summary of Minimum Safety Factors

The minimum safety factors engineers should adopt in the design of piled quays and jetties to ensure that each of the Ultimate Limit States is not exceeded are defined in Part 3 of this ROM 0.5.

When describing each of the failure modes in earlier sections, reference was made to the section of this ROM 0.5 where the calculation procedure and the safety factor to be adopted are defined. Both aspects (calculation method and safety factor) are interconnected and should not be dissociated. Nevertheless, Table 4.3.1 gives a summary of the minimum recommended safety factors for each failure mode. Engineers must be aware of the associated analysis methods before using them.

Table 4.3.1. Minimum Recommended Safety Factors for the Design of Piled Quays and Jetties, Low SERI Works (5 - 19)

Section where the Associated Calculation Method is Defined	Ultimate Limit States of Geotechnical* (GEO) Failure*	Load Combination		
		Quasi-Permanent F_1	Fundamental or Chracteristics F_2	Accidental or Seismic F_3
3.6.6	Pile bearing failure	1.4 - 2.6	1.3 a 2.3	1.3 - 2
3.6.7	Pile extraction	1.4 - 2.6	1.3 a 2.3	1.3 - 2
3.6.8	Ground failure due to horizontal loads	1.8	1.6	1.5
–	Slope erosion	PM	–	–
3.8	Surface slide of slope	1.4	1.3	1.1
3.8	Overall instability	1.4	1.3	1.1
–	Internal erosion and entrainment	PM	–	–
–	Scour	PM	–	–

* Those mainly governed by ground resistance.
PM In these cases, safety is not usually quantified. The problem can be avoided by taking adequate preventive measures (PM).
Note 1: Before using these safety factors, engineers should be familiar with the associated calculation methods defined in this ROM 0.5, as described in Section 4.3 and in the sections/subsections appearing in the first column.
Note 2: Depending on the nature of the works and the duration of the design situation, the recommended safety factors should be increased or decreased according to the modifications mentioned in Subsections 3.3.8 and 3.3.10.
Note 3: For bearing failure and extraction, the safety factor depends on the analysis method used (see Table 3.6.1).

4.3.6 Serviceability Limit States

Displacements of the quay superstructure, fill settlement or differential movements between the two may be incompatible with the correct operation of the quay.

The displacements of decks of quays and jetties can be estimated using the same structural model used to calculate forces and moments on piles, which must include due consideration of the effect of soil-structure interaction by equivalent springs or by other more complex models.

The earth pressures acting on the back of the quay structure to be used in calculating movements must be compatible with the relative earth-quay displacement.

The earth pressure on the buried part of piles to be considered in displacement calculations may be null, provided that the overall stability safety factor is greater than 1.7. Otherwise, additional earth pressures on the piles will have to be assumed as indicated in 3.6.3.4.3.

In cases where the safety factor against the Ultimate Limit State of overall instability is less than the one indicated, and whenever quay displacement is a critical design aspect, it is recommended to carry out more precise calculations and also that the design documents provide for some kind of monitoring procedure that will enable the results to be confirmed during construction.

Precise calculation of piled quay movements will generally require the construction process to take into account.

4.3.7 Other Recommendations

Specific recommendations for the design of this type of structure are given elsewhere in the ROM Programme. Some recommendations of a general type are now added regarding some geotechnical aspects of quays and jetties supported on piles.

Load Tests

Ground investigation and theoretical calculations to check Ultimate Limit States will enable this type of construction to be properly designed. Load tests, however, can provide a much more precise confirmation of some aspects of particular interest.

Vertical ultimate load tests on piles identical to those planned in the design (or similar, with some reduction in their size to facilitate testing) are of particular value. They enable a more accurate estimate to be made of the bearing capacity of the piles and this can even lead to a reduction in cost (particularly when installing a large number of piles). The minimum safety factors required when the estimation of bearing capacities is endorsed by load tests are lower than in the other cases, as indicated in Subsection 3.6.6 (Table 3.6.1).

Horizontal pull tests on individual piles, or between adjacent piles, are generally less difficult and are always advisable in order to obtain a better estimate of the horizontal reaction moduli of the ground as described in Subsection 3.6.9.2.

If the piles used in these pull tests do not need to be used afterwards as a permanent part of the structure, the horizontal load can be increased and the test extended to failure, thus improving subsequent evaluation of the corresponding Ultimate Limit State.

Monitoring Piles Individually

In quays supported on piles with a high individual bearing capacity, the importance of each member is so great that a structural defect in any part of one of them could have serious consequences. For this reason, it is recommended to specify at the design stage that each pile is to be individually monitored.

In the case of cast in situ concrete piles, apart from the usual quality control techniques, structural integrity tests should be carried out after installing them.

In driven piles and to perform individual monitoring, dynamic electronic instrumentation of pile driving may be adequate, using accelerometers and strain gauges, which will enable the shock wave and its reflections to be recorded.

Construction Sequence

Several different construction sequences can be used for a quay. The most suitable must be chosen at the design stage.

Executing piles before or after partial construction of the fill is one aspect of interest when evaluating potential undesirable parasitic loads on the piles.

The way in which the armour layers of a quay slope are executed has a considerable effect on these loads. Carefully placing the cover layer (with a crane and in horizontal lifts, for example) is always advisable in order to avoid damaging the piles.

Monitoring Displacements and Stresses

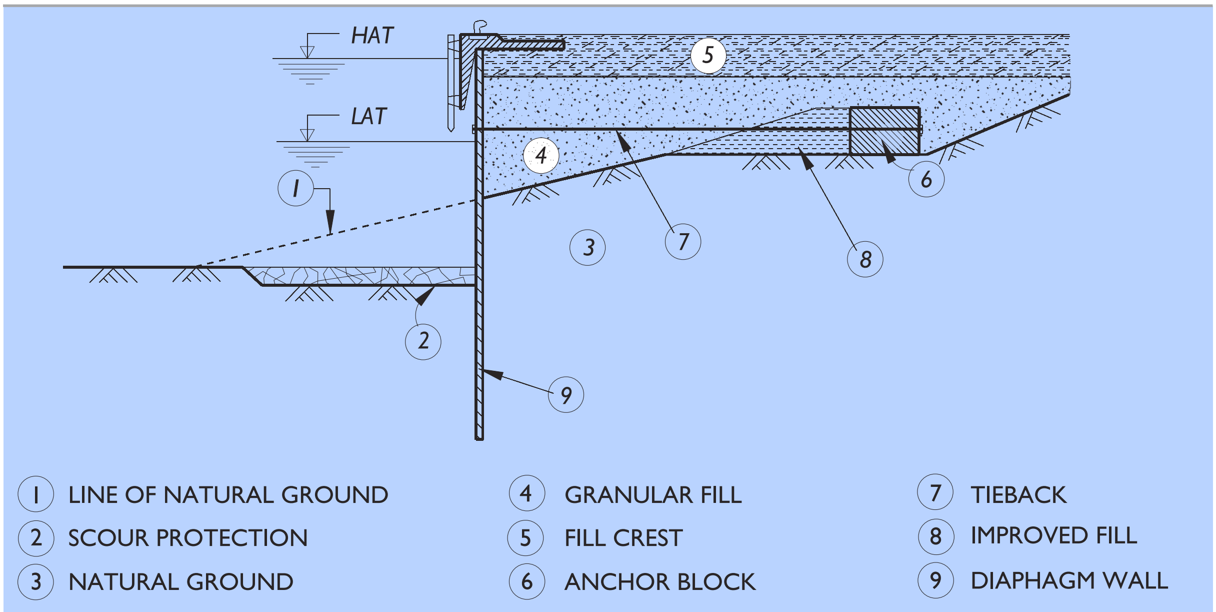
The loads that construction operations can induce in piles after they have been installed are difficult to specify. For this reason, it is recommended not only to monitor the deflection of piles and the quay superstructure, but also to instrument some piles in several sections at different elevations, with several extensometers in each section, so that the stresses can be known as they are generated.

4.4 QUAYS WITH DIAPHRAGM WALLS

4.4.1 Basic Types

The basic type of quay with a diaphragm wall is illustrated in Figure 4.4.1, which highlights the most interesting geotechnical elements.

Figure 4.4.1. ***Diaphragm Quay Wall***



Construction of this type of quay requires executing the diaphragm walls and their anchors as well as the dredging and filling tasks needed to create the appropriate geometry.

Diaphragm walls are often composed of steel sheetpiles, although this structural element can also be made of a cast-in-situ flat concrete wall or by other procedures. The two types of wall are similar with respect to the checks referred to below.

Concrete diaphragm walls can be constructed on virtually any ground. They have the advantage that they can be embedded in firm soil or rock (excavated using a rock hammer or water jet drill), which is more difficult to achieve with driven steel sheetpiles.

The anchorage system usually consists of steel cables or rods conveniently attached to the diaphragm wall and to a rear anchor structure that can consist of another shorter diaphragm wall or a concrete anchor block or *deadman*, as illustrated in the figure, or a vertical or horizontal buried plate. The anchor block or plate may simply be resting on the ground or founded on piles that provide an increased reaction capacity and mean the anchorage length can be shortened.

Other possible anchorage systems are steel rods, cables or micropiles housed in boreholes and attached to the ground by cement or mortar grout.

In short, a wide variety of similar structural arrangements are possible and can be considered to be equivalent for the purpose of the checks to be carried out as recommended in the following sections.

Closed sheetpile enclosures, or cellular cofferdams, and quays formed by two close parallel diaphragm walls, however, must be considered structures of a different kind and are therefore dealt with elsewhere (see Section 4.5).

Numerous factors must be considered for the design and construction of diaphragm wall quays. General recommendations are given in this ROM 0.5 in relation to the geotechnical checks that need to be made, trying to distinguish them from other types of checks that will be the subject of other publications in the ROM Programme in the near future.

4.4.2 Ground Data

The ground where quays with diaphragm walls are to be constructed is a conditioning aspect. The correct behaviour of these quays requires a certain bearing capacity, both vertical and horizontal.

Vertically, the ground will have to support the weight of the quay plus loads and overburdens. Loose ground can give rise to substantial settlement and stability problems.

Horizontally, diaphragm walls require a contribution from the ground in their embedment zone. Loose ground can lead to very long diaphragm walls and very high bending moments on their structure.

Rocky bottoms, on the other hand, can make sheetpile driving difficult and even prevent it altogether and therefore condition this type of solution. It would then be necessary to design diaphragm walls excavated in situ.

The nature of the ground alongside a quay must be known in detail before carrying out the design checks recommended here. In particular, it is specially important to know the horizontal bearing capacity of the ground and this value can be obtained by triaxial tests carried out on undisturbed samples. In-situ tests using static penetrometers or pressuremeters are particularly appropriate.

Of each ground level, it is advisable to determine not only its identification characteristics (grading curve and Atterberg limits) but also its natural state variables (density and moisture content) and its shear strength and deformability.

In the case of rocks on which diaphragm walls could rest (or be embedded in), it is advisable to know their nature, density and moisture content as well as an accurate description of their degree of weathering and jointing.

As the checks to be carried out involve the ground over a large area (overall stability, the effect of deep layers on settlement, etc.), the investigation must be extended vertically and horizontally so that adequate knowledge is obtained of the whole of the surrounding area. Detailed recommendations are given in Part 2 of this ROM 0.5 to these effects.

4.4.3 Source and Quality of Materials

In general and as in the case of gravity or piled quays, it will be necessary to protect the toe of the diaphragm wall against erosion or scour and this will require rockfill, whose origin and quality will have to be investigated at the design stage. The comments made in Subsection 4.2.2 are applicable in this respect.

The backfill of diaphragm walls should preferably be made of granular materials, which exert less pressure and are less susceptible to deferred deformation. The density and moisture content of these backfill materials should be known in the same conditions as expected on site, and also their shear strength, which can be investigated by laboratory tests.

Different materials can be used at the top of the fill, above the water level, placed by spreading and compaction. The geotechnical classification of these materials, the relation between their moisture content when placed and the density they reach in compaction tests and the determination of their bearing capacity (CBR) provide useful data for controlling the fill execution, with a view to designing future pavements.

4.4.4 Structural Behaviour

The following structural elements are part of this type of quay:

- Diaphragm wall.
- Tie-back.
- Capping beam connecting walls sections.
- System of anchorage to the wall.
- System of anchorage in the ground.
- Superstructure and its fenders.

The considerations for dimensioning the diaphragm wall as a structural member are not commented here in full. The only recommendations provided refer to the estimation of forces and moments.

Anchors are only examined here from an overall point of view, but their detailed design will also require taking many factors into account.

The details of the connection between tiebacks and diaphragm wall or with the anchor blocks, the superstructure and its fenders and many other aspects of quay design are considered to fall outside the scope of this ROM 0.5 and will be the subject of future publications of the ROM Programme.

4.4.4.1 Tension in the Tie-back

The tie-backs normally used in sheetpile quays are passive or lightly prestressed. In these circumstances they withstand a load produced as the reaction to the actions on the diaphragm wall. In any event, they can almost always be installed with a certain tension and it is possible to measure and regulate their load.

The anchors will sometimes be active, prestressed with a certain load that can later vary depending on the subsequent deformation of the quay. In this case, there is greater control of the anchor tension.

Anchor forces are one of the principal unknowns in sheetpile quays. It is only possible to determine them accurately by subsequent measurement through works monitoring. It is possible to calculate the anchor force approximately, however, by simplified procedures that are described in the technical literature.

The procedure for estimating anchor loads indicated in Subsection 3.7.11.1.4 is a simple method that is recommended in cases where considerable deformation is expected.

To construct active anchors, to backfill after the anchors are installed or to use very stiff diaphragm walls (made of reinforced concrete, for example) can lead to situations where the earth pressure behind the wall is greater than the active pressure, particularly in the top portion of the diaphragm wall. For this reason, it is recommended either to adopt high safety factors (to be defined in a future ROM publication) when this simplified calculation procedure is used, or else to give due consideration to the effect of soil-structure interaction.

4.4.4.2 Stress Resultants in Diaphragm Walls

The forces and moments acting on a diaphragm wall can be determined once the pressures in its contact with the ground are known.

The contact pressures are normally split into two components, one due to porewater and the other to the soil skeleton. The first can be evaluated, at least in an initial approximation, by the usual soil mechanics methods. The second component is more difficult to specify, since it will depend on the soil deformation.

The water pressure on diaphragm walls can be obtained after construction of the corresponding flownet. Recommendations for drawing steady-state flownets are given in Section 3.4 of this ROM 0.5.

Applied surcharges, or the construction operations themselves, can cause transient excess porewater pressure, which need to be taken into account when calculating some particular cases of pressures on walls.

The effective earth pressure against the diaphragm wall can, in principle, be assumed to be equal to the one corresponding to limit situations of active or passive pressure as appropriate.

Stress resultants on a diaphragm wall calculated with the pressures corresponding to Ultimate Limit States can vary considerably from the forces and moments really occurring in the wall under service conditions. This is particularly important in rigid diaphragm walls or in cases where the anchors allow only small deflections. It also happens when active anchors are installed and generally whenever ground deformations are more restrained. On many occasions, the redistribution of pressures is such that the maximum bending moment due to effective earth pressure is as much as 30% less than the value calculated by this simplified procedure. Bending due to water pressure does not undergo this redistribution.

Engineers should analyse each particular design and evaluate the stresses in the diaphragm wall with an accuracy matching the importance of the works. In general terms, they are recommended to analyse the interaction between diaphragm wall and ground, simulating the construction stages of the works, in all projects.

The simplified calculation procedure indicated in Section 3.7.12.2.2 will provide an approximate idea of the behaviour of the diaphragm wall.

4.4.5 Ultimate Limit States

In order to check the safety of a quay with diaphragm walls, it is a good idea to imagine a series of simple limit situations representing different failure modes. The works will generally be far from such theoretical situations, which could only occur in the event of a serious defect.

To define the location of the water level in the backfill and front face, the indications given in Subsection 4.2.3 (Gravity Walls) apply. To this effect, if no special drainage arrangements are made, sheetpile quay walls should be assumed to be impervious.

The following sections contain some recommendations about the more common theoretical failure modes and state the appropriate analysis procedure and the way in which safety should be introduced to keep the actual behaviour of the wall away from these undesirable failure modes.

Only failure modes governed by ground resistance are considered here. Failure of the anchors, the diaphragm wall or their connections is considered to lie outside the scope of this ROM 0.5.

The failure modes dealt with are illustrated in Figure 4.4.2.

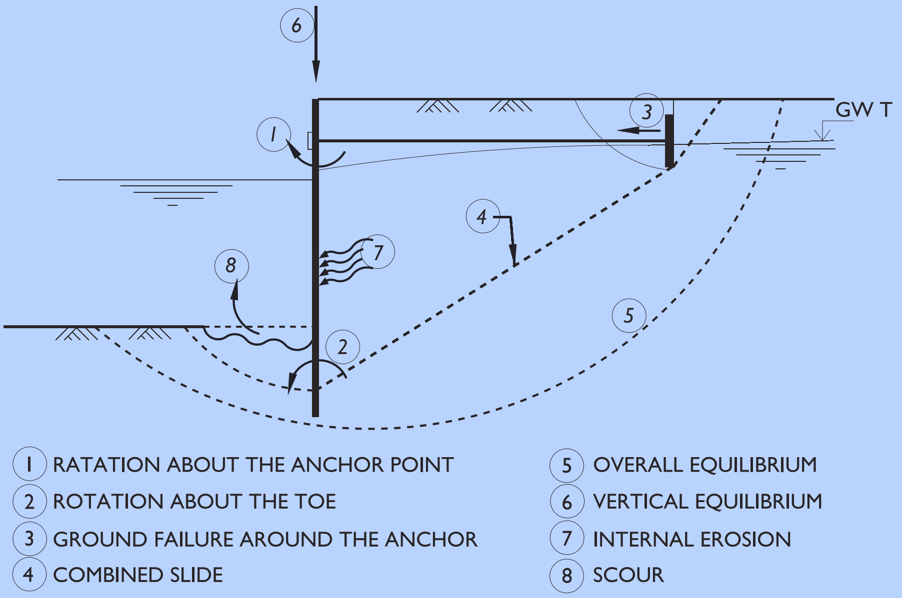
For the purpose of the safety checks indicated below, earth pressures should be calculated in accordance with the relevant recommendations given in Section 3.7.

Engineers can justify the decision to adopt other pressure calculation criteria on a case-by-case basis.

4.4.5.1 Verifying Safety against Rotation about the Anchor

In this theoretical failure mode, the diaphragm wall rotates around a point situated at or above the level of the anchor, as illustrated in Figure 4.4.3. This situation is sometimes called *free earth support*.

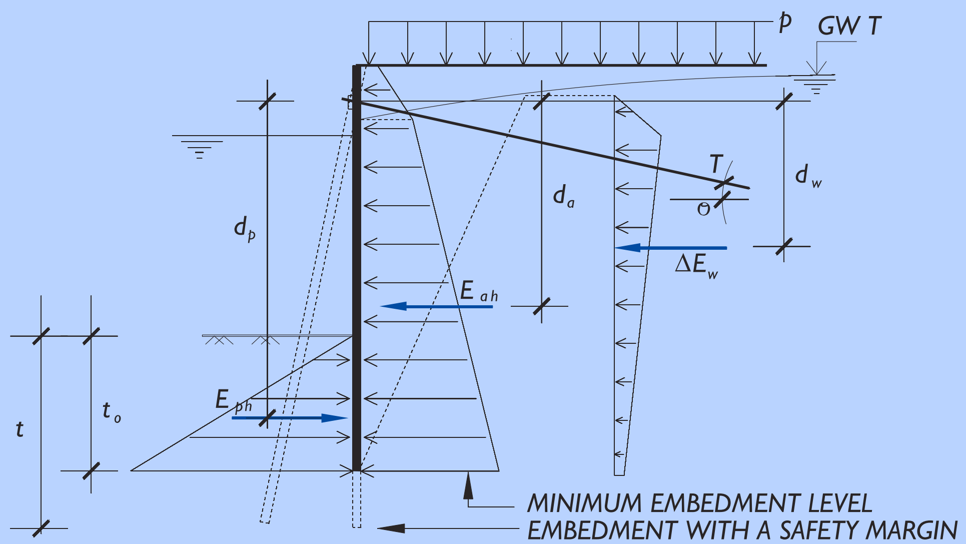
Figure 4.4.2. ***Geotechnical Failure Modes in Quays with Diaphragm Walls***



N.B.: There are other failure modes, which involve exceeding the structural capacity of the diaphragm wall, its anchors, their connections, etc., which are not considered here because they are structural in nature..

Figure 4.4.3. ***Wall Rotation Around the Anchor (Free Earth Support)***

EQUILIBRIUM OF SUFFICIENTLY ANCHORED DIAPHRAGM WALLS



MINIMUM EMBEDMENT CONDITIONS

$E_{ph}\,d_p = E_{ah}\,d_a + \Delta E_w d_w$ EQUILIBRIUM OF MOMENTS (1)

$T\cos\Theta + E_{ph} = E_{ah} + \Delta E_w$ HORIZONTAL EQUILIBRIUM (2)

This failure mode is possible provided that the anchor has sufficient capacity to prevent its own failure.

The diaphragm wall will be buried under the dredging line in a depth, *t*, which must always be greater than the minimum depth necessary to prevent this failure mode, named t_0 in the figure referred to.

For defining safety against rotation around the anchor, it is necessary to know this minimum depth of embedment. To this end, the distributions of horizontal active earth pressure on the back face of the diaphragm wall (landward side) and the horizontal passive earth pressure on the front face (seaward side) should have been obtained.

In order to draw these diagrams, it is necessary to follow the recommendations from Section 3.7, which deals with the study of pressures on retaining structures.

In calculating the earth pressures, it will be conservative to assume that the diaphragm walls are impervious. This should always be done, unless special construction arrangements have been made in the design to ensure a partial water flow.

Once the earth pressure distributions are defined, it is relatively easy to obtain the desired minimum embedment depth by taking moments with respect to the anchorage point for all forces acting on the diaphragm wall. This should be carried with several increasing trial values for depth t_0, until the condition of zero moment around the anchor is reached (Equation 1 in Figure 4.4.3).

Depth t_0 thus found will lead to a definition of the safety factor for the embedment depth, by comparing the real value *t* with the minimum required:

$$F = \frac{t}{t_o}$$

The resulting safety factor for works with a low SERI rating should, in any event, be:

$F_1 \geq 1.3$ Quasi-permanent combination.
$F_2 \geq 1.2$ Characteristic or fundamental combinations.
$F_3 \geq 1.1$ Accidental and seismic combinations..

Depending on the nature of the works (SERI rating) and the duration of the design situation, the modifications mentioned in Subsections 3.3.8 and 3.3.10 can be carried out for increasing or decreasing the recommended safety factors.

If it is justified for engineers to consider an alternative definition of the safety factor for other problems, in this particular case of overturning of the sheetpiles this is an especially important consideration, since there are different procedures for evaluating safety that may be equally acceptable. In the design calculations, nevertheless, evaluation of the safety factor as defined in this section should not be omitted, even though a different minimum safety threshold is used for duly justified reasons.

The calculation just explained will also enable the anchor tension *T* to be obtained when the condition of horizontal equilibrium is imposed for the minimum embedment depth t_0 as calculated above (Equation 2 in Figure 4.4.3).

The *T* value thus obtained will be an initial approximation of the minimum anchor capacity required. This value is also discussed in Subsection 3.7.11.1.3, where a more exact method is given for evaluating T.

If there are two or more anchor levels close to each other, rotation can be analysed around an intermediate point located at distances to them that depend on their respective load capacities (i.e., the point at which the resultant of the ultimate loads of the anchors intersects the wall). An equivalent virtual anchorage should be assumed to be located at this point.

This failure mechanism is not appropriate for diaphragm walls that are tied at different levels and have shallow embedment. Engineers should investigate other potential mechanisms that are more likely, and more appropriate to the particular circumstances and construction process involved.

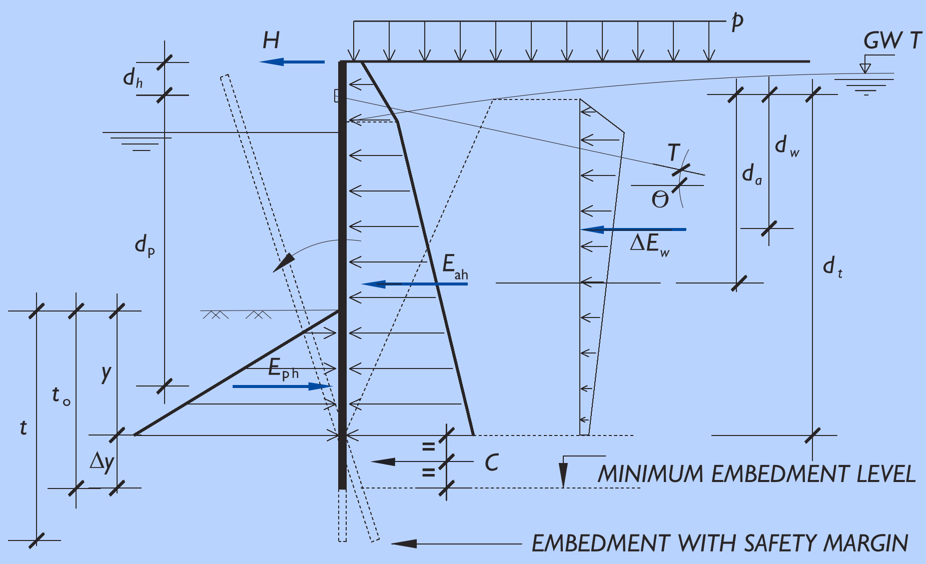
Shear forces and bending moments appearing in the diaphragm wall for this failure mechanism may be very high; in fact, much higher than those on the diaphragm wall when in service. The diaphragm wall may have exceeded its structural capacity before this type of failure can occur.

4.4.5.2 Verifying Safety against Rotation around the Wall Toe

The diaphragm wall can, at least in theory, rotate about a point close to its toe, as drawn in Figure 4.4.4. This failure mode is sometimes known as *fixed earth support*.

Figure 4.4.4. Wall Rotation around its Toe (Fixed Earth Support)

EQUILIBRIUM OF DIAPHRAGM WALLS WITH LOW-CAPACITY ANCHORS



LIMIT EQUILIBRIUM CONDITIONS:

Equilibrium of moments* (about the anchor)
$$M = E_{ph}\, d_p = E_{ah} d_a + \Delta E_w d_w - H\, d_h + C\left(d_t + \frac{1}{2}\Delta y\right)$$

Horizontal equilibrium
$$E = E_{ah} + \Delta E_w + H - E_{ph} = T\cos\Theta - C$$

Resistance
$$C = K_{ph} \cdot \sigma'_v \cdot \Delta y$$

SOLUTION:

$$t_o = \sqrt{d_t^2 + \frac{2M}{K_{ph} \cdot \sigma'_v}} + y - d_t$$

$$T = \frac{1}{\cos\Theta}\left(E + K_{ph} \cdot \sigma'_v (t_o - y)\right)$$

K_{ph} = coefficient of horizontal passive earth pressure beneath the rotation point

σ'_v = vertical effective pressure in the backfill at the elevation of the rotation point

* If no anchorage exist, moments can be taken about any other point
If anchorage does exist, the point shown is the most suitable

For this to occur, the anchor should have a very low capacity compared with the pressures on the diaphragm wall, or else not exist (cantilever walls).

One way of analysing this failure mode is described below. This procedure, like the others that have been considered all along in this ROM 0.5, implies that the structure has sufficient structural capacity to withstand the loads required to cause the ground failure.

In order to gain some knowledge on the safety against rotation of the diaphragm wall about a point close to its toe, several hypothetical situations should be analysed that always differ from the real ones. Safety can be evaluated precisely by comparing them.

These theoretical situations can be defined, in this case, by assuming different rotation points close to the foot of the diaphragm wall.

Having taken a rotation point (at depth *y* in Figure 4.4.4), the problem can be solved as shown in that figure.

By varying the location of the rotation point, a series of values can be obtained for the embedment length, t_0, and the tension on the anchor, *T*, which would place the diaphragm wall in strict equilibrium in relation to this failure mode.

The calculation for each rotation point must begin from the minimum embedment depth as obtained in the previous subsection. Otherwise, negative counterpressures, *C*, will result.

The results of analysing these theoretical situations should be represented in diagram form as illustrated in Figure 4.4.5.

A point *P* can be placed in this safety diagram (such as *A*, *B*, etc.) to represent the real situation of the works, with the embedment depth of the diaphragm walls really planned and with the intended anchor capacity [(1)].

The safety diagram must be completed by the vertical straight line (*t* = constant) corresponding to the failure mode analysed in the previous section.

The present analysis procedure defines the safety factor for this failure mode as the quotient:

$$F = \frac{\overline{OP}}{\overline{OP'}}$$

where $\overline{OP}$ and $\overline{OP}$’ are the distances from points *P* and *P’* to the origin in the safety diagram. *P* represents the actual diaphragm wall data. *P’* is obtained by intersecting line OP with the boundary separating the stable from the unstable zones.

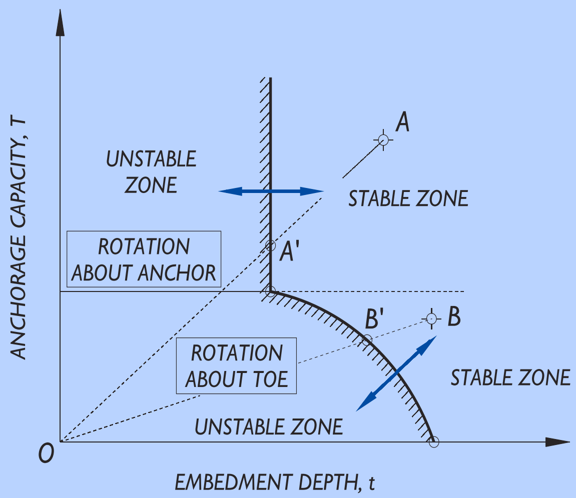
Figure 4.4.5 illustrates the procedure for two different diaphragm walls that would be represented in the safety diagram by points A and B.

Point *A* would represent a situation in which the most likely failure mode corresponds to rotation about the top (this would result from simultaneously and proportionally reducing the embedment length and the anchor capacity). In a situation such as *B*, failure would occur by rotation about the toe.

The minimum safety factors required, when using this calculation procedure, are the same as those indicated in subsection 4.4.5.1 above.

(1) The definition of *anchor capacity* requires certain structural considerations that lie outside the scope of this ROM 0.5. Provisionally and even though other criteria may be adopted in the future in the ROM Programme, anchor capacity is defined –for these purposes only– as one half of the load producing the yield stress in the anchor material.

Figure 4.4.5. **Definition of Safety against Wall Rotation**



SAFETY FACTORS

If the diaphragm wall is represented by Point A:

$$F = \frac{\overline{OA}}{\overline{OA'}}$$

If the diaphragm wall is represented by Point B:

$$F = \frac{\overline{OB}}{\overline{OB'}}$$

The same comments on alternative ways of defining safety made in the previous subsection also apply in this case.

4.4.5.3 Verifying Safety against Failure of the Ground Surrounding the Anchor

Failure in tie-backs can occur by several mechanisms depending on the type of system used.

Piles (or micropiles), rods or steel cables lodged in the ground inside grouted boreholes and used as tie-backs can fail in their contact with the ground or by exceeding their structural capacity.

Tie-backs of non-grouted steel bars or cables (not attached to the ground) may fail either as a result of steel yielding or due to failure of their connections to the diaphragm wall, to the anchor block or anchor wall. These aspects are considered to lie outside the scope of this ROM 0.5.

The recommendations given for piles under tensile loads in Subsection 3.6.7 can be followed with respect to failure along the contact of the ground with anchors that can be assimilated to piles.

Recommendations are given below in relation to the required safety against failure of the ground in the area surrounding anchorage blocks or plates.

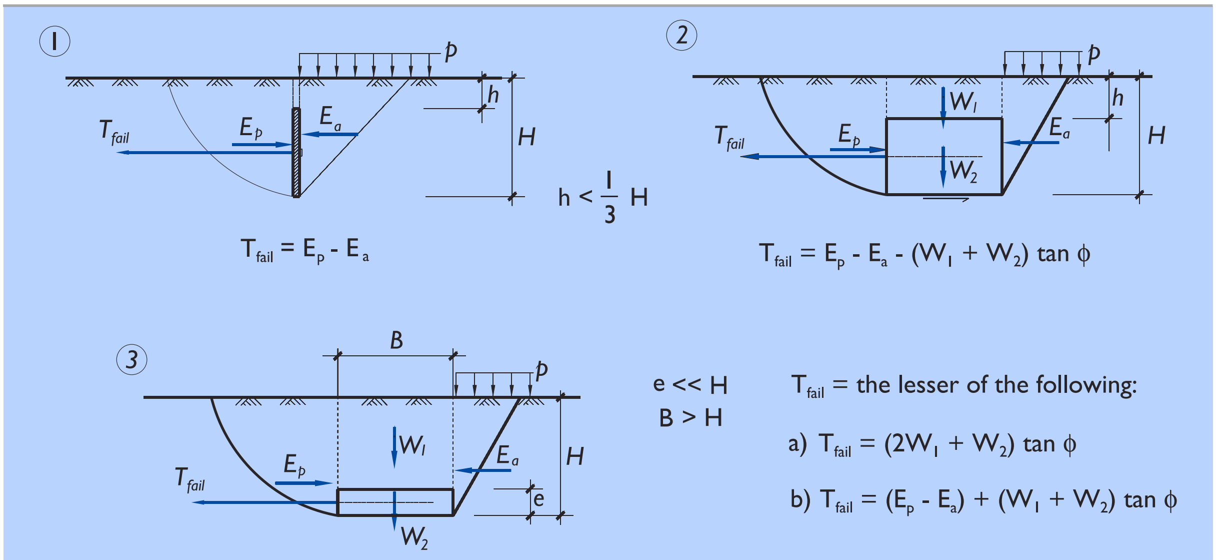
The resistant capacity of anchor blocks (concrete deadmen, buried plates or anchor walls) is generally based on that of the surrounding soil. The quality of the ground around the anchor block is therefore essential when designing an anchorage system.

When the ratio between the earth covering, *h*, and the foundation depth of the anchor block or wall, *H*, is small ($h/H < 1/3$, approximately), the passive earth pressure on the seaward face can be considered as being fully mobilised. These are the top and middle cases illustrated in Figure 4.4.6.

When the earth covering is large compared to the thickness of the deadman (buried slab, as illustrated in the bottom section of Figure 4.4.6), the sliding of the plate along its contact with the ground, on both faces, may possibly prove more critical.

Estimating the active and passive pressures, required to verify safety against sliding of this type of anchorage, must be carried out following the recommendations given in Section 3.7.

Figure 4.4.6. Diagram of Various Anchorage Systems



N.B.: In the three cases indicated, E_p and E_a correspond to the total height *H* and they run parallel to the pull, *T*.

The safety factor against this type of anchor failure is defined as the quotient between the force producing anchor failure and the force acting on the anchor under the design situation considered:

$$F = \frac{T_{(failure)}}{T}$$

This safety factor, for works with a low SERI rating (5 - 19), should be:

$F_1 \geq 2.5$ for quasi-permanent combinations.
$F_2 \geq 2$ for fundamental combinations.
$F_3 \geq 1.8$ for accidental and seismic combinations.

The indications in Subsections 3.3.8 and 3.3.10 should be considered for increasing or decreasing the safety factors recommended, depending on the nature of the works and the duration of the design situation.

Subsection 3.7.9.2 covers other anchorage solutions incorporating discontinuous elements.

4.4.5.4 Verifying Safety against Combined Sliding

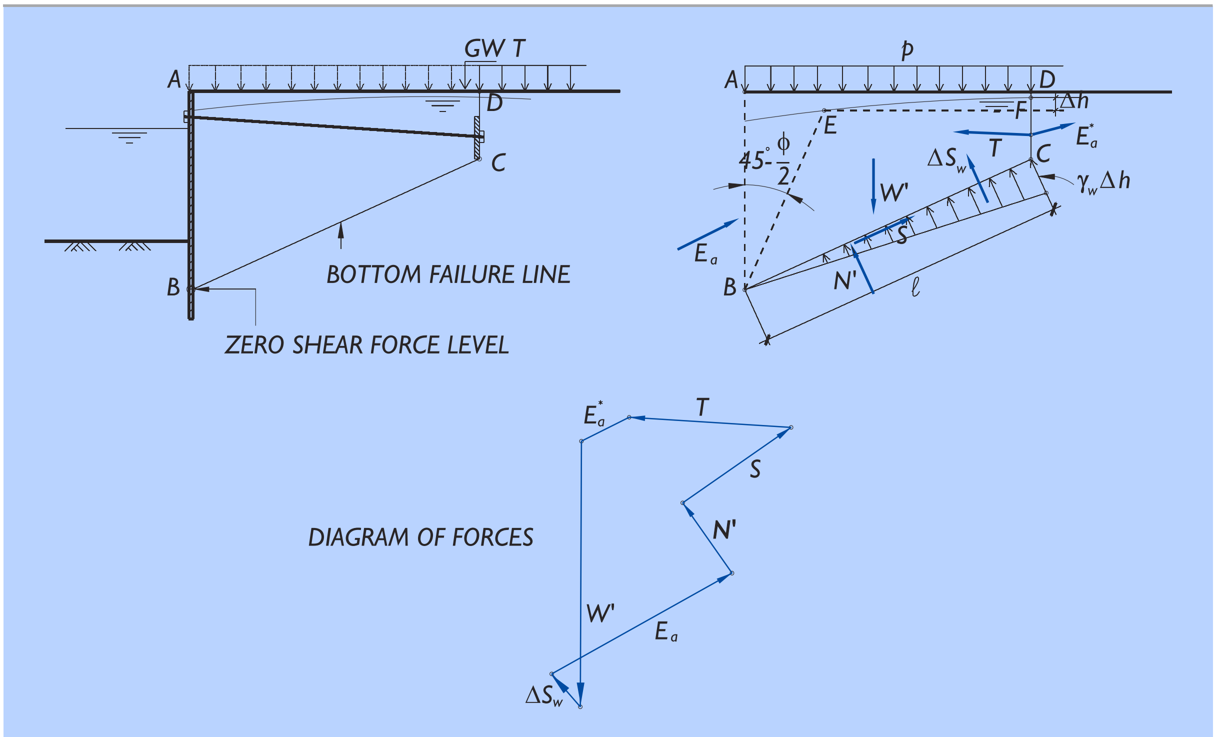
Combined sliding, as considered in this section, is similar to the slip defined in the Kranz method [(2)].

This failure mechanism is applicable to diaphragm walls tied to other shorter walls a certain distance away. This failure mechanism make it possible to evaluate the minimum distance at which these anchor walls must be placed, as a function of the anchorage force necessary.

The sliding mass is represented in Figure 4.4.7. This is the ground between the two diaphragm walls inside the *ABCD* block.

(2) "Uber die Verankerung von Spunwanden". 2 Aufl Berlín 1953.

Figure 4.4.7. **Combined Sliding of Main and Anchor Walls**



Point *B* in the figure is defined as the point on the diaphragm wall where the backfill earth pressure (assumed to be active) is in horizontal equilibrium with the earth pressure in the embedded zone (assumed to be passive) and with the tension of the anchor. The shear force on the diaphragm wall will be zero at this point.

The location of the point of zero shear force is unknown. To carry out these calculations, the depth of this point *B* should be taken as the minimum embedment depth as calculated in Section 4.4.5.1.

The forces acting on the block under study should be calculated as described below.

a. Vertical Back Face. Seaward (Main) Diaphragm Wall

It should be assumed that the active pressure (that will previously have been calculated for other checks) acts on this face (*AB* in the figure). This pressure will generally have two components, the one due to the effective earth pressure (including loads and overburdens) and the one due to the porewater pressure not offset by the level of free water. Calculation of these pressures is detailed elsewhere (Section 3.7). Resultant E_a shown in the figure is the sum of all these pressures.

b. Vertical Face of the Anchor Wall

The ground inside the mass under study is pushed by the anchor pull and by the earth active pressure.

The anchor pull is an unknown in the problem to be solved by analysing this failure mechanism. This will be dealt with later on.

The active pressure on this vertical line (*DC* in the figure) should be calculated following the procedure laid out in Section 3.7.

c. Self-Weight of the Mass

The weight of the sliding mass (*ABCD* in the figure) should include the overburdens that really exist on it. Depending on the inclination of the slip surface (*BC* in the figure), the presence of loads and surcharges may or may not be beneficial. Engineers should examine this aspect under each design situation.

The weight of the mass can be calculated with the apparent densities and then subtracting the uplift at its base. Figure 4.4.7 also suggests an alternative procedure with similar results and which is more suitable (it is compatible with the calculation method based on the concept of earth pressure coefficient defined in Section 3.7 of this ROM 0.5). Above the level marked by point E in the figure, the apparent specific weights should be used and below it, the submerged unit weights. The uplift on the failure line should only be the one due to excess porewater pressure above this level. This is the value ΔS_w shown in the figure.

The equilibrium for these forces is shown in the bottom section of the figure. The relationship between the shear force on the plane of slide (force *S*) and the tension on the anchor, *T*, can be worked out from this equilibrium. If one of them is known, the other can be calculated.

It would be possible to estimate the value of *S* with the failure condition and thus obtain an upper limit for the anchor force that would then be used in some way to define safety. The recommendation here is to do an additional, similar calculation.

The minimum value for the anchor tension *T*, required to guarantee the stability of the diaphragm wall so that the minimum safety requirements are met, must be taken as equal to the value obtained in Subsection 4.4.5.1 (Rotation about the Anchor). Even if the equilibrium can in fact be guaranteed with less anchor force, this reference value is the one that should be used in the following calculation.

For this value of *T*, the corresponding shear force *S* acting on the plane of slide should be calculated. The safety factor against combined sliding is defined as the following quotient:

$$F = \frac{N' \tan\phi + c \cdot \ell}{S}$$

where:

N' = normal effective reaction on the slip plane.
c, ϕ = strength parameters along the slip line.
ℓ = length of the slip line.

The safety factor for works with a low SERI rating (5 - 19) should be:

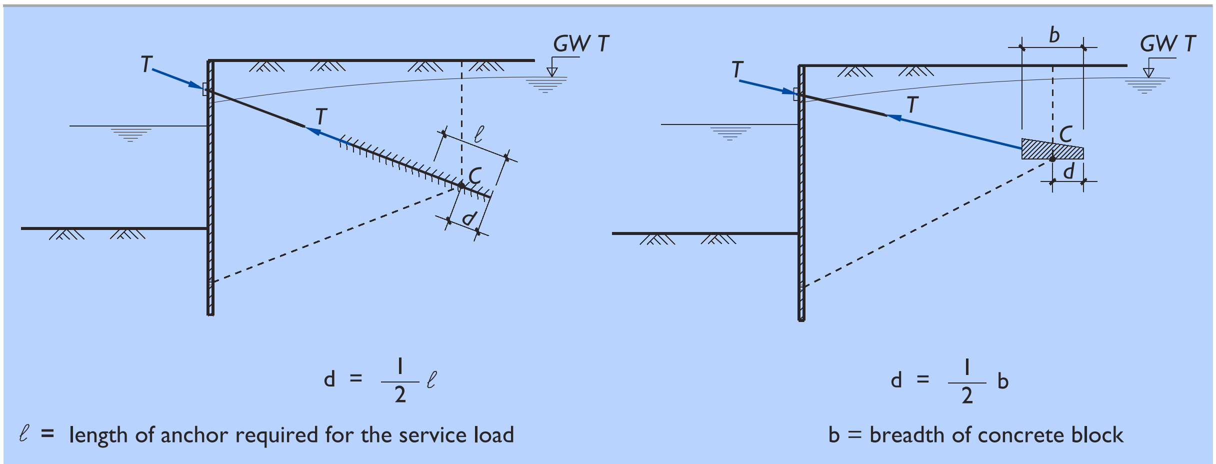
$F_1 \geq 1{,}5$ for quasi-permanent combinations.
$F_2 \geq 1{,}3$ for fundamental combinations.
$F_3 \geq 1{,}1$ for accidental and seismic combinations.

As indicated in Subsections 3.3.8 and 3.3.10 for works of another nature (SERI < 5 or ≥ 20), these safety coefficients must be adapted accordingly.

If this safety factor proves to be less than the minimum required, it will be necessary to site the anchor wall farther away from the main wall or make it deeper. The possibility of reducing the anchorage capacity should only be considered in special cases and even then should be clearly justified by engineers.

When the anchorage system uses cables, rods or micropiles rigidly attached to the ground, the calculation should be similar to the one described. For this purpose, point *C* should be located on the anchor itself at a certain distance from its end, as shown tentatively in Figure 4.4.8. The potential favourable effect of the portion of the anchorage system located beyond the *C* vertical (seawards) should not be taken into account in the calculations.

Figure 4.4.8. ***Position of Point C in other Forms of Anchorage***



4.4.5.5 Verifying Safety against Overall Instability

It could happen that other failure surfaces more distant from the structural elements and that enclose the diaphragm wall and its anchorage elements are more critical than those considered up to now. For this reason, overall equilibrium should be analysed, following the recommendations given in Section 3.8 of this ROM 0.5.

An explicit warning is needed about the potential existence of deep strata with particularly low strength, which must be investigated with caution. If a weak zone is identified, failure mechanisms with a large part of their slip surface inside the weak zone should be analysed. To this end, overall stability analysis procedures may be necessary that are different from the circular slide procedures routinely assumed. Recommendations are also given in Section 3.8 of this ROM 0.5 that may help to guide engineers in these circumstances.

4.4.5.6 Verifying Safety against Vertical Instability

The consideration of inclined active pressure, the self weight of the diaphragm walls, the possible transmission of vertical loads via the superstructure at the wall head and via the possible inclination of the anchors all mean that the diaphragm walls are subjected to axial compression forces.

These vertical compression forces must be offset by the tangential component of the passive earth pressure and by the tip bearing capacity of the diaphragm walls.

Vertical equilibrium must be expressly analysed by calculating the safety factor against bearing failure as shown in the diagram in Figure 4.4.9.

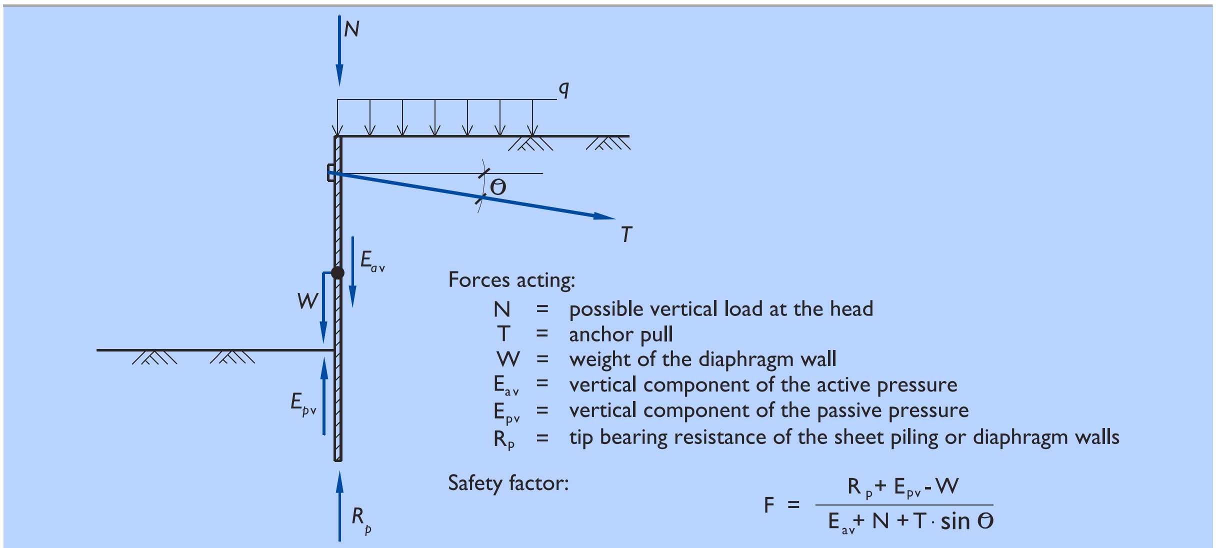
For the purpose of calculating the tip bearing capacity of diaphragm walls, the recommendations given in Section 3.6 of this ROM 0.5 should be followed.

The safety factor obtained for this Ultimate Limit State must be over the minima recommended for piled foundations defined in Subsection 3.6.6 (Table 3.6.1).

4.4.5.7 Verifying Safety against Internal Erosion

The water flow resulting from waves, tidal runs and natural or artificial groundwater tables that are higher on the landward side may cause entrainment of the backfill material.

Figure 4.4.9. **Safety with Respect to Vertical Equilibrium**



A particularly dangerous situation arises when, by some defect in executing the diaphragm walls or the sheetpiles, a gap is left in the wall.

Internal erosion, if localised, may simply lead to sinkholes in the horizontal surface of the backfill. If it is more extensive, it can lead to substantial displacements in the quay or even to its total failure.

Internal erosion must be fought by preventing defects in execution (inspecting the sheetpiles after constructing the diaphragm walls could be of value) and arranging the fill materials in such a way that they fulfil filter conditions at their contacts (see Subsection 3.4.7 for a definition of these conditions). Artificial filters can also be installed across the path of the water.

4.4.5.8 *Verifying Safety against Scour*

The scour that can be caused by the propellers of vessels in berthing manoeuvres, waves and sea currents must be taken into account in the design of quays with diaphragm walls. Scour in this type of quay is particularly important since, as seen above, stability will largely depend on the embedment on the seaward side. A significant reduction in this depth caused by unpredicted scour can reduce the safety of the works, increase their deformation and even lead to their total failure.

The considerations included in Subsections 4.2.3.7 and 4.3.5.7, relating to gravity and piled quays respectively, apply in relation to the preventive measures against scour.

4.4.5.9 *Summary of Minimum Safety Factors*

Part 3 of this ROM 0.5 defines the safety factors engineers should adopt when designing quays with diaphragm walls to ensure that each of the Ultimate Limit States is not reached.

When describing each of the Ultimate Limit States, reference was made to the section of this ROM 0.5 where the calculation procedure and the safety factor to be adopted are defined. The two aspects (calculation method and safety factor) are interconnected and should not be dissociated. Nevertheless, Table 4.4.1 is included here, which summarises the minimum recommended safety factors for each Ultimate Limit State. Engineers must be aware of the associated analysis method before using them.

4.4.6 Limit States of Serviceability

Diaphragm walls are generally highly flexible. Their deflections may cause settlement or displacement in adjacent structures or even in the superstructure that may have been constructed on the quay itself.

Cantilever walls are particularly subject to deformation and their use must be restricted to very small draughts. A free height of two or three metres, in this type of wall, is sufficient for observing horizontal deflections in the order of decimetres in soft ground.

Table 4.4.1. Minimum Recommended Safety Factors for the Design of Diaphragm Quays, Works with Low SERI (5 - 19)

Section where the Associated Calculation Method is Defined	Ultimate Limit States of Geotechnical (GEO) Failure*	Load Combination		
		Quasi-Permanent F_1	Fundamental or Chracteristics F_2	Accidental or Seismic F_3
3.7.11.1.3	Rotation about the anchor	1.3	1.2	1.1
3.7.11.1.3	Rotation about the toe	1.3	1.2	1.1
3.7.9.2	Ground failure around the anchor	3.0	2.4	2
3.8	Combined sliding	1.5	1.3	1.2
3,8	Overall instability	1.4	1.3	1.1
3.7.11.1.3	Vertical instability	2.5	2	1.5
–	Internal Erosion	PM	–	–
–	Scour	PM	–	–

* Those mainly governed by ground resistance.
MP In these cases, safety is not usually quantified. The problem can be avoided by taking adequate preventive measures (PM).
Note 1: Before using these safety factors, the associated calculation methods defined in this ROM 0.5 should be known, as described in Section 4.4 and in the subsections appearing in the first column.
Note 2: Depending on the nature of the works and the duration of the design situation, the modifications from Subsections 3.3.8 and 3.3.10 should be carried out for increasing or decreasing the recommended safety factors.

Deformation of quay diaphragm walls will not only be governed by the quality of the ground involved but also by the system of anchorage. The deflection of these elements may be considerable, particularly in cases where their resistance is based on mobilising passive pressure.

Deformations can be estimated empirically, by analogy with other similar works where displacement has been measured, or analytically by adequate computations of soil-structure interaction.

A study of the cracking will be a crucial element of the design in concrete diaphragm walls.

Diaphragm quays constructed on soft soils can lead to substantial settlement. Natural consolidation of the ground on which fills are dumped and the consolidation of the fills themselves can cause considerable deferred settlement. This matter should always be analysed.

The effect of settlement on the anchorage system should be expressly examined, as it can cause undue bending and even its failure. Anchors must be properly protected against this detrimental effect (for instance placing them inside protective boxes of a suitable height).

Important elements on the quay backfill may require deep foundations, not only to prevent their displacements but also to reduce the pressures on the diaphragm wall.

4.4.7 Other Recommendations

Other aspects of interest are presented here on a provisional basis, pending other indications to appear in future publications in the ROM Programme.

Drainage

Sheetpile walls (and also concrete diaphragm walls) are generally impervious and therefore considerable excess water pressures may be generated in their backfill. Designing an adequate drainage system (with clapper valves, for example) could be useful for reducing the pressures during low tides.

In such cases, it will be absolutely essential to check regularly the proper operation of the valves. In any event, these drainage measures demand that the backfill close to the diaphragm wall is granular and that an adequate filter system is installed.

Sheetpile Lengths

The buried portion of sheetpiles close to their tip is subjected to lower structural stresses than those produced at other points (at the level of the dredging line, for example). For this reason, some of the sheetpiles can be designed to be shorter. Similar considerations apply to the portion close to the head.

A sufficiently safe quay can be designed with alternating shorter sheetpiles (one out of every three or even one out of two) if properly analysed. This savings, however, can entail other constructive complications.

It is not recommended to reduce the embedded length by more than one metre on this account.

It is also possible to construct shorter diaphragm walls resting on regularly spaced piles penetrating deeper into the ground. The deep zone of the piles must be capable of withstanding the forces corresponding to the portion of the diaphragm wall they replace.

Inclination

The head of quay walls will generally tilt seawards as a result of backfill pressure. For compensating this, post tensioning the anchors or driving the sheetpiles with a certain landward inclination are worth considering.

4.5 QUAY WALLS MADE OF SHEETPILE CELLS

The ROM Programme plans to devote a publication to the combined consideration of the structural and geotechnical behaviour of wharfs with sheetpile cellular structures.

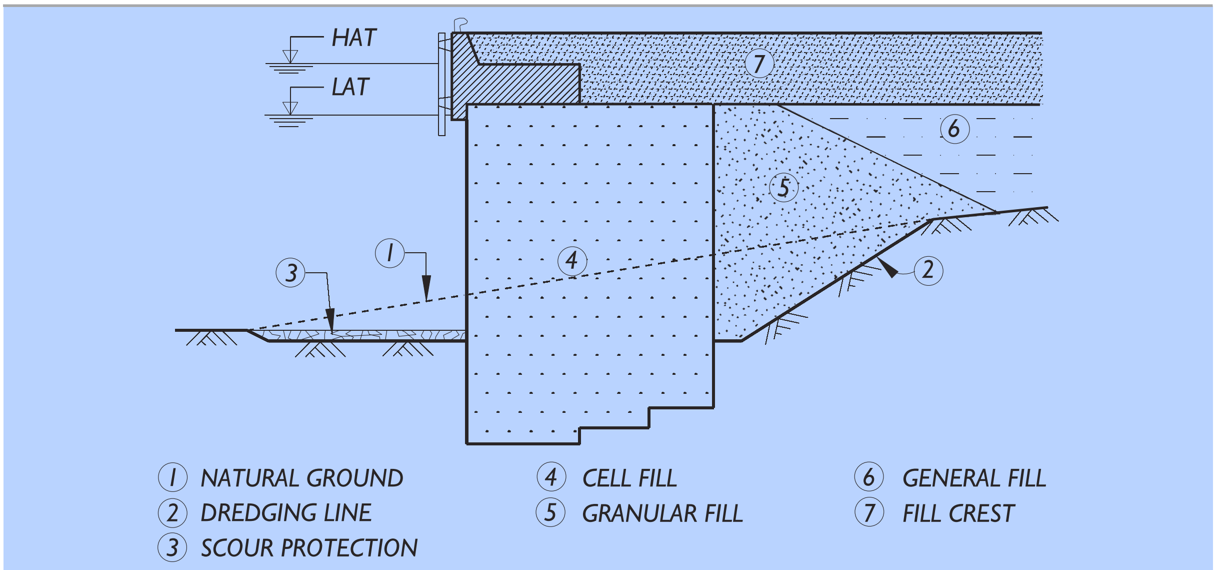
Although it is difficult to separate the two aspects of the problem, some provisional recommendations have been anticipated in this geotechnical ROM 0.5, for the purpose of guiding engineers in the design of such structures.

Therefore, the following recommendations are incomplete. The main aspects of structural behaviour have only been briefly covered.

4.5.1 Types

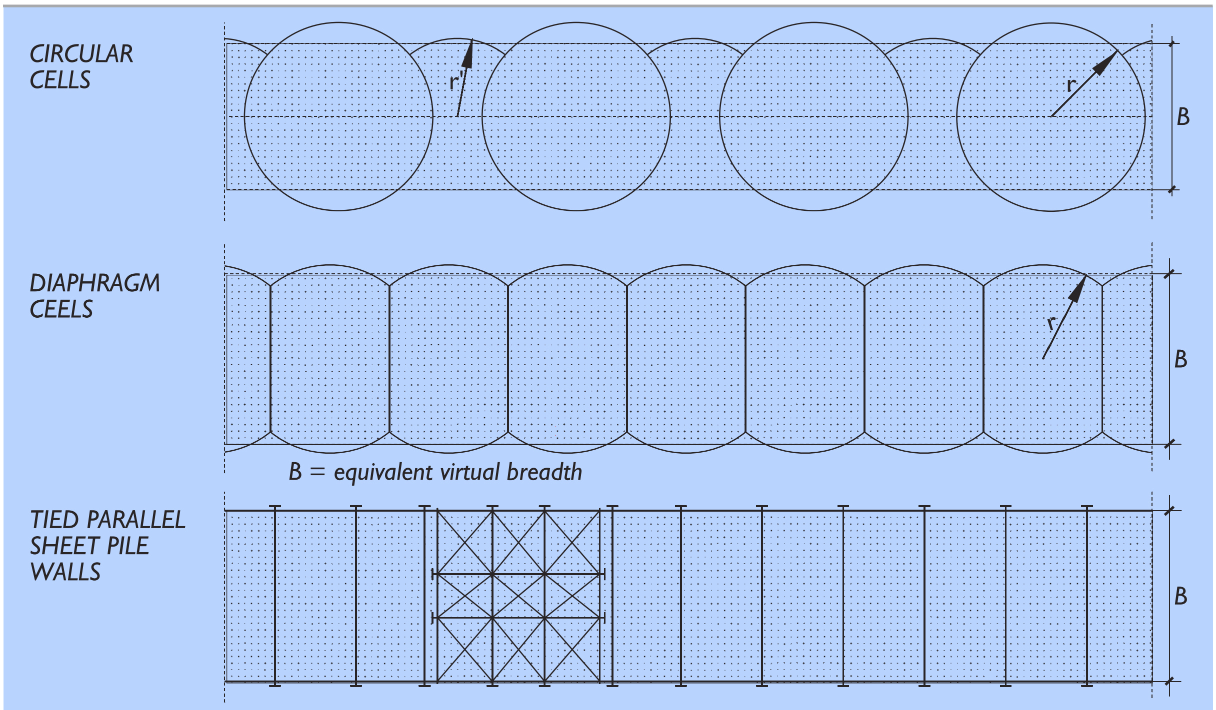
The basic type of sheetpile cellular wall for quays is illustrated in Figure 4.5.1, which highlights the main elements from the geotechnical point of view.

Figure 4.5.1. **Quay with a Wall of Sheetpile Cells**



Cellular structures can be constructed with circular shapes from flat sheetpiles, creating independent cells that are then connected at the front (and possibly also at the back) by arcs of specially shaped sheetpiles. They can also be constructed with diaphragm cells, which have straight transverse walls and curved fronts. The two types are shown in Figure 4.5.2.

Figure 4.5.2. **Plan View of Sheetpile Cellular Structures**



The cells can be filled independently in the case of circular enclosures. With diaphragm cells, the fills must be executed simultaneously (a certain lag is permissible) and a greater number of sheetpiles are necessary. Their potential advantages are based on the lower loads on the sheetpiles (for the same draught).

Quays walls formed by two parallel rows of sheetpiles, braced together at different levels, can also be considered within this category. This type would be a special case of the anchored sheetpile quay wall, in which anchorage is achieved by a second row of sheetpiles similar to the one on the seaward side. It is common with this type of design to include some heavily braced cell that will lend rigidity to the structure and facilitate differential levels between cell fill and backfill.

Diaphragm structures can have curved partitions, thus facilitating the differential fill between adjacent cells.

There are also variations suitable for large draughts such as cells shaped like a four-leafed clover tied along two axes, cells with elliptical shapes or with varying curvature (curves with several different radii), etc.

A comparative study of different types will assist engineers in choosing the most adequate for each case.

For the safety checks referred to later, each of these types should be represented by an equivalent cross-section with breadth B.

This equivalent width must be chosen so that the plan area is maintained.

To this effect, in circular enclosures with no back closing between cells (top diagram in Fig. 4.5.2), a virtual back plane should be assumed, located at a distance $(\pi/4)r$ from the cell axis, where r is the cell radius.

4.5.2 Design Factors

4.5.2.1 Ground Data

Sheetpile cellular walls for quays can be constructed on firm or medium quality ground. Soft soil deposits of considerable thickness may render this type of solution inadequate.

The structure of the natural ground (its type at different spots and at different depths) needs to be known over a sufficiently extensive area and to an appropriate depth, as recommended in Part 2 of this ROM 0.5.

Details of the nature, strength and deformability of each ground level are needed, with greater accuracy close to the foundations, as in the case of other types of quay (see Sections 4.2, 4.3 and 4.4).

A special study should be made of the part of the natural ground that may lie inside the cells. The subsequent decision on whether to dredge it or leave it as part of the internal fill should be based on a good investigation of this zone.

4.5.2.2 Source and Quality of Materials

The material for filling the enclosures should preferably be granular, otherwise large tensile stresses will be produced in the sheetpile walls and this could lead to expensive solutions.

For the cell fill material, its resulting density and the corresponding internal angle of friction -after being placed- should be known in order to estimate the level of safety against the different limit states.

When prospecting for materials, the natural bed should be considered as potential fill for the inside of the cells.

The data to be known about the backfill materials relate to their nature, density, strength and deformability. The comments made concerning gravity quay walls (see Section 4.2) apply in this respect.

Deep compaction (by vibration, for example) of the cell fills could be convenient in sheetpile cellular structures. This aspect must be investigated in each specific case.

The materials to provide protection against scour at the toe of quay walls should be studied as indicated for other types of quay (Sections 4.2, 4.3 and 4.4).

4.5.2.3 Water Levels

The most critical situation for this type of works will normally correspond to a low tide and a high groundwater table on the landward side.

The levels that should be considered are the same as those indicated for gravity quays (see Subsection 4.2.3). To this end and unless special precautions are taken, sheetpile cell walls should be assumed to be impervious.

The potential effects of natural or vessel-induced waves should also be taken into account as indicated in the section corresponding to gravity quay walls (Section 4.2).

4.5.3 Ultimate Limit States

Sheetpile cellular structures have their own types of failure that are still not well known. The most typical failure modes, which must be analysed in any event, are the subject of this section.

Quay walls formed by sheetpile cells work in a very complex manner. During the cell filling stage, the fill produces compression against the cell walls that leads to substantial tensile stress in the steel of the sheetpiles. The critical stage for structural failure (limit stresses in the steel) can occur during construction of the sheetpile enclosures.

Dredging operations carried out on the seaward side (if this is planned in the specific design in question) can increase the stresses in the sheetpiles.

Compression can be produced in the cell walls when dredging is carried out inside the enclosure to remove any unsuitable natural soil from the bottom. This aspect requires special consideration.

During backfilling and particularly when the quay enters service and the surcharges are installed, the limit situations dealt with below usually occur.

For this reason, although analysis of construction operations, in the different construction stages, is essential for checking the stress state in the walls of cells or cellular structures, these situations will not generally need to be considered when evaluating the safety of the works with respect to geotechnical Limit States, i.e., those governed by the nature of the ground (strength, deformability and permeability) and which are covered in this ROM 0.5.

The geotechnical Ultimate Limit States dealt with in this section are simplifications of situations that are similar to those observed in reality, both in laboratory prototype tests and in real instrumented cases, in which the behaviour observed has included failure (laboratory tests) or only slightly approached it.

In laboratory tests, when cells are pushed to failure, this can occur in a wide variety of ways governed by the type of prototype tested and the relation between the eccentricity and inclination of the loads on the base of the cell.

As a general rule, when the eccentricity of these loads is small (pressures applied at a low height), the mechanism appearing to govern cell failure is horizontal sliding.

When the application point of the thrust is high, the failure mechanism will be one of the following:

- Cell overturning.
- Failure due to cell shear.

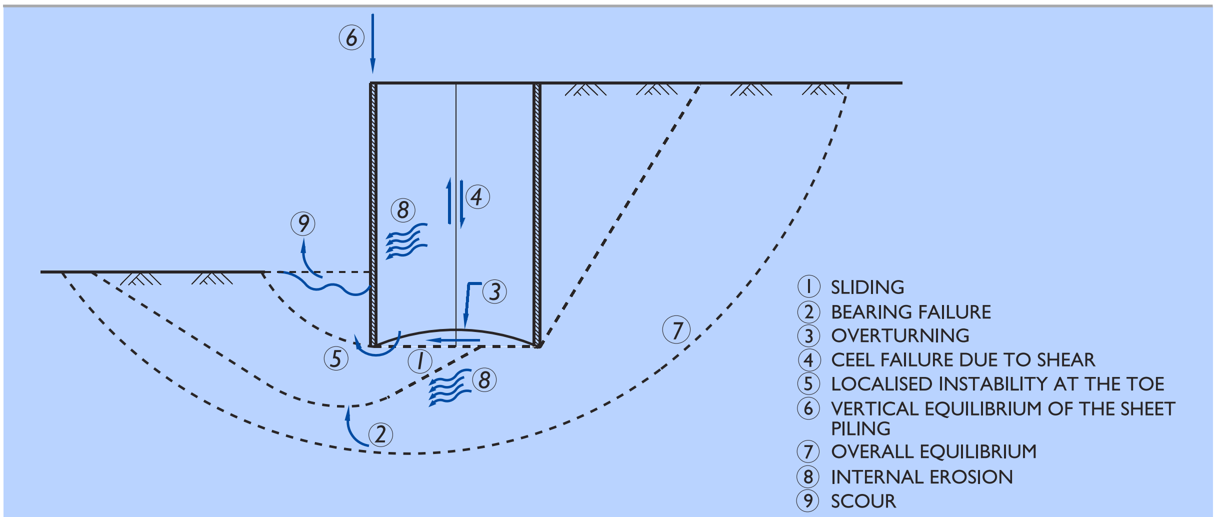
In turn, these theoretically possible failure mechanisms may have a different morphology depending on the relative stiffness of the material filling the enclosures as compared to that of their steel walls.

Independently of these failure modes, bearing failure of the quay wall as a rigid body can occur in cases where the foundations have low resistance. When the ground is weak down to considerable depths, overall failure on a major scale can occur, including the quay and its foundation ground.

Finally, other localised limit situations can occur, such as flow of the fill from the cell towards the front face if the sheetpiles in that zone are only shallow-driven.

These failure modes are illustrated in Figure 4.5.3.

Figure 4.5.3. Geotechnical Failure Modes in Quays with Sheetpile Enclosures



N.B.: Only failure mechanisms of a geotechnical type are included, i.e., those mainly governed by ground characteristics.

These and any other problems that engineers might conceive must be analysed to guarantee that there is sufficient safety against each of them.

As the following subsections only deal with limit situations, the ground parameters that should be used in carrying out these checks are those corresponding to large strain.

To be specific, an active earth pressure should be assumed in the zones where the failure mechanism under analysis involves a horizontal displacement of the cell away from the ground. The pressure will be passive when the relative displacement is in the opposite direction.

Outside the enclosures, the earth pressure acting against their walls should generally be assumed to be inclined at an angle of $2\phi/3$, where ϕ is the angle of friction of the soil at the contact. The direction of the pressure should be consistent with the relative movement assumed in each failure mechanism. The active pressure of the backfill will generally slope downwards while the passive pressure on the front face (the embedded portion on the seaward side) will act upwards, trying to lift the cell.

These general criteria can be somewhat modified for some particular limit states considered here. If this is so, the modification will be expressly mentioned in the corresponding section.

In analysing each possible failure mode, all the actions and load combinations should be taken into account as stipulated in detail in Part 3 of this ROM 0.5 (Section 3.3).

4.5.3.1 Verifying Safety against Sliding

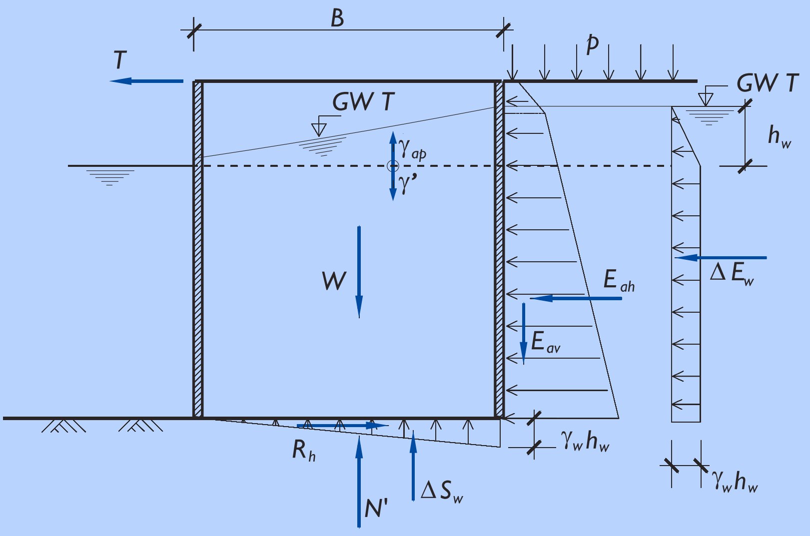
The sliding failure mode considered here is similar to the one described for other gravity retaining structures. The cells will be displaced horizontally, sliding along a horizontal plane close to their base.

In this specific case, a distinction should also be made between two situations depending on the passive pressure that can develop at the front toe. These are the cases referred to as "shallow foundations" and "embedded foundations" described below.

4.5.3.1.1 SHALLOW FOUNDATIONS

When the quay rests on firm soil or rock and self-stabilising, properly tied cellular structures are used, so that the sheetpiles do not need to be driven clearly below the ground surface, the sliding mechanism as a rigid block can be analysed as shown in Figure 4.5.4.

Figure 4.5.4. Safety Factor against Sliding in Non-Embedded Cells



VERTICAL EQUILIBRIUM

$$N' = W + E_{av} - \Delta S_w$$

HORIZONTAL EQUILIBRIUM

$$R_h = E_{ah} + \Delta E_w + T$$

SAFETY FACTOR

$$F = \frac{N' \tan\phi + cB}{R_h}$$

N.B.: a) The weight should be calculated as the product of the areas by the apparent unit weights when the areas are situated above the groundwater table of the front face (γ_{ap}) or by the submerged unit weights (γ') when the areas are below this level.
b) In equilibrium equations, the different horizontal and vertical forces corresponding to each load case will be present in each case.

The geotechnical parameters c and ϕ, to be used in calculating the safety factor against sliding as shown in the figure, can be the ones corresponding to the fill or the foundation ground. The parameters leading to a lower safety factor should be chosen.

The recommendations given in Section 3.7 of this ROM 0.5 should be followed in calculating the backfill active pressure on the cell.

Several procedures can be used to calculate the specific weight of the cells. The procedure illustrated in the figure is considered the simplest and most advisable.

For calculating weights, the apparent unit weights should be used from the free seawater level upwards and the submerged specific weights from that level downwards. The excess uplift (ΔS_w in the figure) must be subtracted later.

The excess uplift will depend on the position of the groundwater table inside the cell, which will generally be curved. It is considered admissible to assume that the uplift at the base of the cell varies linearly from back to front face with limit values equal to $\gamma_w h_w$ on the landward side and zero on the seaward side.

The value of h_w will be the difference in water level between the backfill of the cell and the sea level.

On occasions, when special drainage systems are installed to keep the groundwater table low inside the cells, engineers may justify a lower value for h_w in order to calculate uplifts.

When this aspect is particularly important, it may be desirable to calculate the local flownet beforehand in order to define the effective weight more accurately.

Sometimes the effective weight of the cell may actually be less than the one obtained by the procedure indicated here. This happens when the bottom and walls of the cell are clearly impervious. The excess uplift may be as much as double the amount indicated before (now, with an horizontal groundwater table inside the cell at elevation h_w above the sea level). Engineers should study the possibility of this situation occurring.

4.5.3.1.2 Embedded Foundations

Sheetpile enclosures founded on soils require driving the sheetpiles to a depth that should be decided as a function of several factors, including compliance with the specified requirements for structural behaviour (the subject of a different set of Recommendations in this ROM Programme).

One of the reasons for increasing the sheetpile driving depth may precisely be to increase the safety against sliding of the quay. This improvement, however, is subject to a limit.

If there is collaboration from the passive earth pressure in the whole of the embedment zone, it may happen that at a certain depth the unitary pressure is greater on the outside of the cell than on its inside. The walls of the cell would then cease to operate under tensile stress and the cell could become dangerously deformed.

The depth at which this may occur is difficult to estimate, since the state of stress inside the cell is not well known. The only known fact is that its exterior is subjected (in this failure mechanism) to the active earth pressure on the landward side and the passive pressure on the seaward side and the state of stress inside the cell should lie somewhere between.

From the possible ways of limiting this effect, it seems most adequate to consider only a certain depth t_0, below the level of the ground at the front face.

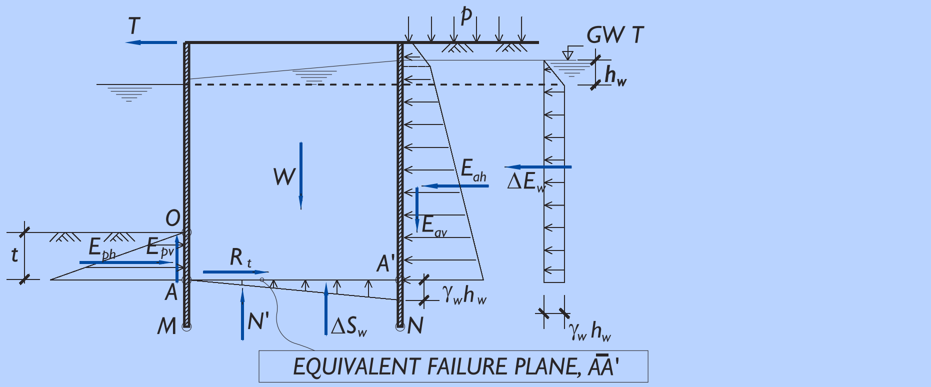
This depth t_0 would be the minimum embedment depth for a virtual diaphragm wall anchored at the free water level and subjected on its back face to the active earth pressure of the cell fill and, at its toe, to the passive pressure of the front face ground. This depth should be estimated as given in Subsection 4.4.5.1 for an anchored diaphragm wall.

As a result and in the absence of specific studies enabling a different design value to be adopted, an embedment depth should be taken for verifying safety against horizontal sliding that is no greater than:

a. The real embedded length.
b. The minimum embedment depth of a virtual diaphragm wall anchored at the water level on the seaward side and necessary to withstand the active earth pressure on its back face, along with any possible vertical loads acting on the enclosure.

Once the plane of sliding has been defined, the calculation procedure will be similar to the other cases of plane slide already analysed. Details of the calculation are illustrated in Figure 4.5.5.

Figure 4.5.5. Safety Factor against Sliding in Embedded Sheetpile Cells



POINT A t = THE LESSER OF t_l AND t_2

t_l = REDUCED EMBEDMENT DEPTH (see text)

t_2 = OM

POINT A´ THE HIGHEST OF THE FOLLOWING TWO:

a) REAR SHEET PILE TOE

b) INTERSECTION OF THE HORIZONTAL LINE THROUGHT "A" AND THE REAR SHEET PILE

$$F = \frac{N' \tan\phi + c \cdot \overline{AA}'}{R_t}$$

N' = normal effective component $\overline{AA}$'.

R_t = tangencial component of the resultant along $\overline{AA}$'.

The factor of safety against sliding will normally increase with the depth of the plane of slide, *t*. If this is not the case, the possible reduction in the embedment that was previously recommended should not be carried out.

Checking that safety against sliding increases with the depth of embedment, as assumed in the calculations and as is normally the case, can be carried out by repeating some calculations or as indicated in the following comment, which will only be valid for slides along horizontal planes.

Comment: *When the forces acting are as shown in Figure 4.5.5, there is a simple analytical expression for the safety factor as a function of the embedment depth,* t.
The derivative of F *with respect to* t *must be greater than zero, so that the safety factor increases with the depth of slide. This will occur when the* F *value obtained in the calculation is less than the critical value defined by:*

$$F_{(critical)} = \frac{\left(\gamma'_c B + \gamma'_a H^* K_{ah} \cdot \tan\delta_A - \gamma'_p\, t\, K_{ph} \cdot \tan\delta_p\right)\tan\phi + B\dfrac{\partial c}{\partial t} + \gamma'_p K_{ph} \cdot t}{\gamma'_a H^*\ K_{ah} + \gamma_w h_w}$$

where:

B = *design equivalent width of the quay (see 4.5.1), (m).*

H^* = *equivalent height of the cell for the purpose of this calculation, obtained from the expression*

$$H^* = \frac{\sigma'_{vo}}{\gamma'_c},\ (\mathrm{m})$$

σ'_{vo} = *effective vertical stress in the rear toe of the cell at the level of the slide plane, (kN/m²)*

K_{ah}, K_{ph} = *horizontal component of the earth pressure coefficients.*

δ_A, δ_p = *inclination of the earth pressures that will both be positive when they have the usual calculation inclination, i.e., with the active pressure pushing downwards and the passive earth pressure lifting upwards.*

$\partial c/\partial t$ = *variation in cohesion with depth (kN/m³)*
γ_w = *specific weight of the water (kN/m³).*
$\gamma'_c, \gamma'_a, \gamma'_p$, = *submerged unit weight of the cell fill, the backfill (in the plane of slide) and the ground on the seaward side (on the same plane) respectively, (kN/m³)*
h_w = *difference in water level between the landward and seaward sides (m).*

In anomalous cases where the safety factor is found to decrease with embedment depth, the calculation for plane sliding must be carried out for several depths until the elevation of the sheetpile toe is reached, in order to ascertain the level at which the minimum safety factor occurs.

If the safety factor continues to decrease, even when the line of plane slide is assumed to pass through the tip of the sheetpiling, special care should be taken to check that the overall equilibrium of the quay fulfils the appropriate requirements.

4.5.3.2 Verifying Safety against Bearing Failure

The bearing failure of the sheetpile cell and its fill, as a rigid body, can be analysed as if it were a shallow foundation on a horizontal plane passing through the toe of the sheetpiles on the seaward side. This recommendation will not apply if the sheetpiling rests on rock.

If the rear sheetpiles (landward side) do not reach this depth, for the purpose of this analysis of safety against bearing failure it can be assumed that they do reach this level and that there is an active earth pressure acting on them that corresponds to the entire height of the quay, from its crest to the horizontal plane where bearing failure is analysed.

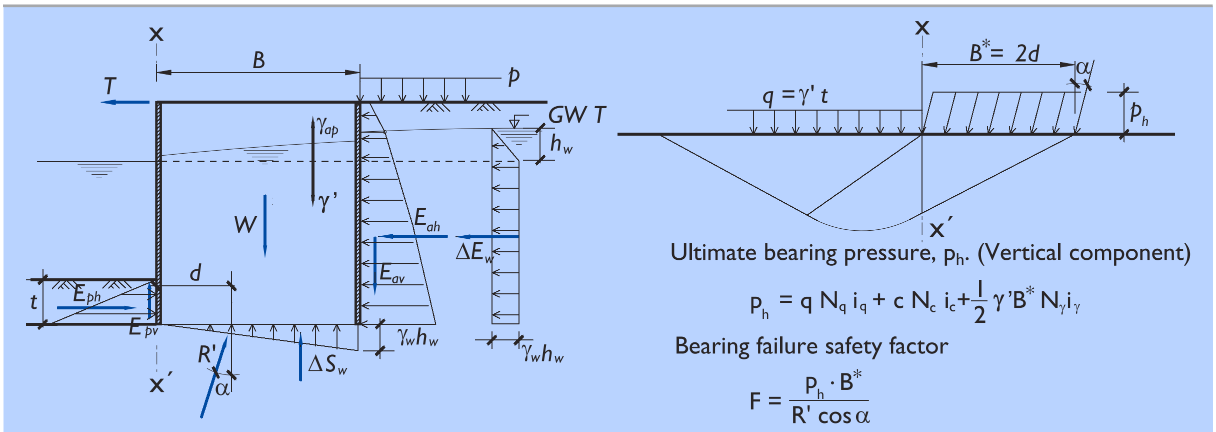
For enclosures embedded deep in the ground, a smaller embedment like the one indicated in Section 4.5.3.1.2 should be assumed for the purpose of these calculations.

Safety against bearing failure will normally increase with the depth of embedment assumed in the calculations. This should be checked and, if the bearing failure safety were to decrease as the depth of the calculation plane increases, the embedment reduction from the previous paragraph must not be applied, but instead the one leading to a minimum safety factor.

This potential anomaly also requires analysing the overall equilibrium of the quay carefully.

Figure 4.5.6 shows the calculation procedure for the factor of safety against bearing failure.

Figure 4.5.6. Evaluation of Safety against Bearing Failure



N.B.: See text for establishing the embedment depth, *t*, to be used in the calculation.

As this type of failure does not necessarily imply any relative vertical displacement in the back face, it should be assumed in the calculations that the active pressure is acting horizontally, unless a different assumption, with a positive angle of inclination (downwards) and less than 2/3ϕ, leads to a lower safety factor.

4.5.3.3 Verifying Safety against Overturning

Sheetpile enclosures often exhibit a distinctive failure along a curved surface close to their base, which develops inside the cell fill itself and passes through the toe of the sheetpiles. The centre of curvature of these lines would be situated below the base of the cell.

If the natural ground is loose, with similar or less strength than that of the fill in the cells, this failure line may curve in the opposite direction. It then develops inside the natural ground and will have a centre of curvature above the base of the cell.

These failure modes will be referred to here as overturning, since they involve a certain rotation of the cell as a rigid body and because they should be distinguished from the plane slide studied in the previous section.

The following forces should be considered as stabilising, counteracting the potential overturning:

- Self-weight of the quay wall corresponding to a fill with a virtual breadth of *B*.
- Passive earth pressure on the seaward side.
- Possible overburdens on the cells and other stabilising loads.

The following overturning forces will contribute to failure:

- Active earth pressure due to backfill and to overburdens.
- Water pressure due to water level difference between the backfill and the front face.
- Seaward horizontal forces transmitted to the enclosure.
- Potential loads and other destabilising actions.

Mode 1 Overturning

The passive earth pressure in the embedded zone on the seaward side is an important stabilising force and, therefore, increasing the embedment depth of the sheetpiles will generally be a measure that ensures stability.

Beyond a certain depth, however, considerable stresses may appear in the bottom of the sheetpiles and, unless special measures are taken and their effectiveness demonstrated, a reduced embedment depth should therefore be taken for calculation purposes as indicated in Subsection 4.5.3.1.2.

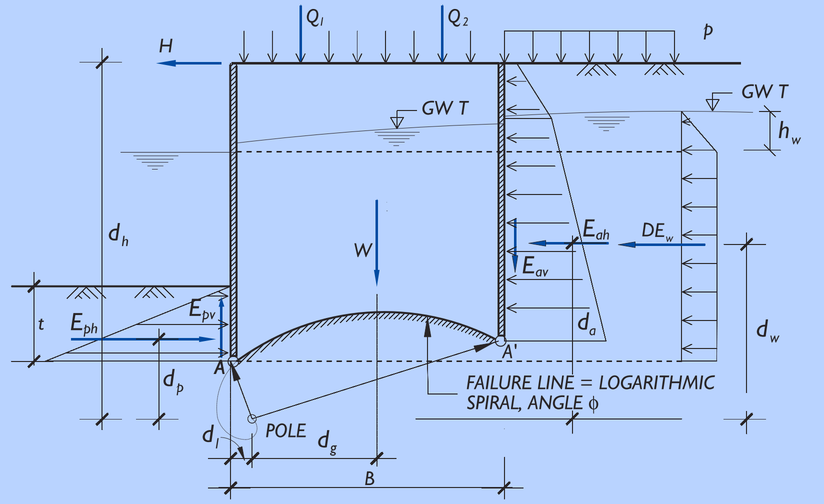
The method of calculating the safety factor for this type of overturning, referred to as Mode 1, is illustrated in Figure 4.5.7.

Mode 2 Overturning

In cases where the real embedment depth is greater than the one used in the preceding calculation (Mode 1 Overturning), and in cases where the natural ground has less strength than that of the cell fill, a further overturning failure mode should be checked, referred to as Mode 2 and illustrated in Figure 4.5.8.

In both cases, the failure line is a logarithmic spiral with an angle, ϕ, equal to the angle of friction along the failure line. This type of mechanism has the basic advantage of simplifying calculations, since in purely frictional soil (c = 0) the resultant of the loads acting against the ground (as that of the ground reactions) must pass through the pole and therefore give a null moment about it when failure occurs.

Figure 4.5.7. **Factor of Safety against Overturning, Mode 1**



$$F = \frac{M\ (\text{stabilising})}{M\ (\text{overturning})} = \frac{M_I}{M_2}$$

Stabilising moments

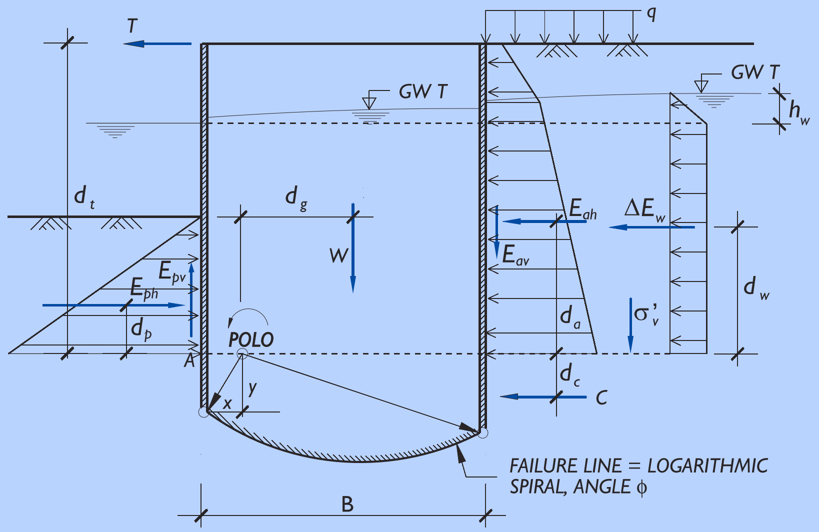
$$M_I = W \cdot d_g + E_{ph} \cdot d_p + E_{pv} \cdot d_l + (\text{cohesion - uplift})\ \text{effect}$$

Overturning moments

$$M_2 = E_{ah} \cdot d_a - E_{av} \cdot (B - d_l) + \Delta E_w \cdot d_w + H \cdot d_h$$

Note: See text for the cohesion-uplift effect".
The effective weights, whose resultant is *W*, should be calculated using the specific weights shown in Figure 4.5.4.
The moment about the pole of other potential loads should be considered as overturning or stabilising depending on whether they tend to stabilise or overturn the cell. See Figure 4.5.5 for the position of points A and A'.

Figure 4.5.8. **Factor of Safety against Overturning, Mode 2**



$$F = \frac{M\ (\text{stabilising})}{M\ (\text{overturning})} = \frac{M_I}{M_2}$$

Stabilising moments

$$M_I = W \cdot d_g + E_{ph} \cdot d_p + E_{pv} \cdot x + C \cdot d_c + (\text{cohesion - uplift})\ \text{effect}$$

Overturning moments

$$M_2 = T \cdot d_t + E_{ah} \cdot d_a - E_{av} \cdot (B - x) + \Delta E_w \cdot d_w$$

Counter-reaction

$$C = K_{ph} \cdot \sigma' \cdot y_v$$

Note: See text for "cohesion-uplift effect". The effective weights, whose resultant is *W*, should be calculated using the specific weights shown in Figure 4.5.4.
The moment about the pole of other potential loads should be considered as overturning or stabilising depending on whether they tend to stabilise or overturn the enclosure.

As a general rule, the moment about the pole produced by the stabilising loads will be greater than the one produced by the destabilising forces and only an increase in the latter can lead to the failure condition.

The safety factor can be calculated, for a particular failure line defined by its pole, as the coefficient by which the overturning forces (or their moments, since the arms are fixed once the failure line is defined) must be multiplied to cancel out the moments about the pole, as shown in the figures referred to.

The safety factor obtained will not be very sensitive to the location of the pole, whose most unfavourable position, however, should be sought by successive trial lines.

Comment: For making it easier to analyse stability along spiral failure lines, several curves of this type are shown in Figure 4.5.9. The loci of the poles of different logarithmic spirals passing through two fixed points are shown in Figure 4.5.10.

Figure 4.5.9. Logarithmic Spirals for Different Angles of Friction

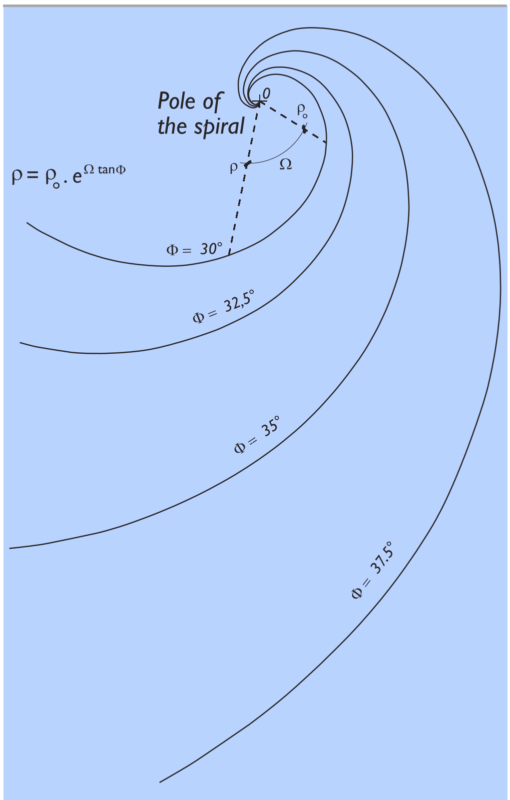
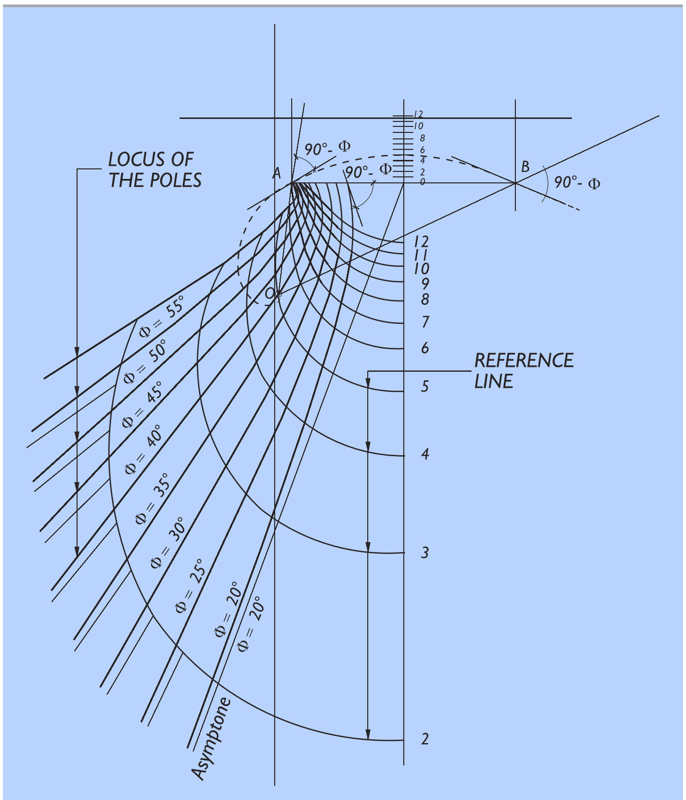


Figure 4.5.10. Loci of the Poles of Logarithmic Spirals (3)



To make stability calculations easier, some practical formulae are included here that may be of assistance.

To consider the potential favourable effect due to cohesion in cases where the failure line intersects cohesive soil, the value of this moment is:

$$M_{(cohesion)} = \frac{1}{2} c \cdot \cot an\phi \left(\rho_1^2 - \rho_0^2\right)$$

(3) Tomado de R. Jelinek, H. Ostermayer. Zur Beruchnung von Fandedämmer und verankerten Spundwänden. Bautechnik. Mai 1967.

where:
c = *cohesion.*
ϕ = *angle of the spiral.*
ρ_o, ρ_1 = *polar radii of the ends of the cohesive zone.*

And the area enclosed between the logarithmic spiral and the chord linking its two end points is:

$$\text{Area} = \frac{1}{4\tan\phi}\left(\rho_1^2 - \rho_0^2\right) - \frac{1}{2}\rho_0\ \rho_1\ \sin\Omega$$

where:
Ω = *angle formed between the two end polar radii* ρ_0 *and* ρ_1.

and where the other parameters have the same meaning as in the previous paragraph..

Uplift on the failure line, A-A', is for the most part taken into account when considering the stabilising weights as submerged under the water level on the seaward side. The excess uplift on the failure line will decrease between a value of $\gamma_w \cdot h_w$ *in the back face and zero in the front face. The effect of this additional uplift can be taken into account by reducing the stabilising moment by the following amount:*

$$M_{(uplift)} = \frac{1}{3}\gamma_w h_w\left(\rho_1^2 - \rho_0^2\right)$$

This is an approximate formula and if this moment is important, a more adequate calculation method will need to be used.

4.5.3.4 Verifying Safety against Cell Failure

Sheetpile enclosures for quay walls need to transmit the loads acting on their backfill and at their head towards the foundation ground with the least eccentricity and least inclination possible.

The body of the cells will be subjected, as a structure, to stress resultants (from integrating the stresses on different planes intersecting it), which must be withstood jointly by the fill and sheetpiles confining it.

The shear force along horizontal planes is relatively easy to calculate. In the same way as was shown in previous sections for horizontal planes close to the base of the enclosures, this can be done for horizontal planes at higher elevations. Failure along these horizontal planes will not only be governed by the cell fill, but also by the shear resistance of the sheetpiles themselves. This is a mechanism that must be evaluated in the context of the structural calculation of the walls.

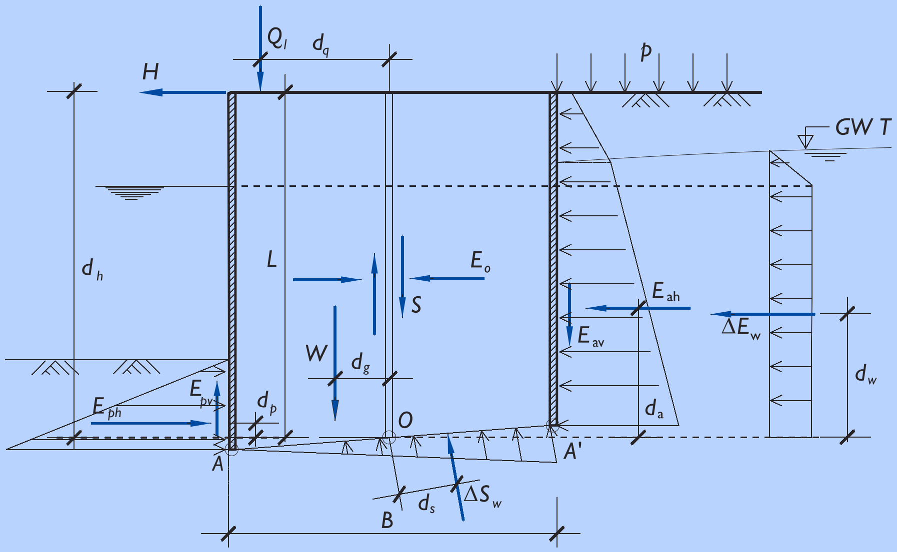
On the central vertical plane, equidistant from the two faces enclosing the cells, there is a shear force, S, and a normal effective compression, E_0, which must be estimated to check that they are far from exceeding the structural capacity of the cells.

The contribution of the sheetpiles to the cell resistance is small in this type of hypothetical shear, since it involves a slide along the sheetpile vertical connection in the middle cell zone, where the tensile stress must not be very high (in enclosures bounded by parallel walls, there are no sheetpiles to be sheared in this central plane).

Neglecting the contribution of the structural resistance to sliding at these connections, the safety factor against vertical shear can be defined as shown in Figure 4.5.11.

The resultant of the loads on the base of the cell will produce an overturning moment M_o about its centre O. This is assumed to be withstood by a pair of forces with an intensity S and an arm of $2B/3$. The centred vertical loads will be borne by uniform reactions that will not generate shear stresses in the vertical plane under study.

Figure 4.5.11. **Safety Factor against Cell Failure**



$$F = \frac{E_o \tan\phi + c \cdot L}{S}$$

The following values should be used:

$$E_o = E_{ah} + \Delta E_w \geq \frac{1}{2} K_o \, W \frac{L}{B}$$

$$S = 1.5 \frac{M_o}{B}$$

where:

$$M_o = E_{ah} \cdot d_a - E_{ph} \cdot d_p - (E_{av} + E_{pv}) \frac{B}{2} + \Delta E_w \cdot d_w + \Delta S_w \cdot d_s + H \cdot d_h + Q_l \cdot d_q + W \cdot d_g$$

After filling the cells and before proceeding to place the backfill, there will be some horizontal effective compression stresses in the fill inside the cells that can be evaluated, for this purpose, as:

$$\sigma'_H = K_o \, \sigma'_v$$

which leads to an estimate of the effective compression in the plane under study:

$$E_o = K_o \int_o^L \sigma'_v \, dz$$

This integral, which extends over the plane in question, can be estimated by assuming a linear variation of the pressures with depth:

$$E_o = \frac{1}{2} K_o \cdot \sigma'_{v(max)} \cdot L = \frac{1}{2} K_o \frac{L}{B} \cdot W$$

This E_0 value is the one shown in Figure 4.5.11. For sand fills, it is normally assumed that $K_0 = 0.4$, which appears to be a lower limit of the vales found in practice.

The cell is compressed by the active earth pressure of the backfill, due to its weight and to existing surcharges, and by the water pressure due to difference in groundwater table levels between the landward and seaward sides. The sum of these thrusts will be similar to the effective compression in the plane under study. Part of this horizontal compression is directly transmitted to the foundation ground but, since the back of the foundation is less heavily loaded, it is assumed that this effect is negligible. The value for the upper limit of E_0, as shown in the figure, was defined on this assumption.

The simplified method here recommended considers this value for the effective compression, provided that it is greater than the one existing prior to backfilling and loading of the quay.

In the absence, therefore, of a more suitable procedure that engineers can justify, the safety factor against vertical shear should be taken as the one defined in Figure 4.5.11.

The embedded length in the front toe should be limited as indicated in Subsection 4.5.3.1.2 for the purpose of carrying out these calculations, unless special arrangements are used.

4.5.3.5 Verifying Safety against Local Instability of the Toe

The area around the front toe of sheetpile cells for quays walls can be particularly problem-ridden when the sheetpiles are not driven deeply.

Engineers must check that safety is sufficient, at least against the Limit States described below, in addition to any others they might conceive.

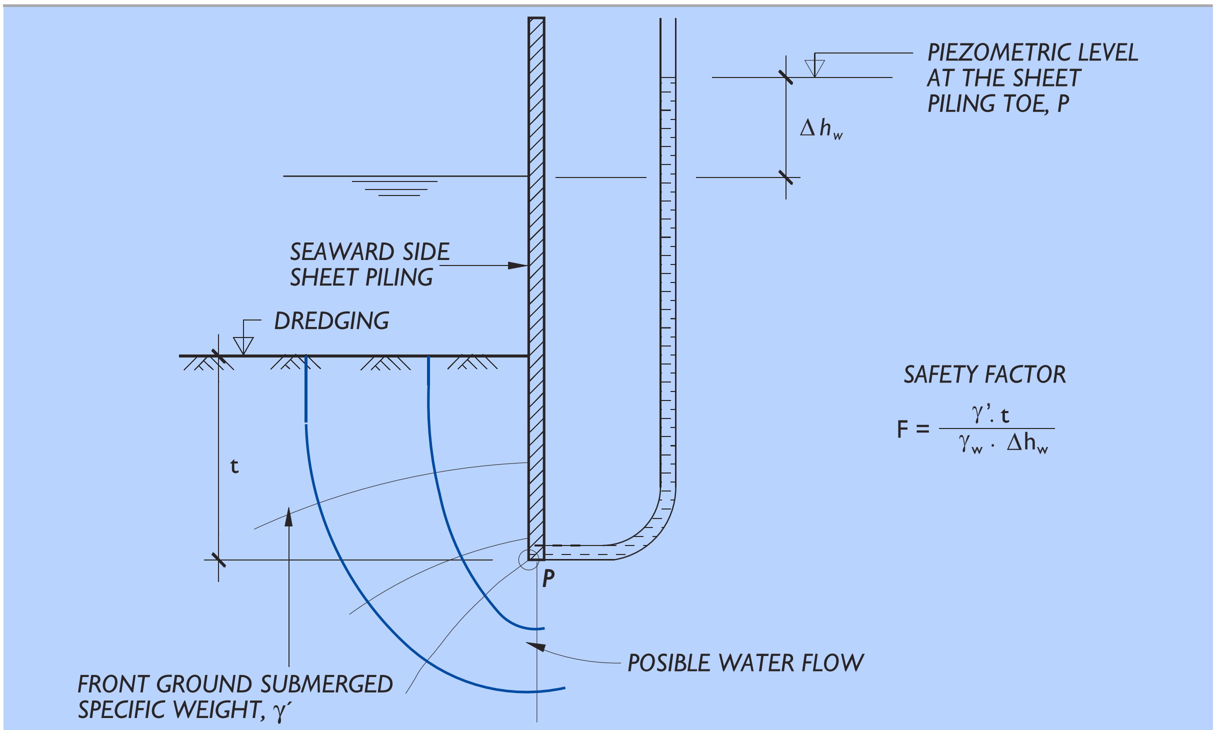
a. Checking Safety against Ground Heave at the Toe

There may be an upward water flow at the front of the quay caused, among other possible reasons, by:

- The effect of tides and waves.
- The effect of rain raising the groundwater table inside the cells.
- Artificial or natural water currents coming from the landward side.

For each design situation, engineers must estimate the maximum piezometric level difference between the toe of the sheetpiles on the seaward side and the dredging line. This is referred to as Δh_w in Figure 4.5.12.

Figure 4.5.12. **Safety against of Ground Heave at the Toe**



The safety factor for this failure mechanism is defined as the quotient between the critical gradient that would cause bottom heave failure and the estimated gradient:

$$I_{(crit)} = \frac{\gamma'}{\gamma_w}$$

where:

γ' = Submerged unit weight of the ground around the toe.
γ_w = Specific weight of the water.

and the estimated gradient:

$$I_{(estimated)} = \frac{\Delta h_w}{t}$$

that is:

$$F = \frac{I_{(critical)}}{I_{(estimated)}}$$

In defining safety in this way, it is assumed that the ground through which the water flows has uniform permeability between point *P* (sheetpile toe) and the seabed surface adjacent to the quay. If this is not the case, the design point *P* should be taken as the point where the enclosure contacts the base of the most impervious stratum that the sheetpiles pass through.

b. Verifying Safety against Toe Plastification

The state of stress inside the sheetpile enclosure close to its front toe can be estimated with the help of the results of the calculations carried out to check safety against bearing failure. In fact, if the resultants of the effective actions on the foundation, with respect to the centre of the cell base, are:

N_o = resultant normal effective compression in the cell base.
H = horizontal component of the resultant.
M_o = moment of the above forces about the centre of the cell base.

The upper bound of the vertical stress near the front toe can be estimated by assuming a linear distribution of stresses, i.e.:

$$\sigma'_{v1} = \frac{N_o}{B} + \frac{6M_o}{B^2}$$

The shear stress can also be estimated. Unless a more adequate procedure is available, it can be assumed to be:

$$\tau = \sigma'_{v1} \cdot \frac{H}{N_o}$$

In the Ultimate Limit State under consideration, the ground will be plastified at this point and this will enable the stresses to be calculated that would exist on a vertical plane, called σ'_H and τ in Figure 4.5.13.

The same stresses, σ'_H and τ, should exist on the vertical plane in the front zone (seaward side). An effective vertical stress σ'_{v2}. will act on a horizontal plane.

The value of σ'_{v2} can be estimated as:

$$\sigma'_{v2} = \gamma^*_2 \cdot t$$

In this case, the design unit weight must be taken as the submerged value, reduced by the effect of a potential upward hydraulic gradient, i.e.:

$$\gamma^*_2 = \gamma'_2 - I_v \cdot \gamma_w$$

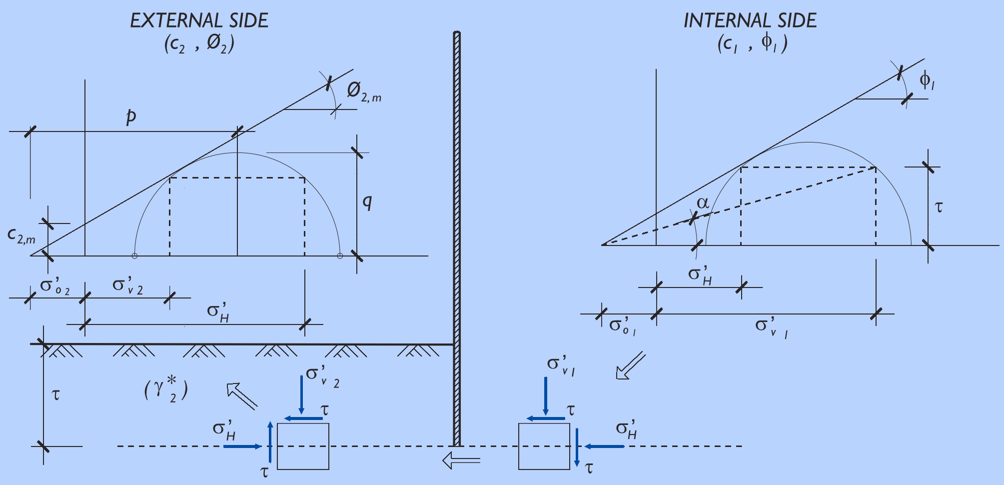
Once the stresses at the toe of the diaphragm wall are known, it is possible to obtain the strength necessary for this zone to be in strict stress equilibrium ($c_{2,m}$, $\phi_{2,m}$).

Comparison of the ground strength close to the sheetpiles (c_2, ϕ_2) in the front toe of the quay with the one needed for the strict stress equilibrium will enable a safety factor to be defined as follows:

$$F = \frac{\tan\phi_2}{\tan\phi_{2,m}} = \frac{c_2}{c_{2,m}}$$

Figure 4.5.13 shows some analytical formulae that can facilitate the calculation associated with this check on toe plastification.

Figure 4.5.13. Stress Check around the Front Toe of a Sheetpile Cell



Initial data. Internal side

$$\sigma'_{v1}, \tau$$

$$\sigma'_{o1} = c_1 \cdot \text{cotan}\,\phi_1$$

$$\tan\alpha = \frac{\tau}{\sigma'_{v1} + \sigma'_{o1}}$$

$$\sigma'_H = \frac{1 + \sin^2\alpha - \cos\alpha\sqrt{\cos^2\alpha - \cos^2\phi_1}}{\cos^2\alpha + \cos\alpha\sqrt{\cos^2\alpha - \cos^2\phi_1}}\left(\sigma'_{v1} + \sigma'_{o1}\right) - \sigma'_{o1} \qquad \text{if } \phi_1 > 0$$

$$\sigma'_H = \sigma'_{v1} - 2\sqrt{c_1^2 - \tau^2} \qquad \text{if } \phi_1 = 0$$

Situation on the external side

$$\sigma'_{v2} = (\gamma' - I_v \cdot \gamma_w) \cdot t \qquad \sigma'_{o2} = c_2 \cdot \text{cotan}\,\phi_2$$

$$q = \frac{1}{2}\sqrt{4\tau^2 + (\sigma'_{v2} + \sigma'_H)^2} \qquad p = \frac{1}{2}\left(\sigma'_{v2} + \sigma'_H + 2\sigma'_{o2}\right)$$

Safety factor

$$F = \frac{1}{q}\sqrt{p^2 - q^2} \cdot \tan\phi_2 \qquad \text{if } \phi_2 > 0$$

$$F = \frac{1}{q} \cdot c_2 \qquad \text{if } \phi_2 = 0$$

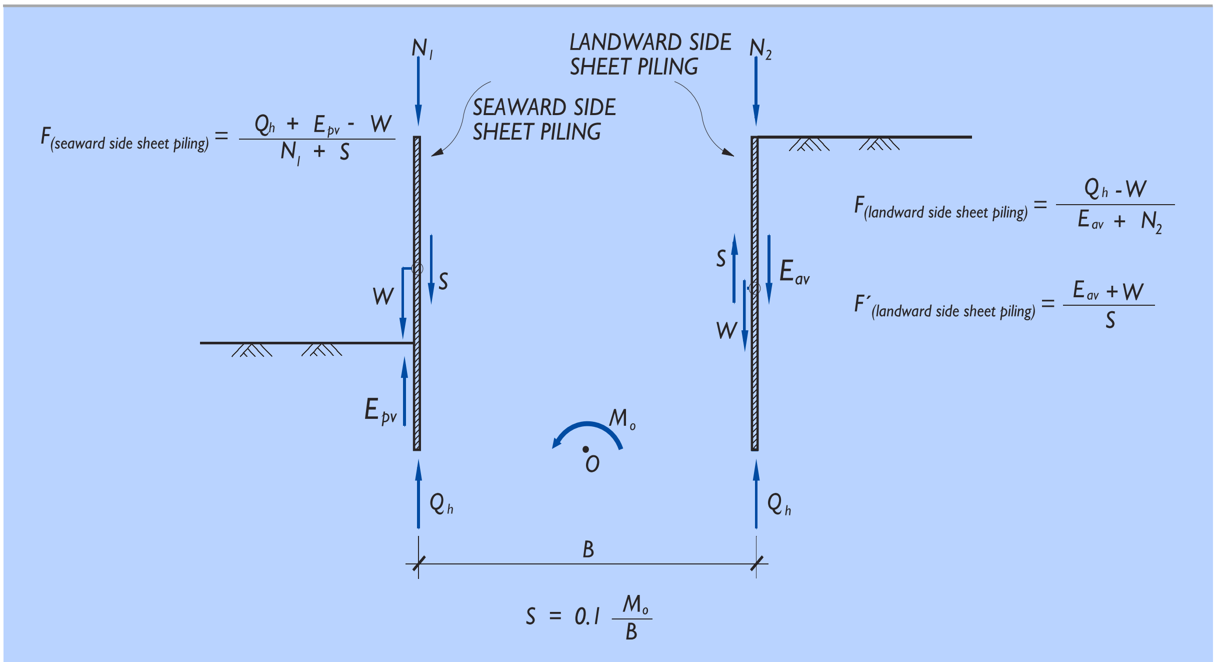
It should be noted, however, that in quays with parallel diaphragm walls not braced at their bottom, failure of the toe caused by horizontal earth pressure should be analysed by the general procedure of anchored diaphragm walls, as indicated in Subsection 4.4.5, in order to obtain the minimum driving depth necessary.

4.5.3.6 *Verifying Safety in Respect of Sheetpile Vertical Equilibrium*

Sheetpiles are subjected to shear forces on their faces and to potential vertical loads on in their heads, so that bearing failure or vertical heave could occur.

The forces that can act vertically per linear metre of sheetpile are shown in Figure 4.5.14.

Figure 4.5.14. Safety with Respect to Sheetpile Vertical Equilibrium



N.B.: M_o = moment of the forces acting per linear metre of quay about the centre of the foundation plane.

Seaward Side Sheetpiles

The sheetpile on the landward side can be loaded with some vertical force on its head, designated N_1, in addition to the possible vertical component of the earth pressure of the cell fill, designated *S*.

For the purpose of this check, it should be assumed that *S* is such that the sheetpiles collaborate, through their vertical structural capacity, in withstanding a small portion of the enclosure's flexural bending. Figure 4.5.14 puts this collaboration at some 10% of the total.

On the seaward side, the vertical component of passive earth pressure estimated in previous calculations can be counted on as a resistant vertical force. The tip bearing capacity –estimated as indicated in Section 3.6 of this ROM 0.5– should be added to the resistant force and the weight of the sheetpiles subtracted from it.

The safety factor against vertical bearing failure for seaward-side sheetpiles is defined as the quotient:

$$F = \frac{Q_h + E_{pv} - W}{N_1 + S}$$

Landward Side Sheetpiles

The sheetpile on the landward side will be mainly subjected to the vertical load due to earth pressure and surcharges, as shown by force E_{av} in Figure 4.5.14. It will also be subject to forces due to the friction of the fill in the cells against its side, i.e., force *S* in the figure referred to.

This force *S* can vary, and will be in a downward direction during construction and upwards in some load combinations that involve substantial bending of the enclosures.

For the purpose of this check, two safety factors should be calculated with respect to vertical displacement. One, under the assumption that this will occur downwards (bearing failure), is defined by:

$$F = \frac{Q_h - W}{E_{av} + N_2}$$

in which no account is taken of any force *S*. The other factor is obtained by assuming that the sliding occurs upwards, in which case the estimated force *S* must be withstood by the friction between the sheetpiles and the backfill, i.e.:

$$F = \frac{E_{av} + W}{S}$$

In cases where other potential vertical forces intervene in the process, the above equations will have to be modified on the same principles that guided their formulation.

4.5.3.7 Verifying Safety against Loss of Overall Equilibrium

The overall equilibrium of sheetpile enclosures in quays is essentially the same as that of gravity quay walls.

The recommendations given in Subsection 4.2.3.5 apply to sheetpile enclosures for quays.

4.5.3.8 Verifying Safety against Internal Erosion

Internal erosion, with fine particles migrating from the fill of the enclosures or the quay backfill, can occur due to water flow, regardless of whether this is natural and pre-existing or created by the presence of the quay itself.

The possible water flow inside the cell fill can bring about entrainment of fine materials passing through defects in the walls or else through the cell bottom.

It is advisable to inspect the enclosures before they are filled to prevent the cell fill being lost as a result of some defect in the joints.

For the purposes of avoiding entrainment through the bottom, a granular material should be arranged as the first fill layer in contact with the bottom, capable of serving as a filter between the natural ground and the bottom of the cells.

The recommendations in Subsection 4.4.5.7 relating to diaphragm wall quays should be followed for the backfill of the cells.

4.5.3.9 Verifying Safety against Scour

The recommendations in Subsection 4.4.5.8 about diaphragm walls in quays should be followed to prevent scour at the front toe.

4.5.3.10 Summary of the Minimum Safety Factors

The minimum recommended safety factors for the different failure modes considered in the preceding sections are shown in Table 4.5.1.

Table 4.5.1. Minimum Safety Factors for Sheetpile Cellular Structures in Quays. Works with Low SERI (5 - 19)

Failure Mode	Load Combination		
	Quasi-Permanent, F_1	Fundamental, F_2	Accidentales or seismic, F_3
Sliding	1.5	1.3	1.1
Bearing failure	2.5	2.0	1.8
Overturning	1.5	1.3	1.1
Cell failure	1.5	1.3	1.1
Local instability at the toe			
a) Heave	1.5	1.3	1.1
b) Plastification	1.5	1.3	1.1
Loss of vertical equilibrium of sheetpiles	2.5	2.0	1.8
Loss of overall equilibrium	1.4	1.3	1.1

Note 1: Before using these safety factors, it is necessary to master the associated calculation methods defined in this ROM 0.5, described in Section 4.5.
Note 2: Depending on the nature of the works and the duration of the design situation, the modifications covered in Subsections 3.3.8 and 3.3.10 should be carried out for increasing or decreasing the recommended safety factors.

4.5.4 Ultimate Limit States of Structural Failure

Structural analysis of sheetpile enclosures for quays should be aimed at deciding on the structural capacity of the sheetpiles, among other aspects.

At each point of the sheetpiling and at each construction stage, different stresses will be acting that will also vary between different types of quay.

In quay walls composed of circular enclosures or enclosures with diaphragm cells, experience has shown that the maximum compression of the fill against the sides is proportional to the vertical effective compression, over a large part of the height of the cells.

The proportionality coefficient, K, is in the order of 0.4 to 0.6 during construction, as far as is known from the few cases from which measurements are available. In addition, this coefficient appears to decrease in the deeper zones, with the result that the maximum horizontal pressure is reached at a depth $z = 3/4H$ in quays supported by rock (cells not embedded) or at the dredging level, i.e. for $z = H$, in partially embedded cells.

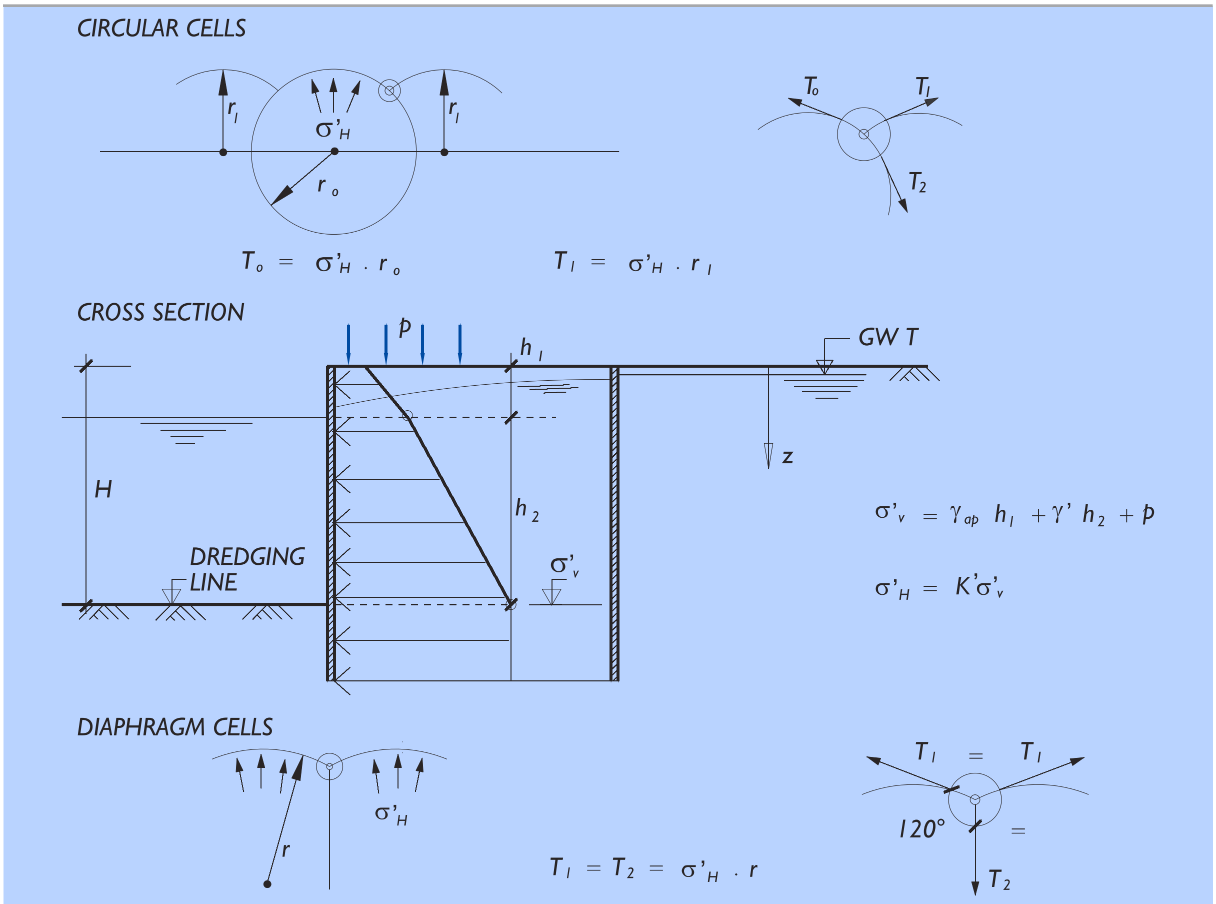
From published experience, it appears that coefficient K decreases somewhat over time, perhaps by 10 or 20%, in the middle and top zones of the cells and increases slightly by a similar amount in the bottom cell zone.

When the quay backfill is placed, the pressures at the two sides of the cell walls tend to balance. Thus, tensile stresses in the walls are reduced except on the seaward side, where the walls continue to be subjected to similar tension.

The horizontal tension between sheetpiles is usually calculated by the simple formula for thin-walled tubes shown in Figure 4.5.15.

In addition to the main horizontal tensile force, there is secondary bending. This produces vertical tension as well as shear forces (see Subsection 4.5.3.4), which must be taken into account in the structural design.

Figure 4.5.15. ***Diagram of Tensile Forces in Walls***



In enclosures composed of two parallel, tied rows of sheetpiles, the steel elements basically work by flexural bending and their structural analysis should be similar to the one that can be carried out in the case of anchored sheetpile quay walls.

Although the structural problem is a key issue in this type of quay, little progress has been made in real knowledge of the stress state in sheetpiles and ample safety factors are therefore usually employed.

In any event, as mentioned, structural design lies outside the scope of this ROM 0.5 and will be the subject of future publications under the ROM Programme.

4.5.5 Serviceability Limit States

Sheetpile enclosures are more deformable than other gravity quay walls (blockwork or caisson). It will sometimes be necessary to verify the limit state of deformation, for which purpose some recommendations are given below.

The movement of a sheetpile cell can be broken down into two parts, one due to the foundation ground and the other due to the cell fill.

Movements due to deformability of the foundation ground (settlement, horizontal displacement and rotation) can be estimated by following the indications relating to gravity quay walls (see Subsection 4.2.4).

Additional movements caused by the deformability of the enclosures can be calculated by assimilating them to a shear beam [4].

At a particular height, *y*, measured from the base, there is a horizontal shear force *Q* in the cell that can be estimated by assuming that the earth pressure on the landward side is active and that on the seaward side only a fraction of the passive earth pressure is acting, since this pressure will only be mobilised in limit cases close to sliding failure.

The angular deformation occurring at level *y* will be:

$$\frac{\partial u}{\partial y} = \frac{Q}{B \cdot G}$$

where *B* is the breadth of the enclosure and *G* the shear modulus of the fill.

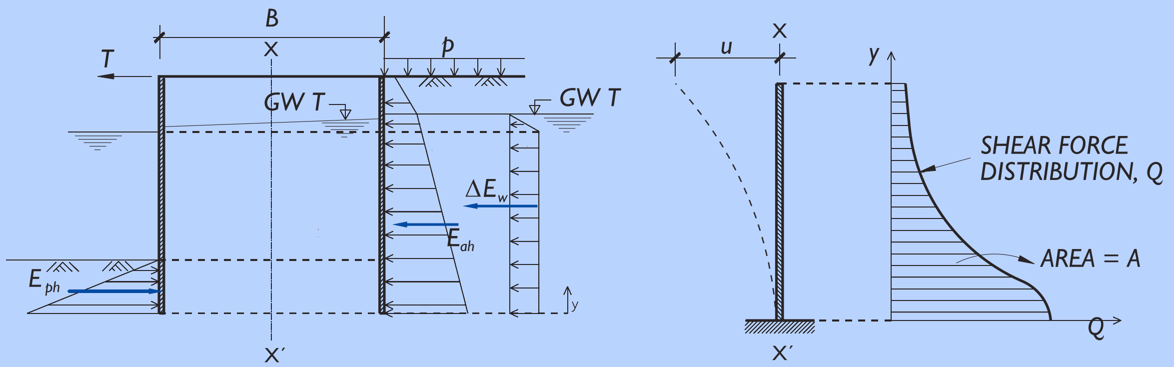
The effect of the cell walls in reducing deformation can be taken into account by increasing the product *B* · *G* in the amount corresponding to the contribution of the sheetpiles.

Integrating this distribution will enable the horizontal displacement of the quay to be calculated at any height.

$$u(y) = \int_{o}^{y} \frac{Q}{B \cdot G} \cdot dz$$

Figure 4.5.16 illustrates this calculation procedure, and shows the solution on the assumption that there is a constant modulus, *G*, throughout the cell height.

Figure 4.5.16. Diagram of a Simplified Deformational Calculation



DISPLACEMENT OF THE HEAD, u (metres)

$$u = \frac{A}{BG}$$

A = area of the shear force distribution (m·kN/m).
B = breadth of the quay (m).
A = shear modulus of the fill (kN/m²).

The horizontal movement of the wall head divided by the total height of the enclosure can amount to a fraction varying from less than 1% for quays with a small draught and very stiff cell fills, to several times that value in very high quays with poor quality fills.

(4) A linear element whose deformation is exclusively the result of shear forces.

As mentioned, these deformations must be added to those that may result from the foundation ground itself, assuming that the quay wall moves as a rigid block when calculating the latter.

If the result of the two deformations leads to head displacement of less than 1% of the wall height, it is recommended to increase the earth pressure (only for the purpose of calculating displacements) and instead of assuming that the pressure is active, to assume that it is closer to earth pressure at rest as recommended in Subsection 4.2.4.

4.5.6 Other Recommendations

From the geotechnical point of view and with no intention to be so comprehensive as future Recommendations of this ROM Programme may be, some ideas are now advanced to complement those given in preceding sections.

Preparing the Foundation Ground

The stability of sheetpile enclosure quays is mainly trusted to the ground resistance in the foundation zone. For this reason, preparation of the ground must be defined with great precision at the design stage and performed with maximum precautions.

In rocky foundations, all remains of clayey material capable of reducing the strength of the contact zone must be cleaned away from inside the cells.

In foundations of less firm soils, it could be worthwhile replacing or improving the topmost zone that has lower strength.

Dredging of the inside of the enclosures, when necessary, may require substantial temporary propping structures.

Cell filling can cause increases in pore water pressure, not only inside the cell but also in the foundation. It is extremely important that this is studied at the time of the design.

Cell Fill

The optimum cell fill is obtained from granular materials with maximum size not too large (some 30 cm at most), formed of unweatherable minerals, resistant and that above all are sufficiently permeable. It is not recommended to use earthfill with fines content (ASTM 200 / 0.080 UNE sieves) over 5% of the weight of the material that is less than 1" (25 mm) in size.

If it is necessary to use a different earthfill of poorer quality or to leave part of the natural ground inside the enclosure, the design must consider potential generation and subsequent dissipation of transient porewater pressures inside the enclosures. It may be advisable to take measures for accelerating the consolidation process.

In seismic zones and for enclosures to be filled with fine sand, the risk of liquefaction in cells should be studied. If this risk exists, the necessary precautionary measures should be taken (compaction by vibro-flotation as the fill is constructed, for example).

Quay Settlements

Sheetpile enclosures in quays may be subject to deformation that can affect installations in the vicinity.

Deformation should be estimated in such a way that reasonable measures can be taken to alleviate the effects of post-construction movements.

Monitoring

Visual monitoring of sheetpile enclosure quay walls is particularly valuable in view of the complex soil-steel interaction governing their behaviour.

To provide an appropriate monitoring system is advisable when designing quays of this type in order to measure the following parameters:

- Quay displacement, which can be monitored by simple surveying techniques.
- Porewater pressures in the fill and the foundations. Vibrating-wire piezometers can be used at the spots of most interest.
- Sheetpile stresses, which can be measured with suitable strain gauges.
- Changes in cell diameter. Long-base horizontal extensometers can be used for these measurements.

Even though they clearly fall within monitoring objectives, procedures for measuring total pressures based on pressure cells are not yet sufficiently well developed. This does not, however, mean that attempts should not be made to measure earth and water pressures on different points of the quay walls. Subsequent interpretation of the readings must be made with caution.

In line with the monitoring plan decided on for a particular quay, the reasonable range of values that each instrument may record must be known as the result of prior calculations to this purpose. No instrumentation system should be implemented without first predicting, albeit roughly, the results that should be obtained for each variable monitored.

4.6 DRY DOCKS AND LOCKS

The shipbuilding industry requires constructing large basins near to the coast capable of being drained for repairing or building vessels. These works generally involve a large volume of earthmoving and substantial dewatering operations plus the construction of very tall earth-retaining structures and base slabs on which vessels can stand and which will therefore have to bear considerable loads.

The problems associated with constructing dry docks are closely linked to geotechnical engineering. Solutions are normally governed by the strength, deformability and, mainly, permeability of the ground.

Navigation locks are similar structures that do not need to be fully drained, although they do have to withstand significant differences in water level as a result of their operation.

Navigation locks may not have a base slab (rocky bottoms) and require two or more gates for ships to pass through. They differ from dry docks in these and other respects.

4.6.1 Classification

The dominant problem in designing dry docks (and, to a lesser extent, in locks) is the effect of uplift. Dry docks can be classified into three types depending on how this problem is solved:

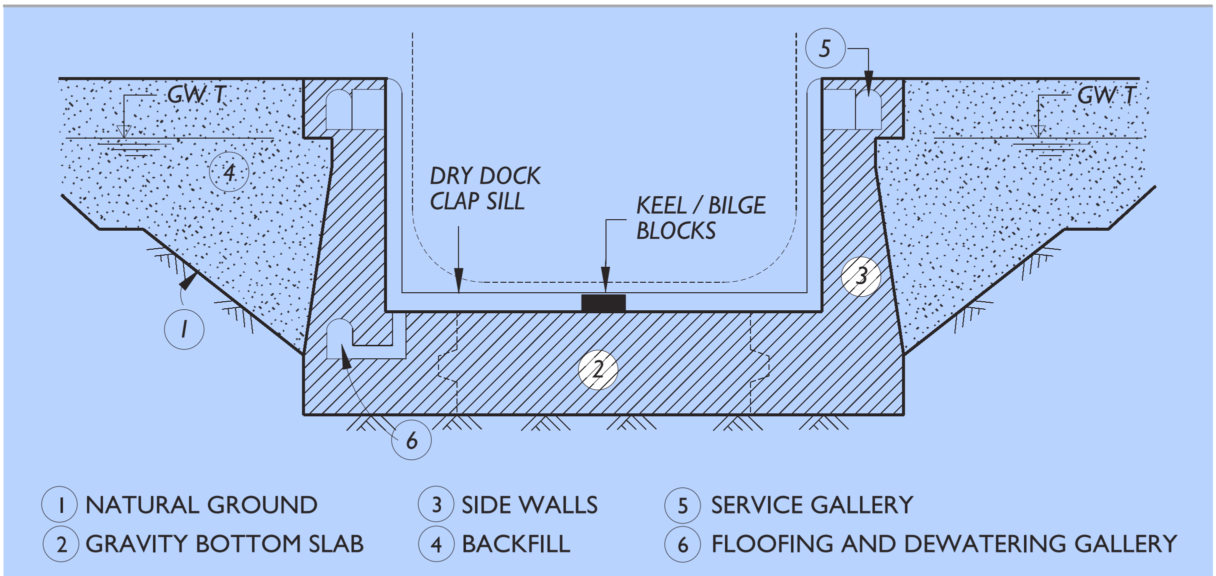
- gravity dry dock
- with drained base slab
- with anchored base slab.

There are obviously combination types where the uplift is principally withstood by gravity with the help of some anchors or drainage system.

These works are normally constructed in dry excavations protected by cofferdams and temporary dewatering systems, but they may also be constructed in previously dredged areas without dewatering. In this latter case, construction can be carried out either with underwater concrete or with precast elements. Dry excavation and construction is generally preferable and should always be considered as a solution to be studied.

The most characteristic elements of dry docks are depicted in Figure 4.6.1. In addition to the elements shown, mention must also be made of the pumping plant for flooding and emptying the dock, which usually requires substantial local excavation, and of the dry dock gate that transmits water pressures to the structure (walls and base slab) when the dock is dry. This creates significant loads on its contour.

FigurE 4.6.1. **Typical Elements of a Dry Dock**



Alongside dry docks, there will be major foundations required for the cranes employed to repair or build vessels. Some of these foundations may rest on the structure of the dry dock or close to it in a way that will condition its design.

4.6.2 Geotechnical Investigation

The geotechnical investigation for a dry dock should be more detailed than in the case of other harbour works on a similar scale. The consequences of a fault in geotechnical investigation for this type of works are generally more important than in others of comparable overall cost.

The primary aspect of designing a dry dock is the manner of counteracting uplift in the base slab. If the type of dry dock has not been decided when planning the geotechnical investigation, this investigation should be carried out taking into account the three possible solutions mentioned in the previous subsection.

In order to study the possibility of permanently draining the base slab and the possibility of carrying out the works in the dry, whatever their type, a hydrogeological study will be necessary. It should cover several kilometres around the works site in order to establish with certainty the water flow regime in the area surrounding the dry dock.

Proximity to the sea will accentuate the importance of seepage problems compared to those of other similar excavations carried out on land. Pumping tests to quantify dewatering discharge rates will be required for the hydrogeological and geotechnical studies. Such investigations should also look for karstic (or microkarstic) cavities, joints in rock and singularities that could bring water into the foundation under low gradients, particularly in dry docks with drained base slabs.

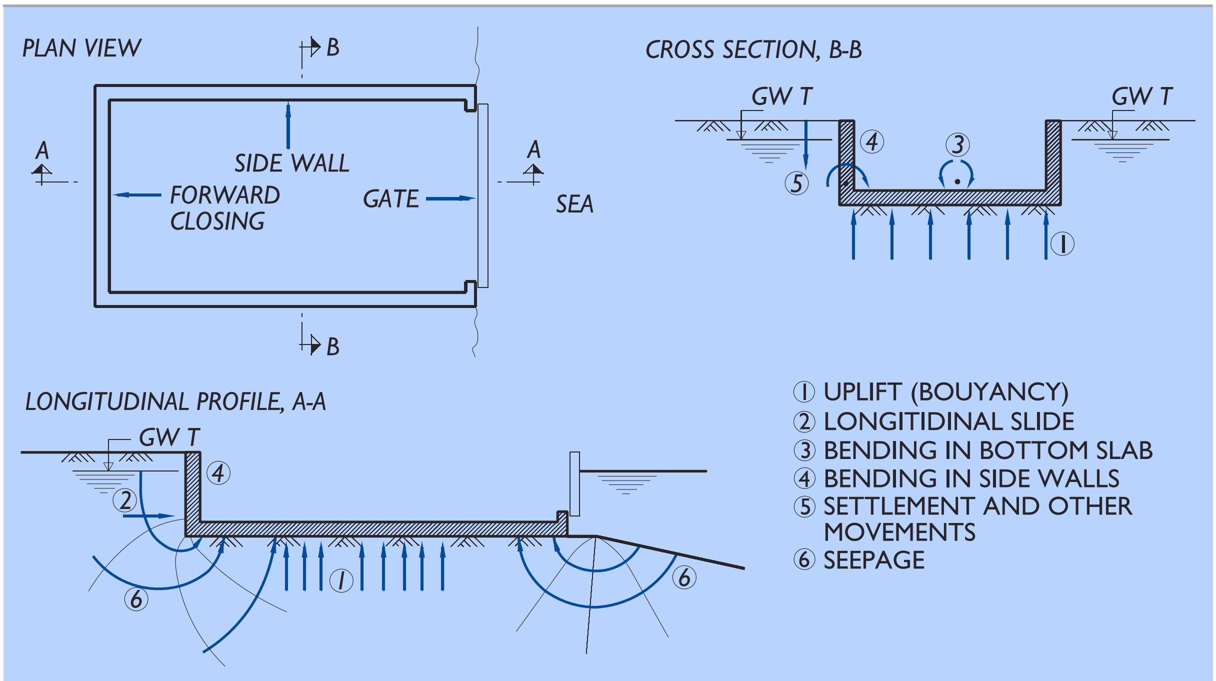
In order to analyse stability and deformation, a geotechnical investigation will be required along the side walls and in the base slab and forward closure areas. The entrance threshold area should also be taken into account, where the gate is supported, along with the pumping station site, since special construction arrangements may be required in these specific places.

General criteria are set out in Part 2 of this ROM 0.5 to provide guidance on geotechnical investigations. The following supplementary aspects should also be considered in this type of dry dock works.

- The analysis of pumping rates required during construction and in service will necessitate detailed sensitivity studies. The geotechnical investigation must provide realistic bounds to possible variations in the parameters governing this problem.
- Excavation of a dry dock or lock will cause a significant decompression in the underlying ground. Ground investigation should include a study of the parameters governing swelling, particularly if the bottom is marly or clayey (overconsolidated clays that could present substantial swelling).
- This type of works may require analysing ground treatments to increase its strength or, more frequently, to modify its permeability.
- The possibility of excavating (mechanically or by blasting) or dredging must also be the subject of geotechnical investigation.
- The investigation must reach sufficient depth to cover not only the hydrogeological aspect but also to evaluate the bearing capacity of possible deep or anchored foundations.
- Backfill for walls, potential structural fills (in precast solutions founded on berms) and the need for concrete aggregates will make it necessary to do a specific study on the source and quality of borrow materials.

Various matters associated with the study of dry docks are illustrated in Figure 4.6.2.

Figure 4.6.2. Some Issues Associated with the Analysis of Dry Docks



4.6.3 Gravity Dry Docks

The most widespread solution, particularly in narrow dry docks, is the gravity type. The weight of the base slab and the side walls (connected to it), plus the earth that can contribute to the weight of the side walls must prevent the dock from floating when emptied.

The most frequent problems governed by ground characteristics concerning this type of dry dock are dealt with below, with some general recommendations for each of them. Given that each project will have its own particular features, engineers must unveil every potential problem and treat each accordingly.

4.6.3.1 Verifying Safety against Dock Flotation Due to Uplift

With gravity base slabs, it must be assumed that the maximum uplift will act over the whole of the bottom face without any reduction whatsoever. To calculate the total weight of the dock, the weight of the side walls and the earth that may move with them should be added to the weight of the base slab.

In very wide dry docks, the base slab can be structurally independent of the side trunks to prevent the considerable bending that would appear as a result of uplift. In such cases, the weight of the base slab alone must counteract the whole of the uplift. Specific details of evaluating safety against dry dock flotation due to uplift will be given in future publications under the ROM Programme.

Safety with respect to flotation is provisionally recommended to be analysed as an UPL-type Ultimate Limit State, as defined in Subsection 3.3.1, and considering the partial factors for fundamental load combinations indicated in 3.3.6. The safety factor can be defined as the quotient between the force required to cause the dock to float –together with the earth jointly attached to it– and the maximum force that uplift can produce.

For works with a low SERI (5 - 19), this safety factor should be:

$F_1 \geq 1.3$ for quasi-permanent combinations.
$F_2 \geq 1.2$ for fundamental combinations.
$F_3 \geq 1.1$ for accidental and seismic combinations.

Furthermore, the specifications in Subsections 3.3.8 and 3.3.10 apply, for the purpose of increasing or reducing the recommended safety factors, depending on the nature of the works and the duration of the design situation.

4.6.3.2 Verifying Safety against Longitudinal Sliding

In docks with a gravity base slab, uplift is so high that the effective compression of the base slab against the ground is considerably reduced.

In normal operation conditions –with the dock empty– even relatively moderate horizontal loads may cause longitudinal sliding of the dock. The earth pressure on the bow closure, for example, can cause longitudinal sliding in some structural layouts.

Engineers should analyse the problem in such a way that a sufficiently ample margin of safety is provided against this potential slide, similar to the one recommended for this type of failure in other parts of this ROM 0.5 (Section 3.5).

4.6.3.3 Verifying Safety against Structural Failure of the Base Slab

With the dock empty, the base slab will be subjected to the bending required to transmit the uplift load to the side walls. The weight of vessels, transmitted through the keel blocks, will generate a localised bending that can be considerable.

Gravity base slabs are generally sufficiently thick not to require substantial reinforcement, although this will obviously depend on the width of the dry dock.

Structural analysis of the base slab will generally be a problem governed by the deformability of the soil; therefore, an analysis of soil-structure interaction is advisable. In such computations the ground can be represented by simple springs. Subsection 3.5.7 shows an appropriate procedure to evaluate the spring constants that represent ground deformability.

In major works, a more accurate computation is recommended, for example, a finite element model in linear elasticity.

4.6.3.4 Verifying Safety in Side Walls

The side walls of gravity dry docks can be of several types. They will generally be made of concrete and consist of either gravity, buttress or simple diaphragm walls.

Side walls should be analysed following the recommendations given in Section 3.7 on earth pressures and retaining structures.

It is not normally necessary to study the sliding stability of these walls, since the base slab will prevent this failure mechanism. The analysis of safety against bearing failure does not normally need to be checked, provided that the connections to the base slab prevent displacement of the side walls. The same applies for overturning, which, as explained in Subsection 3.5.6, is an associated failure mechanism.

Stability conditions during the Construction Stages may be more critical than when the works are in operation and must be expressly considered in the design.

4.6.3.5 Movements

Considerable displacements may occur in dry docks and affect structures supported by them or in their vicinity. Draining the dock can cause the base slab and side walls to heave, which will lead to differential settlement of adjacent structures.

Draining can also induce a certain amount of water seepage from the area surrounding the dock towards its interior, either through the walls or the base slab, and this could lead to further settlement.

Engineers should estimate the movements in the dry dock. The structural model used to analyse stresses could be of assistance in such calculations.

4.6.3.6 Verifying Safety against the Effects of Seepage

Dry docks with drained base slabs can experience seepage through their side walls and base slab when they are empty.

Little dewatering will be required to keep the dock dry, since the seepage will only occur through certain local areas (mainly joints).

Even if it is moderate, seepage can cause fine-particle migration, both from the backfill and from the natural ground itself.

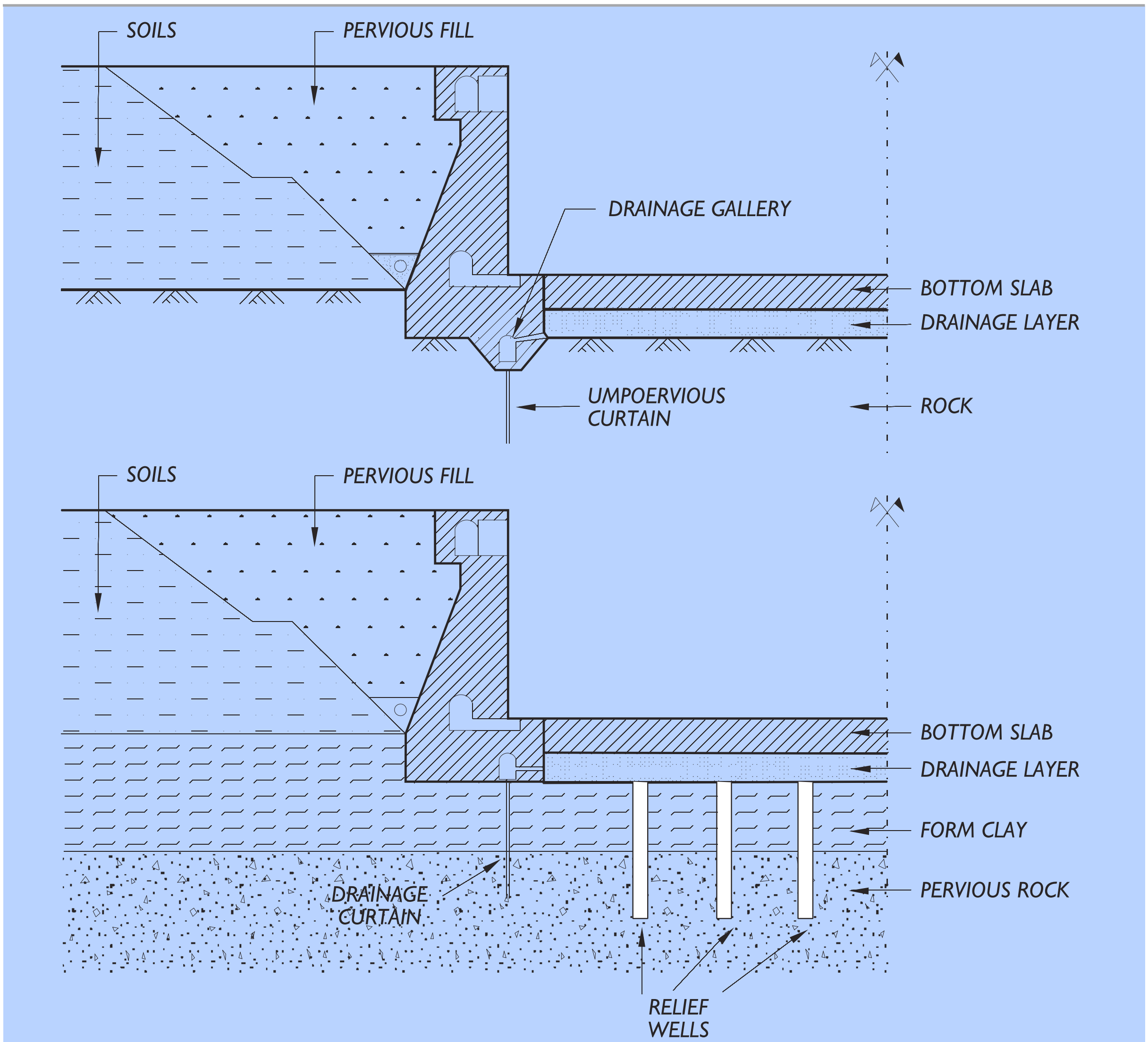
Engineers should examine this aspect and take the appropriate preventive measures (filters next to joints, reinforcement of the watertight strips, localised ground waterproofing, etc.).

4.6.4 Dry Docks with a Drained Base Slab

In very wide dry docks, the cost of the base slabs required to withstand the uplift may be so high that it is worth examining the alternative of draining them permanently.

Various elements of this type of solution are shown in Figure 4.6.3.

Figure 4.6.3. **Possible Layouts for Drained Base Slabs**



4.6.4.1 Reducing Uplift. Verifying Safety against the Effects of Seepage

The fundamental element of this type of solution is the waterproofing and drainage system.

The impermeability of the ground can be improved by constructing deep diaphragm walls around the base slab -made of concrete, steel sheetpiles, bentonite-cement- or cement (or other material) grout curtains.

Behind the seepage barriers, a drainage system must be installed to reduce uplift on the base slab.

The drainage system should generally consist of elements for collecting the seepage, channelling it and pumping out the water.

Seepage must always be collected at the base of the slab by a properly designed drainage layer, so that seepage does not produce particle entrainment or erosion. Some recommended criteria in this respect are given in Subsection 3.4.7.

Seepage can also be collected by auxiliary wells such as those shown in Figure 4.6.3. These wells, situated downstream (in the sense of the seepage flow) of the eventual waterproofing system, can be constructed underneath the side walls or the base slab itself. Water flow towards the drainage system must be kept within certain limits, not only to reduce the operating cost but also to avoid possible particle entrainment (in suspension or solution) by the water being pumped out. In any event, protection against entrainment should be examined and solved by the design of adequate filters.

Base slab drainage needs to be analysed in a sufficiently wide area, considering not only the ground in the vicinity of the base slab but also the distribution of uplift that may exist at greater depths. The presence of an impervious layer at a certain depth below the base slab may cause bottom heave even with zero uplift in the contact between slab and ground.

In any event, drained base slabs should be fitted with safety valves, so that any failure in the uplift reduction system (waterproofing, drainage, piping and pumping) does not involve heave of the base slab and the consequent total failure of the works.

4.6.4.2 Verifying Safety against Structural Failure of the Base Slab

Drained base slabs can be rigidly connected to the side walls or separated from them by watertight structural joints.

Their structural analysis will constitute a soil-structure interaction problem that should be solved as indicated in Subsection 4.6.3.3.

4.6.4.3 Verifying Safety of Side Walls

The side walls should be analysed following the recommendations given in Section 3.7 of this ROM 0.5, which deals with earth pressure retaining structures.

Drained base slabs, even when not very thick, usually have sufficient structural capacity to prevent sliding of the side walls. This point must be checked in each particular case. Bearing failure and overturning of side walls, particularly in cases where the base slab is structurally separated from the side walls by joints, should be analysed by taking the base slab as a non-resistant overburden.

The problem of the overall equilibrium of side walls can be critical, given the presence of the artificial seepage generated by the dewatering system and also due to the external loads that may act in the vicinity of the dock.

4.6.4.4 Movements

There may be substantial displacements in the base slab of docks built on soft soils due to load variations between full and empty states of the dock.

Permanent dewatering of the seepage beneath the base slab can cause subsidence (settlement affecting extensive areas) in the surroundings, which engineers must evaluate at the time of the design. Reflections of the side walls can be analysed as set out in this ROM 0.5 for the different types of retaining structures covered (Section 3.7).

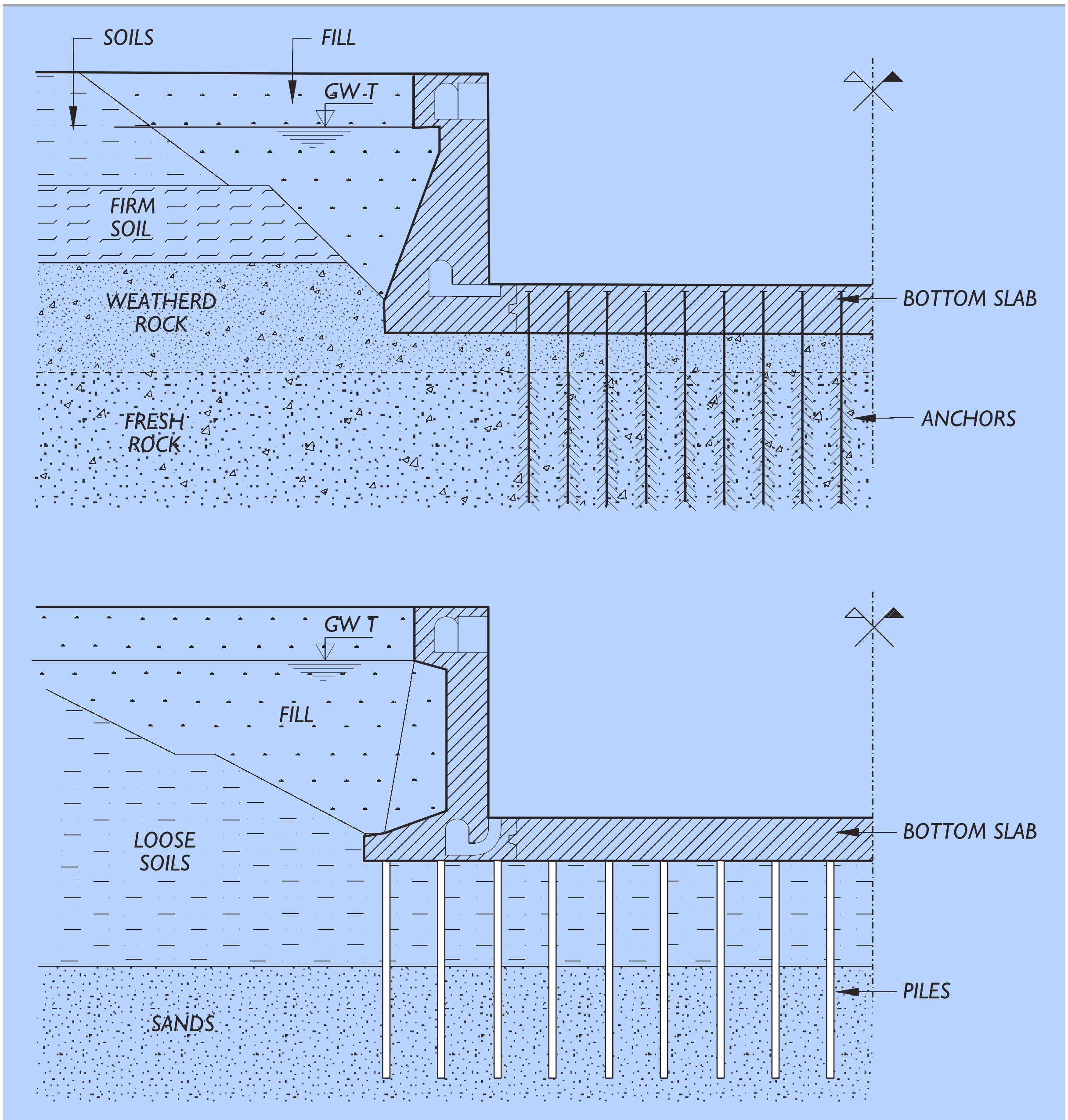
4.6.5 Dry Docks with an Anchored Base Slab

Uplift on the base slab of dry docks can be withstood by tension members rigidly attached to the base slab and the ground.

These tension members can be made of concrete (tensile piles) or steel. It is generally not recommended to use steel elements subjected to large tensile stresses (high strength steel tendons or cables), unless special precautions are taken to prevent the problem of brittleness induced by free atomic hydrogen.

The use of piles may be advisable to assist in transmitting concentrated compression loads underneath the keel or bilge blocks. Various aspects of these anchored or piled base slabs are illustrated in Figure 4.6.4.

Figure 4.6.4. **Possible Layouts for Anchored Base Slabs**



4.6.5.1 Verifying Safety against Base Slab Heave

Each base slab anchor must be capable of withstanding the uplift corresponding to its area of influence. The anchors must be long enough so that their safety against extraction is greater than the value indicated in Subsection 3.6.7.

Joint heave of the base slab and its anchors should be avoided by drilling the anchors to a depth where the weight of the ground mobilised is enough to prevent this massive heave (see the group effect in 3.6.7).

After constructing each tension member, it is recommended to test them individually under the service load. Additional failure tests are advisable on some elements constructed specifically for this purpose.

4.6.5.2 Verifying Safety against Structural Failure of the Base Slab

The structural analysis of the base slab can be done as indicated in 4.6.4.2, by adding some extra springs to the model to represent the anchors or piles.

The spring constants simulating the piles can be estimated as indicated in Subsection 3.6.10.

4.6.5.3 Verifying Safety of the Side Walls

Drained and anchored base slabs are usually constructed with a similar thickness, since this will primarily be conditioned by the service loads. For this reason and for the general study done here, the side walls should be analysed as indicated in Subsection 4.6.4.3.

4.6.5.4 Movements

Displacements of base slabs anchored in rock will generally be moderate and not constitute a critical problem. Base slabs piled through soft soils may be subject to certain movements. Engineers should estimate these following the recommendations given in Subsection 3.6.9.

Deformation of the side walls should be analysed as given in 3.5.7 –if they have shallow foundations– or as given in 3.6.9 -if they are supported on piles.

4.6.5.5 Verifying Safety against the Effects of Seepage

The considerations of Subsection 4.6.3.6 about gravity base slabs apply in this case.

4.6.6 Navigation Locks

Uplift is a less serious problem in locks than it is in dry docks, so gravity base slabs tend to be used -or no base at all in good ground. In any event, the function of the base slab is important as a support for the side walls against the problem of sliding.

4.6.7 Analysis of Construction

The design of a dry dock or navigation lock is intimately connected with the construction process.

The way of carrying out excavations and temporary dewatering, dredging or filling operations, how to found eventual precast solutions, the way in which subsequent filling is performed, etc. are fundamental design aspects.

Engineers must foresee any critical situations during the Construction Stage, such as the potential instability of the excavation slopes, entrainment as a result of temporary dewatering, provisional stability of side walls, etc.

These potentially critical situations will be specific to each individual works and to each construction sequence and should be appropriately identified and evaluated. Sufficiently ample safety factors should be obtained, which in any event must be higher than the values set out in this ROM 0.5 for each failure mode that engineers could anticipate, treating construction as a transient situation.

4.7 BREAKWATERS

Forthcoming publications in the ROM Programme and especially ROM 1.1 will include specific and fully detailed methods for designing and building breakwaters. This section in ROM 0.5, devoted to geotechnical failure modes and factors, only attempts to point out the main geotechnical problems affecting this type of works and to provide some simplified methodologies making it possible to tackle them in the absence of more precise or detailed methods.

Breakwaters and seawalls are structures with the main objective of providing sheltered areas in harbour and coastal zones, safe from marine and atmospheric dynamics. As a general rule, a sheltered harbour is designed to facilitate port and logistical operations related to maritime transport. On the other hand, a sheltered shoreline is designed to facilitate the orderly and sustainable use and exploitation of the coastal environment and can include, *inter alia*, the correction, conservation and regeneration of beaches and bathing areas and the exchange of land-sea cross flows of natural and artificial substances.

4.7.1 Types of Breakwaters

Breakwaters can be classed into the following types:

- rubble-mound (emerged) breakwaters
- vertical breakwaters
- composite breakwaters
- low-crest or reef (submerged) breakwaters
- berm breakwaters.

The main difference between them is the dimensions of each of the sections (foundation, central core and superstructure) and consequently their manner of controlling climatic agents and of transmitting loads to the ground.

The typical cross-section of a *rubble-mound breakwater* comprises a sequence of cover layers making up a transition between the quarry-run core and the primary cover layer, a granular element built of rockfill or artificial concrete armour units. Some breakwaters incorporate toe berms to ensure the stability and shape of the external slope, protect the foundation ground and provide support for the secondary cover layers. In the majority of cases, a wave-return wall or other superstructure completes the breakwater.

Figure 4.7.1 shows a typical section of a rubble-mound breakwater and highlights some of the most interesting elements.

The main characteristic of the typical cross-section of a *vertical breakwater* is that its central section and the superstructure are formed of a single structural element. The face on the seaward side is generally vertical and can be built of precast caissons, plain concrete blocks, sheetpile diaphragm or cellular walls, etc. It generally rests on a quarry-run berm levelled off at a suitable founding depth and that may or may not be protected by an external cover layer depending on whether its stability is affected by sea level oscillations or not. A large-sized block is often constructed on the berm, adjacent to the central body of the breakwater, so as to reduce the peak of the uplift on the seaward edge of the foundation and to protect against scour.

Figure 4.7.1. ***Rubble-Mound Breakwater***

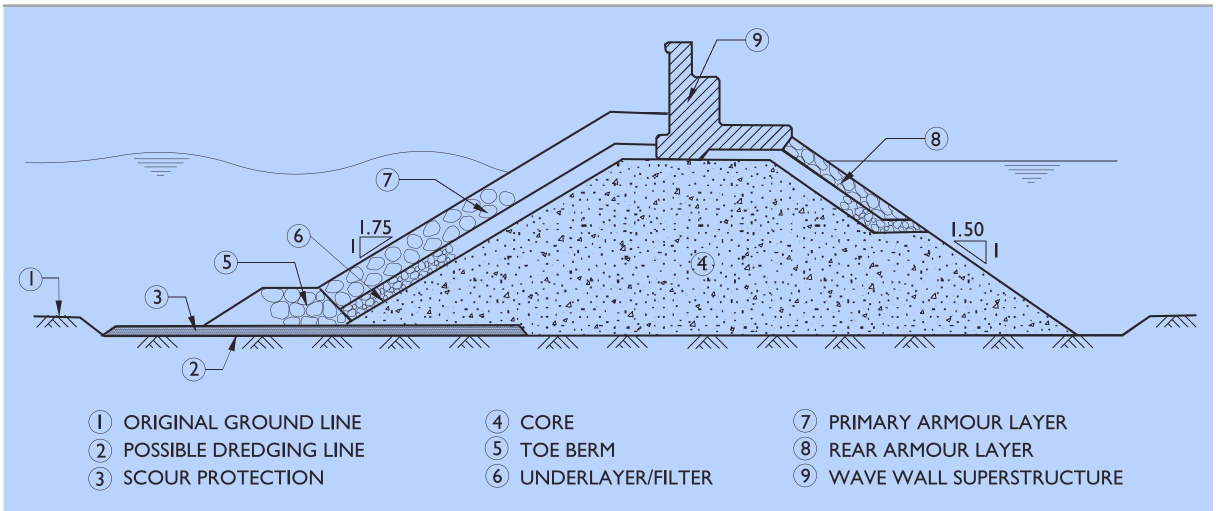
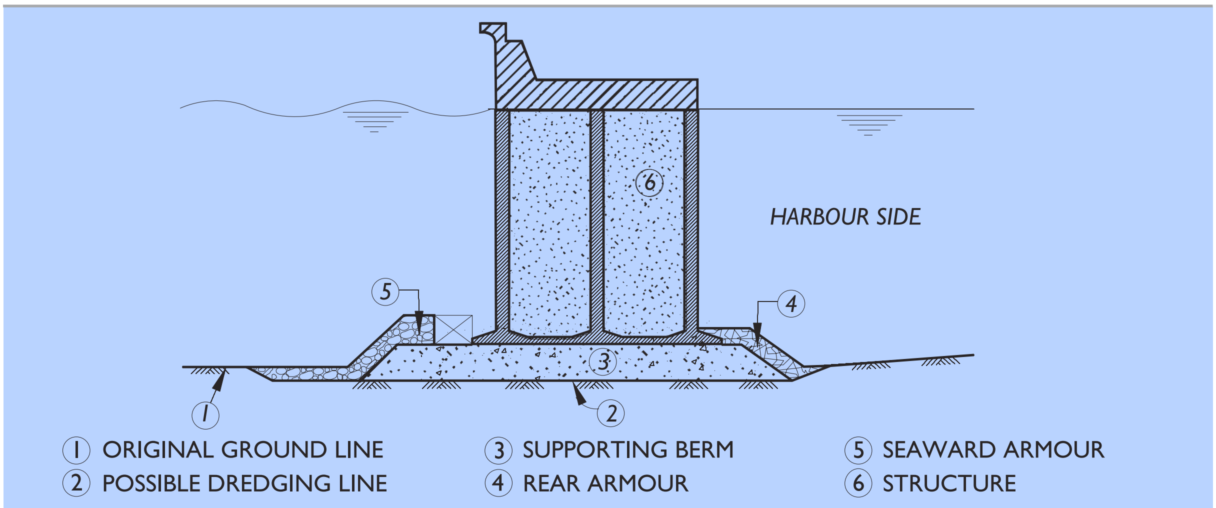


Figure 4.7.2 shows a typical cross-section of a breakwater with a vertical face formed by a precast reinforced concrete caisson.

Figure 4.7.2. ***Vertical Breakwater***



When the foundation berm, as defined in the case of vertical breakwaters, takes up a considerable proportion of the draught, to the extent that its presence significantly modifies the kinematics and dynamics of sea level oscillations, this type is known as a *composite breakwater*. Their protective function is shared between the lower section, which is not just a foundation element, and the central section, which performs the function of a superstructure.

The typical cross-section of submerged *low-crest* or *reef breakwaters* is very similar to that of a rubble-mound breakwater, but without a superstructure. Covering a quarry-run core, a sequence of cover layers is placed up to the external cover layer, which should be extended over the crest and, depending on its width, prolonged along the leeward cover layer. The elevation of the crest determines the behaviour of the section under the waves running over it.

Any type of rubble-mound, vertical or composite breakwater can be overtopped in a sea state if the relative freeboard (F_c/H_T) is less than or equal to 1.0, where H_T is the height of the highest wave for that sea state at the vertical of the toe of the breakwater, considering a high sea level simultaneous and compatible with the state of the sea.

Berm breakwaters are characterised by having the central body as an extension of the foundation and being composed of granular and non-uniformly graded materials, known as *riprap* in coastal engineering. The typical section is built with a very moderate slope in order to guarantee its static stability or else with more steep slopes, allowing for substantial deformations during the Service Stage, until stable profiles are obtained.

The choice of one or other type of breakwater should be made in each specific case in the light of several factors, such as nature of the foundation ground, draught, extent of the harbour zone, availability of materials, construction, maintenance and major repair costs, construction time, maintenance possibilities, environmental impact , etc.

Figure 4.7.3. Composite Breakwater

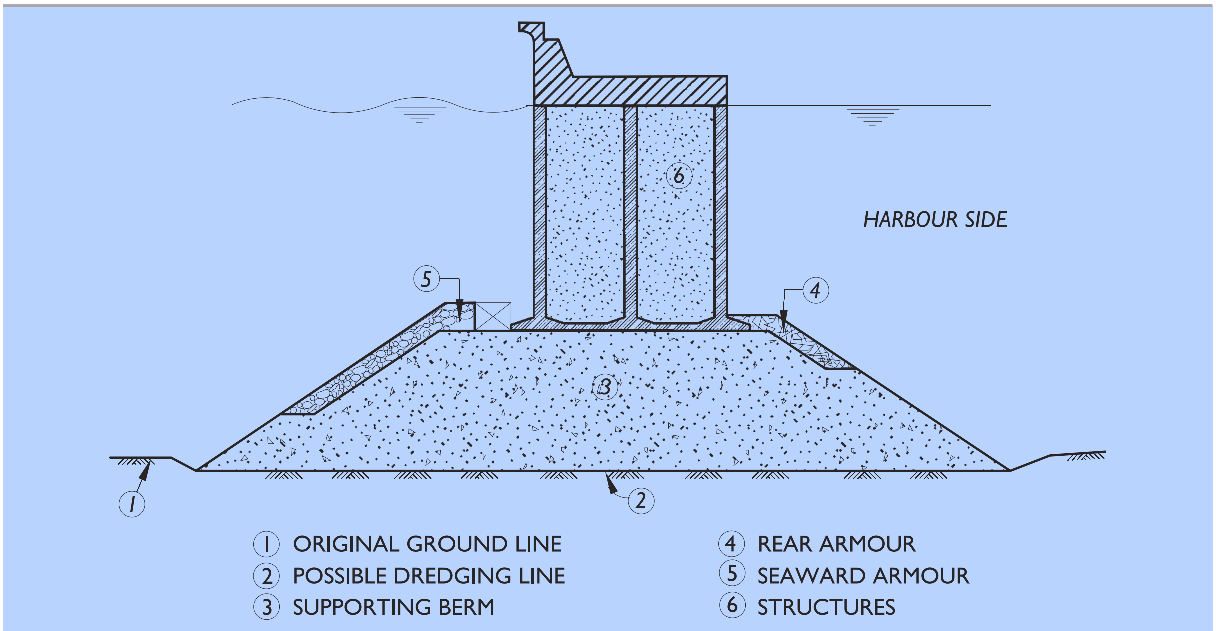


Figure 4.7.4. Reef (Submerged) Breakwater

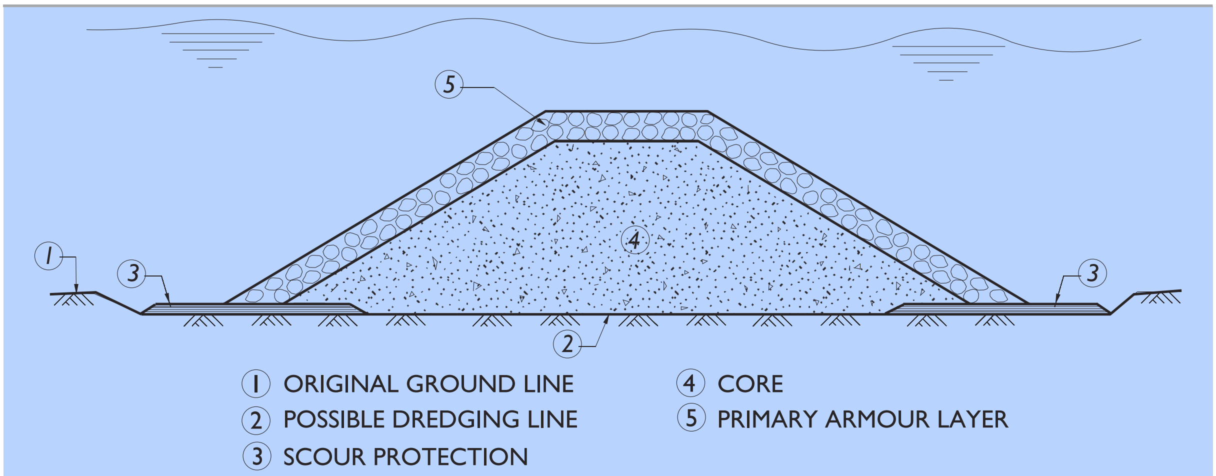
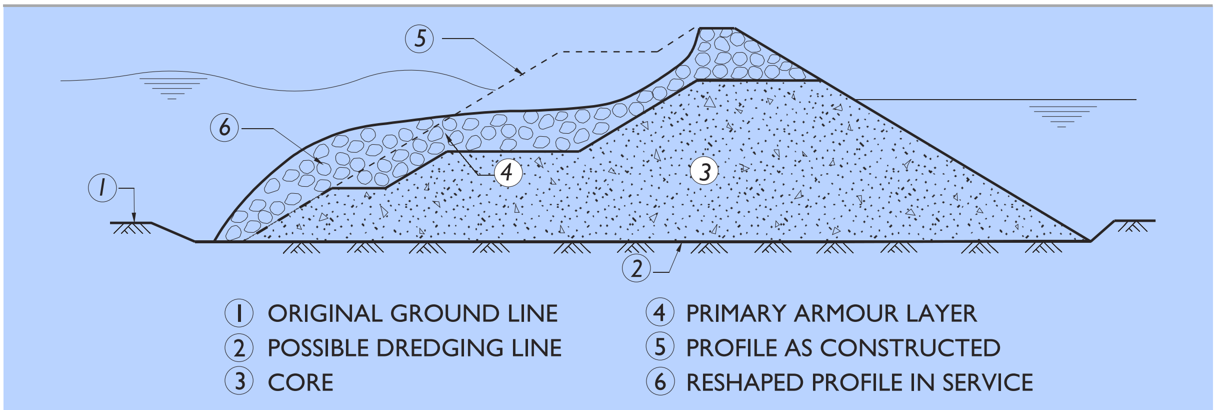


Figure 4.7.5. ***Berm Breakwater***



4.7.2 Geotechnical Information

4.7.2.1 Foundation Investigation

Geotechnical investigation of the area affected by breakwaters is aimed at studying the geotechnical problems associated with this type of structure when facing sea level oscillations. Besides the common soil mechanics problems, they include those associated with erosion, the existence of water flow in the porous medium –which leads to the generation of porewater pressures– and the combined dynamic behaviour of soil and structure, as will be shown below.

Thus, subsequent studies will require knowing the nature, grading curve and corresponding permeability, strength and deformability of each type of soil in the area around the breakwater. In bedrocks, it is advisable to know their nature and structure, jointing and degree of weathering as also their variation with depth.

Recommendations are given in Part 2 of this ROM 0.5 that engineers should follow when planning and carrying out these geotechnical investigations.

Actions due to waves, other sea level oscillations and earthquakes are the main loads acting on breakwaters. They can produce significant dynamic effects in the soil-structure system and modify soil behaviour, both with respect to its resistant capacity and deformational behaviour. Phenomena such as densification, liquefaction and generation of porewater pressures and hydrodynamic seepage forces in the soils can be caused by these actions and should be appropriately explored. Section 3.10 of this ROM 0.5 gives some recommendations worth considering by engineers in this respect.

4.7.2.2 Source and Quality of Borrow Materials

Prospecting for borrow materials required for the construction of breakwaters is an essential aspect of these of works and will normally condition the choice of the most suitable type. Studying the size of stone that can be produced by a particular quarry and the grading (permeability) and mechanical properties (strength and deformability) of the borrow materials is necessary for subsequent analysis of the behaviour of breakwaters subject to sea level oscillations.

a. Rock Quality

It is essential that the rock mass to be quarried is fresh and durable. If only poor quality rock is available (unconfined compressive strength less than 50 MPa and unit weight less than 26 kN/ m^3, for instance), its use should be restricted to the core of the breakwater or at most to the intermediate layers. In such cases, the external armour layer must be constructed with units of a different type.

The quality of the rock can be examined by mineralogical tests to determine its chief components. Rocks with a high clay mineral content should be excluded (marls, for example) as also expansive minerals (anhydrites and some sulphates) and soluble minerals (gypsum and other salts).

Durability can be indirectly determined by laboratory tests as shown in Part 2 of this ROM 0.5 (Subsection 2.11.9). The ROM 0.1 and ROM 1.1 publications set values recommended for the corresponding parameters in each breakwater zone.

The quality of the rock must also be examined to select potential borrow material for concrete aggregates. In this respect, additional tests may be required to check its adequacy.

b. Resistencia al corte de las escolleras

Shear Resistance of Rockfills

Rockfill failure is a complex problem that has still not been properly solved. In regular practice, its strength is represented in calculations by cohesion and friction parameters as in the case of natural granular soils.

There are some basic differences, however, between granular soils and rockfills. Rockfills can have large blocks and open grading, which do not usually occur with natural granular soils. The sharp edges usually existing in the contacts between fragments of rockfill are more rounded in granular soils.

For the same effective pressure, intergranular forces increase with the square of the mean diameter of the fragments making up rockfills. For this reason, crushing of the contacts between particles, which are not common in siliceous granular soils until very high pressures are reached (20 MPa, for example), can occur with large-block rockfill under moderate pressures, close to the service values in maritime applications. Contact crushing can be an important phenomenon under pressures around 1 MPa [(5)].

The shear strength of rockfills is usually defined with a variable angle of friction, in order to take these effects into account:

$$\phi = \phi_o + R \log_{10} \frac{\alpha \cdot q_u}{\sigma'_n} \leq 60°$$

where:

- ϕ_o = basic angle of friction, which will depend on the nature of the rock, and will usually be between 25° and 35°.
- R = reduction in the angle of friction when the effective compression normal to the plane of slide is multiplied by ten. *R* values mainly depend on the shape of the block edges and the degree of compaction and can of vary between zero in the case of fine, not angular rockfill and over 10° for rockfill with a very open grading and very sharp edges.
- α = dimensionless parameter depending on fragment size, which can vary between 0.5 for large-block rockfills to close to 1 for gravel-sized rockfills.
- q_u = unconfined compressive strength of the rock; for these purposes, it can be indirectly estimated by the point load test.
- σ'_n = normal effective compression on the shear plane.

For zones close to the surface, when the effective compression, σ'_n, is small (less than 1 kPa), a small cohesion with a constant value can be assumed (in the order of 0.1 kPa), in addition to the frictional strength. This will circumvent the theoretical problem of zero strength on the surface, without introducing any noticeable errors in the stability calculations.

(5) A similar phenomenon occurs in cacareous sand of organic origin, as described in Section 2.2.8.1.

As can be seen, good-quality rockfill can behave with very high angles of friction in zones close to the surface. This explains why very steep outer slopes, such as 1.5(H):1(V), are stable even in the severe conditions imposed by waves [(6)].

One practical drawback should be mentioned to offset this beneficial effect of the singular resistant behaviour of rockfills. Tests on physical models for representing the hydrodynamic behaviour of rockfills subject to waves are carried out with stress levels lower than the actual ones. Their strength behaviour could prove to be optimistic. In turn, these physical models also fail to reproduce the mechanical behaviour of rockfills correctly, as these properties are not scaled in the model with the Froude number. Consequently, this behaviour cannot be validated with this type of test.

Keeping the above considerations in mind, engineers should investigate the strength of rockfills. It is now possible in Spain to carry out strength tests on rockfills with grading curves and in pressure environments similar to the real ones and this technique could be useful in major projects.

These studies, which could be of less interest for armour layers made of large-block rockfills, can be highly adequate for the secondary cover layers and for breakwater cores.

c. Rockfill Permeability

The flow of water –in the pressure conditions generated around breakwaters and with the usual rockfill grading– may be very far from the laminar regime implicit in Darcy's law, which is recommended in this ROM 0.5 (see Subsection 2.2.6).

Water movement obeys a complicated process of interaction between the water –with trapped air– and the rockfill. The water flow regime, particularly in the contact zone of the sea with the rockfill, will clearly be turbulent ($Re = v \cdot D_{50}/\nu > 1000$).

Nevertheless and in order to facilitate subsequent calculations, it is normal practice to continue to use a coefficient of permeability for rockfills in the same sense as it is used in soil mechanics, i.e., as the quotient between the discharge velocity (water flow divided by gross total area) and the gradient of the water flow potential. Engineers must be aware that, in these cases of turbulent water flow, the coefficient of permeability will depend on this gradient; in fact, it is inversely proportional to the square root of the gradient.

In these turbulent-flow seepage problems, the equivalent coefficient of permeability (i.e., for simplified flownet calculations) of rockfills increases with the average size (D_{50}) of the rockfill and the uniformity of its grading.

Although in natural granular soils (sands and gravels, or mixtures of the two), permeability increases in laminar flow ($Re < 1$) with the square of its effective diameter, this effect of grain size is less pronounced in rockfills subjected to wave action. Increase in permeability in turbulent flow appears to be also governed by some representative size (which could be D_{50}), but increases with the square root of this size.

The equivalent permeabilities of rockfills tend to be in the following order of magnitude (under the turbulent flow being considered):

	Average Size D_{50} (mm)	Equivalent Permeability (m/s) [(7)]
Large blocks	850-2500	1.00
Medium sized blocks	100-300	0.30
Small blocks	10-80	0.10

(6) A simple stability along plane failure surfaces parallel to the slope, and with a descending water flow, shows that equilibrium requires angles of friction in the order of 60°.

(7) AData taken from the CIRIA Manual n° 83. See references.

Engineers must investigate the permeability of the rockfills to be used for their projects, particularly in cores and intermediate cover layers. These data may be required in subsequent design calculations as they are most relevant for defining the flow characteristics through the porous medium and the conditions for generating and dissipating porewater pressure during both the Construction and Service Stages.

Permeability in a turbulent regime can be investigated in specialist laboratories. The test procedures and scale effects are beyond the scope of this ROM 0.5.

Physical models attempting to represent the hydrodynamic behaviour of rockfills subject to waves can produce results containing considerable uncertainties as to the actual works behaviour, due to the difficulties involved in performing an adequate Froude number scaling of the flow characteristics in the porous medium (turbulent or laminar) in each individual material making up the structure. This is especially relevant in structures formed of different-sized materials and in which there can be significant processes of porewater pressure generation and accumulation, because it is very hard to scale –with a Froude similitude– the material constitutive properties simultaneously in areas with large particles and in areas with small particles. A well-thought geometrical scale and the use of more viscous fluids may alleviate some of these problems.

d. Rockfill Deformability

Rockfill deformability is only due in a small part to the elastic compression of the rock fragments composing it. This part of the deformation will be recovered when unloading. There are other causes of deformation such as contact crushing, entrainment or washing out of fines owing to wave action, etc. that give rise to unrecoverable deformation.

Both deformation components should be known when designing breakwaters. Moduli of elasticity or other similar parameters described in Subsection 2.2.10 to represent ground deformation can be used as elastic deformation parameters.

For cyclic, alternating or impulsive loads like those due to direct wave action or to displacements of the resistant structure caused by such action, deformability may vary, as also happens in other types of ground (see Section 3.10).

Non-recoverable deformation should be evaluated by consulting published experience or by carrying out *in situ* or laboratory tests specially designed for the purpose. It should be pointed out that the physical models associated with the hydrodynamic behaviour of rockfills subjected to waves are not representative of their deformability since, as indicated in Point c) above, it is very hard to scale the constitutive properties and particularly the deformation parameters to keep the Froude number. This problem may also cast doubts over the test results on hydrodynamic behaviour in cases where, in respect of deformability, the model behaves very differently from what could be expected in reality.

Stress fluctuations in breakwater rockfills cause permanent deformations (the wave action is said to consolidate them). These deformations may cause substantial post-construction settlement, even up to over 2% of the height of the rubble-mound breakwater (or the thickness of the berm in a vertical breakwater).

4.7.3 Verifying Safety against Geotechnical Failure Modes

As Subsection 3.2.1 of this ROM 0.5 has stated, in certain structural types, including those where wave and other sea-level oscillations or other environmental agents are the predominant agents triggering geotechnical failure modes, it is not possible to allow probabilities of individual occurrence for some of such failure modes that are clearly lower (an order of magnitude less) than the overall failure probability indicated in the ROM 0.0 without this having a relevant influence on the economic optimisation of the works.

In other words, despite the general criterion adopted in this ROM 0.5 of verifying safety against geotechnical failure modes by adopting very low failure probabilities in relation to the joint value admitted for the entire structure, in the above cases some geotechnical failure modes must be considered as principal failure modes and, therefore, their contribution cannot be disregarded in calculating the overall failure probability.

These effects are particularly relevant in breakwaters, where achieving significant increases in the reliability of the structure against certain geotechnical failure modes is very difficult or far from being economically efficient.

4.7.3.1 Choosing the Occurrence Probability for a Geotechnical Failure Mode

For a breakwater, the joint probability of failure, during the Design Stage analysed and with respect to the failure modes assigned to Ultimate Limit States, should be lower than the maximum values recommended in the ROM 0.0 as a function of its social and environmental repercussions impact (SERI), and resulting from economic optimisation.

Having determined the design's overall probability of failure, assigning the occurrence probability corresponding to each failure mode will be possible from the top down in the diagram of failure modes, taking into account the type of each section of the diagram (in series or parallel) and the independence or degree of correlation between the different modes as well as the influence that the failure probability assigned to each mode has on the costs of the works. In this respect, the lowest probabilities should be assigned to the failure modes for which achieving improved reliability does not have a relevant impact on the costs. Consequently, failure probabilities close to the joint failure probability should be assigned to failure modes where improving reliability is difficult or can only be done at the expense of very substantial cost increases.

The ROM 1.1 publication will include the relevant diagrams of failure for each type of breakwater and adequate recommendations for assigning failure probabilities to each one of the failure modes.

In short, for each breakwater section (foundation, central body and superstructure), it can be considered that those geotechnical failure modes for which an increase in reliability has a greater effect on the cost are principal failures. The occurrence probability assigned to each of these modes, considering their different degrees of correlation, can be clearly lower, somewhat lower or even equal to the overall failure probability assigned to each section of the breakwater.

Comment: Considering that in most cases the level of reliability of the foundation and the central body of a breakwater are those most affecting the costs of the works, the simplified solution can be adopted of distributing over these sections the major portion of the maximum failure probability recommended for the SERI rating corresponding to them, as a function of its effect on the cost of the works. Consequently, in line with this subsection, the failure probabilities assigned to the principal geotechnical failure modes associated with each of the breakwater sections have an upper limit that can be taken as equal to the overall failure probability assigned to the said breakwater section (see Fig. 4.7.6). If, for a particular works type and site, the level of reliability of each breakwater section has a different effect on the costs, the joint failure probability should be distributed as a function of these partial costs. For example, the failure probability of each individual section can be taken as a third of the joint failure probability of the works when the effect on the overall cost of each section is similar.

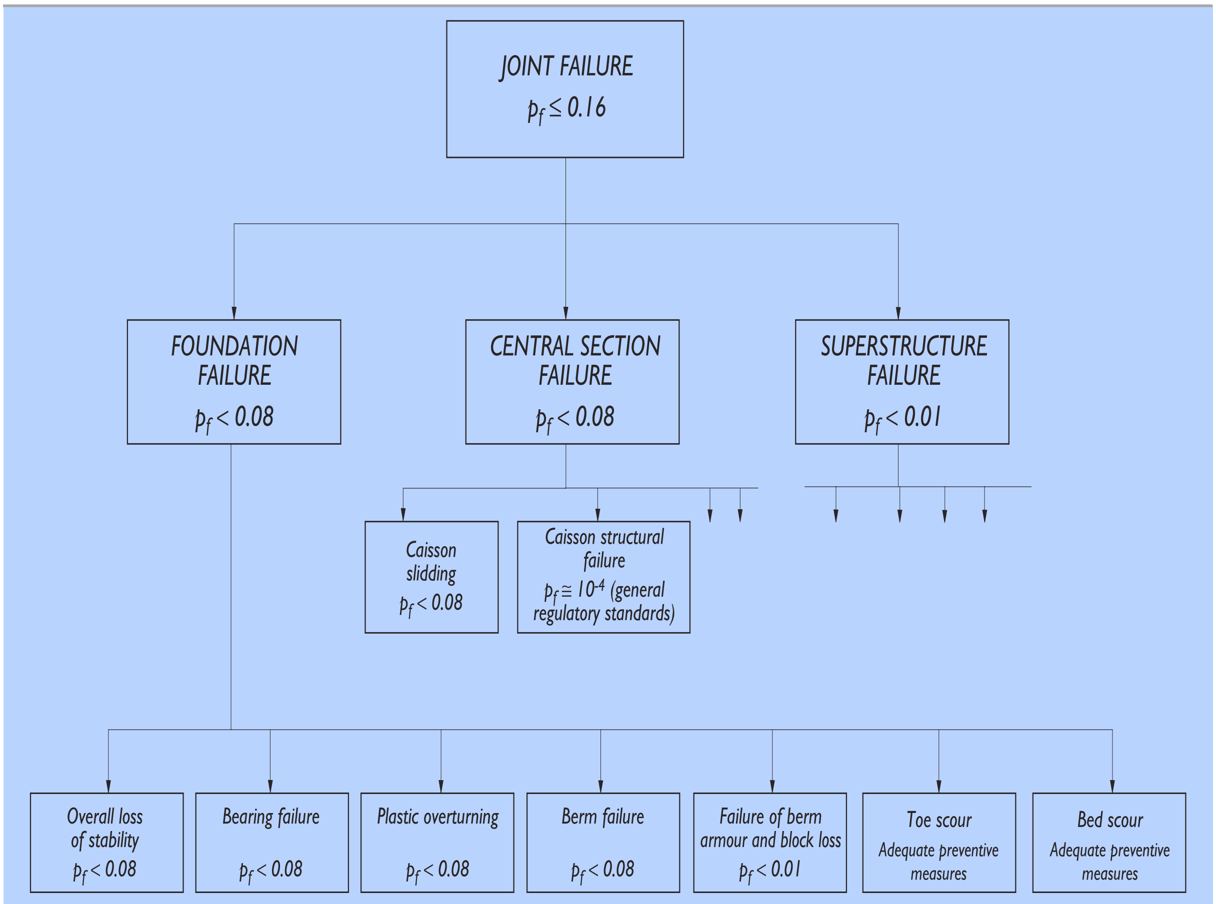
4.7.3.2 Choosing the Verification Procedure for Level-I Calculations

To verify the safety of a breakwater using the Level-I procedure, engineers will need to choose one of the two paths indicated below as a function of the failure probability assigned to the failure mode in question.

The first procedure is the standard approach followed in all the cases covered in this ROM 0.5 and that can apply when the failure probabilities assigned to the failure mode in question are low – in any event under 5%. The method may prove to be fairly inadequate for larger failure probabilities.

The second procedure is specific to the structures where one or more environmental agents play a predominant role in triggering the failure modes, as does the action of waves and the other sea level oscillations in the case of breakwaters. It should be used to analyse failure modes with a failure probability equal to or over 5%, as smaller values will imply considering climatic loads associated with very long return periods, the definition of which can be so imprecise that it makes this checking method as inaccurate as the general method.

Figure 4.7.6. Diagram of Failure for a Vertical Breakwater Associated with Ultimate Limit States. Works with Minor SERI (SERI < 5). Simplified Example of Assigning Failure Probabilities Considering that the Joint Failure Probability Obtained through Economic Optimisation Criteria is 0.16



The difference between these two methods lies in two essential details – definition of the design value of the wave action (1) and the value of the safety factor or safety margin that should result from the verification (2).

Standard ROM 0.5 Approach

This method has been described before in all its details throughout this ROM 0.5. With regard to wave action, according to this standard approach, the design value is equal to the characteristic value multiplied by the load factor and the corresponding coefficient of compatibility, i.e.:

$$E_d = E_n \cdot \gamma_E \cdot \psi$$

The values of the load factor and the coefficients of compatibility are indicated in Part 3 of this ROM 0.5.

The characteristic value of the variable action of waves and the other sea level oscillations should be determined as indicated in Part 3 of this ROM 0.5. The value generally adopted should be the one corresponding to the state of the sea associated with a storm having an annual exceedance probability equal to 2% taken from the extremal regime, as defined for any other variable action (50-year return period).

When using this approach, an extraordinary wave action should also be considered. The characteristic value for this action should be the value corresponding to an annual probability of 0.2% that the wave agent is exceeded, as defined for any other extraordinary action (500-year return period).

The safety factors that should be obtained when verifying each failure mode and the associated calculating procedures are specified in the following sections.

Specific Method

In this procedure, the design value representing the variable wave action, when this is the predominant load, is defined with a different criterion. The design value of the action is formally:

$$E_d = E_n^* \cdot \Psi$$

The characteristic value, in this case E_n^*, is the one associated with the sea state whose storm has an exceedance probability –during the time period assigned to the design situation or state in question– that is precisely equal to the failure probability assigned to the failure mode under analysis. This may lead to the need of defining a different design wave for each design situation or state, and within this, for each failure mode in question.

The characteristic value is not increased by a load factor and is only affected by its combination value, ψ. As the wave action is the predominant load, i.e., waves are the main action of a fundamental combination, the value of the combination factor is equal to 1 ($\psi = 1$).

The safety factor to adopt when proceeding with this method should be lower than the general one defined in this ROM 0.5 and will have to be expressly calculated for each failure mode in question, as is done with wave action.

To obtain a failure probability p_f a safety factor F can be adopted that is given by the following expression:

$$F = \frac{e^{\zeta(\beta_0 + \frac{1}{2}\zeta)}}{1 + \beta_0\, v_A} \geq 1$$

where:

β_0 = reliability index associated with failure probability p_f and given by the expression $\beta_0 = \phi^{-1}(1-p_f)$, in which ϕ is the normalized form of the cumulative normal probability function and ϕ^{-1} is its inverse function.

v_A = coefficient of variation of the load value. In the absence of more precise data, the wave action in the open sea can be taken to have a coefficient of variation of about 0.18 in the Mediterranean and 0.14 in the Atlantic.

ζ = standard deviation of *ln F*. This value is approximately equal to the coefficient of variation of *F*.

Obtaining the value of ζ requires a specific sensitivity analysis as the one indicated in Subsection 3.3.10 or similar. For guidance purposes and for the calculation methods proposed in this ROM 0.5, the following values can be assumed:

ζ = 0.15 to 0.20 problems of overall stability and sliding.
ζ = 0.20 to 0.25 problems of undrained bearing failure.
ζ = 0,30 to 0.35 problems of drained bearing failure.

Within the range indicated for the value of ζ, the larger the coefficient of variation of the action, the larger the chosen value should be. As a general rule and as a result of the foregoing, higher ζ values should be used in the Mediterranean coast than in the Atlantic coast.

It is also possible to use the same *F* values indicated for the standard approach, although this will generally err on the safe side and may not be adequate.

When the failure probability under consideration is greater than or equal to 5%, to simplify the procedure, the required safety factor for exceptional combinations (accidental and seismic) should be taken as equal to 1, provided that the combination value of the variable actions and the characteristic value of the extraordinary and seismic actions are defined by the standard method laid down in this ROM 0.5 (return period in the order of 500 years for extraordinary loads). For greater precision, an alternative fundamental combination could be checked, considering that the seismic action (or another extraordinary load) behaves like a preponderant variable action. Its characteristic value will then be defined as the one whose exceedance probability –during the time assigned to the design situation in question– is equal to the failure probability under analysis. In this case, the required safety factor should be obtained in the same way as defined for waves (8).

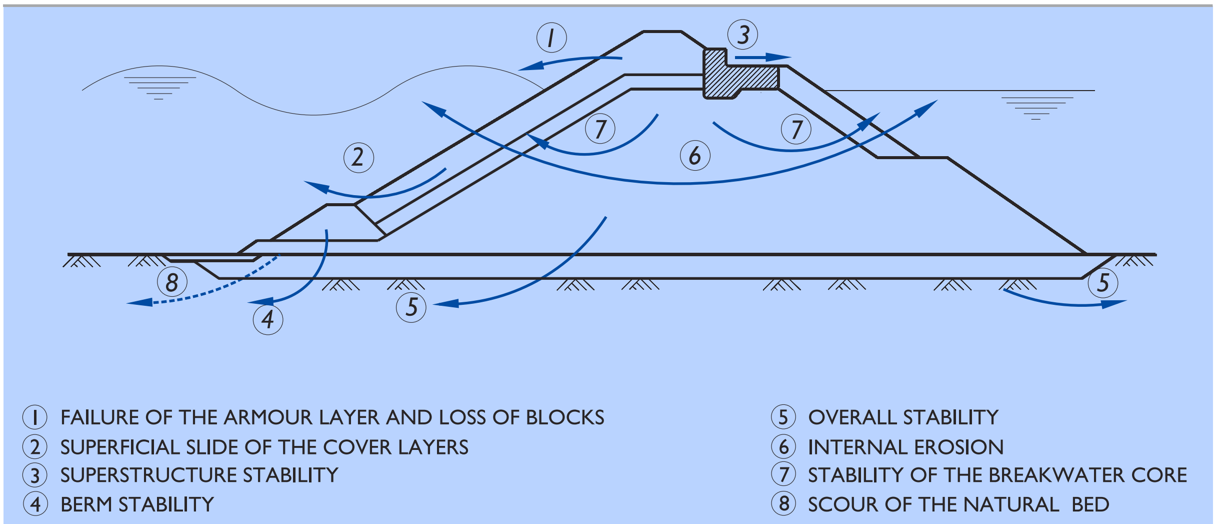
The quasi-permanent combination usually considered in conventional calculations of the general method does not need to be considered when this verification method is used. This combination should only be used, without load factors, to check that behaviour is satisfactory with respect to Limit States of Serviceability.

4.7.4 Rubble-Mound Breakwaters (Emerged)

Each breakwater will have its own set of conditioning factors, which will mean the design engineer has to carry out as many studies as are required to ensure its good performance against all expected failures.

There are, however, some geotechnical aspects that are common to all rubble-mound breakwaters that must always be considered. The ones shown in Figure 4.7.7 are the main geotechnical and hydraulic failure modes assigned to Ultimate Limit States and are commented on in the following sections.

Figure 4.7.7. **Geotechnical and Hydraulic Failure Modes Assigned to Ultimate Limit States and Associated with Emerged Rubble-Mound Breakwaters**



(8) In the absence of more precise data, the basic seismic acceleration associated with a 500-year return period is related with the one corresponding to a different return period by the following formula: $a_{b,T} = a_{b,500}\ (T/500)^{1/2.7}$. In turn, the coefficient of variation of the seismic action can, for this purpose, be taken as 0.2.

4.7.4.1 Verifying Safety against Failure of the Primary Cover Layer

Failure of the armour layer and loss of some of its blocks, dragged by the sea, is the principal hydraulic failure mode to be considered in designing rubble-mound breakwaters and to which most attention is normally devoted.

The armour blocks forming the primary cover layer must be of an adequate size and shape for the design wave. Stability checks are normally carried out by empirical formulae based on the observation of actual breakwater behaviour and on tests run on small-scale physical models (formulae such as Iribarren's, Hudson's, etc.). It is recommended to carry out an experimental laboratory check of the solution adopted.

The equations for verifying safety against this failure mode are covered in detail in the ROM 1.1 publication.

4.7.4.2 Verifying Safety against Surface Slide in Cover Layers

The external armour layer can slide along its contact with the secondary cover layer. This problem can be solved by preventive measures, such as the inclusion of keys (larger sized units lying across this contact zone and introducing a discontinuity), the construction of stabilising berms at the toe of the breakwater or other similar measures. It is not customary practice in Spain to include keys and is not to be recommended for artificial units.

The secondary armour layer may slide along its contact with the breakwater core. If this failure mode occurs, the primary cover layer will also be entrained by the slide.

This almost plane sliding, with a large part of the failure surface running along the contact of rockfills of different grading can be studied using the general calculation procedures indicated in this Section 3.8.

In these calculations, it should be assumed that the shear strength of the contact between layers is purely frictional (c = 0) and that it has a design angle of friction lower than that of the rockfill immediately overlying the plane of slide, i.e.:

$$\phi_{design} = \phi \cdot r^n$$

where:

- ϕ = angle of friction of the rockfill immediately above the sliding plane. This parameter should be defined in the Geotechnical Report as indicated in Subsection 4.7.2.2.
- r = ratio of the average sizes of the rockfill below the plane of slide and the rockfill immediately overlying it: $r = D_{50,\,below} / D_{50,\,above} < 1$.
- n = dimensionless parameter that can be investigated in the laboratory and which, in the absence of better data, can be taken as equal to 0.3 for relatively uniform sizes.

The primary unknown in these calculations is the state of the porewater pressure in the contact, which depends, amongst other parameters, on the permeability of the different cover layers and on the modifications in the hydraulic regime produced by the presence or not of superstructure. As an initial approximation, it can generally be assumed that the breakwater is impermeable beneath the sliding plane and that the rockfill is dragged down by a water flow parallel to the slope of the rubble-mound, and examine the situation at the time when the wave is at its lowest point (wave run-down) in its contact with the breakwater face.

When the slide is very long compared to the thickness of the sliding mass, the simplified formula of plane sliding indicated in Subsection 3.8.4.4 can be used.

Engineers can consider employing more complex calculations that take into account three-dimensional effects and uplift values in line with those expected in reality, which could have been obtained from small-scale model tests.

The ROM 1.1 publication will include detailed methods for verifying this failure mode.

Stability against plane sliding, as described in this section, may not be guaranteed by good behaviour in small-scale model tests. Unless special precautions are taken in preparing the model, it will generally have higher friction angles than those to be found in the breakwater materials.

Safety will be verified when it has been proven that the safety factor obtained is at least higher than the minimum shown farther on in Table 4.7.1.

4.7.4.3 Verifying Safety against Loss of Stability in the Superstructure

The stability of superstructures clearly depends on the potential direct action of wave impacts. The definition of the forces and uplifts acting on crown walls due to wave action can be found in ROM 1.1.

Verifying safety against loss of stability in wave return walls can be approximated by calculating the safety factors corresponding to sliding and overturning, taking as static forces equivalent to wave action those corresponding to maximum pressure, which are obtained from experimentally based empirical formulae (see ROM 1.1.). For this calculation, the general procedure indicated in Section 3.7 (Gravity Walls) can be followed. The safety verification is fulfilled if the safety factors obtained are equal to or higher than those given in Table 4.7.1.

Superstructures will normally rest on a granular fill with enough bearing capacity for not needing to consider the *plastic overturning* and *overall stability* failure modes. But this needs to be checked. If for some particular circumstances this aspect is critical, it should be analysed in detail. Safety can be considered verified if the safety factor against these failure modes reaches or exceeds the minimum values indicated in Table 4.7.1.

4.7.4.4 Verifying Safety against Loss of Berm Stability

The retaining and stabilising berm that may be placed at the toe of a breakwater slope may slide either along its contact with the natural soil or else along deeper failure lines that affect the natural ground.

This stability problem should be analysed following the recommendations given in the following section for calculating overall stability.

4.7.4.5 Verifying Safety against Loss of Overall Stability

Once the stability of the breakwater's armour layers is checked, the failure modes that sliding can cause are the failure of the breakwater core and of its foundation. Both failures can occur along roughly circular lines, as illustrated in Figure 4.7.8.

This section deals with deep failure. If this occurs, it will render the whole works useless. Failure of the core without involving the foundation is analysed in Subsection 4.7.4.7.

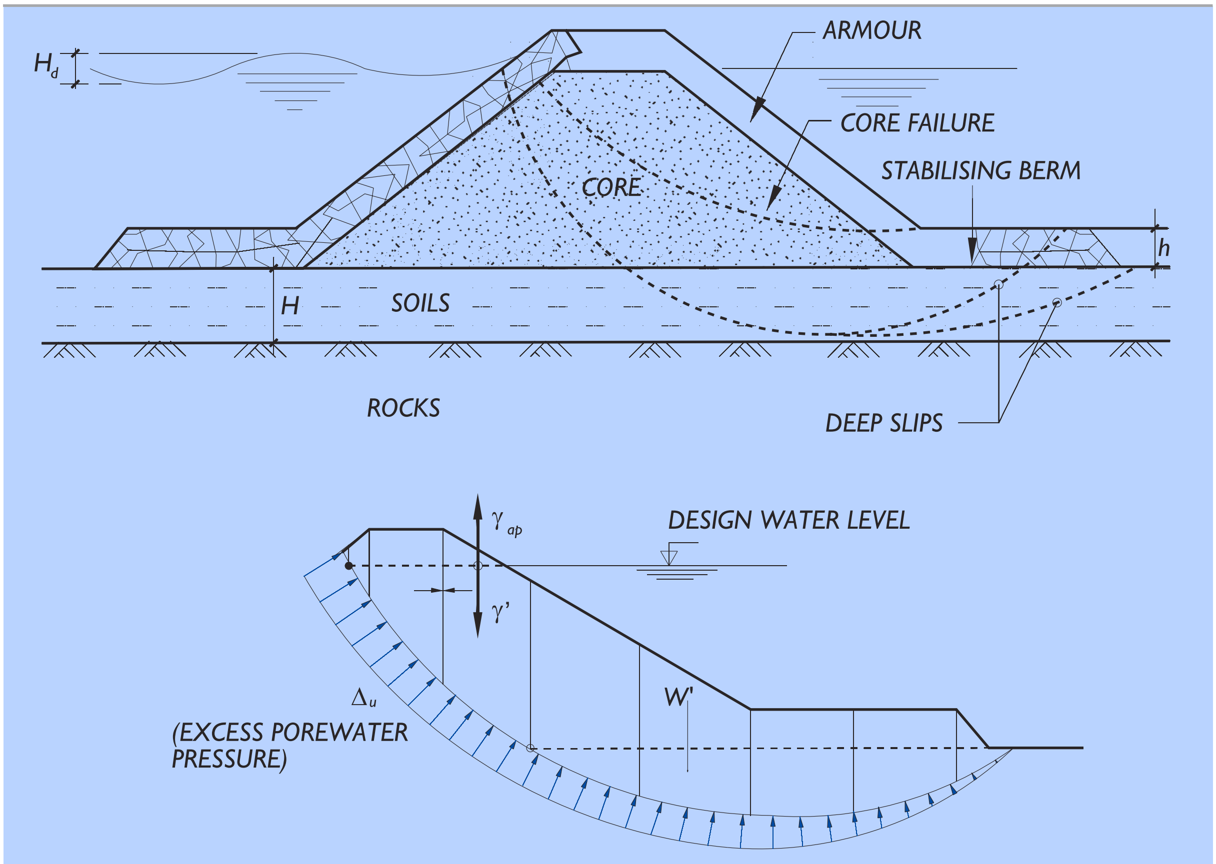
Waves should not cause substantial excesses of porewater pressure in breakwater cores as a result of densification. The porewater pressures that may be generated will be those corresponding to the transient flow created by waves and can be calculated in conditions of elastic deformation in the skeleton. The non-linear nature of this elastic response can be considered.

In order to analyse the problem of overall stability, the following design factors should be defined:

a. Defining Natural Ground Properties

Loss of overall stability can only occur when a breakwater is constructed on weak ground, which will generally also have low permeability.

Figure 4.7.8. **Breakwater Stability. Forces Acting in Ascending Water Flow Situations (Wave Run-up)**



To evaluate breakwater stability, it is necessary to previously calculate the degree of consolidation attained by the foundation. To this end, the recommendations given in Subsection 3.4.8 can be followed, or other more complex procedures adopted.

Once the degree of foundation ground consolidation is known for each design situation under analysis, strength values should be defined that are consistent with this degree of consolidation.

In this respect, three typical situations can be found:

1. Virtually Null Consolidation.

   If the degree of consolidation of the natural ground corresponding to the design situation is low (in the order of 50% or less), there will be foundation zones where the excess pore pressure is very high and the strength gain still remains small. In such cases, it is advisable to represent the foundation with a zero angle of friction and cohesion equal to the undrained shear strength that the foundation would have prior to commencement of the works.

2. Virtually Complete Consolidation.

   If the degree of consolidation of the natural ground corresponding to the design situation is over 90%, and this occurs with conservative design parameters (deduced from oedometer tests, for example), the overall stability can be analysed in terms of effective pressures, taking the cohesion and friction of the natural ground as design parameters. Excess pore pressures should not be taken into account for this reason.

3. Intermediate Situations.

In these cases, the alternatives are either adopting the most conservative assumption or else estimating the excess porewater pressures that are still to be dissipated, which will be different in each point of the natural ground, and introducing them in the calculations. With the latter option, the design parameters should be the friction and cohesion of the ground, the same as if the consolidation process had finished.

For characterising the strength of the natural ground to be considered in calculations, the effects on it caused by the dynamic action of waves and the other sea level oscillations should be taken into account. They can alter ground behaviour, introduce additional porewater pressures and hydrodynamic seepage forces into the natural ground and the central core of the breakwater and, in a word, significantly alter their resistant capacity. Subsection 3.4.11 (Porewater Pressures Generated by Waves and Other Sea Level Oscillations) and Section 3.10 (Dynamic Effects) in this ROM 0.5 include some relevant recommendations. In turn, the ROM 1.1 publication will deal more broadly with the analytical models of the joint behaviour of the soil and the works under wave action, indicating the cases to which each of them can be applied for calculating the evolution of the porewater pressures both in the natural ground and in the different breakwater layers and in its core.

b. Defining Properties of the Breakwater Core

The strength of the breakwater core should be represented in these calculations as indicated in Subsection 4.7.4.7.

c. Defining Design Situations

The most critical design situations that engineers should always analyse are as follows:

- The different Construction Stages.
- The end of construction, when the maximum load is acting on the foundation, which has only been consolidating for a short time.
- The Service Stage.

Engineers should consider other design situations that may be of interest in each specific case.

d. Defining Actions

Self-weight is the fundamental load for analysing stability problems, both localised –inside the breakwater– and overall. The shallower the slide under analysis, the more relevant the wave action will generally be. The characteristic values for the action of waves and other sea level oscillations, and their combination values, are defined in Subsection 4.7.3.2.

As a consequence of waves and the other sea level oscillations, the total pressures in the external contours of a breakwater and its foundation and the porewater pressures generated inside the structure and in the natural ground –needing to be taken into account for stability analysis– are variable in time and space. To facilitate the calculations, the following equivalent static situations can be considered:

- Water flow reaching its maximum level (wave run-up).
- Water flow reaching its minimum level (wave run-down).

with a sea level corresponding both to high tide and to low tide. As a general rule, the low-tide situation will be the most critical with respect to overall stability.

Figure 4.7.8 also represents the actions that should be considered in stability checks.

In that figure, *W'* is the weight of one of the slices of the sliding mass. This weight should be calculated with the submerged specific weights beneath sea level and with the apparent weights above that level. Variables *N'* and *S'*, respectively, are the resultants of the normal effective stresses and shear stresses along the failure line, and Δu is the distribution of excess porewater pressure –over the hydrostatic pressure– generated by waves along the failure line. It should also be assumed that a certain residual excess porewater pressure may exist due to the variation in mean sea level (tide). In any event, the distribution of the different excess (over the hydrostatic) porewater pressures will be dealt with in more detail in the ROM 1.1 publication.

Alternatively, the calculations can be done by eliminating the porewater pressures of the contour of the sliding mass and adding –in its place– the seepage forces corresponding to the corresponding flownet (see Subsection 3.4.5).

The safety factor can be calculated following the recommendations indicated in Subsection 3.8.4. The value obtained should meet the minimum requirement given in Table 4.7.1.

4.7.4.5.1 Stabilising Berms

On the occasions where breakwaters lack sufficient stability, a solution is to install berms made of rockfill or granular materials, duly protected against erosion, on each side of the breakwater.

In such cases, evaluating safety by methods assuming circular slips (Bishop's simplified method, for example) could lead to optimistic results because of the "wedge effect" explained in Subsection 3.8.4.5 of this ROM 0.5. For this reason, either the analysis method should be suitably modified or else methods used that do not involve circular failure lines nor inclinations of inter-slice forces capable of producing the said wedge effect.

Without prejudicing other methods, in such cases it is acceptable to substitute the beneficial effect of the berm by a horizontal external force equal to:

$$E = \frac{1}{2}\gamma' K_{ph}\, h^2$$

where:

γ' = submerged unit weight of the material forming the berm.
K_{ph} = coefficient of horizontal passive pressure, estimated with $\delta = 0$ (see Subsections 3.7.6 and 3.7.7)
h = thickness of the berm at the point where the line of slide emerges.

For the purpose of calculating this thrust, the angle of friction of the berm should be taken as:

$$\tan\phi_{(design)} = \frac{1}{F}\tan\phi$$

where:

$\phi_{(design)}$ = angle of friction for this calculation.
ϕ = angle of friction of the berm.
F = safety factor against sliding along the trial failure line.

Since the safety factor against sliding, *F*, is an unknown at the time of evaluating force *E*, this should be estimated beforehand by assuming a reasonable value for *F* and then adjusted by some simple iterative process.

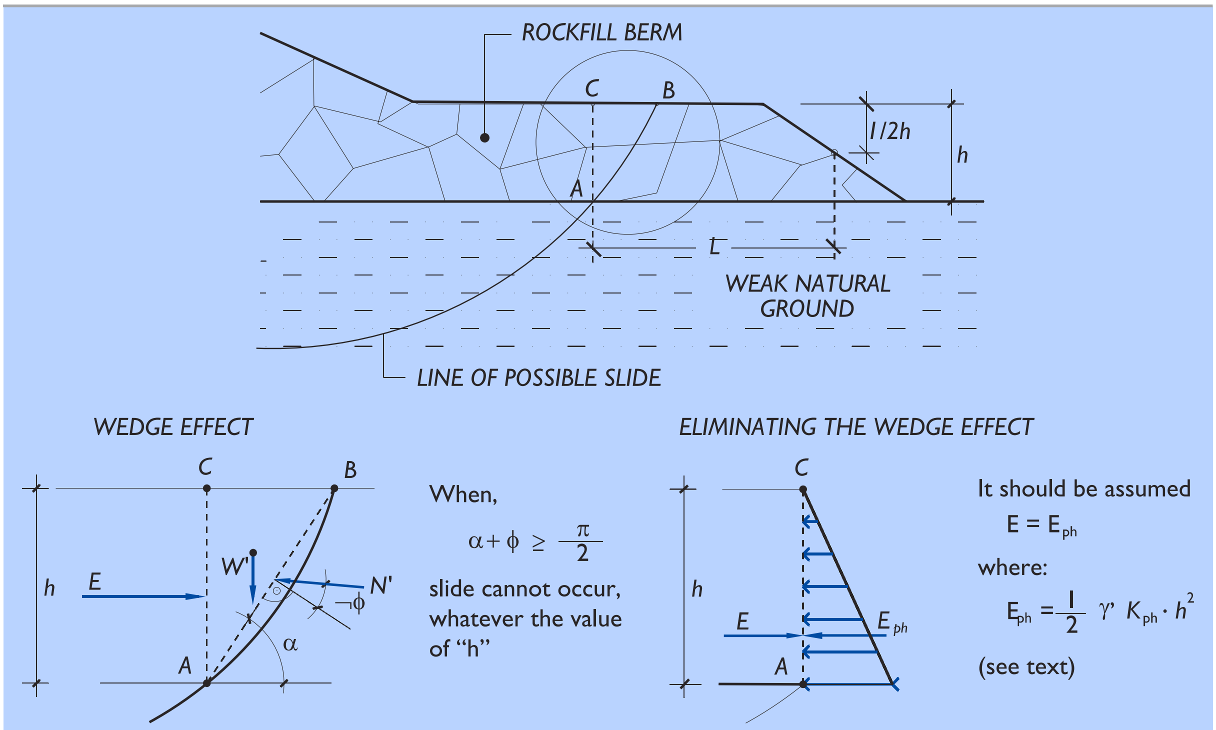
In addition, the thrust *E*, thus calculated, should be checked to remain lower than the one that would give rise to the condition of horizontal sliding in the associated berm section, i.e.:

$$E_{(max)} = \frac{\gamma' \cdot L \cdot h \cdot \tan\phi_s + c_s\left(L + \frac{1}{2}\cdot h\right)}{F} \quad \text{or} \quad E_{(max)} = \frac{\gamma' \cdot L \cdot h \cdot \tan\phi}{F}$$

depending on whether the plane sliding is produced through the natural ground (strength parameters c_s, ϕ_s) or through the berm itself.

This simplified calculation procedure is illustrated in Figure 4.7.9.

Figure 4.7.9. **Overall Stability Improvement using Berms**



4.7.4.6 *Verifying Safety against Internal Erosion*

Moving water around a breakwater will cause flows and backflows in the area close to the faces that can cause entrainment of the material forming the core of the breakwater through the armour layers. The appropriate sequence of grading curves of the different materials must be studied either numerically, as explained in Subsection 3.4.7 of this ROM 0.5, or experimentally by small-scale model tests.

4.7.4.7 *Verifying Safety against Loss of Stability in the Breakwater Core*

Failure of the breakwater along roughly circular slip lines that basically develop inside its core is a failure mechanism that engineers should look into. This type of failure is shown in Figure 4.7.8 (core failure).

The strength of the core, which will normally be built of rockfill or quarry run, will have previously been analysed when prospecting for borrow materials as described in 4.7.2.2.

If there are finer soils in the body of the core that could be sensitive to the alternating loads produced by waves, the recommendations given in Section 3.10 should be followed to analyse their strength.

The verification of safety against loss of stability in a breakwater core should follow the indications laid down in Subsection 4.7.4.5, as it can be treated as a particular case of overall stability. Figure 4.7.8 also represents this failure mode.

As with the verification of safety against overall loss of stability, the definition of the porewater pressures along the failure line is a very complex problem, particularly in the case of cores not behaving as drained, rigid bodies under the dynamic action of waves and other sea level oscillations. Sections 3.4 and 3.10 of this ROM 0.5 and the ROM 1.1 publication give recommendations on the subject.

To analyse core stability, several trial failure lines should be analysed, affecting both seaward and leeward breakwater slopes.

The safety factors to be obtained must meet minimum requirements like those given in Table 4.7.1.

Table 4.7.1. Minimum Safety Factors for Rubble-Mound Breakwaters (for Occurrence Probability of Failure Modes in the Order of 0.01)

Section where the Associated Calculation Method is Defined	Ultimate Limit States of Geotechnical* (GEO)	Load Combination		
		Quasi-Permanent F_1	Fundamental or Chracteristics F_2	Accidental or Seismic F_3
3.8.4.4	Surface sliding of cover layer	1.2	1.1	1
3.5.5 3.5.6 3.8.4.5 and 3.8.4.6	Loss of superstructure stability: Sliding, overturning and overall stability	1.2	1.1	1
3.8.4.5 and 3.8.4.6	Loss of berm stability	1.3	1.1	1
3.8.4.5 and 3.8.4.6	Loss of overall stability	1.3	1.1	1
–	Internal erosion	PM	PM	PM
3.8.4.5 and 3.8.4.6	Breakwater core failure	1.3	1.1	1
–	Scour in natural bed	PM	PM	PM

* Those mainly governed by ground strength.
PM In these cases, safety is not usually quantified. The problem can be circumvented by taking adequate preventive measures (PM).
Note 1: Before using these safety factors, it is necessary to be familiar with the associated analysis methods defined in this ROM 0.5, as described in Section 4.7 and in the subsections appearing in the first column.
Note 2: These safety factors are adequate provided that the occurrence probability admitted for each failure mode is in the order of 0.01 and the standard ROM 0.5 verification approach is applied (see Subsection 4.7.3). For other failure probabilities, the minimum safety factors set in this table should be modified in line with the criteria established in Subsections 3.3.8 and 3.3.10 of this ROM 0.5.
Note 3: In cases where a specific verification method is chosen, because the failure probability under consideration is higher than or equal to 0.05, the minimum required safety factors should be obtained on the basis of the formulation from Subsection 4.7.3.2.

4.7.4.8 Verifying Safety against Scour in the Natural Bed

Erosion of the natural ground in the area close to the toe or the bottom portion of the slope of breakwaters is an important, difficult to quantify problem. It can be assessed (with limited confidence) either empirically by means of equations derived from results obtained in moving-bed physical models, or else by rules of good practice based on experience or on field observations. In spite of the experiments carried out, dependable techniques have still not been developed for estimating the extent of these erosive processes, which appear to depend on the structure and permeability of the slope, the incident wave conditions, the draught and the natural ground characteristics. Its effect can be considered to be greater when the structure's reflection coefficient increases. In other words, in breakwaters with more moderate slopes and greater permeability, scour effects are far smaller. ROM 1.1 will deal with the verification of safety against this failure mode in greater detail.

This erosive action may be accentuated when cross currents act jointly with the waves.

As an order-of-magnitude estimate, during storms and in sandy beds close to the toe of breakwaters, scour or transient movements have been observed up to a depth (beneath the original bed line) greater than the wave height.

Depending on the nature of the seabed, it may be necessary to extend the rubble-mound armour horizontally to cover the natural soils close to the toe and keep zones of potential erosion at a greater distance away from the breakwater. Notwithstanding, this solution may have harmful effects on the breakwater, as it is capable of making the wave steeper and may even go so far as to provoke its breaking.

Subsection 4.3.5.7 makes some recommendations that could be of interest in this case.

4.7.4.9 Minimum Safety Factors

The safety factors against each of these failure modes, covered in preceding sections, have been cited when describing the particular failure mode and making recommendations on the analysis procedure. Table 4.7.1 sums up the minimum safety factors recommended. These factors are adequate only when the failure probability considered for the failure mode is in the order of 0.01 and the standard verification method from this ROM 0.5 is applied. For other failure probabilities, the minimum safety factors fixed in that table should be modified in line with the criteria given in Subsections 3.3.8 and 3.3.10. In turn, in cases where a specific verification method is chosen, because the failure probability under consideration is higher than or equal to 0.05, the minimum required safety factors should be obtained on the basis of the formulation from Subsection 4.7.3.2.

These safety factors are adequate only for the calculation method indicated in the table. Design engineers may justify the use of other calculation procedures and other associated safety requirements for their projects.

4.7.4.10 Settlement

Rockfill breakwaters adapt well to foundation deformation. Settlements must be estimated, nevertheless, in order to:

- Ascertain their potential effect on the elevation of the breakwater crest or its heightening.
- Evaluate their effect on any structures placed on the crest (wave reflecting walls, pavements, etc.).

Deferred settlement occurring after construction and caused by deformation of the natural ground should be calculated by applying adequate soil mechanics methods.

Subsection 4.7.2.2 indicates some causes of deferred settlement originated by rockfill compression.

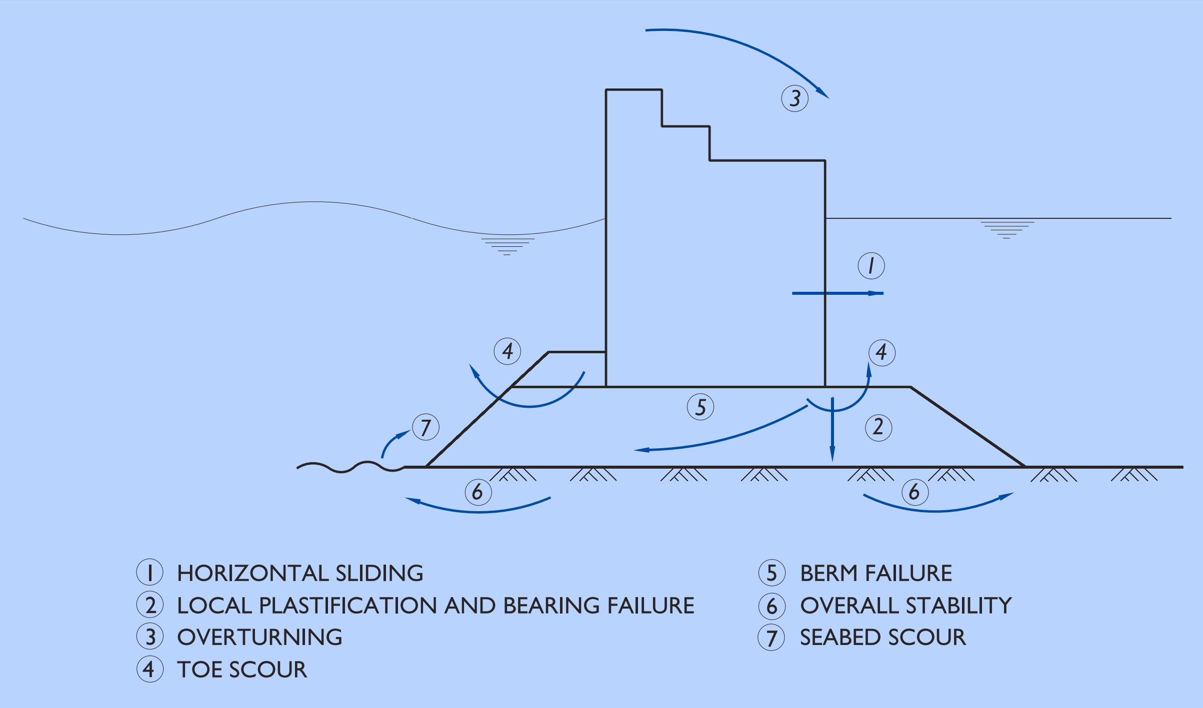
4.7.5 Vertical Breakwaters

Vertical breakwaters may undergo failure modes specific to this type of works. This section analyses the most important geotechnical failure modes assigned to Ultimate Limit States and which, in any event, should be considered in the design (see Fig. 4.7.10).

To verify the safety of vertical breakwaters against geotechnical failure modes, the predominant variable load is the one due to wave action and other sea level oscillations. The characteristic, design and combination values of this action can be obtained on the basis of the criteria defined in Subsection 4.7.3.2.

It is difficult to evaluate the pressures on the breakwater structure or central body and uplifts on its foundation, as also the loads transmitted to the foundation berm and the porewater pressures generated both in the berm and in the natural ground by the design storm. Therefore, specific dynamic computations of soil-structure interaction should be done under this action. Alternatively, simplified equivalent empirical procedures of a static nature and acknowledged validity should be applied and contrasted in each particular case by means of physical model tests. All these aspects will be covered in greater detail in the ROM 1.1 publication, but some specific recommendations are included in Section 3.10 (Dynamic Effects) of this ROM 0.5, due to the cyclic

Figure 4.7.10. **Geotechnical Failure Modes Assigned to Ultimate Limit States and Associated with Vertical Breakwaters**



nature of wave action and the possibility that significant dynamic behaviour can occur in some vertical breakwaters.

The dynamic behaviour of vertical breakwaters depends on the period and magnitude of the wave action and especially on the combined response of the soil and structure, i.e. on their displacements under this load. As have been said in Subsection 3.10.1, this is particularly significant when the wave action has a period close to one of the natural oscillation periods of the soil-structure system. In such cases, substantial dynamic amplifications of the loads transmitted to the foundations can be expected.

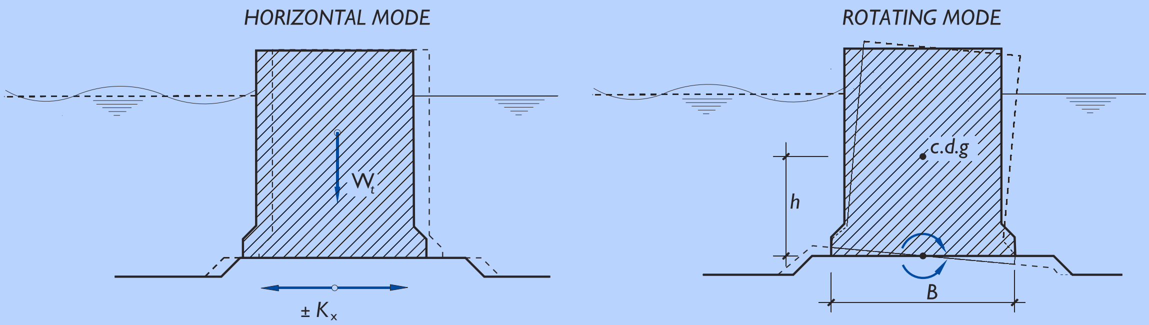
In the absence of a better approximation, the natural oscillation periods of vertical breakwaters corresponding to fundamental modes in the direction the load is acting can be estimated via the formulae included in Figure 4.7.11.

To estimate natural oscillation periods by these formulae, the deformational behaviour of the ground under dynamic loads should be taken into account. In this respect and to define K_x, it is necessary to consider that the parameters defining the soil behaviour have different values under dynamic loads, for which recommendations are made in Section 3.10 of this ROM 0.5.

As a general rule, vertical breakwaters tend to have natural oscillation periods ranging from 0.2 to 2 seconds, far removed from the periods of waves and other sea level oscillations. On the contrary, they come very close to the periods corresponding to the response of the structure under the impulsive load associated with the action of breaking waves, which is why significant dynamic behaviour in vertical breakwaters should be expected only in these cases.

The vertical-face structure making up the breakwater will demand an adequate preparation of the foundation. Gravity structures similar to those described in this ROM 0.5 for quays (Sections 4.2 and 4.4) should follow the recommendations made in these sections and those in Section 4.9 relative to dredging and earthfills beneath structures.

Figure 4.7.11. **Natural Oscillation Periods of Vertical Breakwaters Corresponding to Fundamental Modes**



$$T_h = 2\,\pi \sqrt{\frac{W_t}{g \cdot K_x}}$$

$$T_\Theta = 2{,}2\ T_h \frac{h}{B}$$

W_t = total emerged weight of the structure.
g = gravity acceleration.
K_x = horizontal spring constant of the foundation (see Subsection 3.54.7).

h = height above foundation of the centre of gravity of the total mass of the superstructure.
B = foundation breadth.

Given the cyclic and impulsive nature of the loads acting on the foundation of this type of structure, any type of ground that is particularly sensitive to alternating loads, as indicated in Section 3.10, and that could remain beneath the foundation should be replaced or improved.

4.7.5.1 Verifying Safety against Horizontal Sliding of the Structure

Sliding of gravity structures along their plane of contact with the natural ground or with the levelling berm should be analysed following the general criteria common to all works (Section 3.5, Shallow Foundations), as well as the specific recommendations given in this ROM 0.5 for structures similar to those forming the central body of the breakwater (Section 4.2, Gravity Quay Walls). Notwithstanding, the minimum safety factors will be those defined in Table 4.7.2.

Table 4.7.2. **Minimum Safety Factors for Vertical Breakwaters (for an Occurrence Probability of the Failure Mode in the Order of 0.01)**

Section where the Associated Calculation Method is Defined	Ultimate Limit States of Geotechnical* (GEO)	Load Combination		
		Quasi-Permanent F_1	Fundamental or Chracteristics F_2	Accidental or Seismic F_3
3.5.5	Sliding along the contact of concrete and supporting berm	1.3	1.1	1
3.5.4	Bearing failure	1.8	1.5	1.2
3.5.6 y 3.7.11.1.2	Plastic overturning	1.3	1.2	1.1
3.8	Overall instability	1.3	1.1	1
–	Erosion and scour	PM	PM	PM

* Those mainly governed by ground resistance.
PM In these cases, safety is not usually quantified. The problem can be avoided by taking adequate preventive measures (PM).
Note 1: Before using these safety factors, it is necessary to be familiar with the associated calculation methods defined in this ROM 0.5, as described in Section 4.7 and in the sections and subsections appearing in the first column.
Note 2: These safety factors are valid provided that the occurrence probability admitted for each failure mode is in the order of 0.01 and the standard verification approach from this ROM 0.5 is applied (see Subsection 4.7.3). For other failure probabilities, the minimum safety factors set in this table should be modified in line with the criteria established in Subsections 3.3.8 and 3.3.10 of this ROM 0.5.
Note 3: In cases where a specific verification method is chosen, because the failure probability under consideration is higher than or equal to 0.05, the minimum safety factors should be obtained on the basis of the formulation included in Subsection 4.7.3.2.

4.7.5.2 *Verifying Safety against Bearing Failure, Local Plastification and Plastic Overturning*

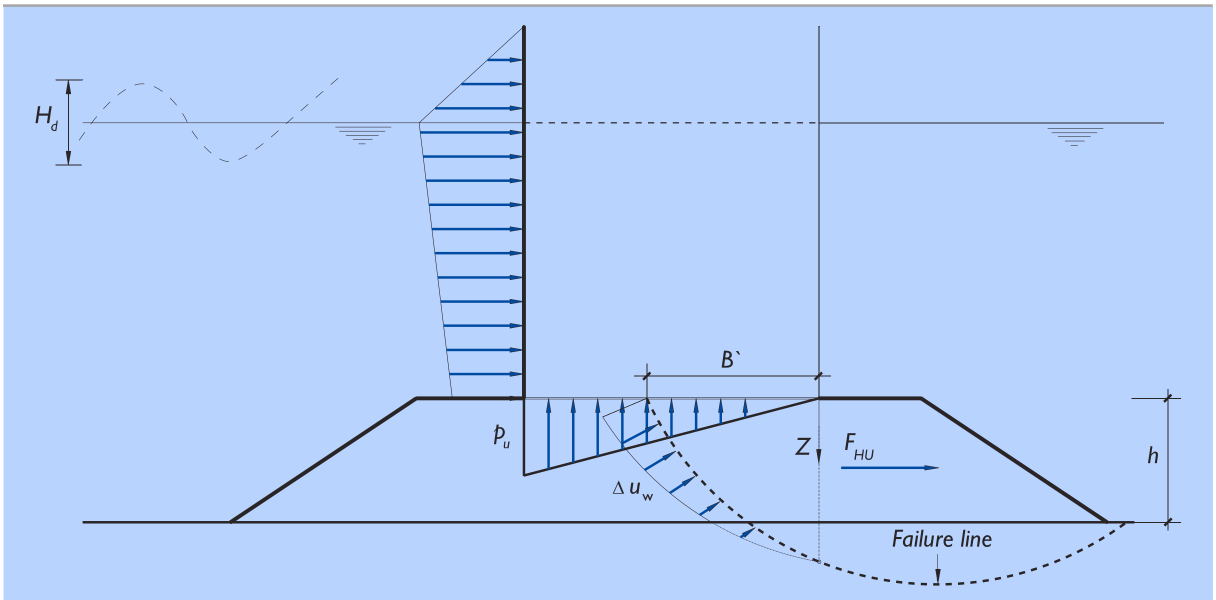
The loads acting on the foundation will cause concentrated stresses in the contact between the superstructure and the levelling berm, whose maxima will lie in the vicinity of the seaward and leeward toes of the structure. These stresses can give rise to local plastification of the rockfill and for this reason should be limited as indicated in Subsection 4.2.5.

Bearing failure and plastic overturning can be checked following the general recommendations given in this ROM 0.5 (Subsections 3.5.4, 3.5.6, and 3.7.11.1.2) and the specific recommendations given in Part 4 for similar harbour structures, introducing the effects of additional porewater pressures generated by the dynamic action of waves. In these cases, the minimum safety factors should be those defined in Table 4.7.2.

Notwithstanding, in calculations for safety factors against bearing failure or plastic overturning of foundations on berms or granular materials, the effect of porewater pressures generated by the dynamic wave action is difficult to take into consideration using this methodology. For these reasons, it is generally more rigorous to tackle them with adequate numerical models or equate these failure modes to those of partial instability (with specific failure shapes) and to use the methods described in Section 3.8 of this ROM 0.5. In the latter case, the porewater pressures generated along the failure surface should intervene in the computations. Moreover, it is necessary to consider not only the equivalent static situations of the passages of the wave crest and trough, but also other intermediate ones, and with the sea level corresponding both to high tide and low tide. Some recommendations are included in Sections 3.4 and 3.10 of this ROM 0.5, as also in the ROM 1.1 publication, for obtaining the distributions of the porewater pressures generated by the cyclic or impulsive action of waves and other sea level oscillations, as a function of whatever behaviour model is used for the structure-berm-natural ground system under this load.

In the case of highly permeable and small-sized berms in relation to the central body of the breakwater, the porewater pressures being generated in the berm by the wave action can be considered to change little vertically. For this reason, when the pressures and uplifts acting on the breakwater are approximated using adequate formulae of acknowledged validity, the distribution of the porewater pressures along the failure line can be estimated based on these dynamic uplift distributions (see Fig. 4.7.12).

Figure 4.7.12. Porewater Pressures Generated by Waves on Berms and the Natural Ground for Verifying Safety (Bearing Failure and Overturning)



$$F_{HU} = \int_0^h \Delta u_w \cdot dz$$

If, in any event, the simplified analysis procedures for bearing failure and overturning included in this ROM 0.5 are used, the effect of porewater pressure can be approximately considered by adding to the loads acting on the foundation a horizontal force, F_{HU}, resulting from the pore water pressures generated by the waves along the failure line in each design situation analysed (see Fig. 4.7.12). In a simplified manner, this horizontal load can be considered to be applied at the level of the foundation plane.

Caissons founded on natural ground composed of fine silty sands can undergo a specially significant change in their foundation behaviour owing to the dynamic wave action, from a drained to an undrained situation -with the corresponding additional increase in porewater pressures. Nevertheless, this effect is not significant in berm gravels and rockfills. In the quarry run inside berm cores, this point will need to be confirmed in each specific case.

In granular soils with a relative density of over 80% and in firm cohesive soils ($q_u > 0.1$ MPa) with over 35% of fines (passing an 0.080 UNE sieve), it will not be necessary to take into account the excess porewater pressures due to the dynamic nature of wave actions.

To chose the foundation strength parameters to be used in checking safety against overturning and bearing failure, the foundation's drained or undrained behaviour under wave action should be taken into account. In the case of cohesive or low-permeability soils, it is also necessary to consider the degree of consolidation attained at the time corresponding to the design situation under study. The recommendations given in Subsection 4.7.4.5 apply in this respect.

4.7.5.3 Verifying Safety against Loss of Overall Stability

The overall equilibrium of breakwaters always needs to be checked whenever soils or weathered rocks of poor strength exist underneath their foundation. The general computation principles to be used are those from Section 3.8 of this ROM 0.5. The porewater pressures generated along the failure line by the dynamic action of waves and other sea level oscillations should be introduced in the calculations, taking as equivalent static situations the wave crest and trough passages, with the sea level corresponding both to high tides and low tides. Some recommendations are included in Sections 3.4 and 3.10 of this ROM 0.5, devoted to dynamic effects, and to be detailed in the ROM 1.1 publication, on how to obtain the distribution of the porewater pressures generated by the cyclic or impulsive action of waves and other sea level oscillations, as a function of whether drained or undrained behaviour is taken for the natural ground under this load.

Figure 4.7.13 represents the distribution of excess porewater pressure (over the hydrostatic values) to be taken into account along the failure line.

In the case of highly permeable and thin berms, the porewater pressures being generated in the berm due to wave action can be considered to change little in the vertical direction. For this reason, when approximating the pressures and uplifts on the breakwater central body using appropriate and sufficiently validated formulae, the distribution of the porewater pressures along the section of the failure line included in the berm can be estimated in a simplified manner from diagrams of dynamic uplift such as that shown in Fig. 4.7.12.

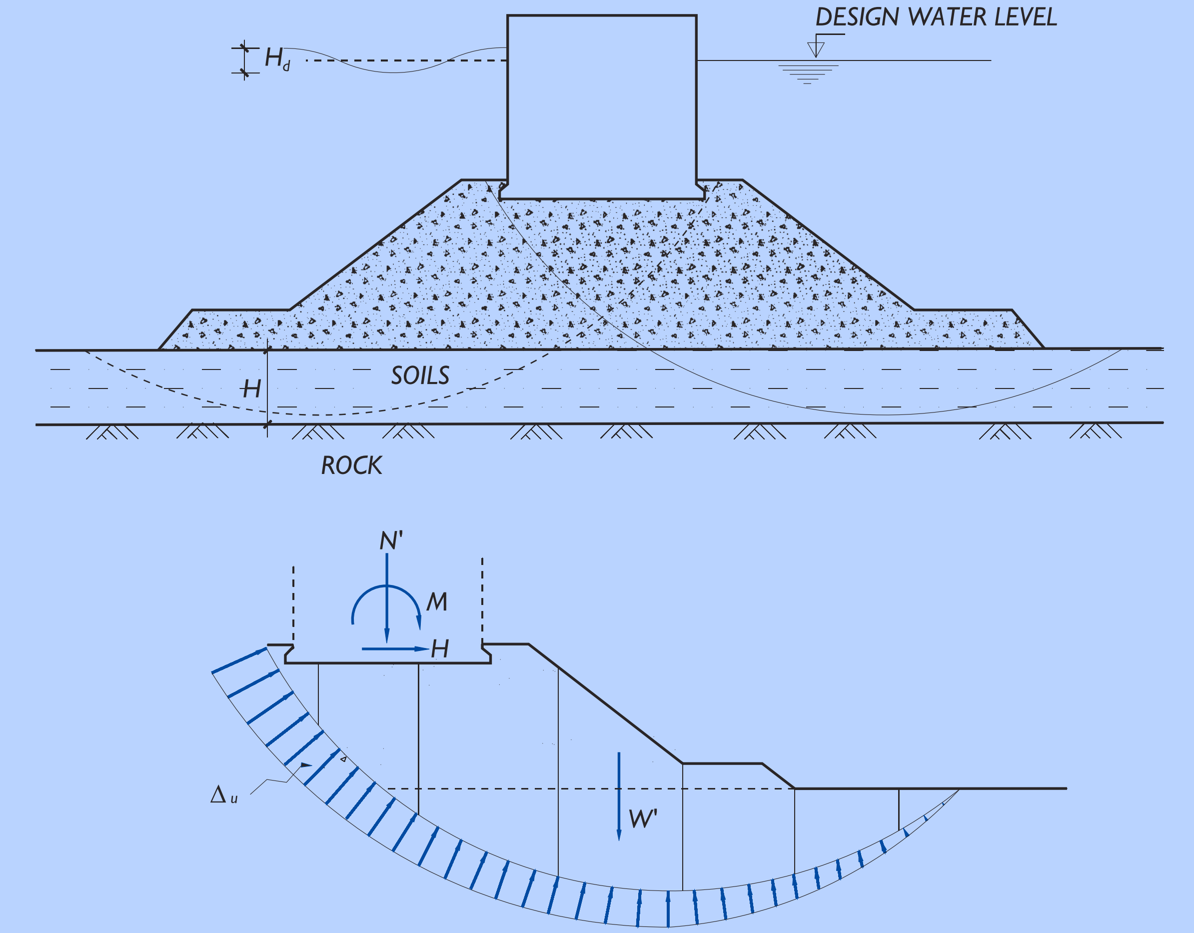
These excess porewater pressures can be taken into account in the calculations either directly or by replacing them with seepage forces producing the equivalent effect (see Subsection 3.4.5).

4.7.5.4 Verifying Safety against Scour and Erosion

Scour in the seabed caused by water currents, modified by the presence of the breakwater, as also by the waves, constitutes an aspect of considerable interest. The remarks made in 4.7.4.8 apply in this case.

Erosion around the toe or in the possible supporting berm of vertical breakwaters can have serious consequences because, if the structure is undermined, this situation would be difficult to repair. The future ROM 1.1 will enlarge on the verification of safety against this failure mode.

Figure 4.7.13. **Porewater Pressures Generated by Waves inside Berms and Natural Ground to Consider for Verifying Safety against Loss of Overall Stability**



Δu = excess porewater pressure over the hydrostatic values corresponding to the design level.

4.7.5.5 Settlement and Deformations

As the breakwater structure may be very stiff and the actions involved are alternating in nature, there exists a certain degree of risk that the breakwater exceeds a Serviceability Limit State as a result of deformations.

Breakwater settlement can be calculated using the conventional soil mechanics procedures, even though the cyclic nature of the loads should be taken into account when defining the deformation moduli, at least indirectly, as it can originate some densification and consequently some settlement.

The rockfill berm can be the cause of rather important post-construction settlement (see Subsection 4.7.4.10 devoted to settlement in rubble-mound breakwaters).

The horizontal displacements and rotations (i.e., pitch) produced by the cyclic water pressures can be calculated with the static formulae of the theory of elasticity and using deformation moduli that take into account the effect of cyclic loads.

When a breakwater has a significant dynamic response, its deformation analysis may require the use of more complex dynamic calculation methods (see Section 3.10).

4.7.5.6 Minimum Safety Factors

The minimum safety factors required must be consistent with the definition of the loads used for calculating them and with the failure probabilities allowed.

Table 4.7.2 summarises the minimum safety factors recommended against each of the failure modes covered in the preceding sections. These factors are valid when the occurrence probability admitted for this particular failure mode is in the order of 0.01 and the standard verification approach from this ROM 0.5 is adopted. For other failure probabilities, the minimum safety factors set in that table should be modified in line with the criteria laid down in Subsections 3.3.8 and 3.3.10. In turn, when a specific verification procedure is chosen, because the failure probability under consideration is higher than or equal to 0.05, the minimum safety factors required should be obtained according to the formulation of Subsection 4.7.3.2.

These safety factors are adequate only for the calculation methods indicated in the table. Design engineers may justify the use of other calculation procedures and other associated safety requirements for their projects.

4.8 OFFSHORE PLATFORMS

Offshore constructions in very deep seas have particular features compared to shoreline constructions, among which the following should be pointed out:

- Decisive importance of wave and wind action compared to other loads.
- High eccentricity of the horizontal loads with respect to the supporting plane.
- Dynamic nature of the loads.
- Far higher cost of geotechnical investigations.

The construction techniques and design methods of these structures require that certain specific matters be taken into account. The most important problems, from the geotechnical point of view, that should be tackled in the design are covered below.

4.8.1 Types

The types of offshore platform commonly constructed vary greatly. Figure 4.8.1 illustrates some of them.

Jack-up rigs are floating structures with large legs (at least three) that, as their name implies, can be jacked up or lowered. They are towed to the site with the legs retracted and once on site are lowered until they rest on the seabed. The floating structure is raised on the legs and an overburden is even applied by temporary water ballast to drive the legs into the sediments on the sea floor.

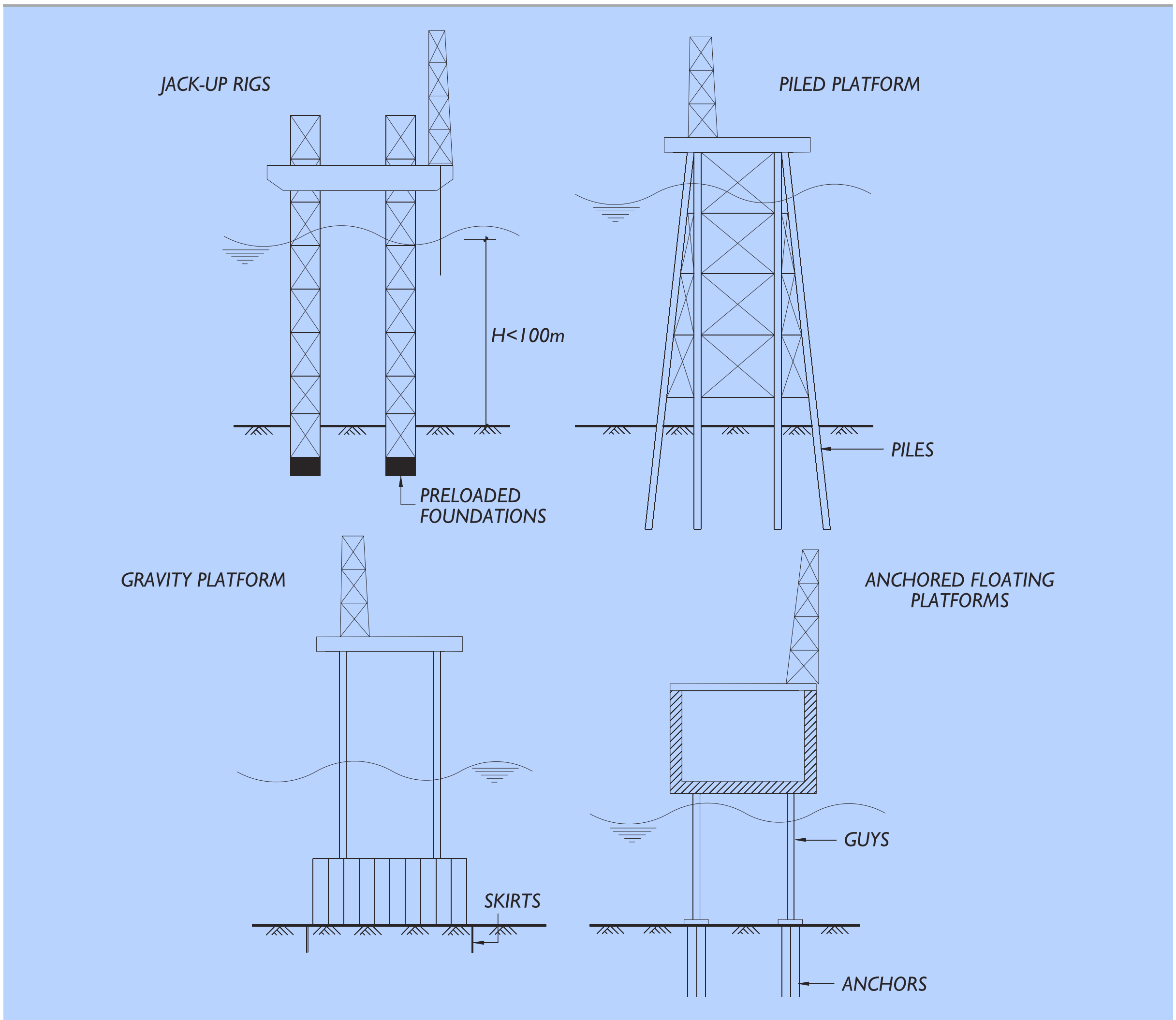
Piled platforms, typical in the Gulf of Mexico, are spatial lattice steel structures supported on the seabed by driven piles.

Gravity base platforms with shallow foundations are the usual type of European constructions in the North Sea.

Anchored floating platforms are also constructed, anchored to the seabed by tethers (tubular steel structures), which in turn transmit their load to the ground by tension piles. Other types of anchorage systems can also be used.

More recently, interest is focused on constructing lightweight structures similar to steel towers, resting on the sea floor and held by guys (lateral tethers anchored to the ground). They have the advantage of their high flexibility and, therefore, of transmitting lower loads to the ground.

Figure 4.8.1. ***Types of Offshore Platforms***



For the purposes of making the following geotechnical recommendations, a distinction will be drawn between two basic types of foundation - shallow and deep.

Jack-up platforms have a somewhat intermediate type of foundation. At the base of their legs, they have special shaped footings that sink into the ground to a certain depth. The techniques for analysing the safety of this type of foundation are far from conventional and are considered to lie outside the scope of this ROM 0.5.

4.8.2 Wave and Wind Action

The main loads on an offshore platform are due to the waves acting on the platform support system and the wind acting on the superstructure.

These actions are clearly dynamic so, when geotechnical calculations are made to check Ultimate Limit States, it will be necessary to apply certain dynamic load factors to the actions if equivalent static calculation procedures are used.

These factors will depend on the energy spectrum of the loads and on the period and damping of the structure response. The solution to this interaction problem is considered outside the scope of this ROM 0.5.

4.8.3 Geotechnical Investigation

Geotechnical investigation of the sea floor at sites where offshore platforms are to be constructed is aimed, as in all other cases, at ascertaining the ground structure and its mechanical properties. Investigation programmes for the study of offshore platforms, however, have special features, including:

- Considerable draughts and exposed sites, necessitating the use of specialised equipment.
- Considerable exploration depths (100-150 m for piled platforms and 100 m for gravity base platforms).
- High costs for the investigation, which is usually structured into two stages. Before utilising direct investigation procedures, geophysical exploration is generally used, mainly seismic reflection.
- The difficulty of guaranteeing the position of the spot tested.
- The difficulty of obtaining quality samples (drilling system, large variation in the hydrostatic pressure between the sampling point and the surface, etc.).
- The difficulty of gaining access to a laboratory. Having a soil laboratory on the geotechnical vessel is desirable.

Direct investigation is normally based on the same general principles as those corresponding to shoreline zones. In other words, it requires boreholes and sampling, in this case with equipment located on the seabed and surface operated from special vessels.

The most frequently used *in situ* tests are static penetrometers (piezocones), pressuremeters, vane shear tests, etc. These tests are described in Part 2 of this ROM 0.5.

The dynamic behaviour of soils in the sea floor is generally more significant for these platforms than for the majority of harbour works. The most common dynamic tests are described in Part 2 of this ROM 0.5 and the characteristics that can be deduced from them are referred to in Section 3.10.

4.8.4 Seabed Stability

The seabed can be affected by instabilities due to a wide variety of causes, including:

- Sea currents.
- Liquefaction caused by the cyclic shear stresses induced by the movement of water on the seabed.
- Instability caused by sediment overburden in riverwash zones.
- Liquefaction due to earthquakes.

Seabed instability can cause sliding in virtually plain areas (it has been observed in areas with slopes as low as 2%) and it can affect a sediment mass of great thickness (some slides have been known to affect depths of over 40 m).

The soils where sliding usually occurs are muds and fine sands. The extent of the slide (distance from head to toe) and the distances these soils can displace are much greater than those usually observed on land. The construction of offshore platforms in areas with an unstable bed is always difficult and risky and so a large part of the general investigation efforts in the surroundings of the platform (topography and geophysics) should be aimed at ensuring that the area in question is naturally stable.

4.8.5 Shallow Foundations

Spread foundations for offshore platforms may take a wide variety of forms. The type considered here will consist of a large support area with similar dimensions to (usually somewhat less than) the draught.

Shallow foundations are normally constructed with minimal or no preparation at all of the contact area. The foundation block is normally precast on land, transported by flotation and founded on the sea floor at the installation site.

Gravity base platforms normally incorporate dowels in their base to guarantee the stability of the structure during the initial founding stages. These dowels penetrate the ground at a time when the platform is still subject to movement as a result of external loads, mainly wave action, and they provide provisional stability.

Shallow foundations are normally protected laterally by diaphragm walls or *skirts* (made of steel, reinforced concrete or other similar elements) that are driven after founding the base. In this way, the edges, which generally need to withstand higher stresses, are subject to a certain amount of lateral confinement. These armour skirts also substantially add to the foundation's sliding resistance.

The confinement achieved by lateral skirts make it possible to subsequently carry out grouting treatments to improve the ground and consolidate the contact zone between the ground and the foundation structure.

Gravity-base offshore platforms with shallow foundations transmit moderate vertical stresses that are no greater than those of other harbour works with much less draught.

The dynamic nature of the loads and the fact that the sea floor cannot be properly prepared mean that direct foundations are restricted to firm soil conditions, i.e., dense sandy soils and overconsolidated clayey soils.

The Limit States for shallow foundations in offshore platforms are largely the same as with the other spread foundations described in this ROM 0.5.

Matters to be considered should include the following:

- Horizontal stability under the least favourable conditions.
- Stability against overturning due to foundation failure under the most unfavourable conditions.
- Overturning and sliding stability under cyclic load conditions.
- Horizontal and vertical deformability under cyclic loads.
- Determining the maximum contact pressures on the base, taking the unevenness of the seabed into account.
- Verifying the condition of non-resonance with external loads.
- Verifying erosion in the area around the structure.
- Analysing the behaviour of the skirts for increasing the ground strength and preventing erosion.

Stability problems should also be checked during the founding stage. These checks will make it possible to properly design the temporary retaining system of dowels.

Some specific remarks are made in the following paragraphs concerning the Ultimate Limit States of these platforms. However, only some of those primarily governed by ground characteristics are considered.

4.8.5.1 Static Analysis

Ultimate Limit States affecting these foundations normally occur as a result of wave and wind action, which originate a temporal history of reactions on the foundation.

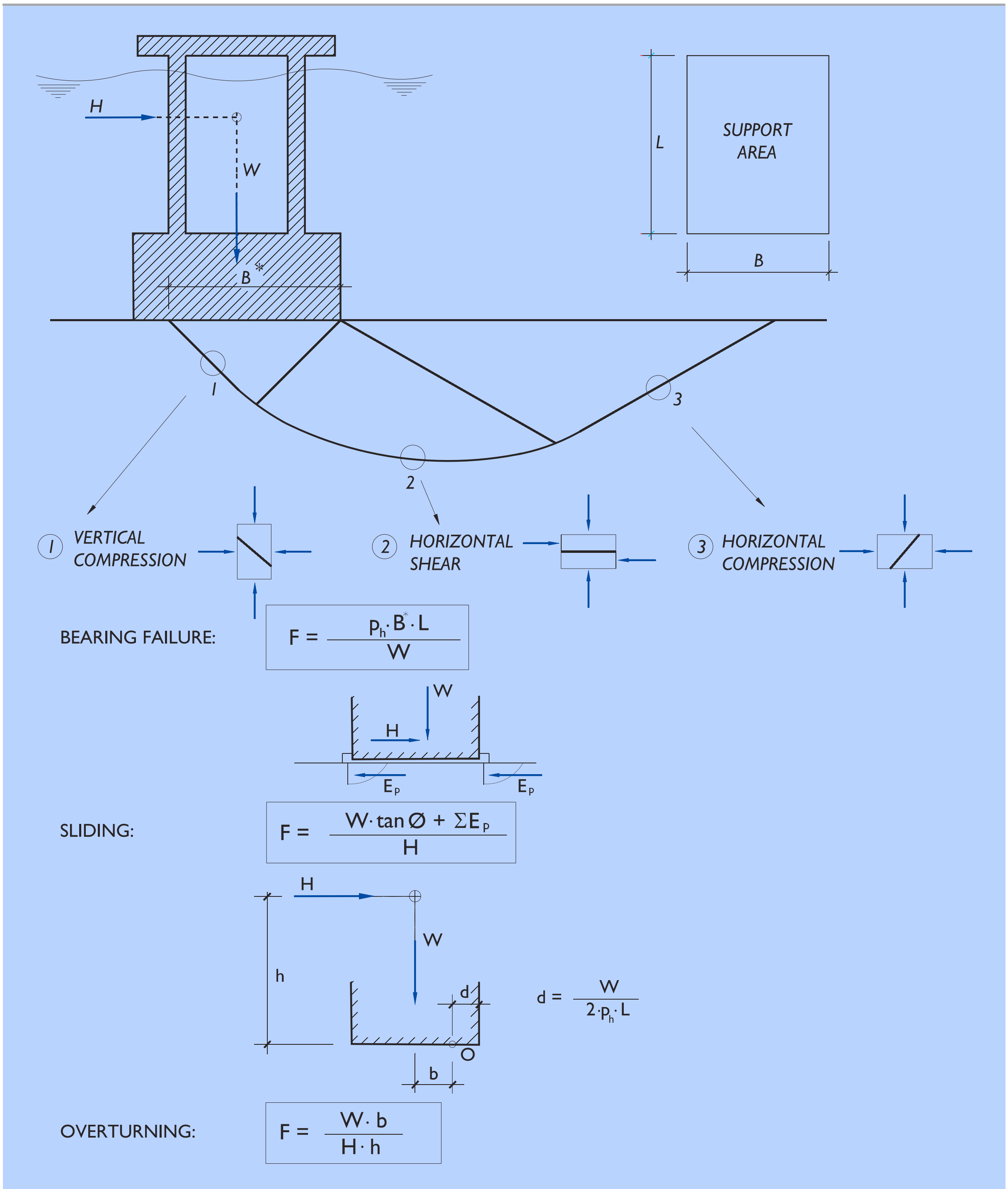
The Ultimate Limit States can be analysed in two clearly different ways. For the purpose of these recommendations only, they will be called *static analysis*, covered in this section, and *dynamic analysis*, covered in the following section.

The basic principle behind static analysis lies in representing the storm capable of bringing about the Limit State by an equivalent static load on the foundation. This load can be considered as due to the maximum wave in the storm or to another wave that could represent, individually, the design storm better. In any event, certain equivalent static forces represent dynamic loads.

The justification for this method is based on current experience of the behaviour of firm soils on which shallow foundations for gravity base platforms are normally located. The shear strength of these soils, under a maximum wave, is independent of the previous passage of other waves. This holds true provided that the soil is far from failure, i.e., provided that the safety factors obtained by this procedure are high.

Once the loads are known, the eccentricity and inclination of the resultant can be obtained. Then, the procedures for analysing safety against bearing failure, overturning and sliding can be applied and the deformation analysed. The typical failure modes of these foundations are shown in Figure 4.8.2.

Figure 4.8.2. ***Typical Failure Modes of Gravity-Base Offshore Platforms***



For the mechanism of horizontal sliding, the passive pressure of all the skirts should be accounted for, provided that this does not imply greater safety than the one corresponding to a deeper-seated slide passing below their tips and provided that the skirts have sufficient structural resistance to transmit the corresponding stresses.

The strength parameters to be used in these calculations (sliding, bearing failure and overturning) differ somewhat, however, from the parameters usually used.

The strength parameters should be obtained by taking the dynamic nature of the loads into account, as shown in Section 3.10.

In firm clays and dense granular soils, the dynamic shear strength may be even greater than the static strength, since the cyclic loads may produce reductions in porewater pressures. This aspect needs to be confirmed for less firm soils.

It will generally be acceptable, therefore, to calculate the failure Ultimate Limit States using the usual parameters in effective stresses, but it will be necessary to check by some specific tests that the variations in pore water pressure during the design storm are negative. Otherwise, the strength will have to be reduced to take this effect into account.

It should be borne in mind that the ground strength will also depend on the manner in which the specimens are loaded (vertical compression, unconfined shear or horizontal compression, as shown in Figure 4.8.2).

The calculation procedures and the minimum required values for the safety factors calculated using this static analysis are similar to those described for conventional shallow foundations (Section 3.5), provided it is possible to admit failure probabilities in the same order of magnitude. The special nature of these works, however, may make it advisable to consider other failure probabilities and therefore other minimum safety requirements against these Ultimate Limit States. In these cases, given that the predominant variable actions (wind and waves) are those caused by the design storm, an acceptable simplification is to follow the specific procedure indicated in Section 4.7 (Breakwaters).

Another typical failure mode worth pointing out is local ground plastification around the tip of the lateral confinement skirts. This failure mode can be analysed using the evaluation procedure described in Subsection 4.5.3.5.

To calculate the maximum deformations produced during the design storm, the procedures indicated in Section 3.5.7 can be followed, using the dynamic moduli described in Section 3.10. As that section recommends, due consideration must also be given to the phenomenon of densification.

4.8.5.2 Dynamic Analysis

The study of offshore platforms requires carrying out dynamic analyses, not only to study the behaviour of the foundation but also, and principally, to know the stresses in the structure.

The dynamic models representing the combination of ground and structure in these computations can be of two types, depending on the way in which the ground is represented. The soil can be simulated by a series of springs and dampers located in the base of the structural model, or by a more detailed discretization of the ground, representing it by a finite-element mesh. Section 3.10.3 gives some details concerning the way the ground should be simulated in these two types of model.

Both models normally assume that the ground behaviour is elastic with equivalent viscous damping to represent its capacity for dissipating energy (which it does by radiation and hysteresis).

Under the conditions referred to, both types of model will lead to similar results in terms of loads acting on the foundation or stresses in the structure. To analyse foundations, the models of springs and dashpots will require additional computations to estimate the stresses at any point in the soil, once the overall loads acting on each spring representing the foundation ground are known.

In any event, the results of calculations using these models will make it possible to know the stresses existing at each point of the ground during the storm corresponding to the design situation under analysis.

In order to know the safety level of the platform foundation, the stresses obtained are compared with the stresses that would lead to ground failure. For facilitating this comparison and simplifying the corresponding laboratory tests, the stress history is usually converted into an equivalent cyclic stress. This would have a certain constant average value and amplitude and would act for a fixed number of cycles, determined in such a way that the same dynamic effect be produced. Several empirical methods exist for setting up this equivalence.

This method will provide a picture of how close the ground is to failure in each foundation zone during the design storm. Engineers can use this picture to assess the structure's failure probability.

It is also possible to utilise more complex soil models representing dynamic behaviour in greater detail and this is done when this aspect is important. Using these models, the degree of safety in each point of the foundation can be obtained without any subsequent computations or processing. This alternative is still at the research stage and is not yet a well-established routine practice.

4.8.6 Deep Foundations

Piles must be used to transmit loads to the seabed when it is composed of soft soils. Given the installations constraints, piles are usually driven and are normally made of steel.

The stability of the works will be conditioned by the behaviour of piles. In this respect, the possible Limit States are the same as those described in Section 3.6 of this ROM 0.5, which applies for these purposes. The studies associated with offshore platform piling are usually related to the following aspects:

- Study of pile driving.
- Bearing capacity.
- Horizontal ultimate load.
- Deformability.
- Loads on piles.

The problems related to seabed stability (sliding, erosion and scour) are the same as in spread foundations and were covered in Subsection 4.8.4.

4.8.6.1 Study of Pile Driving

Piled foundations are always costly and for this reason it is extremely important to carry out a detailed study before work commences.

The depth to which a particular pile can be driven with a specific driving equipment and the bearing capacity corresponding to a particular driven length can be predicted with the dynamic driving formulae and, preferably, by using the wave equation (see Subsection 3.6.5)

In order to increase the driving depth reached in a particular case, certain procedures can be used to assist pile driving (water jets, prior drilling, removing the earth plug inside the pile, etc.).

To increase the bearing capacity of the pile for a given driving depth, it is possible to carry out certain auxiliary operations (broadening the tip zone and subsequently filling in with concrete, cement grouting, etc.).

Generally, a vast range of procedures will be available in driven steel pile technology to make the most of the structural capacity of piles and the ground. Engineers must be aware of them and evaluate them in each specific case during the Stage of Design Studies.

4.8.6.2 Bearing Capacity

The bearing capacity of piles (individually or in a group) should be estimated as shown in Section 3.6.6 of this ROM 0.5.

Considering dynamic effects (rapid and alternating loads) normally leads to the conclusion that the dynamic bearing capacity is virtually the same as the static one, except in certain exceptional circumstances:

a. When the dynamic effects involve reversal of the load, i.e., when some piles work alternately under compression and traction.
b. When the amplitude of the dynamic load is over half of the shaft friction resistance (approximately).

In all cases and particularly in the two above exceptional cases, engineers should check the possible reduction in bearing capacity due to dynamic effects, based on comparable previous experience or by *in situ* load tests (at times, these can be carried out in similar ground formations on land).

4.8.6.3 Ultimate Horizontal Load of the Ground

The ultimate horizontal load of the ground can be estimated by the procedures indicated in Subsection 3.6.8. As a general rule, this problem will only be critical in the case of piles embedded only a short way into the ground.

The dynamic effect on the horizontal failure load is normally introduced by lowering the line of the sea floor for calculation purposes (virtually reducing the pile driven length), to take into account the gap that may exist between pile and ground on the seabed surface.

Engineers should look up similar experiences or explore this effect by adequate field tests. For initial estimates and in the absence of better information, they can assume that, in the horizontal resistance against alternating loads, the ground upper zone -down to a depth of $1.5\ D$, i.e., one and a half times the pile diameter- will contribute to the lateral support of the pile with nothing but its self-weight.

4.8.6.4 Deformability

The alternating nature of the loads means that the ground's subgrade moduli required to study deformation Limit States and calculate forces and moments on the piles may be different from those corresponding to static loading.

On the one hand, the effect of both the axial and transverse loads being cyclic normally is that the ground degrades somewhat and, consequently, the ground-pile system becomes more flexible.

On the other hand, rapid loading causes stiffer ground response than gradual loading.

The combination of these two facts implies that the deformation parameters given in Subsection 3.6.9 are reasonable for preliminary estimates. The spring constants that can represent the piles in dynamic calculations can be estimated, in an initial approximation, using the static formulae indicated in that same subsection.

For detail calculations and particularly for exceptional situations such as those described in a) and b) of Subsection 4.8.6.2, it is advisable to carry out specific horizontal pull tests.

4.8.6.5 Stress Resultants on Piles

Calculating the forces and moments on piles generally constitutes a complex ground-structure interaction problem. To carry out these calculations, each pile can be represented by structural elements fixed to a rigid support by springs and dashpots.

The spring constants indicated in Section 3.6.9 can be of use in these calculations. There are a large number of technical publications that can guide engineers in their choice of the most adequate parameters for the type of calculation they intend to carry out for analysing Ultimate Limit States governed by the structural capacity of piles.

4.9 DREDGING AND FILLS

Although recommendations for earthworks in maritime and harbour engineering will be the subject of specific publications within the ROM Programme, it has seemed advisable to anticipate in this ROM 0.5 –geotechnical in nature and of general purpose– some initial recommendations which can be useful for designing and planning maritime or harbour dredging and filling operations.

4.9.1 Types of Dredging

Dredging can be classified as follows, according to its primary purpose:

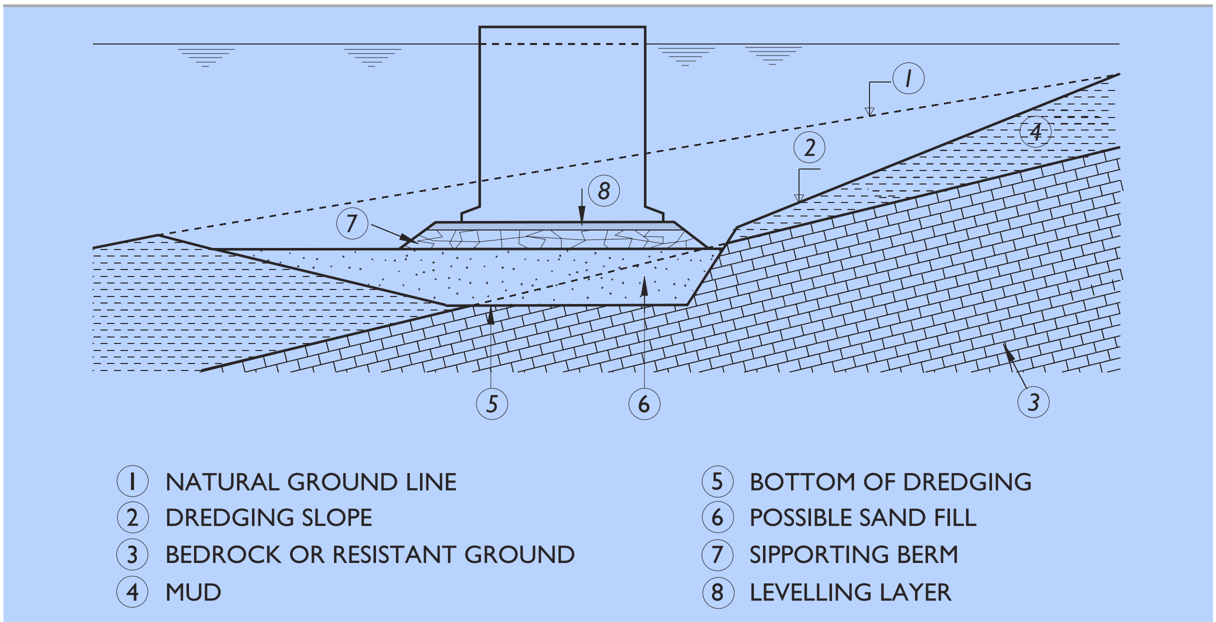
- Those performed to increase draught or depth in docks (or flotation areas) and navigation channels.
- The ones carried out to obtain borrow material for harbour or coastal fill and for beach regeneration.
- Those needed in the execution of maritime structures for reaching better quality foundation soils –by removing softer soil– or which constitute one more stage in the construction process, such as removal of temporary fill or hillocks. This type of dredging is called here *structural dredging*.

The first two types of dredging operations can, in turn, be for first construction or maintenance purposes, although this classification is irrelevant from the geotechnical point of view.

Dredging also tends to be classified in accordance with its extension. *General dredging* is the name given to dredging affecting areas of large length in any plan direction and *trench dredging* refers to those affecting a strip of varying width. Structural dredging normally belongs to this second type.

A construction with structural dredging is shown in Figure 4.9.1.

Figure 4.9.1. ***Diagram of Trench Dredging to Replace Soft Soils***



N.B.: Fill nº 6 can be made of other adequate granular materials, rockfill or quarry run, for example.

4.9.2 Geotechnical Information

4.9.2.1 Ground Investigation

Before designing any dredging operation, adequate geotechnical information needs to be available on the material to be dredged. Part of these data will be essential for the actual dredging itself, as they make it possible to select the most appropriate dredging equipment, to estimate its performance and to predict overexcavation.

Knowledge of the characteristics of the ground to be dredged is also essential in order to:

- Study the environmental impacts, particularly potential polluting effects.
- Analyse the possibility of utilising the dredged material in harbour fills and beach regeneration.
- Study the stability of side slopes in navigation channels and dock basins and the possibility of their affecting adjacent structures.
- Select the site for dumping and evaluate its consequences.

Normally, the programme for investigating the seabed to be dredged will have a general scope so as to investigate all the parameters of interest that intervene in the analysis of the different problems.

In order to study slope stability and above all to estimate the depth of soil to be dredged in foundations, the stratification of the seabed should be known. Each level should be analysed even below the dredging depth initially planned, down to a depth at least double this value.

Although boreholes are more costly at sea than on land, their number should not be reduced without assurance that any possible lack of homogeneity in the soil has been detected.

For each of the different types of soil found, the parameters generally needing to be known are the grading curve, Atterberg limits (in sand, these limits will be null), density, moisture content and strength. In the case of sands, the latter can be indirectly estimated by SPT tests. In clays, their undrained shear strength is needed, obtained either by in situ tests (vane, pressuremeter or static penetration) or by laboratory tests, along with their long-term strength (by triaxial tests).

When dredging rocks, it is advisable to know their nature, their possible stratification and degree of weathering and jointing. The compressive strength of the rock fragments obtained from boreholes is a useful indication in the design of the dredging system.

A description of these and other tests and exploration methods, along with recommendations for their application, can be found in Part 2 of this ROM 0.5.

4.9.2.2 Classifying Ground to Be Dredged

Soils

The soils to be dredged must be accurately described. The Unified Soil Classification System is recommended for this description, as it is widely used in geotechnical practice.

Each soil type should be given a name that will correspond to its primary constituent (cobbles, gravels, sands, silts or clays). Qualifying adjectives should be added relating to the following aspects:

- Grading, to indicate the content of other components (silty clay, for example).

- Nature and shape of the particles in granular soils (*rounded siliceous sands* or *angular conchiferous sands*, for example).

- Compacity or consistency; in this respect, terms such as *loose sands*, *firm silty clays*, etc. can be used. When using such expressions, it should be borne in mind that they could have an ambiguous meaning in common practise. In order to lend some consistency to the meaning of these terms, it is recommended to use Table 4.9.1, where these adjectives are quantified.

Table 4.9.1. Classification of Soil to be Dredged According to its Compacity or Consistency (9)

Granular Soils

Compacity of Sand	SPT N-Index
Very loose	< 4
Loose	4-10
Medium dense	10-30
Dense	30-50
Very dense	> 50

Cohesive Soils

Consistency of Clays		Undrained Shear Strength (kN/m^2)
Very Soft	Easily squeezable between fingers	< 20
Soft	Easily moulded by hand	20-40
Firm	Requires strong pressure to be moulded by hand	40-75
Stiff	Cannot be moulded by hand. Indented by thumbnail	75-150
Hard	Hard. Indented with difficulty by thumbnail	> 150

- Smell and colour, occasionally of interest as indicators to distinguish different soil levels. The smell may serve to differentiate organic soils.

- Other aspects of interest, such as possible cementing, cracking, etc.

When used in written reports and other project design documents, this qualifying data should be backed by appropriate laboratory tests, as indicated in Part 2 of this ROM 0.5.

Rocks

The rocks to be dredged should be described, firstly by indicating their nature. A possible inaccuracy in the "name" given to a particular rock does not usually give rise to problems, provided its characteristics are described well.

The names to be given to rocks must be consistent with their nature. Some common names that can be used are listed in Part 2 of this ROM 0.5.

Description of the weathering and jointing of the rock is an exceptionally important aspect. After investigating the different levels or horizons of the rock, and after testing them in the laboratory to determine their properties, certain adjectives can be used to qualify their degree of weathering, the strength of fresh fragments and their degree of fracturing. Engineers can use the terms most convenient to them, but should always clarify their meaning with quantitative references.

To this end, it is recommended to use Tables 4.9.2, 4.9.3, 4.9.4 and 4.9.5.

(9) Taken from the PIANC Recommendations, Supplement to Bulletin n° 47 (1984).

Table 4.9.2. Terms to Be Used to Define the Degree of Rock Weathering

Class	Description	Symbol	Weathered Mass	Characteristics
I	Fresh	F	0	Compact
II	Slightly weathered	SW	< 10%	Weathered joints
III	Moderately weathered	MW	10-50%	Not friable
IV	Highly weathered	HW	50-90%	Friable
V	Fully weathered	FW	> 90%	Highly friable

Table 4.9.3. Terms to Be Used to Describe Rock Matrix Strength (10)

Term	Unconfined Compressive Strength (MN/m^2)
Very weak	< 1.25
Weak	1.25 to 5
Moderately weak	5 to 12.5
Moderately strong	12.5 to 50
Strong	50 to 100
Very strong	100 to 200
Extremely strong	> 200

Table 4.9.4. Criteria for the Objective Description of the State of Rock Fracturing (11)

Concept	Explanation
Borehole log	The state of the rock *in situ* is very important and for this reason it is essential that the drilling method and diameter employed are stated. In addition, in order to describe the integrity of the rock, various criteria can be used to indicate the fracturing state of rock cores, such as *total core recovery*, *solid core recovery*, *fracture index* and *RQD index* (Rock Quality Designation). All these should be included in the borehole log.
Total core recovery	This is defined as the length of the total amount of core recovered, expressed as a percentage of the length of the core run.
Solid core recovery	This is defined as the length of the core recovered as solid rock cylinders, expressed as a percentage of the length of the core run.
Fracture index	This expresses the number of natural fractures present over an arbitrary length; for example, the number of natural fractures per linear metre of borehole.
RQD index	Rock Quality Designation. This is a quantitative measure of the state of fracture of the rock. The RQD is the sum of the lengths of all the core fragments (100 mm long or more) measured along the core axis and expressed as a percentage of the core length drilled.

Table 4.9.5. Classification of Rock Masses according to their RQD Index

RQD (%)	Mass Quality
90-100	Excellent
75-90	Very good
50-75	Good
25-50	Fair
10-25	Poor

(10) Taken from the PIANC Recommendations, Supplement to n° 47 (1984).
(11) Taken from the PIANC Recommendations, Supplement to n° 47 (1984).

4.9.3 Side Slopes

The stability of side slopes created by dredging should be analysed following the recommendations given in Sections 3.8 and 3.10 of this ROM 0.5.

Dredging operations are specific, however, and special considerations must be taken into account with respect to the general calculating procedures set out there.

At the time of the actual dredging operation, the dredge has a violent action on the ground, particularly in the case of large suction dredges. In soft soils (both sandy and clayey), this usually generates transient excess porewater pressures meaning that stability in undrained conditions can prove critical.

Dense sands and firm clays, however, are usually capable of withstanding dredge action better. Their stability will generally be governed by the drained conditions.

COHESIVE SOILS

Stability in undrained conditions, for cohesive soils, should be analysed by assuming a zero virtual angle of friction, a cohesion equal to the undrained shear strength and a unit weight equal to the submerged specific weight, i.e.:

Undrained conditions:

$$\phi_{design} = 0$$
$$c_{design} = s_u$$
$$\gamma_{design} = \gamma'$$

Calculations in undrained conditions should not take into account the potential effects of porewater pressures generated by dredge operations. In order to take into account, where applicable, the porewater pressures generated in this type of soil by the action of waves and other sea level oscillations, recommendations are given in Subsections 3.4.5, 3.4.11 and Section 3.10 of this ROM 0.5.

In the long term, once the cohesive soil has adapted to the new state of stresses, stability should be checked using strength parameters deduced from CD or CU triaxial tests -measuring and then subtracting porewater pressure- or other equivalent tests, as shown in Part 2 of this ROM 0.5. In other words:

Drained conditions:

$$\phi_{design} = \phi$$
$$c_{design} = c$$
$$\gamma_{design} = \gamma'$$

For calculations in drained conditions and in zones where there may be a porewater seepage gradient from within the slope towards the outside (e.g., slopes in intertidal zones at low tide), the possible excess porewater pressure in the ground (with respect to the free surface of the slope) should be included in the calculations.

As a simplification erring on the safe side and in order to take this effect into account, it is acceptable to multiply the tangent of the friction angle by 1/2 to obtain the tangent of the equivalent virtual friction angle that should be used in the calculation. That is:

$$\tan\phi_{design} = \frac{1}{2}\tan\phi$$

GRANULAR SOILS

The sensitivity of fine sands with low density to dredging action means make the slope stability calculation methods not applicable for analysing their behaviour in undrained conditions. The right slopes can only be approximately estimated by observing their behaviour during the initial stages of dredging and from previous experience.

Subsections 3.4.5, 3.4.11 and 3.10.3 of this ROM 0.5 give recommendations for taking into account, where applicable, the porewater pressures generated in this type of soil by the action of waves and other sea level oscillations.

In denser and coarser sands, it is only necessary to check the theoretical stability in drained conditions.

If the slope is in an area with significant water movement generating a porewater flow from inside the slope to the exterior, potential excess pore water pressures between the ground and the free water alongside the slope should be taken into account.

In cases where the slope is affected by water movement, the simplification - erring on the safe side - of dividing the tangent of the angle of friction by 2 is acceptable in order to obtain the tangent of the equivalent virtual angle of friction that should be used in calculations. In other words:

$$\tan\phi_{design} = \frac{1}{2}\tan\phi$$

If the slope is located in an area also affected by wave action, the porewater seepage pressures generated by this action should also be taken into account for analyses in drained conditions, in line with the indications given in Subsections 3.4.5, 3.4.11 and 3.10.3 of this ROM 0.5.

Rocks

The stability of side slopes in rock dredging can be analysed by the general procedures given in Subsection 3.8.5. The calculations should be made using submerged weights.

In cases where the rock joints are open, potential excess porewater pressures –over the value of the water pressure on the face of the slope– should be taken into account. In the absence of more detailed studies, the effect of water flow from the inside of the slope outwards can be indirectly taken into account by reducing the tangent of the angle of friction in the rock joints by a factor of between 0.5 and 0.8. This reduction will increase with the permeability of the rock joint system and the sea movement in the area surrounding the slope.

Side Slopes Common in Dredging

Table 4.9.6 shows the inclination of dredging slopes from the horizontal that are normally used in designing dredging operations.

Table 4.9.6. *Values of Side Slopes (H/V) Common in Dredging*

Ground Type	Calm Water	Zones with Water Flow from Inside to Outside the Slope
Muds	20 to 6	20 to 10
Fine loose sands	6 to 4	10 to 6
Coarse sands	4 to 3	6 to 4
Clayey sands	3 to 2	4 to 3
Firm clays	2 to 1	3 to 1.33
Stiff clays	1 to 0.5	1.33 to 0.5
Rocks (10)	0.5 to 0.1	0.5 to 0.1

(10) A more moderate side slope, following the stratification, may be advisable in weathered sedimentary rock with unfavourable dipping.

Minimum Safety Factors against Loss of Stability

In any event, the safety of dredging side slopes against loss of stability should be verified in all Design Situations. The minimum required safety factors should be properly set in the design bases for each project, as a function of their importance and the consequences of failure.

If the calculation procedures laid down in Section 3.8 are used, the minimum safety factors shown in Table 4.9.7 are considered allowable in works with a minor SERI rating. As indicated in the general procedure established in this ROM 0.5, these factors are associated with failure probabilities in the order of 10^{-2}.

Table 4.9.7. Minimum Recommended Safety Factors against Loss of Stability in Dredging Slopes, Works with Minor SERI (< 5)

Load Combination	Safety Factors
Quasi-permanent	1.30
Fundamental	1.20
Accidental or Seismic	1.00

For works with a different SERI rating or for other allowable failure probabilities, the minimum safety factor values set in Table 4.9.7 can be adapted as indicated in Subsections 3.3.8.2 and 3.3.10. They can equally be adjusted for transient situations (including short-term geotechnical situations) in line with the provisions of Subsection 3.3.8.1.

In navigation channels and zones affected by waves, water velocities in areas close to dredging side slopes may be high enough to cause erosion problems. In such cases, the most adequate surface protection should be investigated. Subsections 4.2.3.7, 4.3.5.3 and 4.3.5.7 make some recommendations that could apply in this respect.

4.9.4 Types of Fill

Fills can be classified based on several criteria. One common criterion that will be used in the following sections is the classification according to their nature. In this respect, there are as many types of fill as there are soil and rock types, in addition to fills made with unconventional materials (slag, fly ash and other industrial by-products) as also refuse and other urban waste.

For the purpose of the following recommendations, three types of fill should be distinguished, on the basis of their use. These will be referred to as *structural*, *general* and *waste* fills.

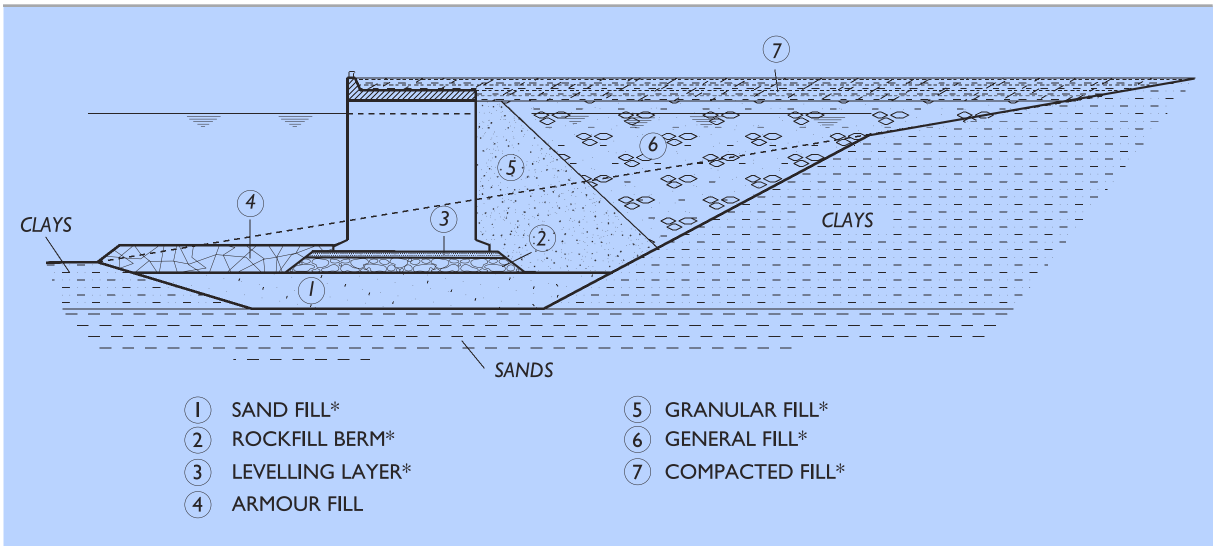
Figure 4.9.2 shows a simplified typical quay section with some common fill zones.

Structural Fills

This ROM 0.5 designates as *structural fill* those used close to structures and whose properties play an important role in the structure stability and deformation.

Structural fills can be placed in the dry or submerged. Filling can take place in the dry provided that the site is above sea level or by artificially lowering the water level in the fill area.

Figure 4.9.2. **Diagram of Possible Types of Fill near a Quay Wall**



N.B.: Fills marked with an asterisk must be considered *structural*.
Fill No. 1 can consist of a different material, rockfill or quarry run, for example.
The crest fill, No. 7, should be considered *structural* in operating and storage zones.

General Fills

The term *general fill* refers to those used in maritime and port works that do not have a structural mission other than serving as support for the crest (generally consisting of a dry-compacted fill) on which a pavement is constructed.

General fill is normally carried out submerged, although it will sometimes be possible to execute part of certain general fills in the dry with artificial measures for lowering the groundwater table.

Waist Fills and Dumpsites

Dumpsites are fill areas where surplus excavation products are deposited that cannot be used in other port zones.

4.9.5 Fill Characteristics

4.9.5.1 Directly Dumped Submerged Fills

Submerged fills, carried out by direct dumping of products controlled at their origin, have the same grading composition and plasticity (the latter in the case of clayey soils) as the values deduced from tests at the point of origin when transported dry (by truck, conveyor belt, etc.). Their density and structure, however, can only be ascertained by subsequent testing. The dumping process can be simulated in the laboratory.

The geotechnical characteristics that can be expected in directly dumped submerged fills are discussed below. Some typical average values are shown in Table 4.9.8.

Clean Granular Materials

Granular fills with a fines content, after placing, of less than 10% (approximately) are compacted by their own weight up to relative densities in the order of 50%.

Table 4.9.8. Typical Values for Some Properties of Recently Dumped Submerged Fills

Type of Material	Dry Specific Weight γ_d (kN/m^3)	Shear Strength			Modulus of Deformation E_2[(1)] (MN/m^2)
		Undrained	Drained		
		s_u (kN/m^2)	c (kN/m^2)	ϕ (°)	
Rockfill	17	–	–	40[(2)]	30
Quarry run	18	–	–	40[(2)]	30
Sandy gravel	18	–	–	35	30
Clean sand	16	–	–	30	20
Silty sand	12	–	–	20	10
Silt (mud)	8	10	0	6[(3)]	1[(4)]
Low-plasticity clay	8	10	0	6[(3)]	1[(4)]
Highly plastic clay	8	10	0	6[(3)]	1[(4)]

(1) Unloading/reloading modulus corresponding to small load increments.
(2) The friction angle of rockfill and quarry run can decrease as the confinement pressure increases (see Subsection 4.7.2.2).
(3) Apparent friction angle for long-term stability analyses accounting for the underconsolidated state.
(4) Compressibility in these cases can be described better by using the compression index, C_c (see Subsection 2.2.10.2).

Their strength, deformability and permeability characteristics can be known by laboratory tests on specimens with the same grading, prepared with the same relative densities as expected on site.

Fine sand fills are sensitive to vibration and can liquefy during earthquakes (see Section 3.10).

The characteristics of submerged granular fills can be improved by different procedures (see Section 3.9).

Dirty Granular Materials

Silty sands (with a fines content of over 10%, approximately) and silts usually give rise to submerged fills with a very low density. Consolidation under their own weight can be slow.

Strength, permeability and deformability characteristics can be investigated in the laboratory on specimens prepared by sedimentation of materials with similar grading to that of the fill under study.

This type of fill is not normally suitable for supporting direct foundations of some importance, unless previously subjected to improving treatment.

These fills may also be susceptible to vibration and can undergo liquefaction. Therefore, improvement techniques may be worth considering in some cases. These aspects are dealt with in Sections 3.9 and 3.10 of this ROM 0.5.

Cohesive Materials

Soft cohesive materials give rise to very low-density fills with very high compressibility, which are difficult to improve by artificial treatment because of their low permeability.

Clods of clay dumped directly into the water can lead to a very open structure. As times passes and as the clods break up, the volume of voids gradually decreases and considerable settlement consequently occurs.

4.9.5.2 Hydraulic Fills

Hydraulic fills are placed on site by sedimentation of the solid particles contained in effluents discharged from confined dredging of relatively watertight enclosures. Such effluents are mainly characterised by their flow rates and suspended-solids contents.

The characteristics of hydraulic fills depend on the nature of the material remaining in the fill after the processes of excavation, transport and sedimentation have taken place.

Hydraulic fills may have very different characteristics from those corresponding to the borrow materials at their point of origin. Sands lose their fines both during dredging and transport operations (if any intermediate handling is involved) and, above all, during the sedimentation process when this occurs over large areas.

When transported by pipe, it is a well-known fact that the coarser soils segregate out alongside discharge areas, whereas the finer particles take longer to deposit and may even be washed out by the effluent flow. For this reason, it is advisable to place the discharge points in areas where better quality fills are required.

If the dredging and filling processes are correctly handled, it is possible to obtain good quality fills of clean sands from silty sand borrow pits. The fines washing process can be controlled at the cost of a series of monitoring tasks at the origin and destination points, including continuous sampling in both the borrow materials and the fill.

The products of dredging clay soils, when transported by pipe and then deposited, give rise to fills in which part of the lumps of clay have not yet broken down and have rounded shapes as a result of transport. The resulting fill has poor strength and high compressibility.

4.9.5.3 Fills Placed in the Dry

After being placed in the dry, fills made up of controlled products have the same on-site grading characteristics and plasticity (or very similar) as those deduced from point-of-origin controls. Therefore, the final characteristics of such fills can be known by analysing the borrow material in advance, by appropriate tests on specimens compacted in the laboratory at specific energies similar to those intended to be used on site.

The typical parameters to be expected of fills compacted in the dry are shown in Table 4.9.9. Further details of their properties must be investigated by laboratory and field tests such as those dealt with in Part 2 of this ROM 0.5.

Table 4.9.9. Typical Values for Some Properties of Fills Compacted in the Dry

Type of Material	Dry Specific Weight γ_d (kN/m^3)	Shear Strength			Modulus of Deformation E_2 (1) (MN/m^2)
		Undrained	Drained		
		s_u (kN/m^2)	c (kN/m^2)	ϕ (°)	
Quarry run	23	–	0	50	200
Sandy gravel	21	–	0	45	150
Clean sand	19	–	0	35	50
Silty sand (2)	18	–	10	32	40
Silt (3)	14	40	20	28	20
Low-plasticity clay	16	200	20	25	20
Highly plastic clay	13	100	20	20	10

(1) Reloading modulus from plate bearing tests with a 30-cm diameter (1 foot).
(2) With a fines content of over 10%.
(3) Silts compacted in the dry can collapse (settle without any change in vertical load) when saturated.

4.9.6 Fill Consolidation

Fills in maritime and harbour works can be used with the degree of consolidation naturally resulting after they have been placed on site. Engineers should, however, study the possibility of improving their characteristics, since this can sometimes be convenient.

General fills can cause substantial settlement and affect pavements constructed on them. Prior to pavement construction, the maximum settlement that the pavement will experience should be estimated. If this is large (see ROM 4.1 - Harbour Pavements), preventive measures will undoubtedly have to be taken.

This section gives some criteria that may help engineers in estimating pavement settlement.

4.9.6.1 Settlement in General Fills

Surface settlement of fills will generally be the result of two components:

- the compressibility of the natural ground on which the fill is built,
- the compressibility of the fill itself.

In both cases, settlement is due to the weight of the natural ground (sometimes still undergoing consolidation before the fill is placed), the weight of the fill itself, of the crest of the fill and of the eventual pavement and to the effect of service loads on the pavement.

Natural or artificial lowering of the groundwater table can cause further settlement.

Estimating settlement and its development is a theoretical problem that engineers should tackle and solve with the help of laboratory tests, preferably oedometer tests, carried out on undisturbed samples of the compressible materials. The compressibility and permeability of fine sands and other materials difficult to sample can be indirectly estimated by other adequate tests. Indications are given in Part 2 of this ROM 0.5 on the exploration methods and tests that can be carried out to determine these properties.

For estimating settlement in heterogeneous or difficult to sample fills, engineers can resort to field tests or loading tests such as the one described below. This should not preclude the above theoretical estimation, based on laboratory tests.

4.9.6.2 Monitoring Procedure

The method proposed in this section is based on the basic premise that, in the long term, settlement varies with time according to a logarithmic law. Such is the best-known law for one-dimensional consolidation for high degrees of consolidation. Similarly, this type of law is usually employed to interpret secondary consolidation processes.

The method involves monitoring the zone over a certain period of time and then applying a load on an approximately circular area with a plan diameter of at least three times the depth of the soils whose compression could cause settlement. The intensity of the load (height of the earthfill) must be such that it produces the same pressure on the ground as the fill remaining to be placed, plus that of the pavement and its future service load.

To this effect, the service load should be converted into an equivalent permanent load. This load can be approximately equal to that of the quasi-permanent combination.

Settlement can be monitored by regular levelling of reference monuments that settle with the fill, such as slightly buried settlement plates.

The fixed reference points should rest on firm ground. To this end, it may be necessary to insert reference monuments anchored at sufficient depth within cased drill holes reaching firm ground.

It is also advisable to install piezometers at specific points (pneumatic or preferably vibrating-wire) in order to monitor variations in the porewater pressure at different depths and at different spots under the load that is to be applied.

a. Monitoring Prior to Applying the Test Load

It is advisable to carry out a prior monitoring of the existing natural consolidation to ascertain whether any movement is occurring before applying the earthfill required for the test.

Before adding the test load, the area should be monitored for long enough to determine the velocity of natural settlement of the fill without any overburden.

If settlement velocity is imperceptible (roughly less than 2 mm/year), this will mean that the fill is not undergoing natural settlement. Preliminary monitoring can conclude and the overburden be applied.

b. Applying the Test Load

Applying the surcharge will cause the previously installed reference monuments to settle and enable settlement data to be gradually obtained from successive levelling campaigns.

The settlement obtained in the different survey points will lead to estimating a maximum settlement that can be assigned to the centre of the test zone for each campaign.

The settlement in the central zone of the test obtained from each measurement campaign will make it possible to calculate the velocity of settlement due to the surcharge at different points in time. For this purpose, the natural settlement velocity, v_o, that was observed prior to applying the test load needs to be subtracted from the settlement rates measured.

Using this data, a graph should be drawn, with time data along the *X* axis and the inverse settlement velocity along the *Y* axis. This is shown in Figure 4.9.3.

The shape of this curve will enable a straight segment to be identified for high values of time. This may require extending the monitoring period; otherwise, pessimistic conclusions may result.

Eliminating the initial data, i.e., the data deviating most from the linear law of evolution of *Y* with *X*, a straight line should be defined (using the least squares method, for example), which is the best fit to the observations made. Hence, parameters *A* and *B* will enable the observed settlement to be extrapolated.

By observing piezometer readings, in addition, it is possible to ensure that the excess porewater pressures that may have been generated by the surcharge have dissipated before deciding to conclude the test. At that moment, the excess pressure (over the one corresponding to a stationary groundwater table) should approximately be less than 10% of the test load applied.

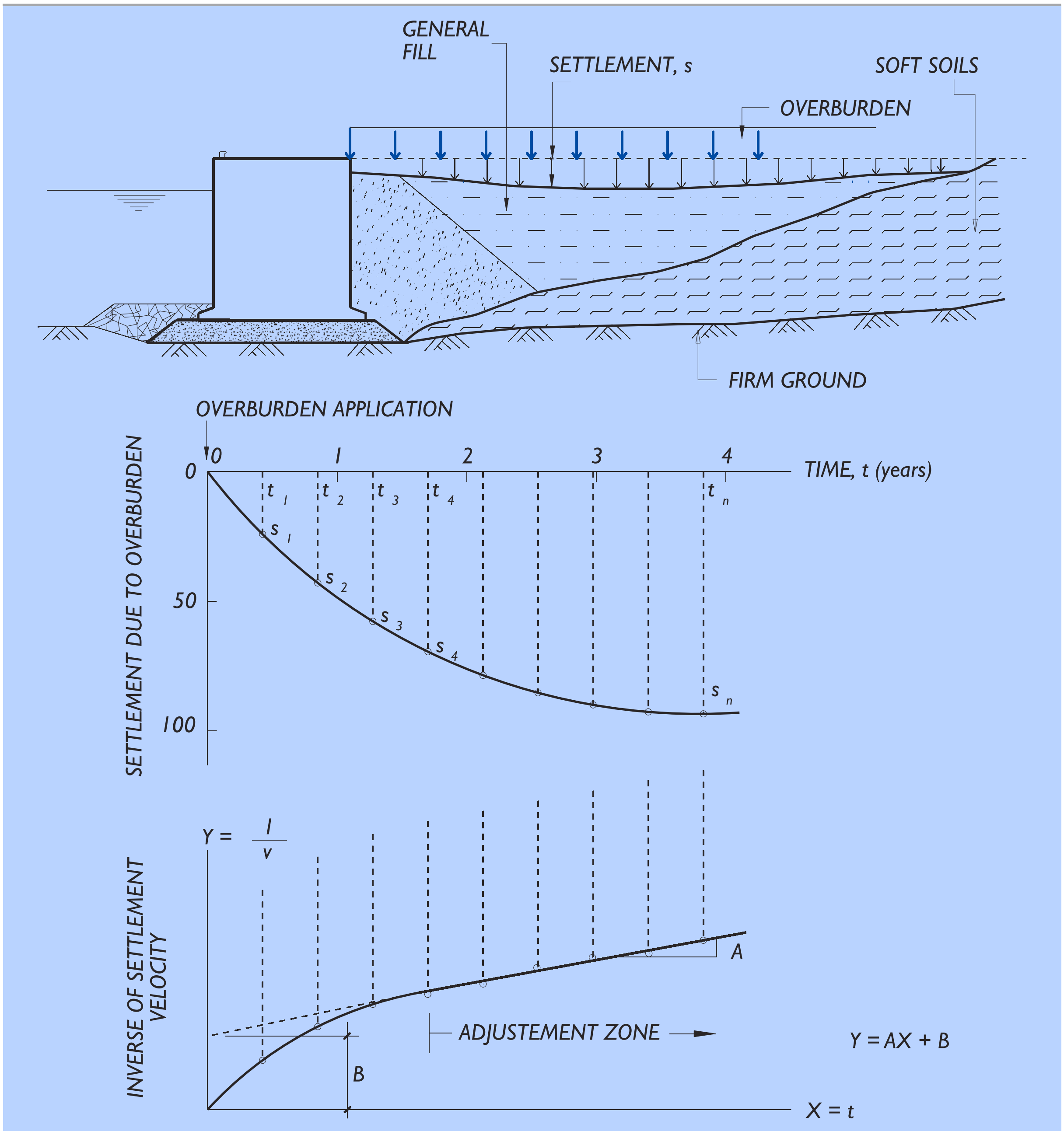
If the piezometers indicate that the excess porewater pressures are still high when the test has concluded, results should be interpreted differently. The method used here could lead to pessimistic conclusions.

c. Pavement Settlement

After concluding the test and analysing the data as indicated above, it possible to estimate the settlement that an eventual pavement would undergo when constructed over the fill under study.

The pavement settlement can be taken as being:

Figure 4.9.3. **Diagram of Fill Settlement**



$$s(t) = s_f + v_o(t - t_f) + \frac{1}{A}\ln\frac{At + B}{At_f + B}, \qquad (t > t_f)$$

where:

A y B = fit parameters obtained from interpreting the settlement caused by the overburden
t = time elapsed since construction of the pavement
s_f = settlement obtained at the end of the load test
v_o = settlement velocity observed prior to applying the test load
t_f = duration of the load test.

As this procedure is based on extrapolating the observed settlement, the accuracy of the results will increase with the length of the test period.

4.9.7 Other Recommendations

Some recommendations are given in this section, which are not intended to be exhaustive. Engineers should take them into account when planning earth-moving operations in their design work.

4.9.7.1 Fill Selection

The fills to be used in harbour zones need to be selected according to their intended use. The possible utilisation of surplus excavation products should particularly be taken into account.

Borrowing sands from the seabed may offer advantages over obtaining other equivalent borrow material from land. For instance, it has a lower visual impact and avoids possibly problematical expropriation or occupation. The effect of the dredging, transport and dumping procedures on the resulting fill quality is significant and must be taken into account when planning the use of borrow material from the seabed.

Some comments about the quality requirements for different types of fill can be found below.

Submerged Structural Fills

Non-granular fills are difficult to compact underwater. Granular fills, however, are compacted under their own weight up to relative densities of around 50% and, if necessary, can be compacted later by one of the procedures indicated in Section 3.9.

The higher the friction angle and permeability of structural fills, the lower their pressure against retaining structures will be. For this reason, it is advisable that structural fills always consist of granular materials, preferably sandy gravels or quarry run.

Unless otherwise justified on the basis of adequate tests, a material should be taken as granular for these purposes when its fines content (UNE 0.080 sieve) is less than 10%. In the case of rockfill or quarry run, where a large part of the weight is made up of particles of over 1 inch (25 mm), this fines content refers precisely to 10% of the weight of the fraction passing a 1” screen.

If non-granular submerged fills are used for structural purposes, their strength should be studied in detail by adequate laboratory tests. If this is not done, the estimate of their strength has to be clearly conservative.

Fills Compacted in the Dry

Any type of material can be used in compacted structural fills, but the following should be excluded:

- Materials containing considerable amounts of organic matter. Over 2% by weight is considered to be excessive.
- Materials containing friable rocks, particularly gypsiferous materials.
- Certain types of slag that may be expansive.
- Highly plastic clayey soils.
- Urban waste (glass, demolition rubble, etc.) that is difficult to compact.
- Soils contaminated by substances that could damage the environment.

Submerged General Fills

General fills will normally be placed submerged, so they will not experience any compaction process other than the one owing to self-weight.

If the borrow material is granular, the resulting fill may be of a reasonable quality. If this is not the case, it will prove to have low strength and be highly compressible.

4.9.7.2 Precautions for Dredging Operations

In trench dredging for the purpose of improving foundations, it is very important to clean out the bottom of the trench carefully, not leaving behind any residue from the dredging itself. If trenches are located at sites where there is silting or continuous inflow of mud, it is advisable to do a dredging operation for cleaning up immediately before laying the rockfill berms or the corresponding structural element, to prevent potential weakening of the contact.

Dredging often has to be carried out very close to existing works, in which case the effect of these operations on their stability must be examined. Very strict tolerances must be observed. Dredging in such cases must be very precise to prevent the side slopes of the dredged zone undermining adjacent structures. Equipment must be chosen that allows for this precision, even if its performance is poorer.

4.9.7.3 Precautions for Fill Operations

Structural fills placed in the dry should be spread and compacted in lifts. In their construction, the maximum densities that can be reached and the associated optimum moisture content should be controlled (by compaction tests) as also the densities actually achieved (in situ densities and moisture content). A degree of compaction of between 95% and 105% of the maximum density from the Standard Proctor compaction test will generally be required, depending on the structural mission of the fill.

The compaction of coarse granular materials can be controlled with the help of plate bearing tests. It is advisable to use large-diameter plates (ϕ 600 mm, for instance).

When fills are compacted alongside retaining structures, transient increases in earth pressure may occur that should be taken into account in the calculations.

Several measures can be taken to reduce the deferred settlement of platforms made with general fills, including:

- Dredging the bottom of the zones to be filled so that the supporting base is of similar or better quality than that of the fill to be placed.
- Using fill materials of the best possible quality.
- Carrying out some ground treatment from the ones indicated in Section 3.9.

4.9.7.4 Analysis of Dumpsites

Some dredging operations not designed to obtain borrow material may give rise to serious problems in relation to disposing of the dredged material at sea. To solve this, it may be necessary to prepare dumpsites consisting of basins bordered by dykes or hillocks and designed to create an enclosure to retain these products.

From the geotechnical point of view, the following three aspects must be analysed in connection with these basins:

- The stability of the dykes and hillocks, by considering the characteristics of the dredged material, the dykes and the seabed, which will often consist of weak soils.

- The possibility of utilising the fills obtained in this manner to support loads, by examining their consolidation.
- The process of effluent sedimentation if hydraulic dumping is used.

4.10 OTHER MARITIME AND HARBOUR WORKS

The wide variety of works usually constructed in port and coastal areas makes it virtually impossible to consider each one individually.

There are some types of works or installations besides the ones considered in previous sections, however, which have some geotechnical aspects worth pointing out now in this ROM 0.5. Future publications in the ROM Programme will deal with these works in more detail.

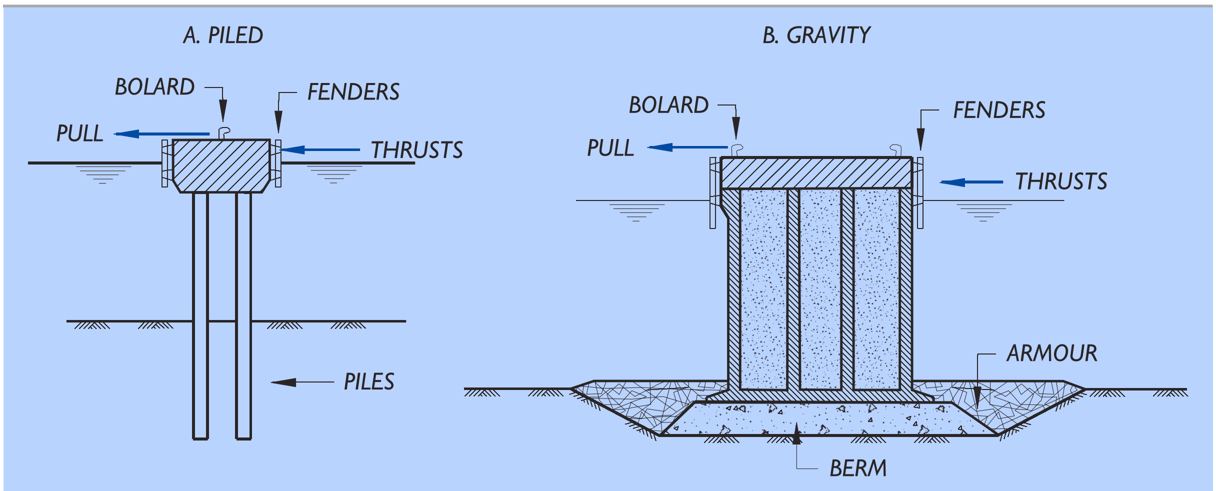
4.10.1 Dolphins

The structures known by this name are usually elements installed in port and harbour areas to withstand horizontal mooring or berthing loads. At times they are constructed to protect other structures (bridge piers in navigation routes, from potential collisions of vessels, for example).

4.10.1.1 Types

The horizontal forces that can act on dolphins can be withstood by gravity structures, generally with shallow foundations, or by structures with deep foundations. Figure 4.10.1 represents these two types.

Figure 4.10.1. **Cross-Section Diagrams of Dolphins**



Both types have a superstructure supporting the fenders for the impacts and/or bollards for applying the pull.

Dolphins can use a wide variety of structural types, as many in fact as the gravity solutions in quay walls (caissons, sheetpile enclosures, etc.) or solutions for executing deep foundations (large isolated piles, groups of smaller piles, piles formed by driven steel sheetpiles, etc.).

Piles with a large structural capacity can be used individually in so-called *monopile* dolphins.

4.10.1.2 Associated Geotechnical Problems

Apart from the geotechnical problems that the foundations of dolphin structures may have in common with other similar structures, the specific geotechnical problem associated with the horizontal component of the load stands out.

Overturning is the critical aspect that will govern the design of gravity dolphins. This does not however mean that all the safety checks referred to in Section 3.5 -on shallow foundations- can be omitted nor that the recommendations for structures of a similar type appearing in this Part 4 can be ignored.

Piled dolphins should primarily be checked against horizontal ground failure (Subsection 3.6.8), which will generally be the critical design condition as far as geotechnical problems are concerned. Dolphins should normally fail structurally before horizontal ground failure occurs. The other Ultimate Limit States for piled structures referred to in Section 3.6 should also be considered in the design of dolphins.

The pull exerted by moorings can have a substantial vertical component. This, combined with the fact that the dolphin structure may include battered piles, may originate tensile forces on the piles. In these circumstances, it may be advisable to install a heavy superstructure on the head of the dolphin to mitigate such tensile stresses.

The problem of horizontal deflection is particularly important, since the energy involved in berthing or impact and the drifting movements of berthed vessels depend on the deformability of the dolphin.

Subsections 3.5.7 and 3.6.9 make recommendations on procedures for estimating the deformability due to the ground in works with shallow and deep foundations that should be taken into account in calculations. The extra deformability of the dolphin structure must be added to that of the foundation ground.

The horizontal loads on dolphins (particularly due to berthing) can be distinctly dynamic. The recommendations given in Section 3.10 should be considered when evaluating their behaviour.

In those piled dolphins that basically work by flexural bending, it is advisable to carry out horizontal pull tests that will characterise their deformability accurately.

4.10.2 Facilities for Launching and Lifting Vessels

Vessels can be built on shipways, ramps with a moderate slope making it easier to launch them by sliding along guiding rails. Sometimes they have side walls and a forward gate to stop water entering at high tide. They are then similar to dry docks.

Lifting vessels out of the water for repair on shore requires constructing slipways. These consist of sloping slabs on which vessels can be moved by supporting them on a rolling device of some kind, hauled by a winch.

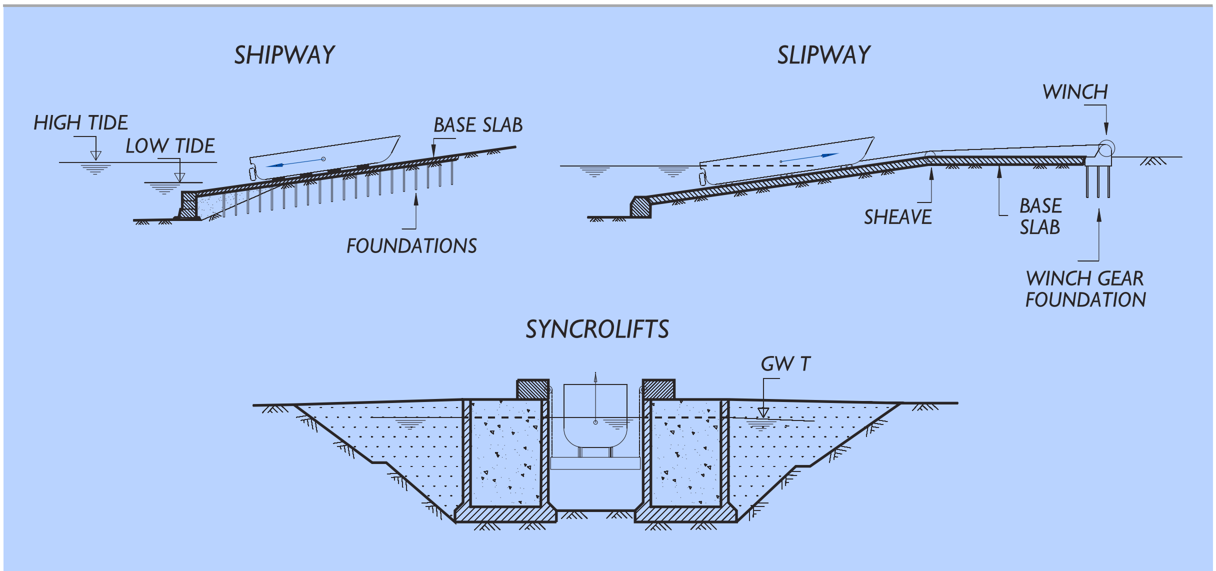
Boats are sometimes lifted vertically by syncrolifts that raise and lower them onto an adequate transport system on the quay platform.

Fixed installations for building floating caissons fulfil a similar function. A vertically mobile platform (or float) enables the weight of the caisson to be supported while it is lowered to the point where it is launched.

Floating docks, also used for these launching and lifting operations, should be considered as vessels, not fixed structures.

Some typical sections of this type of facilities are shown in Figure 4.10.2.

Figure 4.10.2. **Diagram of Launching and Lifting Facilities**



4.10.2.1 Shipways

As mentioned, shipways may have some aspects in common with dry docks, particularly when they have a gate and side walls preventing partial flooding at high tide.

The intense loads on the ramps that support vessels, and above all the specially significant actions of hulls against the base slab at the end of launching operations, make this aspect the dominant one in the design of the base slab.

Alongside the shipways there will be massive foundations for the large cranes used in shipbuilding.

In firm ground, the base slab can be constructed directly on the natural ground, although more frequently, and in view of the intensity of the loads involved, deep foundations by means of piles may be required.

The general recommendations given in Parts 2 and 3 of this ROM 0.5 should be followed in geotechnical investigations and safety analyses for these foundations.

The structural analysis of the base slab will generally require a study of soil-structure interaction similar to the one referred to in Subsections 4.6.3.3, 4.6.4.2 and 4.6.5.2 for the bottom slabs of dry docks.

4.10.2.2 Slipways

The craft normally moved on slipways are smaller in size and the unitary loads on the individual supports are usually restricted, so that they can subsequently be moved on land.

The haul loads on slipways, both longitudinal and transverse, can be estimated as indicated in the ROM 0.2 publication.

Slipway base slabs are usually constructed of reinforced concrete and are between 0.50 m and 1.00 m thick, depending on the intensity of the loads. Deep foundations may only be necessary in ground with a low bearing capacity.

4.10.2.3 Syncrolifts and Caisson Yards

Syncrolifts can hoist vessels at the cost of considerable vertical reactions on the capping beam of quay walls or adjacent ancillary structures.

These vertical loads may be highly eccentric and this may therefore have an unfavourable effect on the bearing failure and overturning of gravity structures or on piles, if deep foundations are used. This effect can be analysed following the procedures given for different types of harbour structures in this Part 4.

Fixed installations in which caissons or other structural elements are constructed –and then launched– are called *caisson yards* here. They can consist of two structural members similar to those of syncrolifts.

Each of these members has to support comparable loads. The foundations of such facilities must be investigated following the recommendations given here for the harbour works to which they can be most closely assimilated.

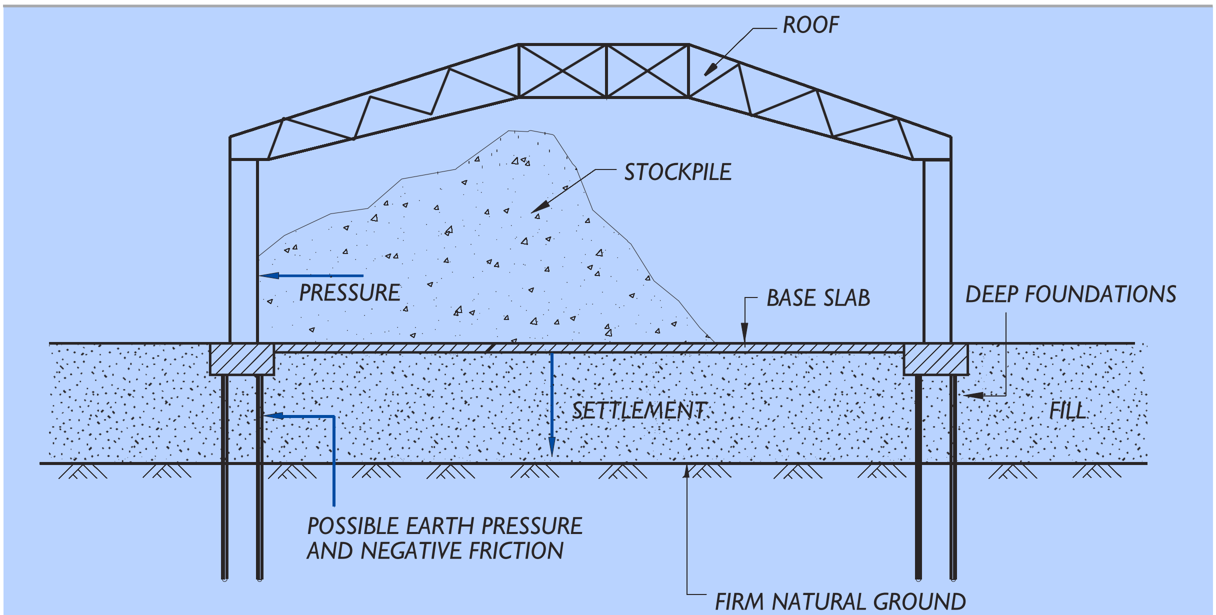
In any event, a check must be made that the rotation of these structures is compatible with the correct operation of the facility.

4.10.3 Transit Sheds, Warehouses and Covered Storage Areas

Handling of goods in port areas requires the construction of this type of facility, often founded on ground with a low bearing capacity (general fills largely made up of directly dumped and submerged materials).

Storage areas in ports can be covered (requiring structures with large spans to support a roof). Figure 4.10.3 shows a typical section of this type of facility.

Figure 4.10.3. ***Diagram of a Covered Storage Area***



Foundations for these structures are conditioned by the considerable eccentricity and inclination of the loads, mainly due to wind action on the roof, and the potential lateral pressure of bulk cargo piled against the walls.

Engineers must consider the alternative of deep foundations, even making use of the uplift capacity of piles, which may be transiently subject to tensile forces. On ground with a good bearing capacity, it can also be advisable to use footings or strip foundations that centre the loads either by their added weight or by eventual deep anchors.

Another particular feature of these structures is usually due to the compressibility of the ground, which settles under the storage loads and can give rise to parasitic loads in deep foundations (see Subsection 3.6.3.4). The bending moments and shear forces in piles can be determining factors in the design.

Ground improvement to reduce settlement caused by storage or delaying the construction of roofs and base slabs (after the primary settlement has occurred) are measures that should be studied in these cases.

The initial load on the base slabs of these structures should be applied as slowly as possible and the settlement monitored. It may be necessary to reconstruct the base slabs once most of the total expected settlement has taken place.

4.10.4 Submarine Outfalls and Other Underwater Pipelines

Sewage works may require the discharge of treated or untreated liquid effluent at considerable distances from the shoreline by sewer mains known as *submarine outfalls*.

Similar problems are found in the construction of oil and gas mains and underwater transmission cables (power, telecommunications, etc.).

4.10.4.1 Geotechnical Problems and Investigation Methods

The geotechnical problems associated with these installations are very varied. Table 4.10.1 attempts to summarize the most common types.

Table 4.10.1. Geotechnical Problems and Investigation Methods Associated with Discharge Outfalls and Underwater Pipelines (13)

Problem to Be Analysed	Information Necessary	Type of Investigation
1. Alignment selection	Geotechnical profiles. Location of obstacles. Rock outcrops. Geotechnical hazards.	Geophysics. Seismic reflection. Representative samples and identification tests.
2. Installation	Ground classification. Ground density. Shear strength.	Undisturbed samples (from vibrocorer or boreholes). Static penetration tests.
3. Buoyancy, bearing failure and transverse sliding	Ground density. Shear strength. Possibility of liquefaction due to to loads on the mains line.	Those indicated in 2. *In situ* vane test. Piezocone. SPTs.
4 Geotechnical hazards	Hazard zones. Ground strength.	Those indicated in 2 and 3. Seismic refraction.
5. Deformations	Stress-strain relation.	Those indicated in 1, 2, 3 and 4. Pressuremeter test.
6. Thermal effects	Ground temperature. Thermal conductivity.	Thermography.
7. Dynamic effects	Dynamic ground strength.	Those indicated in 1, 2, 3 and 4. Dynamic tests.

(13) Adapted from the table by Kolk, M.J. & P.T. Power. "Advances in Geotechnical Investigations and Design for Offshore Pipelines". Offshore Oil and Gas Pipeline Technology Conference. Copenhague. 1983.

Potential geomorphological irregularities should be examined when deciding on the pipe alignment (rock outcrops, changes in slope, etc.) and the presence of artificial obstacles.

The risk of natural instabilities of the seabed (erosion, quicksands, areas with a higher risk of liquefaction, areas close to potentially unstable slopes, etc.) should be investigated and plotted on a geotechnical map in order to select the safest alignment.

Various specific geotechnical investigations will be required to study the pipeline installation (dredging operations, installation of anchors, obtaining borrow material for protecting the installation, etc.). Several recommendations for these are given elsewhere in this ROM 0.5 (see Section 4.9 for dredging and fills).

The ground strength should be known in order to analyse the pipe stability against its different failure modes: bearing failure, buoyancy and transverse displacement.

Once a particular alignment has been defined, it will be necessary to specify the degree of safety against the natural geotechnical hazards referred to above.

Structural analysis of the mains will require knowing the soil reaction to the loads applied, as in the other soil-structure interaction problems covered in this ROM 0.5.

Studying the thermal effects, which can condition the structural design of this type of works, calls for prior knowledge of how natural temperatures vary on the seabed along the pipe alignment.

The external actions on these mains (wave action in coastal areas, sea currents, etc.) are dynamic in nature. Alternating loads may lead to an increase in porewater pressures in the ground, whose dynamic behaviour will have to be studied when the seabed soils are particularly sensitive to these effects (see Section 3.10).

The geotechnical investigation must extend all along the pipe course, with more intensity in singular areas (areas of heterogeneity, areas with a change in slope, where the alignment swerves, etc.).

The general geotechnical investigation methods are described in Part 2 of this ROM 0.5. In the case of these underwater pipes, the recommendations relative to offshore platforms (Section 4.8) should be followed, insofar as they are applicable.

Table 4.10.1 also lists some geotechnical investigation methods that are particularly relevant to underwater pipes and discharge outfalls.

4.10.4.2 Installation Procedures

Pipes can be fixed to the ground by burying them in a trench or by "surface mounting". Figure 4.10.4 shows these two ways of placing discharge outfalls.

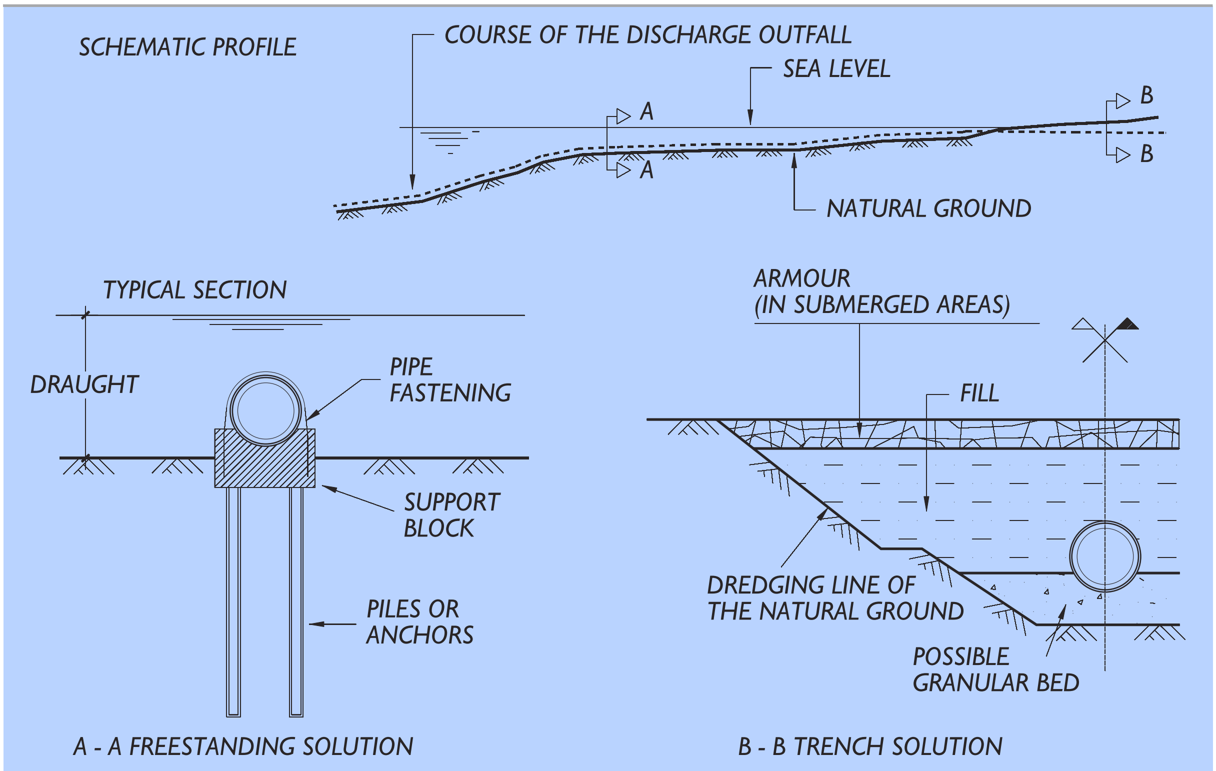
a. Trench Installation

A buried or trench installation can be done with previous dredging or by special excavation procedures (water jet, ground fluidisation, self-burying, etc.).

These installations must be at sufficient depth so that the pipe lies below the maximum depth at which it could be affected by natural movements of sediment as a result of currents, wave action or any other causes. Some recommendations in this respect are given in Sections 4.7 and 4.8 of this ROM 0.5.

Trenches will necessitate previous dredging or excavation. This stage of the works must be investigated and analysed as indicated in Section 4.9, so that stable excavation slopes can be defined during construc-

Figure 4.10.4. **Installation of Submarine Outfalls**



tion, based on the type of ground through which the discharge outfall passes. Recommendations are given in the same section concerning submerged structural fills.

The subsequent fill must be protected against potential scour or erosion caused by currents or wave action. Recommendations are given in Subsection 4.2.3.7 for protecting quay toes that can be of value in these installations.

b. Free-standing or "surface-mounted" pipes

Sea action on free-spanning pipes is considered to fall outside the scope of this ROM 0.5. Engineers need to investigate the drag forces to be considered in design calculations -resulting from currents and wave action- as well as any other possible external agents.

The mains could simply lie on the seabed with the necessary ballast to prevent flotation and to withstand external loads.

The pipeline supports must be capable of receiving the external loads together with the pipe self weight and transmitting them to the ground.

Loads can be transferred to the ground by shallow or deep foundations. In any event, possible scour and movements in the seabed should be considered. The foundation loads must be transmitted to a ground level deep enough not to be affected by these movements.

Piled or anchored foundations are generally more advantageous, particularly in seabed formed by soft soils or where substantial scour is expected.