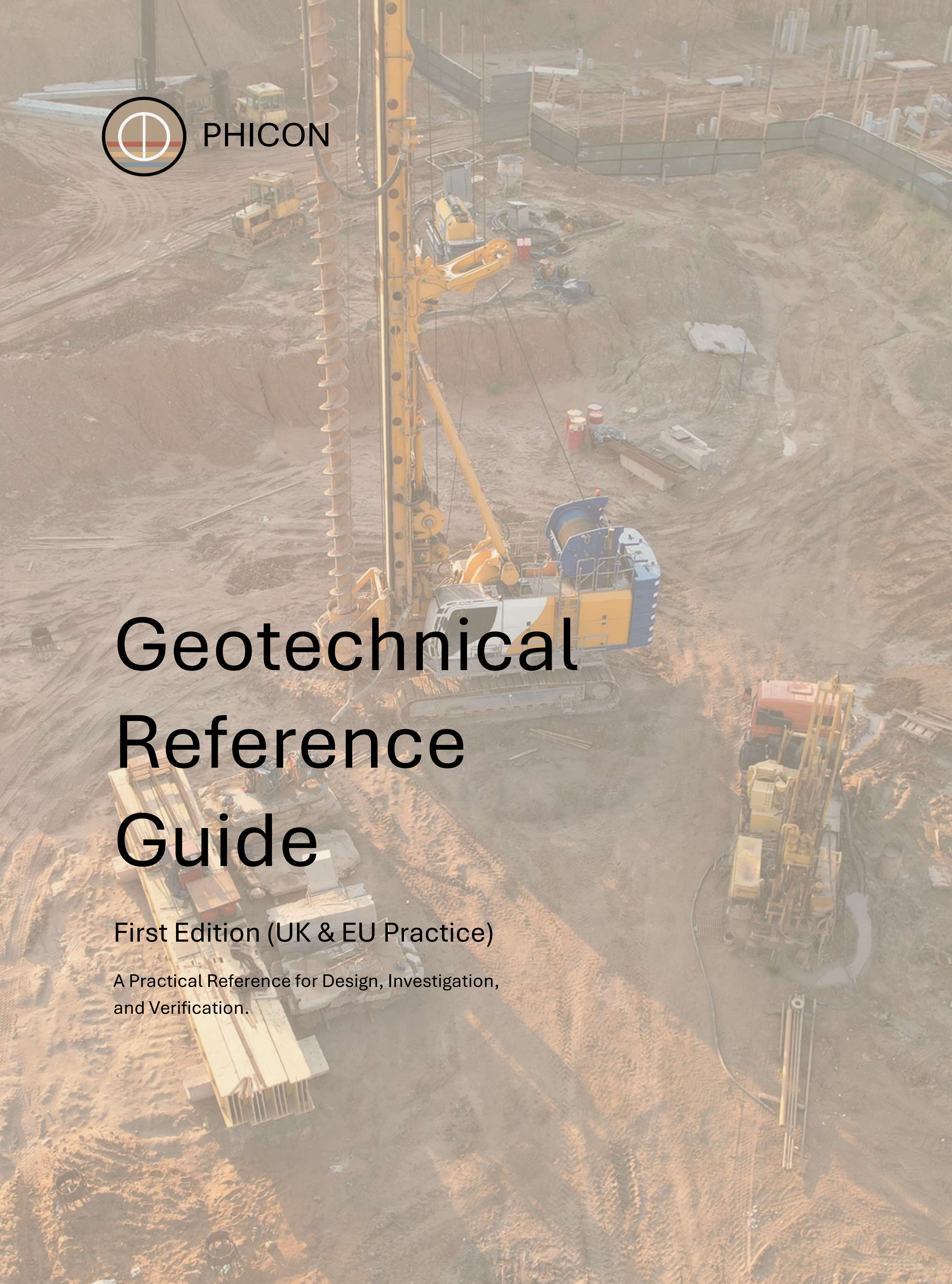

PHICON

Geotechnical Reference Guide

First Edition (UK & EU Practice)

A Practical Reference for Design, Investigation, and Verification.

A Phicon Associate Network Publication

GEOTECHNICAL REFERENCE GUIDE

First Edition (UK & EU Practice)

A Practical Reference for Design, Investigation, and Verification.

Geotechnical Design | Digital Engineering | Technical Assurance

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Preface and Reader's Guide

Purpose of this Reference Guide

This publication has been created as a **compact, technically precise, and practice-oriented reference** for engineers working under **Eurocode 7 (EN 1997)** and its National Annexes.

It distils over a decade of professional geotechnical experience into **concise, auditable guidance** that supports both design office work and field verification.

Unlike a textbook, this Reference Guide is designed to be *used daily* — to check equations, reference correlations, confirm EC7 factor sets, or recall design workflows during active projects.

Scope and Structure

The book follows the sequence of the **geotechnical project lifecycle**, divided into five practical parts:

Part	Focus Area	Typical User Context
Part 1	Fundamentals & Eurocode Framework	Design engineers, reviewers
Part 2	Soil, Site Investigation & Testing	Ground investigation engineers
Part 3	Foundations & Retaining Structures	Design verification, calculations
Part 4	Hydraulic, Earthworks & Ground Improvement	Construction QA, materials engineers
Part 5	Construction, Monitoring & Verification	Resident engineers, inspectors

Each chapter concludes with a **Worked Example** or **Quick Reference Table**, ensuring theoretical content is reinforced through application.

Design Conventions

To maintain alignment with Eurocode and UK practice, all symbols, terminology, and units follow **BS EN 1997**, **BS 5930**, and **ISO 80000**.

Values are presented in **SI base units** (kN, m, MPa) throughout, with key deviations clearly indicated.

Notation	Meaning	Unit
ϕ', c', c_u	Shear strength parameters	°, kPa
γ_G, γ_Q, γ_R	Partial factors on actions / resistance	—
E_d, R_d	Design effects and resistances	kN, kN/m^2
DA1-C1 / DA1-C2	Eurocode 7 design combinations	—
GEO / STR / EQU / UPL / HYD	EC7 limit states	—

Where relevant, diagrams and worked examples follow the **Phicon Calculation Template format**, matching the presentation used in Phicon tools.

Use in Practice

This book is intended to support:

- **Design development:** defining characteristic and design parameters.
- **Technical assurance:** verifying EC7 limit state compliance.
- **Field QA:** checking tolerances, materials, and platform strength.
- **Training and mentoring:** orienting graduates to UK & EU geotechnical design standards.

> **Phicon Note:** The goal is to promote *clarity, consistency, and accountability* in design — ensuring that engineering judgement is both transparent and verifiable.

Integration with Phicon Digital Tools

Each appendix and workflow is designed to integrate with Phicon's internal tool suite:

- **Excel calculators:** bearing capacity, retaining wall, slope, and settlement modules (v78+).
- **Word & PDF templates:** auto-populating verification sheets.
- **Power BI dashboards:** linking site QA data to design assumptions.
- **OneNote Calculation Books:** structured for EC7 clause traceability.

This ensures the same data chain can move seamlessly between **design, supervision, and reporting** — a core Phicon principle of *connected engineering delivery*.

Final Word

The *Geotechnical Reference Guide* is designed to evolve.

Future editions will expand coverage into digital and AI-assisted design workflows, harmonising traditional geotechnics with modern data systems — maintaining the balance between **engineering rigour** and **computational efficiency**.

"Geotechnical design is the art of judgement under uncertainty — this book exists to make that judgement defensible."

—

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PART 1: FUNDAMENTALS & EUROCODE FRAMEWORK

1 Basis of Geotechnical Design (EC7 Context)

1.1 Codes, standards, and frameworks in geotechnical engineering

Before turning to Eurocode 7 itself, it is important to recognise that geotechnical engineering operates within a much broader system of **codes, standards, and statutory frameworks** that have evolved over many decades. These documents collectively establish what is regarded as good geotechnical practice—from ground investigation through to construction and verification—ensuring that the safety, serviceability, and durability of works meet the expectations of society, regulators, and clients alike.

1.1.1 Historical and Statutory Context

The **Building Regulations 2010** (under the *Building Act 1984*) provide the statutory framework for building and civil engineering works in the UK, defining functional requirements for structural safety and performance. Historically, compliance with these regulations was demonstrated through adherence to national standards such as:

- BS 5930: Site Investigations
- BS 8002: Earth Retaining Structures
- BS 8004: Foundations

These standards, published by the **British Standards Institution (BSI)**, long defined accepted practice in UK geotechnical design. With the publication of *Eurocode 7* (BS EN 1997) and its UK National Annex, these earlier codes were formally superseded on **31 March 2010**. Nevertheless, they remain valuable reference sources, often cited as *non-conflicting complementary information (NCCI)* alongside EN 1997.

⚠️ Compliance with Eurocode 7 does not remove the need to consider the guidance of legacy British Standards, particularly where they provide construction, procedural, or safety context not explicitly covered by EN 1997.

1.1.2 Range of Applicable Standards

Geotechnical engineers today must reference a **suite of interrelated standards**, many of which underpin or complement *Eurocode 7*. These can be grouped as follows:

Discipline	Key Standards and Series
Ground Investigation & Testing	BS 5930; BS 1377 (Soil Tests); BS EN ISO 14688, 14689 (Classification); BS EN ISO 17892 (Laboratory Testing); BS EN ISO 22475–22477 (Sampling & Field Testing)); BS EN 1997-2 (*Ground Investigation and Testing*)
Design Codes	BS EN 1997-1 (*Geotechnical Design – General Rules*); BS EN 1993-5 (*Steel Structures – Piling*); BS EN 13793 (*Thermal Design of Foundations*)
Execution (Construction)	BS EN 1536–1538, 12063, 12699, 14199, 14475, 14490, 14731 (*Execution of Special Geotechnical Works* series); BS 8008; BS 8102

Materials & Products	BS EN ISO 13433, 13437, 13438 (Geosynthetics and Durability)

Together, these documents form the **geotechnical code environment**—a structured hierarchy in which *Eurocode 7* governs design philosophy, while the associated BS EN and ISO standards regulate testing, execution, and materials.

1.1.3 International and Complementary Guidance

In parallel with the European standards, a number of **international and UK guidance documents** remain influential in geotechnical practice. Under the *Vienna Agreement* (1991), the **International Organization for Standardization (ISO)** and **Comité Européen de Normalisation (CEN)** coordinate the development of unified geotechnical standards to avoid duplication and ensure compatibility.

Complementary technical references include:

- CIRIA C580 – Embedded Retaining Walls: Guidance for Economic Design
- CIRIA C750 – Groundwater Control
- ICE Specification for Ground Investigation (2nd Edition)
- BRE Digest Series and ICE Manuals of Geotechnical Engineering

A complete reference list of common standards and guidance documents is provided in **Appendix G**.

1.1.4 The Geotechnical Triangle – Understanding the Discipline

Before examining Eurocode 7, it is useful to recall the broader framework of geotechnical engineering. Burland (1987) described four interlinked aspects—**ground profile**, **soil behaviour**, **model**, and **experience**—forming the *geotechnical triangle*, a balance between observation, understanding, and modelling that defines reliable design.

Orr (2008) later extended this to the *geotechnical design triangle*, linking **geometry**, **loads and parameters**, and the **verification method** within the iterative design process.

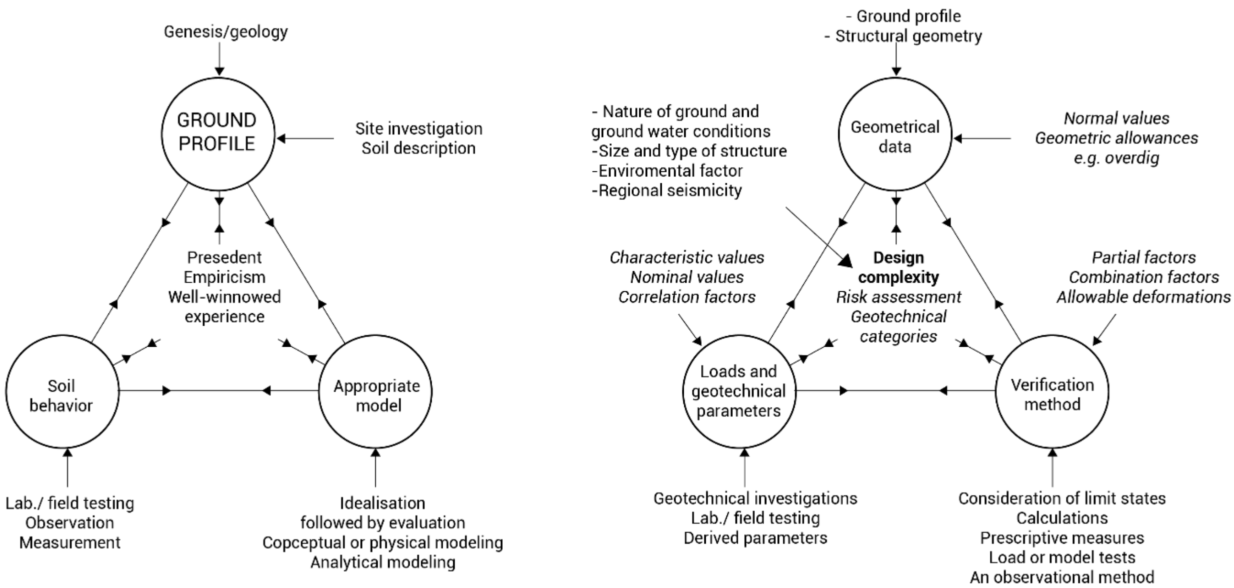


Figure 1: Left) The Geotechnical Triangle, Right) The Geotechnical Design Triangle

1.1.5 Transition to the Eurocode Framework

The **Structural Eurocodes programme**, initiated by the European Commission in 1975, established a unified design philosophy for buildings and civil engineering works across Europe. Within this framework, *Eurocode 7 – Geotechnical Design* represents the harmonised evolution of earlier British practice, integrating a **limit-state design approach**, **partial factors**, and **nationally determined parameters (NDPs)**.

The following sections introduce the **structure and role of Eurocode 7**, its relationship with EN 1990 and EN 1991, and the principles that underpin geotechnical design in accordance with the Eurocode framework.

1.2 Structure of Eurocode 7 and its relation to EN 1990–1999

Eurocode 7 (EN 1997) forms part of the broader family of structural design standards within the Eurocode suite, EN 1990–1999. Together, these provide a unified framework for the design of buildings and civil engineering works across Europe, ensuring consistency in safety, reliability and serviceability across structural materials and geotechnical disciplines.

1.2.1 Overview of the Eurocode Framework

Code	Title / Scope	Relevance to Geotechnical Engineers
EN 1990	Basis of Structural Design	Establishes the fundamental principles, limit states, design situations, and partial factor formats adopted throughout Eurocodes. EC7 relies on EN 1990 for definitions of actions, combinations, and reliability targets.
EN 1991	Actions on Structures	Defines characteristic actions (permanent, variable, accidental, seismic) and load combinations. These form input to geotechnical limit state checks (e.g., foundation loads, surcharge, earth pressures).
EN 1992–EN 1996	Structural Design Codes (Concrete, Steel, Composite, Timber, Masonry)	Used in parallel with EC7 when designing or verifying geotechnical structures involving structural elements such as retaining walls, piles, and anchors.
EN 1997	Geotechnical Design	Governs the design, verification, and execution of geotechnical works including foundations, slopes, earthworks, and retaining structures. Provides design approaches (DA1–DA3) and guidance on limit state verification.
EN 1998	Earthquake Resistance Design	Supplements EC7 for seismic design of geotechnical structures (e.g., liquefaction checks, seismic bearing capacity).
EN 1999	Aluminium Structures	Occasionally relevant for specialist structural applications in geotechnical equipment and temporary works.

1.2.2 Internal Structure of EN 1997

EN 1997 is divided into two core parts, supported by national annexes which adapt the code for each member state:

Part	Title	Focus
EN 1997-1	General Rules	Core principles for geotechnical design, including site investigation, limit state verification (GEO, STR, UPL, HYD), partial factors, and design approaches.
EN 1997-2	Ground Investigation and Testing	Specifies procedures and requirements for field and laboratory testing, classification of results, and derivation of design parameters.

Each country issues its **National Annex (NA)**, providing nationally determined parameters (NDPs) such as partial factors, correlation factors, and procedural preferences.

In the UK, BS EN 1997-1/-2 with UK National Annex is the governing standard, supported by BS 8004 and the ICE Specification for Ground Investigation (3rd Ed.) for local interpretation.

EU states adopt equivalent annexes—for example, Belgium (NBN EN 1997 ANB), Germany (DIN EN 1997 NA), France (NF EN 1997-1/-2 NA)—each modifying the partial factors and preferred design approach.

1.2.3 Relationship Between EN 1990 and EN 1997

EN 1990 establishes the **basis of structural design**, defining the reliability framework applied by all material and geotechnical codes. EN 1997 operates within this framework by:

- Applying **limit state design** principles (ULS and SLS) to ground–structure interaction.
- Adopting **partial factors on actions, materials, and resistances** consistent with EN 1990 reliability targets.
- Integrating with EN 1991 for **actions** and EN 1992–1998 for **structural resistance** checks.
- Allowing alternative **Design Approaches (DA1–DA3)** to ensure equivalent safety levels across national practices.

In effect, EN 1997 bridges **structural reliability** (per EN 1990) with **geotechnical uncertainty**, providing a transparent, auditable framework for design verification and compliance across Europe.

1.2.4 Key Takeaways for Practice

- Always verify the **governing National Annex** to determine applicable partial factors and preferred design approach.
- Treat EN 1990 as the foundation of reliability and load combination philosophy, and EN 1997 as its geotechnical implementation.
- Design deliverables (reports, calculations, and drawings) should clearly reference **EN 1997-1/-2**, relevant **NA**, and supporting standards (BS 8004, ICE, CIRIA).

- Consistent cross-referencing between **actions (EN 1991)**, **structural resistance (EN 1992–1996)**, and **geotechnical verification (EN 1997)** is essential for defensible and harmonised design outputs.

1.2.5 Worked Example – Application of Partial Factors (DA1/C1 and DA1/C2) to a Soil Parameter

Purpose:
To illustrate how characteristic soil parameters are converted into design values under **Design Approach 1 (Combinations 1 and 2)** as defined in **EN 1997-1 §2.4.7.3.4** and the **UK National Annex**.

In DA1 the combinations are (conceptually):

- C1 = A1 + M1 + R1
- C2 = A2 + M2 + R1

Given Data (Characteristic Values)

Parameter	Symbol	Characteristic Value	Unit	Description
Effective angle of shearing resistance	ϕ'_k	30	degrees	Derived from triaxial tests (effective stress)
Effective cohesion	c'_k	5	kPa	Minor apparent cohesion for dense sand–silt mix
Design approach	DA1 (C1 and C2)	—	—	per BS EN 1997-1 / UK NA Table A.4(A)

Partial Factors (UK National Annex)

Factor Set	On Actions	On Material Properties	On Resistance
Combination 1 (C1)	γ_F = 1.35 (G); 1.5 (Q)	$\gamma_{M;\phi,c}$ = 1.00	γ_R = 1.0 (on R)
Combination 2 (C2)	γ_F = 1.0 (G, Q)	$\gamma_{M;\phi,c}$ = 1.25	γ_R = 1.0 (on R)

Step 1 – Convert to Design Values (C1)

For Combination 1, partial factors are applied inferred on **soil strength parameters**.

$$\tan\,\varphi'_d = \frac{\tan\,\varphi'_k}{\gamma_{M,\varphi}}$$
$$c'_d = \frac{c'_k}{\gamma_{M,c}}$$
$$\varphi'_d = \varphi'_k = 30°, c'_d = 5kPa$$

✓ **Design parameters (C1):** ϕ'_n = 30°, c'_n = 5 kPa

Step 2 – Convert to Design Values (C2)

For Combination 2, partial factors are applied directly on **soil strength parameters**.

$$\tan \varphi'_d = \frac{\tan 30°}{1.25} = 0.462 \rightarrow \ tan^{-1}(0.462) \rightarrow \varphi'_d = 24.8°$$
$$c'_d = \frac{5}{1.25} = 4.0kPa$$

✓ **Design parameters (C2):** ϕ'_n = 24.8°, c'_n = 4 kPa

✓ **Resistance design:** $R_d = R_k/1.4$

Step 3 – Interpretation

Combination	Governing Partial Factor Application	Typical Use	Notes
C1	Apply to soil strength	ULS (GEO) when verifying ground failure modes	More conservative for parameter uncertainty
C2	Apply to resistance	ULS (GEO/STR) for combined soil–structure interaction	Aligns with EN 1990 action side factors

Key Practical Points

- In UK practice, **DA1 (C1 + C2)** is typically used for all ULS verifications.
- **Characteristic values** must always be used as input; design values are derived internally by applying partial factors (not user-entered).
- For **design charts or FE models**, C1 and C2 cases are run separately and the **more adverse** result governs.
- The **C1/C2 framework** aligns with EC7's reliability model, giving equivalent global factors of ~1.4–1.6 on bearing capacity.

Summary Table

Parameter	Characteristic	Design (C1)	Design (C2)	Units
ϕ'	30°	30°	24.8°	°
c'	5	5.0	4.0	kPa

Phicon Note:

In spreadsheet or calculator form, always separate *input characteristic parameters* from *derived design values*. This ensures auditable traceability under EC7 and allows quick adaptation for alternative NAs (e.g., Belgian ANB, German DIN NA).

1.3 Design Basis and Limit States (GEO, STR, EQU, UPL, HYD)

Geotechnical design to **Eurocode 7 (EN 1997-1)** follows the unified **design basis of EN 1990 – Basis of Structural Design**, which establishes the reliability framework for all Eurocodes.

A geotechnical structure must be verified for every **relevant limit state**, ensuring safety against collapse (ULS) and satisfactory performance in service (SLS).

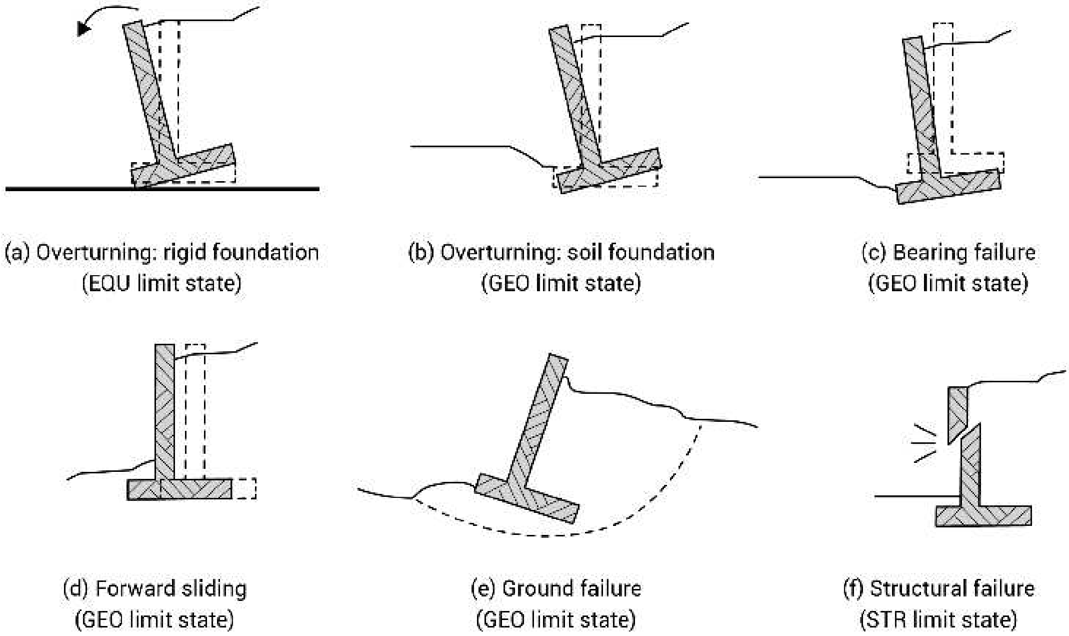


Figure 2: Limit states for cantilever retaining wall

1.3.1 Design Basis (EN 1990 §2 & EN 1997-1 §2.1)

The fundamental requirement is:

$$E_d \leq R_d$$

where:

- E_d= design value of the effect of actions (loads and soil reactions)
- R_d= design value of the resistance

This condition must hold for **all design situations** (persistent, transient, accidental, or seismic).
Reliability is ensured by applying **partial factors** (γ) to cover the uncertainty in loads, material properties, and model assumptions.

1.3.2 Categories of Limit States

Limit State	Symbol	Definition / Failure Mode	Typical Verification	Example Application
Geotechnical failure	GEO	Loss of equilibrium or excessive deformation of the ground	ULS	Bearing capacity of a foundation, slope failure
Structural failure	STR	Failure of a load-bearing structural element	ULS	Sheet-pile bending, RC wall yielding
Overall equilibrium	EQU	Instability of the structure or ground mass as a rigid body	ULS	Sliding or overturning of gravity wall

Uplift failure	UPL	Loss of stability due to upward pore-water pressure	ULS	Raft uplift, basement heave
Hydraulic failure	HYD	Loss of stability due to seepage or piping	ULS	Heave beneath excavation, piping at cut-off

Note: ULS checks guard against *collapse*, while SLS checks limit *movement and distortion* affecting serviceability.

1.3.3 Limit State Philosophy

Type	Purpose	Verification Method	Typical Outputs
Ultimate Limit State (ULS)	Prevent failure or instability	Apply partial factors to loads and resistances	Bearing capacity, sliding, overturning, uplift
Serviceability Limit State (SLS)	Ensure acceptable performance and comfort	Use unfactored (characteristic) loads	Settlement, tilt, lateral movement

Both must be satisfied:

- ULS → Safety
- SLS → Functionality and durability

1.3.4 Reliability and Partial Factors

The **reliability index β** (EN 1990 Annex C) represents the statistical measure of safety corresponding to the probability of failure $P_f = 1 - \Phi(\beta)$, where Φ is the standard normal cumulative distribution. Higher β values indicate lower failure probabilities and greater reliability. Typical target values are β = 3.3 for low, 3.8 for normal, and 4.3 for high consequence structures. In practice, Eurocode partial factors are calibrated to achieve these reliability levels without explicitly calculating β.

EN 1997 achieves equivalent reliability through calibrated **partial factors (γ)** applied to:

Category	Symbol	Typical Range	Covers Uncertainty in
Actions	γ_F	1.0 – 1.5	Loads and surcharges
Material Properties	γ_M	1.0 – 1.25	Soil or rock parameters (ϕ', c', cu)
Resistance	γ_R	1.0 – 1.4	Model or spatial variability of capacity

1.3.5 Design Approaches and Limit State Links

Design Approach	Main Checks	Typical Limit States	Adopted In
DA1 (C1 & C2)	Apply factors to loads + soil strength (C1) and to loads + resistance (C2)	GEO / STR / EQU	UK, France, Belgium
DA2	Apply factors simultaneously to loads and material parameters	GEO / STR	Germany, Netherlands
DA3	Factor loads from structure only; ground properties = characteristic	STR / GEO	Spain, Italy

Each design must demonstrate **adequate safety** in *all relevant limit states*, using the *nationally specified approach*.

1.3.6 Serviceability Limit State (SLS) Considerations

At SLS, **no partial factors** are applied ($\gamma = 1.0$).

Characteristic loads and soil parameters are used to predict movements.

Movement Type	Typical Criterion	Reference Standard
Total settlement (foundations)	25 – 50 mm	BS 8004 § 6.5
Differential settlement	1/500 – 1/300	BS 8004 § 6.5
Angular distortion (sensitive structures)	≤ 1/750	CIRIA C580
Retaining wall rotation (H/height)	≤ 0.5 % H	Experience-based

If deformations exceed these limits, redesign or ground improvement is required.

1.3.7 Worked Example – Limit State Verification of a Pad Foundation

Given:

- $B = 2m, D_f = 1m$
- $G_k = 300kN, Q_k = 50kN, R_k = 450kN$
- Design Approach = DA1 (C1 & C2)

(a) ULS – Combination 1 (C1):

$$V_d = 1.35G_k + 1.5Q_k = 1.35(300) + 1.5(50) = 502.5kN$$
$$R_d = \frac{R_k}{1.0} = 450kN$$

→ $E_d > R_d$ → Not OK

(b) ULS – Combination 2 (C2):

$$V_d = 1.0(G_k + Q_k) = 350kN$$
$$R_d = \frac{R_k}{1.4} = 321kN$$

→ $E_d > R_d$ → Not OK

✓ Both combinations unsafe → increase foundation width or improve bearing stratum.

(c) SLS – Service Check:
At characteristic load: $V_k = 350kN < R_k = 450kN$
Predicted settlement = 18 mm < 25 mm → **Satisfactory**

Phicon Note:

In design documentation, always state:
▪ Which **limit states** have been verified (GEO, STR, EQU, UPL, HYD).
▪ Which **design approach** (DA1, DA2, DA3) was used.
▪ That both **ULS and SLS** checks have been completed and the **governing case identified**.

This clarity allows Category 2/3 checkers and reviewers to trace EC7 compliance without recalculation.

1.4 Design Approaches (DA1, DA2, DA3) and Combination Rules

Eurocode 7 recognises that geotechnical reliability can be achieved by several equivalent routes.
To reflect national practice and calibration history, **three Design Approaches (DA1–DA3)** are permitted under **EN 1997-1 §2.4.7.3**.

Each defines *how* partial factors are applied to actions, soil parameters and resistances when verifying limit states.

1.4.1 The Concept

The same reliability target (β ≈ 3.8 for ULS) can be met by applying partial factors to different quantities.

In all cases, safety is demonstrated when:

$$E_d \leq R_d$$

but the way in which **γ_F, γ_M**, and **γ_R** are applied varies between design approaches.

1.4.2 Summary of the Three Design Approaches

Design Approach	Partial Factors Applied To	Typical Limit States	Used In	Remarks
DA1	Two combinations: ▪ C1 – Apply factors to **actions and soil strength** (γ_F, γ_M) ▪ C2 – Apply factors to **actions and resistance** (γ_F, γ_R)	GEO, STR, EQU	UK, France, Belgium	Both C1 & C2 must be checked; more adverse governs.

DA2	Apply factors simultaneously to **actions and material properties**	GEO, STR	Germany, Netherlands	Equivalent reliability to DA1 (C1 + C2).
DA3	Apply factors to **actions from the structure** only; soil parameters remain characteristic	STR, GEO	Spain, Italy	Often used where structural loads dominate (retaining walls, anchors).

⚠️ **UK Practice:**
BS EN 1997-1/NA specifies **DA1** as mandatory. Designers must evaluate both Combinations 1 and 2.

1.4.3 Combination Rules for DA1

Combination	Factors Applied To	Purpose	Typical Result
C1	γ_F on actions + γ_M on soil parameters	Tests sensitivity to weak soil strength	Usually governs bearing/sliding failures
C2	γ_F on actions + γ_R on resistance	Tests sensitivity to global resistance model	Often governs passive or wall capacity

Both results are compared and the **more critical governs** the design.

1.4.4 Typical Partial Factors (UK National Annex)

Category	Symbol	C1	C2	Reference
Permanent actions (unfavourable)	γ_G	1.35	1.0	EN 1990 Table A1.2(A)
Variable actions	γ_Q	1.5	1.0	EN 1990 Table A1.2(A)
Soil strength parameters (ϕ', c')	γ_M	1.25	1.0	BS EN 1997-1/NA Table A.4(A)
Unit weight (γ')	γ_M	1.0	1.0	—
Resistance (R)	γ_R	1.0	1.4	—

1.4.5 Typical European Variations (for Reference)

Country / Annex	Design Approach	Main Variation from UK NA

Belgium (ANB)	DA1 (C1 & C2)	Slightly reduced $\gamma_M = 1.2$ for ϕ'; DA1 used consistently for all GEO/STR.
Germany (DIN NA)	DA2	Single set of factors ($\gamma_F = 1.35$, $\gamma_M = 1.25$); avoids dual C1/C2 runs.
France (NF EN NA)	DA1	Similar to UK but different ψ-factors on variable actions.
Spain / Italy	DA3	Characteristic soil strengths retained; higher γ_F on structural loads.

These deviations reflect historic practice and calibration against local safety formats (global factor ≈ 1.4–1.6).

1.4.6 Visual Logic – DA1 Framework

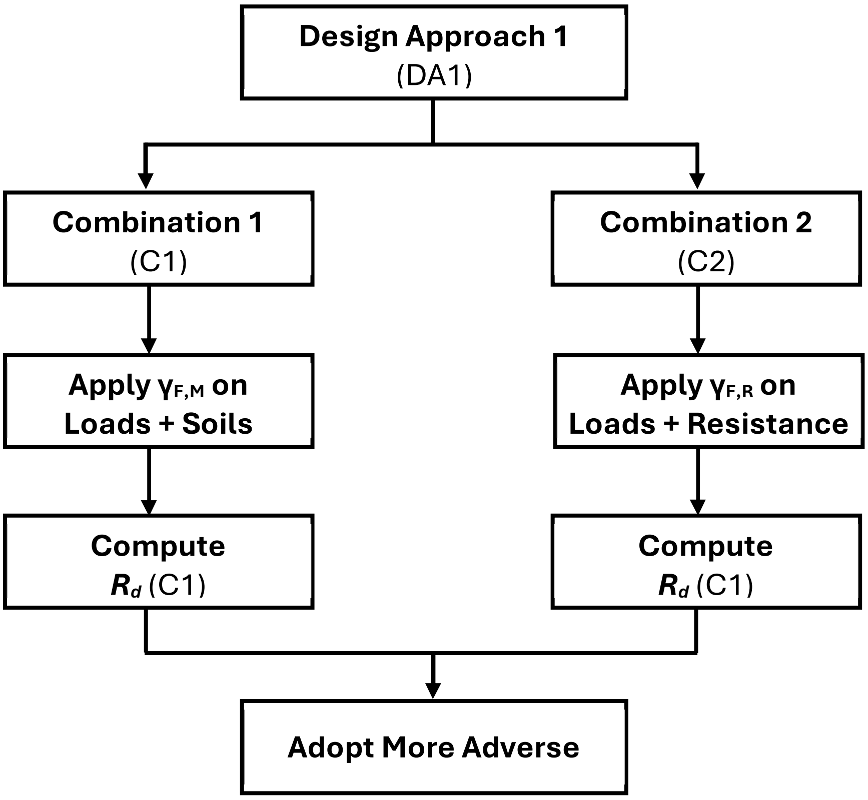


Figure 3: Design Approach Framework

1.4.7 Worked Example – Comparison of DA1 (C1) and (C2)

Given:

- $R_k = 450kN$
- $G_k = 300kN, Q_k = 50kN$

Combination 1:

$$V_d = 1.35G_k + 1.5Q_k = 502.5kN, R_d = 450kN$$

→ Utilisation = 1.12 → **Not OK**

Combination 2:

$$V_d = 1.0(G_k + Q_k) = 350kN, R_d = 450/1.4 = 321kN$$

→ Utilisation = 1.09 → **Not OK**

✓ Governing case = C1 (more adverse).

Increase foundation area or improve soil parameters until $E_d \leq R_d$.

Phicon Note:

In spreadsheets and digital calculators, always **automate both DA1 Combinations (C1 and C2)** with a clear "governing" flag.
This prevents manual input errors and maintains full audit traceability under EC7.
When preparing a design report, quote:

- Design Approach used
- ULS and SLS verified limit states
- National Annex reference (e.g. BS EN 1997-1 UK NA:2019).

1.5 Partial Factors on Actions and Materials (UK NA vs EU)

Partial factors (γ) provide the quantitative link between characteristic and design values.
They compensate for uncertainty in loading, material strength, geometry, and analysis models — ensuring a uniform target reliability across all Eurocodes.

In **EN 1997-1**, partial factors are applied according to the chosen **Design Approach (DA1, DA2, or DA3)**.
Each National Annex specifies its preferred set and magnitude.

1.5.1 Concept

$$\textit{Design Value} = \frac{\textit{Characteristic Value}}{\textit{Partial Factor}}$$

or, for loads:

$$F_d = \gamma_F \cdot F_k$$

Thus, every soil parameter, load, or resistance is factored in the direction that produces a *more adverse design condition*.

1.5.2 Categories of Partial Factors

Category	Symbol	Applied To	Purpose
Actions	γ_F	Permanent (G), variable (Q), accidental (A) loads	Accounts for load magnitude and frequency uncertainty
Material Properties	γ_M	Soil or rock parameters (ϕ', c', cu, γ')	Covers scatter in measured values and model simplifications
Resistance	γ_R	Global bearing, sliding, passive, or pull-out resistance	Covers variability in model accuracy and spatial heterogeneity

1.5.3 Partial Factors – UK National Annex (BS EN 1997-1/NA:2019)

Factor Type	Symbol	Combination 1 (C1)	Combination 2 (C2)	Typical Application
Permanent action (unfavourable)	γ_G	1.35	1.0	Self-weight, overburden
Variable action	γ_Q	1.5	1.0	Surcharge, traffic, crane load
Favourable actions	γ_G, γ_Q	1.0	1.0	Resistive effects (rarely governing)
Shear strength (ϕ', c')	γ_M	1.25	1.0	Reduction applied in C1
Undrained shear (cu)	γ_M	1.4	1.0	UK NA Annex A.4(A)
Unit weight (γ')	γ_M	1.0	1.0	—
Resistance (R)	γ_R	1.0	1.4	Applied in C2

UK Practice:

Design Approach 1 (C1 & C2) is mandatory.

Both combinations are evaluated; the more adverse governs the design.

1.5.4 Comparative Factors – Selected EU National Annexes

Country / Annex	Design Approach	γ_F	γ_M (ϕ')	γ_R	Key Notes
UK (BS NA)	DA1 (C1/C2)	1.35 / 1.5	1.25	1.4	Standard EC7 UK practice
Belgium (NBN EN 1997 ANB)	DA1 (C1/C2)	1.35 / 1.5	1.20	1.4	Slightly less conservative on ϕ'
France (NF EN 1997-1/NA)	DA1 (C1/C2)	1.35 / 1.5	1.25	1.3–1.4	Similar to UK but allows γ_R = 1.3 in some GEO cases
Germany (DIN EN 1997-1/NA)	DA2	1.35	1.25	—	Single combination, no C1/C2 split
Spain / Italy	DA3	1.50	1.00	—	Characteristic soil strengths retained; factors on structural actions only

These differences produce similar **global safety levels** (≈ 1.4 – 1.6) but reflect national calibration and historical practice.

1.5.5 Practical Application

When verifying a design, always apply partial factors on the correct side of the equation:

Design Component	Equation Form	Applied Factors
Actions	$E_d = \gamma_F\, E_k$	γ_G, γ_Q
Material Strength	$\tan\ \varphi'_d = \tan\ \varphi'_k/\gamma_M$	γ_M
Resistance	$R_d = R_k/\gamma_R$	γ_R

At the design output stage, verify that $E_d \leq R_d$ for each combination.

1.5.6 Worked Example – Factoring Actions and Soil Strength (UK DA1)

Given:

- $\varphi'_k = 30°,\ c'_k = 5kPa,\ R_k = 450kN$
- $G_k = 300kN,\ Q_k = 50kN$

Combination 1 (C1):

$$V_d = 1.35G_k + 1.5Q_k = 502.5kN$$
$$\tan\ \varphi'_d = \frac{\tan\ 30°}{1.25} = 0.462 \Rightarrow \varphi'_d = 24.8°$$
$$c'_d = 5/1.25 = 4kPa$$

✓ Use reduced soil strength; $\gamma_R = 1.0$

Combination 2 (C2):

$$V_d = 1.0(G_k + Q_k) = 350kN$$
$$R_d = R_k/1.4 = 321kN$$

✓ Use characteristic soil strength; $\gamma_R = 1.4$

Result:

C1 → $\frac{E_d}{R_d} = 1.12$

C2 → $\frac{E_d}{R_d} = 1.09$

Governing Case: C1

Phicon Note:

▪ Always document which **combination (C1 or C2)** governs.

▪ Avoid user-editable partial factors in calculators — embed UK NA defaults and show reference tables for traceability.

▪ When reviewing EU designs, note that **γ_M** may differ; never compare raw results without checking the governing National Annex.

1.6 Serviceability Limit State Concepts

While **Ultimate Limit States (ULS)** protect against collapse, **Serviceability Limit States (SLS)** ensure the structure continues to perform as intended throughout its life — without unacceptable deformation, cracking, or functional loss.

In **EN 1997-1**, SLS checks are equally important to client assurance and long-term durability.
They typically govern foundation and retaining wall design in soft or compressible ground.

1.6.1 Basis of SLS Verification (EN 1997-1 §2.4.8)

At SLS, **characteristic values** of loads and material properties are used without factoring:

$$E_k \leq R_k$$

or, more practically,

Predicted deformation ≤ Allowable deformation

All partial factors are taken as **γ = 1.0**.

The emphasis is on *performance*, not safety margin.

1.6.2 Typical SLS Checks in Geotechnical Design

Structure / Element	SLS Parameter	Check Type	Verification Tool
Shallow foundations	Total and differential settlement	Elastic, consolidation or empirical	Elastic theory, oedometer data, PLT correlation
Pile foundations	Elastic shortening and group settlement	Analytical or numerical	t-z, q-z curves, load-test correlation
Retaining walls	Rotation, lateral movement, wall deflection	FEM or simplified methods	Rankine/Coulomb with ΔH limits
Slopes / embankments	Long-term creep and surface settlement	Time-dependent models	Limit-equilibrium or numerical
Underground structures	Lining distortion, ground loss	Empirical, analytical, or FE	Volume-loss analysis, ground settlement analysis

1.6.3 Movement and Deformation Limits (Indicative Values)

Criterion	Allowable Value	Reference / Source
Total settlement (individual footing)	25 – 50 mm	BS 8004 § 6.5

Differential settlement (adjacent footings)	1/500 – 1/300	BS 8004 § 6.5
Angular distortion (sensitive structures)	≤ 1/750	CIRIA C580
Retaining wall rotation	≤ 0.5 % of retained height	Empirical experience
Lateral displacement (at ground level)	≤ 0.3 % H	Numerical verification
Heave / swelling uplift	< 10 mm (light structures)	BRE Digest 240

Actual limits should be set by the structural engineer in consultation with the geotechnical designer, considering frame stiffness, cladding, utilities, and tolerance of finishes.

1.6.4 Distinction Between Total and Differential Movement

- **Total settlement (s_t):** absolute downward movement of a foundation or slab.
- **Differential settlement (Δs):** relative movement between two points.
- **Angular distortion (β):** Δs / distance between points.

$$\beta = \frac{\Delta s}{L}$$

Serviceability is generally controlled by **β**, as differential movement governs damage potential.

1.6.5 Load Combinations at SLS

EN 1990 defines serviceability combinations using **ψ-factors** to represent realistic load coexistence:

$$E_k = G_k + \psi_{2,i} \cdot Q_{k,i}$$

Typical ψ_2 values (UK NA to EN 1990 A1.1):

Action	ψ_2	Example
Live load	0.3	Buildings
Traffic load	0.3	Pavements, platforms
Earth / water pressure	0.8	Retaining structures
Temperature	0.3	Slabs, bridge decks

1.6.6 Analytical Approaches to SLS

Approach	Best Used For	Key Input Parameters
Elastic (Immediate) settlement	Sands, lightly loaded rafts	E_s, ν, B, L, Δσ

Consolidation settlement	Clays, long-term loading	$C_c, C_r, m_v, H, \Delta\sigma'$
Empirical correlations	Preliminary estimates	SPT N, CPT q_c, PLT E_{V2}
Numerical (FEM/FEA)	Complex interaction	Constitutive model; E, ν, c'–ϕ' or s_u; k; γ; boundary conditions and stages.

1.6.7 Worked Example – Immediate (Short term) Settlement Check

Given:

Pad foundation 2 m × 2 m on over-consolidated clay

$$q = 180kN/m^2, E_s = 20000kN/m^2, \nu = 0.3$$

$$s = \frac{qB(1-\nu^2)}{E_s} = \frac{180 \times 2 \times (1-0.09)}{20000} = 0.0164m = 16mm$$

✓ **Result:** Total settlement = 16 mm < 25 mm → **Satisfactory**

Differential movement between adjacent pads (Δs = 8 mm over 4 m):

$$\beta = \Delta s/L = 8/4000 = 1/500 \Rightarrow \text{within tolerance}$$

Phicon Note:

▪ Always distinguish **ULS safety** from **SLS performance** — the former prevents collapse; the latter ensures durability and client satisfaction.
▪ SLS checks should be included in every calculation pack even when ULS governs.
▪ When uncertain, adopt **conservative allowable movements** from BS 8004 and CIRIA C580 to defend design assumptions.

1.7 Geotechnical Categories (GC1–GC3)

Eurocode 7 classifies geotechnical projects into **three Geotechnical Categories** according to their complexity, ground risk, and consequence of failure.

This classification determines the **extent of investigation**, the **level of design verification**, and the **supervision intensity** required for safe and economical design.

1.7.1 Purpose of Categorisation (EN 1997-1 §2.1)

- To ensure design effort matches risk.
- To define whether simplified, standard, or detailed design is acceptable.
- To determine whether **independent checking** or **enhanced monitoring** is necessary.
- To guide the level of documentation — Ground Investigation Report (GIR), Geotechnical Design Report (GDR), and Verification Records.

1.7.2 Overview of Geotechnical Categories

Category	Typical Project Type	Design Complexity	Verification Level	Examples
GC1	Simple structures on uniform ground	Routine, low risk	Design by experience or prescriptive rules	Small pads, lightly loaded slabs, shallow trenches
GC2	Normal structures with moderate ground variability	Requires analysis and limited numerical checks	Design based on site data and standard EC7 procedures	Typical building foundations, retaining walls, slopes
GC3	Complex or high-risk projects	Advanced analysis and independent review	Detailed investigation, Category 3 (independent) check	Deep excavations, large retaining systems, tunnels, dams, offshore structures

⚠️ **UK Practice:**

Category 2 covers most conventional infrastructure.

Category 3 applies where failure has major consequences or the design is novel, uncertain, or outside precedent.

1.7.3 Defining Geotechnical Category in Design

Assessment Aspect	GC1	GC2	GC3
Ground conditions known from local experience	✓	✓/ ⚠️	⚠️
Ground behaviour simple and predictable	✓	✓/ ⚠️	✗
Standard methods of design applicable	✓	✓	⚠️
Failure consequence minor	✓	⚠️	✗
Design requires specialist expertise	✗	⚠️	✓
Independent checking required	✗	⚠️	✓ (mandatory)
Observational method or monitoring needed	✗	⚠️	✓

1.7.4 Category Checks in UK & EU Practice

Country	Verification Structure	Typical Requirement for GC3
UK (BS EN 1997 + ICE/HA)	Cat I (self-check), Cat II (independent review), Cat III (independent organisation)	Mandatory independent Category III check
Belgium (NBN EN 1997 ANB)	Similar three-tier verification	Cat III = external design review ("contrôle externe")

Germany (DIN 1054)	Prüfbuch system (checking engineer)	"Prüfingenieur für Geotechnik" certifies Category 3 designs
France (NF P 94)	Contrôle technique	Independent technical control of complex or high-risk works

1.7.5 Geotechnical Design Reports (EN 1997-1 §2.8)

Every geotechnical design must be documented through the following deliverables:

Document	Purpose	Typical Content
Ground Investigation Report (GIR)	Summarises site conditions and results	Factual and interpretative GI data, ground model
Geotechnical Design Report (GDR)	Defines design assumptions, verifications, and methods	Limit states, design approach, partial factors, results
Verification Report / Check Certificate	Confirms compliance under GC3	Independent reviewer's confirmation of adequacy

The GDR is the **central auditable record** for all design decisions under EC7 — equivalent to a "Design Statement" under CDM Regulations.

1.7.6 Role of the Observational Method (EN 1997-1 §2.7)

For GC3 projects with high uncertainty, EC7 allows the **Observational Method**, where design is verified progressively during construction.

This requires:

- Defined ranges of acceptable behaviour,
- Pre-established monitoring triggers, and
- A response plan for exceedances.

⚠️ It is **not a substitute for design**, but a structured way to manage ground uncertainty.

1.7.7 Worked Example – Assigning Geotechnical Category

Project	Ground Conditions	Consequences	Category
Small retaining wall (2 m) for car park	Uniform granular soil	Low	GC1
5 m basement excavation in variable clay	Moderate	Medium	GC2
20 m deep diaphragm wall for quay	Variable fill over sand, high water	Major	GC3
Piled raft supporting energy facility	Variable strata, high economic risk	High	GC3

✓ GC2 designs typically require a full GDR with ULS/SLS verification.

✓ GC3 requires an independent Cat III check and a monitoring plan.

Phicon Note:

▪ State the **Geotechnical Category** explicitly in every GDR — it governs the checking and monitoring scope.
▪ For UK public-sector projects, align with **HA HD 22/08** or equivalent guidance for Category 2/3 verification.
▪ Treat the Category as a **risk management tool**: higher categories justify deeper investigation, modelling, and supervision budgets.

1.8 Design Workflow Card – from Characteristic → Design → Verification

Eurocode 7 adopts a consistent and auditable process: define **characteristic inputs**, convert to **design values**, perform **limit-state checks**, and record results in the **Geotechnical Design Report (GDR)**.

The steps below outline a defensible, repeatable workflow for all geotechnical designs.

1.8.1 Step-by-Step Workflow

Step	Task	Basis / Source	Key Output
1	**Define Design Situation** – Persistent / Transient / Accidental / Seismic	EN 1990 §3.2 / EN 1997-1 §2.4.6	Load case list
2	**Establish Ground Model** – from GIR / site data	EN 1997-2 / BS 5930 / ICE Spec GI	Layer properties, γ', ϕ', c', c_u
3	**Select Characteristic Values (k)**	EN 1997-1 §2.4.5	$ϕ'_k$, c'_k, cu_k, E_k, G_k, Q_k
4	**Apply Partial Factors** according to Design Approach (DA1 C1/C2 or DA2 / DA3)	EN 1997-1 §2.4.7	$ϕ'_d$, c'_d, R_d, V_d, H_d, M_d
5	**Analyse Design Model** – bearing, sliding, wall, pile, slope etc.	EN 1997 Part 1 Chs 6–9 / DA chosen	Limit-state effects Ed and Rd
6	**Verify Limit States**	EN 1997-1 §2.4.8	ULS (GEO, STR, EQU, UPL, HYD) → $E_d \leq R_d$; SLS → within tolerances
7	**Identify Governing Combination**	Compare C1/C2 or load cases	Governing result table
8	**Document in GDR** – assumptions, factors, verifications	EN 1997-1 §2.8 / UK NA A.4	Signed record of compliance
9	**Implement Verification Level** (Cat I / II / III)	HA HD 22/08 / BS EN 1997	Check certificate or peer review
10	**Monitor / Review** during construction (if GC 3 or Observational Method)	EN 1997-1 §2.7	Trigger levels + action plan

1.8.2 Flow Diagram – Eurocode 7 Design Logic

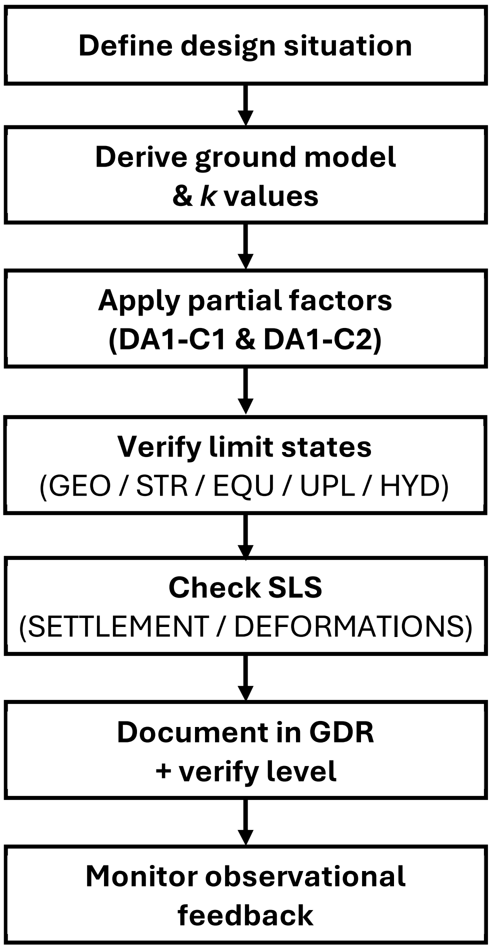


Figure 4: Design flow chart for Eurocode 7

1.8.3 Verification Snapshot (Example Summary Table)

Limit State	Design Approach	Combination	γ-Sets Applied	Result	Status
Bearing (ULS GEO)	DA1	C1	γ_F, γ_M	E_d = 502 kN, R_d = 450 kN	✗ Not OK
Bearing (ULS GEO)	DA1	C2	γ_F, γ_R	E_d = 350 kN, R_d = 321 kN	✗ Not OK
Settlement (SLS)	—	—	$\gamma = 1.0$	s = 16 mm < 25 mm	✓ OK
Sliding (EQU)	DA1	C1	γ_F, γ_M	$F_d < R_d$	✓ OK

Governing case: ULS GEO (C1) → increase foundation size.

1.8.4 Minimum Documentation for Compliance

Document / Record	Purpose
Ground Investigation Report (GIR)	Defines factual and interpreted ground data
Geotechnical Design Report (GDR)	Records assumptions, calculations, and verifications
Cat II/III Check Certificate	Confirms independent technical review
Construction Monitoring Logs	Records observational compliance and trigger actions

Phicon Note:
▪ Use this workflow as a **design checklist** for every calculation sheet or digital tool.
▪ Keep **inputs (characteristic)**, **transformations (partial factors)**, and **outputs (design verifications)** clearly separated.
▪ Ensure the GDR shows traceability between the **Eurocode clause**, the **national annex**, and the **verified result**.
▪ Where multiple associates contribute, this workflow provides a **common QA structure** and proof of EC7 compliance.

PART 2: SOIL, SITE INVESTIGATION & TESTING

2 Soil Fundamentals and Physical Properties

2.1 Nature and Composition of Soils

Soil is a three-phase material composed of **solid particles**, **water**, and **air**. Its engineering behaviour depends on the relative proportions of these phases and the interaction between particles.

2.1.1 Soil as a Three-Phase System

Phase	Symbol	Description	Typical % by Volume
Solids	V_s	Mineral grains (sand, silt, clay, organics)	40–60 %
Water	V_w	Occupies voids when saturated (pore water)	0–50 %
Air	V_a	Remaining void space above water table (pore air)	0–30 %

Total volume, $V = V_s + V_w + V_a$.

The relationship between these volumes defines the soil's **phase relationships** — the basis of most laboratory and field calculations.

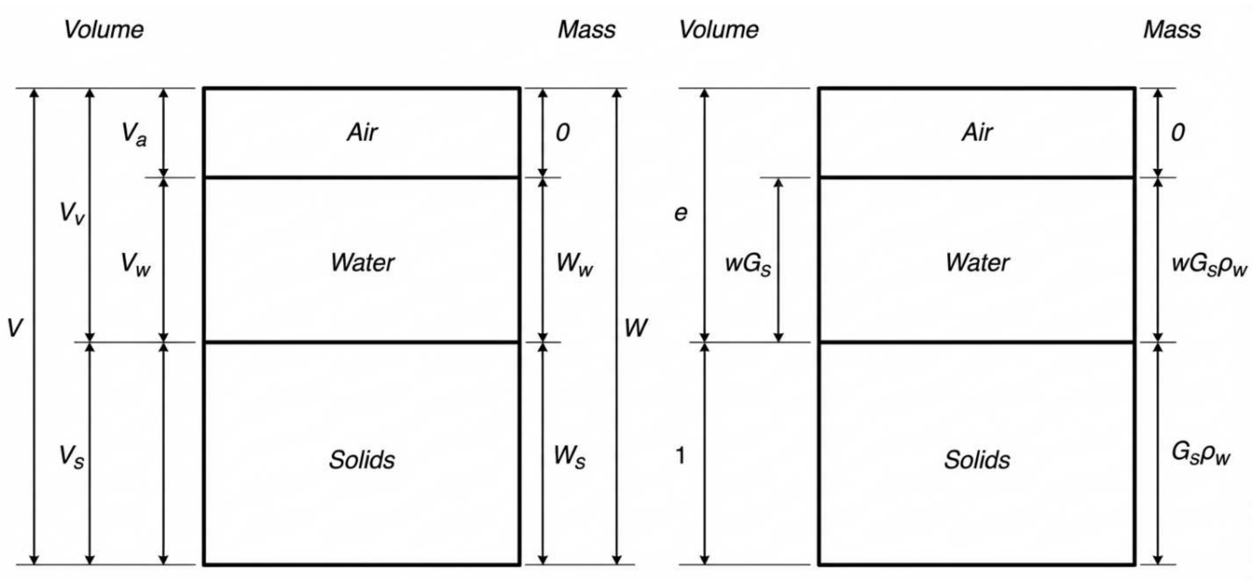


Figure 5: Phase diagram for establishing properties of fill materials (reproduced from Craig 2004)

2.1.2 Key Physical Quantities

Parameter	Symbol	Definition / Formula	Typical Range
Water content	w	$w = \frac{W_w}{W_s} \times 100\%$	5–50 %
Bulk (total) density	ρ	$\rho = \frac{W}{V}$	1.6–2.2 Mg/m^3
Dry density	ρ_d	$\rho_d = \frac{W_s}{V}\ or\ \frac{\rho}{1+\omega}$	1.2–2.0 Mg/m^3
Particle (solid) density	ρ_s	$\rho_s = \frac{W_s}{V_s}$	2.60–2.75 Mg/m^3 (minerals)
Void ratio	e	$e = \frac{V_v}{V_s}$	0.4–1.2
Porosity	n	$n = \frac{V_v}{V}$	0.3–0.6
Degree of saturation	S_r	$S_r = \frac{V_w}{V_v} \times 100\%$	0–100 %
Unit weight	$\gamma = \rho g$	Multiply by $g = 9.81 m/s^2$	

Phase relationships connect **mass–volume–density** parameters.

They are essential for compaction control, settlement prediction, and effective stress evaluation.

Note: In geotechnical practice, mass (kg or Mg) is sometimes used interchangeably with weight (force, γ) for simplicity. From now on this book, wherever weight is used, it implies that mass has been multiplied by the gravitational acceleration ($g = 9.81\ m/s^2$). In such cases, the units of kg or Mg are converted to N or kN, respectively, and the term "**mass–volume–density**" becomes "**weight–volume–density**".

2.1.3 Typical Mineral Composition

Soil Type	Dominant Minerals	Engineering Behaviour
Gravel / Sand	Quartz, feldspar	High strength, permeable, low compressibility
Silt	Quartz, mica, fine rock flour	Moderate plasticity, frost-susceptible
Clay	Kaolinite, illite, montmorillonite	Plastic, low permeability, time-dependent
Organic / Peat	Humic matter	Highly compressible, low strength
Chalk / Weak rock	Calcite	Variable strength, crushable

2.1.4 Classification of Natural Deposits

Deposit Type	Origin	Typical Examples (UK / EU)	Engineering Notes
Residual	Weathered in-situ	Clay-with-flints, saprolites	Heterogeneous; depth varies
Transported – Glacial	Ice, meltwater	Till, glaciofluvial sands	Variable strength; boulders
Fluvial / Alluvial	Rivers	River gravels, floodplain silts	Stratified; may be loose or soft
Marine / Estuarine	Tidal deposition	Clays, silts, sands	Often normally consolidated
Aeolian	Wind-blown	Loess, dune sands	Loose, collapsible on wetting
Made Ground	Human-placed	Fill, rubble, reworked soils	Highly variable; chemical risks

2.1.5 Worked Example – Phase Relationship and Bulk Density

Given:
A soil sample has:

- Total (wet) mass = 1850 g
- Dry mass = 1500 g
- Volume = 1.0×10^{-3} m^3

Find: water content w, bulk density ρ, and dry density ρ_d.

$$w = \frac{1850 - 1500}{1500} = 0.233 = 23.3\%$$

$$\rho = \frac{1850}{0.001} = 1{,}850{,}000\ g/m^3 = 1.85\ Mg/m^3$$

$$\rho_d = \frac{\rho}{1 + w} = \frac{1.85}{1.233} = 1.50\ Mg/m^3$$

Phicon Note:

• Always check phase relationships when reviewing laboratory results— they form the backbone of density, compaction, and consolidation calculations.
• In digital QA systems, auto-derive all inter-linked parameters ($w, \rho, \rho_d, e, n, S_r$) from two known values to minimise transcription errors.
• Adopt $\rho_s = 2.65\ Mg/m^3$unless site-specific mineralogy suggests otherwise (chalk, coal, ironstone, etc.).

2.2 Phase Relationships and Index Properties

The **phase relationships** link the fundamental parameters describing soil's weight–volume composition.

They form the basis for calculating densities, void ratios, and degrees of saturation — essential for laboratory reporting, compaction control, and settlement prediction.

2.2.1 Fundamental Relationships

Let

$$V = V_s + V_v = V_s + V_w + V_a$$

and

$$W = W_s + W_w$$

Symbol	Parameter	Definition / Formula	Typical Range
e	**Void ratio**	$e = \frac{V_v}{V_s}$	0.3 – 1.5
n	**Porosity**	$n = \frac{V_v}{V} = \frac{e}{1+e}$	0.25 – 0.60
S_r	**Degree of saturation**	$S_r = \frac{V_w}{V_v} = \frac{w\rho_s}{e\rho_w}$	0 – 1 (or 0–100 %)
ρ_d	**Dry density**	$\rho_d = \frac{\rho}{1+w}$	1.2 – 2.0 Mg/m^3
γ	**Unit weight**	$\gamma = \rho g$	17 – 22 kN/m^3
γ'	**Effective (submerged) unit weight**	$\gamma' = \gamma_{sat} - \gamma_w$	8 – 11 kN/m^3

⚠️ **Tip:** Knowing any **three** independent parameters (e.g., ρ_d, w, ρ_s) allows all others to be computed.

2.2.2 States of Soil

Condition	Description	Typical Indicator
Dry	All voids filled with air ($S_r = 0$)	$\rho = \rho_d$
Partly saturated	Voids contain both air and water	$\rho_d < \rho < \rho_{sat}$
Fully saturated	All voids filled with water ($S_r = 1$)	$\gamma = \gamma_{sat}$

2.2.3 Index Properties and Their Significance

Index Property	Symbol	Determination (Standard)	Engineering Relevance
Natural water content	w	Oven-drying (BS EN ISO 17892-1)	Indicates current condition vs OMC
Liquid limit	w_L	Casagrande / Cone penetrometer (BS EN ISO 17892-12)	Upper plastic limit of soil behaviour
Plastic limit	w_P	Thread rolling (BS EN ISO 17892-12)	Lower boundary of plastic range
Plasticity index	$I_P = w_L - w_P$	—	Measures clay plasticity and compressibility
Shrinkage limit	w_S	Linear shrinkage test	Assesses potential for volume change
Specific gravity (solids)	G_s	Pycnometer method (BS EN ISO 17892-3)	Used in all density/void calculations

2.2.4 Typical Index Property Ranges

Soil Type	Liquid Limit (%)	Plasticity Index (%)	Shrinkage Limit (%)
Clean sand / gravel	–	Non-plastic	–
Silty sand / sandy silt	25–35	< 10	–
Low-plasticity clay (CL)	35–50	10–20	8–12
Medium-plasticity clay (CI)	50–70	20–35	8–10
High-plasticity clay (CH)	> 70	> 35	5–8
Organic soil / peat	> 100	Variable	Low (< 5)

2.2.5 Worked Example – Determine Void Ratio and Saturation

Given:

$$\rho_s = 2.70\ Mg/m^3,\ \rho_d = 1.60\ Mg/m^3,\ w = 15$$

Find: void ratio (e) and degree of saturation (S_r).

$$e = \frac{\rho_s}{\rho_d} - 1 = \frac{2.70}{1.60} - 1 = 0.69$$

$$S_r = \frac{w\rho_s}{e\rho_w} = \frac{0.15 \times 2.70}{0.69 \times 1.00} = 0.59 = 59\%$$

Phicon Note:

• Maintain consistency between **lab-reported** densities and **design-used** parameters — always check if results are in *wet* or *dry* units.
• Many calculation errors arise from mixing γ, ρ, and ρ_d— record units explicitly (kN/m^3 or Mg/m^3).
• Store phase-relationship formulas in digital templates for auto-validation of test certificates and compaction records.

2.3 Particle Size and Grading Curves

Particle size distribution (PSD) describes the proportion of different grain sizes within a soil.

It is one of the most fundamental classification tools in geotechnical engineering — indicating **drainage, compaction behaviour, strength, and frost susceptibility**.

2.3.1 Standard Methods of Determination

Method	Test Standard	Typical Range	Purpose / Output
Sieve analysis	BS EN ISO 17892-4	Coarse soils: > 63 µm (sand, gravel)	Percent passing by mass vs sieve size
Hydrometer / Sedimentation test	BS EN ISO 17892-4 (Annex D)	Fine soils: < 63 µm (silt, clay)	Particle size distribution from Stokes' Law
Laser diffraction	ISO 13320 (supplementary)	< 500 µm (research, QC)	Rapid PSD for fine soils or suspensions

Sieve and hydrometer data are combined to form a continuous grading curve covering sand–silt–clay fractions.

2.3.2 Key Grain-Size Definitions

Fraction	Size Range (mm)	Typical Description
Gravel	63 – 2.0	Coarse / fine gravel
Sand	2.0 – 0.063	Coarse / medium / fine sand
Silt	0.063 – 0.002	Non-plastic fine soil
Clay	< 0.002	Plastic fine soil

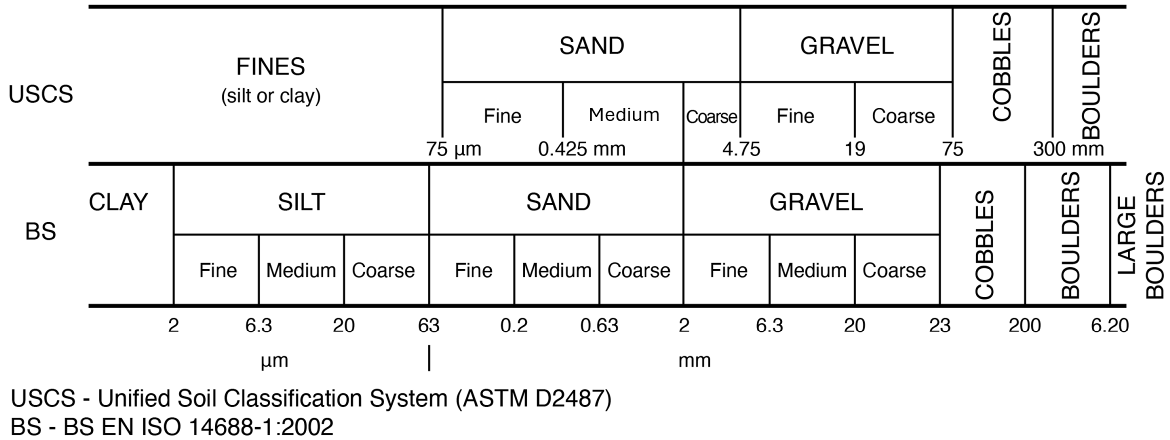


Figure 6: Particle size distribution classification (reproduced from Barnes 2010)

2.3.3 Characteristic Particle Sizes (D-values)

Symbol	Definition	Purpose / Use
D_{10}	Particle diameter at 10 % passing	Effective size — controls permeability
D_{30}	Particle diameter at 30 % passing	Used for curvature coefficient
D_{60}	Particle diameter at 60 % passing	Used for uniformity coefficient

All are obtained directly from the PSD curve plotted on log scale (sieve size on x-axis, % passing on y-axis).

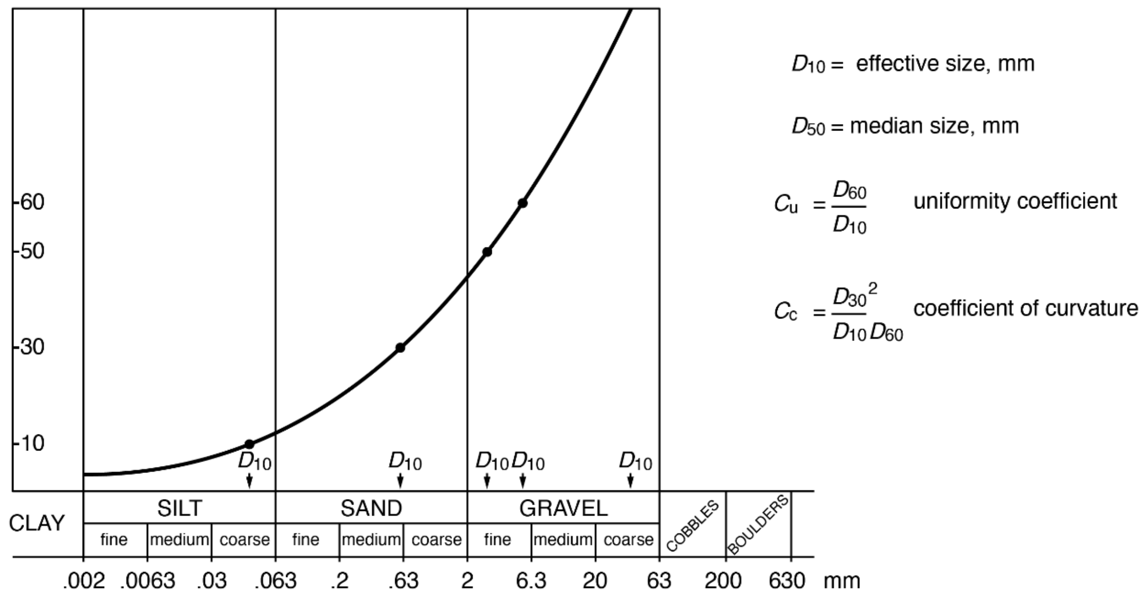


Figure 7: Particle size distribution parameters (reproduced from Barnes 2010)

2.3.4 Gradation Coefficients

Parameter	Formula	Indicates	Typical Range
Uniformity Coefficient	$C_u = \frac{D_{60}}{D_{10}}$	Spread of particle sizes	< 2 (uniform), > 6 (well-graded)
Coefficient of Curvature	$C_c = \frac{(D_{30})^2}{D_{60} \cdot D_{10}}$	Shape / smoothness of curve	1–3 (well-graded)

Interpretation:

• *Well-graded* = wide range of sizes, good compaction, lower permeability.
• *Uniform* = single size, poor compaction, higher permeability.
• *Gap-graded* = missing intermediate sizes; may segregate under vibration.

2.3.5 Example Classification by Grading

Soil Description	C_u	C_C	Grading Type
Clean uniform sand	1.8	1.0	Uniform
Mixed sand & gravel	8.0	2.0	Well-graded
Filter sand	2.5	0.9	Narrow range
Gap-graded fill	15.0	0.4	Gap-graded

2.3.6 Worked Example – Determine Grading Parameters

Given PSD data:

$$D_{10} = 0.20\ mm,\ D_{30} = 0.45\ mm,\ D_{60} = 1.20\ mm$$

$$C_u = \frac{D_{60}}{D_{10}} = \frac{1.20}{0.20} = 6.0$$
$$C_c = \frac{(D_{30})^2}{D_{60} \cdot D_{10}} = \frac{(0.45)^2}{1.20 \times 0.20} = 0.84$$

✓ Interpretation:

$C_u = 6.0$ (well-graded) but $C_c = 0.84 < 1.0$→ slightly gap-graded sand.

2.3.7 Practical Engineering Implications

Characteristic	Effect of Particle Size Distribution
Permeability	Increases with larger D_{10} and uniform grading
Shear strength (ϕ')	Higher for dense, well-graded sands
Compressibility	Lower for well-graded soils
Compaction	Easier with a wide size range (better particle interlock)
Filter design	Relies on D_{15}–D_{85} relationships between soil and filter
Frost susceptibility	High for silty soils with $D_{10} \approx 0.02$ mm

Phicon Note:

• Always request both **raw sieve data** and **log-scale PSD plots** in laboratory reports — visual trends often reveal segregation or contamination.
• Record D_{10}, D_{30}, D_{60} values in your GDR for traceability; they underpin hydraulic and compaction calculations.
• For site QA, verify that imported fills fall within **target grading envelopes** from the project specification or BS EN 13285 (Unbound Mixtures).

2.4 Plasticity and Soil Classification (BS EN ISO 14688-1/2; USCS)

The **plasticity** of a fine-grained soil describes its ability to change shape without cracking or volume loss.

It is primarily controlled by **clay mineral type**, **water content**, and **exchangeable cations**.Laboratory determination of Atterberg limits provides an index of soil behaviour under changing moisture conditions.

2.4.1 Atterberg Limits – Definitions

Limit	Symbol	Definition	Typical Range (Clays)
Liquid limit	w_L	Water content at which soil changes from plastic to liquid state	30–120 %
Plastic limit	w_P	Water content at which soil changes from semi-solid to plastic state	15–60 %
Plasticity index	$I_P = w_L – w_P$	Range of water content over which soil is plastic	10–60 %
Shrinkage limit	w_S	Water content below which no further volume change occurs on drying	5–15 %

Tests per BS EN ISO 17892-12 (cone penetrometer / thread rolling).

2.4.2 Interpretation of Plasticity

Plasticity Index (I_p)	Soil Consistency	Indicative Clay Type	Compressibility
0–10	Slightly plastic	Kaolinite / silt	Low
10–20	Low	Illite / mixed	Low–moderate
20–35	Medium	Illite	Moderate
35–50	High	Montmorillonite / smectite	High
> 50	Very high	Expansive clay	Very high

2.4.3 Plasticity Chart (Casagrande)

- **A-line:** $I_P = 0.73(w_L – 20)$
- **U-line:** $I_P = 0.9(w_L – 8)$

Soils plotted **above the A-line** are **clays (C)**; below are **silts (M)**.
Soils lying near the A-line are **intermediate (CL-ML)**.

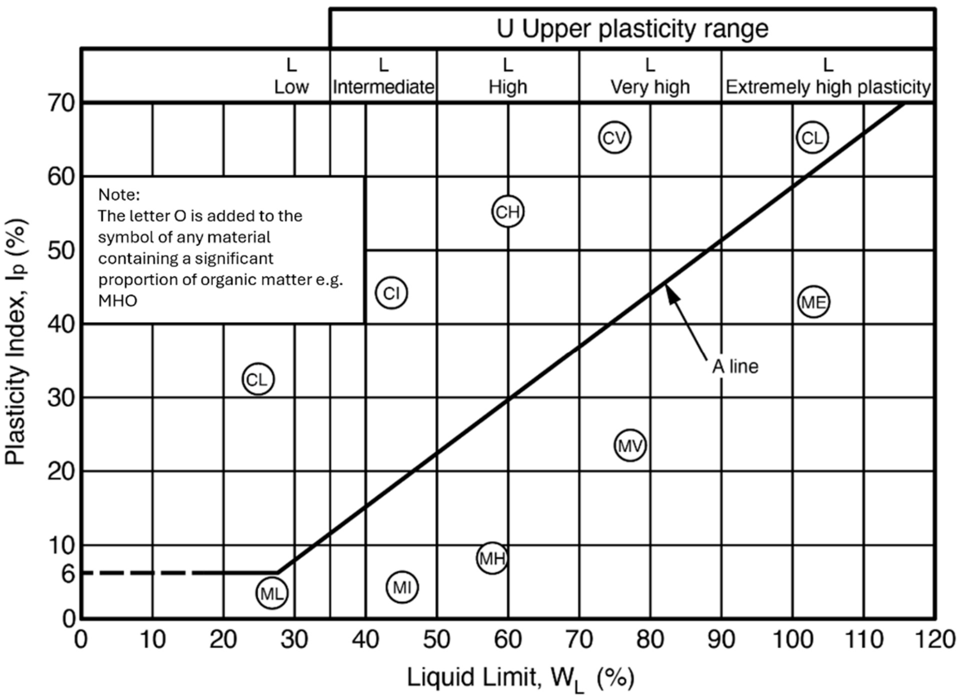


Figure 8: Plasticity chart showing the A line (BS EN ISO 14688-2)

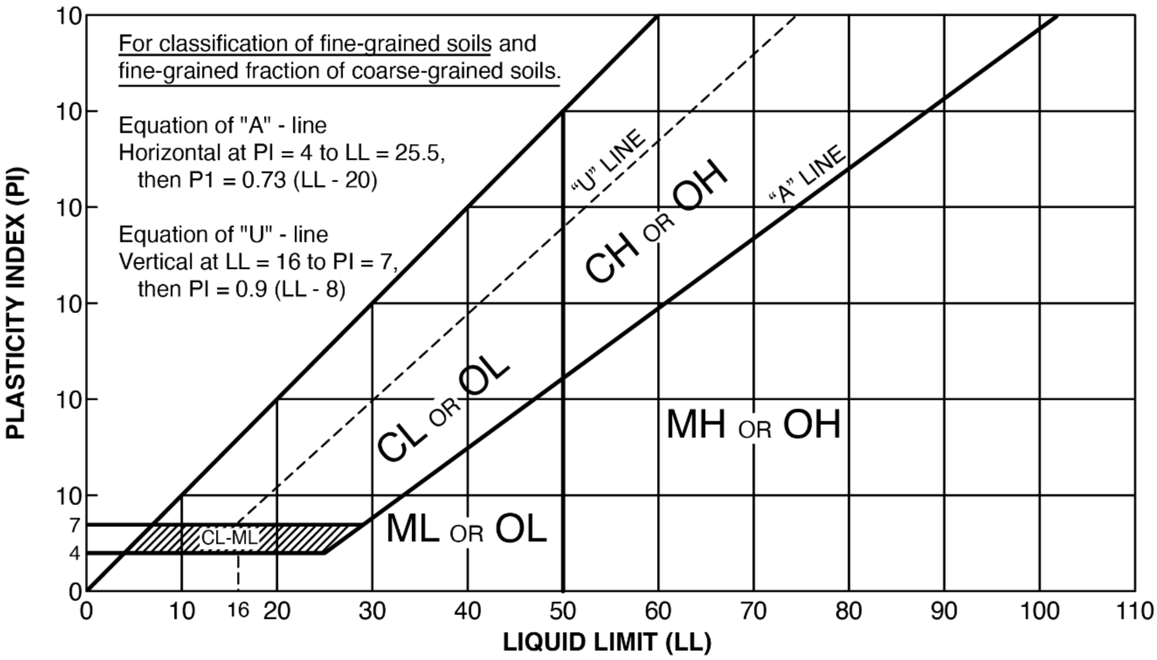


Figure 9: Plasticity chart showing the A line (USCS, ASTM D2487)

USCS: Criteria for Assigning Group Symbols and Names using Laboratory Tests

Classification (For particles smaller than 75 mm and based on estimated weights)			Group Symbol	Group Name [B]	Laboratory Classification		
					Percent finer than 0.075mm	Other Criteria	
Coarse grained soils (More than 50% of the material retained on No. 200 sieve (0.075 mm)	**Gravels** (More than 50%of coarse fraction retained on No. 4 sieve (4.75 mm)	Clean gravels	GW	Well graded gravels, sandy gravels, sand gravel mixture, little or no fines.[D]	< 5 [E]	$C_u \geq 4$ and $1 \leq C_z \leq 3$ [C]	
			GP	Poorly graded gravels, sandy gravels, Sand gravel mixture, little or no fines.[D]		$C_u < 4$ and/or $1 > C_z > 3$ [C]	
		Gravel with fines	GM	Silty gravels, silty sandy gravels.[D, F, G]	> 12 [E]	Ip< 4 or the limit values below 'A' line of plasticity chart	For 4> Ip > 7 and limit values above 'A' line, dual symbol required*
			GC	Clayey gravels, silty clayey gravels. .[D, F, G]		Ip >7 and the limit values above 'A' line of Plasticity Chart	
	Sands (over 50% of coarse fraction smaller than 4.75 mm)	Clean Sands	SW	Well graded sand, gravelly sand, little or no fines.[H]	< 5 [E]	$C_u \geq 6$ and $1 \leq C_z \leq 3$ [C]	
			SP	Poorly graded sands, gravelly sand, little or no fines.[H]		$C_u < 6$ and/or $1 > C_z > 3$ [C]	
		Sands with fines	SM	Silty sand, poorly graded sand silt mixtures.[F, G, H]	> 12 [E]	Ip < 4 or the limit values below 'A' line of Plasticity chart	For 4 > Ip >7 and limit values above A-line, dual symbols required.
			SC	Clayey sand, sand clay mixtures.[F, G, H]		Ip >7 and the limit values above 'A' line of plasticity chart	
Fine grained soils (Over 50% of the material smaller than 0.075 mm)	**Silts & Clays** $w_L < 50$	Inorganic	ML	Silt of low to medium compressibility, very fine sands, rock flour, silt with sand.[K, L, M]	Limit values on or below 'A' line of plasticity chart & Ip <4		
			CL	Clays of low to medium plasticity, gravelly clay, sandy clay, silty clay, lean clay.[K, L, M]	Limit values above 'A' line of plasticity chart and/or Ip > 4		
		Organic	OL	Organic clay[K, L, M, N] and Organic silt[K, L, M, O] of low to medium plasticity	$\frac{\text{Liquid limit (oven dried)}}{\text{Liquid limit (undried)}} < 0.75$		
	Silts & Clays $w_L \geq 50$	Inorganic	MH	Silt of high plasticity, micaceous fine sandy or silty soil, elastic silt.[K, L, M]	Limit values on or below 'A' line of plasticity chart		
			CH	High plastic clay, fat clay.[K, L, M]	Limit values above 'A' line of plasticity chart		
		Organic	OH	Organic clay of high plasticity.[K, L, M, P]	$\frac{\text{Liquid limit (oven dried)}}{\text{Liquid limit (undried)}} < 0.75$		
Soils of high organic origin			PT	Peat and highly organic soils.[K, L, M, Q]	Identified by colour, odour, fibrous texture and spongy characteristics.		

NOTES:
A Based on the material passing the 3-in. (75-mm) sieve *B* If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name. *C* $C_u = D_{60}/D_{10}$, $C_Z = (D_{30})^2 / (D_{10} \times D_{60})$ *D* If soil contains ≥ 15 % sand, add "with sand" to group name. *E* Gravels with 5 to 12 % fines require dual symbols: GW-GM well-graded gravel with silt GW-GC well-graded gravel with clay GP-GM poorly graded gravel with silt GP-GC poorly graded gravel with clay *F* If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM. *G* If fines are organic, add "with organic fines" to group name. *H* If soil contains ≥ 15 % gravel, add "with gravel" to group name. *I* Sands with 5 to 12 % fines require dual symbols: SW-SM well-graded sand with silt SW-SC well-graded sand with clay SP-SM poorly graded sand with silt SP-SC poorly graded sand with clay. *J* If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay. *K* If soil contains 15 to 29 % plus No. 200, add "with sand" or "with gravel," whichever is predominant. *L* If soil contains ≥30 % plus No. 200, predominantly sand, add "sand " to group name. *M* If soil contains ≥ 30 % plus No. 200, predominantly gravel, add "gravelly" to group name. *N* PI ≥ 4 and plots on or above "A" line. *O* PI < 4 or plots below" A" line. *P* PI plots on or above "A" line. *Q* PI plots below "A" line. If desired, the percentages of gravel, sand, and fines may be stated in terms indicating a range of percentages, as follows: Trace – Particles are present but estimated to be less than 5 % Few – 5 to 10 % Little – 15 to 25 % Some – 30 to 45 % Mostly – 50 to 100 %

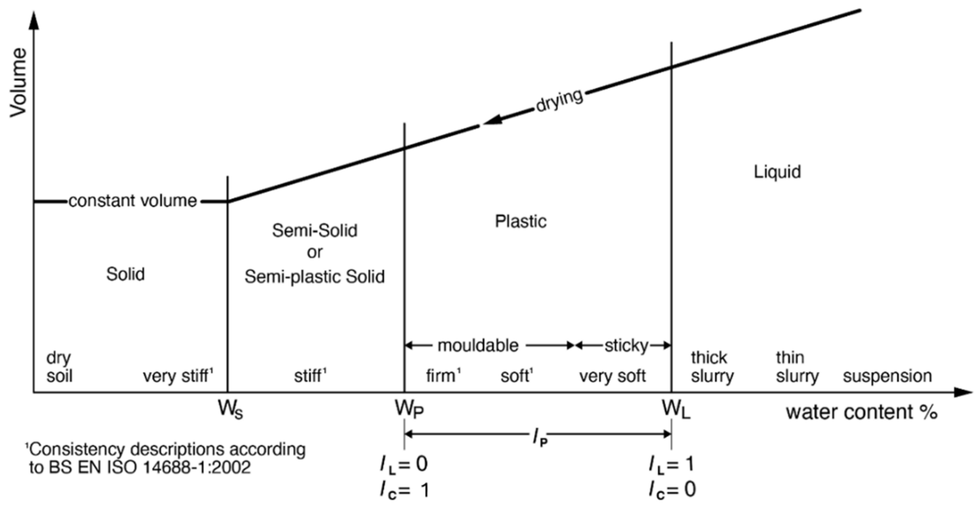


Figure 10: Relationship between volume and water content in terms of Atterberg limits (reproduced from Barnes 2010)

2.4.4 BS EN ISO 14688-1/2 Classification (Simplified)

Group	Designation	Criteria	Typical Description
Coarse soils	G, S	≥ 35 % retained on 63 μm sieve	Gravel or sand; add suffix *w* (well-graded) or *u* (uniform)
Fine soils	M, C	≥ 35 % passing 63 μm sieve	Silt (M) or Clay (C) determined from plasticity
Organic soils	O	Dark, fibrous, odorous, high LOI	Peat (PT) or organic clay/silt (OH, OL)

Descriptive format: [grading prefix] + [main soil type] + [modifiers]
→ e.g. *Slightly gravelly CLAY (CL) or Well-graded SAND with silt (SW-SM)*

2.4.5 Unified Soil Classification System (USCS) – Quick Reference

Group Symbol	Typical Description	Plasticity / Grading Criteria
GW / GP	Well- or poorly-graded gravel	< 5 % fines
SW / SP	Well- or poorly-graded sand	< 5 % fines
GW-SM / GP-SM	Well- or poorly-graded gravel	< 5 % non-plastic fines < 12%
SW-SM / SP-SM	Well- or poorly-graded sand	< 5 % non-plastic fines < 12%
GW-SC / GP-SC	Well- or poorly-graded gravel	< 5 % plastic fines < 12%
SW-SC / SP-SC	Well- or poorly-graded sand	< 5 % plastic fines < 12%
GM / SM	Silty gravel / silty sand	< 50 % non-plastic fines
GC / SC	Clayey gravel / clayey sand	< 50 % plastic fines
ML	Low-plasticity silt	Below A-line, $I_p < 10$
CL	Low-plasticity clay	Above A-line, $I_p < 20$
CI / CH	Intermediate / high-plasticity clay	Above A-line, $I_p > 20$
MH	Elastic silt or micaceous silt	Below A-line, $I_p > 10$
OL / OH / PT	Organic or peat soils	Identified by colour / odour / loss-on-ignition

2.4.6 Worked Example – Soil Classification from Atterberg Limits

Given:

Liquid limit $w_L = 58$ Plastic limit $w_P = 29$

$$I_P = w_L - w_P = 58 - 29 = 29$$

On Casagrande chart:

A-line value at $w_L = 58$: $I_{P,A} = 0.73(58 - 20) = 27.7$

→ Point lies *slightly above A-line* → **clay (C)**

$w_L = 58$→ medium-to-high plasticity → **CI–CH range**

✓ **Classification:** Medium-high plasticity CLAY (CI)

✓ **Description (BS EN 14688):** Slightly silty CLAY, medium plasticity

2.4.7 Practical Engineering Implications

Property	Low Plasticity	High Plasticity
Permeability	High	Low
Compressibility	Low–medium	High
Shrink–swell potential	Negligible	Significant
Shear strength (ϕ')	High	Low
Compaction sensitivity	Low	High
Construction issues	Easy handling	Requires moisture control

Phicon Note:

• Always quote both **BS EN ISO 14688** and **USCS** classifications in reports — most international clients and contractors recognise USCS.

• For plastic clays, record w_L, w_P, I_P, and **liquidity index** $I_L = (w - w_P)/(w_L - w_P)$in design spreadsheets for consistency.

• Plot all fine-grained soils on a digital plasticity chart to visually flag anomalous results (e.g. data points below A-line but labelled "clay").

2.5 Compaction Characteristics and Optimum Moisture Content

Compaction increases soil density by reducing air voids through mechanical effort.

It is one of the most important processes in earthworks, embankment construction, and pavement subgrades — improving **strength, stiffness, and durability**. Laboratory compaction testing determines the **Optimum Moisture Content (OMC)** and **Maximum Dry Density (MDD)** for use in specification and field control.

2.5.1 Objectives of Compaction

- Achieve adequate shear strength and bearing capacity.
- Minimise compressibility and post-construction settlement.
- Reduce permeability and improve frost resistance.
- Ensure consistent performance under traffic and cyclic loads.

2.5.2 Laboratory Compaction Tests (BS EN 13286-2)

Test Type	Energy Level	Typical Use	Equivalent ASTM
Standard Proctor	600 kN·m/m^3	Fine-grained fills, general embankments	ASTM D698
Modified Proctor	2700 kN·m/m^3	Heavily trafficked or high-strength layers	ASTM D1557
Vibrating hammer	1200–2000 kN·m/m^3	Coarse granular fills	BS EN 13286-4

2.5.3 Typical Procedure (Standard Proctor)

1. Air-dry and sieve soil (≤ 20 mm fraction).
2. Compact in a 1 L mould in **three layers**, each given **27 blows** of a 2.5 kg rammer falling 300 mm.
3. Measure **wet density** and **moisture content** for each test point.
4. Plot dry density vs moisture content curve.
5. Identify the **peak** → **MDD** and corresponding **OMC**.

2.5.4 Interpretation of the Compaction Curve

Parameter	Symbol / Definition	Typical Range	Remarks
Optimum Moisture Content	OMC = moisture content at maximum dry density	8–18 % (granular), 15–25 % (clays)	Best water–air balance for compaction
Maximum Dry Density	MDD = peak dry density (Mg/m^3)	1.6–2.2	Increases with compactive effort
Zero Air Void (ZAV) Line	Theoretical upper density at 100 % saturation	—	Indicates achievable limit

Air-voids lines (5–15 %)	Secondary guide on chart	—	Used for QA interpretation

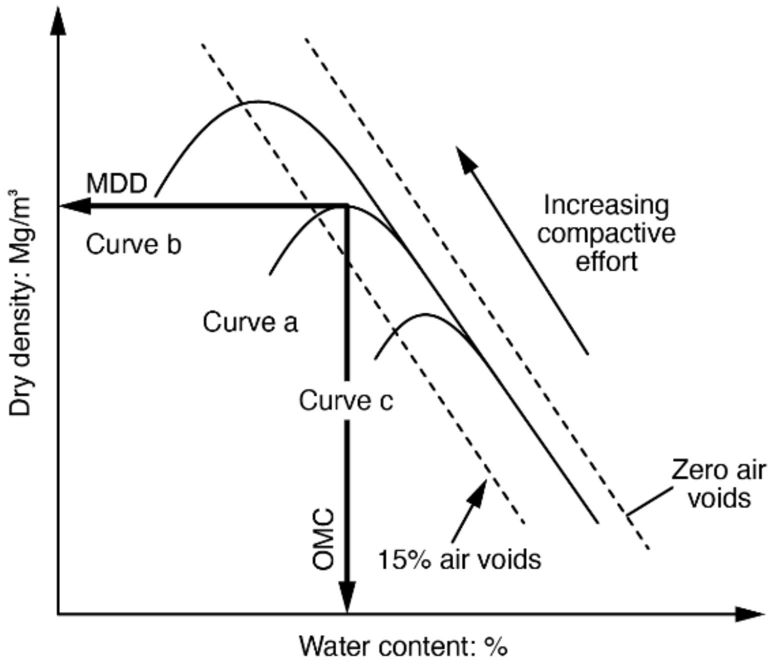


Figure 11: Effect of compactive effort on compaction properties (reproduced from Nowak & Gilbert 2015)

2.5.5 Factors Affecting Compaction

Influence	Effect on MDD / OMC
Soil type	Coarse soils → high MDD, low OMC; fine soils → low MDD, high OMC
Compactive effort	Increased energy raises MDD and reduces OMC
Grading / particle shape	Well-graded soils compact better
Moisture content	Too dry → lack of lubrication; too wet → pore pressure, low density
Soil temperature / additives	Lime or cement stabilisation alters OMC–MDD relationship

Compaction curve shape provides insight into soil type — flat for sandy soils, sharp for cohesive clays.

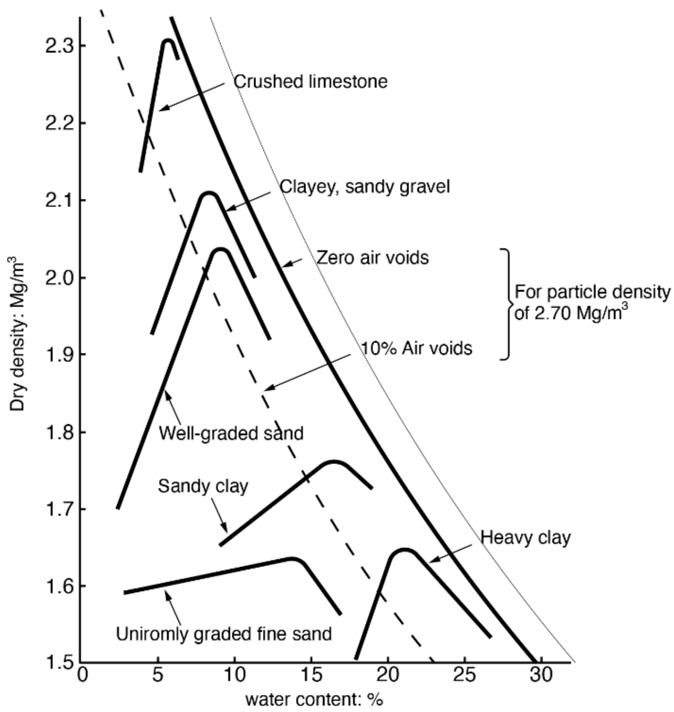


Figure 12: Typical compaction curves for different soil types (reproduced from Parsons 1992)

2.5.6 Field Compaction Control

Test	Standard	Output	Typical Acceptance
Nuclear Density Gauge (NDG)	BS EN 17892-2 / ASTM D6938	In-situ dry density & moisture	≥ 95 % of MDD
Sand Replacement Test	BS 1924 / BS EN ISO 17892-11	Dry density	≥ 90–95 % of MDD
Light / Heavy Weight Deflectometer (LWD/HWD)	DIN 18134 / ASTM E2583	EV_2 modulus	Target stiffness (EV_2 ≥ 50–120 MPa)
Plate Bearing Test (PLT)	BS 1377-9	EV_1, EV_2, EV_2/EV_1	≤ 2.0 ratio = good compaction

2.5.7 Worked Example – Determine OMC and MDD

Moisture content (%)	***6***	***8***	***10***	***12***	***14***	***16***
Dry density (Mg/m^3)	*1.72*	*1.83*	*1.90*	*1.93*	*1.90*	*1.84*

✓ OMC = 12 %, MDD = 1.93 Mg/m^3

Field specification: 95 % compaction
→ Required dry density = 0.95 × 1.93 = **1.83 Mg/m^3**

2.5.8 Typical Ranges by Soil Type

Soil Type	OMC (%)	MDD (Mg/m^3)
Gravel	5–8	2.1–2.3
Sand	8–12	1.9–2.1
Silt	10–18	1.7–1.9
Clay	15–25	1.6–1.8
Chalk (engineered fill)	12–18	1.6–1.9

Phicon Note:

• Always compare **laboratory MDD/OMC** against **field-compacted densities** at placement — results differ with grading, weather, and plant.
• For cohesive fills, specify compaction **within ±2 % of OMC**.
• For QA dashboards, link **EV$_2$**, **CBR**, and **density ratio** automatically using calibration data from site correlation trials.

3 Groundwater and Effective Stress

Groundwater plays a controlling role in nearly all geotechnical problems.

Its pressure, movement, and variation with time affect **stability, strength, consolidation, and durability**.

Understanding the interaction between **pore water pressure**, **permeability**, and **effective stress** is essential for both design and construction management.

3.1 Pore Pressure, Permeability and Flow

3.1.1 Pore Water Pressure (u)

Pore water pressure is the pressure exerted by water within the soil voids.

$$u = \gamma_w \cdot h_w$$

where:

- u= pore water pressure (kPa)
- γ_w= unit weight of water (≈ 9.81 kN/m^3)
- h_w= height of water above the point of interest (m)

Condition	Typical Range (kPa)	Remarks
Below water table	10 to 200	Hydrostatic increase with depth
Above water table (capillary zone)	0 to –80	Negative (suction) pressure
Artesian conditions	> hydrostatic	Uplift pressure risk

Measurement: standpipes piezometers, vibrating-wire cells piezometers (BS EN ISO 22476-11).

3.1.2 Permeability and Darcy's Law

Water flow through soil is governed by **Darcy's Law** (1856):

$$q = k\,i\,A$$

where:

- q= discharge (m^3/s)
- k= coefficient of permeability (m/s)
- i= hydraulic gradient = Δh / L
- A= flow area (m^2)

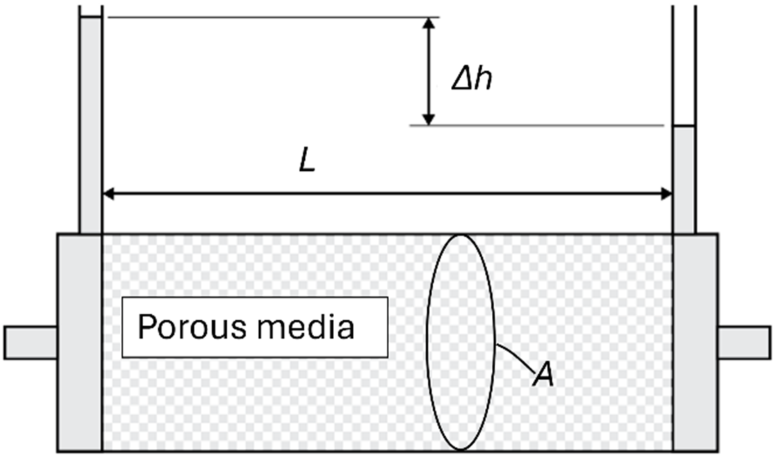


Figure 13: Calculation of flow through a porous media

3.1.2.1 Typical Permeability Ranges

Soil Type	Permeability (k, m/s)	Remarks
Gravel	$10^{-2} – 10^{-3}$	Freely draining
Sand	$10^{-3} – 10^{-5}$	Permeable
Silt	$10^{-5} – 10^{-7}$	Semi-permeable
Clay	$10^{-9} – 10^{-11}$	Practically impervious
Peat	$10^{-4} – 10^{-5}$	High but variable

Laboratory tests: constant-head (coarse soils), falling-head (fine soils) — *BS EN ISO 17892-11*.

Field tests: piezocone dissipation, Lefranc, Lugeon, or pumping tests.

3.1.3 Hydraulic Gradient and Flow Conditions

Type of Flow	Hydraulic Gradient (i)	Typical Application
Laminar (Darcy)	i < 0.1	Fine soils, low velocities
Transitional	0.1–1.0	Medium sands
Turbulent	> 1.0	Coarse gravels, filters

Flow remains **Darcy-type (linear)** until velocity becomes high enough for inertial effects to dominate.

3.1.4 Seepage Forces

Downward or upward flow through soil generates additional body forces:

$$F_s = i \cdot \gamma_w$$

Acting on each unit weight of soil mass, these reduce or increase effective stress depending on direction.

Direction of Flow	Effect
Downward	Increases effective stress (stabilising)
Upward	Reduces effective stress (destabilising)

Upward seepage under excavations can cause hydraulic heave or piping (see § 3.3).

3.1.5 Worked Example – Calculate Seepage Flow

Given:

- Hydraulic conductivity $k = 2 \times 10^{-5}\ m/s$
- Hydraulic head difference $\Delta h = 3$ m
- Flow path length L = 15 m
- Flow area A = 12 m^2

$$i = \frac{\Delta h}{L} = \frac{3}{15} = 0.20$$
$$q = k \cdot i \cdot A = 2 \times 10^{-5} \times 0.20 \times 12 = 4.8 \times 10^{-5}\ m^3/s$$

Result: Seepage discharge = 4.8×10^{-5} m^3/s (≈ 0.17 m^3/h)

3.1.6 Typical Field Indicators of Groundwater Behaviour

Observation	Possible Interpretation	Hydraulic Parameter/Condition	Implication for Excavation/Design
Rapid inflow into trial pit	High permeability soil (e.g., gravel, clean sand)	High Hydraulic Conductivity *k*	Significant dewatering effort required (high pump capacity) to maintain a dry excavation. Potential for 'running sand' conditions
Delayed recovery in standpipe	Low permeability soil (e.g., clay, silt, silty sand)	Low Hydraulic Conductivity *k*	Standpipe response time is slow; measurements may not reflect true *k*. vibrating wire piezometer may be needed for accurate pore pressure. Dewatering may be slow but sustained pumping rates are low
Rising pore pressure in excavation base	Confined aquifer beneath the excavation.	Artesian or semi-Artesian conditions. Piezometric surface is above the excavation base	High risk of base heave due to upward hydraulic gradient. Requires relief wells or deep well dewatering to lower the pressure head below the base
Seasonal variation in water level	Shallow unconfined water table (or perennial perched water)	Recharge/Discharge effects on Unconfined Aquifer	Need to design foundations based on the highest anticipated water table elevation, typically measured after a wet season. Corrosion and buoyancy risks must be assessed for basements
Continuous drawdown during works	Pumping effects in a high *k* aquifer	Induced flow/Drainage	Indicates the dewatering system (or a nearby existing drain/well) is effectively drawing down the water table. The stability of nearby structures may need

Observation	Possible Interpretation	Hydraulic Parameter/Condition	Implication for Excavation/Design
			monitoring for settlement
Water encountered above main water table	Discontinuous low-permeability layer trapping water.	Perched Groundwater Table.	This water body may drain quickly or be localized. It must be assessed if it impacts short-term excavation stability or if it will be encountered by footing level

Phicon Note:

• Always obtain **representative in-situ permeability values** — laboratory specimens can underestimate k due to sample disturbance.
• Include groundwater regime in the **Geotechnical Design Report (GDR)** with at least one **piezometric profile** and **anticipated seasonal fluctuation band**.
• For temporary works, verify **uplift and heave checks** against the **worst-case groundwater level**, not the observed one.

3.2 Seepage and Flow Nets (Graphical / Numerical)

Seepage through and beneath earth structures (embankments, cut-offs, retaining walls) can be evaluated using **flow nets** — graphical representations of **equipotential** and **flow lines** satisfying Laplace's equation for steady-state flow.

They are a core visual tool for estimating **pore pressure distribution**, **seepage quantity**, and **hydraulic gradients**.

3.2.1 Fundamentals of Flow Nets

Under steady, two-dimensional flow:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

The flow field is represented by two orthogonal families of curves:

Curve Type	Symbol	Definition	Physical Meaning
Flow lines	ψ	Path along which a particle of water moves	Tangent to velocity vector
Equipotential lines	ϕ	Lines of equal total head	Perpendicular to flow lines

In a correctly drawn net, the "flow elements" (squares) are approximately equal in aspect ratio.

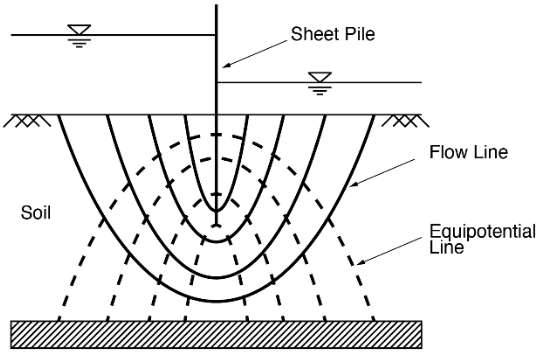


Figure 14: Flow net for seepage calculation beneath a sheet pile (reproduced from Das 2011)

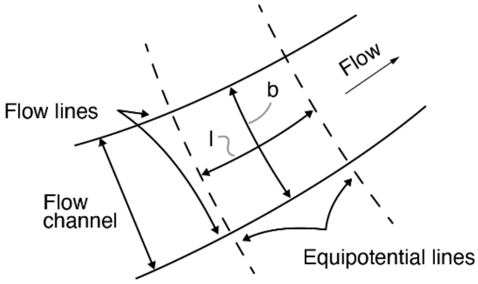


Figure 15: Square section of a flow net (reproduced from Smiths 2014)

3.2.2 Basic Boundary Conditions

Boundary Type	Condition	Examples
Impermeable boundary	Flow line (no normal flow)	Base of clay layer, concrete cutoff
Constant head boundary	Equipotential line	Reservoir or river water surface
Phreatic surface	Variable head, atmospheric pressure	Seepage face, embankment slope
Interface between soils	Head and flux continuity	Layered deposits beneath wall

3.2.3 Components of a Flow Net

Parameter	Symbol	Definition / Meaning
Number of flow channels	N_f	Count of distinct flow tubes between equipotentials
Number of potential drops	N_d	Number of equipotential intervals from upstream to downstream
Total head difference	h	$h = h_1 - h_2$
Hydraulic gradient per drop	$i = h/(N_d \cdot L)$	Determines seepage force

Flow per unit width	$q = k \cdot h \cdot (N_f / N_d)$	Total seepage discharge

3.2.4 Example Applications

Structure	Typical Flow Net Purpose
Earth dam / embankment	Locate phreatic line and estimate seepage loss
Sheet-pile wall	Assess uplift pressure beneath excavation
Cut-off trench	Confirm efficiency of barrier
Retaining wall	Evaluate seepage-induced lateral forces
Groundwater lowering system	Predict drawdown pattern

3.2.5 Worked Example – Seepage under Sheet-Pile Wall

Given:

- $k = 1 \times 10^{-5}\ m/s$
- Head difference across wall $h = 5\ m$
- $N_f = 3,\ N_d = 12$

$$q = k \cdot h \cdot \frac{N_f}{N_d} = 1 \times 10^{-5} \times 5 \times \frac{3}{12} = 1.25 \times 10^{-5}\ m^3/s \cdot per \cdot m$$

✓ **Result:** Seepage per metre length = **1.25×10^{-5} m^3/s** (≈ 0.045 m^3/h).

If width of excavation = 10 m → total flow ≈ **0.45 m^3/h**.

3.2.6 Numerical Flow Modelling

When geometry is complex or stratified, analytical flow nets become unreliable.
Use **finite-element or finite-difference models** (e.g. PLAXIS, SEEP/W, MIDAS) to compute heads and velocities.

Method	Advantages	Limitations
Analytical / hand-drawn	Rapid visual insight; transparent assumptions	Approximate, steady-state only
Numerical (FEM)	Handles anisotropy, 3D effects, transient flow	Requires calibration and data on k, boundaries

3.2.7 Common Errors in Flow-Net Construction

- Distorted element shapes → violates Laplace's condition.
- Incorrect boundary assumption (impermeable vs seepage face).
- Ignoring variable permeability layers.
- Assuming steady-state when drawdown is transient.

Phicon Note:

• For critical works (dams, deep excavations), **compare hand-sketched and numerical flow nets** — consistency between both increases confidence.
• Record N_f, N_d, k, h, q values in GDRs for traceability.
• Always check the **exit gradient** (i_e) near the downstream face against **critical gradient** ($i_c = (G_s - 1)/(1 + e)$) to prevent **piping or heave** (see § 3.3).

3.3 Critical Hydraulic Conditions (Heave, Piping)

When upward seepage forces approach the submerged weight of the soil, **effective stress is reduced**, potentially to zero.

This can trigger **hydraulic heave**, **boiling**, or **piping** — common failure modes beneath excavations and retaining structures.

3.3.1 Mechanisms

Failure Mode	Description	Typical Situation
Hydraulic heave	Upward seepage pressure exceeds submerged weight; soil lifts or fluidises	Base of deep excavations, sheet-pile cofferdams
Boiling / quick condition	Effective stress = 0; particles suspended in flow	Sandy layers under artesian conditions
Piping	Progressive removal of soil along flow path (channel formation)	Beneath dams, cut-off walls, river embankments

All modes arise when pore pressure gradients exceed the soil's self-weight resistance.

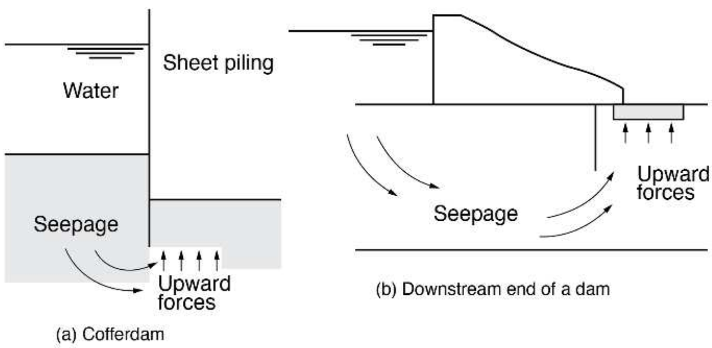


Figure 16: Examples where piping can occur (reproduced from Smiths 2014)

3.3.2 Critical Hydraulic Gradient

The **critical hydraulic gradient** i_c defines the onset of instability:

$$i_c = \frac{\gamma'_{soil}}{\gamma_w} = \frac{(G_s - 1)}{1 + e}$$

where:

- G_s= specific gravity of solids (≈ 2.65)
- e= void ratio (–)
- γ_w= unit weight of water (9.81 kN/m^3)
- Typical Values

Soil Type	e	i_c	Remarks
Dense sand	0.5	1.1	Safe under moderate gradients
Medium sand	0.7	0.95	Marginal safety below walls
Loose sand	1.0	0.80	High risk of boiling
Silty sand	0.9	0.85	Sensitive to uplift
Clay (impervious)	—	> 2.0	Rarely critical

3.3.3 Factor of Safety Against Heave

$$F_h = \frac{i_c}{i_{actual}} = \frac{i_c}{\Delta h/L}$$

where:

- Δh= head difference (m)
- L= seepage path length (m)

Design is considered safe if $F_h \geq 1.5$ (temporary) or ≥ 2.0 (permanent works).

3.3.4 Uplift Pressure and Effective Stress Check

At base of excavation:

$$\sigma'_v = \gamma' \cdot z - u$$
$$u = \gamma_w \cdot h_w$$

Stability requires **positive effective stress** $\sigma'_v > 0$.
If $\sigma'_v \to 0$, soil becomes “quick” and may fluidise.

3.3.5 Worked Example – Check Against Hydraulic Heave

Given:

- Excavation depth = 6.0 m
- Water level outside wall = ground surface
- Water level inside excavation = 1.0 m below surface
- Soil: medium sand ($e = 0.7$, $G_s = 2.65$)
- Base thickness below wall tip $L = 4.0\ m$

$$\Delta h = 6.0 - 1.0 = 5.0\,m$$
$$i_{actual} = \frac{\Delta h}{L} = \frac{5.0}{4.0} = 1.25$$
$$i_c = \frac{(2.65 - 1)}{1 + 0.7} = 0.97$$
$$F_h = \frac{i_c}{i_{actual}} = \frac{0.97}{1.25} = 0.78 < 1.0$$

✗ **Unsafe** — uplift / heave likely.
✓ **Mitigation:**

- Increase embedment depth (L)
- Lower external water level (well points)
- Provide drainage blanket or pressure relief wells

3.3.6 Piping and Cut-off Design Considerations

Measure	Purpose / Effect
Cut-off wall / toe extension	Increases seepage path length (↑ L → ↓ i_{actual})
Upstream filter / drainage blanket	Dissipates excess pore pressure
Relief wells / sub-drains	Reduces Δh and uplift
Weighted blanket / berm	Increases stabilising weight
Impermeable membrane / grout curtain	Reduces permeability (↓ k)

3.3.7 Typical Eurocode Design Checks

Limit State	Symbol	Verification	Clause Reference

Hydraulic heave (UPL/HYD)	UPL / HYD	$i_{actual} < i_c/\gamma_{UPL}$	EN 1997-1 §10.2
Uplift of base slabs	UPL	$V_d < R_d$(uplift vs weight + anchors)	EN 1997-1 §2.4.7
Piping / erosion	HYD	Empirical check using Bligh or Lane method	EN 1997-1 §10.2(3)
Safety against global	GEO	Slope / wall stability including pore pressure	EN 1997-1 §11

Phicon Note:

• Always document groundwater conditions at time of design — *temporary drawdowns can reverse*, producing higher uplift pressures later.

• Avoid "paper safety" by assuming higher embedment depth without checking constructability.

• For **sheet-pile excavations**, cross-check both **heave (i)** and **uplift (σ')** conditions under DA1 (C1 & C2) to ensure consistency with EC7 UPL/HYD limit states.

3.4 Effective Stress Principle and Total Stress Relationships

The effective stress principle is the foundation of soil mechanics.

It defines how total stress, pore pressure, and effective stress interact to control shear strength, stiffness, and volume change.

First formalised by **Terzaghi (1936)**, it remains fundamental to all geotechnical analysis and design under **Eurocode 7**.

3.4.1 The Effective Stress Equation

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

where:

- σ'= effective stress (kPa)
- σ= total stress (from self-weight + external loads)
- u= pore water pressure (kPa)

Only the **effective stress** contributes to shear strength and deformation behaviour of the soil skeleton.

3.4.2 Total Stress Distribution with Depth

Component	Equation	Typical Range / Note
Total vertical stress	$\sigma_v = \gamma_{bulk} \cdot z$	Increases linearly with depth
Pore water pressure	$u = \gamma_w \cdot z$(below water table)	Hydrostatic; linear with depth
Effective vertical stress	$\sigma_v' = \sigma_v - u$	Governs strength and settlement
Submerged (buoyant) unit weight	$\gamma' = \gamma_{sat} - \gamma_w$	Usually 8–11 kN/m^3

3.4.3 Stress Zones in Soil Profile

Zone	Condition	Pore Pressure (u)	Stress Relationship
Above water table	Unsaturated	Negative (suction)	$\sigma' = \sigma - u_{neg}$→ effective stress > total
At water table	Transition	0	$\sigma' = \sigma$
Below water table	Saturated	Positive (hydrostatic)	$\sigma' = \sigma - u$

Suction in unsaturated soils increases apparent cohesion (useful for temporary stability of excavations and slopes).

3.4.4 Drained vs Undrained Conditions

Condition	Time Scale	Pore Pressure Response	Analysis Type	Typical Soil / Scenario
Drained	Long-term (months–years)	Dissipated	Effective stress	Sands, long-term stability
Undrained	Short-term (immediate)	Generated, no dissipation	Total stress	Clays under rapid loading
Partially drained	Intermediate	Partial dissipation	Coupled	Intermediate silty soils

⚠️ **EC7 Note:** choose drained or undrained parameters according to design situation, not soil type alone.

3.4.5 Pore Pressure Changes Due to Loading

- **Compression** → increases pore pressure in undrained clays
- **Shear or unloading** → decreases pore pressure (suction)
- **Consolidation** → gradual dissipation of excess pore pressure; effective stress rises

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

3.4.6 Worked Example – Effective Stress Profile Beneath Ground Surface

Given:

- Water table at 2.0 m depth
- $\gamma_{unsat} = 18\ kN/m^3, \gamma_{sat} = 20\ kN/m^3$
- Compute total and effective vertical stresses at 4.0 m depth

Solution:

Depth (m)	Layer	γ (kN/m^3)	Total stress σ (kPa)	Pore pressure u (kPa)	Effective stress σ' (kPa)
0–2	Above WT	18	18×2 = 36	0	36
2–4	Below WT	20	36 + (20×2) = 76	9.81×2 = 19.6	76 – 19.6 = **56.4**

✓ **At 4 m depth:**
σ = 76 kPa, u = 19.6 kPa, σ' = 56.4 kPa

3.4.7 Effective Stress in Design Context

Application	Use of Effective Stress	Design Parameter
Bearing capacity (drained)	Use γ', ϕ', c' in effective stress terms	EC7 §6.5
Slope stability (long-term)	Use effective shear strength parameters	EC7 §11
Retaining walls (active/passive)	Earth pressures from effective unit weights	EC7 §9.5
Settlement analysis	Based on Δσ' distribution	EC7 §6.6
Uplift / heave checks	Balance total vs pore pressure	EC7 §10.2

3.4.8 Stress Path Concept (Overview)

In advanced analysis, effective stress paths (ESPs) track changes in σ' and u during loading.

They provide insight into whether failure occurs under:

- **Drained conditions** (constant u → σ' increases), or
- **Undrained conditions** (constant volume → u increases).

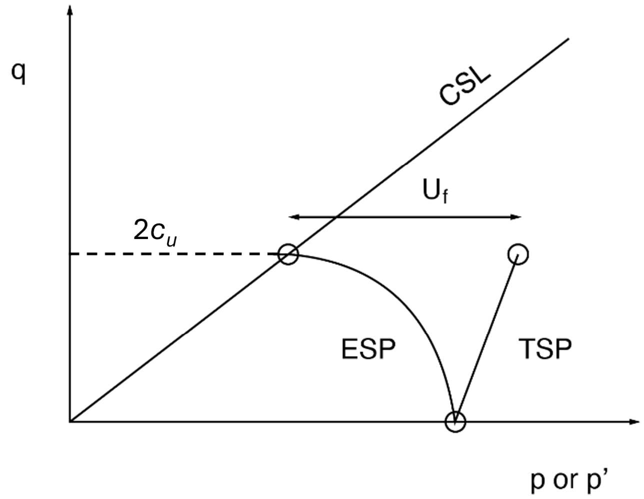


Figure 17: Effective and total stress paths

Phicon Note:

• Always check which **stress condition (total or effective)** a design model uses — mixing parameters leads to major errors.
• Store both **total** and **effective stress profiles** in digital models for each design case; these feed directly into **bearing, settlement, and stability** checks.
• For reporting clarity: present **σ, u, σ'** in one table or chart at key elevations — this allows independent reviewers to validate assumptions quickly.

4 Shear Strength and Critical State Concepts

The **shear strength** of soil is its resistance to sliding or failure along internal planes.

It controls bearing capacity, slope stability, and lateral pressure behaviour — making it one of the most critical parameters in geotechnical design.

Eurocode 7 (§2.4.3 & §6.5) requires that design is based on **characteristic shear strength parameters** (c'_k, φ'_k) or **undrained strength** ($c_{u,k}$), depending on drainage conditions and loading time.

4.1 Mohr–Coulomb Failure Criteria

The Mohr–Coulomb criterion defines the soil's ultimate shear strength as a linear combination of its effective cohesion (c') and its frictional resistance governed by the angle of internal friction (φ').

4.1.1 Basic Equation

$$\tau_f = c' + \sigma' \tan \varphi'$$

where:

- τ_f = shear strength at failure (kPa)
- c'= effective cohesion (kPa)
- σ'= effective normal stress (kPa)
- φ'= effective friction angle (°)

This linear model (Mohr–Coulomb) is used in most design codes and limit equilibrium analyses.

4.1.2 Total vs Effective Stress Strength Parameters

Condition	Equation	Applicable Soil / Situation	Notes
Drained (effective)	$\tau_f = c' + \sigma' \tan \varphi'$	Sands, long-term clays	Pore pressure fully dissipated
Undrained (total)	$\tau_f = c_u$	Short-term clay, rapid loading	$\varphi_u \approx 0°$, constant volume
Partially drained	$\tau_f = c' + (\sigma - u) \tan \varphi'$	Transitional	Requires pore pressure measurement or estimation

4.1.3 Graphical Representation (Mohr's Circle)

At failure, the Mohr's circle is tangent to the **failure envelope** defined by the line $\tau = c' + \sigma' \tan \varphi'$.
Each test point (from direct shear or triaxial test) plots a circle with radius = τ_f.

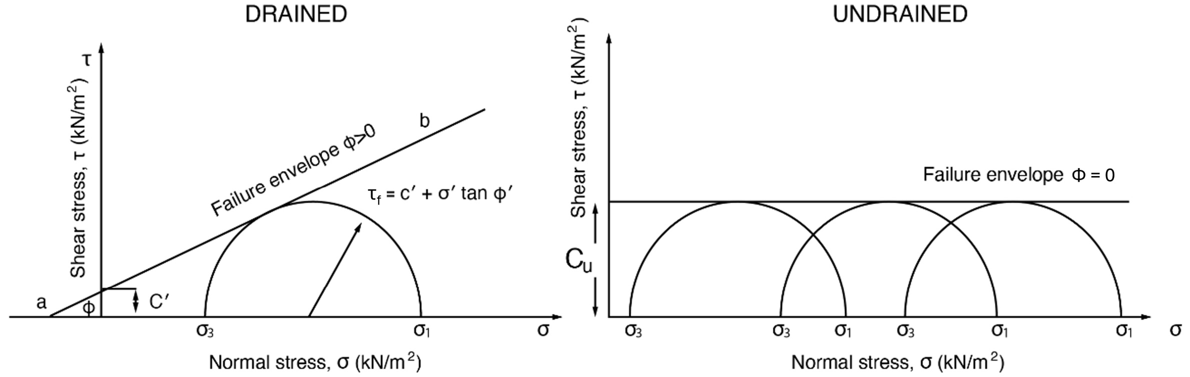


Figure 18: Typical Mohr circles for drained and undrained shear conditions

4.1.4 Typical Shear Strength Parameters

Soil Type	c' (kPa)	ϕ' (°)	c_u (kPa)	Comments
Dense sand	0	35–42	—	High strength, dilatant
Medium sand	0	32–36	—	Common design ϕ' = 34°
Loose sand	0	28–32	—	Sensitive to disturbance
Silty sand	0–5	28–35	—	Moderate ϕ'
Clay (low plasticity)	0–5	22–28	50–100	ϕ'' for long-term checks
Clay (medium)	0–10	20–25	75–150	c' small; strength from c_u
Clay (high plasticity)	0–15	18–22	25–75	Low ϕ'; time-dependent
Glacial till	10–30	26–34	75–200	Often fissured, variable
Rockfill / crushed stone	0	40–45	—	High interlock, dilative

4.1.5 Relationship Between ϕ' and Density (Granular Soils)

Relative Density (D_r)	Typical ϕ' (°)
0–20 % (very loose)	27–30
20–50 % (loose–medium)	30–34
50–80 % (dense)	34–38
80–100 % (very dense)	38–42

In granular soils, ϕ' primarily reflects inter-particle friction and dilatancy.

4.1.6 Drained vs Undrained Envelope Behaviour

Aspect	Drained (ϕ'–c')	Undrained (c_u)
Pore pressure	Dissipated	Generated
Volume change	Occurs	None (ΔV = 0)
Parameter basis	Effective stress	Total stress
Time scale	Long-term	Short-term
Failure envelope	Sloping line	Horizontal line

4.1.7 Worked Example – Direct Shear Test Interpretation

Given test results:

Normal stress (kPa)	*Shear stress at failure (kPa)*
100	*75*
200	*115*
300	*150*

Plot τ vs σ' → best-fit line gives:

$$c' = 30\ kPa, \varphi' = 26°$$

✓ Shear strength equation:

$$\tau_f = 30 + \sigma'\tan\ 26°$$

For σ' = 150 kPa:

$$\tau_f = 30 + 150\ \tan\ 26° = 30 + 73 = 103\ kPa$$

4.1.8 Influence of Drainage and Loading Rate

Condition	Effect on Strength
Rapid loading in clay	Undrained → c_u controls
Long-term consolidation	Drained → ϕ' controls
Cyclic / repeated loading	Reduces effective ϕ' (structure breakdown)
Overconsolidation	Increases c' and ϕ' temporarily
Fissures / soft layers	Reduce mobilised shear strength

Phicon Note:

• Always define **which shear strength** (total or effective) your design uses — and match it to the correct **limit state** (ULS/SLS).
• For clays: adopt **c_u (undrained)** for short-term and **c', ϕ' (drained)** for long-term stability.
• Document test type, drainage condition, and parameter derivation in the **Geotechnical Design Report (GDR)** — EC7 §3.2 requires this traceability.
• In spreadsheets, flag whether "ϕ" and "c" are total or effective — mix-ups are one of the most frequent causes of design error.

4.2 Drained vs Undrained Conditions

The **drainage condition** determines which strength parameters apply — either the drained **effective stress** parameters (c', φ') or the undrained **total stress** shear strength (c_u).

Selecting the wrong one is one of the most common design errors in practice.

Eurocode 7 (§2.4.3) requires that the designer identifies whether pore pressures **dissipate** or **remain** during loading.

4.2.1 Concept Summary

Condition	Definition	Timeframe / Rate	Relevant Parameters
Drained	Pore water can dissipate during loading	Slow / long-term	c', φ'
Undrained	No drainage; water pressure changes occur instantly	Rapid / short-term	c_u
Partially drained	Intermediate; partial pore-pressure dissipation	Medium-term	Combination of c', φ'+ u model

4.2.2 Typical Soil Behaviour

Soil Type	Immediately After Loading	Long-Term Condition
Sand / Gravel	Drained (no excess u)	Drained
Silty Sand / Silt	Partially drained	Drained
Clay (soft–stiff)	Undrained	Drained
Peat / Organic soil	Undrained	Drained but with creep

Clays fail undrained in the short term (excavation, loading) but behave drained in long-term equilibrium.

4.2.3 Time Scale for Drainage

$$t_{50} = \frac{H_{dr}^2}{C_v}$$

where:

- t_{50}= time for 50 % consolidation (s)
- H_{dr}= drainage path (m)
- C_v= coefficient of consolidation (m^2/s)

C_v (m^2/s)	H_{dr} (m)	t_{50} (approx.)	Interpretation
$1×10^{-3}$	1.0	15 min	Sand – fully drained during loading
$1×10^{-7}$	1.0	4 months	Clay – undrained during construction
$1×10^{-9}$	2.0	60 years	Very soft clay / peat – creep governs

4.2.4 Selection of Design Parameters

Scenario	Loading Nature	Design Condition	Parameter Set
Shallow foundation on sand	Slow, long-term	Drained	φ', c'
Shallow foundation on clay	Short-term (immediate load)	Undrained	c_u
Slope stability (temporary cut)	Rapid stress change	Undrained	c_u
Slope stability (long-term)	Creep / consolidation	Drained	φ', c'
Pile design (static)	Depends on soil type	Mixed	β- or α-method
Embankment loading	Variable rate	Time-dependent	Coupled analysis

4.2.5 Relationship Between c_u and Effective Parameters

For normally consolidated clays (approximation):

$$c_u \approx 0.22 \cdot \sigma'_v \cdot \tan\ \varphi'$$

or, more empirically:

$$\varphi' \approx 25°–30°, c_u/\sigma'_v \approx 0.2–0.3$$

For over-consolidated clays, use higher ratios (0.3–0.5) depending on **OCR**.

OCR	Typical c_u/σ'_v
1 (NC)	0.2
2	0.25
4	0.35
8	0.45

4.2.6 Example – Determining Design Condition

Scenario:
A 2 m-wide pad foundation on stiff clay, loaded to 200 kPa within 2 days.
Site data: $C_v = 1 \times 10^{-7}\ m^2/s,\ H_{dr} = 1.5\,m$

$$t_{50} = \frac{1.5^2}{1 \times 10^{-7}} = 2.25 \times 10^7 s = 260\ days$$

✓ **Interpretation:**
Loading time $< t_{50}$→ **undrained condition** → use $c_u$for bearing check.
Long-term settlement → check with φ', c' after consolidation.

4.2.7 Combined Analysis in Practice

Design Stage	Limit State	Condition	Check
Construction (short-term)	ULS (GEO/STR)	Undrained	Stability, heave
Operation (long-term)	SLS	Drained	Settlement, rotation
Decommissioning / temporary cut	ULS	Undrained	Base heave / wall stability

Phicon Note:

• Always justify in the GDR whether analyses are **drained** or **undrained**; reviewers must see that time-scale and permeability have been considered.
• For clays, include both sets of parameters in spreadsheets and toggle with a *drained/undrained switch* to avoid duplication.
• When using numerical software (e.g. PLAXIS), verify consolidation coefficients and loading sequence — *steady-state drained ≠ instantaneous undrained*.

4.3 Laboratory Strength Tests (Triaxial, Shear Box, Vane)

Laboratory shear strength testing provides the core parameters for design:

- **Drained parameters** c', φ' for long-term or granular soils
- **Undrained strength** c_u for short-term or cohesive soils

Each test type simulates stress and drainage conditions under controlled loading.

4.3.1 Overview of Common Strength Tests

Test Type	Standard	Drainage Condition	Primary Outputs	Typical Application
Triaxial compression	BS EN ISO 17892-8	UU / CU / CD	c', φ', c_u, stress–strain, pore pressure	Clay and silt (undrained / drained)
Direct shear (shear box)	BS EN ISO 17892-10	Drained / consolidated	c', φ'	Sand, gravel, stiff clay
Vane shear	BS EN ISO 17892-7	Undrained	c_u	Soft to firm clays
Simple shear (direct / ring)	ASTM D6528 (rare in UK)	Drained / undrained	τ_f, G, u	Offshore / cyclic analysis

4.3.2 Triaxial Compression Test (UU, CU, CD)

Apparatus and Principle

Cylindrical specimen encased in rubber membrane within a pressure cell.

Axial load applied under controlled confining pressure σ_3.

Shear strength deduced from failure deviator stress $\Delta\sigma = \sigma_1 - \sigma_3$.

Type	Condition	Measured Parameters	Use
UU (Unconsolidated Undrained)	No drainage during consolidation or shearing	c_u (total stress)	Short-term clay stability
CU (Consolidated Undrained)	Consolidated under σ_3; undrained during shearing; pore pressure measured	c', φ' (via total → effective conversion)	General use in clays
CD (Consolidated Drained)	Fully drained throughout; slow shear rate	c', φ' (effective stress)	Long-term behaviour, sands

4.3.3 Key Equations (Effective Stress Interpretation)

From CU or CD test:

$$\tau_f = c' + \sigma' \tan \varphi'$$
$$\sigma' = \sigma - u$$

where u= pore pressure at failure.

4.3.4 Typical Failure Envelopes

Material	Envelope Shape	Typical Parameters
Dense sand	Linear or curved	$\varphi' = 38°–42°, c' = 0$
Normally consolidated clay	Linear	$\varphi' = 23°–30°, c' \approx 0$
Over-consolidated clay	Non-linear (curved)	$\varphi' = 25°–35°, c' = 10–30\ kPa$

4.3.5 Direct Shear (Shear Box) Test

To determine the Mohr–Coulomb parameters (c' and φ'), a 60 × 60 mm split box with specimen is placed. A constant vertical normal stress (σ) is applied while a controlled relative horizontal displacement is introduced, forcing the soil to fail along the central shear plane.

Output	Calculation	Use
Shear stress at failure	$\tau = \frac{P}{A}$	Defines strength envelope
Friction angle	$\varphi' = \tan^{-1}\frac{\Delta\tau}{\Delta\sigma'}$	From τ–σ' plot
Cohesion intercept	c'	y-intercept from plot

Shear box gives approximate drained parameters but may not control drainage or normal stress precisely.

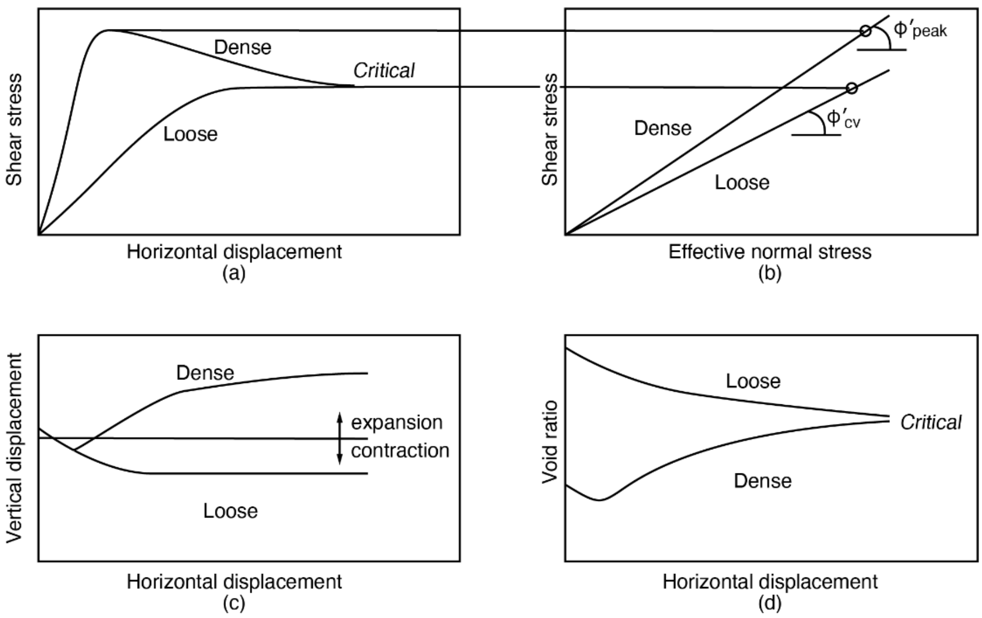


Figure 19: Typical shear box results on loose and dense samples of the same soil (reproduced from Smiths 2014)

4.3.6 Field and Laboratory Vane Shear Tests

Type	Application	Output	Notes
Laboratory (BS EN ISO 17892-7)	Soft to firm clays	$c_u = \dfrac{T}{\pi D^2 H/2 + D^3 \pi/6}$	Small sample, quick estimate
Field (BS 1377-9)	Borehole / excavation	c_u profile with depth	Sensitive to disturbance

4.3.7 Correction factors

Condition	Correction Factor
Remoulded strength	$c_{ur} = \alpha \cdot c_{u,insitu}$with $\alpha \approx 0.7\text{–}0.8$
Sensitivity (St)	$St = \dfrac{c_u}{c_{ur}}$

4.3.8 Typical Ranges of Measured Strength

Material	Test Type	c_u (kPa)	ϕ' (°)	Remarks
Soft clay	UU, vane	15–50	—	Rapid undrained loading
Firm clay	CU	50–100	20–25	Common design range
Stiff clay	CU, CD	100–250	25–30	Moderate fissuring effects
Dense sand	CD	—	38–42	Fully drained
Loose sand	CD	—	28–32	Low density
Silty sand	CU / CD	—	30–35	Partial drainage
Peat / organic	UU / vane	5–20	—	Sensitive, high compressibility

4.3.9 Worked Example – Deriving c' and ϕ' from Triaxial Data

σ_3 (kPa)	σ_1 at failure (kPa)	u (kPa)	σ'_3 (kPa)	σ'_1 (kPa)
100	250	30	70	220
200	370	45	155	325
300	480	60	240	420

Plot (σ'_1, σ'_3) points and fit failure line:

$$\varphi' = 27°, c' = 15\ kPa$$

✓ Strength Equation:

$$\tau_f = 15 + \sigma' \tan 27°$$

4.3.10 Common Errors and Good Practice

Error	Consequence
Using total stress values instead of effective in CD/CU	Overestimation of ϕ'
Ignoring sample disturbance	Reduced c', ϕ' values
Using CU without measuring pore pressure	Misleading “total ϕ” values
Overconsolidated samples not reconsolidated	Excessive strength
Mislabelled drainage condition	Incorrect design approach

Phicon Note:

• Always record **drainage condition (UU, CU, CD)**, **sample quality class**, and **pore pressure measurement** in the GDR.
• For cohesive soils, use **multiple test types** — confirm c_u (UU) against c'–ϕ' (CU/CD) for consistency.
• When importing results into spreadsheets or EC7 tools, use **characteristic values** (5th percentile/95% confidence mean) before applying **partial factors (γ_M)**.
• Maintain full traceability: test ID → sample depth → derived parameter → design input.

4.4 Peak and Residual Strength

Soil strength can decrease after initial failure due to **particle reorientation**, **shear displacement**, or **pore-pressure buildup**.

Eurocode 7 (§2.4.3 & §11.5) distinguishes between **peak**, **mobilised**, and **residual** shear strengths depending on strain level and failure history.

4.4.1 Definitions

Term	Symbol	Meaning / Description	Where used
Peak strength	τ_p	Maximum shear resistance reached before strain softening begins	First-time failure or dense soils
Mobilised strength	τ_m	Portion of peak strength required for equilibrium at current strain	Working stress / SLS checks
Residual strength	τ_r	Minimum post-failure resistance after large displacement	Reactivated slips, fissured or slickensided clays

4.4.2 Behaviour After Peak

- **Dense sands / over-consolidated clays:** exhibit strain softening — sharp drop from $\tau_p \rightarrow \tau_r$.
- Loose sands / normally consolidated clays: often strain harden — no distinct peak.
- **Residual state:** particles aligned parallel to shear plane → reduced ϕ'.

4.4.3 Parameter Relationships

Strength State	Equation	Typical Use
Peak	$\tau_p = c' + \sigma'\tan\,\varphi'_p$	Initial failure, design of new slopes
Residual	$\tau_r = c'_r + \sigma'\tan\,\varphi'_r$	Re-activated slips, long-term equilibrium
Mobilised	$\tau_m = \tau_p/F_s$	SLS or partial-mobilisation analyses

4.4.4 Typical Values of Residual Friction Angle (ϕ'_r)

Material	ϕ'_p (°)	ϕ'_r (°)	ϕ'_r/ϕ'_p Ratio	Comments
Dense sand	38–42	34–38	0.9–1.0	Minor reduction
Medium sand	32–36	30–34	0.9	Dilatancy lost
Over-consolidated clay	25–35	10–20	0.5–0.7	Slickensided, re-sheared zones
Normally consolidated clay	22–28	18–25	0.8–0.9	Small softening
Glacial till	26–34	18–24	0.7	Highly variable
Clay shale / mudstone	20–30	8–16	0.5	Use residual for long-term slopes

Residual ϕ' is best measured in ring-shear or multi-reversal shear tests.

4.4.5 Testing for Residual Strength

Test Type	Standard / Method	Use
Ring-shear test	BS 1377-7 / Bromhead	Determines ϕ'_r directly under large displacement
Reversed shear box	Modified BS 17892-10	Estimates ϕ'_r for over-consolidated clays
Back-analysis	Slope monitoring / failure data	Derives ϕ'_r from known geometry & stability

4.4.6 Worked Example – Residual Strength in Slope Stability

A pre-existing slip in fissured clay is to be reassessed.

Lab tests show $\varphi'_r = 14°$, $c'_r = 0\ kPa$.

Effective stress along slip plane $\sigma' = 80\ kPa$.

$$\tau_r = \sigma' \tan \varphi'_r = 80 \tan 14° = 20\ kPa$$

If peak condition was $\varphi'_p = 28°$:

$$\tau_p = 80 \tan 28° = 42\ kPa$$

✓ Strength reduction ≈ 50 % — design must use ϕ'_r for long-term check.

4.4.7 Use in EC7 Design

Situation	Limit State	Recommended Parameter Set
New slope / first construction	ULS (GEO)	Peak φ'_p, c'_p
Serviceability deformation	SLS	Mobilised $\varphi'_m \approx 0.9\varphi'_p$
Long-term equilibrium / old slip	ULS (GEO)	Residual φ'_r, c'_r
Re-activation or creep	SLS / Observational	Residual + monitoring feedback

4.4.8 Indicative Safety Factors

Case	Typical F_s (static)	Basis
First-time slope (ϕ'_p)	1.3 – 1.5	Peak parameters
Long-term slope (ϕ'_r)	1.2 – 1.3	Residual parameters
Temporary works	1.1 – 1.2	Observational control
Critical infrastructure	≥ 1.5	Conservative design

Phicon Note:

• Always document whether a slope analysis uses **peak** or **residual** ϕ'; they are not interchangeable.
• For re-activated or historical landslides, use **residual parameters** validated by back-analysis or ring-shear testing.
• In EC7 ULS verification, treat residual ϕ' as the **characteristic value** for long-term stability — then apply γ_M = 1.25 per UK NA.
• Maintain separate parameter tables in your GDR: *Peak / Mobilised / Residual*, linked to corresponding limit states for clarity during Cat II/III checks.

4.5 Pore Pressure Parameters (A, B)

When soils are loaded under undrained conditions, **pore pressures** develop because water cannot escape.

The magnitude of this excess pore pressure determines the **effective stress path** and thus the **undrained shear strength**.

Skempton (1954) introduced **pore pressure coefficients A and B** to quantify this response.

4.5.1 Fundamental Relationship

Under undrained loading:

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

where:

- Δu= change in pore pressure (kPa)
- $\Delta\sigma_1, \Delta\sigma_3$= changes in major and minor principal stresses (kPa)
- A, B= empirical pore pressure coefficients

This expression accounts for both **isotropic compression** and **deviatoric loading** contributions to pore pressure change.

4.5.2 Skempton's Coefficients

Coefficient	Definition	Typical Range	Meaning / Use
B	$\frac{\Delta u}{\Delta\sigma_3}$under isotropic loading	0 → 1	Measures degree of saturation
A	$\frac{\Delta u - B\Delta\sigma_3}{B(\Delta\sigma_1 - \Delta\sigma_3)}$	–0.5 → 1	Describes pore pressure due to shear (deviatoric) loading

4.5.3 Interpretation of B Coefficient

B Value	Condition / Soil State
0	Completely dry
0.5	Partially saturated
0.95–1.0	Fully saturated (typical test target)

In CU triaxial tests, B ≥ 0.95 indicates the specimen is fully saturated — a requirement before shearing.

4.5.4 Typical A Values

Soil Type / Condition	A (approx.)	Behavioural Interpretation
Normally consolidated clay	1.0	Pore pressure increases during shear
Over-consolidated clay	0 → –0.5	Negative pore pressure (suction, dilatant)
Loose sand (undrained)	1.0	Contractive, generates u
Dense sand (undrained)	0 → –0.3	Dilative, reduces u
Silt	0.5–0.9	Intermediate behaviour

4.5.5 Example – Estimating Pore Pressure Change

Given:
CU triaxial test on fully saturated clay (B = 1.0, A = 0.8)
Loading increments:

$$\Delta\sigma_1 = 200\ kPa,\ \Delta\sigma_3 = 100\ kPa$$
$$\Delta u = B(\Delta\sigma_3) + A(\Delta\sigma_1 - \Delta\sigma_3)$$
$$\Delta u = 1.0(100) + 0.8(200 - 100) = 100 + 80 = 180\ kPa$$

✓ **Result:**
Pore pressure rise = **180 kPa** → effective stress reduction = 180 kPa.

4.5.6 Influence on Effective Stress Path

During undrained triaxial shearing:

$$\sigma'_1 = \sigma_1 - u, \sigma'_3 = \sigma_3 - u$$

- **Contractive soils (A > 0.5):** u increases, σ' decreases → may lead to failure at low deviator stress.
- **Dilative soils (A < 0):** u decreases, σ' increases → higher apparent strength.

4.5.7 Use in Design and Modelling

Application	Use of A, B Parameters
CU triaxial interpretation	Converts total → effective stresses for ϕ', c'
Undrained analysis (simplified)	Estimates Δu due to loading
Numerical models (PLAXIS, FLAC)	Define pore pressure response in undrained materials
Rapid loading / excavation	Evaluate short-term effective stress reduction

Liquefaction checks (advanced)	Helps model cyclic pore pressure buildup

4.5.8 Typical Design Practice

- For effective stress design (DA1/DA2), pore pressures should be measured or modelled, not assumed.
- **A = 1.0, B = 1.0** is acceptable for quick conservative checks in normally consolidated clays.
- For **dense granular soils**, consider negative A (–0.2 to –0.4) to reflect dilatancy.
- Record whether pore pressure correction has been applied when deriving c', φ' from CU data.

Phicon Note:

• Before interpreting CU triaxial results, always confirm the **B-value** used in saturation — otherwise ϕ' and c' may be invalid.
• In back-analysis or FEM models, use **A–B formulation** when direct pore pressure data are unavailable — it provides realistic undrained response trends.
• Store derived A, B, and u values with stress–strain data for traceable calibration in future parametric studies.

5 Planning and Scope of Site Investigation (EN 1997-2)

A geotechnical design is only as reliable as the ground data behind it.

Eurocode 7 Part 2 (EN 1997-2) establishes the framework for designing, executing, and interpreting site investigations to ensure that **ground conditions are properly characterised**, **geotechnical risks are identified**, and **parameters for design are derived on a sound basis**.

A well-planned investigation progresses from conceptual understanding → targeted exploration → design verification.

5.1 Desk Study, Walkover and Preliminary Data

5.1.1 Purpose

The **desk study** and **site reconnaissance (walkover)** form the first phase of any investigation.

Their aim is to develop a **ground model hypothesis** before intrusive work begins.

This early synthesis often determines **scope, risk level, and cost efficiency** of the entire project.

5.1.2 Desk Study – Information Sources

Category	Examples / Typical Sources	Key Insights Gained
Topographic & cadastral	Ordnance Survey, LiDAR, aerial imagery	Surface gradients, drainage, existing infrastructure
Geological	BGS mapping, borehole records, memoirs	Stratigraphy, structure, faults, superficial deposits
Hydrogeological	BGS/Hydrology datasets, EA groundwater maps	Water table depth, aquifers, recharge zones
Historical	Old OS maps, archive drawings, previous reports	Past land uses, made ground, contamination, mining
Geotechnical	Previous GI reports, local authority records	Typical strengths, densities, variability
Environmental / mining	Coal Authority, DEFRA, Envirocheck	Hazards, mine entries, contamination potential
Utilities & services	Statutory undertakers, asset maps	Constraints for boreholes and trial pits
Planning / design inputs	Drawings, loadings, alignment data	Anticipated foundation and excavation levels

Collate data into a concise **Desk Study Summary Table** for easy reference in the Geotechnical Design Report (GDR).

5.1.3 Walkover Survey

The **walkover** validates desk study findings on site and captures near-surface conditions not evident from records.

Observation	What to Record	Relevance
Ground form	Slopes, hollows, scarps, depressions	Slope stability, fill, drainage
Drainage	Ditches, ponds, seepage zones	Water levels, infiltration behaviour
Surface cover	Vegetation type, hardstanding, contamination	Infiltration, ground disturbance
Structures	Foundations, retaining walls, services	Evidence of distress or differential movement
Access / constraints	Overhead lines, utilities, ecological zones	Planning for GI execution
Evidence of movement	Cracks, bulges, scarps, leaning trees	Existing instability
Groundwater clues	Wet patches, springs, iron oxide staining	Indicative water table level

Photograph key features and annotate on a site plan — these often explain anomalies in test results later.

5.1.4 Deliverables of the Preliminary Phase

Output	Purpose
Conceptual ground model (Stage 1)	Stratified sketch / cross-section showing likely layers, groundwater, and hazards
Preliminary hazard list	Expansive clay, soft ground, contamination, voids, etc.
Preliminary design assumptions	Indicative bearing strata, water table, likely design approach
Proposed investigation plan (draft)	Borehole / pit locations, target depths, key tests
Risk register (initial)	Identifies unknowns and uncertainties guiding the GI scope

5.1.5 Typical Red-Flag Findings

Observation	Possible Risk
Historical landfill or infill	⚠️ Gas, differential settlement
Old industrial site	⚠️ Contamination, buried obstructions
Chalk with sinkholes	⚠️ Solution features, voids
Peaty ground	⚠️ High compressibility, low bearing
Groundwater seepage near excavation	⚠️ Instability, basal heave risk
Colliery / mine entries	⚠️ Collapse, subsidence potential

5.1.6 Integration into the Geotechnical Design Report (GDR)

In accordance with EN 1997-1 §3.3 and CIRIA C574, the desk study and walkover results form **Section 1 – Ground Conditions** of the GDR.

Include:

- Summary of sources reviewed
- Ground model sketch or profile
- Key risks and assumptions
- Recommended next investigation phase

Clarity at this stage prevents mis-scoped investigations and unnecessary cost later.

Phicon Note:

• Treat the desk study as the **first design deliverable**, not just a paperwork step — it sets the hypothesis your GI must test.
• Maintain a **traceable data file** (maps, PDFs, coordinates) under project folders for future verification.
• Always cross-reference findings with current BGS 3D viewer and nearby borehole logs — subtle layer trends often define the design category (GC1–3).

5.2 Investigation Planning and Scope Definition

5.2.1 Purpose

The **planning stage** transforms the conceptual ground model into a **targeted investigation plan**.

Its goal is to obtain sufficient, reliable data for:

- Design parameter derivation
- Verification of assumptions from the desk study
- Assessment of geotechnical risks

Eurocode 7 (§3.2) requires the investigation to be **sufficiently detailed** for each design situation, taking account of **geotechnical category**, **structure type**, and **consequence of failure**.

5.2.2 Geotechnical Categories (GC1–GC3)

Category	Description	Typical Examples	Investigation Expectation
GC1 – Simple	Small, low-risk structures with well-known ground conditions	Single-storey buildings, light foundations	Desk study + minimal trial pits or DCPs
GC2 – Normal	Routine civil works with moderate complexity	Retaining walls, basements, embankments, industrial buildings	Desk study, boreholes, lab tests, standard field tests
GC3 – Complex / high-risk	Unusual design, difficult ground, or high consequences	Dams, tunnels, large quay walls, deep excavations	Multi-phase investigation, advanced in-situ/lab testing, FEM modelling

The **geotechnical category** determines the extent of investigation, level of checking (Cat II/ III), and documentation required in the GDR.

5.2.3 Investigation Stages

Stage	Objective	Typical Activities
Preliminary (Phase 1)	Define ground model and main risks	Desk study, walkover, shallow sampling
Design (Phase 2)	Derive parameters for design verification	Boreholes, in-situ tests, lab testing
Construction (Phase 3)	Confirm actual conditions and quality	Inspection, density/strength testing, instrumentation
Post-construction (Phase 4)	Monitor behaviour / performance	Piezometers, inclinometers, settlement plates

5.2.4 Defining the Scope

A well-scoped investigation should answer:

1. **What needs to be known?** – foundation level, groundwater, parameters, hazards.
2. **Where and how much data?** – location and depth coverage.
3. **What accuracy is needed?** – depends on structure sensitivity.

Aspect	Design Consideration	Typical Practice
Exploration depth	Extend to depth of stress influence (1–2 × foundation width)	Increase in vertical stress caused by the new foundation load normally until Δ_σ < 10 % σ'_v
Investigation spacing	Capture variability across site	1–2 boreholes per structure; grid 25–50 m for earthworks
Borehole vs trial pit	Accessibility, depth, sampling class	Pits ≤ 4 m; boreholes > 3 m or below water table
In-situ testing	Needed where sampling is difficult	CPT, SPT, DMT, PMT
Sampling quality	Depends on soil type	Class 1 (undisturbed) for fine soils; Class 5 for gravels
Groundwater	Install standpipes or piezometers	Minimum one per stratum influencing design

5.2.5 Test Selection by Design Requirement

Design Parameter	Field Tests	Laboratory Tests
Strength (ϕ', c', c_u)	CPT, SPT, vane, PMT	Triaxial, shear box, unconfined
Compressibility (mv, C_c, E)	CPT, PMT, PLT	Oedometer, triaxial (E_{50})
Density / Compaction	NDG, PLT	Moisture, density, PSD
Permeability	Piezometer, pump / falling-head	Permeameter
Chemical / Aggressivity	–	Sulphate, pH, chloride tests
Contamination	–	Suite per BS 10175, CLEA guidance

5.2.6 Typical Minimum Investigation Depths

Structure Type	Minimum Depth Below Formation
Shallow foundation	1–2 × foundation width or to firm stratum
Retaining wall	1.5–2 × height or to base of slip circle
Embankment	1.5 × height or to competent foundation

Pile	2–3 × pile diameter below toe
Basement / excavation	Below invert to low-permeability stratum
Pavement	1.5 m minimum, extend to weak layer if present

5.2.7 Determining the Number of Locations

Site Size / Uniformity	Indicative Minimum Locations
Small uniform site (< 0.5 ha)	3–4 points
Moderate site (0.5–5 ha)	5–10 points
Large or variable site	10 + points at 25–50 m grid spacing
Linear works (roads, pipelines)	1 per 50–100 m or per change in strata

These are indicative only — adjust based on variability and criticality.

5.2.8 Planning Considerations

- Plan boreholes along **structural alignments** and at **critical interfaces** (cut/fill transitions, changes in geology).
- Ensure early **utility clearance** and **ecological screening** before intrusive works.
- Specify **sample quality class** (1–5) and **disturbance rating** for each soil type.
- Include **contingency instructions** for extending depth or adding locations if unexpected strata are encountered.

5.2.9 Documentation

The investigation plan should include:

- Site plan with coordinates, ground levels, and proposed exploratory holes
- Investigation table (ID, method, target depth, sampling/test schedule)
- Health & Safety and environmental controls
- Supervision and logging responsibilities
- Reporting deliverables (factual → interpretative → design data)

⚠️ Use BS 5930's proforma for Exploration Records; attach as appendix to the GDR.

Phicon Note:

• Always define investigation scope in measurable terms — **how many holes, what depths, which tests, why**.

• Reference EN 1997-2 Annex B and BS 5930 for exploration frequency; deviations must be justified in the GDR.
• Capture scope rationale in a **one-page Investigation Planning Sheet** for sign-off by designer and client — this forms part of geotechnical risk management.
• Where sites are large or variable, adopt a **phased investigation strategy**: broad reconnaissance first, then targeted design-stage testing.

5.3 Exploration Methods (Boreholes, Trial Pits, CPT, Geophysics)

The exploration method defines how ground data are obtained and the quality of samples and measurements achievable.

EN 1997-2 (§3.3–3.4) and BS 5930 (Clauses 5–8) classify methods into **intrusive** and **non-intrusive**, depending on whether they disturb the ground.

The selection of method depends on depth, access, expected strata, and design requirements.

5.3.1 Summary of Common Methods

Method	Type	Typical Depth	Key Outputs	Advantages	Limitations
Trial pits	Intrusive	≤ 4–5 m	Visual inspection, sampling, groundwater observation	Low cost, direct inspection	Shallow only, collapse risk below WT
Boreholes (cable percussive / rotary)	Intrusive	5–60 m +	Continuous logs, sampling, in-situ tests	Deep access, Class 1–3 samples	Cost, access, disturbance
CPT / PCPT (cone penetrometer test)	Intrusive (push-in)	≤ 40 m	q_c, f_s, u_2, derived ϕ', c_u, OCR	Fast, continuous data	No sample recovery
SPT (standard penetration test)	In-situ (within borehole)	As borehole	N-value → ϕ', c_u correlation	Simple, cheap	High scatter, poor control
Pressuremeter (PMT / Ménard /OYO/ self-boring)	In-situ	1–30 m	E_m, limit pressure, K_0	Stiffness and limit load directly	Specialist, costly
Dilatometer (DMT)	In-situ (push-in)	≤ 25 m	E_d, K_0, OCR	Intermediate stiffness and ϕ'	Sensitive to operator
Geophysics (seismic, resistivity, GPR)	Non-intrusive	Variable	Layer thickness, anomalies, stiffness (V_s)	Rapid coverage, no disturbance	Needs calibration

Dynamic probing (DPSH, DPH, DPL)	Intrusive	≤ 10–15 m	Blow count → SPT N_{60} correlation	Quick, portable	Empirical only
Hand auger / window sampler	Intrusive	≤ 5–6 m	Disturbed / small undisturbed samples	Light equipment	Depth and stability limited

5.3.2 Trial Pits

- Ideal for shallow foundations, pavements, and fills.
- Allow visual logging, in-situ density and small shear tests.
- Safe working: support or batter sides > 1.2 m deep; dewater if necessary.
- Record soil profile, groundwater seepage, inclusions, and strata contacts.

⚠️ Include photos and hand-drawn logs — these often provide the most intuitive understanding of site conditions.

5.3.3 Boreholes

Method	Typical Use
Cable percussive	General soils, alluvium, drift; SPT every 1.5 m
Rotary core (open-hole / core-barrel)	Weak rock, glacial till, chalk
Rotary percussive (DTH)	Hard rock, dense gravels
Windowless sampling / dynamic auger	Light access, shallow investigations

Key outputs:

- Continuous soil/rock description
- Sample recovery and quality (Class 1–5)
- SPT, Vane, PMT testing
- Groundwater level observations (standpipe or piezometer installation)

For soft soils, use low-energy boring methods (shell & auger) to minimise disturbance.

5.3.4 Cone Penetration Test (CPT / PCPT)

CPT involves continuous measurement of:

- Cone resistance (q_c)
- Sleeve friction (f_s)
- Pore pressure (u_2) (if piezocone)

Derived Parameters	Typical Correlations
Drained ϕ' (sand)	$\varphi' = f(q_c/\sigma'_v)$
Undrained c_u (clay)	$c_u = (q_t - \sigma_v)/N_k, N_k \approx 15–20$
OCR (over-consolidation ratio)	Empirical from u_2/q_c ratio
Soil behaviour type	Ic index (Robertson chart)

CPT gives the most continuous, repeatable soil strength profile — essential for parametric modelling and risk zoning.

5.3.5 Geophysical Methods

Non-intrusive surveys provide rapid areal coverage and supplement intrusive data.

Technique	Measured Property	Typical Use
Seismic (MASW, refraction, cross-hole)	Shear wave velocity (V_s)	Stiffness profiles, dynamic moduli, V_{s30} for EC8
Electrical resistivity (ERT)	Resistivity	Detect moisture, contamination, voids
Ground-penetrating radar (GPR)	Reflection interface	Pavement layers, shallow utilities
Magnetometry / EM	Magnetic field / conductivity	Locate buried structures or UXO
Microgravity	Density anomalies	Caves, dissolution features

⚠️ Always calibrate geophysics with at least one intrusive reference point.

5.3.6 Integration and Selection Matrix

Design Goal	Preferred Method(s)	Supplementary
Identify bearing stratum	Borehole + SPT	CPT
Estimate ϕ' or c_u	CPT, PMT	Triaxial / Shear Box
Determine stiffness (E)	PMT, DMT	Oedometer / PLT
Locate weak horizons / voids	CPT, Geophysics	Boreholes
Map contamination / fill	Trial pits	Lab testing
Assess liquefaction potential	CPT + seismic	SPT + V_s

5.3.7 Quality Control

- Maintain continuous logs and sample depth control.
- Calibrate CPT cones and geophysical equipment before use.
- Assign **unique ID and coordinates** to every exploratory hole.
- Document sample recovery rate (%), disturbance class, and water strikes.
- Cross-check strata consistency between nearby points.

5.3.8 Minimum Data for Design

Structure Type	Minimum Exploration Requirement (Typical)
Shallow foundations	3 + boreholes or pits to twice footing width
Retaining wall / basement	2 + deep boreholes through potential failure plane
Embankment / slope	Boreholes + CPTs along alignment (spacing ≤ 50 m)
Piles	One deep borehole per 250 m^2 footprint or per pile group
Working platform / pavement	Trial pits + PLTs + CBRs at representative zones

Phicon Note:

• Select methods for **data quality, not convenience** — e.g. use CPTs for parameter derivation, not just description.
• Combine **borehole logs + CPT plots + lab data** to build a robust 3D ground model — this supports later design categories (bearing, stability, settlement).
• Record all tests, IDs, and coordinates in a **GI master register**; this forms the traceable dataset referenced in the Geotechnical Design Report (GDR).

5.4 Sampling Quality and Disturbance Classes

5.4.1 Purpose

The **quality of sampling** directly governs the reliability of laboratory test results and derived design parameters.

Eurocode 7 Part 2 (EN 1997-2 §3.4) and BS EN ISO 22475-1 classify samples by **degree of disturbance**, **representativeness**, and **suitability for specific tests**.

A test is only as valid as the **sample quality** it is based on.

5.4.2 Classification of Samples

Type	Description	Typical Recovery Method	Remarks
Intact / undisturbed	Preserves structure, void ratio, and in-situ water content	Thin-walled tubes, piston samplers, block samples	Needed for strength and compressibility tests
Disturbed	Structure altered but composition representative	Split-spoon, auger, bulk grab	Suitable for classification and index tests
Bulk	Mixed material sample	Hand pit, bucket, auger	For PSD, Atterberg, chemistry
Incremental / continuous	Series of discrete samples	Core barrel, window sampler	Useful for stratigraphic logging
Core	Continuous cylindrical record of strata	Rotary coring	Required in rock and weak rock

5.4.3 EN 1997-2 Sample Quality Classes

Class	Description	Typical Sampling Method	Suitable Tests
1	*Very high quality* — minimal disturbance, natural structure preserved	Piston sampler, block sample	Strength, stiffness, compressibility (triaxial, oedometer)
2	*High quality* — small disturbance, structure largely intact	Thin-walled tube, stationary piston	Shear strength, compressibility, permeability
3	Intermediate quality — some disturbance	Split-spoon, window sampler	Index tests, approximate strength
4	*Disturbed* — soil composition only representative	Auger or bulk grab	Classification, chemical testing
5	Highly disturbed / non-representative	Blown samples, spoil, wash cuttings	Description only (not for testing)

⚠️ Class 1–2 samples are required wherever design relies on measured parameters (e.g. c', ϕ', E, m_v).

5.4.4 Practical Sampling Guidance

Soil Type	Recommended Method	Expected Class	Notes
Soft clay / silt	Piston or thin-walled Shelby tube	1–2	Push rate < 20 mm/s
Firm to stiff clay	100 mm open-tube or stationary piston	2	Trim ends before sealing
Granular soils	Split-spoon (SPT) or bulk grab	3–4	Use in-situ tests for parameters
Gravelly soil	Bulk or disturbed grab	4–5	Sieve analysis only
Peat	Block or piston sample	1	Handle gently, keep saturated
Rock / weak rock	Rotary core (triple tube preferred)	1–2	Aim for high RQD (%)

Soil properties	**Quality classes of soil samples for lab testing**				
Unchanged soil properties	**1**	**2**	**3**	**4**	**5**
Soil type	•	•	•	•	•
Particle size	•	•	•	•	
Water content	•	•	•		
Density, density index, permeability	•	•			
Compressibility, shear strength, stiffness	•				
Properties that can be determined					
Sequence of layers	•	•	•	•	•
Boundaries of strata - broad	•	•	•	•	
Boundaries of strata - fine	•	•			
Atterberg limits, particle density, organic content	•	•	•	•	
Water content	•	•	•		
Density, density index, porosity, permeability	•	•			
Compressibility, shear strength, stiffness	•				
Sampling categories	**A**				
		B			
			C		
				D	
					E

Figure 20: Sample quality classes from BS EN ISO 22475-1:2021 Annex H

5.4.5 Disturbance Effects

Form of Disturbance	Caused By	Influence on Test Result
Excess stress / vibration	Drilling energy, drop hammer	Lowers strength and stiffness
Loss of moisture	Exposure, poor sealing	Alters density and volume change
Air entry	Poor sealing, vacuum	Alters pore pressure and effective stress
Smearing at tube walls	High push rate, dull cutting edge	Reduces permeability and stiffness
Incomplete recovery	Collapse or fall-in	Breaks stratigraphy continuity

Keep disturbance <10 % strain equivalent for triaxial or oedometer samples.

5.4.6 Sample Handling and Preservation

- Label clearly: project, hole ID, depth, date, sampler type, recovery.
- Seal immediately using paraffin wax, foil, cling film, or end caps.
- Mark the top and bottom (undisturbed sample) to preserve the soil's natural orientation.
- Transport upright and cushioned; avoid vibration.
- Store at 4–10 °C if testing is delayed >7 days.
- Record recovery ratio:

$$R = \frac{L_r}{L_p} \times 100\%$$

where L_r= recovered length, L_p= penetration length.
$R \geq 95\%$ indicates good-quality sampling.

5.4.7 Visual Indicators of Disturbance

Indicator	Interpretation
Excess water on sample	Disturbed or remoulded
Glossy / smeared surfaces	Tube friction effects
Fissured or shattered appearance	Desiccated or over-drilled
Slumped soil in tube	Over-driven or vibration damage
Continuous intact structure	Good quality (Class 1–2)

5.4.8 Documentation for the GDR

Include in the Geotechnical Design Report (GDR):

- Summary table of sample types, depths, and quality class

- Sample handling procedure (reference to BS EN ISO 22475-1)
- Any anomalies or observed disturbance
- Correlation between sample class and parameter derivation

This traceability allows reviewers to confirm whether measured parameters are supported by appropriate sample quality.

Phicon Note:

• Always record **sample class and recovery** on each log — it directly controls the credibility of derived design parameters.
• For weak clays and silts, specify **Class 1 sampling** at critical depths — cost is small compared with uncertainty reduction.
• Link every test parameter in your design sheet to the **sample ID and class** — this creates an auditable data chain.
• Never interpret oedometer or triaxial results from Class 3 or poorer samples for design use.

5.5 Groundwater and Hydrogeological Monitoring

Groundwater conditions govern **effective stress**, **seepage**, **uplift**, and **long-term stability**.

Accurate observation and monitoring are essential throughout investigation and construction to confirm design assumptions and manage risk.

5.5.1 Purpose of Groundwater Monitoring

Objective	Design Relevance
Determine groundwater level(s) and seasonal variation	UPL/HYD limit states, bearing checks
Identify artesian or perched conditions	Excavation and dewatering design
Characterise hydrogeological regime	Flow direction, permeability, drainage
Support design models	Input for consolidation and seepage analyses
Detect changes during construction	Observational method and QA verification

5.5.2 Methods of Observation

Method	Description / Principle	Typical Use	Remarks
Open standpipe (observation well)	Slotted PVC pipe in backfilled borehole, head equilibrates	Routine groundwater level measurement	Simple and reliable; equilibrium may take days
Piezometer (Casagrande tip)	Porous tip connected by riser tube to surface	Short-term response, multiple depths	Measures total head; used in layered soils

Vibrating-wire piezometer (VWP)	Electronic sensor measuring pressure	Continuous / automated monitoring	For embankments, basements, long-term data
Pneumatic piezometer	Air pressure counterbalances water head	Fast response, high-pressure ranges	Often used in deep excavations
Standpipe with transducer	Combines traditional tube + datalogger	Temporary real-time data	Ideal for dewatering verification
Seepage observation	Visual logging in trial pits or boreholes	Preliminary estimate	Useful for identifying perched zones

5.5.3 Installation Details

Component	Specification / Good Practice
Filter tip	Sand or porous stone matching adjacent stratum
Sealing layers	Bentonite plugs above and below filter zone to isolate horizon
Backfill	Clean sand around filter tip; bentonite pellets above seal
Headworks	Flush or raised cap with label showing ID and depth
Surveying	Record reduced level (RL) of top of pipe or measuring point to ±10 mm
Depth to water	Measured by dip-meter or datalogger; convert to RL for comparison

⚠️ Always install separate standpipes for each significant stratum (e.g., upper sand, lower gravel) to identify vertical head differences.

5.5.4 Observation Periods and Frequency

Monitoring Phase	Typical Duration / Frequency
Initial equilibration	24–72 hours after drilling
Short-term observation	Weekly for 1–2 months
Long-term monitoring	Monthly or quarterly for 6–12 months
During construction	Continuous (VWPs or transducers)
Post-construction	6–24 months for embankments or basements

⚠️ At least one full annual cycle is ideal to establish maximum and minimum groundwater levels.

5.5.5 Interpretation of Results

Observed Pattern	Likely Interpretation
Constant level across depths	Unconfined aquifer
Head increases with depth	Confined / artesian aquifer
Upper zone only partially saturated	Perched water table
Rising level during rainfall	Recharge response
Falling level near excavation	Drawdown due to pumping
Differential heads between standpipes	Upward or downward seepage gradient

5.5.6 Groundwater Parameters from Field Tests

Test Type	Purpose / Output	Typical Application
Pumping test	Determines hydraulic conductivity (k), transmissivity, storativity	Dewatering design, confined aquifers
Rising / falling head test	Quick estimate of k for single hole	Small projects, low-permeability soils
Slug test	Instantaneous head change, monitor recovery	Rapid checks in standpipes
Piezocone dissipation test (CPTu)	u_2 decay → permeability and OCR	Continuous profiling in soft clays

⚠️ Use at least one test per hydrostratigraphic unit affecting design water pressures.

5.5.7 Reporting and Presentation

Include in the Geotechnical Design Report (GDR):

- Table of monitoring points (ID, coordinates, screened depths, strata)
- Graphs of water level vs. time (and rainfall correlation if available)
- Groundwater contour or flow direction map
- Discussion of regime type (unconfined, confined, perched, artesian)
- Design water level adopted for calculations (characteristic vs. worst-case)

Plot both “observed” and “design” levels — this distinction is essential for transparency in UPL/HYD checks.

5.5.8 Design Application

Design Check	Groundwater Data Required
Bearing capacity (drained)	Depth to WT for γ' calculation
Retaining wall (active/passive)	Pore pressure distribution behind wall
Uplift / heave (UPL limit state)	Maximum groundwater head below structure
Slope stability	Pore pressure profile or Ru ratio
Settlement analysis	Consolidation starting pore pressures
Permeability / drainage	*k* values, flow direction, recharge conditions

Phicon Note:

• Record both **depth to water (m BGL)** and **elevation (m AOD)** — elevations allow direct comparison across the site.
• Always identify **perched** and **confined** systems separately; mixing them leads to errors in uplift and stability checks.
• For excavations or basements, use **real-time VWPs** during construction to confirm pore pressure assumptions in UPL/HYD limit state checks.
• Adopt the **maximum expected water level** (not average) when verifying Eurocode UPL, HYD, and EQU limit states — this is explicitly required in EN 1997-1 §10.2.

5.6 Reporting

A geotechnical report is the formal record linking **investigation data → parameter derivation → design verification**. Prior to these being written, a **basis of design** and occasionally a **baseline report** is written.

EN 1997-1 §3.3 and EN 1997-2 §3.4 require that all results from investigation and testing are documented in a traceable manner and summarised in a **Geotechnical Design Report (GDR)**.

5.6.1 Purpose of Reporting

Stage	Purpose	Key Users
Baseline Report	Define the agreed-upon geotechnical conditions and establish common ground for risk allocation	Contractor, Client, Designer
Factual report	Record all field and lab data exactly as obtained	Designer, reviewer

Interpretative report	Derive parameters, assess variability, update ground model	Design engineer
Geotechnical Design Report (GDR)	Demonstrate compliance with EC7 and design approaches	Client, verifier, regulator

Each level builds upon the previous; factual → interpretative → design verification.

5.6.2 Geotechnical Baseline Report (GBR)

The Geotechnical Baseline Report defines the contractual and interpretative ground conditions that form the *baseline* for design, construction, and risk allocation.

It provides a single, agreed statement of subsurface conditions against which variations, claims, and unforeseen ground events are measured.

A GBR consolidates factual investigation data, design interpretations, and assumptions into a concise, auditable document.

Typical contents include:

Section	Description
Introduction	Purpose, scope, and contractual status of the GBR.
Site Conditions Summary	Geological, hydrogeological, and environmental context.
Baseline Stratigraphy	Defined layer sequence, thickness, and spatial limits derived from investigation data.
Baseline Properties	Characteristic parameters (strength, stiffness, permeability) used for design.
Groundwater Regime	Baseline levels, expected seasonal variation, and pressures.
Construction Assumptions	Anticipated behaviour under excavation, support, dewatering, or piling.
Out-of-Baseline Conditions	Definition of what constitutes a material variation or “differing site condition.”

While not an explicit Eurocode deliverable, the GBR supports the EN 1997 requirement for *“adequate knowledge of the ground”* (§2.1 P) and aligns with BS EN 1997-2 data interpretation principles.

It bridges the **Factual Ground Investigation Report (GIR)** and the **Geotechnical Design Report (GDR)**, ensuring consistency between investigation results and design assumptions.

5.6.3 Basis of Design Report (BoD)

The Basis of Design Report sets out the *technical framework* governing geotechnical and structural design.

It records design philosophy, applicable standards, load definitions, and key assumptions that control subsequent calculations and verifications.

Typical sections include:

Section	Description
Project Scope and Interfaces	Boundaries of the design package, responsibilities, and adjacent works.
Applicable Standards	EN 1997-1/-2 with UK NA, EN 1990–1998, BS 8004, ICE Specifications, CIRIA C-series, etc.
Design Life and Reliability	Target design life, consequence class, and reliability index per EN 1990 Annex B.
Design Approach	Declared EC7 Design Approach (e.g., DA1 – Combinations 1 & 2) and National Annex factors.
Limit States	Definition of all relevant ULS (GEO, STR, EQU, UPL, HYD) and SLS checks.
Design Parameters	Summary of characteristic values and derivation sources (from GBR).
Actions and Load Combinations	Dead, live, environmental, and construction loads per EN 1991.
Verification Methods	Analytical, numerical (FE/LE), or empirical methods to be used.
Monitoring and Observational Method	Criteria for updating design based on field data.
Deliverables and Review Process	Calculation sheets, drawings, check categories (Cat II/III), QA workflow.

The BoD is the controlling reference for all geotechnical design calculations and forms the opening section of the **Geotechnical Design Report (GDR)**.

It satisfies EN 1997-1 §2.1 (1) P, which requires documentation of design assumptions, verification methods, and safety formats.

During construction, it provides traceability for design intent and links directly to QA records and limit-state verification sheets.

5.6.4 Factual Report (FR)

Scope: purely descriptive — no engineering judgement. Collation of all test certificates and borehole logs finalised in a single report.

Prepared by the contractor or supervising engineer upon completion of the field and laboratory works.

Contents (typical):

- Introduction (scope, dates, personnel, methods)
- Location plan and borehole/trial pit schedule
- Exploration logs (BS 5930 format)
- Laboratory test certificates and summary tables
- In-situ test results (SPT, CPT, PMT, DMT, PLT, etc.)
- Groundwater monitoring records
- Chain of custody, calibration and QA certificates

All values must be recorded as **measured**, with units, dates, and sample identifiers.

⚠️ Where a number of factual reports from various sources are combined into one large factual report, this is often referred to as a Ground Investigation Report (GIR) not to be confused with the ground interpretation report. Where a GIR exists on a project the interpretation report can be referred to as the ground investigation interpretation report (GIIR).

5.6.5 Interpretative Report (Ground Interpretation Report, GIR)

Adds engineering interpretation to the factual data to form the **ground model**.

Section	Typical Content
Ground conditions summary	Stratigraphy, thickness, variability
Groundwater regime	Type, seasonal range, hydraulic behaviour
Derived parameters	ϕ', c', c_u, E, m_v, γ, k, etc., with rationale
Hazards and risks	Swell, collapse, contamination, voids, instability
Recommended further work	Supplementary testing or monitoring
Preliminary design inputs	Characteristic values and soil profiles

⚠️ Parameter derivation must be clearly traceable to specific test data and sample quality (EN 1997-2 §3.4.3).

5.6.6 Geotechnical Design Report (GDR)

The GDR is the **formal design record** required by EN 1997-1 §3.3. It summarises the design basis, derived data, calculations, and verification of all geotechnical structures.

Typical GDR structure:

Section	Core Content
1. Introduction	Project details, scope, applicable standards
2. Ground conditions	Summary of geology, hydrogeology, hazards
3. Design situations & limit states	GEO, STR, UPL, EQU, HYD considered
4. Characteristic values	Derived from GIIR, with partial factors
5. Design approach & combinations	DA1 (C1/C2), DA2, DA3 justification
6. Verification results	Bearing, settlement, stability, etc.
7. Monitoring & observational method	Trigger levels, instrumentation
8. Risk register	Residual uncertainties and mitigations
9. Conclusions & recommendations	Summary and design validity

The GDR forms part of the permanent design record and should be updated through design revisions and construction.

5.6.7 Link Between Reports

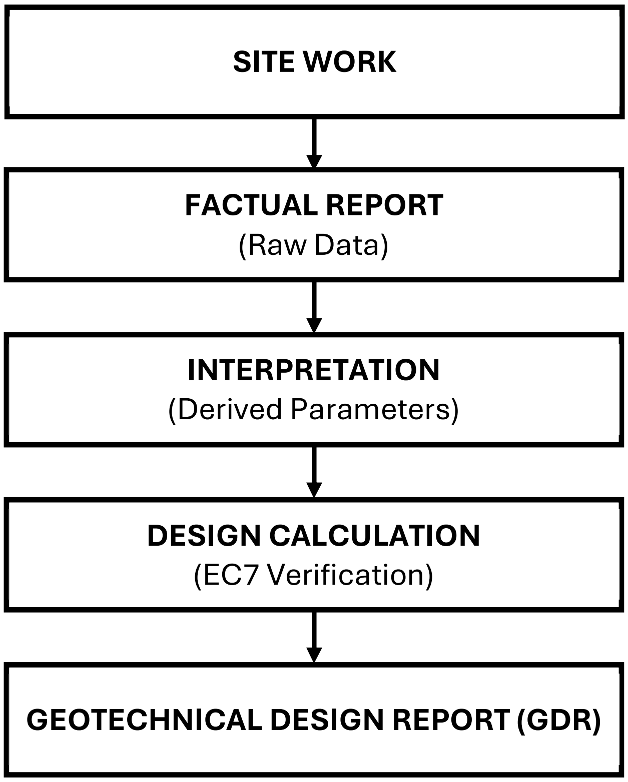


Figure 21: Sequential stages of document production

Each stage must retain **traceability**: sample ID → test result → parameter → design value → verification outcome.

5.6.8 Digital and QA Integration

- Maintain a **master GI register** (hole ID, test ID, sample class, result type).
- Store lab data in machine-readable format (CSV/XLSX) for re-use in dashboards or parameter plots.
- Ensure document QA: revision control, reviewer initials, and date of approval.
- For large projects, integrate factual and interpretative data into GIS or BIM platforms.
- Keep metadata on coordinate system, datum, and file version in all submissions.

5.6.9 Common Reporting Issues

Issue	Effect
Factual and interpretative data combined	Loss of audit trail
Derived parameters not traceable to test IDs	Unverifiable design basis
Missing groundwater records	Invalid UPL/HYD checks
No record of sample class	Unreliable stiffness/strength
Unreferenced design approach	Non-compliance with EC7

5.6.10 Report Integration and Traceability

Geotechnical reporting under Eurocode 7 follows a sequential hierarchy linking **investigation**, **interpretation**, **design**, and **verification**.

Each document has a distinct function but must remain traceable to the previous stage to ensure that design assumptions are evidence-based and auditable.

Document Chain

Report	Primary Function	Key Standard / Reference	Output or Interface
Ground Investigation Report (GIR)	Presents factual field and laboratory data only – no interpretation.	BS EN 1997-2 §4; BS 5930	Logs, test data, and laboratory results.
Geotechnical Baseline Report (GBR)	Establishes contractual and interpretative baseline ground conditions for design and risk allocation.	EN 1997-1 §2.1 P (adequate ground knowledge)	Baseline stratigraphy, parameters, and groundwater regime.
Basis of Design Report (BoD)	Defines the design philosophy, standards, approaches, load combinations, and verification methods.	EN 1997-1 §2.4; EN 1990	Design inputs and safety framework.
Geotechnical Design Report (GDR)	Documents detailed design calculations, limit-state checks, and compliance with EN 1997-1.	EN 1997-1 §2.8	Verified design results and drawings.
Geotechnical Completion / Construction Report (GCR)	Records as-built conditions, QA/QC evidence, and comparison with design assumptions.	EN 1997-1 §4	Verification of design validity during and after construction.

Integration Principles

- Each report shall **reference the preceding document**, maintaining a continuous trace from investigation to construction.
- Any revision to the GBR or BoD during design or construction must trigger a **controlled update** to the GDR and QA documentation.
- Design verification sheets (ULS/SLS) should clearly cite the parameter source (GIR/GBR) and factor set (BoD).
- The full chain of reports forms the **auditable EC7 compliance record**, enabling Category 2 or 3 design checks without data duplication.

Phicon Note:

• Treat the GBR as the *risk boundary* between investigation and construction. It should express the baseline in measurable terms (levels, strengths, groundwater heads) and exclude interpretative optimism. Clear definition of the baseline reduces dispute potential and supports the Observational Method.

• Maintain a single, controlled BoD throughout the design life of the project. Update it only through formal revision when design assumptions change (e.g. revised groundwater level, material parameters, or load cases).

• Treat the **GDR as both a design and legal document** — it evidences compliance with EC7 and your professional duty of care.

• Maintain full **traceability chain**: *sample → test → parameter → design → verification → outcome*.

• Always separate **measured**, **derived**, and **design** values in tables — this clarity simplifies audits and Cat III reviews.

• Version and archive GDRs throughout project phases (concept → detailed → as-built) for robust record management.

• A consistent naming convention (e.g. “PHI-GIR-001”, “PHI-GBR-002”, “PHI-BoD-003”, etc.) ensures transparency and rapid cross-referencing between data, assumptions, and verification outcomes.

6 Field Testing Methods

Field testing forms the link between **site investigation** and **parameter derivation**.

It provides in-situ data on **strength, density, stiffness**, and **layer variability**, often where sampling quality is limited.

EN 1997-2 (§4) and BS EN ISO 22476 series define the principal test methods used in design.

6.1 Standard Penetration Test (SPT)

6.1.1 Principle

The **Standard Penetration Test (SPT)** measures soil resistance to dynamic penetration of a **split-spoon sampler** driven into the ground by a 63.5 kg hammer falling 760 mm.

The test is carried out in boreholes, typically at **1.5 m depth intervals**, or at **changes in strata**.

Measured output:
Number of blows required to drive the sampler 300 mm after an initial 150 mm seating —

$$N = N_{150\text{–}450}$$

This blow count is then **corrected** to standard energy conditions to give N_{60}.

6.1.2 Equipment Summary

Component	Specification	Notes
Hammer	63.5 kg, 760 mm drop	Automatic release preferred
Rods	1 m steel rods	Should be straight and clean
Sampler	Split-spoon, 35 mm ID, 50.8 mm OD	Cutting shoe and drive shoe fitted
Borehole casing	Maintains hole stability	Avoid water pressure effects
Energy calibration	60 % standard efficiency	Record actual system efficiency (η)

In UK practice, automatic trip hammers are standard; cathead systems are discouraged due to variable energy delivery.

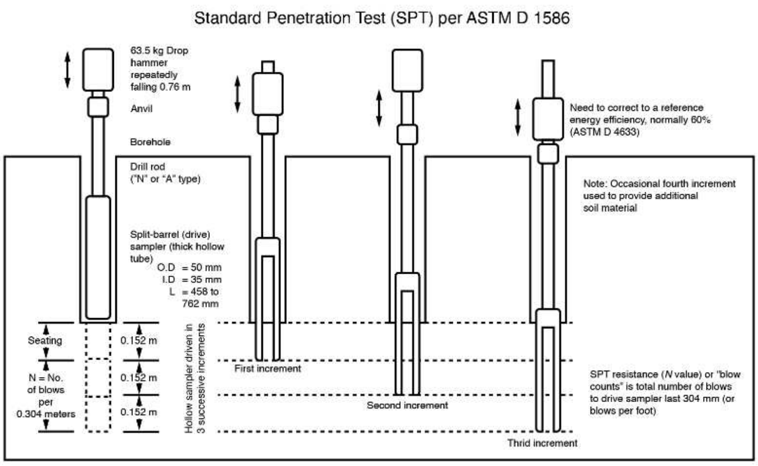
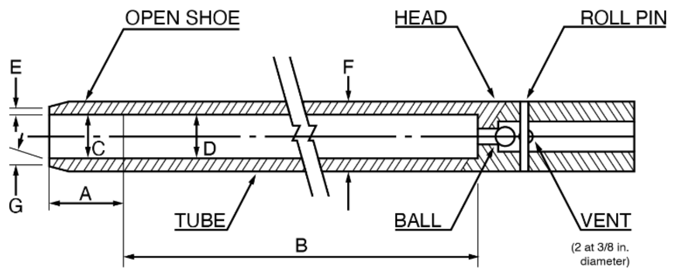


Figure 22: Standard Penetration Test: SPT Sampler Penetration Procedure



A = 1.0 to 2.0 in. (25 to 50 mm)
B = 18.0 to 30.0 in. (0.457 to 0.762 m)
D = 1.50 ± 0.05-0.00 in. (38.1 ± 1.3-0.00 mm)
E = 0.10 ± 0.02 in. (2.54 ± 0.25 mm)
F = 2.00 ± 0.05-0.00 in. (50.8 ± 1.3-0.00 mm)
G = 16.0° to 23.0°

Figure 23: Standard SPT Split Spoon Sampler

6.1.3 Test Procedure (BS EN ISO 22476-3)

1. Seat sampler 150 mm into the base of borehole.
2. Record blows for each 75 mm penetration.
3. Ignore first 150 mm; add last 300 mm = test result N.
4. If > 50 blows for any 75 mm, terminate test (“refusal”).
5. Record groundwater conditions, hammer type, energy ratio.

$$N_{60} = N \times \frac{E_r}{60} \times \frac{C_N \cdot C_B \cdot C_S \cdot C_R}{1}$$

where:

- E_r= measured energy ratio (%)
- C_N= overburden correction
- C_B, C_S, C_R= borehole, sampler, rod corrections (normally 1.0 for UK automatic hammers)

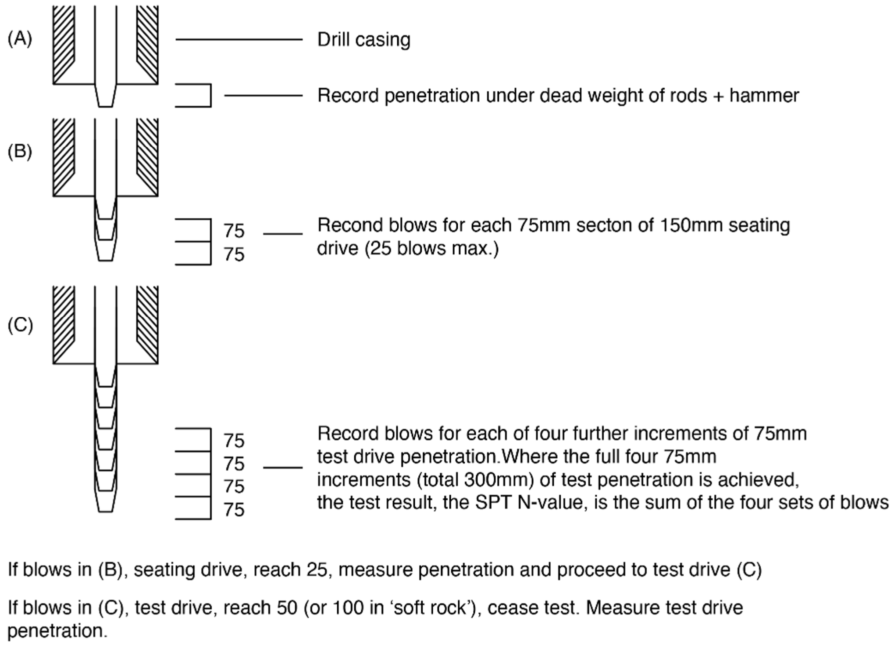


Figure 24: Standard Penetration Test blow count procedure (reproduced from Structural Soils 2025)

6.1.4 Corrections

(a) Overburden Correction (CN)

Accounts for confining pressure effects:

$$C_N = (\frac{P_a}{\sigma'_v})^{0.5}$$

(usually limited to $C_N \leq 1.7$)

where $P_a = 100\ kPa$, σ'_v= effective overburden pressure.

6. (b) Energy Correction (Er)

If energy ratio ≠ 60 %:

$$N_{60} = N \times \frac{E_r}{60}$$

6.1.5 Typical Ranges of N_{60} Values

Soil Type	N_{60}	Interpretation (BS 5930)
Very loose sand	< 4	Very low density
Loose sand	4–10	Low density
Medium-dense sand	10–30	Medium density
Dense sand	30–50	High density

Very dense sand / gravel	> 50	Very high density / refusal
Soft clay	< 4	Very low strength
Firm clay	4–8	Low strength
Stiff clay	8–15	Medium strength
Very stiff clay	> 15	High strength

6.1.6 Correlations with Design Parameters

Parameter	Typical Correlation	Notes
Undrained shear strength (clay)	$c_u = 5N_{60}\ (kPa)$	Rough estimate for cohesive soils
Friction angle (sand)	$\varphi' = 27° + 0.3N_{60}^{0.5}$	Peck, Hanson & Thornburn (1974)
Relative density (Dr)	$D_r = 15(N_{60})^{0.5} - 20$	Approx. for clean sands
Young's modulus (E)	$E = (2–5)N_{60}\ MPa$	Lower range for fine sands
Bearing capacity (q_{ult})	$q_u lt = 15N_{60}\ kPa$(shallow)	Preliminary use only

⚠️ Use correlations only where verified locally and supported by parallel testing (CPT, triaxial, etc.).

6.1.7 Limitations and Good Practice

Potential Issue	Effect / Mitigation
Gravel or cobbles	Refusal before valid penetration — use DPSH or CPT instead
Very soft clays	Excess pore pressure → overestimation of strength
Partially saturated soils	Energy losses → inconsistent N
Water-filled holes	Use casing or bail water before test
Operator error / variable drop	Record equipment and energy ratio

⚠️ Always note test anomalies in the log — SPT results must be interpreted in context.

6.1.8 Reporting Requirements

Each test record should include:

- Borehole ID, depth, and strata description
- Blow counts per 75 mm interval
- Groundwater level and borehole diameter
- Hammer type and energy efficiency
- Corrected N_{60} values
- Interpretation remarks

Include in the **factual report** and cross-reference in the **GIIR** for parameter derivation.

6.1.9 Example Calculation

Given:
Measured N = 25 at 6 m depth, $\sigma'_v = 120\ kPa, E_r = 70$

$$C_N = (\frac{100}{120})^{0.5} = 0.91$$
$$N_{60} = 25 \times \frac{70}{60} \times 0.91 = 26.5$$

✓ **Corrected N_{60} = 26.5** → medium-dense sand (ϕ' ≈ 33°).

> **Phicon Note:**
>
> • Treat SPTs as **semi-empirical indicators**, not precise measurements — their value lies in pattern recognition across a site.
> • For **granular soils**, average N_{60} over a few points in the bearing zone to derive ϕ'_k; for **clays**, confirm c_u from triaxial or vane tests.
> • Always document **energy efficiency and overburden corrections** — uncorrected N-values are not valid for design.
> • Integrate SPT data into your **ground model plot** — it's a quick visual check for consistency of layer boundaries and density trends.

6.2 Cone Penetration Test (CPT / PCPT)

6.2.1 Principle and Application

The **Cone Penetration Test (CPT)** is a continuous in-situ test measuring soil resistance to cone penetration under controlled rate.

It provides a **rapid, repeatable, and continuous profile** of soil behaviour and mechanical properties.

When equipped with a pore pressure sensor (u_2), it becomes the **Piezocone Penetration Test (PCPT)** — the modern standard per **EN ISO 22476-1:2022**.

Measured Parameters	Symbol	Units	Description
Cone resistance	q_c	MPa	Tip resistance to penetration
Sleeve friction	f_s	MPa	Friction along cone sleeve
Pore pressure	u_2	kPa	Measured just behind cone tip (PCPT only)
Depth	z	m	Continuous measurement every 20 mm

Penetration rate: 20 ± 5 mm/s (standardised).

6.2.2 Equipment and System Types

System Type	Typical Use	Notes
Electric CPT (piezocone)	Modern standard	Records q_c, f_s, u_2 digitally
Mechanical CPT	Legacy (q_c only)	No pore pressure data
Seismic CPT (SCPT)	Adds V_s measurement	For dynamic stiffness / EC8
Cone size	10 cm^2 (standard) or 15 cm^2	Larger cones for gravels
Friction ratio	$R_f = 100 \cdot (f_s/q_c)(\%)$	Used for soil type classification

Ensure saturation of the u_2 filter element — improper saturation causes large errors in clay interpretation.

6.2.3 Field Procedure (BS EN ISO 22476-1)

1. Level and anchor the CPT rig.
2. Advance cone at constant rate (20 mm/s).
3. Record q_c, f_s, u_2 continuously with depth.
4. Stop penetration at target depth or refusal (>60 MPa qc).
5. Withdraw cone; verify zero readings.
6. For PCPT, perform **dissipation test** by pausing at selected depth and monitoring pore pressure decay vs time.

⚠️ Always plot q_c, f_s, and u_2 vs depth in the field to identify strata boundaries before leaving site.

6.2.4 Derived Parameters and Corrections

(a) Corrected Cone Resistance

$$q_t = q_c + u_2(1 - a)$$

where;

- a= cone area ratio (≈ 0.80 for standard cones).

(b) Friction Ratio

$$R_f = 100 \times \frac{f_s}{q_c}$$

Used to classify soil type and behaviour.

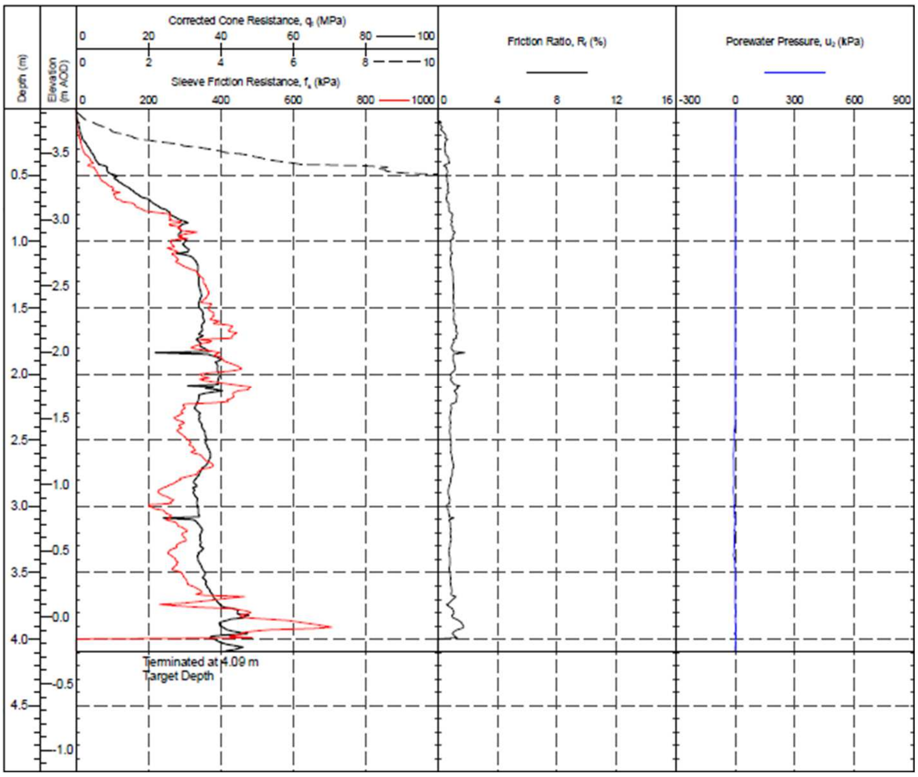


Figure 25: Typical CPT field measurements taken on a compacted uniform sand

6.2.5 Soil Behaviour Type (SBT) Classification

Using Robertson (2010) charts with **normalized** parameters:

$$Q_t = \frac{(q_t - \sigma_v)}{\sigma'_v}, F_r = \frac{f_s}{q_t - \sigma_v} \times 100$$

Zone	Typical Soil Type	Indicative ϕ' (°)	I_c
1	Sensitive CLAY	< 25	NA
2	Organic soils – CLAY	25–28	>3.6
3	Clays – silty CLAY to CLAY	28–30	2.95
4	clayey SILT to silty CLAY	30–32	2.60
5	Silty SAND to sandy SILT	32–38	2.05
6	Clean SAND to silty SAND	> 38	1.31
7	Gravelly SAND to dense SAND	> 40	<1.31
8	Very stiff SAND to clayey SAND	>38	NA
9	Very stiff, fine grained	30–32	NA

Plot points on the Robertson I_c chart to confirm soil behaviour index (I_c). Typical boundary: I_c = 2.6 between sand and clay behaviour.

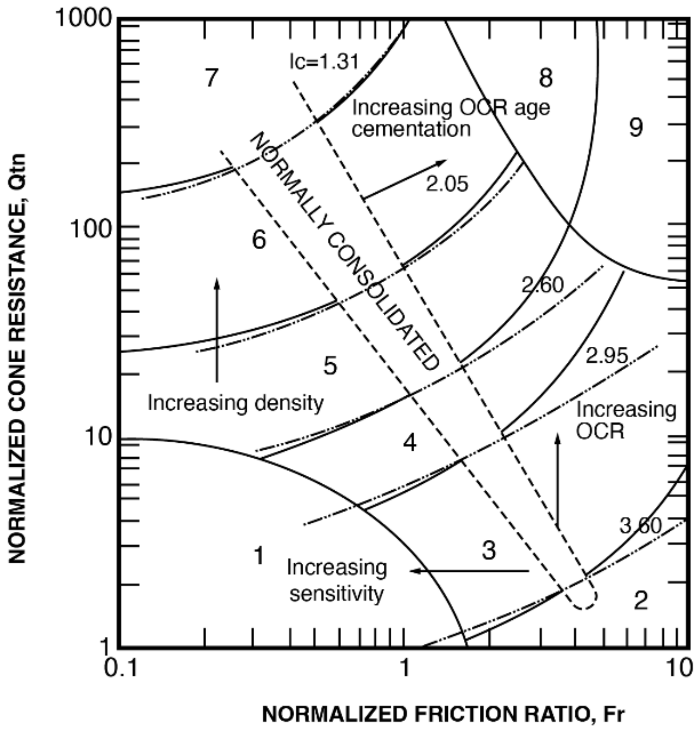


Figure 26: Normalised soil behaviour chart

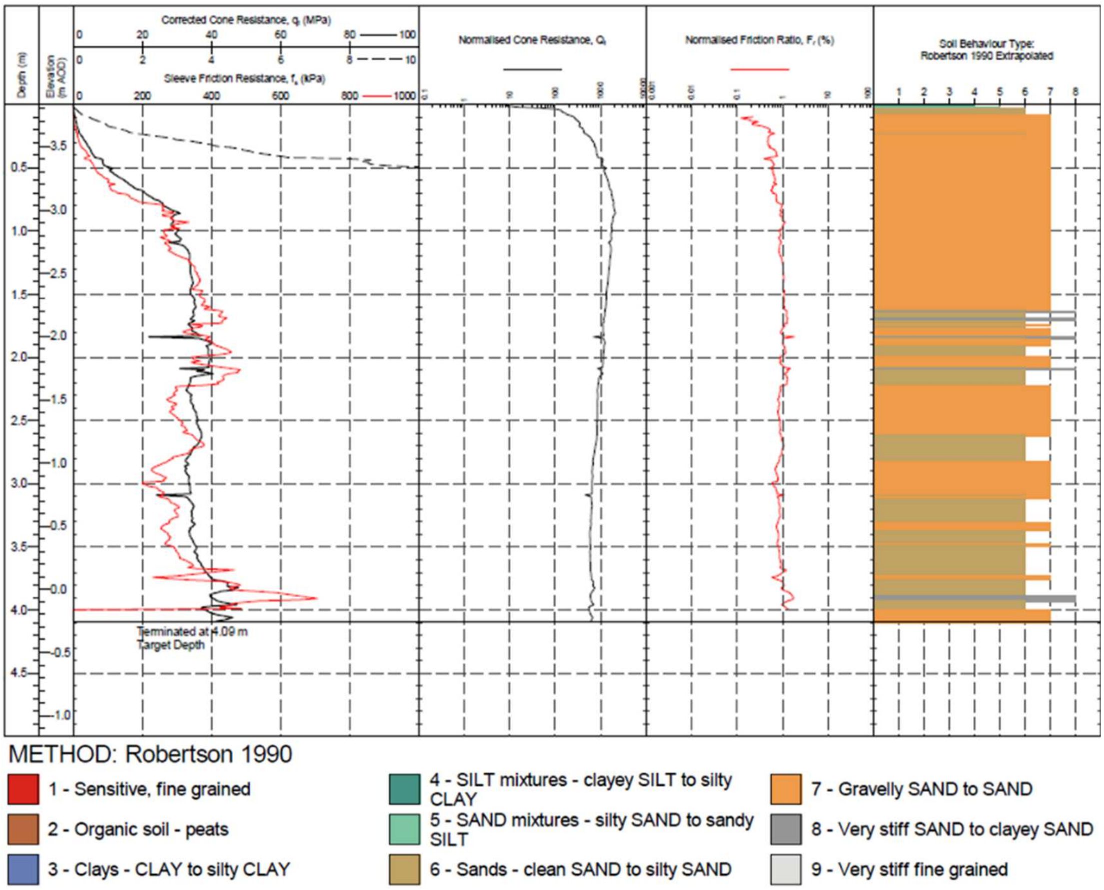


Figure 27: CPT normalised soil behaviour interpretation

6.2.6 Correlations for Strength and Stiffness

Parameter	Correlation (typical)	Applicability / Notes
Undrained shear strength (c_u)	$c_u = (q_t - \sigma_v)/N_k$ with $N_k = 15–20$	Soft to firm clays
Effective friction angle (ϕ')	$\varphi' = f(q_c/\sigma'_v)$, e.g. Bolton (1986): $\varphi' = 17 + 11 \cdot \log_{10}(q_c/\sigma'_v)$	Sands
OCR (Overconsolidation ratio)	$OCR = 0.25(q_t/\sigma'_v)^{0.8}$	Clays (Schmertmann, 1978)
Modulus (E_{50})	$E_{50} = (2.5–5) \cdot q_c$	General stiffness estimation
Young's modulus (E_s)	$E_s = (2–4) \cdot q_c$	Use for sand foundation settlement
Unit weight (γ)	$\gamma = 16 + 0.25 \cdot \log_{10}(q_c)$(approx)	Empirical; check vs lab data

⚠️ Use site-specific correlations where possible — published formulae vary widely by soil type and calibration.

6.2.7 Dissipation (u_2) Test

- Conducted by stopping cone at depth and measuring decay of u_2 with time.
- Used to estimate coefficient of consolidation (c_v) and permeability (*k*).
- The time to 50% dissipation, t_{50}, is read from u_2–t curve.

$$c_v = T_{50} \cdot \frac{r^2}{t_{50}}$$

where r= cone radius, T_{50}= theoretical time factor (≈0.245).

6.2.8 Example Calculation – Clay Layer

Given:
$q_c = 1.2\ MPa, \sigma_v = 0.1\ MPa, a = 0.8, u_2 = 0.05\ MPa$
Then:

$$q_t = 1.2 + 0.05(1 - 0.8) = 1.21\ MPa$$

Assume $N_k = 18$:

$$c_u = (1.21 - 0.1)/18 = 0.062\ MPa = 62\ kPa$$

✓ Characteristic undrained shear strength = 62 kPa.

6.2.9 Presentation and Reporting

Include in the **Factual Report:**

- CPT location plan and IDs (coordinates, ground level)
- q_c, f_s, and u_2 logs vs depth
- Derived parameters and SBT chart
- Dissipation test results and interpreted $c_{v,k}$ values

In the GIIR / GDR:

- Summarise derived ϕ'_k, c_{uk}, and stiffness (E_k) values with justification
- Reference any empirical correlations used
- State assumed correlation factors (e.g., N_k = 18, correlation source)

Cross-validate CPT data with borehole logs and lab tests for stratigraphic consistency.

6.2.10 Typical CPT Parameter Ranges (UK Practice)

Soil Type	q_c (MPa)	ϕ' (°)	c_u (kPa)	E_{50} (MPa)
Soft clay	< 1	—	20–50	3–10
Firm clay	1–4	—	50–100	10–20
Stiff clay	4–8	—	100–200	20–40
Loose sand	2–5	28–32	—	20–50
Medium sand	5–10	32–35	—	50–100
Dense sand	10–20	35–40	—	100–200
Very dense sand / gravel	> 20	> 40	—	200+

Phicon Note:

• CPT/PCPT data form the backbone of most **EC7 parameter derivations** — use them wherever possible to quantify ϕ', c_u, OCR, and stiffness continuously.
• For **Eurocode Design Approach 1**, characteristic ϕ'_k or c_{uk} should be derived from representative CPT trends, then reduced using γ_M from NA tables.
• Always cross-check CPT behaviour type against **sample descriptions and SPT trends** to validate the ground model.
• Retain all raw CPT files (.cor, .xls, .cdf) — these are often required for Cat III review or digital archive.

6.3 Field Vane and Hand Shear Tests

6.3.1 Purpose

Field vane and pocket (hand) shear tests provide a **direct in-situ measure of undrained shear strength (c_u)** in soft cohesive soils where sampling and laboratory testing risk disturbance.

They are particularly valuable for:

- **Soft to firm clays** below 0.5–15 m depth
- Embankment and slope assessment
- Excavation and working platform verification

6.3.2 Equipment Summary

Test	Typical Equipment	Measurement Range	Notes
Field vane (FVST)	4-blade cruciform vane (standard 100 × 50 mm) + torque head + depth rods	5–200 kPa	Standard per BS EN ISO 22476-9
Hand shear (pocket shear)	Spring-loaded torsional device with calibrated dial	5–100 kPa	Rapid indicative checks only

⚠️ The field vane is the only in-situ test that directly measures undrained shear strength; pocket shear gives comparative readings for QA.

6.3.3 Principle

The vane is inserted into the soil and rotated at a slow, steady rate (≈ 6°/min).

The torque required to cause shear failure on a **cylindrical surface** is recorded.

For a standard vane:

$$T = \frac{\pi D^2 H}{2} \cdot \tau + \frac{\pi D^3}{6} \cdot \tau$$

Where:

- T: torque at failure,
- D: vane diameter,
- H: vane height,
- τ: shear stress on cylindrical + end surfaces.

Simplifying for the 2:1 height-to-diameter ratio:

$$c_u = \frac{T}{\pi D^2 (H/2 + D/6)}$$

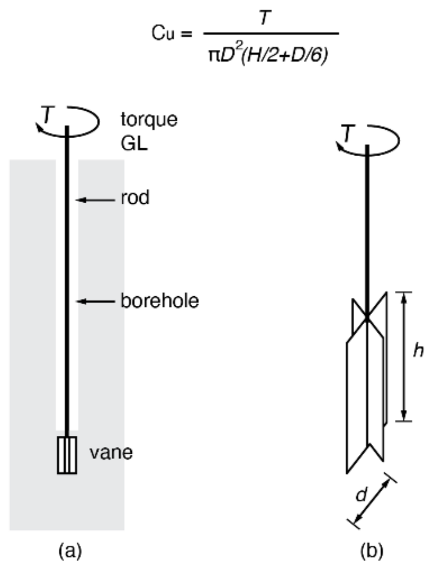


Figure 28: Vane shear test: (a) in a borehole and (b) vane.

6.3.4 Test Procedure (BS EN ISO 22476-9)

1. Push vane gently to test depth to avoid disturbance.
2. Record depth, strata, and water level.
3. Rotate vane at 6 ± 3 °/min; note torque vs angle.
4. Record maximum torque T_f at failure (undrained strength).
5. Immediately re-rotate soil at high speed to remould, and record residual torque T_r.
6. Compute sensitivity:

$$S_t = \frac{T_f}{T_r}$$

7. Extract vane, clean, and repeat at next interval.

6.3.5 Interpretation and Corrections

Correction	Factor / Typical Range	Purpose
Plasticity correction	$c_{u,corr} = c_u \times C_p$, where $C_p \approx 0.8–1.0$	Adjust for strain rate effects
Rate of rotation	Slow rotation → higher accuracy	Too fast → excess pore pressure
Rod friction	Subtract measured friction or use dummy test	Neglect can overestimate τ
Over-consolidation (OCR)	Vane often gives higher c_u in OC clays	Apply factor 0.7–0.8 if confirmed OC
Fissuring / anisotropy	Can reduce true strength	Check repeatability at nearby depths

⚠️ Where laboratory triaxial data exist, apply site-specific correction factors to align FV_{ST} and lab c_u values.

6.3.6 Typical Results and Classification

c_u (kPa)	Consistency (BS 5930)	Sensitivity S_t	Interpretation
< 25	Very soft	1–2	Normally consolidated clay
25–50	Soft	2–4	Low-plasticity OC clay
50–100	Firm	4–8	Moderately sensitive
100–200	Stiff	8–16	Sensitive structured clay
> 200	Very stiff / hard	> 16	Highly sensitive or fissured clay

⚠️ Plot c_u vs depth to check for increasing trend with overburden; flat or erratic profiles suggest disturbance.

6.3.7 Use in Eurocode 7 Design

Design Context	Application of FVST Data
Bearing (shallow / embankment)	Use representative mean of FV_{ST} c_u within bearing stratum → derive characteristic c_{uk} (5 % fractile or cautious mean)
Stability (slope / excavation)	Use c_{uk} from FV_{ST} profile; apply γ_m = 1.4 (UK NA, DA1-C1)
Correlations	Compare with CPT (Nk ≈ 15–20) and triaxial data to validate
Residual shear (post-failure)	Use remoulded $c_{ur} = c_u / S_t$

⚠️ For EC7 verification, always state source, test ID, depth, and derived characteristic c_{uk} in the GDR parameter table.

6.3.8 Pocket (Hand) Shear Test

- Used at shallow depths or exposed faces (trial pits, excavations).
- Measures peak torque on a small vane (≈ 20 mm) pressed against soil face.
- Useful for QA/QC and relative comparison, not formal design.
- Readings typically ±20 % accuracy; average several per location.

Inconsistent readings (> ±30 %) often indicate heterogeneity or poor contact.

6.3.9 Example Calculation

Given: D = 50 mm, H = 100 mm, T = 5.8 N·m

$$c_u = \frac{5.8}{\pi(0.05)^2(0.1/2 + 0.05/6)} = 53\ kPa$$

Remoulded torque $T_r = 1.9\ N \cdot m \rightarrow S_t = 5.8/1.9 = 3.1$

✓ Undrained shear strength ≈ 53 kPa, Sensitivity ≈ 3 → soft, moderately sensitive clay.

6.3.10 Reporting and QA

Include in Factual Report:

- Test IDs, depths, soil description, moisture condition
- T_f, T_r, c_u, S_t values
- Equipment details and calibration date
- Notes on disturbance or test anomalies

Summarise in Interpretative Report / GDR:

- Mean and characteristic c_{uk} values per stratum
- Sensitivity trends with depth
- Comparison with triaxial and CPT-derived strengths

Phicon Note:

• FV_{ST} gives reliable c_u only in soft cohesive soils — do **not** use in silty or fissured clays without corroboration.
• Correlate field vane profiles with **CPT and triaxial** to confirm consistency.
• Always quote **test ID + depth + sample description** alongside c_u values — this traceability is essential for EC7 verification.
• Use **remoulded vane strength** to assess stability during excavation or soft ground improvement phases.

6.4 Pressuremeter and Dilatometer Tests (PMT / DMT)

6.4.1 Purpose

Pressuremeter (PMT) and Flat Dilatometer (DMT) tests measure **in-situ deformability and strength** of soils. They are the most direct means of obtaining **stress–strain response** for design parameters such as:

- Deformation modulus (E_m / E^D)
- Limit pressure (p_l)
- In-situ horizontal stress (p_0 or K_0)

These parameters feed directly into **settlement**, **bearing**, and **retaining wall** analyses under Eurocode 7.

6.4.2 Types of Pressuremeter

Type	Installation	Typical Use	Standard
Pre-bored (Ménard)	Inserted into prepared borehole	Soft to stiff clays, sands	BS EN ISO 22476-4

Self-boring (SBPM)	Cuts into soil without pre-drilling	High-quality modulus data	BS EN ISO 22476-5
Push-in / full displacement	Advanced directly (CPT-based)	Rapid continuous profiling	BS EN ISO 22476-6
Rock pressuremeter	Borehole type with high pressure range	Weak rocks, chalk	EN ISO 22476-4 Annex C

Ménard PMT is most widely used in UK and Europe for foundation design; SBPM preferred in research and sensitive clays.

6.4.3 Test Principle

The test measures radial expansion of a cylindrical probe under controlled pressure increments, recording applied pressure (p) vs cavity volume change (ΔV).

A typical curve has three zones:

1. **Initial compression:** seating and system expansion
2. **Pseudo-elastic phase:** linear portion → defines modulus E_m
3. **Yield / plastic phase:** curve steepens until limit pressure (p_l)

6.4.4 Key Parameters

Symbol	Definition	Typical Range	Use
$\boldsymbol{p_0}$	In-situ horizontal stress (pressure at zero volume change)	50–400 kPa	Determines K_0
$\boldsymbol{E_m}$	Ménard modulus (slope of pseudo-elastic phase)	1–200 MPa	Foundation stiffness
$\boldsymbol{p_l}$	Limit pressure (plastic yield)	0.3–5 MPa	Ultimate bearing capacity
$\boldsymbol{p_l/p_0}$	Pressure ratio	2–6	Relative density / OCR indicator

6.4.5 Calculation of Parameters

- (a) Ménard Modulus (E_m)

$$E_m = 2(1+\nu) \cdot \frac{\Delta p}{\Delta \varepsilon}$$

where Δp= pressure increment (elastic range),

$\Delta\varepsilon$= radial strain increment, Poisson's ratio, $\nu \approx 0.33$.

Alternatively, from manufacturer software directly as slope of linear section.

- (b) Limit Pressure (p_l)

Taken as pressure at which curve deviates sharply or reaches 25% volume increase.

- (c) Bearing Capacity

$$q_{ult} = K_p(p_l - p_0)$$

where K_p = 1.2–1.5 (empirical for spread footings).

- (d) Settlement Estimation

$$s = \frac{q \cdot B(1 - \nu^2)}{E_m}$$

for small-strain response under footing width B.

6.4.6 Typical Values

Soil Type	E_m (MPa)	p_l (MPa)	ϕ' (°) (approx.)
Soft clay	2–6	0.5–1	—
Firm clay	6–15	1–2	—
Stiff clay	15–30	2–4	—
Loose sand	10–20	1–3	30–32
Medium sand	20–50	3–5	33–36
Dense sand	50–120	5–10	36–40
Gravel / dense fill	80–200	8–15	40–45

6.4.7 Dilatometer Test (DMT)

The **Flat Dilatometer Test** (DMT), per **BS EN ISO 22476-11**, involves a **stainless-steel blade** with a 60 mm circular membrane inserted into the soil.

The membrane is inflated at two pressures:

Symbol	Pressure	Meaning
$\boldsymbol{p_A}$	Pressure to lift membrane (contact pressure)	Reflects horizontal stress (K_0)
$\boldsymbol{p_B}$	Pressure to expand membrane 1.1 mm	Reflects stiffness
$\boldsymbol{p_0 = p_A - u_0}$	Effective horizontal stress index	—

DMT Moduli:

$$E_D = 34.7(p_B - p_A)$$

$$K_D = \frac{p_B - u_0}{\sigma'_v}$$

$$I_D = \frac{p_B - p_A}{p_A - u_0}$$

6.4.8 Interpretation of DMT Results

Parameter	Indicative Range	Interpretation
$\boldsymbol{I_D < 0.6}$	Clay / silt behaviour	Compressible
$\boldsymbol{0.6 < I_D < 1.8}$	Intermediate soils	Mixed plasticity
$\boldsymbol{I_D > 1.8}$	Sand / dense soils	Frictional
$\boldsymbol{K_D > 2}$	Overconsolidated or dense soils	High stiffness
$\boldsymbol{E_D/\sigma'_v = 150–600}$	Typical range	Used for settlement analysis

6.4.9 Correlations and Comparisons

Parameter	From PMT	From DMT	Notes
Small-strain modulus	E_m	E_D	Both can be used for settlement checks
In-situ stress ratio	$K_0 = p_0/\sigma'_v$	$K_D/1.5$	Correlated with OCR
Bearing capacity	$q_{ult} = K_p(p_l - p_0)$	–	PMT only
OCR (clay)	$p_l/p_0 \approx 1.2 \cdot OCR^{0.5}$	$K_D \approx 0.5 \cdot OCR^{0.8}$	Indicative
ϕ' (sand)	$\varphi' = f(p_l/p_0)$	$\varphi' = f(K_D)$	Calibrated empirically

6.4.10 Example Calculation (Ménard PMT)

Given:
Measured $p_0 = 120\ kPa, p_l = 900\ kPa, q = 200\ kPa, B = 2.0\ m.$

$$E_m = 20\ MPa$$

Settlement:

$$s = \frac{q \cdot B(1 - \nu^2)}{E_m} = \frac{200 \cdot 2 \cdot (1 - 0.33^2)}{20000} = 0.034\ m$$

✓ Predicted settlement = 34 mm

6.4.11 Use in Eurocode 7 Design

Design Aspect	Parameter Source	Application
Bearing capacity (GEO)	p_l, p_0	Compute ultimate and characteristic pressures
Settlement (SLS)	E_m, E_D**	Direct use in deformation analysis
K_0 for lateral pressure	From DMT or PMT p_0/σ'_v	Input for retaining wall and slope models
Verification	Compare with CPT/SPT derived ϕ' or c_u	Ensure consistency in ground model

For EC7 Design Approach 1, use characteristic $E_{m,k}$ and apply partial factor $\gamma_M = 1.0$ for stiffness-based serviceability checks.

6.4.12 Reporting Requirements

Include in Factual Report:

- Test type, depth, and equipment calibration data
- Pressure–volume (PMT) or pressure–displacement (DMT) curves
- Derived parameters: E_m, p_0, p_l or E_D, K_D, I_D
- Groundwater level and test environment

In Interpretative Report / GDR:

- Parameter tables by stratum
- Summary of representative moduli and bearing pressures
- Correlation with CPT / triaxial data
- Notes on applicability and limitations

Phicon Note:

• PMT and DMT data often provide the **most reliable in-situ stiffness values** — use them for settlement, deformation, and wall deflection checks.
• Always verify the **elastic range** on pressure–volume curves before adopting E_m.
• In granular soils, CPT–PMT cross-checks refine ϕ'; in clays, DMT–FVST comparisons improve c_u consistency.
• For large foundations, interpolate E_m spatially — **averaging over the loaded area** reduces local variability effects.

6.5 Plate Load Tests and EV_2 / CBR Correlation

6.5.1 Purpose

The **Plate Load Test (PLT)** is an in-situ method to determine the **load–deformation behaviour** of soil or compacted fill.

It is the standard verification tool for:

- Working platform and pavement stiffness (EV_2 criteria)
- Foundation bearing checks at shallow depth
- Earthworks and compaction QA

In UK practice, the PLT is executed in accordance with **BS 1377-9:1990**, and evaluated using **EV_1** and **EV_2** moduli as per **SHW Clause 612**.

6.5.2 Test Equipment

Component	Specification	Notes
Bearing plate	Typically 300 mm Ø (steel)	Larger plates (600 mm Ø) for coarse fills
Loading system	Hydraulic jack reacting against truck or frame	Capable of incremental loads to 1.5× design stress
Settlement measurement	Dial gauges or LVDTs (0.01 mm precision)	3 or more equally spaced gauges
Load measurement	Calibrated pressure gauge / load cell	Accuracy ±1 %
Seating	Flat, level surface; remove loose material	Plate seated on prepared test area

Ensure rigid reaction and vibration isolation for repeatable results.

6.5.3 Test Procedure (BS 1377-9)

1. **Seating:** Apply ~10 % design load for 1 min, then release.
2. **Loading:** Apply load in equal increments (usually 5–7 stages) up to design stress or plate yield.
3. **Recording:** At each increment, record settlement at 0.25, 0.5, 1, 2, 4, 8, 16 min, then every 5 min until rate < 0.02 mm/min.
4. **Unloading / Reloading:** Remove load in steps, then reload to 1.5× design load to obtain **Ev_2**.
5. **Plot:** Pressure (q) vs settlement (s).

6.5.4 Determination of Moduli

(a) First Loading Modulus (EV_1)

$$E_{V1} = \frac{\Delta q \cdot (1 - \nu^2) \cdot r}{\Delta s}$$

(b) Reloading Modulus (EV_2)

$$E_{V2} = \frac{\Delta q \cdot (1 - \nu^2) \cdot r}{\Delta s}$$

Where;

- Δq= stress interval in elastic range (kPa)
- Δs= corresponding settlement (mm)
- r= plate radius (mm)
- ν= Poisson's ratio (typically 0.3)

EV_2 is always taken from the **second loading curve**, representing soil response after seating or compaction.

EN 1997-2:2007, Annex K.2 gives the plate-settlement modulus as

$$E_{PLT} = \frac{\Delta p}{\Delta s} \frac{\pi b}{4} (1 - \nu^2)$$

where b is the **plate diameter**, ν is **Poisson's ratio**, Δp is the load increment, and Δs is the corresponding settlement increment (secant modulus).

6.5.5 Acceptance Criteria (SHW Clause 612)

Application	Minimum EV_2 (MPa)	EV_2 / EV_1 Ratio	Typical Notes
Capping (6F5 / 6F4)	≥ 120	1.2–2.0	Indicates well-compacted granular fill
Sub-base (Type 1)	≥ 180	1.2–2.0	Before pavement construction
Working platform (BRE 470)	≥ 100–150	1.1–2.0	Depends on loading class
Formation (cohesive)	≥ 60	≤ 2.5	Higher ratio → plastic or wet material
Earthworks (general)	As per project spec	—	Varies with class and use

⚠️ If $EV_2/EV_1 > 2.5$, the layer is under-compacted or moisture-sensitive.

6.5.6 Correlation with CBR

Approximate relationship for compacted granular layers:

$$EV_2 = 17.6 \times CBR^{0.64}$$

or, conversely,

$$CBR = 0.04 \times (EV_2)^{1.56}$$

EV_2 (MPa)	Equivalent CBR (%)	Indicative Soil
30	2	Soft clay / silty fill

60	5	Firm cohesive
120	10	Well-compacted capping
180	15	Sub-base (Type 1)
300	30	Crushed rock / concrete platform

⚠️ These are broad correlations; verify using site-specific paired testing if possible.

6.5.7 Interpretation and Checks

Observation	Implication
Non-linear first cycle, steep second	Normal compaction response
High EV_1, low EV_2	Disturbance / oversaturation
High $EV_2/EV_1 > 2.5$	Weak sub-layer or excess fines
Settlements > 5 mm at design load	Re-compact layer
Gradual curvature on second load	Progressive yielding

6.5.8 Worked Example

Given:
Plate $\text{Ø} = 300\text{ mm} \rightarrow \text{r} = 150\text{ mm}$, $\Delta q = 250\ kPa$, $\Delta s = 0.60\ mm$, $\nu = 0.3$

$$E_{V2} = \frac{250 \times (1 - 0.3^2) \times 150}{0.60} = 43{,}800\ kPa = 43.8\ MPa$$

✓ EV_2 = 44 MPa, correlates to CBR ≈ 6 % → marginal for formation layer.

6.5.9 Reporting

Include in Factual Report:

- Test location plan and coordinates
- Plate size, reaction system, date, operator
- Load–settlement data and curve
- EV_1, EV_2, and EV_2/EV_1 ratios
- Moisture and material description

Summarise in GIIR / QA report:

- Average EV_2 by layer or zone
- Non-conformances and remedial actions
- Comparison with specification thresholds

6.5.10 Design Use under Eurocode 7

Design Context	Use of PLT Data
Bearing (ULS GEO)	Derive characteristic $q_{(k)}$ from load–settlement curve (≤ 25 mm settlement at working load)
Serviceability (SLS)	Determine stiffness ($E_k \approx E_{V2}$) for settlement analysis
Earthworks / platform QA	Direct compliance with SHW / BRE 470 requirements
Model calibration	Validate finite-element soil modulus (E_{50})

⚠️ When used for EC7 verification, clearly state whether EV_2 represents drained (long-term) or undrained (short-term) stiffness conditions.

Phicon Note:

• Always perform at least **two repeat tests per material zone** — spatial variability is often greater than instrument error.
• Compare EV_2 with moisture trends and compaction test results (NDG, MCV) for QA correlation.
• For working platforms, link EV_2 data to the **BRE 470 design check sheet** — this provides full traceability between field stiffness and design load capacity.
• Capture load–settlement curves digitally; they provide invaluable data for calibrating constitutive stiffness in future projects.

6.6 Permeability and Pumping Tests

6.6.1 Purpose

Permeability (hydraulic conductivity, *k*) governs **seepage, drainage, and uplift pressures** in design.

Field permeability tests determine *in-situ k* more reliably than laboratory methods, particularly in **heterogeneous or fissured strata**.

They support EC7 verification of:

- UPL / HYD limit states
- Dewatering and drawdown design
- Slope and embankment stability under seepage
- Seepage through dams, cut-offs, and retaining walls

6.6.2 Test Methods Overview

Method	Typical Strata	Depth Range	Standard / Reference	Key Output
Falling-head (standpipe)	Silts, fine sands, clayey soils	≤ 10 m	BS 1377-5 §5	*k* for low permeability soils
Rising-head	Similar to falling-head	≤ 10 m	BS 1377-5 §5	Same principle, inverse head change

Method	Typical Strata	Depth Range	Standard / Reference	Key Output
Packer (in-hole / Lugeon)	Rock, chalk, stiff clay	3–50 m	EN ISO 22282-3	*k* or Lugeon ($\times 10^{-7}$ m/s)
Slug / Bail test	Standpipes or piezometers	3–30 m	ASTM D4044 (practice)	Rapid estimate of *k*
Pumping test	Sand, gravel, aquifers	5–100 m+	EN ISO 22282-4	*k*, T (transmissivity), S (storativity)
CPTu dissipation	Soft clays	≤ 30 m	EN ISO 22476-1	c_v, *k* from u_2 decay

6.6.3 Falling-Head / Rising-Head Tests

Performed in standpipes or boreholes.

Measure the time for water level to fall (or rise) between two points under gravity flow.

$$k = \frac{aL}{A\Delta t}\ln\left(\frac{h_1}{h_2}\right)$$

Where;

- a = cross-sectional area of standpipe (m^2)
- A = area of flow (borehole or sample) (m^2)
- L = length of test zone (m)
- h_1, h_2 = initial & final head difference (m)
- Δt = time interval (s)

Typical Ranges	Soil Type
$10^{-3} – 10^{-5}$ m/s	Sands, gravels
$10^{-6} – 10^{-7}$ m/s	Silts
$10^{-8} – 10^{-9}$ m/s	Clays
10^{-10} m/s ↓	Very impermeable soils

Repeat tests until log (h) vs t is linear → laminar flow confirmed.

6.6.4 Packer Tests (in Rock or Stiff Strata)

Single or double-packer assemblies isolate a test section of borehole.
Water is injected at constant pressure and flow rate (Q) measured.

$$k = \frac{Q}{A \cdot i}$$

where A = test section area, $i = \Delta h/L$.

Lugeon value = flow rate (L/min/m) at 10 bar pressure.

Lugeon (LU)	Interpretation
< 1	Tight / impermeable rock
1–5	Slightly fractured
5–20	Moderately fractured
> 20	Highly permeable / open joints

Correct flow for water viscosity (temperature) and ensure steady-state reached before next pressure step.

6.6.5 Slug / Bail Tests

A rapid test in piezometers — instant change of head (adding or removing water) and measurement of level recovery over time.

$$k = \frac{r_w^2}{2L} \frac{\ln\left(h_0/h_t\right)}{t}$$

where r_w = well radius, L = screen length.

Use for reconnaissance or construction QA.

6.6.6 Pumping Tests

Principle:
Water is pumped from a well at constant rate (Q) and drawdown (s) observed in the pumped and observation wells.

Steady-state (Dupuit-Thiem):

$$k = \frac{Q \cdot \ln\left(r_2/r_1\right)}{\pi H (h_2 - h_1)}$$

Where;

- r_1, r_2 = radial distances of observation wells,
- h_1, h_2 = corresponding heads,
- H = aquifer thickness.

Transient (Theis) methods yield transmissivity (T = kH) and storativity (S).

Test Duration	Objective
2–4 h	Indicative k for construction dewatering
24–72 h	Reliable steady-state assessment
> 7 days	Regional or aquifer scale characterisation

Always include a recovery phase — recovery data are often more diagnostic than drawdown.

6.6.7 Typical Hydraulic Conductivity Ranges

Material	k (m/s)	Flow Behaviour
Clean gravel	$10^{-2} – 10^{-3}$	Free draining
Sand	$10^{-3} – 10^{-5}$	Permeable
Silt	$10^{-6} – 10^{-7}$	Low permeability
Clay	$10^{-8} – 10^{-10}$	Impermeable

Chalk (karstic)	$10^{-4} – 10^{-6}$	Highly variable
Weathered rock	$10^{-6} – 10^{-8}$	Variable fracture flow

6.6.8 Interpretation and Use in Design

Design Aspect	Parameter Required	Typical Use in EC7 Checks
Uplift / heave (UPL)	k and head gradient (i)	Verify hydraulic equilibrium beneath raft/basement
Hydraulic heave (HYD)	k and Δh/L	Compute factor of safety against heave
Seepage and drainage	k field profile	Design of cut-offs, filters, and drainage blankets
Consolidation / cv	Derived from k via $k = cv \cdot mv \cdot \gamma_w$	Settlement calculations
Dewatering	T = kH for flow rate and radius of influence	Pump and well design

6.6.9 Reporting and QA

Include in Factual Report:

- Test type, location, depth, method, duration
- Head–time data and plots (log h vs t)
- Derived k, T, S values
- Temperature and viscosity corrections
- Groundwater conditions before/after test

In GIIR / GDR:

- Representative k_k per stratum (5 % fractile/percentile or cautious mean with 95% confidence)
- Groundwater regime and flow direction summary
- Implications for uplift, heave, and drainage design

6.6.10 Worked Example (Falling-Head Test)

Given:
$a = 4.91 \times 10^{-4}$ m^2, A = 0.028 m^2, L = 1.2 m, h_1 = 1.0 m, h_2 = 0.25 m, Δt = 7200 s

$$k = \frac{(4.91 \times 10^{-4}) \cdot 1.2}{0.028 \cdot 7200} \ln\left(\frac{1.0}{0.25}\right) = 2.6 \times 10^{-7}\ m/s$$

✓ Hydraulic conductivity = 2.6×10^{-7} m/s → low permeability silt / clayey sand.

Phicon Note:

• Permeability governs both **strength** and **stability** — always relate *k* to pore pressure behaviour in your ground model.
• Adopt the **highest credible *k*** for UPL/HYD checks (worst-case drainage) and **lowest *k*** for consolidation/settlement modelling.
• Never mix **lab and field *k*** without reconciliation — field tests capture macro-fissuring not seen in samples.
• Store all head–time data digitally; these trends are valuable for later model calibration and observational-method verification.

7 Laboratory Testing & Parameter Derivation

Laboratory testing converts physical samples into **quantitative design parameters**.

Eurocode 7 (§3.4.3) requires that test results be used to derive **characteristic values** of ground properties, with due regard to sample quality (EN 1997-2 §3.4.2).

7.1 Index Tests (Moisture, Atterberg, PSD)

7.1.1 Purpose

⚠️ Index tests describe the **physical state and composition** of soils — not strength or stiffness directly, but the indicators that control them.

They underpin classification, correlation, and assessment of **engineering behaviour** (compressibility, permeability, swell, etc.).

7.1.2 Moisture Content (w)

Test Standard	BS 1377-2 / EN ISO 17892-1
Principle	Dry representative sample at 105 ± 5 °C to constant mass.
Formula	$w(\%) = \frac{m_w}{m_s} \times 100 = \frac{m_1 - m_2}{m_2 - m_t} \times 100$
Use	Assess compaction state, volume-change potential, and compare with OMC from Proctor test.
Typical Ranges	Clay 25–80 %; Silt 10–40 %; Sand < 10 %.

Always express alongside sample class and depth — moisture variation often reveals drainage boundaries or desiccation.

7.1.3 Particle Size Distribution (PSD)

Test Standard	BS 1377-2 / EN ISO 17892-4
Method	Sieve (> 63 µm) + Hydrometer (< 63 µm).
Outputs	% passing vs sieve size (µm or mm); D_{10}, D_{30}, D_{60}; Uniformity Cu = D_{60}/D_{10}; Curvature Cc = $D_{30}^2/(D_{60}D_{10})$.
Classification Basis	% sand/silt/clay fractions (BS EN ISO 14688-1).
Engineering Use	Drainage, filter design, permeability, frost susceptibility.

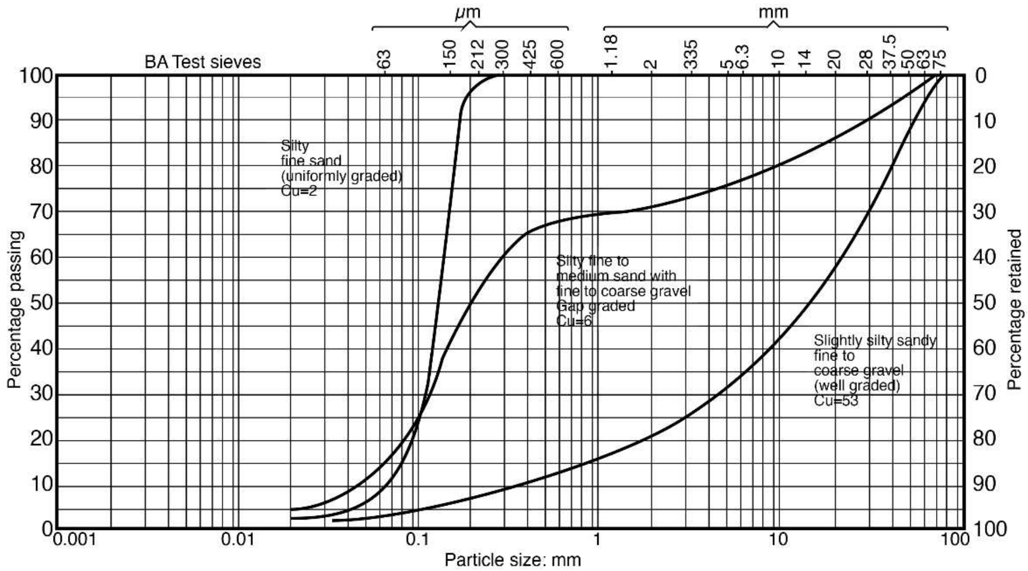


Figure 29: Terminology used to describe a PSD curve (reproduced from Nowak & Gilbert 2015)

7.1.4 Typical Indicators

Soil Type	D_{10} (mm)	Cu	*k* (m/s) (approx.)
Clean sand	0.2–0.6	2–5	10^{-3}–10^{-4}
Silty sand	0.06–0.2	5–15	10^{-5}–10^{-6}
Sandy silt	0.02–0.06	10–20	10^{-6}–10^{-7}
Clayey silt / clay	< 0.002	–	$< 10^{-8}$

⚠️ A gap-graded PSD (high Cu > 20) signals potential segregation or filter instability.

7.1.5 Atterberg Limits

Test Standard	BS 1377-2 / EN ISO 17892-12
Parameters	Liquid limit (w_L), Plastic limit (w_P), Shrinkage limit (w_S).
Derived Index	Plasticity Index $PI = w_L - w_P$.
Classification Chart	Casagrande A-line → distinguishes clays (A-line above) vs silts (below).

7.1.6 Interpretation (Typical Values)

Soil	w_L (%)	PI (%)	Behaviour
Silty clay / low plasticity	30–40	10–20	Low shrink–swell
Medium plastic clay	40–60	20–35	Moderate volume change
Highly plastic clay	> 60	> 35	High swell / low strength
Organic soil	> 80	variable	Very compressible

⚠️ Plotting on the Plasticity Chart is essential — it guides selection of undrained strength correlations (e.g., c_u–PI).

7.1.7 Derived Relationships

Purpose	Empirical Correlation	Range / Notes
Liquidity Index	$LI = (w - w_P)/(w_L - w_P)$	LI ≈ 0 → stiff; LI ≈ 1 → soft.
Activity (A)	$A = PI/$%clay (< 2 µm)	< 0.75 inactive; 1–2 active; > 2 very active (swelling risk).
Shrink–swell category	per NHBC & BS 5930	Class A (low) → Class D (high).
Permeability approx.	$k \approx 10^{-(PI/20)}$ m/s	Indicative for cohesive soils.

7.1.8 Reporting and Interpretation

Include in Factual Report:

- Moisture, PSD, and Atterberg data tables.
- Graphs: moisture vs depth; PSD curves; Plasticity chart.

Include in Interpretative Report / GDR:

- Soil classification per BS EN ISO 14688-2.
- Consistency limits vs depth.
- Identification of problematic zones (e.g., expansive clay, loose silty fill).

Cross-reference each test to its sample ID and class — this underpins EC7 traceability for derived design parameters.

Phicon Note:

• Index results form the **language of the ground model** — review them before strength or stiffness testing.
• Always inspect **moisture–PI–density** trends for compaction and volume-change assessment.
• Use **activity (A)** and **plasticity chart position** to screen for expansive or sulphate-susceptible materials.
• Maintain a single summary sheet per stratum listing **mean, min, max, and characteristic** values — this becomes your EC7 parameter base table.

7.2 Strength Tests (UU, CU, CD Triaxials; Shear Box)

7.2.1 Purpose

Strength tests determine the **shear resistance of soil** under controlled drainage and stress conditions.

Results are used to derive **undrained shear strength (c_u)** and **effective strength parameters (c', ϕ')** for Eurocode limit-state verification.

Eurocode 7 (§3.4.3) requires that parameters be derived from representative tests on **samples of known quality** (EN ISO 17892-8 to -10).

7.2.2 Triaxial Test Types

Test Type	Drainage Condition	Measured Parameters	Typical Use
UU – Unconsolidated Undrained	No drainage, no consolidation	Total stress (c_u only)	Short-term undrained strength of clays
CU – Consolidated Undrained (with pore pressure)	Consolidated under σ_3, then sheared undrained	Effective stress (c', ϕ'), pore pressure	Both total and effective analyses
CD – Consolidated Drained	Fully drained during shear	Effective stress (c', ϕ')	Long-term drained stability (sand, stiff clay)

All triaxial tests require known sample area, height, and confining pressure; pore pressure transducers are essential for CU tests.

7.2.3 Typical Test Procedure (CU Example)

1. Apply cell pressure (σ_3) and allow volume change → consolidation.
2. Record pore pressure (u) until rate < 1 kPa/min.
3. Shear at constant strain rate (≈ 0.5–2 %/hr).
4. Record deviator stress ($\sigma_1 - \sigma_3$) vs axial strain and pore pressure.
5. Plot *Mohr circles* and obtain effective parameters:

$$\tau = c' + \sigma' \cdot \tan \varphi'$$

where $\sigma' = (\sigma_1 – u_1 + \sigma_3 – u_3)/2$.

7.2.4 Example Result Summary

Specimen	Test Type	σ_3 (kPa)	$\sigma_1 - \sigma_3$ (kPa)	u (kPa)	c' (kPa)	ϕ' (°)	c_u (kPa)
Clay A (1)	UU	0	—	—	—	—	42
Clay A (2)	CU	100	180	60	12	26	—
Sand B (1)	CD	50	260	0	0	34	—

7.2.5 Quick-Use Correlations

Soil	Approximate ϕ' (°)	c_u (kPa)	Notes
Very soft clay	—	< 25	Immediate condition after excavation
Firm clay	—	25–75	Moderate sensitivity
Stiff clay	—	75–150	Typically PI 20–40
Loose sand	28–32	—	Drained ϕ'
Medium sand	32–35	—	Dense 35–38°
Dense gravel	38–42	—	Crushed rock 42–45°

For cohesive soils, ϕ' ≈ 0° is acceptable for short-term ULS; for drained design, always use effective ϕ'.

7.2.6 Shear Box Test (Direct Shear)

Standard	**BS 1377-7 / EN ISO 17892-10**
Principle	Horizontal shear along a defined plane under constant normal stress.
Outputs	τ–σ curve; intercept c' and slope tan ϕ'.
Typical Sample	60 mm square × 20 mm thick (cohesionless) or 100 mm × 25 mm (cohesive).

Test sequence:

1. Apply normal stress (e.g., 50, 100, 200 kPa).
2. Consolidate until volume change rate < 0.01 mm/min.
3. Shear at constant displacement rate (≈ 0.2–1 mm/min).
4. Record peak and residual shear stresses.

7.2.7 Peak vs Residual Strength

Parameter	**Definition**	**Design Relevance**
Peak (ϕ'_p, c'_p)	First failure on τ–σ curve	For intact or lightly disturbed soils
Residual (ϕ'_r, c'_r)	Fully softened, post-slip strength	For reactivated slopes, slickensides
Fully softened	Before residual reached	For long-term cut slopes or fissured clay

⚠️ Typical ranges (clays): ϕ'_p = 24–28°, ϕ'_r = 16–20°.

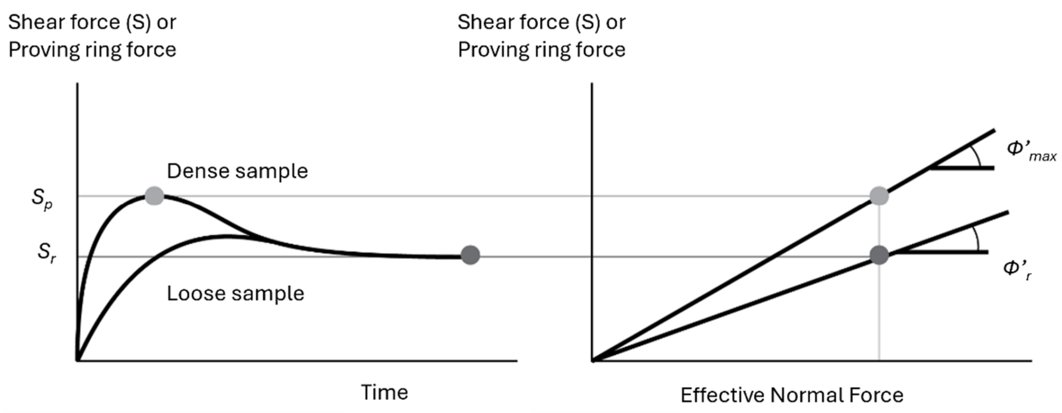


Figure 30: Typical dense and loose shear box test results

7.2.8 Sample Quality Considerations

Class	Recommended Test	Comment
1	CU/CD triaxial	High-quality undisturbed
2	UU triaxial / shear box	Moderate disturbance
3–5	Recompacted / index only	Classification tests only

Use Class 1–2 samples for parameter derivation; record recovery and disturbance factors in GDR tables.

7.2.9 Design Use in Eurocode 7

Design Limit State	Appropriate Parameter	Notes
GEO (short-term)	c_{uk} (from UU / FVST)	Apply γ_m = 1.4 (UK NA, DA1-C1)
GEO (long-term / drained)	c'_k, ϕ'_k (from CU/CD)	Apply γ_m = 1.25–1.5
STR (structural)	ϕ'_k, E_k → bearing / sliding	Coupled with EN 1992/1993 checks
SLS (settlement)	E_{50} or E_m from CU/CD	γ_m = 1.0 (stiffness parameter)

⚠️ Always quote both total and effective parameters and the test conditions under which they were obtained.

7.2.10 Typical Presentation (GDR Table Extract)

Stratum	Test Type	c_{uk} (kPa)	c'_k (kPa)	ϕ'_k (°)	E_{50} (MPa)	Sample Class
Made Ground	—	—	—	—	—	4–5
Soft clay	UU	35	—	—	5	2
Firm clay	CU	—	10	25	12	1
Dense sand	CD	—	0	36	45	2

Phicon Note:

- Always align **drainage condition** with the **design scenario** (short-term → total, long-term → effective).
- Verify ϕ' and c' trends with depth — inconsistent values often reveal sample disturbance or mixed strata.
- For layered deposits, use **weighted averages** of c_u or ϕ' by thickness for bearing and stability analyses.
- Record every derived parameter in a **traceable data sheet** (Sample ID → Test → Result → Design Value) for Cat III reviews.

7.3 Compressibility and Consolidation (Cc, Cv, m_v)

7.3.1 Purpose

Compressibility and consolidation testing quantifies the **volume-change behaviour** of soils under one-dimensional loading.

Results determine:

- **Compression index (Cc)** – magnitude of primary consolidation
- **Coefficient of consolidation (Cv)** – rate of settlement
- **Coefficient of volume compressibility (m_v)** – stiffness parameter for SLS analysis

These parameters feed directly into **Eurocode 7 Serviceability Limit State (SLS)** settlement checks for shallow foundations, embankments, and excavations.

7.3.2 Test Reference and Apparatus

Test	Standard	Typical Sample	Notes
Oedometer (1-D consolidation)	BS 1377-5 / EN ISO 17892-5	75 mm Ø × 20 mm H undisturbed clay/silt	Rigid ring, porous stones, incremental vertical loading

The test is one-dimensional – lateral strain is prevented, simulating vertical loading beneath wide foundations or embankments.

7.3.3 Principle

1. Apply successive load increments (typically doubling each step: 12.5 → 25 → 50 → 100 → 200 kPa).
2. Measure vertical compression (Δh) with time under each increment.
3. Plot **void ratio (e)** versus **log effective stress (σ'v)** → defines compressibility.
4. From time–settlement curves, derive rate parameter **Cv** by the square-root-of-time or log-of-time methods.

7.3.4 Key Parameters and Equations

Symbol	Definition	Units	Equation / Derivation
C_c	Compression index (slope of virgin line)	–	$C_c = \dfrac{\Delta e}{\Delta \log\ \sigma'}$
C_r (or C_s)	Recompression index (slope of rebound line)	–	$C_r = \dfrac{\Delta e_r}{\Delta \log\ \sigma'}$
e_0	Initial void ratio	–	$e_0 = V_v/V_s$
m_v	Coefficient of volume compressibility	m^2/kN	$m_v = \dfrac{\Delta \varepsilon_v}{\Delta \sigma'} = \dfrac{\Delta h/H}{\Delta \sigma'}$
C_v	Coefficient of consolidation	m^2/s	$C_v = \frac{T_v H_{dr}^2}{t_{90}}$ (typically Tv = 0.848 for 90 % consolidation)
C_c'	Normalised compressibility	–	$C_c' = C_c/(1 + e_0)$

a_v	Compression ratio	m^2/kN	$a_v = \Delta e/[(1+e_0)\Delta\sigma']$→ ≈ m_v

7.3.5 Typical Parameter Ranges

Soil Type	Cc	Cv (m^2/s)	m_v (m^2/kN)	Remarks
Dense sand	0.02–0.05	10^{-3}–10^{-4}	< 0.05 × 10^{-3}	Immediate (elastic) settlement only
Silty clay	0.20–0.35	10^{-7}–10^{-8}	0.2–0.6 × 10^{-3}	Moderate consolidation
Normally consolidated clay	0.25–0.50	10^{-7}–10^{-9}	0.4–1.2 × 10^{-3}	Dominant primary consolidation
Over-consolidated clay	0.05–0.15	10^{-7}–10^{-8}	0.1–0.3 × 10^{-3}	Limited compressibility
Peat / organic soil	1.00–3.00	10^{-6}–10^{-8}	> 5 × 10^{-3}	High and long-term settlement

7.3.6 Derived Relationships

Correlation	Approximation / Range	Use
$\boldsymbol{C_c \approx 0.009(w_L - 10)}$	For clays where no test data	Quick estimate of virgin compressibility
$\boldsymbol{m_v \approx \Delta h/(H\Delta\sigma')}$	From field plate or oedometer	Settlement modulus
$\boldsymbol{E_s \approx 1/m_v}$	Young's modulus equivalent	Used for SLS design (drained)
$\boldsymbol{C_c/C_r \approx 5\text{–}10}$	Typical ratio	Indicates pre-consolidation effect

7.3.7 Settlement Analysis (Eurocode 7 SLS)

The three-formula proposed by Terzaghi & Peck (1948) are widely used for calculating primary consolidation settlement. These formulas relate the settlement to the preconsolidation stress (σ'_p), initial (σ'_1) & final (σ'_2) effective stress.

For normally consolidated clay ($\sigma'_1 \geq \sigma'_p$) layer of thickness *H*, with final effective stress σ'_2 and initial σ'_1:

$$S_c = \frac{C_c \cdot H}{1+e_0} \log_{10}\left(\frac{\sigma'_2}{\sigma'_1}\right)$$

For over-consolidated soils ($\sigma'_2 \leq \sigma'_p$) (recompression zone):

$$S_c = \frac{C_r \cdot H}{1+e_0} \log_{10}\left(\frac{\sigma'_2}{\sigma'_1}\right)$$

If over-consolidated by the condition of $\sigma'_1 < \sigma'_p < \sigma'_2$:

$$S_c = \frac{C_r \cdot H}{1+e_0} \log_{10}\left(\frac{\sigma'_p}{\sigma'_1}\right) + \frac{C_c \cdot H}{1+e_0} \log_{10}\left(\frac{\sigma'_2}{\sigma'_p}\right)$$

Rate of Settlement

$$t_{90} = \frac{T_v H_{dr}^2}{C_v}$$

where H_{dr} = drainage path (half layer thickness if double drainage).

7.3.8 Worked Example

Given:
e_0 = 0.85, Cc = 0.30, Cr = 0.06, H = 4 m, σ'_1 = 80 kPa, σ'_2 = 160 kPa, σ'_p = 100 kPa

Here, $\sigma'_1 < \sigma'_p < \sigma'_2$, so the third condition applies,

$$S_c = \frac{0.06 \times 4}{1 + 0.85} \log_{10}\left(\frac{100}{80}\right) + \frac{0.30 \times 4}{1 + 0.85} \log_{10}\left(\frac{160}{100}\right) == 144.97\ mm$$

✓ Total primary settlement ≈ 145 mm

If Cv = 3×10^{-7} m^2/s, H_{dr} = 2 m →
$t_{90} = 0.848 \times 2^2/(3 \times 10^{-7}) \approx 11.3 \times 10^6 s \approx 130\ days.$

90 % of settlement in ≈ 4 months.

7.3.9 Design and Reporting

Report Stage	Key Outputs	Notes
Factual	Load–time & e–log σ' plots; derived Cc, Cv, mv per sample	Include full loading history
Interpretative / GDR	Representative Cc_k, Cv_k, mv_k by stratum	Use 5 % fractile (conservative)
EC7 Application	SLS settlement calc → compare S_{total} vs allowable (typically 25–50 mm)	Use γ_m = 1.0 for stiffness
Observational method	Compare predicted vs monitored settlements	Update mv and Cv if required

7.3.10 Typical Allowable Settlements (Reference)

Structure	Max Total (mm)	Max Differential (mm/m)
Raft / mat foundation	50	1/500
Pad foundations	25	1/300
Machine bases	10–20	1/1000
Embankments (long-term)	150+	n/a

Phicon Note

• Use **Cc/Cv/m_v** trends with depth to validate stiffness profile – outliers often indicate sample disturbance.
• Always distinguish **pre-consolidated vs normally consolidated** zones; over-consolidation can mask potential future settlement.
• Apply **Cv** conservatively for construction timescales – field drainage conditions rarely equal ideal lab drainage.
• When data are sparse, estimate **Cc from wL** and cross-check with **SPT/CPT-derived Es** for stiffness consistency.
• Incorporate measured settlements in an **Observational Method** feedback loop for ongoing design verification.

7.4 Chemical & Sulphate Testing

7.4.1 Purpose

Chemical testing determines the **aggressivity of the ground and groundwater** to buried concrete and metalwork and identifies materials that may cause **volumetric instability** (e.g., sulphate or pyrite attack).

Results guide:

- Concrete class selection (BRE SD1 / Design Sulphate Class)
- Material acceptability under SHW Series 600
- Mitigation measures for contamination or re-use of excavated material

7.4.2 Key Tests and Standards

Parameter	Symbol	Standard / Method	Purpose / Notes
Water-soluble sulphate	WSS	BS EN 1744-1, Cl.10	Determines readily soluble sulphates affecting early hydration
Total sulphur	S_t	BS EN 1744-1, Cl.11	Indicator of total sulphide and sulphate content
Acid-soluble sulphide (ASS)	S_a	BS EN 1744-1, Cl.13	Replaces obsolete oxidisable sulphide (OS); detects pyrite (FeS_2)
pH	—	BS 1377-3, Cl.9.5	Concrete protection / corrosion risk
Chloride content	Cl^-	BS EN 1744-1, Cl.7	Corrosion of reinforcement and metalwork
Organic matter	OM	BS 1377-3, Cl.4	Decomposition → settlement / gas potential
Electrical resistivity	ρ	ASTM G57 (indicative)	Low ρ → high corrosion potential
Sulfate in groundwater	SO_4^{2-} (mg/L)	BS EN 27888 / lab analysis	Combine with soil test for aggressivity class

⚠️ The **Acid-Soluble Sulphide (ASS)** test supersedes Oxidisable Sulphides (OS) — now the industry standard since 2022.

7.4.3 Assessment Frameworks

Two main guidance systems are used in UK practice:

Purpose	Reference	Primary Output
Concrete in aggressive ground	BRE Special Digest 1 (SD1)	Design Sulphate Class (DS) + Aggressive Chemical Environment for Concrete (ACEC)
Highway earthworks & fill	SHW Series 600	Compliance with material acceptability (e.g., sulphate ≤ 1% for Class 6F5)

7.4.4 BRE SD1 Assessment Flow

1. Determine soluble sulphate (2:1 soil:water extract) and pH.
2. Calculate equilibrium sulphate concentration (g/L) and classify into Total Potential Sulphate (TPS) or Water-Soluble Sulphate (WSS) range.
3. Combine with groundwater sulphate and pH to determine the Design Sulphate Class (DS1–DS5).
4. Assign **ACEC Class (AC1–AC5)** → used to specify concrete quality and protective measures.

7.4.5 Typical Design Sulphate Classes (BRE SD1)

WSS (g/L)	pH Range	Design Sulphate Class (DS)	ACEC Class	Indicative Concrete Class (per BS 8500)
< 0.2	> 5.5	DS-1	AC-1	DC-1, normal concrete
0.2–0.5	5.0–5.5	DS-2	AC-1s	DC-2, mild protection
0.5–1.9	4.5–5.0	DS-3	AC-2	DC-3, sulphate-resisting
1.9–3.7	4.0–5.0	DS-4	AC-3	DC-3, high-grade SRC or
> 3.7	< 4.0	DS-5	AC-4/5	DC-4/5, specialist

⚠️ When DS class exceeds DS-3, consult specialist concrete supplier; blended cements (CEM III/B) and low-permeability design become essential.

7.4.6 SHW Series 600 Material Compliance

Material Type	Test / Limit	Specification Reference	Notes
6F5 capping	WSS ≤ 1%	Clause 612.10	Typically < 0.3% achieved for crushed rock
Fill to structures	WSS ≤ 1.9 g/L; pH > 5	Clause 601.14	Avoid sulphate-rich industrial wastes
Earthworks general	Comply with Table 6/1	SHW 600	Requires sulphate & organic tests
Lime/cement treatment	Sulphate < 0.25%	HA 74/00	High sulphate impairs pozzolanic reaction

7.4.7 Interpretation and Engineering Implications

Finding	Risk / Consequence	Action
High WSS (> 1.9 g/L)	Sulphate attack on concrete	Upgrade DS class and specify sulphate-resisting mix
High ASS (> 0.03%)	Pyritic expansion (heave)	Avoid re-use or encapsulate under non-permeable layer
Low pH (< 4.5)	Acidic ground	Use acid-resisting materials / coatings
High Cl^- (> 0.4%)	Corrosion of reinforcement	Increase concrete cover or use cathodic protection
High OM (> 2%)	Decomposition / settlement	Remove or treat with lime/cement
Low resistivity (< 100 Ω·m)	Corrosion potential	Isolate metallic elements

7.4.8 Example Summary Table (Factual Report Extract)

Parameter	Lower Criteria	Upper Criteria	Result (Range)	Assessment
WSS (%)	0	1.9	0.12–0.34	PASS
Total Sulphur (%)	—	2.0	0.22	PASS
ASS (%)	—	0.03	0.00	PASS
pH	≥ 4.5	—	7.5–8.1	PASS
Chloride (%)	—	0.4	0.02	PASS

✓ Ground considered non-aggressive to concrete → DS-1 / AC-1 classification.

7.4.9 Reporting Requirements

Factual Report:

- List raw lab data for each sample (WSS, ASS, S_t, pH, Cl^-).
- Record extraction ratios and test methods (2:1 or 1:1).
- Identify sample depth and lithology.

Interpretative Report / GDR:

- Provide summary table by stratum.
- Define Design Sulphate and ACEC classes per BRE SD1.
- Comment on suitability of fills, treatment requirements, or blending strategy.

Phicon Note:

• Always specify **ASS instead of OS** — it detects both pyrite and reduced sulphur phases accurately.
• Combine **WSS + pH + groundwater chemistry** for reliable aggressivity classification.
• When in doubt, **adopt the higher DS class**; upgrading concrete mix is cheaper than remedial replacement.
• For re-used fills or capping, sulphate and pH data are as critical as compaction — ensure testing forms part of every approval certificate.
• Document laboratory certificates and interpretation traceably — they are often required at handover and warranty stage.

7.5 Result Interpretation and Parameter Selection

7.5.1 Purpose

The final step in ground characterisation is to convert **test results** into **design parameters** consistent with **EN 1997-1 §2.4.5.2** and **EN 1997-2 §3.4.3**.

This requires engineering judgement to define **characteristic values** that are:

- Representative of the stratum (mean trend)
- Conservative relative to variability and uncertainty
- Consistent across multiple test types (laboratory and in-situ)

7.5.2 Parameter Derivation Workflow

Step	Input	Action	Output
1	Field & lab data (FVST, CPT, CU, CD, oedometer, etc.)	Review quality, check consistency	Acceptable dataset
2	Stratigraphic grouping	Define zones of uniform soil behaviour	Layer model
3	Statistical summary	Mean, range, standard deviation, % fractile	Characteristic estimate
4	Apply EC7 principle	Select cautious value (5 % fractile for strength, mean for stiffness)	X_k– characteristic parameter
5	Apply partial factor γ_m	$X_d = X_k/\gamma_m$	Design value for verification
6	Document in GDR	Traceable reference (sample ID, test type, stratum)	Design-ready table

7.5.3 Characteristic vs Design Values (Eurocode 7)

$$X_k = \text{characteristic value (representative cautious estimate)}$$
$$X_d = \frac{X_k}{\gamma_M}$$

Parameter Type	Typical Partial Factor (UK NA, DA1-C1)	Notes
Undrained shear strength (c_u)	$\gamma_m = 1.4$	Apply to ULS GEO checks
Effective cohesion (c')	$\gamma_m = 1.25$	–
Effective friction angle (ϕ')	$\tan \phi'/\gamma_m$	Equivalent to $\phi'_d = \tan^{-1}(\tan \phi'/\gamma_m)$
Unit weight (γ, γ')	$\gamma_m = 1.0$	Usually taken as mean
Modulus (E, mv, Es)	$\gamma_m = 1.0$	For serviceability checks
Permeability (k)	$\gamma_m = 1.0$	Hydraulic checks (SLS)

EC7 requires consistency — partial factors apply to material parameters, not to test results.

7.5.4 Selecting Representative Parameters by Soil Type

Soil Type	Characteristic Strength Parameter	Characteristic Stiffness / Compressibility
Soft to firm clay	c_{uk} = mean of lower 1/3 FVST or UU tests; cross-check with CPT (Nk = 15–20)	mv_k = cautious mean from oedometer; $E_k = 2.5–4\ c_{uk}$ (short-term)
Stiff over-consolidated clay	$c_{uk} = 0.7 \times$ mean FVST (accounting fissuring); ϕ'_k from CU/CD tests	$E_k = 1000–2500\ c_{uk}$ or derived from PLT/PMT
Sand / gravel	ϕ'_k = mean − 2° from CPT/SPT correlations	$E_k = 2.5 \times q_{(c)}/\tan \phi'$ (approx.); use PMT/DMT where available
Silt / mixed soils	Select ϕ'_k conservatively (28–32° typical)	E_k = 20–60 MPa (correlate with EV_2 / DMT)
Fill / made ground	Assume $c'_k = 0$, ϕ'_k = 28–35° depending on compaction	E_k from PLT (EV_2) or empirical chart

7.5.5 Parameter Cross-Checking

⚠️ Always verify **inter-parameter consistency** to ensure a coherent ground model:

Check	Expected Range / Relation	Comment
ϕ' vs density	Denser → higher ϕ'	Compare with SPT N or CPT $q_{(c)}$
c_u vs PI	$c_u \approx 5 \times$ PI (kPa)	For normally consolidated clays

E vs c_u	$E \approx (500–1500) \times c_u$	Short-term stiffness for clays
E vs EV_2	$E \approx (2–3) \times EV_2$	Convert field modulus to design value
k vs PSD	Coarser → higher k	Check hydraulic consistency

7.5.6 Presentation in the Geotechnical Design Report (GDR)

Parameter Summary Table Example

Stratum	γ (kN/m^3)	c_{uk} (kPa)	c'_k (kPa)	ϕ'_k (°)	E_k (MPa)	mv_k (m^2/kN)	k_k (m/s)	Remarks
Made Ground	19	–	0	30	25	–	1×10^{-5}	Compact fill
Soft Clay	17.5	35	–	–	5	8×10^{-4}	2×10^{-7}	NC behaviour
Firm Clay	18.0	60	10	25	12	4×10^{-4}	1×10^{-7}	Slightly OC
Dense Sand	19.5	–	0	36	45	1×10^{-4}	3×10^{-5}	Drained
Weathered Rock	20.5	–	15	40	120	–	1×10^{-6}	Intact / fractured

Each entry must trace back to:

- Sample/test IDs
- Source method (lab/field)
- Test reference (BS/EN standard)
- Derivation note (e.g., "mean −1σ", "lower quartile")

This ensures full transparency for **Category 2 or 3** design reviews.

7.5.7 Typical Conservatism Levels

Soil Behaviour	Recommended Characteristic Estimate	Rationale
Homogeneous deposit, low variability	Mean – 1 SD	Slightly cautious
Moderate variability	Lower 10th percentile	EC7 conservative
Heterogeneous / fill	Mean of lowest third	Greater uncertainty
Critical interface zone	Lowest observed or inferred	Design-driving

⚠️ Avoid arbitrary reductions — document the rationale for each parameter.

7.5.8 Integration with Limit-State Design

Limit State	Primary Parameters Used	Verification Type
ULS – GEO	c_{uk}, c'_k, ϕ'_k, γ, k	Bearing, sliding, overturning
ULS – STR	c'_k, ϕ'_k, E_k	Section bending / shear capacity
ULS – UPL/HYD	k, γ_w, i	Uplift and heave
SLS	E_k, mv_k, Cc, Cv	Settlements and deflections
OBS / Monitoring	$c_u(t)$, E(t), pore pressure	Observational method (update model)

7.5.9 Reporting to Eurocode 7 Requirements

Each parameter must be traceable to its **source, derivation, and justification**. Eurocode 7 mandates that the GDR states explicitly:

- How each **characteristic value** was derived
- Which tests, correlations, or assumptions were used
- The **reliability class** of data (sample quality, test quality, representativeness)

Phicon Note:

• Always triangulate field and lab results — consistency builds confidence, inconsistency demands investigation.
• Avoid "best guess" averages — characteristic means *represent cautious lower bounds*, not optimistic fits.
• Document every assumption; a transparent parameter table is the first line of defence in any design audit.
• For complex or variable strata, define **parameter envelopes** (upper/lower) to support sensitivity or observational analysis.
• Ensure that final parameters in the GDR align exactly with those input into **design spreadsheets or FE models** — traceability is part of EC7 compliance.

8 Problematic Soils and Hazards

8.1 Expansive / Shrink–Swell Clays

8.1.1 Purpose

Expansive (shrink–swell) clays exhibit **significant volume change** with fluctuations in moisture content.

They cause differential heave and seasonal movement that can **damage foundations, slabs, and services**. Identification and mitigation are essential in **low-rise housing, pavements, and lightly loaded structures**.

8.1.2 Mechanism of Volume Change

Shrink–swell is driven by changes in **soil suction** and **water content** within the active zone (typically 1–2 m depth).

$$\Delta V/V \;\propto\; \Delta w \times \text{mineral activity}$$

Key influences:

- Clay mineral type (montmorillonite ≫ illite ≫ kaolinite)
- Plasticity (PI) and activity (A = PI / % clay < 2 μm)
- Seasonal moisture variation and vegetation
- Permeability and desiccation cracking

8.1.3 Identification

Indicator	Method / Reference	Typical Range / Criterion
Plasticity Index (PI)	Atterberg limits (BS 1377-2)	PI > 20 → potentially expansive
Activity (A)	PI / % clay < 2 μm	A > 1 → active clay
Mineralogy	X-ray diffraction / methylene blue	Montmorillonitic → high risk
Field evidence	Desiccation cracks, domed pavements, foundation distortion	Visual confirmation
Shrinkage Limit (w_S)	BS 1377-2	$w - w_S > 10$ % → susceptible

8.1.4 Classification (BS 5930 / NHBC Table 4.1)

Plasticity Index (%)	Shrink–Swell Potential	Volume Change Category	Typical Soils
< 20	Low	Class A	Silty / sandy clays
20 – 40	Medium	Class B	Clayey drift, glacial clay
40 – 60	High	Class C	London / Weald Clay
> 60	Very high	Class D	Gault / Lias clays, bentonitic

8.1.5 Quantifying Movement

1. **Shrinkage strain (ε_s)** from lab shrinkage limit:

$$\varepsilon_s = \frac{w_i - w_s}{(1 + e_0)} \times \rho_w / \rho_s$$

2. Approximate ground movement (BRE Digest 240):

$$\Delta H = \varepsilon_s \times H_a$$

where H_a = depth of active zone (typically 1.0–1.5 m).

Shrink–Swell Class	ε_s (%)	Expected ΔH (mm)
A (Low)	< 0.5	< 15
B (Medium)	0.5 – 1.0	15–30
C (High)	1.0 – 2.0	30–60
D (Very high)	> 2.0	> 60

8.1.6 Design Implications (EC7 & NHBC 4.2)

Aspect	Design Guidance
Foundation depth	Increase to extend below seasonal moisture zone (≥ 1.0 m typ., up to 3 m adjacent trees).
Tree influence	Use NHBC Table 4.4 tree foundation depth guidance — depends on mature height and PI.
Rafts / slabs	Reinforce to limit differential movement (max angular distortion ≈ 1/500).
Drainage control	Maintain surface falls and soakaways away from structures.
Rehydration risk	Avoid rapid rewetting after desiccation (e.g. following removal of large trees).
Subgrade design	For pavements: ensure formation moisture ≈ OMC ± 2 %; allow for suction changes.

8.1.7 Mitigation and Remediation

Issue	Mitigation Measure
Tree influence	Root barriers, selective felling, rehydration monitoring
Shallow foundations	Deepen or adopt stiffened raft / piles
New fill on clay	Pre-wet and compact; use geogrid-reinforced capping
Service routes	Flexible joints, trench fill with low-shrink material

Existing distress	Controlled re-watering, underpinning, or partial replacement

8.1.8 Testing and Monitoring

Parameter	Test / Standard	Purpose
Atterberg limits	BS 1377-2	Plasticity / activity classification
Shrinkage limit	BS 1377-2 Cl. 6	Estimate ε_s
Suction measurement	Filter paper / thermocouple	Assess moisture regime
Seasonal movement	Settlement / heave pins	Observational method verification

8.1.9 Example Assessment

Site: Over-consolidated clay with PI = 45, e_0 = 0.9, H_a = 1.5 m
From correlation: $\varepsilon_s \approx 1.4$ % →

$$\Delta H = 0.014 \times 1500 = 21\ mm$$

✓ High shrink–swell potential (Class C): foundations ≥ 1.2 m deep, reinforced slab recommended.

Phicon Note:

• Treat **moisture variation** as a design load — it induces cyclic stress on foundations comparable to seasonal temperature.
• Verify **tree influence radius** using both NHBC and BRE Digests; double-check where trees are retained post-construction.
• During earthworks, maintain **controlled moisture** (± 2 % OMC) — compaction outside this range exacerbates future heave.
• Record PI, moisture, and vegetation data in the **GDR hazard register** to maintain traceability through design and construction.

8.2 Organic and Peaty Soils

8.2.1 Purpose

Organic and peaty soils are **highly compressible, weak, and moisture-sensitive** materials formed from decayed vegetation in waterlogged conditions.

They pose risks of **excessive settlement, instability, and degradation** when loaded or exposed to air. Typical engineering responses include **avoidance, removal, or ground improvement**.

8.2.2 Identification

Indicator	Method / Reference	Typical Criterion / Range
Organic content	Loss-on-ignition (BS 1377-3 Cl.5)	> 6 % organic → organic soil; > 20 % → peat
Colour & odour	Visual / tactile	Dark brown–black, fibrous, strong odour
Bulk density	BS EN ISO 17892-2	< 1.2 Mg/m^3 typical
Moisture content	Oven-dry (BS 1377-2)	Often > 150 % for peat
Fibre content	Von Post scale (H1–H10)	H1–H4 fibrous → soft; H7–H10 amorphous
Shear strength (FVST)	EN ISO 22476-9	< 20 kPa, often < 10 kPa
Compressibility (mv)	Oedometer	> 5×10^{-3} m^2/kN (very high)

Organic odour, fibrous texture, and low density are immediate field indicators of potential peat horizons.

8.2.3 Engineering Behaviour

Characteristic	Typical Range / Observation	Implication
High water content	150–800 %	Rapid consolidation / settlement
Low strength	c_u < 15 kPa	Bearing and slope instability
High compressibility	Cc = 1–3	Long-term creep settlement
Organic decay	Biochemical oxidation over time	Progressive loss of volume and stiffness
Permeability	$k = 10^{-6}–10^{-4}$ m/s	Slow drainage but potential for seepage paths

8.2.4 Typical Problem Mechanisms

- **Immediate settlement** under low stress (soft matrix).
- **Secondary compression (creep)** continues for years after loading.
- **Lateral instability** at embankment toes.
- **Differential settlement** due to variable thickness and decomposition.
- **Oxidation shrinkage** if desiccated during construction.

8.2.5 Eurocode 7 Design Implications

Design Aspect	Recommendation / Check
Bearing capacity (GEO)	Avoid using peat for foundation support; if unavoidable, use floating raft or piles to firm stratum.
Settlement (SLS)	Compute primary + secondary settlement; adopt Cc = 1–3, $C\alpha \approx 0.02–0.05$ Cc.
Stability	Verify low undrained shear strength (c_{uk} = 5–10 kPa); use staged construction or basal reinforcement.
Drainage	Consider prefabricated vertical drains (PVDs) + surcharge loading to accelerate consolidation.
Replacement	Excavate peat where depth < 2 m and lateral extent limited.

8.2.6 Treatment and Mitigation Options

Technique	Principle	Typical Use
Excavation & replacement	Remove peat; backfill with compacted granular material	Shallow pockets (< 2 m thick)
Preloading / surcharge	Apply temporary load to induce consolidation	Moderate depths (2–5 m)
Vertical drains (PVDs)	Shorten drainage path, speed up consolidation	Thick deposits (> 3 m)
Lightweight fill	Reduce load (e.g., EPS, foamed concrete, tyre chips)	Road embankments
Basal reinforcement	Geogrid or basal mattress distributes load	Embankments over soft peat
Deep mixing / vibro replacement	Improve stiffness locally	Limited success — check organic interference
Floating raft / piles	Transfer load to firm strata	Structural foundations

Choice of method depends on thickness, moisture regime, access, and environmental sensitivity.

8.2.7 Typical Parameter Ranges for Design

Property	Symbol	Range (Indicative)
Bulk density	γ	10–13 kN/m^3
Undrained shear strength	c_u	5–15 kPa
Compression index	C_c	1.0–3.0
Coefficient of consolidation	C_v	1×10^{-7}–1×10^{-8} m^2/s
Coefficient of secondary compression	C_α/C_c	0.02–0.05
Permeability	k	10^{-6}–10^{-4} m/s
Void ratio	e_0	3–10
Water content	w	150–800 %

8.2.8 Construction Considerations

- **Pre-loading & monitoring:** record settlements; allow full primary consolidation before final surfacing.
- **Seasonal moisture control:** avoid stripping or heavy trafficking in dry periods (oxidation).
- **Environmental handling:** peat is a carbon-rich material — treat excavation and re-use under waste and environmental licensing guidance (EA/SEPA).

- **Instrument monitoring:** settlement plates, extensometers, and piezometers for rate control.

8.2.9 Worked Example (Indicative)

Given: H = 4 m of peat, Cc = 2.2, e_0 = 6.0, σ'_1 = 20 kPa, σ'_2 = 40 kPa

$$S_c = \frac{C_c \cdot H}{1 + e_0} \log_{10} \left(\frac{\sigma'_2}{\sigma'_1}\right) = \frac{2.2 \times 4}{1 + 6} \log_{10} \left(\frac{40}{20}\right) = 0.38\, m$$

✓ Predicted primary settlement ≈ 380 mm → unacceptable without preloading or replacement.

Phicon Note:

• Peat is not a foundation material — treat it as a **hazardous stratum** requiring removal or engineering control.
• Where removal is uneconomic, **design for deformation**, not strength — adopt flexible structures or staged construction.
• Always include **secondary compression (creep)** in settlement forecasts — it dominates long-term movement.
• Map peat extents in plan and section — its **thickness and lateral continuity** control the risk envelope more than its strength alone.
• Retain test records (loss-on-ignition, FVST, oedometer) in the **hazard register** for traceable design review.

8.3 Made Ground and Fill Materials

8.3.1 Purpose

“Made Ground” refers to **anthropogenic deposits** — either engineered fills placed under controlled conditions, or uncontrolled historical deposits.

They vary widely in composition, density, and strength, and present risks of **differential settlement**, **instability**, and **contamination**.

Understanding the **origin, composition, and compaction quality** of fill is critical to determine whether it may:

- Support foundations or pavements directly; or
- Require remediation, improvement, or replacement.

8.3.2 Classification (BS 5930 / EN ISO 14688)

Category	Description	Engineering Implication
Engineered fill	Placed and compacted under controlled specification (e.g., SHW Series 600)	Predictable stiffness and bearing capacity
Non-engineered fill	Random tipped, uncontrolled compaction	Unreliable behaviour; avoid founding directly

Category	Description	Engineering Implication
Industrial / demolition waste	Includes brick, ash, slag, concrete	Variable density; chemical attack potential
Municipal waste / landfill	Organic / putrescible	Compressible, gas-producing; unsuitable for support
Reworked natural material	Cut–fill re-used	Acceptable if moisture-controlled and tested
Recycled aggregate	Processed crushed concrete / asphalt	Acceptable under compliance testing (6F5 / Type 1)

8.3.3 Key Risks

Hazard	Mechanism	Engineering Consequence
Variable density	Inconsistent placement / composition	Differential settlement
Decomposition	Organic or sulphide material	Long-term volume loss
Voids / inclusions	Timber, metal, rubble	Local collapse under load
Contamination	Industrial residues	Chemical attack / vapour migration
Gas generation	Methane, CO_2 from degradation	Explosion / asphyxiation risk
Corrosion potential	Acidic or sulphate-rich	Concrete and steel deterioration
Poor drainage	High fines or debris	Pore pressure build-up / instability

8.3.4 Field Identification and Investigation

- **Trial pits / boreholes:** Record composition, inclusions, colour, odour, and moisture condition.
- **Dynamic probing / SPT:** Identify variability; refusal layers often mark buried rubble or concrete.
- **Permeability tests:** Assess drainage and groundwater control potential.
- **Laboratory testing:** PSD, Atterberg limits, sulphate/pH, and organic content.
- **Gas and vapour monitoring:** Methane, carbon dioxide, flow rate, and oxygen content (BS 8576).

Treat fills as highly variable; increase test frequency and apply engineering judgement to interpolate between results.

8.3.5 Engineering Behaviour (Typical Ranges)

Parameter	Uncontrolled Fill	Engineered Fill (compacted granular)	Notes
γ (kN/m^3)	16–20	18–21	Depends on composition
c_u (kPa)	5–30	50–150	Fines or cohesive content dependent
ϕ' (°)	25–35	30–40	Lower for silty/clayey fill
E (MPa)	5–30	50–150	Field PLT (EV_2) commonly 30–150 MPa

k (m/s)	10^{-8}–10^{-4}	10^{-6}–10^{-3}	Highly variable; check drainage
Sulphate (%)	—	< 1.0 (SHW 600)	Verify by testing
Gas (CH_4 / CO_2 %)	variable	< 1 (negligible)	Ventilation or barrier may be required

8.3.6 Design and Eurocode 7 Considerations

Design Aspect	Guidance / Approach
Bearing capacity (GEO)	Avoid direct founding unless proven by PLT or CPT; otherwise transfer to natural stratum.
Settlement (SLS)	Treat as compressible; use empirical modulus from PLT or back-calculated EV_2.
Slope stability	Check fill interfaces; compaction planes can act as slip surfaces.
Drainage	Provide underdrains or geocomposite layers to avoid trapped pore pressures.
Chemical aggressivity	Test WSS, pH, ASS; ensure concrete and steel protection per BRE SD1.
Geotechnical category	Category 2 or 3 depending on variability and consequence.

When fills form part of a structure (e.g., embankment, platform), design verification and certification per EN 1997-1 §2.1(8) are mandatory.

8.3.7 Construction and Verification (SHW Series 600)

Stage	Key Control	Testing / Acceptance
Material selection	Free from organic / deleterious matter	PSD, sulphate, pH, organic content
Placement	Layer thickness ≤ 225 mm (granular)	Visual + in-situ density checks
Compaction	Achieve ≥ 95 % MDD (BS 1377-4)	NDG, sand replacement, or MCV correlation
Verification	Surface stiffness	Plate Load (EV_2 ≥ 120–180 MPa typical)
Documentation	Daily placement / test logs	Required for QA sign-off

8.3.8 Differential Settlement and Compatibility

Interface Type	Design Consideration
Fill–natural soil	Step change in stiffness → risk of differential movement

Fill–structure	Provide slip layer or compressible inclusion
Fill–service trench	Over-compact backfill or provide flexible joints
Variable fill thickness	Preload or lightweight fill to minimise strain differentials

⚠️ Settlement potential increases sharply with layer heterogeneity — stratify model by fill type and placement phase.

8.3.9 Contamination and Gas Hazard

Assessment Standard	Parameter	Design Implication
BS 8576 / CIRIA C665	CH_4, CO_2, flow rate	Determine gas protection class (1–6)
BS 10175	Chemical contamination	Determine remediation or encapsulation
CL:AIRE Definition of Waste	Fill re-use classification	Determines regulatory compliance
BRE SD1	WSS, pH	Concrete protection
EA/SEPA guidance	Hydrocarbon / metal content	Groundwater risk

⚠️ Adopt multidisciplinary review (geo-environmental + geotechnical) before re-use or acceptance.

8.3.10 Example Case

Scenario: Old industrial fill, 2.5 m thick, compacted gravelly clay with rubble inclusions.
Plate Load Test → EV_2 = 80 MPa, pH = 7.5, WSS = 0.12 %.
Design for pad foundation (width 2.0 m) on fill with 100 kPa working stress. Assume Poisson's ratio 0.3.

Settlement check (simplified):

$$s = \frac{q \cdot B(1-\nu^2)}{E} = \frac{100 \cdot 2(1-0.3^2)}{80000} = 2.28\, mm$$

✓ Acceptable settlement (< 25 mm).
Bearing and stiffness acceptable with QA documentation.

Phicon Note:

• Treat all Made Ground as **variable until proven otherwise** — even "engineered fill" requires verification.
• For existing sites, **perform PLT or CPT checks** before relying on design assumptions.
• Always test for **chemical and gas hazards**; even inert-looking fill can be aggressive or gassing.
• Maintain **layer-by-layer QA records** — these are key evidence for compliance under SHW 600.
• Avoid founding on **mixed fill–natural interfaces** without transition layers or ground improvement.

8.4 Sulphates, Pyrite and Chemical Attack

8.4.1 Purpose

Certain soils and fills contain **sulphate or sulphide minerals** (notably gypsum, anhydrite, and pyrite) that can react with water, air, and cement hydration products, causing **swelling, cracking, and long-term deterioration** of buried concrete and steel.

This section summarises field recognition, laboratory testing, interpretation, and mitigation procedures for **aggressive ground conditions**.

8.4.2 Mechanisms of Chemical Attack

Mechanism	Reaction Process	Consequence
Sulphate attack	SO_4^{2-} ions react with tricalcium aluminate (C_3A) in cement → ettringite formation	Expansion, cracking, strength loss
Pyritic oxidation	$FeS_2 + O_2 + H_2O \rightarrow Fe(OH)_3 + H_2SO_4$	Generates sulphuric acid → concrete & metal corrosion
Acidic ground	Low pH (< 4.5) dissolves $Ca(OH)_2$ from concrete	Accelerated reinforcement corrosion
Chloride presence	Cl^- ions promote corrosion of embedded steel	Loss of section, bond failure

8.4.3 Laboratory Tests and Standards

Parameter	Symbol	Standard / Clause	Purpose / Notes
Water-soluble sulphate	WSS	BS EN 1744-1 Cl. 10	Quantifies immediately available SO_4^{2-} in soil
Total sulphur	S_t	BS EN 1744-1 Cl. 11	Measures total sulphide + sulphate content
Acid-soluble sulphide	ASS	BS EN 1744-1 Cl. 13	Detects pyrite; replaces obsolete oxidisable sulphide (OS)
pH	—	BS 1377-3 Cl. 9.5	Assesses acidity / alkalinity
Chloride	Cl^-	BS EN 1744-1 Cl. 7	Corrosion risk for steel
Groundwater sulphate	SO_4^{2-}(aq)	BS EN ISO 5667 / Lab analysis	Combined with soil data for BRE SD1 classification

Key change: since 2022 the **ASS** test supersedes the older **OS** method — now standard across SHW Series 600 contracts.

8.4.4 Interpretation Frameworks

Two parallel frameworks are applied:

Context	Reference	Output
Concrete durability	BRE Special Digest 1 (SD1)	Design Sulphate Class (DS1–DS5) + ACEC Class (AC1–AC5)
Highway / earthworks compliance	SHW Series 600 (Table 6/1, Clause 601.14)	Material acceptability limits for fill and capping

8.4.5 BRE SD1 Assessment Procedure

1. Determine WSS (2:1 soil:water extract) and pH of soil.
2. Measure **groundwater SO_4^{2-}** concentration (mg/L).
3. Compare results with BRE SD1 Table C1 to assign **Design Sulphate Class (DS1–DS5)**.
4. Determine Aggressive Chemical Environment for Concrete (ACEC) class (AC1–AC5).
5. Select appropriate **concrete designation (DC-class)** per BS 8500-1 / -2.

Typical Range (2:1 Extract)	pH	DS	ACEC	Concrete Class
WSS < 0.2 g/L	> 5.5	DS-1	AC-1	DC-1 (normal)
0.2–0.5 g/L	5.0–5.5	DS-2	AC-1s	DC-2
0.5–1.9 g/L	4.5–5.0	DS-3	AC-2	DC-3 (SRC cement)
1.9–3.7 g/L	4.0–5.0	DS-4	AC-3	DC-3/4
> 3.7 g/L	< 4.0	DS-5	AC-4/5	DC-4/5 + barrier membrane

8.4.6 Pyrite and Acid-Soluble Sulphides (ASS)

Result	Interpretation	Recommended Action
ASS < 0.03 %	Non-pyritic	Normal classification
0.03–0.5 %	Slightly pyritic	Avoid lime/cement treatment; monitor oxidation
0.5–2.0 %	Moderately pyritic	Potential heave on oxidation; encapsulate or remove
> 2.0 %	Highly pyritic	Treat as hazardous; remove or isolate with HDPE membrane

Oxidation of FeS_2 generates sulphuric acid → secondary gypsum → expansive heave. Always assess in conjunction with pH and moisture regime.

8.4.7 Typical SHW 600 Material Limits

Material Type	Parameter / Limit	Reference	Acceptance Notes
6F5 / 6F4 capping	WSS ≤ 1.0 %	Cl. 612.10	Commonly < 0.3 % in crushed rock
Fill to structures	WSS ≤ 1.9 g/L, pH > 5	Cl. 601.14	Reject sulphate-rich industrial wastes
Lime/cement treatment soils	WSS < 0.25 %	HA 74/00	High sulphate impairs pozzolanic reaction
Recycled concrete aggregate	ASS < 0.03 %	EN 1744-1 Cl. 13	Ensures no pyrite expansion risk

8.4.8 Design and Mitigation

Issue	Effect	Recommended Mitigation
High WSS or DS > 3	Sulphate attack on concrete	Use sulphate-resisting cement (CEM III/B), waterproof membrane, or lined blinding
Pyritic shale / fill	Oxidation heave	Encapsulate with inert fill + drainage; avoid exposure to air
Acidic ground (pH < 4.5)	Concrete / steel corrosion	Neutralise or isolate; pH > 5.5 required
High Cl^- (> 0.4 %)	Rebar corrosion	Increase cover or specify low-chloride concrete
Mixed fill (industrial)	Variable chemistry	Perform suite testing (WSS, ASS, Cl^-, pH) per layer
Groundwater sulphate	Migratory attack	Provide sub-base drainage / cut-off trench

8.4.9 Example Summary (Factual Report Extract)

Parameter	Lower Criteria	Upper Criteria	Result (Range)	Assessment
WSS (%)	0	1.9	0.12–0.34	PASS
Total S (%)	—	2.0	0.22	PASS
ASS (%)	—	0.03	0.00	PASS
pH	≥ 4.5	—	7.3–8.1	PASS
Cl^- (%)	—	0.4	0.02	PASS

✓ Ground non-aggressive → Design Sulphate Class DS-1 / ACEC AC-1.

8.4.10 Reporting Requirements

- Record sample ID, depth, and extraction ratio (2:1 vs 1:1).
- Include **WSS, ASS, S_t, pH, Cl^-** in Factual Report.
- Summarise by stratum in the **GDR**, assigning DS and ACEC classes with rationale.
- Comment on fill acceptability per SHW 600 and any material management implications.

Phicon Note:

• Always use **acid-soluble sulphide (ASS)** — it correctly captures pyritic potential; OS is obsolete.
• Combine **WSS + pH + groundwater SO_4^{2-}** for full aggressivity assessment.
• When uncertain between two DS classes, **adopt the higher** — increasing concrete class is cheaper than remedial works.
• Avoid **lime/cement stabilisation** in materials with detectable pyrite or high sulphate.
• Document laboratory certificates and interpretations in the **Geotechnical Design Report** — they form part of the permanent QA record for asset durability.

8.5 Collapsible and Loess Soils

8.5.1 Purpose

Collapsible soils are metastable, open-structured materials that maintain apparent strength when dry but undergo sudden volumetric collapse upon wetting or loading.

This behaviour leads to differential settlement, loss of bearing capacity, and structural distortion — particularly in arid or semi-arid deposits such as loess, silty drift, and certain aeolian or alluvial fills.

8.5.2 Mechanism of Collapse

Cause	Effect on Soil Fabric	Consequence
Wetting (rise in degree of saturation)	Reduction of suction and cementation bonds	Rapid pore collapse, volume loss
Loading (increase in σ')	Particle rearrangement under reduced interparticle stress	Settlement under constant effective stress
Combination (wetting + load)	Loss of metastable structure	Catastrophic differential movement

Collapse typically occurs between 1–5 m depth where infiltration or groundwater change affects loosely packed, silty soils.

8.5.3 Typical Occurrence

- **Loess deposits** (wind-blown silt; Central & Eastern Europe)
- **Alluvial / aeolian silts** with weak carbonate or clay bonding
- **Old industrial fills** with desiccated fines
- **Tropical residual soils** with cemented microstructure
- Unsaturated granular fills compacted dry of optimum

8.5.4 Identification

Indicator	Test / Reference	Criterion
Dry density (ρ_d)	BS EN ISO 17892-2	< 1.6 Mg/m^3 → potentially collapsible
Moisture content (w)	BS 1377-2	< OMC – 2 % → metastable
Atterberg limits	BS 1377-2	PI < 12 → silt-dominated
Collapse potential (CP)	Double-oedometer test (ASTM D5333)	CP > 2 % → collapse risk
Structure / fabric	Thin-section / SEM	Open, honeycomb, calcareous bonding
Field evidence	Sinkage, cracked slabs post-wetting	Confirmation of active collapse

8.5.5 Collapse Potential (ASTM D5333 / Modified Oedometer Test)

A sample is loaded twice — once at natural moisture, once after saturation — under identical stress.

$$\text{Collapse Potential (CP)} = \frac{\Delta h}{h_0} \times 100$$

CP (%)	Collapse Severity	Indicative Foundation Implication
< 1	None	Normal design
1–5	Slight	Check shallow foundations
5–10	Moderate	Pre-wet or compact
10–20	Severe	Ground improvement or deep foundation
> 20	Very severe	Avoid or excavate/replace

8.5.6 Eurocode 7 Design Implications

Aspect	Guidance / Check
Bearing capacity (GEO)	Verify using post-collapse parameters (c'_k, ϕ'_k, γ_{sat}). Reduce by factor reflecting collapse strain.
Settlement (SLS)	Include collapse strain ε_c = CP/100 × H_a. For CP > 5 %, consider mitigation.
Hydraulic change (UPL/HYD)	Assess effect of raised water table on effective stress and fabric stability.
Material acceptance	Avoid collapsible fills for engineered platforms unless pre-wetted and re-compacted.
Geotechnical category	Treat as Category 2 or 3 with observational follow-up.

8.5.7 Mitigation Measures

Method	Principle	Typical Application
Pre-wetting / inundation	Collapse soil under controlled conditions before construction	Shallow deposits (< 3 m)
Dynamic compaction	High-energy impact densifies and collapses loose structure	Loessic or sandy silt fills
Re-compaction (wet of OMC)	Eliminates voids, increases density	Platform or road formation
Replacement	Remove and backfill with engineered material	Thin lenses or isolated zones
Grouting (cement / chemical)	Binds structure, reduces permeability	Under slabs or local remediation
Deep foundations	Transfer loads below collapsible horizon	Thick or heterogeneous layers
Drainage control	Limit infiltration near structures	Long-term stability measure

8.5.8 Example Calculation

Given: CP = 6 %, depth of collapsible zone = 3 m

$$s_c = \frac{CP}{100} \times H = 0.06 \times 3.0 = 0.18\, m$$

Predicted collapse settlement ≈ 180 mm → unacceptable for shallow footing; pre-wetting or raft/pile required.

8.5.9 Typical Design Parameters (Post-Collapse)

Property	Before Wetting	After Wetting (Design)
Unit weight γ (kN/m^3)	17.0	18.5
ϕ' (°)	35	30
c' (kPa)	10	0
E (MPa)	40	15
k (m/s)	1×10^{-6}	5×10^{-6}

Use post-collapse values for ULS bearing and SLS settlement verification.

8.5.10 Construction & QA Notes

- Test bulk samples for CP wherever **silty, low-plasticity materials** are encountered.
- Maintain compaction **wet of OMC** to reduce susceptibility.
- Monitor post-construction settlement where pre-wetting has been employed.
- Provide drainage to **avoid long-term infiltration** from roof runoff or service leaks.
- Document collapse hazard in the **GDR Hazard Register** and update during construction.

Phicon Note:

• Collapse is a *wetting-induced structural failure* — it cannot be “designed away” without altering soil fabric.
• Always assess both **short-term bearing** and **long-term hydraulic** scenarios — raising groundwater often triggers delayed movement.
• Verify compaction moisture and density on engineered platforms; a “dry of optimum” placement is a red flag.
• Record CP data in the site register and integrate with the geotechnical model to flag potential *collapse zones* for monitoring or exclusion.

8.6 Mining, Voids and Sinkholes

8.6.1 Purpose

Natural or man-made underground voids — from **mining, dissolution, or buried infrastructure** — pose a risk of sudden **collapse or subsidence** at ground surface.

Early recognition and risk management are essential to prevent structural or geotechnical failure.

Common causes include:

- **Historical mining** (coal, chalk, ironstone, gypsum, salt)
- **Solution features** (karstic limestone, chalk, gypsum)
- **Natural cavities / swallow holes**
- **Man-made voids** (tunnels, culverts, cellars, buried tanks)
- **Backfilled shafts / wells**

8.6.2 Classification of Void Hazards

Origin	Mechanism	Example Locations (UK/EU)
Mining voids	Collapse of unsupported workings or shafts	Midlands, N. England, S. Wales coalfields
Solution features	Dissolution of soluble strata (chalk, limestone, gypsum)	Southern & Eastern England, Northern France, Rhineland
Natural karst / caves	Progressive roof failure due to erosion or drainage	Central Europe, Balkans
Man-made tunnels / basements	Ageing masonry or backfilled chambers	Urban brownfield areas
Collapsed services / utilities	Leakage undermining fine-grained soils	Common in older infrastructure

8.6.3 Geotechnical Risk Mechanism

Stage	Description	Impact
Void formation	Excavation, dissolution, or decay of buried material	Creates hidden cavity
Crown thinning	Progressive collapse of roof into overlying soils	Loss of support zone

Cavity migration	Arching failure moves upward (chimneying)	Surface settlement or sinkhole
Surface collapse	Sudden or progressive failure	Structural damage, loss of serviceability

⚠️ Risk increases where **low-density granular soils** overlie void-prone rock or fill.

8.6.4 Investigation Methods

Method	Application	Comment
Desk study	Review mining plans, BGS records, Coal Authority, historical maps	Identify known shafts / workings
Walkover survey	Evidence of depressions, subsidence, drainage anomalies	Early surface screening
Probing / drilling	Rotary open-hole with collapse watch; percussive probing	Detect voids to ~20 m depth
Geophysics (microgravity, resistivity, GPR, ERT)	Non-intrusive void mapping	High sensitivity to density anomalies
Downhole camera / sonar	Confirm void geometry	Requires safe access
Trial pits	Shallow verification only	Unsafe in suspected void zones

Combine multiple methods — correlation between geophysics and borehole control gives highest reliability.

8.6.5 UK-Specific Regulatory Framework

Authority / Document	Applies To	Notes
The Coal Authority	Coalfield zones (UK)	Provides mining reports, permissions, and stabilisation advice
BGS GeoIndex / BRITPITS	All mineral and quarry records	Free mapping of historical workings
NHBC Chapter 4.1	Housing development	Requires mining & void risk assessment
CIRIA C758D (Abandoned Mines and Cavities)	Engineering best practice	Investigation, treatment, and monitoring procedures

8.6.6 Eurocode 7 Implications

Aspect	Guidance / Requirement
Ground model	Identify voided zones as *geotechnical hazard layers* (GEO)
Design approach	Treat as Category 3 (requires detailed investigation & monitoring)

Bearing capacity (GEO ULS)	Apply reduction for voided strata; consider loss of bearing layer
Serviceability (SLS)	Allowable angular distortion < 1/500; consider sudden collapse mechanisms
Observational method	Justified for stabilisation works with ongoing monitoring (EN 1997-1 §2.7)

8.6.7 Risk Assessment Framework

Risk Level	Indicators	Typical Mitigation
Low	No recorded workings, stable strata	Standard design; no special measures
Moderate	Old shallow workings within 100 m	Investigation + local probing
High	Recorded mine entries / solution features < 30 m	Grouting, reinforcement, or piling
Very High	Known voids directly beneath site	Exclusion, capping, or major stabilisation works

⚠️ Apply a geotechnical risk register (GDR) including confidence level and residual hazard.

8.6.8 Mitigation and Treatment Measures

Technique	Principle	Typical Application
Pressure grouting	Fill and stabilise voids with cementitious grout	Small to moderate cavities
Compaction grouting	Densify and reinforce overlying soils	Over collapsed workings
Capping / bridging slabs	Reinforced raft distributing load	Light structures above shallow voids
Piled foundations	Transfer load below affected zone	Deep or uncertain void depth
Exclusion zones	Re-plan site layout	Where voids cannot be treated
Ground improvement	Stone columns or dynamic replacement	Non-mining, compressible infill zones
Drainage management	Prevent dissolution in soluble strata	Gypsum / chalk regions

8.6.9 Example Case

Scenario: Development in former coalfield with recorded shallow seam at 18 m depth.

- Rotary probe: 50 mm loss of flush return at 17.8 m (possible cavity).
- Grouting trial: 8 m^3/m injection confirms void infill.
- Verification: Post-treatment probe refusal at 16.9 m.
  ✓ Residual risk mitigated; adopt raft foundation with 25 kN/m^2 allowable bearing.

8.6.10 Design Verification and Monitoring

- **Pre-construction:** Borehole probing, geophysics, and risk classification.
- **During construction:** Supervised grouting and verification coring.
- **Post-construction:** Settlement monitoring pins, inclinometers, and drainage checks.
- **Reporting:** Include all records in the Geotechnical Design Report (GDR) and hazard register, with residual risk clearly stated.

Phicon Note:

• Treat all suspected void-prone sites as **Category 3** — failure consequences are often disproportionate.
• Always verify Coal Authority or local mining records; undocumented shafts are common.
• When using grouting, require **supervised volume and pressure logs** as evidence of infill efficiency.
• Maintain **perimeter drainage** to prevent solution reactivation in chalk or gypsum.
• Clearly define residual risk in the **Geotechnical Risk Register** — post-construction monitoring is integral to EC7 compliance.

PART 3: FOUNDATIONS & RETAINING STRUCTURES

9 Shallow Foundations: Bearing Capacity

9.1 Foundation Types and Typical Construction Methods

9.1.1 Purpose

Shallow foundations transfer structural loads to the ground at relatively small depths — typically $D_f < B$ (embedment less than foundation width).

They are used where competent strata occur within a few metres of the surface and where **settlement, bearing and stability** can be verified by conventional geotechnical design methods.

9.1.2 Common Types

Type	Typical Form / Use	Construction Method	Remarks / Design Features
Pad footing	Individual column support	Excavate local pit → blinded base → reinforcement cage → concrete pour	Economical for isolated loads; check punching and differential settlement
Strip footing	Line loads (walls, frames)	Trench excavation → level blinding → continuous concrete	Suitable for masonry or retaining walls; ensure uniform bearing
Combined footing	Two or more columns close together	RC pad spanning between columns	Used where column spacing prevents individual pads
Raft / mat foundation	Large footprint structure	Shallow excavation → uniform reinforced slab	Reduces differential settlement; redistributes load
Trench-fill footing	Deep narrow trench filled with concrete	Common in low-rise housing	Minimises labour; control shrinkage cracking
Eccentric / offset footing	Near boundaries or walls	Use tie-beam or strap footing	Prevents overturning from eccentricity
Blinding / levelling layer	Cement-sand layer 50–75 mm thick	Provides clean working surface	Not counted in bearing design

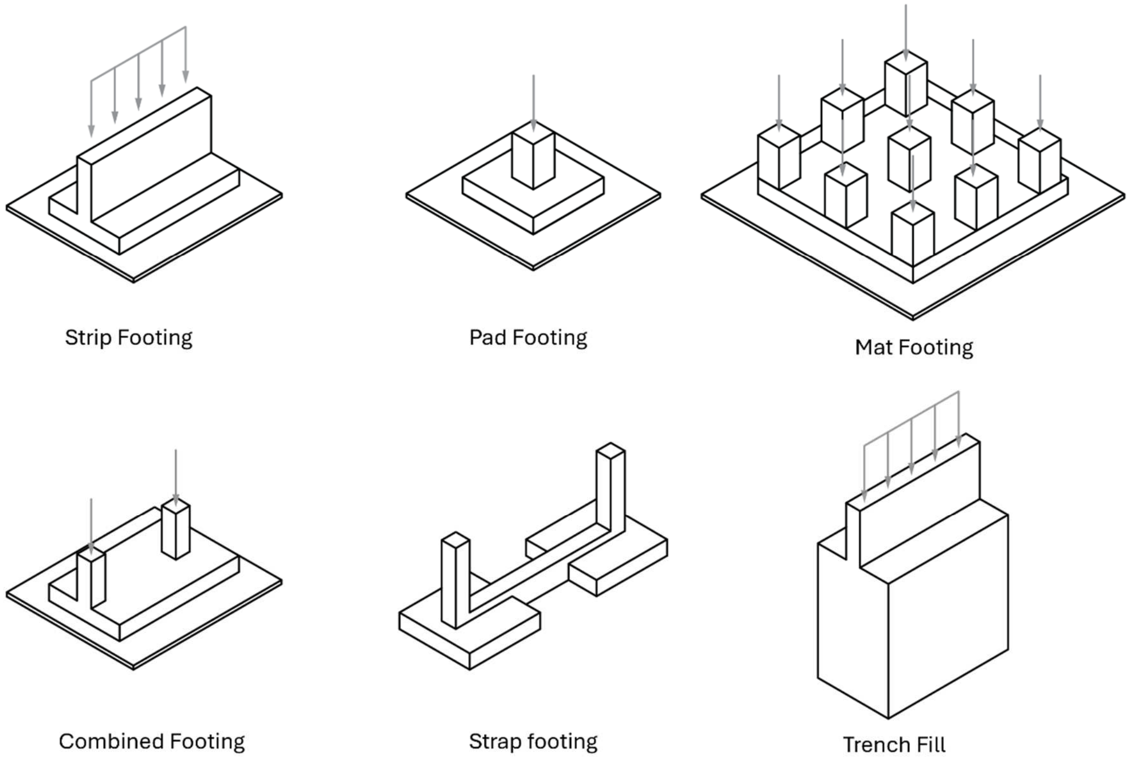


Figure 31: Common types of shallow foundations

9.1.3 Selection Considerations

Criterion	Influence on Type Selection
Load magnitude and spacing	Pad vs raft choice; close-spaced heavy loads favour rafts
Soil bearing capacity and stiffness	Weak or variable soils → raft or piles
Groundwater conditions	May require drained excavation, waterproofing or uplift checks
Differential settlement risk	Stiff or rafted foundations minimise angular distortion
Construction logistics	Access, plant size, programme and cost constraints
Environmental / reuse factors	Excavation arisings, carbon, and spoil re-use considerations

9.1.4 Design Framework (Eurocode 7 Context)

Verification	Limit State	Relevant Parameters	Typical Partial Factors (DA1-C1)
Bearing failure	ULS – GEO	c'_k, ϕ'_k, γ, D_f, B, L	γ_m = 1.25 (ϕ'), 1.25 (c')
Sliding / overturning	ULS – EQU	H_k, V_k, M_k, ϕ'_k	γ_r = 1.0 (resistance)
Structural adequacy	ULS – STR	Concrete & steel capacity	EN 1992 / EN 1993
Settlement / tilt	SLS	E_k, m_{vk}, Cc, D_f	γ_m = 1.0

⚠️ Shallow foundations typically fall under **Design Approach 1 (Combinations 1 & 2)** in the UK NA, with characteristic actions factored for ULS and unfactored for SLS.

9.1.5 Typical Dimensions and Embedment

Foundation Type	D_f (m)	B (m)	Notes
Pad footing	0.8–2.0	1.0–3.0	Below frost & desiccation depth
Strip footing	0.6–1.5	0.4–1.5	Uniform bearing soil required
Raft	0.5–1.5	—	Entire footprint loaded
Trench fill	0.9–2.5	0.3–0.6	Concrete up to ground level
Reinforced raft (industrial)	1.0–2.0	—	For variable or compressible soils

⚠️ **Minimum embedment depth (D_f)** to the top of foundation should exceed local frost depth (≈ 0.45 m UK) and account for seasonal moisture variation in shrink-swell clays (NHBC 4.2).

9.1.6 Construction QA / QC

Stage	Check / Control	Key Acceptance Criteria
Site Clearing	Debris removal, vegetation, compaction	Area clear of obstructions; sub-grade compacted to specification
Setting Out	Location, orientation, and level	Tolerance ±10 mm horizontally; levels checked against finished floor level (FFL) datum
Excavation	Depth, base level, stability	±25 mm level tolerance; base undisturbed
Bearing inspection	Proof test or visual confirmation	No soft pockets, water, or loose material
Blinding / bedding	50 mm cement–sand or lean mix	Provides clean, even surface
Formwork	Dimensions, verticality, bracing, internal cleanliness	Internal dimensions ±5 mm; Plumb ±3 mm per meter; adequate support against concrete pressure
Reinforcement placement	Cover & spacing	As per EN 1992 & drawings
Concrete casting	Continuous pour	Curing & surface finish verified
As-built record	Dimensions, bearing level, test results	Included in QA pack & GDR

9.1.7 Field Verification of Bearing Stratum

- **Dynamic probing / SPT:** Identify density or strength variability.
- **Plate load test (PLT):** Confirm deformation modulus (EV_2) ≥ 120–200 MPa for typical foundations.
- **Trial pits:** Direct visual inspection for soft lenses or fill interfaces.
- **Proof rolling:** Identify yielding areas in granular subgrade.
- **Excavation log:** Record strata sequence, depth, and bearing level for traceability.

9.1.8 Practical Construction Observations

- Avoid **over-digging**; backfilling weakens support.
- Excavate only immediately before concreting to prevent drying or slaking.

- Dewater as required — standing water reduces bearing capacity.
- Protect excavations against **frost and rainfall**.
- Verify **level and clean base** before casting — contamination layers invalidate design bearing assumptions.

9.1.9 Typical Load Ranges

Foundation Type	Typical Allowable Bearing Pressure	Indicative Ultimate Capacity (γ_m = 1.25)
Firm clay	100–200 kPa	250–400 kPa
Dense sand / gravel	200–400 kPa	500–900 kPa
Weak rock	500–2000 kPa	1–5 MPa
Engineered fill	100–300 kPa	250–600 kPa
Peat / organic	< 50 kPa	Not suitable

Phicon Note:

• Foundation design must integrate **site investigation data** with construction practicality — treat bearing strata confirmation as a live verification process.
• Always log **actual founding depth and material** for traceable compliance with EC7 §2.4.2.
• Avoid generic bearing values unless confirmed by field testing or published presumptive data (BS 8004 Table 2.1).
• For large or variable footprints, consider **raft or piled raft** to reduce differential settlement.
• Shallow foundations remain the most cost-effective form of support when soil conditions are predictable and QA is enforced.

9.2 Failure Modes (General, Local, Punching)

9.2.1 Purpose

The bearing capacity of a shallow foundation depends not only on soil strength but also on **the mode of failure mobilised beneath the footing**.

Recognising the correct failure mode is fundamental to selecting appropriate factors, applying correlations, and interpreting field tests.

9.2.2 Modes of Failure

Failure Mode	Typical Soil Behaviour	Characteristic Features	Engineering Implications
General shear failure	Dense sand / stiff clay	Distinct continuous shear surfaces; heaving of soil on both sides; sudden load drop in test	Well-defined peak; classical bearing capacity theory applies

Local shear failure	Medium-dense sand / firm clay	Partial shear zone; limited surface bulging; gradual load–settlement curve	Use **reduced ϕ' and c'** (approx. ϕ' × 0.8); lower bearing factor
Punching shear failure	Loose sand / soft clay	No surface heave; narrow shear zone under footing; progressive settlement without peak	Bearing governed by **soil compression** and settlement, not ultimate failure

9.2.3 Conceptual Failure Mechanisms

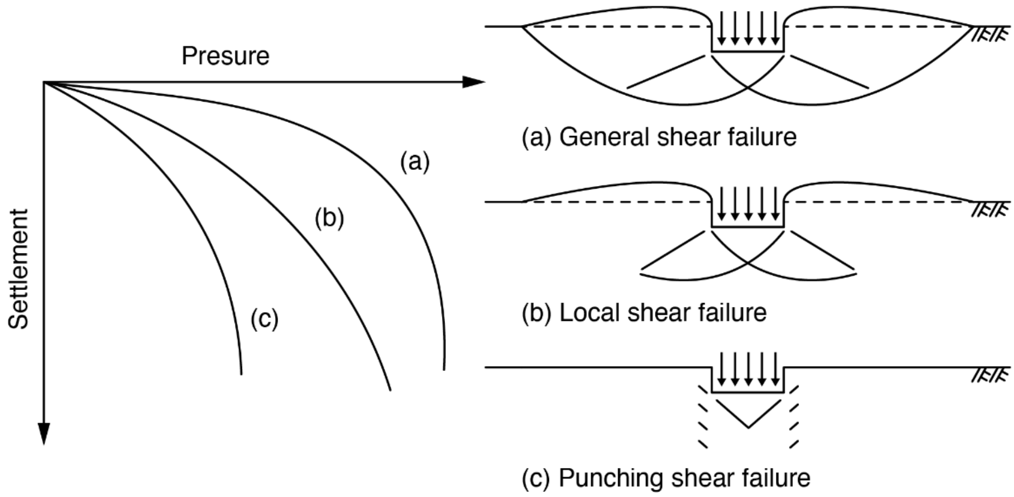


Figure 32: Failure mechanisms considered in bearing capacity

9.2.3.1 Practical Observations

- **General shear** is desirable — provides warning and ductility.
- **Local shear** common in granular fills or lightly over-consolidated soils; moderate safety factors appropriate.
- **Punching shear** frequent in soft clay or loose sand; must check settlement and stiffness, not just ultimate capacity.
- Field verification through **plate load, CPT, or SPT trends** helps confirm mode of failure.

9.2.4 Recognition in Field or Test Data

Indicator	General	Local	Punching
Plate Load Test (PLT) curve	Sharp peak and drop	Rounded, gradual peak	Continuous settlement, no peak
Settlement magnitude	< 10 % B at failure	10–20 % B	> 20 % B without failure
Surface observation	Edge heave, radial cracking	Slight bulging	None visible
Typical soils	Dense granular / stiff cohesive	Intermediate	Soft cohesive / loose granular

9.2.5 Design Implications (Eurocode 7)

- EC7 assumes **general shear failure** as the governing case for ultimate limit state (ULS) checks.
- Where local or punching shear is expected, the designer must **adjust shear parameters** conservatively to represent partial mobilisation.

Condition	Adjustment
Local shear	$\phi'_{local} = 0.8 \times \phi'$; $c'_{local} = 0.67 \times c'$
Punching behaviour	Treat as settlement-limited; check SLS first
Reinforced footing / raft	Shear governed by concrete capacity (STR)

9.2.6 Flowchart for Calculation of Bearing Capacity to EC7

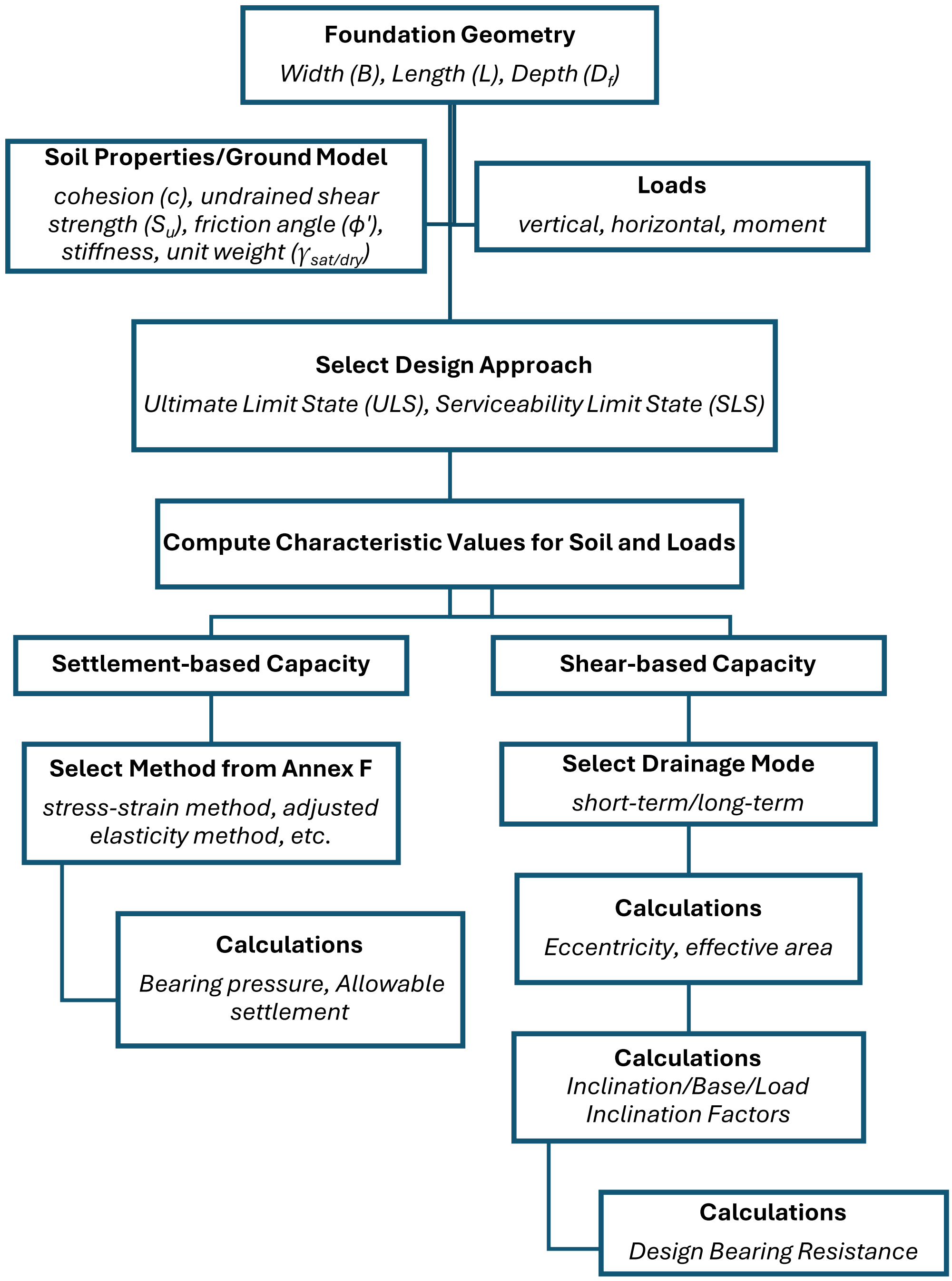


Figure 33: Flowchart for Calculation of Bearing Capacity to EC7

9.2.7 Bearing Pressure–Settlement Relationship

Typical load–settlement response:

1. **Elastic region:** Linear, small settlement (immediate).
2. **Plastic development:** Curved region, partial failure.
3. **Peak (general shear):** Rapid settlement increase.
4. **Post-peak (collapse):** Softening or punching.

For design, EC7 recommends using the **design value of bearing resistance** (R_d) at the **onset of rapid settlement**, not necessarily the full peak.

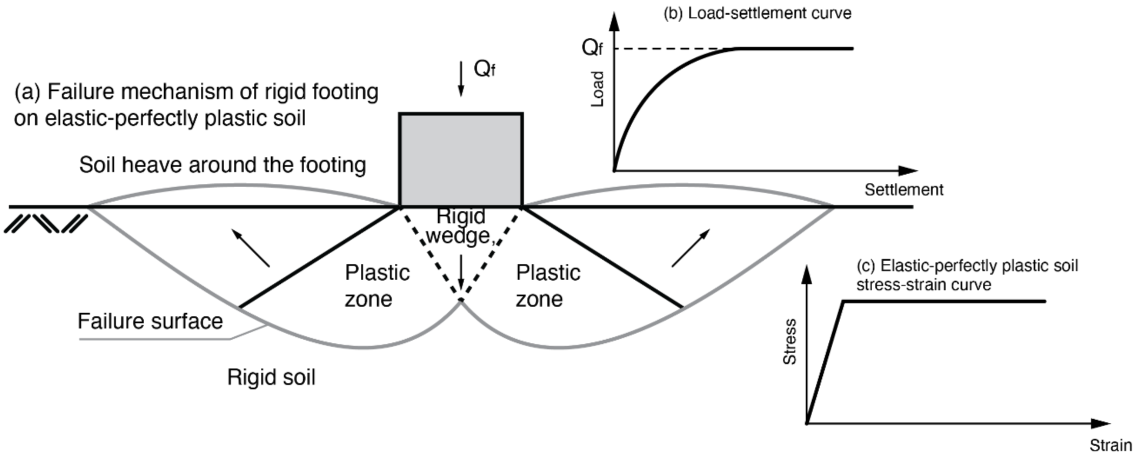


Figure 34: Failure mechanism of shallow foundation on elastic-perfectly plastic soil

9.2.8 Integration with Foundation Type

Foundation Type	Typical Failure Mode	Design Consideration
Pad / isolated footing	General → local	Use ϕ' reduction if settlement continues without peak
Strip footing	General	Width governs failure depth
Raft / mat	Punching under column zones	Check both overall bearing and local punching
Trench fill	Local	Narrow width limits lateral confinement
Combined / strap	Mixed	Analyse each loaded area individually

9.2.9 Typical Bearing Pressure at Failure

Soil Type	Expected Mode	Approx. Settlement at Failure (% B)
Dense sand	General	5–10 %
Medium-dense sand	Local	10–20 %
Loose sand	Punching	> 20 %
Stiff clay	General	6–10 %
Firm clay	Local	12–20 %

Soft clay	Punching	> 25 %

Phicon Note:

• **Identify the governing failure mode first** — it determines whether design is governed by ultimate strength or deformation.
• When uncertainty exists between local and general failure, adopt the **more conservative local parameters**.
• In design verification, ensure **ULS bearing** and **SLS settlement** are both satisfied — the governing limit state often depends on failure mode.
• Keep clear, annotated PLT or CPT evidence in the **GDR**, linking observed behaviour to design assumptions.

9.3 Bearing Capacity Equations (Terzaghi, Meyerhof, Hansen)

9.3.1 Purpose

Bearing capacity theory estimates the **ultimate load-carrying capacity** of soil beneath a shallow foundation before general shear failure occurs.

Classical formulations (Terzaghi → Meyerhof → Hansen → Vesic) remain the foundation of EC7-based design, with **partial factors applied to actions and material strengths** to obtain design resistance.

Term	Symbol	Description / Definition	Typical Use / Stage
Presumed Bearing Capacity	q_p	An approximate bearing capacity value adopted from experience, empirical correlations, or published tables (e.g., based on soil type or SPT values). Used as an initial estimate before detailed testing or analysis.	Preliminary design / feasibility stage
Ultimate Bearing Capacity	q_u	The theoretical maximum pressure that the ground can sustain without shear failure (collapse) of the soil beneath the foundation. Determined by analysis or test (e.g., Terzaghi, Meyerhof, or plate load test).	Derived from soil parameters at characteristic state
Net Ultimate Bearing Capacity	q_{nu}	The increase in bearing pressure that can be applied by the foundation above the existing overburden pressure: $q_{nu} = q_u$ - \γ D_f	Used to isolate the foundation-induced stress increment
Safe Bearing Capacity	q_s	The maximum pressure that can be applied to the soil without risk of shear failure, obtained by applying an appropriate factor of safety (typically 2.5–3): $q_s = q_u / F_s$	Traditional (pre-Eurocode) design approach
Allowable Bearing Capacity	q_{allow}	The lower of: (a) safe bearing capacity against shear failure and (b) pressure limited by permissible settlement criteria (usually 25 mm–50 mm for shallow foundations).	Serviceability check; used in allowable stress design

Characteristic Bearing Resistance	R_k	The representative value of bearing resistance derived from test data or calculations, such that an unfavourable deviation has a low probability of occurrence.	Input to Eurocode 7 limit-state design
Design Bearing Resistance	R_d	The factored (reduced) bearing resistance used in design, obtained by dividing the characteristic resistance by the material partial factor: $R_d=R_k/\gamma_R$	Final design value at ULS under EC7

9.3.2 General Expression

Ultimate bearing resistance at failure:

$$q_u = cN_c s_c d_c i_c + \gamma z N_q s_q d_q i_q + 0.5\,\gamma B\, N_\gamma s_\gamma d_\gamma i_\gamma$$

where:

- *c*: Cohesion
- *γ*: Effective unit weight
- *B*: Foundation width
- N_c, N_q, N_γ: Bearing capacity factors (function of ϕ')
- *s, d, i:* Shape, depth, and inclination factors (≤ 1.0 for unfavourable loading)

9.3.3 Classical Formulations

Author	Applicable Soils	Bearing Factors	Notes
Terzaghi (1943)	General shear failure in strip footing	$N_q = e^{\pi tan\varphi'} tan^2(45 + \varphi'/2)$; $N_c = (N_q – 1)/tan\varphi'$; $N_\gamma = 2(N_q + 1)tan\varphi'$	Valid for ϕ' ≤ 40°, $D_f/B \le 1$
Meyerhof (1951–63)	Extension for other shapes & depths	Adds s, d, i factors; increases N_γ for wide footings	Basis for most design charts
Hansen (1970)	Unified approach	Introduces load inclination, base, and ground-slope corrections	Commonly used for EC7 adaptation
Vesic (1973)	Refinement for ϕ' > 30°	Smooth N_γ curve; widely used in codes	Default for many modern tools

9.3.4 Typical Bearing Capacity Factors

φ' (°)	N_q	N_c	N_γ
0	1.0	5.7	0
20	6.4	17.7	5
25	12.7	25.1	9
30	18.4	30.1	19
35	41.4	57.8	42
40	81.3	100.4	80

9.3.5 Shape Factors (Meyerhof)

Foundation Type	s_c	s_q	s_γ
Strip (L ≫ B)	1.0	1.0	1.0
Rectangular	$1 + 0.2B/L$	$1 + 0.1B/L$	1–$0.4B/L$
Circular	1.3	1.2	0.6
Square	1.3	1.2	0.8

9.3.6 Depth Factors

Condition	d_c	d_q	d_γ
$D_f/B \leq 1$	$1 + 0.2D_f/B$	$1 + 0.1D_f/B$	1.0
$D_f/B > 1$	$1 + 0.2tan\varphi'(D_f/B)$	$1 + 0.1tan\varphi'(D_f/B)$	1.0

9.3.7 Inclination Factors (Hansen)

For load inclination angle α (H/V):

$$i_q = (1 - \alpha/90°)^2$$

$$i_c = i_q - (1 - i_q)\frac{tan\varphi'}{N_q - 1}$$

$$i_\gamma = (1 - \alpha/90°)^2$$

Simplified:

- For vertical load: $i = 1.0$
- For **10° inclination:** reduce q_u by ≈ 10–15 %

9.3.8 Example – Strip foundation (ϕ' = 30°, c_u = 60 kPa)

Considering: $\gamma' = 19\ kN/m^3, D_f = 1.0\ m, B = 2.0\ m$

a) Short-term (undrained)

$$q_u = c_u N_c s_c d_c + \gamma z N_q s_q d_q + 0.5\,\gamma' B\, N_\gamma s_\gamma d_\gamma$$

$$d_c = 1 + 0.2 \times \frac{1}{2} = 1.1, \qquad d_q = 1 + 0.1 \times \frac{1}{2} = 1.05, \qquad d_\gamma = 1.0$$

$$q_u = (60 \times 5.7\ \times 1\ \times 1.1) + (19\ \times 1 \times 1 \times 1 \times 1.05) + (0.5 \times 19 \times 2 \times 0 \times 1 \times 1)$$

$$= 376.2 + 19.95\ + 0 = 396\ kPa$$

b) Long-term (drained):

$$q_u = c N_c s_c d_c + \gamma z N_q s_q d_q + 0.5\,\gamma' B\, N_\gamma s_\gamma d_\gamma$$

$$d_c = 1.1, \qquad d_q = 1.05, \qquad d_\gamma = 1.0$$

$$q_u = (0 \times 30.1\ \times 1\ \times 1.1) + (19\ \times 1 \times 18.4 \times 1 \times 1.05) + (0.5 \times 19 \times 2 \times 19 \times 1 \times 1)$$

$$= 0 + 367.1\ + 361 = 728\ kPa$$

✓ Governing (undrained) ultimate capacity ≈ 396 kPa (characteristic).

9.3.9 Eurocode 7 Application

Under Design Approach 1 (UK NA):

$$R_d = \frac{R_k}{\gamma_R}$$
$$R_k = q_u \times B \times L$$

with partial factors applied on **actions** or **soil parameters** per combination:

- **C1:** Actions factored (γ_F = 1.35 / 1.5), materials unfactored.
- **C2:** Materials factored (γ_M = 1.25 for ϕ', 1.25 for c'), actions unfactored.

Verification:

$$E_d \le R_d$$

9.3.10 Practical Notes

- Use **effective stress parameters** for drained (long-term) conditions; total stress (c_u) for short-term.

- Limit N_γ for ϕ' > 40° — classical equations overestimate resistance.
- For **square and circular** pads, apply shape factors cautiously — over-conservatism preferred for variable soils.
- For **strip footings in layered ground**, evaluate controlling stratum or apply weighted average.
- Verify results against field **plate load test (PLT)** where available.

Phicon Note:

• The governing design depends on *how ϕ' is mobilised* — dense sands rarely reach full Nγ before settlement governs.
• Always include **depth, shape, and inclination corrections** explicitly in calculations — do not embed them into empirical "bearing pressure" values.
• For mixed clay–sand profiles, analyse both **total and effective stress conditions** and adopt the lower design value.
• Cross-check analytical results with **presumptive values (BS 8004 Table 2.1)** and, where feasible, **PLT back-analysis** for QA consistency.

9.4 c–ϕ Soils and Groundwater Influence

9.4.1 Purpose

Many natural soils exhibit both **cohesive and frictional strength components**, expressed as **c–ϕ soils**.

The presence of **groundwater** modifies the effective stress and consequently reduces the available shear strength and ultimate bearing capacity.

Correct treatment of both aspects is critical to reliable design.

9.4.2 Combined Cohesion–Friction Behaviour

For general shear failure under drained conditions:

$$q_u = c' N_c s_c d_c i_c + q' N_q s_q d_q i_q + 0.5\gamma' B N_\gamma s_\gamma d_\gamma i_\gamma$$

Where both **c'** and **ϕ'** are non-zero, all three terms contribute:

- **Cohesion term:** Dominates in cohesive (clayey) soils or small footings.
- **Friction term:** Governs dense sands or large footings.
- **Unit weight term:** Important in wide footings or deep embedment.

The *transition from a combined c–ϕ behaviour (undrained) to pure frictional (drained)* is gradual; parameter selection should reflect likely drainage and rate of loading.

9.4.3 Effective vs Total Stress Approach

Condition	Design Parameters	Typical Application	Notes
Drained (effective stress)	c', ϕ', γ'	Long-term bearing, granular or stiff clays	Consolidation complete; pore pressures dissipated
Undrained (total stress)	c_u, $\phi = 0$	Short-term loading, saturated clays	Rapid loading; excess pore pressures
Partially drained / c–ϕ	Both c' and ϕ'	Intermediate states, layered or fissured soils	Often relevant near groundwater level or in partially saturated materials

⚠️ Always verify which condition governs each design stage — long-term (drained) or short-term (undrained).

9.4.4 Groundwater Position and Effective Stress

Effective overburden pressure:

$$q' = \gamma' D_f = (\gamma_{\text{soil}} - \gamma_w) D_f$$

If groundwater rises into the bearing zone, effective stress and bearing resistance decrease approximately in proportion to the **submerged unit weight**.

Groundwater Condition	Effective Unit Weight γ' (kN/m^3)	Effect on Bearing Capacity
Water table well below foundation	$\gamma' = \gamma$	No reduction
Water table at foundation base	$\gamma' = \gamma - \gamma_w$	Typically 40–50 % reduction
Water table above base (surcharge submerged)	Use γ' for both q' and γ terms	Major reduction; re-check stability and settlement

When the water table fluctuates seasonally, adopt the **highest expected groundwater level** in design.

9.4.5 Simplified Correction for Groundwater

For ϕ' soils ($c' = 0$) (drained):

$$q_u = q' N_q + 0.5 \gamma' B N_\gamma$$

where
$q' = \gamma' D_f$ and $\gamma' = \gamma - \gamma_w$.

Typical reduction factors:

Water Table Depth Below Base (B)	Reduction Factor (R_q)
≥ 1 × B	1.0
0.5 × B	0.75
At base (0 × B)	0.5

These empirical factors can be applied to the total bearing resistance to account for partial submergence.

9.4.6 Example – c–ϕ Soil with Shallow Groundwater

Given:

- $\varphi' = 28°, \; c' = 10\, kPa, \gamma' = 19\, kN/m^3, D_f = 1.0\, m, B = 2.0\, m$
- Water table at foundation base → $\gamma' = (19 - 9.81) = 9.2\, kN/m^3$

From charts:

$$N_q = 14.8, N_c = 25.1, N_\gamma = 12.3$$

$$q_u = (10 \times 25.1) + (9.2 \times 1.0 \times 14.8) + (0.5 \times 9.2 \times 2.0 \times 12.3)$$
$$q_u = 251 + 136 + 113 = 500\, kPa$$

✓ *Ultimate (characteristic) bearing capacity = 500 kPa*
If water table were **well below** the base, $\gamma' = 19$→ $q_u \approx 750\, kPa$.
Thus, **bearing reduced by ~33%** due to groundwater.

9.4.7 Influence of Cohesion in c–ϕ Soils

Soil Type	Typical c' (kPa)	Typical ϕ' (°)	Behavioural Note
Stiff clay (overconsolidated)	5–20	25–30	ϕ' term dominates at large B
Firm clay (normally consolidated)	0–10	20–26	Mixed c–ϕ response
Sandy clay / clayey sand	0–15	28–34	Frictional term primary
Glacial till / mixed soil	10–40	20–35	High apparent cohesion from suction or cementation

Apparent cohesion may diminish upon saturation — apply lower bound c' in design.

9.4.8 Pore Pressure and Effective Stress in Design

For groundwater near the foundation level:

- Use **pore pressure ($u = \gamma_w h$)** to reduce total stresses → $\sigma' = \sigma - u$.
- Recalculate effective surcharge and bearing terms with γ'.

- In soft clays or fine sands, rising water levels can also **reduce stiffness**, increasing settlement.

⚠️ EC7 requires **verification under adverse pore pressure conditions** (e.g. following heavy rainfall or dewatering rebound).

9.4.9 Typical Design Checks

Check	Parameter Basis	Notes
ULS – GEO	c'_k, ϕ'_k (with γ_M = 1.25) and γ'	Bearing capacity reduction due to water table
SLS – Settlement	E_k, m_{vk} (adjusted for saturation)	Settlements increase with rising groundwater
UPL – Uplift / buoyancy	γ_w, i	Verify stability where D_f < groundwater level
HYD – Heave	γ_w, permeability (k)	Check basal stability for excavations

9.4.10 Practical Design Tips

- For **strip or pad foundations**, check both **dry and submerged cases** — groundwater fluctuation often controls.
- When **c'** is small (≤ 10 kPa), treat soil as ϕ'-only for simplicity; minor cohesion offers limited benefit.
- Apply **drainage layers** or cut-offs to control seepage and maintain dry working conditions during construction.
- Record **groundwater observations** in boreholes during site investigation — not just static levels but also response during drilling.
- Consider **transient effects** — e.g. dewatering rebound, seasonal high water table, or perched layers.

Phicon Note:

• Groundwater reduces both effective stress and strength — treat the **highest credible groundwater level** as the design case.
• For mixed c–ϕ soils, adopt the **lower-bound combination** of c' and ϕ' consistent with field data and saturation.
• Apply **γ' instead of γ** systematically across surcharge and unit weight terms — many design errors stem from partial adjustment.
• Include a note in the **Geotechnical Design Report (GDR)** describing groundwater influence assumptions; auditors often trace bearing discrepancies back to this step.
• Always check for **long-term uplift or heave** where dewatering or backfilling alters hydrostatic equilibrium.

9.5 Eccentric and Inclined Loading

9.5.1 Purpose

Foundations rarely carry purely vertical loads.
Eccentric or inclined loading occurs from:

- **Horizontal forces** (wind, earth pressure, crane surge, traffic braking)
- **Moments** from frame action or eccentric column positioning
- **Uneven bearing** under differential stiffness or geometry

The result is a **non-uniform stress distribution** beneath the footing, reducing the effective area and increasing the risk of rotation or sliding.

9.5.2 Definitions

Symbol	Description	Units
$\boldsymbol{V}$	Vertical design load	kN
$\boldsymbol{H}$	Horizontal load (at base level)	kN
$\boldsymbol{M}$	Moment about foundation centroid	kN·m
$\boldsymbol{B}, \boldsymbol{L}$	Foundation width & length	m
$\boldsymbol{e_x}, \boldsymbol{e_y}$	Load eccentricities in plan	m
$\boldsymbol{q_{max}}, \boldsymbol{q_{min}}$	Maximum / minimum contact pressures	kPa

9.5.3 Eccentricity and Effective Area Concept

Resultant load not passing through the centroid → **eccentric bearing**.

$$e_x = \frac{M_y}{V}, e_y = \frac{M_x}{V}$$

If $e < B/6$ and $e < L/6$:
Full contact maintained (no uplift).

Effective dimensions:

$$B' = B - 2e_x, L' = L - 2e_y$$

Effective area:

$$A' = B'L'$$

Design bearing stress:

$$q_d = \frac{V}{A'}$$

When $e > B/6$ or $L/6$, part of the base is in tension — uplift or rocking may occur, and stability must be checked under the **EQU limit state**.

9.5.4 Inclined Loading

Inclined loads reduce bearing resistance due to partial mobilisation of shear strength. The reduction is quantified using load inclination factors which multiply the respective terms in the general bearing capacity equation. J. Brinch Hansen's (1970) inclination factors are included in general bearing capacity equation:

$$i_q = (1 - \frac{H}{V})^2, i_c = i_q - (1 - i_q)\frac{\tan\ \varphi'}{N_q - 1}, i_\gamma = (1 - \frac{H}{V})^2$$

For small H/V < 0.2, $i \approx$ 1–2(H/V) gives a close approximation.

9.5.5 Modified Bearing Capacity for Eccentric and Inclined Loads

$$q_u = c'N_c s_c d_c i_c + q'N_q s_q d_q i_q + 0.5\gamma'B'N_\gamma s_\gamma d_\gamma i_\gamma$$

Use reduced width B' (and length L') and appropriate inclination factors i_x.

EC7 requires that **the design value of applied action effect** $E_d = V_d, H_d, M_d$ is compared with **the design resistance** R_d using these reduced parameters.

9.5.6 Bearing Pressure Distribution

For rectangular footing under vertical + moment load:

$$q_{max,min} = \frac{V}{BL}(1 \pm \frac{6e}{B})$$

- **Uniform pressure** → e = 0
- **Zero pressure at edge** → e = B/6
- **Partial contact** → e > B/6 → triangular pressure block only

Limit for serviceability:

$$\frac{q_{max}}{q_{min}} \leq 2.0$$

(to maintain tolerable angular distortion).

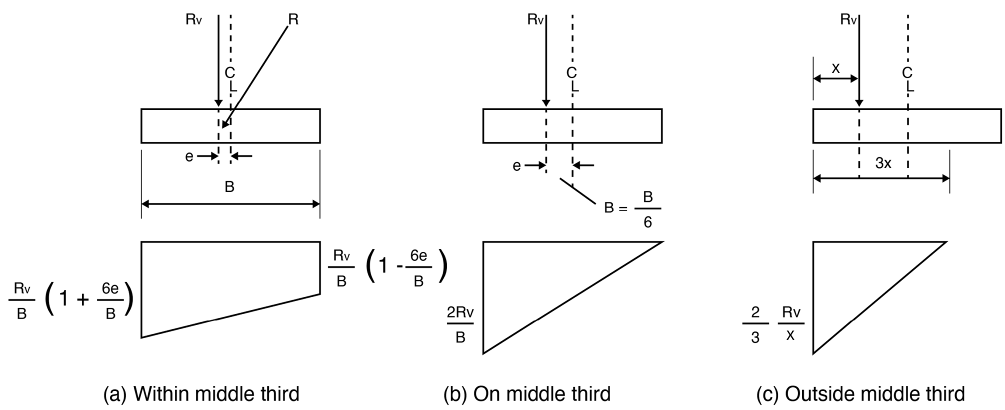


Figure 35: Bearing pressure distributions under high moment loading (reproduced from Smiths 2010)

9.5.7 Worked Example

Given:
Pad footing B = 2.0 m, L = 2.5 m; V = 600 kN; M = 100 kN·m about L-axis; ϕ' = 30°; γ' = 19 kN/m^3; D_f = 1.0 m

$$e = \frac{M}{V} = \frac{100}{600} = 0.167m < \frac{B}{6} = (0.33m) \rightarrow OK$$
$$B' = 2 - 2 \times 0.167 = 1.67\ m$$
$$q_d = \frac{600}{1.67 \times 2.5} = 144\ kPa$$

Bearing capacity (Meyerhof, ϕ' = 30°, D_f = 1 m):
$$q_u = q'N_q + 0.5\gamma'B'N_\gamma = 349 + 300 = 649\ kPa$$

Safety ratio = $649/144 = 4.5 > \ 3.0$→ **adequate.**

9.5.8 Eurocode 7 Verification Summary

Limit State	Primary Check	Key Partial Factors	Typical Acceptance Criteria
GEO (ULS)	Bearing failure	γ_m = 1.25 (ϕ', c')	$E_d \leq R_d$
EQU (ULS)	Overturning / uplift	γ_G = 1.35, γ_Q = 1.5	$\Sigma M_{res} \geq 1.5\Sigma M_{act}$
SLS	Rotation / tilt	γ = 1.0	θ ≤ 1/500–1/300

9.5.9 Practical Recommendations

- Keep $e \leq B/6$ and $H/V \leq 0.2$ where possible.
- Provide **tie beams** or combined footings to counterbalance eccentric loads.
- For crane bases or retaining walls, **include transient load cases** (wind, braking).
- Verify **sliding** with:

$$H_d \leq (V_d tan\delta + c'B'L')$$

where δ ≈ (2/3 ϕ').

- In overturning checks, ensure resultant remains within the **kern** (e<B/6).
- Record **eccentricities and applied moments** in the GDR — they directly affect bearing verification.

Phicon Note:

• Eccentric or inclined load design is fundamentally a **geometry correction problem** — focus on locating the resultant accurately.
• For transient conditions (wind, seismic, equipment), check both **instantaneous ULS** and **long-term SLS**.
• Where moments are large, prefer **rafts or combined footings** — they distribute stresses efficiently and simplify EC7 verification.
• Keep **field as-built dimensions** recorded; minor deviations in **B** and **e** can change bearing pressures significantly.

9.6 Rock and Weak Rock Bearing Capacity

9.6.1 Purpose

Foundations on rock generally achieve **high bearing resistance** and **minimal settlement**, but their performance depends on:

- The **quality and continuity** of the rock mass,
- The **spacing and condition** of joints or bedding planes, and
- The **weathering, infilling, and groundwater** conditions at the bearing interface.

⚠️ Eurocode 7 recognises that rock design differs fundamentally from soil design — discontinuities control behaviour rather than the intact material alone.

9.6.2 Governing Mechanisms

Mechanism	Description	Design Implication
Intact rock crushing	Exceeding compressive strength at contact	Rare except under concentrated loads
Block sliding / toppling	Movement along inclined joints or bedding planes	Check joint orientation and friction
Differential settlement	Variable weathering or infilled joints	Governed by deformation modulus
Punching into weak rock	Thin crust or soft weathered band	Apply reduced bearing value or raft
Rock mass dilation	Opening of discontinuities under stress	Typically minor for small foundations

9.6.3 Rock Strength Classification (EN ISO 14689-1)

Class	Description	Unconfined Compressive Strength (UCS, MPa)	Typical Examples
R1	Very strong	> 100	Basalt, granite, limestone
R2	Strong	50–100	Dense sandstone, dolerite
R3	Moderately strong	25–50	Cemented sandstone, mudstone
R4	Weak	5–25	Weak sandstone, marl, weathered shale
R5	Very weak	1–5	Highly weathered mudstone, chalk
R6	Extremely weak	< 1	Claystone, very soft chalk, cemented soil

R4–R6 classes behave more like stiff or hard soils and require soil-type bearing checks.

9.6.4 Bearing Capacity Estimation Methods

9.6.4.1 UCS-Based Correlation

Empirical relationships between UCS and allowable bearing pressure (q_a):

$$q_{allow} \approx (0.2\text{–}0.4) \times \text{UCS}$$

$$\text{or } q_{allow} = \frac{\text{UCS}}{3\text{–}5}$$

Rock Class	UCS (MPa)	Allowable Bearing (kPa)	Design Comment
R1	> 100	> 10,000	Settlement usually governs
R2	50–100	5,000–10,000	Verify bedding orientation
R3	25–50	2,000–5,000	Adequate for most structures
R4	5–25	500–2,000	Treat as weak rock / hard soil
R5	1–5	100–500	Soil bearing checks apply
R6	< 1	< 100	Treat as clay or chalk

Apply partial factors on **resistance (γ_R = 1.0)** and **material (γ_M = 1.25)** as per EN 1997-1 §2.4.7.3.

9.6.4.2 Rock Mass Rating (RMR) or Q-System Correlations

RMR Class	Rock Quality	Approx. q_a (kPa)	Foundation Comment
> 80	Very good	10,000–20,000	Rock-like; settlement negligible
60–80	Good	5,000–10,000	Direct bearing feasible
40–60	Fair	2,000–5,000	Check joint spacing, infill
20–40	Poor	500–2,000	Requires improvement / grouting
< 20	Very poor	< 500	Treat as weak rock or soil

9.6.5 Discontinuities and Bearing Failure

Joint Condition	Parameter	Design Guidance
Rough, clean	$\delta \approx 0.8\ \phi'_{rock}$	Good frictional resistance
Smooth, planar	$\delta \approx 0.5\ \phi'_{rock}$	Risk of sliding; provide key or dowels
Clay-filled	$\delta \approx 0.3\ \phi'_{rock}$ or 10–15°	Use shear strength of infill
Open joints	Reduce area proportionally	Possible local punching
Inclined strata	Check dip vs load direction	Stability may control design

If bedding dips toward the loaded edge, **verify block sliding**:

Block sliding check (bedding dipping toward the loaded edge)

$$F_s = \frac{W\cos\,\alpha(\tan\,\delta + \frac{c_j}{W})}{W\sin\,\alpha} = \cot\,\alpha(\tan\,\delta + \frac{c_j}{W}) \geq 1.5$$

Where;

- α= bedding dip toward the loaded edge (radians or degrees),
- δ= joint/interface friction angle,
- c_j= cohesive **force** acting along the joint (N).
  *If you have cohesion as a **stress** c_a(kPa) over a contact area A(m^2), use $c_j = c_a A$.*
- W= vertical load (N).

Notes (concise):

- Valid for an **inclined base**; if $\alpha = 0$ use the standard form $F_s = (V\tan\ \delta + c_a A)/H$.
- Use **design values** under EC7 (i.e., factored W, δ_d, $c_{a,d}$) if documenting to the code.
- The target $F_s \geq 1.5$ is a conventional limit-state factor; align with project/NA requirements.

9.6.6 Settlement Behaviour of Rock Foundations

Settlement usually small (1–5 mm), but can be higher in **weak or fissured rock**.
Approximate deformation modulus (E):

Rock Type	E (MPa)	Expected Settlement
Very strong (R1–R2)	5,000–20,000	< 1 mm
Moderately strong (R3)	1,000–5,000	1–5 mm
Weak (R4)	200–1,000	5–15 mm
Very weak (R5–R6)	50–200	10–50 mm

Settlement verification per EC7 §2.4.8 (SLS) should use

$$s = \frac{qB(1 - \nu^2)}{E}$$

with Poisson's ratio $\nu = 0.2$–0.3.

9.6.7 Weak Rock and Chalk Behaviour

Material	Design Approach	Notes
Chalk (Grade D–E)	Treat as very weak rock or stiff soil; use ϕ' = 30°, c' = 0–10 kPa	Allowable q_a 100–400 kPa
Weathered mudstone / marl	Use c–ϕ soil parameters with partial factors	Avoid soft bands at interface
Cemented tills / weak sandstone	Use UCS or PLT correlation	Verify by field load testing

Highly fractured strata	Base design on RMR ≤ 40 → q_a ≤ 2000 kPa	Settlement and block failure dominate

9.6.8 Groundwater and Weathering

- **Groundwater flow** along joints can reduce effective cohesion (c') and soften infill materials.
- **Differential weathering** can lead to stepped bearing surfaces; maintain minimum **bench depth ≥ 0.3 m** into sound rock.
- For **gypsum or evaporite rocks**, check for dissolution and cavity formation (see Section 8.6).

When water pressure acts along discontinuities, include it in **EQU** and **HYD** checks per EC7 §2.4.7.4.

9.6.9 Construction and QA Considerations

- Excavate to sound, unweathered rock; remove softened or infilled seams.
- Clean bearing surface before concreting; use lean concrete blinding if irregular.
- Where bedding is inclined, step or key foundation into rock.
- Carry out **plate load tests (PLT)** for weak rocks (R4–R6) to verify deformation modulus and allowable pressure.
- Document rock class (R1–R6), weathering grade, discontinuity condition, and UCS data in the GDR.

9.6.10 Example Calculation

Given: Moderately strong sandstone (R3), UCS = 30 MPa, foundation 2.0 × 2.0 m.

$$q_{allow} = 0.3 \times 30 = 9\ MPa = 9000\ kPa$$

Apply EC7 reduction:

$$q_d = \frac{9000}{\gamma_M} = \frac{9000}{1.25} = 7200\ kPa$$

✓ Design bearing capacity = 7.2 MPa (ULS); settlement negligible (< 2 mm).

Phicon Note

• Always classify rock quality in the field — **bearing design is meaningless without an R or RMR designation**.
• The **UCS correlation method** is reliable for R1–R3 rocks; for R4–R6, switch to c–ϕ soil-based bearing checks.
• Incorporate **joint orientation and groundwater conditions** explicitly — most failures occur along pre-existing planes.
• Where uncertainty exists, verify by **plate load test** or **borehole shear testing**.
• Maintain a **rock bearing log** (depth, class, weathering, joint condition) within the GDR for traceability.

10 Shallow Foundations: Settlement

10.1 Allowable Movements and Differential Limits

10.1.1 Purpose

Settlement control is a **serviceability design requirement** under Eurocode 7.

The **ultimate limit state (ULS)** ensures bearing failure does not occur, whereas the **serviceability limit state (SLS)** ensures the structure remains functional and serviceable without undue distortion or cracking.

Excessive settlement or differential movement can cause:

- Structural cracking or tilting,
- Malfunction of doors, joints, or machinery,
- Leakage or service failure in tanks and pipelines,
- Visual or aesthetic damage.

10.1.2 Settlement Components

Type	Mechanism	Timescale	Applicable Soils
Immediate (elastic)	Elastic deformation of dry soil or undrained clay	Instantaneous	Sands, silts, stiff clays
Primary consolidation	Volume decrease from dissipation of excess pore pressure	Weeks to years	Saturated clays
Secondary compression (creep)	Continued deformation under constant load	Long-term (years)	Organic clays, peats, fills

In granular soils, settlements occur rapidly; in fine-grained cohesive soils, they may continue for years after construction.

10.1.3 Total vs Differential Settlement

Term	Definition	Typical Structural Consequence
Total settlement (s_t)	Overall vertical movement of a foundation	Affects elevation and alignment
Differential settlement (Δs)	Difference between settlements of adjacent foundations	Causes tilting and cracking
Angular distortion (β)	Ratio Δs / L (where L = spacing between foundations)	Main serviceability measure in EC7

10.1.4 Eurocode 7 Framework

EN 1997-1 §2.4.8 requires verification that:

$$s_d \leq s_{lim}$$

Where:

- s_d: calculated design settlement
- s_{lim}: limiting or allowable settlement for the structure

Design values are determined using **characteristic stiffness parameters** (E_k, m_{vk}) with **γ_m = 1.0** (no partial factor at SLS).

10.1.5 Typical Allowable Settlements (s_{lim})

Structure Type	Maximum Total Settlement (mm)	Maximum Differential Settlement (mm/m)	Reference
Heavy industrial / framed buildings	50–75	1/500	BS 8004, Burland (1995)
Offices, commercial buildings	25–50	1/500–1/400	EN 1997 commentary
Brickwork / masonry	25–40	1/500–1/300	CIRIA C580
Machine foundations	≤ 10	≤ 1/1000	Manufacturer's criteria
Water tanks, silos	25	1/1000	API 650 / BS 8007
Pavements / slabs	25–50	—	TRL 113, HA 74/07
Bridges (abutments, piers)	50–100	1/600–1/1000	DMRB CD 225
Residential housing	25	1/300	NHBC 4.2

⚠️ Angular distortion limits are generally more critical than absolute settlements.

10.1.6 Differential Settlement Criteria (Angular Distortion)

Burland & Wroth (1975) proposed limits based on building sensitivity:

Structure Type	Angular Distortion (Δs/L)	Typical Effect / Remarks
Precision / Laboratory Buildings	≤ 1/1000–1/1500	Very stringent; prevents misalignment of sensitive or calibrated equipment.
Framed Buildings (Steel or RC)	≤ 1/500	Tolerable for most framed structures; minor cracking in finishes possible.

Structure Type	Angular Distortion (Δs/L)	Typical Effect / Remarks
Masonry or Brittle Structures	≤ 1/750–1/1000	Limits stepped cracking in walls and finishes; suitable for heritage or load-bearing masonry.
Commercial / Residential Buildings	≤ 1/500–1/750	Acceptable for typical office or housing structures; minimal visible cracking.
Industrial / Warehouse Buildings	≤ 1/400–1/500	Robust finishes; minor cosmetic cracking acceptable.
Bridges and Long-Span Structures	≤ 1/600–1/750	Controls joint and bearing misalignment; maintains serviceability of deck and parapets.

10.1.7 Foundation Spacing and Group Effects

Differential movements depend on **relative foundation stiffness** and **spacing**.

Adjacent footings founded on similar strata exhibit smaller Δs due to stress overlap.

Spacing Ratio (S/B)	Interaction Factor (α)	Comment
> 4	0 (independent)	Footings behave separately
2–4	0.3–0.5	Moderate interaction
1–2	0.5–0.8	Partial overlap of stress zones
< 1	0.8–1.0	Behave as raft or group

$$\Delta s = \alpha(s_1 - s_2)$$

For raft foundations, consider **global deflection** rather than individual settlements.

10.1.8 Long-Term Considerations

- **Secondary compression** may add 10–20 % to primary settlement in cohesive soils.
- **Groundwater rise** can reduce effective stress and increase settlement.
- **Seasonal desiccation / heave** affects shrink–swell clays — refer to NHBC 4.2.
- For **fills or soft alluvium**, settlements may continue for years unless preloaded or improved.

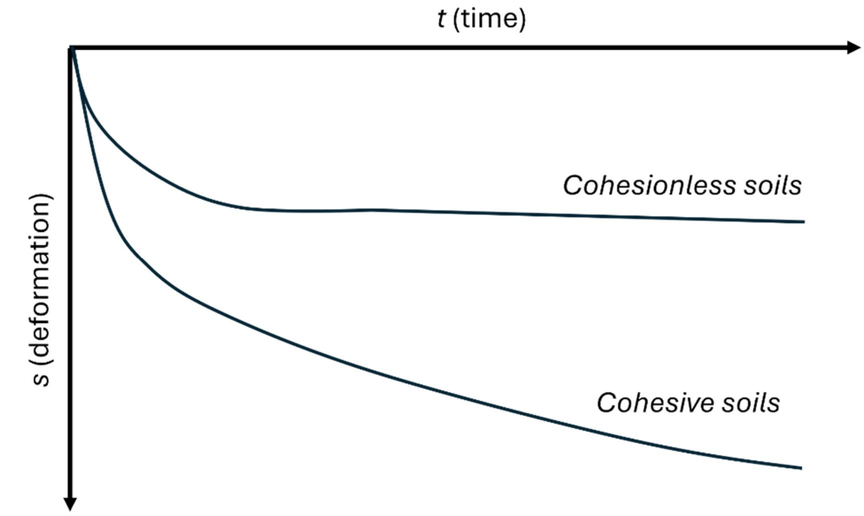


Figure 36: Schematic settlement curve for cohesive and cohesionless soils

10.1.9 Verification Procedure (Eurocode Practice)

1. **Determine characteristic stiffness** (E_k or m_{vk}) from lab or field tests.
2. **Compute total settlement** = immediate + consolidation (+ creep if relevant).
3. **Compute differential settlement** between adjacent footings.
4. **Compare with allowable limits** from §10.1.5–10.1.6.
5. **Document results** in GDR under *Serviceability Verification*.

Check: $s_{calc} \leq s_{lim}$ and $\beta_{calc} \leq \beta_{lim}$

10.1.10 Practical Guidance

- For **stiff clays and dense sands**, immediate settlement often governs; use elastic methods (§10.2).
- For **soft clays**, consolidation settlement dominates; check time–rate curves (§10.3).
- **Raft or strip footings** provide settlement compatibility; isolated pads risk greater differential.
- Always verify **drainage and groundwater conditions**, as post-construction wetting can double settlements.
- Use field testing (PLT, CPT) correlations where lab data are limited (§10.4).

Phicon Note:

• Treat settlement as a **serviceability check, not a factor of safety** issue — it relates to structural performance.
• Always quote **angular distortion** alongside absolute settlement in reports; it's the criterion auditors check first.
• For critical structures, verify both **short-term elastic** and **long-term consolidation** components.
• Include **tolerable movement limits** in the Geotechnical Design Report (GDR) to inform structural detailing.
• Where ground improvement or preloading is used, include predicted vs monitored settlements as part of EC7 verification.

10.2 Immediate (Elastic) Settlement

10.2.1 Purpose

Immediate (elastic) settlement occurs as soon as load is applied — resulting from distortion of the soil mass without change in water content.

It governs foundation performance on **granular soils, overconsolidated clays**, and **rock-like materials**, and forms the first component of total settlement under SLS.

EN 1997-1 §2.4.8 requires the calculated **design settlement (s_d)** to remain below the **limiting value (s_{lim})** appropriate for the structure.

10.2.2 Fundamental Expression

For a uniformly loaded flexible footing:

$$s_i = qB\frac{(1-\nu^2)}{E_s}I_s$$

Where;

- s_i: Immediate settlement (m)
- q: Net foundation pressure (kPa)
- B: Foundation width (m)
- ν: Poisson's ratio of soil (-)
- E_s: Young's modulus (characteristic) (kPa)
- I_s: Influence factor (shape + depth + rigidity) (-)

For **rectangular foundations**, $I_s$varies from 0.6–1.2 depending on geometry and embedment.

Use **characteristic stiffness values** (E_k) from tests or correlations, without partial factors.

10.2.3 Influence Factors (Meyerhof, Poulos & Davis)

Footing Shape	Depth Ratio (D_f/B)	Poisson's Ratio (ν)	Influence Factor (I_s)
Strip footing	0	0.3	1.00
Circular	0	0.3	1.12
Square	0	0.3	1.06
Rectangular (L/B = 2)	0	0.3	1.02
Any shape (D_f/B = 1)	—	—	+10–20 % for depth confinement

Simplified:

$$I_s \approx 1.0 \pm 0.2$$

depending on shape and embedment.

10.2.4 Elastic Modulus from Field Correlations

Test Type	Parameter	Correlation (approx.)	Soil Type
SPT (N_{60})	E_s (MPa) = 2.5–4 × N_{60}	Sands / gravels	
CPT (q_c)	E_s (MPa) = (2–4) × (q_c/1000)	Sands / silts	
PLT (E_{v_2})	E_s = (1.5–2.0) × E_{v_2}	Any compacted material	
Oedometer (m_v)	E_s = 1/m_v	Clays (small strain)	

Typical ranges:

Soil Type	E_k (MPa)	ν
Loose sand	10–25	0.3
Medium-dense sand	25–60	0.3
Dense sand / gravel	60–120	0.25
Soft clay	3–8	0.45
Firm clay	8–20	0.40
Stiff clay	20–60	0.35
Hard clay / weak rock	60–200	0.30

10.2.5 Immediate Settlement under Uniform Pressure

For a **square footing** (flexible base, ϕ' soil):

$$s_i = \frac{qB(1-\nu^2)}{E_s} \times I_s$$

Example:

- q = 200 kPa, B = 2 m, ν = 0.3, E_s = 40 MPa, $I_s = 1.06$

$$s_i = \frac{200 \times 2 \times (1-0.3^2)}{40{,}000} \times 1.06 = 0.0096\ m = 9.6\ mm$$

✓ Immediate settlement ≈ 10 mm (within most structural limits).

10.2.6 Effect of Embedment

Embedment increases confinement and reduces strain:

$$s_{i,embedded} = s_i(1 - 0.5\frac{D_f}{B})$$

Approximate reduction factors:

D_f/B	Reduction Factor
0	1.00
0.5	0.75
1.0	0.50

10.2.7 Influence of Rigidity and Layered Ground

- **Flexible foundations** (mats, rafts) → more uniform pressure; use mean E_s.
- **Rigid foundations** (RC pads) → corner settlements lower, centre higher; I_s slightly smaller.
- Layered soils:
    - Compute weighted average modulus E_{eq}:

$$E_{eq} = \frac{\sum(E_i \Delta z_i)}{\sum \Delta z_i}$$

up to a depth ≈ **2B**.

- Alternatively, integrate settlement using layer-by-layer summation.

10.2.8 Simplified Design Formulae (Approximate)

Soil Type	Approximate Formula (mm)	Applicable Range
Sand / gravel	$s_i = 0.25q(B/E_s)$	q in kPa, B in m
Clay (undrained)	$s_i = 0.5q(B/E_s)$	Short-term loading
Weak rock / dense soil	$s_i < 5\ mm$	Bearing often governs

10.2.9 Eurocode 7 Verification Steps

1. **Select soil stiffness** (E_k) from field/lab tests.
2. **Compute elastic settlement** s_i.
3. **Sum with consolidation / creep** (if any) to obtain total s_d.
4. **Compare to allowable limits** (§10.1.5).
5. **Document** within GDR as part of *Serviceability verification*.

$$s_d = s_i + s_c + s_s$$

10.2.10 Practical Observations

- Granular soils show most settlement during loading — confirm via **plate load tests**.
- In soft clays, "immediate" undrained deformation may represent only the initial phase of long-term settlement.
- For footings on **engineered fills**, stiffness depends strongly on compaction control (EV_2 > 120–150 MPa recommended).
- Record foundation stiffness assumptions in the calculation sheet — they are frequently reviewed in CAT II/III checks.

Phicon Note:

• Immediate settlement usually controls design on **granular or compacted soils**, especially under transient or cyclic loads.
• Use **field-derived stiffness** (PLT, CPT, SPT) rather than textbook values where available — the variability is often ±50%.
• For multi-footing structures, calculate both **absolute and differential** settlements using the same E-profile.
• Include influence factor I_s explicitly in spreadsheets — don't embed as constants.
• Check that **E_k values are compatible with stress level** (linear elasticity assumption valid only up to ~25–50% of $q_{(u)}$).

10.3 Consolidation and Secondary Settlement

10.3.1 Purpose

Consolidation settlement arises when excess pore water pressure, generated by load application, **dissipates over time**, causing volume reduction of saturated cohesive soils.

It governs the **long-term deformation** of foundations on **soft to firm clays, silts, and organic soils**, and is assessed as part of the **Serviceability Limit State (SLS)** under EN 1997-1 §2.4.8.

10.3.2 Primary Consolidation

(a) Fundamental Equation (Terzaghi, 1D Theory)

$$S_c = \frac{C_c \cdot H}{1 + e_0} \log_{10}\left(\frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}\right)$$

where:

- s_c: Primary consolidation settlement
- C_c: Compression index
- e_0: Initial void ratio
- H: Thickness of compressible layer
- σ_0': Initial effective overburden pressure
- $\Delta\sigma'$: Increase in effective vertical stress at mid-depth

(b) For Overconsolidated Clays

$$s_c = \begin{cases} \dfrac{C_r \cdot H}{1+e_0} \log_{10}\left(\dfrac{\sigma'_p}{\sigma'_0}\right) + \dfrac{C_c \cdot H}{1+e_0} \log_{10}\left(\dfrac{\sigma'_0 + \Delta\sigma'}{\sigma'_p}\right), & for\ \sigma'_0 + \Delta\sigma' > \sigma'_p \\ \dfrac{C_r \cdot H}{1+e_0} \log_{10}\left(\dfrac{\sigma'_0 + \Delta\sigma'}{\sigma'_0}\right), & for\ \sigma'_0 + \Delta\sigma' \leq \sigma'_p \end{cases}$$

where:

- σ'_p: preconsolidation pressure
- C_r: recompression index (≈ 0.1–0.2 × C_c)

The transition from recompression to virgin compression defines whether consolidation is elastic or plastic.

10.3.3 Coefficient of Volume Compressibility

Volume compressibility is defined as:

$$m_v = \frac{\Delta e}{(1+e_0)\Delta\sigma'}$$

or from the e–log p curve in oedometer tests:

$$m_v = \frac{C_c}{(1+e_0)\ln(10)(\sigma'_2 - \sigma'_1)}$$

Typical ranges:

Soil Type	m_v (m^2/MN)	$C_{(c)}$	$C_{(r)}$
Soft clay	0.5–2.0	0.2–0.6	0.02–0.08
Firm clay	0.2–0.5	0.15–0.3	0.02–0.05
Stiff clay	0.05–0.2	0.08–0.2	0.01–0.03
Organic clay / peat	2–10	0.6–2.0	0.05–0.1

m_v provides a direct link between oedometer results and settlement under field stress increments.

10.3.4 Average Degree of Consolidation and Time–Rate Behaviour

Terzaghi's one-dimensional time–rate relationship:

$$U = \frac{s_t}{s_c}$$
$$T_v = \frac{C_v t}{H_{dr}^2}$$

Degree of Consolidation (U%)	Time Factor (T_v)
50	0.197
60	0.287

70	0.403
80	0.567
90	0.848
95	1.129

Rearranging:

$$t = \frac{T_v H_{dr}^2}{C_v}$$

where:

- C_v: coefficient of consolidation (m^2/s)
- H_{dr}: maximum drainage path (m)
- t: time for given U

Typical $C_v$values:

Soil Type	C_v (m^2/s)
Soft clay	$1\times10^{-8} – 5\times10^{-8}$
Firm clay	$1\times10^{-7} – 5\times10^{-7}$
Silt	$5\times10^{-7} – 2\times10^{-6}$
Sand	$>1\times10^{-5}$ (drained instantly)

Consolidation time is highly sensitive to drainage path — double drainage (top & bottom) halves the time to reach U = 50%.

10.3.5 Secondary Compression (Creep)

After dissipation of pore pressure, deformation continues under constant effective stress due to **viscous creep**.
Secondary settlement (s_s):

$$s_s = \frac{C_\alpha H}{1 + e_p} \log_{10}(\frac{t_2}{t_1})$$

where:

- C_α: coefficient of secondary compression (typically 0.01–0.05 C_c)
- e_p: void ratio at end of primary consolidation
- t_1, t_2: start and end times of secondary phase

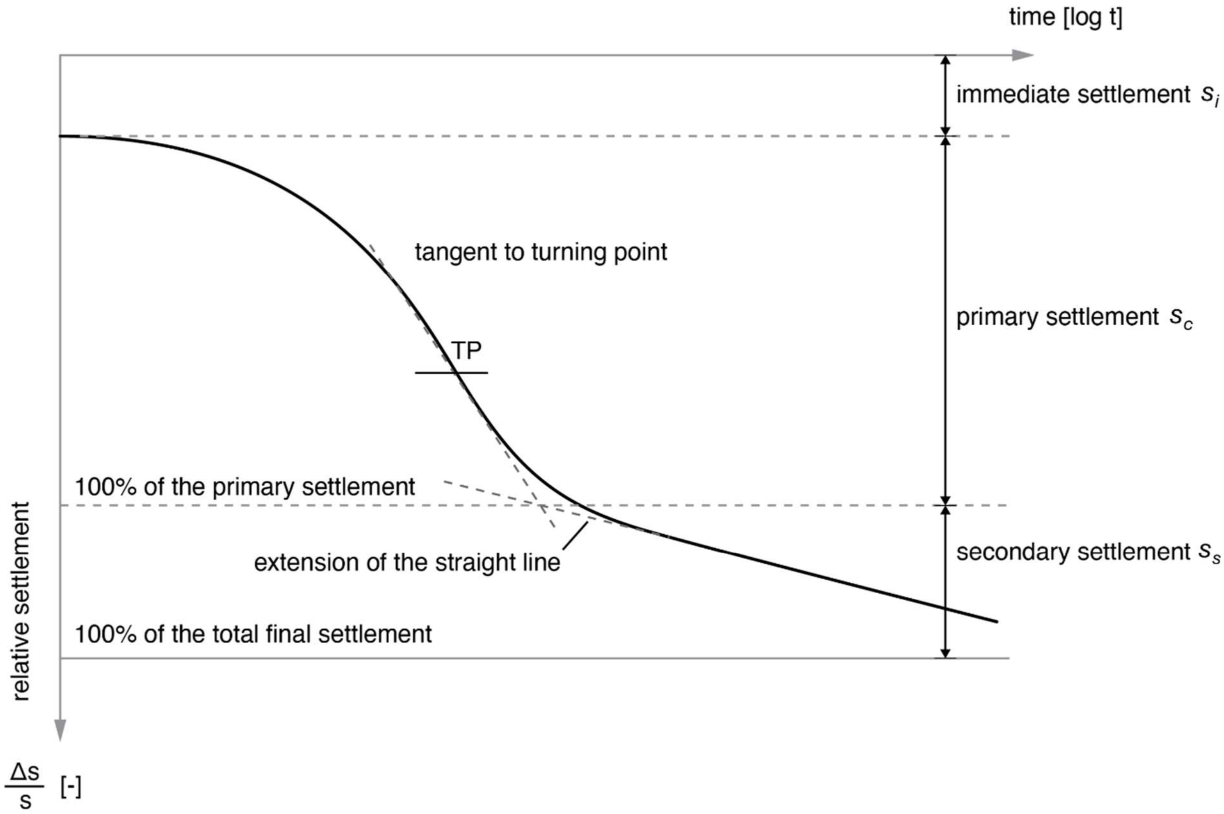


Figure 37: Time-settlement curve for a typical cohesive soil

Approximation for organic clays or peat:

$$s_s \approx (0.1–0.3)s_c$$

10.3.6 Worked Example

Given:

- Soft clay, H = 6 m, e_0 = 1.0, $C_{(c)}$ = 0.3, σ'_0 = 80 kPa, $\Delta\sigma'$ = 60 kPa

$$s_c = \frac{0.3}{1+1.0} \times 6 \times \log_{10}\left(\frac{80+60}{80}\right) = 0.15 \times 6 \times 0.243 = 0.22\ m$$

$C_v = 2 \times 10^{-8}\ m^2/s, H_{dr} = 3\ m$ (double drainage).
Time for 90 % consolidation:

$$t_{90} = \frac{0.848 H_{dr}^2}{C_v} = \frac{0.848 \times 9}{2 \times 10^{-8}} = 3.82 \times 10^8\ s = 12\ years$$

✓ Predicted primary settlement = 220 mm over ~12 years.
If $C_\alpha/C_c = 0.03$, secondary adds ≈ 30–40 mm → total ≈ 250 mm.

10.3.7 Practical Design Treatment

Stage	Action / Input	Purpose
1	Derive m_v or Cc from oedometer tests	Compressibility
2	Compute stress increase ($\Delta\sigma'$) using influence factors (2:1 or Boussinesq)	Loading distribution
3	Calculate s_c, s_s	Primary and secondary components
4	Apply time–rate analysis (C_v, H_{dr})	Duration to reach SLS
5	Compare s_{total} with structural limits (§10.1)	Verification
6	Assess improvement measures (drains, preloading)	If excessive

10.3.8 Ground Improvement and Acceleration

To control excessive consolidation:

- **Prefabricated vertical drains (PVDs):** shorten drainage path, increase C_v.
- **Preloading / surcharge:** accelerate consolidation before construction.
- **Lightweight fill:** reduce stress increment $\Delta\sigma'$.
- **Raft foundations:** distribute loads to reduce differential movement.
- **Vacuum preloading:** alternative where surcharge impractical.

10.3.9 Eurocode 7 Verification Framework

EN 1997-1 §2.4.8:

$$s_{calc} \leq s_{lim}$$

At SLS:

- γ_M = 1.0 (no partial factors on soil stiffness or parameters).
- Use characteristic parameters (Cc, Cv, mv) from tests or published data.
- Document assumptions for drainage conditions, time dependency, and ground improvement in the GDR.

10.3.10 Typical Design Ranges

Soil Type	Expected s_c (mm)	s_s (mm)	Duration (yrs)
Soft alluvial clay	100–400	20–80	5–20
Marine clay	200–600	30–100	10–30
Firm clay	50–150	10–20	1–5

Peat / organic	300–1000	100–300	5–30

Phicon Note:

• Always confirm whether **double or single drainage** applies — it's the largest driver of settlement duration.
• Consolidation is a **time-dependent verification**, not a one-off static value — report both *magnitude* and *timescale*.
• Use laboratory C_v and m_v cautiously; field-scale values may be an order of magnitude higher due to macro-fissures.
• For organic or preloaded soils, include **secondary compression (creep)** explicitly in total settlement prediction.
• If total settlements exceed serviceability limits, evaluate **drainage, preloading, or raft redistribution** before resorting to piling.

10.4 Empirical Methods (SPT/CPT/PLT Correlations)

10.4.1 Purpose

Empirical methods enable settlement estimation directly from **in situ test results**, providing practical guidance where site investigation data are limited.

They are particularly useful for:

- Preliminary design,
- QA verification of bearing capacity and stiffness,
- Foundation checks on granular or mixed soils where oedometer tests are impractical.

These methods are **semi-empirical** — they must be applied with engineering judgement, calibrated to local experience, and verified against Eurocode 7 serviceability limits.

10.4.2 General Form

Settlement is expressed as:

$$s = \frac{qB(1 - \nu^2)}{E_s} I_s$$

The challenge is determining a representative **modulus of deformation (E_s)** from field test results.

10.4.3 A. Settlement from SPT Correlations

10.4.3.1 10.4.3 SPT-Based Modulus Correlations

Soil Type	E_s (MPa)	Approximate Relation	Reference
Loose sand	10–25	$E_s = 2.5 \times N_{60}$	Peck et al. (1974)
Medium-dense sand	25–60	$E_s = 4 \times N_{60}$	Bowles (1996)

Dense sand / gravel	60–120	$E_s = 6 \times N_{60}$	NAVFAC DM 7.1
Soft clay	3–8	$E_s = 15 \times N_{60}$	Stroud (1974)
Stiff clay	10–25	$E_s = 30 \times N_{60}$	Stroud (1974)

where N_{60}= corrected SPT blow count (energy = 60 %).

⚠️ These are indicative ranges — always compare with local calibration data or correlations derived from known test performance (PLT, EV_2).

10.4.3.2 Schmertmann (1970, 1978) Simplified SPT Method

For shallow foundations on sand:

$$s = C_1 C_2 \frac{qB}{E_s}$$

where:

- $C_1 = 1 - 0.5(q/q_u)$(shape factor)
- $C_2 = 1 + 0.2\frac{Df}{B}$(depth correction)
- $E_s = 2.5\text{–}4.0 \times N_{60}$ (MPa)

For typical cases:

- $s \approx 0.25(qB/E_s)$
- Use upper bound for preliminary check.

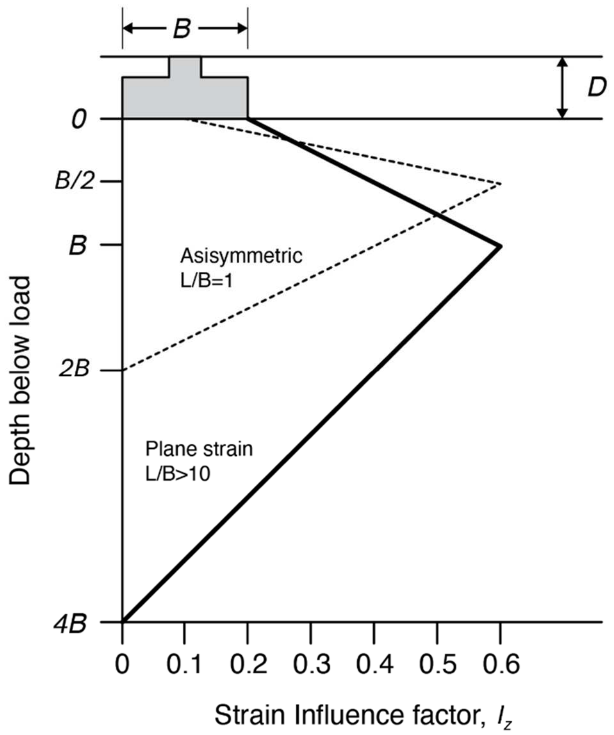


Figure 38: Strain influence factors from Schmertmann et al. (1978)

10.4.4 B. Settlement from CPT Correlations

10.4.4.1 CPT Modulus Correlations

Soil Type	E_s (MPa)	Relation	Source
Sand / gravel	$E_s = (2–5) \times (q_c / 1000)$	Schmertmann (1978)	
Silt / fine sand	$E_s = (1–3) \times (q_c / 1000)$	De Beer (1970)	
Clay (drained)	$E_s = (1–2) \times (q_c / 1000)$	Lunne & Kleven (1981)	
Soft clay (undrained)	$E_s = (150–300) \times c_u$	NAVFAC (1986)	

Typical example:
If $q_c = 8\ MPa$ (dense sand), $E_s = 3 \times 8 = 24\ MPa$.
At q = 200 kPa, B = 2 m, ν = 0.3, $I_s = 1.0$:

$$s = \frac{200 \times 2 \times (1 - 0.3^2)}{24000} = 0.012\ m = 12\ mm$$

✓ Predicted settlement = 12 mm.

10.4.4.2 Schmertmann's CPT Strain Influence Method

A more rigorous approach for sands:

$$s = C_1 C_2 \int_0^{z_i} \frac{\Delta\sigma_z I_z}{E(z)} dz$$

where:

- $\Delta\sigma_z$: vertical stress increment beneath foundation,
- I_z: strain influence factor (≈ 0.5 at z/B = 0, decreasing to 0 at z/B = 2),
- $E(z)$: modulus profile from CPT q_c,
- C_1: time factor (1 for immediate, 1.2–1.5 for long-term),
- C_2: shape factor (≈ 1.0 for square).

Approximation:

75–80 % of settlement occurs within 2B depth.
Integration can be done in layers using spreadsheet or numerical tools.

10.4.5 C. Settlement from Plate Load Tests (PLT / EV_2)

10.4.5.1 Plate Load Modulus and Scaling

PLT provides a direct measure of ground stiffness (EV_2).
To scale from plate diameter D_p(typically 300–760 mm) to foundation width B:

$$E_s = E_{V2}(\frac{B}{D_p})^n$$

with $n = 0.3–0.5$.

Alternatively, approximate:

$$s_{foundation} = s_{PLT}(\frac{B}{D_p})^{1-n}$$

Soil Type	n	Comment
Sand / gravel	0.3	Settlement increases moderately with size
Clay / silt	0.5	Settlement increases more strongly
Mixed fill	0.4	Use intermediate value

Where available, the PLT correlation provides the **most direct validation** of design settlements.

10.4.5.2 EV_2 – CBR Relationship (for Fills and Platforms)

Approximate equivalence (BRE 470, TRL 113):

$$EV_2(MPa) \approx 17 \times CBR^{0.64}$$

Typical range:

EV_2 (MPa)	Equivalent CBR (%)	Use
50	3–4	Weak fill
100	8–10	Working platform
150	15–20	Sub-base or well-compacted fill
250	30–40	Granular foundation
400+	60+	Rock fill / dense gravel

Use EV_2-derived stiffness for immediate settlement checks of rafts or spread footings on engineered layers.

10.4.6 Eurocode 7 Treatment

EN 1997-1 §2.4.8:

- Use characteristic stiffness (E_k) or measured EV_2 directly.
- No partial factors applied to material properties at SLS (γ_m = 1.0).
- Document source of correlations (SPT, CPT, PLT) and any calibration used.
- Confirm **correlation consistency** with soil type and stress range.

$$s_d \leq s_{lim}$$

is verified using settlement from the chosen empirical approach.

10.4.7 Comparative Overview

Method	Input Data	Typical Accuracy	Strengths	Limitations
SPT	N_{60}	±40 %	Widely available, quick	Sensitive to energy & equipment
CPT	q_c, f_s	±25 %	Continuous profile, accurate	Equipment access / calibration
PLT	Load–settlement	±15 %	Direct stiffness measure	Localised, small test area
Oedometer	m_v, C_c	±25 %	Theoretical basis	Time-consuming, small samples

10.4.8 Phicon Field Practice Tips

- Cross-check stiffness from **CPT q_c**, **SPT N**, and **PLT EV_2** to ensure consistency.
- Always relate empirical data to the **stress level** — moduli vary nonlinearly with applied pressure.
- For QA of working platforms and shallow footings, **EV_2 > 120 MPa** (CBR ≈ 10–15%) is typically sufficient.
- Use **local calibration plots** (EV_2 vs. settlement) to refine project-specific modulus relationships.
- Report all **test-derived correlations** in the Geotechnical Design Report (GDR), stating uncertainty and reference source.

Phicon Note:

• Field-based methods are most powerful when used in combination — CPT gives continuous stiffness, PLT validates local response, and SPT benchmarks older datasets.
• Treat empirical correlations as **tools for verification**, not replacements for parameter derivation.
• Under Eurocode 7, field stiffness correlations serve the **SLS verification** (γ_m = 1.0); they must still be consistent with the characteristic parameter concept.
• Always note whether correlations are **drained (long-term)** or **undrained (short-term)** before applying them to settlement predictions.
• Maintain traceable records of how E_s, EV_2, or q_c were converted to settlements — they are among the most scrutinised aspects in CAT II/III reviews.

11 Deep Foundations: Piles

11.1 Pile Types and Installation Methods (Driven, CFA, Bored, Micro)

11.1.1 Purpose

Piles transfer load from a structure to deeper, stronger strata where near-surface soils are too weak, compressible, or variable.

Design must address:

- Axial capacity (compression and tension),
- Settlement and load–displacement response,
- Group and interaction effects, and
- Installation-induced changes in soil structure and pore pressure.

Eurocode 7 requires verification of both **geotechnical (GEO)** and **structural (STR)** limit states for each pile.

11.1.2 Classification by Load Transfer Mechanism

Type	Load Transfer Behaviour	Typical Application
End-bearing pile	Load carried primarily on tip resistance	Dense sand, rock, or stiff clay at depth
Friction pile	Load transferred mainly by shaft friction	Deep deposits of soft clay or medium sands
Combined pile	Mix of friction and end-bearing	Most practical cases in layered ground
Tension / anchor pile	Resists uplift or overturning	Basements, retaining structures, towers

In reality, most piles act as **combined** friction and end-bearing elements; relative contributions vary with soil type and construction.

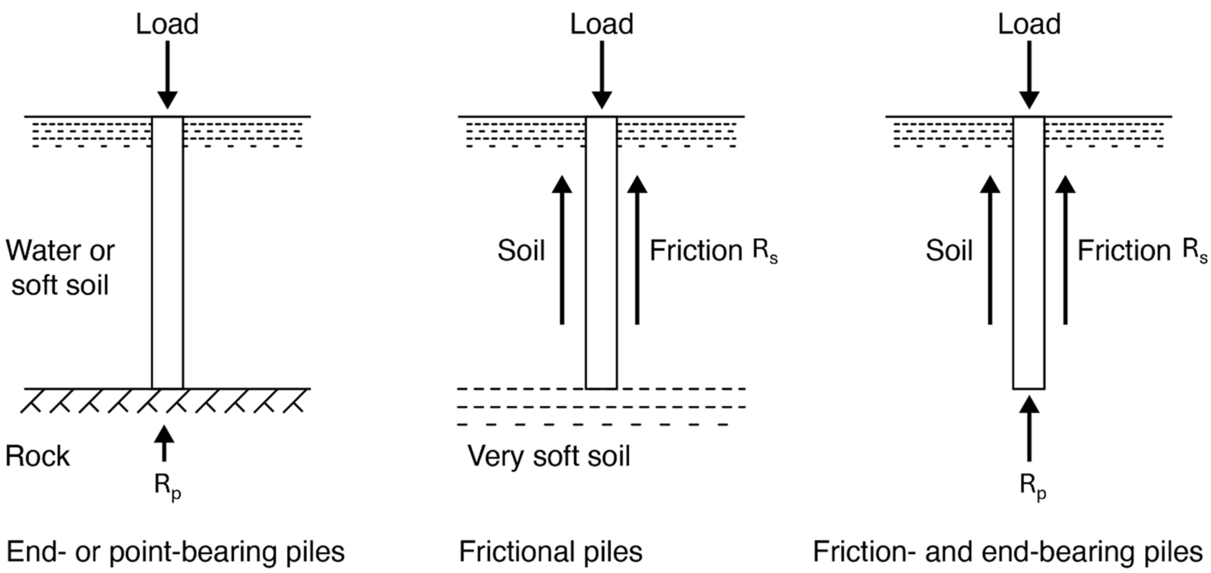


Figure 39: Load bearing Mechanism of pile

11.1.3 Classification by Installation Method

Category	Sub-type / Example	Key Features	Advantages	Limitations
Driven displacement	Precast RC, steel H-piles, closed-ended tubular, timber	Driven by hammer or vibration	High capacity; good quality control	Noise/vibration; displacement effects
Driven non-displacement	Open-ended steel tube, cast-in-situ (DCIS)	Soil enters pile during driving	Suitable in dense soils; variable base	Difficult QA; limited toe inspection
Bored (replacement)	Rotary bored, underreamed, CFA	Soil removed, concrete cast in hole	Minimal vibration; adaptable diameters	Spoil removal; slower installation
Continuous flight auger (CFA)	Auger bored, concrete pumped through stem	Continuous operation; cast in situ	Fast, low noise	Difficult toe cleaning; QC via logs
Micropiles / mini piles	Grouted steel bar/tube ($D \leq 300$ mm)	Installed by drilling, low disturbance	Restricted access, tension capacity	Limited compressive load, corrosion control
Screw / helical piles	Torque-installed steel shafts	Minimal spoil; reusable	Light to medium loads	Corrosion and quality variability

11.1.4 Pile Materials

Material	Common Diameter / Size	Notes / Advantages
Reinforced concrete (precast)	250–400 mm square	High durability; suitable for high bending
Bored concrete (in situ)	450–1800 mm	Common for heavy loads or retaining walls
Steel H-pile / tubular	150–1000 mm	High capacity; easy splicing; tension use
Timber	150–350 mm	Temporary or light structures; limited durability
Micropile (steel + grout)	75–300 mm	Versatile for underpinning or restricted sites

Corrosion, cover, and durability checks are carried out under EN 1992 (concrete) and EN 1993-5 (steel), with design life typically ≥ 50 years.

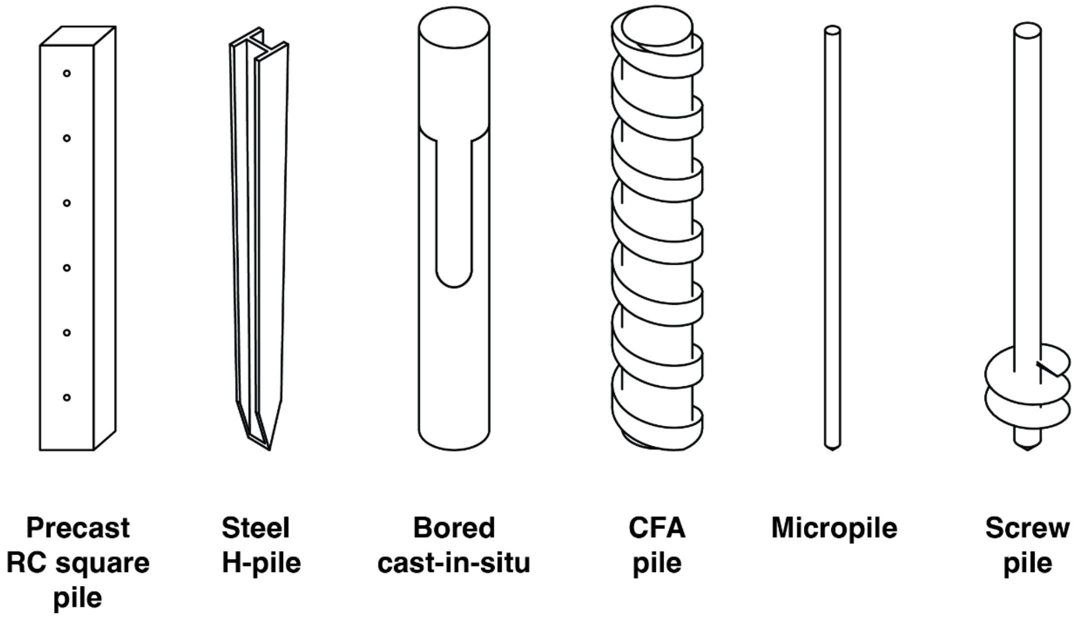


Figure 40: Common pile types used in UK/EU

11.1.5 Installation Effects on Soil Behaviour

Method	Effect On Soil	Design Implication
Driven Displacement	Densification in sands, pore pressure rise in clays	Temporary reduction in capacity during dissipation; long-term increase in dense soils
Bored Replacement	Disturbance and loosening around shaft	Lower interface friction; cleaning of base critical
Cfa Piles	Partial replacement; residual spoil along shaft	Quality depends on continuous auger concreting and pressure monitoring
Micropiles	Minimal displacement; grout–soil bond governs	Use grouted bond capacity, not skin friction formulae
Screw Piles	Shear mobilisation through helix plates	Correlation through torque–capacity relationship

⚠️ Always record **installation parameters** (hammer energy, auger torque, grout pressure, concreting logs) — these form part of the *Execution Record* per EN 1536 / EN 12699.

11.1.6 Typical Pile Sizes and Capacities

Pile Type	Diameter / Section	Typical Working Load (kN)	Remarks
Precast RC square pile	250–400 mm	400–1500	Driven; used widely in housing / infrastructure
Steel H-pile	203×203×46 to 305×305×137	500–2500	High axial + tension; reusable
Bored cast-in-situ	450–1800 mm	600–10,000+	Used for bridge piers, buildings
CFA pile	300–900 mm	400–2000	Medium capacity; quick installation
Micropile	100–300 mm	100–800	Underpinning, tension, restricted access
Screw pile	100–400 mm	100–600	Temporary or light structures

11.1.7 Eurocode 7 Design Framework

Limit State	Description	Relevant EN Reference	Verification Requirement
ULS – GEO	Pile–soil capacity (shaft + base)	EN 1997-1 §7.6.2	$R_d \geq E_d$
ULS – STR	Structural strength of pile section	EN 1992 / EN 1993-5	Check axial + bending resistance
SLS	Settlement, load–displacement	EN 1997-1 §2.4.8	Verify movement ≤ limit
EQU	Overall stability (uplift, overturning)	EN 1997-1 §2.4.7	$\Sigma M_{res} \geq \Sigma M_{act}$
HYD	Hydraulic heave or piping (for tension piles)	EN 1997-1 §10	Where groundwater critical

Partial factors (UK NA, DA1):

- γ_M = 1.25 on ϕ' and c' (C2)
- γ_R = 1.0 on resistances
- γ_F = 1.35 / 1.5 on actions (C1)

11.1.8 Construction QA and Verification

Stage	Quality Control Measures	Typical Records / Output
Pre-installation	Check rig calibration, pile design vs ground	Pile log sheets, rig inspection
During driving / boring	Monitor depth, energy, concreting pressure	Installation records, dynamic data
Post-installation	Integrity testing, dynamic or static load testing	PDA / SLT reports
Completion	As-built pile schedule, verification summary	Included in Geotechnical Design Report (GDR)

⚠️ Eurocode 7 §4.1 requires that *test piles* be used to verify design parameters whenever feasible, particularly for critical structures or variable ground conditions.

11.1.9 Summary of Execution Standards

Standard	Application	Title / Notes
EN 1536	Bored piles	Execution of special geotechnical works – bored piles
EN 12699	Displacement	Driven cast-in-place piles
EN 14199	Micropiles	Execution of micropiles
EN 1997-2	Field testing	Specifies pile load, integrity, dynamic and static test procedures
BS 8004	UK guidance	Provides complementary bearing and settlement guidance
CIRIA C580 / C574	Best practice	Piling QA, design, and verification references

Phicon Note:

• Always differentiate between **installation type** and **load-transfer mechanism** — both affect design parameters and verification tests.
• For EC7 documentation, describe **execution method**, **pile type**, **nominal dimensions**, **installation parameters**, and **expected performance** in the GDR.
• Site testing is essential — no amount of theoretical calculation replaces a well-documented **static load test (SLT)**.
• Pile behaviour is strongly influenced by construction quality — consider **installation-induced variability** as part of design robustness (EN 1997-1 §2.4.9).
• Keep a record of **as-built pile lengths**, **cut-off levels**, and **ground conditions**; they form the factual basis for Category 2/3 verification.

11.2 Axial Capacity in Cohesive Soils (α-method)

11.2.1 Purpose

The **α-method** estimates the **undrained (total stress) axial capacity** of piles in cohesive soils.

It is used where pile behaviour is governed by **shaft adhesion** rather than tip resistance — typically in soft to firm clays, silty clays, or layered deposits.

Under **short-term (undrained) loading**, pore pressures prevent effective stress mobilisation, so strength is represented by the **undrained shear strength (c_u)**.

This approach is widely accepted for **temporary or initial service conditions** in UK and EU design.

11.2.2 Total Axial Capacity (Compression)

$$Q_{ult} = Q_s + Q_b$$
$$Q_s = \alpha \, c_u \, A_s, Q_b = N_c \, c_u \, A_b$$

where;

- Q_{ult}: Ultimate axial load capacity (kN)
- Q_s: Shaft resistance (kN)
- Q_b: Base (end-bearing) resistance (kN)
- α: Adhesion factor (0–1.0) (-)
- c_u: Undrained shear strength (average along shaft) (kPa)
- A_s: Shaft surface area (m^2)
- A_b: Base area (m^2)
- N_c: Bearing factor (typically 9 for deep piles) (-)

11.2.3 Adhesion Factor (α) – Typical Values

Empirically derived from field load tests (Tomlinson 1957; BS 8004:2015 §7.4.2).

Pile Type / Installation	Soil Consistency	Typical c_u (kPa)	α-factor	Comment
Driven precast / steel	Very soft clay	< 25	1.0	Full adhesion; low remoulding
	Soft–firm clay	25–75	0.8–1.0	Common UK design range
	Stiff clay	75–150	0.6–0.8	Adhesion reduces with stiffness
Bored (replacement)	Soft–firm clay	25–75	0.6–0.8	Partial remoulding during boring
	Stiff clay	75–150	0.4–0.6	Shaft cleaned but roughness reduced
CFA / cast-in-situ	Soft–firm clay	25–75	0.5–0.7	Depends on concrete pressure, cleanliness
Micropile (grouted)	Any	—	0.4–0.6	Use measured bond stress where available

As a rule of thumb:

$$\alpha \approx \min\left(1, \frac{120}{c_u}\right)$$

but always limited to 0.3–1.0.

11.2.4 Base (End-Bearing) Resistance

$$Q_b = N_c\, c_u\, A_b$$

Typical factors (Skempton, 1951):

Pile Type	N_c	Comment
Driven	9	Fully mobilised end bearing
Bored / CFA	6	Partially disturbed base
Underreamed	9–10	Enhanced base area and confinement

In soft clay, base contribution is often < 20 % of total capacity.
In firm–stiff clay, it can exceed 30–40 % depending on embedment and base cleanliness.

11.2.5 Design Approach per Eurocode 7

For Design Approach 1 (Combination 2) (UK NA default for piles):

$$R_d = \frac{Q_{ult}}{\gamma_R}, \text{with } \gamma_R = 1.0$$

and

$$c_u, d = \frac{c_u, k}{\gamma_M}, \gamma_M = 1.25$$

Verification:

$$E_d \leq R_d$$

Partial factors are applied on **soil parameters**, not resistances, under DA1/C2.

Use **characteristic cu profiles** derived from field or laboratory tests (Vane, UU triaxial, CPT qc correlations).

11.2.6 Worked Example

Given:
Bored pile in firm clay

$$D = 0.6\,m, L = 18\,m, c_u = 75\,kPa, \alpha = 0.6, N_c = 6$$

$$A_s = \pi DL = \pi \times 0.6 \times 18 = 33.9\,m^2, A_b = \frac{\pi D^2}{4} = 0.283\,m^2$$
$$Q_s = 0.6 \times 75 \times 33.9 = 1526\,kN, Q_b = 6 \times 75 \times 0.283 = 127\,kN$$
$$Q_{ult} = 1526 + 127 = 1653\,kN$$

$$Q_d = \frac{1653}{1.25} = 1320\ kN$$

✓ Design axial resistance (ULS, GEO) = 1320 kN.

- 11.2.7 Tension (Uplift) Capacity

$$Q_{t,ult} = \alpha_t\ c_u\ A_s$$

Pile Type	α_t (uplift adhesion)	Notes
Driven	0.7–1.0 α	Rough surface, good bond
Bored / CFA	0.5–0.7 α	Reduced interface roughness
Micropile (grouted)	0.6–0.8 α	Dependent on grout–soil adhesion

Design per EC7 DA1/C2:

$$R_{t,d} = \frac{Q_{t,ult}}{\gamma_M}\ (\gamma_M = 1.25)$$

11.2.7 Settlement and Load–Displacement Behaviour

- Load–settlement response in clay is typically **non-linear**:
  - Initial elastic stage → **adhesion mobilisation** → **yield at top** → full mobilisation near base.
- Typical working load = **≤ 50–60 % of $Q_{(ult)}$** to limit settlement < 10–20 mm.
- For bored piles in soft clay, consolidation beneath tip may cause additional long-term settlement (check §10.3).

11.2.8 Field Validation and Testing

Test Type	Purpose	Standard / Reference
Static load test (SLT)	Confirm $Q_{(s)}$ + $Q_{(b)}$ mobilisation	EN ISO 22477-1
Dynamic (PDA)	Estimate capacity during driving	EN ISO 22477-2
Integrity test (SIT)	Detect defects or necking	EN ISO 22477-4
Instrumented pile test	Separate shaft / base load shares	CIRIA C580

Design correlations should be checked against **test results** from at least one trial pile where feasible.

Phicon Note:

• The α-method remains the **most practical and robust** approach for cohesive soils where full drainage cannot occur during loading.
• Choose α conservatively based on **installation type and remoulding level** — never assume α = 1.0 without test evidence.
• Always distinguish between **total stress (undrained)** and **effective stress (drained)** design cases — they are **not interchangeable**.
• Verify design with **load testing or back-analysis** from similar projects; adjust α accordingly for local conditions.
• Record **c_u profile, α-factor, and $N_{(c)}$ value** clearly in the Geotechnical Design Report (GDR) for traceability and future calibration.

11.3 Axial Capacity in Granular Soils (β-method and Point Resistance)

11.3.1 Purpose

In sands and gravels, pile capacity derives mainly from **shaft friction** and **tip resistance** mobilised under drained (effective stress) conditions.

The **β-method** expresses shaft resistance as a function of **effective vertical stress** and **interface friction**, while the base resistance depends on **end bearing within the dense stratum**.

Eurocode 7 treats both as **GEO limit state checks**, verified using characteristic effective stress parameters and appropriate partial factors.

11.3.2 Governing Equations

$$Q_{ult} = Q_s + Q_b$$
$$Q_s = \sum(\beta_i\ \sigma'_{v,i}\ A_{s,i}), Q_b = q_b\ A_b$$

Where;

- β_i: Shaft friction coefficient (dimensionless) (-)
- $\sigma'_{v,i}$: Effective overburden stress at mid-depth of layer i (kPa)
- $A_{s,i}$: Shaft area in layer i (m^2)
- q_b: Base (end-bearing) pressure (kPa)
- A_b: Base area (m^2)

11.3.3 Shaft Resistance – β-Method

Empirically;

$$\tau_s = \beta\ \sigma'_v$$

and typically $\beta = K\tan\ \delta$

where;

- K= coefficient of horizontal stress (≈ 0.5–1.0),
- δ= pile–soil interface friction angle (≈ 0.6–0.8 ϕ').
- Typical β-values (effective stress design)

Soil Density / Relative Density (Dr)	Driven Pile	Bored / CFA Pile	Notes
Loose sand (ϕ' = 28°)	0.25–0.35	0.15–0.25	Limited confinement
Medium sand (ϕ' = 32°)	0.35–0.55	0.25–0.40	Common UK range
Dense sand / gravel (ϕ' = 36°–40°)	0.5–0.8	0.35–0.6	Driven piles densify soil
Silty sand / fine sand	0.25–0.45	0.15–0.30	Sensitive to fines and saturation
Weathered rock / chalk	—	—	Use end-bearing correlation (R-class)

Driven piles generally achieve higher β due to dilation and densification of surrounding soil.
Bored and CFA piles mobilise lower shaft friction because of wall loosening and smear.

11.3.4 Base Resistance (Point Bearing)

$$q_b = N_q\,\sigma'_{v,b} + 0.5\,\gamma' B\,N_\gamma$$

with ϕ' = effective friction angle at base.

Soil Density	N_q	N_γ	Comment
Loose sand (28°)	12	10	Low resistance, settlement governs
Medium sand (32°)	22	18	Typical design range
Dense sand (36°)	40	35	Fully mobilised end bearing
Gravel (40°)	80	70	Use for hard-driven piles on dense strata

Base resistance is typically limited to **5–15 MPa** in design to prevent excessive settlement or punching of dense layers.

11.3.5 Effective Stress Distribution

Effective vertical stress at pile base:

$$\sigma'_{v,b} = \gamma'(z_b - z_{wt})$$

where
$\gamma' = \gamma - \gamma_w$ and z_b= embedment depth.

In layered sands, compute $\sigma'_{v,i}$ for each stratum.
If groundwater fluctuates, use **highest credible water level** for design (conservative per EN 1997-1 §2.4.6.1).

11.3.6 Design Approach under Eurocode 7

For Design Approach 1 – Combination 2 (UK default):

$$c'_d = \frac{c'_k}{\gamma_M}, \varphi'_d = \tan^{-1}(\frac{\tan\,\varphi'_k}{\gamma_M}), \gamma_M = 1.25$$
$$R_d = \frac{Q_{ult}}{\gamma_R}, \gamma_R = 1.0$$

Verification:

$$E_d \le R_d$$

At SLS (γ = 1.0), confirm **load–settlement** relationship from static load test or empirical correlation (see §11.5).

11.3.7 Worked Example

Given:
Driven steel tubular pile, D = 0.6 m, L = 18 m, dense sand (ϕ' = 38°), γ' = 10 kN/m^3.
Assume $\beta = 0.6, N_q = 50.$

$$\sigma'_{v,avg} = \gamma' \times L/2 = 10 \times 9 = 90\ kPa$$
$$A_s = \pi DL = 33.9\ m^2, A_b = 0.283\ m^2$$
$$Q_s = \beta\sigma'_{v,avg}A_s = 0.6 \times 90 \times 33.9 = 1830\ kN$$
$$q_b = N_q\sigma'_{v,b} = 50 \times 10 \times 18 = 9000\ kPa, Q_b = 9000 \times 0.283 = 2550\ kN$$
$$Q_{ult} = 1830 + 2550 = 4380\ kN$$
$$Q_d = \frac{4380}{1.25} = 3500\ kN$$

✓ Design axial capacity (ULS, GEO) = 3500 kN.

- 11.3.8 Practical Observations
- Shaft friction in sands mobilises rapidly (within 5–10 mm settlement).
- End bearing may require 1–3 % B settlement for full mobilisation.
- In **CFA or bored piles**, settlement at working load (≈ 30 % $Q_{(ult)}$) often < 10 mm.
- Driven piles in dense sands gain capacity over time (“setup”) as pore pressures dissipate — often +10–30 % within weeks.

11.3.8 Field Validation

Test Type	Purpose	Standard / Reference
Static compression test	Verify load–settlement behaviour	EN ISO 22477-1
Dynamic test (PDA)	Estimate capacity during driving	EN ISO 22477-2
Instrumented test pile	Determine shaft vs. base contribution	CIRIA C580
Pile driving records	Correlate set-per-blow with capacity	ICE SPERWall / Hiley formula

Phicon Note:

• Apply the β-method for **effective stress (drained)** design, and the α-method for **total stress (undrained)** — never mix the two in a single check.
• Always derive β and Nq from **site-specific ϕ'** and **installation type**; published values vary ±50 %.
• For driven piles in sand, consider **setup effects** only if supported by load test evidence.
• Verify pile toe level and end-bearing stratum during construction — a few decimetres of deviation can halve $Q_{(b)}$.
• Record all correlations, assumptions, and parameter sources in the **GDR**; reviewers focus heavily on how β was justified.

11.4 Pile Load Testing (Static, Dynamic)

11.4.1 Purpose

Pile load testing provides the **most direct verification** of pile design assumptions, construction quality, and load–displacement behaviour.

EN 1997-1 (§7.6.2.3) requires that, where feasible, **static load tests** be performed to verify design parameters — particularly for high-risk or non-standard piling systems.

“No analytical method provides greater confidence than a well-executed static load test.”

11.4.2 Types of Pile Load Tests

Test Type	Purpose	Typical Application
Preliminary (Trial) Test	Determine ultimate capacity and load–displacement behaviour	Prior to production to confirm design parameters
Working (Proof) Test	Verify capacity of selected working piles	Routine production QA
Verification / Proof to Failure	Investigate anomalies or confirm marginal design	Selected cases
Dynamic Test (PDA)	Estimate capacity during or after driving	Driven piles, rapid QA
Integrity Test (SIT, CSL)	Check for defects (voids, necking, segregation)	Routine non-destructive QA

11.4.3 Static Load Test (Compression)

(a) Principle

Vertical load applied to pile head via hydraulic jack(s) reacting against:

- Kentledge (dead weight),
- Reaction piles,
- Tension anchors, or
- Structural frame.

Displacement is measured using dial gauges, LVDTs, or laser sensors on an independent reference beam.

(b) Loading Procedure

Stage	Loading Ratio (of Working Load)	Hold Period	Observation
1	0.25	15–30 min	Initial elastic response
2	0.5	30 min	Pore pressure rise begins
3	1.0	60 min	Working load displacement recorded
4	1.5	60–120 min	Approaching yield
5	2.0 or to failure	Until rate < 0.25 mm/h	Ultimate capacity mobilised
Unloading	Reverse in 25–50 % steps	15–30 min per step	Elastic rebound observed

Total test duration may range from **6 – 24 hours**, depending on soil type.

(c) Interpretation Methods

Method	Definition of Failure Load ($Q_{(u)}$)	Comment
Davisson offset	Load at which total settlement exceeds elastic line + 4 mm + D/120	Widely used in UK practice
De Beer (log–log)	Break point on log Q vs. log s curve	Suitable for all soil types
Chin (hyperbolic)	$s/Q = a + bs$; ultimate = 1/b	Smooths noisy data
Butt / Hansen 80 %	Load at 80 % of asymptotic capacity	Used for soft clays

Example plot below illustrates Davisson offset line vs. measured settlement curve.

(diagram placeholder – load–settlement curve with offset line)

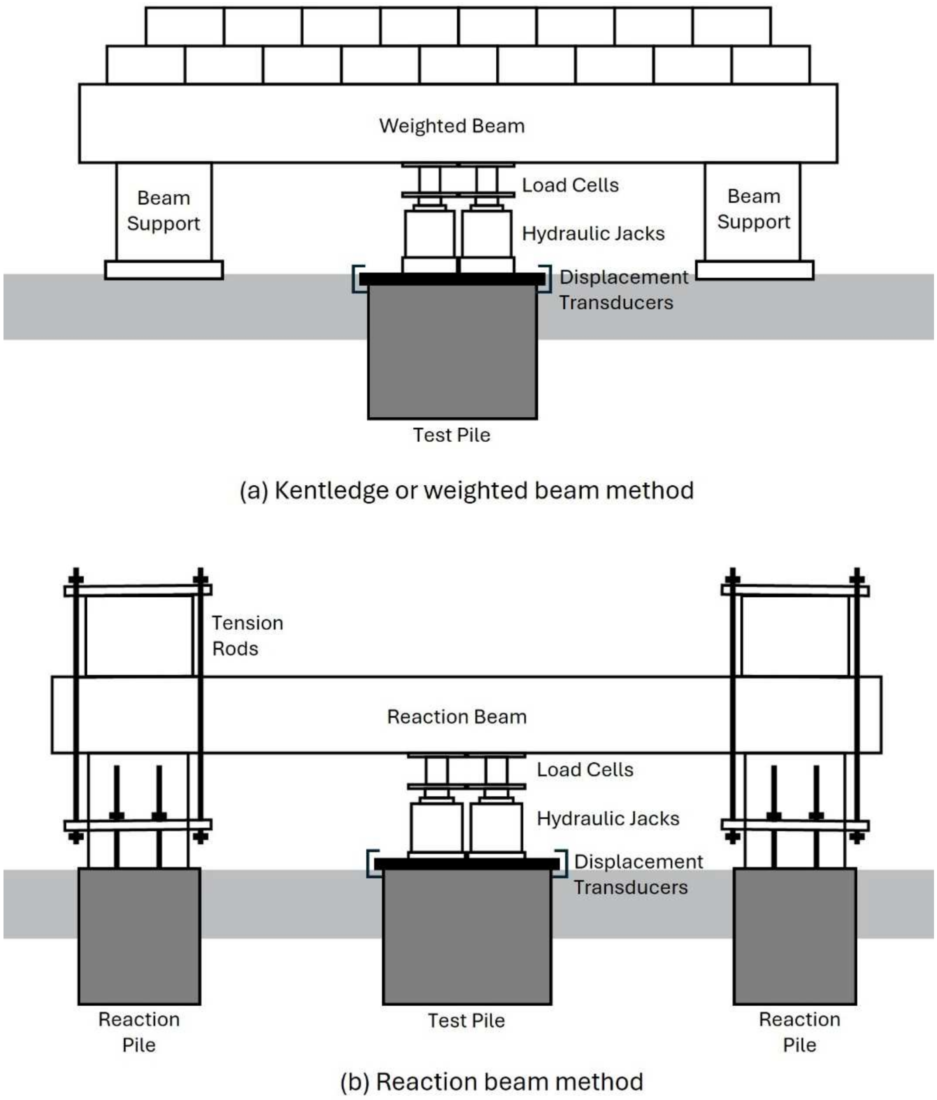


Figure 41: Static axial compression pile load test setup

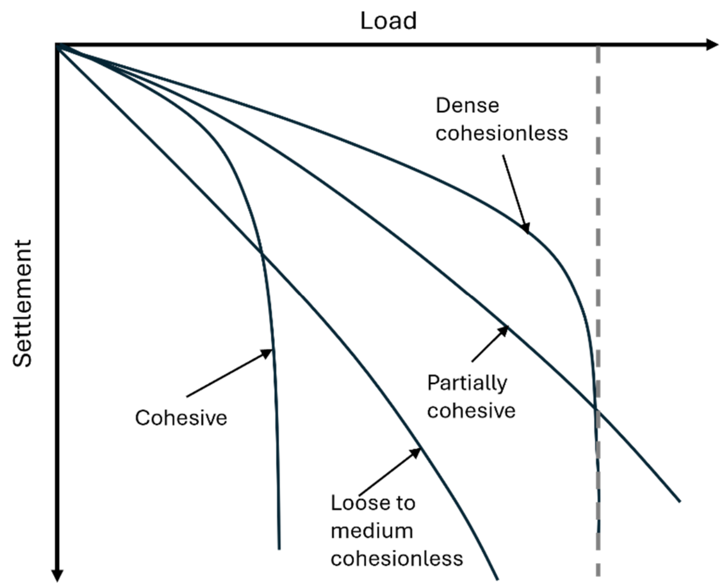


Figure 42: Typical load-settlement curve from static axial compression pile load test

11.4.4 Static Load Test (Tension / Uplift)

Procedure analogous to compression test but:

- Load applied upward (using reaction beams or anchors).
- Displacement measured at pile head.
- Typically tested to 1.3–1.5 × design uplift load.
- Load–displacement curve defines **adhesion mobilisation**.

Tension capacity:

$$Q_{t,ult} = \alpha_t c_u A_s$$

(see §11.2.7).

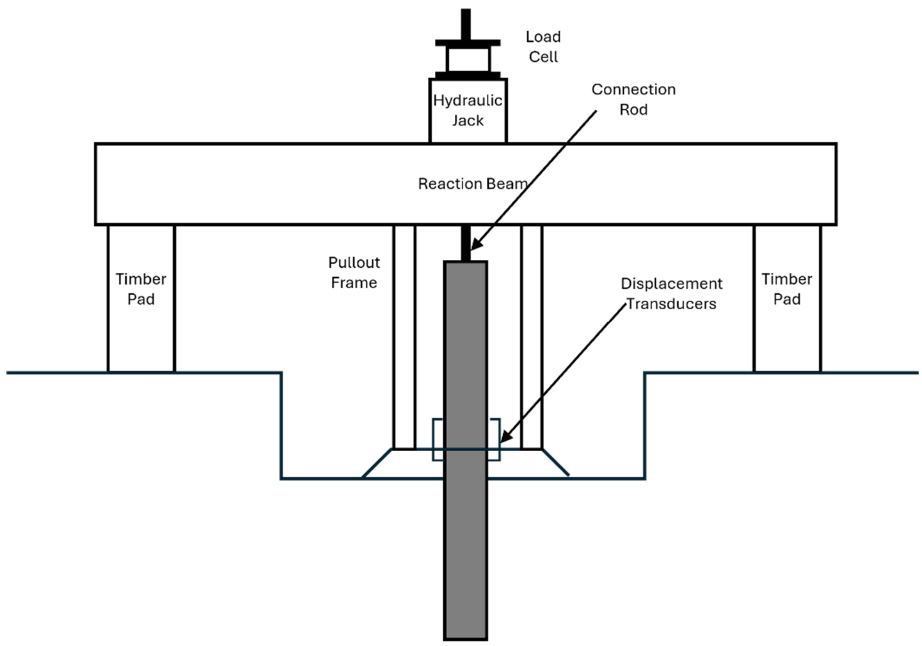


Figure 43: Static uplift pile load test setup

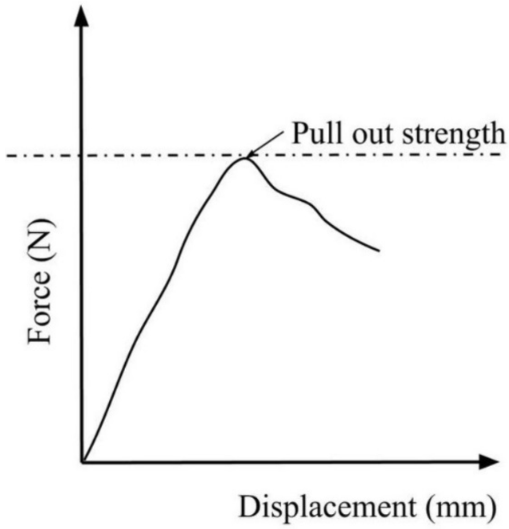


Figure 44: Typical load-displacement curve from static axial tension pile load test

11.4.5 Lateral Load Tests

Used for piles resisting horizontal loads (e.g., retaining walls, jetty piles).

- Incremental load applied laterally at cut-off level.
- Displacements recorded at top and along length (if instrumented).
- Compare measured deflection with predicted from **p–y** or **subgrade modulus** models (see §11.6).

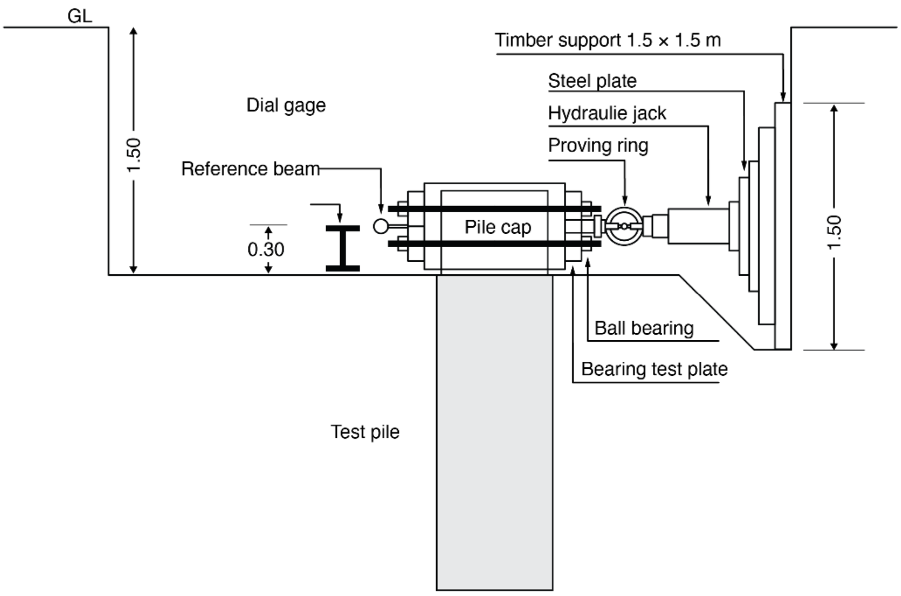


Figure 45: Setup for lateral load test on pile

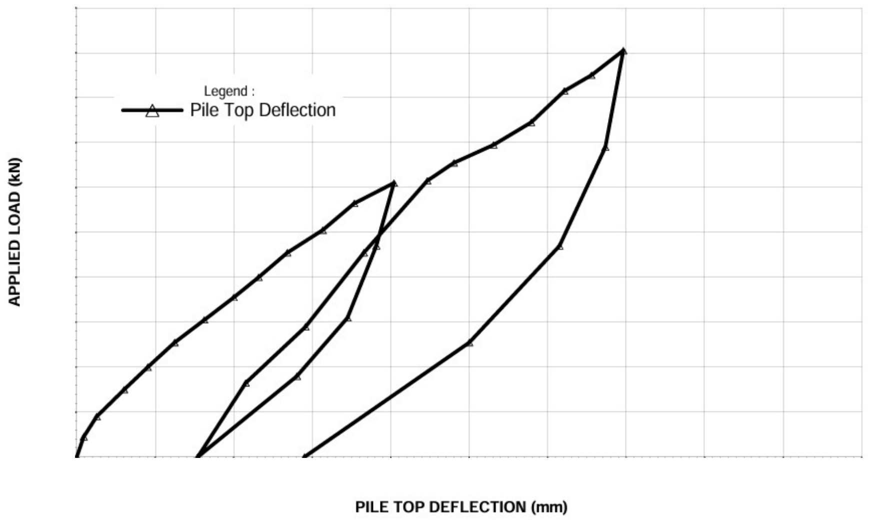


Figure 46: Typical load-deflection (pile head) curve from pile lateral load test

11.4.6 Dynamic Load Testing (PDA Method)

(a) Principle

The **Pile Driving Analyzer (PDA)** measures acceleration and strain near the pile head during hammer impact to estimate:

- Ultimate capacity ($Q_{(u)}$),
- Shaft vs. base contribution,
- Energy transfer and driving efficiency.

(b) Analysis

Dynamic formula (simplified Hiley approach):

$$Q_{dyn} = \frac{Wh\eta_h\eta_b}{S + C/2}$$

Where;

- W= ram weight,
- h= drop height,
- η_h, η_b= hammer and blow efficiency,
- S= final set per blow,
- C= elastic compression.

Modern PDA systems use the **Case Method (CAPWAP)** for refined analysis.

(c) Advantages

Pros	Cons / Limitations
Rapid, repeatable, economical	Requires calibration by static test
Suitable for production control	Unsuitable for bored/CFA piles
Indicates hammer energy efficiency	Interpretation depends on wave speed, damping

11.4.7 Instrumented Load Tests

- Strain gauges or load cells placed along the pile to separate **shaft** and **base** resistance.
- Useful for calibrating α-, β-, or Nq-methods for future design.
- Data plotted as mobilisation curves (τ vs. displacement) for each segment.

CIRIA C580 recommends at least one **instrumented test pile** for critical projects or novel piling systems.

11.4.8 Eurocode 7 Framework

Requirement	Reference	Summary
Pile testing for design verification	EN 1997-1 §7.6.2.3	Trial or proof piles shall confirm design parameters
Test procedure	EN ISO 22477-1 / -2	Compression, tension, and dynamic testing
Data interpretation	Annex D (informative)	Defines failure load determination methods
Reporting	EN 1997-2 §4.2.3	Include test setup, calibration, results, and analysis

At ULS:

$$R_d = \frac{Q_{ult,test}}{\gamma_R}$$

At SLS:

$$s_{measured} \leq s_{limit}$$

with $\gamma_R = 1.0$ (UK NA).

11.4.9 Typical Acceptance Criteria

Test Type	Acceptance Limit	Comment
Working test	Settlement ≤ 10 mm at 1× WL	For most buildings
Preliminary test	Settlement ≤ 10 % B at 2× WL	Determine ultimate
Uplift test	Movement ≤ 5 mm at WL	Limited by adhesion mobilisation
Lateral test	Deflection ≤ 5–10 mm at WL	Depends on pile stiffness

Always define acceptance criteria in the **Pile Test Method Statement** before testing begins

> **Phicon Note:**
>
> • Static load testing remains the **primary calibration tool** for all empirical design methods (α, β, or Nq).
> • In UK and EU practice, at least one **preliminary test pile** should be specified per site unless reliable local experience exists.
> • Dynamic testing is valuable for **production QA** but should never replace static testing for **design verification**.
> • Use test results to **back-analyse derived parameters** (α, β, Nq) — they often feed directly into future design refinements.
> • Archive raw data, calibration records, and interpreted curves in the **Geotechnical Design Report (GDR)** for audit and traceability.

11.5 Settlement and Serviceability Checks

11.5.1 Purpose

Settlement checks ensure that pile deformations under service loads do not impair the **function, appearance, or durability** of the structure.

Under **Eurocode 7 (EN 1997-1 §2.4.8)**, these are verified at the **Serviceability Limit State (SLS)** using **characteristic values** of soil stiffness and load — i.e. *no partial factors* are applied to actions or resistances.

11.5.2 Components of Pile Settlement

Total pile settlement at the pile head is the sum of three main components:

$$s_{total} = s_{shaft} + s_{base} + s_{elastic}$$

Component	Mechanism	Typical Contribution	Remarks
$\boldsymbol{s_{shaft}}$	Shear strain along pile–soil interface	30–60 %	Mobilised early (small load increments)
$\boldsymbol{s_{base}}$	Compression or yielding below pile tip	20–50 %	Mobilised gradually; depends on soil stiffness at toe
$\boldsymbol{s_{elastic}}$	Elastic shortening of pile material	10–20 %	Function of pile length and modulus

11.5.3 Elastic Shortening of Pile

$$s_{elastic} = \frac{QL}{AE_p}$$

Where;

- *Q*: Working (service) load on pile (kN)
- *L*: Embedded pile length (m)

- A: Cross-sectional area of pile (m^2)
- E_p: Elastic modulus of pile material (RC ≈ 30 GPa; steel ≈ 200 GPa) (kN/m^2)

For a 600 mm RC pile, L = 20 m at Q = 1500 kN, $s_{elastic} \approx 3–5 mm$.

11.5.4 Load–Settlement Behaviour

Typical load–settlement response observed in static tests:

1. **Initial Elastic Phase** – settlement proportional to load.
2. **Non-linear Mobilisation Phase** – increasing slope as shaft and base resistances develop.
3. **Yield / Plastic Phase** – rapid displacement with little load increase.

At **working load** (~ 50–60 % of ultimate), settlements should generally remain below **10–20 mm** for most structures.

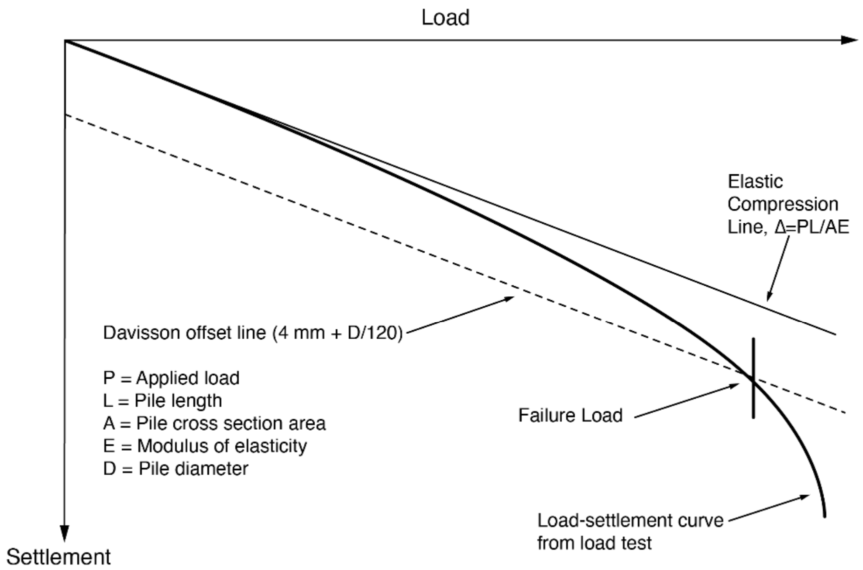


Figure 47: Typical behaviour of a load-settlement curve from static axial compression pile load test

11.5.5 Analytical Estimation of Settlement

(a) Shaft Deformation

$$s_{shaft} = \sum \frac{\tau_i}{G_i} \Delta z_i$$

where $\tau_i = \beta_i \sigma'_{v,i}$ and $G_i \approx E_s/[2(1+\nu)]$.
Integrate along pile shaft in layers (e.g., 2 m intervals).

(b) Base Deformation

Approximation after Vesic (1977):

$$s_{base} = \frac{q_b B}{E_s} I_b$$

$I_b \approx 0.6–0.9$ depending on pile rigidity and base embedment.

(c) Elastic Shortening

From § 11.5.3. Combine all three to obtain total settlement at working load.

11.5.6 Simplified Empirical Approaches

Source	Method / Formula	Applicable to	Typical s @ 1×WL
CIRIA C580	$s = 0.4 \times (qB/E_s)$	Bored piles in stiff clay / sand	5–15 mm
Poulos & Davis (1980)	Elastic solution for floating pile	General	0.5–2 % of D
NAVFAC DM 7.1	$s = Q/(k_s A)$	Driven piles in sand	5–10 mm
Tomlinson (1994)	$s \approx Q/(2E_s B)$	Sand & gravel	5–15 mm

11.5.7 Allowable Settlements (Structural Criteria)

Structure Type	Maximum Total Settlement	Maximum Differential Settlement	Typical Reference
Raft / mat foundation	50 mm	1/500–1/750	BS 8004:2015, Table 2
Building frame (on piles)	25 mm	1/500	CIRIA C580
Heavy industrial or tank base	75 mm	1/300	API / CIRIA
Bridge abutment / pier	20 mm	1/1000	BD 74/00 / DMRB
Sensitive equipment	10 mm	1/1000+	Project-specific

These are **indicative values** only — final tolerances should be agreed with the **structural designer** per EN 1990 Annex A2.

11.5.8 Group and Interaction Effects

Pile groups settle more than single piles under the same load due to stress overlap in the soil.

$$s_{group} = \mu s_{single}$$

where $\mu \approx 1.2–2.0$depending on pile spacing and soil type.

- **Spacing < 3D** → full interaction (μ > 1.5).
- **Spacing > 5D** → complete independent behaviour.

Use influence factors from Poulos & Davis charts or FE analysis for precise estimates.

11.5.9 Eurocode 7 Verification Steps

1. **Determine** characteristic ground stiffness (E_k, m_v) from tests or correlations (§§ 6 & 7).
2. **Compute** settlements at service load using unfactored values.
3. **Compare** with allowable or structural limits.
4. **Document** calculation method, parameter source, and assumptions in GDR.
5. **Verify** against measured load-test data if available.

At SLS:

$$s_{calc} \leq s_{lim}$$

with $\gamma_M = 1.0, \gamma_F = 1.0$.

11.5.10 Calibration with Field Tests

Test	Measured Output	Use in SLS Verification
Static Load Test	Load–settlement curve	Confirms stiffness assumed in design
Dynamic Test (PDA)	Equivalent static capacity	Compare predicted vs. back-analysed Q–s
Plate Load Test (EV_2)	Ground modulus near surface	Input for CFA / short piles
Instrumented Pile	Shaft/base strain	Derive E_s and β for future projects

11.5.11 Practical Considerations

- For **bored piles** in clay, part of settlement may occur gradually due to consolidation.
- **Driven piles** in sand often show negligible settlement (< 5 mm).
- Record **permanent set** after unloading; it indicates plastic deformation or soil relaxation.
- Compare **predicted vs. observed** settlements to refine design parameters for future works.

Phicon Note:

• Settlement verification is the **most defensible SLS check** under Eurocode 7 — it must be supported by explicit calculations and field data.
• The **calculated stiffness (E_s)** must always reflect the **stress level at service load**, not small-strain lab values.
• For pile groups, evaluate **both total and differential settlements** across the raft or pile cap.
• Record the **assumed load share** between piles in a group; differences in length or stiffness can cause rotation.
• When presenting results, include a **Q–s curve overlay** showing working-load settlement and ultimate capacity — this format is preferred in CAT II/III reviews.

11.6 Lateral Load Behaviour and Deflection Limits

11.6.1 Purpose

Piles resist lateral loads through **bending and shear** in the pile shaft and **reaction pressures** from surrounding soil.

Such loading arises from:

- Earth pressures (retaining walls, abutments)
- Wind, wave, or current forces (marine or tower structures)
- Horizontal components of inclined loads or eccentricities

Design ensures that:

1. The pile's structural bending strength (STR) is not exceeded, and
2. **Lateral deflections and rotations** remain within acceptable serviceability limits (SLS).

11.6.2 Governing Behaviour

The pile–soil system acts as a **flexible beam on an elastic foundation**, where soil resistance is modelled by subgrade reaction proportional to lateral displacement:

$$p = k_h y$$

Where;

- p: soil reaction pressure (kN/m^2)
- k_h: modulus of horizontal subgrade reaction (kN/m^3)
- y: lateral deflection (m)

This simple linear model provides the basis for both analytical and numerical design.

11.6.3 Modulus of Subgrade Reaction (k_h)

The modulus varies linearly with depth:

$k_h = n_h z$	n_h (MN/m³ per m)	k_h at 1 m depth (MN/m³)	Comment
Soft clay	5–10	5–10	Low lateral resistance
Firm clay	10–30	10–30	Moderate resistance
Stiff clay	30–80	30–80	Non-linear response
Loose sand	15–40	15–40	Compressible
Medium-dense sand	40–100	40–100	Typical for CFA piles
Dense sand / gravel	100–300	100–300	High stiffness, strong rebound

Values depend on strain level and pile diameter.
Empirical correlations exist between k_h and **SPT N**, **CPT q_c**, or **EV_2** where site-specific data are limited.

11.6.4 Broms' Simplified Design Method

A long-established analytical method for preliminary design. Assumes *rigid–plastic soil response* and ultimate conditions.

(a) Free-Head Piles (Unrestrained Top)

$$H_u = 2P_u L_e$$
$$M_{max} = 0.4 P_u L_e$$

where $P_u = 3c_u D$ (for clay) or $P_u = K_p \gamma' D L_e$ (for sand).

(b) Fixed-Head Piles (Restrained Top)

$$H_u = 3P_u L_e$$
$$M_{max} = 0.25 P_u L_e$$

L_e = effective embedded length (portion of pile mobilising full resistance).
Typically $L_e \approx 2$–$4D$ for stiff clay, 4–$6D$ for sand.

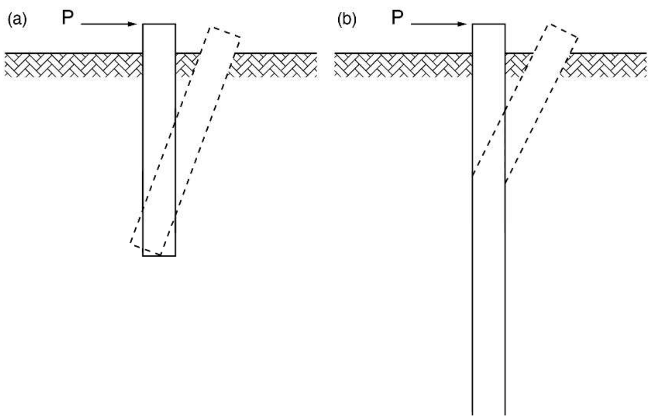


Figure 48: Failure modes for free headed piles: (a) rotation; (b) hinge failure (Broms, 1964)

11.6.5 p–y Curve Method

Modern design (per EN 1997-1 and API RP 2A) uses **non-linear soil springs** representing real soil behaviour.

(a) Basic Relationship

$$p = p_u \tanh\left(\frac{y}{y_{50}}\right)$$

Where;

- p_u: ultimate soil resistance,
- y_{50}: lateral displacement at 50% of p_u.

(b) Typical Parameters

Soil Type	$p_{(u)}$ (kPa)	y_{50} (mm)	Comment
Soft clay	3 c_u	25–50	Time-dependent response
Stiff clay	9 c_u	10–20	Quick mobilisation
Loose sand	30–40 γ'z	5–10	Linear to 10 mm
Dense sand	80–120 γ'z	5–8	Non-linear hardening

These are applied as distributed springs along the pile length in **p–y analysis software** (e.g. LPILE, PLAXIS, Wallap).

11.6.6 Deflection and Moment Profiles

For design, key parameters are:

- Maximum lateral deflection (y_{max}) at pile head,
- Maximum bending moment (M_{max}), and
- Point of zero bending moment (inflection).

Typical profiles (from p–y analysis) show:

- M_{max} occurs at depth ≈ 0.5–1.0D below ground.
- Rotation occurs mainly in top 5–10D zone.
- Below 10D, soil reactions largely balanced.

11.6.7 Serviceability Deflection Limits

Application	Limit (Head Deflection)	Rotation (at ground)	Reference / Practice
Building foundations	≤ 10 mm	≤ 1/500	BS 8004, CIRIA C580
Bridge abutments	≤ 15 mm	≤ 1/300	BD 74/00
Retaining wall supports	≤ 25 mm	≤ 1/200	C580, EC7 commentary
Offshore / marine piles	≤ 50 mm	≤ 1/100	API RP 2A
Temporary works	≤ 75 mm	—	Engineer's judgement

These limits ensure both structural integrity and operational function (alignment, drainage, bearings, etc.).

11.6.8 Eurocode 7 Verification Framework

At **ULS (STR + GEO):**

- Verify combined moment, shear, and axial load within section capacity:

$$\frac{N_d}{N_{Rd}} + \frac{M_d}{M_{Rd}} \leq 1.0$$

- Include lateral earth pressure and surcharge as design actions (γ_F = 1.35 or 1.5).

At **SLS:**

- Check head deflection ≤ limit from Table §11.6.7.
- Apply characteristic stiffness (E_k, k_h, G_s) with γ = 1.0.
- Document results in **GDR**, including plots of y–z, M–z, and p–y relationships.

11.6.9 Numerical Modelling

Finite element or beam-spring models (e.g. **PLAXIS 2D/3D**, **Oasys Pile**, **Wallap**) simulate full interaction:

- Apply layered soil stiffness and interface properties.
- Verify both **deflection** and **bending moment** envelopes.
- For groups, apply **interaction factors** or model equivalent raft stiffness (see §12.2).

11.6.10 Example Check (Simplified)

Given:
CFA pile, D = 0.6 m, L = 15 m, medium-dense sand (γ' = 10 kN/m^3, k_h@1m = 60 MN/m^3).
Applied horizontal load = 100 kN, free head.

Approximate head deflection (Poulos & Davis, 1980):

$$y_0 = \frac{H}{2E_pI_p/L_e^3 + k_hL_e}$$

Taking $L_e = 5D = 3\ m$, $E_pI_p = 30 \times 10^9 \times \pi(0.6^4/64) = 0.19 \times 10^9$.

$$y_0 \approx \frac{100}{\left(2 \times 0.19 \times \frac{10^9}{27}\right) + 60 \times 10^6 \times 3} = 100/(14.14 \times 10^6 + 180 \times 10^6) = 0.00051\ m = 0.5\ mm$$

✓ Predicted head deflection = 0.5 mm — well within serviceability limit.

Phicon Note:

• For routine design, use **Broms' method** for quick checks and **p–y analysis** for detailed verification.
• Always distinguish between **free-head** and **fixed-head** conditions — this changes both capacity and deflection by up to a factor of 3.
• Confirm that assumed **soil stiffness (k_h)** is appropriate for the strain range (< 0.5 %); overly stiff models underestimate deflection.
• In the GDR, include plots of **y–z** and **M–z** for at least one representative pile; reviewers expect traceable curves.
• For groups or rows of raked piles, consider **interaction and group shadowing effects** — lateral stiffness may reduce by 30–50 %.

12 Pile Groups, Rafts & Piled Rafts

12.1 Group Efficiency and Block Failure

12.1.1 Purpose

When piles act in a **group**, the interaction between individual pile stress zones alters both the **ultimate capacity** and **settlement behaviour**.

Design must therefore check:

1. **Individual pile capacity** (from single-pile analysis), and
2. **Group capacity** (as a block or equivalent mass of soil–pile system).

The **lower** of these governs the **group ultimate resistance (R_g)** for Eurocode 7 verification.

12.1.2 Pile Group Mechanisms

Two principal mechanisms govern pile group behaviour:

Mechanism	Description	Typical Condition
Individual Pile Action	Each pile mobilises its own shaft and base resistance independently	Large spacing (s ≥ 3D)
Block / Group Action	Piles and enclosed soil act as one large block	Close spacing (s ≤ 2.5D) or cohesive soils
Intermediate Behaviour	Partial interaction between adjacent piles	Most practical group layouts

For design, both mechanisms must be considered — and the smaller resistance adopted (EN 1997-1 §7.6.3.3).

12.1.3 Group Efficiency (η_g)

Group efficiency defines the ratio of actual group capacity to the sum of individual pile capacities:

$$\eta_g = \frac{Q_{group}}{nQ_{single}}$$

Typical efficiency ranges:

Soil Type	Pile Spacing (s/D)	η_g (Typical)	Comments
Dense sand	3D–4D	0.9–1.0	Interaction small
Medium sand	2.5D–3D	0.7–0.9	Overlap of stress bulbs
Soft clay	2.5D	0.6–0.8	Strong block tendency
Firm clay	3D	0.8–0.9	Moderate overlap
Stiff clay	≥ 3D	0.9–1.0	Piles behave independently

Where piles are arranged in rows, **shadowing effects** (leading row taking higher load) should be considered in structural design.

12.1.4 Block Failure Check

If the group behaves as a single block, ultimate capacity may be obtained using the **block method**:

$$Q_{block} = c' N_c A_b + 0.5\gamma' B_{eq} N_\gamma A_b + \gamma' D_f N_q A_b$$

for **drained (sand)** conditions, or

$$Q_{block} = c_u N_c A_b$$

for **undrained (clay)** conditions.

where A_b= plan area enclosed by pile group (outer perimeter).

The **block width (B_{eq})** is taken as the group width, and the **depth (H)** equals the pile embedment.

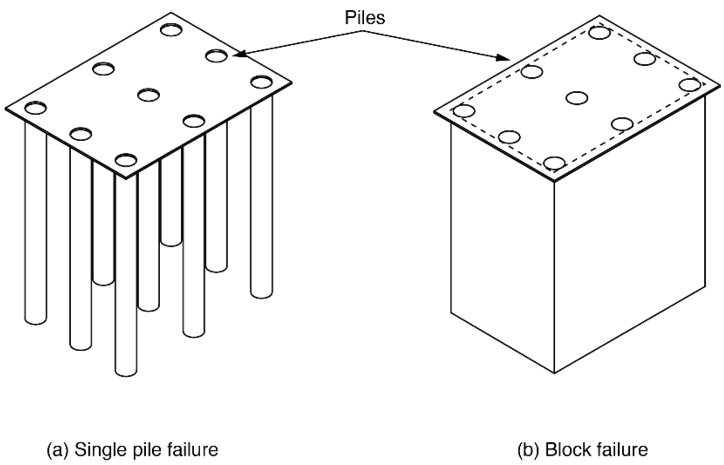


Figure 49: Failure modes for pile groups (reproduced from O'Brien 2012)

12.1.5 Shaft and Base Sharing

In cohesive soils (α-method):

$$Q_{group} = \alpha c_u (A_{shaft,group} + N_c A_{base,group})$$

where:

- $A_{shaft,group}$= perimeter × pile length,
- $A_{base,group}$= group block area.

Driven groups often exhibit higher α due to soil densification; bored piles typically lower.

12.1.6 Empirical Group Efficiency Equations

(a) Converse–Labarre Formula (1936)

$$\eta_g = 1 - m(\frac{D^2}{s^2})$$

where $m = 0.5–0.7$ depending on soil compressibility.

(b) Prakash & Sharma (1990) – for clays

$$\eta_g = 1 - 0.1(n_r - 1)(n_c - 1)\frac{D}{s}$$

where n_r, n_c are number of piles in rows and columns.

Use as **approximate guidance only** — actual efficiency should be validated by field test data or numerical analysis.

12.1.7 Settlement of Pile Groups

Group settlement typically exceeds that of a single pile under the same load due to overlapping stress zones.

$$s_g = \mu s_{single}$$

where $\mu = 1.2–2.0$ depending on pile spacing and soil stiffness (see §11.5.8).

In clays, settlement is dominated by **consolidation** of soil beneath the group base; in sands, by **elastic deformation** of soil around the piles.

12.1.8 Design Approach under Eurocode 7

At **ULS (GEO):**

- Check both individual pile capacity and block capacity:

$$R_{group,d} = \min\left(\eta_g n R_{single,d}, R_{block,d}\right)$$

- Adopt characteristic values of ϕ', c', or c_u with partial factor γ_m = 1.25.

At **SLS:**

- Use characteristic stiffness (E_k, m_v) without partial factors.
- Verify **settlement and rotation** within serviceability limits.

12.1.9 Worked Example (Simplified)

Given:
4 × 4 pile group (16 piles), D = 0.6 m, s = 2.4 m (s/D = 4), ϕ' = 35°, γ' = 10 kN/m^3.
Single pile $Q_{ult} = 2500\ kN, \eta_g = 0.9$.

$$Q_{group} = 0.9 \times 16 \times 2500 = 36{,}000\ kN$$

Group block width: $B_{eq} = (Pile\ Number - 1) \times Spacing + Dia. = (n - 1) \times 2.5D + D = (4 - 1) \times 2.5 \times 0.6 + 0.6 = 5.1\ m$

Block base capacity check (using Nq = 40, D_f=18 m):

$$Q_{block} = N_q \gamma' D_f A_b = 40 \times 10 \times 18 \times 5.1^2 = 187{,}272\ kN$$

✓ Governed by individual-pile group efficiency (36,000 kN).

12.1.10 Practical Design Notes

- If $s/D \geq 3$, piles behave largely independently — use $\eta_g \approx 1.0$.
- For dense sand, block failure is rare; for soft clay, always check both.
- When group efficiency < 0.7, consider **larger spacing**, **increased length**, or **raft integration**.
- In layered soils, ensure piles penetrate through soft strata to competent layers to avoid **group punching**.

Phicon Note:

• Always evaluate both **individual pile** and **block** capacity modes — EC7 verification must adopt the *least favourable*.
• Document η_g, s/D, and assumed load-sharing in the GDR for transparency in CAT II/III reviews.
• Consider group settlement at SLS even if ULS capacity is satisfactory; serviceability often governs.
• For closely spaced bored piles, **negative skin friction** and group down-drag can significantly reduce working capacity (address in §12.2).
• Where feasible, verify model assumptions using **instrumented test piles** or **static load tests** on groups.

12.2 Group Settlement and Interaction

12.2.1 Purpose

When piles act as a group, their settlement is typically greater than that of a single pile under the same load because the **stress bulbs beneath each pile overlap**, reducing the effective stiffness of the surrounding soil.

Design must therefore predict both:

- **Total group settlement** (absolute movement), and
- **Differential settlement** across the pile cap or raft.

Eurocode 7 (§2.4.8) requires settlement checks to be carried out at **SLS using characteristic values** of soil stiffness and unfactored loads.

12.2.2 Key Influencing Factors

Factor	Effect on Settlement	Typical Trend
Pile spacing (s/D)	Controls degree of interaction	s/D ↑ → interaction ↓
Soil compressibility (E_s, m_v)	Governs deformation modulus	Soft clay → high settlement
Pile length and stiffness (E_pI_p)	Determines load transfer ratio	Longer / stiffer → less settlement
Group geometry (rows × columns)	Influences load distribution	Edge piles less confined
Load share (axial vs moment)	Alters deformation pattern	Eccentric loading amplifies differential movement

12.2.3 Group Settlement Ratio (μ)

The **group settlement ratio (μ)** defines the increase in settlement relative to a single pile:

$$\mu = \frac{s_{group}}{s_{single}}$$

Typical values:

Soil Type	s/D	μ (range)	Comment
Dense sand	≥ 4	1.0–1.2	Little interaction
Medium sand	3–4	1.2–1.5	Moderate overlap
Soft clay	2–3	1.5–2.0	Strong group effect
Firm clay	3–4	1.3–1.6	Common UK practice assumption

Approximation (Tomlinson, 1994):

$$\mu = 1 + 0.5(\frac{D}{s})$$

but capped at μ ≤ 2.0.

12.2.4 Poulos & Davis Elastic Interaction Method

The most widely accepted analytical approach to predict group settlement and load sharing.

$$s_g = \frac{Q}{nk_p(1 + \Sigma\alpha_{ij})}$$

Where;

- Q= total group load,
- n= number of piles,
- k_p= stiffness of a single isolated pile,
- α_{ij}= interaction factor between piles *i* and *j*.
- Interaction Factor ($\alpha_{(ij)}$):

Depends on:

$$\alpha_{ij} = f(\frac{s}{L}, \frac{E_s}{E_p}, \frac{\nu_s}{\nu_p})$$

Typical ranges:

- 0.1–0.3 for s/L ≥ 2 (small interaction)
- 0.4–0.6 for s/L ≤ 1 (strong interaction)

For preliminary design, total settlement ≈ 1.5 × single pile settlement for s/D ≈ 3 in clay.

12.2.5 Randolph's Equivalent Raft Concept (1983)

For large groups, the pile cap and group behave as an **equivalent raft** founded at an "equivalent depth" z_{eq} below the cap.

The raft stiffness K_r and pile stiffness K_p act in parallel:

$$K_{total} = K_p + K_r$$

The **load share** carried by piles vs. soil is:

$$\eta_p = \frac{K_p}{K_p + K_r}, \eta_s = \frac{K_r}{K_p + K_r}$$

Typical values:

- $\eta_p \approx 0.6–0.8$ for heavily piled rafts,
- $\eta_p \approx 0.4–0.6$ for moderately piled rafts.

This approach underpins modern **piled raft design**, described further in §12.3.

12.2.6 Simplified Empirical Expressions

- **(a) Soft Clay (Poulos, 1979)**

$$s_g = s_{single}(1 + 0.5\frac{B_g}{s})$$

where B_g = group width.

- **(b) Sand (Vesic, 1969)**

$$s_g = s_{single}(1 + 0.2\frac{B_g}{s})$$

These formulas approximate average settlement for square or rectangular groups.

12.2.7 Estimating Settlement from Load Distribution

Settlement beneath a pile group can also be assessed using the **stress distribution beneath the pile cap**:

$$\sigma_z = q(\frac{1}{1 + (z/B_g)^2})^{1.5}$$

and

$$s_g = \int_0^{2B_g} \frac{\sigma_z}{E_s} dz$$

This "equivalent raft" integration can be implemented easily in spreadsheets to estimate SLS settlement without FE software.

12.2.8 Eurocode 7 Verification Framework

At **ULS (GEO):**

$$R_{group,d} = \eta_g n R_{single,d} \text{ or } R_{block,d}$$

(check both per §12.1).

At **SLS:**

$$s_{calc} \leq s_{lim}$$

- Use characteristic stiffness (E_k, m_v),
- Apply γ_m = 1.0,
- Include **interaction factor (μ)** explicitly in design report.

Document:

- Input parameters (E_s, s/D, η_g, μ),
- Method used (Poulos–Davis, Randolph, empirical),
- Comparison with test or case history data.

12.2.9 Typical Settlement Ranges

Foundation Type	Soil Type	s_{total} (mm)	Notes
Isolated pile (bored, stiff clay)	Clay	5–15	Immediate + consolidation
4×4 group, s/D = 3	Clay	15–30	Group interaction
Pile raft (moderately piled)	Sand	10–25	Combined stiffness effect
Large piled raft (tall building)	Mixed	20–50	Usually within limits

12.2.10 Practical Field Observations

- Settlement often governed by **soil consolidation** beneath pile tips, not by pile elasticity.
- In groups, **outer piles settle less** than central piles — causing potential bending in pile cap.
- **PVDs or preloading** can reduce long-term group settlement in soft clays.
- Use **instrumented test piles** or **settlement plates** to calibrate numerical predictions.

Phicon Note:

• For EC7 SLS verification, always include both **individual-pile** and **group-interaction** assessments.
• The **Poulos–Davis interaction factor approach** is recommended for spreadsheet-based verification; Randolph's method for FE or piled raft design.
• Keep pile spacing ≥ 3D where possible — below this, settlement amplifies rapidly.
• Compare predicted vs. measured settlements; discrepancies >30 % suggest re-evaluation of E_s or μ.
• Always record **E_s profiles, pile spacing, μ, and η_g** values in the GDR for traceability and CAT III review compliance.

12.3 Raft Design & Differential Settlement Control

12.3.1 Purpose

Raft (or mat) foundations distribute structural loads over a wide area of soil, reducing bearing pressure and differential movements between columns or walls.

They are often adopted where:

- Multiple closely spaced columns make isolated footings impractical,
- Soils are moderately compressible (clays, silts),
- Differential settlement control is critical (tanks, basements, framed buildings), or
- They serve as **piled rafts** combining raft and pile support (see § 12.4).

Eurocode 7 (§ 2.4.8) treats rafts as **shallow foundations**, verified for both **ultimate limit states (ULS)** and **serviceability limit states (SLS)**, with differential settlement limits defined by structural tolerance.

12.3.2 Load Transfer Mechanism

Raft load distribution depends on **relative stiffness** between the raft and subsoil.

Condition	Description	Design Assumption
Rigid raft	Uniform rotation and settlement; soil pressures vary linearly	Simplified elastic or Winkler model sufficient
Flexible raft	Curvature and variable deflection across plan	Requires FE or plate–spring analysis
Piled raft	Combined stiffness from soil + piles	Modelled as parallel springs or full 3D continuum

⚠️ In practice, most reinforced concrete rafts behave *semi-rigidly* — deflection pattern lies between uniform and flexible extremes.

12.3.3 Governing Equations (Elastic Analysis)

For an elastic raft on a homogeneous soil:

$$s = \frac{qB(1-\nu_s^2)}{E_s} I_s$$

where I_s is the shape influence factor (≈ 1.0 ± 0.2).

For variable stiffness across the raft, use **finite-element plate analysis** (PLAXIS, SAFE, Robot, etc.) with soil modulus $E_s$or subgrade modulus k_s.

12.3.4 Modulus of Subgrade Reaction (k_s)

Soil Type	Typical k_s (MN/m^3)	Notes
Soft clay	10–30	Time-dependent, check consolidation
Firm clay	30–80	Common for mat foundations
Dense sand / gravel	80–200	High stiffness, low long-term creep
Engineered fill (EV_2 ≈ 120–180 MPa)	50–100	Back-calculated via $k_s \approx E_s/B$

For rafts > 10 m wide, k_s decreases with increasing width:

$$k_s \approx k_{s,1m}(\frac{1}{B})^{0.5}$$

12.3.5 Settlement Distribution and Differential Movement

For uniform raft load:

$$s_{max} = \frac{qB(1-\nu_s^2)}{E_s}$$

For variable pressure (centre-loaded raft or asymmetric geometry), use influence charts or FE output.

Angular distortion (β):

$$\beta = \frac{\Delta s}{L}$$

Structure Type	Limiting β (rad)	Approx. Ratio (1/β)	Reference
Heavy masonry	1/1000–1/1500	0.001–0.0007	Skempton & MacDonald (1956)
Steel / RC frames	1/500–1/750	0.002–0.0013	BS 8004, CIRIA C580
Sensitive equipment	1/2000	0.0005	Project-specific
Tanks / reservoirs	1/1000	0.001	API 650 / Eurocode guidance

⚠️ Differential settlement, not total, typically governs raft performance.

⚠️ Check both **short-term (elastic)** and **long-term (consolidation)** components.

12.3.6 Verification Workflow (Eurocode 7)

1. **Define inputs:** soil stiffness (E_k or m_v), Poisson's ratio, raft geometry, applied loads.
2. Analyse raft response:
    - Rigid or elastic model → compute settlements and rotations.
3. Check limit states:

   - ULS: bearing and punching checks (see § 9.3).
   - SLS: $s_{calc} \le s_{lim}, \beta_{calc} \le \beta_{lim}$.
4. **Document** results and assumptions in GDR.
5. **If excessive:** modify thickness, stiffness, or introduce piles (see § 12.4).

Partial factors at SLS:

$$\gamma_F = 1.0, \gamma_M = 1.0$$

At ULS (DA1/C2):

$$\gamma_F = 1.35\text{–}1.5, \gamma_M = 1.25$$

12.3.7 Reducing Differential Settlement

Approach	Effect	Typical Application
Increase raft thickness	Improves flexural rigidity	Medium → large rafts
Vary reinforcement / stiffness	Reduces bending concentration	Irregular loading
Stiffen with ribs or beams	Redistributes load	Long strips / wall lines
Provide piles under high-load zones	Hybrid piled raft	Bridges, tanks, towers
Preload or compact fill	Reduces future consolidation	Embankments, slabs
Use compressible inclusion layers	Absorbs differential movement	Basements on variable ground

12.3.8 Modelling Considerations

- **Subgrade model:** Winkler (spring) for quick checks; continuum (FE) for detailed design.
- **Soil modulus:** choose value corresponding to working stress level (≤ 25–50 % $q_{(u)}$).
- **Layered ground:** use weighted modulus or assign zones in FE mesh.
- **Water pressure:** include buoyant effects when below groundwater.
- **Creep / secondary settlement:** consider for organic or compressible clays.

12.3.9 Typical Settlement Ranges

Ground Type	Approx. $s_{(tot)}$ (mm)	Comments
Dense sand / gravel	5–15	Often within construction tolerance
Firm clay	15–40	Differential may govern
Soft clay	40–100+	Consider ground improvement or piles

Engineered fill	10–25	Verify compaction (EV_2 > 120 MPa)

12.3.10 Worked Example (Simplified)

Given:
Raft 12 m × 12 m, thickness = 0.6 m, load = 250 kPa, $E_s = 30\ MPa, v_s = 0.3$.

$$s = \frac{250 \times 12(1 - 0.3^2)}{30{,}000} = 9.1 \times 10^{-2} = 0.091\ m = 91\ mm$$

Differential over half span (6 m):

$$\beta = 91/6000 = 1/66 \approx 0.0152\ rad$$

✓ Exceeds limit for typical building (1/500 = 0.002 rad) → increase thickness or introduce piles.

12.3.11 Construction and QA Notes

- Confirm **bearing stratum continuity** via trial pits or boreholes before casting.
- Record **level survey points** for as-built verification and post-construction monitoring.
- If constructed over variable fill, consider **rigid inclusions** or **piled transition zones**.
- For waterproof or basement slabs, check that differential movement does not compromise waterstops or joint sealing.

Phicon Note:

• Raft design is often governed by **differential movement**, not bearing pressure — check rotations explicitly.
• Use **equivalent spring stiffness models** for quick assessments, then refine using FE analysis.
• For large basements, integrate **structural stiffness and soil interaction** within the same FE model — Eurocode 7 encourages soil–structure interaction (SSI) verification.
• Always state **basis of modulus selection (E_k)**, loading stage, and **drainage condition** in the GDR.
• Include both **absolute** and **differential settlement** results in summary tables — these are routinely reviewed during CAT II/III checks.

12.4 Hybrid Piled Rafts

12.4.1 Purpose

A **hybrid piled raft** combines a raft foundation with a limited number of piles to control **settlement and differential movement** rather than to increase ultimate bearing capacity.

This composite system takes advantage of:

- The raft's **wide-area bearing** and bending stiffness, and
- The piles' **vertical stiffness** and **load transfer** to deeper competent strata.

Eurocode 7 treats piled rafts as a *composite geotechnical system*, verified for both **ULS** (bearing and block failure) and **SLS** (settlement and angular distortion).

12.4.2 Behaviour and Load Sharing

Under load, both raft and piles deform simultaneously.
The **load share** depends primarily on:

- Relative stiffness of piles, raft, and soil,
- Pile spacing and arrangement,
- Soil compressibility, and
- Pile–raft interaction (stress overlap and group effects).

Define:

$$Q_{total} = Q_p + Q_r$$

$$\eta_p = \frac{Q_p}{Q_{total}}, \eta_r = \frac{Q_r}{Q_{total}}$$

with $\eta_p + \eta_r = 1$.

Typical load-sharing ranges:

- **η_p ≈ 0.5–0.8** → piles carry majority of load (dense sand or stiff clay)
- **η_p ≈ 0.3–0.5** → balanced system (soft to firm clay, wide raft)
- **η_p < 0.3** → raft-dominant, piles act as settlement control only

12.4.3 Advantages

Aspect	Benefit
Economy	Fewer piles required than fully piled foundations
Performance	Reduced total and differential settlement
Structural synergy	Raft carries more load once piles mobilise
Redundancy	Provides reserve capacity if individual piles underperform
Adaptability	Suitable for variable ground or partial soft zones

12.4.4 Design Philosophy (EN 1997-1 Context)

EN 1997-1 §7.6.2 states that piled foundations must be verified for both:

- **ULS** — bearing resistance, shaft capacity, structural strength; and
- **SLS** — settlements and load distribution.

For piled rafts:

1. The raft is **treated as a flexible plate** resting on a spring-supported medium (subgrade reaction, k_s).
2. Piles are **represented as vertical springs** with stiffness k_p, positioned at their actual plan locations.

3. Combined response is evaluated for:
    - Load distribution: η_p, η_s
    - Settlements and rotations
    - Bending moments in raft
    - Axial load in piles

12.4.5 Analytical Modelling (Randolph & Wroth, 1978; Poulos, 2001)

(a) Equivalent Spring Method

- Raft and piles modelled as linear springs in parallel.
- Settlement compatibility enforced:

$$Q_p = k_p(s - s_0), Q_r = k_r s$$

where s_0 is the relative displacement between pile head and raft underside.

(b) Load Sharing Ratio

$$\eta_p = \frac{k_p}{k_p + k_r}, \eta_r = \frac{k_r}{k_p + k_r}$$

(c) System Stiffness

$$K_{total} = K_p + K_r$$

with $K_p = \sum k_{p,i}$ and $K_r = k_s A_r$.

These relationships form the basis for most commercial software (e.g. PLAXIS 3D, Repute, GARP, PIGLET).

12.4.6 Typical Stiffness Ratios and Performance

Ground Type	Pile Type	η_p	Typical Reduction in Settlement
Soft clay	Bored piles (L = 15–25 m)	0.6–0.8	50–70 %
Firm clay	CFA piles (L = 10–20 m)	0.4–0.6	40–60 %
Dense sand	Driven piles (L = 10–15 m)	0.3–0.5	30–40 %
Variable fill over rock	Mini-piles / micropiles	0.2–0.4	20–30 %

12.4.7 ULS Verification (Eurocode 7, §7.6.2.3)

- **Check 1:** Bearing resistance of raft:

$$V_{d,raft} \leq R_{d,raft}$$

- **Check 2:** Axial capacity of piles (individual and group):

$$Q_{p,i,d} \leq R_{p,i,d}$$

- **Check 3:** Global block or overall failure (raft + pile zone):

$$V_{total,d} \leq R_{block,d}$$

Apply partial factors:

- **Actions:** γ_F = 1.35 (permanent), 1.5 (variable)
- **Materials:** γ_M = 1.25 (for ϕ', c', or c_u)

12.4.8 SLS Verification

At service load:

- No partial factors applied ($\gamma_F = \gamma_M = 1.0$).
- Compute total and differential settlement from soil–structure interaction model.
- Verify:

$$s_{calc} \leq s_{lim}, \beta_{calc} \leq \beta_{lim}$$

- Check load share:

$$\eta_p = \frac{\Sigma Q_{piles}}{Q_{total}} \text{ within expected range (0.4–0.8)}$$

- Ensure no pile exceeds its elastic capacity at SLS.

12.4.9 Differential Settlement Control

Technique	Effect	Application
Vary pile lengths	Stiffens high-load zones	Towers, transfer structures
Adjust pile spacing	Controls stiffness gradient	Rafts under asymmetric loads
Raft thickening or ribs	Reduces bending and local settlement	Basements, tanks
Localised pile clusters	Support heavy columns	Portal frames
Geogrid or stone columns below raft	Improves subgrade stiffness	Industrial slabs, fills

The goal is *settlement compatibility*, not zero movement — uniform deformation is acceptable provided differential limits are met.

12.4.10 Design Example (Simplified)

Given:
Raft 15 m × 15 m, load = 450 kPa.

12 bored piles, D = 0.6 m, L = 20 m.
$E_s = 25\ MPa$, $k_s = 50\ MN/m^3$, $k_p = 800\ MN/m$.

$$\eta_p = \frac{k_p}{k_p + k_r} = \frac{12\ \times 800}{12\ \times 800 + 50 \times 15 \times 15} = 0.46$$

⇒ Piles carry 46 % of total load; raft carries 54 %.

Total settlement (from elastic model):

$$s_{total} = \frac{Q}{K_p + K_r} = \frac{15 \times 15 \times 450}{(800 \times 12 \times 1000) + (50 \times 225 \times 1000)} = 0.004856\ m = 4.86\ mm$$

✓ Within SLS limit for framed structure (25 mm total, 1/500 differential).

12.4.11 Construction and QA

- Confirm **pile continuity into competent layer**; verify via static test or sonic integrity.
- Record **load–settlement tests** on at least one working pile.
- Survey **raft level grid** post-pour and periodically post-loading.
- Ensure **drainage, waterproofing, and joint details** accommodate minor elastic movement.
- For sensitive structures, install settlement pins or hydrostatic levelling systems.

Phicon Note:

• Hybrid piled rafts are most effective where **settlement control**, not capacity, governs design.
• Always model both **raft–soil** and **pile–soil** stiffness explicitly — this is a key EC7 expectation under SLS verification.
• Avoid over-piling; use piles strategically where loads or settlements are highest.
• Include **load share ratio (η_p)** and **predicted settlement contour plots** in the GDR for clarity.
• In CAT III checks, reviewers expect to see both **p–y or t–z data** and **raft reaction plots** — ensure traceable assumptions.

13 Lateral Earth Pressure Theory

13.1 At-Rest (K_0), Active (K_a), and Passive (K_p) Conditions

13.1.1 Purpose

Lateral earth pressure is the **horizontal stress exerted by soil** on a vertical or near-vertical boundary, such as a retaining wall, basement, or pile.

Understanding and correctly evaluating these pressures is fundamental to the design of retaining and embedded structures under Eurocode 7 (EN 1997-1 §9).

The magnitude of lateral pressure depends primarily on:

- The soil's shear strength (ϕ', c'),
- The degree of wall movement, and
- The **stress history and density** of the retained soil.

13.1.2 States of Lateral Pressure

State	Notation	Wall Movement	Soil Behaviour	Typical Application
At-rest	K_0	None	Elastic, no strain	Basement walls, rigid frames
Active	K_a	Wall moves *away* from soil	Shear failure develops behind wall	Cantilever walls, gravity walls
Passive	K_p	Wall moves *towards* soil	Shear failure develops in front of wall	Resistance to sliding or overturning

(See Fig. 13.1 – Conceptual diagram of K_0, K_a, and K_p conditions.)

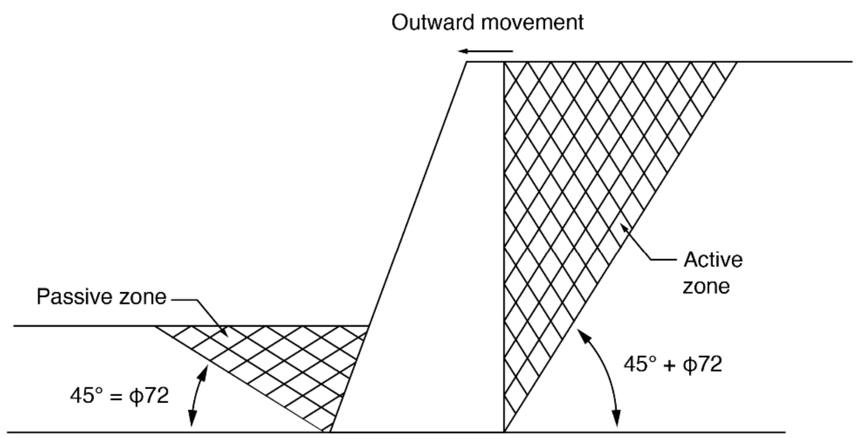


Figure 50: Active and passive wedges of a gravity retaining wall

13.1.3 Relationship Between Vertical and Horizontal Stress

$$\sigma_h = K\sigma_v'$$

where;

- σ_h: horizontal effective stress,
- σ_v': vertical effective stress,
- K: lateral earth pressure coefficient (K_0, K_a, or K_p depending on condition).

13.1.4 Rankine Theory (Simplified, Frictionless Wall)

Assumes a **planar failure surface** and no wall friction ($\delta = 0$).

For a soil with effective friction angle φ':

$$K_a = \tan^2(45° - \frac{\varphi'}{2})$$
$$K_p = \tan^2(45° + \frac{\varphi'}{2})$$

and for at-rest:

$$K_0 = 1 - \sin\ \varphi'$$

ϕ' (°)	K_0	K_a	K_p
25	0.58	0.41	2.46
30	0.50	0.33	3.00
35	0.43	0.27	3.69
40	0.36	0.22	4.60

Rankine theory is appropriate for **vertical walls with horizontal backfill** and is widely used in preliminary design.

13.1.5 Coulomb Theory (Inclined Backfill and Wall Friction)

Coulomb's method extends Rankine's to include wall friction (δ) and backfill slope (β).

$$K_a = \frac{\cos^2(\varphi' - \beta)}{\cos^2\beta\cos(\delta + \beta)} \times \frac{1}{[1 + \sqrt{\frac{\sin(\varphi' + \delta)\sin(\varphi' - \alpha)}{\cos(\delta + \beta)\cos(\alpha - \beta)}}]^2}$$
$$K_p = \frac{\cos^2(\varphi' + \beta)}{\cos^2\beta\cos(\delta - \beta)} \times \frac{1}{[1 - \sqrt{\frac{\sin(\varphi' + \delta)\sin(\varphi' + \alpha)}{\cos(\delta - \beta)\cos(\alpha + \beta)}}]^2}$$

Where;

- α = wall inclination from vertical,
- β = ground surface inclination from horizontal,
- δ = wall–soil friction angle (≈ 0.5–0.67 ϕ' for granular soils).

Coulomb theory predicts lower K_a and higher K_p values compared to Rankine for the same ϕ' when δ > 0.

13.1.6 Typical Values for Design

Soil Type	ϕ' (°)	K_0	K_a	K_p	δ (°)
Soft clay	—	1.0	—	—	—
Firm clay (c' = 10 kPa, ϕ' = 20°)	20	0.66	0.49	2.04	0
Loose sand	30	0.50	0.33	3.00	15
Dense sand	38	0.38	0.24	4.20	20
Gravel	42	0.33	0.21	4.80	25

13.1.7 Cohesive Backfill Adjustment

For purely cohesive soils (ϕ' = 0):

$$\sigma_h = K_a \sigma_v' - 2c'\sqrt{K_a}$$
$$\sigma_h = K_p \sigma_v' + 2c'\sqrt{K_p}$$

⚠️ *Negative pressures at shallow depths are not physically sustained* — tension zones should be neglected in design (EN 1997-1 §9.5.2).

13.1.8 Influence of Wall Movement

Wall Displacement (Δ/H)	Approximate Condition	Pressure Coefficient
0	At-rest	K_0
0.0005–0.001	Active	K_a
0.01–0.02	Passive	K_p

Thus, a lateral movement of only **0.1–0.2 % of wall height** is sufficient to mobilise active conditions in most granular soils.

Stiff or braced systems (e.g., basements, sheet piles with props) generally remain near $\mathbf{K_0}$ conditions; cantilever walls may reach $\mathbf{K_a/K_p}$ states.

13.1.9 Eurocode 7 Context

EN 1997-1 §9.5 requires that:

- **At-rest pressures (K_0)** be used where movement is restrained,
- **Active pressures (K_a)** be used where movement allows soil expansion,
- **Passive pressures (K_p)** may only be relied upon where sufficient movement can occur to mobilise resistance.

Partial factors (per UK NA):

- On actions (γ_F): 1.35 or 1.5 (permanent/variable)
- On soil strength parameters (γ_M): 1.25 (ϕ', c')
- On resistances (γ_R): 1.0 (for equilibrium limit states)

Phicon Note:

• Always determine which pressure state is physically mobilised — do not assume K_a or K_p without considering wall stiffness and movement.
• For braced excavations and basements, adopt **K_0–K_a envelope** from construction sequence analysis.
• For passive resistance checks, **limit δ to ≤ 2/3 ϕ'** and apply reduction for partial mobilisation (often 50 %).
• Include groundwater effects separately (hydrostatic + seepage components).
• Always show in GDR which **earth pressure theory (Rankine/Coulomb)** was used and justify parameter selection (ϕ', δ, β).

13.2 Rankine and Coulomb Methods

13.2.1 Purpose

The Rankine and Coulomb methods are classical approaches to quantify **active and passive earth pressures** on retaining walls.

They differ in:

- The **assumed geometry** of the failure surface,
- Whether **wall friction (δ)** and **wall inclination (α)** are included, and
- Their **applicability** to different field conditions.

Both methods are compatible with Eurocode 7 (EN 1997-1 §9.5), provided parameters (ϕ', c', δ, β) are selected as **characteristic values** and factored appropriately under the relevant design approach.

13.2.2 Rankine's Theory (Simplified Approach)

- Assumptions
- Vertical wall with smooth (frictionless) interface.
- Horizontal backfill with uniform soil properties.
- Failure surface planar and inclined at $45° \pm \varphi'/2$.
- No surcharge, or uniform surcharge only.
- Active and Passive Coefficients

$$K_a = \tan^2\left(45° - \frac{\varphi'}{2}\right), K_p = \tan^2\left(45° + \frac{\varphi'}{2}\right)$$

- Active Earth Pressure at Depth $\boldsymbol{z}$

$$\sigma_{h,a} = K_a(\gamma' z + q)$$

where q= uniform surcharge (kPa).

- Passive Earth Pressure

$$\sigma_{h,p} = K_p \gamma' z$$

- Resultant Force (Triangular Pressure Distribution)

For wall height H:

$$P_a = \frac{1}{2} K_a \gamma' H^2$$
$$P_p = \frac{1}{2} K_p \gamma' H^2$$

The resultant acts at **H/3** above the base.

- Total Pressure with Uniform Surcharge

$$P_{a,total} = \frac{1}{2} K_a \gamma' H^2 + K_a q H$$

(triangular + rectangular components)

13.2.3 Rankine Pressure Diagram

Component	Distribution	Resultant	Lever Arm
Soil weight	Triangular (γ'_z)	$½K_a\gamma'H^2$	H/3
Surcharge	Rectangular (K_{aq})	K_aqH	H/2

(Figure 13.2 – Rankine active pressure diagram)

Simple, conservative, and widely used for **cantilever, gravity, and embedded walls** where δ ≈ 0.

13.2.4 Coulomb's Theory (Generalised Approach)

Coulomb (1776) extended Rankine's theory to include:

- Wall friction (δ),
- Wall inclination (α),
- Sloping backfill (β).

The method assumes **planar failure wedges** satisfying limit equilibrium of forces.

- Active Pressure Coefficient:

$$K_a = \frac{\cos^2(\varphi' - \beta)}{\cos^2\beta \cos(\delta + \beta)} \times \frac{1}{[1 + \sqrt{\frac{\sin(\varphi' + \delta)\sin(\varphi' - \alpha)}{\cos(\delta + \beta)\cos(\alpha - \beta)}}]^2}$$

- Passive Pressure Coefficient:

$$K_p = \frac{\cos^2(\varphi' + \beta)}{\cos^2\beta \cos(\delta - \beta)} \times \frac{1}{[1 - \sqrt{\frac{\sin(\varphi' + \delta)\sin(\varphi' + \alpha)}{\cos(\delta - \beta)\cos(\alpha + \beta)}}]^2}$$

For δ = 0 and β = 0, Coulomb reduces to Rankine.

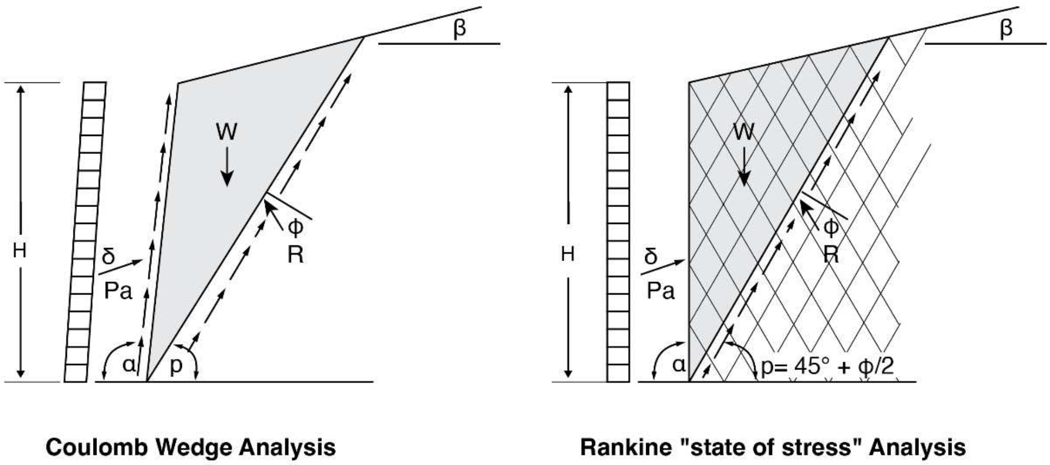


Figure 51: Comparison of methods (reproduced from Keystone Retaining Wall Systems)

13.2.5 Wall Friction (δ) and Backfill Slope (β) Influence

Parameter	Effect on K_a	Effect on K_p	Comment
δ ↑ (more wall friction)	↓	↑	Wall friction relieves active pressure, increases passive resistance
β ↑ (sloping backfill)	↑	↓	Sloping backfill increases active pressure
α ↑ (inclined wall)	↑	Variable	Steeper wall increases pressure

13.2.6 Typical Ranges

ϕ' (°)	δ (°)	β (°)	K_a (Coulomb)	K_a (Rankine)
30	0	0	0.33	0.33
30	15	0	0.27	0.33
30	20	10	0.37	0.33
35	0	0	0.27	0.27
35	15	0	0.22	0.27

⚠️ *Including wall friction (δ)* typically reduces K_a by 10–25 % for granular soils.

Designers often limit δ ≤ 2/3 ϕ' for stability and constructability consistency.

13.2.7 Line of Thrust and Pressure Distribution

- **Active state:** pressure increases linearly with depth (triangular distribution).

- **Passive state:** similar shape but reversed direction.
- For *non-homogeneous soils*, calculate pressure layer-by-layer using depth-dependent γ' and ϕ'.

For reinforced or propped walls, combine $\mathbf{K_0}$ (upper portion) and $\mathbf{K_a/K_p}$ (mobilised zones) to form realistic pressure envelopes.

13.2.8 Surcharge and Additional Loads

For uniform surcharge q:

$$\sigma_h = Kq$$

For strip or line loads (Boussinesq):

$$\sigma_h = K \times \frac{2q}{\pi} \tan^{-1}\left(\frac{H}{x}\right)$$

where x = horizontal offset from load.

In Eurocode verification, surcharges should be included as *unfavourable permanent or variable actions* with partial factors per NA Table A.1.

13.2.9 Cohesion in Active Pressure

For cohesive soils:

$$\sigma_h = K_a(\gamma' z) - 2c'\sqrt{K_a}$$

However, any **tensile zone** (negative σ_h) at the top is neglected — pressure is set to zero at the tension crack depth:

$$z_c = \frac{2c'}{\gamma'\sqrt{K_a}}$$

and effective pressure below this depth is recalculated linearly.

13.2.10 Comparison Summary

Aspect	Rankine	Coulomb
Wall friction (δ)	Ignored	Included
Wall inclination (α)	Vertical only	Any inclination
Backfill slope (β)	Horizontal only	Any slope
Failure surface	Planar	Planar (assumed)
Complexity	Simple, closed-form	More complex

Typical use	Preliminary checks	Detailed wall design
Conservatism	Slightly higher pressures	Slightly lower pressures (δ>0)

13.2.11 Eurocode 7 Context

EN 1997-1 §9.5.2 requires:

- Use of characteristic ϕ', c', and γ' values,
- Verification of both **ULS (EQU, GEO)** and **SLS**,
- Consideration of water pressures and construction sequence,
- Selection of K-values consistent with wall movement conditions.

At **ULS:**

$$\gamma_F = 1.35–1.5, \gamma_M = 1.25$$

At **SLS:**

$$\gamma_F = \gamma_M = 1.0$$

and deflections checked using characteristic parameters.

Phicon Note:

• Use **Rankine** for simple geometries and conceptual estimates; **Coulomb** for inclined or frictional interfaces.
• Always confirm **assumed wall movement** justifies active/passive conditions — many walls remain close to **K_0** during early stages.
• Include **surcharge and groundwater effects explicitly**; water pressures act independently of earth pressure state.
• Present **pressure diagrams with key depths (H/3, H/2)** in the GDR for clarity.
• When in doubt, adopt **conservative K_a (Rankine)** and **reduced K_p (0.5 × Coulomb)** for preliminary design; refine later with FE models.

13.3 Earth Pressure due to Surcharge, Slope, and Water

13.3.1 Purpose

In most practical retaining wall problems, lateral earth pressure is not governed solely by the soil's self-weight.

Additional contributions arise from:

- Surface surcharges (traffic, storage, structures),
- Sloping backfills or irregular ground levels, and
- Water pressures (static, seepage, or hydrostatic head).

Correctly combining these effects is essential for both **ULS (stability)** and **SLS (deflection)** checks under Eurocode 7 (§9.5).

13.3.2 Superposition Principle

Lateral pressures from multiple sources are **additive** if they act simultaneously and independently:

$$\sigma_{h,total} = \sigma_{h,soil} + \sigma_{h,surcharge} + u + \sigma_{h,hydraulic}$$

Where;

- $\sigma_{h,soil}$: from self-weight,
- $\sigma_{h,surcharge}$: from surcharges or loads,
- u: pore water pressure,
- $\sigma_{h,hydraulic}$: from seepage or uplift gradients.

Each component is evaluated using the **same K-value** (K_a, K_0, or K_p) appropriate to the wall's movement condition.

13.3.3 Earth Pressure due to Uniform Surcharge

For a uniform surface surcharge q(kPa):

$$\sigma_{h,surcharge} = Kq$$

This acts as a **rectangular pressure distribution** over the wall height.

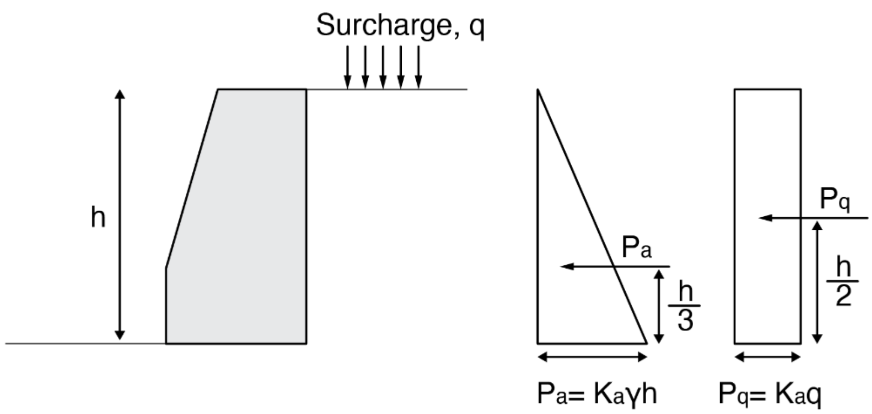


Figure 52: Effect of uniform surcharge on a retaining wall (reproduced from Smiths 2014)

Load Type	Typical q (kPa)	Comment
Light traffic / footpath	10–15	Landscaping or pedestrian areas
Car park / light vehicle	20–25	BS EN 1991-1-1 (Table 6.3)
Heavy vehicle / HGV	40–60	Industrial yards
Storage / stockpile	50–100+	May require 3D modelling
Rail surcharge	20–40	Depending on axle spacing

Apply **γ_F = 1.35–1.5** for unfavourable surcharges under **ULS GEO/EQU** (UK NA to EN 1997-1, Table A.1).

13.3.4 Line or Strip Loads

For a **line load (Q, kN/m)** located at a horizontal distance *x* behind the wall, the horizontal stress increment at depth *z* (Boussinesq solution):

$$\Delta\sigma_h = \frac{2Q}{\pi}\frac{z^2}{(x^2+z^2)^2}$$

For **strip loads** of finite width *B*:

$$\Delta\sigma_h = \frac{q}{\pi}[\tan^{-1}(\frac{x+B}{z}) - \tan^{-1}(\frac{x}{z})]$$

In practice, these are often simplified to an **equivalent uniform surcharge** over the wall height using K-values.

13.3.5 Sloping Backfill Effects

A sloping backfill increases the active pressure due to higher resultant stress acting parallel to the slope.

For backfill inclined at angle β to the horizontal, Rankine gives:

$$K_a = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\varphi'}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\varphi'}}$$

ϕ' (°)	β (°)	K_a (slope)	K_a (horizontal)
30	0	0.33	0.33
30	10	0.37	—
30	20	0.42	—
35	10	0.30	0.27
40	15	0.28	0.22

For steep slopes (>15°), use Coulomb's method to include both β and δ (wall friction).
Check that surcharges applied on sloping backfill are **projected normal to the slope surface**.

13.3.6 Earth Pressure from Surface Loads near Slopes

When retaining structures are located **below embankments or loaded slopes**, vertical stress from the embankment should be combined with **K_a** to derive lateral pressure.

Approximation (after NAVFAC DM 7.2):

$$\sigma_h = K_a(\gamma H + q_{emb}) + \gamma H \sin\beta$$

where q_{emb}= equivalent surcharge from slope self-weight.

For steep slopes, use numerical or limit equilibrium slope–wall interaction analysis rather than direct K-values.

13.3.7 Water Pressure and Hydrostatic Effects

Groundwater exerts **isotropic hydrostatic pressure**, independent of soil stress:

$$u = \gamma_w h$$

where $\gamma_w = 9.81$ kN/m^{3}and *h* = head of water above the point of interest.

- The pressure distribution is **linear with depth**.
- The resultant hydrostatic force = ½$\gamma_w h^2$.
- Water pressure **adds directly** to soil pressure at the same depth.

Where drainage is provided, design for both **short-term (undrained)** and **long-term (drained)** cases.

13.3.8 Seepage and Hydraulic Gradients

If seepage occurs through the backfill, horizontal hydraulic forces act in addition to hydrostatic pressure:

$$\Delta u = i\gamma_w z$$

where *i* = hydraulic gradient.

For upward flow (e.g., behind cut-off walls or basements), **effective stress decreases**, increasing risk of instability or uplift.

Designers should:

- Check hydraulic gradient ≤ critical ($i_{(crit)} \approx 1.0$), and
- Include **hydraulic uplift (UPL)** and **piping (HYD)** verifications per EN 1997-1 §10.4.

13.3.9 Combined Pressure Envelope

The total lateral pressure profile is obtained by **superimposing**:

1. Soil pressure ($K_a \gamma' z$),
2. Surcharge pressure ($K_a q$),
3. Water pressure ($u = \gamma_w z$), and
4. Any seepage or construction load effects.

Example for drained granular backfill (H = 5 m, ϕ' = 30°, q = 20 kPa, water table at base):

Component	Expression	Value at Base (kPa)
Soil	$K_a\gamma' H = 0.33 \times 10 \times 5 =$	16.5
Surcharge	$K_a q = 0.33 \times 20 =$	6.6
Water	$\gamma_w H = 9.81 \times 5 =$	49.0
Total	sum	≈ 72 kPa

13.3.10 Eurocode 7 Verification Context

Limit State	Checks Required	Typical Partial Factors
ULS (GEO, EQU)	Wall sliding, overturning, bearing	γ_F = 1.35 (perm.), 1.5 (var.); γ_M = 1.25
SLS	Wall deflection, joint opening, cracking	$\gamma_F = \gamma_M$ = 1.0
UPL / HYD	Base uplift, heave, piping	γ_F = 1.35; γ_M = 1.25; γ_R = 1.0

All relevant combinations must consider **worst-case drainage condition** (drained, undrained, rapid drawdown).

13.3.11 Design Notes for Practice

- Always include surcharge loads (vehicular, stockpile, construction) explicitly in calculations.
- For **temporary conditions**, assume no effective drainage — include full hydrostatic head.
- For **permanent works**, specify drainage layer or weepholes and justify any reduction in pore pressure.
- Check **earth pressure envelopes** graphically — EC7 reviewers expect pressure diagrams showing soil, surcharge, and water components.
- For **prop or anchor design**, use total pressure including water head below groundwater.

Phicon Note:

• Treat surcharge, slope, and water pressures as distinct but additive — document each source separately in design sheets.
• Use **K_a (drained)** for long-term serviceability and **K_0 (undrained)** for short-term staged excavation checks.
• For partial drainage conditions, interpolate between K_0 and K_a based on measured wall movement.
• Always superimpose **hydrostatic pressure** independently of soil stress — they are not mutually exclusive.
• Include clear **pressure diagrams** (soil, surcharge, water) in GDRs — these are the primary basis for CAT II/III verification.

13.4 Wall Movement and Mobilisation

This section builds on the preceding theory, explaining how **actual wall displacement** governs the degree of mobilisation of active and passive earth pressures in practice, particularly relevant to **retaining walls, basements, sheet piles, and temporary works**.

13.4.1 Purpose

Earth pressures derived from Rankine or Coulomb theory assume the wall has moved sufficiently to fully mobilise the **active (K_a)** or **passive (K_p)** state.

In reality, most retaining structures deform less than required, resulting in intermediate conditions between **at-rest (K_0)** and **active/passive limits**.

Understanding and quantifying this mobilisation is essential to:

- Select realistic design pressures,

- Avoid over-conservatism in stiff systems, and
- Ensure serviceability and structural safety under Eurocode 7 verification.

13.4.2 Movement Required to Mobilise Earth Pressures

Experimental and field data show that only **small wall displacements** (as a fraction of wall height H) are required to mobilise lateral states.

Relative Wall Movement (Δ/H)	Mobilised Condition	Typical Pressure Coefficient
0	At-rest	K_0
0.0005–0.001 (0.05–0.1 % H)	Fully active	K_a
0.005–0.02 (0.5–2 % H)	Fully passive	K_p

Thus, for a 6 m wall:

- Active condition mobilised at ~3–6 mm movement,
- Passive condition at ~30–120 mm movement.

Flexible walls (sheet piles) typically achieve K_a/K_p; stiff or restrained walls (basements) remain closer to K_0.

13.4.3 Pressure–Displacement Relationship

The **mobilised earth pressure coefficient** (K_m) varies non-linearly between K_a and K_0:

$$K_m = K_a + (K_0 - K_a)e^{-\alpha(\Delta/H)}$$

where $\alpha \approx 500$–1000 (empirical constant).

A similar exponential relationship applies between K_0 and K_p for compression (wall pushed into soil).

This relationship allows intermediate K-values to be estimated based on expected deflection, especially useful for serviceability design of stiff walls.

13.4.4 Typical Pressure Envelopes

(a) Rigid Cantilever Wall (free to yield):

- Full active (K_a) on retained side.
- Passive (K_p) at toe.
- Triangular pressure distribution.

(b) Braced or Propped Wall (restrained):

- Reduced movement.
- Pressure distribution approximately trapezoidal or parabolic (K_m between K_0 and K_a).

(c) Fully Fixed Basement Wall:

- Essentially no rotation or translation.
- Near K_0 pressure distribution; peak often occurs mid-height.

13.4.5 Mobilisation in Granular vs. Cohesive Soils

Soil Type	Mobilisation Behaviour	Comments
Granular (sands, gravels)	Rapid mobilisation of shear strength; pressure–movement relationship steep	Small movement sufficient to reach K_a/K_p
Cohesive (clays)	Strain-dependent strength; time lag due to pore pressure dissipation	Larger movement and time required for full mobilisation
Overconsolidated clay	Exhibits high K_0; may dilate, causing slight negative pore pressures	K_m may exceed K_0 temporarily

⚠️ Always check drainage condition: *undrained* = short-term (K_0), *drained* = long-term (K_a/K_p).

13.4.6 Practical Mobilisation Levels by Structure Type

Wall Type / Condition	Movement Characteristic	Likely Pressure State
Gravity or mass concrete wall	Moderate rotation at base	K_a (active)
Cantilever RC wall	Rotates about base	K_a (active)
Flexible sheet pile (anchored)	Bending + toe rotation	Mixed K_a–K_p distribution
Propped / braced excavation	Minimal lateral deflection	Between K_0 and K_a
Basement wall cast against slab	Fully restrained	K_0
Embedded diaphragm wall	Depends on excavation sequence	K_a near top, K_p near base

13.4.7 Effect on Design Forces

Partial mobilisation changes both **magnitude** and **distribution** of bending moments and shear in the wall.

- Assuming $\mathbf{K_0}$ for flexible walls → overly conservative bending moments.
- Assuming $\mathbf{K_a/K_p}$ for stiff walls → unconservative displacements.
- EC7 therefore encourages using **observational methods** or **staged analysis** to update design pressures as construction progresses.

13.4.8 Eurocode 7 Context

EN 1997-1 §9.5.1–9.5.2 requires:

- Wall design to account for the deformations necessary to mobilise soil resistance,
- Verification at both **ULS (GEO/STR)** and **SLS**,
- Use of characteristic parameters with appropriate partial factors.

At ULS:

$$\gamma_F = 1.35–1.5, \gamma_M = 1.25$$

At SLS:

$$\gamma_F = \gamma_M = 1.0$$

Designers should demonstrate that assumed earth pressures correspond to *achievable* wall movements (e.g., ≤ H/500 at SLS for serviceability).

13.4.9 Observational and Finite Element Methods

Modern designs increasingly use **numerical analysis** (e.g. PLAXIS 2D/3D, Wallap, Oasys Pile) to model mobilisation directly.

These tools:

- Compute stress redistribution as wall deflects,
- Allow combination of staged excavation, groundwater, and soil non-linearity,
- Produce pressure–deflection profiles consistent with EC7 performance-based design.

Where FE analysis is not practical:

- Use **simplified pressure envelopes** (e.g., CIRIA C580 design charts),
- Calibrate stiffness (E_{50}, $G_{0\cdot7}$) from field or lab data.

13.4.10 Field Observations and Monitoring

- Lateral wall deflections in stiff RC basement walls typically range **1–5 mm**.
- Cantilever sheet piles: **H/500–H/250** lateral deflection (10–25 mm for 5 m wall).
- Ground movements should be verified by **inclinometer monitoring** where adjacent structures or utilities are present.
- Compare measured and predicted deflections; adjust K-values in back-analysis for future designs.

Phicon Note:

• Always link assumed K-values to expected wall movement — this is the single most common inconsistency in design audits.
• For **temporary works**, it is safer to assume **partial mobilisation** ($K_m \approx 0.7K_0–K_a$).
• For **permanent basements**, adopt **K_0 or K_m = 0.9K_0** unless flexibility is demonstrated.
• Include a table in GDRs summarising: wall type, movement range (Δ/H), and corresponding K-value used in design.
• For flexible walls, cross-check deflections using both **simplified (C580)** and **numerical** methods — reviewers expect corroboration.

14 Retaining Walls: Types and Design

14.1 Common Wall Types and Construction Methods

14.1.1 Purpose

Retaining walls support soil or backfill at a higher elevation and must resist **lateral earth pressures**, **surcharge loads**, and **water pressures** while maintaining **global and structural stability**.

The design selection depends on:

- Required retained height and loading conditions,
- Ground type (granular, cohesive, or rock),
- Construction constraints (top-down vs. bottom-up), and
- Durability, service life, and reuse potential.

Eurocode 7 requires verification of both **geotechnical (GEO)** and **structural (STR)** limit states for all wall types, supported by appropriate construction control and documentation in the Geotechnical Design Report (GDR).

14.1.2 Retaining Wall Classification

Category	Primary Resistance Mechanism	Typical Height Range (m)	Common Applications
Gravity walls	Self-weight	1–5	Landscaping, low retaining walls
Cantilever (reinforced concrete)	Flexural resistance	2–7	Basements, roads, bridge abutments
Counterfort / buttressed	Flexural + shear truss	5–12	Tanks, high retaining structures
Embedded (sheet / secant / diaphragm)	Passive earth pressure	4–20+	Deep excavations, quay walls
Mechanically Stabilised Earth (MSE)	Reinforced backfill	2–25	Highway embankments, bridge abutments
King pile / Berliner / soldier pile walls	Embedded soldier piles + lagging	3–8	Temporary or semi-permanent works

Where groundwater is present, embedded or drained systems are generally required.

14.1.3 Gravity and Mass Concrete Walls

14.1.3.1 Description

Massive walls that rely entirely on **self-weight** to resist overturning and sliding. Constructed from plain concrete, masonry, or modular blocks, often with weep holes for drainage.

14.1.3.2 Typical Characteristics

- Simple design (no reinforcement).
- Economical up to 3–4 m retained height.
- Require firm foundation bearing (q_a > 150 kPa).
- Often stepped or battered at the rear face.

- Verification (EC7 Framework)

1. **ULS (EQU):** Overturning stability:

$$\frac{M_{resist}}{M_{driving}} \geq 1.5$$

2. **ULS (GEO):** Sliding and bearing capacity checks.
3. **SLS:** Settlement and rotation within acceptable limits.

14.1.4 Reinforced Concrete Cantilever Walls

14.1.4.1 Description

A reinforced concrete stem and base slab acting as a cantilever; resistance derived from **weight of backfill above the heel** and the **wall's self-weight**.

14.1.4.2 Construction Notes

- Cast in situ or precast.
- Usually constructed on compacted granular base.
- Drainage layer or weepholes essential to relieve pore pressure.

Design Aspects

- Stem acts as vertical cantilever in bending.
- Base resists overturning and sliding through friction and self-weight.
- Structural design per EN 1992-1-1 (ULS bending, shear, service crack width).
- Practical Height Range: up to 6–7 m.

14.1.5 Counterfort and Buttressed Walls

14.1.5.1 Description

For higher walls (>6 m), **counterforts** or **buttresses** reduce bending moments by tying the stem to the base.

- *Counterfort wall:* Triangular web on backfill side (tension in web).
- *Buttressed wall:* Web on exposed face (compression in web).

Advantages

- Economical for tall walls (5–12 m).
- Reduced reinforcement demand in stem and base.

Design Considerations

- Complex formwork; suited to permanent works only.
- Requires granular, well-drained backfill.
- Verify shear, flexure, and crack control in webs per EN 1992-1-1.

14.1.6 King Pile / Berliner (Soldier Pile) Walls

14.1.6.1 Description

H-section steel piles (king posts) spaced typically 1.5–3.0 m apart, infilled with:

- Timber planks (temporary works),

- Precast concrete panels (semi-permanent), or
- Shotcrete (urban excavations).

Applications

- Temporary retaining walls, shafts, basements, underpasses.
- Re-usable steel sections make it economical for repeat works.

Construction Sequence

1. Install H-piles (driven or bored).
2. Excavate in lifts, installing infill panels progressively.
3. Provide temporary props or anchors if required.

Design Notes

- Design as **cantilever or propped beam** (BS EN 1993-5).
- Earth pressures typically between K_a and K_0 depending on stiffness.
- Check connection capacity between pile and lagging.
- Drainage essential to prevent build-up of water pressure.

14.1.7 Mechanically Stabilised Earth (MSE) Walls

14.1.7.1 Description

Segmental facing (concrete panels, blocks, or gabions) connected to horizontal reinforcement layers (geogrids or steel strips) within compacted backfill.

14.1.7.2 Mechanism

Internal stability achieved by tensile reinforcement; external stability by weight of the reinforced mass.

Advantages

- Rapid construction and flexible system.
- Performs well under differential settlement.
- Aesthetically adaptable.
- Design Standards

BS EN 14475 (Reinforced fill structures) and HA 68/94 for UK practice.

14.1.8 Embedded Wall Systems

14.1.8.1 Types

- **Sheet pile walls:** Steel interlocking piles driven to form watertight wall.
- **Secant pile walls:** Overlapping bored piles (reinforced / unreinforced).
- **Diaphragm walls:** Reinforced concrete panels constructed in slurry trench.

Applications

- Basements, deep excavations, quay walls, cut-and-cover tunnels.
- Construction Modes

System	Support	Construction Mode	Typical Depth (m)
Cantilever sheet pile	Self-supporting	Bottom-up	≤ 6
Propped / anchored wall	Struts, ground anchors	Top-down / bottom-up	6–20
Diaphragm wall	Stiff concrete panel	Top-down	10–40+

Design Aspects

- Governed by both **GEO** (soil strength, passive resistance) and **STR** (bending, shear).
- Verified by limit equilibrium or numerical analysis (e.g., PLAXIS, WALLAP).
- Structural design per EN 1993-5 or EN 1992-1-1 as applicable.

14.1.9 Drainage and Hydrostatic Relief

Regardless of wall type:

- **Drainage layer or weepholes** must prevent pore pressure build-up.
- Include **filter media** to avoid migration of fines.
- Where drainage cannot be provided, full hydrostatic head must be included in design (γ_w h).

Phicon Note:

• Wall type selection must consider constructability, ground conditions, and adjacent assets before analytical design begins.
• Always confirm **movement tolerance** — flexible (sheet pile) vs. rigid (RC) walls behave differently under identical loads.
• Include **construction method statement and sequence** within the GDR — EC7 expects this linkage for wall stability verification.
• For mixed soil profiles, prefer **embedded solutions** or **MSE systems** with reinforced zones extending below weak layers.
• Annotate **wall cross-sections** in design sheets showing toe depth, drainage details, and reinforcement zones — a standard CAT II/III review requirement.

14.2 Stability Checks (Sliding, Overturning, Bearing)

14.2.1 Purpose

Stability checks ensure that a retaining wall and its foundation can safely resist the **resultant earth pressures, surcharges, and water loads** without global or local failure.

Per EN 1997-1 (§9.2.1, §9.5.1), verification is required for:

Limit State	Objective	Typical Failure Mode
EQU (Equilibrium)	Prevent gross instability or overturning	Rotation about toe
GEO (Geotechnical)	Ensure soil bearing and sliding resistance	Sliding, bearing failure
STR (Structural)	Verify internal strength of wall	Bending, shear, tension

These checks are normally carried out using **Design Approach 1 (DA1)** — combinations 1 and 2 — in UK practice.

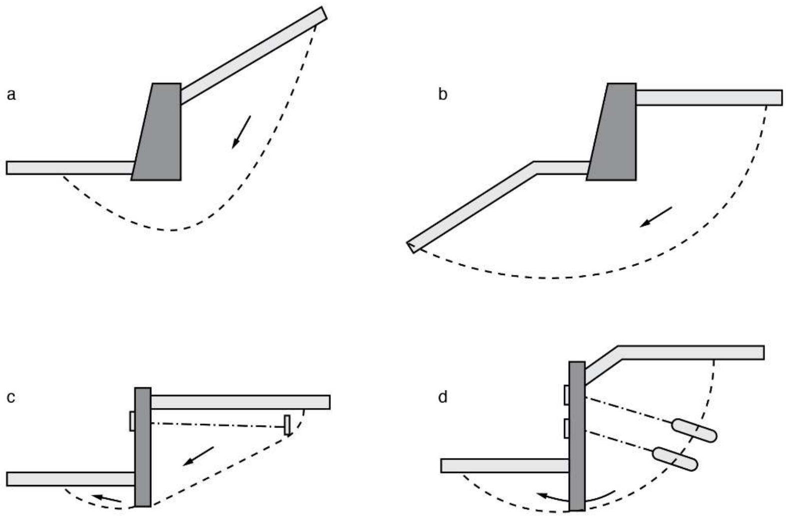


Figure 53: GEO Overall stability of retaining structure (based on Fig 9.1, EN 1997-1:2004)

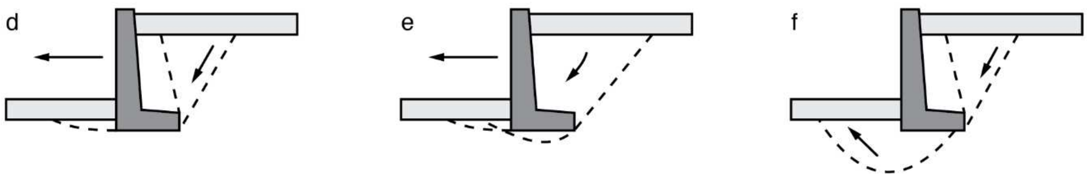


Figure 54: GEO Foundation failures of gravity walls (based on Fig 9.2, EN 1997-1:2004)

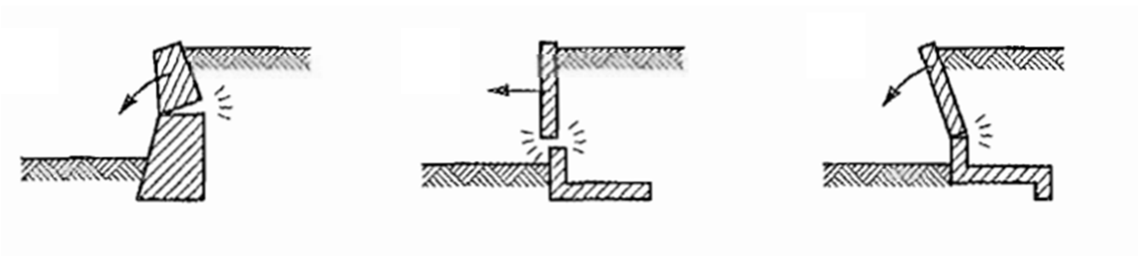


Figure 55: STR Structural failure of retaining structures (based on Fig 9.5, EN 1997-1:2004)

14.2.2 Verification Overview

Check Type	Primary Resistance	Typical Mode	Eurocode Clause
Overturning	Wall self-weight + soil reaction	Rotation about toe	EN 1997-1 §9.2.1(1)
Sliding	Base friction + passive resistance	Translational movement	EN 1997-1 §9.2.1(1)
Bearing	Soil bearing strength under resultant	Bearing or toe failure	EN 1997-1 §6.5.2
Global Stability	Overall slope failure	Combined wall & soil slip	EN 1997-1 §11.5

14.2.3 Design Approach (DA1) Application

Combination 1 (DA1/C1):

- Partial factors on **actions** only.

$$\gamma_F = 1.35 \text{ (perm), } 1.5 \text{ (var), } \gamma_M = 1.0$$

Combination 2 (DA1/C2):

- Partial factors on soil properties/resistances.

$$\gamma_F = 1.0, \gamma_M = 1.25 \text{ (ϕ', c', etc.), } \gamma_R = 1.0$$

Both combinations are checked; the more adverse governs.

14.2.3.1 A. Overturning Stability

Concept

The wall must resist overturning moments caused by lateral earth and water pressures about the toe.

$$M_{resist,d} \geq M_{driving,d}$$

or equivalently

$$\frac{M_{resist,d}}{M_{driving,d}} \geq 1.0$$

At working (service) conditions, a **minimum factor of safety of 1.5** is customary.

Governing Moments

Moment Source	Action	Lever Arm (about toe)
Earth pressure (P_a)	Driving	H/3
Surcharge (qK_aH)	Driving	H/2
Water pressure (u)	Driving	H/3
Wall self-weight	Resisting	From centroid of section
Soil above heel	Resisting	From centroid of heel load

The **resultant line of action** should fall within the **middle third** of the base width to avoid tension at the heel.

Verification Example (Simplified)

For wall height H = 5 m, backfill γ' = 18 kN/m^3, ϕ' = 30° (K_a = 0.33):

$$P_a = 0.5 \times 0.33 \times 18 \times 5^2 = 74.3 \text{ kN/m}$$

Acting at H/3 = 1.67 m.
Assuming self-weight (W) = 200 kN/m at 1.0 m from toe,

$$M_{driving} = 74.3 \times 1.67 = 124.1$$

$$M_{resist} = 200 \times 1.0 = 200$$
$$\text{FOS}_{OT} = \frac{M_{resist}}{M_{driving}} = 1.61 \geq 1.5$$

✓ Satisfactory.

14.2.3.2 B. Sliding Stability

Concept

Sliding occurs when the horizontal forces exceed the base friction and passive resistance at the toe.

$$\Sigma H_d \leq R_{d,friction} + R_{d,passive}$$

Resisting Forces:

$$R_{friction} = (W + Q_v)\tan\ \delta_b$$
$$R_{passive} = 0.5 K_p \gamma' H^2$$

Where;

- δ_b= interface friction angle at base (≈ 2/3 ϕ'),
- Q_v= vertical component of surcharge or structural load.
- Design Check **(EQU/GEO)**:

$$\frac{R_{d,friction} + R_{d,passive}}{H_d} \geq 1.0$$

In serviceability (unfactored), target FOS ≈ 1.5–2.0.

Notes on Passive Resistance

- Passive resistance should be used **cautiously** and only where mobilisation is assured.
- For permanent structures, **reduce K_p by 50 %** unless verified by movement capacity.
- Neglect passive pressure in front of wall if excavation may occur later.

Example Check

Given H = 5 m, W = 200 kN/m, P_a = 74.3 kN/m, ϕ' = 30°:

$$R_{friction} = 200\tan\ (20°) = 73\ \text{kN/m}$$
$$R_{passive} = 0.5(3.0)(18)(5^2) = 675\ \text{kN/m}$$

Even with 50 % reduction → 338 kN/m,

$$\Sigma R = 73 + 338 = 411 > 74.3$$

✓ Sliding stability adequate.

14.2.3.3 C. Bearing Capacity

Concept

Check that resultant vertical stress beneath the base does not exceed the **design bearing resistance** of the foundation soil and remains **within base width (no uplift)**.

$$\sigma_{max,d} \leq q_d$$

and

$$e \leq B/6$$

where e= eccentricity = ($M_{resist,d}$ - $M_{driving,d}$) / V_d).

Bearing Pressure Distribution

For resultant within middle third:

$$\sigma_{max} = \frac{V}{B}(1 + \frac{6e}{B})$$
$$\sigma_{min} = \frac{V}{B}(1 - \frac{6e}{B})$$

If σ_{min} < 0 → tension zone at heel → modify geometry or add shear key.

Design Bearing Resistance (EN 1997-1 §6.5.2)

$$q_d = \frac{1}{\gamma_{Rd}}(c'N_c s_c i_c + q'N_q s_q i_q + 0.5\gamma' B N_\gamma s_\gamma i_\gamma)$$

with partial factors:

$$\gamma_M = 1.25, \gamma_R = 1.0$$

For drained soils, $c' = 0$, ϕ' governs.
For undrained (short-term) check, use $c_u$and corresponding $N_c \approx 6$.

14.2.4 Serviceability Checks

At **SLS** (γ = 1.0):

- Verify no uplift (σ_{min} ≥ 0).
- Check rotation ≤ 1/500–1/300 for wall type.
- Confirm **differential settlements** between wall sections are within tolerance (usually < 25 mm).

14.2.5 Global Stability

Global (overall) stability must also be checked (EN 1997-1 §11.5).
The failure surface may pass:

- Below the wall foundation (deep-seated slip), or
- Through the backfill (compound slip).

Evaluate using limit equilibrium software (e.g. SLOPE/W, LimitState:GEO) or analytical methods (Bishop, Janbu).

Partial factors as per DA1 (C2):

$$\gamma_M = 1.25 \text{ on } \varphi', c'$$

Target **global factor of safety ≥ 1.2–1.4** under characteristic loading.

14.2.6 Summary Table – Stability Verification

Mode	Equation / Criterion	Check	Typical FS / γ	Limit State
Overturning	$M_{resist} \geq M_{drive}$	e ≤ B/6	FS = 1.5	EQU
Sliding	$\Sigma R \geq \Sigma H$	Δ ≤ tolerable	FS = 1.5–2.0	GEO
Bearing	$\sigma_{max} \leq q_d$	No uplift	γ_m = 1.25	GEO
Global stability	Factor ≥ 1.2–1.4	No slip	γ_m = 1.25	GEO
Serviceability	Deflection, rotation	Within limits	γ = 1.0	SLS

Phicon Note:

• Always document *EQU, GEO, and STR* verifications distinctly — each uses different governing equations and partial factors.
• Use **DA1/C1 and DA1/C2 side-by-side**; present the more critical result in the GDR summary table.
• Where groundwater is present, add **hydrostatic head to all horizontal forces** and confirm resultant lies within base.
• For mixed backfill (granular over clay), analyse in **layered sections** using appropriate ϕ', c', and γ' per layer.
• For CAT II/III designs, reviewers expect a clear **moment and force summary table** and **base pressure diagram** attached to each calculation sheet.

14.3 Drainage and Hydrostatic Relief

14.3.1 Purpose

Groundwater and surface infiltration behind retaining structures can increase lateral pressure by up to **100% or more** if not relieved.

Drainage provisions are therefore an **integral part of wall design**, reducing hydrostatic head and improving long-term performance and durability.

EC7 requires that the **design pore pressure distribution** is justified, either by assuming full hydrostatic head (conservative) or by demonstrating an effective drainage system.

14.3.2 Water Pressure Mechanism

Water pressure acts **independently** of soil stress:

$$u = \gamma_w h$$

Where;

- u= pore water pressure (kPa),

- γ_w= unit weight of water (9.81 kN/m^3),
- h= vertical head of water above point considered.

At-rest or active soil pressures are **added to** this hydrostatic component, not substituted.

Designers should explicitly superimpose water pressure in the lateral load diagram.

14.3.3 Drainage Objectives

1. **Prevent hydrostatic head accumulation** behind the wall.
2. **Reduce seepage forces** that can cause base heave or piping.
3. **Maintain stability of backfill** (avoid saturation and softening).
4. **Limit serviceability effects** (cracking, leakage, corrosion).
5. **Ensure maintainability** for long-term operation.

14.3.4 Types of Drainage Systems

Drainage Component	Function	Typical Application
Weep holes	Direct relief of hydrostatic pressure through wall	Gravity & cantilever walls
Filter layer	Prevent migration of fines into drainage medium	Behind masonry/RC walls
Perforated collector drain	Collects and discharges seepage to outlet	Along wall base (heel or toe)
Geocomposite drainage sheet	Thin alternative to granular drainage	Basements, diaphragm walls
Relief wells / subdrains	Relieve uplift pressures below slabs	Basements and rafts
Surface interceptor drains	Prevents surface runoff entering backfill	Top of retained ground

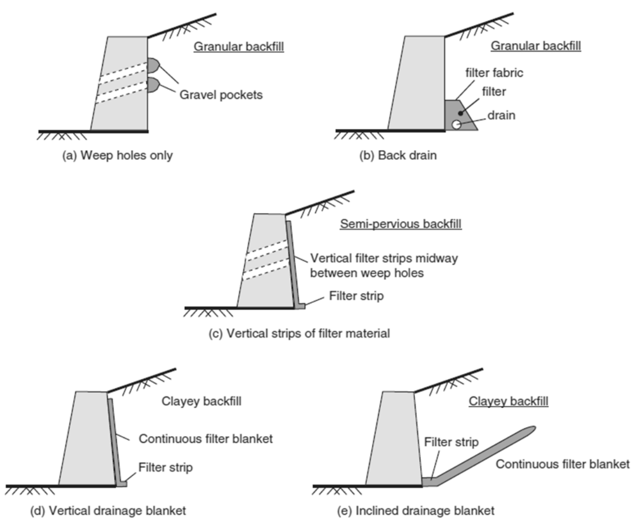


Figure 56: Common drainage systems for retaining walls (reproduced from Smiths 2014)

14.3.5 Granular Drainage Layer

A **vertical drainage zone** behind the wall typically:

- **Thickness:** 300–500 mm (minimum 150 mm for small walls),
- **Material:** Clean, free-draining gravel (10–40 mm),
- **Filter fabric:** Geotextile separator between soil and drainage medium.
- Filter Criteria (Terzaghi):

To prevent migration of fines:

$$\frac{D_{15,filter}}{D_{85,soil}} \leq 4$$

and to ensure permeability:

$$\frac{D_{15,filter}}{D_{15,soil}} \geq 4$$

Use locally available graded stone or recycled material compliant with **BS EN 13242** or SHW Series 500.

14.3.6 Weep Holes

14.3.6.1 Function

Provide direct drainage through the wall to relieve water head behind the stem.

14.3.6.2 Typical Specification

- Diameter: 75–100 mm PVC or HDPE pipe.
- Spacing: 1.0–1.5 m horizontally and vertically staggered.
- Gradient: minimum 1V:20H outwards.
- Protected by gravel pocket or geotextile to prevent clogging.

In marine or contaminated conditions, use **non-corroding materials** (HDPE, GRP).
Always provide **visible outlets** with inspection access.

14.3.7 Collector Drains and Discharge

Perforated collector drains (100–150 mm Ø) are laid at the **toe or heel** of the wall:

- Surrounded by clean 10–20 mm gravel (150 mm minimum envelope).
- Laid to **fall of 1–2%** towards outlet.
- Connected to surface or subsoil drainage system.

For highway and basement walls, design discharge per **BS EN 752** (minimum flow capacity ≈ 0.5–1.0 L/s·m).

Where gravity discharge is not feasible, use **pumped sump systems** with standby provision.

14.3.8 Geocomposite Drainage Layers

Modern permanent structures (e.g. basements, diaphragm walls) often use **prefabricated geocomposite drainage sheets** instead of granular backfill.

These systems combine:

- **Filter geotextile** + **drainage core** + **backing film**,
  and achieve equivalent flow capacity in a compact thickness (typically 10–20 mm).

Advantages:

- Space-saving and lightweight.
- Rapid installation against formwork or shotcrete.
- Compatible with waterproof membranes.

Comply with **BS EN 13252** (drainage geocomposites).

Verify long-term flow capacity under design load per manufacturer data.

14.3.9 Hydrostatic Pressure Verification

If no drainage is provided, assume **full hydrostatic head**:

$$u_{max} = \gamma_w H$$

Add directly to soil pressure:

$$\sigma_{h,total} = K_a \gamma' z + \gamma_w z$$

Resultant water force:

$$P_w = 0.5 \gamma_w H^2$$

acting at H/3 above base.

Always include in both **ULS (EQU/GEO)** and **SLS** verifications unless fully drained conditions are demonstrably permanent.

14.3.10 Design for Partial Drainage

If partial drainage is provided (e.g. limited permeability or partial clogging risk),
use **reduced head assumption**, e.g.:

$$h_{eff} = \eta H$$

where η= proportion of full head (0.3–0.7 typically).

Document the assumption in the GDR and justify based on permeability and maintenance regime.

14.3.11 Maintenance Considerations

- Provide **accessible outlets** for flushing and inspection.
- Avoid **blind discharges** into fill.
- Include **backflow protection** if connected to stormwater networks.
- Inspect periodically (especially after heavy rainfall or freeze–thaw cycles).
- Record maintenance plan in the **operation and maintenance manual**.

14.3.12 Eurocode 7 Context

EN 1997-1 §9.5.2:

"Where pore water pressures are significant, their variation with time and drainage conditions shall be considered.
The designer shall justify the assumed pore pressure distribution in the design."

Partial factors:

- $\gamma_F = 1.35$ (unfavourable hydrostatic head)
- $\gamma_M = 1.0$ (if derived from measured data)
- $\gamma_M = 1.25$ (if based on assumed parameters)

Phicon Note:

• **Never neglect water pressures** unless full relief is guaranteed and maintainable.
• Always include a **hydrostatic pressure diagram** in design drawings and GDR.
• Use **dual drainage strategy** for permanent works: primary collector + secondary relief layer.
• When using geocomposites, verify long-term creep resistance and flow capacity under soil load (EN ISO 12958).
• Provide explicit **maintenance access** and include this in the design risk register (CDM requirement).
• For temporary works, assume **worst-case undrained condition** unless pumping is monitored and controlled.

14.4 Global Stability and Base Shear

This section explains how the **overall (global) stability** of a retaining wall and its surrounding soil mass is verified under Eurocode 7, including methods for **compound slip analysis** and **base shear checks**. It extends beyond the local wall–foundation interaction to evaluate the **combined wall–soil system** as a whole.

14.4.1 Purpose

Local checks for overturning, sliding, and bearing (Section 14.2) ensure *internal wall stability*.

However, the **global stability** assessment verifies that the *entire soil mass + wall system* remains stable against deep-seated or rotational failure that passes beneath or around the wall base.

This is essential where:

- Retained heights > 4 m,
- Weak or layered foundation soils exist,
- Sloping ground or surcharge is present behind the wall,
- Wall friction or toe resistance may not control failure, or
- Embedded walls extend below soft strata.

EC7 §11.5 requires a *global stability check* whenever the failure surface extends beyond the structural elements directly analysed.

14.4.2 Nature of Global Stability Failures

Typical potential failure mechanisms include:

- **Rotational (circular)** slips passing beneath the wall foundation.
- **Compound slips** intersecting the backfill and foundation soil.
- **Translational** failure along weak layers (often in soft clays).
- **Deep-seated** failure including part of the retained ground and wall base.

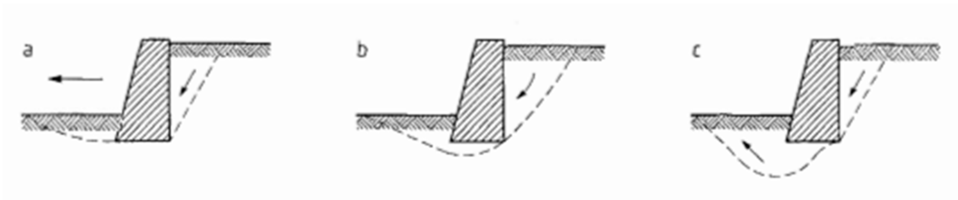


Figure 57: GEO Global failures around gravity walls (based on Fig 9.2, EN 1997-1:2004)

For flexible systems (sheet piles, diaphragm walls), a global check also considers toe embedment and passive resistance mobilisation.

14.4.3 Analytical and Numerical Methods

Method	Description	Typical Use
Limit Equilibrium (LEM)	Classical circular or non-circular slip surface analysis (Bishop, Janbu, Morgenstern–Price).	Standard approach for slopes and retaining structures.
Finite Element (FE)	Stress–strain modelling with shear strength reduction (e.g. PLAXIS).	Complex geometries, soil–structure interaction, staged construction.
Hybrid (LEM + FE)	FE-derived pore pressures / stress fields used in limit equilibrium models.	Deep basements and staged excavations.

Both approaches are acceptable under EC7, provided characteristic parameters are used and partial factors applied to strength (ϕ', c').

14.4.4 Design Verification Format (EN 1997-1 §11.5)

At ultimate limit state (ULS), the requirement is:

$$E_d \leq R_d$$

or equivalently,

$$\text{FOS}_{global} \geq 1.0$$

Under characteristic conditions (unfactored) for *concept checks*, typical minimum global safety factors are:

Design Condition	Recommended FOS (Characteristic)	Partial Factors (ULS)
Temporary works	1.2	γ_M = 1.25 (ϕ', c')
Permanent works	1.3–1.5	γ_M = 1.25
Sensitive structures / weak ground	1.5–1.7	γ_M = 1.25–1.4

FOS values are often reported for information even when partial factors are explicitly applied (EC7 DA1/C2).

14.4.5 Inputs for Global Stability Modelling

- Wall geometry and base width / embedment.
- Backfill and retained soil parameters (γ', ϕ', c').
- Foundation soil layers, pore pressures, and interfaces.
- Surcharges (uniform, strip, or embankment).
- Water table and seepage conditions.
- Drainage provisions and phreatic line location.

Groundwater levels are usually the *most sensitive input*. Always test several scenarios (drained, undrained, rapid drawdown).

14.4.6 Example (Concept Check)

For a 6 m-high cantilever wall on stiff clay (c' = 10 kPa, ϕ' = 20°, γ' = 18 kN/m^3):

- Slip circle analysis (Bishop simplified) yields FOS = 1.48 (characteristic).
- Applying γ_M = 1.25 (DA1/C2):

$$\varphi'_d = \tan^{-1}\left(\frac{\tan \varphi'}{\gamma_M}\right) = 16°$$

Re-analysis gives FOS ≈ 1.18.
✓ Acceptable for permanent condition.

14.4.7 Base Shear Check

While global stability examines deep slip surfaces, **base shear** checks verify that:

- The wall base or embedment can resist the *shear transmitted* from retained soil,
- No local sliding or shear plane forms immediately below the base.

$$V_d \leq \tau_{resist,d} = c'_d A + N'_d \tan \varphi'_d$$

where:

- V_d= design horizontal shear at base,
- N'_d= design vertical effective load,
- A= base area,
- c'_d, φ'_d= design parameters after factoring.

Particularly important for **embedded walls** and **raft-type retaining structures** with low normal stress at the base.

14.4.8 Compound and Multi-Layer Failures

In layered ground, failure surfaces may cross zones of different ϕ', c', and γ'.
In such cases:

- Conduct *multi-layer limit equilibrium analysis*, summing slice forces layer by layer.
- Apply **reduced ϕ', c'** to weaker strata only (γ_M = 1.25).
- Include **interface reduction factors** (R_i ≈ 0.7–0.9) for soil–structure or soil–soil boundaries.

14.4.9 Hydrostatic and Seepage Influence

Rising water levels reduce effective stress and hence stability.
Always include:

- Seepage forces ($u = \gamma_w h$),
- Rapid drawdown condition (worst case),
- Drained vs. undrained analyses as appropriate.

If groundwater cannot be effectively drained, verify:

$$i \le i_{crit} = \frac{\gamma'}{\gamma_w}$$

to prevent piping or hydraulic heave (see Chapter 15B).

14.4.10 Numerical Verification (FE Methods)

Finite element methods (e.g. PLAXIS, MIDAS GTS NX, ZSoil):

- Use **Shear Strength Reduction (SSR)** technique to compute FOS directly,
- Capture wall–soil interaction, stiffness effects, and non-linearity,
- Allow staged excavation and loading sequences (required for deep basements).

Outputs include:

- Displacement fields,
- Plastic strain zones,
- Safety factor vs. shear strength reduction curve.

When using FE methods, report the reduction factor at failure (FOS = 1.0 / reduction factor at collapse).

14.4.11 Reporting and Presentation

A complete stability verification should include:

1. **Input table** — layers, parameters, γ_M factors.
2. **Slip surface diagram** — showing critical and secondary surfaces.
3. **Water table position** — all conditions tested.
4. **FOS summary table** — per case (temporary, permanent, rapid drawdown).
5. **Interpretation note** — governing failure mode and remedial measures.

EC7 reviewers expect clear traceability between characteristic data → design parameters → verification results.

Phicon Note:

• Always include at least one **global stability plot** in GDR submissions for retained heights > 4 m.
• Explicitly check **toe slip** and **deep-seated rotational surfaces** passing below the wall.
• For **embedded walls**, ensure base fixity and passive resistance are included in global analysis.
• Use FE verification (PLAXIS, LimitState:GEO) for *complex geometry, layered ground, or staged excavation*.
• Present **characteristic and design safety factors side by side** — this improves audit clarity.
• When unsure of long-term drainage performance, treat water table as fully raised (worst-case design).

15 Embedded Walls & Support Systems

15.1 Embedded Walls: Sheet, Secant, and Diaphragm Walls

15.1.1 Purpose

Embedded walls retain soil and groundwater by mobilising **passive resistance** below the excavation level.

They are used where conventional gravity or cantilever walls would be uneconomical or infeasible due to:

- Restricted site space or adjacent structures,
- High retained heights (>4 m),
- Deep excavations below groundwater, or
- Temporary support requirements during staged construction.

Eurocode 7 treats embedded walls as **combined soil–structure systems**, requiring verification at both the *geotechnical limit state (GEO)* and *structural limit state (STR)*, including staged construction effects.

15.1.2 Principal Types of Embedded Wall

Type	Material & Construction	Typical Depth (m)	Advantages / Limitations
Steel Sheet Pile Wall	Interlocking rolled steel sections driven or pressed	4–20+	Fast installation; reusable; limited stiffness; vibration-sensitive
Secant Pile Wall	Overlapping bored piles (hard–soft or hard–hard)	8–30	Watertight; good stiffness; slow construction; expensive
Diaphragm Wall	Reinforced concrete panels cast in slurry trench	10–50+	Very stiff; watertight; suitable for deep basements
Contiguous Pile Wall	Spaced bored piles (typically 100 mm gap)	4–15	Economical; not watertight; suitable for temporary works

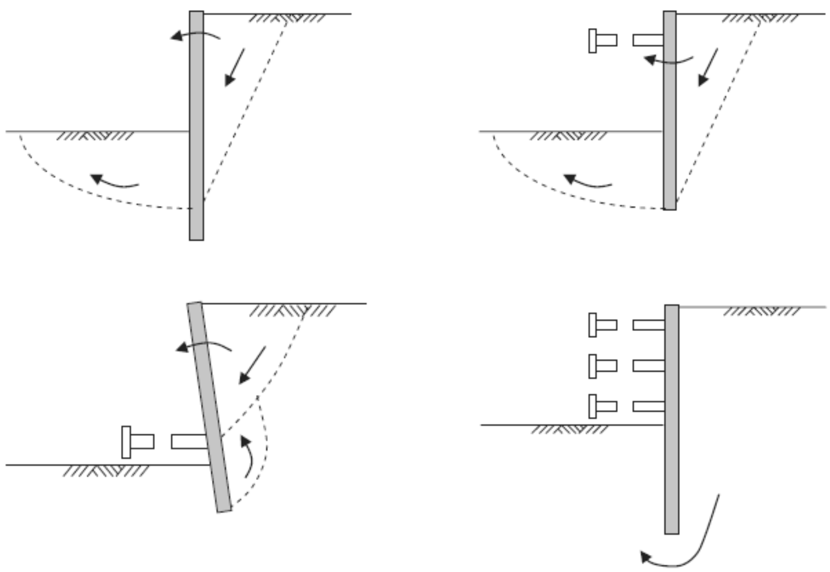


Figure 58: GEO Rotational failures of embedded walls (based on Fig 9.3, EN 1997-1:2004)

15.1.3 Steel Sheet Pile Walls

15.1.3.1 Description

Thin interlocking steel sections forming a continuous wall, commonly used for:

- Basements, quays, cofferdams, and temporary excavations.
  Sections include **U-profiles, Z-profiles,** or **straight web piles** (EN 10248).

Installation Methods

- Vibratory driving (fast, minimal pre-augering).
- Impact driving (dense soils, marine).
- Press-in systems (silent, low-vibration urban work).

Advantages

- Reusable, lightweight, adaptable.
- Rapid installation and extraction.
- Can act **as permanent wall (corrosion** protected).

Limitations

- Noise and vibration sensitive.
- Limited stiffness for deep cuts (>8 m) unless propped/anchored.
- May require toe sealing (grout or plug).

Structural Design

- Verified per EN 1993-5 (Steel Piling Design).
- Check bending, shear, and interlock tension.
- Typical **modulus range:** 400–2500 cm^3/m.

15.1.4 Secant Pile Walls

15.1.4.1 Description

Constructed by **drilling and casting overlapping bored piles**.
Alternate *female (soft)* and *male (hard)* piles interlock to form a watertight wall.

Type	Sequence	Strength
Hard–Soft	Primary piles unreinforced (low strength)	5–15 MPa
Hard–Hard	Both reinforced concrete piles	15–30 MPa

Installation

- Rotary bored with temporary casing or polymer slurry.
- Overlap typically 100–150 mm.
- Reinforcement cage inserted in secondary piles only (for Hard–Soft).

Advantages

- Minimal vibration and noise.

- Can cut through boulders and old foundations.
- High stiffness → low deflections.
- Excellent watertightness (when overlap is continuous).

Limitations

- Slow, expensive.
- Construction tolerances critical (risk of gaps or overcut).
- Requires high accuracy in pile alignment and sequence.

Structural Design

- Reinforced concrete design to **EN 1992-1-1**.
- Earth pressures per **EC7 DA1/C1 & C2**.
- Verify global stability including toe resistance.

15.1.5 Diaphragm Walls

15.1.5.1 Description

Cast-in-situ reinforced concrete panels excavated under **bentonite or polymer slurry** to maintain stability. Commonly used for **deep basements**, **cut-and-cover tunnels**, and **marine quay walls**.

15.1.5.2 Construction Steps

1. Excavation in panels using grab or hydro-mill.
2. Placement of steel cage.
3. Tremie concreting from base upward.
4. Repeat for adjacent panels (with stop-end pipes or joints).

15.1.5.3 Key Parameters

Parameter	Typical Range
Panel width	0.6–1.2 m
Panel length	2.5–6 m
Panel depth	up to 50 m
Concrete strength	35–50 MPa

Advantages

- Very high stiffness → minimal deflection.
- Watertight with proper joint detailing.
- Can act as permanent wall + basement structure.

Limitations

- High cost and slow production rate.
- Requires slurry management and plant access.
- Difficult to verify verticality and joint sealing.

Design Considerations

- Geotechnical verification (embedment depth, toe fixity).
- Structural design as continuous RC wall (EN 1992-1-1).
- Control crack width ≤ 0.2 mm for watertight walls (BS EN 1992-3).

15.1.6 Contiguous Pile Walls

15.1.6.1 Description

Series of bored piles with small gaps (typically 50–150 mm) left between piles.
Not watertight but economical for **temporary or low-risk permanent** works.

Construction

- Continuous auger or rotary bored.
- Reinforced cages in every pile or alternate.
- Gaps may be sprayed with shotcrete for soil retention.

Advantages

- Simple and fast to construct.
- No vibration, low noise.
- Flexible alignment and pile layout.

Limitations

- Not water-retaining.
- Requires facing system for granular soils.
- Limited lateral stiffness compared to secant/diaphragm walls.

15.1.7 Design Parameters and Soil Interaction

The **soil–structure interaction** for embedded walls depends on:

- Wall stiffness (EI),
- Embedment depth (D),
- Retained height (H),
- Support system (props, anchors, berms),
- Soil strength (ϕ', c'),
- Drainage conditions.
- Mobilised Earth Pressures
- Active (K_a) above excavation level.
- Passive (K_p) below excavation level.
- At-rest (K_0) in stiff or braced systems.

Partial mobilisation often occurs in staged construction — use simplified envelopes (e.g. CIRIA C580 charts) or FE analysis.

15.1.8 Embedment Depth Determination

For a free-cantilever wall:

$$P_a H = P_p \frac{(2/3)H^2}{D}$$

Iteratively solve for *D* ensuring adequate moment equilibrium.

For propped or anchored walls:

- Use CIRIA C580 “free” and “fixed” earth pressure envelopes,
- Add embedment ≥ 10–20 % of excavation depth for construction tolerance,
- Verify minimum toe penetration below dredge or excavation line.

15.1.9 Corrosion and Durability

Wall Type	Design Life	Protection Measures
Steel sheet piles	25–100 yrs	Coatings, sacrificial thickness, cathodic protection
RC secant/diaphragm walls	50–100 yrs	Concrete cover per EN 1992-1-1 (≥ 75 mm below ground)
Temporary walls	≤ 2 yrs	Minimal protection; inspect before reuse

15.1.10 Eurocode 7 Design Context

Under **EN 1997-1 §9.5**, embedded walls must be verified for:

1. **EQU (equilibrium)** — overall stability, overturning, sliding.
2. **GEO (geotechnical)** — passive/active resistance, base heave, global stability.
3. **STR (structural)** — bending, shear, crack control (EN 1992 / EN 1993-5).
4. **UPL / HYD** — uplift, piping, seepage (see Chapter 15B).

Use Design Approach 1 (DA1):

- **C1** – apply factors to actions (γ_F = 1.35–1.5).
- **C2** – apply factors to soil parameters (γ_M = 1.25).

Phicon Note:

• Select wall type early based on *depth, ground type, and access*. Re-engineering wall type mid-design often leads to uncoordinated verification gaps.
• Always cross-reference **wall stiffness (EI)** and **support method** — these govern mobilisation and deflection envelopes.
• Minimum embedment depth should include **tolerance + hydraulic safety margin** (≥ 0.5 m below failure surface).
• Specify **corrosion protection**, **joint detailing**, and **construction sequencing** explicitly in the GDR.
• For EC7 verification sheets, present wall geometry, soil parameters, and bending moment envelopes side-by-side for transparency.
• Include a schematic showing **K_a/K_p zones**, **toe depth**, and **support locations** — this is a standard CAT II/III requirement.

15.2 Support Systems (Props, Struts, Anchors, Berms)

This section describes the **types, functions, and design principles** of support systems used to stabilise embedded retaining walls during excavation or in permanent conditions. It covers **props, struts, ground anchors, and berms**, with reference to **Eurocode 7 (EN 1997-1 §9.8)** and **CIRIA C580** for staged excavation design and load sharing.

15.2.1 Purpose

Support systems reduce wall bending and deflection by providing **horizontal restraint** during excavation.

They act as reaction points for the mobilised **active and passive earth pressures**, ensuring stability at both **ULS** and **SLS**.

EC7 requires that embedded wall designs explicitly consider construction sequence and mobilised stiffness of the support system, rather than assuming fully active pressures at all stages.

15.2.2 Typical Support Systems

Support Type	Description	Application Range
Internal struts / props	Steel compression members spanning excavation	Cofferdams, basements, shafts
Ground anchors	Prestressed steel tendons grouted into soil or rock	Deep basements, permanent or temporary walls
Top-down slabs	RC slabs acting as internal props	Permanent basements (top-down method)
Raking struts	Inclined compression members	Temporary shafts, cofferdams
Berms / batters	Temporary retained soil benches	Shallow or staged excavations

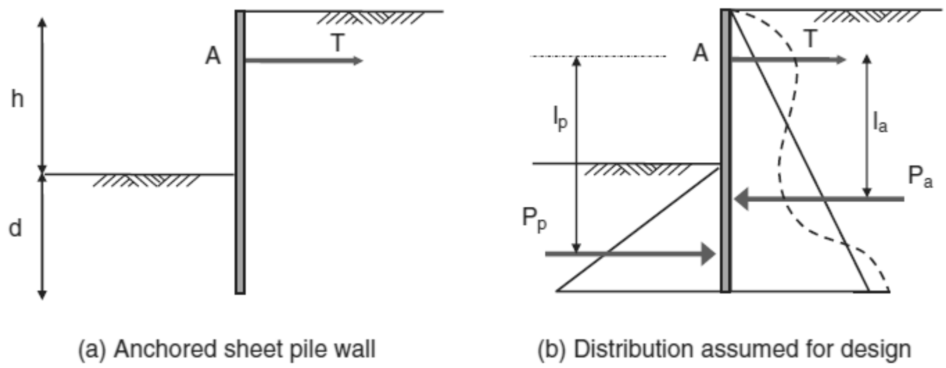


Figure 59: Free earth support method for anchored sheet piled walls (reproduced from Smiths 2014)

15.2.3 Effect of Support on Pressure Distribution

Adding supports modifies the earth pressure diagram significantly:

- **Cantilever wall:** triangular active pressure ($K_a\gamma'z$).
- **Single-propped wall:** triangular with reversal below prop (zero net moment at prop).
- **Multi-propped wall:** near-uniform pressure distribution between supports.

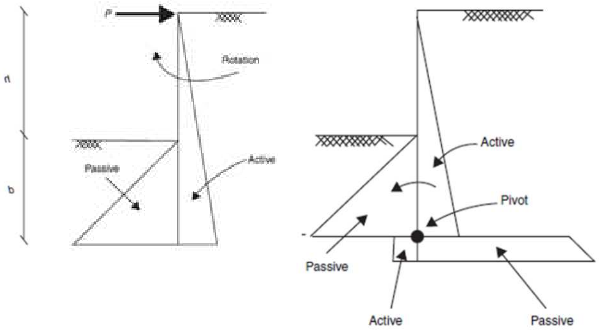


Figure 60: Fixed earth and free earth embedded retaining wall pressure diagrams

Support position strongly affects wall bending moment envelope — prop near the top reduces bending but increases prop load.

15.2.4 Design Workflow for Supported Excavations

1. **Define geometry and support levels** (excavation depth, wall embedment, prop/anchor elevations).
2. **Estimate K-values** (K_a, K_0, K_p) consistent with expected wall movement.
3. **Draw soil pressure envelope** (CIRIA C580 charts or Rankine/Coulomb theory).
4. **Compute prop/anchor loads** by area–moment or stiffness analysis.
5. Determine wall bending moments and shears.
6. **Check embedment depth** for sufficient passive resistance and toe fixity.
7. Verify equilibrium (**ULS GEO/EQU**) and service deflection (SLS).

For staged excavations, recalculate at each stage using updated soil conditions and support activation sequence.

15.2.5 Internal Props and Struts

15.2.5.1 Function

Provide temporary or permanent **horizontal restraint** by spanning across excavation between opposing walls or frames.

15.2.5.2 Design Principles

- Compression member design per **EN 1993-1-1** (buckling, axial load, slenderness).
- Connections designed for eccentricity and pin rotation.
- Allow for **preload (5–10% of design load)** to limit wall movement.
- Include allowance for **prop shortening** under load ($\Delta = N L / EA$).
- Typical Strut Layouts

Span (m)	Member Type	Section Example
< 6	Circular hollow section (CHS)	273×10
6–12	H-section	UC 254×254×73
12–20	Box / built-up strut	Fabricated
> 20	Prestressed truss or cable system	Custom design

Temporary props are usually designed for **3–4% lateral deflection limit** and safety factor ≥ 1.5 on axial capacity.

15.2.6 Raking Struts

Inclined props bearing at the base or mid-level of the excavation.
Advantages:

- Avoid internal obstruction to excavation.
- Provide combined vertical and horizontal restraint.
  Limitations:
- Uneven load distribution, eccentricity, and foundation reaction at toe.

Verify **buckling about both axes**, ensure adequate **bearing pads** at base contact.

15.2.7 Ground Anchors

15.2.7.1 Description

Tension elements transferring load from the wall to stable ground beyond the active wedge.

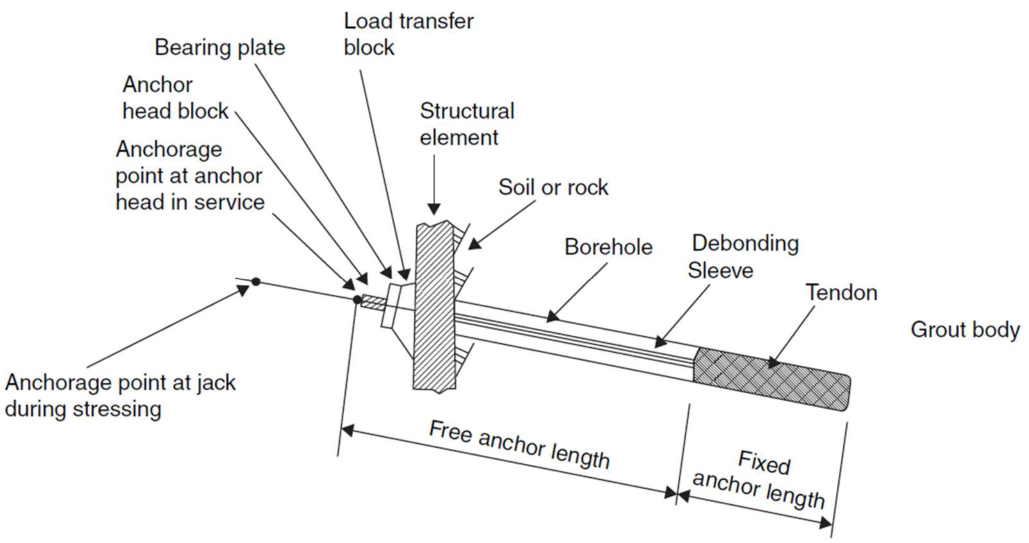


Figure 61: Typical ground anchor detail (reproduced from CIRIA C580)

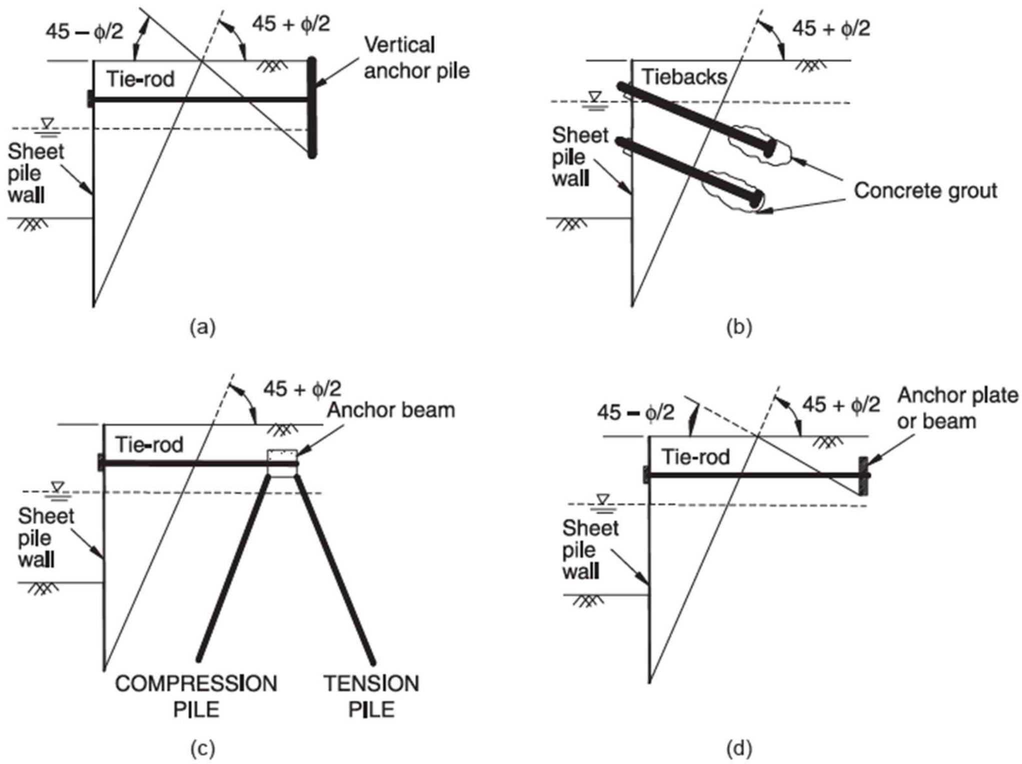


Figure 62: Different types of anchors: (a) vertical anchor pile, (b) tieback, (c) anchor beam, and (d) anchor plate. (reproduced from Das 2011)

Comprise:

- **Free length:** unbonded steel tendon under tension.
- **Fixed length:** grouted bond zone.
- **Head assembly:** anchor plate and stressing jack connection.

Design Considerations

- **Bond length:** 3–6 m in granular soil, 2–4 m in stiff clay.
- **Angle:** 10–25° below horizontal.
- **Spacing:** 1.2–2.5 m (horizontal and vertical).
- **Load transfer:** via friction and adhesion in fixed zone.
- Design Load

$$T_d = T_k \times \gamma_F$$

where $\gamma_F = 1.35$ (ULS), T_k= characteristic tendon load.

Apply **reduction factors** for time-dependent losses:

- Anchorage slip (ΔL ≈ 1–3 mm).
- Relaxation (2–5%).
- Creep in grout (1–3%).

Permanent anchors require **redundant corrosion protection** (EN 1537) — e.g. double sheathing, epoxy coating, sealed trumpet head.

15.2.8 Anchor Testing

Before acceptance:

- **Proof Test:** up to 1.25 × design load.
- **Performance Test:** staged loading to 1.5 × design load.
- **Creep Acceptance:** < 2 mm over 10 min hold period.

Record test results in QA schedule and include calibration certificates for jacks and gauges.

15.2.9 Top-Down Construction Supports

For **top-down basement construction**, the permanent RC slabs act as props during excavation:

- Slab cast on ground, then excavation proceeds beneath.
- Each slab supports the wall before next excavation stage.

Advantages:

- Early superstructure start.
- No need for temporary props.

Disadvantages:

- Complex sequencing, limited inspection access.

EC7 and EN 1992-1-1 require slabs to be checked for both *construction load case* and *final structural case*.

15.2.10 Berms and Temporary Benches

Where soil strength permits, **berms** or **staged batters** can provide temporary stability.

- Common in stiff clays and granular soils.
- Bench width 0.5–1.5 m typical; height 1.5–2.0 m.
- Reduces wall height and earth pressure incrementally.

⚠️ Always verify interim stability of each stage using short-term undrained parameters.

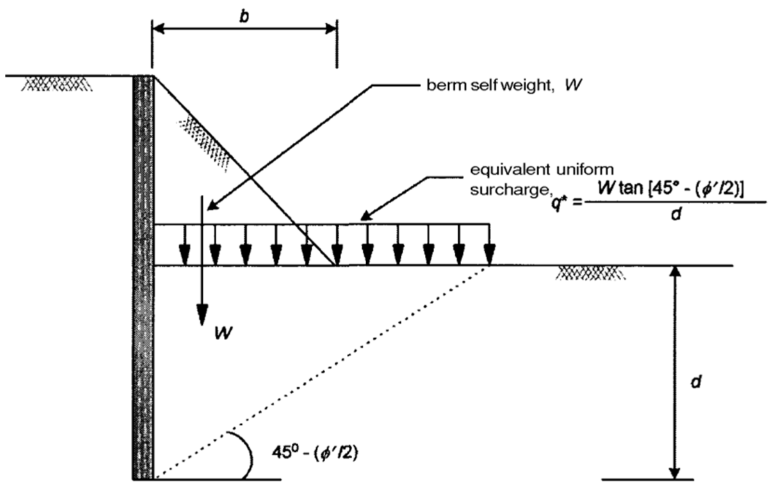


Figure 63: Representation of a berm as an equivalent surcharge (reproduced from Fleming 1994)

15.2.11 Eurocode 7 Design Context

EN 1997-1 §9.8:

"The design of supported excavations shall take into account the interaction between wall and supports, the sequence of construction, and the stiffness of the support system."

Partial factors (UK NA, DA1):

- **ULS (C1):** γ_F = 1.35 (perm.), 1.5 (var.); γ_M = 1.0
- **ULS (C2):** γ_F = 1.0; γ_M = 1.25
- **SLS:** $\gamma_F = \gamma_M$ = 1.0 (deflection check)

Support reactions are normally determined via:

- Simplified hand analysis (CIRIA C580 moment–area method), or
- Stiffness-based numerical analysis (e.g. WALLAP, PLAXIS).

15.2.12 Serviceability Considerations

- Limit wall deflection typically to **H/500–H/750** (urban excavations).
- Prop or anchor load variations < 20% between supports.
- Control pre-stress losses and monitor movements during excavation.
- Record actual loads and deflections — observational verification.

Long-term monitoring is mandatory for permanent anchors and basement props under *Observation Method* (EC7 §2.7).

Phicon Note:

• Always confirm **support stiffness compatibility** — overly stiff anchors or props can over-attract load and overstress wall sections.
• Define the **sequence of excavation and support activation** within design drawings; EC7 verification depends on it.
• Present **prop/anchor load tables** alongside pressure and moment envelopes in GDRs.
• Apply **load sharing correction** for asymmetric geometry or prop eccentricity.
• For temporary works, re-verify after each stage — e.g. *Stage 1 (unpropped) → Stage 2 (1st prop engaged) → Stage 3 (final excavation)*.
• Include a **QA/testing schedule** for anchors and struts as part of the design package — CAT II/III reviewers routinely request it.

15.3 Construction Sequencing (Top-down / Bottom-up)

This section describes how **construction sequence** affects the **mobilisation of earth pressures**, **support reactions**, and **wall deflections** in embedded wall systems. It outlines procedures for **bottom-up** and **top-down** construction, and explains how these are considered in **Eurocode 7 (EN 1997-1 §9.8)** design verification and observational control.

15.3.1 Purpose

Unlike gravity walls, embedded retaining structures rely on **interaction between wall, soil, and supports**, which develops progressively as excavation proceeds.

Correct sequencing ensures:

- Earth pressures are mobilised in the intended order,
- Supports engage at the right stage,
- Movements remain within service limits, and
- Staged analyses in design reflect actual construction practice.

EC7 §9.8 requires that "the design shall reflect the construction sequence and the staged mobilisation of earth and water pressures."

15.3.2 Construction Sequence in Design

Every embedded wall design must define:

1. Excavation stages and depths,
2. **Timing and activation** of props, anchors, or slabs,
3. Temporary water control measures,
4. Soil condition (drained or undrained) at each stage, and
5. Final retained height and permanent condition.

Each stage should be checked separately for both **ULS** (stability) and **SLS** (deflection), using earth pressures consistent with the expected wall movement at that stage.

15.3.3 Bottom-Up Construction

15.3.3.1 Description

The traditional method where the full excavation is carried out in lifts from the surface downward, installing supports as each level is reached.

Typical stages:

1. Install wall to full depth (sheet, secant, or diaphragm).
2. Excavate to first support level; install prop/anchor.
3. Repeat for subsequent levels.
4. Excavate to final formation; cast base slab or blinding.

15.3.3.2 Advantages

- Straightforward sequencing.
- Easier QA/QC and visual inspection.
- Simpler drainage control.

15.3.3.3 Limitations

- Excavation open until all levels supported — larger short-term wall movement.
- Temporary works may obstruct access or working space.

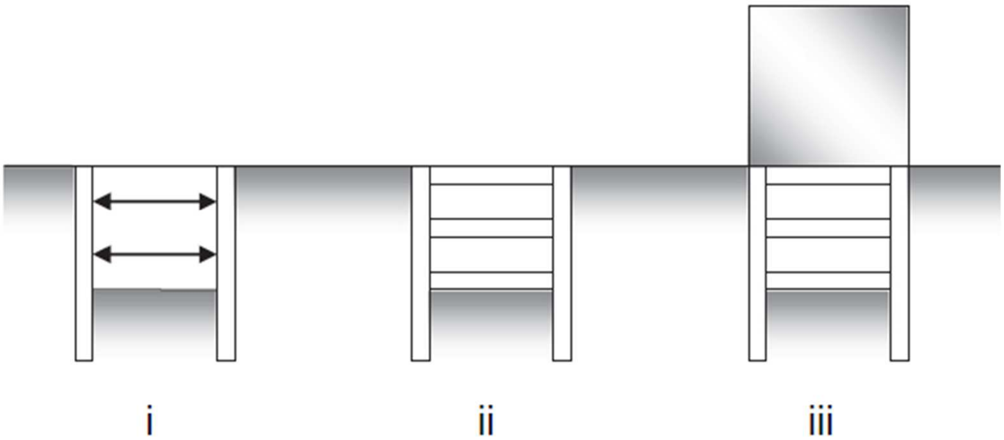


Figure 64: Typical bottom-up excavation sequence with two props (reproduced from Ingram 2012)

15.3.4 Top-Down Construction

15.3.4.1 Description

Permanent slabs (floor or roof) act as horizontal supports as excavation proceeds beneath them.

Typical stages:

1. Construct guide walls and install diaphragm or secant wall.
2. Cast ground-floor slab with openings for future excavation.
3. Excavate beneath slab to next level, install next slab, etc.
4. Proceed downward until final base slab cast.

15.3.4.2 Advantages

- Early superstructure start.
- Minimal movement to adjacent structures (slabs provide early restraint).
- No temporary props required.

15.3.4.3 Limitations

- Restricted headroom and access.
- Complicated ventilation and spoil removal.
- Difficult to inspect wall and joints during excavation.

Common in congested urban basements and transport stations where deformation control is critical.

15.3.5 Sequential Mobilisation of Earth Pressures

Stage	Active Side (Retained Soil)	Passive Side (Toe)	Support Condition	Typical K-Value
Stage 1 – before excavation	At-rest (K_0)	K_0	Fully restrained	K_0
Stage 2 – partial excavation, before prop	Mobilising toward K_a	Increasing to K_p	Free top	K_a / K_p partial

Stage 3 – first prop installed	Reduced movement	Passive increases below prop	Single restraint	K_a–K_0 mix
Stage 4 – second prop / final level	Fully developed pressure envelope	Passive fully mobilised	Multi-prop / fixed base	K_a (active zones), K_p (toe)

As supports are installed, the wall stiffens and earth pressures shift from triangular to trapezoidal or near-uniform shapes.

15.3.6 Moment Envelope Evolution

- **Cantilever stage:** Maximum moment near excavation level (concave shape).
- **Single-prop stage:** Moment reduced and shifted upward.
- **Two-prop stage:** Moments alternate between supports, forming *reversal points*.
- **Final stage:** Nearly symmetrical moment profile with reduced peak moments.

Designers should present **moment diagrams per stage**, showing progressive reduction and critical bending locations.

15.3.7 Analysis Approaches

Method	Description	Use
Classical limit-equilibrium (CIRIA C580)	Simplified linear or parabolic pressure envelopes for each stage.	Concept and verification design.
Stiffness-based (WALLAP, PLAXIS)	Accounts for soil and support stiffness; automatically simulates stage activation.	Detailed design and deflection prediction.
Observational approach	Wall movements measured and design adjusted during excavation.	Complex or high-risk urban excavations.

For critical projects, EC7 encourages comparison between simplified and numerical results for consistency.

15.3.8 Water Control and Sequencing

- **Dewatering** may precede or accompany each stage (well-points, sump pumping).
- Maintain **drawdown below excavation base** by ≥ 0.5 m to prevent base heave (see 15B.2).
- **Relief wells or drainage blankets** used to control pore pressures during staged excavation.
- Always consider **transient pore pressure** re-rise after pump shutdown in design.

15.3.9 Documentation and EC7 Verification

EC7 §9.8 requires that construction sequence effects be explicitly represented in both design and reporting.

A compliant design package should include:

1. **Stage table:** excavation depth, prop/anchor levels, construction sequence.
2. **Pressure diagrams:** one per stage (K_a/K_0/K_p zones).
3. Bending and shear envelopes: annotated by stage.

4. **Support load table:** design and anticipated measured loads.
5. **Deflection limits:** SLS compliance summary.
6. **Contingency notes:** remedial action if monitoring triggers are exceeded.

15.3.10 Serviceability and Observational Control

Monitoring during excavation provides direct feedback for verifying assumptions:

- **Inclinometers:** wall deflection profiles.
- **Load cells:** prop or anchor load variation.
- **Piezometers:** groundwater level control.
- **Settlement markers:** adjacent structure movement.

Trigger levels (Typical):

- **Alert:** 75 % of predicted deflection.
- **Action:** 100 % of predicted deflection or differential settlement > 10 mm.

Data are compared against staged FE or WALLAP predictions and reported weekly in QA dashboards.

Phicon Note:

• Always model excavation in *stages* — single-stage “final depth only” analysis is non-compliant with EC7.
• For **multi-propped walls**, ensure each prop is *activated at correct depth* in analysis; incorrect sequencing is a frequent CAT III audit failure.
• Clearly distinguish **temporary (construction)** vs **permanent (final)** load cases in the GDR.
• Use top-down sequencing where **deflection control** or **site access constraints** dominate.
• Include **monitoring trigger tables** and expected load ranges for each support in the verification package.
• Cross-link sequencing diagrams to **CDM risk assessments** and the **Construction Method Statement** — EC7 reviewers expect alignment between design and site execution.

15.4 Water Pressure and Uplift Considerations

This section details the assessment of **hydrostatic pressures, uplift, and hydraulic stability** for embedded retaining walls and deep excavations, in accordance with **Eurocode 7 (EN 1997-1 §§10.4 & 10.5)** and **CIRIA C580**. It explains how to check **uplift (UPL)** and **hydraulic heave/piping (HYD)** limit states, both of which are critical when groundwater lies above the excavation base.

15.4.1 Purpose

Water pressure is often the **governing action** in deep excavations and basements.
If uncontrolled, it can cause:

- Uplift of base slabs,
- **Hydraulic heave** (upward flow through soil beneath the wall),
- **Piping or erosion** of fine material, and
- **Global instability** of the retained system.

Eurocode 7 requires explicit verification of *UPL (uplift)* and *HYD (hydraulic failure)* limit states wherever water may induce upward flow beneath the excavation or structure.

15.4.2 Water Pressure Components

Total pore pressure at depth *z* below the water surface:

$$u = \gamma_w h$$

Where;

- u= hydrostatic pressure (kPa),
- γ_w = 9.81 kN/m^3,
- h= vertical head of water above point considered.

Pore pressure distributions are linear under static conditions, but may be **non-linear under seepage** where head gradients exist beneath the excavation (e.g. due to cut-off walls, drains, or relief wells).

15.4.3 Uplift of Base Slabs (UPL Limit State)

- Design Requirement

$$V_d \geq U_d$$

Where;

V_d: design downward load (self-weight + surcharge + structure),
U_d: design uplift due to pore water pressure.

or equivalently,

$$\frac{V_d}{U_d} \geq 1.0 \text{ (ULS)}$$

and

$$\text{FOS}_{SLS} \geq 1.2\text{–}1.3$$

- Actions Included
- Self-weight of base slab and overlying structure,
- Weight of retained soil or pavement layers,
- Pore pressure head beneath slab (hydrostatic + seepage).

Include a **minimum 0.5 m head margin** (or 10 % of total head, whichever is greater) to cover uncertainty in water levels.

15.4.4 Design Example – Base Uplift

Base slab at depth 6 m, groundwater at surface:

$$U = \gamma_w h = 9.81 \times 6 = 58.9 \text{ kPa}$$

If self-weight + structure = 70 kPa →

$$\text{FOS}_{SLS} = \frac{70}{58.9} = 1.19 < 1.2$$

Increase thickness or add tie-down anchors.

✓ Minimum FOS achieved when $V \geq 1.2U$ (SLS) and $V \geq 1.0U$ (ULS).

15.4.5 Hydraulic Heave beneath Excavations (HYD Limit State)

- Mechanism

Upward seepage through the base causes loss of effective stress.
Heave occurs when the **vertical seepage force** equals the submerged weight of soil:

$$i_{crit} = \frac{\gamma'}{\gamma_w}$$

Where;

i= hydraulic gradient (Δh / L),
γ'= effective (submerged) unit weight of soil.

- Design Requirement

$$i \leq i_{crit}/\gamma_F$$

with γ_F = 1.35 (unfavourable water head).

Typical Values

Soil Type	γ' (kN/m^3)	$i_{(crit)}$	Comment
Loose sand	10	1.0	Very susceptible to piping
Medium dense sand	11	1.1	Heave likely if uncut
Dense sand / gravel	12–13	1.2–1.3	Safer
Clay (impermeable)	—	—	Not applicable (drained uplift dominates)

Safety is improved by deeper wall embedment, berms, drainage, or lowering the external water level.

15.4.6 Verification of Hydraulic Heave (EC7 §10.4.3)

If water can flow under the wall toe, check for uplift of soil immediately beneath the excavation base.

For flow path length L and head difference Δh:

$$i = \frac{\Delta h}{L}$$

If $i \geq i_{crit}$, base instability or piping may occur.
For cohesive or layered soils, effective stress analysis may be used instead.

For cut-off walls, use seepage models (2D flow nets, finite element) to compute Δh beneath the toe and verify gradients.

15.4.7 Piping and Internal Erosion

Progressive removal of fine particles under upward seepage leads to **piping**.
Check that:

- Filter or drainage layer meets **Terzaghi filter rule**,
- Exit hydraulic gradient at downstream face $i_{exit} \leq 0.5\, i_{crit}$,
- Provide graded filter or geotextile to retain fines.

For coarse fill foundations, add a **graded filter layer (D_{15}/D_{85} ≤ 4)** or geotextile per **BS EN 13252**.

15.4.8 Cut-off and Relief Systems

Mitigation measures when hydraulic gradients exceed limits:

Method	Description	Effect on Flow Path
Deeper wall embedment	Extends seepage length below base	Reduces Δh/L
Grouted cut-off wall	Impermeable barrier under toe	Blocks direct flow path
Relief wells / pressure relief drains	Allows controlled upward flow into sump	Reduces head under base
Drainage blanket	Free-draining layer below slab	Reduces pore pressure by 30–70 %
Under-reamed anchors or tension piles	Resist uplift by tensile capacity	Increases V_d/U_d ratio

15.4.9 Design of Tie-Down Anchors for Uplift

When uplift exceeds structural weight, install **tension piles or rock anchors** through base slab.

Design load per anchor:

$$T_d = \frac{U_d - V_d}{n}$$

where *n* = number of anchors.

- Verify bond length, pull-out resistance, and creep (EN 1537).
- Allow for partial factors:

$$\gamma_{R;anchor} = 1.3 \text{ (temporary)}, 1.5 \text{ (permanent)}$$

15.4.10 Seepage and Finite-Element Analysis

For deep basements or marine cut-offs:

- Construct **2D or 3D seepage model** (e.g. PLAXIS, SEEP/W).
- Apply head boundaries at internal/external water levels.
- Extract uplift pressure distribution beneath slab.
- Verify both *UPL* and *HYD* limit states using derived pressures.

For rapid drawdown or transient pumping, use transient flow analysis to simulate pore pressure rebound.

15.4.11 Eurocode 7 Verification Summary

Limit State	Criterion	Typical Partial Factors (UK NA)	Target FOS
UPL (Uplift)	$V_d \geq U_d$	γ_F = 1.35 (unfavourable), γ_M = 1.0	1.0 (ULS) / 1.2 (SLS)
HYD (Heave)	$i \leq i_{crit}/\gamma_F$	γ_F = 1.35	≥ 1.0 (ULS)
Piping	$i_{exit} \leq 0.5 i_{crit}$	γ_F = 1.35	≥ 1.5 (SLS)

All uplift and seepage verifications must correspond to *the same assumed phreatic surface* used for wall design.

Phicon Note:

• Never assume drained conditions below formation without evidence — always start with *fully saturated hydrostatic head*.
• Present **uplift, heave, and piping checks together** on one sheet with common head diagram.
• Where monitoring is feasible, install **piezometers beneath the slab** to validate design pore pressures.
• For EC7 reporting, use clearly labelled **UPL** and **HYD** limit-state tables — many reviewers require explicit separation.
• Include **transient drawdown scenario** if groundwater control depends on temporary pumping.
• For permanent basements, always provide *dual safety mechanism*: structural resistance + drainage relief.

16 Structural Checks for Geotechnical Elements (STR)

16.1 Design Codes and Interfaces (EN 1992, EN 1993-5)

This section outlines how **structural design verification** integrates with geotechnical design under **Eurocode 7 (EN 1997)**, **Eurocode 2 (EN 1992)**, and **Eurocode 3 Part 5 (EN 1993-5)**.

It defines the **STR limit state**, the relevant design interfaces, and practical cross-checks required for piles, walls, anchors, and reinforced concrete or steel members forming part of a geotechnical system.

16.1.1 Purpose

While Eurocode 7 governs **soil–structure interaction** and overall stability (GEO, EQU, UPL, HYD), the **structural integrity** of the elements themselves is verified under the **Structural Limit State (STR)**.

This ensures that:

- Bending, shear, and axial capacities of concrete or steel components are not exceeded.
- Serviceability requirements (deflection, crack control, durability) are satisfied.
- Structural design parameters are compatible with geotechnical boundary conditions (e.g., reaction pressures, load transfer zones).

EC7 §2.4.7.3 distinguishes STR checks (material failure) from GEO checks (ground failure). Both must be verified separately.

16.1.2 Design Interfaces Between EC7, EC2, and EC3-5

Element Type	Geotechnical Design Code	Structural Design Code	Interface Definition
Reinforced concrete wall / raft / slab	EN 1997-1 (GEO/UPL)	EN 1992-1-1 (STR)	Soil reaction → design bending, shear, crack width
Steel sheet pile	EN 1997-1 (embedment, earth pressures)	EN 1993-5 (cross-section strength, interlocks)	Earth pressure → design moment / shear in pile
Ground anchor	EN 1997-1 (bond, pull-out)	EN 1993-1-8 (steel tension member) / EN 1537	Anchor force → tendon stress and bond verification
Pile (bored, CFA, driven)	EN 1997-1 §7	EN 1992-1-1 / EN 1993-5	Axial / bending load → section resistance and detailing

The design interface defines where **GEO actions** from soil pressures or reactions become **design effects (E_sd)** in the structural verification under EC2/EC3.

16.1.3 Structural Limit State (STR)

The STR limit state is satisfied when:

$$E_d \leq R_d$$

Where;

- E_d= design effect of actions (bending, shear, axial force) from geotechnical analysis,
- R_d= design resistance of the structural section (reinforced concrete or steel).

Design actions are obtained from verified EC7 calculations (e.g., PLAXIS, WALLAP, or hand analysis envelopes), then applied directly to structural models.

16.1.4 Partial Factors (UK NA, DA1)

Parameter	C1 (Actions)	C2 (Materials)
Permanent unfavourable action	$\gamma_F = 1.35$	—
Variable action	$\gamma_F = 1.5$	—
Material strength (concrete, steel)	—	$\gamma_M = 1.15$
Combination rule	C1 or C2 gives governing case	—

EC7 Design Approach 1 (DA1) ensures that both *action-factored* and *resistance-factored* combinations are tested.

16.1.5 Structural Inputs from Geotechnical Design

From EC7 Design	To EC2 / EC3 STR Check
Earth pressure envelope	Bending moment & shear diagrams
Passive / active soil reaction	Base pressure & axial load distribution
Embedment depth & fixity	Boundary conditions for bending
Effective stress at base	Axial compression or uplift
Water pressure	Buoyancy load on slab or pile
Support reactions (props / anchors)	Connection forces & detailing loads

These must be imported consistently — e.g., design moment at pile cut-off or wall mid-span should match the governing GEO case (DA1/C2).

16.1.6 Verification Process

1. **Transfer of loads:** Extract design effects E_d(moment, shear, axial) from EC7 outputs.
2. **Material resistance:** Compute $R_d = f_y A / \gamma_M$(steel) or $M_{Rd} = f_{cd} z$(RC).
3. **Compare:** Check $E_d \leq R_d$.
4. **Serviceability:** Verify crack widths, deflection, and stress limits under characteristic (unfactored) combinations.
5. **Durability:** Check covers, corrosion allowances, and exposure classes per BS EN 1992-1-1 / BS EN 1993-5.

16.1.7 Coordination of Design Disciplines

- The **geotechnical designer** supplies *reaction envelopes* and *soil pressures* to the structural engineer.
- The **structural designer** checks reinforcement, section sizing, and service performance.
- The **GDR** (Geotechnical Design Report) must clearly define **responsibility boundaries** — e.g., "Wall bending moments per EC7 verified by Phicon; reinforcement design per EC2 by RC designer."

Misalignment of envelopes is a common audit issue — ensure both parties use the same **design case (C1 or C2)** and **partial factors**.

16.1.8 Software Workflow (Typical)

Stage	Software / Output	Purpose
GEO verification	PLAXIS / WALLAP / LimitState:GEO	Determine soil pressures, moments, shear envelopes
STR verification	Tedds / IDEA StatiCa / Robot / RC spreadsheets	Check RC/steel section capacities
Integration	Excel / PDF summaries	Combine GEO and STR checks for submission

16.1.9 Durability and Detailing

Environment	Code Clause	Minimum Cover / Protection
Buried RC basement / wall	EN 1992-1-1 Table 4.4N	75 mm (XC3–XC4)
Marine or tidal zone (steel sheet piles)	EN 1993-5 §7.4	1.5–3 mm/yr corrosion allowance or coating
Anchors / tie rods	EN 1537 / EN 1993-1-8	Double corrosion protection (permanent)

⚠️ Always link cover and corrosion provisions back to **intended design life** and **inspection frequency** — EC7 §2.4.1(5) requires that durability be verified over the design period.

16.1.10 Reporting and QA Expectations

- Include both **GEO** and **STR** design sheets in calculation packs.
- Annotate **load path diagrams** showing how geotechnical actions translate to structural effects.
- Reference design clauses (e.g. EN 1992-1-1 §6.1 for flexure).
- Provide summary table of design actions and capacities:

Element	Action E_d	Resistance R_d	Ratio E_d/R_d	Status
Wall (mid-span)	320 kN.m	390 kN.m	0.82	✓ OK
Pile (axial)	820 kN	940 kN	0.87	✓ OK
Anchor tendon	350 kN	420 kN	0.83	✓ OK

Phicon Note:

• Always show which **Design Approach (DA1/C1 or DA1/C2)** governs — this ensures traceability between GEO and STR verifications.
• For **sheet piles**, quote section modulus and steel grade directly from EC3-5 tables.
• For **RC elements**, clearly note the **effective cover**, **exposure class**, and **bar spacing** used in crack checks.
• Maintain one **combined GEO + STR summary page** for Cat II/III submissions — reviewers expect both limit states side-by-side.

• Where bending envelopes from GEO software exceed section capacity, adjust wall thickness or reinforcement before resubmission — do not modify soil parameters to suit.
• Archive all software output plots (pressure, moment, shear) as PDFs linked to the GDR index for audit traceability.

16.2 Sheet Pile Section Bending & Shear Capacity

This section describes the verification of bending and shear strength for steel sheet piles under the Structural Limit State (STR), following EN 1993-5: Eurocode 3 – Design of Steel Structures, Part 5: Piling.

It provides simplified procedures to check section modulus adequacy, combined bending–shear effects, and durability provisions, as typically required for embedded wall design to Eurocode 7.

16.2.1 Purpose

Once the geotechnical (GEO) analysis defines the **design bending moment (M_n)** and **shear force (V_n)** along the wall (e.g., from WALLAP or PLAXIS), the **section resistance (R_n)** of the chosen sheet pile profile must be verified at the **STR limit state**.

The check ensures:

- The pile section resists the design moment and shear with appropriate partial factors,
- Interlocks can transfer the required shear between piles, and
- Buckling, plastic hinging, and corrosion losses are accounted for.

EC7 provides the design actions; EC3-5 provides the resistance checks. The STR limit state confirms that the steel section itself will not fail before the ground does.

16.2.2 Governing Design Equation

For each critical section of the wall:

$$M_{Ed} \leq M_{Rd} \text{ and } V_{Ed} \leq V_{Rd}$$

Where;

- M_{Ed}, V_{Ed}: design effects (from geotechnical analysis),
- M_{Rd}, V_{Rd}: design resistances per EN 1993-5.

If both act together:

$$\frac{M_{Ed}}{M_{Rd}} + \frac{V_{Ed}}{V_{Rd}} \leq 1.0$$

16.2.3 Design Moment Resistance (EN 1993-5 §6.2.1)

$$M_{Rd} = W_{pl,eff} \times \frac{f_y}{\gamma_{M0}}$$

Where;

- $W_{pl,eff}$: effective plastic section modulus (cm^3/m),
- f_y: yield strength of steel (typically 355 MPa for S355GP),
- $\gamma_{M0} = 1.0$(partial factor for material resistance).

16.2.3.1 Worked Example (Typical Verification)

Parameter	Value	Unit
Design bending moment, M_{Ed}	420	kNm/m
Steel grade	S355GP	—
f_y	355	MPa
Required $W_{pl,req}$	$M_{Ed}/f_y = 420 \times 10^6/355 = 1.18 \times 10^6$	mm^3/m

Hence, choose section with $W_{pl,eff} \geq 1.2 \times 10^6$ mm^3/m→ e.g. **Arcelor AZ18-700** (W_p = 1.26 × 10^6 mm^3/m). ✓ **OK** at STR limit state.

16.2.4 Shear Resistance (EN 1993-5 §6.2.6)

Shear strength check:

$$V_{Rd} = A_{v,eff} \times \frac{f_y}{\sqrt{3} \times \gamma_{M0}}$$

Where;

- $A_{v,eff}$: effective shear area (mm^2/m),
- f_y: yield stress.

Typical for sheet piles:

$$A_{v,eff} \approx 0.8A$$

⚠️ Shear rarely governs in practice; bending is the usual controlling limit state unless toe loads or impact occur.

16.2.5 Bending–Shear Interaction (EN 1993-1-1 §6.2.8)

When both bending and shear occur:

$$\frac{M_{Ed}}{M_{Rd}} + \frac{V_{Ed}}{V_{Rd}} \leq 1.0$$

If $V_{Ed} < 0.5V_{Rd}$, interaction can be neglected.

Design programs (WALLAP, Repute, or IDEA StatiCa) typically apply this automatically.

16.2.6 Deflection and Serviceability

Service deflection limit (SLS):

$$\delta_{max} \leq H/500 \text{ to } H/750$$

Where, H = retained height.

Deflection usually verified by stiffness-based analysis (e.g., PLAXIS).

For excessive movements, increase section stiffness (EI) by:

- Selecting higher modulus section (e.g., AZ20 → AZ26),
- Using double pile configuration, or
- Adding tie rods or props.

16.2.7 Corrosion and Section Loss

Design Life Considerations

Environment	Corrosion Rate (mm/yr)	Design Life (yrs)	Allowance (mm/face)
Inland groundwater	0.01–0.02	50	0.5–1.0
Tidal zone	0.03–0.05	50	1.5–2.5
Splash zone	0.06–0.10	50	3.0–5.0
Temporary (≤2 yrs)	Negligible	—	None

Effective section modulus reduction:

$$W_{pl,eff,cor} = W_{pl,nom}(1 - \frac{2t_{cor}}{t_{web}})$$

where t_{cor}= corrosion loss per face.

For permanent marine structures, either apply **sacrificial thickness** or **protective coating (zinc, epoxy, cathodic)** per EN 1993-5 §7.4.

16.2.8 Steel Grades and Section Properties

Grade	Yield Strength (f_y, MPa)	Typical Use
S270GP	270	Temporary works, short walls
S355GP	355	Standard permanent applications
S390GP	390	High load walls, marine
S430GP	430	Special high-strength applications

Common profiles (Arcelor, JFE, ESC, etc.):

Section	W_p (cm^3/m)	A (cm^2/m)	Weight (kg/m^2)
AZ14-700	930	107	84
AZ18-700	1260	137	106
AZ26-700	1820	194	151
AZ32-750	2250	230	179
AU25	1750	197	153

Always quote the *effective* W_p after corrosion deduction in verification sheets.

16.2.9 Interlock Shear and Watertightness

- **Interlock shear capacity** is typically > 10% of section shear; check for concentrated loads or marine impact.
- **Watertightness** can be enhanced by:
  - Hot-melt bitumen,
  - Polyurethane sealant,
  - Hydrophilic strip (for permanent basements).

For press-in piles, interlock friction reduction factors (≈ 0.6–0.8) may be used in shear verification.

16.2.10 Presentation of STR Verification

Each wall should include a **section verification sheet** summarising:

Location	M_ed (kN.m/m)	V_ed (kN/m)	Section	M_rd (kN.m/m)	V_rd (kN/m)	Status
Mid-span	420	60	AZ18-700 (S355)	448	202	✓ OK
Near toe	270	95	AZ18-700 (S355)	448	202	✓ OK

Attach as an appendix to the GDR under **STR verifications**.

Phicon Note:

• Always select the **governing moment** from GEO (DA1/C2) — never understate bending by relying on C1 results alone.
• Maintain **consistent units (kNm/m)** across GEO and STR sheets.
• Deduct **corrosion allowance** before calculating $W_{pl,e}$ff.
• Verify both **M-V interaction** and **deflection criteria (SLS)**.
• Include **steel grade, coating system, and expected design life** explicitly in the GDR.
• For **permanent marine works**, confirm compliance with **BS EN ISO 12944** corrosion category (C4–CX).
• Always include a **section modulus table** and **strength-utilisation chart** in STR appendices — this is a CAT III expectation for major ports and basements.

16.3 Reinforced Concrete Walls and Rafts – Cover, Crack Control, Durability

This section provides concise guidance for verifying **reinforced concrete (RC) retaining walls, rafts, and basement slabs** under the **Structural Limit State (STR)**, consistent with **Eurocode 2 (EN 1992-1-1)** and UK National Annex provisions.

It focuses on **cover, crack control, and durability**, which are often the governing design checks for permanent substructures in contact with soil and groundwater.

16.3.1 Purpose

Reinforced concrete elements in geotechnical applications must satisfy:

1. **Ultimate limit state (ULS)** – adequate flexural and shear capacity,
2. **Serviceability limit state (SLS)** – control of crack widths and deflection,
3. **Durability** – corrosion resistance, water tightness, and long-term integrity.

EC2 treats crack control and cover not merely as detailing rules but as essential design verifications linked to exposure class, design life, and construction quality.

16.3.2 Applicable Codes and References

Code / Standard	Title / Scope
EN 1992-1-1	Design of Concrete Structures – General rules and buildings
EN 1992-3	Liquid retaining and containment structures (waterproofing guidance)
BS 8500-1 / -2	Concrete – Complementary UK guidance on durability and mix design
CIRIA C766	Basement waterproofing design and detailing (UK best practice)
NHBC Chapter 5.4	Waterproofing of basements and substructures

16.3.3 Exposure and Durability Classes

Environment	Exposure Class (EN 206 / BS 8500)	Example Location	Min. Cover (mm)
Buried, damp soil (no chloride)	XC2 / XC3	Retaining wall below grade	50–60
Constantly wet soil / groundwater	XC4	Basement wall, raft slab	65–75
Marine splash or tidal	XS2 / XS3	Quay wall, marine basement	75–90
Aggressive chemical soil (sulphate class 3)	XA2–XA3	Sulphate-bearing clay, landfill	65–85

⚠️ Minimum covers assume C35/45 concrete, 50-year design life, and good compaction quality (execution class 2). Increase cover by +10 mm if tolerances or quality are uncertain.

16.3.4 Crack Width Control (EC2 §7.3)

Crack width limits depend on **exposure** and **function**:

Condition	Limit w_k (mm)	Purpose
Dry internal environment	0.3	Aesthetic / serviceability
Buried wall, non-water-retaining	0.3	Durability
Basement wall resisting groundwater (Type B waterproofing)	0.2	Watertightness
Liquid-retaining wall (Type A waterproofing)	0.1	Tightness / leakage prevention

Crack width must be checked under **characteristic quasi-permanent load combination** (SLS), using design stress in reinforcement.

16.3.5 Simplified Check (EN 1992-1-1 Eq. 7.9N):

$$w_k = s_{r,max}(\varepsilon_{sm} - \varepsilon_{cm})$$

Where;

- $s_{r,max}$: maximum crack spacing,
- ε_{sm}: mean steel strain,
- ε_{cm}: mean concrete strain.

Alternatively, for practical design:

$$\sigma_s \leq \frac{k_3 c + k_1 k_2 \varphi}{\rho_{p,eff}} \times 1.3 w_{k,limit} E_s$$

(EC2 Table 7.3N provides constants k_1–k_3.)

16.3.6 Simplified UK Practice (Basement Wall, C35/45, Type B):

Bar dia. (mm)	Spacing (mm)	Max σ_s (MPa) for w_k = 0.2 mm
12	150	160
16	200	180
20	250	200

If higher stress is expected, increase bar area or reduce spacing.

16.3.7 Minimum Reinforcement (EN 1992-1-1 §9.2.1.1)

For crack control (tensile zones):

$$A_{s,min} = k_c k f_{ct,eff} A_{ct} / f_y k$$

Typical simplification (C35/45 concrete):

$$A_{s,min} \approx 0.002A_{ct}$$

(≈ 0.2% of gross concrete area in tension zone.)

For temperature/shrinkage reinforcement in slabs:

$$A_{s,min} = 0.001A_c$$

with spacing ≤ 3 × thickness or 400 mm (whichever smaller).

16.3.8 Water-Retaining and Basement Structures

For basement slabs and walls under hydrostatic head:

- Adopt **Type B waterproofing** (BS 8102:2022): water-resistant concrete + crack control.
- Verify **uplift (UPL)** separately (see Section 15.4).
- Crack width limit **w_k ≤ 0.2 mm**, ideally 0.15 mm if water pressure > 5 m.
- Use **minimum wall thickness 300 mm** to achieve low permeability.
- Waterstops at joints (PVC or hydrophilic) are mandatory.

CIRIA C766 recommends controlling early-age thermal cracking by staged pour sequences or low-heat cement mixes.

16.3.9 Bending and Shear Capacity Verification (STR)

At ULS:

$$M_{Ed} \leq M_{Rd} = A_s f_{yd} z \text{and} V_{Ed} \leq V_{Rd} = 0.6(1 - f_{ck}/250)bdf_{cd}$$

Design values per EC2:

- $f_{cd} = f_{ck}/\gamma_c$, with $\gamma_c = 1.5$,
- $f_{yd} = f_y/\gamma_s$, with $\gamma_s = 1.15$.

For embedded RC walls, bending is often controlled by crack-width and serviceability rather than ultimate strength.

16.3.10 Thermal and Shrinkage Effects

Temperature gradients and autogenous shrinkage induce restrained stresses.
EC2 requires that restraint be considered explicitly where wall or slab is restrained by adjacent elements.

Crack Risk Mitigation	Recommended Practice
Construction joints	Max 7.5–10 m spacing
Reinforcement continuity	Provide additional temperature steel at mid-depth
Cooling / low-heat cement	For thick rafts (>600 mm)
Staged pour sequence	Reduce thermal gradient differential
Controlled curing	Maintain surface temperature uniformity

Include early-age crack control verification for thick pours (>0.6 m) or when water tightness is critical.

16.3.11 Typical Reinforcement Layouts

Element	Face	Typical Steel Ratio (ρ)	Spacing
Basement wall (H ≤ 3.5 m)	Inner + outer	ρ = 0.25–0.35 %	T12–T16 @ 150–200 mm
Raft slab (300–600 mm)	Top + bottom	ρ = 0.2–0.3 %	T16–T20 @ 200–250 mm
Retaining wall stem	Back (tension face)	ρ = 0.3 %	T16 @ 150 mm

16.3.12 Durability Detailing Checklist

✓ Provide adequate **cover** per exposure class and execution tolerance.
✓ Ensure **minimum reinforcement ratios** for crack control.
✓ Maintain bar spacing ≤ 300 mm where waterproofing is required.
✓ Place **waterstops** at all joints (horizontal and vertical).
✓ Seal penetrations with cast-in or welded sleeves.
✓ Specify **cement type** and **admixtures** for reduced permeability.
✓ Ensure all reinforcement laps and bends are consistent with EC2 §8.7.

Phicon Note:

• Always document **exposure class**, **cover**, and **crack-width limit** directly on drawings — this is the first item checked in CAT II/III reviews.
• For permanent basements, include a **Durability and Waterproofing Schedule** in the GDR: concrete grade, cover, bar size, joint type, and protection system.
• For **Type B structures**, perform both **UPL** and **SLS (crack)** verifications on the same sheet.
• Crack control governs most RC wall designs — optimise rebar spacing before increasing thickness.
• Ensure **service combination** used for w_k check (unfactored, long-term) is clearly stated.
• Align **joint layout** with pour sequence and waterstop detailing.
• Record all cover checks and bar layouts in PDF output for QA traceability.

16.4 Anchors and Ties – Steel Strength, Bond Length, Corrosion Allowance

16.4.1 Purpose

Ground anchors and tie rods resist tensile actions generated in retaining and embedded wall systems. Their design ensures the **steel capacity**, **bond transfer**, and **corrosion protection** are adequate for the design life.

Verification follows **Eurocode 7 (EN 1997-1 §9)** for geotechnical actions and **Eurocode 3 Part 5 (EN 1993-5 §6)** for steel design, with **EN 1537** governing anchor execution and testing.

Correct detailing is critical to maintain load transfer, prevent long-term creep, and ensure durability under permanent groundwater exposure.

16.4.2 Anchor Behaviour and Components

Zone	Function	Typical Material	Design Focus
Fixed Length	Transfers load to soil/rock through grout	Cement grout	Bond stress, creep, pull-out
Free Length	Allows tendon extension to mobilise load	Plastic-sheathed tendon	Friction minimisation
Anchor Head	Transfers force to wall/plate	Bearing plate & nut	Steel capacity, corrosion seal
Protection System	Maintains long-term durability	Double Corrosion Protection (DCP)	Watertight encapsulation

Typical parameters influencing capacity:

- Soil or rock shear strength (τ_k)
- Grout quality and pressure
- Tendon area $A_s$and yield strength f_y
- Bond length $L_f$and diameter d
- Corrosion environment and service life requirement

16.4.3 Design Equations and Checks

(a) Steel Strength (EN 1993-5 §6.2)

$$T_{Rd} = \frac{A_s f_y}{\gamma_{M0}}$$

where
A_s= steel area (mm^2), f_y = characteristic yield strength (MPa), $\gamma_{M0} = 1.00.$

Ultimate resistance:

$$T_{Rd,u} = \frac{A_s f_u}{\gamma_{M2}}$$

with f_u= ultimate tensile strength, $\gamma_{M2} = 1.25.$

Design load must satisfy $T_{Ed} \leq 0.6\, A_s f_y$to limit relaxation and creep.

(b) Bond Resistance (EN 1537 §7.3)

$$T_{Rd,\,bond} = \pi d L_f \tau_{Rd}$$
$$\tau_{Rd} = \frac{\tau_k}{\gamma_{R,bond}}$$

Where, τ_k= characteristic bond stress, $\gamma_{R,bond} = 1.25.$

Ground Type	τ_k(kPa)	Comment
Dense sand / gravel	150 – 250	Pressure-grouted anchors
Stiff clay	75 – 150	Fully grouted
Weak clay / silt	25 – 75	May exhibit creep
Sound rock	300 – 1000	Roughness dependent

(c) Free Length and Elongation

Ensure sufficient extension to mobilise tension:

$$\Delta = \frac{T_{Ed} L_{free}}{A_s E_s}$$

where E_s≈ 200 GPa.
Total elongation Δshould be ≤ 1 % of L_{free}.
Lock-off load typically = 50–70 % of design load after creep stabilisation.

16.4.4 Typical Design Values

Parameter	Temporary Anchor	Permanent Anchor (DCP)	Reference
Steel grade	S355 / 1770 MPa strand	S460 / 1770 MPa strand	EN 10025 / EN 10138
Partial factor $\gamma_{(}$M0$_{)}$	1.10	1.00	EN 1993-5
Bond safety $\gamma_{R,bond}$	1.25	1.25	EN 1537
Service stress ratio (T_{serv} / T_{Rd})	≤ 0.60	≤ 0.55	EN 1537
Design life	< 2 years	≥ 50 years	BS 8081
Corrosion protection	Single grout cover	Double Corrosion Protection (DCP)	EN 1537 §8

16.4.5 Testing and Verification (EN 1537 §10)

Test Type	Purpose	Load Level	Typical Application
Suitability Test	Confirms design parameters	1.5 × T_{serv}	Trial anchors
Acceptance Test	Verifies installed capacity	1.25 × T_{serv}	Each working anchor
Creep Test	Checks long-term load loss	1.1 × T_{serv}	Selected anchors

Creep rate ≤ 2 mm per log cycle of time after stabilisation.
Residual load after unloading ≥ 90 % of lock-off load.

16.4.6 Corrosion Allowance and Durability

(a) Temporary Anchors

Allow corrosion loss on exposed steel faces:

Environment	Rate (μm/yr)	6-Month (mm)	1-Year (mm)
Inland	50	0.03	0.05
Marine / splash	120	0.07	0.12

Increase design area or reduce working stress accordingly.

(b) Permanent Anchors (DCP)

Comprise:

1. Individual sheath around tendon filled with grease or grout.
2. Outer grout column to borehole wall.
3. Sealed anchor head assembly.

✓ No corrosion allowance required — durability achieved by complete encapsulation and QA inspection (pressure test of sheath to 0.5 MPa).

16.4.7 Worked Example — permanent strand anchor (DCP)

Given

- Design tensile effect from EC7/EN 1990: $T_{Ed} = 600$ kN.
- 7-wire strand bundle, effective steel area $A_s = 300$ mm^{2}per strand; use 8 strands → $A_s = 2400$ mm^2.
- Strand $f_y = 1570$ MPa, $f_u = 1770$ MPa; $\gamma_{M0} = 1.00, \gamma_{M2} = 1.25$.
- Fixed bulb diameter $d = 150$ mm; characteristic bond in dense sand $\tau_k = 200$ kPa; $\gamma_{R,bond} = 1.25$.

Steps

1. Steel capacity (yield and ultimate):

$$T_{Rd,y} = \frac{A_s f_y}{\gamma_{M0}} = \frac{2400 \times 1570}{10^3} = 3{,}768 \text{ kN (OK)}$$
$$T_{Rd,u} = \frac{A_s f_u}{\gamma_{M2}} = \frac{2400 \times 1770}{1.25 \times 10^3} = 3{,}398 \text{ kN (OK)}$$

2. Bond resistance → required fixed length L_f:

$$\tau_{Rd} = \frac{200}{1.25} = 160 \text{ kPa}, T_{Rd,bond} = \pi d L_f \tau_{Rd}$$

Set $T_{Rd,bond} \geq T_{Ed} = 600$ kN:

$$L_f \geq \frac{600 \times 10^3}{\pi \times 0.15 \times 160 \times 10^3} = 7.96 \text{ m}$$

Adopt $L_f = 8.5$ m (allowance for construction tolerances).

3. **Service stress & lock-off:**

   Target $T_{serv} \leq 0.55\, A_s f_y = 0.55 \times 3{,}768 = 2{,}072$ kN (well above 600→ OK).

   Set lock-off at 60–70 % of T_{Ed} after creep test → 360–420 kN.

4. **Free length elongation (check):**

   Assume $L_{free} = 12$ m, $E_s = 200$ GPa:

$$\Delta = \frac{T_{Ed} L_{free}}{A_s E_s} = \frac{600 \times 10^3 \times 12}{2400 \times 200 \times 10^3} = 0.015 \text{ m} = 15 \text{ mm}$$

$\Delta / L_{free} = 15/12000 = 0.13\%$ (mobilisation adequate, movement acceptable).

Result
8-strand DCP anchor provides ample steel capacity; fixed length 8.5 msatisfies bond; free-length extension is reasonable.

16.4.8 Practical Application Notes

- Use **grout pressures ≥ 300 kPa** in coarse soils to ensure full bond.
- Maintain free-length smoothness; avoid kinks or bends at transition.
- Protect anchor heads immediately after stressing; seal with cap and mastic.
- For critical structures > 6 m deep, design two or more anchor levels.
- Record every anchor test (load – extension curve) in the Construction Verification File.

Phicon Note:

• Treat anchors and tie rods as **structural (STR)** verifications under **EN 1993-5 §6**, with actions imported from **EC7 DA1-C1** analysis.
• In Phicon calculators:
  – default γ_{M0} = 1.00 (permanent), 1.10 (temporary);
  – bond factor $\gamma_{(R,bond)}$ = 1.25.
• Permanent anchor templates assume **Double Corrosion Protection**, with QA prompts for sheath integrity and grout continuity.
• Include in design summary:
  "Anchor/tie verified per EN 1993-5 §6 and EN 1537 (Execution of Ground Anchors). Actions derived from Eurocode 7 DA1-C1 analysis."
• For CAT II / III designs, summarise anchor system type, test regime, and corrosion protection in the Geotechnical Design Report (GDR).

PART 4: HYDRAULIC, EARTHWORKS & GROUND IMPROVEMENT

17 Hydraulic Uplift & Basal Heave (UPL/HYD)

Hydraulic stability forms a critical verification step in deep basements, excavations, and raft foundations where **groundwater pressure** can act upward against the base.

Eurocode 7 treats both **uplift (UPL)** and **hydraulic heave (HYD)** as distinct ultimate limit states requiring explicit checks alongside structural and geotechnical verifications.

17.1 Uplift of Basements and Rafts

17.1.1 Purpose

Basement slabs, rafts, and buried structures must resist **hydrostatic uplift** caused by groundwater acting on the underside of the structure.

If unchecked, uplift can result in:

- **Structural failure** (cracking, delamination, flotation),
- **Loss of serviceability** (differential heave), or
- **Water ingress** (loss of watertightness).

⚠️ EC7 §10.4 requires verification that the **design downward load** exceeds the **design uplift pressure** for all credible water level scenarios.

17.1.2 Design Requirement (Eurocode 7 Format)

At Ultimate Limit State (ULS):

$$V_d \geq U_d$$

At Serviceability Limit State (SLS):

$$\frac{V_k}{U_k} \geq 1.2 \text{ to } 1.3$$

where:

- V_d = design vertical resistance (self-weight + structure + surcharge),
- U_d = design uplift pressure (γ_F × γ_w × h),
- V_k, U_k = characteristic values.

Apply DA1 combinations (C1 and C2) per UK NA:

– **C1:** factor actions, keep material parameters unfactored;
– **C2:** factor resistances (γ_M = 1.0 for water, 1.25 for soil).

17.1.3 Components of Vertical Resistance

Source	Abbreviation	Description	Unit	Notes / Partial Factors
Self-weight of structure		γ_{conc} × thickness	kN/m2	γ_G = 1.35 (ULS)
Overburden soil		γ' × depth	kN/m2	γ_G = 1.35 (ULS)
Structural load		Column/wall reaction on slab	…	γ_Q = 1.5 (ULS)
Frictional skin (if embedded)		Mobilised side resistance	…	Often neglected in conservative design
Anchor or pile tension		Tie-down resistance	…	γ_R = 1.3–1.5 (ULS)

At least **two independent resisting mechanisms** (e.g. self-weight + anchors) should be provided for permanent works.

17.1.4 Calculation of Uplift Pressure

$$U = \gamma_w \times h$$

where

- $\gamma_w = 9.81$ kN/m^3,
- h= vertical head difference between water table and underside of structure.

For sloping phreatic surfaces or partial submergence, integrate pressure distribution linearly.

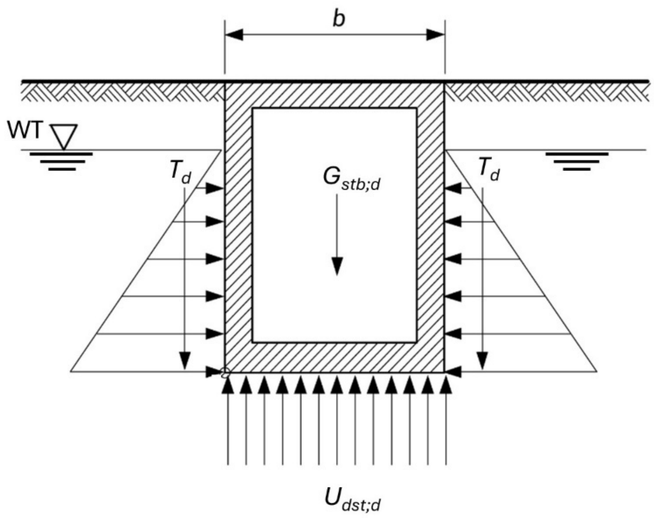


Figure 65: Uplift of a buried hollow structure (reproduced from §10.2 BS EN 1997-1:2004)

17.1.5 Example Verification

Parameter	Symbol	Value
Groundwater head above base	h	5.5 m
Water unit weight	γ_w	9.81 kN/m^3
Uplift pressure	U	54.0 kPa
Downward load (structure + overburden)	V	72.0 kPa
FOS (SLS)	V/U	1.33 ✓ OK

✓ Acceptable uplift safety margin at SLS (≥1.2).
If FOS < 1.2, consider:

- Thicker raft or ballast,
- Permanent tie-down anchors,
- Controlled drainage or under-drain.

17.1.6 Tie-Down Anchors and Tension Piles

Where uplift exceeds self-weight, **tension piles** or **ground anchors** are used to transfer load to deeper strata.

-
- Design Load per Anchor

$$T_d = \frac{U_d - V_d}{n}$$

where *n* = number of anchors.

- Verification (EN 1537, EN 1997-1 §9.7):
- Bond length ≥ 3 × pile diameter (granular) or 2 × (cohesive).
- Design resistance $R_{b,d} = \pi D L \alpha c_u / \gamma_M$.
- γ_M = 1.25 (DA1/C2).

Factor of safety ≥ 2.0 (temporary) or 2.5 (permanent). The expression for Rb_d is provided for cohesive soils, but the corresponding equation for granular soils should also be included for completeness. Furthermore, the use of a Factor of Safety (FoS) reflects an Allowable Stress Design (ASD) approach.

Under Eurocode 7, FoS is not used; instead, design is based on partial factors applied to actions, material properties, and resistances. **Any FoS-based checks in the document should therefore be reviewed and, where appropriate, reframed within the Eurocode 7 partial-factor design logic.**

Anchors should include double corrosion protection and proof testing to 1.25 × design load.

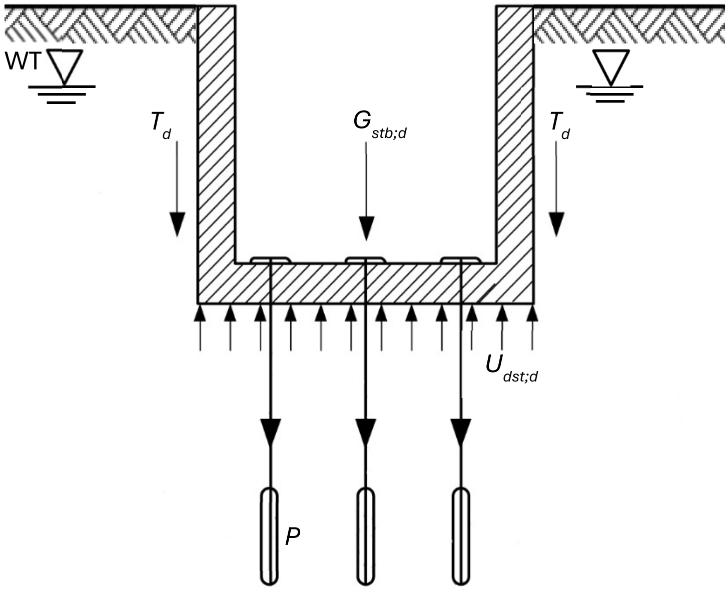


Figure 66: Tie down anchors to resist uplift (reproduced from §10.2 BS EN 1997-1:2004)

17.1.7 Structural Considerations

Basement slabs must also satisfy **structural STR** checks:

- Flexural strength under combined hydrostatic + self-weight load.
- Crack-width control $w_k \leq 0.2$ mmfor watertightness (see §16.3).
- Shear punching checks around pile heads or walls.

Use finite-element modelling or simplified plate bending (Winkler foundation) for large rafts.

17.1.8 Design Life and Monitoring

System Type	Design Life	Monitoring / Inspection
Reinforced concrete raft	50–100 years	Annual inspection of joints, drains
Permanent anchors	≥ 50 years	Load testing + corrosion protection
Temporary dewatering	< 2 years	Continuous head monitoring

Install piezometers beneath slab to confirm pore pressure reduction and monitor uplift risk during and post-construction.

17.1.9 Common Failure Modes

Symptom	Likely Cause
Slab heave / uplift	Underestimated water table or rapid rebound post-pumping

Crack along wall junction	Differential uplift / restraint incompatibility
Leakage through joint	Waterstop displacement or poor compaction
Anchor creep / loss of tension	Grout failure or corrosion

Phicon Note:

• Always assess *highest credible groundwater level* (e.g., +1.0 m above recorded) for uplift design.
• Use combined safety from **structural weight + hydro-relief**; never rely on dewatering alone for permanent stability.
• Specify anchor bond length, test load, and corrosion protection explicitly in GDR appendices.
• Include uplift pressure plots and V/U summary table for each design case (temporary and permanent).
• Confirm uplift safety for both *unfactored* (SLS) and *factored* (ULS) cases — EC7 reviewers often request both side by side.
• For permanent basements, design slab as *composite waterproofing element* — watertight concrete + secondary drainage = robust UPL resistance.

17.2 Hydraulic Heave at Excavations

Hydraulic heave is a **geotechnical instability** occurring when upward seepage beneath an excavation base causes the **effective stress** within the soil to reduce towards zero.

It is particularly critical in **deep cuts below the groundwater table**, such as shafts, cofferdams, or basements with open excavations.

Eurocode 7 classifies heave verification under the **Hydraulic (HYD) limit state**, alongside uplift (UPL), requiring checks against **loss of stability due to seepage flow**.

17.2.1 Mechanism of Hydraulic Heave

When water flows upward beneath an excavation (or through a confined layer), seepage forces oppose the weight of the soil.

If the **upward hydraulic gradient** $i = \frac{\Delta h}{L}$exceeds the **critical gradient** $i_{crit} = \frac{\gamma'}{\gamma_w}$, the soil may lose its effective stress and behave as a fluid — leading to **boiling** or **heaving** of the base.

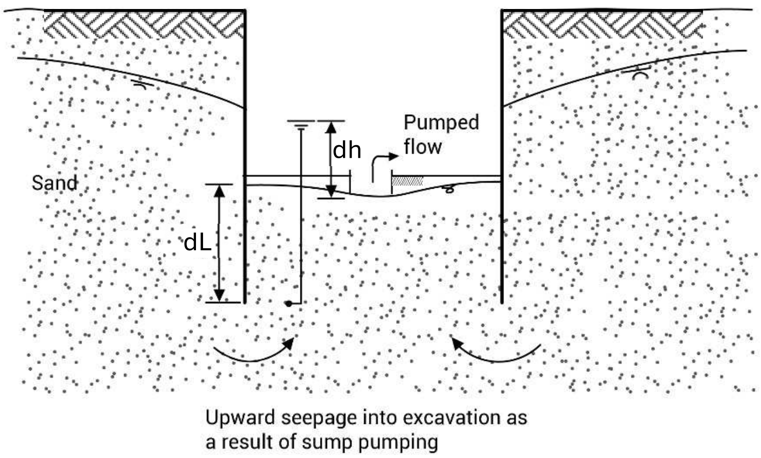


Figure 67: HYD Hydraulic gradient dimensions for base stability calculation

The failure is not structural but **geotechnical**, typically involving sand "boil" or sudden upward displacement of clay.

17.2.2 Design Verification Requirement (EC7 §10.4.3)

The hydraulic heave check must demonstrate that:

$$i \leq \frac{i_{crit}}{\gamma_F}$$

where:

- $i = \Delta h/L$= actual hydraulic gradient,
- $i_{crit} = \gamma'/\gamma_w$,
- $\gamma_F = 1.35$(unfavourable action, UK NA).

Alternatively, define a **Factor of Safety**:

$$FOS = \frac{i_{crit}}{i} \geq 1.0 \text{ (ULS)}, \geq 1.25 \text{ (SLS)}$$

17.2.3 Typical Parameters

Soil Type	γ (kN/m³)	γ' (kN/m³)	$i_{(crit)}$ = γ'/γ_w	Susceptibility
Loose sand	18	8	0.82	High
Medium dense sand	19	9	0.92	Moderate
Dense sand / gravel	20	10	1.02	Low
Silt / very fine sand	17	7	0.71	Very high
Clay	—	—	Not applicable (undrained)	—

Where multiple strata exist, the lowest-strength (highest permeability) layer beneath the base governs.

17.2.4 Simplified Hand Calculation

Let:
Δh = difference between external and internal water levels (m),
L = vertical flow path beneath wall toe (m).

$$i = \frac{\Delta h}{L}$$

Example:

- Δh = 3.0 m
- L = 2.5 m
- γ' = 9.0 kN/m³ → $i_{(crit)}$ = 0.92

- i = 3.0 / 2.5 = 1.20 → FOS = 0.92 / 1.20 = 0.77 ✗ Unsafe

Mitigation:

- Increase embedment (L) to 4.0 m → i = 0.75 → FOS = 1.23 ✓ Safe.

17.2.5 Flow Net and Seepage Modelling

Hydraulic heave is best assessed using **flow nets** or numerical seepage models (e.g. SEEP/W, PLAXIS):

- Determine equipotential lines and flow channels beneath excavation.
- Compute Δh across each equipotential drop.
- Calculate vertical head loss per flow line and resulting i-values.
- Identify maximum exit gradient near base.

For circular shafts or complex geometries, 2D axisymmetric models provide more accurate pressure distributions than planar approximations.

17.2.6 Heave vs. Piping vs. Uplift

Failure Type	Driving Mechanism	Typical Material	Key Check
Hydraulic Heave	Upward seepage → reduced effective stress	Sands / silts	$i < i_{crit}/\gamma_F$
Piping / Erosion	Particle removal along flow path	Fine sands	$i_{exit} < 0.5\, i_{crit}$
Uplift	Buoyancy on structure	Clays / slabs	$V_d \geq U_d$

(See §17.3 for piping criteria and §17.1 for uplift checks.)

17.2.7 Mitigation Measures

Measure	Principle	Effect
Deeper wall embedment	Increases flow path length (L)	Reduces hydraulic gradient
Cut-off wall / grout curtain	Reduces permeability or flow	Reduces Δh
Relief wells / drainage blanket	Relieves pore pressure	Reduces Δh at base
Heavier base slab	Adds stabilising weight	Resists uplift / heave
Underpinning berms	Increases passive resistance	Adds shear resistance to upward flow

⚠️ Always check *both* temporary and long-term conditions — temporary drawdown or re-pressurisation after pumping can trigger heave even in apparently stable soils.

17.2.8 Construction Considerations

- Install **standpipes or piezometers** to monitor groundwater levels during excavation.
- Avoid sudden **dewatering cut-off**; re-pressurisation can cause delayed heave.
- Maintain **constant head differential** where pumping is required.
- For excavations >6 m deep, always confirm **base factor of safety ≥ 1.25** before advancing.

17.2.9 Eurocode 7 Summary

Verification	Equation / Criterion	Partial Factors (UK NA)	Target Safety
Hydraulic heave (HYD)	$i \leq i_{crit}/\gamma_F$	$\gamma_F = 1.35$	≥1.0 (ULS), ≥1.25 (SLS)
Combined uplift / heave	$V_d \geq U_d + i < i_{crit}$	$\gamma_F = 1.35$	Both satisfied
Drained base resistance	$\gamma'_{eff} > 0$(at base)	—	Positive effective stress

Phicon Note:

• Always verify **heave** using *worst-case head differential* (max water outside, drained excavation inside).
• Include **flow net sketch or seepage contour plot** in GDR — CAT III reviewers expect visual verification.
• Combine **HYD** and **UPL** checks on one sheet; they are interdependent limit states.
• Consider **rapid drawdown** scenarios in temporary works — particularly where external head reduction can reverse seepage direction.
• For marine cofferdams, always include **tidal head fluctuation** in heave sensitivity analysis.
• Log **pore pressure recovery** post-pumping to ensure base stability before slab casting.

17.3 Piping Criteria and Safety Ratios

Piping refers to the **progressive removal of fine soil particles** under upward or horizontal seepage flow.

It can initiate at small exits (toe of an excavation, drain outlet, or cut-off joint) and develop into a **continuous flow channel**, leading to **loss of confinement**, **base instability**, or **catastrophic failure**.

Eurocode 7 (EN 1997-1 §10.4.3) requires designers to verify the **piping safety ratio** as part of the **Hydraulic (HYD)** limit state wherever seepage gradients exist.

17.3.1 Mechanism of Piping

When water emerges from a permeable layer under pressure, the **exit gradient** at the downstream face can exceed the **critical hydraulic gradient** $i_{crit} = \gamma'/\gamma_w$.

Once upward drag exceeds particle weight, erosion begins, forming “boils” or sand vents.

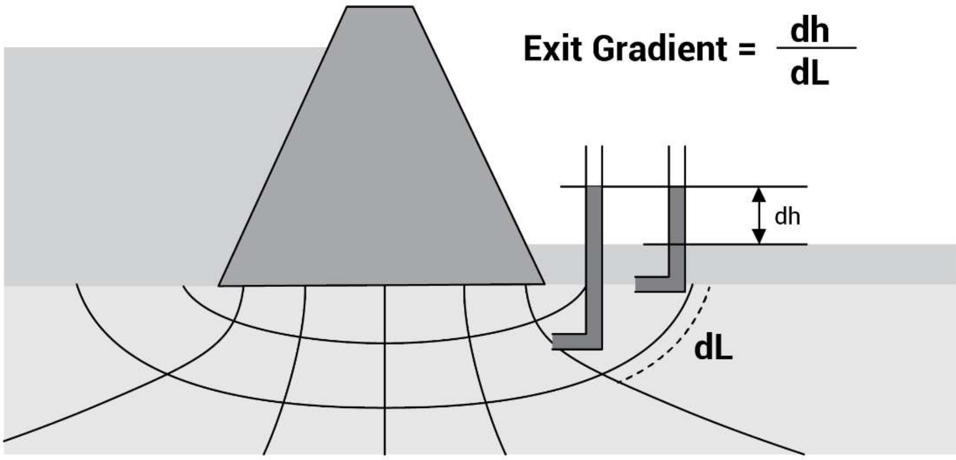


Figure 68: Piping failure beneath a hydraulic structure

Piping is most common in **fine sands, silty sands**, and **heterogeneous fills** where no graded filter or cut-off barrier is present.

17.3.2 Design Verification Requirement

The safety against piping is verified using the **exit gradient criterion**:

$$i_{exit} \leq \frac{i_{crit}}{\gamma_F}$$

where

- i_{exit}: maximum gradient at point of exit (typically at wall toe or downstream end),
- $i_{crit} = \gamma' / \gamma_w$: critical gradient,
- $\gamma_F = 1.35$: partial factor (unfavourable water action, UK NA).

Alternatively expressed as a **Factor of Safety**:

$$FOS_{piping} = \frac{i_{crit}}{i_{exit}} \geq 1.0 \text{ (ULS)or } \geq 1.5 \text{ (SLS)}$$

17.3.3 Typical Ranges of Critical Gradient

Soil Type	**γ (kN/m³)**	**γ' (kN/m³)**	**$i_{(crit)}$ = γ'/γ_w**	**Piping Risk**
Fine sand	17–18	7–8	0.7–0.8	Very high
Silty sand	17	7	0.7	High
Medium sand	18–19	9	0.9	Moderate
Dense sand / gravel	20	10–11	1.0–1.1	Low
Clay / silt (low k)	—	—	—	Negligible (impermeable)

The most critical location is normally **at the downstream toe of the wall** or **beneath a cut-off termination** where head loss is concentrated.

17.3.4 Determining the Exit Gradient

- Simplified Formula (Rectangular Flow Path):

$$i_{exit} = \frac{\Delta h}{N_d \, \Delta l}$$

where

- Δh = total head loss (m),
- N_d = number of potential drops (from flow net),
- Δl = distance between equipotential lines near exit.

In practice, i_{exit} can be estimated as:

$$i_{exit} = \frac{\Delta h_{local}}{L_{path}}$$

with L_{path} being the seepage length beneath the cut-off wall toe or under-drain.

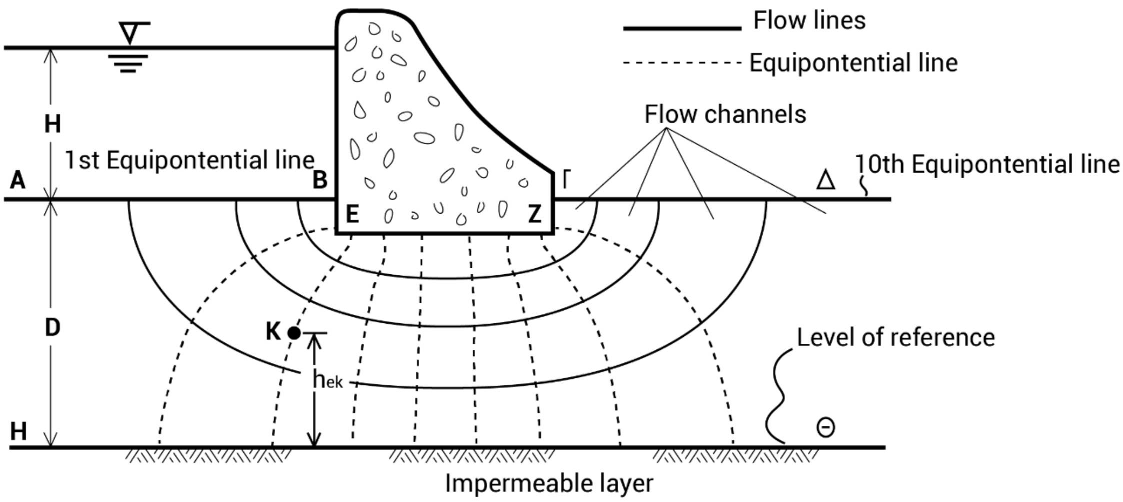


Figure 69: Flow net showing equipotential drops and exit gradient zone

17.3.5 Example Calculation

A diaphragm wall retains 6 m of water; seepage path under toe ≈ 3 m.

$$i_{exit} = \frac{\Delta h}{L} = \frac{6}{3} = 2.0$$

For medium dense sand $\gamma' = 9.5$, $i_{crit} = 0.97$.

$$FOS = \frac{0.97}{2.0} = 0.49 \Rightarrow \text{Unsafe.}$$

Mitigation: extend wall embedment to L = 7 m → $i_{exit} = 0.86, FOS = 1.13$✓ Acceptable.

17.3.6 Filter Design and Erosion Control

Where piping risk exists, filters or drainage layers must prevent migration of fines while allowing seepage.

- Terzaghi Filter Criteria

$$\frac{D_{15(filter)}}{D_{85(base)}} \leq 4 \text{and} \frac{D_{15(filter)}}{D_{15(base)}} \geq 4$$

Filter Type	Material	Purpose
Graded granular filter	Clean sand / fine gravel	Retains base fines, dissipates head
Geotextile filter (EN 13252)	Non-woven or woven fabric	Alternative to sand filter in confined space
Drainage blanket	≥ 300 mm thick gravel layer	Equalises pressure beneath base
Relief wells	Vertical wells through base	Reduce Δh and exit gradient

Filters must satisfy both **hydraulic compatibility** and **geotechnical stability** — avoid segregation or clogging by organic material.

17.3.7 Cut-off and Flow Path Modification

Increasing seepage length is an effective way to reduce the exit gradient.

Method	Function	Typical Reduction in i_{exit}
Deeper wall embedment	Extends flow path	30–50 %
Grout curtain	Seals pervious zones	40–70 %
Clay blanket	Adds low-permeability layer	20–40 %
Relief drain / well	Lowers downstream head	40–80 %
Toe berm	Adds weight and increases confining stress	10–20 %

Always re-evaluate uplift and heave simultaneously when modifying seepage paths.

17.3.8 Combined Heave–Piping Stability Ratio

For layered ground, both upward gradient and particle stability must be satisfied:

$$FOS_{combined} = \min\left(\frac{i_{crit,base}}{i_{up}}, \frac{i_{crit,exit}}{i_{exit}}\right) \geq 1.0$$

If either condition fails, **hydraulic instability** may initiate — treat both as governing for the **HYD limit state**.

17.3.9 Observational Control

During excavation or dewatering:

- Observe for sand boils, turbid discharge, or pulsing groundwater.
- Measure pore pressures and flow rate at drains / sumps.
- Immediately suspend lowering operations if sudden turbidity appears (indicating particle migration).
- Install **pressure relief wells** to reduce local Δh.

Continuous piezometric monitoring near wall toes is a Category 2 requirement for basements > 6 m deep under UK practice (CIRIA C580, C750).

17.3.10 17.3.10 Eurocode 7 Summary

Check Type	Criterion	Partial Factor (UK NA)	Typical Target FOS
Piping / erosion	$i_{exit} \leq i_{crit}/\gamma_F$	$\gamma_F = 1.35$	≥ 1.0 (ULS)
Hydraulic heave	$i \leq i_{crit}/\gamma_F$	$\gamma_F = 1.35$	≥ 1.0 (ULS), ≥ 1.25 (SLS)
Uplift	$V_d \geq U_d$	—	≥ 1.2 (SLS)

Phicon Note:

• Always check **piping risk** whenever groundwater is lowered through permeable strata or where drainage exits are concentrated (toe, sump, culvert).
• For long-term basements, specify a **graded filter or geotextile liner** under the raft — this is often overlooked in EC7 submissions.
• When using seepage models, report **maximum exit gradient contours**, not only uplift pressure.
• Combine **HYD + Piping + UPL** checks in one sheet for traceability; reviewers expect consistent head assumptions.
• Add a **relief drain sketch or section** in GDR drawings where gradient reduction forms part of the design intent.
• Field control: monitor **discharge clarity** and **piezometric head** daily during active dewatering — practical confirmation of hydraulic stability.

17.4 Relief Drains and Cut-off Walls

Relief drains and cut-off walls are **hydraulic control measures** used to manage pore water pressures and seepage flow beneath foundations or excavations.
Their purpose is twofold:

1. **To reduce uplift or exit head** by providing controlled drainage paths (relief drains);
2. **To limit seepage flow and gradient** by lengthening the flow path or reducing permeability (cut-off walls).

Eurocode 7 recognises both as acceptable measures for **HYD and UPL verifications**, provided the designer demonstrates long-term hydraulic stability and durability.

17.4.1 Function and Design Objectives

Measure	Primary Function	Design Objective
Relief drain	Dissipate pore pressure under slabs or behind walls	Reduce Δh and uplift pressure
Cut-off wall	Interrupt or deflect seepage flow	Increase seepage length (L) and reduce i
Gravel blanket / drainage layer	Equalise local pore pressure	Prevent local uplift or boiling
Relief wells	Vertically drain trapped water	Maintain safe hydraulic gradient

The design must balance **hydraulic control** and **construction feasibility** — excessive drainage may compromise waterproofing or lead to settlement.

17.4.2 Relief Drains Beneath Rafts and Basements

Relief drains are installed below rafts or slabs to **relieve hydrostatic pressure** by providing a free-draining layer connected to sumps or external discharge points.

3. Typical Arrangement

- 150–300 mm thick **clean granular layer** (e.g. 10–20 mm gravel),
- Wrapped in **geotextile filter (EN 13252)** to prevent clogging,
- Connected to perimeter collector drains and sump pumps,
- Non-return valves to prevent backflow during pump shutdown.

17.4.3 Design Checks

Check	Requirement
Hydraulic capacity	$Q = kiA \geq Q_{inflow}$
Filter stability	Terzaghi filter criteria satisfied
Pump reliability	Redundancy (2 × duty/standby)
Discharge	Legal consent & pollution prevention (EA/SEPA)

Where waterproofing relies on the slab (Type B), drainage must be secondary — the structure should remain stable if drains fail.

17.4.4 Relief Wells

Vertical wells (Ø150–300 mm) penetrating low-permeability strata allow pore water to escape into overlying drains. Used for temporary or permanent groundwater control beneath deep basements, shafts, or underpasses.

17.4.5 Design Considerations

- Screened length set below potential heave zone.
- Head loss through well governed by **Darcy flow** and well efficiency.
- Spacing determined by desired Δh reduction:

$$s = \sqrt{\frac{Q}{\pi k h}}$$

- Wells to be valved or sealed post-construction if not permanent.

Relief wells must be maintainable and accessible for cleaning; silting and biofouling are common long-term issues.

17.4.6 Cut-off Walls

Cut-off walls act as **impermeable barriers** to reduce flow and lower the hydraulic gradient beneath or around a structure.

They can be **temporary (steel sheet pile)** or **permanent (diaphragm, secant, jet-grouted wall)**.

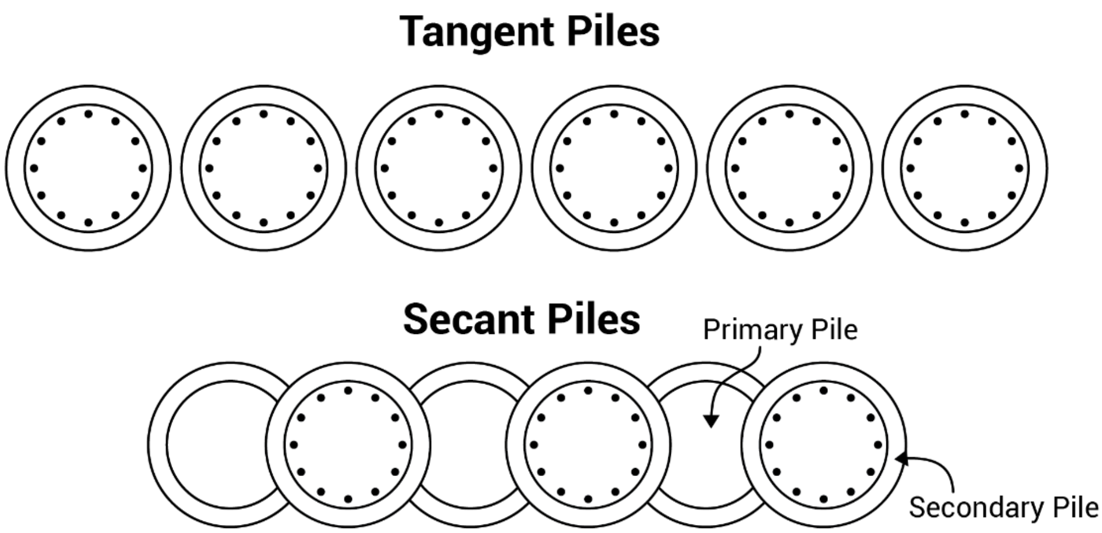


Figure 70: Difference between tangent and secant piles

Type	Material / Construction	Typical k (m/s)	Design Life
Steel sheet pile	Interlocked steel sections	$10^{-5} – 10^{-6}$	50+ years
Plastic concrete wall	Bentonite–cement mix	$10^{-7} – 10^{-8}$	100 years
Jet-grouted column barrier	Jet-grout soil mix	$10^{-6} – 10^{-8}$	50–75 years
Bentonite slurry trench	Low-permeability backfill	$10^{-7} – 10^{-8}$	50 years
Secant pile wall	Intersecting concrete piles	$10^{-7} – 10^{-9}$	100 years

17.4.7 Design Approach

- Embed below pervious layer to form full cut-off.
- Verify reduction in head loss Δh via flow net or seepage model.
- Apply partial factor $\gamma_F = 1.35$ to residual head for ULS.
- Consider permeability degradation or cracking in long-term performance.

17.4.8 Combined Systems

Often, relief drains and cut-off walls are combined for robust control:

Combination	Function	Application
Cut-off + drain	Reduces inflow and allows pressure relief	Deep basements, shafts
Cut-off + pumping	Temporary dewatering	Cofferdams
Drain + tie-down anchors	Balances uplift with drainage	Permanent rafts
Cut-off + clay blanket	Passive control (no pumps)	Dams, lagoons

Combined systems are recommended for **Type C waterproofing** (drained cavity systems) where continuous head reduction is part of the design intent.

17.4.9 Permeability Requirements and QA Testing

Test	Purpose	Typical Acceptance Criteria
In-situ falling head / Lefranc test	Verify permeability of cut-off trench	$k \le 1\times10^{-7}$ m/s
Laboratory permeability test	Confirm mix design of plastic concrete	$k \le 1\times10^{-8}$ m/s

Pump test / drawdown trial	Confirm system performance	Head reduction ≥ design Δh
Drainage inspection (CCTV)	Confirm flow and filter integrity	No blockage or sedimentation

⚠️ Always record and retain QA results within the Geotechnical Design Report (GDR) for traceability under EC7 §4.1(6)P.

17.4.10 Maintenance and Monitoring

- Install **piezometers** to monitor uplift pressure beneath slabs and adjacent to cut-offs.
- Record **flow rate and head** monthly (permanent) or daily (temporary).
- Design systems for accessible flushing points.
- Plan **periodic jetting or replacement** for drainage media every 5–10 years in aggressive conditions.
- Where discharge is pumped, include **high-level alarms and backup power**.

EC7 requires the designer to demonstrate **durability, inspectability, and maintenance strategy** for all groundwater control measures (see §2.4.7.3 of EN 1997-1).

17.4.11 Design Documentation

Each hydraulic control system should include:

- Flow net or seepage model output showing head reduction.
- Drainage and cut-off layout drawings.
- Summary of assumed permeability and boundary conditions.
- Long-term inspection and maintenance schedule.
- Confirmation of discharge consents (EA/SEPA).

Include a Hydraulic Control Verification Table in the GDR:

Component	k (m/s)	Δh (m)	$i_{(max)}$	FOS (HYD)	Status
Cut-off wall	1×10^{-8}	3.0	0.3	1.6	✓ OK
Under-raft drain	5×10^{-3}	1.5	0.4	1.8	✓ OK
Relief wells	—	2.5	0.5	1.4	✓ OK

Phicon Note:

• Use relief drainage only as **secondary defence** — the slab or structure must remain stable if the drain fails.
• Always demonstrate the **system's hydraulic effect** (Δh reduction) via calculation, not by assumption.
• Specify **filter gradings and geotextile class** in construction drawings to ensure compatibility.
• For permanent cut-offs, verify **permeability testing and QA certificates** before backfilling.
• Include **inspection and maintenance notes** in the handover documentation — often neglected but critical for long-term performance.
• Remember: **cut-off walls reduce flow; drains reduce pressure** — both may be required for EC7 compliance.

17.5 Worked Example – Basal Heave Safety Check beneath an Excavation

(Hydraulic Limit State – HYD; Eurocode 7 DA1, UK NA)

17.5.1 Problem Statement

Design a temporary excavation supported by a diaphragm wall in water-bearing sand. Verify **hydraulic heave** safety at the base using both a **simplified gradient method** and a **flow-net approximation**, and demonstrate mitigation to achieve EC7 compliance.

17.5.2 Geometry & Groundwater

- Excavation depth (to formation): **H = 7.0 m** below ground level (bgl)
- Diaphragm wall toe: **D = 3.5 m** below formation (embedment)
- External groundwater: **at ground level** (outside the wall)
- Internal groundwater: **at formation level** (open excavation; no internal drawdown)
- Total head difference across base: **Δh = 7.0 m**

17.5.3 Soil Data (governing stratum beneath base)

- Soil: medium dense sand
- Total unit weight: $\gamma = 19$ kN/m^3
- Submerged unit weight: $\gamma' = 9.5$ kN/m^3
- Water unit weight: $\gamma_w = 9.81$ kN/m^3
- Critical hydraulic gradient:

$$i_{crit} = \frac{\gamma'}{\gamma_w} = \frac{9.5}{9.81} = 0.97$$

Eurocode factors (HYD): Unfavourable water action **γ_F = 1.35** → ULS criterion $i \leq i_{crit}/\gamma_F = 0.97/1.35 = 0.72$.

17.5.3.1 A) Simplified Gradient Method

Take the **vertical seepage path length** below the wall toe as the embedment depth beneath formation (**L ≈ D = 3.5 m**).

$$i = \frac{\Delta h}{L} = \frac{7.0}{3.5} = 2.00$$

- ULS requirement: $i \leq 0.72$→ 2.00 > 0.72 ⇒ FAIL
- **Characteristic FOS (informative):** $FOS = \frac{i_{crit}}{i} = \frac{0.97}{2.00} = 0.49$⇒ **Unsafe**

Interpretation: With this toe depth, upward seepage can reduce effective stress to near zero at the base → high risk of basal heave/boiling.

17.5.3.2 B) Flow-Net Approximation (planar section)

Construct a quick flow net under the toe (concept check):

- Adopt **N_d = 7** potential drops (coarse but reasonable)
- Near-toe **exit element length** $\Delta l \approx 0.60$ m(tightest spacing)

The local exit gradient:

$$i_{exit} = \frac{\Delta h / N_d}{\Delta l} = \frac{7.0/7}{0.60} = \frac{1.0}{0.60} = 1.67$$

- ULS requirement: $i_{exit} \leq 0.72$→ 1.67 > 0.72 ⇒ FAIL
- Piping FOS (informative): $\frac{0.97}{1.67} = 0.58$⇒ Unsafe

Interpretation: Flow concentration at the toe yields even **higher** local gradients than the simplified check; mitigation is required.

17.5.3.3 C) Mitigation Options & Re-check

We trial three practical modifications and re-verify:

- Option 1 – Increase embedment to D = 7.0 m
- **L = 7.0 m**, $i = 7.0/7.0 = 1.00$
- ULS limit: 0.72 → 1.00 > 0.72 ⇒ FAIL
- **FOS (char.):** $0.97/1.00 = 0.97$→ marginal on characteristic, still **fails at ULS**

Deeper toe alone is **not sufficient** for ULS compliance in this head condition.

17.5.3.4 Option 2 – Deeper toe (D = 7.0 m) + under-base relief drainage

Assume under-raft/gravel blanket + sump lowers **Δh to 4.0 m** (demonstrable by pump test or seepage model).

- **L = 7.0 m**, $i = 4.0/7.0 = 0.57$
- ULS limit: 0.72 → 0.57 ≤ 0.72 ⇒ PASS
- FOS (char.): $0.97/0.57 = 1.70$→ Comfortable

17.5.3.5 Option 3 – Original toe (D = 3.5 m) + plastic concrete cut-off (k ↓) + relief wells

Assume combined measures reduce effective head at the base to **Δh = 3.0 m** and lengthen path to **L_{eff} ≈ 5.0 m** (verified by flow model).

- $i = 3.0/5.0 = 0.60$
- ULS limit: 0.72 → 0.60 ≤ 0.72 ⇒ PASS
- FOS (char.): $0.97/0.60 = 1.62$→ Acceptable

17.5.3.6 Summary Table (HYD Verification)

Case	Δh (m)	L (m)	i = Δh/L	ULS limit i ≤ 0.72	Char. FOS = $i_{(crit)}/i$	Status
Base design (D=3.5 m)	7.0	3.5	2.00	Fail	0.49	χ
Option 1 (D=7.0 m)	7.0	7.0	1.00	Fail	0.97	χ
Option 2 (D=7.0 m + drain)	4.0	7.0	0.57	Pass	1.70	✓
Option 3 (cut-off + wells)	3.0	5.0	0.60	Pass	1.62	✓

(Flow-net exit gradients at the toe were also checked qualitatively: Options 2 & 3 show exit gradients below ULS limit when re-drawn with reduced head.)

17.5.3.7 D) Additional Checks (Good Practice)

- **UPL (17.1):** If a slab or blinding is present, verify $V_d \geq U_d$ at both temporary and permanent stages.
- **Piping (17.3):** Confirm **exit gradient** at the downstream toe i_{exit} satisfies ULS and SLS; provide **filter/geotextile** where drains discharge.
- **Observational method:** Install **piezometers** inside/outside to confirm Δh assumptions; define **trigger–action** levels.

17.5.3.8 E) Recommended Design (for this example)

- Adopt **Option 2**: Increase embedment to **D = 7.0 m** and install an **under-base drainage blanket** (≥150–300 mm gravel, wrapped in EN 13252 geotextile) connected to a **sump with duty/standby pumps**.
- Document the **assumed head reduction** (Δh = 4 m) and confirm by **commissioning pump test** prior to full-depth excavation.
- Provide **filter detailing** at outlets; specify **maintenance access** and alarms.
- Re-run a **2D seepage model** to record maximum **i** and $\mathbf{i_{exit}}$ contours for the GDR.

Phicon Note:

• In UK practice, **HYD ULS** often governs deep urban excavations — do not rely on toe depth alone; combine **cut-offs** and **relief drainage**.
• Always show both **hand calc (Δh/L)** and a **flow-net/FE seepage** check; reviewers expect corroboration.
• Treat Δh conservatively (e.g., **highest credible external head**) and verify that pumping/relief systems can **sustain** the required head reduction.
• Couple HYD with **UPL** and **piping** checks on the same sheet using a **common head diagram** — it avoids inconsistent assumptions.
• Install **piezometers** early; make head the leading indicator in the observational plan (alert/action thresholds).

18 Earthworks & Slope Stability

18.1 Cuttings, Embankments and Failures (Rotational, Planar)

This section sets out the **failure mechanisms**, **quick-check formulae**, and **Eurocode 7 context** for slopes in **cut** (removal) and **fill/embankment** (placement). Use it to scope analyses before detailed limit-equilibrium (LE) or FE modelling.

18.1.1 Purpose & EC7 Context

EN 1997-1 §11: Slopes must be verified at **ULS (GEO)** using characteristic ground parameters and appropriate partial factors (DA1/C2), and checked at **SLS** for deformations/erosion durability.

Consider **construction stage** (short-term undrained vs long-term drained), **groundwater**, **surcharge**, and **seismic/pseudo-static** where relevant.

18.1.2 Slope Types & Typical Scenarios

Slope type	Typical geometry	Governing behaviour	Notes
Cutting (into natural ground)	Height H = 3–20 m; face angle β	Often **rotational** in clays; **planar/wedge** in granular or bedded strata	Transient pore-pressures critical in overconsolidated clays
Embankment / Fill	Side slopes 1V:2H–1V:3H	Settlement & **global stability**; foundation shear	Construction control (lift thickness, compaction, drainage) governs performance
Engineered bunds	Low (<4 m)	Erosion + local slips	Provide surface drainage and erosion protection

18.1.3 Failure Mechanisms (taxonomy)

1. **Rotational (circular) failure** – common in homogeneous clays and some fills.
    1. **Face**
    2. **Toe**
    3. **Base**
2. **Non-circular / compound** – passes through weak seams or interfaces.
3. **Planar (infinite slope)** – translational along a plane parallel to slope (granular/stratified soils).
4. **Wedge** – two-plane intersection (often in rock or stiff fissured clays).
5. **Base failure** – embankment on soft foundation squeezes and heaves.

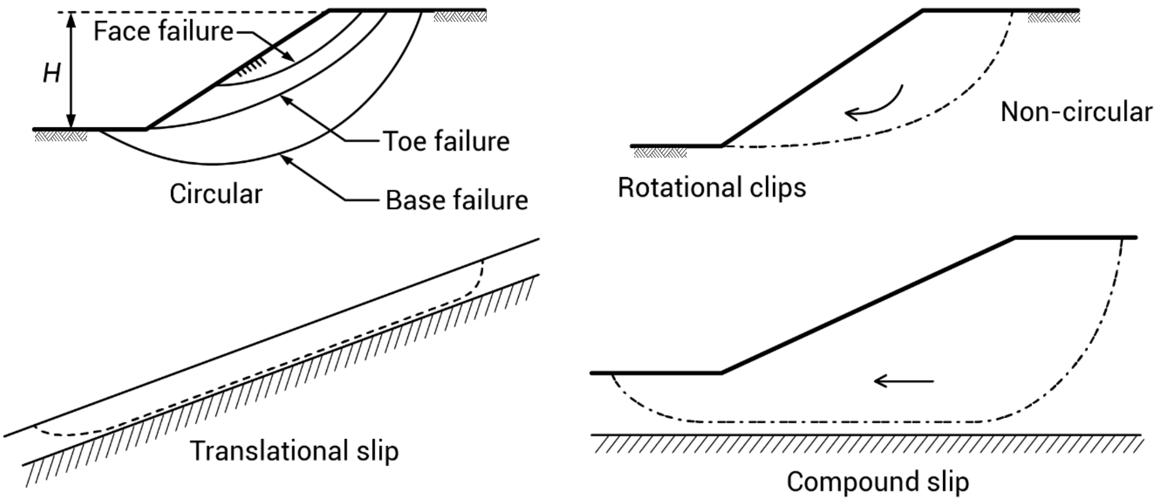


Figure 71: Typical failure mechanisms of slopes

18.1.4 Quick Checks – Factors of Safety (hand methods)

- (A) Circular (moment) – undrained short-term ($\phi_u = 0$)

For a trial circle (Bishop simplified, concept):

$$F \approx \frac{\sum c_u\, b}{\sum W \sin\ \alpha}$$

where c_u: undrained shear; b: slice width; W: slice weight; α: base angle.

Use for cuts in clay during/shortly after excavation.

- (B) Planar / Infinite slope – drained (dry)

$$F = \frac{c'}{\gamma z \sin^2 \beta} + \frac{\tan \phi'}{\tan \beta}$$

z: depth to potential plane; β: slope angle; γ: unit weight.

- (C) Planar – seepage parallel to slope (steady state)

$$F = \frac{c'}{\gamma' z \sin^2 \beta} + \frac{\gamma'}{\gamma}\ \frac{\tan \phi'}{\tan \beta}$$

where $\gamma' = \gamma - \gamma_w$.

Use for embankments at full saturation / long wet periods.

- (D) Undrained embankment on soft foundation (circul. concept)

Approximate:

$$F \approx \frac{c_{u,found}\ N_c}{\gamma H}$$

with $N_c \sim 5 - 6$ for broad circles (concept), H= embankment height.

Confirm with LE software.

These are **screening** formulae. For design, use LE (Bishop, Janbu, Morgenstern–Price) with DA1/C2 parameters and water conditions consistent with the scenario.

18.1.5 Parameter Sets & Design Situations

Situation	Parameters	Water condition	Notes
Short-term cut in clay	c_u(undrained), γ	Undrained; K_0	Rapid works; risk of delayed softening
Long-term cut	$c', \phi', \gamma', u(z)$	Drained; steady GW	Check for fissure softening & weathering
New embankment on soft clay	c_u(foundation), γ	Undrained foundation response	Reinforcement/widening to increase F
Consolidated long-term	$c', \phi'$for fill & foundation	Drained	Include improved strength after consolidation
Seismic (pseudo-static)	$k_h W$horizontal action	Drained/undrained per duration	Use $k_h$per project spec; reduce F targets appropriately

18.1.6 Indicative Stable Slopes (concept/preliminary)

(Final design must be by analysis; values below are **rules-of-thumb**)

Material (characteristic)	Dry / well drained	Poorly drained / with seepage
Dense sand/gravel (ϕ' = 38–42°)	β ≈ 35–40° (≈1V:1.2–1.4H)	β ≈ 30–35° (≈1V:1.7–1.4H)
Medium sand (ϕ' = 32–35°)	β ≈ 30–34° (≈1V:1.7–1.5H)	β ≈ 26–30° (≈1V:2.0–1.7H)
Firm–stiff clay (c' ≈ 5–15 kPa, ϕ' ≈ 20–25°)	1V:2H–1V:3H	1V:3H–1V:4H
Soft clay / peat	Not self-supporting	Use berms/reinforcement; staged loading

18.1.7 Embankments on Soft Ground – stability levers

- **Wider toe / berms** → increases resisting moment and lengthens slip.
- **Lightweight fill** (foams, EPS) → reduces driving weight.
- **Basal reinforcement (geogrid/geo-textile)** → increases tensile restraint along slip.
- **Staged construction + PVDs/preload** → gains strength via consolidation (cv, t_{90}).
- **Shear keys / trench improvement** → intercepts slip in soft strata.

18.1.8 Drainage & Groundwater

- **Cut slopes:** intercept drains at crest; toe drains; avoid perched water.
- **Embankments:** chimney & blanket drains; maintain freeboard; assess transient drawdown.
- **Analysis:** model **u(z)** properly—steady vs transient; consider **rapid drawdown** where applicable.

18.1.9 Target Factors / Partial Factors (UK NA – guide)

- **ULS (DA1/C2):** apply $\gamma_M = 1.25$on $c', \tan\ \phi'$; actions per NA.
- Characteristic FOS (informative, un-factored):
  - Permanent slopes: ≥ **1.3–1.5**
  - Temporary works: ≥ **1.2–1.3**
- **SLS:** check deformations, erosion robustness, and drainage capacity.

State clearly in the GDR whether you report **partial-factor format** only, and if any **informative FOS** is provided for context.

18.1.10 Common Red Flags

- Ignoring **construction sequence** (short-term undrained in clay).
- Using **drained ϕ'** while modelling **undrained pore pressures** (inconsistent).
- Omitting **water table rise** scenarios / blocked drains.
- Not checking **global stability** when adding berms or toe walls.
- Assuming infinite slope for **finite-height** cut without verifying circular/compound slips.

18.1.11 Documentation (what reviewers expect)

1. Geometry, stratigraphy, **parameter table** (char → design).
2. Water conditions (sections with phreatic line(s)).
3. Method(s): LE type, software, slice assumptions, search grid.
4. Results: **critical surface plot**, minimum F (by case).
5. Sensitivity: ±10–20% on ϕ', c', γ', and water level.
6. Mitigation layout (drains, berms, reinforcement) and re-check.

Phicon Note:

• Start with **hand checks** (infinite slope / undrained circle) to size the problem, then confirm with LE/FE.
• Always separate **short-term** and **long-term** cases; many failures occur in the interim.
• Treat water honestly: show the phreatic line; test **seepage-parallel** case for sands and **pore-pressure build-up** in clays.
• For embankments on soft ground, put **basal reinforcement** and **staged preload** on the table early—often the most economical route to F ≥ 1.3–1.5.
• Include **erosion protection** (vegetation/mats/rip-rap) and **surface drainage** in drawings; stability without durability is incomplete.
• Record **trigger–action** monitoring (inclinometers, piezos) for high slopes or urban interfaces—align with Chapter 22.

18.2 Limit Equilibrium Methods (Bishop, Janbu, Morgenstern–Price)

This section summarises the principal **Limit Equilibrium (LE)** methods used to determine **slope stability** under Eurocode 7, compares their assumptions, and outlines practical workflow for consistent application in both **hand** and **software** analyses.

18.2.1 Purpose

LE methods compute the **Factor of Safety (FoS)** or **design verification ratio** by equating the **driving moments or forces** along a potential slip surface with available **shear strength** of the soil.

In Eurocode 7 terms:

- Use **design parameters** (factored c_d, ϕ_d) and **design actions** → verify $F_d \geq 1.0$ at ULS.
- Alternatively, in **informative (unfactored)** terms → check $FoS \geq 1.3–1.5$ for permanent slopes.

18.2.2 Method Families

Method	Type	Equilibrium enforced	Typical Use	Accuracy / Notes
Swedish / Ordinary	Circular	Moment only	Rapid screening	Crude (neglects interslice forces)
Bishop Simplified (1955)	Circular	Moment + vertical	Homogeneous clays / sands	Reliable; default for circular slips
Janbu Simplified (1954)	Non-circular	Force only	Layered soils, complex geometry	Good for non-circular but conservative
Spencer (1967)	General	Moment + force (constant ratio of interslice forces)	Complex geometries, accurate	Benchmark reference method
Morgenstern–Price (1965)	General	Moment + force (user-defined interslice function)	All soils, variable u(z), anisotropy	Most rigorous; industry standard
GLE (General Limit Equilibrium)	General	Moment + force	Software implementation of above	Equivalent to M–P or Spencer depending on assumptions

18.2.3 Core Equation (General LE Form)

For any slice along a slip surface:

$$\tau_{mobilised} = \frac{W\sin\,\alpha - U\sin\,\beta - X}{F}$$
$$\tau_{available} = c' + (\sigma' \tan\,\varphi')$$

Solve for **F** (global factor) such that **Σ moments or forces are in equilibrium** across all slices.

Where:

- W: slice weight,
- α: base inclination,
- U: pore pressure,
- X: interslice shear,
- F: overall safety factor (if using characteristic inputs).

18.2.4 In EC7 design format, the comparison is instead $E_d \leq R_d$using factored γ_F, γ_M.Input Requirements

Parameter	Typical Input	Notes
Geometry	Slope profile, boundaries	From section drawing
Stratigraphy	Layer thickness, c'–ϕ'	Use design values (DA1/C2)
Unit weights	γ (total), γ' (submerged)	Consistent with water table
Pore pressure	u(z), ru, or phreatic line	Include perched or artesian heads
Loading	Surcharge, seismic (k_h)	Include load cases separately
Reinforcement	Tensile capacity, depth	Modelled as line or distributed elements

⚠️ Maintain consistency between phreatic line and γ' selection — never mix drained and undrained assumptions.

18.2.5 Method Selection Guidance

Situation	Preferred Method	Rationale
Homogeneous clay cut	Bishop Simplified	Simple circular, undrained
Layered fill on soft base	Janbu or Morgenstern–Price	Non-circular, layered
Deep excavation / bermed slope	Spencer / M–P	High accuracy, interslice forces
Reinforced slope / geogrid	M–P / GLE	Can handle line elements
3D valley / complex	3D GLE or FE (PLAXIS, SLOPE3D)	Avoid planar simplification

In practice, **Morgenstern–Price (GLE)** is the reference for EC7 verification due to force + moment equilibrium and flexibility for complex pore pressures.

18.2.6 Numerical Workflow (PHICON Summary)

Step 1 — Define Geometry

- Import section from CAD or draw in software (e.g., Slope/W, Slide, LimitState:GEO).
- Delimit soil layers, interfaces, and water table(s).

Step 2 — Assign Parameters

- From **characteristic** dataset → convert to **design** per DA1/C2:
  $c_d = c'/1.25$; $\tan\ \varphi_d = \tan\ \varphi'/1.25$.
- Assign γ_d = 1.0 × characteristic unless otherwise required.

Step 3 — Define Slip Search

- Circular (Bishop) → define grid or auto-search between crest & toe.
- Non-circular (Janbu/M–P) → polygonal or multi-radius surfaces.
- Refine until F converges (<0.01 tolerance).

Step 4 — Water Pressure

- Define phreatic line; specify ru values if transient pore pressures (typically ru = 0.2–0.5 in OC clays).
- For seepage parallel to slope, use steady-state flow calculation.

Step 5 — Calculate

- Run for DA1/C2 → obtain **E_d/R_d ratio** or **FOS**.
- Re-run for sensitivity (±10% ϕ', ±10% c', ±1 m phreatic shift).

Step 6 — Output

- Record critical surface, slice output, and summary plot.
- Report both ULS design verification and informative FOS.

18.2.7 Presentation of Results (GDR Standard)

Case	Analysis Method	FOS (char)	ULS check	Status
ST (undrained)	Bishop Simplified	1.43	$E_d/R_d = 0.87$	✓ OK
LT (drained)	M–Price (GLE)	1.52	$E_d/R_d = 0.93$	✓ OK
Rapid drawdown	M–Price	1.12	$E_d/R_d = 1.05$	⚠ Near limit

Attach **critical slip plot** and **table of slices** in Appendix.
Ensure output shows **FOS trend vs water level** for traceability.

18.2.8 Advantages & Limitations (summary)

Method	Strengths	Limitations
Bishop	Quick, circular, simple inputs	Not valid for non-circular / layered

Janbu	Handles non-circular, complex	Conservative for curved slips
Spencer / M–P	Full equilibrium, robust	Slightly higher computation demand
GLE / Software	Flexible, any geometry	Must check convergence & mesh quality
FE (e.g. PLAXIS)	Can couple seepage & deformation	Requires experience; not direct FOS unless strength reduction used

LE methods remain standard for EC7 verification; FE is supplementary for deformation and strain compatibility checks.

18.2.9 Quality and Validation Checks

- Compare **two independent methods** (e.g., Bishop vs M–P).
- Verify slip passes through expected weak zone; inspect stress diagram.
- Check interslice force ratio distribution (M–P): should be smooth, not erratic.
- Ensure **slice width <0.5 m** for steep slopes to maintain accuracy.
- Validate FOS by simple hand calc or spreadsheet using mean c'–ϕ'.

Phicon Note:

• Always state which **LE method** you've used — and why — in the GDR. CAT III reviewers expect traceability.
• Pair **short-term Bishop (ϕ_u = 0)** with **long-term M–Price (c'–ϕ')** for most cuttings.
• For reinforced slopes, document the **geogrid pullout capacity** and model it explicitly rather than adding "lumped resistance."
• Include both **ULS ratio (E_d/R_d)** and **informative FOS**; it clarifies EC7 compliance vs conventional metrics.
• Remember: an elegant M–Price analysis is useless without correct pore pressures — spend time on **u(z)** definition first.
• Use **sensitivity plots (ϕ' ±5°, water ±1 m)** to demonstrate robustness — these charts strengthen the GDR QA.

18.3 Short-Term (Undrained) vs Long-Term (Drained) Analyses

The stability of slopes and embankments is strongly dependent on **pore pressure behaviour over time**. Eurocode 7 requires separate verification of **short-term (undrained)** and **long-term (drained)** conditions to ensure safety during **construction** and **service life**.

This section explains the distinction, how to select parameters, and how to check both cases in practice.

18.3.1 Basis and Timescale

Stage / Duration	Soil Condition	Analysis Type	Typical Case

During excavation / fill placement (hours–weeks)	No time for drainage	Undrained (ϕ_u = 0)	Cut in clay, early embankment
After consolidation / long-term service (months–years)	Excess pore pressures dissipated	Drained (c'–ϕ')	Permanent slope, cut after pore dissipation

The key is whether **drainage and consolidation** can occur within the timescale of loading.

For intermediate stages (partial drainage), numerical coupled consolidation or staged LE analyses may be required.

18.3.2 Parameter Selection

Parameter Type	Notation	Applicable To	Typical Source / Test
Undrained shear strength	c_u	Short-term clay	Triaxial UU / CU (test), field vane, CPT correlations
Effective strength parameters	c', φ'	Long-term	CD triaxial, ring shear, correlations
Pore pressure ratio	$r_u = u/(\gamma h)$	Rapid loading	Use if no direct u(z) available
Undrained modulus	E_u	Deformation checks (ST)	CU/UU triaxial, empirical (500 c_u)
Effective modulus	E'	Settlement / SLS	Oedometer / CPT / empirical correlations

Rule of thumb:
$c_u \approx 0.22\sigma_v'$for normally consolidated clay;
$c_u \approx$ 0.25–$0.35\sigma_v'$for lightly overconsolidated clay (OCR = 2–4).

18.3.3 Undrained (Short-Term) Analysis

Assume **no drainage**; pore pressures rise to resist volume change.

- Strength: $\tau = c_u$(ϕ_u = 0).
- Unit weight: total γ.
- Water: neglected in shear; included in total stresses.
- EC7 check: GEO ULS using $\gamma_M = 1.25$on c_u.

Simplified slope stability (short-term clay):

$$F = \frac{c_u N_c}{\gamma H}$$

$N_c \approx$ 5–6for circular failure (Bishop).

Key risks:

- Rapid excavation triggers transient excess pore pressures → delayed failure ("progressive softening").
- Watch for softening or fissuring in OC clays (London, Mercia Mudstone).

Always verify intermediate **construction stages**: the most critical condition may occur **mid-excavation**, not at final geometry.

18.3.4 Drained (Long-Term) Analysis

After pore pressures dissipate:

- Use effective stress parameters c', φ'.
- Include groundwater explicitly via u(z).
- Verify at both high and low water levels.
- EC7 ULS uses $\gamma_M = 1.25$on $c', tan\varphi'$.

Infinite-slope form (for screening):

$$F = \frac{c'}{\gamma' z \sin^2 \beta} + \frac{\tan \varphi'}{\tan \beta}$$

In dense granular soils, long-term often governs.

In clays, long-term is usually **more stable** due to pore-pressure dissipation, but weathering or erosion may reduce c' over time.

18.3.5 Transition & Partial Drainage Cases

Where the rate of loading ≈ rate of consolidation (e.g., staged embankment), neither assumption is fully valid.

Approach	Implementation
Pore-pressure ratio (r_u method)	Approximate u = r_u γ H $\cos^2$ β (0.2–0.5 typical)
Coupled consolidation analysis	Time-dependent FE or LE with cv, mv, Δt steps
Stage-construction model	Incremental raise (e.g., PLAXIS, Slope/W), update c_u gained per stage

For critical embankments, use staged analysis with consolidation and observational verification of pore-pressure dissipation (Chapter 22).

18.3.6 Rapid Drawdown (Special Case)

Occurs when reservoir or river levels drop faster than pore pressures dissipate in the slope.
Use:

- Effective stresses for seepage; undrained strength in slope body.
- Apply "steady-state" pore pressure in soil + reduced external head.
- Target FOS ≥ 1.2 (informative).

Include transient seepage model if dam or flood-bank related (CIRIA C742 guidance).

18.3.7 Example Comparison (conceptual)

Condition	Parameter set	Computed FOS (char)	Governs?
Short-term cut (undrained)	$c_u = 45kPa$	1.25	✓ Temporary works
Long-term (drained)	$c' = 8kPa, \varphi' = 25°$	1.60	— Permanent slope
Rapid drawdown	Mixed	1.05	⚠ Critical case

18.3.8 Documentation Requirements

Each slope section in the **GDR** should include:

1. Table of soil layers and both parameter sets (ST/LT).
2. **Water conditions** per stage (initial, excavated, long-term).
3. **Verification summary** (ULS ratio or FOS for both).
4. **Commentary** on which case governs.
5. Staged construction chart if applicable.

18.3.9 Typical Consolidation Times (order of magnitude)

Soil Type	Coefficient cv (m^2/yr)	Time to 90 % consolidation for 3 m drainage path
Soft clay	0.5	2–3 years
Firm clay	2	6–8 months
Silt / sandy clay	10	1–2 months
Sand / gravel	> 100	Immediate (fully drained)

Phicon Note:

• Always separate **short-term (construction)** and **long-term (service)** checks in the GDR — they use **different material models**.
• In cuts, the most dangerous moment is **immediately after excavation**, not after stabilisation.
• For embankments, the least safe is **just before excess pore pressure dissipates**; use staged models or instrument feedback.
• Specify **piezometers** and **standpipes** to confirm drainage assumptions — these validate which design case is real.
• Do not mix **undrained c_u** and **drained ϕ'** within the same LE model unless justified (hybrid methods can mislead).
• Record **Δu assumptions** and, if possible, tie them to laboratory cv data or empirical time–depth plots.

18.4 Reinforced & Stabilised Slopes

Reinforced and stabilised slopes combine **soil strength with tensile reinforcement** (geogrids, geotextiles, nails, soilcrete) to achieve steeper angles or improve stability on weak foundations.

Design follows **BS 8006-1**, **EN 14475**, and the Eurocode 7 framework for **GEO/STR limit states**, supported by site-specific ground models.

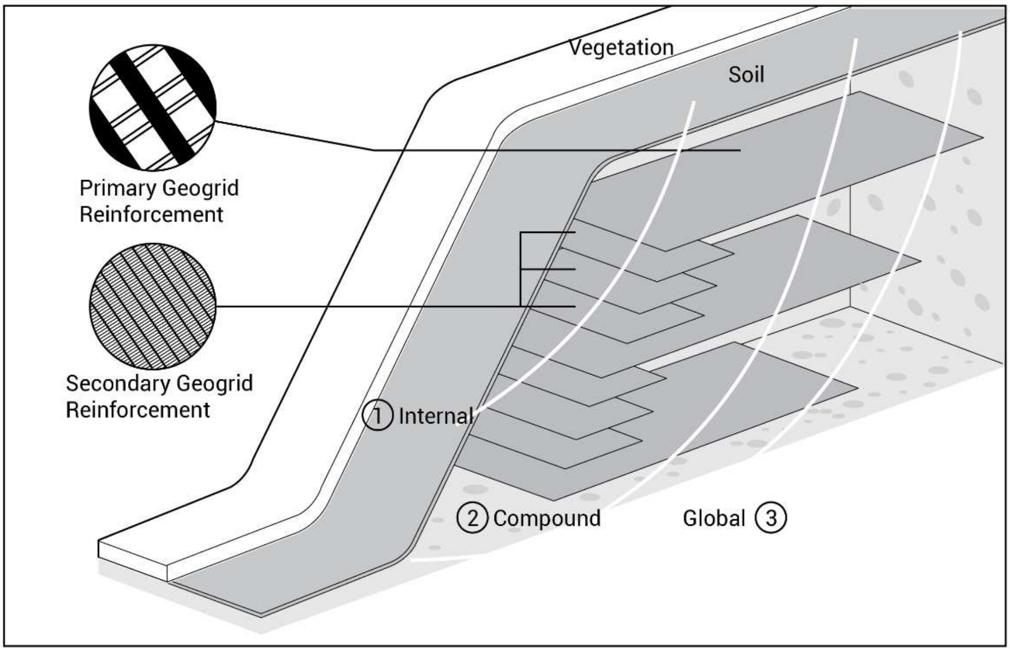


Figure 72: GEO failure mechanisms within reinforced slope

18.4.1 Purpose and Applications

Application	Typical Height	Slope Angle	Reinforcement Type
Embankment face steepening	< 10 m	up to 70°	Geogrid layers in engineered fill
Cut slope stabilisation	5–25 m	45–80°	Soil nails or anchors
Retained earth structures	5–40 m	70–90°	Mechanically Stabilised Earth (MSE) wall or wrap-around face
Temporary works (excavation support)	variable	60–90°	Shotcrete + soil nails / mesh
Slope repair / re-profiling	variable	variable	Geogrid or geocell with surface drainage

18.4.2 Design Principles

Reinforced slopes are checked for both **external** and **internal** stability.

Check	Failure Mode	Typical Analysis Method
External stability	Overall circular / compound slip	Limit equilibrium (Bishop, M-Price) including reinforcement
Internal stability	Pull-out, rupture of reinforcement	BS 8006 / EN 14475 design equations
Facing stability	Local sliding, connection strength	Empirical / manufacturer design data
Bearing / settlement	Excessive deformation of foundation	EC7 settlement checks (SLS)

The overall design must show **ULS (GEO/STR) ≥ 1.0** using design parameters (DA1/C2), and confirm **serviceability deformations are acceptable**.

18.4.3 Reinforcement Mechanisms

1. **Tensile Resistance** – reinforcement carries tension across potential slip surfaces.
2. **Pull-out Resistance** – friction or bond along embedded length.
3. **Confinement & Stress Redistribution** – restrains lateral strain, increases apparent ϕ'.
4. **Facing / Anchorage** – restrains local erosion and surface deformation.

18.4.4 Governing Design Equations (BS 8006-1:2010 / EN 14475)

(a) Tensile Load per Layer

$$T_d = \frac{S_v \, \gamma \, H}{N_{layers}} \times \text{(mobilisation factor)}$$

where S_v= vertical spacing, H= slope height, γ= unit weight.

Each layer must satisfy:

$$T_d \leq \frac{T_{ult}}{\gamma_{RF}}$$

with partial resistance factor $\gamma_{RF} = \gamma_{mb} \times \gamma_{cr} \times \gamma_{dur} \times \gamma_{doc}$.

Factor	Meaning	Typical Value
γ_{mb}	Manufacturing	1.05–1.10
γ_{cr}	Creep	1.2–1.5
γ_{dur}	Durability (chemical/biological)	1.1–1.2
γ_{doc}	Installation damage	1.1–1.3

Combined $\gamma_{RF} \approx$ 1.5–2.0 typical.

(b) Pull-Out Resistance

$$R_{pd} = 2L_e(c' + \sigma_v' \tan\,\delta)/\gamma_P$$

where L_e= embedment length beyond slip, δ= interface friction angle, $\gamma_P = 1.1–1.3$.
Design criterion: $R_{pd} \geq T_d$.

(c) Connection Strength

At the facing (wrap-around block, panel, or nail head):

$$T_d \leq R_{conn,d} = \frac{R_{conn,k}}{\gamma_{conn}}; \gamma_{conn} = 1.3–1.5$$

18.4.5 Reinforcement Layout Guidance

Parameter	Typical Range	Notes
Vertical spacing (S_v)	0.4–0.8 m	Closer near slope face
Embedment length (L_e)	≥ 0.7–0.8 × slip radius beyond surface	Ensure adequate bond length
Reinforcement length (total)	0.7–1.0 × slope height	Depends on ϕ' and face inclination
Interface friction (δ)	0.7–0.9 ϕ'	Use measured or conservative estimate
Reinforced zone back slope	Typically 1H:1V to 0.5H:1V	Steeper possible with high-strength grids

⚠️ Check global stability including both reinforced zone and retained ground — not just reinforcement limit states.

18.4.6 Soil Nailing (Cut Slope Stabilisation)

Soil nails act as **passive tension members** mobilised as the slope deforms. Design to **EN 14490** (Execution of Soil Nailing) and BS 8006 concepts.

Parameter	Typical Value / Range
Nail inclination	10–20° below horizontal
Nail spacing	1.0–2.0 m (horiz. & vert.)
Bond length	4–6 m or beyond slip plane
Bar size / strength	25–40 mm, f_n = 500–700 MPa
Design safety factors	γ_M = 1.25 (GEO), γ_S = 1.15 (STR)

Check:

1. Nail **pull-out** (bond shear ≤ capacity),
2. Tensile yield of bar,
3. **Global stability** with nails included as line elements,
4. **Facing** (shotcrete or mesh) for local stability.

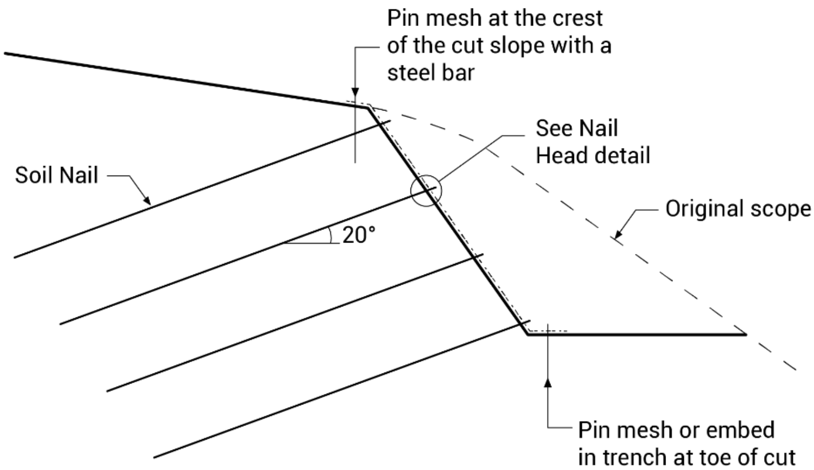


Figure 73: Typical soil nail cross section within cutting (reproduced from Whitbread 2012)

18.4.7 Reinforced Soil vs. Retaining Wall Selection

Aspect	Reinforced Slope	Retaining Wall (MSE / RC)
Typical face angle	≤ 70°	≥ 70–90°
Foundation requirements	Wider base, flexible	Higher bearing, rigid
Settlement tolerance	Higher	Low

Drainage	Integral (free draining)	Often requires behind-wall drainage
Aesthetics	Natural / vegetated	Structural
Cost (approx.)	60–80 % of RC wall	—

Reinforced slopes are efficient where space permits; MSE walls where footprint is limited.

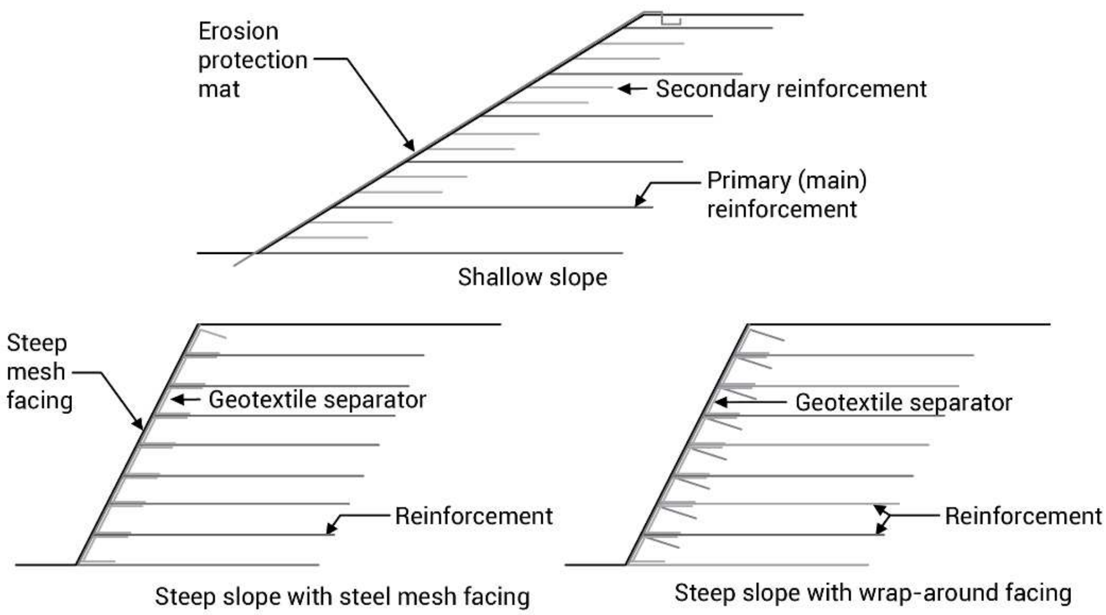


Figure 74: Typical reinforced soil slope configurations (reproduced from Manceau et al. 2012)

18.4.8 Construction Considerations

- **Compaction control:** light compaction near face to prevent geogrid damage.
- **Drainage:** provide rear chimney drains; no ponding in reinforced zone.
- **Face detail:** geogrid wrap-around, modular blocks, or gabion fascia.
- **Sequence:** layer placement and tensioning per manufacturer's method statement.
- **Quality assurance:** check grid orientation (machine direction), overlaps, and continuity.

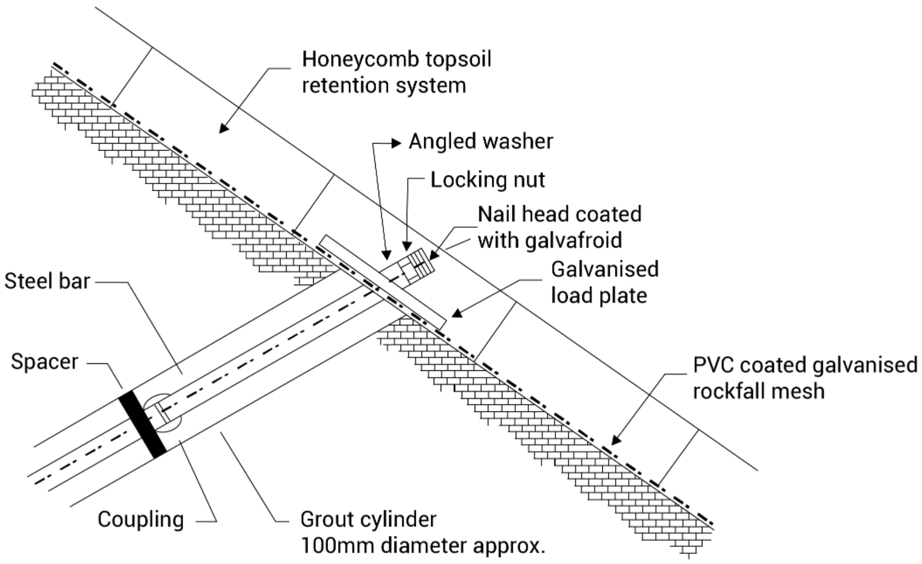


Figure 75: Typical soil nail head detail (reproduced from Whitbread 2012)

18.4.9 EC7 Verification Summary (GEO / STR)

Limit State	Check	Design Equation	Partial Factors (UK NA)
GEO (global)	Stability of entire slope	$E_d \leq R_d$(LE or FE)	γ_M = 1.25 on c', tan ϕ'
STR (reinforcement)	Tensile and pull-out resistance	$T_d \leq R_d$	γ_S = 1.15 (steel), γ_M = 1.25 (soil)
SLS	Deformation, face displacement	≤ 1/100 height	Informative
DUR	Long-term degradation	Strength after 50–100 yr	Manufacturer data / BS 8006 Table 13

18.4.10 Documentation Checklist (for GDR)

✓ Geometry and section drawings.
✓ Soil design parameters (char & design).
✓ Reinforcement type, layout, and long-term reduction factors.
✓ EC7 verification summary (ULS & SLS).
✓ Drainage and facing details.
✓ QA plan: testing, inspection, pull-out checks, certificates.

Phicon Note:

• Always distinguish **reinforced slope** (soil structure) from **retaining wall** (structural element) in reporting — the limit states differ.
• In the GDR, show both **reinforced zone outline** and **critical slip circle** from the global analysis.
• Never rely solely on manufacturer software output — cross-check using LE method with equivalent tension lines.
• Specify **minimum embedment and wrap detail** on drawings; site teams frequently shorten these.
• Include **durability design life** (50–100 yrs) and **reduction factors** transparently — reviewers expect traceability to BS 8006 tables.
• Verify **internal pull-out** (within the zone) and **external overall stability** (outside it) separately.
• In vegetated faces, include **temporary erosion matting** until vegetation is established.

18.4.11 Worked Example – Circular Slip Analysis of Clay Embankment

(Eurocode 7 DA1/C2; Short-Term ϕ_u = 0 and Long-Term c'–ϕ')

Problem Statement

Assess the stability of a **new roadway embankment** on clay. Verify **short-term (undrained)** during construction and **long-term (drained)** after consolidation. Provide **EC7 DA1/C2** checks and an **informative (unfactored) FoS** for context.

A) Geometry, Ground & Water

- Embankment height: **H = 6.0 m**; side slopes **1V:2.5H** (β ≈ 21.8°)
- Crest width: **10 m**
- Foundation soil: uniform **soft to firm clay** of **thickness > H** (governing layer)
- Groundwater: **at existing ground level** (phreatic at toe); for long-term, steady seepage assumed
- Materials (Characteristic values)

Material	γ (kN/m^3)	γ' (kN/m^3)	Parameters
Embankment engineered fill	19	9.5	Long-term ϕ' = 32°, c' = 0*
Foundation clay – short-term	19	—	c_u,k = 35 kPa (UU/CU)
Foundation clay – long-term	18	8.5	c' = 8 kPa, ϕ' = 22°

⚠️ Sometimes software cannot understand a 0 value for cohesion and computes incorrect outputs therefore a 0.1 kPa is reccomended in complex analysis.

EC7 DA1/C2 design values:

- $c_{u,d} = c_{u,k}/1.25 = 28\text{kPa}$
- $\tan\ \varphi'_d = \tan\ \varphi'/1.25 \Rightarrow \varphi'_d \approx 18.0°$
- $c'_d = 8/1.25 = 6.4\text{kPa}$
- Actions: γ_F = 1.0 (C2); unit weights unchanged

B) Short-Term (Undrained) – ϕu = 0

Method: Bishop simplified (circular), but a screening **c_u–Nc** estimate guides the target.

For wide embankments on uniform clay, an indicative **bearing-type circle** gives:

$$F \approx \frac{c_u N_c}{\gamma H} \text{with} N_c \approx 5.5\text{–}6.0$$

- Informative (unfactored) FoS:

$$F_{char} \approx \frac{35 \times 5.7}{19 \times 6} = \frac{199.5}{114} = 1.75$$

- EC7 DA1/C2 Design Check (ULS):

$$F_d \approx \frac{c_{u,d} N_c}{\gamma H} = \frac{28 \times 5.7}{19 \times 6} = \frac{159.6}{114} = 1.40 \quad (\geq 1.0) \quad \checkmark$$

(A quick Bishop run with ϕu = 0 and the above geometry typically returns $F_d \approx 1.35\text{–}1.45$; adopt the governing software result in the GDR.)

⚠️ **Comment:** Short-term stability is satisfactory without berms. If $c_{u,k}$ were ≤ 30 kPa, berming or basal reinforcement may be needed.

C) Long-Term (Drained) – c'–ϕ' with groundwater

Method: Bishop/Morgenstern–Price (circular). Pore pressures from the phreatic line at the slope face (steady seepage parallel to slope through fill and into clay).

- Screening (infinite slope in fill, conservative):

$$F_{fill} = \frac{c'}{\gamma' z \sin^2 \beta} + \frac{\tan \phi'}{\tan \beta}$$

At the face, $c' = 0$(granular fill), z →shallow →

$$F_{fill} \approx \frac{\tan\ 32°}{\tan\ 21.8°} = \frac{0.624}{0.399} = 1.56$$

- Whole system (circle passing into clay)

Adopt design parameters for foundation clay (controls along deeper arc):

- **Informative (unfactored) FoS (Bishop):** $F_{char} \approx 1.45$(typical for these inputs)
- **DA1/C2 design run:** reduce to $c'_d = 6.4$kPa, $\varphi'_d = 18°$→ $F_d \approx 1.12\text{–}1.18$(software dependent) → **OK (≥ 1.0)**

If phreatic line is conservatively raised +1 m, sensitivity shows F_d ↓by ≈ 0.05–0.10. Still typically ≥ 1.05 here.

D) Sensitivity (recommended for GDR)

Variation	ST F_d(ϕu=0)	LT F_d(c'–ϕ')	Note
Base case	1.40	1.15	—
$c_{u,k} - 20\% \rightarrow c_{u,d} = 22.4$**kPa**	1.12	—	May require berm/reinforcement
ϕ' − 3° (to 19° char; ϕ'_d ≈ 15.3°)	—	1.06	Near-limit; drainage/berm helps
Water table +1 m	—	1.07	Add chimney/toe drains

E) Measures if Long-Term Margin is Tight

- **Chimney + toe drain** (lower phreatic line through fill).
- **Wider toe / berm** (increases resisting moment).
- **Basal geogrid** across foundation (tensioned membrane effect).
- **Lightweight fill** at upper slope to reduce driving weight.
- Staged construction + waiting period for consolidation strength gain.

F) EC7 DA1/C2 Verification Summary (to place in GDR)

Case	Parameter set	Method	$F_d$or E_d/R_d	Status
Short-term (construction)	ϕu=0, $c_{u,d} = 28$kPa	Bishop	1.40	✓ GEO ULS OK
Long-term (service)	$c'_d = 6.4$kPa, $\varphi'_d = 18°$	M–Price	1.15	✓ GEO ULS OK
Sensitivity: WT +1 m	as LT	M–Price	1.07	✓ but close

Attach: critical slip plots, phreatic line, slice table, and sensitivity charts.

Phicon Note:

• Separate **ST ϕu=0** and **LT c'–ϕ'** checks — they govern at different times.
• Put **water** front and centre: draw the phreatic line and test **+1 m** scenario.
• If LT margin < 1.10, add **drainage (chimney+toe)** or a **modest berm**; usually the most economical fix.
• For soft foundations, consider **basal reinforcement** or **staged preload** to gain shear strength before full height.
• Record both **DA1/C2 results** and **informative FoS**; reviewers expect clarity on EC7 compliance vs traditional safety factors.
• Instrument with **piezometers and settlement plates** if sensitivity is high (hand-off to Chapter 22 for monitoring triggers).

19 Earthworks Specification & Compaction (QA/QC)

19.1 Earthworks Classes and Material Acceptability

This section gives a field-ready acceptance framework for earthworks materials, linking classification & suitability to specification clauses and QA testing. It aligns with UK/EU practice drawing on BS EN 16907 (Earthworks), BS 6031 (Code of practice for earthworks), BS EN ISO 14688 (soil classification), BS 1377 (testing), and (where used) the UK Specification for Highway Works (SHW) Series 600/800 material classes.

19.1.1 Purpose

- Define **what can and cannot go into the earthworks**, and **where** (fill, capping, embankment core, drainage).
- Set **simple, checkable limits** (moisture, PI, grading, contaminants) for **accept/reject** on site.
- Tie acceptability to **compaction method & QA tests** later in Chapter 19.

19.1.2 Material Families (quick map)

Family	Typical sources	Use	Key risks
Granular (sands, gravels, crushed rock)	Quarries, site-won sand & gravel, recycled aggregate	Structural fill, capping, drainage	Degradation, excess fines, sulphate
Cohesive (silts/clays)	Site-won glacial/estuarine clays	Embankment core, landscape	Moisture sensitivity, shrink–swell
Engineered/by-products	Crushed concrete, slag, PFA, stabilised soils	Capping, selected fill	Variability, leachate, durability
Problematic	Topsoil/peat/organics; chalk at high moisture; highly plastic clays	Generally unsuitable	Collapse, excessive settlement, durability

19.1.3 Acceptability Matrix (selection at a glance)

19.1.3.1 A. Structural/Engineered Fill (e.g., under pavements, platforms, foundations)

Criterion	Granular (selected)	Cohesive (engineered)
LL / PI	LL not critical; **fines (P0.063)** ≤ 12–15% typical	**PI ≤ 20** (≤ 25 if controlled), **LL ≤ 60** (project specific)
Moisture	Within **OMC ±2%**	**OMC to OMC+2%** (avoid dry clod)
Organic / deleterious	**≤ 1%** organics; no wood, gypsum, asphalt lumps	**≤ 1%** organics; no degradables
Sulphate (water-soluble)	As per concrete class/DRS; typically ≤ **0.3%** for near-concrete	Same; assess **CES** (total potential sulphate)

Compaction	Heavy compaction permitted	Avoid excessive energy that laminates lifts

19.1.3.2 B. Capping / Working Platform Granular (e.g., 6F1/6F2 type)

Property	Typical limit (guide)
Particle size	Max 125–150 mm; controlled fines (≤ 15%)
Los Angeles (LA) or MDE	LA ≤ 35 (durable); MDE ≤ 30
Plasticity of fines	Non-plastic to **PI ≤ 6**
Frost susceptibility	F1 or better (project specific)

19.1.3.3 C. Drainage / Filter (see §14.3 filter criteria)

- Clean, hard **10–20 mm** gravel, **< 1%** passing 0.063 mm, non-plastic fines.
- Geotextile to **EN 13252** where separation/filtering required.

⚠️ **Project specifications take precedence.** The above are common ranges used to draft acceptance tables.

19.1.4 UK SHW Classes (if adopting SHW language)

SHW class (typical)	Description / use	Notes
1 / 2	General cohesive/granular fill	Embankments/landscaping (not structural)
6N / 6P	Selected cohesive (N) / granular (P) engineered fill	Embankment shoulders/structural zones
6F1 / 6F2	Granular capping / platform	Well-graded, low fines; recycled often 6F2
6C / 6G	Selected granular (formation) / selected fill	For formation improvement
1A/1B	Acceptable arisings (general)	Moisture/PI limits govern
7A–7D	Materials requiring treatment (lime/cement, etc.)	Use §20 for stabilisation design

If not using SHW, **map** SHW classes to your **EN 16907 material specifications** in the project spec.

19.1.5 Exclusions & Conditional Use

Material	Status	Notes / Conditions
Topsoil, peat, organics > 2%	Not acceptable	Strip and reuse in landscaping only
Made ground with deleterious (ash, wood, gypsum, tar)	Conditional / not acceptable	Only after processing & testing; segregate hot spots
Highly plastic clays (PI > 40, LL > 80)	Conditional	Use only with tight moisture control or stabilisation
Chalk	Conditional	Accept as engineered chalk **only**: low moisture, tight lift thickness, avoid trafficking when wet
Sulphate-bearing (e.g., colliery spoil)	Conditional	Chemical testing; consider **concrete class** & ground improvement
Contaminated soils	Not acceptable (structural)	Risk assessment; treat/replace

19.1.6 Minimum Testing for Acceptance (lot-based)

Test	Standard	When / frequency (typ.)
Classification (LL, PL, PI)	BS 1377-2	1 per **500–1000 m^3** per source/type
Grading & fines (P0.063)	BS EN 933	1 per 500–1000 m^3
Moisture content (w)	BS 1377-2	**Daily** per active face + each lot
Organic content / LOI	BS 1377-3	If suspect, else 1 per **2000 m^3**
Sulphate (water-soluble & total)	BS 1377-3 / BRE SD1	Near concrete or suspect strata
MCV (moisture condition value)	TRL	**Every 200–400 m^3** for cohesive fills
Compaction (density/air voids)	BS 1377-4 / NDG	2–4 tests per 1000 m^2 per layer set
Plate load / EV_2	DIN 18134	Working platforms, formation checks

Frequencies are **typical**; set project-specific lot sizes and escalate if variability is high.

19.1.7 Moisture & Workability Windows (field rules)

Cohesive engineered fill

- Target OMC to OMC + 2%; if w > OMC + 3–4%, reject or condition (aerate, lime).
- Avoid trafficking that kneads and laminates layers; **loose lift 150–225 mm** (compacted ~125–175 mm).

Granular engineered fill

- Use appropriate plant (sheepsfoot for well-graded with fines; smooth drum/vibrating for clean granular).
- Maintain **uniform grading**; avoid segregation (manage drop heights, windrows).

19.1.8 Acceptance Flow (site quick card)

1. **Identify**: source, visual classify (granular/cohesive), note moisture.
2. **Screen**: LL/PI (cohesive) or grading/fines (granular), organics/sulphate.
3. **Assign**: material to **use class** (e.g., 6F2 capping; 6N; selected granular).
4. **Set window**: compaction method, lift thickness, **OMC window**.
5. **Prove**: MCV/NDG/EV_2 as applicable; record lot & location.
6. **Accept/Reject/Condition**: document action; track by delivery ticket/stockpile ID.

19.1.9 Recycled & Secondary Aggregates (RSA)

- Require **source certification** (WRAP/CE) and **harmonised testing**: grading, LA/MDE, fines plasticity, sulphate, organic contaminants.
- **Crushed concrete** generally suitable for **6F2** if fines are controlled and deleterious removed.
- Manage variability by blending and tighter test frequency.

19.1.10 Durability & Chemical Environment

- Tie **sulphate class** to **concrete class** (e.g., BRE SD1 / BS 8500) where fill is adjacent to RC.
- In aggressive soils (XA classes), specify **cement types** and **protective details**; avoid gypsum-rich fines behind RC walls.
- For **frost** exposure (formation/capping), specify **non-frost-susceptible** materials (project criteria).

19.1.11 Documentation – What reviewers expect

- **Material schedule** mapping source to **use class** with **limits** (LL/PI, P0.063, sulphate, organics).
- **Lot control plan**: layer thickness, plant, test frequency, acceptance criteria.
- **Non-conformance route**: condition (dry/aerate/lime), re-test, or reject.
- **Traceability**: stockpile IDs, delivery tickets, test certificates, location records (chainage/grid).

Phicon Note:

• Keep the **acceptability table** to one page in the spec: PI/LL limits, fines %, moisture window, and where the material may be placed.
• If variability is high, **tighten lot size** and **increase test frequency** rather than arguing about borderline results.
• **Moisture control is king** for cohesive fills — plan conditioning (lime bays, drying pads) from the outset.
• For working platforms and capping, insist on **graded, low-fines granular** with EV_2/PLT proof — it saves plant incidents.
• Always link **sulphate limits** to nearby **concrete durability** requirements.
• Chalk and very plastic clays are **special cases** — either detail strict windows or exclude from structural zones.

19.2 Specification Types (Method, End-Product)

Specifications define **how earthworks must be built** and **how compliance is proven**.
Eurocode 7 and BS EN 16907-1 emphasise that requirements must be *measurable, verifiable,* and *tied to performance intent*.

This section outlines the two main specification philosophies — **Method** and **End-Product** — and how to blend them effectively for UK infrastructure works.

A **Performance-based specification** can be viewed as a sub-set of an end-product specification, focusing on the measurable behaviour of the completed works rather than the construction process. In geotechnical design, it defines serviceability criteria such as limits on total and differential settlement or allowable bearing pressure, which represent performance outcomes of the constructed ground. These criteria form part of the overall end-product requirements, linking in-service behaviour to the physical quality and compliance of the finished works.

19.2.1 Purpose

- Clarify whether quality is controlled by **process** (method) or **result** (end-product).
- Define **responsibilities** between Designer, Contractor, and Supervisor.
- Align site QA testing (density, EV_2, MCV) with the **specification style**.

19.2.2 Specification Philosophy

Specification Type	Basis of Compliance	Used When	Examples
Method Specification	Compliance = Follow defined procedure	When soil behaviour is variable or hard to measure quantitatively	Earthworks in cohesive clay, lime-treated materials, environmental fills
End-Product Specification	Compliance = Achieve target property (density, EV_2, air voids, etc.)	When uniform material, repeatable process	Granular fills, capping, working platforms

In practice, large infrastructure projects use **hybrid specifications** — method-based for cohesive materials, end-product for granular zones.

19.2.3 Method Specification (Process-Based)

The designer prescribes **plant, lift thickness, and moisture window**, assuming that following this procedure ensures the required performance.

⚠️ Testing focuses on **process control** rather than numerical result.

Typical Clause Content	Example
Compaction plant	"Use 10-tonne sheepsfoot roller; minimum 4 passes per layer."
Lift thickness	"Lay fill in 200 mm loose layers."

Moisture window	“Compact between OMC and OMC + 2 %.”
Visual checks	“Reject layers with visible laminations or clod sizes > 75 mm.”

Verification:

- Record plant type, passes, and moisture readings.
- Carry out occasional density/MCV tests for confidence.

Advantages:

- Works well with variable or fine-grained materials.
- Reflects long UK experience with cohesive fills.

Limitations:

- Doesn’t quantify stiffness; not suited to structural fills where performance (EV_2, CBR) matters.

19.2.4 End-Product Specification (Performance-Based)

The contractor chooses means and plant to achieve defined **target results**, such as compaction, stiffness, or bearing value.

Typical Clause Content	Example
Density ratio	≥ 95 % of MDD (BS 1377-4 or 13286-2)
Air voids	≤ 5 % (or ≤ 10 % for cohesive)
Plate load / EV_2	≥ 120 MPa for capping; EV_2/EV_1 ≥ 2.0
CBR	≥ 5 % (formation); ≥ 15 % (sub-base)
MCV	≥ 10 (general fill); ≥ 15 (structural fill)
ΔS_{max}	< 1:200 over 10m; 5 years after construction
ρ_{max}	<25mm rut depth; 1 yea during operations

Verification:

- Field testing each lot: NDG, sand replacement, PLT, or LWD.
- Acceptance based on achieving numerical thresholds.

Advantages:

- Encourages innovation in plant and sequencing.
- Produces quantifiable performance.

Limitations:

- Requires reliable and consistent material source.
- High testing demand; not suitable for very variable fills.

19.2.5 Hybrid Specification (Best Practice)

A **hybrid approach** ties together the benefits of both systems.

Typical structure:

1. **General method**: plant type, layer thickness, moisture window.
2. **Performance trigger**: minimum EV_2, MCV, or density.
3. **Fallback route**: if testing fails → re-work, aerate, or compact additional passes.

Example hybrid clause:

> *"Compact cohesive fill in 200 mm layers using 10 t sheepsfoot roller.*
> *Moisture shall be within OMC ± 2 %.*
> *Achieve **MCV ≥ 10** or **dry density ≥ 92 %** of MDD.*
> *Failing lots shall be re-worked until results comply."*

19.2.6 Selection Guidance

Material	Recommended Specification Type	Typical Verification
Cohesive engineered fill (clay)	Method + MCV trigger	Moisture, MCV, NDG (trend)
Granular fill (6F2, capping)	End-product	EV_2 / NDG density
Working platforms	End-product	PLT or LWD (target EV_2, EV_2/EV_1)
Stabilised soils	Method + Performance	Lime content, pH, UCS, EV_2
Topsoil / landscaping	Method only	Visual; moisture & thickness
General backfill	Method + occasional check	Moisture, compaction record

19.2.7 Managing Non-Conformance

If field results fall short of the specification:

Action	Example Measures
Review procedure	Increase roller passes, adjust moisture, change plant type
Condition material	Aerate (drying) or lime-treat to improve workability
Re-test after remedial	New NDG / MCV / EV_2 test
Record & disposition	NCR log; identify chainage, layer, date, test ID

All actions and re-tests must be logged and signed off by the **Engineer's Representative or Earthworks Supervisor** under QA procedures (see §19.7).

19.2.8 EC7 QA Context

Eurocode 7 (§2.7 & §4.1) requires that construction **methods and controls** used in design assumptions are verified in the field.
Therefore:

- If a **method specification** was assumed in design → verify compliance with that method.
- If design assumed a **performance level (EV_2, stiffness)** → verify by end-product testing.

Failure to maintain this link can invalidate design assumptions.

19.2.9 Typical Acceptance Targets (illustrative)

Application	Parameter	Target / Limit	Verification Method
Embankment core (clay)	MCV	≥ 10	MCV / NDG
Structural fill	Density	≥ 95 % MDD	NDG / Sand replacement
Capping (6F2)	EV_2	≥ 120 MPa	PLT / LWD
Working platform	EV_2/EV_1	≥ 2.0	PLT
Formation	CBR	≥ 5 %	CBR / PLT
Sub-base	EV_2	≥ 180 MPa	PLT / LWD

Target EV_2 and CBR values are project-specific and depend on the asset type (highway, rail, airfield), traffic loading and layer function (formation, capping, sub-ballast, sub-base). The values in this table are illustrative and should be replaced by the governing client or national specification for the project.

Phicon Note:

• Choose specification type based on *variability and consequence*. For cohesive fills, method-based control avoids chasing inconsistent test values.
• Always define **what constitutes compliance** — measurable value, tolerance, or visual condition.
• Keep clauses concise: *"compact with 10 t roller, OMC ± 2 %, MCV ≥ 10"* tells site teams exactly what to do and test.
• If adopting end-product control, include a **fallback clause** (re-work, drying, retest).
• Record **actual compaction plant** and **moisture window**; these confirm design-to-construction consistency under EC7.
• Treat QA testing as **statistical evidence**, not punishment — consistency trends are more valuable than individual outliers.

19.3 Compaction Theory (Dry Density, Air Voids, OMC, MCV)

Compaction is the most critical process in earthworks.

It transforms loose or wet material into a **dense, stable mass** capable of sustaining design loads with minimal deformation.

This section outlines the **fundamentals of compaction**, **key parameters** used for field control, and **Eurocode-consistent verification logic** for design and construction QA.

19.3.1 Purpose of Compaction

Compaction improves:

- **Shear strength** (increase c' and ϕ'),
- **Stiffness** (increase E, EV_2, CBR),
- **Volume stability** (decrease in settlement and creep),
- **Permeability control** (decrease in k, beneficial in clays).

In EC7 terms, compaction ensures the constructed fill meets the **design assumptions** used in slope, bearing, and settlement calculations (GEO and SLS verifications).

19.3.2 Fundamental Concepts

Compaction re-arranges soil particles by mechanical energy, expelling air (but not water).

The relationship between **dry density ($\rho_{(}d_{)}$)** and **moisture content (w)** is used to define the **Optimum Moisture Content (OMC)** — the moisture at which maximum dry density (MDD) is achieved for a given compactive effort.

$$\rho_d = \frac{\rho}{1 + w}$$

where ρ= bulk density, w= water content.

19.3.3 Standard Laboratory Tests

Test	Standard	Compactive Effort	Use
Standard Proctor	BS 1377-4 / BS EN 13286-2	600 kN·m/m^3	Light compaction (landscape fills)
Modified Proctor (Heavy)	BS 1377-4 / BS EN 13286-2	2700 kN·m/m^3	Structural fills, platforms, capping
Vibrating hammer (coarse)	BS 1377-4 Cl. 4.4	Coarse aggregate fills	Equivalent to Heavy Proctor
MCV–Density correlation	TRL Report 1132	Field validation	For cohesive control, avoids Proctor

19.3.4 Dry Density–Moisture Relationship

Typical curve:

- **Left of OMC** → too dry, insufficient cohesion, risk of particle breakage.
- **At OMC** → maximum achievable density.
- **Right of OMC** → excess water; pore pressure buildup and low stiffness.

Moisture Condition	Field Consequence	Acceptable Range
< OMC − 2 %	Hard, dusty, poor binding	Reject or wet
≈ OMC	Optimum	✓
> OMC + 2 %	Soft, pumping, laminates	Reject or aerate

19.3.5 Air Voids Criterion

Air voids (%Va) indicate compaction efficiency independent of soil type.

$$V_a = 100(1 - \frac{\rho_d(1+w)}{\rho_s})$$

where ρ_s= particle density (typically 2.65 Mg/m^3 for soils).

Material Type	Target Va (%)	Equivalent Compaction
Granular fills	≤ 5	≈ 98 % MDD
Cohesive fills	≤ 10	≈ 95 % MDD
Low permeability cores	≤ 8	—

Controlling **air voids** rather than %MDD gives a consistent stiffness/strength proxy, especially across variable sources.

19.3.6 Moisture Condition Value (MCV)

MCV (Moisture Condition Value) provides a **rapid field indicator** of soil workability and compactability, especially for cohesive fills.

MCV Range	Interpretation (BS 6031 / TRL)
< 5	Too wet; not suitable for compaction
5–8	Marginal; drying/aeration or lime required
8–10	Suitable for general fill (method spec)
10–15	Suitable for engineered fill / embankment
> 15	Very dry or stiff; difficult to compact uniformly

Empirical relationships (approximate):

$$MCV \approx 10 + 4(OMC - w) \text{or} MCV \leftrightarrow \rho_d \text{ via site correlation chart}$$

A site-specific correlation between **MCV, moisture content, and dry density** should be established during trial compaction.

19.3.7 Compactive Effort vs Density (field implications)

Compaction Plant	Energy Level	Typical Use / Material
Smooth drum roller (vibrating)	High	Granular fill, capping
Sheepsfoot / padfoot roller	Medium–high	Cohesive clay fills
Pneumatic tyred roller	Moderate	Mixed / fine granular
Tracked dozer (non-vibrating)	Low	Only for spreading, not compaction
Hand rammer / trench roller	Localised	Narrow backfill, reinstatements

19.3.8 Field Density Verification

Method	Standard	Typical Material	Notes
Sand replacement	BS 1377-9	Cohesive / fine soils	Accurate; slow
Nuclear density gauge (NDG)	ASTM D6938 / BS EN 17892-12	All except >40 mm	Rapid; calibration needed
LWD / PLT (EV_2)	DIN 18134	Granular fills	Stiffness proxy; correlates to density
MCV trend line	TRL	Cohesive fills	Rapid process control

Results are compared against the **MDD / OMC curve** from laboratory or site calibration.

19.3.9 Influence of Moisture & Structure

Cohesive soils:

- At OMC → flocculated structure → good strength.
- Wet of OMC → dispersed → low permeability but weak.
- Dry of OMC → stiff but brittle; can trap voids.

Granular soils:

- Moisture acts as lubricant; too dry → segregation; too wet → pore pressure/pumping.

19.3.10 Compaction Criteria – Summary Table

Material	Specification Basis	Target (End-Product)	Typical OMC window
Granular (6F2, capping)	Heavy Proctor	≥ 95–98 % MDD; EV_2 ≥ 120 MPa	± 2 %
Cohesive (6N/6P)	Method + MCV	≥ 92–95 % MDD or MCV ≥ 10	OMC to +2 %
Structural fill (formation)	Heavy Proctor	≥ 95 % MDD; Air voids ≤ 10 %	± 2 %
Lime-treated soils	Method + UCS/EV_2	UCS ≥ 1 MPa (7d) or EV_2 ≥ 80 MPa	—
Working platform	End-product	EV_2 ≥ 120–150 MPa; EV_2/EV_1 ≥ 2.0	N/A

19.3.11 Common Non-Conformances

Issue	Cause	Mitigation
Low density / high air voids	Under-compaction, too wet	Additional passes or drying
Laminated layer	Over-wet clay, excess roller energy	Scarify and re-compact
Density high but low EV_2	Poor interlock / segregation	Adjust grading; re-work
NDG <95 % but PLT OK	Local variability; dry patch	Cross-check with EV_2 trends

19.3.12 EC7 and QA Context

EC7 (§4.1.1 & §4.2.3) requires that **construction verification** confirms soil properties used in design. For fills, this means verifying:

- **γ (unit weight)** used in design matches achieved field density.
- **Stiffness (E, EV_2)** used in settlement or platform design is validated by test.
- **Water content** and **void ratio** remain within design assumptions.

Phicon Note:

• Always link the **Proctor reference test** to the specific compaction plant and material — not just "95 % MDD" in isolation.
• If site conditions differ from lab (coarser grading, weathering), run a **site trial compaction** and produce a fresh MDD–OMC curve.
• Control **moisture first, density second** — density will follow.
• MCV is an excellent rapid tool for cohesive fills; correlate it early with NDG or EV_2 results.
• Always plot a **trend of density vs moisture vs EV_2** — visual consistency reveals more than isolated pass/fail results.
• Document your assumptions (energy, layer thickness, plant) in the **GDR verification table** per EC7 §4.1.

19.4 Compaction Plant and Layer Control

This section provides practical guidance on selecting **compaction plant**, controlling **layer thickness**, and recording **field operations** to achieve consistent, verifiable compaction results. It ties laboratory compaction data (OMC–MDD) to **site implementation** and **Eurocode 7 verification requirements** (§4.1 & §4.2.3).

19.4.1 Purpose

- Match **plant type and energy** to the soil's grading and plasticity.
- Define **layer thickness** and **number of passes** needed for uniform density.
- Establish a **QA record** linking design assumptions to field execution.

19.4.2 Principles of Effective Compaction

1. **Correct moisture** – within OMC window.
2. **Sufficient energy** – achieved via correct plant and passes.
3. **Appropriate layer thickness** – allows full energy penetration.
4. **Progressive testing** – confirm compliance (MCV, NDG, EV_2).
5. **Continuous visual inspection** – surface condition, rutting, laminations.

If any of the above fail, compaction will be inconsistent regardless of effort.

19.4.3 Typical Compaction Plant by Material Type

Material	Recommended Plant	Passes (typical)	Comments / Notes
Granular fill (6F2, 6G, capping)	Vibratory smooth-drum roller (8–12 t)	6–10	Use full vibration; multiple frequencies for thick lifts
Cohesive clay (6N, 6S)	Sheepsfoot / padfoot roller (10–12 t)	4–8	Operate slightly damp of OMC; allow drying between lifts
Silts / mixed materials	Pneumatic-tyred roller (12–20 t)	6–8	Effective kneading and sealing; avoid excessive passes when wet
Crushed rock / recycled aggregate	Vibratory smooth-drum or tamping roller	6–10	Ensure no breakdown of aggregate; control fines

Chalk	Non-vibratory smooth-drum / pneumatic	6–8	Compact only at moisture below saturation (MCV > 8)
Backfill around structures	Hand-guided plate or trench roller	As required	Avoid overstressing structure; limit drop height
Working platforms	Vibratory roller (10–15 t)	As per design trial	Confirm EV_2 ≥ target (120–150 MPa)

Plant selection is critical: excessive vibration on cohesive or saturated soils causes **pumping** and **layer lamination**.

19.4.4 Layer Thickness and Energy Penetration

Material Type	Typical Loose Layer Thickness	Compacted Thickness	Notes
Granular (well-graded)	250–300 mm	200–250 mm	Decrease thickness if density < target
Cohesive (clay/silt)	200 mm	150–175 mm	Use thin layers to prevent trapped water
Crushed rock / 6F2	300–400 mm	250–325 mm	Adjust for aggregate size and roller type
Lime/cement treated	200 mm	150 mm	Compact immediately after mixing
Working platform	300 mm typical	250 mm	Prove by EV_2 test per 1000 m^2

Energy penetration depth ≈ **0.6 × drum diameter**; ensure total lift ≤ this value.

19.4.5 Field Compaction Control Workflow

Step 1 – Pre-start setup

- Review design compaction requirements (MDD/OMC or EV_2).
- Confirm plant availability and lift plan.
- Mark control lots (chainage, grid, layer).

Step 2 – Execution

- Compact in even passes across full width; stagger roller tracks.
- Maintain moisture by spraying or aerating as needed.
- Record plant, passes, and visible behaviour (heave, ruts, bounce).

Step 3 – Verification

- Test 1–2 points per 500 m^2 (NDG/MCV/EV_2).
- Compare with target value (≥95 % MDD or EV_2 ≥ 120 MPa).
- Rework and retest any failing lot.

Step 4 – Documentation

- Record in Compaction Log Sheet: date, lot no., material, OMC, w, density/MCV, passes, comments.
- Submit with daily Earthworks QA report.

19.4.6 Recognising Field Indicators

Observation	Likely Cause	Action
Roller "bounce" / slippage	Too dry / stiff	Add moisture or reduce passes
Shear planes / laminations visible	Too wet	Aerate, rework after drying
Surface pumping	Saturation or vibration on cohesive soil	Reduce amplitude or change plant
Segregation (coarse lenses)	Over-dumping, poor spreading	Remix, control drop height
Glossy surface / shear polish	Over-compaction	Scarify surface before next lift

19.4.7 Field QA Records (GDR Verification)

The Geotechnical Design Report (GDR) should demonstrate that compaction quality aligns with design assumptions.

Minimum documentation:

1. Compaction Log Sheets (daily, per lot)
   - Source & type of material
   - Layer thickness, no. of passes, moisture range
   - Plant type/weight/frequency
   - Test IDs and results
2. Summary Table of achieved densities / EV_2 / MCV.
3. Trial compaction certificate for each material type.
4. Photographic record (plant and layer surface condition).

Retain all records to demonstrate **EC7 verification** under clause §4.1(5): "execution shall be monitored to ensure design assumptions remain valid".

19.4.8 Compaction Trials (Recommended Practice)

Before mass placement, carry out **trial compaction area** (minimum 30 m × 10 m).
Test for:

- MDD achieved vs. OMC.
- EV_2 or density after each pass.
- Visual condition (no laminations or rutting).

Purpose: calibrate plant, passes, and moisture windows — results define the "approved method" for subsequent production work.

Trial Outcome	Use
Passes vs. EV_2 chart	Sets required number of passes
OMC vs. MCV correlation	Defines on-site acceptability window
Layer thickness performance	Confirms max lift per layer

19.4.9 Interaction with Other Elements

- **Adjacent structures:** avoid over-compaction near retaining walls or culverts; maintain minimum standoff (1.0 m typical).
- **Services & ducts:** use hand-guided plant in 150 mm lifts.
- **Backfill behind walls:** compact gently from structure outward; prevent arching.
- **Weather:** halt operations if standing water or frozen surfaces.
- **Slope formation:** compact from toe upward, maintaining drainage.

19.4.10 Typical Compaction Log (summary form)

Lot No.	Material	OMC (%)	Measured w (%)	Plant / Passes	Density / MCV / EV_2	Result	Remarks
EW-01	6N cohesive fill	17	18	Padfoot 10 t / 6 passes	MCV = 11	✓	OK – within OMC + 1 %
EW-02	6F2 capping	—	—	Vib. drum 12 t / 8 passes	EV_2 = 135 MPa	✓	Compaction uniform
EW-03	Recycled aggregate	—	—	Vib. drum 10 t / 10 passes	EV_2 = 115 MPa	⚠	Rework 1 pass, retest

Phicon Note:

• Select plant **based on material, not convenience** — a 12 t vibratory roller is not a universal solution.
• Verify **layer thickness** during trial — thick lifts hide non-compaction.
• Field inspectors should walk the layer between passes: feel for bounce, rutting, or pumping.
• Always maintain **drainage at formation**; compacting over ponded water wastes effort.
• Record **plant ID, weight, and passes** — these confirm construction consistency to EC7.
• Compaction QA is not about hitting 95 %; it's about repeatability, traceability, and confidence in performance.

19.5 Testing: NDG, PLT, CBR, EV_2

This section summarises the **key in-situ field tests** used to verify earthworks compaction and formation quality, their **interpretation under Eurocode 7**, and typical **target values** adopted in UK practice.

It links each test to the type of material, specification basis (Method or End-Product), and stage of work (formation, capping, working platform, etc.).

19.5.1 Purpose

To confirm that:

- The as-constructed fill meets the density, stiffness, or strength assumed in design;
- The ground conditions at formation are uniform and stable;
- The **compaction process** delivers repeatable performance.

Each test provides a **different performance proxy** — no single test suits all materials.

Select the method based on **soil type, particle size, and consequence of failure.**

19.5.2 Overview of Common Tests

Test	Measures	Used For	Soil Type	Typical Standard
NDG (Nuclear Density Gauge)	Dry density, moisture	Routine compaction control	All fine–medium soils	ASTM D6938 / BS EN 17892-12
Sand replacement	In-situ density	QA of cohesive fills	Fine / cohesive	BS 1377-9
MCV (Moisture Condition Value)	Workability proxy	Cohesive control	Clay / silt	TRL Report 1132
PLT (Plate Load Test)	Stiffness (EV_1, EV_2), bearing ratio	Formation, capping, working platforms	Granular / mixed	DIN 18134
LWD (Light Weight Deflectometer)	Equivalent EV_2	Rapid spot check / QA	Granular	ASTM E2835 / TRL
CBR (California Bearing Ratio)	Penetration resistance	Pavement subgrade / formation	Granular or cohesive	BS 1377-4 / BS EN 13286-47

19.5.3 Nuclear Density Gauge (NDG)

Purpose: Rapid field determination of in-situ **dry density** and **moisture**.

Principle: Gamma radiation attenuation correlates to soil density; neutron backscatter gives moisture.

Typical Use:

- Verification of %MDD (95–98 %) in compacted fill.
- Trending density consistency between lots.

Parameter	Symbol / Formula
Dry density	$\rho_d = \frac{\rho_{bulk}}{1+w}$
Compaction ratio	$\frac{\rho_{d(field)}}{\rho_{d(max)}} \times 100\%$

Acceptance:

- ≥ 95 % MDD for structural fills (granular/cohesive).
- ≥ 98 % for high-stiffness zones (working platforms).

QA Notes:

- Calibrate daily using standard block.
- Not reliable on materials with >40 mm aggregate or voided structure.
- For cohesive soils, cross-check with MCV or moisture readings.

19.5.4 Plate Load Test (PLT – DIN 18134 & EN1997-1)

Purpose: Determine surface stiffness (EV_1, EV_2) and check for non-progressive settlement. Used widely for **capping**, **formation**, and **working platforms**.

- Procedure Summary
- 300–600 mm diameter plate loaded via hydraulic jack in **two cycles**.
- Record pressure–settlement curve; derive:
    - **EV_1** (first load cycle),
    - **EV_2** (second cycle; reloading stiffness).
- Key Calculations

$$E_v = \frac{(1-\nu^2)\,\Delta\sigma}{\Delta\varepsilon} \text{ where } \Delta\sigma = \frac{\Delta P}{A},\ \Delta\varepsilon = \frac{\Delta s}{D}$$

and:

$$E_{V2}/E_{V1} \geq 2.0 \text{(densification criterion)}$$

EN 1997-2:2007, Annex K.2 gives the plate-settlement modulus as

$$E_{PLT} = \frac{\Delta p}{\Delta s}\ \frac{\pi\, b}{4}\ (1-\nu^2)$$

where b is the **plate diameter**, ν is **Poisson's ratio**, Δp is the load increment, and Δs is the corresponding settlement increment (secant modulus).

Target EV_2 (MPa)	Application
≥ 45	Subgrade (formation)
≥ 80	Selected fill (below capping)
≥ 120	Capping layer
≥ 150	Working platform (tracked plant)
≥ 180	Sub-base (pavement)

If **$EV_2/EV_1 < 2.0$**, material may still be deforming — require extra passes.

Advantages:

- Direct stiffness measure; repeatable.
- Immediate result.

Limitations:

- Slower than LWD; not suitable on cohesive, saturated ground.
- Plate size and seating affect consistency — use same setup each time.

19.5.5 Light Weight Deflectometer (LWD)

Portable dynamic version of PLT — applies repeated 10–15 kg falling weight.

Provides **E_{vd}** (equivalent dynamic modulus) correlated to EV_2.

Correlation (approx.)	Use
$EV_2 \approx 1.1–1.3 \times E_{vd}$	Quick check for capping / platform
E_{vd} target 100–120 MPa	~EV_2 120–150 MPa equivalent

Advantages:

- Rapid (2–3 min/test).
- Useful for 100 % coverage surveys.
- Lightweight and low-impact — ideal for granular platforms.

Limitations:

- Sensitive to surface roughness and seating; unsuitable for cohesive clay.
- Calibrate vs PLT at start of project.

19.5.6 California Bearing Ratio (CBR)

Purpose: Assess **bearing capacity** of subgrades or fills for pavement design.
Principle: Ratio of measured penetration resistance to that of a crushed stone reference.

$$CBR = \frac{P_{measured}}{P_{standard}} \times 100$$

Typical Target Values:

Application	CBR (%)
Subgrade (formation)	≥ 2–5
Selected fill	≥ 5–10
Capping	≥ 15
Sub-base	≥ 30

Quick Correlations:

- EV_2 ≈ 17.6 × CBR (approx., valid for 2–12 % range).
- EV_2 (MPa) / CBR (%) ≈ 15–20 typical ratio.
- $E = 10 \times CBR$(MPa) for low ranges (design approximation).

Advantages:

- Simple, direct strength measure; widely used for highway works.
- Laboratory test (remoulded samples) gives reference trend.

Limitations:

- Time-consuming; requires large sample; not sensitive for dense granular fills.
- Field CBR correlation via DCP (Dynamic Cone Penetrometer) preferred for rapid QA.

19.5.7 Moisture Condition Value (MCV) – Recap

Re-stated for context (see §19.3.6):

MCV ≥ 10 generally acceptable for compacted clay; MCV trend used to check fill consistency and control moisture.

Establish **project-specific calibration** between MCV, density, and EV_2 — improves QA correlation across tests.

19.5.8 Test Selection Matrix

Material / Application	Primary Test	Alternative / Support	Typical Frequency
Cohesive engineered fill	MCV, NDG	Sand replacement	1 per 500 m^3 or 500 m^2
Granular capping (6F2)	PLT (EV_2)	LWD	2 per 1000 m^2
Working platform	PLT / LWD	—	3–4 per 1000 m^2
Formation (final)	PLT or CBR	NDG	2 per 1000 m^2
Lime-treated	PLT + UCS cores	—	1 per 500 m^3

Recycled aggregate	PLT + NDG	LWD	2 per 1000 m^2

Increase frequency if test variability > ±10 % of target or if material changes.

19.5.9 EC7 Interpretation and Reporting

Under EC7 §4.1(5) and BS EN 16907-3:

- Field testing must **confirm design parameters** (γ, E, ϕ' proxies).
- Use **measured EV_2 or CBR** to check that serviceability (settlement) and bearing assumptions remain valid.
- If measured stiffness < design assumption → re-evaluate design (GEO/SLS).

Report in GDR or verification record:

Test ID	Location / Lot	Method	Result	Requirement	Status
PLT-12	EW-Capping 4+200	DIN 18134	EV_2 = 128 MPa	≥120	✓
NDG-07	EW-6N Layer 3	ASTM D6938	96 % MDD	≥95 %	✓
CBR-05	Formation 3+050	BS 1377-4	7.5 %	≥5 %	✓
LWD-04	Platform area	ASTM E2835	Evd = 110 MPa	≥100	✓

19.5.10 Correlation Reference (Quick Guide)

Parameter Pair	Approximate Correlation	Typical Use
EV_2 (MPa) ≈ 17.6 × CBR (%)	Road formation equivalence	Pavement design QA
EV_2 (MPa) ≈ 1.15 × Evd (MPa)	LWD to PLT calibration	Platform QA
E (MPa) ≈ 10 × CBR	SLS settlement modelling	FE / analytical design
EV_2 ≈ 100–150 × MCV	Indicative for cohesive fill	Site correlation trend

⚠️ (Correlations are indicative; confirm with project-specific calibration tests.)

Phicon Note:

• Choose the test based on **material behaviour** — NDG for uniform fine soils, PLT/LWD for granular stiffness, CBR for pavement bearing.
• Use **EV_2/EV_1 ratio** to assess compaction efficiency — <2.0 means more passes required.
• For cohesive fills, **MCV + NDG trend** is faster and more reliable than chasing a lab-based 95 % density.
• Always include **test locations on drawings** (chainage/grid) — traceability is key for EC7 verification.
• Avoid comparing unlike tests directly; instead, establish **on-site calibration curves** early in the project.
• Treat outliers as an opportunity to inspect workmanship — poor compaction rarely hides from visual evidence.

19.6 Material-Specific Controls (Chalk, PFA, Contaminated Fills)

Certain materials encountered on UK sites require **special handling and verification controls** due to their **sensitivity to moisture**, **chemical composition**, or **variable engineering behaviour**.

This section outlines practical guidance for dealing with **chalk**, **pulverised fuel ash (PFA)**, and **contaminated or marginal fills**, drawing from BS EN 16907, BS 6031, HA 207, BRE SD1, and CIRIA reports.

19.6.1 Overview

Material	Primary Risk	Control Approach
Chalk	Loss of strength on saturation or trafficking; crushing under compaction	Strict moisture control, thin layers, low energy
PFA / industrial by-products	Variable strength gain; leachate or sulphate attack	Conditioning, blending, chemical testing
Contaminated or Made Ground	Chemical or geotechnical unpredictability	Risk assessment, segregation, treatment or replacement

19.6.2 Chalk (UK-Specific Considerations)

Chalk is unique to UK earthworks and can behave anywhere between **hard limestone** and **weak clay** depending on grade, saturation, and compaction.

Design and acceptance follow **HA 207/07 (Design and preparation of chalk fill)** and **BS EN 16907-2**.

1. Chalk Material Grades (per CIRIA C574 / HA 207)

Grade	Description	Typical Moisture Sensitivity	Suitability for Fill
A	Hard, intact chalk, blocks up to 150 mm	Low	Excellent
B	Moderately strong, slightly weathered	Moderate	Good if dry
C	Weak / highly weathered	High	Conditional
D	Very weak, structureless, putty chalk	Very high	Unsuitable without drying/treatment

19.6.3 Key Controls for Chalk Fill

Aspect	Control / Limitation
Moisture condition	Compact only when **MCV ≥ 8** or **w ≤ OMC**; reject saturated chalk.
Compaction	Non-vibratory smooth-drum or pneumatic roller; **no vibratory compaction** on saturated chalk.
Layer thickness	≤ 300 mm loose (preferably 200 mm); ensure visible interlock between layers.
Trafficking	Prohibit heavy plant until layer proof-rolled and dry crust forms.
Testing	PLT or EV_2 after 24 h rest; $EV_2 \geq 80$ MPa typical.
Drainage	Provide lateral and underdrainage to prevent ponding or saturation.
Frost	Protect against freezing; chalk is frost-susceptible when saturated.

⚠️ Compact chalk *"as a rock, not as a soil"* — aim for mechanical interlock, not plastic deformation. Once the chalk begins to remould, its long-term stiffness drops sharply.

19.6.4 Pulverised Fuel Ash (PFA) and Similar By-Products

PFA (fly ash) and other industrial residues (e.g., **GGBS**, **bottom ash**, **slag**, **recycled fines**) can perform well if **chemically benign and properly conditioned**.

These materials fall under **BS EN 16907-4** (Materials Treatment and Reuse) and **SHW Series 600 / HA 151**.

19.6.5 Typical Behaviour

Type	Description	Engineering Notes
PFA (fly ash)	Fine, silt-like, pozzolanic when mixed with lime/cement	Low density (~1.1–1.4 Mg/m^3); gains strength on drying; self-hardens slowly
FBA (bottom ash)	Coarser, granular, inert	Drains well; suitable as subgrade/capping fill
GGBS / slag	Granular, variable hardness	Can expand if poorly aged or sulphide-bearing
Cement kiln dust / furnace fines	Variable chemistry	Require testing for sulphate, free lime

19.6.6 Key Controls for PFA and Industrial By-Products

Aspect	Control / Limitation
Moisture condition	Place between 15–25 % moisture (friable, not slurry).
Compaction	Light vibratory roller or pneumatic; avoid over-compaction (risk of crusting).
Lift thickness	200–300 mm loose (reduce if slurry-like).
Strength gain	Allow curing / drying time before trafficking (typically 24–72 h).
Chemical testing	Check pH, sulphate, chloride, loss on ignition (LOI), and leachate (BRE SD1).
Sulphate class	Limit to **< 0.3 % water-soluble** (WS) for near-concrete; total potential ≤ 1.9 %.
Durability	If pozzolanic, specify design life and UCS at 7 and 28 days.
Environmental	Verify against **EA Waste Recovery Plans**; ensure material qualifies as *non-waste*.

⚠️ Always trial compact and check moisture sensitivity — many ashes work best as **semi-dry fills**, but behaviour varies widely between sources.

19.6.7 Indicative Performance Targets

Material	Dry Density (Mg/m^3)	EV_2 (MPa)	MCV (target)	Remarks
PFA (conditioned)	1.3–1.5	60–100	9–12	Self-hardening; allow curing time
Bottom ash	1.5–1.8	80–130	—	Behaves like granular fill
Stabilised PFA (lime/cement)	1.5–1.7	100–150	—	High stiffness after 7 days
GGBS / slag	1.6–1.9	100–160	—	Check expansion and sulphate

19.6.8 Contaminated, Made Ground and Marginal Fills

Key Hazards

- **Chemical aggressivity** (sulphates, acids, organics, metals) → affects concrete and long-term durability.
- **Gas generation** (methane, CO_2, VOCs) → confined space / confined pressure risks.
- **Settlement** from heterogeneous composition.
- **Instability** under compaction or wetting (collapsible / putrescible layers).
- Risk Assessment Framework

Follow **BS 10175** (Investigation of potentially contaminated sites) and **CLR11 (EA Model Procedures)**.

Stage	Purpose	Output
Phase 1 – Desk & walkover	Identify potential contaminants	Preliminary Conceptual Model (PCM)
Phase 2 – Intrusive sampling	Quantify hazards (chem/geotechnical)	Risk assessment, reuse criteria
Phase 3 – Design & management	Define mitigation / re-use plan	Verification strategy

19.6.9 Re-use Controls (Geotechnical)

Criterion	Control / Test	Threshold / Guidance
Organic content	BS 1377-3 / LOI	≤ 2 % for structural fill
Sulphate (WS)	BS 1377-3 / BRE SD1	≤ 0.3 % (near RC)
Total Sulphur	BS 1377-3	< 1 % typical
pH	BS 1377-3	6–10 acceptable range
Asbestos	Visual / laboratory	Must be < 0.001 % (non-detectable)
CBR / EV_2	Field test	Meets zone requirement

⚠️ Any fill containing *made ground, ash, or demolition material* must have a Material Management Plan (MMP) under the CL:AIRE Definition of Waste Code of Practice (DoWCoP).

19.6.10 Treatment and Engineering Options

Issue	Treatment	Verification
High moisture / low strength	Lime/cement stabilisation	UCS ≥ 1 MPa (7d), EV_2 ≥ 80 MPa
Variable grading	Blending with granular material	Grading conformity (BS EN 933)
Chemical aggressivity	Capping / encapsulation	Sulphate & pH recheck
Gas generation	Venting layers / membranes	Gas monitoring (BS 8485)
Organics / topsoil	Screening / replacement	LOI retest

All treatments must be logged in the **Earthworks Verification Plan** and certified by a competent person (CGeol / CEng MICE).

19.6.11 Verification & QA

Stage	Verification Step	Typical Evidence Required
Source approval	Chemical / geotechnical testing	Certificates, lab reports
Placement control	Moisture / compaction QA	MCV, NDG, EV_2
Post-placement check	Strength / stiffness / drainage	PLT or UCS
Record keeping	MMP log / chainage plan	Batch references, test data

Phicon Note:

• Chalk should **never be compacted wet or vibrated** — moisture control and surface drainage are the keys to success.
• For PFA and ashes, **treat each batch as a new material** — variability between sources is huge. Always test before use.
• Use **lime or cement stabilisation** for wet ashes or marginal silty fills, but verify chemical compatibility (avoid excessive sulphate).
• All reuse of made ground or waste-derived materials must comply with **DoWCoP** and have a traceable testing record.
• In the GDR, **clearly distinguish engineered re-use** from **disposal or environmental management** — reviewers expect to see this separation under EC7 and CDM.
• Include a **contingency plan** in the QA schedule — e.g., alternative source if moisture, sulphate, or contamination thresholds are exceeded.

19.7 Quality Control vs Assurance Records

Earthworks verification under Eurocode 7 and the Specification for Highway Works (SHW Series 600) depends not only on meeting test targets, but on demonstrating that construction processes and results were properly controlled and recorded.

This section clarifies the distinction between **Quality Control (QC)** and **Quality Assurance (QA)**, describes their relationship to the Geotechnical Design Report (GDR), and presents a practical framework for documentation on UK infrastructure projects.

19.7.1 Purpose

To ensure that:

- The **constructed ground** performs as assumed in design (GEO/SLS).
- The design process and verification testing are traceable.
- The **client and designer** have confidence that works comply with standards.

19.7.2 QC vs QA – Distinction

Aspect	Quality Control (QC)	Quality Assurance (QA)
Focus	Operational — “Are we building it right?”	Systemic — “Do we have evidence it’s right?”
Responsibility	Site team, earthworks supervisor	Contractor, independent checker / designer
Timing	During construction	Throughout design–build–handover
Outputs	Test results, compaction logs, inspection records	Signed reports, verification certificates
Verification Level	Lot-by-lot	Project-wide system compliance

QC ensures the work is done correctly; QA ensures that **the proof is recorded and auditable**.

19.7.3 EC7 Context

Under **EN 1997-1 §4.1(5)** and **§4.2.3**, construction must be monitored and verified to confirm design assumptions.

The **UK National Annex (NA.4.2.1)** requires that the level of supervision be proportionate to the geotechnical category and risk.

Geotechnical Category	Verification Expectation
GC1 – Simple, low risk	Visual QA, minimal testing
GC2 – Normal structures	Full QC testing and QA certification
GC3 – Complex or high-risk	Independent checker; full audit trail, continuous monitoring

19.7.4 QC Testing – Routine Controls

QC testing provides **numerical confirmation** that the fill meets specification.

19.7.4.1 Typical components:

Test Type	Purpose	Typical Frequency
NDG / sand replacement	Density / compaction	1 per 500 m^3 or 500 m^2
MCV	Moisture condition (clay)	1 per 250 m^3

PLT / EV_2	Stiffness check	2–4 per 1000 m^2
CBR	Formation bearing	1 per 1000 m^2
Visual inspection	Layer thickness, bonding	Continuous
Material verification	Grading, moisture, contamination	Each source change

Results are plotted against specification criteria to demonstrate **trends** and detect **outliers** early.

19.7.5 QA Documentation – Assurance Records

QA documentation is the **structured evidence** that QC results were valid, complete, and approved. These records should be maintained daily and summarised at handover.

19.7.5.1 Essential QA Records:

Document Type	Purpose	Maintained By
Compaction Log Sheets	Record of material, plant, passes, results	Supervisor
Test Certificates	Traceable lab/field results	Testing agency
Inspection & Hold Point Records	Approval checkpoints before cover-up	Engineer / QA rep
Non-Conformance Reports (NCRs)	Document failures & remedial actions	QA manager
Verification Summary Sheet	Collated results per zone/layer	Contractor QA lead
GDR Verification Summary	Confirms design assumptions achieved	Designer / checker

19.7.6 The GDR Verification Table

The **Geotechnical Design Report (GDR)** should include a concise verification summary (see Appendix E) linking design values to achieved field results:

Design Parameter	Design Value	Verification Method	Measured Range	Conformity
γ (unit weight)	20 kN/m^3	NDG	19.2–20.4 kN/m^3	✓
EV_2	120 MPa (capping)	PLT	125–138 MPa	✓
ϕ' (equiv.)	35°	Material classification	Met	✓
w (moisture)	OMC ±2 %	MCV trend	Within range	✓

This table provides direct EC7 traceability — linking design inputs → field verification → compliance evidence.

19.7.7 Data Management and Traceability

All QC and QA data should be stored under a controlled, revision-tracked system (e.g. OneDrive / CDE). Each test record should have:

- **Unique ID** (chainage, grid, layer, lot no.)
- Date & operator
- Test type and standard
- Reference to material source / location
- Signature or digital approval

File naming convention (recommended):

- EW_[Section/Lot]_[TestType]_[Date]_RevX.pdf
- Example: EW_L3_PLT_2025-04-21_RevA.pdf

19.7.8 QA Review and Hold Points

Hold points provide control at **critical transitions** (e.g. completion of a formation layer or before structural element placement).

Stage	Hold Point	Release Criteria
Formation approved	Before capping	$EV_2 \geq 45$ MPa / CBR ≥ 5 %
Capping complete	Before sub-base	$EV_2 \geq 120$ MPa / visual inspection
Platform layer	Before piling/crane use	$EV_2 \geq 150$ MPa / $EV_2/EV_1 \geq 2.0$
Backfill	Before reinstatement	MCV ≥ 10 / 95 % MDD
Non-conformance	Before acceptance	NCR closed / retest passed

19.7.9 QA Hierarchy in Earthworks Projects

Level	Responsible Party	Typical Role
Designer	Defines required verification and design values	Specifies testing matrix
Contractor	Executes QC testing, records data	Performs field QA
Earthworks Supervisor	Monitors compliance, interprets results	Approves lot completion
Independent Checker (if GC3)	Audits QA system	Signs GDR verification
Client / Employer's Rep	Final acceptance	Witness checks

19.7.10 Common Non-Conformances and Responses

Non-Conformance	Root Cause	Corrective Action
Density < 95 % MDD	Over-wet, under-compacted	Rework layer; adjust moisture
EV_2 < target	Insufficient energy or weak material	Add passes; condition or replace
Incomplete test coverage	Missed area or weather restriction	Supplement with alternative test
Missing signatures	Admin gap	QA revalidation; record update
Wrong test reference	Mislabelled lot	Cross-check chainage; correct records

19.7.11 QA Audit Trail

For Cat 2 and Cat 3 works, maintain a **QA Audit Folder** containing:

- QA Plan and Inspection Test Plan (ITP)
- Daily site records and photo evidence
- Test summaries and certificates
- NCR log with closures
- Verification sign-off sheets

This folder forms the as-built evidence supporting the Final Completion Report or Geotechnical Completion Certificate (GCC).

Phicon Note:

• **QC tells you what happened — QA proves it was correct.**
• A perfect test result without a record is worthless in EC7 verification terms.
• Keep **QA records simple and templated** — one format across all layers avoids data gaps.
• Always link test data back to **design assumptions** in the GDR (e.g. “$EV_2 \geq 120$ MPa confirms bearing stiffness used in settlement check”).
• QA review should happen daily — not as a final audit. Small corrections early avoid rework later.
• Ensure that all QA documentation is **digital and traceable**; handwritten logs are acceptable only if scanned the same day.

19.8 Working Platforms for Tracked Plant (BRE 470)

Working platforms form the **interface between construction plant and the ground**, and are therefore **temporary geotechnical structures** that must be designed, verified, and maintained under Eurocode 7.

This section summarises the design intent, verification process, and QA/maintenance requirements for **pile rigs, cranes, and tracked plant** operating on granular platforms in accordance with **BRE Report 470**, **FPS guidance**, and **BS EN 16907-6**.

19.8.1 Purpose and Design Basis

Working platforms distribute plant loads safely to prevent:

- **Bearing failure** (punching or shear),
- Excessive rutting or settlement,
- Plant instability or overturning, and
- Damage to underlying formation or services.

Under **EC7 (EN 1997-1 §2.1, §9.2)**, a platform is treated as a *temporary geotechnical structure*, verified for **ULS (GEO/STR)** and **SLS (rutting, stiffness)** limit states.

- Design References
- BRE Report 470 (2004): Working Platforms for Tracked Plant
- FPS/TWF Working Platform Guidance (2022)
- BS EN 16907-6:2018 Earthworks – Landform and Construction Platforms
- **CDM 2015** – defines platform design and maintenance as a *safety-critical system*

19.8.2 Typical Platform Composition

Layer	Function	Typical Thickness
Surface layer	Bearing layer for plant; even load distribution	0.3–0.6 m
Capping / sub-base	Secondary load spread; drainage	0.3–0.5 m
Formation	Natural ground or engineered fill beneath platform	Variable
Geotextile (optional)	Separates platform from formation	As required

Materials: Typically well-graded crushed stone (6F1/6F2), recycled aggregates, or coarse granular Type 1; compacted to **EV_2 ≥ 120–150 MPa** and **EV_2/EV_1 ≥ 2.0**.

19.8.3 Platform Design Inputs

19.8.3.1 Key Design Parameters

Parameter	Symbol	Typical Value / Source
Plant load (track pressure)	q_p	From manufacturer (kPa)
Track length / width	L_t, B_t	—
Ground strength (formation)	c_u, φ', γ	Site investigation data
Platform material properties	γ, φ', E	Lab / empirical

Factor of safety (bearing)	$FOS_{ULS} \geq 2.0$	BRE 470 recommendation
Target deformation	≤ 25 mm	SLS comfort limit

19.8.3.2 Verification Checks (per BRE 470)

1. **Bearing capacity** (GEO):

$$q_{p,d} \leq \frac{q_{ult}}{\gamma_R}$$

Where $\gamma_R = 1.4$(typical) for temporary works.

2. **Settlement / rutting** (SLS):

$$s \leq s_{lim} = 25\text{–}50 \text{ mm (depending on rig type)}$$

3. **Edge stability:**
   Maintain ≥ 1.0 m lateral margin beyond track width.
4. **Slope stability:**
   Platform gradient ≤ 1:20 unless verified.

19.8.4 Platform Design Procedure (Summary)

Step	Description	Reference
1	Obtain rig/plant data (max track load, spacing)	FPS Rig Data Sheet
2	Define platform material and strength	Lab/field data, compaction specs
3	Calculate bearing capacity using Meyerhof/Terzaghi	BRE 470 Appendix A
4	Check for rutting and settlement under cyclic loads	Empirical stiffness approach
5	Add formation improvement or geogrid if required	BS EN 16907-4
6	Document design in Platform Certificate	FPS Form WP1

19.8.5 Platform Verification Testing

Verification ensures that as-built conditions meet design assumptions.

Field testing should confirm:

- EV_2 ≥ design stiffness target (usually 120–150 MPa).
- **EV_2/EV_1 ≥ 2.0** to demonstrate compaction efficiency.
- **Uniform thickness** and **moisture range** verified through inspection.

Test	Purpose	Typical Frequency
PLT (DIN 18134) or LWD	Stiffness verification	2–3 per 1000 m^2
NDG	Density check	1 per 500 m^2
MCV	Moisture condition	1 per 250 m^3
Level survey	Surface regularity	Each lift

If EV_2 < design target, carry out additional compaction or replace material.

19.8.6 Inspection and Certification (FPS / BRE Process)

Role	Responsibility
Platform Designer	Designs to BRE 470; issues **Certificate WP1**
Contractor / Installer	Builds and compacts platform per design
Rig Operator / User	Checks surface condition before use
Principal Contractor	Ensures inspection regime and maintenance
Temporary Works Coordinator (TWC)	Verifies certificates and site implementation

Documentation:

- **Working Platform Certificate (Form WP1)** — must be completed *before plant mobilises*.
- **Maintenance Plan** — defines frequency of visual checks and repair thresholds.

19.8.7 Wet-Weather and Degradation Management

Platforms are **sensitive to moisture and trafficking** — strength and stiffness can drop by >50 % when saturated.

19.8.7.1 Recommended Controls

Condition	Action
Prolonged rainfall / standing water	Cease operations; drain and re-compact
Soft spots / rutting observed	Mark area, excavate and replace
Pumping or loss of fines	Add capping or temporary geogrid
Surface contamination (spoil, slurry)	Scrape clean before further use
Frost / freeze-thaw	Restrict heavy plant; check stiffness post-thaw

⚠️ Daily inspection by Site Supervisor — minimum every **1000 m^2 or change of area**.

19.8.8 Platform Maintenance & Inspection Checklist

Item	Requirement	Pass/Fail Criteria
Surface level	±50 mm of design	✓ if within
Standing water	None visible	✓ if dry
Rutting depth	≤ 50 mm	✓ if < limit
Edge margin	≥ 1.0 m from rig track	✓ if maintained
Material degradation	None (no fines pumping)	✓ if intact
Certificates available	WP1 signed and filed	✓

Frequency:

- Daily by Supervisor;
- Weekly by Engineer/TWC;
- Re-inspection after heavy rain or plant change.

19.8.9 Example Target Values (Design & Verification)

Plant Type	Typical Track Pressure (kPa)	Recommended EV_2 (MPa)	Platform Thickness (m)
Crawler crane (≤80 t)	200–300	≥120	0.6–0.8
Rotary pile rig	300–400	≥150	0.8–1.0
Continuous flight auger (CFA)	400–500	≥180	1.0–1.2
Excavator / general plant	100–200	≥100	0.4–0.6

⚠️ (Values indicative — always verify against rig data and BRE 470 equations.)

19.8.10 QA and Record-Keeping Requirements

Record Type	Purpose	Retained By
WP1 Certificate	Confirms design & approval	Designer / Principal Contractor
As-built test results	Confirms platform meets stiffness spec	QA file
Inspection logs	Maintenance evidence	Site Supervisor

Photographs	Visual condition record	Daily record
Repairs log	Details of rework or replacements	Contractor
Removal record	Confirm safe disposal or reuse	Completion report

⚠️ All documentation should be incorporated into the **Geotechnical Design Report (GDR)** or **Earthworks Completion Certificate** for EC7 compliance.

Phicon Note:

• Treat every working platform as a **temporary geotechnical structure**, not "just hardcore."
• Always request or issue a **WP1 certificate** — failure to do so transfers liability to the site operator.
• **EV_2 and EV_2/EV_1** provide direct stiffness and densification checks — target $EV_2 \geq 150$ MPa for rigs or heavy tracked plant.
• After heavy rain, always **inspect, regrade, and retest** — degradation is rarely visible but highly consequential.
• Document inspection intervals and results — EC7 verification depends on proving design assumptions remained valid during use.
• Consider digital QR-linked inspection sheets or Power BI dashboards for traceable daily sign-offs.

20 Ground Improvement Techniques

Ground improvement methods enhance soil strength, stiffness, or drainage to meet design performance criteria **without full replacement**.

They are essential where natural ground cannot satisfy the **GEO/SLS limit states** required by Eurocode 7 and are widely applied beneath **embankments, foundations, working platforms, and slabs-on-grade**.

This chapter outlines method selection, design checks, and applicability guidance under **BS EN 16907-5**, **EC7 §11**, and **CIRIA C573 / C727**.

Ground improvement is a critically important component of geotechnical engineering. For this reason, including clear visual illustrations of the different methods is essential for understanding how the ground is being improved, whether it involves displacement or in-situ treatment. Visuals sourced from reputable ground-improvement contractors can also be used, provided proper attribution is given.

20.1 Selection and Applicability

20.1.1 Purpose of Ground Improvement

To improve one or more of the following:

- **Shear strength** → increase bearing or slope stability.
- **Stiffness** → reduce settlement or differential movement.
- **Permeability** → accelerate consolidation or reduce seepage.
- **Homogeneity** → mitigate variable or problematic ground.
- **Constructability** → enable plant access, reduce working platform thickness.

Ground improvement is classed as **temporary or permanent geotechnical work**, depending on whether the improved zone remains part of the final load path.

20.1.2 Eurocode Context

Under **EN 1997-1 §11.1**, ground improvement must be:

1. **Designed** using appropriate soil parameters (post-treatment).
2. **Verified** by field trials and control testing.
3. **Documented** in the **Geotechnical Design Report (GDR)**, identifying residual risks and inspection requirements.

Verification involves confirming:

$$R_d \geq E_d \text{ and } S_d \leq S_{lim}$$

where R_d= design resistance, E_d= design action, S_d= predicted settlement.

20.1.3 Method Categories

Category	Technique Type	Primary Mechanism	Typical Application
Mechanical	Dynamic compaction, vibro-replacement, preloading	Densification / rearrangement	Sands, gravels, loose fills

Hydraulic / Drainage	Prefabricated vertical drains (PVDs), vacuum consolidation	Pore pressure dissipation	Soft clays, silts
Chemical	Lime, cement, fly ash treatment	Pozzolanic bonding, strength gain	Silts, clays, weak fills
Grouting	Permeation, jet, compensation grouting	Void filling, strength gain	Sands, fissured clays, masonry
Reinforcement	Stone columns, geogrids, basal reinforcement	Load sharing, tension restraint	Soft ground under embankments
Thermal / Freezing	Artificial ground freezing	Temporary strength increase	Shafts, tunnels, cofferdams

20.1.4 Applicability Matrix

Ground Type	Limiting Problem	Feasible Techniques	Key Considerations
Soft clay (cu < 30 kPa)	Low strength, high compressibility	PVD + preload, stone columns, deep mixing	Allow consolidation time; check lateral stability
Loose sand	Low density	Vibro-compaction / replacement	Requires saturated condition
Silt / fine sand	Liquefaction risk	Densification, grouting	Sensitivity to vibration
Collapsible / fill	Void collapse	Dynamic compaction, replacement	Pre-wetting may be needed
Organic / peat	Very high compressibility	Replacement, deep mixing, PVD	Often uneconomic for thick deposits
Reclaimed or ash fill	Heterogeneity	Stone columns, cement stabilisation	Check sulphate and chemical reactivity
Karst / voided ground	Cavities	Compensation grouting	Structural monitoring essential

20.1.5 Selection Criteria

When selecting a technique, evaluate:

Criterion	Key Question
Geotechnical	What is the target shear strength, stiffness, and permeability?
Depth of influence	How deep must improvement extend?
Structural	What load or deformation limits apply?
Hydrogeological	Will improvement alter drainage or groundwater regime?
Construction / access	Can the method operate safely on available platform?
Environmental	Will chemical use or vibration cause impact?
Programme / cost	Is curing or surcharge time acceptable?

⚠️ Treat selection as an optimisation exercise — the best technique balances *improvement effectiveness, constructability, and QA verifiability*.

20.1.6 Verification & Control (EC7 §11.4)

Every improvement technique requires:

1. **Pre-construction trial** (field or pilot area).
2. Specification of performance criteria — e.g. cu ≥ target, EV_2 ≥ target.
3. **Monitoring regime** — pore pressures, settlement, strength gain.
4. **Acceptance testing** — post-treatment CPTs, vane, PLT, or UCS.

Improvement Objective	Typical Verification Parameter
Increase strength	cu (vane, triaxial), ϕ' (CPT correlation)
Reduce settlement	E (PLT, oedometer), EV_2
Accelerate consolidation	Pore pressure vs time (piezometers)
Improve drainage	Permeability (k), settlement rate curves
Stabilise fill	UCS, MCV, dry density

20.1.7 Performance Acceptance Criteria (Illustrative)

Parameter	Typical Target Value	Verification Method
cu (post-treatment)	≥ 50–80 kPa	Field vane / CPT
EV_2	≥ 80–120 MPa	PLT / LWD
Settlement	≤ 50 mm residual	Monitoring points
Permeability (k)	≤ 1×10^{-8} m/s (cut-off)	Falling-head test
Strength gain (lime/cement)	UCS ≥ 1 MPa (7d)	UCS cubes
Excess pore pressure	≤ 10 % of initial	Piezometer monitoring

20.1.8 Design and Risk Integration

Ground improvement introduces **residual uncertainty** due to soil variability and treatment efficiency. Per **EC7 §2.4.7.4(3)**, residual variability must be accounted for via:

- Partial factors on material properties (γM),
- Statistical validation of trial data,
- **Robust QA monitoring** throughout construction.

Typical risk mitigation:

- Run **CPT or plate test grid** post-installation.
- Implement **instrumentation** (settlement plates, piezos).
- Define **trigger levels** for performance review.
- Include contingency (e.g., increase treatment depth or add columns).

20.1.9 Example Design Framework (Simplified)

Step	Action	Output
1	Define performance target (bearing, settlement, strength)	Numerical criteria
2	Review ground model and variability	Parameter zones
3	Select improvement method	Preliminary design
4	Trial / calibration	Measured improvement data
5	Update design model	Verified properties
6	Implement QA testing regime	Construction control
7	Validate in GDR	Verification summary table

Phicon Note:

• Always classify improvement as **temporary** or **permanent** in the GDR — affects design approach and liability.
• Never assume target cu or EV_2 gains without **site-specific validation** — published correlations are indicative only.
• Specify **acceptance criteria and test frequency upfront**; "improve until good" is not defensible under EC7.
• Integrate **monitoring and observational design** for surcharge/PVD projects; settlement curves are the most powerful verification tool.
• Use **Eurocode 7 combination factors** (DA1-C1 or DA1-C2) consistently — improvement methods often form part of the GEO resistance chain.
• In reports, explicitly link the "improved" parameters to **source test evidence** — reviewers expect that traceability.

20.2 Mechanical Compaction & Vibro Replacement (Stone Columns)

Mechanical densification methods improve weak or compressible soils by **rearranging or replacing soil particles** to increase density, stiffness, and drainage.

Two key approaches used in UK and EU practice are **Dynamic / Vibro-Compaction** (densification) and **Vibro-Replacement** (stone columns).

This section summarises their principles, design checks, and verification procedures in line with **EN 1997-1 §11**, **BS EN 14731**, and **ICE Specification for Ground Treatment (2012)**.

20.2.1 Purpose

To improve bearing capacity, reduce settlement, and mitigate liquefaction or instability by:

- Increasing effective stress and density in granular soils, or
- Providing a **reinforced columnar system** (stone + soil matrix) in cohesive or mixed strata.

Both methods create a composite ground whose design parameters depend on the area replacement ratio, column stiffness, and improved soil strength.

20.2.2 Method Summary

Technique	Typical Soil Type	Mechanism	Equipment	Typical Depth
Dynamic Compaction	Granular fills, loose sand, reclamation	Repeated impact densification	10–20 t tamper dropped from 10–30 m	5–10 m
Vibro-Compaction (Vibroflotation)	Clean sands, gravels	Densification by vibration and saturation	Downhole vibrator with water flush	10–25 m

Vibro-Replacement (Stone Columns)	Soft clays, silts, mixed fills	Cavity creation + backfill with stone to form composite	Top-feed or bottom-feed vibrator	5–25 m

20.2.3 Vibro-Replacement – Mechanism of Improvement

Stone columns form a **reinforced soil matrix**, transferring load via:

1. **Vertical stress concentration** into the stiffer column, and
2. **Radial confinement** from surrounding soil (passive lateral resistance).

The improved ground behaves as a **homogeneous equivalent medium** with:

$$E_{eq} = E_s[(1 - a_s) + nE_c/E_s \cdot a_s]$$

where:

- E_{eq}= equivalent composite modulus,
- E_s= soil modulus,
- E_c= column modulus (stone),
- a_s= area replacement ratio,
- n = 1–1.2 (empirical adjustment factor).

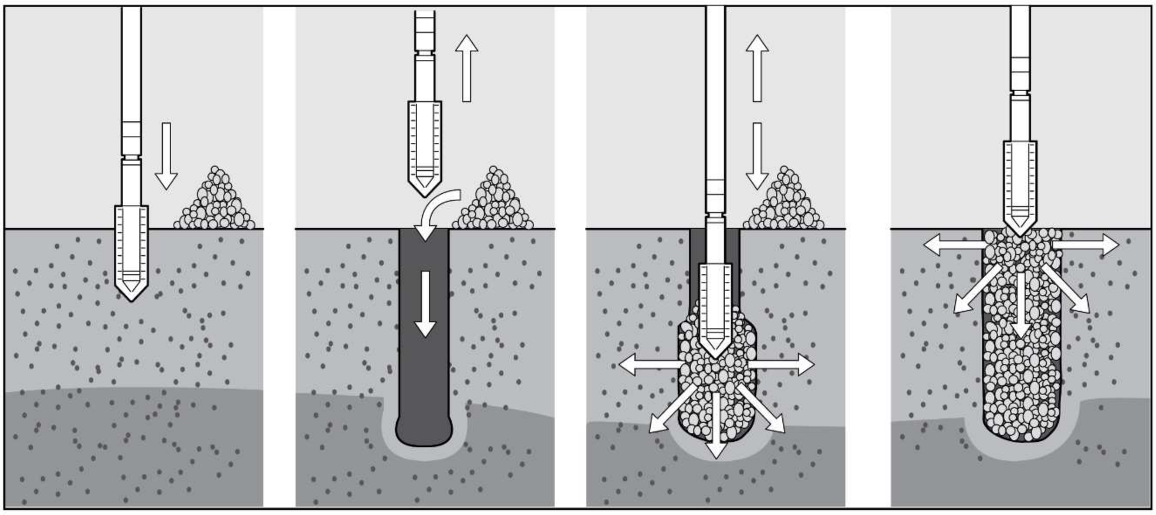


Figure 76: Process of installing vibro-replacement stone columns

20.2.4 Area Replacement Ratio and Spacing

Layout	Area Replacement Ratio (a_s)	Relation to Spacing (S) and Diameter (D)
Triangular grid	$a_s = 0.907(D/S)^2$	Most efficient load sharing
Square grid	$a_s = 0.785(D/S)^2$	Simplifies site setting out

Typical Values	Design Guidance
$\boldsymbol{a_s}$**= 0.10–0.25**	Normal range
$\boldsymbol{D}$**= 0.6–1.0 m**	Dependent on vibrator type
$\boldsymbol{S}$**= 1.8–3.0 m**	Based on design load and soil type

Phicon Note: 10–20 % area replacement provides ~2–3× increase in stiffness for soft cohesive soils.

20.2.5 Bearing Capacity and Settlement Checks

- (a) Bearing Capacity – EC7 GEO Limit State

The improved ground is checked using **equivalent composite strength** or **unit cell analysis** (Priebe method, 1995):

$$q_{u,imp} = q_{u,soil}(1 + \beta a_s)$$

where β= improvement factor (typically 3–5 for stone columns in soft clay).

Design bearing capacity (ULS):

$$q_d = \frac{q_{u,imp}}{\gamma_R}$$

where $\gamma_R = 1.4$(temporary) or 1.6(permanent).

- (b) Settlement Reduction – EC7 SLS

Settlement ratio:

$$\frac{s_{imp}}{s_{orig}} = \frac{1}{1 + m(E_c/E_s - 1)a_s}$$

where m≈ 0.8–1.0 depending on confinement.

Typical outcome:

- Settlement reduction: 50–70 %.
- Bearing capacity improvement: 2–4× natural soil.

- (c) Stability Check for Column Bulging

Columns may fail by **bulging** within 2–3 diameters below surface.
Check using **Hughes & Withers (1974)** method:

$$p_{lim} = 4c_u(1 + \ln \frac{r_m}{r_c})$$

Ensure applied stress ≤ p_{lim}.
If not, reduce spacing or increase column diameter.

20.2.6 Design Parameters

Parameter	Typical Range	Comment
Column diameter D	0.6–1.0 m	Determined from rig energy
Spacing S	1.8–3.0 m	Adjust for a_s
Column modulus E_c	40–80 MPa	From crushed stone gradation
Improved soil modulus E_{eq}	20–60 MPa	Composite stiffness
Undrained shear strength gain	2–4 × original cu	Typical after densification
Drainage improvement	Permeability ↑ by 10–100×	Accelerates consolidation

20.2.7 Quality Control and Verification

(a) Construction QA

- **Rig instrumentation:** current, power, depth, pull-out resistance.
- **Real-time log:** shows densification energy and backfill rate.
- **Column continuity:** confirmed via stone take (tonnage/m).

(b) Verification Testing

Test Type	Purpose	Acceptance / Target
CPT before/after	Assess improvement zone	qc increase ≥ 50 %
PLT / EV_2	Check stiffness	EV_2 ≥ 80–120 MPa
Plate load on unit cell	Load transfer	Settlement ratio ≤ 0.5 × untreated
Trial area (test cell)	Confirm design	Meets predicted stiffness

(c) Documentation

- Column layout and IDs.

- As-built depth and diameter.
- Installation energy and stone volume logs.
- Field test certificates and GDR summary.

20.2.8 Design Example – Indicative

Given	**Soft clay cu = 25 kPa, ϕ' = 0°, γ = 18 kN/m³**
Stone column	D = 0.8 m, S = 2.5 m (triangular grid, $a_s = 0.093$)
Improvement factor	β = 4 → $q_{u,imp} = 25 \times (1 + 4 \times 0.093) = 34.3 kPa$
Design bearing capacity (γR = 1.6)	$q_d = 34.3/1.6 = 21.4 kPa$(ULS)
Settlement reduction ratio (E_c/E_s = 10, a_s = 0.093)	$s_{imp}/s_{orig} \approx 1/(1 + 9 \times 0.093) \approx 0.55$
→ Settlement reduced by ~45 %.	

20.2.9 Practical Construction Notes

- Use **bottom-feed vibrator** where groundwater or contamination risk exists.
- Maintain constant **stone feed and withdrawal rate** to ensure continuous column.
- Ensure surface working platform EV_2 ≥ 120 MPa (see §19.8).
- Avoid overlap with adjacent columns; spacing ≥ 2D.
- Check **bulging and punching** using simplified Priebe or FE modelling for sensitive clays.

Phicon Note:

• Always verify the **"as-improved" parameters** by CPT or PLT — design must not rely solely on assumed β factors.
• Include **installation energy** as a QA metric — low energy or stone take signals underperformance.
• In soft clays, stone columns act primarily as **drains and reinforcement**, not as isolated load-carrying elements.
• Use **DA1–C1** combination for settlement and **DA1–C2** for bearing verification.
• Document improvement efficiency in the **GDR verification table** — reviewers expect to see pre- vs post-CPT comparison plots.
• Maintain **strict wet-weather control** — slurry intrusion during vibration severely weakens column performance.

20.3 Grouting (Permeation, Jet, Compensation)

Grouting techniques enhance or stabilise soil by **injecting fluid materials** into the ground to fill voids, bind particles, or create structural inclusions.

Depending on the method, grouting can provide **strength, stiffness, water cut-off, or settlement control**.

This section summarises grouting types, design principles, Eurocode 7 verification requirements, and practical QA measures per **EN 12715 (Execution of Grouting Works)**, **EN 12716 (Jet Grouting)**, and **CIRIA C514**.

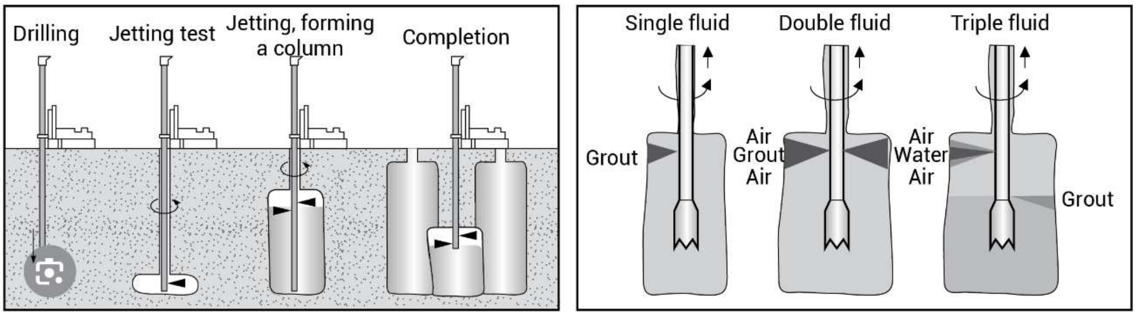


Figure 77: Process of jet grouting

20.3.1 Purpose

Grouting is applied to:

- Reduce permeability (cut-off or curtain works),
- Increase strength / stiffness (foundation improvement),
- Control deformation (compensation grouting beneath sensitive structures),
- Stabilise or fill voids (mining, tunnelling, karst), and
- Support underpinning (massive or jet-grouted columns).

Under **Eurocode 7 §11**, grouting is classified as a *ground improvement method* that requires design verification and monitoring to confirm performance during and after injection.

20.3.2 Classification of Grouting Methods

Type	Mechanism	Typical Soil Type	Typical Pressure	Product / Mix
Permeation grouting	Low-viscosity grout permeates pores	Sands, gravels	< 1 MPa	Cement suspension / silicate
Compaction grouting	Displaces soil; densification	Loose fills, gravels	0.5–2 MPa	Stiff mortar / grout
Fracture grouting	Creates fractures and fills them	Silts, clays	1–3 MPa	Bentonite-cement / resin
Jet grouting	High-energy erosion & mixing	Almost all soils	20–60 MPa jet pressure	Cementitious slurry
Compensation grouting	Controlled injection to counter settlement	Soft clays, sands under structures	0.5–2 MPa	Cement / microfine cement

20.3.3 Design Inputs and Parameters

Parameter	Symbol	Design Consideration
Grout type & rheology	—	Penetrability vs soil permeability
Injection pressure	p_i	Limited to prevent uplift or fracturing
Grout take	V_g	Volume/m^2 or per hole — indicator of success
Treated zone	D_t, S	Diameter & spacing of treatment bulbs
Strength gain	c', φ', E, q_u	Derived from core tests / plate tests
Permeability reduction	$k_{treated}$	Target ≤ 1×10^{-7} m/s for cut-off walls

20.3.4 Design Verification – Eurocode Context

Design must check:

1. ULS (GEO):

$$q_d \le R_d \text{where} R_d = f(c', \varphi', \gamma_R)$$

using treated strength parameters.

2. **SLS (Deformation):**
   Settlement ≤ limit for supported structure or overlying slab.
3. **Hydraulic UPL/HYD:**
   Verify pore-pressure reduction and uplift stability if used for cut-off.
4. **Observational Method:**
   Adjust injection pressure or layout based on real-time ground response (§2.7 EC7).

20.3.5 Permeation Grouting

20.3.5.1 Principle

Injects **low-viscosity grout** that flows through soil pores without disturbing structure — commonly microfine cement, sodium silicate, or polyurethane.

- Design Notes
- Applicable only when **D_{10} > 0.1 mm** and **k > 10^{-5} m/s**.
- Pressure should be less than **effective overburden × 0.5** to avoid fracturing.
- Verify by Lugeon / water-tightness tests before and after.
- Typical improvement:
  - Strength ↑ 20–100 %,
  - Permeability ↓ 1–2 orders of magnitude.

20.3.6 Jet Grouting

20.3.6.1 Mechanism

High-pressure jet (cement slurry 20–60 MPa) erodes and mixes soil to form **grout-soil columns** up to 3 m diameter.

Process Type	Description
Single-fluid	Cement slurry only (common in UK)
Double-fluid	Slurry + air shroud to increase cutting radius
Triple-fluid	Water + air + slurry for maximum reach (weak soils)

20.3.6.2 Design Parameters

Symbol	Typical Range	Comment

Column diameter D_j	0.8–3.0 m	Depends on jet energy & soil
Unconfined compressive strength q_u	1–5 MPa (7–28 d)	Test cores / cubes
Elastic modulus E	1000–2000 MPa	For settlement analysis
Overlap	15–25 %	Ensures continuity
Spacing S	0.7–1.5 m	Grid based on load demand

20.3.6.3 Checks

- Column continuity and geometry via **coring or borehole camera**.
- Verify target **strength (UCS ≥ 1 MPa)** and **diameter** via site trials.
- Apply **γ_R = 1.4–1.6** for temporary vs permanent use.

20.3.7 Compensation Grouting

20.3.7.1 Principle

Low-volume, high-control injections through tubes à manchettes to **counteract settlement** caused by tunnelling or excavation.
Used under existing structures to maintain serviceability ($\Delta s < 10$ mm typical).

20.3.7.2 Monitoring Integration

- Real-time optical or robotic monitoring (prisms / levelling).
- Trigger-response system:
    - **Amber:** movement > 50 % trigger → prepare grout.
    - **Red:** exceedance → inject until recovery.

20.3.7.3 Design Guidance

Parameter	Typical Value / Range
Injection pressure	0.3–2.0 MPa
Grout modulus	50–500 MPa (stiff cement)
Maximum lift per stage	≤ 2 mm
Tube spacing	2–3 m grid
Target permeability post-grout	≤ 10^{-6} m/s

20.3.8 QA / QC Requirements

20.3.8.1 Before Works

- Site investigation and permeability testing (Lugeon / falling-head).
- Grout design mix verified by **laboratory rheology test** (flow cone, viscosity, bleed).
- Establish **trial area or test column** to calibrate pressure, flow, and take.

20.3.8.2 During Works

Parameter Monitored	Purpose	Typical Instrumentation
Injection pressure	Prevent hydraulic fracturing	Pressure gauge / logger
Grout flow & take	Confirm penetration	Flowmeter
Ground heave	Detect uplift	Survey / inclinometer
Pore pressure	Confirm consolidation / safety	Piezometer
Volume vs design	Check efficiency	Automatic log

20.3.8.3 After Works

- Core samples or test pits (UCS, diameter, continuity).
- Water permeability test (falling-head).
- Record "as-built" map of treated zones.

20.3.9 Verification Parameters

Objective	Verification Test	Target / Acceptance
Strength	UCS (28 d)	≥ 1 MPa (general), ≥ 3 MPa for underpinning
Stiffness	PLT / EV_2	≥ 80–150 MPa
Permeability	Falling-head	≤ 10^{-7} m/s
Geometry	Core / borehole camera	Diameter ± 10 %, overlap confirmed
Uplift (SLS)	Levelling / monitoring	≤ design trigger (< 10 mm)

Acceptance must be recorded in the **GDR Verification Table** (see Appendix E) showing *design target vs achieved result*.

20.3.10 Common Design & Execution Issues

Issue	Cause	Mitigation
Excessive heave / fracturing	Injection pressure too high	Reduce pressure, monitor heave
Incomplete penetration	Grout too viscous / fine pores	Adjust mix, use microfine cement
Column necking (jet grout)	Variable withdrawal speed	Control rod speed and rotation
Variable strength	Poor mixing or segregation	Calibrate pump, homogenise slurry

Leakage or uplift	Lack of confinement	Stage injection, reduce rate

20.3.11 Environmental & Health Considerations

- **Cement grouts**: high pH > 12 — containment and PPE essential.
- **Silicate / resin grouts**: check COSHH and groundwater compatibility.
- Spoil disposal: classify under WM3 / EA EWC codes.
- **Groundwater monitoring**: post-works sampling if within Source Protection Zone (SPZ).

Phicon Note:

• Grouting is highly **observational** — real-time monitoring is part of design, not just QA.
• Always start with a **trial panel** to calibrate pressure, take, and radius — it defines realistic design parameters.
• In cohesive or silty soils, avoid assuming permeation; consider **fracture or jet grouting**.
• Keep a **grout take vs pressure log** — deviations > 20 % indicate blockages or leakage.
• EC7 requires that improved strength / stiffness values be **verified by test**, not assumed from literature.
• For compensation grouting, design must include **trigger levels and response plan** within the Geotechnical Control Plan.
• Post-treatment CPT or coring provides the most defensible verification evidence for Category 3 works.

20.4 Deep Soil Mixing (Lime, Cement)

Deep soil mixing (DSM) is a ground improvement technique that **mechanically blends in-situ soil with a stabilising binder**—typically lime, cement, or both—to create a cemented composite with higher strength, lower compressibility, and reduced permeability.

It is widely applied for **embankment foundations, slope stabilisation, excavation support, and treatment of soft clays or organic soils** in the UK and EU.

Design follows **EN 14679 (Execution of Special Geotechnical Works – Deep Mixing)** and **EN 1997-1 §11**, supported by **CIRIA C514** and **EuroSoilStab** guidelines.

Deep Soil Mixing (DSM) is a highly important and widely used ground-improvement technique, so providing sufficient detail on this method is essential. In this context, it is also valuable to reference the FHWA specification prepared specifically for deep soil mixing, as it sets out comprehensive design, construction, and quality-control requirements for DSM works.

https://www.fhwa.dot.gov/publications/research/infrastructure/structures/bridge/13046/001.cfm

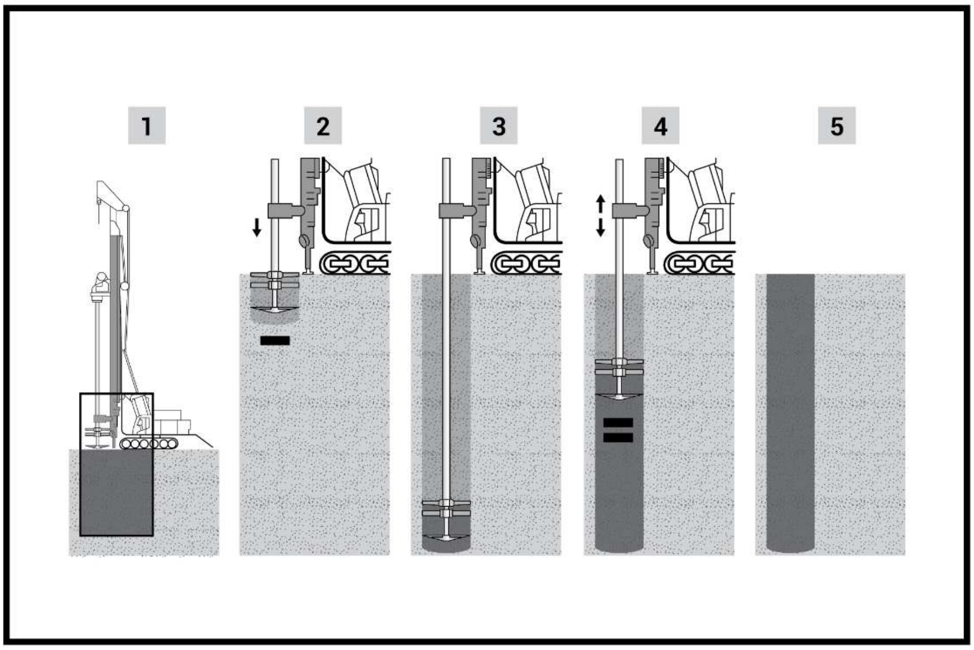


Figure 78: Process of deep soil mixing

20.4.1 Purpose and Mechanism

Soil mixing improves ground by:

- Creating **cylindrical or panel-shaped columns** of treated soil–binder mixture.
- Inducing **pozzolanic and cementitious reactions** that bond particles and reduce voids.
- Increasing undrained shear strength (c_u), stiffness (E), and bearing capacity, while decreasing permeability (k) and compressibility (m_v).

The improvement depends on:

- Binder type and dosage,
- Degree of mixing,
- In-situ water content and organic content,
- Curing time and temperature.

20.4.2 System Types

System	Mixing Principle	Typical Application
Wet mixing	Injects cement slurry during mixing	Soft to very soft clays, peats
Dry mixing	Blends dry binder (lime/cement) using air	Sensitive clays, shallow works
Mass mixing	Surface-level blending by excavator	Stabilising shallow fills
Panel mixing (massive DSM)	Creates large overlapping panels	Excavation support / cut-offs

20.4.3 Design Inputs

Parameter	Symbol	Typical Range / Notes
Column diameter	D_c	0.6–1.5 m
Column spacing	S	1.5–3.0 m (triangular or grid)
Binder content	B_c	100–300 kg/m^3 of soil
Water/binder ratio	w/b	0.8–1.5 (wet mix)
Strength at 28 days	q_u	0.5–2.0 MPa (soft clay), up to 5 MPa (lime-cement)
Elastic modulus	E	100–500 × cu (typically 50–300 MPa)
Target permeability	k	≤ 10^{-7} m/s

20.4.4 Design Process (Summary)

Step	Action	Output
1	Determine soil properties (c_u, w, organic %, pH)	Baseline ground model
2	Select binder type and dosage (laboratory mix trials)	UCS–binder relationship
3	Choose geometry (diameter, spacing, depth)	Layout and area ratio
4	Model improved zone (unit cell or FE)	Composite strength & stiffness
5	Check limit states (ULS, SLS)	Design verification
6	Define QA testing regime	UCS / coring / CPT checks

20.4.5 Strength Development

- Empirical Strength Correlation

$$q_u = k_b \times B_c^n$$

where:

- q_u= unconfined compressive strength (MPa),
- B_c= binder content (kg/m^3),
- k_b, n= empirical coefficients (from lab tests, typically 0.001–0.005 and 0.8–1.0).
- Undrained Shear Strength

$$c_u = \frac{q_u}{2}$$

Typical Strength Gain:

Time	Strength (%)
7 days	40–60 % of 28-day strength
28 days	100 % (design reference)
90 days	120–140 % (long-term increase)

Phicon Note: Always base design on **28-day laboratory-tested mix trials** from site-specific soils — not generic binder charts.

20.4.6 Design Verification Checks (EC7 §11)

- 1. Bearing Capacity (ULS – GEO)

$$q_d = \frac{q_{u,improved}}{\gamma_R}$$

with $\gamma_R = 1.4–1.6$(temporary vs permanent).
Verify that improved zone provides required **design bearing resistance**.

- 2. Settlement (SLS)

Settlement reduction ratio:

$$\frac{s_{DSM}}{s_{orig}} = \frac{1}{1 + (E_{col}/E_{soil} - 1)a_s}$$

where a_s = treated area ratio.
Typically, **settlement reduced by 60–80 %**.

- 3. Global Stability

Check potential slip circles passing through DSM zone using **enhanced cu**.
Include **interface shear strength** between treated and untreated soils (typically 0.5–0.7 × cu,DSM).

- 4. Uplift / Cut-Off (if applicable)

If used as barrier, verify permeability ≤ design limit.

20.4.7 Treated Zone Geometry

Application	Configuration	Typical Layout
Embankment foundation	Grouped columns	Triangular / grid, 1.5–3.0 m spacing
Excavation support	Overlapping panels	Continuous wall, 0.5–1.0 m overlap
Cut-off / containment	Line of DSM panels	Achieve k ≤ 10^{-7} m/s

Liquefaction mitigation	Grid of columns	Area ratio ≥ 20 %
Slope stabilisation	Rows of inclined columns	Designed for shear reinforcement

20.4.8 Area Ratio and Composite Strength

The **treated area ratio (a_s)** defines load sharing:

$$a_s = \frac{\pi D_c^2}{2\sqrt{3}S^2} \text{(triangular layout)}$$

Composite undrained strength:

$$c_{u,eq} = (1 - a_s)c_{u,soil} + a_s c_{u,DSM}$$

20.4.9 Quality Control & Verification

20.4.9.1 During Construction

Parameter	Control	QA Evidence
Mixing energy	Torque, depth, time per metre	Automatic rig log
Binder feed	Weight or volume vs design	Computer log
Mixing uniformity	Visual + sample inspection	Site record
Column geometry	Depth, overlap	As-built log and survey

20.4.9.2 After Construction

Test	Purpose	Target
UCS cubes (7d, 28d)	Strength development	≥ design value
Core samples	Verify column integrity	Diameter ±10 %, continuous
CPT	Strength profile	qc increase ≥ 2–3× untreated
Permeability test	Barrier verification	k ≤ 10^{-7} m/s
Trial panels	Calibration and validation	Confirms binder/strength correlation

Typical QA Testing Frequency:

- 1 core per 250 m^2 or 10–20 columns per batch, minimum one per day of production.

20.4.10 Common Design & Execution Issues

Issue	Cause	Mitigation
Weak or variable strength	Poor binder dispersion / incorrect dosage	Re-mix, adjust torque or rate
Column necking	Insufficient mixing time or withdrawal rate	Maintain overlap and tool speed
Excess water content	In-situ saturation too high	Pre-drying / adjust binder ratio
Organic interference	Inhibits pozzolanic reaction	Increase binder % or use cement–slag blend
Incomplete overlap	Rig deviation	Pre-drill guides, check alignment
Low pH soils	Poor cement reaction	Add lime pre-treatment

20.4.11 Environmental Considerations

- **pH and leachate**: Monitor groundwater; stabilised material can exceed pH 11–12.
- **Binder storage and dust**: Enclose silos; avoid discharge to surface drains.
- Spoil reuse: Verify treated spoil under DoWCoP / WRAP protocol.
- **Long-term durability**: Use blended binders (lime–GGBS) to resist sulphate attack.

20.4.12 Typical Design Values (Guidance)

Soil Type	Binder Mix (kg/m^3)	q_u (28d, MPa)	E (MPa)	k (m/s)
Soft clay (low organic)	150–200 lime–cement (1:1)	1.0–2.0	150–300	10^{-7}–10^{-8}
Organic clay / peat	250–300 cement–slag	0.5–1.0	80–150	10^{-8}–10^{-9}
Silt / alluvium	100–200 cement	2.0–3.0	250–400	10^{-7}
Hydraulic cut-off (clay barrier)	150–250 cement–bentonite	—	—	$\leq 10^{-7}$

20.4.13 Documentation Requirements

- **Design drawings** showing grid, depth, and binder dosage.
- **Rig logs** (binder feed, torque, mixing energy).
- UCS test certificates and curing records.
- **Coring reports / CPT data** confirming column geometry.
- **QA verification table** summarising design vs achieved properties (to be appended to GDR).

Phicon Note:

• Treat DSM as **engineered ground**, not simply treated soil — its behaviour depends on binder chemistry, moisture, and energy input.
• **Pre-trials are mandatory**; mix trials define binder–strength relationships.
• Always check **long-term durability** (sulphates, carbonation, leaching) for permanent structures.
• EC7 verification must show that the **design strength (cu, E)** derives from tested material, not literature.
• Maintain a **digital rig log** — it forms the most defensible QA evidence.
• When treating soft organic deposits, consider **hybrid improvement** (e.g., DSM + PVD) to combine strength and drainage benefits.
• Specify a **minimum 28-day curing before loading**, unless verified otherwise by lab correlation.

20.5 Drainage and Consolidation Methods (PVDs, Pre-loading)

Drainage and consolidation techniques are used to **accelerate the dissipation of excess pore pressure** in low-permeability soils—primarily **soft clays and silts**—to achieve settlement and strength gain before construction.

The most common systems in UK and EU practice are **Prefabricated Vertical Drains (PVDs)** combined with **surcharge or vacuum preloading**.

Design and verification follow **EN 15237 (Execution of Vertical Drainage Works)** and **EN 1997-1 §11**, incorporating the **Observational Method** (OM) for real-time performance control.

20.5.1 Purpose

- Reduce consolidation time from years to months.
- Increase effective stress and undrained strength (cu).
- Minimise post-construction settlement.
- Stabilise embankments or platforms on soft ground.
- **Enable early construction access** for piling, earthworks or slabs.

The process involves **installing drainage paths** through low-permeability clay to reduce the flow distance for pore water dissipation, while applying **surcharge loading** to induce controlled settlement and consolidation.

20.5.2 Mechanism of Improvement

- Primary Consolidation

$$t_{100} \propto \frac{H_d^2}{c_v}$$

where H_d= drainage path length (half layer thickness), c_v= coefficient of consolidation.

PVDs reduce $H_d$by providing **radial drainage** to vertical drains spaced across the site, drastically shortening consolidation time.

- Consolidation with Vertical Drains (Barron Theory, 1948)

$$U = 1 - \exp\left(-\frac{8T_v}{F(n)}\right)$$

where:

- U= degree of consolidation,
- $T_v = \frac{c_h t}{D_e^2}$,
- $F(n) = \ln(n) - 0.75$,
- $n = D_e/D_w$(ratio of influence diameter to drain diameter),
- c_h= horizontal coefficient of consolidation,
- t= time, D_e= equivalent drain influence radius.

20.5.3 Design Inputs

Parameter	Symbol	Typical Range / Notes
Drain spacing	S	1.0–2.5 m (triangular or square)
Drain width × thickness	$w_d \times t_d$	100 × 4 mm typical (band drains)
Equivalent drain radius	r_w	≈ 50–70 mm
Influence radius	r_e	$0.565S$(triangular), $0.564S$(square)
Coeff. of consolidation (horizontal)	c_h	0.5–2.0 × c_v
Compressibility	C_c, m_v	From oedometer test
Unit weight of fill	γ_{fill}	18–21 kN/m^3
Target settlement	s_t	Typically 80–90 % of ultimate
Degree of consolidation	U_t	90–95 % before main works

20.5.4 Design Procedure (Summary)

Step	Action	Output
1	Obtain consolidation parameters (c_v, c_h, m_v) from oedometer data	Soil input
2	Define surcharge load (Δσ') and thickness	Target effective stress
3	Select drain spacing and pattern	Design influence radius
4	Calculate consolidation time using Barron's theory	Time–settlement curve
5	Define target settlement (e.g. 90 %)	Time to completion
6	Integrate monitoring and trigger plan	Observational Method (EC7 §2.7)

20.5.5 Pre-loading Techniques

Technique	Description	Advantages	Considerations
Conventional surcharge	Temporary fill > final load	Simple, proven	Requires settlement waiting period
Vacuum preloading	Negative pressure applied via PVDs and membrane	No high embankment required	Requires tight membrane, pump system
Staged construction	Incremental filling with monitoring	Controls excess pore pressure	Requires instrumentation and trigger levels
Combined vacuum + surcharge	Hybrid method	Faster consolidation	Complex setup, careful monitoring

Phicon Note: Surcharge must always exceed operational load by 10–20 % to allow for post-construction rebound and creep.

20.5.6 Design Example (Indicative)

Given: Soft clay layer = 10 m; $c_h = 2 \times 10^{-7}$ m^2/s; spacing S = 1.5 m (triangular); desired U = 90 %

$$r_e = 0.565S = 0.848m,\ r_w = 0.05m,\ n = 17$$

$$F(n) = \ln(17) - 0.75 = 2.09$$

From Barron theory: $T_v = 0.197$for U = 90 %

Time for 90 % consolidation: $t = \frac{T_v D_e^2}{c_h} = \frac{0.197 \times 0.848^2}{2 \times 10^{-7}} \approx 7.1 \times 10^5 s = 8.2 days$

Consolidation time ≈ **1–2 weeks** versus **>6 months** without PVDs.

20.5.7 Settlement & Strength Gain Relationship

The undrained shear strength gain after surcharge is related to the increase in effective stress:

$$\frac{c_{u,new}}{c_{u,orig}} = \frac{\sigma'_{v,new}}{\sigma'_{v,orig}}$$

Typical **cu increase of 2–3×** observed after full consolidation.

20.5.8 Instrumentation & Monitoring

Monitoring is mandatory under EC7 for Observational Method verification.

Key instruments include:

Instrument	Parameter Measured	Purpose
Settlement plates / rods	Surface & subsurface settlement	Confirm consolidation progress
Piezometers (standpipe / VW)	Pore pressure dissipation	Degree of consolidation
Inclinometers	Lateral movement	Slope or embankment stability
Survey markers	Surface heave / tilt	Compensation monitoring

Trigger Levels:

- **Amber:** U < 70 % at 50 % programme → review rate / surcharge height.
- **Red:** Lateral movement > 50 mm or Δu > 10 kPa → pause loading.

20.5.9 Verification and Acceptance

Criterion	Verification Method	Acceptance Limit
Degree of consolidation (U)	Settlement–time back-analysis	≥ 90 %
Pore pressure	Piezometer data	Δu ≤ 10–20 % of initial
Strength gain	Field vane / CPT	cu ≥ 1.5–2 × original
Surface movement	Survey	Within design tolerance
Time to completion	Data vs design prediction	Within ±20 %

⚠️ All results must be documented in the **Consolidation Performance Report** and appended to the **GDR** for EC7 compliance.

20.5.10 Typical Design & Verification Values

Soil Type	c_v (m^2/s)	c_h/c_v	S (m)	Drain Length (m)	Consolidation Time (90 %)
Very soft clay	1×10^{-8}	2.0	1.0	10	2–3 months
Soft clay	5×10^{-8}	1.5	1.5	10–15	1–2 months
Silt / organic clay	2×10^{-7}	1.0	2.0	15	3–4 weeks

20.5.11 QA / QC Requirements

Activity	Requirement	QA Evidence

Drain spacing	≤ ±50 mm tolerance	As-built survey
Drain length	≥ design depth	Rig log (depth meter)
Filter integrity	No tear or clogging	Site inspection
Surcharge level	Maintain target fill height ±50 mm	Survey record
Vacuum system (if used)	Maintain ≥ 80 % vacuum	Pressure log
Instrumentation readings	Daily during surcharge	Field data sheet

20.5.12 Common Issues & Mitigation

Issue	Cause	Mitigation
Clogged or smeared drains	Excess filter pressure during driving	Use mandrel shoe and low driving rate
Non-vertical drains	Rig deflection	Use guides and maintain verticality
Excessive pore pressure	Over-rapid loading	Staged fill, real-time monitoring
Uneven settlement	Variable subsoil	Adjust surcharge height locally
Vacuum leakage	Membrane defects	Overlap + tape seal inspection

20.5.13 Environmental & Safety Considerations

- **Soft ground access** — use low-bearing working platform (EV_2 ≥ 100 MPa).
- **Flood risk / drainage** — ensure surcharge bund stability and safe water discharge.
- **Waste classification** — surcharge or PVD offcuts managed under **WM3 / EWC codes**.
- **Vacuum pumps** — acoustic shielding and oil management required.

Phicon Note:

• Always link **PVD spacing** to **required construction schedule** — economics are time-driven, not material-driven.
• Incorporate an **observational plan** — update predicted settlement curves weekly based on actual data.
• The **surcharge load** should be treated as a *temporary action* in EC7 verification ($\gamma Q = 1.0$ for SLS; $\gamma Q = 1.3$ for ULS).
• Strength gain must be validated by **post-treatment CPTs or field vane tests** before removing surcharge.
• For long-term cut-off barriers (vacuum + PVD), check **uplift and piping** under §17.3 design equations.
• Always log **drain IDs, depths, and dates** — traceability is mandatory for Category 3 projects.
• Combine consolidation monitoring with **Power BI dashboards or QR-linked site records** to improve transparency and EC7 verification traceability.

20.6 Stabilising Expansive / Collapsible Soils

Expansive and collapsible soils present significant **volume-change hazards** in geotechnical works, leading to foundation movement, cracking, or loss of bearing capacity.

In the UK, this includes **shrink–swell clays (e.g. London Clay, Gault Clay)** and **collapsible loessic or fill materials**.

This section outlines their identification, behaviour, treatment, and design verification in the context of **Eurocode 7 (EN 1997-1 §2.4.7, §11)** and **NHBC / BRE / TRRL guidance**.

20.6.1 Overview

Soil Type	Behaviour	Primary Risk Mechanism
Expansive (shrink–swell)	Volume change due to moisture variation	Heave and shrinkage under foundations
Collapsible (loessic / fill)	Structure collapse upon wetting	Sudden settlement on saturation

These materials require identification during investigation, mitigation in design, and monitoring in construction.

Both are governed by changes in suction and effective stress, not by conventional loading alone.

20.6.2 Identification and Classification

- Expansive Soils

Indicators of swell potential:

- **Plasticity Index (PI)** > 20 % → moderate, > 40 % → high.
- Shrink–swell classification (NHBC 2023):
  - *Low*: PI < 20,
  - *Medium*: 20–40,
  - *High*: 40–60,
  - Very High: > 60.
- **Free swell index (BS 1377-2:1990)** > 50 % = high expansion potential.
- **Mineralogy**: presence of montmorillonite or smectite.
- Collapsible Soils

Indicators of collapse susceptibility:

- **Open, loose structure**; low dry density (<1.5 Mg/m^3).
- Loess, windblown silt, uncompacted fill.
- Collapse potential (CP):

$$CP = \frac{e_1 - e_2}{1 + e_1} \times 100\%$$

where $e_1, e_2$are void ratios before/after wetting.

  - *Low*: < 2 %, *Moderate*: 2–6 %, *High*: > 6 %.

⚠️ Always confirm classification with **oedometer tests under inundation** — visual description alone is insufficient for EC7 verification.

20.6.3 Eurocode 7 Context

Expansive and collapsible soils fall under **"problematic ground"** (EN 1997-1 §2.4.7.4(3)), requiring:

1. Enhanced site investigation and testing.
2. Design assumptions verified through field or laboratory tests.
3. Application of **partial factors on material properties** (γM) reflecting uncertainty.
4. Documentation of residual risk in the **Geotechnical Design Report (GDR)**.

20.6.4 Design Considerations for Expansive Soils

Issue	Design Measure	Reference
Seasonal moisture changes	Maintain stable moisture regime	NHBC Ch. 4.2, BRE 240
Tree influence	Use foundation depth tables / root protection	NHBC 2023
Differential heave	Provide slip layers or void formers	TRRL 380
Floor slabs	Isolate from ground with void or compressible fill	CIRIA C583
Drainage	Direct water away from foundation zone	BS 8103

- Heave Estimation (BRE 240)

$$H = S \cdot I_{pt} \cdot V$$

where:

- H = potential heave (mm),
- S = surface shrinkage potential (mm/m),
- I_{pt} = plasticity index influence factor,
- V = volume change zone influence factor.

Typical potential heave range:

- Low plasticity: < 25 mm,
- *Medium*: 25–75 mm,
- *High*: 75–150 mm,

20.6.5 Design Considerations for Collapsible Soils

Issue	Design Measure	Comment
Wetting-induced collapse	Pre-wet or compact	Achieve >95 % MDD
Open structure	Dynamic compaction / vibro replacement	Reduce voids
Variable fill	Replace or treat locally	Confirm density & grading
Foundation type	Use rafts or stiffened strip	Spread differential movement

Water ingress	Cut-off drains, control infiltration	Prevent saturation
Load testing	Plate tests under wetting	Confirm residual deformation

20.6.6 Stabilisation Methods

(a) Lime or Cement Treatment

- Reduces plasticity and shrink–swell potential.
- Typical addition: 2–5 % lime for clayey soils.
- **Immediate reaction**: flocculation & cation exchange → reduced PI.
- **Long-term pozzolanic reaction**: increased strength & stiffness.
- Verify with Atterberg tests before and after treatment.

(b) Pre-wetting

- Applies controlled moisture to collapse or swell soil in advance.
- Suitable for **collapsible fills** — accelerates settlement before construction.
- Requires monitoring of volumetric strain and surface settlement.

(c) Compaction & Densification

- Dynamic compaction for loess or fills (energy 1000–3000 kN·m).
- Reduces void ratio and collapsibility.
- Verification by PLT or CPT confirming **EV_2 ≥ 80 MPa**.

(d) Replacement or Reinforcement

- Replace with granular fill (e.g. 6F1 / 6F2) to depth ≥ 1 m.
- Reinforce with **geogrids or basal reinforcement** to distribute load.

20.6.7 Laboratory & Field Verification

Parameter	Test	Purpose / Acceptance
Plasticity Index (PI)	BS 1377-2	PI < 35 for low reactivity
Free swell	BS 1377-2	< 40 % after lime stabilisation
Collapse potential	Oedometer wetting	CP < 2 % acceptable
CBR (4-day soak)	BS 1377-4	≥ 5 % for founding layer
EV_2	PLT (DIN 18134)	≥ 80–120 MPa (post-treatment)
pH	Chemical test	10–12 ensures active lime reaction

20.6.8 Construction and QA Measures

Stage	Control Activity	Target / QA Evidence
Pre-treatment	Sample and test for PI, moisture, organic content	Laboratory results
Mixing	Visual uniformity and binder rate check	Site inspection record
Compaction	Achieve ≥ 95 % MDD	NDG / density tests
Curing	Maintain moisture for 3–7 days	Site log
Verification	Post-treatment PI, CBR, EV_2	Field test certificates

20.6.9 Long-Term Maintenance Considerations

- Maintain **surface drainage** and prevent ponding around structures.
- Avoid planting **deep-rooted trees** near foundations on shrink–swell clay.
- Use impermeable pavements or membranes to stabilise moisture.
- Monitor **seasonal movement** via level points for long-term performance.
- Reassess design if water table changes significantly (e.g., due to adjacent developments).

20.6.10 Typical Design Values (Indicative)

Soil Type	Untreated cu (kPa)	After Stabilisation cu (kPa)	EV_2 (MPa)	PI Reduction
High PI clay (PI 50–60)	40	120–200	80–150	50–70 %
Medium PI clay (PI 35–50)	60	150–250	100–180	40–60 %
Collapsible fill	—	—	80–120	—
Lime-treated silt	30	100–150	60–100	—

20.6.11 Eurocode Verification Framework

Limit State	Design Objective	Verification Method
GEO (ULS)	Prevent bearing failure	Verify cu or ϕ' improvement (lab / field)
SLS	Limit heave or settlement	Check predicted vs allowable movement
UPL / HYD	Prevent moisture ingress and uplift	Drainage / cut-off checks
Durability	Maintain strength and stiffness	Periodic inspection (GDR residual risk table)

Phicon Note:

• Treat all high-PI soils as *expansive* until proven otherwise — classification must be supported by lab data.
• Stabilisation should **target PI < 35** or **EV_2 > 80 MPa** before acceptance as founding stratum.
• Avoid over-drying during lime mixing — shrink cracking reduces long-term durability.
• For fills, perform **collapse potential tests** before assuming suitability for re-use.
• Integrate **moisture control and drainage strategy** into foundation design drawings.
• EC7 verification must demonstrate that improvement parameters (cu, EV_2) are **derived from testing**, not generic data.
• Record **treatment boundaries, binder quantities, and PI reductions** — these form essential evidence for Cat 2/3 verification.
• Include long-term monitoring clauses in specifications — particularly for developments on shrink–swell clay or treated fill.

PART 5: CONSTRUCTION, MONITORING & VERIFICATION

21 Construction QA/QC & Supervision

Geotechnical construction quality management ensures that **design intent, safety, and performance criteria** established under Eurocode 7 are maintained during execution.

This chapter outlines the structure and documentation of **Quality Assurance (QA)** and **Quality Control (QC)** systems, focusing on the distinct responsibilities of the designer, contractor, and supervising engineer.

It references **EN 1997-1 §4 (Design Supervision)**, **BS EN ISO 9001**, **BS EN 16228 (drilling and piling)**, and **CIRIA C760** for UK practice.

21.1 Quality Systems for Geotechnical Works (QA vs QC)

21.1.1 Purpose of Quality Management

The aim is to:

- Ensure works are executed in accordance with design assumptions.
- Detect and rectify non-conformities early.
- Provide **traceable evidence** for Eurocode verification.
- Demonstrate compliance with **CDM 2015** duties for "permanent and temporary geotechnical structures."

Quality management for groundworks is both a **safety function** (avoiding instability or collapse) and a **commercial safeguard**, maintaining design accountability throughout construction.

21.1.2 QA vs QC – Distinction

Aspect	Quality Assurance (QA)	Quality Control (QC)
Definition	System of planned procedures to ensure quality requirements will be met.	Operational activities to confirm quality during production.
Objective	Prevent defects through management systems.	Detect and correct defects during execution.
Responsibility	Designer / Principal Contractor / TWC.	Site Engineer / Supervisor / Specialist Subcontractor.
Output	Inspection & Test Plans (ITPs), Method Statements, QA documentation.	Field tests, checklists, measurements, as-built records.

⚠️ QA is *process-focused*; QC is *result-focused*. Both are essential for EC7 compliance.

21.1.3 Eurocode 7 Requirements

EN 1997-1 §4.1(1)P:

"Adequate supervision and control shall be provided during execution to ensure the geotechnical works are constructed to meet the design requirements."

Key EC7 quality obligations:

1. **Verification of design assumptions** (material properties, geometry, groundwater).
2. Supervision category selection (Table 2.1, EC7).
3. **Execution class (EXC)** under EN 1990 / EN 1090 principles.
4. **Reporting and record-keeping** within the Geotechnical Design Report (GDR) and Geotechnical Control Plan (GCP).

21.1.4 QA Documentation Framework

Document Type	Purpose	Prepared By
Quality Plan / ITP	Defines inspection and test regime	Contractor (approved by Designer / TWC)
Method Statement (MS)	Describes construction sequence and controls	Specialist Contractor
Geotechnical Control Plan (GCP)	Integrates design verification, supervision, and testing	Designer / Supervisor
Checklists & Logs	Daily verification records	Site Engineer
Test Certificates	Confirms compliance with specification	Testing Laboratory
Non-Conformance Reports (NCRs)	Records deviations and corrective action	QA Manager
As-built Records	Final verified data	Principal Contractor / Engineer

21.1.5 QA Hierarchy in Temporary and Permanent Works

Role	Primary QA Function
Designer	Defines verification requirements and tolerances.
Principal Contractor (PC)	Implements QA/QC systems and holds overall responsibility.
Temporary Works Coordinator (TWC)	Reviews and approves method statements and certificates.
Site Supervisor (Contractor)	Executes inspections and records results.
Independent Checker (Cat II/III)	Verifies design and QA evidence for compliance.
Client / Principal Designer	Retains duty of ensuring risk management under CDM.

21.1.6 Supervision Categories (EC7 Table 2.1)

Category	Level of Control	Typical Application
1	Routine supervision	Simple shallow foundations, small retaining walls
2	Enhanced supervision	Piling, embedded walls, significant excavations
3	Continuous supervision	Deep basements, complex soil–structure interaction, high-risk slopes

⚠️ For Category 2 and 3 projects, supervision must be undertaken by **competent geotechnical engineers**, and field data must be logged for verification.

21.1.7 QA/QC Flow During Construction

1. Pre-Construction Stage
   - Review GDR, design drawings, and risk register.
   - Approve Method Statements and ITPs.
   - Calibrate and verify test equipment.
2. Execution Stage
   - Carry out inspections and record parameters (depths, pressures, densities).
   - Compare field data vs design assumptions.
   - Implement hold points for critical verifications.
3. Post-Construction Stage
   - Validate as-built against design.
   - Issue Completion Certificate.
   - Archive QA documentation in project data repository.

21.1.8 Hold Points and Witness Points

Type	Definition	Example (Geotechnical Work)
Hold Point (HP)	Work must stop until checked/approved.	Before concreting pile, after excavation to founding level.
Witness Point (WP)	Engineer may observe but work may continue.	Plate load test, soil mixing operation.
Review Point (RP)	Document review only.	Material delivery certificates, calibration records.

All HP/WP/RP must be **clearly listed in the ITP** and referenced in site QA records.

21.1.9 Key QA Testing Interfaces (Examples)

Geotechnical Element	Key Verification Tests	Typical Tolerances / Targets
Earthworks / Fill	Density, EV_2, MCV	≥ 95 % MDD, EV_2 ≥ 80–120 MPa
Shallow foundations	Bearing, formation level	Level ±25 mm, undisturbed surface
Piles	Concrete volume, depth, integrity	Depth ±50 mm, deviation ≤ 1:75
Retaining walls	Toe / heel level, reinforcement	±20 mm on reinforcement placement
Working platforms	EV_2, thickness	EV_2 ≥ 120–150 MPa, thickness ±25 mm
Anchors / nails	Proof load	1.25 × design load (ULS verification)

21.1.10 QA Records – Minimum Contents

Each activity record should contain:

1. Date and chainage/location.
2. Weather and surface condition.
3. Personnel and plant used.
4. Material batch / source.
5. Test results and calibration.
6. Inspector's name and signature.
7. Cross-reference to NCRs and corrective actions.

⚠️ All QA forms should carry unique IDs and QR-linked references for traceability — particularly when using digital inspection systems (e.g. Power BI or mobile site forms).

21.1.11 Digital QA Integration (Recommended Practice)

Digital QA systems (Power BI, FieldView, PlanRadar, or custom PHICON dashboards) can:

- Auto-link test results to chainage and date.
- Flag out-of-spec results automatically.
- Store calibration certificates and images.
- Export inspection summaries directly into the **GDR Verification Tables**.

Advantages include:

- Faster non-conformance tracking.
- Auditable EC7 verification trail.
- Consistent formatting across subcontractors.

21.1.12 Non-Conformance & Corrective Action Procedure

Step	Description
1	Identify and record non-conformance (NCR form)
2	Stop affected work (if critical)
3	Notify TWC and designer
4	Evaluate root cause
5	Define corrective action (rework, replacement, acceptance under concession)
6	Verify and close out (signature by QA Engineer)
7	Update QA register and lessons-learned log

⚠️ **Acceptance under concession** requires written approval from the designer and must be documented in the project QA register.

Phicon Note:

• Treat QA/QC as a **verification process**, not just record-keeping — every test confirms or refutes a design assumption.
• Use **structured ITPs** aligned with EC7 limit states: GEO, STR, EQU, HYD, and UPL.
• Require all subcontractors to submit **method-specific QA records** before mobilisation (e.g. Piling QA Sheets, DSM Logs).
• Retain QA evidence digitally — aim for "zero paper" workflow, with site logs linked to design IDs.
• Ensure **temporary works** (platforms, berms, anchors) receive the same QA rigour as permanent works.
• Record deviations clearly — an undocumented deviation is a liability.
• Embed QA summary tables in each design report and align format with PHICON verification structure for traceability across projects.

21.2 Piling Construction Verification (Bored, CFA, Driven)

Piling verification ensures that installed foundations achieve the **design geometry, material quality, and load performance** assumed in the design.

This process is critical under **Eurocode 7 (EN 1997-1 §7)** and associated execution standards:

- **BS EN 1536** – Bored piles
- **BS EN 12699** – Displacement piles (driven cast-in-place)
- **BS EN 14199** – Micro-piles
- **BS EN 1538** – Diaphragm walls
- ICE Specification for Piling and Embedded Retaining Walls (3rd Edition, 2017)

21.2.1 Purpose of Verification

To confirm that:

1. Each pile is installed within dimensional and positional tolerances.

2. **Concrete or grout integrity** meets strength and continuity requirements.
3. **Bearing layer and length** correspond to design assumptions.
4. Load performance is proven via **static or dynamic testing**.

⚠️ Under EC7, *as-built conformity* and *test evidence* are both verification tools — neither substitutes the other.

21.2.2 Typical QA Framework for Piling Works

Phase	QA Objective	Verification Output
Pre-construction	Confirm design intent and execution plan	Method statement, ITP, rig calibration, mix design approval
Execution	Monitor installation and materials	Daily logs, load tests, concrete/grout records
Post-construction	Verify pile integrity and capacity	Test reports, as-built drawings, completion certificate

21.2.3 Applicable Design and Execution Classes

Pile Type	EC7 Design Category	Execution Class (EN 1536)
CFA / rotary bored	Cat 2–3	EXC2–EXC3
Driven displacement	Cat 2	EXC2
Micro-pile	Cat 2–3	EXC2
Barrettes / large diameter	Cat 3	EXC3

21.2.4 Inspection and Test Plan (ITP) – Core Checks

Activity	Hold / Witness Point	QA Evidence	Typical Tolerance / Target
Set-out & position	HP	Survey record	±75 mm plan position
Drilling depth & diameter	WP	Rig depth meter log	±50 mm depth, ±20 mm diameter
Base cleaning (bored piles)	HP	Visual / slurry test	<100 mm sediment
Reinforcement cage	WP	Site inspection	Cover ±20 mm, cage centralised
Concrete / grout placement	HP	Cube / log	Strength ≥ fck; continuous pour

CFA pile monitoring	WP	Electronic rig log	Verify pressure, volume, extraction rate
Driven pile driving	WP	Blow count log	Compare to set criteria
Pile head trimming	RP	Inspection sheet	Trim to sound concrete
Load testing	HP	Test certificate	Meets design load with ≤ 10 mm creep (SLS)

21.2.5 Essential QA Records

Each pile installation should generate a **traceable digital record** containing:

1. Pile ID, type, location, date
2. Rig / operator
3. Diameter and depth
4. Concrete volume vs theoretical
5. Pressure or torque data
6. Cage details
7. Integrity and load test references

⚠️ Consider using Sharepoint Lists or Microsoft Forms to record and store these digital records.

Digital rig outputs (e.g., Soilmec / Bauer logs) should be archived in the **GDR Verification Appendix** for traceability.

21.2.6 Concrete and Grout Verification

Parameter	Test / Method	Acceptance Criteria
Cube strength	BS EN 12390	≥ fck (typically 35–50 MPa)
Slump / flow	BS EN 12350	150–200 mm (bored), 220 mm (CFA)
Volume delivered vs theoretical	—	±5 % tolerance
Grout density (micro-piles)	Flow cone test	1.8–2.0 Mg/m^3
Integrity	Sonic / cross-hole / thermal	Continuous to full depth

All cube results must be traceable to pile ID and pour batch.

21.2.7 Pile Integrity Testing

21.2.7.1 Types and Purpose

Test Type	Description	Typical Use
Low-strain (PIT)	Sonic echo using hammer impact	Quick integrity screen
Cross-hole Sonic Logging (CSL)	Ultrasonic pulse between tubes	Bored piles > 900 mm dia

Thermal Integrity Profiling (TIP)	Measures hydration heat	CFA or rotary bored piles
Gamma / CSL-CT	Advanced imaging	Research or high-risk projects

Acceptance:

- No major defects > 10 % of diameter or >1.0 m in length.
- Minor anomalies acceptable if below neutral axis and non-critical.

21.2.8 Load Testing and Verification

21.2.8.1 Static Load Tests (EN ISO 22477-1)

Performed to verify axial capacity and load–settlement behaviour.

Test Type	Purpose	Acceptance Criteria (typical)
Maintained Load (ML)	Verify working load behaviour	≤10 mm creep in final load cycle
Quick Load (CRP)	Define load–displacement curve	Settlement < 10 % D at failure
Tension / uplift	Check anchor / tie performance	1.1–1.25 × design load
Lateral test	Serviceability check	Δ < 10 mm at working load

EC7 allows reduction of pile group safety factor if individual test evidence confirms reliability.

21.2.8.2 Dynamic Load Tests (EN ISO 22477-4)

Used for driven piles and CFA verification:

- **PDA (Pile Driving Analyser)** captures strain & acceleration data.
- Must be calibrated against at least one static test (per EC7 §7.6.2).
- Acceptance: predicted static capacity within ±10–15 % of static test.

21.2.9 Tolerances (ICE Specification Reference)

Parameter	Tolerance
Plan position	±75 mm (≤600 mm dia), ±100 mm (>600 mm dia)
Verticality	1:75 (bored), 1:150 (CFA/driven)
Cut-off level	±25 mm
Pile length	+300 / −100 mm
Concrete cover	±20 mm
Reinforcement cage alignment	≤25 mm eccentricity

21.2.10 Non-Conformance and Remedial Action

Issue	Typical Cause	Corrective Action
Reduced length / refusal	Obstruction	Redesign or supplement pile
Excess concrete volume	Cavity / overbreak	Check base cleaning; sonic test
Integrity defect (neck / bulge)	Poor tremie control	Verify via PIT / CSL, repair if accessible
Pile deviation	Soft pockets / poor control	Adjust layout or apply group reduction
Inadequate test results	Material or QA failure	Retest or install replacement pile

All corrective measures must be **documented in NCR register** and **signed off by designer** before re-commencement.

21.2.11 Verification Testing Frequency (Indicative)

Test Type	Frequency (Typical)	Applicability
Static load test	1 per 250 piles (min 1 per type)	ULS verification
Dynamic load test	1 per 50–100 driven piles	Production monitoring
PIT integrity	100 % (for CFA / bored piles)	Integrity screening
Cross-hole / thermal	10–20 % of test piles	Large diameter bored piles
Cube strength	1 set per 25 m^3 concrete	Material verification

21.2.12 Record Deliverables

- Daily Pile Installation Log (per rig).
- As-built pile schedule and plan.
- Test reports (static/dynamic/integrity).
- Concrete cube strength certificates.
- Calibration certificates for rigs and sensors.
- Final Pile Verification Summary Table (design vs achieved).

All must be referenced in the Geotechnical Completion Report (GCR) or GDR Verification Appendix.

Phicon Note:

• QA is the *design check in action* — each pile log validates one line of the design spreadsheet.
• Ensure **rig telemetry data** (torque, pressure, extraction rate) are stored digitally — they provide the most reliable performance evidence.
• Reject piles on data gaps, not just on visual condition; missing logs = unverifiable work.

• Load test results should always be **benchmarked against EC7 partial factor design loads (DA1-C2)**.
• Always perform a **Category 2 / 3 verification review** before load test mobilisation.
• Integrate test results into the **PHICON QA dashboard** for traceability across chainage and date — EC7 verification is only defensible if datasets are cross-referenced.
• When pile defects are identified, issue NCRs promptly and document both the **engineering assessment** and **decision rationale** for acceptance or replacement.

21.3 Embedded Wall Construction Checks

Embedded retaining walls—**sheet pile, secant pile, or diaphragm**—form critical temporary or permanent geotechnical structures where wall performance depends on ground–structure interaction.

Their verification involves confirming that the **constructed geometry, materials, and groundwater controls** match the design assumptions set out under **Eurocode 7 (EN 1997-1 §9)**, **BS EN 1538 (Diaphragm walls)**, **BS EN 12063 (Sheet piles)**, and **ICE Specification for Piling and Embedded Retaining Walls (2017)**.

21.3.1 Purpose of Verification

To ensure:

1. The installed wall achieves the designed embedment, verticality, and continuity.
2. Materials (concrete, reinforcement, joints) meet specified strength and permeability requirements.
3. **Ground and water pressures** are controlled to prevent failure or excessive movement.
4. The wall behaves within EC7 **ULS (GEO/STR)** and **SLS (deformation, HYD)** limits during construction and service.

⚠️ Embedded walls must be verified as *constructed systems*—geometry, groundwater, and stiffness all form part of the resistance chain.

21.3.2 Common Wall Types and Execution Methods

Type	Execution Standard	Typical Application
Sheet pile wall	BS EN 12063	Temporary or permanent cuttings, cofferdams
Secant pile wall	BS EN 1536 / 1538	Deep basements, soft ground support
Diaphragm wall	BS EN 1538	Deep excavations, water-retaining cut-offs
Contiguous pile wall	ICE Specification	Temporary or drained permanent walls
Cutter soil mix wall (CSM)	EN 14679	Low-permeability cut-offs / retaining walls

21.3.3 QA/QC Sequence

Stage	Activity	QA Evidence / Record
1. Pre-construction	Verify design drawings, wall layout, and datum	Design review, survey setting-out sheet
2. Excavation / piling	Depth, verticality, overlap	Rig log, inclinometer data
3. Support / stability	Slurry density, viscosity, or casing control	Slurry tests (Marsh cone, density)
4. Reinforcement & joints	Bar placement, cages, joints	Inspection sheet, photographic record
5. Concreting / tremie	Continuous pour verification	Cube results, concrete log
6. Water tightness	Joint sealing / integrity tests	Permeability or pumping records
7. Instrumentation	Install inclinometers, piezometers	Installation report and calibration
8. As-built verification	Position, thickness, depth	Survey and record drawings

21.3.4 Key QA Parameters by Wall Type

Parameter	Sheet Pile	Secant / Diaphragm Wall
Plan position tolerance	±50 mm	±50 mm
Verticality tolerance	1:150	1:200 (up to 1:250 for guide-walled panels)
Toe / embedment depth	+300 / −100 mm	+300 / −100 mm
Overlap	≥ 150 mm	100–200 mm (secant)
Joint permeability	Watertight ($\leq 10^{-7}$ m/s)	Watertight ($\leq 10^{-7}$ m/s)
Concrete / grout strength	≥ fck (35–50 MPa)	≥ fck (40–50 MPa)
Reinforcement cover	±20 mm	±20 mm

21.3.5 Excavation Stability and Support Control

Slurry-Supported Walls (Diaphragm, Secant):

Parameter	Test	Target / Limit
Bentonite density	Mud balance	1.03–1.12 Mg/m^3
Viscosity	Marsh cone	32–50 s
Sand content	Sand test kit	≤ 4 %
Fluid loss	Filter press	≤ 30 ml/30 min
pH	Paper or probe	8–11

Slurry properties must be tested at least **every 4 h** of operation and before concreting.

Cased / CFA Walls:

- Maintain positive head of concrete or grout during extraction.
- Ensure minimum **overlap length** between primary and secondary piles is achieved.
- Record **torque, pressure, extraction rate** on rig telemetry (QA evidence for uniform pile body).

21.3.6 Reinforcement and Joint Verification

- **Reinforcement cages:** centralised, tied, and braced to prevent float during tremie pour.
- **Concrete spacers:** every 2–3 m vertical spacing.
- **Joints (diaphragm walls):** waterstops or hydrophilic strips at panel joints.
- **Secant walls:** confirm hard–soft or hard–hard sequencing achieved; record pile overlap depth.

QA Records: cage inspection sheet, bar schedule cross-check, photos of joints before concreting.

21.3.7 Concreting QA and Verification

Parameter	Check	Acceptance Criteria
Concrete continuity	Tremie pour log	Continuous to full depth
Concrete volume	Delivery vs theoretical	±5 % tolerance
Cube strength	BS EN 12390	≥ fck (28 d)
Bentonite contamination	Visual / flow test	None observed
Temperature (mass pours)	Thermal monitoring	<70 °C to avoid cracking

Discontinuity or loss of head during tremie pour must trigger **NCR + verification coring** before wall acceptance.

21.3.8 Groundwater and Watertightness Verification

If wall acts as a **cut-off** or **basement perimeter**, verify permeability post-construction via:

- Pumping or permeability test between wells.
- Standpipe response test (drawdown recovery).
- Inclinometer movement vs pore-pressure monitoring (HYD verification).

Acceptance limit: **$k \le 1\times10^{-7}$ m/s** or no measurable inflow at design head difference.

21.3.9 Instrumentation and Deformation Monitoring

Instrument	Purpose	Typical Trigger / Action
Inclinometers	Lateral deflection	25 mm (amber), 50 mm (red)
Piezometers	Groundwater pressure	±10 kPa vs baseline
Strain gauges	Bending moment verification	Check vs design envelope
Settlement markers	Adjacent structures	≤10 mm (SLS threshold)

Monitoring frequency:

- **Daily** during active excavation,
- **Weekly** post-excavation until final props / slabs cast.

21.3.10 QA Testing and Verification Summary

Test / Check	Standard / Reference	Frequency	Acceptance
Slurry density & viscosity	BS EN 1538 Annex B	Every batch / 4 h	Within limits
Cube strength	BS EN 12390	1 set / 25 m^3	≥ fck
Verticality (DGPS / inclinometer)	—	Each panel	≤ 1:200
Panel depth / overlap	Rig log	Every panel	Within ±100 mm
Permeability (if watertight)	BS EN ISO 22282	1 per 100 m	$k \le 10^{-7}$ m/s
Deformation monitoring	BS 5930 / EC7	As per plan	Within triggers

21.3.11 Non-Conformance and Corrective Actions

Issue	Cause	Action
Wall out of verticality	Rig drift, guide wall misalignment	Evaluate bending capacity; install compensating piles

Excessive overlap void	Poor sequencing / timing	Grout infill or secondary cut
Soft base / toe	Over-excavation, weak stratum	Excavate / clean; re-pour tremie or extend depth
Slurry contamination	Inadequate cleaning	Remove spoil, re-mix slurry
Leak through joint	Poor seal / displacement	Inject with resin / cement grout
Excessive wall movement	Unbalanced excavation sequence	Install props earlier, review stage design

All deviations must be recorded in **NCR log** and evaluated by the **Geotechnical Designer** before acceptance.

21.3.12 Documentation Deliverables

Deliverable	Purpose
Daily excavation and concreting logs	Confirm wall geometry and sequence
Slurry test sheets	Demonstrate stability compliance
Reinforcement and joint inspection reports	As-built verification
Concrete cube strength certificates	Material compliance
Inclinometer / piezometer logs	SLS verification
Final as-built drawings	Geometry and depth record
QA summary table (design vs achieved)	EC7 verification documentation

Phicon Note:

• Always record **panel-by-panel QA data**—embedded walls are verified by continuity, not just by spot tests.
• Digital rig telemetry and slurry logs are crucial evidence under **Design Approach 1, Combination 2 (DA1-C2)** verifications.
• Maintain **"live" inclinometer plots** linked to chainage and stage — deformation data must feed directly into Observational Method updates.
• EC7 requires proof that design assumptions (embedment, stiffness, permeability) remain valid — if construction alters any of these, issue a **Design Change Note** immediately.
• Ensure **temporary props / anchors** are tested and logged under the same QA framework as permanent elements (§21.4).
• For permanent basement walls, specify **post-construction permeability checks** and integrate results in the **GDR residual-risk register.**

21.4 Ground Anchor and Soil Nail Testing

Ground anchors and soil nails are structural–geotechnical elements used to provide **tensile resistance** for retaining walls, slopes, and excavations.

Their design and verification must confirm that the **bond length, load transfer, and lock-off capacity** meet EC7 limit state requirements for both **GEO (pull-out resistance)** and **STR (steel capacity)**.

This section aligns with **EN 1997-1 §8.5**, **BS EN 1537 (Ground Anchors)**, **BS EN 14490 (Soil Nailing)**, and the **ICE Specification for Piling & Embedded Retaining Walls (2017)**.

21.4.1 Purpose of Verification

To demonstrate that:

1. Anchors and nails are installed to design geometry and bond length.
2. Tendon materials and corrosion protection meet specification.
3. Test results confirm design load, proof load, and creep limits.
4. Construction performance matches assumptions used in the **GDR and EC7 verification**.

⚠️ Every tested anchor validates a design assumption — QA logs are treated as *design verification records*, not merely site checks.

21.4.2 System Overview

System Type	Primary Use	Bond Mechanism	Typical Load Range
Ground anchor (active)	Retaining walls, basements	Cement grout bond to soil/rock	300–1500 kN
Soil nail (passive)	Slope stabilisation, cuts	Frictional bond mobilised by deformation	50–300 kN
Micro-anchor / bar	Lightweight retaining or temporary works	Short bonded length, smaller load	50–150 kN
Self-drilling nail (Hollow bar)	Temporary support, fine soils	Combined drilling + grouting	50–200 kN

21.4.3 Eurocode Verification Framework

Limit State	Design Check	Primary Verification Method
GEO (Pull-out)	Bond resistance $R_{bond} \geq E_d/\gamma_R$	Proof / creep testing
STR (Tendon capacity)	$N_{design} \leq f_{yk}A_s/\gamma_M$	Factory / material certificates
SLS (Creep / movement)	ΔL ≤ allowable at working load	Acceptance testing
EQU	Stability against rotation / uplift	Inclination, head geometry checks

21.4.4 Types of Testing (BS EN 1537)

Test Type	Purpose	When Conducted	Typical Frequency
Suitability test	Confirms design and bond assumptions	Pre-production (trial anchors)	≥ 1 per 20–30 anchors
Acceptance test	Confirms installation quality	During works	100 % of permanent anchors
Proof test	Confirms working performance	For temporary works / nails	≥ 5 % of total
Creep test	Checks load-time behaviour	On selected anchors	≥ 1 per 10–20 anchors

21.4.5 Test Load Definitions

Symbol	Description	Typical Magnitude
$\boldsymbol{P_0}$	Lock-off load (service load)	0.6 × design load
$\boldsymbol{P_p}$	Proof load	1.25–1.5 × design load
$\boldsymbol{P_m}$	Maximum test load	1.5–1.7 × design load
$\boldsymbol{P_f}$	Failure load	Beyond design scope, used for suitability test only

5. Typical Load Application Sequence (Acceptance Test)
1. Apply incrementally: 0.25 P_p → 0.5 P_p → 0.75 P_p → P_p.
2. Hold each step for 1–2 minutes.
3. At proof load, maintain for 10 minutes.
4. Record load vs displacement (ΔL).
5. Reduce to lock-off P_0.

Acceptance:

- Load–extension curve stable (no load drop).
- Movement at proof ≤ 6 mm (temporary) or ≤ 2–3 mm (permanent).
- Residual movement after 10 min ≤ 1 mm.

21.4.6 Creep Testing

Performed on selected anchors to assess **long-term deformation under constant load**.

Stage	Load Level	Duration	Criterion
1	0.8 × P_p	10 min	—
2	1.0 × P_p	10 min	—
3	1.1 × P_p	60 min	ΔL ≤ 1 mm in final 10 min

Failure to meet criterion → review bond length or grouting quality.

21.4.7 Installation QA Controls

Parameter	Verification Method	Acceptance / Tolerance
Drill diameter	Measurement / rig log	±5 mm
Bore inclination	Survey	±2° from design
Bond length	Measurement	±100 mm
Grout quantity	Volume record	±10 % theoretical
Tendon length	Measurement	±50 mm
Free length sleeve	Inspection	Continuous, undamaged
Grout pressure	Gauge log	≤ 1.5 MPa (soil), ≤ 3.0 MPa (rock)
Grout density	Flow cone test	1.8–2.0 Mg/m^3
Head hardware	Torque / lock-off record	Within manufacturer limits

21.4.8 Soil Nail Verification (BS EN 14490)

Parameter	Typical Range / Acceptance
Nail length	±0.1 × design
Diameter	±5 mm
Grout strength	≥ fck 25 MPa
Pull-out load	≥ 1.5 × design (proof)
Displacement at proof	≤ 5 mm

Corrosion protection	Double corrosion barrier for permanent works

⚠️ **QA Evidence:** nail ID, installation depth, grout take, pull-out record, cube strength — appended to GDR Verification Table.

21.4.9 QA Documentation Requirements

Each anchor or nail must have a **unique record sheet** containing:

1. ID number and location (chainage, level, inclination).
2. Drilling log (depth, diameter, strata).
3. Grouting log (mix, pressure, take).
4. Bar / tendon details (material cert., batch no.).
5. Lock-off load and date.
6. Test results (proof / creep / suitability).
7. Inspector and witness signatures.

All data must be cross-referenced to:

- **Calibration certificates** (load cell, jack, gauge).
- Cube strength certificates.
- **Anchor test summary sheet** for design verification.

21.4.10 Testing Frequency (Indicative Guidance)

Structure Type	Suitability Tests	Acceptance / Proof Tests
Basement retaining wall	1 per 20 anchors	100 %
Slope stabilisation (soil nails)	1 per 50 nails	5–10 %
Temporary excavation	1 per 30 anchors	50 %
Permanent anchors (>500 kN)	1 per 10	100 % + 10 % creep tests

21.4.11 Common Non-Conformances and Mitigation

Issue	Possible Cause	Mitigation / Action
Low test load capacity	Incomplete bond, voids	Re-drill / re-grout, extend bond length
High creep movement	Weak bond or low grout strength	Replace or supplement anchor
Damaged corrosion protection	Handling / installation fault	Replace tendon
Excess grout pressure	Obstruction / too rapid pumping	Slow injection, relieve pressure

Anchor slip during lock-off	Poor wedge seating	Re-tension or reset
Test equipment out of calibration	Expired cert.	Recalibrate before retest

All non-conformances must be entered in the **NCR Register** with photographic and test evidence.

21.4.12 Acceptance Criteria Summary (Typical)

Parameter	Limit / Requirement	Reference
Max displacement at proof load	≤ 6 mm (temp), ≤ 3 mm (perm)	BS EN 1537
Creep movement	≤ 1 mm in final 10 min	BS EN 1537
Load hold loss	≤ 10 %	EN 1537 / ICE Spec
Lock-off load	0.6–0.7 × design load	Manufacturer guidance
Factor of safety on bond	≥ 2.0 (temp), ≥ 2.5 (perm)	EC7 DA1-C2 GEO
Corrosion system	Double barrier (perm)	BS EN 1537 Annex B

21.4.13 Reporting and Verification

Each test must have a **signed report** including:

- Load–displacement plot (log-linear scale).
- Summary of applied loads, hold times, and creep measurements.
- Pass/fail statement with reference to acceptance criteria.
- Commentary on anomalies and proposed actions.

All data are compiled in the **Anchor and Nail Verification Register** and cross-linked to:

- EC7 Limit State Table (GEO / STR).
- GDR Verification Summary.
- Residual risk section (e.g., potential for long-term creep).

Phicon Note:

• Treat every anchor test as a *data point in the design model* — record, plot, and trend analysis are key for verification.
• Always test the first anchors in each area to **calibrate bond parameters** before mass installation.
• Creep readings beyond limits often indicate **grout defects**, not soil issues.
• EC7 verification must document both **load capacity** (GEO) and **tendon strength** (STR); omitting either invalidates compliance.
• Implement a **digital anchor register** — unique QR-coded ID, linking installation, testing, and calibration records for traceable verification.
• Lock-off loads must be verified at ambient temperature; re-check after 24 hours for relaxation.
• Do not reduce testing frequency on temporary works — under EC7, verification is independent of service life.

21.5 Working Platform and Earthworks Verification

Working platforms and earthworks are temporary or permanent layers designed to safely distribute loads from plant, foundations, or structures into the ground.

Verification confirms that **thickness, compaction, and stiffness (EV_2, CBR, density)** meet design assumptions, ensuring safety and serviceability under **Eurocode 7 (EN 1997-1 §4.1 & §6.5.4)** and **BS EN 16907-6 (Earthworks – Field Testing)**.

UK guidance is provided in **BRE 470**, **ICE Specification for Piling & Embedded Walls**, and **HA 44/91**.

21.5.1 Purpose of Verification

To confirm that:

1. Platform or fill layers meet specified **bearing stiffness and compaction**.
2. The material type, thickness, and drainage correspond to design intent.
3. **Field testing results** provide objective evidence of compliance.
4. Risks of bearing failure, rutting, or excessive settlement are mitigated.

⚠️ Verification must confirm not only that the platform meets specification — but that it matches the assumptions used in EC7 limit state design (GEO/UPL/EQU).

21.5.2 Applicable Standards and References

Standard / Guidance	Scope
EN 1997-1:2004 + A1	Basis of geotechnical design (ULS/SLS verification)
BS EN 16907-6:2018	Earthworks – Field testing
BRE 470:2011	Working platforms for tracked plant
HA 44/91	Earthworks compaction control
BS 1377-4	Soil compaction and CBR
BS 1924	Stabilisation with lime/cement

21.5.3 Verification Stages

Stage	Objective	QA Evidence
Pre-construction	Confirm design assumptions (bearing, thickness, material class)	Platform design sheet / BRE 470 check
During construction	Monitor layer placement, thickness, compaction	Field test logs, inspection sheets
Post-construction	Verify achieved stiffness and compliance	PLT / LWD / NDG test certificates

21.5.4 Working Platform Design Overview (BRE 470)

The design bearing capacity or stiffness of a platform depends on:

- Subgrade strength (EV_2, CBR, or cu).
- Platform thickness and material (typically crushed granular or recycled).
- Load from plant or crane (kN/m^2).
- Factor of safety (1.5–2.0 for working platforms).

Verification must confirm:

- Platform material matches the **design input (γ, ϕ', grading)**.
- **Thickness** within ±25 mm of design.
- **Stiffness** not less than design EV_2 or CBR.

21.5.5 Field Test Methods

Test	Standard / Reference	Purpose	Key Output
Plate Load Test (PLT)	BS 1377-9 / DIN 18134	Bearing stiffness	EV_1, EV_2, EV_2/EV_1 ratio
Light Weight Deflectometer (LWD)	ASTM E2835	Rapid stiffness screening	EV_2 (approx.)
Nuclear Density Gauge (NDG)	ASTM D6938	Compaction control	%MDD, moisture
Sand Replacement / Core Cutter	BS 1377-9	Density verification	%MDD
CBR (soaked)	BS 1377-4	Bearing correlation	CBR (%)
Moisture Condition Value (MCV)	TRL 447	Workability and compaction	MCV value

21.5.6 Acceptance Criteria

Test Parameter	Typical Acceptance Value	Reference
EV_2 (Working platform)	≥ 120 MPa (tracked plant)	BRE 470
EV_2 (Road sub-base)	≥ 80 MPa	BS EN 16907-6
EV_2/EV_1 ratio	1.2–2.0	DIN 18134
CBR (%)	≥ 5–15 % depending on design	BS 1377
Dry density	≥ 95 % MDD	HA 44/91
Moisture content	±2 % OMC	HA 44/91
MCV	≥ 10 for compaction, ≥ 8 for acceptability	TRL 447

⚠️ Where platform performance is critical (e.g. piling rigs, cranes), **EV_2 ≥ 150–200 MPa** is typically targeted to provide redundancy against variable subgrade conditions.

21.5.7 Testing Frequency (Indicative)

Activity / Area	Recommended Frequency	Minimum No. of Tests
Platform area < 250 m^2	1 per 100 m^2	≥ 3 tests
Platform area > 250 m^2	1 per 250–400 m^2	≥ 6 tests
Earthworks / embankment	1 per 500 m^3	—
Working platform (piling)	1 per 250 m^2 or per rig pad	—
After rainfall or repair	Re-test affected zones	—

All testing must be referenced by **chainage, grid, and date**, and results plotted in **QA heat maps** for review.

21.5.8 Compaction and Layer Control

Parameter	Typical Range / Requirement	Verification Method
Layer thickness	≤ 300 mm compacted	Survey / spot check
Number of passes	4–6 (roller type dependent)	Site log
Material grading	Conforms to class (6F1 / 6F2 / 1A)	Lab PSD
Moisture control	±2 % OMC	NDG / MCV
Compaction plant	Registered and approved	Plant log

Edge compaction	Full coverage, no bridging	Visual / density test

21.5.9 Typical Test Correlations

Parameter	Approximate Correlation	Source / Note
CBR (%) = 0.07 × EV_2	for granular materials	Empirical UK correlation
EV_2 (MPa) = 17 × CBR	inverse relation	—
EV_2 / EV_1 ratio	Compaction quality indicator	<1.2 = under-compacted; >3 = soft subgrade
E' = EV_2 / (1 + v)	Modulus for FE models	Assume v = 0.35 for granular

21.5.10 Documentation and QA Records

Each test or inspection record should contain:

1. Test ID, date, chainage, coordinates.
2. Material type and layer designation.
3. Weather and surface condition.
4. Operator and equipment ID.
5. Raw test data and calculated parameters.
6. Pass/fail assessment and remarks.
7. Digital link to map or plan.

Field data should be logged directly into **digital QA dashboards** (Power BI, FieldView) for live visualisation of pass/fail status and area coverage.

21.5.11 Non-Conformance and Remediation

Issue	Possible Cause	Action / Mitigation
Low EV_2 or CBR	Inadequate compaction / wet surface	Recompact or dry layer
High EV_1/EV_2 ratio	Soft subgrade or lack of bedding	Replace soft material / add thickness
Excess moisture	Poor drainage / rainfall	Aerate, regrade surface
Variable test results	Material inconsistency	Segregate / reblend material
Differential settlement	Poor edge compaction	Reinstate with graded fill, compact in layers

Remedial areas must be **retested** and recorded as separate test IDs with updated QA maps.

21.5.12 Verification for EC7 Limit States

Limit State	Verification Focus	Typical Method
GEO (bearing)	Confirm EV_2 or CBR ≥ design	PLT / LWD
EQU (stability)	Level, edge restraint	Visual & survey
UPL (uplift)	Adequate drainage / pore control	Drain inspection
SLS (settlement)	Stiffness ≥ design E’	Plate load curve
DUR (durability)	Material quality, fines control	PSD / visual / certificate

21.5.13 As-Built and Verification Deliverables

Document	Purpose
Platform verification summary table	Confirms EV_2, thickness, compliance
Field test certificates (PLT / LWD / NDG)	Objective QA evidence
Test location plan	Visual coverage record
Material delivery and lab test records	Source verification
Platform inspection sheet	Visual QA / photographic log
Final as-built drawing	Design vs achieved geometry
Non-conformance register	Traceable record of corrective actions

All verification data are compiled into the **Geotechnical Completion Report (GCR)** or appended to the **GDR Verification Appendix**.

Phicon Note:

• Treat **working platform verification** as a *Category 2 check* under EC7 — inadequate stiffness or thickness invalidates all dependent design assumptions (e.g. pile verticality, rig stability).
• Require **minimum EV_2 targets** per platform type and maintain permanent digital traceability.
• Re-test platforms after **rainfall, reinstatement, or heavy trafficking** — surface softening is a latent risk not visible to the eye.
• Always cross-reference test results to **chainage, elevation, and date**; unreferenced data cannot be used in verification.
• Maintain a **QA coverage ratio ≥ 95 %** for critical areas — missing tests create unverified zones.
• When using recycled or stabilised materials, ensure **post-treatment tests (PI, EV_2)** demonstrate equivalent stiffness and durability.
• Integrate platform verification results into the **Observational Method feedback loop** if future excavation or piling will alter the platform structure.

21.6 Non-Conformance and Remediation Process

Non-conformance management is a formal process to identify, document, and resolve any deviation from **design intent, specification, or quality requirement** during geotechnical construction.

It ensures that corrective actions are **controlled, traceable, and approved by competent persons** so that Eurocode 7 verification remains valid at completion.

This process underpins **safety, accountability, and audit readiness** across both temporary and permanent works.

21.6.1 Purpose

To provide a structured method to:

1. Detect and record deviations early.
2. Prevent unverified works from proceeding.
3. Evaluate the engineering significance of each deviation.
4. Implement approved corrective or concessionary actions.
5. Maintain a verifiable audit trail for EC7 limit-state compliance.

⚠️ Under EC7, non-conformance control is a *design verification safeguard*—not an administrative formality.

21.6.2 Definitions

Term	Meaning
Non-Conformance (NCR)	Any work, material, or test result not meeting specification, drawing, or design assumption.
Corrective Action	Work performed to restore compliance (e.g. re-test, re-work, replacement).
Preventive Action	Steps taken to prevent recurrence (e.g. training, procedural update).
Concession	Formal acceptance of deviation that does not compromise safety or serviceability, authorised by the Designer.
Observation	Minor deviation not affecting function, logged for monitoring.

21.6.3 EC7 Context

EN 1997-1 §4.1 (1)P:

"Adequate supervision and control shall be provided during execution to ensure that the geotechnical works are constructed to meet the design requirements."

Non-conformance control provides the documentation that this requirement has been met.

Each NCR serves as part of the **ULS/SLS verification evidence chain**, confirming that design assumptions have been either maintained or re-validated after correction.

21.6.4 Escalation Hierarchy

Role	Responsibility in NCR Process
Site Engineer / Supervisor	Identify and raise NCR immediately when defect or deviation is detected.
Temporary Works Supervisor (TWS)	Verify classification and recommend containment or suspension of affected works.
Temporary Works Coordinator (TWC)	Assess engineering significance, decide whether to escalate to Designer.
Designer / Geotechnical Engineer	Evaluate structural or geotechnical impact; issue corrective or concession decision.
QA Manager / Principal Contractor	Maintain NCR register, close-out tracking, and audit trail.
Client / Independent Checker	Review for major non-conformances or residual risk entries in GDR.

21.6.5 NCR Workflow

1. **Detection:** Field or lab identifies deviation (test fail, wrong material, dimension error).
2. **Containment:** Suspend or isolate affected area until assessment complete.
3. **Notification:** NCR issued with description, location, date, responsible person.
4. **Investigation:** Determine cause (design, workmanship, materials, testing).
5. **Disposition:** Designer classifies as *Rework*, *Use-As-Is*, or *Concession*.
6. **Corrective Action:** Implement remedial works or retesting.
7. **Verification:** Check that correction achieves specification.
8. **Closure:** All signatures obtained; NCR filed in QA register and linked to GDR.

⚠️ No NCR should be closed without quantitative verification (test, survey, or designer confirmation).

21.6.6 Classification of Non-Conformance

Category	Definition	Typical Examples	Action Level
Critical	Potential safety or stability risk	Pile defect, wall instability, inadequate bearing	Immediate suspension, designer review
Major	Affects performance or serviceability	EV_2 below spec, out-of-tolerance geometry	Designer concession / rework
Minor	No safety impact, aesthetic or procedural	Late test log, missing ID tag	QA closure only

Observation	Potential trend requiring watch	Slight surface rutting, moisture deviation	Monitor and note in QA record

21.6.7 Example NCR Record Content

Field	Example Entry
Project / Activity	Piling Platform EV_2 Verification
NCR No.	PHC-NCR-021-2025
Date / Time	14 Mar 2025 – 10:30
Location	Grid E5–E7
Description	2 of 6 PLTs < 120 MPa; surface softened by rain
Raised By	Site Engineer – J. Lee
Containment Action	Area cordoned, no rig movement
Root Cause	Inadequate surface drainage
Corrective Action	Regrade + recompact, retest
Verification	EV_2 = 165 MPa (PASS) 15 Mar 2025
Designer Decision	Accepted – Rework successful
Closed By / Date	QA Manager / 16 Mar 2025

21.6.8 Common Geotechnical NCR Scenarios

Category	Typical Example	Immediate Action
Piling	Incomplete depth, high concrete take	Suspend rig, notify Designer
Earthworks	Compaction below 95 % MDD	Re-compact, retest
Working Platform	EV_2 < 120 MPa	Reinstate layer, retest
Anchors / Nails	Proof load fail or high creep	Hold subsequent installations
Retaining Walls	Toe level out of tolerance	Design review for stability
Instrumentation	Inclinometer out of calibration	Re-survey baseline
Drainage / HYD	Blocked relief pipe	Clear and re-verify

21.6.9 Corrective Action Hierarchy

Option	When Used	Verification Required
Rework / Re-test	When defect can be rectified	Repeat original test or equivalent
Replacement	When repair impossible or uneconomic	Full new QA record
Concession	When deviation is minor and safe	Designer approval with note in GDR
Use-As-Is	Only if deviation immaterial	Engineer sign-off and risk note
Preventive Action	To avoid recurrence	Update ITP, training, toolbox talk

21.6.10 Documentation and Traceability

Every NCR shall:

1. Have a **unique identifier** (e.g. PHC-NCR-###).
2. Be logged in the **QA Register** with status (Open / Closed / Concession).
3. Cross-reference relevant test certificates, photos, drawings, or survey data.
4. Be digitally stored and indexed by **chainage / element / date** for future audits.
5. Feed into the **GDR Residual Risk Register**, if not fully eliminated.

21.6.11 Designer Involvement and Verification

The Designer must:

- Review NCRs classified as *Critical* or *Major*.
- Assess structural or geotechnical implications (e.g. reduction in factor of safety).
- Decide if revised design calculation or re-analysis is required.
- Document rationale in the **Design Change Note (DCN)**.
- Confirm closure through written approval (email or NCR form).

21.6.12 QA Audit and Close-Out

Before project completion:

1. Verify that **all NCRs are closed** or transferred to residual risk.
2. Compile **summary table** (ID / Status / Action / Verification) within QA report.
3. Include NCR metrics (count, type, closure rate) in project KPI dashboard.
4. Archive in **digital QA folder** aligned with the EC7 verification hierarchy.

Closed NCRs constitute evidence of *effective supervision* and are often reviewed during Cat III checks or external audits.

21.6.13 Integration with Digital QA Systems

Phicon recommends all NCRs be logged in **Power BI / SharePoint integrated forms**:

- Auto-assign sequential IDs.
- Embed photo, test, and map location metadata.
- Link directly to EV_2, CBR, PLT, or anchor test tables.

- Generate live dashboards (open vs closed NCRs).

Benefits:

- Instant visibility for TWC and Designer.
- Reduced administrative backlog.
- Permanent digital record for EC7 verification.

Phicon Note:

• Non-conformance tracking is part of *limit-state verification*: each deviation must be resolved or explicitly accepted under engineering judgement.
• Always apply the rule: **"no record, no compliance."**
• Never classify a geotechnical defect as "minor" without quantitative evidence—small geometry errors can undermine stability.
• Record *both* the technical cause and the management response; the learning loop prevents recurrence.
• Integrate NCR data into monthly QA summaries and project dashboards.
• All concessions must be logged in the **GDR Residual Risk Table**—these are auditable and must be defensible.
• A closed NCR with full evidence trail demonstrates compliance, accountability, and professional diligence under EC7 §4.

21.7 Close-Out Reports and Data Preservation

Close-out and data preservation mark the formal transition from **construction verification** to **as-built certification**.

The objective is to demonstrate that all works have been completed, tested, and documented in accordance with **design assumptions, Eurocode 7 requirements, and project specifications**.

This section outlines the essential structure, content, and control of final geotechnical QA deliverables — ensuring **traceability, accountability, and long-term retrievability** of data.

21.7.1 Purpose

To ensure:

1. All construction and test records are compiled, signed, and archived.
2. As-built information is consistent with design drawings and EC7 verification basis.
3. Outstanding NCRs or risks are transferred into the **Residual Risk Register**.
4. Permanent documentation exists to support future audits, re-use, or modification.

⚠️ A complete close-out package is not just a project deliverable — it's a *defensible record of professional verification.*

21.7.2 Eurocode 7 Context

EN 1997-1 §4.1 (3)P:

"Records of construction and ground behaviour shall be sufficient to verify that design requirements have been met."

Close-out documentation is therefore an integral part of EC7 verification, linking design intent, construction evidence, and post-works confirmation.

Each element — pile, wall, platform, embankment — must have a demonstrable chain of compliance from **input data → construction → verification → acceptance**.

21.7.3 Core Deliverables

Deliverable	Purpose	Prepared By / Owner
Geotechnical Completion Report (GCR)	Summarises verification evidence and outcomes	Designer / Geotechnical Engineer
As-Built Drawings	Confirm geometry, depths, levels	Contractor / Surveyor
QA Summary Tables	Tabulate test results, tolerances, outcomes	Site Engineer / QA Manager
Material Certificates	Verify conformity to specification	Suppliers / Lab
NCR Register (closed)	Document non-conformances and resolutions	QA Manager
Residual Risk Register	Identify remaining risks post-completion	Designer
Instrumentation Records	Confirm monitoring and performance data	Geotechnical Engineer
Digital Archive (Data Room)	Centralised access to all verified data	Project Manager / Client

21.7.4 Geotechnical Completion Report (GCR) – Typical Contents

1. Project and Design Summary
    - Description of works and geotechnical category.
    - Applicable standards and design approaches (DA1, DA2, etc.).
2. Design Assumptions Recap
    - Characteristic soil parameters and partial factors.
    - Limit state combinations verified (GEO, STR, EQU, HYD, UPL).
3. Verification Evidence Summary
    - Table of field/lab tests by type (EV_2, CBR, PLT, PIT, CSL, etc.).
    - Compliance statement vs specification and design inputs.
4. Non-Conformance Summary
    - Reference list of NCRs and close-out actions.
    - Confirmation that no residual impact on design validity remains.
5. Residual Risks and Recommendations
    - Any unmitigated risks transferred to operation or maintenance.
6. Supporting Appendices
    - QA test certificates, survey reports, calibration certificates.
    - Final drawings and photographic evidence.

⚠️ Each GCR should be concise (10–20 pages typical) but *data-rich*—cross-referencing digital QA sheets for traceability.

21.7.5 Digital Data Preservation Framework

Data Type	Format / File Type	Retention Recommendation
QA test results (PLT, LWD, NDG)	CSV / XLSX / PDF	≥ 12 years (permanent works)
Calibration certificates	PDF	Until next calibration cycle + 2 years
As-built drawings	DWG / PDF	≥ 15 years
GCR / GDR documents	PDF (signed)	Permanent archive
Instrumentation logs	CSV / XLSX	Life of structure
NCR register	XLSX / Power BI dataset	Permanent record
Photos / geotagged images	JPEG / GeoJSON metadata	≥ 10 years
3D ground model (if produced)	AGS / BIM IFC	Permanent digital archive

Data should be stored in a **version-controlled repository** (e.g. SharePoint, OneDrive, or client CDE) with unique file identifiers and metadata (revision, author, approval date).

21.7.6 Verification of Data Completeness

Before final submission:

1. Confirm **all test IDs** are reconciled against site logbook.
2. Validate **geo-spatial references** (chainage, coordinates, level).
3. Confirm cross-references between design calculations and field evidence.
4. Reconcile any duplicate or missing test results.
5. Ensure **signatures / approvals** are captured on each QA sheet.
6. Include a **QA compliance matrix** (Design Input → Verification Evidence → Status).

21.7.7 Residual Risk Register

Residual risks represent conditions where:

- Full compliance was not achievable, but stability/serviceability remain adequate; or
- Long-term uncertainty remains (e.g. groundwater, consolidation, monitoring).

Risk ID	Description	Residual Effect	Mitigation / Monitoring	Owner
RR-01	Potential soft zone under pile group	Slight differential settlement	Include in I&M regime	Client
RR-02	Elevated sulphate in fill	Durability concern	Concrete class DS-3 / monitoring	Designer
RR-03	Minor wall deflection > design SLS	Cosmetic only	Continue inclinometer readings	Engineer

Residual risk items must be clearly transferred to the **Client H&S File (CDM Regulation 12)** and referenced in the GCR.

21.7.8 Lessons Learned and Feedback Loop

After close-out:

- Conduct a **QA debrief** between Designer, TWC, and QA Manager.
- Identify recurring NCR trends or specification ambiguities.
- Feed findings into **company QA system** and future project templates.
- Update checklists, method statements, or digital dashboards accordingly.

Lessons learned should be treated as part of *continuous improvement* — they are vital for EC7 Category 2/3 verification readiness.

21.7.9 EC7 Verification Closure Statement (Example Wording)

"Based on review of QA evidence, test results, and construction records, the works described herein are considered to satisfy the design requirements of EN 1997-1 for the verified limit states (GEO, STR, EQU, UPL, HYD). All non-conformances have been resolved or recorded as residual risks. The verification evidence is complete, traceable, and retained within the project data archive."

Signed:
Chartered Geotechnical Engineer (CEng MICE)
Date: ___________________
Phicon Verification Reference: ___________________

21.7.10 Archiving and Handover

Stage	Deliverable	Recipient
Internal archive	QA data room, GCR, NCR register	PHICON Project Folder
Client handover	Digital close-out pack (GCR, as-builts, certificates)	Client / PM
Regulatory handover	CDM Health & Safety File	Principal Designer
Long-term archive	Read-only cloud folder (SharePoint/OneDrive)	Company Record

Ensure all files are:

- Locked to prevent post-approval edits.
- Tagged with metadata (title, date, revision, author).
- Linked to project code and GDR reference for retrieval.

Phicon Note:

• Every close-out document must be auditable: **if it's not referenced, it doesn't exist.**
• Combine design, construction, and verification data into a single digital chain — it forms your *EC7 defence file*.
• Retain GCRs and verification tables permanently; they are part of the **geotechnical design record under CDM**.
• Digital archiving is preferable to PDF-only: structured data (e.g. CSV / Power BI) allows trend re-use and re-analysis in future.
• Ensure all residual risks are formally handed over to the Client — unresolved risks left undocumented become latent liabilities.
• Where PHICON acts as Designer, final sign-off must always reference **limit states verified** and **data completeness status** — never just a "general compliance" statement.
• The strength of a geotechnical practice lies in its *data traceability* — make every calculation, test, and record recoverable.

22 Geotechnical Monitoring & Observational Method

Monitoring and the observational method form the **feedback loop between design and behaviour** in geotechnical works.

They allow designers and constructors to verify that **ground response remains within acceptable limits** and to adjust construction if required.

This chapter covers monitoring principles, trigger levels, and implementation of the **Observational Method (OM)** as formalised in **EN 1997-1 §2.7**, **CIRIA C760**, and **ICE Specification for Geotechnical Works (2017)**.

22.1 Monitoring Principles and Trigger Levels

22.1.1 Purpose

To ensure that:

1. Ground and structural movements remain within **predicted envelopes** from design.
2. Any deviation from expected performance is **detected early and acted upon**.
3. The behaviour of the works provides data for back-analysis and verification.
4. Construction control decisions are **data-driven** and defensible under EC7.

⚠️ Monitoring is not just "data collection" — it is a **real-time verification tool** embedded in the design process.

22.1.2 Eurocode 7 Framework

EN 1997-1 §2.7 (Observational Method):

"The design shall be reviewed during construction, and the observed performance of the structure or the ground shall be compared with the predictions, in order to verify validity of the design assumptions."

Monitoring therefore forms part of the EC7 verification path and is explicitly linked to **limit state compliance** (GEO, STR, HYD, UPL, SLS).

22.1.3 Elements of a Monitoring Scheme

Component	**Purpose**
Parameter	What is measured (e.g. displacement, pore pressure, strain)
Instrument	How it is measured (e.g. inclinometer, piezometer)
Location / Chainage	Where it is measured
Frequency	How often readings are taken
Trigger levels	When action is required
Responsible engineer	Who interprets and acts
Action plan	What response occurs if trigger is exceeded

All must be defined in a **Monitoring Schedule or Instrumentation & Monitoring (I&M) Plan**, approved by the designer and principal contractor.

22.1.4 Common Monitoring Parameters

Parameter	Purpose	Typical Instrument	Indicative Limit (SLS)
Lateral wall deflection	Verify stability and stiffness	Inclinometer / tilt sensor	25–50 mm
Ground settlement	Check ground loss or compression	Settlement plate / levelling	10–25 mm
Pore pressure	Confirm drawdown or excess pore dissipation	Standpipe / VW piezometer	±10 kPa from baseline
Anchor load	Confirm load transfer and lock-off stability	Load cell / strain gauge	±10 % of design
Structural strain	Detect overstress or cracking	Strain gauge / fibre optic	±20 % of predicted
Water level	Check hydrostatic conditions	Datalogger / pressure transducer	±0.5 m
Surface movement	Global stability check	Survey markers	< 1:1000 slope of line

22.1.5 Trigger Level Hierarchy

Trigger Level	Meaning	Typical Response
Green	Within expected range	Continue monitoring
Amber	Approaching design prediction / tolerance	Increase frequency, review data, inform Designer
Red	Exceeds design threshold or rate	Stop affected works, investigate, implement contingency

Example trigger wording:

- *Amber:* Deflection > 75 % of design prediction.
- *Red:* Deflection > design prediction or rate > 5 mm/week.

⚠️ Trigger levels must be **quantitative and pre-agreed**—not subjective statements like “significant movement”.

22.1.6 Frequency of Monitoring (Indicative)

Parameter	During Active Works	Post-Construction
Wall deflection / inclinometers	Daily / twice weekly	Weekly → monthly
Groundwater / piezometers	Twice weekly	Weekly
Surface settlement	Weekly	Monthly
Anchor loads	Daily during stressing	Monthly thereafter
Structural strain	Continuous (auto-loggers)	Monthly
Survey levelling	Weekly	Monthly or after rainfall events

Frequency should increase automatically after any trigger exceedance or unexpected trend.

22.1.7 Monitoring Data Management

All readings must be:

1. Timestamped and geo-referenced.
2. Entered into a **secure database or dashboard** (e.g. Excel, FieldView, Power BI, InstruNet).
3. Linked to instrument ID, chainage, and elevation.
4. Plotted against **predicted envelopes** from design.
5. Reviewed by the **geotechnical engineer** within 24 h of data upload.

Automated dashboards should include:

- Trend analysis and rate-of-change alarms.
- Colour-coded trigger visualisation.
- Cross-links to construction stages (e.g. excavation depth).

22.1.8 Quality and Calibration Controls

Control	Requirement / Frequency
Instrument calibration	On installation and at defined intervals (typically 6–12 months)
Baseline readings	Minimum 3 sets before excavation or loading
Redundancy	At least 1 spare sensor per cluster
Cross-check	Manual reading against logger once per week
Verification	Independent data audit at project milestones

Calibration certificates form part of the **QA verification record** under EN 1997-1 §4.

22.1.9 Interpretation and Reporting

Report Type	Frequency	Content / Action
Daily / Weekly Summary	During active works	Key readings vs design predictions; any Amber/Red alerts
Monthly Report	Ongoing verification	Graphs, rate of change, commentary
Trigger Event Report	As required	Investigation summary, photographs, mitigation
Close-out Summary	Post-construction	Final plots, residual movements, compliance statement

All reports should be **signed by the responsible geotechnical engineer** and appended to the Geotechnical Completion Report (GCR).

22.1.10 Example Monitoring Summary Table

Instrument ID	Type	Chainage / Level	Parameter	Baseline	Amber	Red	Current	Status
IN-01	Inclinometer	Ch 40 / RL +6.5	Deflection	0	20	35	18 mm	🟩 Green
ST-03	Settlement plate	Ch 65	Settlement	0	15	25	21 mm	🟧 Amber
PP-05	VW Piezometer	RL +4.2	Pore pressure	45	±10	±15	+9 kPa	🟩 Green
AL-02	Anchor load cell	RL +7.0	Load	350	±10	±15	390 kN	🟥 Red

22.1.11 EC7 Verification Perspective

Monitoring confirms that the structure's behaviour remains within the **design assumptions used at ULS and SLS**.

A trigger exceedance does **not automatically mean failure**, but it requires:

1. Verification of readings (repeat measurement).
2. Assessment vs design model.
3. Decision whether modification or stabilisation is necessary.
4. Record of actions and design re-verification.

This process fulfils EC7 §2.7's requirement for **adaptive design based on observation**.

22.1.12 Digital Integration

Phicon recommends:

- **Power BI dashboards** linked to live instrument feeds.

- **Automated alert emails** for Amber/Red triggers.
- **Versioned plots** embedded into GDR appendices.
- Cloud-stored CSVs for re-analysis and EC7 audit.
- QR-coded site boards linking to live monitoring graphs.

Benefits:

- Rapid decision support.
- Transparent traceability of readings.
- Integration with residual risk and QA systems.

Phicon Note:

• Monitoring is verification in motion — every reading checks a design assumption.
• Baseline readings are as critical as construction data; missing baselines invalidate comparisons.
• Set trigger levels using design predictions + serviceability limits (not arbitrary numbers).
• Trend rate is often more important than total value — *movement rate acceleration* is the earliest warning.
• Maintain close communication between monitoring engineer, designer, and TWC; data without action is meaningless.
• Always log *context*: excavation level, rainfall, adjacent works — interpretation requires understanding of stage sequence.
• Archive data in both visual and numerical formats; future designers may need to re-analyse performance decades later.

22.2 Instrumentation Types (Piezometers, Inclinometers, Extensometers)

Instrumentation provides the **quantitative link** between design predictions and actual ground behaviour. Proper selection, installation, and calibration are essential to ensure readings are **accurate, representative, and traceable** under the Eurocode 7 framework for **observational verification (EN 1997-1 §2.7)**.

This section outlines the most common geotechnical instruments used for monitoring pore pressure, deformation, and strain in soil and rock.

22.2.1 Purpose

To measure:

1. **Groundwater and pore pressure** changes (effective stress control).
2. Lateral or vertical movement of soil or structures.
3. **Deformation and settlement** in excavations, embankments, or foundations.
4. **Strain or load** within structural or ground elements.

⚠️ An instrument is only as valuable as its calibration and context — every installation must correspond to a **defined design verification parameter** (e.g. pore pressure at base of excavation, wall deflection at mid-depth).

22.2.2 Overview of Instrument Types

Instrument	Measured Quantity	Typical Application	Typical Accuracy

Piezometer	Pore water pressure (u)	Excavations, embankments, slopes	±2–5 kPa
Inclinometer	Lateral displacement	Retaining walls, slopes	±1 mm / 10 m
Extensometer	Vertical displacement between anchors	Tunnels, settlement basins	±0.1 mm
Settlement plate	Surface settlement	Embankments, fills	±2 mm
Vibrating wire strain gauge	Strain / stress	Concrete, anchors	±0.5 % FSR
Pressure cell	Earth or water pressure	Retaining walls, fills	±2 % FSR
Load cell	Force in anchor/tie	Walls, ground anchors	±1 % FSR

22.2.3 Piezometers

Type	Principle / Construction	Use
Casagrande standpipe	Porous tip, water-filled riser	Long-term groundwater level, slow response
Pneumatic piezometer	Gas pressure balance	Medium response, reliable under depth
Vibrating wire (VW) piezometer	Frequency shift under diaphragm deflection	High precision, automated monitoring
Hydraulic / Push-in (BAT type)	Hydraulic fluid system	Short-term pore pressure, CPT-based installation

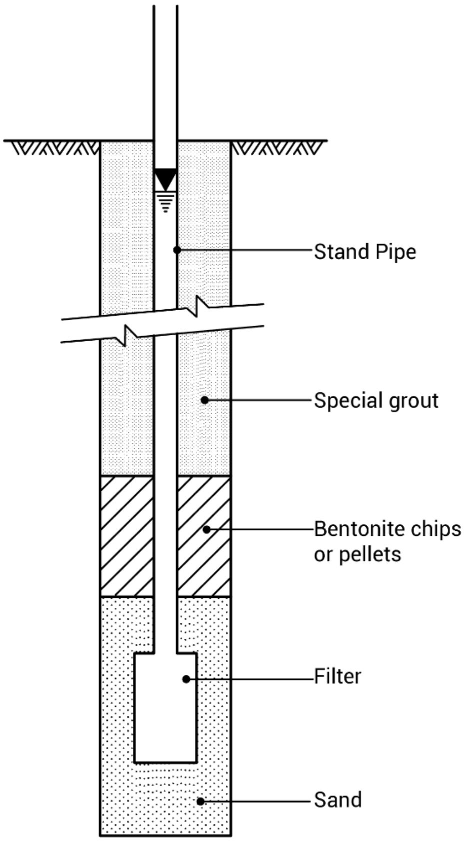


Figure 79: Schematic detail of a Casagrande standpipe

22.2.3.1 Key QA Controls

Stage	Check / Requirement
Installation depth	Confirmed by borehole log (±50 mm)
Backfill	Sand filter and bentonite seals at correct intervals
Calibration certificate	Traceable to manufacturer, valid within 12 months
Baseline readings	3 consistent readings before construction activity
Logging frequency	Twice weekly → daily during excavation / drawdown
Drift check	Recalibrate at mid-project or ±2 kPa tolerance drift

$$u = \gamma_w \cdot h$$

where u= pore pressure, γ_w= unit weight of water, h= hydraulic head difference.

Typical plotting:

- Compare measured vs predicted pore pressure dissipation curve.
- Calculate effective stress: $\sigma' = \sigma - u$.
- Verify heave / drawdown control under EC7 UPL/HYD checks.

22.2.4 Inclinometers

Measure **horizontal displacement** vs depth to detect:

- Bending of retaining walls or piles.
- Shear zones or slip surfaces in slopes.
- Ground relaxation behind excavations.

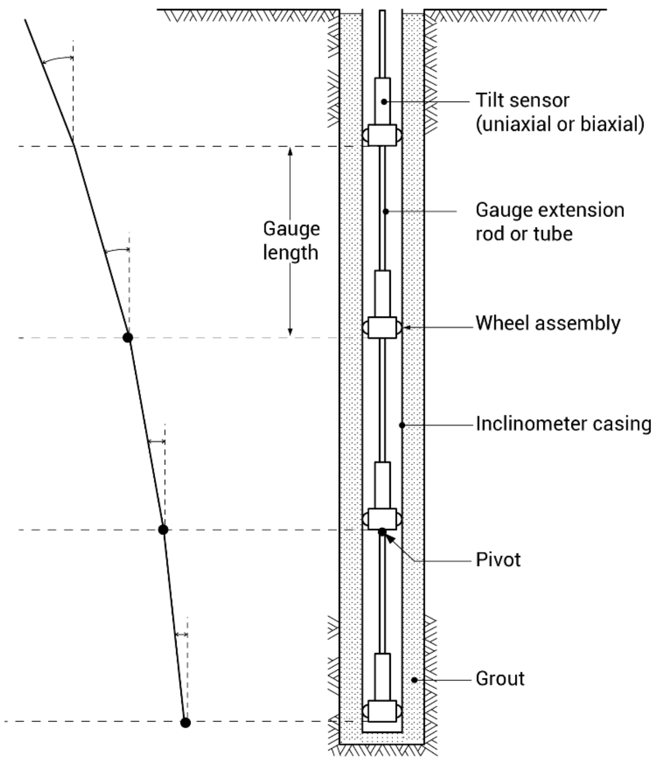


Figure 80: Schematic of in-place inclinometer (reproduced from Dunnicliff 2012)

22.2.4.1 Installation

Requirement	Typical Value / Practice
Borehole diameter	70–90 mm
Casing	ABS or aluminium grooved casing
Embedment	Min 2–3 m below expected movement zone
Orientation	A-axis perpendicular to wall face
Grout backfill	Bentonite–cement mix (w:c ≈ 1.5:1)
Surface collar	Secure, sealed, labelled with ID and RL
Calibration	Manufacturer certificate; zero check before insertion

22.2.4.2 Monitoring Procedure

- Initial **baseline profile** before any excavation.
- **Incremental displacement** readings (0.5 m intervals typical).
- Compare successive profiles to detect movement patterns.
- Plot deflection and cumulative movement vs depth and time.

⚠️ Readings must be taken in **the same orientation** and temperature conditions — rotation or reversal of probe can invalidate data trends.

22.2.4.3 Acceptance / Trigger Guidance

Parameter	Indicative Limit	Action
Cumulative lateral deflection	25–50 mm (SLS typical)	Amber / review design
Deflection rate	> 5 mm/week	Red / stop excavation
Rotation at toe	> 1°	Design reassessment
Unexpected reversal	—	Check for instrument error

22.2.5 Extensometers

Purpose:

Measure **vertical or internal deformation** within soil or rock mass — useful for:

- Detecting settlement layers within embankments.
- Monitoring compression in weak strata.
- Verifying consolidation rate predictions.

22.2.5.1 Types

Type	Construction	Accuracy / Use
Rod extensometer	Series of rods anchored at depth, reference head at surface	±0.1 mm; reliable, common in tunnels
Multipoint VW extensometer	Vibrating wire sensors at multiple depths	±0.05 mm; automated long-term monitoring
Magnet extensometer	Sliding magnets along access pipe	±0.5 mm; used in soft clay embankments

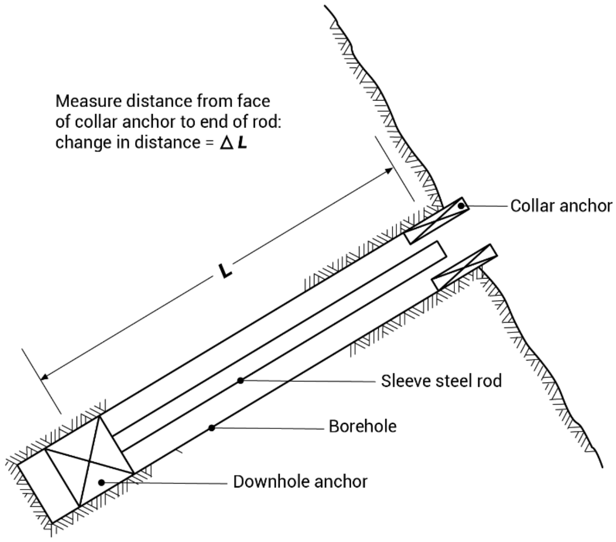


Figure 81: Fixed borehole (rod) extensometer (reproduced from Dunnicliff 2012)

22.2.5.2 Installation

Parameter	Verification
Anchor depth	Confirmed against borehole log
Central alignment	Measured during installation
Calibration	Each sensor certified ±0.1 % FSR
Baseline readings	≥ 3 consistent datasets pre-construction
Reference datum	Benchmarked to site grid / fixed survey point

22.2.5.3 Interpretation

Displacement profiles identify zones of compression, creep, or failure planes.
Used to validate settlement predictions under EC7 SLS checks.

Example correlation:

$$\Delta h = \Sigma \Delta L_i$$

where total settlement Δh= sum of segment displacements ΔL_i.

22.2.6 Typical Instrument Layout in Excavations

Instrument	Placement	Purpose
Inclinometer	Behind wall, extending 2–3 m below excavation base	Wall deflection / toe rotation
Piezometer	Beneath base and behind wall	Check drawdown, HYD stability
Anchor load cell	On top of anchor heads	Verify lock-off load

Settlement marker	Surface or building proximity	Ground movement
Extensometer	In underlying soft strata	Vertical compression
Strain gauge / pressure cell	On wall reinforcement	Structural verification

22.2.7 Data Recording and QA

Requirement	**Description**
Unique ID	Each sensor labelled and logged with coordinates and depth.
Baseline file	Initial stable dataset for comparison.
Timestamped entries	Every reading dated and time-recorded.
Temperature correction	Applied to VW sensors where required.
Cross-check	Manual verification against survey data weekly.
QA verification	Comparison to predicted behaviour from design models.

All instrument data must be **backed up daily** and uploaded to the central project database or dashboard.

22.2.8 Calibration and Maintenance

Instrument	**Calibration Frequency**	**Procedure**
Vibrating wire sensors	6–12 months	Zero check / reference pressure
Inclinometer probe	Quarterly	Comparison against calibration jig
Data logger	6 months	Battery / channel verification
Manual standpipe	—	Cross-check with adjacent sensors
Extensometer head	Annual	Micrometer verification
Load cell	6 months	Load calibration using proving ring

⚠️ Calibration records form mandatory QA evidence under EN 1997-1 §4 supervision requirements.

22.2.9 Common Faults and Troubleshooting

Issue	Possible Cause	Action
Erratic VW readings	Cable damage / temperature effect	Inspect cable, apply correction
Noisy inclinometer data	Probe reversal / debris in casing	Clean casing, recheck orientation
Loss of piezometric head	Clogged tip / seal failure	Re-develop or replace piezometer
Sudden zero shift	Logger reset or mechanical slip	Check calibration and fix baseline
Gradual drift	Creep or gas accumulation	Purge lines, confirm with duplicate sensor

22.2.10 Data Interpretation and EC7 Verification

Design Aspect	Verification Parameter	Instrument
Stability of excavation	Wall deflection within predicted envelope	Inclinometer
Heave / uplift	Pore pressure < design hydrostatic	Piezometer
Settlement control	Vertical compression within tolerance	Extensometer / settlement plate
Structural stress check	Load or strain within design limit	Load cell / strain gauge
Consolidation rate	Pore pressure dissipation vs time	Piezometer ($\Delta u/\Delta t$)

Data must be summarised in **Monitoring Summary Tables** linked to EC7 limit state references (ULS, SLS, HYD, UPL).

Phicon Note:

• Each instrument must answer a **specific design question** — avoid “monitoring for its own sake.”
• Always pair piezometers and inclinometers near key sections — pressure and movement together provide diagnostic clarity.
• Redundant sensors improve reliability and are mandatory for high-risk excavations.
• Automated VW systems improve continuity but require rigorous calibration — manual spot checks remain essential.
• Ensure instrument IDs match as-built drawings and dashboard references; mismatch breaks traceability.
• Under EC7, monitoring data are verification evidence — missing or unverified sensors equal *design non-compliance*.
• Archive raw data and processed plots in both CSV and PDF for long-term retrievability.

22.3 Observational Method in Design and Construction

The **Observational Method (OM)** is a cornerstone of modern geotechnical design philosophy under **Eurocode 7 (EN 1997-1 §2.7)**.

It provides a structured process for managing **uncertainty in ground conditions and behaviour** through *design prediction, monitoring, and adaptive control*.

Unlike empirical "watch and see" approaches, the OM requires that **contingency actions and trigger criteria** are pre-defined, documented, and verifiable.

22.3.1 Purpose and Context

The method is applicable where:

1. Ground conditions cannot be fully determined prior to construction.
2. Behaviour depends on complex soil–structure interaction.
3. It is practicable to monitor and react before failure occurs.
4. Modifying works is safer and more economical than over-design.

Typical applications:

- Deep excavations and retaining walls.
- Tunnels and shafts in variable strata.
- Large embankments and fills.
- Slope stabilisation and groundwater drawdown works.

⚠️ The OM converts uncertainty into a managed design variable — *it does not replace design with observation*.

22.3.2 Eurocode 7 Definition (EN 1997-1 §2.7.1)

The OM comprises the following essential steps:

1. Establishing acceptable limits of behaviour.
2. Preparing a design based on these limits and predicted performance.
3. Instrumenting and monitoring the structure and ground.
4. Comparing observed and predicted behaviour.
5. Modifying the design or construction if the behaviour diverges from prediction.

The method therefore merges **design verification** and **construction supervision** into one continuous loop of observation and adjustment.

22.3.3 Step-by-Step Framework

Stage	Objective	Verification Deliverable
1. Prediction	Model expected ground and structural behaviour (settlement, deflection, pore pressure)	Baseline predictions and trigger thresholds
2. Acceptable limits	Define quantitative limits for serviceability and safety	Trigger Level Table (Green/Amber/Red)

3. Instrumentation design	Select sensors, locations, frequency	Monitoring Plan and layout drawings
4. Construction plan	Define stages and sequences with hold points	Construction Method Statement
5. Monitoring & interpretation	Record and assess data vs predictions	Weekly/monthly monitoring reports
6. Action plan	Detail contingency actions for exceedances	Observational Action Plan (OAP)
7. Review & closure	Confirm design remains valid or adapt	GDR Verification update

22.3.4 Prediction and Acceptable Behaviour

22.3.4.1 Predictions

Prepared using design analyses (finite element, limit equilibrium, empirical) giving expected:

- Deflection profiles (mm vs depth)
- Settlement troughs
- Pore pressure dissipation curves
- Anchor/tie load trends

22.3.4.2 Acceptable Limits

- Set in relation to **serviceability criteria** and adjacent structure tolerances.
- Quantitative: e.g., deflection ≤ 40 mm, settlement ≤ 25 mm, pore pressure ±10 kPa.
- Each limit must link to a measurable parameter and instrument.

22.3.5 Trigger–Response Framework

Trigger Level	Description	Example Limit	Required Response
Green	Behaviour within prediction	Deflection < 75 % design value	Continue works
Amber	Trend approaching prediction	Deflection > 75 %, rate ↑	Increase frequency, review analysis
Red	Exceedance or rapid deviation	Deflection > 100 % predicted	Stop works, implement contingency

Responses should be clearly defined in the **Observational Action Plan (OAP)** and authorised by the **Geotechnical Designer / TWC**.

22.3.6 Instrumentation & Data Requirements

Monitoring must be:

- **Sufficiently frequent** to detect deviations early.
- Accurate and calibrated (see §22.2).
- Spatially representative of critical zones.
- Recorded and reviewed by competent engineers.

A minimum of three pre-construction baseline readings are required to establish natural variability.

Automated systems (e.g. VW or total-station networks) must include independent manual cross-checks at least weekly.

22.3.7 Contingency Design and Action Plans

Each OM application must include a **pre-defined contingency design**, typically embedded in the GDR or Construction Method Statement.

22.3.7.1 Examples

Observed Issue	Potential Cause	Contingency Action
Excessive wall movement	Underestimated active pressure	Add temporary prop / reduce excavation step
High pore pressures	Insufficient drainage	Install relief well or re-sequence excavation
Rapid settlement	Soft layer mobilisation	Pause works, install monitoring extensometer
Anchor load loss	Bond creep or slip	Re-stress or add redundant anchor
Increased heave	Groundwater rebound	Delay base pour, adjust drainage

All actions must be **trigger-linked**, not reactive.

⚠️ A “contingency plan” is an engineered design state — it should be verified for feasibility before works start.

22.3.8 Documentation Framework

Document	Purpose / Content
Observational Method Statement (OMS)	Defines scope, monitoring, triggers, and contingencies
Monitoring Schedule	Instrument type, ID, frequency, responsible engineer
Trigger Level Matrix	Quantitative thresholds for each parameter
Action & Communication Flowchart	Defines roles and escalation (Site → TWC → Designer → Client)
OM Verification Log	Ongoing record of readings vs predictions

Design Update Sheet	Record of any modification following exceedance

All forms should be **digitally cross-referenced** within the project QA system and appended to the GDR.

22.3.9 Role Responsibilities

Role	Responsibility in OM
Designer	Define predictions, triggers, and contingency design.
Principal Contractor / TWC	Implement and monitor data collection, enforce actions.
Monitoring Engineer	Record, interpret, and flag deviations.
Site Engineer	Maintain construction records linked to stages.
Client / PM	Review trigger responses and approve major design changes.

22.3.10 Review and Adaptation Process

1. Compare observed vs predicted data (plots, rates, envelopes).
2. Assess significance: is deviation within acceptable margin?
3. Re-analyse design if deviation exceeds Amber threshold.
4. Modify construction (sequence, support, drainage) if Red trigger reached.
5. Document changes in the **Design Change Note (DCN)** and GDR.
6. Reset baseline or new prediction if modification implemented.

EC7 allows adaptation of design *provided the structure remains safe at all stages* and documentation is complete.

22.3.11 Integration with Digital Systems

Phicon recommends:

- **Automated Power BI dashboards** linking live data to trigger logic.
- Instant alert emails for Amber/Red exceedances.
- Embedded "trend envelopes" showing predicted vs observed.
- Digital audit trail between monitoring data and GDR verification tables.

Benefits:

- Immediate visibility for designers.
- Transparent, timestamped verification record.
- Simplified close-out and residual risk transfer.

22.3.12 Limitations and Good Practice

- OM is unsuitable where **failure can occur suddenly or without warning** (e.g., brittle collapse).
- Requires **direct control of works** and real-time communication.

- Should not be used to justify **incomplete design** — initial design must be safe within reasonable bounds.
- Must be backed by a documented monitoring system and decision authority chain.
- All design modifications must be re-verified under EC7 limit state rules.

Phicon Note:

• The Observational Method is an **active design tool**, not a fallback; treat it as a designed feedback system with engineered boundaries.
• The *critical outputs* are not the readings — they are the **decisions made in response**.
• Always define the monitoring frequency so that **action can be taken before instability develops**.
• Document all decisions in the OM log, cross-referenced to chainage, time, and instrument ID.
• EC7 compliance requires **quantified trigger levels and a pre-approved contingency plan** — general statements like "monitor and review" are not compliant.
• Where Phicon acts as designer, the OM should form a **standalone appendix** to the GDR and remain live until post-construction verification is complete.
• The OM represents the highest form of geotechnical professionalism — combining design judgement, data interpretation, and controlled adaptability.

22.4 Real-Time Monitoring Dashboards and QA Integration

Real-time dashboards transform geotechnical monitoring from static data collection into **active decision support**.

When combined with the EC7 observational framework (§2.7) and QA verification records (§4.1), dashboards provide immediate visibility of **ground behaviour, trigger status, and compliance evidence**.

This section outlines digital integration principles for monitoring, verification, and close-out under PHICON practice.

22.4.1 Purpose

To enable:

1. **Continuous verification** of design assumptions against observed performance.
2. **Instant visibility** of trigger breaches and rates of change.
3. **Automatic data traceability** between instruments, design models, and QA records.
4. **Auditable EC7 verification chains** linking construction, monitoring, and response actions.

⚠️ Real-time data does not replace engineering judgement — it ensures the right information reaches the right engineer at the right time.

22.4.2 Digital Workflow Architecture

Modern geotechnical monitoring systems rely on integrated digital workflows that connect field instruments, data platforms, and verification processes into a single information stream. The architecture typically comprises several linked layers:

1. **Data Acquisition** – Sensors and loggers collect readings automatically or manually, with each record time-stamped, georeferenced, and stored in a central repository.

2. **Data Validation and Structuring** – Incoming data are checked for accuracy, range, and consistency before being organised by instrument type and location, ensuring traceability.
3. **Integration** – Validated data are linked to design models, QA documentation, and trigger action plans to enable direct comparison between observed and predicted behaviour.
4. **Visualisation and Analytics** – Dashboards display trends, trigger levels, and rates of change using automated calculations and colour-coded alerts for rapid assessment.
5. **Response and Archiving** – Exceedances trigger notifications and response actions that are recorded digitally, creating a verifiable audit trail for design verification and asset management.

This digital architecture creates a **closed feedback loop** between construction, monitoring, and design verification—supporting faster decisions, transparent records, and compliance with the observational method.

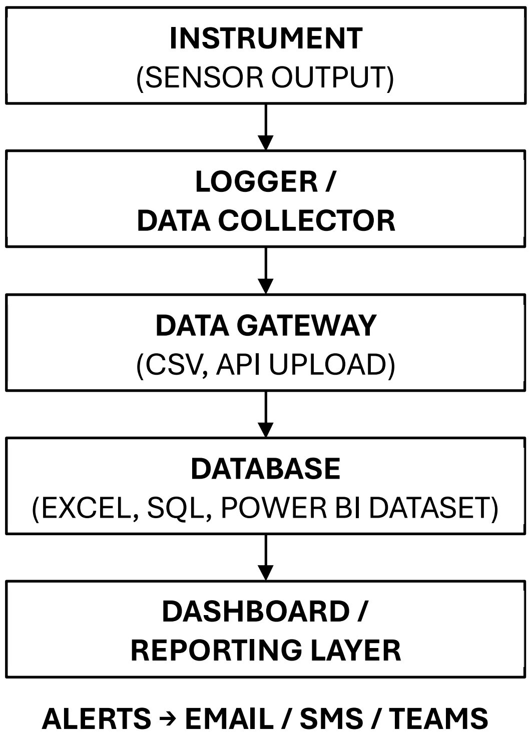


Figure 82: Data Flow Overview

Each layer must include **QA checkpoints** for validation, timestamping, and version control.

Stage	Example Tool	Verification Requirement
Instrument logging	InstruNet, Senceive, GeoLogger	Manufacturer calibration certificate
Data collection	CSV / Excel macro import	Date-time and unit consistency check
Data storage	SQL / SharePoint / OneDrive	Unique ID and metadata tags
Dashboard visualisation	Power BI / Grafana	Correct trigger colour logic
Alert distribution	Outlook / Teams / SMS	Auto-log of recipient and timestamp

22.4.3 Dashboard Structure (Typical Layout)

Section	Content / Functionality
Header	Project ID, excavation stage, last update timestamp
Map Panel	Interactive plan with instrument locations and IDs
Live Charts	Real-time plots of displacement, pore pressure, settlement
Trigger Table	Green–Amber–Red matrix with exceedance flags
Rate-of-Change Graphs	Trend analysis over 7-day and 30-day windows
QA Verification Status	Links to latest field reports, calibration logs
NCR / Action Log	Active non-conformances linked to instruments
Residual Risk Summary	Persistent exceedances flagged for GDR inclusion

22.4.4 Colour Logic (Trigger Display)

Status	Condition	Colour / Symbol	Automatic Action
Green	Within predicted range	🟩	Normal monitoring
Amber	>75 % design value or acceleration trend	🟧	Notify Designer, increase frequency
Red	≥100 % predicted or rapid exceedance	🟥	Halt works, issue trigger alert
Grey	Offline / no data	⚪	Verify instrument or logger

All alerts must automatically generate a **record entry** (timestamp, instrument ID, parameter, action taken).

22.4.5 Data Validation and QA Checks

Check	Purpose	Frequency
Timestamp check	Confirms data continuity	Daily
Range check	Prevents false readings (spikes, zero drift)	Daily
Baseline lock	Ensures stable reference value	On installation

Calibration cross-check	Confirms sensor accuracy	Weekly–monthly
Data redundancy check	Verifies duplicate sensors behave consistently	Weekly
Audit trail backup	Permanent record of raw and processed data	Continuous

All QA verification logs should be exportable to PDF for **traceability**.

22.4.6 Linking Monitoring to EC7 Verification

Each parameter must correspond to a **limit state** and design assumption.

Measured Parameter	Limit State Verified	Verification Evidence
Pore pressure (u)	HYD / UPL	Piezometer readings vs design head
Wall deflection	GEO / STR	Inclinometer trend vs predicted envelope
Settlement	SLS	Settlement plate data vs design limit
Anchor load	STR	Load cell readings vs lock-off design
Water level	EQU	Comparison to hydrostatic design

Dashboards should automatically reference each parameter to the **ULS/SLS combination** used at design stage (e.g. DA1-C1, DA1-C2).

22.4.7 Automation Features

Modern Phicon dashboards include:

- **Auto-trigger notifications** via email / Teams within 5 minutes of breach.
- **Dynamic plots** showing observed vs predicted envelopes.
- **Daily data roll-up tables** for rapid field QA checks.
- **Embedded NCR log forms** linked to each instrument ID.
- **Automatic GDR export** (PDF + CSV snapshot at monthly intervals).
- **Integration with Power Automate** for document filing in SharePoint.

22.4.8 Typical KPI Outputs

Indicator	Description	Target / Use
% instruments online	Active sensors vs total installed	> 95 %
Trigger exceedance ratio	Amber/Red events per week	Trend analysis
Average response time	Time between alert and action logged	< 24 h
NCR closure rate	% of NCRs closed within timeframe	> 90 %

Data completeness	Valid readings / expected readings	> 98 %
Report submission delay	Days between reading and upload	< 2 days

22.4.9 Integration with QA & GDR Documentation

Every dashboard element should link to its **QA record** or **verification document**:

Dashboard Link	QA Source Document
Calibration button	Instrument calibration certificate
Data trend plot	Weekly monitoring report
NCR reference	NCR form with corrective action
Stage log	Construction diary / excavation log
GDR reference	EC7 verification table (ULS/SLS)
Residual risk tab	GCR residual risk summary

These links create a **live verification environment** that satisfies EC7's requirement for traceable supervision (§4.1).

22.4.10 System Security and Data Retention

- Data stored on **encrypted cloud environments** (SharePoint, OneDrive, Azure).
- Access controlled by user permissions (Designer, TWC, Client).
- Auto-backups every 24 h; version history retained for ≥ 10 years.
- No manual overwriting of raw data — processed outputs saved separately.
- All data exports timestamped with unique revision IDs (e.g. "PHC-MTR-2025-10-24-v03").

22.4.11 Example Power BI Dashboard Layout

Page 1 – Summary View

- Map overview (instrument IDs)
- Traffic-light trigger panel
- 7-day event history

Page 2 – Detailed View

- Graphs of displacement, pore pressure, and settlement vs time
- Predicted vs observed envelopes
- Notes log with attached site photos

Page 3 – QA & NCR Log

- Table of open/closed NCRs
- Cross-links to corrective actions and test certificates

Page 4 – EC7 Verification Report

- Table of instruments vs verified limit state
- Automatic summary: “All monitored parameters within EC7 design predictions (DA1-C1).”

Phicon Note:

• Dashboards should be treated as *live verification tools*, not presentation graphics.
• Each data point must have **source, timestamp, and calibration chain** documented.
• Integrate dashboards with QA registers and NCR logs for complete EC7 traceability.
• Always pair real-time visualisation with **weekly engineering review**—automation without interpretation is unsafe.
• Use dashboards to demonstrate compliance during Cat III or client audits—every reading, trend, and trigger becomes part of your **limit-state verification record**.
• The final archived dashboard dataset (CSV + PDF) forms the *digital verification annex* to the Geotechnical Design Report (GDR).

APPENDIX A: EUROCODE PARTIAL FACTORS AND DESIGN CHECKS

This appendix summarises the Eurocode 7 partial factors, limit-state combinations, and verification formats used in geotechnical design.

It serves as a quick reference for converting characteristic actions and material parameters into design values, enabling consistent application of the Design Approaches (DA1–DA3) under both UK National Annex (NA) and typical EU National Annex (ANB) practice.

This appendix summarises the **Eurocode 7 partial factors, limit-state combinations, and verification formats** used in geotechnical design.

It serves as a quick reference for converting **characteristic actions and material parameters** into **design values**, enabling consistent application of the **Design Approaches (DA1–DA3)** under both **UK National Annex (NA)** and **typical EU National Annex (ANB)** practice.

A.1 Partial Factors on Actions & Materials (DA1, DA2, DA3)

A.1.1 Design Philosophy

All EC7 designs are based on:

$$E_d \leq R_d$$

where:

- E_d= design effect of actions (e.g. moment, shear, bearing stress)
- R_d= design resistance (soil or structural)

Both are derived from **characteristic values** using **partial factors**:

$$E_d = \sum(\gamma_F \cdot F_k), R_d = \frac{R_k}{\gamma_R}$$

Design safety is achieved by combining the correct factors according to the **Design Approach (DA)** and **Combination (C1/C2)**.

A.1.2 Overview of Design Approaches

Design Approach	Applied Factors	Typical Application	Used In (NA Examples)
DA1 (C1 & C2)	Separate factoring of actions and resistances	Foundations, retaining walls	UK, BE, FR, DK

Design Approach	Applied Factors	Typical Application	Used In (NA Examples)
DA2	Actions + resistance on soil strength (combined)	Central Europe, some bridge design	DE, NL
DA3	Partial factors mainly on actions and loads	Temporary works, slope stability (simplified)	SE, NO

⚠️ The UK National Annex (BS EN 1997-1:2004+A1:2013) adopts **DA1** as the default approach for all design situations.

A.1.3 DA1 – Combination 1 (C1)

Applied when verifying **ultimate limit states** governed by **load effects** (e.g. equilibrium, overturning, sliding).

Factor Type	Symbol	Typical Value (UK NA)	Description
Permanent actions (unfavourable)	$\gamma_{G,unf}$	1.35	Increases destabilising effect
Permanent actions (favourable)	$\gamma_{G,fav}$	1.00	No reduction applied
Variable actions	γ_Q	1.50	Typically horizontal loads, surcharge
Soil strength parameters	γ_{ϕ}', γ_c'	1.00	Characteristic values used
Soil resistance	γ_R	1.00	Not applied in C1
Water pressure	γ_u	1.00	Design pore pressure = characteristic

C1 therefore checks **equilibrium of actions**, with strength parameters unfactored.

A.1.4 DA1 – Combination 2 (C2)

Applied when verifying **ultimate limit states** governed by **resistance** (e.g. bearing, sliding resistance, passive pressure).

Factor Type	Symbol	Typical Value (UK NA)	Description
Permanent actions	γG	1.00	Characteristic actions used
Variable actions	γQ	1.30	Reduced variability factor
Soil strength parameters	γϕ', γc'	1.25 (ϕ'); 1.40 (c')	Reduced strength
Undrained shear strength	γcu	1.40	Applied to su,k

Soil resistance	γ_R	1.00	Typically unity unless national annex modifies
Water pressure	γ_u	1.00	Characteristic values

C2 therefore checks **capacity of soil resistance**, with strength parameters reduced.

⚠️ For bearing capacity, the governing design is usually **DA1-C2**, as it controls the reduction of c', ϕ', or su.

A.1.5 DA2 – Typical EU Practice

In DA2, the same partial factors are applied to both actions and resistances.

Factor Type	Symbol	Typical Value	Remarks
$\gamma_{G,unf}$	1.35	Permanent (unfavourable)	
γ_Q	1.50	Variable	
$\gamma_{\phi'}$	1.25	Soil strength reduction	
$\gamma_{c'}$	1.25–1.40	Cohesion reduction	
γ_R	1.40	Applied directly to resistance	
γ_{cu}	1.40	Undrained strength reduction	

DA2 typically results in slightly higher safety margins and is used in Germany, Netherlands, and Austria.

A.1.6 DA3 – Alternative Simplified Format

Used in some Nordic or temporary works contexts, DA3 applies higher load factors but often omits material factoring.

Factor Type	Symbol	Typical Value	Comment
$\gamma_{G,unf}$	1.10	Small increase on dead load	
γ_Q	1.30	Variable actions	
$\gamma_{\phi'}$, $\gamma_{c'}$	1.00	Characteristic values used	
γ_R	1.30	Resistance factor	
γ_{cu}	1.00	For short-term undrained states	

⚠️ DA3 should only be used when justified by national guidance or temporary design justification.

A.1.7 Water and Hydraulic Factors

Parameter	Symbol	Typical Value	Description

Pore water pressure	γ_u	1.00	Use characteristic hydrostatic or measured
Seepage / uplift	γ_U	1.35 (ULS)	EC7 recommends factoring for safety against uplift or piping
Hydraulic gradient	γ_i	1.00	Usually unity unless transient pressures anticipated

A.1.8 Serviceability Limit States (SLS)

For SLS checks (settlement, deformation, movement):

$$E_k \leq R_k$$

Partial factors are **not applied** to either actions or material strengths — characteristic values are used directly.

Safety is achieved by limiting deformation to acceptable service levels.

Limit State	Typical Application	Factors Used
SLS – deformation	Foundations, rafts	γ = 1.0
SLS – pore pressure	Excavations, basements	γ = 1.0
SLS – settlement	Slabs, embankments	γ = 1.0

A.1.9 UK vs EU Partial Factor Comparison

Parameter	UK NA (DA1)	Typical EU ANB (DA2)	Comment
γϕ'	1.25	1.25	Consistent
γc'	1.40	1.25	UK more conservative
γcu	1.40	1.40	Aligned
γG,unf	1.35	1.35	Aligned
γQ	1.50	1.50	Aligned
γR	1.00	1.40	EU adds resistance factor
γu	1.00	1.00	Aligned

A.1.10 Typical Application Table

Design Case	Design Approach / Combination	Limit State	Relevant Factors
Shallow foundation (bearing)	DA1-C2	GEO	γ_{ϕ}', γ_c', γ_{cu}
Retaining wall stability	DA1-C1	EQU	γ_G, γ_Q
Passive pressure check	DA1-C2	GEO	γ_{ϕ}', γ_c'
Pile capacity	DA1-C2	GEO	γ_{cu}, γ_R
Slope stability	DA2 (EU)	GEO	γ_{ϕ}', γ_c', γ_R
Uplift / hydraulic heave	DA1-C1	UPL / HYD	γ_U, γ_G, γ_u
Settlement	SLS	SLS	$\gamma = 1.0$

A.1.11 Note

Phicon Note:

• Always start from **characteristic values**; apply partial factors only for verification at ULS.
• The UK NA's adoption of DA1 ensures **balanced safety** between action and resistance.
• Never mix factor sets between DAs — maintain internal consistency.
• For parametric studies, state clearly which **combination (C1 or C2)** applies to each check.
• For hydraulic problems, apply $\gamma U = 1.35$ to uplift or piping verifications.
• At SLS, characteristic values are used un-factored — EC7 requires serviceability checks to verify *function*, not *safety*.
• Phicon calculation sheets auto-apply DA1-C1 and DA1-C2 logic to ensure traceability and prevent user error.

A.2 – ULS Check Matrix (GEO, STR, EQU, UPL, HYD)

This appendix summarises the **Ultimate Limit State (ULS)** verification framework defined by **Eurocode 7 (EN 1997-1 §2.4.7)**.

It presents a consolidated matrix of design checks for **geotechnical (GEO)**, **structural (STR)**, **equilibrium (EQU)**, **uplift (UPL)**, and **hydraulic (HYD)** limit states, with the associated partial factor applications and verification formats.

A.2 Purpose

To provide a unified reference enabling engineers to:

1. Identify the **governing limit state** for each geotechnical design element.
2. Apply the correct **Design Approach (DA1-C1 / DA1-C2)** or equivalent EU combination.
3. Verify compliance through the **EN 1990 design inequality**:

$$E_d \leq R_d$$

4. Document the corresponding **factors, parameters, and equations** for audit traceability.

A.2.1 Limit State Overview

Limit State	Code Reference	Purpose	Typical Examples
GEO	EN 1997-1 §2.4.7.3	Ground failure or deformation of soil	Bearing capacity, slope stability, passive pressure
STR	EN 1997-1 §2.4.7.2	Structural failure of geotechnical element	Sheet pile bending, anchor tensile rupture
EQU	EN 1997-1 §2.4.7.1	Loss of overall equilibrium	Overturning, sliding, rotation
UPL	EN 1997-1 §10.4	Uplift or flotation by hydrostatic pressure	Basement slabs, pile caps
HYD	EN 1997-1 §10.5	Hydraulic failure of ground (heave, piping)	Excavations, cut-offs, drains

A.2.2 Verification Matrix (Summary)

Limit State	Failure Mode	Typical Structure / Check	Design Approach (UK NA)	Factored Parameters / Partial Factors	Verification Format
EQU	Overturning / sliding	Retaining wall, abutment	DA1-C1	$\gamma_{G,unf}$ = 1.35; γ_Q = 1.50	∑(stabilising) ≥ ∑(destabilising)
GEO	Bearing failure	Footing, raft	DA1-C2	γ_ϕ' = 1.25; γ_c' = 1.40; γ_{cu} = 1.40	V_d / (BL) ≤ q_d
GEO	Slope instability	Embankment, cutting	DA2 (EU) / DA1-C2 (UK)	γ_ϕ' = 1.25; γ_c' = 1.40	FOS ≥ 1.0 at ULS
STR	Bending or tension failure	Sheet pile, anchor, RC wall	EN 1993-5 / EN 1992-1-1	γ_{M0} = 1.00–1.10 (steel); γ_c = 1.50 (RC)	$M_d \leq R_d$
UPL	Flotation / uplift	Basement, raft	DA1-C1	γ_G = 1.35 (unfavourable); γ_U = 1.35	$W_d \geq U_d$
HYD	Heave / piping	Excavation base	DA1-C1	γ_G = 1.35; γ_U = 1.35; γ_R = 1.00	FOS ≥ 1.0 at ULS

A.2.3 GEO – Geotechnical Limit State

Purpose: Prevent failure or excessive deformation of soil or rock mass supporting the structure.

Verification Equation:

$$E_d \leq R_d = \frac{R_k}{\gamma_R}$$

Typical checks:

- Bearing resistance:

$$q_d = \frac{q_k}{\gamma_{\phi'} \text{ and } \gamma_{c'}}$$

- Sliding resistance:

$$V_d \leq \mu' \cdot N_d$$

- Passive resistance:

$$P_p / \gamma_{\phi'} \geq E_h$$

Parameter	Partial Factor	UK NA Value
γ_ϕ'	1.25	Strength reduction
γ_c'	1.40	Cohesion reduction
γ_{cu}	1.40	Undrained shear strength
γ_R	1.00	Resistance factor

A.2.4 STR – Structural Limit State

Purpose: Ensure geotechnical structural elements (piles, walls, anchors, slabs) resist applied design actions without structural failure.

Verification Equation:

$$S_d \leq R_{d,\text{structural}}$$

Material	Code Reference	γ_M	Typical Design Check
Reinforced concrete	EN 1992-1-1	γ_c = 1.50; γ_s = 1.15	Flexure, crack width, durability
Structural steel	EN 1993-5	γ_{M0} = 1.00–1.10	Section bending and shear

Ground anchor tendon	EN 1997-1 §8.5	γ_M = 1.15–1.25	Ultimate tensile capacity
Sheet pile	EN 1993-5	γ_{M1} = 1.10	Bending moment and shear

A.2.5 EQU – Equilibrium Limit State

Purpose: Prevent overall instability (e.g. rotation, toppling, global sliding) where both soil and structure remain essentially rigid.

Verification Equation:

$$\Sigma M_{stabilising,d} \geq \Sigma M_{destabilising,d}$$

or

$$\Sigma V_{resisting,d} \geq \Sigma V_{driving,d}$$

Action Type	Partial Factor (γ_F)
Permanent (unfavourable)	1.35
Permanent (favourable)	1.00
Variable	1.50

The EQU check normally corresponds to **DA1-C1** with characteristic soil strengths.

A.2.6 UPL – Uplift Limit State

Purpose: Prevent flotation or uplift of structures subjected to hydrostatic pressure.

Verification Equation:

$$\Sigma W_d + \Sigma R_d \geq \gamma_U \cdot U_k$$

where:

- W_d= design weight of structure (including overburden)
- R_d= design anchorage or friction resistance
- U_k= characteristic uplift force

Parameter	Typical Factor	Notes
$\gamma_{G\,(unfavourable)}$	1.35	Weight of structure (unfavourable)
$\gamma_{G\,(favourable)}$	1.00	Weight acting against uplift
γ_U	1.35	Applied to water uplift

γ_R	1.00	Resistance factor

A.2.7 HYD – Hydraulic Limit State

Purpose: Prevent hydraulic heave, piping, or internal erosion beneath excavations or cut-offs.

Verification Equation:

$$i_{c,d} \leq i_{allowable} = \frac{\gamma_s'}{\gamma_w}$$

or equivalently,

$$\Sigma W_d \geq \gamma_U \cdot \Sigma U_d$$

Parameter	Factor	Typical Value
γ_U	1.35	Design uplift or seepage pressure
γ_R	1.00	Resistance factor
γ_G	1.35	Permanent load for stability

Verification often involves **critical hydraulic gradient**:

$$i_c = \frac{\gamma_s'}{\gamma_w}$$

and safety ratio:

$$F_{HYD} = \frac{i_c}{i_d} \geq 1.0$$

A.2.8 Recommended ULS Safety Margins

Design Case	Verification Format	Minimum Target Ratio
Bearing capacity (GEO)	$q_d \geq q_{d,applied}$	≥ 1.0
Sliding / overturning (EQU)	$V_{res} \geq V_{drive}$	≥ 1.0
Hydraulic heave (HYD)	FHYD = i_c / i_d	≥ 1.0
Uplift (UPL)	$W_d \geq U_d$	≥ 1.0
Pile structural capacity (STR)	$N_d \leq R_d$	≥ 1.0

A.2.9 Integration with Serviceability Checks

EC7 requires that SLS checks are carried out **in parallel** with ULS verification.

Typical workflow (DA1 context):

1. **ULS** – Apply partial factors (DA1-C1 / DA1-C2).
2. **SLS** – Re-run using characteristic values ($\gamma = 1.0$).
3. **Compare displacements, settlements, or pore pressures** to allowable limits.

Phicon Note:

• Each limit state represents a *different failure mechanism* — always identify the correct governing one before applying factors.
• GEO failures are governed by **soil resistance**; EQU by **global stability**; STR by **structural strength**.
• UPL and HYD checks must include **pore pressure verification** using field measurements where possible.
• Never mix factor sets between checks — use C1 for equilibrium, C2 for resistance.
• Document limit-state compliance in tabular form for each foundation or wall; this becomes the traceable verification summary within the GDR.
• Phicon design sheets automatically apply factor combinations per limit state and highlight governing condition.

A.3 Recommended Tolerances for Construction

Construction tolerances define the **allowable deviations** from design dimensions, levels, and alignments that can occur during geotechnical works **without compromising safety or serviceability**.

They ensure that constructed geometry remains compatible with the assumptions used in **Eurocode 7 design verification (EN 1997-1 §4.1)** and associated British Standards such as **BS 8004:2015**, **BS EN 1536:2010 (bored piles)**, and the **ICE Specification for Geotechnical Works (2017)**.

⚠️ Tolerances are not arbitrary allowances — they are **design-linked control limits** that must be enforced through QA inspection and survey verification.

A.3.1 General Principles

- Tolerances apply to **final as-built positions**, not temporary stages.
- Deviations beyond tolerance must trigger **engineering assessment** (potential NCR).
- Cumulative effects (e.g. multiple piles out of position) must be assessed collectively.
- Where tighter tolerances are required (e.g. under existing structures), these must be specified on drawings and ITPs.

A.3.2 Foundations – Pads, Strips and Rafts

Parameter	Typical Tolerance (mm)	Reference / Notes
Plan position (pad centreline)	±50	BS 8004:2015, Cl. 7.3
Width / length of pad	±50	ICE Spec §7.4

Level of top of foundation	±15	At finished blinding or bearing surface
Thickness of foundation	±10	Structural control tolerance
Level of underside	±25	For bearing control or anchor interface
Surface flatness	±10	Over 3 m straightedge
Edge batter / slope	±5 %	Based on design inclination
Reinforcement cover	±10	EN 1992-1-1 (concrete durability)

Where eccentricity or rotation tolerance exceeds design assumptions, re-verification under EQU may be required.

A.3.3 Piles (Bored, CFA, Driven)

Parameter	Tolerance	Reference / Notes
Pile head position (plan)	±75 mm (isolated) / ±50 mm (group)	BS EN 1536:2010 §11.2
Pile verticality (L/H ratio)	1:75 (normal) / 1:100 (critical)	ICE Spec §9.2
Cut-off level	±25 mm	After trimming
Pile length	+300 / –100 mm	Unless founded on rock socket
Pile diameter	±5 %	As-constructed check
Reinforcement cage alignment	±50 mm lateral	Before concreting
Test pile head elevation	±5 mm	For load testing datum consistency

A.3.4 Retaining Walls and Embedded Elements

Parameter	Tolerance	Reference / Notes
Wall alignment (horizontal)	±25 mm over 10 m	BS 8002:2015 §8.4
Wall verticality	1:200	ICE Spec §12.3
Top of wall level	±10 mm	Structural tolerance
Base level	±25 mm	Excavation control
Panel joint width	±10 mm	For precast / diaphragm wall panels

Anchor head position	±50 mm	Relative to wall grid
Prop / strut level	±5 mm	To maintain design force geometry
Bored wall overlap (secant)	50 mm minimum	QA verification at each panel

Excessive deviation in wall alignment directly affects bending and earth pressure distribution — reassess STR and GEO checks if limits exceeded.

A.3.5 Earthworks and Platforms

Parameter	Tolerance	Reference / Notes
Formation level	±25 mm (critical) / ±50 mm (general)	SHW Series 600
Compacted layer thickness	±10 %	Relative to design lift
Slope gradient	±2 % (engineered)	As-built survey confirmation
Batter position	±150 mm	At toe or crest
Working platform level (rig / crane)	±25 mm	BRE 470, Table 2
Surface evenness	≤ 25 mm under 3 m straightedge	Visual QA check
CBR / EV_2 target	As specified (±10 %)	Field testing verification

A.3.6 Structural / Interface Elements

Element	Tolerance	Notes
Anchor inclination	±2°	Relative to design angle
Anchor head projection	±10 mm	For stressing equipment alignment
Tie rod line	±10 mm	Horizontal alignment
Drain / weephole location	±50 mm	Consistent spacing and fall
Waterproofing termination	±10 mm	Interface QA inspection
Instrument reference point	±5 mm	To ensure monitoring repeatability

A.3.7 Slope and Embankment Construction

Parameter	Tolerance	Reference / Notes

Embankment height	±50 mm	Measured at crest or midpoint
Side slope	±0.5°	Visual survey or total-station check
Crest width	±50 mm	Geometric control
Toe position	±100 mm	From design line
Surface settlement marker elevation	±5 mm	For monitoring baseline

A.3.8 QA Verification and Non-Conformance Triggers

Tolerance checks form part of the **Inspection & Test Plan (ITP)**.
Exceedance of tolerance requires:

1. Immediate NCR or observation record.
2. Assessment of impact on limit state assumptions (GEO, EQU, STR).
3. Designer review and acceptance or instruction for correction.
4. Updated as-built survey to confirm rectification.

DEVIATION TYPE	TRIGGER ACTION
≤ TOLERANCE	Acceptable – record QA pass
SLIGHT EXCEEDANCE	Observation – monitor and record
MAJOR EXCEEDANCE	NCR – suspend works and seek design review

A.3.9 Survey & Verification Methodology

- All measurements referenced to a control grid and permanent benchmark.
- As-built surveys to **±10 mm vertical** and **±20 mm horizontal** accuracy using total station or 3D scan.
- Survey data logged in QA database (chainage, date, element ID).
- Tolerances should be displayed on **site drawings and dashboards** for visibility during inspection.

Phicon Note:

▪ Construction tolerances are a **design control tool**, not a workmanship allowance — they maintain compatibility with EC7 verification.
▪ Always relate tolerances to **design assumptions** (e.g., eccentricity limits, bearing depth, wall toe).
▪ QA inspectors should check *functionally critical dimensions first* — not purely geometric neatness.
▪ For working platforms and founding strata, use **both geometric and performance tolerances** (EV_2, CBR).
▪ Embed tolerances into ITPs, survey forms, and Power BI dashboards for traceable compliance.
▪ Phicon digital templates auto-flag exceedances for NCR issue and designer notification.
▪ Tolerances exceeding design limits without assessment are treated as **non-conformance** under EC7 §4.

APPENDIX B – SOIL PROPERTIES & CORRELATIONS

This appendix provides indicative correlations between **index, strength, and stiffness parameters** used in Eurocode 7–based geotechnical design.

Values represent **characteristic ranges** derived from UK & European practice (Tomlinson 1983 [1]; Clayton et al. 1995 [2]; BS 5930 [3]; BS EN ISO 14688 [4]; NAVFAC DM-7 [5]).

They are intended to guide preliminary design and parameter selection pending site-specific test data.

B.1 Typical Soil Parameters (c', ϕ', cu, E, ν)

Soil Type	Density γ (kN/m^3)	Effective Cohesion c' (kPa)	Friction Angle ϕ' (°)	Undrained Strength cu (kPa)	Young's Modulus E (MPa)	Poisson's Ratio ν	Typical References
Soft clay	15–17	0 – 5	20–23	20–50	3–8	0.45	[1],[2],[3]
Firm clay	17–19	0 – 10	22–26	50–100	8–20	0.45	[1],[2]
Stiff clay	19–20	5 – 15	25–30	100–200	20–60	0.40	[1],[3]
Very stiff / hard clay	20	10–20	27–32	> 200	60–120	0.35	[1],[5]
Loose sand	16–18	0	28–32	—	10–25	0.30	[2],[3]
Medium dense sand	17–19	0	32–36	—	25–60	0.30	[1],[2]
Dense sand	19–21	0	36–42	—	60–150	0.28	[1],[4]
Gravel (well-graded)	20–22	0	38–45	—	100–200	0.25	[1],[3],[5]
Silt (low plasticity)	17–19	0 – 5	27–32	30–80	5–20	0.40	[2],[3]
Peat / organic soil	10–14	0	15–25	10–30	1–3	0.45–0.50	[2],[3]

⚠️ Adopt the **lower quartile** of published ranges for characteristic (ck) design; upper quartile for back-analysis.

B.2 Empirical Correlations (SPT N, CPT q_c, PLT EV_2)

a) SPT N Correlations

Parameter	Empirical Correlation	Typical Range	Notes / Reference
ϕ' (°) – sand	ϕ' = 27 + 0.3 N_{60} (≤ 40°)	N_{60} = 10 → 30°; 30 → 36°	Peck et al. 1974 [6]; BS 8004 App.C
cu (kPa) – clay	cu = 5 × N_{60} (kPa)	± 25 % scatter	Stroud 1974 [7]
E' (MPa) – clay	E' = 250 × cu (kPa)	Soft–stiff clays	[7],[3]
E' (MPa) – sand	E' = (2–5) × qc / 1000	qc in kPa	[8],[1]

Use **energy-corrected N_{60}** (ASTM D6066; BS EN ISO 22476-3).

b) CPT qc Correlations

Parameter	Correlation	Reference / Comment
ϕ' (°) – sand	ϕ' = 17.6 + 11 × $\log_{10}$(qc / pa)	Robertson & Campanella 1983 [9]
cu (kPa) – clay	cu = (qc – σvo) / Nkt ; Nkt = 15–20	BS EN ISO 22476-1; Lunne et al. 1997 [10]
OCR – clay	OCR ≈ 0.5 × $(qc / pa)^{0.5}$	Lunne & Christoffersen 1983 [10]
Es (MPa) – sand	Es ≈ 2.5 × (qc / 1000)	De Ruiter & Beringen 1979 [11]

c) Plate Load Test (PLT EV_2) Correlations

Correlation	Equivalent Property	Reference / Range
EV_2 = 1.5 × EV_1 (minimum ratio per DIN 18134)	Elastic rebound criterion	DIN 18134 (2012)
EV_2 ≈ 2.5 × $E_o$ed	Settlement modulus comparison	Meyer & Schmidt 1978 [12]
EV_2 / CBR ≈ 17 – 22	CBR estimation	TRL Report 1132 (UK HA 68/94)
E' ≈ EV_2 / (1 – v^2)	Equivalent elastic modulus	Hooke conversion

Field QA: EN 1997-2 Table A.16 suggests EV_2 ≥ 120 MPa for piling platforms (> 40 t rig).

B.3 Earth Pressure Coefficients (K_0, K_a, K_p)

Soil Type	K_0 (At Rest) **(Jaky)**	K_a **(Rankine)**	K_p **(Rankine)**	Reference
Normally consolidated clay	1 – sin ϕ' ≈ 0.6–0.7	$\tan^2$(45 – ϕ'/2)	$\tan^2$(45 + ϕ'/2)	Jaky 1944 [13]

Overconsolidated clay (OCR = 4)	$K_0 \approx (OCR)^{0.5}(1 - \sin\phi')$	—	—	Wroth 1975 [14]
Dense sand (ϕ' = 38°)	0.45	0.24	4.2	[1],[2]
Loose sand (ϕ' = 30°)	0.67	0.33	3.0	[1],[5]

B.4 Subgrade Modulus and EV_2–CBR Conversion

Test Parameter	Correlation / Conversion	Reference
EV_2 (MPa) ≈ 17 × CBR (%)	UK Highways Agency HA 68/94 [15]	
ks (kN/m^3) ≈ 40 × CBR (%)	Bowles 1988 [16]	
E (MPa) ≈ 10 × CBR (%)	Tomlinson 1983 [1]	
k30 ≈ EV_2 / 30	Simplified Westergaard approximation	[16]

B.5 Typical Modulus Ratios and Relationships

Parameter Relationship	Typical Range	Reference
Eoed / Eu	0.25 – 0.5	Clayton et al. 1995 [2]
E50 / Eoed	1.5 – 2.0	Schmertmann 1978 [17]
E' / qc	2.5 – 5.0 MPa/MPa	De Ruiter & Beringen 1979 [11]
v (granular)	0.25 – 0.35	BS 8002 [18]
v (cohesive)	0.40 – 0.48	[2],[3]

Phicon Note:

• Correlations are **indicative**, not substitutes for testing — apply conservatively for preliminary design.
• Use site-specific data wherever available; field and lab tests should take precedence over empirical trends.
• When deriving design values, apply **Eurocode 7 partial factors** to parameters, not to correlations.
• Document all correlation sources within the GDR for audit traceability.
• Avoid extrapolating beyond the correlation's empirical range — especially for highly variable soils (chalk, glacial tills).
• For chalk and weak rock, refer also to BS EN 1997-2 Annex H and CIRIA C574.
• Phicon calculation sheets embed these correlations with citation tags for each parameter used.

References

[1] Tomlinson M.J. (1983) *Foundation Design and Construction*, 5th Ed., Longman.
[2] Clayton C.R.I., Matthews M.C., Simons N.E. (1995) *Site Investigation*, Blackwell Science.
[3] BS 5930 (2015) *Code of Practice for Ground Investigations*.
[4] BS EN ISO 14688-1 & -2 (2018) *Soil Classification for Engineering Purposes*.
[5] NAVFAC DM-7 (1982) *Soil Mechanics Design Manual 7.01*.
[6] Peck, Hanson & Thornburn (1974) *Foundation Engineering*.
[7] Stroud M.A. (1974) "The Interpretation of SPT Tests", *Géotechnique*, Vol. 24(4).
[8] Lambe & Whitman (1979) *Soil Mechanics*.
[9] Robertson & Campanella (1983) *Canadian Geotechnical Journal*, 20(4).
[10] Lunne, Robertson & Powell (1997) *CPT in Geotechnical Practice*.
[11] De Ruiter & Beringen (1979) *Proc. 7th ECSMFE, Brighton*.
[12] Meyer & Schmidt (1978) *Die Plattendruckversuch nach DIN 18134*.
[13] Jaky J. (1944) *J. Soc. Hungarian Architects & Eng.*, 78(22).
[14] Wroth C.P. (1975) *Géotechnique*, 25(1).
[15] Highways Agency (1994) *HA 68/94 – Design Methods for CBR and* EV_2.
[16] Bowles J.E. (1988) *Foundation Analysis and Design*, McGraw-Hill.
[17] Schmertmann J.H. (1978) *Settlement of Shallow Foundations on Sand*.
[18] BS 8002 (2015) *Earth Retaining Structures – Design Code*.

APPENDIX C – CONVERSION FACTORS & CONSTANTS

This appendix provides **standard metric units, conversion factors, and physical constants** commonly used in geotechnical design and reporting.
All values are consistent with **SI conventions** defined in **BS 350: Part 1**, **BS EN ISO 80000-1:2020**, and **Eurocode 7 Annex A (EN 1997-1)**.

⚠️ All calculations, reports, and QA documents within Phicon practice use **SI base units** as per Eurocode 0 (EN 1990) — conversions are provided only for reference or legacy datasets.

C.1 Metric (SI) Units for Geotechnical Use

Quantity	Symbol	Base Unit	Equivalent SI Expression
Length	L	metre (m)	—
Area	A	m^2	—
Volume	V	m^3	—
Mass	m	kilogram (kg)	—
Force	F	newton (N)	1 N = 1 kg·m/s^2
Stress / Pressure	σ, p	pascal (Pa)	1 Pa = 1 N/m^2
Density	ρ	kg/m^3	—
Unit Weight	γ	kN/m^3	1 kN/m^3 = 1000 N/m^3
Time	t	second (s)	—
Angle	ϕ, θ	degree (°) or radian (rad)	1 rad = 180/π°
Modulus	E	MPa	1 MPa = 10^6 Pa
Energy	W	joule (J)	1 J = 1 N·m
Velocity	v	m/s	—
Acceleration	a	m/s^2	—
Permeability	k	m/s	—
Strain	ε	dimensionless	—
Poisson's Ratio	ν	dimensionless	—
Hydraulic Gradient	i	dimensionless	—

Temperature	T	°C	—

C.2 Conversion Factors (Imperial ↔ Metric)

Quantity	To Convert From (Imperial)	Multiply By	To Obtain (Metric)
inch (in)	—	25.4	millimetre (mm)
foot (ft)	—	0.3048	metre (m)
yard (yd)	—	0.9144	metre (m)
mile	—	1.609	kilometre (km)
square foot (ft^2)	—	0.0929	square metre (m^2)
cubic foot (ft^3)	—	0.0283	cubic metre (m^3)
cubic yard (yd^3)	—	0.7646	cubic metre (m^3)
gallon (UK)	—	4.546	litre (L)
pound (lb)	—	0.4536	kilogram (kg)
ton (UK long)	—	1.016	tonne (t)
psi	—	6.895	kN/m^2 (kPa)
ton/ft^2	—	95.76	kN/m^2 (kPa)
ft of water	—	2.989	kN/m^2 (kPa)
in of water	—	0.249	kN/m^2 (kPa)
pcf (lb/ft^3)	—	0.1571	kN/m^3
kip (1000 lb)	—	4.448	kN
°F to °C	—	(°F – 32) × 5/9	°C

C.3 Physical Constants & Material Densities

a) Universal Constants

Quantity	Symbol	Value	Unit

Acceleration due to gravity	g	9.80665	m/s^2
Density of water (fresh, 4°C)	ρw	1000	kg/m^3
Unit weight of water	γw	9.81	kN/m^3
Atmospheric pressure	patm	101.325	kPa
π (pi)	π	3.1416	—
Conversion: 1 MPa	—	1000	kN/m^2
Gas constant for air	R	287	J/kg·K

b) Typical Material Densities and Unit Weights

Material	Density ρ (kg/m^3)	Unit Weight γ (kN/m^3)	Reference / Note
Fresh water	1000	9.81	Standard reference
Sea water	1030	10.1	Salinity 35 ppt
Air (at 20°C, 1 atm)	1.2	0.012	—
Dry sand	1600–1800	15.7–17.7	[1],[2]
Saturated sand	1900–2100	18.6–20.6	[1],[2]
Soft clay (saturated)	1500–1700	14.7–16.7	[3]
Stiff clay	1800–2000	17.7–19.6	[3]
Gravel	2000–2200	19.6–21.6	[1],[2]
Crushed rock	2200–2400	21.6–23.6	[1]
Concrete	2400	23.5	EN 1992-1-1 Table 3.1
Reinforcing steel	7850	77.0	EN 1993-1-1 Table 3.1
Structural steel	7850	77.0	—
Granite	2650	26.0	[2],[3]
Chalk (moderate density)	1700–2100	16.5–20.5	CIRIA C574
Peat / organic soil	900–1200	8.8–11.8	[2],[3]

Bituminous surfacing	2300	22.6	BS 594987:2015

c) Derived Constants for Geotechnical Analysis

Property	Typical Symbol / Formula	Value / Range	Unit	Comment
Unit weight of dry soil	$\gamma_d = (G_s \gamma_w) / (1 + e)$	13–19	kN/m^3	$G_s = 2.65$ typical
Submerged unit weight	$\gamma' = \gamma_{sat} - \gamma_w$	8–10	kN/m^3	For buoyancy corrections
Hydraulic gradient (critical)	$i_c = \gamma' / \gamma_w$	0.9–1.1	—	For piping check
Earth pressure coefficient (Jaky)	$K_0 = 1 - \sin\phi'$	0.4–0.7	—	For NC soils
Specific gravity of solids	G_s	2.65 ± 0.05	—	Silicate minerals
Conversion: kPa to m head of water	$h = p / (\gamma_w)$	0.102	m/kPa	10 m ≈ 98.1 kPa

d) Temperature and Viscosity Relationships (Hydraulic Use)

Temperature (°C)	Dynamic Viscosity μ ($\times 10^{-3}$ N·s/m^2)	Kinematic Viscosity ν = μ / ρw ($\times 10^{-6}$ m^2/s)
0	1.79	1.79
10	1.31	1.31
20	1.00	1.00
30	0.80	0.80
40	0.66	0.66

Used in seepage and permeability corrections (Darcy's law temperature adjustment).

e) Energy and Pressure Equivalence Table

Pressure / Stress (kPa)	Equivalent Head (m water)	Equivalent Stress (ton/ft^2)	Equivalent Energy (J/m^3)
10	1.02	0.10	1.0×10^4

50	5.10	0.50	5.0×10^4
100	10.2	1.0	1.0×10^5
500	51	5.0	5.0×10^5
1000	102	10.0	1.0×10^6

Phicon Note:

• Always maintain **unit consistency** between input and derived parameters — mixing kN/m^2 and MPa remains the most common error in site calculations.
• Adopt **kN, m, and MPa** as PHICON standard working units across all EC7 calculation sheets.
• Use γw = 9.81 kN/m^3 consistently — not rounded to 10 kN/m^3 unless in field calculations.
• When converting legacy imperial data, apply significant figures consistently (never mix rounded conversions).
• Calibration of permeability or pressure transducers should include **temperature and unit conversion corrections**.
• Phicon dashboards and templates enforce unit metadata for each parameter, ensuring automated QA traceability.

References

1. BS EN ISO 80000-1:2020 Quantities and Units – Part 1: General.
2. Tomlinson M.J. (1983) *Foundation Design and Construction*, 5th Ed., Longman.
3. Clayton C.R.I., Matthews M.C., Simons N.E. (1995) *Site Investigation*, Blackwell Science.
4. BS 350: Part 1 (2004) Conversion Factors and Units.
5. Eurocode 0 (EN 1990:2002) Basis of Structural Design.
6. CIRIA C574 (2002) Engineering in Chalk.

APPENDIX D – SYMBOLS, UNITS & NOTATION

This appendix compiles the **standardised symbols, abbreviations, and units** used throughout this Reference Guide and Phicon's geotechnical calculation suite.

It aligns with the conventions defined in:

- BS EN 1997-1:2004+A1:2013 (Eurocode 7 – Geotechnical Design)
- BS EN ISO 14688-1/2:2018 (Soil Classification)
- BS 5930:2015 (Code of Practice for Ground Investigation)
- ISO 80000 Series (Quantities and Units)

⚠️ Consistent notation is a prerequisite for traceable EC7 design. Each PHICON spreadsheet, report, and OneNote template uses this symbol set to ensure standardisation across the practice.

D.1 Geometrical and Physical Quantities

Symbol	Parameter	Typical Unit	Remarks
L	Length	m	General dimension
B	Foundation width	m	Shorter plan dimension
L_f	Foundation length	m	Longer plan dimension
H	Depth / height	m	From surface to point of interest
z	Depth below surface	m	Vertical coordinate
A	Area	m^2	Plan area or section
V	Volume	m^3	—
α	Inclination angle	°	Of load or surface
β	Ground slope angle	°	From horizontal
θ	Rotation / deflection angle	°	—
π	Pi constant	3.1416	—

D.2 Material and Index Properties

Symbol	Parameter	Typical Unit	Remarks
w	Natural water content	%	(mass of water / dry mass) × 100

G_s	Specific gravity of solids	—	2.65 typical for mineral soils
e	Void ratio	—	Volume of voids / solids
n	Porosity	%	Volume of voids / total volume
S_r	Degree of saturation	%	(Vw / Vv) × 100
γ	Total unit weight	kN/m^3	Includes air + water
γ_d	Dry unit weight	kN/m^3	$= \gamma / (1 + w)$
γ_{sat}	Saturated unit weight	kN/m^3	—
γ'	Submerged (effective) unit weight	kN/m^3	$= \gamma_{sat} - \gamma_w$
γ_w	Unit weight of water	$9.81\ kN/m^3$	Reference constant
ρ	Density	kg/m^3	—
LL	Liquid limit	%	Atterberg limit test
PL	Plastic limit	%	—
PI	Plasticity index	%	= LL – PL
wL, wP	Water content at limits	%	—

D.3 Strength and Stress Parameters

Symbol	Parameter	Typical Unit	Remarks
σ	Total stress	kPa	= total pressure on soil element
σ'	Effective stress	kPa	$= \sigma - u$
τ	Shear stress	kPa	—
c'	Effective cohesion	kPa	For drained condition
c_u	Undrained shear strength	kPa	For total stress analysis
ϕ'	Effective friction angle	°	Peak or critical state
ϕ_u	Total stress friction angle	°	Undrained equivalent
δ	Interface friction angle	°	Soil–structure interaction
K_0	At-rest earth pressure coeff.	—	$= 1 - \sin\phi'$ (Jaky)

K_a	Active earth pressure coeff.	—	Rankine / Coulomb
K_p	Passive earth pressure coeff.	—	Rankine / Coulomb
OCR	Overconsolidation ratio	—	= σ'p / σ'v
σ'_p	Preconsolidation pressure	kPa	Oedometer test
s_u	Undrained shear strength	kPa	Alternative notation for cu

D.4 Stiffness and Compressibility Parameters

Symbol	Parameter	Typical Unit	Remarks
E'	Drained Young's modulus	MPa	Used in elastic analyses
E_u	Undrained Young's modulus	MPa	For total stress analysis
E_s	Modulus of subgrade reaction	kN/m^3	For beams / slabs on grade
E_{oed}	Oedometer modulus	MPa	From consolidation curve
m_v	Coefficient of volume compressibility	m^2/MN	= Δεv / Δσ'
C_v	Coefficient of consolidation	m^2/s	Time–rate of settlement
C_c	Compression index	—	From e–logσ' plot
C_s	Swelling (recompression) index	—	—
v	Poisson's ratio	—	0.25–0.48 typical
κ	Elastic unloading parameter	—	For critical state models

D.5 Hydraulic and Flow Parameters

Symbol	Parameter	Typical Unit	Remarks
k	Coefficient of permeability	m/s	—
i	Hydraulic gradient	—	= Δh / L
h	Hydraulic head	m	= ψ + z
u	Pore water pressure	kPa	= γw × hw
h_w	Pressure head	m	Water column equivalent

q_w	Seepage discharge per unit width	m^2/s	Darcy flow rate
n	Porosity	—	Volume fraction of voids
γ_w	Unit weight of water	9.81 kN/m^3	—
i_c	Critical hydraulic gradient	—	= γ' / γw

D.6 Loading and Design Variables

Symbol	Parameter	Typical Unit	Remarks
F	Force	kN	General load symbol
Q	Variable (live) load	kN	—
G	Permanent (dead) load	kN	—
M	Moment	kN·m	—
H	Horizontal load	kN	—
V	Vertical load / shear	kN	—
e	Load eccentricity	m	M / V
γ_F	Partial factor on actions	—	EC7 Table A.1
γ_M	Partial factor on materials	—	EN 1993 / EN 1992
γ_R	Partial factor on resistance	—	EC7 Annex A
E_d	Design effect of actions	—	Factored
R_d	Design resistance	—	Factored
ψ	Combination factor	—	EN 1990 Table A.1.1
ξ	Model uncertainty factor	—	Optional in reliability formats

D.7 Settlement and Deformation Parameters

Symbol	Parameter	Typical Unit	Remarks
s	Total settlement	mm	—
s_i	Immediate (elastic) settlement	mm	From elastic theory

s_c	Consolidation settlement	mm	From oedometer curve
s_s	Secondary (creep) settlement	mm	—
Δ	Differential settlement	mm	Between two points
ρ	Rotation due to differential settlement	radians	—
δ_h	Lateral wall movement	mm	For retaining structures
w_{max}	Maximum deflection	mm	For SLS checks

D.8 Dimensionless Coefficients and Ratios

Symbol	Parameter	Typical Range	Notes
N_c, N_q, N_γ	Bearing capacity factors	5–120	Depends on ϕ'
α, β	Adhesion / friction coefficients	0.5–1.0	For pile design
N_{kt}	CPT cone factor	10–20	For cohesive soils
M	Slope stability moment factor	—	Derived numerically
μ	Friction coefficient	0.3–0.6	tanδ
r	Radius of influence	m	For settlement zone
F_s	Factor of safety	≥ 1.0 (ULS)	Or >1.3 (SLS)
η	Efficiency factor	—	For group or load distribution

D.9 Abbreviations and Acronyms

Abbreviation	Meaning
EC7	Eurocode 7 – Geotechnical Design
GDR	Geotechnical Design Report
GI	Ground Investigation
CPT	Cone Penetration Test
SPT	Standard Penetration Test
PLT	Plate Load Test

PMT	Pressuremeter Test
NDG	Nuclear Density Gauge
MCV	Moisture Condition Value
WPL	Working Platform Load
BRE	Building Research Establishment
QA / QC	Quality Assurance / Quality Control
NCR	Non-Conformance Report
DA1–3	Design Approaches 1–3 (EC7)
GEO / STR / EQU / UPL / HYD	EC7 Limit States
OCR	Overconsolidation Ratio
OMC	Optimum Moisture Content
EV_2	Second load cycle modulus (PLT)
CBR	California Bearing Ratio
FOS	Factor of Safety
DMT	Dilatometer Test
PVD	Prefabricated Vertical Drain
DCP	Dynamic Cone Penetrometer
PPR	Pore Pressure Ratio (CPT)

Phicon Note:

• Each Phicon calculation sheet automatically labels input and derived parameters using these symbols — ensuring compatibility across Excel, Word, and OneNote deliverables.
• Follow this notation consistently when preparing interpretative reports and design summaries; it aligns with EC7 clause numbering and BS 5930 definitions.
• In combined analyses (e.g. FE models), document all symbol definitions in the input sheet to ensure traceability.
• Avoid mixing ϕ' and ϕu, or c' and cu — always state whether analysis is **drained or undrained**.
• All graphs, charts, and Excel plots should label parameters using **standard EC7 notation** for consistency with design verification reports. Phicon standard templates apply this naming convention across:

-Bearing capacity, slope stability, and wall design spreadsheets

-OneNote calculation books

-Word report macros for GDR appendices

APPENDIX E – EC7 DESIGN WORKFLOW CARDS (UK NA & BE ANB)

These workflow cards condense the **Eurocode 7 (EN 1997-1)** design process into **traceable, step-by-step verification sequences** for everyday use in foundation, retaining, and slope design.

They distinguish between **Design Approach 1 (DA1 – UK default)** and **alternative National Annex practices** (e.g. Belgium, Germany, Netherlands).

Each workflow represents the structure of Phicon's digital calculation sheets. The intent is to make Eurocode verification *visual, auditable, and fast to follow* — suitable for both spreadsheet and on-site verification.

E.1 Overview of the EC7 Design Process

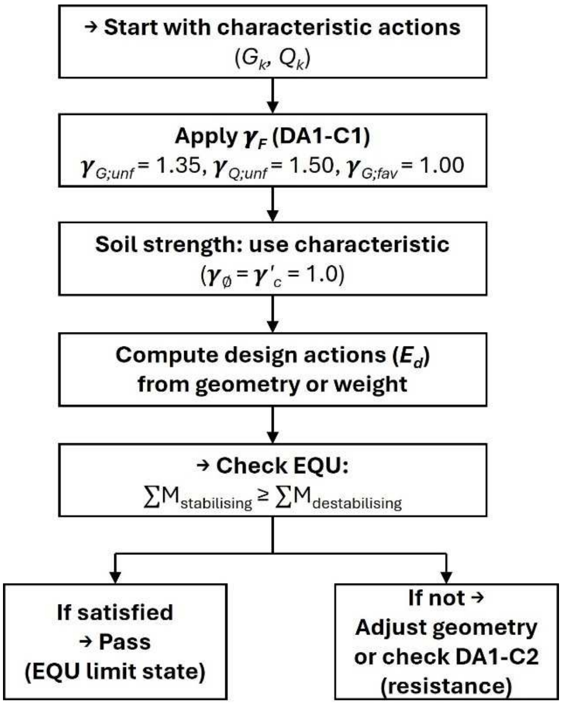
Stage	Action	Description
1	Define problem	Establish geometry, loading, and limit state to be checked (GEO, STR, EQU, UPL, HYD).
2	Identify soil parameters	Derive characteristic parameters (c'_k, ϕ'_k, cu_k, γ_k, E_k, etc.) from test data.
3	Select Design Approach	Apply DA1 (UK default) – Combination 1 (C1) or Combination 2 (C2) as relevant.
4	Apply partial factors	Modify actions, soil parameters, or resistances according to EN 1997-1 Annex A.
5	Perform analysis	Compute effects of actions (Ed) and resistances (Rd).
6	Verify limit state	Check Ed ≤ Rd (ULS) or serviceability limits (SLS).
7	Document results	Record characteristic inputs, design values, combination used, and governing case.

E.2 Design Approach 1 – UK National Annex Workflow (Default)

Combination 1 (C1) – EQU / Sliding / Overturning Checks

Purpose: Verifies *equilibrium of actions* where failure is governed by overall stability or geometry (not soil strength).

Workflow Card:

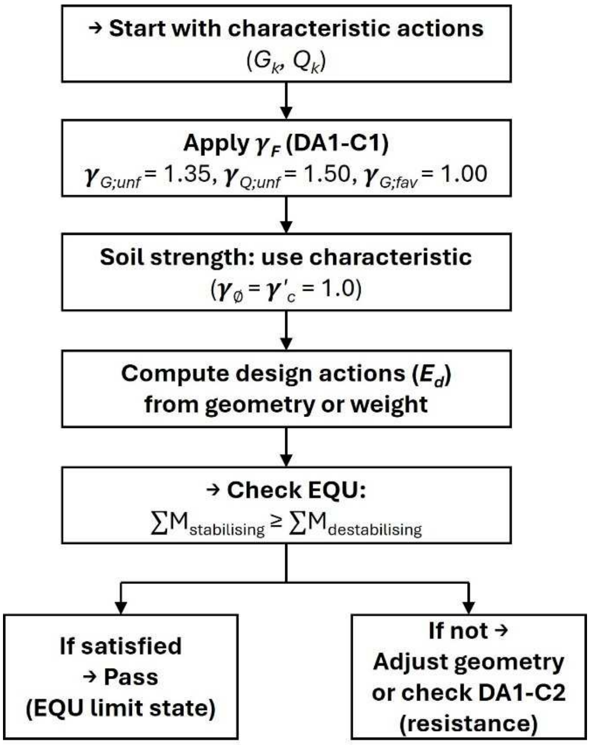


Typical applications:

- Retaining wall overturning or sliding
- Global stability of wall + soil block
- Uplift checks for basements
- Temporary earthwork equilibrium
- Combination 2 (C2) – GEO Resistance Checks

Purpose: Verifies *soil resistance* (bearing, passive, sliding) where failure occurs in the ground mass.

Workflow Card:



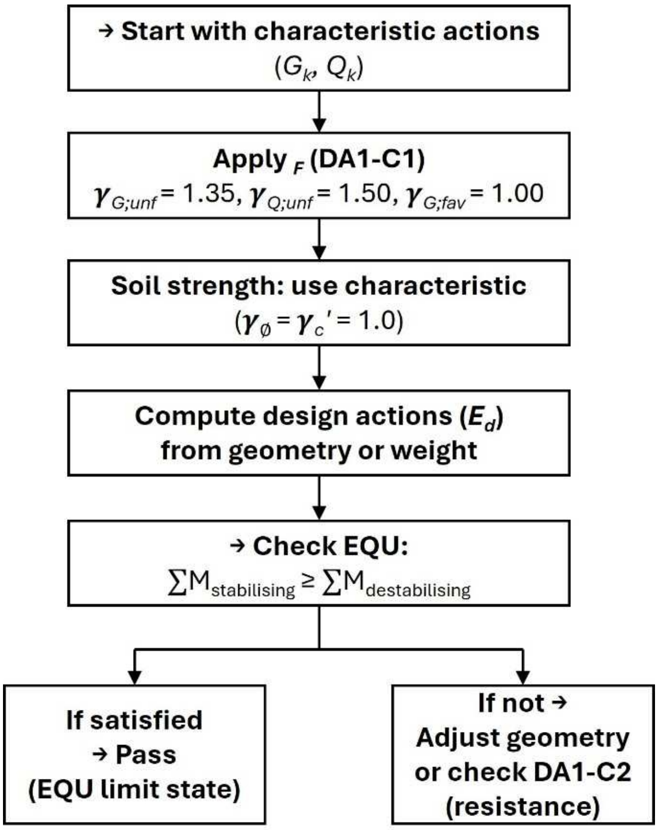
Typical applications:

- Bearing capacity of footings and piles
- Passive resistance of retaining walls
- Sliding along soil–structure interface
- Pile shaft or base resistance

E.3 Structural Limit State (STR) Workflow

Purpose: Ensures geotechnical structures themselves have adequate **section capacity** under design actions.

Workflow Card:

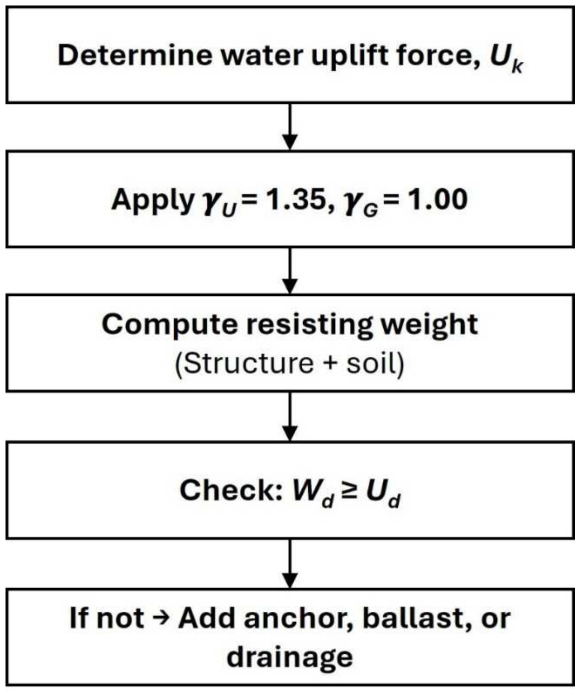


Typical applications:

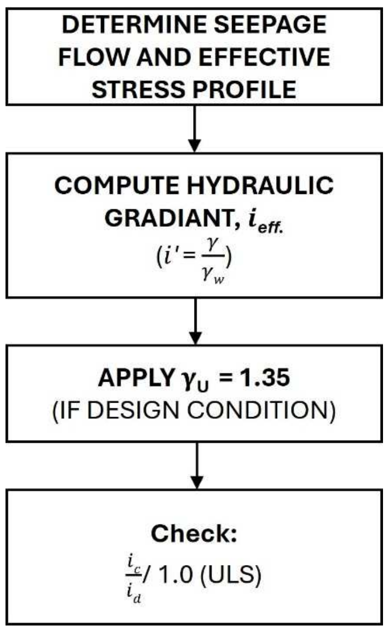
- Sheet pile bending
- RC wall or raft flexure
- Anchor and tie bar capacity

E.4 Uplift and Hydraulic (UPL / HYD) Workflow

a) Uplift (Basement / Raft)



b) Hydraulic Heave / Piping (Excavation Base)



→ Determine seepage flow and effective stress profile

↓

Compute hydraulic gradient i_{eff}

↓

Compare with critical gradient: $i_c = \gamma' / \gamma_w$

↓

Apply γ_U = 1.35 (if design condition)

↓

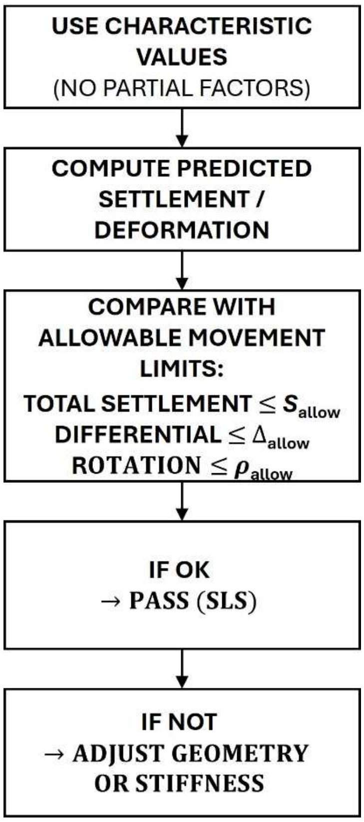
Check: $i_c/i_d \geq 1.0$ (ULS)

Typical applications: deep basements, cut-off walls, drained excavations.

E.5 Serviceability Limit State (SLS) Workflow

Purpose: Ensures that deformation, settlement, or deflection does not impair function or service life.

Workflow Card:



→ Use characteristic values (no partial factors)

↓

Compute predicted settlement / deformation

↓

Compare with allowable movement limits:

- Total settlement ≤ Sallow
- Differential ≤ Δallow
- Rotation ≤ ρallow

↓

If OK → Pass (SLS)

If not → Adjust geometry or stiffness

Typical limits (BS 8004:2015 Table 7):

Structure Type	Total (mm)	Differential (mm/m)
Isolated footings	25–50	1:500–1:300
Raft / mat foundation	50–100	1:500
RC framed structure	50	1:500
Masonry / brittle finish	25	1:750–1:1000

E.6 Alternative National Annex (BE / DE / NL) Workflows

Design Approach	Action Factors	Soil / Resistance Factors	Typical Use	Notes
DA2 (BE / DE / NL)	$\gamma_G = 1.35$, $\gamma_Q = 1.5$	$\gamma_R = 1.4$, $\gamma_{\phi'} = 1.25$	Retaining walls, slopes	Resistance factored directly
DA3 (Nordic)	$\gamma_G = 1.1$, $\gamma_Q = 1.3$	$\gamma_R = 1.3$	Temporary works	Simplified approach
UK NA (DA1)	$\gamma_G = 1.35$, $\gamma_Q = 1.5$ (C1) or 1.3 (C2)	$\gamma_{\phi'} = 1.25$, $\gamma_{c'} = 1.4$	Default (BS EN 1997-1)	No γ_R applied separately

⚠️ For cross-border designs, adopt the **most conservative** combination where annex uncertainty exists unless client or national code dictates otherwise.

E.7 Worked Example – Workflow Cross-Reference

Design Type	Limit State	Workflow	Combination / DA
Pad foundation bearing	GEO	§E.2 (C2)	DA1-C2
Retaining wall sliding	EQU	§E.2 (C1)	DA1-C1
Sheet pile bending	STR	§E.3	EN 1993-5
Basement uplift	UPL	§E.4a	DA1-C1
Excavation heave	HYD	§E.4b	DA1-C1
Settlement check	SLS	§E.5	Characteristic only

Phicon Note:

• EC7 requires both **combinations (C1 & C2)** under DA1 to be checked; the governing result controls design.
• Always record which **combination governs** and why — this provides audit trail compliance under EN 1997-1 §2.7.

• Design values must *always* be traceable from characteristic values; never enter pre-factored data.
• For field verification, PHICON spreadsheets automatically display both combinations side by side.
• Use **flowcharts** in project GDRs and OneNote pages for reviewer clarity — they help demonstrate EC7 compliance at a glance.
• Include partial factors (γF, γM, γR) visibly in outputs to meet EC7 §2.4.7 and UK NA audit expectations.

APPENDIX F – QA CHECKLISTS & INSPECTION FORMS

This appendix provides **ready-to-use quality assurance (QA)** and **inspection checklists** to support geotechnical construction verification under **Eurocode 7, BS 22475, BS 8004, BS EN 1536, and ICE Specification for Geotechnical Works (2017)**.

They are intended for use during **supervision, monitoring, and record documentation** phases as described in **EN 1997-1 §4.1–4.3** and **BS 5930:2015 Cl. 25**.

These checklists are formatted for field or digital entry — matching Phicon's QA templates used in OneNote and Power BI dashboards. They capture traceable parameters required for compliance and post-construction verification.

F.1 Pile Installation Verification Checklist

Scope: Applies to bored, CFA, and driven piles.

Verification must confirm pile geometry, material quality, and construction method in accordance with **BS EN 1536**, **ICE Spec §9**, and **EN 1997-1 §8.4**.

Item	Check Description	Acceptance / Criteria	Verification Method	Pass / Fail / Comment
Pile ID / Location	Position per grid coordinates	±50 mm (group) / ±75 mm (isolated)	Survey check	
Verticality	Measured deviation from plumb	≤ 1:75 (normal) / ≤ 1:100 (critical)	Inclinometer / spirit level	
Cut-off Level	Elevation after trimming	±25 mm	Survey	
Diameter	As per design	±5 % tolerance	Tape / calliper	
Boring Depth	Reaches design founding stratum	Within +300 / –100 mm	Log / measurement	
Cleanliness of Base	No loose debris or slurry	Visual / camera		
Concrete Volume	Theoretical + 5 % tolerance	Calculated vs. delivered		
Reinforcement	Length, cover, and cage position	Cover ≥ 50 mm	Visual before pour	
Concrete Cube Strength	≥ Specified fck	Test report		

Integrity Testing	Sonic / cross-hole	No major defect	Report review
Load Test (if applicable)	Meets test load criteria	No failure / ≤ specified settlement	Load-deflection plot
Record Sheet Complete	Date, rig, operator, conditions	Signed / dated	Supervisor QA

For CFA piles, record auger torque, penetration rate, and concrete pressure continuously (EN 1536 Annex B).

Ensure logs are exported to PDF and linked to the GDR as part of EC7 §8.4 verification.

F.2 Retaining Wall Monitoring Checklist

Scope: Embedded, cantilever, or anchored retaining walls.

Inspection confirms installation geometry, support forces, wall deflection, and pore pressure performance.

Item	Check Description	Acceptance / Criteria	Verification Method	Pass / Fail / Comment
Wall Alignment	Deviation from design line	±25 mm over 10 m	Survey	
Wall Verticality	Deviation from plumb	≤ 1:200	Survey / inclinometer	
Toe Embedment Depth	Achieved as per design	±100 mm	Boring log	
Support Installation	Level / pre-load of strut / anchor	Within ± 5 mm level	Gauge reading	
Anchor Test	Lock-off load vs. design	± 10 %	Test record	
Excavation Level	Stepwise sequence confirmed	Per method statement	Survey / log	
Inclinometer Reading	Deflection vs. predicted	≤ design limit (e.g. 25 mm)	Instrumentation	
Piezometer Reading	Pore pressure below design line	≤ target	Monitoring log	
Drainage Function	Weepholes / geocomposite clear	Flow confirmed	Visual	
Crack / Distress	Visual damage check	None	Inspection	

Record Complete	As-built drawing and instrument log	Checked	Supervisor sign-off	

Incorporate inclinometer and piezometer data into a **trigger-level dashboard** (Green / Amber / Red) aligned with the Observational Method (EN 1997-1 §2.7).

F.3 Working Platform Inspection (BRE 470 Verification)

Scope: Working platforms for tracked plant, piling rigs, and cranes. Verification ensures bearing strength, thickness, and drainage adequacy per **BRE 470 (2004)** and **ICE Spec §17**.

Item	Check Description	Acceptance / Criteria	Verification Method	Pass / Fail / Comment
Platform Reference ID	Matches drawing and ITP	Correct ID	Record	
Platform Level	As per design level	±25 mm	Survey	
Layer Thickness	Per design / compaction lift	±10 %	Level check	
Material Type	Compliant with platform spec (e.g. Type 1, 6F4)	As specified	Delivery ticket / inspection	
Moisture Condition	Within OMC ± 2 %	NDG / MCV		
Compaction	≥ 95 % MDD (BS 1377-4)	Test report		
EV_2 / CBR Value	≥ Specified (e.g. 120 MPa EV_2 or 15 % CBR)	PLT / LWD		
Surface Evenness	≤ 25 mm under 3 m straightedge	Survey		
Drainage / Run-off	Adequate fall (> 1 %)	Visual / level		
Edge Protection	Timber / kerb restraint in place	Secure		
Inspection Frequency	Daily visual; weekly test	As per ITP		
Platform Maintenance	No rutting > 50 mm	Visual		
Sign-off	Completed by geotechnical supervisor	Date / name		

Test locations and results should be geo-referenced. EV_2 data are automatically plotted against platform zones in Phicon dashboards for traceable compliance.

F.4 Earthworks Compaction & Fill Verification (QA Snapshot)

Item	Check Description	Acceptance / Criteria	Verification Method	Pass / Fail / Comment
Material Type	As per specification class (1A–7B)	Conforming	Delivery / log	
Layer Thickness	≤ 250 mm (granular) / ≤ 200 mm (cohesive)	Test		
Moisture Condition	Within OMC ± 2 %	NDG / MCV		
Compaction Energy	Roller / passes as specified	Plant record		
In-situ Density	≥ 95 % MDD	NDG / sand replacement		
CBR / EV_2	≥ Specified	PLT / LWD		
Slope Profile	As per design gradient	Survey		
Non-Conformance	Material rejection / re-compaction	As instructed		
Record Complete	Signed QA form	Supervisor		

F.5 Observation & Trigger-Level Monitoring Form

Purpose: To implement the *Observational Method* (EN 1997-1 §2.7) through measurable response triggers.

Instrument Type	Parameter	Trigger Levels (Example)	Action
Inclinometer	Lateral wall movement	Green < 15 mm / Amber 15–25 mm / Red > 25 mm	Notify designer; review wall stability model
Piezometer	Pore pressure rise	Green < design line / Amber + 10 kPa / Red > 20 kPa	Review drainage / pumping
Settlement gauge	Vertical settlement	Green < 20 mm / Amber 20–40 mm / Red > 40 mm	Review fill placement / delay lifts

Strut load cell	Prop load	Green < 1.0 × Ppred / Amber 1.1 × / Red > 1.2 × Ppred	Assess pre-load, re-stress
Surface marker	Differential movement	Green < 1:1000 / Amber 1:750 / Red < 1:500	Inspect and record NCR if exceeded

Monitoring should be recorded digitally with *auto-trigger notifications* to the design engineer when amber or red thresholds are exceeded.

F.6 QA Documentation and Sign-Off Summary

Minimum QA Records Required for EC7 Compliance (EN 1997-1 §4.3):

Document Type	Stage	Typical Content
Site Inspection Reports	Construction	Photos, daily checks, NCRs
Test Certificates	Testing	NDG, PLT, CBR, concrete cubes
Survey Reports	Setting-out	Levels, coordinates
As-built Drawings	Completion	Verified geometry
Monitoring Logs	Observational	Trigger compliance
NCR Register	QA control	Corrective actions
Material Delivery Tickets	Construction	Type, source, batch ID
QA Summary Sheet	Handover	Final verification sign-off

All QA documentation should be traceable to the GDR via version-controlled digital folders (e.g. OneDrive / SharePoint) and linked in the project QA dashboard for transparency during Cat II/III reviews.

Phicon Note:

• Quality control is an integral part of geotechnical design under EC7 — not a separate activity.
• EC7 §4 requires verification of design assumptions during construction; these checklists provide the evidential link.
• QA forms should be tailored by project stage and incorporated into the *Inspection and Test Plan (ITP)*.
• PHICON digital QA templates automatically assign non-conformance severity (Observation / NCR) based on tolerance thresholds.
• All inspection forms must include **engineer name, date, weather, and chainage reference** for traceability.
• For critical elements (anchors, piles, platforms), QA data should be reviewed **daily** and uploaded to the central design register.
• Final acceptance should confirm that design assumptions remain valid (as-built geometry, stiffness, water pressure).

APPENDIX G – COMMON DOCUMENTS & STANDARDS

G.1 British Standards Institution (BSI)

Document	Year / Edition	Scope & Relevance
BS EN 1997-1: Eurocode 7 – Geotechnical Design, Part 1: General Rules	2004 + A1:2013	Core European geotechnical design standard establishing limit state design principles (STR, GEO, UPL, HYD, EQU), design approaches (DA1–DA3), and verification methods. UK National Annex provides partial factors and local guidance.
BS EN 1997-2: Eurocode 7 – Part 2: Ground Investigation and Testing	2007	Specifies field and laboratory test procedures, data reporting, and derivation of design parameters. Foundation for GI planning, classification, and interpretation.
BS 5930: Code of Practice for Ground Investigations	2020	Comprehensive UK code covering all stages of site investigation from desk study to reporting. Essential reference for planning, execution, and quality of GI.
BS 8004: Code of Practice for Foundations	2015	Complements EC7 with UK-specific guidance on shallow and deep foundation design, construction, and testing. Bridges between EC7 and traditional design approaches.
BS 8002: Code of Practice for Earth Retaining Structures	2015	Provides design methods for retaining walls, soil pressures, stability, and construction considerations. Works alongside EC7 for temporary and permanent works.
BS 8006-1: Code of Practice for Strengthened/Reinforced Soils	2010	Guidance on design and construction of reinforced soil and mechanically stabilized earth (MSE) structures. Commonly used for bridge abutments, slopes, and embankments.
BS EN ISO 14688 & 14689	2018–2019	Classification and identification of soils and rocks (Part 1 & 2). Replaces BS 5930 soil description clauses. Fundamental for consistent logging and reporting.
BS EN ISO 22476 Series	2012–2022	Field testing standards (CPT, SPT, pressuremeter, vane, DMT, etc.). Define procedures, equipment, and data reporting—integral to GI compliance.
BS EN 1990 & BS EN 1991	2002–2011	Basis of structural design and loading standards underpinning EC7 combinations (e.g. DA1-C1/C2).

G.2 European Committee for Standardization (CEN) / Eurocode System

Document	Year / Edition	Scope & Relevance
EN 1997-1 & EN 1997-2 (Eurocode 7)	2004 / 2007	Core EC7 documents governing geotechnical design and testing. Currently under revision (2nd Generation Eurocodes, expected 2027–2028).
EN 1990–1999 Series (Eurocodes 0–9)	Various	Unified framework for structural and geotechnical design. EC7 interfaces primarily with EC1 (actions), EC2 (concrete), and EC3 (steel).
EN ISO 17892 Series – Laboratory Testing of Soil	2014–2023	Harmonized European test procedures (Atterberg limits, shear box, triaxial, oedometer, etc.) aligning lab methods with EC7 parameter derivation.
EN 16907 Series – Earthworks	2018	Covers design and construction of earthworks, including material classification, compaction, and verification. Important for highways, rail, and embankments.

G.3 CIRIA (Construction Industry Research and Information Association)

Document	Year / Edition	Scope & Relevance
C574 – Engineering in Chalk	2002	Definitive UK reference on chalk engineering behaviour, classification, and design implications. Used with EC7 and BS 5930 for chalk sites.
C580 – Embedded Retaining Walls – Guidance for Economic Design	2003	Practical guidance on cantilever and propped walls, including design charts and construction advice; complements BS 8002 and EC7.
C750 – Guidance on Embedded Retaining Wall Design (2nd Edition)	2019	Updated guidance aligning with EC7 principles and modern construction methods (diaphragm, secant, and sheet pile walls).
C760 – Guidance on Eurocode 7: Geotechnical Design	2019	Key interpretation document for EC7 implementation in the UK. Explains design approaches, partial factors, and verification procedures.
C641 – Construction over Abandoned Mine Workings	2006	Guidance for risk assessment and treatment of mine voids—used across infrastructure projects.
C675 – Piling and Embedded Retaining Walls (Piling Handbook)	2007	Practical reference on design, installation, and QA for piles and retaining walls.

G.4 Institution of Civil Engineers (ICE)

Document	Year / Edition	Scope & Relevance
ICE Specification for Piling and Embedded Retaining Walls (SPERW)	3rd Ed., 2017	UK standard specification defining materials, workmanship, tolerances, and testing for piling and wall works. Widely referenced in contracts.
ICE Manual of Geotechnical Engineering (Vol. 1 & 2)	2012	Comprehensive reference on soil mechanics, site investigation, foundation design, and construction methods under EC7 framework.
ICE Manual of Construction Materials	2009	Supporting text on properties and behaviour of construction materials, including fills and aggregates.
ICE Specification for Ground Investigation	2nd Ed., 2011	Standard specification (with AGS format compatibility) defining scope, logging, and reporting standards for GI contracts.

G.5 Building Research Establishment (BRE)

Document	Year / Edition	Scope & Relevance
BRE Digest 240 – Foundations on Fill Materials	1993	Guidance for identifying and treating fill prior to foundation design. Common reference for housing and redevelopment projects.
BRE Digest 365 – Soakaway Design	2016	Standard method for infiltration testing and soakaway design, widely used for SuDS compliance.
BRE Report BR470 – Working Platforms for Tracked Plant	2004	Essential for temporary works and piling mats. Defines design methodology and verification (now commonly implemented in "MPM" design tools).
BRE Digest 298 – Sulfate and Acid Resistance of Concrete in Soils and Groundwater	2005	Basis for concrete class determination in aggressive ground conditions.

G.6 Network Rail

Document	Year / Edition	Scope & Relevance
NR/L2/CIV/086 – Geotechnical Design Manual	Issue 6, 2023	Network Rail's implementation of EC7 and company standards for design assurance and approval (A/B/C categories).
NR/L2/CIV/071 – Management of Geotechnical Risk	2020	Defines risk management, ground model development, and Category 1–3 design control processes.
NR/L3/CIV/065 – Design and Checking of Civil Engineering Works	2021	Sets requirements for design assurance, checking levels, and certification—vital for Cat II/III checks.

G.7 Transport for London (TfL)

Document	Year / Edition	Scope & Relevance
TfL Civil Engineering Design Standards – S1050 Geotechnical Design	Rev. 3, 2020	TfL-specific requirements aligned with EC7, covering site investigation, retaining walls, tunnelling interfaces, and foundation verification.
TfL S1052 – Earthworks and Retaining Structures Specification	Rev. 2, 2019	Defines construction quality control, materials, and testing for TfL infrastructure.

G.8 Highways England / National Highways

Document	Year / Edition	Scope & Relevance
CD 622 (formerly HA 68/94) – Managing Geotechnical Risk	2020	Sets out mandatory procedures for ground risk management in highway projects, including GIR and GDR stages.
CD 622 & HD 22 – Earthworks Design and Construction	2020	Governs earthworks classification, design, compaction, and testing for road schemes.
MCHW Volume 1 – Series 600 Earthworks / Series 800 Fencing and Safety Barriers	Ongoing	Part of the Specification for Highway Works, defining material specifications, testing, and acceptance criteria.

G.9 Association of Geotechnical and Geoenvironmental Specialists (AGS)

Document	Year / Edition	Scope & Relevance
AGS Data Format (Latest v4.1)	2022	Defines electronic data transfer format for GI data (boreholes, lab tests, CPTs). Standard across UK consultants and contractors.
AGS Guidance on Reporting and Validation	Ongoing	Best practice on QA/QC and data exchange between laboratories, designers, and contractors.

G.10 Other Notable References

Publisher	Document	Year	Scope & Relevance
ICE / CIRIA / BRE Joint	*SPERW Companion Notes*	2018	Practical companion providing commentary on SPERW clauses and QA implementation.
Tomlinson & Woodward – ***Foundation Design and Construction***	7th Ed., 2008	Classic reference text widely used for conceptual design, correlations, and practical methods.	
ICE / Thomas Telford	*Geotechnical Engineering Principles and Practice*	2010	Professional reference summarising EC7 design and verification methods.

G.11 Usage Overview by Practice Stage

Stage	Primary Documents
Desk Study & GI Planning	BS 5930, ICE Spec for GI, NR/L2/CIV/071, CD 622
Ground Testing & Classification	BS EN ISO 14688/22476/17892 Series
Design & Analysis	BS EN 1997-1/2, CIRIA C760, BS 8002/8004
Construction & QA	BRE BR470, SPERW, MCHW, TfL S1052
Risk & Reporting	AGS Format, NR/L2/CIV/086, CD 622, CIRIA C574

Closing Foreword

This first edition of *The Phicon Geotechnical Reference Guide* has been developed as a **practical, standards-aligned reference** for engineers engaged in design, investigation, and verification across the UK and European markets.

Its structure follows the **Eurocode 7 framework (EN 1997-1/2)**, integrating the corresponding **UK and EU National Annexes** with everyday field practice and design office application.

The book's intent is not to replace primary standards or textbooks but to **bridge the gap between design theory and deliverable output** — from preliminary investigation through to verification on site.

Each section has been written to emphasise:

- **Traceability** – every parameter and factor is linked back to EC7 clause, UK NA, or published reference.
- **Practicality** – guidance is presented in concise, field-usable format with worked examples and checklists.
- **Digital integration** – all tables and workflows correspond to PHICON's Excel, Word, and OneNote templates for calculation and QA automation.
- **Professional accountability** – aligning with BS 8004 and ICE Specification expectations for demonstrable compliance.
- About PHICON

Phicon Ltd is a specialist geotechnical engineering consultancy operating from the Highlands of Scotland and serving the wider UK and EU infrastructure sectors.

Its focus lies in the **intersection of engineering design and digital enablement** — developing auditable, Eurocode-compliant calculation tools, dashboards, and automated templates to make geotechnical decision-making faster, clearer, and more defendable.

The practice combines **Chartered Engineer oversight** with a collaborative network of associates to deliver:

- Geotechnical design and verification to Eurocode 7 (DA1/C1 & C2).
- Ground investigation design, interpretation, and reporting.
- QA and observational method implementation.
- Bespoke digital tools for data traceability and design reporting.

Phicon's mission is to simplify and modernise geotechnical delivery — ensuring design logic remains transparent from field data through to structural verification.

Acknowledgements

This Reference Guide draws on the collective knowledge embodied in the UK and European standards, long-established references such as **Tomlinson (1983)**, **Clayton et al. (1995)**, **CIRIA guidance**, and the experience of practitioners who continue to apply these principles in the field.

Special thanks are due to Phicon's associate engineers, collaborators, and reviewers who contributed their practical insight during drafting — helping shape this work into a tool for both **young engineers learning EC7** and **senior designers seeking quick recall**.

Citation

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Disclaimer

This publication is intended as a **practical engineering aid** and is not a substitute for professional judgement or for the official versions of the standards cited herein.

Phicon Ltd accepts no liability for loss or damage arising from reliance on this material without appropriate verification or competent review.

Designers must always consult the relevant Eurocodes, National Annexes, and project specifications.

Colophon

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