The use of WALLAP in the context of Eurocode 7
(EN 1997-1, Eurocode 7:Geotechnical Design)

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Introduction

WALLAP version 6 (June 2012) introduces a new feature to simplify the preparation of data in accordance with the different Limit State analyses required by EC7. Separate data sets (SLS and ULS parameters) can be generated in order to fully investigate the various Limit States. There are also options for user-defined Limit States for those who wish to design according to codes other than EC7.

WALLAP carries out Limit Equilibrium and Soil-Structure Interaction analyses at each stage of the construction sequence. This allows the user to investigate both stability and structural forces at all stages of construction. Seismic and Accidental situations after the end of construction can also be investigated.

The purpose of this note is to summarise the requirements of EC7 as they relate to embedded retaining walls and the use of WALLAP in their design. . The note does not describe the operational aspects of WALLAP which are set out in the WALLAP Help system

The requirements of EC7 are not always easy to understand or implement and in practice one encounters a variety of interpretations. This note endeavours to set out and justify some of the choices which must be made during the design process so that the WALLAP user can

a) arrive at a rational design.
b) justify it to the wider world.

Notation

a_{ULS}, a_{SLS}	ULS and SLS seismic accelerations	SLS	Serviceability Limit State
agR	Peak Ground Acceleration (PGA)	ULS	Ultimate Limit State
A	Accidental action	X	Generic material property
c'	Drained shear strength	X_k	Characteristic value of a material property
c'_k	Characteristic value of c'	X_d	Design value of a material property
c_U	Undrained shear strength	δ_a , δ_p	Angle of wall friction (active, passive)
c_{Uk}	Characteristic value of c_U	ϕ'	Drained angle of shearing resistance
c_{SOFT}	Undrained shear strength of softened soil at excavation level	$\phi_{cv} = \phi_{cs}$	Constant Volume (= Critical State) angle of shearing resistance
c_{SOFTk}	Characteristic value of C_{SOFT}	ϕ_d	Design angle of shearing resistance
DA1	Design Approach 1	ϕ_k	Characteristic angle of shearing resistance
E_U	Young's modulus of soil - undrained	ϕ_{peak}	Peak angle of shearing resistance
E'	Young's modulus of soil - drained	γ_F	Partial factor on an action
F	Generic action	γ_{Cu}	Partial factor on undrained shear strength
F_k	Characteristic value of an action	γ_G	Partial factor on a permanent action
F_{rep}	Representative value of an action	γ_M	Partial factor on a material property
F_d	Design value of an action	γ_{Mw}	Partial factor on bending strength of wall
G	Permanent action	γ_{Pp}	Partial factor on strut/anchor strength
K_o	Coefficient of in situ earth pressure	γ_Q	Partial factor on a variable action
M_{Wd}	Design BM acting on the wall	γ_{Pp}	Partial factor on strength of strut/anchor
M_{Wult}	Ultimate bending resistance of wall	γ_ϕ	Partial factor on tan ϕ
P_{Pd}	Design prop force	γ_γ	Partial factor on weight density of soil
P_{Pult}	Ultimate prop resistance	ν	Poisson's ratio
N_{60}	SPT N value	ψ	Load combination factor
Q	Variable action		

1.0 References

Referred to as

EN 1990 Eurocode: Basis of structural design
Underlying principles of Eurocodes for all types of structures

EN 1991 Eurocode 1: Actions on structures
Design guidance and definitions of Actions for all types of structures

EN 1997-1 Eurocode 7: Geotechnical design - General rules — **EC7**
General principles and requirements to ensure safety, stability and durability of earthworks and foundations. Not an easy read.
EN 1997-2 (Ground investigation and testing) is not discussed in this note

EN 1998: Eurocode 8: Design of structures for earthquake resistance — **EC8**
Part 1: General rules, seismic actions and rules for buildings.
Part 5: Foundations, retaining structures and geotechnical aspects
Additional rules for design in seismic regions

BS8102:1990 Protection of structures against water from the ground — **BS8102**

Eurocode 7: National Annex Nationally determined parameters
Country-specific data
Choice of Design Approaches
References to Non-contradictory, complementary information

CIRIA Report 104, Design of retaining walls embedded in stiff clays — **CIRIA 104**
C.J.Padfield and R.J.Mair CIRIA, 1984
The predecessor of C580. Now superseded.

CIRIA Report C580, Embedded retaining walls - guidance for economic design — **C580**
A.R.Gaba, B.Simpson, W.Powrie, D.R.Beadman.
CIRIA, 2003.
The UK National Annex to Eurocode 7 lists C580 as a source of Non-contradictory, complementary information. (B+H pp.402-403)
An invaluable source of guidance and information.

Designers' Guide to EN 1997-1, Eurocode 7: Geotechnical design - General rules — **DG**
R.Frank, C.Bauduin, R.Driscoll, M.Kavvadas, N.Krebs Ovesen, T.Orr and B.Schuppener. Thomas Telford Ltd, 2004
Interesting historical background to the evolution of Eurocode and Design Approaches. Much useful detail, comment and worked examples but not totally self-contained.

Decoding Eurocode 7 — **B+H**
Andrew Bond and Andrew Harris
Taylor and Francis, 2008
A readable guide to the Eurocode geotechnical labyrinth. With this, C580 and the Designers' Guide (above) you may never need to read EN 1997-1. Good coverage of general principles. Excellent presentation (and resolution) of conflicting interpretations of EC7. Precise references to Eurocode itself.

Bridge Manual 2nd Edition 2003 Document Code SP/M/022
New Zealand Transport Agency (Transit NZ), PO Box 5084, Wellington
http://www.transit.govt.nz/technical

Kramer, Steven L. (1996)
Geotechnical Earthquake Engineering. Prentice Hall

2.0 Definitions

2.1 Limit States

B+H p.29 §2.5

Serviceability Limit State — is concerned with functioning of the structure (and adjacent structures), the comfort of people and the appearance of the works. B+H p.35 §2.8

Ultimate Limit State — is concerned with collapse or instability of the works which may affect the safety of people or the structure, or cause major economic loss. EC7 identifies several different ULSs which must be verified. Each is denoted by an acronym: B+H p.32 §2.7

Limit state Acronym	Description	Relevant to embedded walls	Information obtained from Wallap analyses
EQU	Loss of equilibrium e.g. toppling	No	n/a
STR	Failure of structural members by excessive deformation, formation of a mechanism or rupture	Yes. Bending failure of walls. Tensile or pull-out failure of anchors. Strut failure	Bending moments and strut forces
FAT	Fatigue or creep failure	Maybe. In very stiff clays with high K_o values, active pressures which have relaxed during excavation may recover to K_o levels in the long term.	Long term stresses can be modelled by a resetting of soil properties at the end of construction
GEO	Failure or excessive deformation of the ground	Yes. Active or passive failure of soil. Ground heave.	Soil pressures and horizontal displacements are given in the WALLAP output. Vertical displacements are not calculated by WALLAP and must be assessed separately
UPL	Loss of equilibrium due to uplift by water pressure	Yes	Warning issued in extreme cases. Uplift pressures must be assessed separately
HYD	Hydraulic heave, internal erosion or piping due to hydraulic gradients	Yes	None. Hydraulic gradients must be assessed separately

Table 1

2.2 Actions, Effects, Resistances and Material properties

Actions are loads and other phenomena (e.g. thermal stresses, impacts, vibrations) which act on the structure. Actions are divided into: B+H p.36 §2.9

Direct actions — Forces applied to a structure e.g. Self weight, water pressure, pre-stress, temperature, wind, snow, impact.

Indirect actions — Imposed deformations or accelerations

Effects are the stresses and bending moments within the soil mass and structural members (wall and struts) due to the design loads (actions) which will tend to cause failure of the soil / structure.

Resistance of a structural member is its capacity to withstand actions without failing e.g. the moment resistance of a wall, passive resistance of a soil mass. The resistance of a member is a function of its geometry and the strength of the material(s) of which it is made.

Material properties (e.g. tensile strength of steel or shear strength of soil) determine the available resistance of parts of the structure.

Generic values of actions, effects, resistances and material properties are denoted by the symbols:

Actions	F
Effects	E
Resistances	R
Material properties	X

A safe design is achieved by applying partial factors to some or all of the above. For a simple structure e.g. a cable supporting a single tensile load, the Effect (tensile force in the cable) is proportional to the Action (the tensile load) and the Resistance (load capacity of the cable) is proportional to the Material strength (tensile strength of the cable). In such a simple case we would achieve the same margin of safety and the same design whether we apply our partial factor to the Action, Effect, Resistance or Material property. However, for retaining walls there are significant choices to be made in the application of partial factors. Traditional methods of retaining wall design have usually balanced Effects (active pressures) against factored Resistances (passive pressures). EC7, by contrast, prescribes the use of partial factors on Actions and Material properties.

2.3 Actions

2.3.1 Types of actions B+H p.37 §2.9.1

Actions are classified according to their variability over time.

Type of action	Symbol	Examples
Permanent	G	Self weight** of the structure and permanent loads, water pressure (under normal conditions)
Variable (live)	Q	Traffic, snow, wind, thermal load
Accidental	A	Accidental removal of a strut, impact, fire, seismic load
Self weight** But **not** self weight of **soil**		

Table 2

2.3.2 Design situations B+H p.30 §2.6

Eurocode defines four design situations, Persistent, Transient, Accidental and Seismic. Table 3 summarises the actions and limit states to be considered for each design situation.

Design Situation	Description	Relevant actions	Limit States to be considered	
			ULS	SLS
Persistent	Normal use	G + Q	√	√
Transient	During construction or repair	G + Q	√	√
Accidental	Accidental removal of a strut, impact, fire, seismic load	G + A (+Q)	√	x
Seismic	475 year return period quake	G + A (+Q)	√	x
	95 year return period quake	G + A (+Q)	X	√

√ calculation required
x not applicable

Table 3

Persistent and Transient situations both include Permanent and Variable actions. Seismic loads are often described as Accidental actions requiring only a ULS analysis. However, one expects structures:

- to resist destruction (ULS) under the most severe design earthquake.
- and also to resist significant damage (SLS) under more frequent smaller earthquakes.

Thus Table 3 includes the requirement of a SLS analysis for less severe earthquakes. See Section 4.10 for further treatment of the parameters required for seismic design.

2.3.3 Combinations of actions B+H p.39 §2.9.2

For each design situation (Persistent, Transient etc…) there may be more than one load combination to consider. An individual action is denoted by its Characteristic value, F_k. Actions are combined in various proportions according to the Design Situation and Limit State being considered. The Representative value,

$$F_{rep} = \psi . F_k$$

where ψ is a combination factor chosen according to the relative importance of the action in a particular load combination and whether it is favourable or unfavourable.

Where several variable actions may occur *independently* one considers each action to take the "Lead" in turn. In each combination the Lead action has its full characteristic value (ψ = 1.0) while all the other variable actions are scaled down (ψ < 1.0).

Unfavourable variable loads applied to a retaining wall are often a relatively small part of the total loading and to simplify matters they can all be assigned a combination factor, ψ = 1 without loss of economy of design. In other cases, e.g. bridge abutments, variable actions can be very significant and it will be important to consider the full range of combinations with their respective ψ values. See B+H §2.9.2, p.42 for details

For permanent, accidental and seismic actions, ψ = 1.0

For favourable variable actions, ψ = 0

2.3.4 Partial factors on actions

The Design value of an action is obtained from its representative value,

$$F_d = \gamma_F . F_{rep}$$

where γ_F is a partial factor whose value depends on -

a) the Limit State under consideration - SLS or ULS
b) whether the action is favourable or unfavourable.
c) the Design Approach adopted

Different sets of partial factors are defined for actions and material properties for each of the situations and limit states. Accidental and Seismic situations being less likely to occur require partial factors (typically 1.0) which are lower than for Persistent and Transient situations. Generally speaking all partial factors on actions are unity for the SLS case. (see also Section 3.3).

2.4 Material properties B+H p.129 §5.2; DG pp.24-30; EC7 §2.4.5

2.4.1 Characteristic values

Material properties are derived from test results.

Test results	Derivation → (e.g. c_U from SPT N value)	Derived values, X	Characterisation → (reduction of many test values to a single value)	Characteristic values X_k

Material properties e.g. soil strength, are introduced into the calculation as characteristic values (X_k). For non-geotechnical materials (e.g. steel and concrete) characteristic strengths are taken as the value which would be expected to be exceeded by 95% of samples. Due to the great variability of geotechnical materials and the difficulties in obtaining representative samples, EC7 redefines the characteristic value as

a cautious estimate of the value affecting the occurrence of the limit state. B+H §5.3.2

Bond and Harris (p.138) propose that "cautious estimates" can be equated with the "representative values" defined in BS8002 as

conservative estimates … of the properties of the soil as it exists in situ *… properly applicable to the part of the design for which it is intended* B+H §5.3.3

BS 8002 prescribes that for parameters such as weight density which show little variation in value, the representative value "should be the mean of the test results". Where greater variations occur (e.g. soil strength) or where values cannot be fixed with confidence, the representative value "should be a cautious assessment of the lower limit …of the acceptable data". This seems a sensible approach to adopt for WALLAP analyses.

Note. The use of the term "representative" in BS 8002 is quite different from that in EC7 - see Section 2.3.3 above.

The Eurocode "cautious estimate" can also be equated with the definition in C580 of the term "moderately conservative"

> *A cautious estimate of soil parameters….. Worse than the probabilistic mean but not as severe as a worst credible parameter value. Sometimes termed a conservative best estimate.*

To summarise:		Characteristic values / Cautious estimates	(EC7)
	=	Representative values	(BS 8002)
	=	Moderately conservative values	(C580)

2.4.2 Parametric exploration of worst cases.
Much of the discussion of Characteristic values in EC7 and the above Section 2.4.1 assumes that "less is worse" when it come to the selection of strength and stiffness of soils and structural components. Experience and common sense lead to the conclusion that for redundant structures this is not necessarily so. A stiff wall attracts more moment than a flexible wall. In most cases stiffer soil will lead to lower bending moments but the possibility of the reverse situation is worth exploring especially for seismic loading. The effect of varying Ko is particularly unpredictable and if there is significant uncertainty about its value, a range of plausible values should be explored.

2.4.3 Partial factors on material properties
The Design value of a material property is obtained from its Characteristic value,

$$X_d = X_k / \gamma_M \qquad \text{B+H p.51}$$

where γ_M is a partial factor whose value depends on -

a) the Limit State under consideration - SLS or ULS
b) whether drained or undrained conditions operate
b) the Design Approach adopted

C580 (Design Approach B) requires the use of "worst credible" parameters i.e. the worst value of soil parameters that the designer realistically believes might occur. These may be regarded as comparable to Eurocode design values (ULS case) i.e. the characteristic value divided by its (ULS) partial factor.

3.0 ULS Design Approaches

DG pp.3-5; B+H p.403

The following discussion relates only to ULS calculations. For SLS calculations all partial factors are unity i.e. SLS analysis is based on unfactored characteristic values of actions and material properties. But note that some non-European codes require SLS partial factors greater than unity e.g. the New Zealand Bridge Manual requires a SLS load factor on “normal” traffic load of 1.35 .

The ULS Design Approach discussed here relates to:

- Permanent, Transient, Accidental and Seismic situations
- Walls, footings and slopes.
  Bearing pile design requires a different design approach on account of its highly empirical nature.

A design approach is the method by which the required margin of safety is achieved. One can enhance the applied loads or decrease material strengths or consider a combination of these. EC7 has not managed to achieve a unified approach in relation to ULS design. Instead it offers a choice of three distinct Design Approaches which reflect the traditions, geology and preferences of the member countries. Each country has developed a National Annex to EC7 which specifies choices, parameters and information relevant to the practice in that country.

The EC7: UK National Annex specifies the use of DA1. Design Approaches 2 and 3 will not be discussed further in this note.

3.1 Design Approach 1 (DA1) - Permanent and Transient Situations

DA1 requires the engineer to consider two different combinations of partial factors:

- Combination 1 — places emphasis on the factoring of actions while using unfactored material properties
- Combination 2 — places emphasis on the factoring of material properties while using unfactored actions (except for variable [live] actions)

We do not factor all parameters in one combination on the grounds that it would be unduly pessimistic to assume that loads **and** soil strength might have worst credible values at the same time

Design Approach 1			Combination 1			Combination 2		
			A1	M1	R1	A2	M2	R1
Permanent actions (G)	Unfavourable	γ_G	1.35			1.0		
	Favourable	$\gamma_{G,fav}$	1.0			1.0		
Variable actions (Q)	Unfavourable	γ_Q	1.5			1.3		
	Favourable	$\gamma_{Q,fav}$	0			0		
Coeff. of shearing resistance (tan φ)		γ_φ		1.0			1.25	
Effective cohesion (c′)		$\gamma_{c'}$		1.0			1.25	
Undrained strength (c_u)		γ_{cu}		1.0			1.4	
Unconfined compressive strength (q_u)		γ_{qu}		1.0			1.4	
Weight density (γ)		γ_γ		1.0			1.0	
Resistance (R)		γ_R			1.0			1.0

Table 4 (after B+H Fig.6.11)

Strictly (according to EC7) one must explore both Combinations and take the worse case for design. This note suggests that a simpler approach is appropriate and that in most cases a safe and economical design will be achieved by considering Combination 2 alone.

3.2 DA1: Combination 1 - different interpretations

3.2.1 A straight forward interpretation of Combination 1

Looking at B+H Fig. 6.11 (above) we would think that all that is required in Combination 1 is to factor external loads i.e. surcharges, wind load, impact load. According to the table, self weight of soil is not factored as it is both a favourable and an unfavourable load. It is clear that for most retaining walls such an interpretation of Combination 1 would be much less onerous than Combination 2 and so there would generally be no point in carrying out this extra analysis unless external loads (wind or structural) were very significant.

3.2.2 Combination 1 as interpreted by the Designers' Guide

Nevertheless, in order to comply strictly with the requirement to factor "actions" it has become widely accepted that we should somehow factor the ***effect*** of self weight of soil. This is achieved by factoring total wall pressures and hence the resulting bending moments and strut forces. The main source for this approach is the Designers' Guide (DG) Section 9.7 p.158:

> All soil parameters, water pressures, and permanent actions enter the calculation with their Characteristic values. Variable loads and surcharges enter the calculation with their Characteristic values multiplied by γ_Q/γ_G (see Table 4 i.e. 1.5/1.35 = 1.1). The resulting bending moments and strut forces are regarded as unfavourable permanent actions which are then multiplied by γ_G (1.35) to obtained their design values.

It would seem reasonable to allow for Overdig in the data input although DG does not make explicit mention of it.

The above approach is also presented by B+H Section 12.5.1 p.420.

3.2.3 Comment

We note that, apart from Overdig and Variable loads, which are pre-factored by 1.1, the input parameters are identical to the input for a SLS analysis. Similarly the resulting bending moments and strut forces get factored by 1.35 to obtain their design values. Looking at Section 7.1.1 we see that the DG interpretation of Combination 1 leads to design bending moments and strut forces which are only marginally greater than those obtained from an SLS design. We also note that any FoS obtained from this (Combination 1) analysis would be a lumped factor for which it is difficult to define a suitable design value. However we still have Combination 2 to ensure that stability is achieved and that design bending moments are safe.

Moreover the DG interpretation of Combination 1 is in essence **not** a proper ULS analysis. The point of a ULS analysis is to factor the input parameters as close to their source as possible whereas this scheme lumps all the unfactored parameters together and then factors the resulting structural forces.

Amongst reputable consulting engineers opinion is divided. The evangelical Euro-zealots carry out the above Combination 1 analysis as a matter of routine and may even insist that others do so too (in addition to SLS and DA1-2 analyses). The Euro-sceptics consider the implications of DA1-1 and find that they can generally ignore it if Applied Loads are not significant.

3.2.4 Combination 1 options in WALLAP

Out of respect for published interpretations of Combination 1, WALLAP offers the Designers' Guide approach as "standard" under the heading **DA1 Combination 1** in the FoS options. However our own "straight forward" interpretation of Combination 1 (as outlined above) is also available under the heading **DA1 Comb. 1 (Alternative)**. This option is accessed under User Defined Limit State No.1 when you enter the "Limit State description".

3.3 Accidental and Seismic Situations

Accidental situations include fire, impact, explosion and accidental removal of a strut. In Accidental and Seismic design situations one adopts a single Design Approach in which all Permanent Actions and Material Properties are given their unfactored characteristic values (see Section 4.10.1).

The situation in regard to Variable Actions is not so simple. It would be unreasonably pessimistic to assume that all Variable Actions (e.g. traffic, snow, wind) operate simultaneously at the moment of the Accidental or Seismic event. Detailed recommendations, which will depend on the type of structure (e.g. bridge abutment, harbour wall) and the nature of the accidental load, are beyond the scope of this note.

4.0 Selection of parameters

For each parameter you will need to provide its characteristic value (subscript k) for the SLS case and its design value (subscript d) for the ULS case (see Section 2.4.1). For many parameters such as soil strength, the characteristic value is derived from test data and the design value is derived from the characteristic value by applying a partial factor from Table 4.

Other parameters such as water pressure often have to be estimated for both cases and there is no simple factor which relates the ULS case to the SLS case. Each case is estimated on its own merits.

Note the terminology used here:

- Characteristic (unfactored) strengths (subscript k) are used for SLS analysis
- Design (factored) strengths (subscript d) are used for ULS analysis

4.1 Geometry of the structure

Wall dimensions are taken as nominal values for all cases. In the case of steel piles due allowance must be made for corrosion.

The depth of the wall is usually taken as its nominal design value but one should consider the possibility that piles driven into a hard stratum might not achieve the anticipated penetration.

4.1.1 Bending strength of the wall

WALLAP offers the possibility of defining the ultimate bending resistance of the wall. This can be useful where local failure in bending is anticipated under accidental or seismic loading or, occasionally, under working conditions. Thus this facility can be used as part of SLS as well as ULS calculations. The characteristic strength, M_{Wult}, should be entered in the data. Values of Ultimate Moment Capacity of standard sheet pile sections are given in the WALLAP help system.

Caution should be exercised when allowing concrete walls to mobilise their full characteristic strength in SLS analyses as cracking may impair the durability of the wall

4.2 Excavation levels

The SLS case is analysed using nominal excavation levels including any extra planned excavation e.g. trenches for the installation of drains.

4.2.1 Unplanned excavation

B+H p.402 §12.3.2

This applies to the ULS case only. The allowance for over-excavation depends on the level of site supervision. For normal levels of supervision one allows for an unplanned excavation, ΔH, which is the lesser of

a) 10% of the retained height (for cantilever walls)
10% of the height below the lowest prop (for propped walls)

or

b) 0.5m

Larger values should be used if there is a high level of uncertainty about excavation levels e.g. dredging. Smaller values may be appropriate if strict supervision is in place.

4.3 In situ lateral pressure

C580 (§5.4.3) provides detailed advice on the measurement and derivation of K_o values. The most well known of the formulae (Jaky 1944) relates specifically to normally consolidated soils and is therefore relevant only in a minority of situations. Proper consideration of K_o is essential if one is to obtain meaningful results, especially in highly overconsolidated clays. The following rough and ready advice is proffered by C580

- for normally consolidated soils: $K_o = 1 - \sin\phi'$ (Jaky 1944)
- for overconsolidated fine grained soils: $K_o = 1.0$
- for overconsolidated coarse grained soils: $K_o = 1.0$ for walls installed by non-displacement methods (eg bored pile walls, diaphragm walls).

The value of K_o affects the displacement required to mobilise the fully active and passive condition. Thus high values of K_o can have a significant effect on calculated bending moments and displacements in the SLS case. As it is often difficult to assess K_o precisely it is not uncommon to explore a range of values so that one is aware of the implications of errors in K_o.

ULS analyses should be carried out (at least initially) using the characteristic values used for the SLS analysis. ULS bending moments will generally be less sensitive to K_o and there is normally no need to explore a range of values.

4.4 Water pressures

Water pressure can represent a large proportion of the total pressure on the retained side and particular care must be taken in determining suitable design values. All estimates of water pressure (for both SLS and ULS analyses) must take account of:

1. The permeability and, most importantly, the ***relative*** permeability of the various strata which may give rise to perched water tables.
2. Permeability of the wall.
3. Penetration of the wall into an impermeable stratum - if relevant.
4. Distribution of water pressures round the toe of the wall. WALLAP has a convenient facility for modelling a simple linear drop of gradient where this is appropriate in sufficiently uniform strata. Flow nets or other calculations are required for more complex conditions.

Extreme water pressures such as might arise from a burst water main are regarded as an Accidental situation (see Section 3.3).

4.4.1 Moderately Conservative and Worst Credible water pressure profiles

The concepts of Moderately Conservative (SLS) and Worst Credible (ULS) values apply to water pressures just as they do to material strength (Section 2.4.1). There is much discussion about how these values might be arrived at. Two main approaches are often mentioned:

1. Make separate estimates of Moderately Conservative and Worst Credible water pressures according to the known circumstances of the structure and its environment.
2. Estimate Moderately Conservative water pressures from the available data and derive Worst Credible values by factoring (as with soil strength).

This note recommends the use of separate estimates as discussed by B+H (Section 3.4.4). EC7 does provide for obtaining ULS water pressures from SLS water pressures by the application of a partial factor but there is no rational basis for this and separate estimates are to be preferred.

The terms Moderately Conservative and Worst Credible require some clarification in order to obtain actual values for design. EC7 [Clause 2.4.6.1(6)P] defines them as follows:

> **Moderately Conservative** water pressures (SLS) are the most unfavourable values which could occur in ***Normal Circumstances***.
>
> **Worst Credible** water pressures (ULS) represent the most unfavourable values that could occur during the ***Design Lifetime*** of the structure.

The estimation of "most unfavourable" water pressures during Normal Circumstances and Design Lifetime is strongly influenced by the type of soil being retained as described in the following sections.

4.4.2 Impermeable and semi-permeable soils

In impermeable soils and in the absence of reliable drainage EC7 specifies that water level should normally be taken at the surface of the retained material for both the SLS and ULS cases (Normal Circumstances and Design Lifetime)

Long standing British practice (BS8102: 1990) is more lenient. The depth of the water table in the retained soil can be assumed to be 25% of the retained height of soil but in any case not greater than 1m.

It is suggested by some authors that EC7 is unrealistically strict in this respect and that the BS8102 approach is to be preferred. Whichever approach is adopted one uses **the same water pressure profiles for the SLS and ULS cases**.

4.4.3 Water filled tension crack in undrained medium or stiff clay

If there is the possibility of a water-filled tension crack in undrained cohesive soil then this must be assumed to occur **in both the SLS and ULS cases**. Even though the crack is assumed to fill to ground level, the crack itself will not extend to the toe of the wall. WALLAP allows the user to specify the maximum depth of water filled tension cracks which will usually be (much) less than the theoretical maximum depth and generally not greater than 2 or 3m. See C580 (§4.1.6) for further advice.

When modelling long term drained conditions in clay the water-filled tension crack option is not applicable and then the criteria of Section 4.4.2 should be adopted. However, even in the long term one should check for undrained behaviour with a water-filled tension crack as this may be more severe.

4.4.4 Permeable soils or in the presence of a reliable drainage system

For the SLS case (Normal Circumstances) the assessment of water pressures is based on:

1. Water pressures observed during the period (say a year or two) immediately prior to construction e.g. standpipes, piezometers, tide levels etc..
2. Reasonably foreseeable changes in ground water regime due to climatic variation and long term effects of construction.

For the ULS case (Design Lifetime) one takes the SLS as a base line and estimates the rise in the water table under the most adverse conceiveable conditions.

4.4.5 Water pressures for DA1 - Combination 1 (ULS)

DA1-1 is a ULS case but the generally adopted procedure (see Section 3.2) is to carry out the WALLAP analysis using unfactored (SLS) parameters and then factor the resulting bending moments and strut forces.

Thus, although the above discussion has referred consistently to using Moderately Conservative water pressures for the SLS case and Worst Credible water pressures for the ULS case, you will see that in the summary tables of SLS and ULS parameters, you are advised to use Moderately Conservative water pressures for DA1 Combination 1.

This approach assumes that the effect of factoring bending moments makes due allowance for the difference between SLS and ULS water pressures. One is unlikely to incur gross errors as DA1 Combination 2 considers ULS water pressures.

4.5 Soil strength

Characteristic (as defined in Section 2.4.1) values of soil strength should be derived in the usual way from field tests, laboratory tests or extrapolation from data from similar sites. C580, §5.4.4 gives much useful information on the derivation of strength parameters.

The decision on whether to carry out a drained or undrained analysis at any particular stage of the construction sequence is discussed in the WALLAP User Guide, in C580 and in any standard text on retaining wall design. It is not a matter which is addressed by EC7.

4.5.1 Angle of friction

There is some debate as to whether the design of retaining walls should employ characteristic strengths based on peak or critical state values of ϕ' (B+H p.431). The use of critical state (or even residual) values of ϕ' for characteristic strength represents a very cautious approach and is only appropriate in specific circumstances - for example:

- in soils which exhibit brittle behaviour or marked strain softening e.g. highly overconsolidated clays and very dense granular soils (SPT N value > 40).
- where it is impossible to obtain reliable estimates of peak strength from in situ tests or undisturbed samples.
- where wall installation or other construction activities have reduced the strength below its peak value
- where pre-existing shear surfaces make the use of in situ or laboratory measurements inapplicable.
- where a progressive failure mechanism means that peak strength is not mobilised simultaneously throughout the soil mass.

For most situations, characteristic values of ϕ' can safely be based on ϕ_{peak} . Any concerns about brittle behaviour will be addressed by the use of critical state values in the ULS analysis.

The next question is how to derive the ULS design value from the characteristic value. One could simply apply the prescribed factor (1.25) in Table 4. However for very dense granular soils, peak ϕ' may be much greater than ϕ_{cv} and it might be prudent to use ϕ_{cv} as the ULS design value.

To summarise, it is recommended that SLS calculations are normally based on characteristic values of ϕ_{peak} and that ULS calculations are based on either factored values of ϕ_{peak} or unfactored values of ϕ_{cv} where this is less than factored ϕ_{peak} .

4.5.2 Drained cohesion

Drained cohesion is not a fundamental soil property and values should be obtained from tests within the appropriate stress range. Values of apparent cohesion obtained from triaxial tests should be used with caution. High values generally indicate too high a rate of testing. Characteristic values of cohesion can be derived directly from the triaxial data.

4.5.3 Undrained cohesion

Characteristic values of undrained cohesion are often derived form in situ test (SPT values). There is often a great deal of data with a large scatter. Advice on interpreting these data can be found in Decoding Eurocode 7 Ch.5.

Design values for ULS analysis are obtained by applying the partial factors in Table 4 to the characteristic values.

4.5.4 Strength of softened soil at excavation level

Undrained cohesive soils are liable to softening at excavation level on the retaining side during construction. It is customary to allow for a reduction in strength of up to 30% within the top 0.5m or so. C580, §5.9.1 offers guidance on the degree and depth of softening. The actual amount of softening to be allowed will depend on many factors including permeability of the soil, control of ground water and speed of construction.

The softened strength should be regarded as the characteristic strength since it represents a moderately cautious view of conditions which will actually pertain during in construction. The softened soil will in most cases be removed by excavation before completion of the works. Nevertheless the softened strength will feature in the SLS analysis of the permanent works. Any movements and bending moments which develop as a result of softening will form part of the cumulative bending moments and displacements.

The ULS strength should, in principle, be obtained by factoring the characteristic strength. There is no clear guidance on this point and it must therefore remain a matter of judgement as to whether the application of the usual partial factor (1.4) would lead to unreasonably conservative strength values.

4.5.5 Strength of the founding stratum
Where the wall penetrates only a small distance (say 1m or so) into a particularly strong or stiff stratum the calculated bending moments will very sensitive to the fixity provided by that stratum. Caution must be exercised in assumptions concerning

- the actual penetration of the founding stratum which may be achieved in practice
- the strength of the founding stratum bearing in mind any disturbance which might occur during wall installation.

A range of situations should be considered and characteristic values should be selected corresponding to the worst plausible case.

4.6 Wall friction

B+H p.405 §12.3.4

C580 and EC7 prescribe ULS wall friction values based on ϕ_{cv} rather than ϕ_k as has been the practice for many years according to CIRIA 104 and BS8002. The amount of wall friction also depends on the wall material. Recommended ULS values of $\tan\delta$ are given below:

Wall material	ULS Wall friction, $\tan\delta$ C580 and EC7		Wall friction, $\tan\delta$ CIRIA 104 and BS8002	
	Active	Passive	Active	Passive
Steel	$\tan(\frac{2}{3}\phi_{cv})$	$\tan(\frac{2}{3}\phi_{cv})$	CIRIA 104 $\tan(\frac{2}{3}\phi_k)$ BS8002 $\frac{3}{4}\tan(\phi_k)$	CIRIA 104 $\tan(\frac{1}{2}\phi_k)$ BS8002 $\frac{3}{4}\tan(\phi_k)$
Cast concrete	$\tan(\phi_{cv})$	$\tan(\phi_{cv})$		
Pre-cast concrete	$\tan(\frac{2}{3}\phi_{cv})$	$\tan(\frac{2}{3}\phi_{cv})$		

Table 5 (after B+H p.406)

Note that ϕ_{cv} is already a conservative estimate of strength relative to the characteristic value and no further factor is required for the ULS case. In terms of actual design, this does not represent a radical departure from CIRIA 104 and BS8002 since the earth pressures (or the moments derived from them) in those calculations were then factored to achieve a suitable margin of safety.

An allowance for wall friction is not always appropriate e.g. where there are large bearing or pull-out loads on the wall. See C580 (§4.1.4) for further discussion of these situations.

SLS values of $\tan\delta$ are not explicitly mentioned in C580 or EC7 but we can achieve a consistent approach by substituting ϕ_k for ϕ_{cv} in Table 5.

Note that one can no longer make direct use of traditional earth pressure tables to find ULS values of K_a and K_p in terms of δ/ϕ_{cv} and ϕ_d since the ϕ values are not the same. To use such tables you have to work out your δ value from Table 6, calculate δ/ϕ_d, and look that up in your K_a or K_p tables. To avoid this cumbersome procedure the WALLAP help facility has been modified to provide values of K_a and K_p directly in terms of δ and ϕ.

You can still use traditional earth pressure tables to find SLS values of K_a and K_p in terms of δ/ϕ_k and ϕ_k since the ϕ values are the same.

4.7 Soil stiffness

Subgrade reaction and quasi-FE analyses require values of Young's modulus of the soil. Direct in situ measurements using a self-boring pressure meter can provide good indications of modulus but such data are rarely available. More usually we estimate Young's modulus via correlations with SPT N values and undrained strength. The following rules of thumb may be adopted:

$E_U = M.c_U$ for stiff overconsolidated clay
where M is between 500 and 1000

$E' = 0.8\ E_U$

$E' = F.N_{60}$ (MPa) for coarse grained soils
where F= 1.0 for normally consolidated soils
and F= 2.0 for overconsolidated soils

For further discussion of correlations between undrained shear strength and Young's modulus see the WALLAP User Guide and also C580 (§5.4.5).

The values obtained from the above correlations are characteristic values suitable for the SLS analysis. EC7 does not specify partial factors for soil modulus but C580 recommends that moduli for ULS calculations should be taken at 50% of their SLS values. This reflects the non-linear nature of soil elasticity and the lower modulus at higher strains. (B+H p.419 §12.5.1)

Young's modulus under seismic conditions is the subject of much debate. Seismic events are associated with high strain rates and large total strains. High strain rates are associated with increased modulus compared to static conditions whereas large strains are associated with lower modulus. It is sometimes assumed that the increase in modulus due to high strain rate cancels out the reduction due to large strains and so one uses SLS values of modulus to model seismic conditions. More detailed advice can be found in Chapter 6 of Kramer (1996).

Poisson's Ratio for drained soils lies in the range 0.1 to 0.3. The same value is used for SLS and ULS calculations. Poisson's Ratio for undrained soil is 0.5 by definition although for mathematical reasons WALLAP like all FE programs can only handle a value close to 0.5.

4.8 Surcharges and loads applied to the wall

4.8.1 Surcharges applied to the ground

C580 states:

> *For flat ground and walls retaining heights greater than 3m, it is recommended that a minimum surcharge of 10kPa should be applied to the surface of the retained ground in design. For walls retaining less than 3m, this surcharge load may be reduced provided the designer is confident that a minimum surcharge of 10kPa will not apply, during the life of the structure.*

This nominal surcharge is regarded as an "unfavourable variable" surcharge. 10kPa is its characteristic value. For Combination 2 (see Section 3.1) apply a partial factor of 1.3 and for Combination 1 (if considered) apply a partial factor of 1.5.

Other unfavourable variable surcharges e.g. traffic and cranes, are treated similarly.

Unfavourable permanent surcharges e.g. self weight of building, are only factored in DA1 Combination 1.

4.8.2 Horizontal and moment loads applied to the wall

Unfavourable loads applied directly to the wall are treated in the same way as surcharges. Characteristic values are used for the SLS case and partial factors are applied in the ULS case. Some loads (e.g. moment loads applied to the wall) may be both favourable and unfavourable e.g. increasing bending moments but decreasing deflections. If in doubt a range of values should be considered.

Strut and anchor pre-stress loads are entered as their characteristic (nominal) values (in both SLS and ULS analyses) and are never factored as they are regarded as permanent favourable actions.

4.8.3 Favourable variable surcharges and loads

Favourable variable loads are not included (partial factor = zero). If a Favourable variable load has a minimum value then that is regarded as a Permanent load with a partial factor of 1.0

4.9 Strut and anchors

Strut/anchor dimensions and modulus are taken as characteristic (nominal) values for the SLS and ULS cases. The pull-out resistance and tensile strength of anchors (or the compressive strength of struts) are not mentioned in WALLAP. It is the designer's responsibility to ensure that struts/anchors have sufficient capacity to withstand the loads calculated by WALLAP for both the SLS and ULS cases.

Strut and anchor pre-stress loads are entered as their characteristic (nominal) values and are never factored as they are regarded as permanent favourable actions.

4.10 Seismic loads

Seismic loading is introduced into a WALLAP analysis as a special type of construction stage. The effect of the earthquake is represented by a horizontal (and/or vertical) acceleration applied to the soil mass i.e. the dynamic reality is modelled as equivalent static forces. Design values of acceleration are selected according to location, type of structure and foundation soil, following local codes of practice and experience. Within the EU the relevant information can be found in the EC8 Annex for the particular country.

EC8: Part 1 adopts a performance-based seismic design (PBSD) which relates levels of damage (inelastic displacements) to the return period of the earthquake. EC8 considers two levels of damage:

No-collapse requirement:

"The structure shall…withstand the design seismic action without local or global collapse, thus retain its structural integrity and a residual load bearing capacity."

Damage limitation requirement:

"…to withstand an earthquake without occurrence of damage and limitations of use".

The first is clearly a ULS criterion and is deemed to be satisfied by designing for an earthquake with a return period of 475 years. Longer return periods may be required for critical structures such as hospitals or schools. The second is a SLS criterion and is deemed to be satisfied by designing for an earthquake with a return period of 95 years.

The WALLAP analysis does not automatically take account of the effect of the earthquake on soil properties or water pressures (e.g. liquefaction). These effects must be calculated separately and introduced into the construction sequence as additional stages (see Section 4.10.3).

4.10.1 ULS analysis

The first step is to determine the Reference Peak Ground Acceleration (PGA, or agR) for the required return period (usually 475 years) at the relevant location. This will usually be obtained from a seismic hazard map.

Two modifications need to be made to the PGA before it can be used in design

- A response factor to take account of the geometry of the structure and amplification of the vibrations by the soil
- An importance factor related to the purpose of the building.

The resulting acceleration is taken as the design value for ULS calculations. A PGA of less than 0.05g lies outside the provisions of EC8 and no special design measures are required.

One does not combine seismic forces with *factored* loads and soil strengths as this would be unreasonably pessimistic (B+H §2.13.2 p.52). Thus although the soil strengths are at their characteristic values the resulting analysis is, in effect, a ULS analysis as it is believed that the combination of characteristic strengths with seismic load represents a worst conceivable scenario.

4.10.2 SLS analysis

The procedure is as for the ULS analysis but starting with the PGA for an earthquake with a 95 year return period. If this data is not directly available it can be obtained by extrapolation from the 475 year return period PGA.

4.10.3 The WALLAP construction sequence

Seismic loading would normally appear at the end of the WALLAP construction sequence. The following additional adjustments to the data may be required:

- If clay deposits are involved it might be appropriate to change the properties of clay layers to their undrained values before applying the seismic loading stage.
- Cyclic loading due to seismic vibrations might:

  increase water pressures
  decrease soil strength and stiffness
  decrease wall friction

One should make cautious estimates of the new parameters and introduce them into the calculation as characteristic values. No further factoring of these parameters is called for as, again, this would be unreasonably pessimistic.

Kramer (1996) discusses the effects of cyclic loading in great detail. Particular care must be taken in respect of water pressure as this is likely to be the single most significant contributor to loss of stability. The properties of the affected layers and the new water pressure profiles should be implemented as additional construction stages *before* applying the seismic loading stage.

Local yield of the wall can be an important feature of seismic design. The characteristic (un-factored) yield strength of the wall is entered in the "Wall properties" section of the WALLAP data.

One could consider more than one seismic event e.g. separate or combined vertical and horizontal accelerations and these can be placed in sequence.

5.0 Analysis options

5.1 Factor of safety calculation

Strictly speaking, Factor of Safety calculations have no place in a Limit State approach as all possible failure mechanisms of the soil will come to light in the ULS analysis (DA1 Combination 2). WALLAP does however carry out the FoS calculation in parallel with the SLS bending moment calculation analysis and also DA1 Combination 2. The FoS calculation is always switched off for DA1 Combination 1.

WALLAP offers a variety of methods for calculating factors of safety for cantilever walls and single propped walls. The Strength Factor method is the only method which is consistent with the limit state approach of EC7. The Strength Factor method should be used for all calculations **but** please note that the Strength Factor method used in conjunction with the Wedge Analysis option (for active and passive limits) increases computing times significantly.

Limit State options

WALLAP offers a choice of Limit States with preset partial factors of safety in order to simplify the implementation of EC7 requirements as set out in Table 6. For Temporary and Permanent situations one uses the SLS and ULS Limit States. For Accidental situations one can add extra stages to an SLS analysis or implement one of the User Defined Limit States with partial factors of ones own choice.

Minimising computing time with the Wedge Analysis option

If you need to use the Wedge Analysis option (Seismic situation or complex backfill) then you can use one of the other FoS methods (say Burland and Potts) noting that in the ULS case you only need to achieve an overall FoS of unity and that all methods of analysis agree with each other when FoS = 1.0. i.e. if you have achieved FoS > 1 by the Burland and Potts method then you know that the FoS by the Strength Factor Method is also greater than unity.

5.2 Bending moment and displacement calculations

WALLAP calculates bending moments and displacements by an elastic-plastic spring model. Two types of model are available.

- Subgrade reaction - independent springs
- 2D-Finite Element analysis - interactive springs

Either method may be used but the 2D-FE analysis is be more realistic and usually gives smaller (but realistic) bending moments because it can include soil arching.

6.0 Summary of required Limit State calculations

There are 3 distinct Limit State calculations to be considered.

- Serviceability Limit State
- Ultimate Limit State - DA1: Combination 2
- Ultimate Limit State - DA1: Combination 1 (only if required)

Combination 1 will not normally be considered except as mentioned in Section 3.2

Parameters for the different Limit States are set out in Table 6

Footnotes for Table 6	
1	The most unfavourable values that could occur during the design life-time of the structure [EC7 § 2.4.6.1(6)P]
2	The most unfavourable values that could occur during normal circumstances [EC7 § 2.4.6.1(6)P]
3	Actions i.e. Surcharges applied to the ground and structural loads applied to the wall (not soil or water pressure)
4	Wall friction values are the maximum permitted and may need to be reduced having regard to the direction and amount of movement of the wall relative to the ground.
5	A Partial Factor of 2.0 on Soil Modulus is specified by C580 for the ULS case but not by EC7. The basis for this approach is that modulus at the large strains associated with ULS calculations is expected to be lower. Note: Subscript **k** denotes a Characteristic value

Parameters for Persistent and Transient Situations

SLS and ULS Design Approach 1 Combination 2

Parameter		Values to be adopted for each Limit State, including their Partial factors and other allowances	
Parameter Description	**Symbol**	**Serviceability Limit State (SLS)**	**Ultimate Limit State: Design Approach 1 Combination 2** Factors applied (mainly) to soil strengths
Wall geometry (including toe elevation)	-	Nominal values	Nominal values
Excavation levels	-	Nominal values	Overdig levels
In situ earth pressure coefficient	K_o	Characteristic values	Characteristic values
Water pressures	-	Worst values in Normal Circumstances[2]	Worst values during Lifetime of Structure[1]
Density of soil	γ	γ_k	γ_k
Drained soil friction angle	$\tan\phi'$	$\tan\phi_k$	Lesser of $\tan\phi_{cv}$ or $(\tan\phi_k)/1.25$
Drained soil cohesion	c'	c'_k	$c'_k/1.25$
Undrained soil cohesion	c_U	c_{Uk}	$c_{Uk}/1.40$
c_U of softened soil at excav. level (see C580, §5.9.1)	c_{SOFT}	c_{SOFTk}	$c_{SOFTk}/1.40$
Wall friction - Steel [4]	$\tan\delta_a$ $\tan\delta_p$	$\tan(⅔\phi_{cv})$	$\tan(⅔\phi_{cv})/1.25$
Wall friction - Cast concrete [4]	$\tan\delta_a$ $\tan\delta_p$	$\tan(\phi_{cv})$	$\tan(\phi_{cv})/1.25$
Wall friction - Pre-cast concrete [4]	$\tan\delta_a$ $\tan\delta_p$	$\tan(⅔\phi_{cv})$	$\tan(⅔\phi_{cv})/1.25$
Drained Young's modulus [5]	E'	E'_k	E'_k
Undrained Young's modulus [5]	E_U	E_{Uk}	E_{Uk}
Poisson's ratio	ν	ν_k	ν_k
Partial Factor on Permanent Actions [3] — Unfavourable	γ_G	1.0	1.0
Partial Factor on Permanent Actions [3] — Favourable	γ_G	1.0	1.0
Partial Factor on Variable Actions [3] — Unfavourable	γ_Q	1.1	1.3
Partial Factor on Variable Actions [3] — Favourable	γ_Q	0	0
Strut and anchor properties and pre-stress	-	Nominal values	Nominal values
Design bending moment of the wall	M_{Wd}	see Sections 7.1.1 and 7.2.1	$M_{W\text{-}ULS}$

Table 6a

Parameters for Persistent and Transient Situations

ULS Design Approach 1 Combination 1

Parameter			Values to be adopted for each Limit State, including their Partial factors and other allowances	
Parameter Description		Symbol	Ultimate Limit States: Design Approach 1 (ULS - DA1)	
			Combination 1 **Designers' Guide interpretation** Factors applied to effects of actions	**Combination 1** **Alternative interpretation** Factors applied to actions
Wall geometry (including toe elevation)		-	Nominal values	Nominal values
Excavation levels		-	Overdig levels	Overdig levels
In situ earth pressure coefficient		K_o	Characteristic values	Characteristic values
Water pressures		-	Worst values in Normal Circumstances[2]	Worst values during Lifetime of Structure[1]
Density of soil		γ	γ_k	γ_k
Drained soil friction angle		$\tan\phi'$	$\tan\phi_k$	$\tan\phi_k$
Drained soil cohesion		c'	c'_k	c'_k
Undrained soil cohesion		c_U	c_{Uk}	c_{Uk}
c_U of softened soil at excav. level (see C580, §5.9.1)		c_{SOFT}	c_{SOFTk}	c_{SOFTk}
Wall friction - Steel [4]		$\tan\delta_a$ $\tan\delta_p$	$\tan(⅔\phi_{cv})$	$\tan(⅔\phi_{cv})$
Wall friction - Cast concrete [4]		$\tan\delta_a$ $\tan\delta_p$	$\tan(\phi_{cv})$	$\tan(\phi_{cv})$
Wall friction - Pre-cast concrete [4]		$\tan\delta_a$ $\tan\delta_p$	$\tan(⅔\phi_{cv})$	$\tan(⅔\phi_{cv})$
Drained Young's modulus [5]		E'	E'_k	E'_k
Undrained Young's modulus [5]		E_U	E_{Uk}	E_{Uk}
Poisson's ratio		ν	ν_k	ν_k
Partial Factor on Permanent Actions [3]	Unfavourable	γ_G	1.0	1.35
	Favourable	γ_G	1.0	1.0
Partial Factor on Variable Actions [3]	Unfavourable	γ_Q	1.10	1.50
	Favourable	γ_Q	0	0
Strut and anchor properties and pre-stress		-	Nominal values	Nominal values
Design bending moment of the wall		M_{Wd}	$1.35 \times M_{W\text{-}ULS}$	$M_{W\text{-}ULS}$

Table 6b

6.1 Construction sequences and data files

WALLAP can model the whole construction process in one continuous sequence. This may include

- Excavation and surcharge application before the wall is installed
- Construction activities with the wall in place
  (excavation, dewatering, strut installation and removal)
- Drained and undrained conditions and changes from one to the other
- Post-construction stress changes
  e.g. equilibriation of pore pressures
  soil relaxation
  relaxation of the wall
- Accidental and seismic loads in the post-construction phase

One construction sequence occupies one WALLAP data file. So generally you will need to create at least two versions of the data file, one with SLS values and one with ULS values (DA1 Combination 2) according to the values in Table 6. Details of data entry for Limit State analysis are given in the WALLAP Help System and the User Guide.

For Accidental situations one can add extra stages to an SLS analysis or implement one of the User Defined Limit States with partial factors of ones own choice.

7.0 Assessment of results and verification of design

At each stage of an analysis WALLAP calculates

- Bending moments and shear forces in the wall
- Strut/anchor forces
- Displacements of the wall
- A factor of safety (if applicable)

An example of the "Detailed results" for an individual stage is shown in Table 7

Node no.	Y coord	Nett pressure kN/m2	Wall disp. m	Wall Rotation rad.	Shear force kN/m	Bending moment kN.m/m	Strut forces kN/m	Applied moments kN.m/m
1	11.00	5.54	0.031	-2.34E-03	0.0	0.0		
2	10.00	10.49	0.033	-2.41E-03	8.0	5.6	82.61	
		10.49			-74.6	5.6		
3	9.50	12.97	0.034	-2.26E-03	-68.7	-29.9		
4	8.25	39.92	0.036	-3.78E-04	-35.7	-90.5		
5	7.00	45.62	0.035	2.537E-03	17.8	-96.1	-0.00	
6	6.50	31.26	0.033	3.648E-03	37.0	-81.7		
7	6.00	37.16	0.031	4.525E-03	54.1	-58.6		
8	4.50	-17.05	0.023	5.671E-03	29.2	-2.5		
9	3.00	-2.91	0.015	5.280E-03	14.3	23.4		
10	1.60	15.00	0.008	4.251E-03	22.7	35.4		
11	1.00	18.08	0.006	3.600E-03	32.6	51.3		
		-47.35			32.6	51.3		
12	-0.50	-16.09	0.002	1.866E-03	-14.9	41.2		
13	-2.00	35.99	-0.000	1.095E-03	0.0	0.0		

Strut force at elev. 10.00 = 82.6 kN/m run = 247.8 kN per strut (horiz.)
= 251.6 kN per strut (incl.)

The strut at elev. 7.00 is slack

Table 7

It is the responsibility of the WALLAP user to verify the design by ensuring that values of these parameters do not exceed their permitted maximum value for the relevant Limit State. If initial results show that limiting values are exceeded then the design should be modified and re-analysed. Excessive bending moments and displacements can be accommodated by increasing the wall thickness or reducing strut spacings.

Table 8 summarises the design information obtained form the ULS and SLS analyses. A detailed discussion is given in Sections 7.1 and 7.2.

	Type of analysis	
	Limit Equilibrium	**Soil-Structure Interaction**
Design information obtained with ULS parameters	**FoS > 1 indicates stability (for cantilever and single prop cases)**	**Equilibrium BM solution indicates stability.** **ULS bending moments. ULS strut forces.**
Design information obtained with SLS parameters	**FoS > 1.25 (drained) or FoS > 1.4 (undrained)** **is a partial indication of stability**	**SLS bending moments and displacements. SLS strut forces.**

Table 8

7.1 Verification of Ultimate Limit State

7.1.1 Bending moment

C580 (§6.6.2) states the ULS verification procedure as follows:

> *The ULS wall bending moments* (M_{Wd}) *and shear forces for use in the structural design of the wall should be obtained as the* ***greater*** *of:*
>
> - *the maximum bending moment from the ULS analyses* (M_{ULS})
> - *1.35 times the maximum bending moment from the SLS analyses* (M_{SLS})

What is being said here is that we have two versions of the design bending moment (M_{Wd}). One is derived directly from the ULS analysis. The other is derived by considering the bending moment from the SLS analysis to be the characteristic value of a permanent action (the bending moment) which is multiplied by 1.35 (Table 4, Column A1) to obtain the design bending moment. To summarise:

$$M_{Wd} \text{ is the greater of } M_{ULS} \text{ or } 1.35M_{SLS}$$

The ultimate bending resistance of the wall, M_{Wult}, is related to the design bending moment, M_{Wd}, by:

$$M_{Wd} \leq M_{Wult} / \gamma_{Mw}.$$

For steel walls γ_{Mw} = 1.0 i.e. subject to the provisos below you can use the Ultimate Moment Capacities straight out of the "Table of steel pile moment capacities" in the WALLAP help system. There is no distinction between moment capacities for temporary and permanent situations.

For reinforced concrete walls a proper design must be carried out for the concrete and steel section. Eurocode 2 specifies

$\gamma_M = \gamma_c = 1.5$ for concrete
$\gamma_M = \gamma_s = 1.15$ for reinforcing bars

Bear in mind that:

- Bending and shear resistance are influenced by vertical loads in the wall.
- the characteristic bending strength of the wall may reduce with time e.g. due to corrosion.

7.1.2 Prop forces (struts and anchors)

C580 (§7.4) states the ULS verification procedure as follows:

> *The ULS prop load* (P_{Pd}) *for use in the design of the struts/anchors should be determined as the* ***greater*** *of:*
>
> - *the prop force from the ULS analyses* (P_{ULS})
> - *1.35 times the maximum prop force from the SLS analyses* (P_{SLS})

The logic concerning the alternative forces is the same as for the bending moments (Section 7.1.1). To summarise:

$$P_{Pd} \text{ is the greater of } P_{ULS} \text{ or } 1.35P_{SLS}$$

Having obtained the design load, P_{Pd}, one must design an anchor or strut to withstand that load. The design of props (struts or anchors) lies outside the scope of this note. A full treatment of prop design can be found in C580 (§7.3) and in B+H (Ch.14). The following remarks are for general guidance only.

- It is the designer's responsibility to ensure that anchor is of sufficient length such that the deadman or grouted anchor length (and the passive zone associated with it) lies outside any potential active failure wedge.
- Anchor strength depends on the strength of the tendon and resistance to pull-out. Pull-out resistance falls off rapidly after its peak value i.e. pull-out failure is brittle. Tendon strength falls off gradually after peak. Therefore anchors should be designed so that anchorage strength is greater than tendon strength as this gives a less brittle design.
- Anchors can be designed by calculation or on the basis of pull-out tests.

7.1.3 Displacements

Calculated displacements in ULS analyses are likely to very large as the structure is on the verge of failure. There is no prescribed maximum displacement in the ULS condition.

7.1.4 Factor of safety

Factors of Safety at all stages should normally be greater than unity. However the limit equilibrium analysis does not take account of arching action and so the bending moment and displacement calculation may often find an equilibrium solution for propped walls even when the FoS is less than unity. This situation is perfectly satisfactory.

7.2 Verification of Serviceability Limit State

7.2.1 Bending moment

C580 (§6.6.1) states the SLS verification procedure thus:

> *The calculated SLS bending moments and shear forces should be used to check compliance with:*
>
> - *crack width criteria for reinforced concrete walls*
> - *and allowable stress criterion for steel sheet pile walls (if applicable)*

The allowable stress criterion for steel is not relevant to EC7 as that is taken care of by the ULS bending moment check (see Section 7.1.1).

7.2.2 Strut/anchor forces

There is no verification of SLS prop loads. The ULS design of props ensures adequate performance.

7.2.3 Displacements

Maximum permitted displacements vary greatly according to circumstances. Criteria may relate to

- Acceptable displacements of the new structure.
- Damage to neighbouring services or foundations
- Damage to finishes of neighbouring structures
- Unacceptable differential settlement of neighbouring structures in relation to performance of services or machinery.

In the absence of any specific criteria, maximum wall displacements should normally be limited to 0.5% of the excavated height.

7.2.4 Factor of safety

Factor of Safety is not part of the verification of a SLS. Generally you will need to refer to the FoS calculations of the ULS analysis in order to verify the ULS condition.

However, the factor of safety (by the Strength Factor method) calculated in a SLS analysis may used to provide *partial indication* of the ULS condition in some circumstances:

- if all the soils are drained then a FoS of 1.25 or greater indicates that the ULS is satisfied
- if all the soils are undrained then a FoS of 1.4 or greater indicates that the ULS is satisfied

This verification would only be partial because only the soil strengths have been factored while, water pressures, excavation levels and surcharges all have their SLS values.