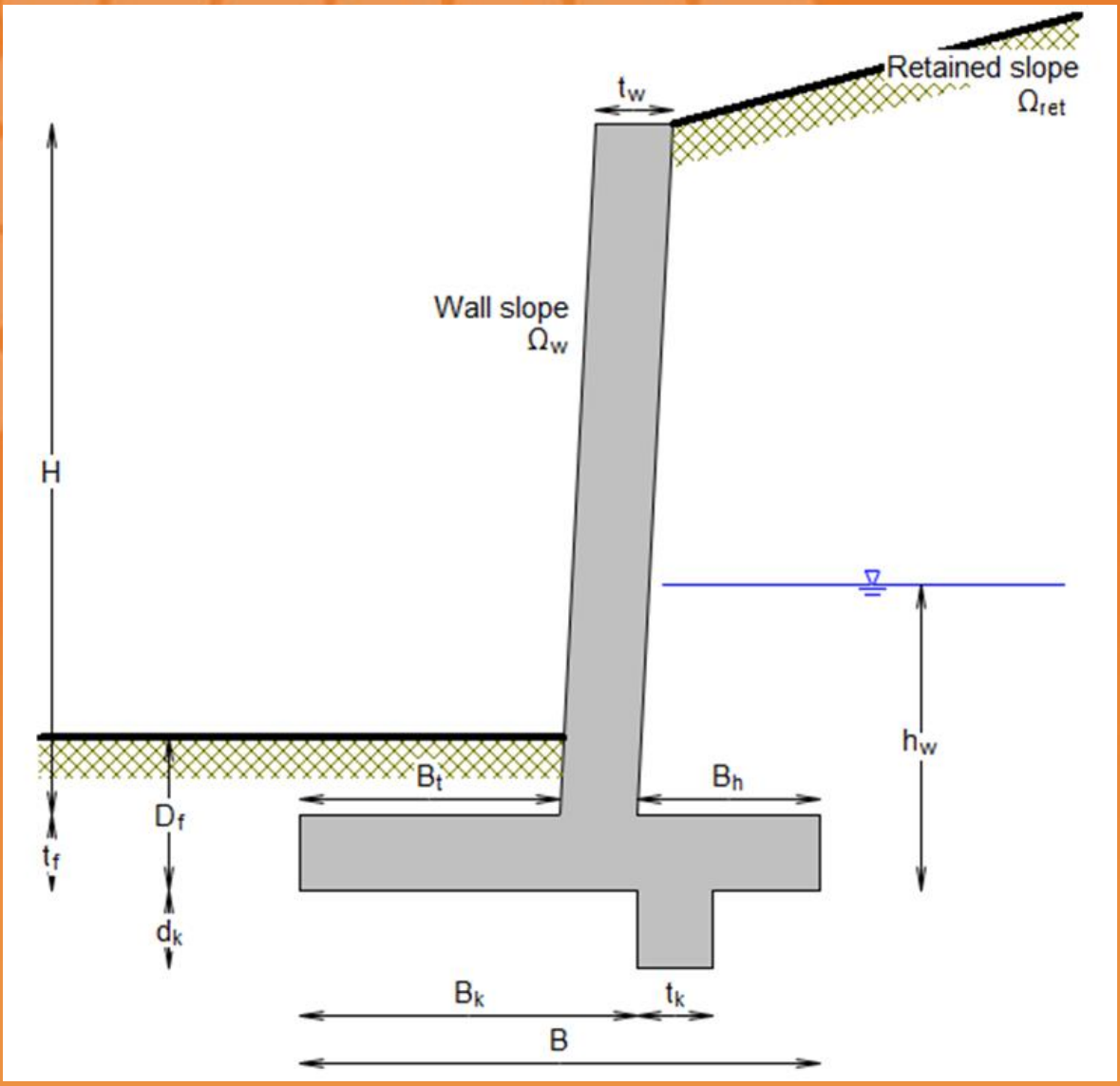
DESIGN GUIDE



SESOC

Concrete Cantilever Retaining Walls

SESOC Design Guide
Concrete Cantilever Retaining Walls

September 2026

Simplified Structural Design Guide
Concrete Cantilever Retaining Walls

September 2026

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SESOC's work includes the sponsorship of design guides, the provision of educational, advisory and information services, the dissemination of technical knowledge to the construction industry, participation in the activities of relevant national and international bodies, and promotion of design and construction standards and codes of practice.

Disclaimer

This design guide has been prepared to assist the competent (structural) engineer in the design of concrete cantilever retaining walls. Geotechnical input must be obtained for soil properties, and any other aspect outside the knowledge of the designer. Please note in particular the assumptions and limitations underpinning this guide and recommendations for seeking the advice of a Geotechnical Engineer described in Section 2.

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1. Introduction

1.1 Design guide

New Zealand does not have a design standard for reinforced concrete cantilever retaining walls, nor is there any generally agreed industry standard. MBIE/NZGS [6] does provide some good guidance with support from the Engineering NZ seminar series.

This document outlines a design procedure based on a static mechanics pressure block approach for concrete cantilever retaining walls that adheres to the concepts of Limit State design. While adhering to sound geotechnical principles, in accordance with its target audience – the Structural Engineering fraternity – this document is presented in a manner with intentional structural bias.

This document was originally prepared as one of two design guides, the other being for cantilever timber pole retaining walls [2].

1.2 Scope

This document is intended to be a stand-alone guide for the geotechnical aspects of standard concrete cantilever retaining walls, presenting guidance for Structural Engineers in simplified, step by step procedures. It is intended to be sufficient for practical use by Structural Engineers, and detailed enough that, given a set of input parameters, a single design result will be obtained.

The scope (described more fully in Section 2 below) is intended to cover:

- Free-standing cantilever walls only
- Cohesive or cohesionless founding conditions
- Static long-term loading
- Seismic loading, modelled using pseudo-static loads with appropriate short-term load factors
- Optional footing key, including positional capacity benefits (location dependent)
- Water table
- Retained slope
- Vertical or sloping walls
- Distributed superimposed loading
- Cohesionless (drained) backfill soil, with assumed cohesionless behaviour of all retained soil – refer also Section 3: Fundamental assumptions

This design guide is not intended to provide exhaustive coverage of all aspects of reinforced concrete wall design, but rather to document the core principles for a

majority of simple retaining walls. Concrete section analysis is specifically not included in this document.

Also, while the guide is fundamentally oriented towards the 'technical design' aspects, we have sought to include practical/construction aspects where appropriate. Nevertheless, this guide must not be taken as presenting all aspects of concrete cantilever retaining walls design and construction – e.g. practical construction sequencing, provision for drainage, etc are not covered.

While the content of this guide is intended to cover typical scenarios, specific Geotechnical Engineering advice must be sought where any aspects of the guide are not achievable, or any aspects are beyond the expertise of the designer. The reader is reminded that a significant portion of PI claims against Structural Engineers relate to inadequate geotechnical designs.

Users of this guide should note the assumptions and limitations of the documented methodology (given in Section 2 below).

1.3 How to use this design guide

This design guide has been written to progressively introduce the broad design principles, gradually providing increasing levels of detail so as to provide the reader with clarity of principle in an understandable and easy-to-read manner.

1.4 Compliance and regulatory matters

This guide has, at its core, a goal to present a best practice design methodology for 'simple' concrete cantilever retaining walls, yet also covering the multiple design facets potentially necessary for a 'typical' retaining wall. It has been prepared with input from numerous contributors across both the Geotechnical and Structural disciplines, and has sought to include, as much as possible, the basis and principles of a wide range of design examples, yet coalesced into a single, coherent whole.

While prepared in accordance with the principles of B1/VM1, nevertheless, this is presented as an 'alternative design method', in the absence of any national standard, or other widely accepted design methodology at this time.

2. Assumptions and limitations

This section sets out the assumptions made by the design methodology defined in this guide, together with limitations beyond which the methodology may not apply.

Where site conditions, soil behaviour, groundwater effects, stability concerns, or other geotechnical matters fall outside the stated limits or the Structural Engineer's own area of professional competence, appropriate advice from a Geotechnical Engineer should be sought before proceeding.

The following generic graphic shows a typical wall, along with associated dimensions and parameters.

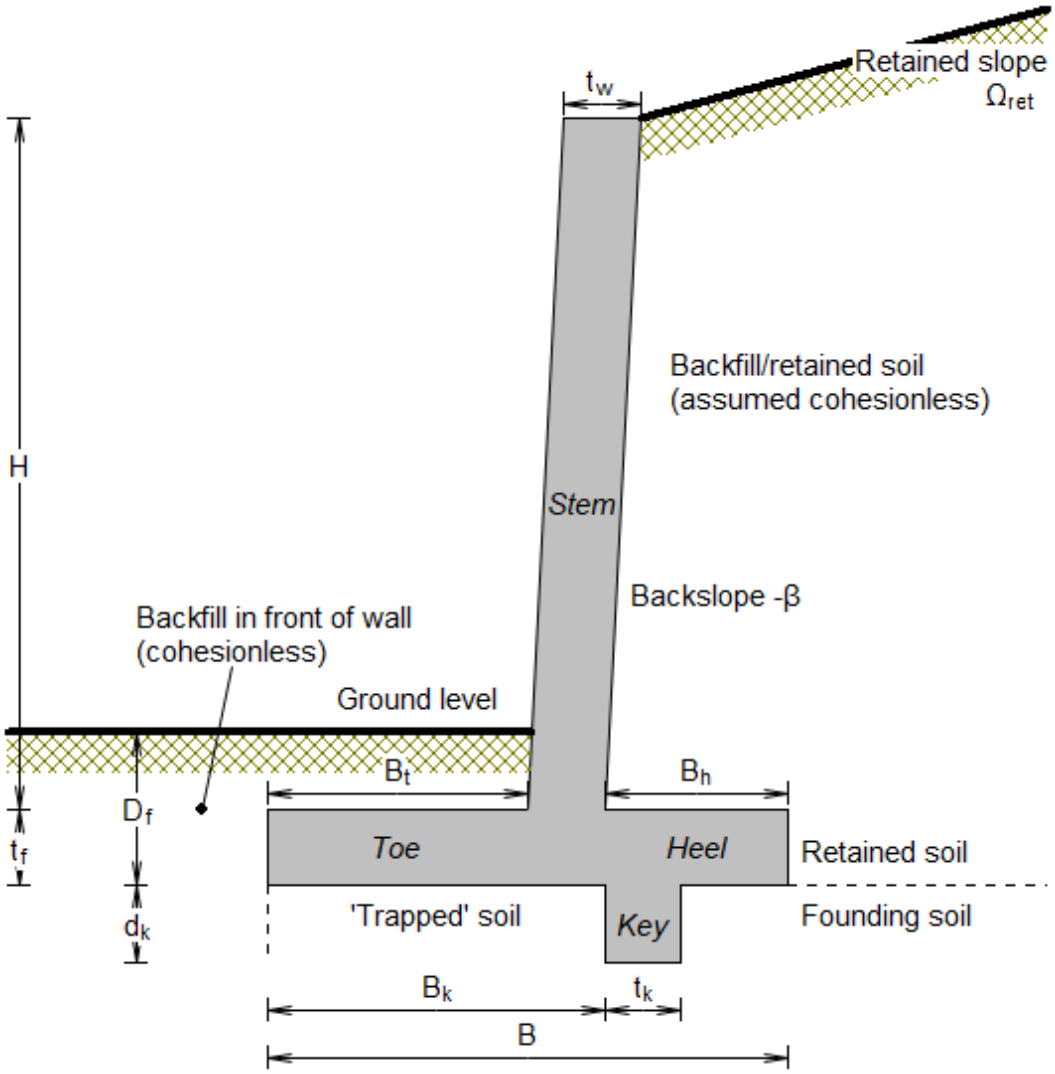


Figure 2-1: Wall geometry

Fundamental assumptions

- The structure is a reinforced concrete cantilever retaining wall
- No anchors or other supporting structures are employed
- Superimposed loading is distributed and 'normal', and does not include heavy traffic loading, crane loading, etc

- The wall is not displacement-sensitive. Generally, this means that no significant structures or displacement-sensitive services are on the bank immediately above the wall, or within a distance equal to the height of the wall measured from the top of the wall. While superimposed dead and live loads above the wall are allowed for, roads and driveways sensitive to ground movement are not
- Displacement of the wall is a base assumption for the design. The client should be made aware that an originally vertical wall may displace over time, and the wall may look tilted forward and potentially out-of-vertical
- 'Plane strain' conditions, with the wall and foundation assumed to be long (length significantly greater than breadth) and continuous, noting that bearing capacity calculations do include a length factor

Geotechnical assumptions

- 'Good ground' as defined by MBIE [7]
- Homogenous properties for the:
    - founding soil (below footing)
    - retained soil
    - soil in front of the wall
- Backfill soil is cohesionless (drained); assumed cohesionless behaviour of all retained soil – refer Appendix G.1 for further detail
- The water table, if present, is assumed to be on the retained side of the wall, but not in front of the wall
- The water table under seismic conditions is no higher than the ground level in front of the wall
- Together, the wall and soil are considered sufficiently 'flexible' such that use of K_a, the active pressure coefficient, is appropriate when calculating the earth pressure from the backfill. The designer should be aware of expected displacements and their implications on the design

Geometrical limitations

- Vertical or near-vertical wall (backslope less than 14°)
- Retained height H not exceeding 3.0m
- Retained soil slope is limited to half the effective friction angle of backfill material, i.e. $\Omega_{ret} \leq ½\varphi'_{bf}$
- Minimum extent of slope distances above and in front of wall:
    - retained slope is limited to Ω_{ret} for a distance behind the wall of at least 3 times the wall height
    - soil is horizontal in front of the wall for a distance of at least 2 times its retained height (H) and 8 times the height over which passive earth pressures act in front of the wall, i.e. $8(D_f + d_k)$

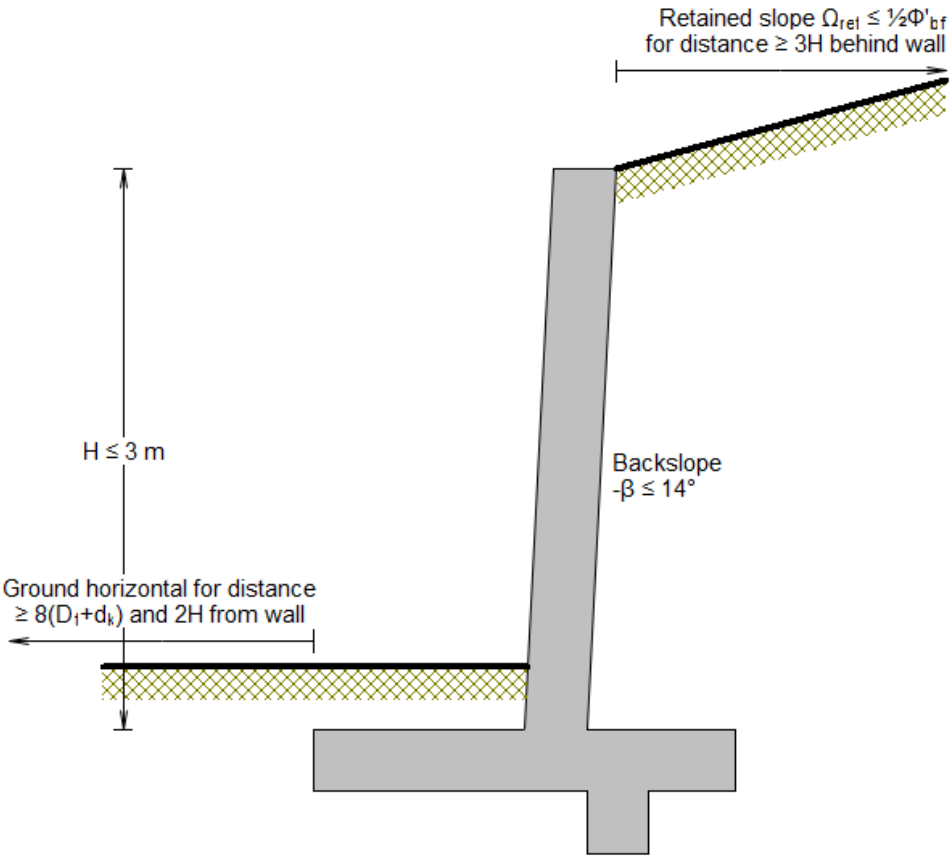


Figure 2-2: Geometrical limitations

Not covered by this guide

- Tiered walls (walls located within twice their height)
- Basement structures
- Matters related to temporary stability and safety
- Overall stability; deep-seated rotational or sliding failure in the vicinity of the wall
- Effect on adjacent structures, services etc

Seeking advice from Geotechnical Engineer

In many retaining wall situations, there may be geotechnical issues present which warrant a Structural Engineer seeking the advice of a Geotechnical Engineer. Such geotechnical issues include, but are not limited to, the following:

- Global geotechnical instability
- Defects or discontinuities in the soil or rock matrix
- Changes in soil properties with depth
- Significant or long-term presence of groundwater
- Liquefaction
- The impact of any ground movements on neighbouring properties, buildings, roads or buried infrastructure

- Undesirable levels of wider ground movement, both in the static and seismic load case scenarios
- Constructability of the wall type in the given ground conditions, and/or,
- Confirmation of the safest construction sequence(s) to manage health and safety risks

In summary, the Structural Engineer must ensure they are not operating beyond the bounds of their competence, and seek professional Geotechnical Engineering advice as appropriate.

Where advice is sought from a Geotechnical Engineer, this should be initiated prior to undertaking the design, rather than seeking a 'rubber stamp' upon completion.

For the purposes of this document Geotechnical Engineer shall mean a Chartered Professional Engineer and/or Professional Engineering Geologist, as appropriate, who is appropriately qualified, experienced and specifically assessed by Engineering New Zealand as specialising in the Geotechnical practice area.

All readers of this document are reminded that in New Zealand a significant proportion of engineering consultancy professional indemnity insurance claims relate to the geotechnical inadequacy or failure of retaining wall systems and/or the wider site, and such failures often feature Structural Engineers practising outside their field of expertise and/or level of geotechnical competence.

3. Design basis

The design principle adopted in this guide is that of Load Resistance Factored Design (LRFD). This process involves factoring-up load components (design action) and factoring-down resistance components in order to provide a satisfactory margin of safety. A satisfactory design is indicated where the factored design action is less than the dependable design capacity, i.e.

$$S^* \leq R^*$$

or

$$\sum \alpha_i \cdot F_i \leq \sum \varphi_j \cdot R_j$$

where

S^* = factored design action
R^* = dependable design capacity
α_i = load factors
F_i = applied loads
Φ_j = strength reduction factors
R_j = ideal capacities or strengths

Though partial load factors are not used in this design guide per se, some intermediate calculations are based on factored design action and resistance forces. For example, superimposed loading above the wall has relevant dead and live load factors applied to it prior to calculating the resulting pressure on the wall (which is in turn subject to a further earth pressure load factor).

Depending on the type of founding soil and the duration/type of loading, the wall is analysed either for a 'drained' or 'undrained' situation. Soil and water pressures exerting a force on the retaining wall are described by a series of geometrically-defined load blocks.

Both static and seismic load cases are covered – with seismic loads modelled using 'pseudo-static' earth pressures.

While it is expected that design results using this guide will be similar to those presented in the example by MBIE/NZGS [6], this document is intended to provide guidance across a range of geotechnical, geometrical, and soils loading parameters.

4. Failure modes

Possible modes of failure for reinforced concrete cantilever retaining walls are illustrated in Figure 4-1, from MBIE/NZGS (2021) Appendix C: Worked Example 2 [6].

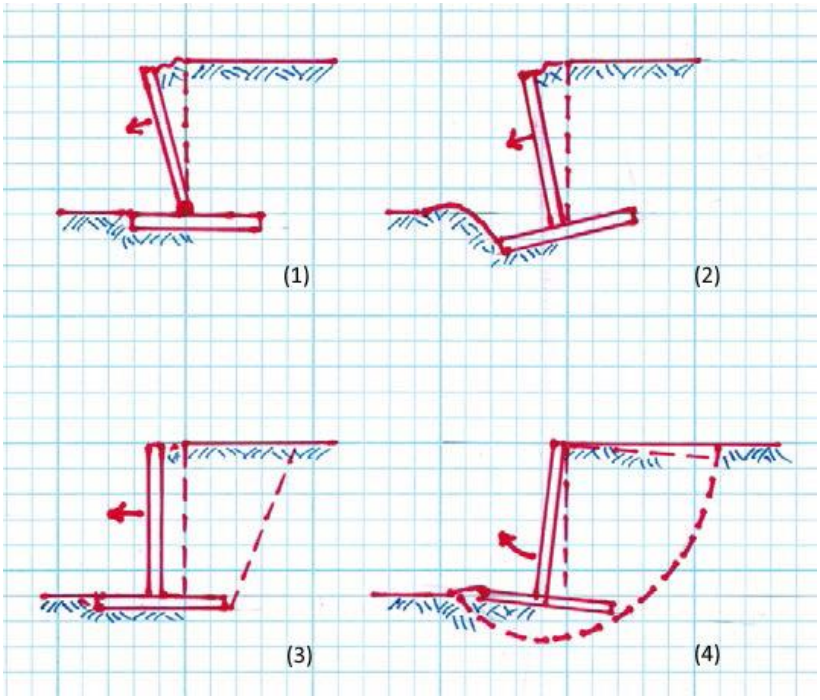


Figure 4-1: Possible modes of failure

1. Wall stem structural failure: The wall stem fails in bending, usually at the base of the wall where the stem connects to the foundation
2. Foundation bearing failure: A bearing failure of the soil under the toe of the foundation and a forward rotation of the wall
3. Sliding failure of wall: This is typically associated with non-cohesive soils. The wall moves outwards with passive failure of the soil in front of the foundation and active failure of the soil behind the wall. Often a key is required beneath the foundation to prevent this failure mode
4. Deep seated rotational failure: This is typically associated with cohesive soils. It is unlikely to govern unless the wall is embedded into sloping ground with a sloping backfill or there is a weak layer below the wall

This design guide is focussed almost exclusively on Modes 2 and 3, i.e. 'local' geotechnical aspects. While the guide provides equations for the bending moment at the base of the stem, the strength of wall under Mode 1 should be checked with respect to the concrete structures standard (NZS 3101).

A complete design must address each of these modes of failure where appropriate. Should the reader have any concerns about these – or any other aspects – advice must be sought from a Geotechnical Engineer.

5. Soil Model

This section provides further detail around the soil parameters and mechanics aspects, including loading.

5.1 Soil type/loading conditions

For analysis, retained soil is treated as cohesionless, as is any material in front of the toe.

Foundation soil may be referred to as either cohesive or cohesionless, and this defines the strength parameters used in design. If cohesive, no consolidation occurs under short-term/seismic loading, so calculations are in terms of undrained shear strength (s_u) and total stress. Full consolidation occurs under long-term loading, so calculations are done in terms of drained shear strength and effective stress (c' and φ').

Where a short-term live load portion of the surcharge represents a substantial portion of the load on the wall, static design for cohesive soils should be on a short term (undrained) basis. The approach used in this guide for a seismic analysis (but with k_h set to 0) and with appropriate design factors could be used in this case.

If cohesionless, only the soil internal friction contributes to the shear strength, and drained strength is used for all load cases.

Founding Soil Type	***Loading***	***Analysis Type***	***Strength Parameter***
Cohesive	Long-term static	Drained	φ', c'
Cohesive	Seismic, or short-term static	Undrained	s_u
Cohesionless	Long-term static	Drained	φ'
Cohesionless	Seismic, or short-term static		

Or, to present in another way, the analysis type is determined by the loading scenario plus the founding soil type:

Loading	***Analysis type***	
	Cohesive founding soil	***Cohesionless founding soil***
Long-term	Drained	Drained
Seismic, short-term	Undrained	Drained

It may be necessary for the designer to allow for multiple soil models when designing a retaining wall, e.g. when it is not clear:

- whether the founding soil is acting as a cohesive or cohesionless soil
- if a load is 'short-term' or 'long-term', and therefore whether the analysis type should be drained or undrained
- how well the soil parameters have been estimated (e.g. c' may be difficult to assess)

Moderately conservative soil parameters should be chosen by the designer.

5.2 Founding soil cohesion

Whether cohesive or cohesionless, the founding soil is assumed to rely on its long-term, drained shear strength (φ') under most static loading situations. While any cohesive part of the soil's drained strength, c' can have a significant effect on the overall capacity, it can be conservatively ignored. However, designs may then be unduly conservative, particularly for the sliding limit state.

Additionally, in this Design Guide the contribution of adhesion between soil and concrete to sliding resistance on a cohesive founding soil is included. This uses parameters c' (or s_u) and r_A (adhesion/cohesion ratio). Refer to Appendix E for further details.

5.3 Lateral resistance from backfill in front of toe

It is common to (conservatively) neglect the passive soil resistance of the backfill in front of the toe. While the contribution is included in the equations given in this Guide, the designer should only include this where there is a high degree of confidence in the backfill soil properties, and that this soil will remain in place for the design life of the wall.

5.4 Water table

The equations in this guide assume that, if present at all, the water table is below the footing level in front of the wall, but at a height h_w above the base of the footing behind the wall.

The presence of water, even at ground level, can have a major impact on the design of the wall, and hence overall capacity and stability. A sensible assessment of risk must be undertaken by the designer.

The designer **must** take a conservative view, considering various contributory aspects, including:

- Construction type
- Drainage system, and confidence in future maintenance (if any)
- Backfill material
- Type of retained soil further back from the wall, potentially affecting the assumed effective strength of the backfill
- Site topography
- Augured exploratory investigation hole results (this should be carried out even for low walls)
- Possible future changes to the site

6. Design parameters and factors

This section gives an overview of the load factors and capacity reduction factors that typically apply to a concrete cantilever retaining wall. Refer to Appendix B – Design factors for additional information including typical/recommended values.

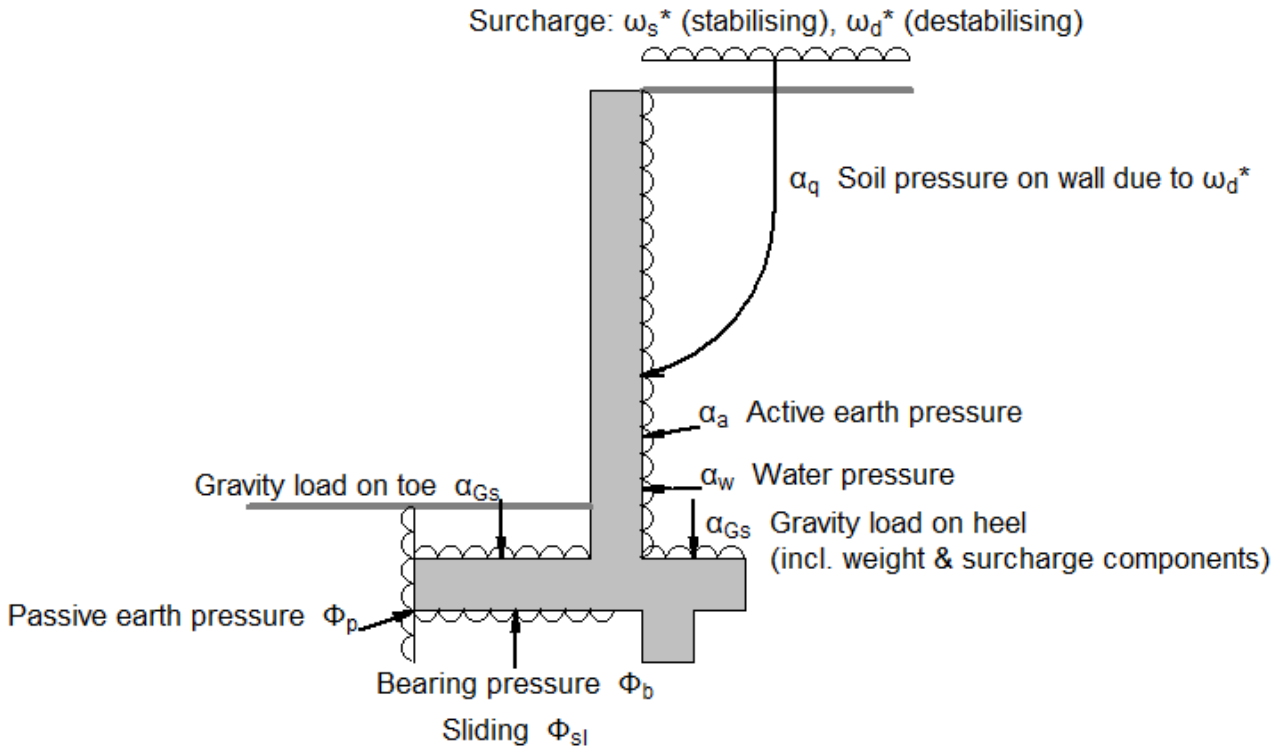


Figure 6-1: Design factors used in concrete wall analysis

6.1 Strength reduction factors

Strength reduction factors should be related to the type and degree of site investigation. Relevant reduction factors for concrete cantilever retaining walls are:

- Φ_b bearing reduction factor
- Φ_p passive resistance reduction factor
- Φ_{sl} sliding resistance reduction factor

Typical values are given in Appendix B.2 Strength reduction factors.

6.2 Load factors

Separate load factors are applied for loads arising from the following, for both static and seismic load cases:

- retained soil
- pressure from superimposed surcharge (dead and live) loading
- groundwater

Relevant load factors for concrete cantilever retaining walls are:

- α_a active soil pressure
- α_{Gs} gravity load factors (treated as stabilising)
- α_q soil pressure on wall due to superimposed loading (this factor applies in addition to any superimposed dead and live load destabilising factors)
- α_w ground water pressure

Typical values are given in Appendix B.3 Load factors.

In geotechnical work overall factors of safety for static load conditions are usually taken to be at least 2.0, so in general this would result in a design with an ideal capacity to demand ratio of 2.0, assuming all load and material factors are unity.

6.3 Seismic coefficient

The designer is advised to read Section 6.6 of MBIE/NZGS [6] which provides good coverage of seismic loading aspects, plus guidance on calculating k_h, incorporating wall displacement and topographic modification factors.

In summary, the peak ground acceleration (PGA) is determined using the loading standard, and then modified to determine the design horizontal earthquake coefficient, k_h. k_h is the result of the base PGA value times a multiplier – often of some value below unity, incorporating, where applicable :

- A_{topo} – a topographic amplification factor
- W_d – a reduction factor allowing for wave scattering, lack of coherency in the backfill accelerations, and acceptable (possibly permanent) wall displacement

In terms of the applicable loading standard, while NZS 1170.5 is the currently cited standard, in the NZGS advice on geotechnical seismic hazard document (NZGS 2026 [9]), NZGS advocate for the use of TS1170 as representing the latest in terms of current knowledge of seismic hazard.

While a value in the range of 0.3 to 0.7 x PGA may often be used, it is the responsibility of the designer to justify the chosen value, based on, amongst other things, wall situation, expected wall displacement, retained slope angle, etc. Refer also to Section 5.3 of MBIE/NZGS [6] and the worked example therein.

7. Load and resistance components

This section provides information around the various load and resistance components, active and passive pressure blocks, soil pressures at depth and the resultant forces etc.

7.1 Soil model

This design guide uses a simplified model of soil and other pressures on the retaining wall. These are graphically described in the following diagram, including the 'virtual back of wall' approach.

The loads on the foundation allowed for are as follows:

- Weight of concrete wall W_{conc}, including stem (W_s), foundation (W_f), and key (W_k)
- Backfill above the heel, including any sloping backfill (W_{bf})
- Overburden above the toe (W_{bft})
- Soil trapped in front of key (below foundation slab) ($W_{trapped}$)
- Vertical loading on the heel from a uniform superimposed load (W_{sur}), calculated from distributed surcharge dead load ω_D and live load ω_L
- Buoyancy adjustment of gravity loads due to presence of water
- Pressure forces P_a from backfill ($P_{bf,a}$) and superimposed load ($P_{sur,a}$)
- Pressure forces due to water (P_w)
- Inertial forces from seismic action on wall, backfill and superimposed load ($H_{inertia}$)

The x-y origin of convenience for calculation of overturning and restoring moments is at the base of the foundation slab at the front of the toe.

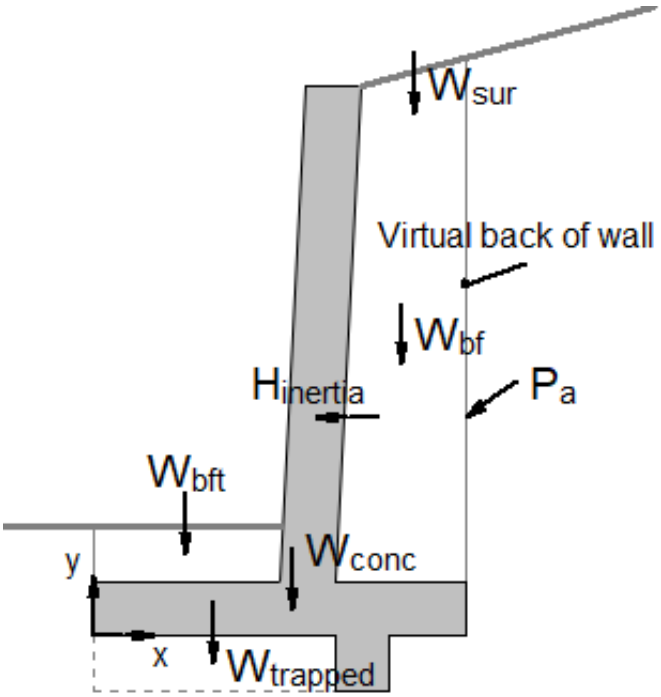


Figure 7-1: Load diagram

7.2 Virtual back of wall

Soil pressure calculations are simplified by assuming that the pressure acts on a 'virtual' back of wall, represented by a vertical plane directly up from the back of the heel (see Figure 7-1 above).

The soil behind the concrete stem and above the heel is assumed to act integrally with the wall. Active pressure from the backfill ($P_{bf,a}$) and any superimposed load ($P_{sur,a}$) is deemed to act on the virtual wall, rather than directly on the concrete wall. Also, the weight of the backfill based on the soil volume directly above the heel (W_{bf}) and the superimposed load (W_{sur}) between the concrete wall and the virtual wall are assumed to assist with the stability of the wall for both overturning and sliding.

Rather than using β_w (the actual slope of the back face of the concrete wall) and δ'_{bf} (the angle of interface friction between the backfill and the concrete), effective values β_{eff} (effective slope of the vertical wall) and δ_{eff} (effective friction along the vertical virtual wall) are used in analysis.

Vertical virtual wall is an artifice

It is acknowledged that a vertical virtual back of wall is an artifice, and typically does not represent a true failure plane. For a typical wall, a failure plane often develops running from the top of the concrete wall stem to the back of the heel, e.g. see Figure 7-2 below.

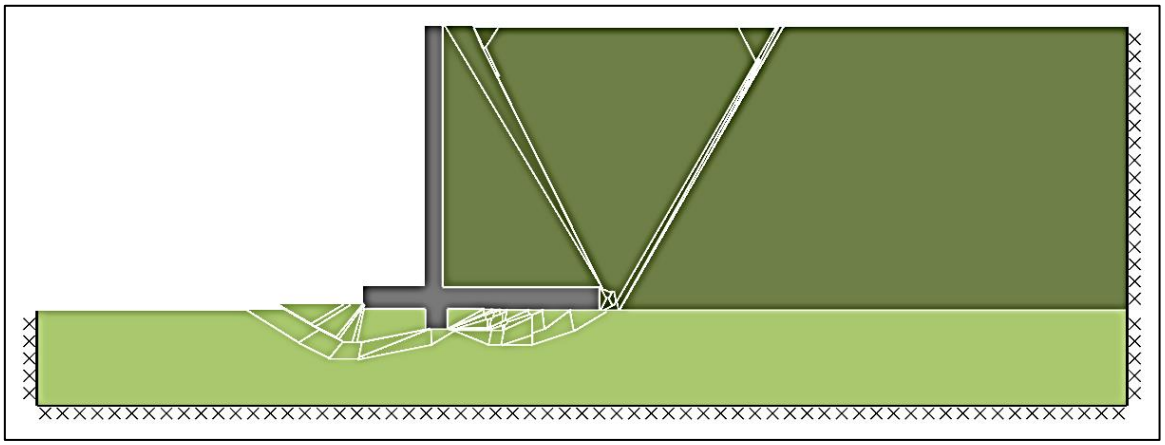


Figure 7-2: From finite element analysis with cohesionless backfill

A question then arises about how to determine the effective δ_{eff} using a vertical virtual wall – refer to a discussion on this in O'Sullivan & Creed [12]. For a horizontally retained slope, a naive use of $\delta_{eff} = \Phi'_{bf}$ (reasoning that the vertical virtual wall is along a soil/soil interface) will typically result in an unconservative statically inadmissible solution, since the friction along the actual failure plane may then be required to be greater than Φ'_{bf}. O'Sullivan & Creed conclude that a vertical virtual wall may be safely used if δ_{eff} is less than some small (calculated) value. For simplicity, at this time we have taken the approach that δ_{eff} should be set to zero, while recognising that a higher δ_{eff} may be justifiable for walls with a retained slope and long heel.

Heel size effects

Heel size affects the interaction of the virtual vertical back of wall, effective soil-wall interface friction, etc. In summary, this design guide classifies a heel into one of three categories:

- Long heel – where the heel breadth exceeds that forming an angle λ shown in the following diagram equal to the 'Rankine angle' (45° + $\Phi'_{bf}/2$)
- Short heel – any between long heel and no heel
- 'No' heel – where the top of the wall coincides vertically with the rear of the heel (λ=90°)

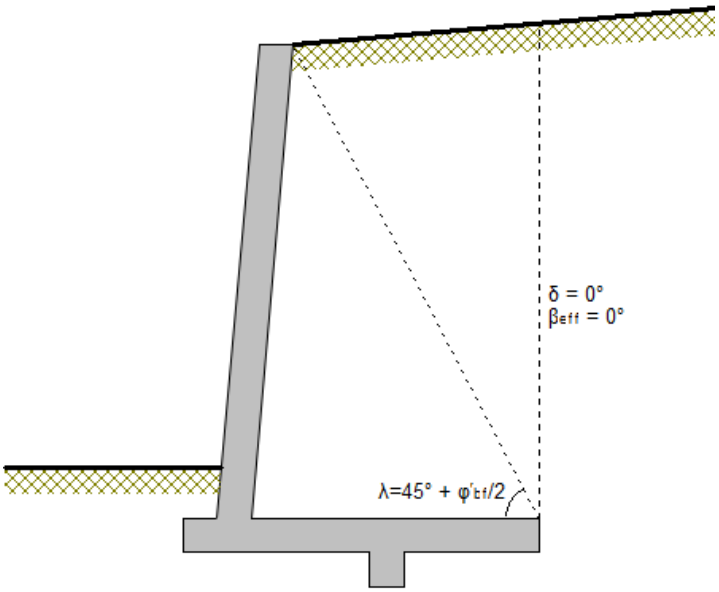


Figure 7-3: Crossover between Short and Long heel when angle λ = 45° + $\Phi'_{bf}/2$

Where the heel is 'long', this guide assumes that $\delta_{eff} = \beta_{eff} = 0$, otherwise δ_{eff} is linearly interpolated between $\delta_{eff} = \delta'_{bf}$ (for no heel) and 0 (for the crossover from short to long heel).

Heel breadth	***Effective wall slope β_{eff}***	***Effective friction angle δ_{eff}***
'No' heel: top of stem directly above back of heel (λ = 90°)	0	δ'_{bf}
Short heel: $B_h < B_{h,crossover}$ (λ > 45° + $\Phi'_{bf}/2$)	0	Interpolate between δ'_{bf} and 0
Long heel (λ ≤ 45° + $\Phi'_{bf}/2$)	0	0

Depth of active soil pressure

The depth of the active soil pressure (i.e. the height of the virtual back-of-wall) is assumed to extend down to the:

- base of the footing for overturning calculations
- top of the footing for calculating the moment at the base of the stem
- (for sliding calculations with a key):
    - base of the footing if the key is at the front of the footing
    - base of the key if the key is right at the back of the footing
    - a linear interpolation if in between

Refer, for example, to indicative Figure 7-4: Earth horizontal pressure blocks (pressures under sliding conditions shown dashed).

7.3 Superimposed loading

This guide assumes that the direct vertical load on the heel (W_{sur}) resulting from the superimposed load is the distributed load multiplied by the horizontal distance between the top of the wall and the back of the heel. W_{sur} may be zero, for example if there is no heel. However, ω_D and ω_L will still contribute to the pressure force $P_{sur,a}$ even if W_{sur} is zero.

Superimposed loads can be both stabilising (W_{sur} assists resistance to overturning and sliding) and destabilising ($P_{sur,a}$ increases the active horizontal pressure causing overturning and sliding). Different load factors are used for each of these situations:

$$\omega_s^* = \omega_D \cdot \alpha_{Ds} + \omega_L \cdot \alpha_{Ls} \quad \text{(static, stabilising)}$$

$$\omega_{se}^* = \omega_D \cdot \alpha_{Dse} + \omega_L \cdot \alpha_{Lse} \quad \text{(seismic, stabilising)}$$

$$\omega_d^* = \omega_D \cdot \alpha_{Dd} + \omega_L \cdot \alpha_{Ld} \quad \text{(static, destabilising)}$$

$$\omega_{de}^* = \omega_D \cdot \alpha_{Dde} + \omega_L \cdot \alpha_{Lde} \quad \text{(seismic, destabilising)}$$

7.4 Pressure coefficients – basis

The active earth pressure coefficients for static and seismic cases (K_a and K_{ae} respectively) and the active pressure coefficients for superimposed loading (K_{aq} and K_{aqe}) used to calculate pressure at depth are taken from Coulomb's adjusted equation for rough wall and sloping backfill, modified by Mononobe-Okabe for seismic loading.

δ_{eff} and β_{eff} as defined in Section 7.2 above are used directly in these equations.

The passive earth pressure coefficients for static and seismic cases (K_p and K_{pe} respectively) are based on a formulation by Mylonakis et al 2007 [8].

Note: the Coulomb/Mononobe-Okabe formula based on wedge theory should not be used since it significantly overestimates the passive pressure component).

Refer Appendix C – Pressure coefficients for more details.

7.5 Soil pressure at depth

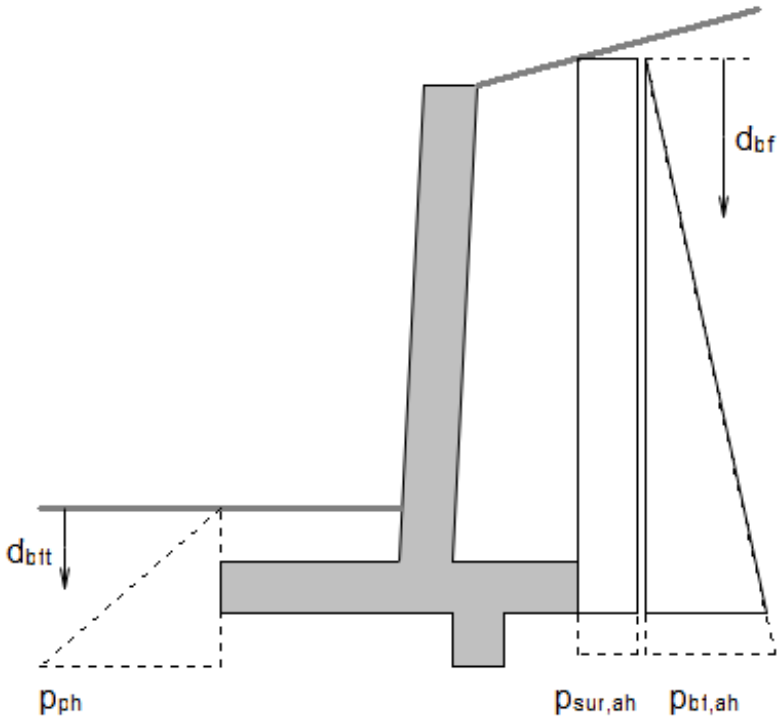


Figure 7-4: Earth horizontal pressure blocks (pressures under sliding conditions shown dashed)

Active pressure – static

The active pressure from the soil at depth d_{bf}, applied at an angle corresponding to the friction angle of the soil (plus the slope of the back of the wall where a virtual back of wall is not assumed), is:

$$p_{bf,a} = K_a \cdot \gamma_{bf} \cdot d_{bf}$$

The active pressure resulting from any superimposed (surcharge) loading is:

$$p_{sur,a} = K_{aq} \cdot \omega^*$$

The (destabilising) horizontal components of these pressures are used when calculating pressure forces on the wall:

$$p_{bf,ah} = K_{ah} \cdot \gamma_{bf} \cdot d_{bf}$$
$$p_{sur,ah} = K_{aqh} \cdot \omega_d^*$$

where

$$K_{ah} = K_a \cdot \cos(\delta_{eff} + \beta_{eff}) \text{ and } K_{aqh} = K_{aq} \cdot \cos(\delta_{eff} + \beta_{eff})$$

This document assumes that the pressure resulting from a superimposed (surcharge) load acts at the same angle as the pressure due to the soil itself.

The (stabilising) vertical components of these pressures are given by:

$$p_{bf,av} = K_{av} \cdot \gamma_{bf} \cdot d_{bf}$$
$$p_{sur,av} = K_{aqv} \cdot \omega_s^*$$

where

$K_{av} = K_a \cdot \sin(\delta_{eff} + \beta_{eff})$ and $K_{aqv} = K_{aq} \cdot \sin(\delta_{eff} + \beta_{eff})$

Active pressure – seismic

Similarly, the horizontal and vertical components of the active pressures under seismic loading are:

$$p_{bf,aeh} = K_{aeh} \cdot \gamma_{bf} \cdot d_{bf}$$
$$p_{sur,aeh} = K_{aqeh} \cdot \omega_{de}^*$$
$$p_{bf,aev} = K_{aev} \cdot \gamma_{bf} \cdot d_{bf}$$
$$p_{sur,aev} = K_{aqev} \cdot \omega_{se}^*$$

where

$K_{aeh} = K_{ae} \cdot \cos(\delta_{eff} + \beta_{eff})$, and $K_{aqeh} = K_{aqe} \cdot \cos(\delta_{eff} + \beta_{eff})$

$K_{aev} = K_{ae} \cdot \sin(\delta_{eff} + \beta_{eff})$, and $K_{aqev} = K_{aqe} \cdot \sin(\delta_{eff} + \beta_{eff})$

In this guide, seismic loads are deemed to act at the same height and same angle as the equivalent static horizontal pressure loads. This is considered to be a reasonable assumption for a concrete cantilevered retaining wall that has no anchors or other restraining structures.

Passive pressure

The front of the footing is assumed to have been backfilled with cohesionless soil, but should only be included in the analysis where there is a high degree of confidence in the soil properties, and that this soil will remain in place for the design life of the wall.

Note that the unfactored passive pressure force is reduced under seismic loading, since the instantaneous dynamic component acts in the opposite direction from the static component. The available passive resistance under seismic loading is therefore less than that available under static loading.

The horizontal components of pressure under static and seismic loading are:

$p_{ph} = K_{ph} \cdot \gamma_{bft} \cdot d_{bft}$ (static)

$p_{peh} = K_{peh} \cdot \gamma_{bft} \cdot d_{bft}$ (seismic)

where

$K_{ph} = K_p \cdot \cos(\delta'_{bf} - \beta)$, $K_{peh} = K_{pe} \cdot \cos(\delta'_{bf} - \beta)$ (β normally zero here)

Formulas for K_p and K_{pe} are given in Appendix C.

For simplicity, the vertical component of the passive pressure is ignored because it is small and can only be reliably attributed to the footing portion anyway.

7.6 Water pressure at depth

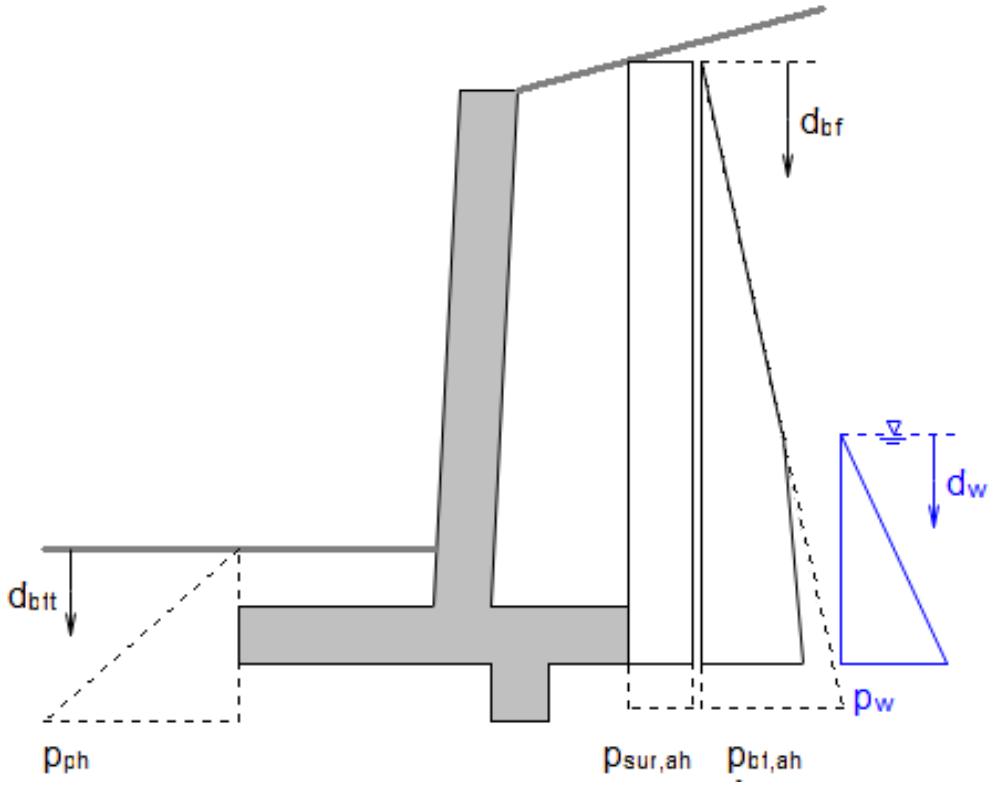


Figure 7-5: Horizontal pressure blocks with water table (pressures under sliding conditions shown dashed) (refer to Figure 8-1 for additional nomenclature)

Change to applied pressures due to water table

Any water table behind the retaining wall modifies the net pressure below the water table as follows, referring to Figure 8-1 for geometrical dimensions.

For static:

a) Adding a horizontal static water pressure on the virtual wall

$$p_w = \gamma_w \cdot d_w$$

b) Reducing the static active soil pressure below the water table (the unit weight of submerged soil $\gamma_{bf,sub} = \gamma_{bf,sat} - \gamma_w$ is used in the calculation for $p_{bf,ah}$ instead of γ)

$$\Delta p_{bf,ah} = -K_{ah} \cdot (\gamma_{bf} - \gamma_{bf,sub}) \cdot d_w$$

c) Reducing the static passive soil pressure in front of the toe (the water table in front of the toe can be no higher than D_f)

$$\Delta p_{ph} = -K_{ph} \cdot (\gamma_{bft} - \gamma_{bft,sub}) \cdot d_{bft} \qquad (when\ h_w > D_f)$$

$$\Delta p_{ph} = -K_{ph} \cdot (\gamma_{bft} - \gamma_{bft,sub}) \cdot d_w \qquad (when\ h_w \leq D_f)$$

For seismic:

d) In addition to the contribution from static water pressure, a water table behind the wall affects the pressure loading on the wall by:
   - modifying the contribution of the backfill to $H_{inertia}$
   - increasing the effective value of k_h for seismic loading

   Any hydrodynamic effects are minor for low values of k_h (e.g. less than 0.2), but can be significant for values of $h_w > H/3$. Given the assumptions and limitations given in Section 2 above, no allowance is made for this.

Buoyancy adjustment of gravity loads due to presence of water

Gravity loads (concrete, backfill, and toe overburden) need to be adjusted due to the presence of water.

a) Concrete buoyancy adjustment
$$\Delta W_{conc} = -\gamma_w \cdot (volume\ of\ concrete\ below\ water\ table)$$
b) Backfill buoyancy adjustment
$$\Delta W_{bf} = (\gamma_{bf,sub} - \gamma_{bf}) \cdot (volume\ of\ backfill\ below\ water\ table)$$
c) Toe overburden buoyancy adjustment
$$\Delta W_{bft} = (\gamma_{bft,sub} - \gamma_{bft}) \cdot (volume\ of\ overburden\ below\ water\ table)$$

Assumptions regarding water pressure

This guide assumes that:

- the water table in front of the toe is at the same level as the water table behind the wall, although with a maximum height of the ground level in front of the wall
- any downward water pressure on the footing heel is balanced by upward water pressure beneath, noting that any (minor) pressure variation due to the footing thickness is likely to be negated by a reducing pressure gradient from the rear to the front of the footing
- due to the typically impervious nature of reinforced concrete walls, the direct static pressure from the water table behind the wall is assumed to act under seismic loading in the same manner as for static loading
- a water table substantially below the underside of the foundation is assumed to have no effect on the design – the designer must validate this assumption and calculate bearing capacity as appropriate. The equations in this guide assume that the water table is either not present, or is at least as high as the underside of the foundation, but refer to Bowles (1995) [3] for intermediate depths
- water table loading is temporary, rather than significant long-term loading

Water table effects can be significant, especially for higher seismicity regions. However, allowance for a sustained water table above ground level under seismic loading is beyond the intended scope of this guide.

7.7 Pressure forces

The following are presented as the base unfactored loads, with relevant load or reduction factors applied later, as appropriate.

Active pressure forces – horizontal component

For backfill, the active pressure force is:

$$P_{bf,ah} = \tfrac{1}{2} \cdot p_{bf,ah} \cdot H_v - \tfrac{1}{2} \cdot \Delta p_{bf,ah} \cdot h_w \quad \text{(static)}$$

$$P_{bf,aeh} = \tfrac{1}{2} \cdot p_{bf,aeh} \cdot H_v - \tfrac{1}{2} \cdot \Delta p_{bf,aeh} \cdot h_{we} \quad \text{(seismic)}$$

For dead and live surcharge loading, the active pressure force is:

$$P_{sur,ah} = p_{sur,ah} \cdot H_v \quad \text{(static)}$$

$$P_{sur,aeh} = p_{sur,aeh} \cdot H_v \quad \text{(seismic)}$$

Active pressure forces – vertical component

Downward 'friction' forces due to soil pressure are calculated similarly to the above, as follows:

$$P_{bf,av} = \tfrac{1}{2} \cdot p_{bf,av} \cdot H_v - \tfrac{1}{2} \cdot \Delta p_{bf,av} \cdot h_w \quad \text{(static)}$$

$$P_{bf,aev} = \tfrac{1}{2} \cdot p_{bf,aev} \cdot H_v - \tfrac{1}{2} \cdot \Delta p_{bf,aev} \cdot h_{we} \quad \text{(seismic)}$$

$$P_{sur,av} = p_{sur,av} \cdot H_v \quad \text{(static)}$$

$$P_{sur,aev} = p_{sur,aev} \cdot H_v \quad \text{(seismic)}$$

These vertical forces are assumed to apply at the virtual back of wall.

Active pressure forces – overturning vs sliding

The same formulas given above are used to calculate pressure forces for both overturning and sliding analyses, except that for a sliding analysis, the virtual wall can extend below the underside of the foundation due to the key (refer Section 7.2 above). In this case, H_v in the above equations is replaced by $H_{v,sl}$.

Passive pressure forces

Horizontal component

For cohesionless soil in front of the toe, the horizontal component of the passive soil force resisting sliding is:

$$P_{ph} = \tfrac{1}{2} \cdot p_{ph} \cdot (D_f + d_k) \; - \tfrac{1}{2} \cdot \Delta p_{ph} \cdot h_w \qquad when\ h_w \le D_f \text{ (static)}$$

$$P_{ph} = \tfrac{1}{2} \cdot p_{ph} \cdot (D_f + d_k) \; - \tfrac{1}{2} \cdot \Delta p_{ph} \cdot d_{bft} \qquad when\ h_w > D_f \text{ (static)}$$

$$P_{ph} = \tfrac{1}{2} \cdot p_{peh} \cdot (D_f + d_k) \; - \tfrac{1}{2} \cdot \Delta p_{peh} \cdot h_{we} \qquad when\ h_{we} \le D_f \text{ (static)}$$

$$P_{ph} = \tfrac{1}{2} \cdot p_{peh} \cdot (D_f + d_k) \; - \tfrac{1}{2} \cdot \Delta p_{peh} \cdot d_{bft} \qquad when\ h_{we} > D_f \text{ (static)}$$

Vertical component

As the vertical component of the passive pressure is minor, this design guide ignores it.

Water pressure forces

The horizontal force due to static water pressure, applicable to both static and seismic load cases, is:

$$P_w = \frac{1}{2} \cdot p_w \cdot h_w$$

7.8 Inertia forces

The seismic load, $H_{inertia}$, is calculated by applying the design earthquake coefficient k_h (refer to Section 6.3) to the gravity loads (concrete, soil, and applied surcharge). The earthquake vertical acceleration coefficient is assumed to be 0.

$$H_{inertia} = k_h \cdot (W_{conc,e} + W_{bf,e} + W_{bft,e} + W_{sur,e})$$

7.9 Middle third rule

To avoid the rear of the footing lifting under static loads (assuming a triangular rather than rectangular pressure distribution), the point of application of the reaction force on the underside of the foundation would have to be in the middle third of the footing. Arguably, this middle third rule has come from a working stress background, on the premise that load placement within the middle third ensures that no tension exists under the pad.

Even under factored load design, compliance with the middle third rule is still encouraged, especially for static and low-level seismic analyses. For higher-level seismic demands (e.g. a PGA of more than 0.4g), middle-third compliance may be impractical.

Further, where this rule is satisfied, the full foundation breadth is available for the sliding capacity for cohesive soils.

7.10 Load factors for intermediate calculations

As the reader will appreciate, load factors (≥1.0) and strength reduction factors (≤1.0) represent a construct in order to achieve a suitable margin between demand (S*) and capacity (R*). As such, it is generally beneficial to calculate unfactored demand and resistance values, and then apply the relevant load factors at the limit state combination stage. This particularly applies to 'intermediate'-type calculations, where factored versus unfactored loads may give wildly different answers, e.g. for the bearing capacity load inclination factors in Appendix D, where unfactored loads are used.

8. Calculation process

This section provides an overview of the calculation process.

8.1 Overview

In summary, the design process is undertaken in the following order, with the steps expanded in the following sections:

- Define inputs (Section 8.2)
- Calculate forces and moments on wall (Section 8.3)
- Check for overturning stability (Mode 2) (Section 8.4)
- Check for bearing capacity failure (Mode 2) (Section 8.5)
- Check for sliding failure (Mode 3) (Section 8.6)
- Calculate design actions at base of stem (Section 8.7)
- Check wall structural capacity (not covered in this document)

8.2 Define inputs

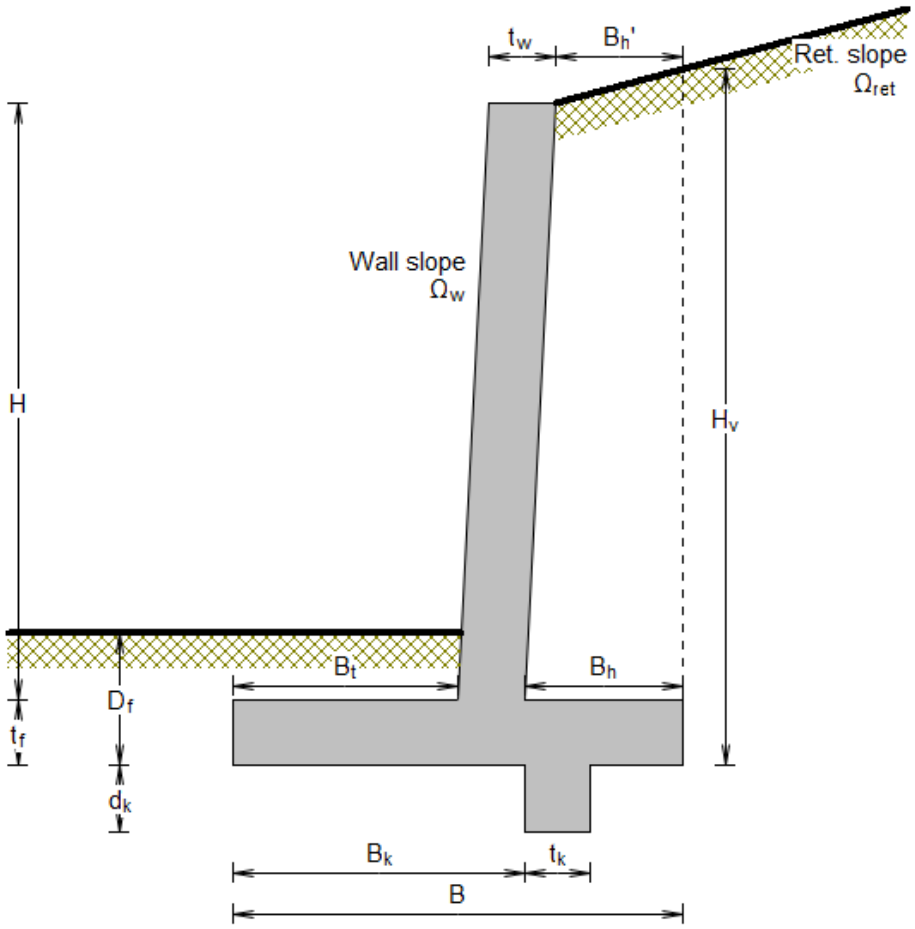


Figure 8-1: Wall geometry, showing virtual back of wall

Inputs need to be defined in the following areas (refer Appendix A – Nomenclature):

- design parameters
- wall geometry
- site geometry
- soil properties
- surcharge, water table and seismic loading

8.3 Calculate forces and moments on wall

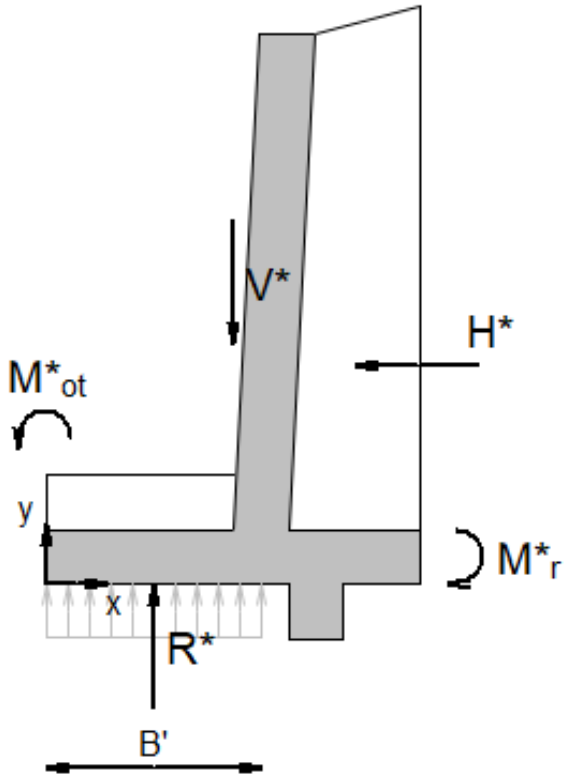


Figure 8-2: Force diagram for overturning action

The overturning moment M_{ot}* caused by the applied horizontal loading H* is balanced by the restoring moment M_r* from the applied vertical loading, and the upwards reaction R* from the soil. Stability against overturning is checked by:

- calculating the effective width of bearing on the underside of the foundation, and then
- checking soil bearing capacity

Vertical (stabilising) forces

- W_{conc}: wall self-weight, after any allowing for buoyancy
- W_{bf}: weight of soil above the heel, using buoyant soil weight for any portion below the water table

- W_{bft}: weight of soil above the toe (usually stabilising, but the designer should check if there is only a small toe, in which case this soil may be destabilising)
- W_{sur}: superimposed load above the backfill

$$W_{sur} = \omega_s^* \cdot Bh' \quad \text{(static)}$$

$$W_{sur,e} = \omega_{se}^* \cdot Bh' \quad \text{(seismic)}$$

- Total weight:

$$W = (W_{conc} + W_{bf} + W_{bft} + W_{sur})$$

- P_{av}: vertical component of soil pressure at interface, applied at the virtual back-of-wall if there is one, and at the height of the total pressure force otherwise
- Total factored applied vertical load (conservatively, α_{Gs} is applied to P_{av}):

$$V^* = (W + P_{bf,av} + P_{sur,av}) \cdot \alpha_{Gs} \quad \text{(static)}$$

$$V^* = (W + P_{bf,aev} + P_{sur,av}) \cdot \alpha_{Gse} \quad \text{(seismic)}$$

Horizontal (de-stabilising) forces

- $P_{bf,ah}$: force due to pressure from the retained soil
- $P_{sur,ah}$: force due to pressure from the superimposed load above the backfill
- P_w: static force from any water pressure behind the wall, applied at 1/3 * the height of the water table
- $H_{inertia}$: seismic load, only applicable to seismic load cases
- Total factored horizontal load:

$$H^* = P_{bf,ah} \cdot \alpha_a + P_{sur,ah} \cdot \alpha_q + P_w \cdot \alpha_w \quad \text{(static)}$$

$$H^* = P_{bf,aeh} \cdot \alpha_{ae} + P_{sur,aeh} \cdot \alpha_{qe} + P_{we} \cdot \alpha_{we} + H_{inertia} \quad \text{(seismic)}$$

8.4 Overturning failure – stability

Stability equations

The above forces lead directly to the following stability equations, where x_i and y_i are the horizontal and vertical distances respectively from the origin to the relevant force:

- Vertical equilibrium:

$$R = V^*$$

- Restoring moment (for static and seismic respectively):

$$M_r^* = (W_{conc} \cdot x_{conc} + W_{bf} \cdot x_{bf} + W_{bft} \cdot x_{bft} + W_{sur} \cdot x_{sur} + P_{bf,av} \cdot x_{bf,a} + P_{sur,av} \cdot x_{sur,a}) \cdot \alpha_{Gs}$$

$$M_{re}^* = (W_{conc} \cdot x_{conc} + W_{bfe} \cdot x_{bfe} + W_{bft} \cdot x_{bft} + W_{sur} \cdot x_{sur} + P_{bf,aev} \cdot x_{bf,ae} + P_{sur,aev} \cdot x_{sur,ae}) \cdot \alpha_{Gse}$$

- Overturning moment:

$$M_{ot}^* = P_{bf,ah} \cdot y_{bf} \cdot \alpha_a + P_{sur,ah} \cdot y_{sur,a} \cdot \alpha_q + \frac{1}{3} P_w \cdot h_w \cdot \alpha_w \quad \text{(static)}$$

$$M^*_{ote} = P_{bf,aeh} \cdot y_{bfe} \cdot \alpha_{ae} + P_{sur,aeh} \cdot y_{sur,ae} \cdot \alpha_{qe} + \frac{1}{3} P_{we} \cdot h_{we} \cdot \alpha_{we} + H_{inertia} \cdot y_H \quad \text{(seismic)}$$

For stability, the restoring moment must be greater than the overturning moment, i.e.

$$M^*_r - R \cdot \frac{B'}{2} \geq M^*_{ot}$$

where B′ is the effective width of the bearing.

Effective bearing width

Based on the limit state equations above:

$$R \cdot \frac{B'}{2} + M^*_{ot} - M^*_r = 0$$

Since R = V*, the effective width of bearing is:

$$B' = 2 \cdot \frac{M^*_r - M^*_{ot}}{V^*}$$

Note that B′ must be calculated separately for both static and seismic load cases.

Check 'middle third'

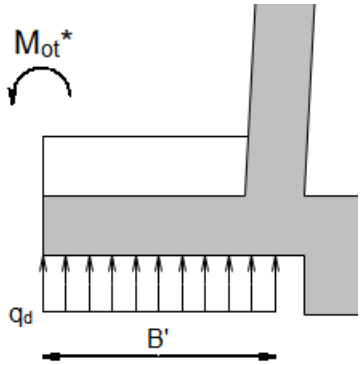
To achieve 'good design', the point of application of the reaction force on the underside of the foundation should not be closer to the edge than B/3, i.e.:

$$\frac{B'}{2} \; must\; be \geq \frac{B}{3}$$

Refer also to Section 7.9 above.

8.5 Overturning failure – bearing

Check soil bearing capacity



The design bearing pressure is determined by dividing the design vertical load by the effective bearing area of the foundation (assuming a rectangular stress block):

$$q_d = V^*/B'$$

The ultimate bearing strength (soil capacity) q_u is calculated according to:

$$q_u = c\lambda_{cs}\lambda_{cd}\lambda_{ci}N_c + q\lambda_{qs}\lambda_{qd}\lambda_{qi}N_q + \frac{1}{2}\gamma_{f,eff}B'\lambda_{\gamma s}\lambda_{\gamma d}\lambda_{\gamma i}N_\gamma$$

Terms in this equation, based on the founding soil properties and foundation slab geometry, are defined in Appendix D.

The strength reduction factor φ_b is defined in Appendix B, and a satisfactory design in terms of overturning capacity is therefore where:

$$q_d \leq \varphi_b . q_u$$

i.e. the bearing capacity ratio must be less than 1:

$$CR = \frac{q_d}{q_u . \varphi_b} \leq 1$$

8.6 Sliding failure

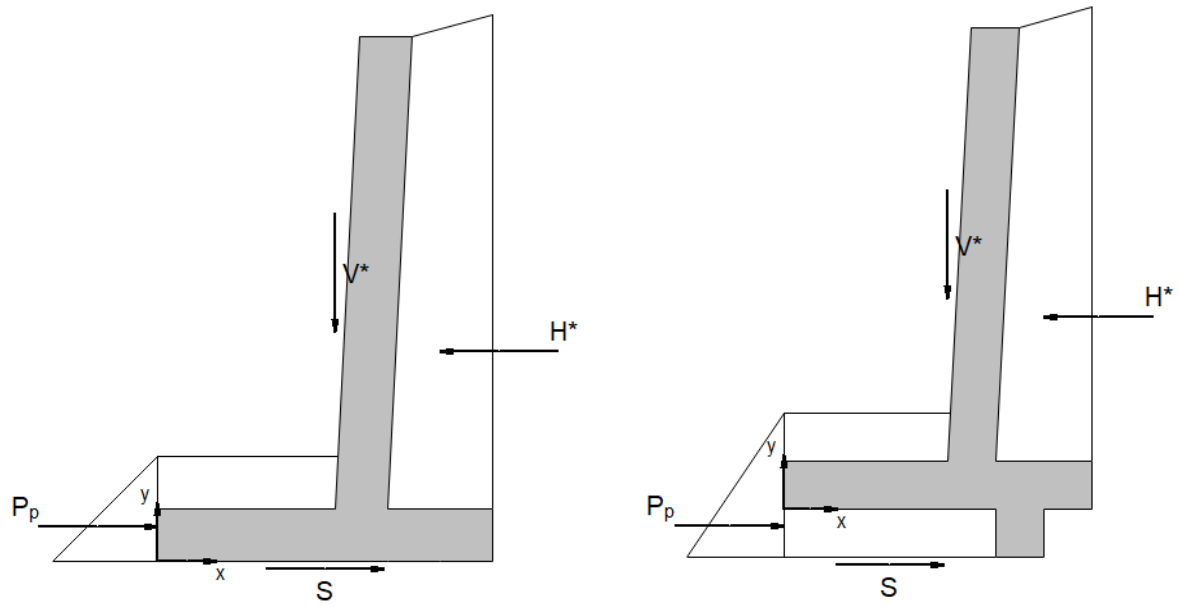


Figure 8-3: Force diagrams for sliding action
(refer to Figure 8-1 for nomenclature)

Sliding resistance

For most typical concrete cantilever retaining walls, sliding failure is the critical limit state criterion.

Resistance to sliding is provided by three sources (refer also to Bowles [3]):

- lateral passive resistance from the soil in front of the footing (P_p)
- sliding resistance from soil friction (S_f)
- sliding resistance from adhesion between the soil and the underside of the foundation slab (S_a)

Where there is a key, the soil trapped below the foundation in front of the key assists with the sliding (friction) resistance for a drained analysis. This is due to the increased gravity load, as well as a higher friction resistance (soil to soil vs soil to concrete).

$$R_{sl} = R + W_{trapped} \cdot \alpha_{Gs}$$

The various sliding cases are summarised below, with details of each case (key present or not present, key located at front or back, drained or undrained analysis) given in Appendix E.

Resistance from soil in front of the footing

Lateral resistance from passive earth pressure is given by:

$$P_p = 0.5 \cdot K_p \cdot \gamma_{bft} \cdot \left(D_f + d_k\right)^2$$

Friction with soil below foundation slab

Where there is no key, the ultimate shear strength S_f for a drained analysis relies on friction between the soil and the underside of the slab, and is given by:

$$S_f = R_{sl} \cdot tan\ \delta_f' \qquad \text{(drained only)}$$

A greater friction resistance is enabled when a key is present (provided it is at least, say, 0.2m deep), since the sliding failure plane is now within the founding soil rather than between soil and concrete.

The capacity also depends on where the key is located – the greatest benefit is when the key is located at the back of the heel. The ratio of the soil-to-soil interface width to the soil-to-concrete interface width (r_s) is used to weight the two components respectively:

$$Ratio\ r_s = MIN[\frac{B_k}{B_{sl}'}, 1]$$

where the effective bearing width used for friction resistance calculations, B'_{sl}, is given by:

- if overturning stability is such that the middle-third rule is complied with (i.e. B'/2 ≥ B/6), then the full foundation breadth is available for friction resistance, i.e. $B'_{sl} = B$
- otherwise, $B'_{sl} = B'$ (refer Section 8.4).

Friction resistance (for drained conditions only) is then as follows:

$$S_f = R_{sl} \cdot [r_s \cdot tan\ \varphi_f' + (1 - r_s) \cdot tan\ \delta_f']$$

Adhesion between soil and foundation slab

(Cohesive soils only)

$$S_a = B'_{sl} \cdot c'[r_s + (1 - r_s) \cdot r_A] \quad \text{(drained)}$$

$$S_a = B'_{sl} \cdot s_u[r_s + (1 - r_s) \cdot r_A] \quad \text{(undrained)}$$

Check for sliding stability

A satisfactory design in terms of sliding is where the design horizontal load H* is less than the dependable capacity:

$$H^* \leq \varphi_{sl} \cdot (S_f + S_a) + \varphi_p \cdot P_p$$

i.e. the sliding capacity ratio must be less than 1:

$$CR = \frac{H^*}{\varphi_{sl} \cdot (S_f + S_a) + \varphi_p \cdot P_p} \leq 1$$

8.7 Calculate design actions at base of stem

This will typically have to be calculated separately for the static and seismic load cases.

For design actions on the stem base, forces and moments are calculated based on the pressure down to the top of the footing (as distinct from the base of the footing used for overturning calculations and the base of the key used for sliding calculations).

As this document is targeted at Structural Engineers, and the geotechnical principles and loading calculations already clearly laid out, for simplicity's sake further detail is considered unnecessary, as is calculation of the structural capacity aspects.

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10. Amendment History

Version	*Date*	*Description*
0.9	17/12/2025	First public (DRAFT) release
1.0	30/9/2026	First formal release

Appendices

Appendix A – Nomenclature

A.1 Wall Geometry

B	Breadth of the footing
B_h, B_t	Breadth of the heel, breadth of the toe Note that $B_t + t_{wb} + B_h = B$
B_h'	Vertical projection of the superimposed load on to the heel (same as B_h if the slope of the back of the wall is zero, and = 0 if the top of the wall overhangs the back of the heel, e.g. if there is no heel)
d_k	Depth of the key (below the footing)
B_k	Distance from the front of the footing to the front of the key
D_f	Depth of the footing (distance from top of the soil in front of the wall to the base of footing)
H, H_t	Height of the wall from the top of the footing (H) or underside of the footing (H_t)
L_f	Length of footing (and wall)
t_f	Footing thickness
t_k	Thickness of the key
t_w	Wall stem thickness (at top of wall)
t_{wb}	Wall stem thickness (at base of stem = top of footing), usually = t_w
β_w	Angle of the back face of the wall from vertical, negative for a wall leaning back towards the retained soil. Normally calculated based on Ω_w and any variance in the wall thickness
Ω_w	Slope of the front of the wall, positive for a wall leaning back towards the retained soil

A.2 Site

Ω_{ret}	Slope of retained soil
Ω_{site}	Site slope in front of the wall, assumed 0 in this guide

A.3 Soil properties

Founding soil	
c'	Effective cohesion of the founding soil (cohesive soils only)
c_A	Adhesion of founding soil to underside of concrete foundation = r_A * c′ (cohesive soils only)
q_d	Design bearing pressure
q_u	Ultimate bearing strength (soil capacity)
r_A	Ratio of cohesive founding soil adhesion to cohesion, e.g. 0.65, typically 0.5-1.0
s_u	Undrained shear strength of the founding soil (cohesive soils only)
δ'_f	Angle of interface friction between the founding soil and the concrete, e.g. • φ'_f for calculating shearing resistance with cast in-situ concrete foundations • 2/3 φ'_f for calculating shearing resistance with smooth precast foundations Note that this value will be the lowest of all interfaces (i.e. if there is a blinding layer or DPS, then that should be considered)
γ_f	Drained unit weight of founding soil
$\gamma_{f,sat}$	Saturated unit weight of founding soil
$\gamma_{f,sub}$ (or γ'_f)	Unit weight of submerged founding soil = $\gamma_{f,sat}$ - γ_w
$\gamma_{f,eff}$	Effective founding soil density at the base of the foundation, used for bearing strength calculations
φ'_f	Effective angle of founding soil shearing strength (internal friction angle)

Backfill (assumed always cohesionless)	
G_s	Soil specific gravity, being the ratio of the mass of solids in the soil and the mass of water with the same volume. For the typical range of soils used as backfill, this can be taken as = 2.65
δ'_{bf}	Angle of interface friction between the backfill soil and the back of the wall, e.g. 2/3 φ'_{bf} for calculating an active pressure coefficient K_a Note that δ'_{bf} may be as low as 0, depending on what is between the backfill soil and the concrete stem Refer also below for the definition of δ_{eff}, the effective friction angle at the virtual back of wall
γ_{bf}	Drained unit weight of backfill soil
$\gamma_{bf,sat}$	Saturated unit weight of backfill soil
$\gamma_{bf,sub}$ (or γ'_{bf})	Unit weight of submerged backfill soil = $\gamma_{bf,sat}$ - γ_w
φ'_{bf}	Effective angle of backfill soil shearing strength (internal friction angle)
Backfill on toe	
δ'_{bft}	Angle of interface friction between toe backfill soil and front of foundation slab, usually 0.5φ'_{bft} for calculating the passive pressure coefficient K_p
γ_{bft}	Drained unit weight of toe backfill soil
φ'_{bft}	Effective angle of toe backfill soil shearing strength (internal friction angle)

A.4 Ground water

h_w	Retained soil water level above base of footing
γ_w	Unit weight of water = 9.81 kN/m^3

A.5 Applied loading

k_h	Design earthquake coefficient (horizontal acceleration); the earthquake coefficient for vertical acceleration is assumed to be 0
$k_{h,eff}$	Effective value of k_h where there is a water table behind the wall
ω_D, ω_L	Unfactored uniform superimposed dead and live load; uniform distribution per unit horizontal width, assumed continuous over the failure wedges Include the effect of any line or point load surcharge by converting to equivalent uniform load

A.6 Design factors

$\Phi_{b,\ sl\ etc}$	Strength reduction factor – refer to Appendix B.2 for additional documentation and recommended values
$\alpha_{a,\ q,\ etc}$	Load factor – refer to Appendix B.3 for additional documentation and recommended values

A.7 Derived properties

Derived dimensions	
d_{bf}, d_{bft}, d_w	Depth dimension from backfill surface above virtual wall, toe overburden surface, and top of water table respectively (see Figures 7-2 and 7-3)
D_{wz}	Wedge zone depth, refer Appendix D
r_s	Key location ratio for sliding resistance (for walls with a key): ratio of the soil-to-soil interface width to the soil-to-concrete interface width (see Section 8.6)
Gravity loads	
W_*	Unfactored weight of concrete wall components, soil and superimposed dead + live load (surcharge) (see Figure 7-1) as follows: W_s: Stem W_f: Footing W_k: Key

	W_{bf}: Soil backfill above heel to top of wall including any sloping backfill W_{bft}: Overburden above toe $W_{trapped}$: Soil 'trapped' in front of key (below foundation slab) W_{sur}, $W_{sur,e}$: Total vertical load on heel due to superimposed load under static and seismic conditions respectively
W_{conc}	Unfactored total concrete loading, i.e. $W_{conc} = W_s + W_f + W_k$
W	Total gravity load
Virtual back of wall	
$B_{h,crossover}$	Breadth of heel where the angle of the 'Rankine' active failure plane $\lambda = 45° + \Phi'_{bf}/2$
H_v	Height of the virtual back of wall from the underside of the footing to the backfill ground surface used for an overturning analysis. This is the same as H_t if the slope of the backfill is zero, or the wall has a very short heel – refer Section 7.2
$H_{v,sl}$	Height of the virtual back of wall to the backfill ground surface used for a sliding analysis. When there is a key below the foundation, $H_{v,sl}$ will be larger than H_v
β_{eff}	Effective slope of the virtual back of wall Varies between 0 and the concrete wall β_w, depending on the length of the heel
δ_{eff}	Effective friction angle at the virtual back of wall Varies between 0 and the concrete/backfill friction angle δ'_{bf}, depending on the length of the heel
λ	Angle of a failure plane (measured from the horizontal) from the back of the heel to the top of the stem At the crossover heel length, $\lambda = 45° + \Phi'_{bf}/2$, also known as the Rankine active failure plane

Soil pressure coefficients	
K_{a*}, K_{p*}	Coefficients of active (K_{a*}) and passive (K_{p*}) earth pressure Coefficients under seismic loading have an "e" suffix Horizontal and vertical components are given by "h" and "v" suffixes respectively e.g. K_{aeh} is the horizontal component of the active earth pressure coefficient under seismic loading
K_{aq*}	Coefficients of active earth pressures due to superimposed loads Coefficients under seismic loading have an "e" suffix Horizontal and vertical components are given by "h" and "v" suffixes respectively e.g. K_{aqeh} is the horizontal component of the active pressure coefficient (for superimposed loads) under seismic loading
Force calculations	
B'	Effective width of bearing on the underside of the foundation, based on unfactored loads and a rectangular pressure distribution, refer Section 8.4
B'_{sl}	Effective bearing width used for friction resistance calculations, refer Section 8.6
$H_{inertia}$	Seismic load, calculated by applying the design earthquake coefficient to the gravity loads (concrete, soil, and applied surcharge) $H_{inertia} = k_h * W$
P_a	Unfactored net pressure force from backfill ($P_{bf,a}$) plus superimposed loading ($P_{sur,a}$) (see Figure 7-1) P_{ah}: Horizontal component of P_a P_{av}: Vertical component of P_a
P_p	Unfactored net resistance from passive earth pressure
q, q_e	Vertical stress at depth Df (underside of the foundation at toe) under static and seismic conditions respectively
q_d	Design bearing pressure on soil

R	Reaction force on underside of foundation under overturning, based on factored loads and a rectangular pressure distribution
R_{sl}	Reaction force on underside of foundation under sliding, including effect of weight of soil trapped under the foundation in front to of the key (if any)
S_f, S_a	Unfactored net sliding resistance from friction (S_f) and from adhesion of the underside of footing with the base soil (S_a)
x_i, y_i	The horizontal and vertical distances, respectively, from the origin to the relevant force, used when calculating moments about the x-y axes
Factored design loads	
CR	Ratio of factored design loads to dependable bearing or sliding capacity
H*	Design horizontal load = factored horizontal load from soil pressure + inertial load (e/q only) + water pressure where the water table is above the base of the foundation
H^*_{stem}	Design horizontal load (= shear) at base of stem
M_{ot}^*	Overall design overturning moment ('driving' moment) about the base of the toe caused by the applied horizontal loads, modified by relevant load factor
M_r^*	Overall restoring moment about the base of the toe from the applied vertical loads, modified by the relevant load factor
M^*_{stem}	Design moment at base of stem
V*	Effective design vertical load = factored gravity load + factored vertical load from pressure
ω_s^*, ω_d^*	Factored superimposed dead plus live load, uniform distribution per unit horizontal width. Subscript "s" indicates stabilising load, "d" indicates destabilising load

Note: unless otherwise indicated, use of subscripts "u" and "e" indicate undrained founding soil conditions and seismic loading respectively

Appendix B – Design factors

This document takes an LRFD approach (refer Section 2), factoring up loads (α load factors) and factoring down capacity (Φ reduction factors). This is similar to the majority of New Zealand design codes. Note that a factor-of-safety approach can be used by appropriate modification of the factors.

Historically, there has been some disparity and variation in the design parameters used in various design examples and guides. SESOC has sought to provide a consistent set of values going forward, the basis for which is provided in Section B.4 below, a study undertaken by John Wood for SESOC.

B.1 Summary of typical values

Recommended typical values for strength reduction and load factors are shown in the diagrams below. Refer to the following sections B.2 and B.3 for further detail and description. Refer to Appendix B.4 for industry background and SESOC recommended basis.

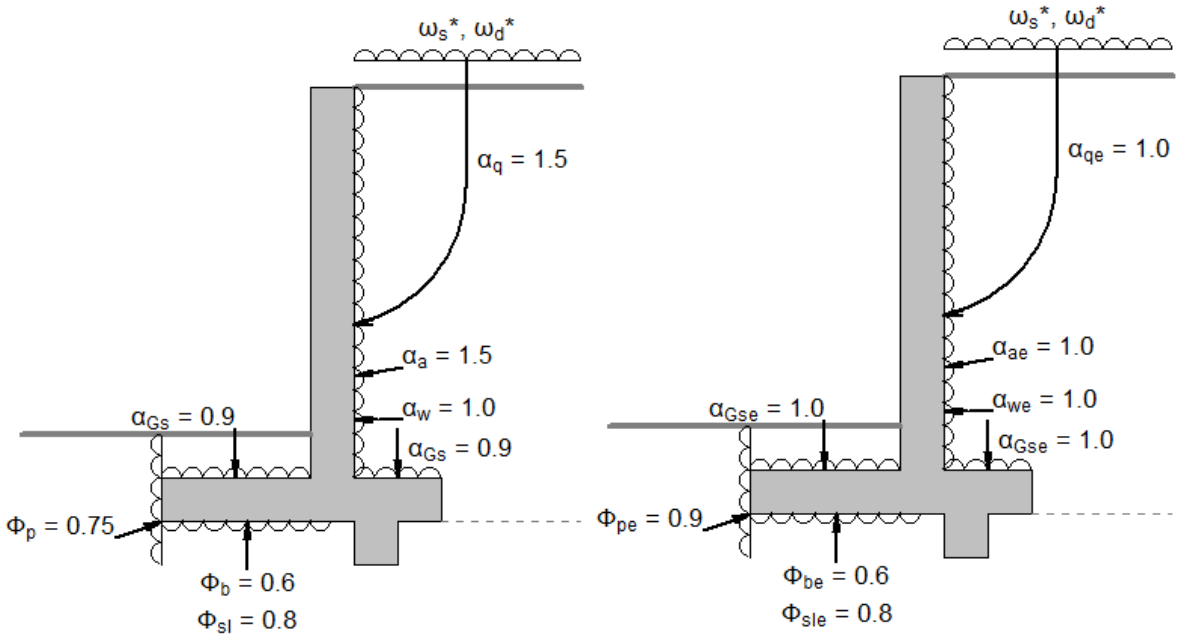


Figure B-1: Recommended design factors

B.2 Strength reduction factors

Item	*Typical value*	*Description*
Bearing/passive		Factors applied to obtain reliable resistance based on bearing or passive soil pressure
Φ_b	0.6	Strength reduction factor for static bearing failure: foundation rotation (failure mode 2 in Section 4 above)
Φ_{be}	0.6	Strength reduction factor for bearing failure under seismic loading
Φ_p	0.75	Strength reduction factor for passive earth pressure (assumed always cohesionless)
Φ_{pe}	0.9	Strength reduction factor for passive earth pressure under seismic loading (assumed always cohesionless)
Sliding		Factors applied to obtain reliable resistance to sliding based on friction
Φ_{sl}	0.8	Strength reduction factor for sliding failure (failure mode 3)
Φ_{sle}	0.8	Strength reduction factor for sliding failure under seismic loading

B.3 Load factors

Item	*Typical value static/seismic*	*Description*
Gravity load		Factors applied to gravity loads, that can be destabilising or stabilising. Generally determined in accordance with AS/NZS 1170.0, e.g. see 4.2(a)
α_{Gd}/α_{Gde}	1.5/1.0	Load factor for destabilising gravity loads under static/seismic loading
α_{Gs}/α_{Gse}	0.9/1.0	Load factor for stabilising gravity loads (wall, backfill, and superimposed dead load) under static/seismic loading

Superimposed loading		Factors applied to uniform dead and live loads (ω_D and ω_L) superimposed on the retained soil. Different factors are used depending on whether the superimposed load is stabilising or destabilising. Note that calculation of pressure on the wall resulting from the destabilising loading is subject to an additional α_q factor. Refer to NZGS/SESOC (2025) [10].
α_{Dd}/α_{Dde}	1.0/1.0	Load factor for superimposed permanent (dead) loading causing destabilising effect under static/seismic loading
α_{Ld}/α_{Lde}	1.0/0.4	Load factor for superimposed live loading causing destabilising effect under static/seismic loading
α_{Ds}/α_{Dse}	0.9/0.9	Load factor for superimposed permanent (dead) loading causing stabilising effect under static/seismic loading. Determine according to AS/NZS 1170.0, e.g. see 4.2(a)
α_{Ls}/α_{Lse}	0.9/0.0	Load factor for superimposed live loading causing stabilising effect under static/seismic loading. Determine according to AS/NZS 1170.0, e.g. 0.0 is implied by 4.2(a)
Active earth pressure		Factors applied to the pressure loads resulting from the retained soil and from any superimposed loading. Refer to NZGS/SESOC (2025) [10].
α_a/α_{ae}	1.5/1.0	Load factor for active loads (earth pressure) This factor is intended to cover uncertainty in the soil density, soil properties, and the soil-load pressure behaviour
α_q/α_{qe}	1.5/1.0	Load factor for earth pressure resulting from superimposed loads under static/seismic loading This factor is intended to cover uncertainty in the soil properties and in the soil-load pressure behaviour (transfer of superimposed load resulting in pressure on the wall), and has been chosen with due consideration of the cumulative effect of applying an active pressure

		load factor in addition to the destabilising dead and live load factors
Water pressure		
α_w/α_{we}	1.0/1.0	Load factor for water pressure, provided a conservative h_w is used to calculate water pressures under static/seismic loading AS/NZS 1170.0:2002 4.2.3(e) specifies a load factor of 1.2 to be applied to ground water assuming the water level is at ground level or, where information is available, the ground water level with an annual probability of exceedance of 1 in 50. Provided the backfill and drainage is known to be good, and particularly if a conservative h_w is used, this guide recommends a load factor of 1.0 (use of a 1.2 factor combined with an already-conservative h_w can be overly conservative)

B.4 Comparison and discussion

The following represents a summary of work kindly undertaken by John Wood for SESOC. John undertook a study of commonly used codes, guidelines and examples, followed by preparation of a recommended consistent set of design factors. This information is summarised in the table below.

SESOC recommended				***Industry Examples***							
				MBIE Ex 2		***B1/VM2 and earlier B1/VM4 wall example***		***Caltrans: design note***		***NZTA Bridge Manual***	
Strength reduction factors		***G***	***G+E***	***G***	***G+E***	***G***	***G+E***	***G***	***G+E***	***G***	***G+E***
Sliding	Φ_{sl}	0.8	0.8	0.8	1.0	0.8	0.8	-	-	0.6 max	0.75 max
Bearing Drained	Φ_b	0.6	0.6	0.5	1.0	0.45	0.45	-	-		
Bearing Undrained			0.6								
Passive Cohesionless	Φ_p	0.75	0.9	0.5	1.0	0.45	0.45	-	-		
Passive Cohesive		0.65	0.8								
Load factors		***G***	***G+E***	***G***	***G+E***	***G***	***G+E***	***G***	***G+E***	***G***	***G+E***
Earth pressure	α_a	1.5	1.0	1.5	1.0	1.6	1.0	1.5	1.0	1.82	1.35
Earth pressure due to superimposed loading	α_q	1.5	1.0	1.5	1.0	1.6	1.0				
Gravity (earth): destabilising	α_{Gd}	1.5	1.0	1.2	1.0	-	-	-	-	1.35	1.0
Gravity (earth): stabilising	α_{Gs}	0.9	1.0	0.9	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Superimposed dead load: destabilising	α_{Dd}	1.0	1.0	1.2	1.0	-	-	1.75	0	2.25	0
Superimposed dead load: stabilising	α_{Ds}	0.9	0.9	0.9	0.9	-	-				
Superimposed live load: destabilising	α_{Ld}	1.0	0.4	0.4	0.3	-	-				
Superimposed live load: stabilising	α_{Ls}	0.9	0	0	0	-	-				

- Superimposed load would usually be split into two sections to consider both stabilising and destabilising effects. The section above the heel of the footing would usually be taken as stabilising and the section behind the virtual back plane as destabilising.
- For superimposed load behind the virtual back plane, gravity and earth pressure effects should be factored as per normal, but with a unity factor applied to the virtual back plane (stabilising) friction component. For SESOC recommended, superimposed dead and live load factors are applied in addition to α_q.
- In the MBIE example the LL was assumed to be 3 kPa for stabilising effects (0.9G, across the heel) and 5 kPa (1.2G + 0.4Q) for the horizontal earth pressure load. That is, the LL was effectively split into stabilising and destabilising components.
- In the MBIE example for the gravity load case the vertical components of the active thrust on the back plane were not factored. The horizontal thrust was factored by 1.5 to allow for uncertainty in the soil properties but it was argued that this should not affect the vertical component. Although inconsistent this approach is conservative and perhaps reasonable.
- In a particular analysis it would not be logical to apply different load factors to the same mass. For example, a soil mass on the toe should have the same load factor for both gravity and the EQ inertia loads. This is not normally an issue as the load factors for EQ are usually all taken as 1.0.
- The load and resistance factors used in the MBIE Example 2 give an overall FoS for sliding of 2.1 and about 3.3 for bearing.
- Following discussions between SESOC and NZGS in June 2025, α_D has been changed from 1.2 to 1.0, α_L has been changed from 1.5 to 1.0, and α_q has been changed from 1.3 to 1.5 for v0.7.2 of this Design Guide – refer NZGS/SESOC (2025) [10]

Appendix C – Pressure coefficients

While generally in this guide the wall is assumed to be vertical or near-vertical, the equations in this section allow for a sloping back of wall (angle β from vertical, assumed negative where the wall is leaning back towards the retained soil). For a constant thickness wall, $\beta = -\Omega_w$, unless a virtual back of wall is assumed, in which case effective values of the wall slope β and friction angle δ should be used in the equations below (see Section 7.4). The equations also allow for a sloping backfill (angle Ω_{bf}).

C.1 Active earth pressure coefficient

Where the retained soil properties are φ_{bf} (angle of shearing resistance), δ_{bf} (friction angle of soil with wall), and Ω_{bf} (slope of retained surcharge), the active pressure coefficient K_a is calculated using the Coulomb failure wedge approach (modified by Mayniel for wall friction, and by Muller-Breslau for a non-horizontal backfill and non-vertical soil-wall interface):

$$K_a = \frac{\cos^2(\varphi_{bf} - \beta)}{\cos^2\beta \cdot \cos(\delta_{bf} + \beta) \cdot \left[1 + \sqrt{\frac{\sin(\varphi_{bf} + \delta_{bf}) \cdot \sin(\varphi_{bf} - \Omega_{bf})}{\cos(\delta_{bf} + \beta) \cdot \cos(\Omega_{bf} - \beta)}}\right]^2}$$

The seismic active pressure coefficient K_{ae} is calculated based on Mononobe-Okabe theory (this gives a formula similar to that for K_a except incorporating $\theta = \tan^{-1} k_h$). This formula reduces to the same as for K_a when $k_h = 0$.

$$K_{ae} = \frac{\cos^2(\varphi_{bf} - \beta - \theta)}{\cos\theta \cdot \cos^2\beta \cdot \cos(\delta_{bf} + \beta + \theta) \cdot \left[1 + \sqrt{\frac{\sin(\varphi_{bf} + \delta_{bf}) \cdot \sin(\varphi_{bf} - \Omega_{bf} - \theta)}{\cos(\delta_{bf} + \beta + \theta) \cdot \cos(\Omega_{bf} - \beta)}}\right]^2}$$

Note that if k_h is such that $\varphi_{bf} - \Omega_{bf} - \theta < 0$, the above equation has no real solution. In this case, the slope may fail above the wall and shallow failures will tend to form at the surface of the backfill (refer also MBIE/NZGS (2017) [6]). It is unlikely that K_{ae} will increase significantly above its maximum value when this occurs, and so the following equation applies:

$$K_{ae} = \frac{\cos^2(\varphi_{bf} - \beta - \theta)}{\cos\theta \cdot \cos^2\beta \cdot \cos(\delta_{bf} + \beta + \theta)}$$

The active earth pressure (static and seismic) acts an angle of $\delta_{bf} + \beta$ from the horizontal.

C.2 Active pressure coefficient for superimposed loading

For a vertical wall and horizontal backfill, the active pressure coefficient for superimposed loading is the same as the active earth pressure coefficient. Otherwise (e.g. refer to Mylonakis et al 2007 [8]):

$$K_{aq} = K_a \cdot \frac{\cos\beta}{\cos(\beta - \Omega_{bf})}$$

And similarly, the seismic active pressure coefficient for superimposed loading is:

$$K_{aqe} = K_{ae} \cdot \frac{\cos\beta}{\cos(\beta - \Omega_{bf})}$$

The active pressure (static and seismic) due to superimposed loading acts at an angle of $\delta_{bf} + \beta$ from the horizontal (same as for the active earth pressure coefficient).

C.3 Passive earth pressure coefficient

This guide recommends that the passive pressure coefficients K_p and K_{pe} values be based on the following closed-form 'stress plasticity' formulation by Mylonakis et al 2007 [8]. This is preferred to:

- the Coulomb/Mononobe-Okabe formula, which overestimates K_p, particularly where $\delta > \varphi/2$
- the commonly-used Rankine approach (which is strictly applicable only for horizontal backfill, vertical walls and smooth soil–wall interface, i.e. $\Omega_{bf} = \beta = \delta_f = 0$)

Under static loading:

$$K_p = \frac{\cos(\beta - \Omega_{bf})\cos\Omega_{bf}}{\cos^2\beta\cos\delta_f} \cdot \left[\frac{1 + \sin\varphi_f \cos(\Delta_2 + \delta_f)}{1 - \sin\varphi_f \cos(\Delta_1 - \Omega_{bf})}\right] e^{2\vartheta \tan\varphi_f}$$

where

$$\sin\Delta_1 = \frac{\sin\Omega_{bf}}{\sin\varphi_f}$$

$$\sin\Delta_2 = \frac{\sin\delta_f}{\sin\varphi_f}$$

$$2\vartheta = \Delta_2 + (\Delta_1 + \delta_f) + \Omega_{bf} - 2\beta$$

And under seismic loading:

$$K_{pe} = \frac{\cos(\beta - \Omega_{bf})\cos(\Omega_{bf} - \theta)}{\cos\theta\cos^2\beta\cos\delta_f} \cdot \left[\frac{1 + \sin\varphi_f \cos(\Delta_2 + \delta_f)}{1 - \sin\varphi_f \cos\left(\Delta_1^* - (\Omega_{bf} - \theta)\right)}\right] e^{2\vartheta_E \tan\varphi_f}$$

where

$$\sin \Delta_1^* = \frac{\sin\left(\Omega_{bf} - \theta\right)}{\sin \varphi_f}$$

$$2\vartheta_E = \Delta_2 + \left(\Delta_1^* + \delta_f\right) + \Omega_{bf} - 2\beta + \theta$$

Note that the value of K_{pe} is less than K_p – the above formula for K_{pe} assumes that inertia acts away from the wall, minimising passive resistance. The formula reduces to the same as for K_p when $k_h = 0$.

Passive earth pressure (static and seismic) acts at an angle of $\delta_f - \beta$ from the horizontal.

Note that the Mylonakis formula used above is conservative (i.e. underestimates the pressure coefficient) for high φ_{bf} and high δ_{bf} / φ_{bf}. For a more accurate (but not closed-form) solution, please refer to Soubra 2011 [15].

Appendix D – Bearing strength factors

D.1 Introduction

For this design guide, the approach developed by Vesic (1975) [16] has been adopted. This is better referenced and more defensible than a B1/VM2-based implementation [7], and has already been adopted in Module 6 (MBIE/NZGS (2021) [6]), for example.

The ultimate foundation soil bearing strength is based on contributions from the soil shear strength, the vertical pressure at the foundation level, and the self-weight of the soil, and is given by the following equation, using the notation in MBIE (2025) [7], but with removal of ground inclination and base tilt factors):

$$q_u = c\lambda_{cs}\lambda_{cd}\lambda_{ci}N_c + q\lambda_{qs}\lambda_{qd}\lambda_{qi}N_q + \frac{1}{2}\gamma_{f,eff}B'\lambda_{\gamma s}\lambda_{\gamma d}\lambda_{\gamma i}N_\gamma$$

where:

c is 0 for cohesionless soil, c' for cohesive (static) and s_u for cohesive (seismic) (undrained analysis)

q is the vertical stress at depth D_f (underside of the foundation at toe)

= $\gamma_{bft} * D_f$ for seismic loading and for foundations above water table (total stress)

= $\gamma_{bft} * (D_f - h_w) + \gamma_{bft,sub} * h_w$ for water table below ground level in front of the wall, but above the bottom of the foundation

= $\gamma_{bft,sub} * D_f$ otherwise (vertical effective stress)

$\gamma_{f,eff}$ is the effective founding soil density at the base of the foundation, using a simple linear interpolation approach

= γ_f if any water table is at least D_{wz} beneath the foundation

= $\gamma_{bft,sub}$ if the water table is at foundation level or higher

= linear interpolation of γ_f and $\gamma_{bft,sub}$ otherwise

D_{wz} is the wedge zone depth, based on based on Bowles (1995) [3] p249

N_* are bearing strength factors defined below, as in Vesic (1975) [16]

λ_* are foundation shape (λ_{*s}), depth (λ_{*d}) and load inclination (λ_{*i}) factors related to cohesion (λ_{c*}), loading (λ_{q*}), and soil density ($\lambda_{\gamma*}$), all as in Vesic (1975) [16]

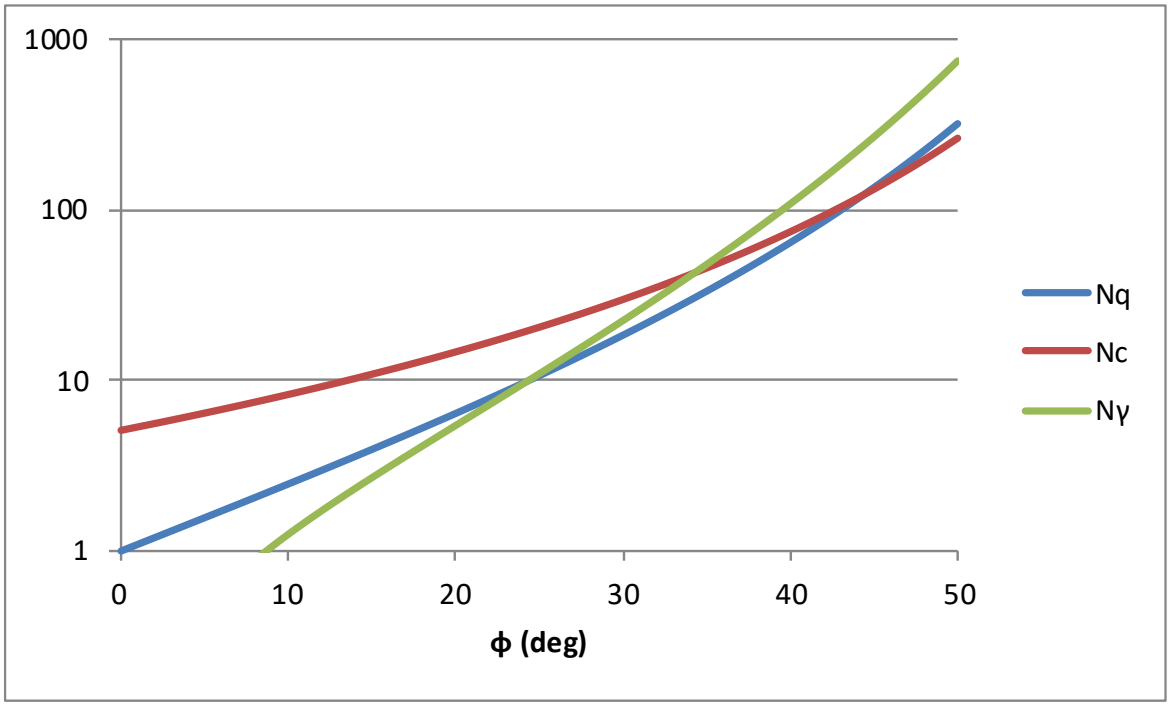
D.2 Bearing strength factors

Calculate N_q, N_c and N_γ (refer Vesic p127, 128):

$$N_q = e^{\pi \tan \varphi} \cdot \tan^2(45 + \varphi/2), \varphi \; in \; degrees$$

$$N_c = \frac{(N_q - 1)}{\tan \varphi} \; (= 5.14 \; for \; \varphi = 0)$$

$$N_\gamma = 2(N_q + 1) \cdot \tan \varphi \; \; (= 0 \; for \; \varphi = 0)$$



D.3 λ factors

Note that unfactored horizontal and vertical design loads (H_{uf} and V_{uf}) are used for the calculation of the load inclination factors.

Foundation shape factors

Refer Vesic Table 3.2, p129 for a rectangular base:

$$\lambda_{cs} = 1 + \left(\frac{B'}{L_f}\right)\left(\frac{N_q}{N_c}\right)$$

$$\lambda_{qs} = 1 + \left(\frac{B'}{L_f}\right)\tan \varphi$$

$$\lambda_{\gamma s} = 1 - 0.4\left(\frac{B'}{L_f}\right)$$

Footing depth factors

In line with Vesic's advice for the design of shallow foundations (p133), footing depth factors are all set to 1.

$$\lambda_{cd} = \lambda_{qd} = \lambda_{\gamma d} = 1$$

Load inclination factors

Vesic recommends (p130, 131) that the expressions derived from plain-strain analyses of a very long foundation be modified based on the foundation breadth to depth ratio as follows:

$$m = \left(2 + \frac{B}{L_f}\right)\left(1 + \frac{B}{L_f}\right)$$

$$if\ \varphi = 0\ then\ \lambda_{qi} = 1, \qquad \lambda_{\gamma i} = 1, \quad \lambda_{ci} = 1 - \frac{mH_{uf}}{A' \cdot \mathrm{c} \cdot N_c}$$

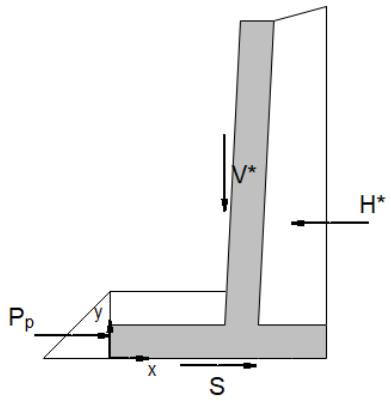
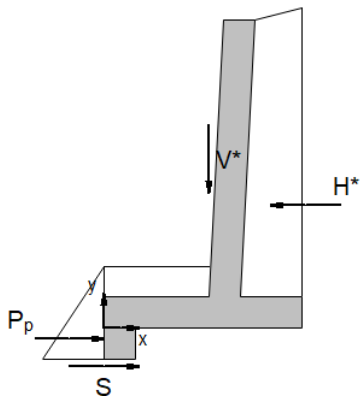
$$else\ \lambda_{qi} = \left[1 - \frac{H_{uf}}{V_{uf} + A' \cdot \mathrm{c} \cdot \cot\varphi'}\right]^{\mathrm{m}}, \lambda_{\gamma i} = \left[1 - \frac{H_{uf}}{V_{uf} + A' \cdot \mathrm{c} \cdot \cot\varphi'}\right]^{\mathrm{m+1}},$$

$$\lambda_{ci} = \lambda_{qi} - \frac{1 - \lambda_{qi}}{N_c \cdot \tan\varphi'}$$

Appendix E – Models for sliding resistance

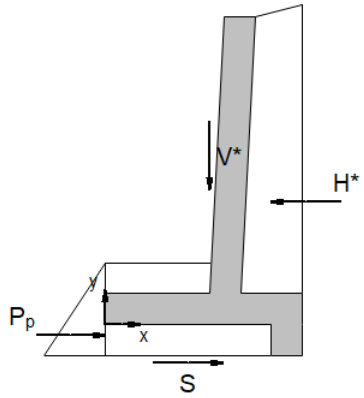
Refer to Section 8.6, and Figure 3-1 for geometrical dimensions. Components of resistance are:

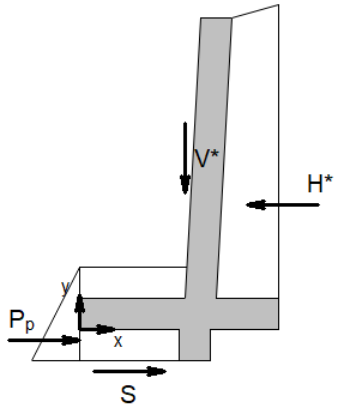
- Passive resistance from cohesionless soil (P_p)
- Sliding resistance from friction (S_f)
- Sliding resistance from adhesion (S_a)

Cohesionless analysis	***Drained cohesive analysis***	***Undrained analysis***
No key		
		
$P_p = 0.5 \cdot K_p \cdot \gamma_{bft} \cdot {D_f}^2$		
$S_f = R_{sl} \cdot tan\ \delta'_f$		$S_f = 0$
$S_a = 0$	$S_a = r_A \cdot c' \cdot B'_{sl}$	$S_a = r_A \cdot s_u \cdot B'_{sl}$
Key at front		
		
$P_p = 0.5 \cdot K_p \cdot \gamma_{bft} \cdot \left(D_f + d_k\right)^2$		
$S_f = R_{sl} \cdot tan\ \delta'_f$		$S_f = 0$
$S_a = 0$	$S_a = r_A \cdot c' \cdot B'_{sl}$	$S_a = r_A \cdot s_u \cdot B'_{sl}$

Where the key is not at the front of the footing, sliding resistance varies depending on its location. In general, resistance is made up of a combination of relatively high resistance in front of the key (multiplied by ratio r_s) and a lower resistance behind the key (multiplied by $(1\text{-}r_s)$).

$$Ratio\ r_s = MIN[\frac{B_k}{B_{sl}^{'}}, 1]$$

<table>
<tr><th colspan="3">Key at back</th></tr>
<tr><td colspan="3">
</td></tr>
<tr><td colspan="3">$P_p = 0.5 \cdot K_p \cdot \gamma_{bft} \cdot \left(D_f + d_k\right)^2$</td></tr>
<tr><td colspan="2">$S_f = R_{sl} \cdot [r_s \cdot tan\ \varphi_f' + (1 - r_s) \cdot tan\ \delta_f']$</td><td>$S_f = 0$</td></tr>
<tr><td>$S_a = 0$</td><td>$S_a = B'_{sl} \cdot c'[r_s + (1 - r_s) \cdot r_A]$</td><td>$S_a = B'_{sl} \cdot s_u[r_s + (1 - r_s) \cdot r_A]$</td></tr>
</table>

<table>
<tr><th colspan="3">Key not at front or back</th></tr>
<tr><td colspan="3">
</td></tr>
<tr><td colspan="3">$P_p = 0.5 \cdot K_p \cdot \gamma_{bft} \cdot \left(D_f + d_k\right)^2$</td></tr>
<tr><td colspan="2">$S_f = R_{sl} \cdot [r_s \cdot tan\ \varphi_f' + (1 - r_s) \cdot tan\ \delta_f']$</td><td>$S_f = 0$</td></tr>
<tr><td>$S_a = 0$</td><td>$S_a = B'_{sl} \cdot c'[r_s + (1 - r_s) \cdot r_A]$</td><td>$S_a = B'_{sl} \cdot s_u[r_s + (1 - r_s) \cdot r_A]$</td></tr>
</table>

Appendix F – Other matters

F.1 Assumptions around homogeneous cohesionless retained soil

This design guide is predicated on cohesionless backfill, and assumption of cohesionless behaviour of the retained soil block (refer assumptions and limitations in Section 2 above). This assumption is made for simplicity, irrespective of whether the retained soil is cohesionless or cohesive, or differing φ values.

It is not the intent at this time to delve further into this aspect, i.e. narrow cohesionless backfill and larger cohesive retained mass. The designer needs to validate the approach for their design, and could include calculation of the angle and the failure forces on the wall for both the granular backfill and a clay retained soil. Judgement can then be used to determine the appropriate design loading.

Refer to Shukla (2015) [14], which gives a procedure for carrying out the M-O analysis in a c-φ soil.

Appendix G – Worked design example

Worked design example to be included once SESOC Retaining Wall software (RetWall) containing concrete cantilever retaining walls module is finalised (expected November 2026).


SESOC