DESIGN GUIDE



SESOC

Timber Pole Retaining Walls

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July 2025

Simplified Structural Design Guide
Timber Pole Retaining Walls

July 2025

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Disclaimer

This design guide has been prepared to assist the competent (structural) engineer in the design of timber pole 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 3.

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    - for its contribution via the first author's (voluntary) time spent on this initiative
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1. Introduction

1.1 Design guide

New Zealand does not have a design standard for cantilever timber pole retaining walls, nor is there any generally agreed industry standard. MBIE/NZGS [5] 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 timber pole 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 concrete retaining walls [2].

1.2 Scope

This document is intended to be a stand-alone guide for the geotechnical aspects of standard timber pole 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 3 below) is intended to cover:

- Variable pole size and spacing
- Cohesive or cohesionless founding conditions
- Static long-term loading
- Seismic loading, modelled as static loading with appropriate loading and short-term load factors
- Water table
- Retained slope
- Vertical or sloping poles
- 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 cantilever timber pole design, but rather to document the core principles for a majority of simple cantilever retaining walls.

Also, while the guide is fundamentally oriented towards the 'technical design' aspects, i.e. structural mechanics pressure blocks etc, we have sought to include

practical/construction aspects where appropriate. Nevertheless, this guide must not be taken as presenting all aspects of cantilever timber pole design and construction. There are multiple construction aspects etc which are not covered by this guide, e.g. practical construction sequencing, provision for drainage, etc.

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' cantilever 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. 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 capacity, i.e.

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

or

$$\sum \alpha_i \cdot F_i \leq \varphi \cdot R$$

where

S^* = factored design action
R^* = factored (dependable) resistance
α_i = load factor
F_i = applied load
φ = capacity reduction factor
R = calculated ideal resistance or strength

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. This includes, for example, calculation of:

- depth to point of foundation rotation
- depth to maximum below-ground moment on the pole

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.

Note that while the approach used is limited to static analysis only, both static and seismic loads are covered (seismic loads are treated as 'pseudo-static' earth pressures on the wall).

While it is expected that design results using this guide will be similar to those presented in the example by MBIE/NZGS [5], this document is intended to provide guidance across a range of geotechnical, geometrical, and soils loading parameters.

3. Assumptions and limitations

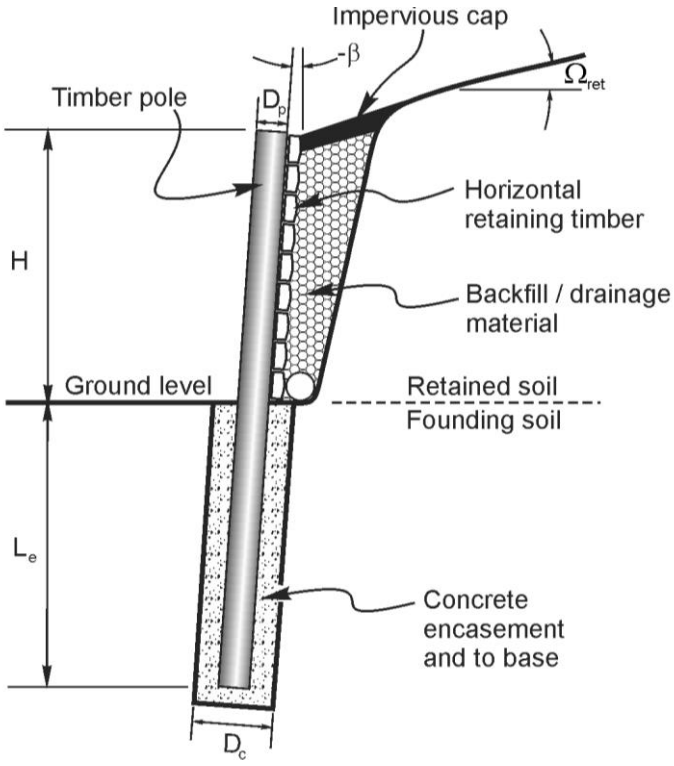


Figure 3-1: Generic wall layout

The following are the assumptions that underpin this design guide:

Fundamental assumptions

- 'Good ground' as defined by MBIE [6]
- Homogenous profile for founding soil (below ground level); homogeneous profile for retained soil
- Backfill soil is cohesionless (drained); assumed cohesionless behaviour of all retained soil – refer Appendix E.1 for further detail
- The structure is a cantilevered retaining wall, with longitudinal (horizontal) retaining timber spanning between and supported by spaced timber poles
- No anchors or any 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 and adjacent soil is a base assumption for the design, both on initial loading and potentially seasonally. The client should be made aware that the wall will be displaced once the conditions impose an at-rest earth

pressure state, and in time an originally vertical wall may look tilted forward and potentially out-of-vertical
- Normal 'active' soil pressures; where the retaining structure is on a boundary, and/or reducing displacement is required, use of a higher loading pressure (e.g. 'at-rest") may be necessary
- Deep-seated rotational failure of the wall does not occur
- 'Plane strain' conditions, with the wall and foundation assumed to be long and continuous
- Poles are founded into undisturbed ground, and that (for concrete-encased poles) sides of bored hole have not slumped prior to concreting
- If composite action of concrete-encased poles is to be utilised (not normally recommended, refer Section 9.1 and Appendix E.3), full adhesion is achieved between the concrete and the timber such that the ratio between the design flexural strength of the composite pole and that of the bare pole is a constant (Cc)

Geotechnical assumptions

- Soil and pore water are homogenous
- Soil does not lose strength, e.g. liquefy in the event of the design earthquake
- Triangular/ trapezoidal pressure distribution on the back face of the wall. Arching action and local increases due to compaction are ignored
- Under seismic/short-term loading conditions with cohesive founding soil, an equivalent fluid unit weight is applied as an active pressure even when water is not expected to develop behind the wall (refer Section 5.3 below)
- The level of the water table, if present, is assumed to be at a height h_w above ground level between 0 and H/3 on the retained side of the wall, and at ground level at the front
- There is no water table present under seismic conditions for cohesionless founding soil
- Cohesive soils: The first Zt (e.g. 0.25 to 0.5m) distance of the pole below ground level provides no lateral resistance from cohesion (i.e. is unsupported in cohesive soil) (refer also Section 5.2 below)

Geometrical limitations

- Vertical or near-vertical wall (backslope less than 14°)
- Retained height (H) not exceeding 3.0m
- Timber pole diameter not exceeding 450mm
- Spacing of poles not exceeding 6x the embedded pole diameter
- Retained soil slope is limited to half the effective friction angle of backfill material

- 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 8 times the embedded pole diameter (refer also Appendix E.2 and Wood 2021 [14])
    - specific geotechnical advice should be sought where these limits may not be achievable

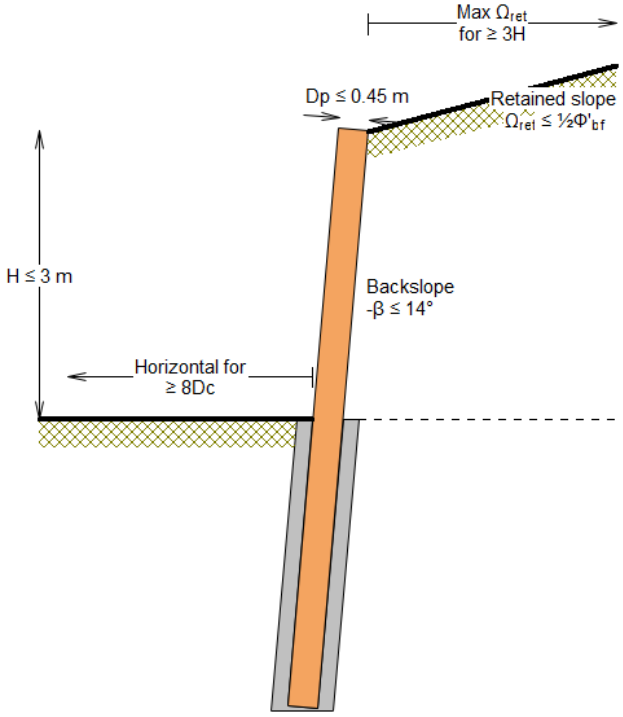


Figure 3-2: Geometrical limitations

Construction comments

- Sufficient space for concrete flow surrounding pole, including allowance for large end diameter of tapered pole
- Temporary stability of retained slope as lagging and drainage aggregate is installed
- Surface water to be diverted if appropriate

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 effective strength due to earthworks
- Groundwater

- Liquefaction potential
- Undesirable levels of wall deflection, both in the static and seismic load case scenarios
- Undesirable levels of wider ground movement, both in the static and seismic load case scenarios
- The impact of any ground movements on neighbouring properties, buildings, roads or buried infrastructure
- Where peat or peaty soils occur below ground level
- Soft or cohesionless soils which may require temporary support of pile holes
- 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 advice as appropriate.

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 usually feature Structural Engineers practising outside their field of expertise and/or level of geotechnical competence.

4. Failure modes

Possible modes of failure for cantilever pole retaining walls are illustrated in Figure 4-1, taken from the MBIE/NZGS 2021 Worked Example 1 [5]. A complete design must address each of these modes of failure where appropriate.

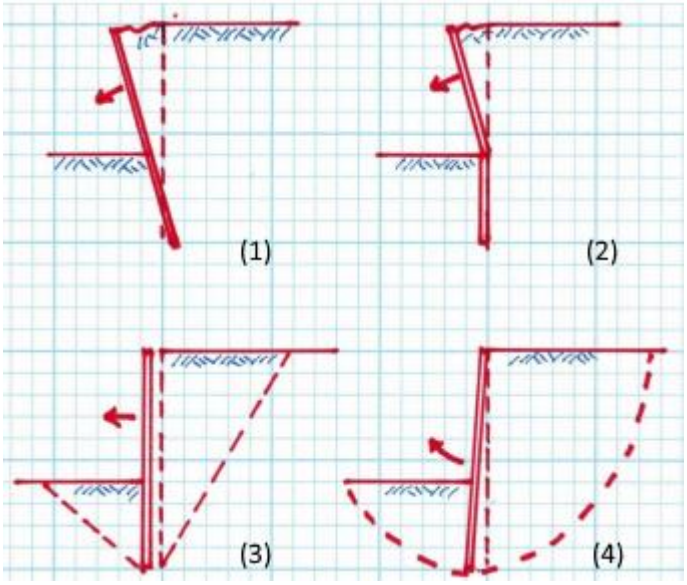


Figure 4-1: Possible modes of failure

1. Foundation failure: The embedded pole foundations rotate through the soil (usually accompanied by some outward sliding displacement)
2. Pole structural failure: The poles fail in bending. The most likely location is at the ground surface where the poles are embedded in substantial concrete foundations; otherwise, it may be below the ground surface at the point of maximum below-ground moment
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 wall and active failure of the soil behind the wall. This is generally considered to be very rare, and unlikely to govern design for typical cases
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 at the toe of the wall

This design guide is focussed almost exclusively on Modes 1 and 2, i.e. 'local' geotechnical aspects plus structure. As is noted above, Mode 3 is very unlikely within the geometrical and geotechnical limitations of this guide (particularly note the assumption that the ground is horizontal in front of the wall). Mode 4 should only occur if there is a weak layer of soil in the vicinity of, or below, the bottom of the retaining structure.

Should the reader have any concerns about these – or any other aspects – advice must be sought from a Geotechnical Engineer.

5. Soil Mechanics

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

5.1 Soil type/loading conditions

For analysis, foundation soil is assumed to be either cohesive or cohesionless (if in doubt, the analysis should be carried out for both soil types). If cohesive, no consolidation occurs under short-term 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').

If cohesionless, consolidation occurs very quickly, so drained strength is used for all load cases.

Founding Soil Type	Loading	Analysis Type	Strength Parameter
Cohesive	Static, long-term	Drained	φ', c'
Cohesive	Seismic, short-term	Undrained	s_u
Cohesionless	Static, long-term	Drained	φ'
Cohesionless	Seismic, short-term	Drained	φ'

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

	Analysis Type for:	
Loading	**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:

- if a load is 'short-term' or 'long-term', and therefore whether the analysis type should be drained or undrained
- how cohesive the founding soil is (c' may be difficult to assess)

5.2 Lateral resistance from cohesion close to ground level

The lateral resistance of cohesive soil against the top part of the pole is assumed to be ineffective for a depth Zt below ground level, where Zt is often taken as 0.25 to 0.5m. Note that Broms [3](a) uses 1.5Dc for Zt, some analysis software assumes zero, 0.25m is used by Pender [11] and 0.5m is used by MBIE [5] in its example calculations.

Wood [14] notes that assuming the ineffective depth is 0.5Dc appears acceptable and gives good agreement with the average of test load capacities. A top ineffective depth of

0.5Dc would correspond to about 300 mm or about 0.1H for typical pole walls that have H/Dc ratios of about 5.

In a related matter, e.g. for very expansive soils, crack depth may exceed this value. Summer crack depths may develop up to a depth of 0.75 to 1.0m, especially in the presence of trees (which soak up additional moisture). In these cases, Zt may need to be increased, with geotechnical advice sought as appropriate.

5.3 Cohesive soil crack assumptions

This section covers the aspect of active soil pressures below ground level for cohesive soil. Of note are the different facets affecting this, not limited to but potentially including:

- tension and/or shrinkage cracks during construction
- outward displacement of the wall in order to generate the passive (restoring) pressures, etc

Consideration of such is complex, "in view of the uncertainties regarding tension cracks" (CIRIA C580 sections 4.1.6 and 5.5.1 [4]) along with differing views from various parties.

In one view, an unconservative analysis would ignore any active pressures acting below ground level (due to the formation of tension cracks). Conversely, an overly conservative analysis would assume that the backfill soil extends down to the point of rotation.

This document proposes that the active pressure from cohesive soil below ground is the sum of:

- a 'minimum active pressure' of 5z kPa, applied across the full effective pressure width, EPW (refer Section 5.5 below). Refer also CIRIA C580 sections 4.1.6 and 5.5.1 [4]
- water pressure (if present), acting on the embedded pole width only

5.4 Strength parameters

Soil parameters: φ', c'

This design guide is predicated on cohesionless backfill, and an assumption of cohesionless behaviour of the retained soil block. Therefore, only the density plus the effective angle of shearing resistance, φ', are used for calculating backfill loads (refer Appendix E.1).

Whether cohesive or cohesionless, the founding soil relies 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 in the presence of water. This design guide includes c' in the calculation of passive pressure resistance, noting that it the responsibility of the designer to determine a reliable c' design value.

Soil parameters: s_u

Under seismic loading conditions, a cohesive founding soil is assumed to behave in an undrained manner (relying on s_u).

5.5 Pole spacing effects

This guide takes a simplified approach to allowing for different pole spacings. Calculations are done on the basis of a conventional continuous embedded wall, but below-ground pressure forces are adjusted based on a (reduced) effective pressure width for widely-spaced poles. Note that this approach is only deemed applicable for a maximum centre-to-centre pole spacing of 6x the embedded pole diameter. At greater spacing, isolated pole analysis (e.g. using Brom's equations) should be considered, and is not documented in this guide.

For poles spaced at 3.5x embedment diameter or less, the below-ground soil pressure bulbs (driving and resisting) are assumed to provide continuous pressure along the wall (see diagram below). In this case, the effective pressure width can be taken as the pole spacing dimension. When poles are spaced at greater than $3.5D_c$, the soil no longer provides that pressure over the full distance between poles. In this situation, the effective pressure width (EPW) is reduced linearly based on the pole spacing, i.e. EPW = MIN($3.5D_c$, Sp).

It is assumed that the same effective pressure width applies to all below-ground active and passive soil pressures. Additionally, it is assumed that below-ground water pressures only act on the embedded pole diameter.

To be clear, the above reduction only applies to below-ground soil pressure forces; when calculating above-ground forces, the full tributary areas and loads must be included irrespective of pole spacing.

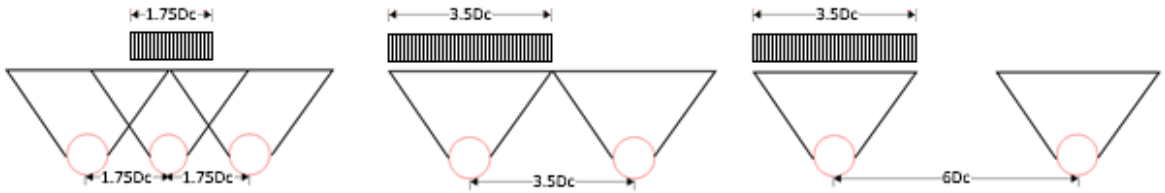


Figure 5-1: Effective pressure widths for below-ground soil pressures

5.6 Water table

The equations in this guide assume that, if present at all, the water table is at ground level in front of the wall, but at a height h_w (between 0 and H) behind the wall. While any water table level above ground level is often treated as short term loading, this may not always be the case.

Due to a range of soil types, construction types, and variance of opinion, it is not possible to provide specific guidance about the water table level. The presence of water, even at ground level, can have a major impact on the design of the wall in terms of required embedment length, and hence overall stability and safety. While it can be argued that a water level behind a cantilever timber wall with timber walers and cohesionless backfill is unlikely, particularly under seismic loading, nevertheless a sensible assessment of risk must be undertaken by the designer. This is a significant decision – perhaps the single most important!

The designer should consider various contributory aspects, including:

- Construction type – closely-fitting or spaced horizontal retaining timber
- 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

The designer **must** take a conservative view, and in the absence of specific information to the contrary, may wish to allow for load cases where:

- the water table behind the wall is up to 1/3 the height of the wall (perhaps as a short-term load with reduced load factors)
- the water table is at ground level in front of the wall

In summary, do NOT ignore the effects of a water table without having carried out an appropriate risk assessment and considering relevant factors. The reader is reminded that a significant portion of PI claims against structural engineers relate to inadequate geotechnical designs. If in doubt, check it out – with a geotech engineer!!

Treatment of water pressure – drained vs undrained

In a drained situation, the diagram in Fig 7-1 shows water pressures acting from h_w down to the full embedment depth. The net drained water pressure below ground is assumed conservatively to be rectangular down to the point of foundation rotation, and ignored below this point.

For the undrained situation, only water pressure from h_w above ground level is directly included in the equations, which may be contrary to expectations. The basis for this is that below-ground water table effects are already included in the undrained founding soil properties. Note that the undrained case (short term loading) applies to a saturated clay and the effective unit weight of the soil is not changed significantly by the location of the water table.

Water table design limitations

As the reader will appreciate, the purpose of this design Guide – and the associated design tool – is to provide a rational, 'formularised' basis according to recognised geotechnical principles. Of course, this must also align with established independent software – typically rather more sophisticated commercial software, and generally employing rather more complex FEA analyses.

In terms of alignment, there is one aspect where our design model is potentially unconservative – of design in the presence of elevated water table levels. Design with the water table at ground level (GL) generally aligns with other external software. However, for water above GL the model above appears potentially unconservative – notwithstanding the considerable step change in embedment depth from 'no water' to water at GL, especially for cohesionless soils.

Accordingly, we have made the decision for now to limit water table design levels to H/3. In addition, pending further verification, seismic design with a water table is limited to cohesive founding soil.

In general, it is expected that timber pole retaining wall design in the presence of elevated water table levels is likely the exception. Where the H/3 limitation may impact a particular design, the designer is advised to use recognised, independent, special purpose GeoTech software, e.g. Wallap or LSGeo. Further, initial indications are that the presence of elevated water table levels may give rise to much higher wall displacement values – an aspect that should be specifically addressed, and an additional reason for independent rigorous analysis.

6. Design parameters and factors

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

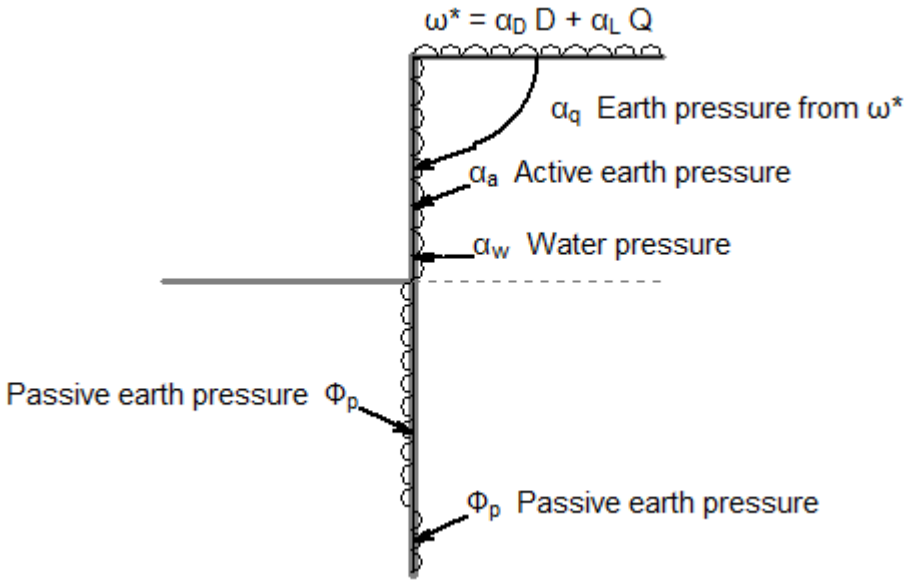


Figure 6-1: Design factors used in pole wall analysis

6.1 Capacity reduction factors

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

- Φ_p passive resistance reduction factor
- Φ_t timber material strength reduction factor

Typical values are given in Appendix B.2 Recommended capacity reduction factors.

6.2 Load factors

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

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

Relevant load factors for cantilever timber pole walls are:

- α_a active soil pressure
- α_D, α_L superimposed dead and live load factors
- α_q soil pressure on wall due to superimposed loading (this factor applies in addition to the superimposed dead and live load factors)

- α_w ground water pressure

Typical values are given in Appendix B.3 Recommended 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 a minimum capacity to demand ratio of 2.0, assuming all load and material factors are unity.

6.3 Seismic design basis

Seismic design is based upon active seismic loading applied to the complete retained plus below-ground structure, right down to the rotation point. This loading is applied to the full pole spacing (Sp) above ground and full effective pressure width (EPW) below ground.

The peak ground acceleration (PGA) should be determined using NZS 1170.5, and then modified to determine the design horizontal earthquake coefficient, k_h. k_h is sometimes taken as 0.5 x PGA to allow for wave scattering, lack of coherency in the backfill accelerations and acceptable (possibly permanent) outward movement. A value of 0.5 to 0.7 x PGA is normally more appropriate for small walls where some of the reduction effects might not be as significant.

MBIE/NZGS [5] also provides some guidance on calculating k_h.

While the methodology in this design guide does not currently incorporate displacement calculations, some of the (independent software) validation work carried out indicates there may be significant permanent displacements at higher seismic loads and/or where the embedment depth approaches that for minimum equilibrium.

For this reason, the methodology is as follow:

- seismic limitations of $k_h \leq 0.3$ and $K_{aeh} \leq 0.65$ are imposed
- embedment depth: while additional depth to allow for mitigation of (permanent, residual) seismic displacement was considered, we have not formally implemented this

Further notes to these aspects:

- the background to the $k_h \leq 0.3$ relates to several aspects, but primarily:
    - we had a number of convergence issues with the verification software at higher seismicities
    - there is a lack of industry clarity (i.e. multiple opinions) on the applied seismic loading below ground
    - permanent residual displacement risk appears to increase exponentially with higher k_h
    - please note this limit relates to 'effective' seismic actions, not the PGA

- the purpose of the K_{aeh} limitation is to account for retained slopes, an aspect not included in the simple $k_h \leq 0.3$
- in terms of embedment depth under seismic conditions, some initial analyses indicate that a 25% increase in embedment depth facilitates a rather higher decrease in likely permanent seismic displacement of approximately 50%. On this basis, especially:
    - in higher seismic risk regions and/or
    - where permanent residual displacement in the event of a major earthquake should be minimised

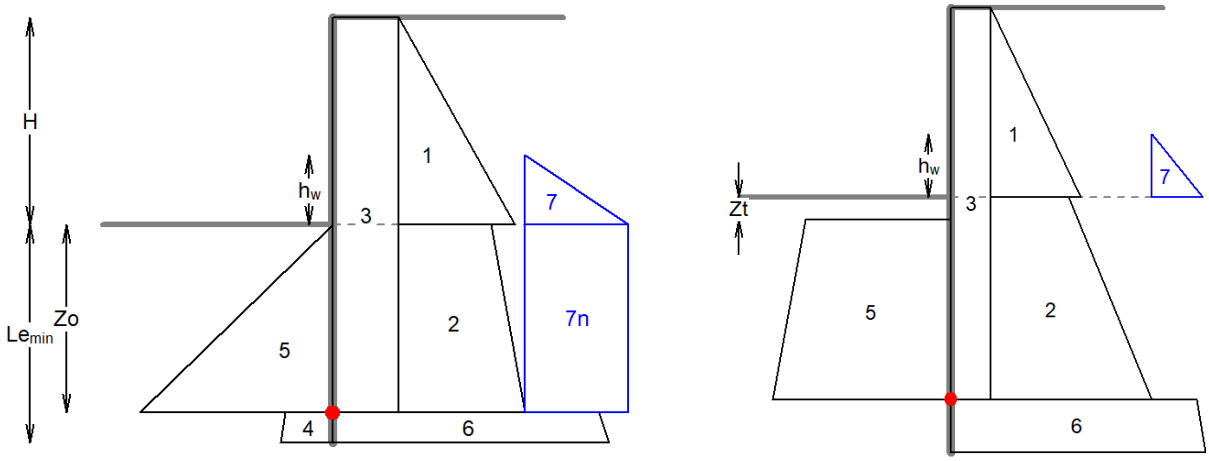
  the designer should consider increasing the embedment depth below the design minimum in order to mitigate such effects

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 Pressures and forces on wall

This design guide uses a simplified model of soil and other pressures on the retaining wall. These are graphically described in the following diagrams. Soil blocks 1-4 contribute to overturning moment in the wall, and soil blocks 5 and 6 resist those moments. Figure 7-1 (left) shows the pressures arising in a drained (cohesionless) situation, and Figure 7-1 (right) shows the load and resistance in an undrained situation.



Drained (for c'=0) **Undrained**

Figure 7-1: Load and resistance diagrams

Pressure block	***Description***
1	Active pressure on back of wall from retained soil above ground level
2	Active pressure on back of wall from founding soil from ground level down to rotation point
3	Active pressure on back of wall/pole due to surcharge loading
4	(Small) active pressure in front of pole below rotation point
5	Passive pressure of founding soil in front of pole (above rotation point) providing resistance to overturning
6	Passive pressure of founding soil on back of wall (below rotation point) providing resistance to overturning

7	Water pressure on back of wall
7n	Net pressure from below-ground water at front and back of pole

7.2 Calculation of required embedment length

Based on the defined pressure blocks, this guide's approach is to solve the simultaneous horizontal pressure force equilibrium and overturning moment equilibrium equations. An iterative process can be used to find Zo, the depth to the point of rotation, and Le_{min}, the required embedment length. A similar process can then be used to find Zbm, the depth to the maximum moment in the pole at some depth below ground level.

A detailed calculation process is laid out in Section 8 below.

7.3 Superimposed loading

Note that ω_D and ω_L have different load factors applied based on the relevant superimposed gravity load factors:

$$\omega^* = \omega_D \cdot \alpha_D + \omega_L \cdot \alpha_L \qquad \text{(static)}$$

$$\omega_e^* = \omega_D \cdot \alpha_{De} + \omega_L \cdot \alpha_{Le} \qquad \text{(seismic)}$$

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.

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 [7]. (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 Pressure at depth

Active pressure – static

The active pressure at depth d_{bf} below the top of the wall, applied at an angle corresponding to the friction angle of the soil plus the slope of the back of the wall, is:

$$p_a = K_a \cdot \gamma_{bf} \cdot d_{bf}$$

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

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

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

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

where

$$K_{ah} = K_a \cdot \cos(\delta'_{bf} + \beta) \text{ and } K_{aqh} = K_{aq} \cdot \cos(\delta'_{bf} + \beta)$$

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.

Active pressure – seismic

Similarly, the horizontal components of the active pressures under seismic loading (following Mononobe-Okabe) are:

$$p_{bf,aeh} = K_{aeh} \cdot \gamma_{bf} \cdot d_{bf}$$

$$p_{sur,aeh} = K_{aqeh} \cdot \omega_e^*$$

where

$$K_{aeh} = K_{ae} \cdot \cos(\delta'_{bf} + \beta), \text{ and } K_{aqeh} = K_{aqe} \cdot \cos(\delta'_{bf} + \beta)$$

In this guide, seismic loads are deemed to act at the same height and same angle as the equivalent static horizontal pressure loads.

Note that for a stiff wall, however, the seismic pressure component is greater than that given by the Mononobe-Okabe equations as well as acting higher than H/3. Pole walls with no anchors or other restraining structures will generally have sufficient flexibility for the Mononobe-Okabe force acting at H/3 assumption to be an acceptable approximation.

Passive pressure

Passive pressure from the soil below ground level provides the sole resistance to overturning. The passive pressure profiles are shown as blocks 5 and 6 in the diagrams below. Note that:

- passive resistance is improved when an analysis can take advantage of the founding soil's effective cohesion (c') under static loading, and undrained shear strength (s_u) under seismic loading
- the top Zt of the cohesive resistance is ignored (see Section 5.2)
- the depth component of passive pressure is the only one available for cohesionless founding soil
- this depth component is proportional to the weight of the soil, which is greatly reduced if there is a water table ($\gamma_{f\text{-}w} = \gamma_f - \gamma_w$)

The horizontal components of passive pressure at depth z below ground level under static and seismic loading are calculated as follows:

$$p_z = K_{ph} \cdot \gamma_{f-w} \cdot z \qquad \text{(static, cohesionless)}$$

$$p_{zc} = K_{ph} \cdot \gamma_{f-w} \cdot z + 2c'\sqrt{K_{ph}} \qquad \text{(static, cohesive)}$$

$$p_{ze} = K_{peh} \cdot \gamma_{f-w} \cdot z \qquad \text{(seismic, drained)}$$

$$p_{zue} = \gamma_{f-w} \cdot z + 2s_u \qquad \text{(seismic, undrained)}$$

where

$$K_{ph} = K_p \cdot \cos(\delta'_f - \beta),\ K_{peh} = K_{pe} \cdot \cos(\delta'_f - \beta)$$

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

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.

7.6 Pressure on wall – expanded pressure blocks

In the following, the horizontal pressures and forces are labelled with subscripts corresponding to the pressure blocks shown in Figure 7-1: Load and resistance diagrams. For clarity, the pressure blocks are presented as conventional limit state pressures, e.g. Pender [11].

Zo is the distance below ground level to the rotation point, and Le_{min} is the required length of embedment for force and moment equilibrium.

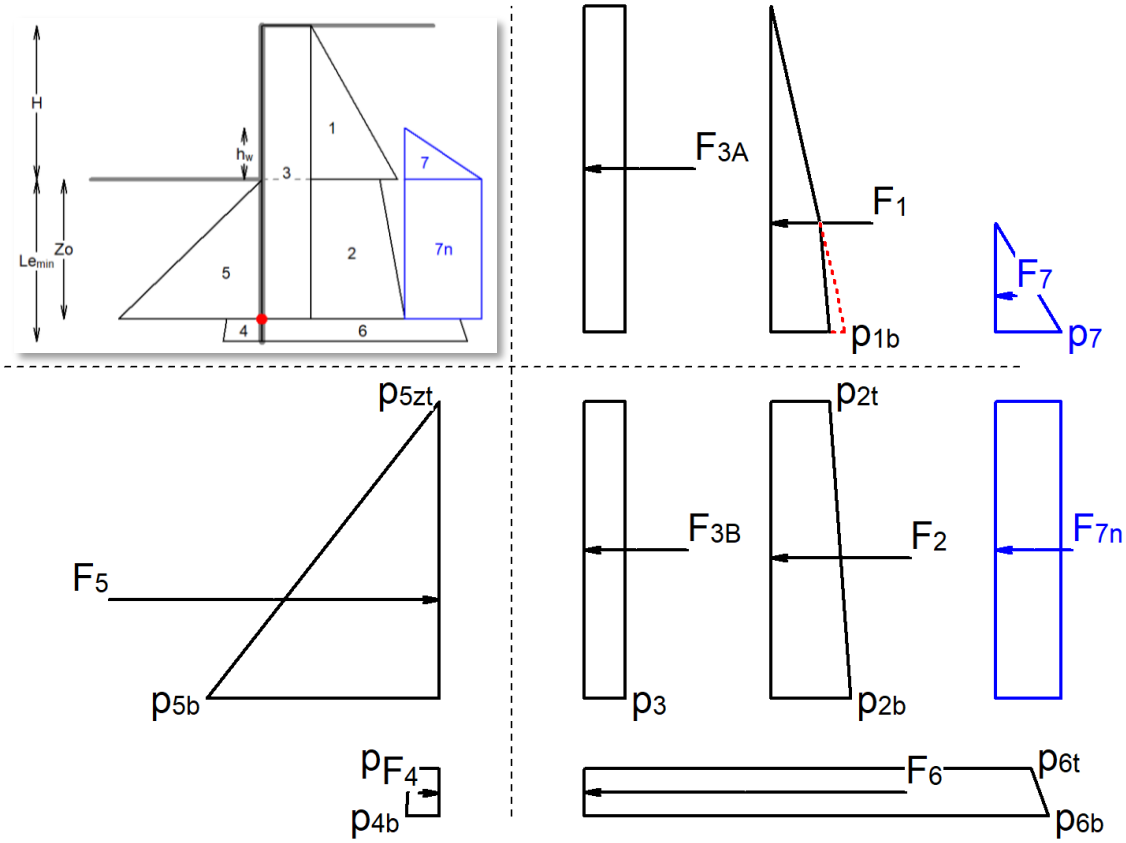


Figure 7-2: Expanded force diagram for drained (cohesionless) founding soil (c'=0)

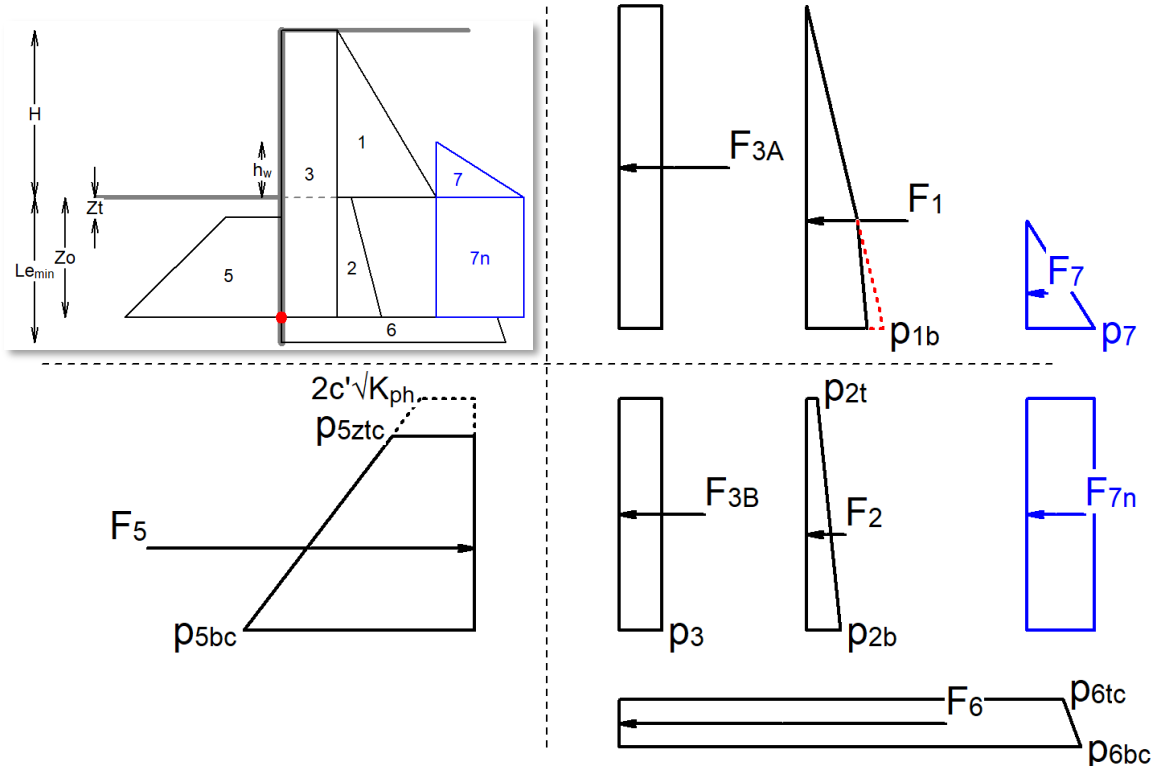


Figure 7-3: Expanded force diagram for drained (cohesive) founding soil (c' > 0)

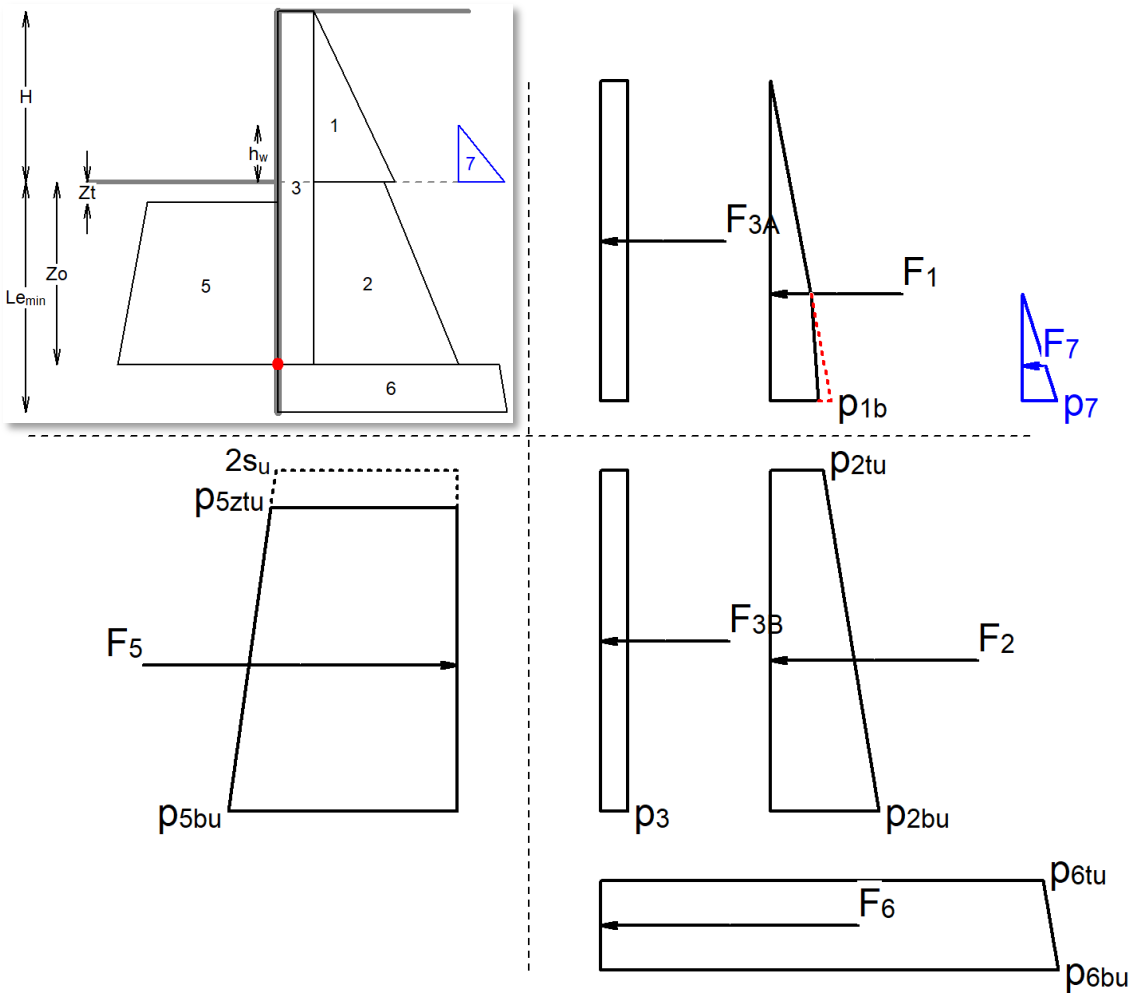


Figure 7-4: Expanded force diagram for undrained founding soil

Note that, for drained:

- the effects of the water on the retained side and that on the other side are combined below ground level (pressure block 7n), and conservatively modelled as a rectangle down to the point of rotation (if the water table is at ground level, there is no net water pressure on the pole)

For undrained:

- refer Section 5.3 above for reference on the minimum active pressure used in pressure block 2
- pressure block 3 is assumed to be uniform, i.e. rectangular, although strictly for the undrained analysis it would be more correct to set $K_{ah,f}$ = 1.0 below ground level but then reduce the pressure due to cracking in the cohesive soil
- no adjustment is made for water behind the wall when calculating blocks 2 and 6

7.7 Implementation and alignment

Notwithstanding the long-standing and broadly accepted Geotech principles upon which this above content is based, it must nevertheless be recognised that the structural mechanics model is just that – a model, an approximation, of the real-world behaviour. Further, the model must still align with best practice and generally accepted results by other tools or methods.

For this reason, since the outset, the authors have been very much aware of the need for validation and verification. Validation, of the methodology – including nuances, and verification, or (quantifiable) alignment with current best practice.

To this end, a comprehensive series of analysis models have been run, for both drained and undrained conditions, across multiple parameters, including surcharge loading, seismic loading, water, retained slope angle, strong/weak soils, etc. These analyses have been independently undertaken using multiple software applications and approaches.

While not documented in this guide, suffice to say that while quite time-consuming, they have also challenged a number of facets of this guide, necessitating refinement and improvement of the methodology.

In one particular aspect, we understand that a 'limit state stress block' is, fundamentally, an engineering convenience only. In terms of an embedded pole, we also implicitly understand that in the real world the passive pressure does not switch from limit state passive maxima in front of the pile to limit state passive maxima behind the pile over an infinitesimal distance at the rotation point, e.g. the stress block transition between stress blocks 5 and 6 in Figure 7-2 to Figure 7-4.

For this reason, we have tapered the limit state passive pressure block from its original value down to zero at the rotation point. This better represents the transition zone, and, in fact, the real pressure forces.

The reason for this is two-fold:

1. to ensure that, absent any load factors, the methodology is not unconservative, and
2. to broadly align results with that of our independent verification software suite

Our overall goal is to be accurate or conservative – though not unduly so, but certainly not unconservative. Understandably, perfect alignment across multiple software applications implementing different methodologies is unlikely. Nevertheless, broad alignment is important.

To elaborate in terms of our verification suite of software applications, we have used two independent, widely-used and recognised GeoTech programs (Wallap, and Limit State Geo), each using two different approaches, as well as another recently developed 'isolated pile' methodology [14].

In terms of 'specific results' comparison, we have used Wallap as the software of record, in part because of its detailed analysis outputs, but also due to its implementation of a pseudo non-linear displacement assessment.

In addition to the obvious strength/stability aspect, we have also considered limit state displacements – particularly for seismic analyses, refer Section 6.3.

In terms of soil pressures, Wallap shows below-ground pressure somewhat different from the idealised triangular or trapezoidal pressure block diagrams. The differences mainly occur adjacent to the nominal rotation point, and are roughly linear. And so, in order to better align with Wallap's soil pressures and net results, we have introduced the following two passive pressure modifiers:

- 'Tapered Ratio' (λ): the limit state stress block pressure is tapered from an intercept based on the depth ratio down to zero at the rotation point
- 'Pressure Reduction Multiplier' (PRM): amount the pressure block values are scaled down by

These adjustments are demonstrated below for passive pressure distributions of blocks 5 (triangular) and 6 (trapezoidal).

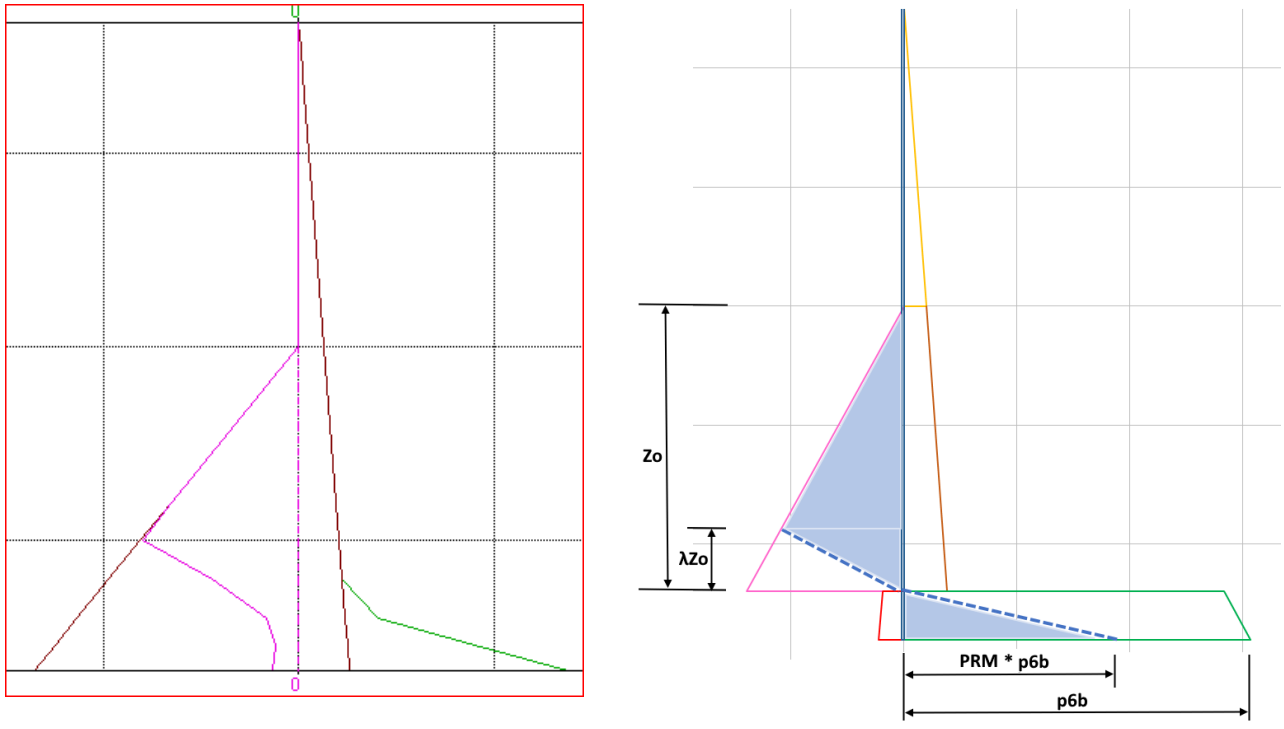


Figure 7-5: Combined active/pressure diagram from Wallap vs modified pressure blocks 5 and 6 used in this guide

7.8 Effect of passive pressure modifiers

Refer to Appendix A.6 for values of λ and PRM that have currently been adopted for this guide.

For passive pressure above the rotation point, i.e. block 5, the pressure magnitudes generally align well with Wallap, hence the PRM factors of unity leave the pressure magnitudes unchanged. However, the amount of taper varies according to the loading and soil type.

For passive pressures below the rotation point, i.e. block 6, our verification analyses all showed passive pressures considerably below the (theoretical) unmodified passive pressures, hence PRM values for block 6 are set well below unity. Further, the verification pressures uniformly presented essentially as a triangle, hence the tapered ratio values of $\lambda_6 = 1.0$, i.e. 100% tapered.

Upon reflection, this makes sense, as the pole rotations are small, the rotational (Zo to Le) distance required to achieve sufficient resistive capacity is also small, hence the displacement even at the bottom of the pole is insufficient to generate the theoretical maximum passive pressures assumed. Note that, broadly, the area of (reduced) pressure block 6 must be similar to the theoretical (unreduced) block 6, but that it is achieved over a slightly greater depth.

The following demonstrates the change to the shape of passive pressure blocks for an undrained analysis as a result of using a block 5 Tapered Ratio (λ_{5u}) of 0.7 and a block 6 Pressure Reduction Multiplier (PRM_6) of 0.65.

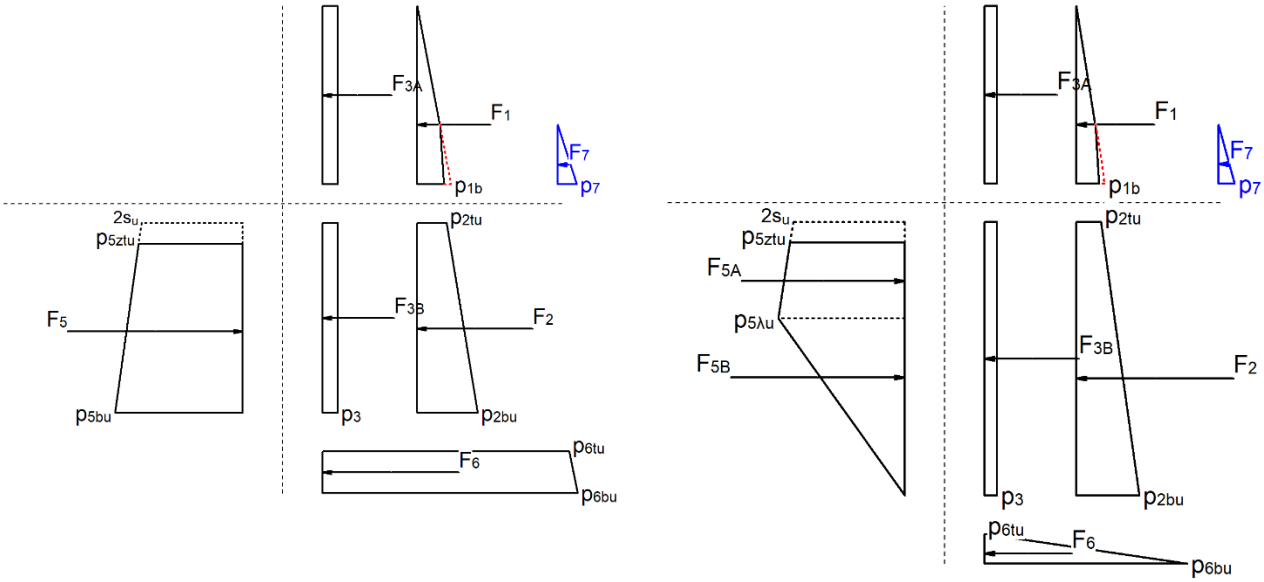


Figure 7-6: Effect of passive pressure modifiers for undrained analysis (before and after adjustments)

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:

- Define inputs (Section 8.2)
- Determine effective pressure width factor for below-ground soil pressures (Section 8.3)
- Calculate minimum depth to prevent overturning (Section 8.4)
- Check for sliding failure (Section 8.5)
- Calculate design actions on pole (Section 8.6)
- Check pole structural capacity (Section 9)

These steps are expanded in the following sections.

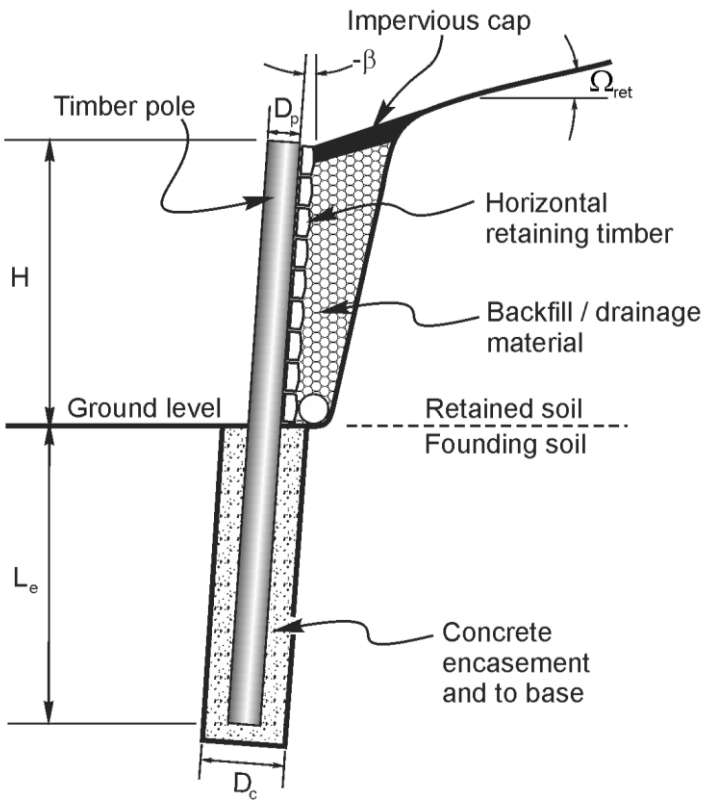


Figure 8-1: Generic wall layout

8.2 Define inputs

Inputs need to be defined in the following areas:

- wall geometry
- surcharge and seismic loading
- pole properties
- site geometry
- soil properties
- design parameters

A full list of required inputs is given in Appendix A – Nomenclature.

8.3 Determine effective pressure width

The effective pressure width EPW = MIN(3.5D_c, Sp) is used for all below-ground soil pressures when calculating forces applied to the pole.

8.4 Calculate minimum depth to prevent overturning failure

Calculate Zo, the distance below ground to the rotation point for equilibrium, and L_e, the required embedment depth, using the following iterative process:

1. Assume an initial estimate for Zo and Le_{min}
2. Calculate soil and water pressures at depth according to Appendix D.1 below
3. Calculate the resultant (factored) pressure forces on the wall F_1-F_8 (refer Appendix D.2), using:
    a. active earth pressures for driving forces and passive earth pressures for resisting forces
    b. an effective pressure width EPW for all below-ground soil pressure forces
    c. load factors α_a, α_{ae}, α_w, α_{we}, α_q, α_{qe} for driving forces
    d. capacity factors Φ_p, Φ_{pe} for resisting forces
4. Calculate the factored driving moment about Zo (M^*d) (refer Appendix D.3)
5. Calculate the dependable restoring moment about Zo (M^*r) (refer Appendix D.3)
6. Use equilibrium equations for forces and moments about the rotation point to refine Zo and Le_{min} (refer Appendix D.4)

This process may have to be done separately for the static long-term, static short-term, and seismic load cases, due to changes in the:

- design surcharge load
- analysis type (drained vs undrained)
- pressure coefficients (K_a and K_p vs K_{ae} and K_{pe})

The above algorithm assumes an automated implementation and could be further simplified for hand calculation.

8.5 Check for sliding failure

This is a possible failure mode for non-cohesive soils but is unlikely to govern design for typical cases.

Provided the equation for horizontal force equilibrium is used in Section 8.4 above, and the worst-case minimum embedded depth Le_{min} is chosen from the various load cases, there is generally no need to carry out a separate check for sliding failure.

8.6 Calculate design actions on pole

The pole strength must be considered for the relevant critical location. This is most likely at the ground surface where it is embedded in concrete AND composite action is justified (refer Section 9.1 and Appendix E.3); otherwise for driven piles it is likely to be below the ground surface at the point of maximum below-ground moment. Design actions are calculated as follows:

1. (For critical location at ground surface) calculate the factored design shear V^*p and moment M^*p at ground level (refer Appendix D4)
2. (For critical location below ground surface) iterate to find Zbm, the distance below ground to the point of zero shear (i.e. the point of maximum moment) in the pole, based on design actions. Hence calculate the design moment M^*pz at Zbm

Once again, this check may have to be done separately for the static long-term, static short-term, and seismic load cases.

Refer to Appendix D4 for details of this calculation.

8.7 Minimum depth 'rules of thumb'

Based on advice from experienced industry practitioners, the following table gives some rules of thumb minimum embedded length for areas of low seismicity.

Founding soil	***Rule of thumb minimum embedded length Le_{min}***
Cohesive	1.0H + Zt
Cohesionless, no water table	1.5H
Cohesionless, water at ground level	2.0H

9. Timber structural capacity checks

This section addresses some key aspects relating to structural capacity. For additional detail, the designer should refer to the relevant material design standard.

9.1 Composite strength increase factor Cc

In this guide, Cc is a factor applied to the bare pole flexural strength to obtain the composite timber/concrete strength in cases where the pole is embedded in concrete. There are conflicting points of view on the appropriateness of including composite action from the surrounding concrete in the strength of the pole. This guide provides versatility of use according to the knowledge of the designer and the state of knowledge at the time.

The main design benefit of concrete-encasement should be seen as the increased passive capacity of the soil (due to the increased effective diameter of the pole), rather than any composite pole strength improvement. The Cc factor should be set to 1.0 unless the design engineer has undertaken an appropriate risk analysis, including consideration of relevant 'decision factors', and is confident that an increased Cc is justified. Refer Appendix E.3 for additional information on this issue.

9.2 Pole dependable strength

Design standard

The following is based on design of pole moment and shear to NZS 1720.1 [8].

Flexure

Design timber pole strength (refer Appendix A.8 for notation):

$$f_{bpd} = k_{21}*k_{22}*k_{23}*f_{bp}$$

Timber pole section modulus:

$$Zp = \pi*D_p^3/32$$

Bare pole design flexural strength:

$$\Phi Mn = \Phi_t*k_1*f_{bpd}*Zp \quad \text{(long-term loading)}$$

$$\Phi Mn_e = \Phi_{te}*k_{1e}*f_{bpd}*Zp \quad \text{(seismic/short-term loading)}$$

Composite pole design flexural strength (only required if pole is concrete-encased):

$$\Phi Mn_c = \Phi Mn*Cc \quad \text{(long-term loading)}$$

$$\Phi Mn_{ce} = \Phi Mn_e*Cc \quad \text{(seismic/short-term loading)}$$

Shear

Design timber shear strength:

$$f_{spd} = k_{21}*k_{22}*k_{23}*f_s$$

Gross cross-sectional area of timber pole:

$$A_g = \pi*D_p^2/4$$

Shear area:

$$A_v = 0.75\ A_g$$

Bare pole design shear strength:

$$\Phi Vn = \Phi_t*k_1*f_{spd}*A_v \quad \text{(long-term loading)}$$

$$\Phi Vn_e = \Phi_{te}*k_{1e}*f_{bpd}*A_v \quad \text{(seismic/short-term loading)}$$

Timber standard factors

Refer to Appendix A.8 for definitions of k_1, k_{21}, k_{22} & k_{23}.

9.3 Check structural capacity

Pole bending strength

Check that the design moment is not more than the relevant dependable strength as follows:

Concrete-encasement	Critical location	Design check
Pole is driven	Below ground-level	$M*p_z <= \Phi Mn_z$ $M*p_{ze} <= \Phi Mn_{ze}$
Pole is concrete-encased below ground	May be at ground level or below ground-level	$M*p <= \Phi Mn$ $M*p_e <= \Phi Mn_e$ $M*p_{cz} <= \Phi Mn_{cz}$ $M*p_{cze} <= \Phi Mn_{cze}$

Pole shear strength

Check the design shear at ground-level, although this is unlikely to govern in typical cases:

	Critical location	Design check
Pole is driven or concrete-encased	At ground-level	$V*p <= \Phi Vn$ $V*p_e <= \Phi Vn_e$

10. References

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11. Amendment History

Version	*Date*	*Description*
1.0	29/4/2024	First public release
1.1	18/12/2024	Update formulas for pressure block 4 Fix formula typos for passive pressure coefficients Update design example Updates to Section 6.3 Amend φ_{pe} to 0.8/0.9 in Appendix B.2 Limitation of design with water table present (section 5.6)
1.2	23/6/2025	Update pole dependable strength calculations in Sections 9.2 and 9.3 and corresponding nomenclature in Appendix A.8 in accordance with NZS 1720.1 [8] Update Appendix B recommended load factors: following discussions between SESOC and NZGS, 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 Update Appendix F (design example) using the new load factors

Appendices

Appendix A – Nomenclature

A.1 Wall Geometry

D_p	Diameter of timber pole at ground level
D_c	Diameter of embedded (normally concrete-encased) pole
d_{bf}	Depth dimension below backfill surface
H	Height of pole above ground (i.e. retained height)
Le	Actual length of embedment. Le should be at least 0.75H for cohesive founding soil, and at least 1.0H for cohesionless founding soil
Sp	Centre-to-centre spacing of poles
β	Angle of back face of the wall from vertical, negative for a wall leaning back towards the retained soil, so $\beta = -\Omega_w$
Ω_w	Slope of the front of the wall, positive for a wall leaning back towards the retained soil

A.2 Site

Zt	Depth to start of passive bearing. For cohesive soils, the first Zt (e.g. 0.25 to 0.5m) depth of the resisting soil is considered to provide no resistance (refer Section 5.2 for more details)
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 founding soil (cohesive soils only)
s_u	Undrained shear strength of founding soil (cohesive soils only)

δ_f'	Angle of interface friction between the founding soil and the back of embedded poles, e.g. $0.5\varphi'_f$
γ_f	Drained unit weight of founding soil
$\gamma_{f\text{-}w}$	Unit weight of founding soil below water table (= γ_f - γ_w with water table present, = γ_f where there is no water table)
$\gamma_{min} \cdot z$	Minimum active pressure for undrained soil at depth z
φ'_f	Effective angle of founding soil shearing strength (internal friction angle)
Backfill (assumed always cohesionless)	
δ'_{bf}	Angle of interface friction between backfill soil and back of wall, e.g. $0.5\varphi'_{bf}$
γ_{bf}	Drained unit weight of backfill soil
$\gamma_{bf\text{-}w}$	Unit weight of backfill soil below water table = γ_{bf} - γ_w
φ'_{bf}	Effective angle of backfill soil shearing strength (internal friction angle)
Ground water	
γ_w	Unit weight of water = 9.81 kN/m^3
h_w	Retained soil water level, with a maximum of H/3 for static loading and a maximum of 0 for seismic loading

A.4 Applied loading

k_h	Design earthquake coefficient (horizontal acceleration)
ω_D, ω_L	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.5 Design parameters

Φ_p, Φ_t	Capacity reduction factors – refer to Appendix B.2 for additional documentation and recommended values
$\alpha_{a, q, etc}$	Load factors – refer to Appendix B.3 for additional documentation and recommended values

A.6 Passive pressure modifiers

λ_n	Tapered ratio – proportion of passive pressure block n that is tapered. The pressure is set in a straight line from the point defined by λ to zero at the rotation point Zo (refer to Section 7.7) λ can vary from 0 (no taper) to 1 (block is fully tapered)
PRM_{n*}	Pressure reduction multiplier – passive pressure block n values are scaled down by multiplying by the PRM_{n*} (* subscript d=cohesionless drained, c=cohesive, u=undrained) (refer to Section 7.7)

See Section 7.7 for descriptions of these parameters, and the table below for values.

Block	***Analysis***	***Tapered Ratio λ_n***	***Pressure Reduction Multiplier PRM_{n*}***
5	Cohesionless (5d)	0.1	1.0
	Cohesive (5c)	0.4	1.0
	Undrained (5u)	0.7	1.0
6	Cohesionless (6d)	1.0	0.8
	Cohesive (6c)	1.0	0.6
	Undrained (6u)	1.0	0.65

A.7 Derived properties

d_{bf}	Depth dimension below backfill surface behind wall
EPW	Effective pressure width for below-ground soil pressures = MIN(4Dc, Sp)
F_n	Net force from soil or water pressure in pressure block n (see Figure 7-2 to Figure 7-4)
K_{a*}	Coefficients of active 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 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
K_{p*}	Coefficients of passive 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_{pev} is the vertical component of the passive earth pressure coefficient under seismic loading
Le_{min}	Minimum required length of embedment, calculated to prevent foundation failure
M^*d	Overall design overturning moment (‘driving’ moment) about Zo, modified by relevant load factor
M^*r	Overall restoring moment about Zo, modified by relevant capacity factor
M^*p	(Factored) design moment on pole at ground level

M^*pz	(Factored) design moment on composite pole below ground level
M_n	Moment about Zo corresponding to pressure block n
M_{nz}	Moment about Zbm corresponding to pressure block n
p_n	Defining soil or water pressure in pressure block n (see Figure 7-2 to Figure 7-4)
$p_{v,z}$	Vertical soil pressure at z below ground level (after allowing for buoyancy) At ground level: $p_{v,GL} = \gamma_{bf} H - \gamma_w h_w$ At Zo: $p_{v,Zo} = p_{v,GL} + \gamma_{f\text{-}w} Zo$ At Le_{min}: $p_{v,Le} = p_{v,GL} + \gamma_{f\text{-}w} Le_{min}$
V_{nz}	Shear in pole at depth Zbm from pressure block n
V^*p	(Factored) design shear on pole at ground level
z	Depth dimension below ground level (in front of wall)
z=Zbm	Distance below ground to point of zero shear (maximum moment) in pole, based on factored design actions. Zbm varies according to the relevant loading and analysis type under consideration: static drained, short-term static undrained, seismic drained, seismic undrained
z=Zo	Distance below ground to rotation point for equilibrium, based on factored design actions. Zo varies according to the relevant loading and analysis type under consideration: static drained, static cohesive, seismic drained, seismic undrained
Zc_n	Depth below ground to centroid of pressure block n
$Z\lambda_n$	Depth below ground to start of passive pressure block n taper
ω^*, ω_e^*	Factored superimposed dead plus live load, uniform distribution per unit horizontal width

Note: unless otherwise indicated, use of suffixes “c”, “u” and “e” indicate cohesive soil, undrained conditions and seismic loading respectively

A.8 Pole properties

A_g	Gross cross-sectional area of pole at ground level
A_v	Shear area of pole at ground level
Cc	Composite strength increase factor, default 1.0, for concrete-encased poles (where applicable). Equals the ratio between the design flexural strength of the composite pole and that of the bare pole Refer to Appendix E.3 Pole strength increase due to composite action
f_{bp}	Characteristic bending stress for poles; NZS 1720 has normal density (38 MPa) and high density (52 MPa)
f_{bpd}	Design bending strength of timber pole
f_{sp}	Characteristic shear stress for poles; NZS 1720 has normal density (3.1 MPa) and high density (3.5 MPa)
f_{spd}	Design shear strength of timber pole
k_1, k_{1e}	Timber load duration factor (=0.57 for gravity long-term, =1.0 for seismic or gravity short-term)
k_{21}	Peeling or shaving factor, refer NZS 1720 Table ZZ6.2 [8], normally 0.8 for bending, 1.0 for shear
k_{22}	Seasoned use factor, refer NZS 1720 Table ZZ6.4 [8], normally 1.25 for bending, 1.06 for shear
k_{23}	Preservative treatment involving steaming factor, refer NZS 1720 Table ZZ6.3 [8], normally 0.85 for bending, 0.9 for shear
ΦMn	Dependable pole flexural strength
ΦMn_c	Dependable composite pole flexural strength
ΦVn	Dependable pole shear strength
Zp	Timber pole section modulus
Zpc	Composite section modulus = Zp * Cc

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 capacity 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.

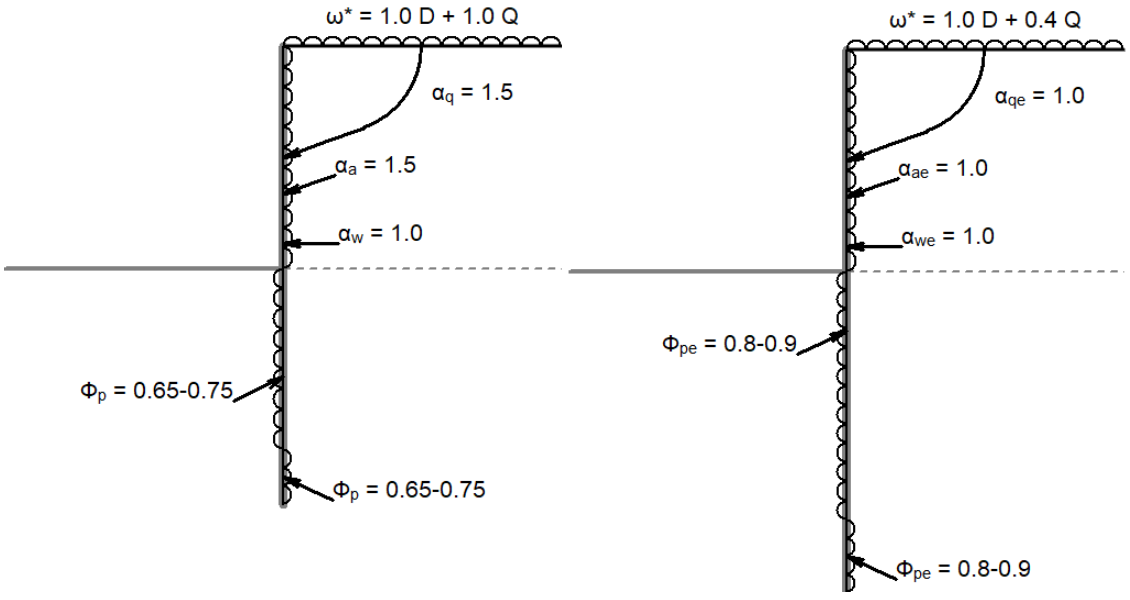


Figure B.1-1: Recommended design factors

B.2 Recommended capacity reduction factors

Item	*Typical value*	*Description*
Passive soil resistance		Reliable resistance factors based on passive soil pressure
Φ_p	0.65 (cohesive) 0.75 (cohesionless)	Capacity factor for foundation failure: pole foundation rotation (failure mode 1 in Section 4 above)
Φ_{pe}	0.8 (undrained) 0.9 (drained)	Capacity factor for pole foundation rotation failure under seismic loading
Timber strength		Timber strength reduction factors
Φ_t	0.8	Timber strength reduction factor (failure mode 2)
Φ_{te}	0.8	Timber strength reduction factor under seismic loading

B.3 Recommended load factors

Item	*Typical value static/seismic*	*Description*
Superimposed loading		Factors applied to uniform dead and live loads (ω_D and ω_L) superimposed on the retained soil. Note that calculation of pressure on the wall resulting from the destabilising loading is subject to an additional α_q factor
α_D/α_{De}	1.0/1.0	Load factor for superimposed permanent (dead) loading causing destabilising effect under static/seismic loading. Updated June 2025
α_L/α_{Le}	1.0/0.4	Load factor for superimposed live loading causing destabilising effect under static/seismic loading. Updated June 2025

Active earth pressure		Factors applied to the pressure loads resulting from the retained soil and from any superimposed loading
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 Updated June 2025
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. In particular, we wanted to benchmark and align our methodology with several 'industry' examples.

Unsurprisingly, we found some variation of approach and results. Of more concern, however, was the sometimes-significant variation between a 'factor of safety' (working stress) and factored (ultimate) designs – for the same basic problem.

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 below.

Code	*Active earth pressure α_a*	*Earth pressure from super-imposed α_q*	*Dead Load Factor[1]*	*Live Load Factor[1]*	*Drained Reduction Factor*	*Undrained Reduction Factor*	*Effective FoS (for no superimposed LL)*	
							Drained	*Undrained*
Pender 2000							2.0	
MBIE Example Gravity	1.0	1.0	1.2	0.4	0.67		1.5	
MBIE Example Seismic	1.0	1.0	1.0	0.3		1.0		1.0
Hong-Kong Guidelines			1.0	1.5	0.83	0.5	1.2	2.0
AS 4678: Wall Category B			1.25	1.5	0.9	0.5	1.4	2.5
EN 1998-5:2004					0.8	0.71		
CIRIA C580			1.35	1.35	0.83	0.67	1.6	2.0
B1/VM4 Gravity/Sliding	1.6	1.6			0.8	0.8	2.0	2.0
B1/VM4 EQ/Sliding	1.0	1.0			0.8	0.8	1.3	1.3
B1/VM4 Gravity/Bearing	1.6	1.6			0.45	0.45	3.6	3.6
B1/VM4 EQ/Bearing	1.0	1..0			0.45	0.45	2.2	2.2
SESOC Recommended Gravity	1.5				0.75[2(a)]	0.65[2(b)]	2.0[s(a)]	2.31[2(b)]
SESOC Recommended EQ	1.0				0.9	0.8	1.11	1.25
SESOC Recommended Gravity		1.5[3]	1.0[3]	1.0[3]	0.75[2(a)]	0.65[2(b)]	2.0[2(a)]	2.31[2(b)]
SESOC Recommended EQ		1.0	1.0	0.4	0.9	0.8	1.11	1.25

Notes:

1. For SESOC recommended, superimposed dead and live load factors are applied in addition to α_q. For other examples, meaning of dead and live load factors is unknown
2. (a) Values for cohesionless soil (b) values for cohesive soil under static loading
3. 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 v1.2 of this Design Guide

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 wall (angle β from vertical, assumed negative where the wall is leaning back towards the retained soil). 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_{ret} (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 wall):

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

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 $k_h = 0$.

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

Note that if k_h is such that $\varphi_{bf} - \Omega_{ret} - \theta < 0$, 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 (2021) [5]). It is unlikely that K_{ae} will increase significantly above its maximum value when this occurs, and so the following equation applies:

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

The active earth pressure (static and seismic) acts at 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 [7]):

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

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

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

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 [7]. 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. $\Omega_{ret} = \beta = \delta_f = 0$)

Under static loading:

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

where

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

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

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

And under seismic loading:

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

where

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

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

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 $\delta_{bf} / \varphi_{bf}$. For a more accurate (but not closed-form) solution, please refer to Soubra 2011 [13].

Appendix D – Calculation of design actions

The following equations are formulated explicitly for non-seismic load cases. However, unless otherwise noted, by substituting the relevant pressure coefficients, z=Zo, Le_{min}, and load and capacity reduction factors they equally apply for:

- static or seismic loading
- drained or undrained analysis

A "$*$" subscript in any of the following equations (e.g. F_{5*}) indicates that it is only applicable to cohesive soil analyses – drained ($*$ = c) or undrained ($*$ = u).

Refer Figure 7-2 to Figure 7-4 for block numbering.

D.1 Soil pressures for drained and undrained analyses

(Set z=Zo)

Pressure block	***Drained analysis***	***Undrained analysis***
Active soil pressure (driving):		
1	$p_{1b} = K_{ah,bf}\ \gamma_{bf}\ H$ $-\Delta p_{1b} = -K_{ah,bf}\ \gamma_w\ h_w$ (buoyancy adjustment) (i.e. $p_{1b} - \Delta p_{1b} = K_{ah,bf}\ p_{v,GL}$)	
2	Cohesionless: $p_{2t} = K_{ah,f}\ p_{v,GL}$ $p_{2b} = p_{2t} + K_{ah,f}\ \gamma_{f\text{-}w}\ z$ Cohesive (where p_{2tc} and p_{2bc} >=0): $p_{2tc} = K_{ah,f}\ p_{v,GL} - 2c'\sqrt{K_{ah,f}}$ $p_{2bc} = K_{ah,f}\ p_{v,GL} + K_{ah,f}\ \gamma_{f\text{-}w}\ z - 2c'\sqrt{K_{ah,f}}$	$p_{2tu} = \text{Max}[\gamma_{min}\ H,\ p_{v,GL} - 2s_u]$ $p_{2bu} = \text{Max}[\gamma_{min}\ (H+z),\ p_{v,GL} + \gamma_{f\text{-}w}\ z - 2s_u]$
3	$p_3 = K_{aqh,bf}\ \omega^*$	
4	Cohesionless: $p_{4t} = K_{ah,f}\ \gamma_{f\text{-}w}\ Zo$ $p_{4b} = K_{ah,f}\ \gamma_{f\text{-}w}\ Le_{min}$ Cohesive (where p_{4tc} and p_{4bc} >=0): $p_{4tc} = K_{ah,f}\ \gamma_{f\text{-}w}\ Zo - 2c'\sqrt{K_{ah,f}}$ $p_{4bc} = K_{ah,f}\ \gamma_{f\text{-}w}\ Le_{min} - 2c'\sqrt{K_{ah,f}}$	$p_{4u} = 0$

Passive soil pressure (restoring):		
5	Taper start location: $Z\lambda_5 = Zt + (1-\lambda)(Zo - Zt)$ $= (1-\lambda)\,Zo$ for drained Cohesionless: $p_{5b} = K_{ph}\,\gamma_{f\text{-}w}\,z$ (for no tapering) $p_{5\lambda} = K_{ph}\,\gamma_{f\text{-}w} * Z\lambda_5$ Cohesive (top Zt is ineffective): $p_{5ztc} = K_{ph}\,\gamma_{f\text{-}w}\,Zt + 2c'\sqrt{K_{ph}}$ $p_{5bc} = K_{ph}\,\gamma_{f\text{-}w}\,z + 2c'\sqrt{K_{ph}}$ (for no tapering) $p_{5\lambda c} = K_{ph}\,\gamma_{f\text{-}w}\,Z_{5\lambda} + 2c'\sqrt{K_{ph}}$	$p_{5ztu} = \gamma_{f\text{-}w}\,Zt + 2s_u$ $p_{5bu} = \gamma_{f\text{-}w}\,z + 2s_u$ (for no tapering) $p_{5\lambda u} = \gamma_{f\text{-}w}\,Z_{5\lambda} + 2s_u$
6	Taper start location: $Z\lambda_6 = Zo + \lambda\,(Le_{min} - Zo)$ $= Le_{min}$ for $\lambda = 1$ (fully tapered) Cohesionless: $p_{6t} = K_{ph}\,p_{v,Zo} * PRM_{6d}$ (for no tapering) $p_{6b} = K_{ph}\,p_{v,Le} * PRM_{6d}$ Cohesive: $p_{6tc} = (K_{ph}\,p_{v,Zo} + 2c'\sqrt{K_{ph,f}}] * PRM_{6c}$ (for no tapering) $p_{6bc} = (K_{ph}\,p_{v,Le} + 2c'\sqrt{K_{ph,f}}) * PRM_{6c}$	$p_{6tu} = (\gamma_{bf}\,H + \gamma_{f\text{-}w}\,Zo + 2s_u) * PRM_{6u}$ (for no tapering) $p_{6bu} = (\gamma_{bf}\,H + \gamma_{f\text{-}w}\,Le_{min} + 2s_u) * PRM_{6u}$
Water pressure:		
7	$p_7 = \gamma_w * h_w \quad (0 \le h_w \le H)$	

As noted earlier, Zo is the distance below ground level to the rotation point, and Le_{min} is the required length of embedment for force and moment equilibrium. Zo and Le_{min} vary according to the relevant loading and analysis type under consideration:

- static or seismic loading
- drained (cohesionless or cohesive) or undrained analysis

The same equations (with z=Zbm instead of z=Zo where required) can be used when calculating pressures and hence shears and moments in the pole at any point above Zo.

These pressure blocks are shown diagrammatically in Section 7.6 above.

D.2 Pressure forces on wall

(Set z=Zo)

Block	***Description***	***Equation***
Unfactored driving forces per pole (active pressures)		
1	Force from back of wall retained soil pressure	$F_1 = ½p_{1b} * H * Sp$ $-\Delta F_1 = -½\Delta p_{1b} * h_w * Sp$ (buoyancy adjustment)
2	Force from back of wall founding soil pressure	$F_2 = ½(p_{2b} + p_{2t}) * z * EPW$
3	Force from surcharge	$F_{3A} = p_3 * H * Sp$ (above ground-level) $F_{3B} = p_3 * z * EPW$ (below ground-level)
4	Force from soil pressure below Zo	$F_4 = ½(p_{4b} + p_{4t})* (Le_{min} - z) * EPW$
7, 7n	Force from water pressure	$F_7 = ½p_7 * h_w * Sp$ $F_{7n} = p_7 * z * D_c$ (drained only)
Unfactored restoring forces per pole (passive pressures)		
5	Force from passive pressure above (1-λ)Zo (non-tapered): Cohesionless: Cohesive, top Zt ineffective: Force from passive pressure above Zo but below (1-λ)Zo (i.e. the triangular tapered section)	$F_{5A} = ½p_{5\lambda} * z * EPW$ $F_{5A} = ½(p_{5\lambda*} + p_{5zt*}) * (z - Zt) * EPW$ $F_{5B} = ½p_{5\lambda*} (1 + z - Zo) * [z – (1-\lambda)Zo] * EPW$ (for z=Zo) $F_{5B} = ½p_{5\lambda*} \lambda Zo * EPW$
6	Force from passive pressure below Zo:	$F_6 = ½p_{6b} * (Le_{min} - z) * EPW$

D.3 Driving and restoring moments about z

(Set z=Zo)

Block	*Description*	*Equation*
Unfactored driving moments per pole		
	Total driving moment (factored)	$M^*d = (M_1 - \Delta M_1 + M_2 + M_4) * \alpha_a + (M_{3A} + M_{3B}) * \alpha_q + (M_7 + M_{7n}) * \alpha_w$
	where:	
1	Moment from back of wall retained soil pressure	$M_1 = F_1 * (z - Zc_1)$ $-\Delta M_1 = -\Delta F_1 * (z - Zc_{\Delta 1})$ (buoyancy adjustment)
2	Moment from back of wall foundation pressure	$M_2 = F_{2A} * (z - Zc_2)$
3	Moment from surcharge	$M_{3A} = F_{3A} * (z - Zc_{3A})$ $M_{3B} = F_{3B} * (z - Zc_{3B})$
4	Moment from soil pressure below Zo	$M_4 = F_4 * (Zo - Zc_4)$
7, 7n	Moment from water pressure	$M_7 = F_7 * (z - Zc_7)$ $M_{7n} = F_{7n} * (z - Zc_{7n})$ (drained only)
Unfactored restoring moments per pole		
	Total restoring moment (dependable)	$M^*r = (M_{5A} + M_{5B} + M_6) * \Phi_p$
	where:	
5	Moment from passive pressure above Zo	$M_{5A} = F_{5A} * (z - Zc_{5A})$ $M_{5B} = F_{5B} * (z - Zc_{5B})$
6	Moment from passive pressure below Zo	$M_6 = F_6 * (Zo - Zc_6)$

D.4 Check equilibrium equations

Block	*Description*	*Equation*
Horizontal forces per pole (factored active forces + dependable resisting forces)		
	Horizontal force equilibrium	$(F_1 - \Delta F_1 + F_2 - F_4) * \alpha_a + (F_{3A} + F_{3B}) * \alpha_q + (F_7 + F_{7n}) * \alpha_w$ $= (F_{5A} + F_{5B} - F_6) * \Phi_p$
Moments about Zo per pole (factored driving moments + dependable restoring moments)		
	Moment equilibrium	M*d = M*r

D.5 Calculate design actions on pole

Calculate shear and bending moment at ground level (z=0), and maximum moment below ground level (z=Zbm). Shears and moments down to z=Zbm are calculated as above, but only considering that portion of the pressure blocks that are above z. Note that Zbm is always above Zo, so pressure blocks 4 and 6 can be ignored.

Block	*Description*	*Equation*
Shear and Moment at ground level per pole		
	Total shear at ground level (factored)	$V^*p = (F_1 - \Delta F_1) * \alpha_a + F_{3A} * \alpha_q + F_7 * \alpha_w$
	Total moment at ground level (factored)	$M^*p = [F_1* H/3 - \Delta F_1 * h_w/3] * \alpha_a$ $+ F_{3A}* H/2 * \alpha_q + F_7 * h_w/3 * \alpha_w$
Depth where shear is zero (point of maximum bending moment below ground level)		
	Total factored shear (= 0 at position of maximum bending moment)	$V*pc = (V_{1z} + V_{2z}) * \alpha_a + V_{3z} * \alpha_q + (V_{7z} + V_{7nz}) * \alpha_w$ $- V_{5z} * \Phi_p$ (cohesionless) $V*pc_* = (V_{1z} + V_{2uz}) * \alpha_a + V_{3z} * \alpha_q + V_{7z} * \alpha_w - V_{5*z} * \Phi_p$ (cohesive drained/undrained)
	where:	

	Depth to zero shear	Iterating to find Zbm:
1	Shear from retained soil pressure driving force	$V_{1z} = F_1 - \Delta F_1$
2	Shear from back of wall founding soil driving force	$V_{2z} = F_{2(z=Zbm)}$
3	Shear from surcharge/live load driving force	$V_{3z} = F_{3A} + F_{3B(z=Zbm)}$
7, 7n	Shear from water pressure	$V_{7z} = F_7$ $V_{7nz} = F_{7n(z=Zbm)}$ (drained only)
5	Shear at depth from passive soil resistance	$V_{5z} = F_{5(z=Zbm)}$
Maximum bending moment below ground level		
	Total maximum moment at depth (factored)	$M^*pc = (M_{1z} - \Delta M_{1z} + M_{2z}) * \alpha_a + (M_{3Az} + M_{3Bz}) * \alpha_q + (M_{7z} + M_{7nz}) * \alpha_w - M_{5z} * \Phi_p$
	where:	
1	Moment at depth from soil active pressure	$M_{1z} = F_1 * (Zbm - Zc_1)$ $-\Delta M_{1z} = -\Delta F_1 * (Zbm - Zc_{\Delta 1})$ (buoyancy adjustment)
2	Moment at depth from backfill soil pressure	$M_{2z} = F_{2(z=Zbm)} * (Zbm - Zc_{2B(z=Zbm)})$
3	Moment at depth from surcharge	$M_{3Az} = F_{3A} * (Zbm - Zc_{3A})$ $M_{3Bz} = F_{3B(z=Zbm)} * (Zbm - Zc_{3B(z=Zbm)})$
7, 7n	Moment at depth from driving water pressure	$M_{7z} = F_7 * (Zbm - Zc_7)$ $M_{7nz} = F_{7n(z=Zbm)} * (Zbm - Zc_{7n(z=Zbm)})$ (drained only)
5	Moment at depth from passive soil resistance	$M_{5z} = F_{5(z=Zbm)} * (Zbm - Zc_{5(z=Zbm)})$

Appendix E – Other matters

E.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 3 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) [12], which gives a procedure for carrying out the M-O analysis in a c-φ soil.

E.2 An approach to sloping site in front of wall

The equations in this design guide assume a horizontal site slope in front of the wall so that full passive load resistance may be achieved. A distance of 8 times the embedded pole diameter is proposed as sufficient to satisfy these criteria. An approach for a sloped site is to use an 'effective' ground-level at the worst-case lowest location between the wall and 8Dc from the wall – see diagram below.

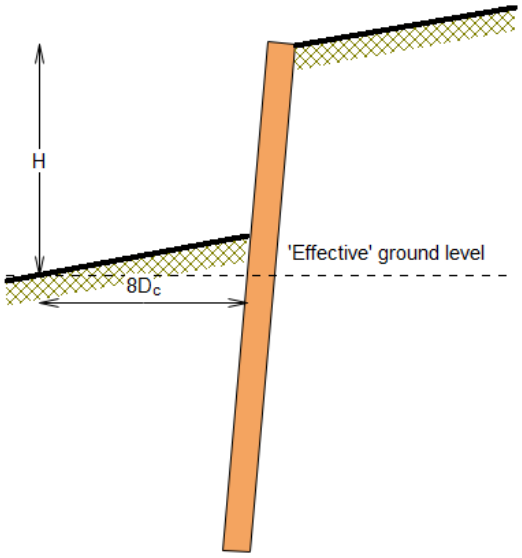


Figure E-1: Treatment of sloping site in front of wall

Refer to Wood 2021 [14] for further discussion on this matter. The reader is also encouraged to seek specific Geotechnical advice, as appropriate, and as advised elsewhere in this guide.

E.3 Pole strength increase due to composite action

The extent of the contribution of any concrete encasement to the flexural strength requires specific evaluation by the engineer for each design. The Cc factor used in this design guide should be set to 1.0 (even if the pole is concrete-encased) unless the design engineer has undertaken an appropriate risk analysis, including consideration of relevant 'decision factors', and is confident that an increased Cc is justified.

Decision factors might include some or all of the following:

- The consequences of failure and difficulty in undertaking effective remedial work where performance failures occur
- The source of the lateral earth pressure applied to the wall, and the extent to which its magnitude can be accurately assessed, over both the short and long term
- The uncertainties which exist around the assumptions surrounding the assumed composite action from the concrete encasement
- Whether the retaining wall structure is temporary or permanent and/or whether the lateral earth pressure load to be resisted is short term or long term
- Nature of the encasement concrete, including the presence of an additive such as fibre and/or other reinforcing to enhance its dependable strength in tension
- The reliance on inspections and/or supervision by the design engineer to ensure that assumptions made in design are reflected in what has been constructed. As-built dimensions of concrete encasement, extent of contamination of concrete by soil during placement and/or compaction, condition of timber/concrete interface, e.g. the presence or absence of mechanical shear keys are examples of matters requiring on site verification
- Space for surrounding concrete between large end diameter and augured hole
- Temporary stability of retained slope as lagging and drainage aggregate is installed – especially if structures above
- Surface water to be diverted if appropriate
- Foreseeable (and possibly unforeseen) occurrences affecting the design, e.g. mounding of ground water profiles, or unauthorised excavation or re-profiling of ground in front of the wall
- The range in performance margins ("safety index") recommended in design of structures of this type in particular loading situations, particularly where importance levels (IL) are higher than normal

E.4 Timber pole – other issues

Pole specification

In addition to the strength calculations outlined in Section on page 339 above, the reader is reminded of the need for vigilance regarding specification of the poles and compliance with the requisite codes. In particular, the presence of multiple knots in a local area will likely result in a significant reduction in capacity – and has been the cause of cantilever timber pole wall failures in the past.

The designer must also consider the effects of pole 'trimming' and the moisture conditions (i.e. 'wet') the poles are to operate in over their life.

Note that poles are commonly supplied with a taper (e.g. 6mm/metre of pole), which the designer should make appropriate allowance for.

Construction issues

The designer should consider potential construction issues when deciding on the type of wall (driven or bored). This includes a wall installed in a relatively permeable soil with a high groundwater level. The concrete within a bored timber pole wall is unable to be tremie poured below the groundwater level. If the permeability of the soil is sufficiently high, the augured hole is likely to at least partially fill with water after a pump has been removed and the pole installed and positioned. Consideration should therefore be given to using a driven pole.

Horizontal retaining timber

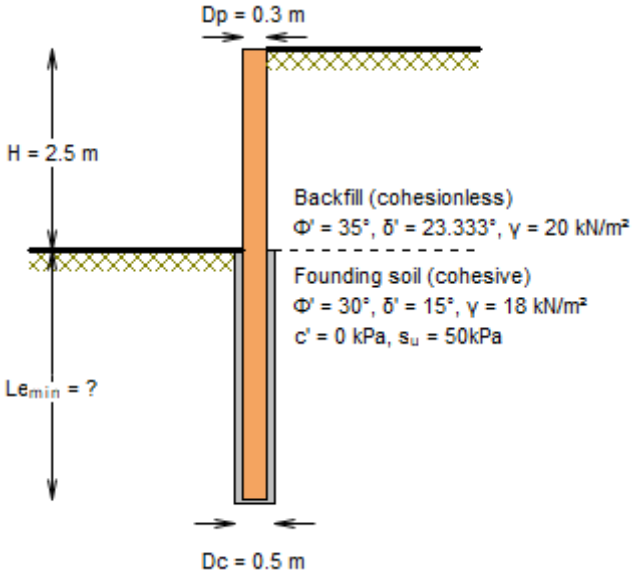
The horizontal retaining timber (rails/walers/lagging) spanning between poles should be specifically designed for strength and to prevent 'bowing' (the designer should allow for long-term deflection in the presence of moisture).

Appendix F – Worked design example

This example is loosely based on the MBIE/NZGS (2021) [5] Example 1 for a cantilever timber pole retaining wall calculation. The design parameters do not necessarily represent a typical wall.

The following calculations indicate the broad workflow taken from the SESOC Retaining Wall software (RetWall).

GEOMETRY



Pole diameter Dp	0.3 m
Embedded pole diameter Dc	0.5 m
Retained height H	2.5 m
Pole spacing Sp	1.2 m
Wall slope Ω_w	0°

SOIL PROPERTIES	**Founding soil**	**Backfill**
Soil type	Cohesive	Cohesionless
Internal friction angle φ'	30°	35°
Friction angle with wall δ'	15°	23.3°
Drained soil density γ	18 kN/m^3	20 kN/m^3
Effective cohesion c'	0 kPa	
Undrained shear strength s_u	50 kPa	
Slope of retained soil Ω_{ret}	0°	
Depth to start of bearing Zt	0.5 m	(c' = 0 so ignore Zt for static loading)

LOADING

Seismic coefficient kh	0.2
Dead load surcharge ω_D	0 kN/m
Live load surcharge ω_L	2.5 kN/m

Assumed no water table present

DESIGN FACTORS

Load factors:	***Static Destabilising***	***Seismic Destabilising***
Active α_a	1.5	1.0
Pressure from surcharge α_q	1.5	1.0
Superimposed dead load α_D	1.0	1.0
Superimposed live load α_L	1.0	0.4

Reduction factors:	***Drained (static)***	***Undrained (seismic)***
Passive Φ_p	0.75	0.8
Pole timber Φ_t	0.8	0.8

DERIVED PROPERTIES

Angle of back face of wall

β = -0° (negative for back-leaning wall)

Effective pressure width for below-ground soil pressures

EPW = MIN[3.5 * Dc, Sp]
= MIN[3.5 * 0.5, 1.2]
= 1.2 m

Unit weight of founding soil (no water table)

$\gamma_{f\text{-}w} = \gamma_f$
= 18 kN/m^3

Factored load surcharge

$\omega^* = \alpha_D \omega_D + \alpha_L \omega_L$
= 1.0 * 0.0 + 1.0 * 2.5 = 1.0 * 0.0 + 0.4 * 2.5
= 2.5 kN/m (static) = 1.0 kN/m (seismic)

Equivalent angle of seismic loading used to calculate Kae

$\theta = \tan^{-1}kh$ = 11.31°

Minimum active pressure for undrained soil

$\gamma_{min}.z$ = 5z kPa

Vertical soil pressure at ground level (behind wall)

$p_{v,GL} = \gamma_{bf} H$
= 50.0 kPa

PRESSURE COEFFICIENTS

Active coefficients based on Coulomb/M-O, and passive coefficients based on Mylonakis et al (2007).

Pressure coefficients used for each pressure block are given in the following table:

Active pressure block (from earth)	***φ' (°)***	***δ' (°)***	***β (°)***	***Ω_{ret} (°)***	***K_{ah}***	***K_{aeh}***
Backfill (p1)	35	23.3	0	0	0.224	0.353
Founding soil (p2)	30	15	0	0	0.291	0.437
Founding soil (p4)	30	15	0	0	0.291	0.437
Active pressure block (from superimposed load)					***K_{aqh}***	***K_{aqeh}***
(due to surcharge) (p3)	35	23.3	0	0	0.224	0.353
Passive pressure block					***K_{ph}***	***K_{peh}***
Founding soil (p5)	30	15	0	0	4.29	3.73
Founding soil (p6)	30	15	0	0	4.29	3.73

FORCE AND MOMENT EQUILIBRIUM

For horizontal force equilibrium:

$$\sum F^* = 0$$

$$\text{i.e. } 0 = \sum F_{active} * \alpha_a + \sum F_{surcharge} * \alpha_q + \sum F_{water} * \alpha_w + \sum F_{passive} * \Phi_p$$

$$= (F_1 + F_2 - F_4) * \alpha_a + (F_{3A} + F_{3B}) * \alpha_q + (F_7 + F_{7n}) * \alpha_w + (-F_{5A} - F_{5B} + F_6) * \Phi_p$$

For moment equilibrium about Zo:

$$\sum M^* = 0$$

$$\text{i.e. } 0 = \sum M_{active} * \alpha_a + \sum M_{surcharge} * \alpha_q + \sum M_{water} * \alpha_w + \sum M_{passive} * \Phi_p$$

$$= (M_1 + M_2 + M_4) * \alpha_a + (M_{3A} + M_{3B}) * \alpha_q + (M_7 + M_{7n}) * \alpha_w + (-M_{5A} - M_{5B} - M_6) * \Phi_p$$

For force and moment equilibrium: (solve for Zo and Le_{min})	Static loading (drained analysis)	Seismic loading (undrained analysis)
Zo =	2.49 m	1.99 m
Le_{min} =	3.01 m	2.59 m

SOIL PRESSURES (z = Zo)	Static pressure (kPa) (drained analysis)	Seismic pressure (kPa) (undrained analysis)
$p_{1b} = K_{ah}.\gamma_{bf}.H$	11.2	17.6
$p_{2tc} = K_{ah,bf}\ p_{v,GL}$	14.6	
$p_{2tu} = Max[\gamma_{min}\ H, \gamma_{bf}\ H - 2s_u]$		12.5
$p_{2bc} = K_{ah,bf}\ p_{v,GL} + K_{ah,f}\ \gamma_{f\text{-}w}\ z$	27.6	
$p_{2bu} = Max[\gamma_{min}\ (H + z), \gamma_{bf}\ H + \gamma_{f\text{-}w}\ z - 2s_u]$		22.4
$p_3 = K_{aqh}\ \omega^*$	0.561	0.353

$p_{4tc} = K_{ah,f}\ \gamma_{f\text{-}w}\ Zo$	13.1	
$p_{4bc} = K_{ah,f}\ \gamma_{f\text{-}w}\ Le_{min}$	15.8	
$p_{4tu} = p_{4bu} = 0$		0.0
$Z\lambda_5 = Zt + (1 - \lambda)\ (Zo - Zt)$	(2.24 m)	(0.946 m)
$p_{5ztc} = K_{ph}\ \gamma_{f\text{-}w}\ Zt$	0.0	
$p_{5ztu} = \gamma_{f\text{-}w}\ Zt + 2s_u$		109
$p_{5\lambda c} = K_{ph}\ \gamma_{f\text{-}w}\ \lambda\ Z\lambda_5$	173	
$p_{5\lambda u} = \gamma_{f\text{-}w}\ \lambda\ Z\lambda_5 + 2s_u$		117
$p_{6bc} = (p_{v,GL} + \gamma_{f\text{-}w}\ Le_{min}).PRM_c$	357	
$p_{6bu} = (\gamma_{bf}\ H + \gamma_{f\text{-}w}\ Le_{min} + 2s_u).PRM_u$		128
PRESSURE FORCES ON WALL (z = Zo)	Static force (kN) (drained analysis)	Seismic force (kN) (undrained analysis)
$F_1 = ½ * p_{1b} * H * Sp$	16.8	26.5
$F_2 = ½ * (p_{2b} + p_{2t}) * z * EPW$	63.1	41.7
$F_{3A} = p_3 * H * Sp$	1.68	1.06
$F_{3B} = p_3 * z * EPW$	1.68	0.841
$F_4 = ½ * (p_{4b} + p_{4t}) * (Le_{min} - Zo) * EPW$	8.90	0.0
$F_{5A} = ½ * (p_{5\lambda*} + p_{5zt}) * (z - Zt) * EPW$	233	60.5
$F_{5B} = ½ * p_{5\lambda} * \lambda * z * EPW$	25.9	73.1
$F_6 = ½ * p_{6b} * (Le_{min} - Zo) * EPW$	110	46.1
MOMENTS ABOUT Zo (z = Zo)	Static moment (kNm) (drained analysis)	Seismic moment (kNm) (undrained analysis)
$M_1 = F_1 * (z - Zc_1)$	56.0	74.6
$M_2 = F_2 * (z - Zc_2)$	70.6	37.5
$M_{3A} = F_{3A} * (z - Zc_{3A})$	6.30	3.43
$M_{3B} = F_{3B} * (z - Zc_{3B})$	2.09	0.836
$M_4 = F_4 * (Zo - Zc_4)$	2.36	
$M_{5A} = F_{5A} * (z - Zc_{5A})$	233	76.3
$M_{5B} = F_{5B} * (z - Zc_{5B})$	4.31	50.7
$M_6 = F_6 * (Zo - Zc_6)$	37.8	18.5

POLE DESIGN ACTIONS

Location of maximum moment (= location of zero shear)		Static loading (drained analysis)	Seismic loading (undrained analysis)
	Zbm	1.52 m	0.91 m

Actions on pole - critical sections Depth below ground level (m)		Static loading Shear (kN)	Static loading Moment (kNm)	Seismic loading Shear (kN)	Seismic loading Moment (kNm)
z = G.L. =	0.0	27.8	24.2	27.5	23.4
Zbm (static) =	1.52	0.0	62.7		
Zbm (seismic) =	0.91			0.0	46.6

Note: For this design example, the founding soil c' was set to zero even though the soil was nominally cohesive. Where c' is zero, the RetWall software treats the soil as cohesionless for static loading (drained conditions), which means that:

- Zt is assumed to be zero
- Passive Φp is 0.75 not 0.65


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