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Concrete Masonry – Gravity Retaining Walls

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Concrete Masonry Association of Australia

Segmental Concrete
Gravity Retaining Walls

Preface

Standards Australia has published AS 4678–2002 for the design of earth retaining structures, which include segmental concrete gravity retaining walls. It encompasses the following features:

- Limit state design
- Partial loading and material factors
- Compatibility with the general approach in AS 1170 SAA Loading code[Note1]
- Compatibility with the structures standards such as AS 3600 *Concrete structures* and AS 3700 *Masonry structures*.

This Guide provides Australian designers and contractors with a comprehensive approach to the design and construction of segmental concrete gravity retaining walls based on:

- The design and construction rules set out in AS 4678-2002
- An analysis method developed by the Concrete Masonry Association of Australia (CMAA) to fit Australian experience.

It includes:

- A description of the principal features of the Australian Standard
- A description of the analysis method
- A design example which demonstrates the use of the Australian Standard and analysis method
- A site investigation check list
- A detailed construction specification.

NOTES:
1   When published in early 2002, AS 4678 included load factors which were compatible with the load factors on the version of AS 1170 that was then current. However, changes to AS 1170 in late 2002 have meant that exact similarity of load factors no longer exists.

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Segmental Concrete
Gravity Retaining Walls

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1.1 General

1.1 General

For many years reinforced masonry cantilever retaining walls, relying on gravity loads to resist the overturning forces due to soil pressure, have been constructed with reinforced concrete masonry stems (steel reinforcement grouted into hollow concrete blockwork) which is built on a reinforced concrete footing.

Segmental concrete gravity retaining walls, consisting of concrete masonry units dry-stacked against a soil slope and resisting overturning by virtue of their own weight became more widespread in the 1980's and early 1990's. This system provided very attractive embankment finishes, but is limited in height.

A further development during the 1990's has been the incorporation of geogrids into the soil mass behind the structure to create dry-stacked masonry reinforced soil structures. Provided the system is correctly designed and there is access to the soil mass at all levels, such a system can be constructed several metres high.

This Guide covers the design of segmental concrete gravity retaining walls. The other two forms of construction are covered in separate Guides.

In 1990 the Concrete Masonry Association of Australia (CMAA) published *Masonry Walling Guide No 4 Design For Earth Loads - Retaining Walls*, which set out a design methodology for earth retaining structures. It included:

■ Ultimate load design with material factors based on characteristic soil properties, partial load factors consistent with AS 1170.1 and structure designs to AS 3700 and AS 3600.

■ Coloumb analysis of the back fill

■ Bearing analysis using the Meyerhoff approach (including tilt and inclined load factors).

■ Sliding analyses that account for friction, passive pressure and (if appropriate) base adhesion.

These design and analysis features were a considerable improvement on the working stress/assumed bearing capacity/Rankine analysis that was then in common use. These principles have been adopted in Standards Australia
AS 4678–2002, which is appropriate for the design of these structures.

1.2 Scope

This manual makes use of the following design and analysis methods.

■ Coulomb Method for all walls over 1200 mm high;

■ Rankine-Bell Method for walls, in some situations, in the range 800 mm to 1200 mm high;

■ Simple depth to height proportions for walls, in some situations, under 800 mm high.

NOTES:
2   This Guide uses the terminology "dead load" to indicate permanent loads and "live load" to indicate imposed loads. This terminology is consistent with the convention adopted in AS 4678-2002.

1.3 Glossary

Loads and limit states:

Design life
The time over which the structure is required to fulfil its function and remain serviceable.

Dead load[Note 2]
The self-weight of the structure and the retained soil or rock.

Live load[Note 2]
Loads that arise from the intended use of the structure, including distributed, concentrated, impact and inertia loads. It includes construction loads, but excludes wind and earthquake loads.

Wind load
The force exerted on the structure by wind, acting on either or both the face of the retaining wall and any other structure supported by the retaining wall.

Earthquake load
The force exerted on the structure by earthquake action, acting on either or both the face of the retaining wall and any other structure supported by the retaining wall.

Stability limit state
A limit state of loss of static equilibrium of a structure or part thereof, when considered as a rigid body.

Strength limit state
A limit state of collapse or loss of structural integrity of the components of the retaining wall.

Serviceability limit state
A limit state for acceptable in-service conditions. The most common serviceability states are excessive differential settlement and forward movement of the retaining wall.

Components:

Concrete masonry units
Concrete blocks manufactured to provide an attractive, durable, stable face to a retaining wall. They are available in a variety of shapes and sizes, with varying degrees of interlock.

Geotextile
A permeable, polymeric material, which may be woven, non-woven or knitted. It is commonly used to separate drainage material from other soil.

Retained material
The natural soil or rock, intended to be retained by a retaining wall.

Foundation material
The natural soil or rock material under a retaining wall.

Infill material
The soil material placed immediately behind the retaining wall facing.

Drainage material
The crushed rock, gravel or similar material placed behind a retaining wall to convey ground water away from the wall and foundations. It is commonly used in conjunction with other drainage media, such as agricultural pipes.

Soil types:

Cohesive fill
Naturally-occurring or processed materials with greater than 50% passing the 75 μm Australian standard sieve, a Plasticity Index of less than 30% and a Liquid Limit of less than 45%.

Controlled fill Class I
Soil, rock or other inert material that has been placed at a site in a controlled fashion and under appropriate supervision to ensure the resultant material is consistent in character, placed and compacted to an average density equivalent to 98% (and no test result below 95%) of the maximum dry density (standard compactive effort) for the material when tested in accordance with AS 1289.5.1.1. For cohesionless soils, material compacted to at least 75% Density Index is satisfactory.

Controlled fill Class II
Soil, rock or other inert material that has been placed in specified layers and in a controlled fashion to ensure the resultant material is consistent in character, placed and compacted to an average density equivalent to 95% (and no test result below 92%) of the maximum dry density (standard compactive effort) for the material when tested in accordance with AS 1289.5.1.1. For cohesionless soils, material compacted to at least 75% Density Index is satisfactory. Generally the layer thickness is specified as a maximum of 300 mm.

Uncontrolled fill
Soil, rock or other inert material that has been placed at a site and does not satisfy the materials included above.

Insitu material
Natural soil, weathered rock and rock materials.

GW
Well-graded gravel as defined by the *Cassegrande extended classification system*. Generally in the range of 2 mm to 60 mm, and graded such that the smaller particles pack into the spaces between the larger ones, giving a dense mass of interlocking particles with a high shear strength and low compressibility.

SW
Well-graded sand as defined by the *Cassegrande extended classification system*. Generally in the range of 0.06 mm to 2 mm, and graded such that the smaller particles pack into the spaces between the larger ones, giving a dense mass of interlocking particles with a high shear strength and low compressibility.

GP
Poorly-graded gravel as defined by the *Cassegrande extended classification system*. Generally in the range of 2 mm to 60 mm, and of a single size. This material has good drainage properties provided it is protected from infiltration by silts and clays.

No-fines concrete
Concrete with a crushed rock aggregate of nominal size 12 to 20 mm, a characteristic compresive strength greater than 10 MPa and a density between 1600 and 1800 kg/m^3.

1.4 Behaviour of Segmental Concrete Gravity Retaining Walls

If unrestrained, a soil embankment will slump to its angle of repose. Some soils, such as clays, have cohesion that enables vertical and near-vertical faces to remain partially intact, but even these may slump under the softening influence of ground water. When an earth-retaining structure is constructed, it restricts this slumping. The soil exerts an active pressure on the structure, which deflects a little and is then restrained by the friction and adhesion between the base and soil beneath, passive soil pressures in front of the structure and the bearing capacity of the soil beneath the toe of the structure.

If water is trapped behind the retaining structure, it exerts an additional hydraulic pressure. This ground water also reduces the adhesion and bearing resistance. If massive rock formations are present, immediately behind the structure, these will restrict the volume of soil which can be mobilised and thus reduce the pressure.

The walls described in this Guide are gravity, earth-retaining structures, consisting of dry-stacked segmental concrete units. The retained soil exerts an active pressure on the wall. Overturning is resisted by the vertical load of the structure.

Drain (minimum 1 in 100 fall) to permanent stormwater system

Optional capping unit

NOTE:
Facing may be vertical, stepped or sloped, depending on the particular units used

Concrete segmental units

Bearing pad

Compacted foundation

Compacted clay or similar to seal surface. 150 mm minimum thickness

Minimum fall 1 in 100

Excavation line

Infill material as per specification, if required

Geotextile filter material, if required

Drainage material (In some cases, no-fines concrete may be used to increase wall stability)

Slotted PVC ag. pipe draining to stormwater at minimum 1 in 100 fall

Figure 1.1   *Typical Segmental Concrete Gravity Retaining Wall*

1.5 Importance of a Geotechnical Report

The design of a retaining wall includes two essential parts:

- Analysis of the adjacent ground for global slip, settlement, drainage and similar global considerations; and
- Analysis and design of retaining wall structure for strength.

These analyses must be based on an accurate and complete knowledge of the soil properties, slope stability, potential slip problems and groundwater. A geotechnical report by a qualified and experienced geotechnical engineer should be obtained.

Such a report must address the following considerations, as well as any other pertinent points not listed.

- Soil properties;
- Extent and quality of any rock, including floaters and bedrock;
- Global slip and other stability problems;
- Bedding plane slope, particularly if they slope towards the cut;
- Effect of prolonged wet weather and the consequence of the excavation remaining open for extended periods;
- Effect of ground water;
- Steep back slopes and the effect of terracing;
- Effect of any structures founded within a zone of influence.

1.6 Safety and Protection of Existing Structures

Whenever soil is excavated or embankments are constructed, there is a danger of collapse. This may occur through movement of the soil and any associated structures by:

- rotation around an external failure plane that encompasses the structure,
- slipping down an inclined plane,
- sliding forward, or
- local bearing failure or settlement.

These problems may be exacerbated by the intrusion of surface water or disruption of the water table, which increase pore water pressures and thus diminish the soil's ability to stand without collapse.

The safety of workers and protection of existing structures during construction must be of prime concern and should be considered by both designers and constructors. All excavations should be carried out in a safe manner in accordance with the relevant regulations, to prevent collapse that may endanger life or property. Adjacent structures must be founded either beyond or below the zone of influence of the excavation. Where there is risk of global slip, for example around a slip plane encompassing the proposed retaining wall or other structures, or where there is risk of inundation by ground water or surface water, construction should not proceed until the advice of a properly qualified and experienced Geotechnical Engineer has been obtained and remedial action has been carried out.

1.7 Global Slip Failure

Soil-retaining structures must be checked for global slip failure around all potential slip surfaces or circles (Figure 1.2).

Designers often reduce the heights of retaining walls by splitting a single wall into two (or more) walls, thus terracing the site. Whilst this may assist in the design of the individual walls, it will not necessarily reduce the tendency for global slip failure around surfaces encompassing all or some of the retaining walls.

The designer should also take into account the effects of rock below or behind the structure in resisting slip failure.

Analysis for global slip is not included in this guide and it is recommended that designers carry out a separate check using commercially available software.

1.8   Differential Settlement

Techniques to reduce or control the effects of differential settlement and the possibility of cracking include (Figure 1.3):

- Articulation of the wall (by discontinuing the normal stretcher bond) at convenient intervals along the length.
- Excavating, replacing and compacting areas of soft soil.
- Limiting the stepping of the base to a maximum of 200 mm.

1.9   Importance of Drainage

This Guide assumes that a properly-functioning drainage system is effective in removing hydraulic pressure. If this is not the case, the designer will be required to design for an appropriate hydraulic load.

Based on an effective drainage system, it is common to use drained soil properties. For other situations, the designer must determine whether drained or undrained properties are appropriate. In particular, sea walls that may be subject to rapid draw-down (not covered in this Guide) require design using undrained soil properties.

1.10 Passive Pressure

For completeness, this Guide gives consideration to passive pressure, which, in some circumstances, could contribute to the resistance to forward sliding.

Because the soil in front of a retaining wall can be excavated, eroded or otherwise disturbed, it is strongly recommended that passive pressure in front of the wall be ignored in design.
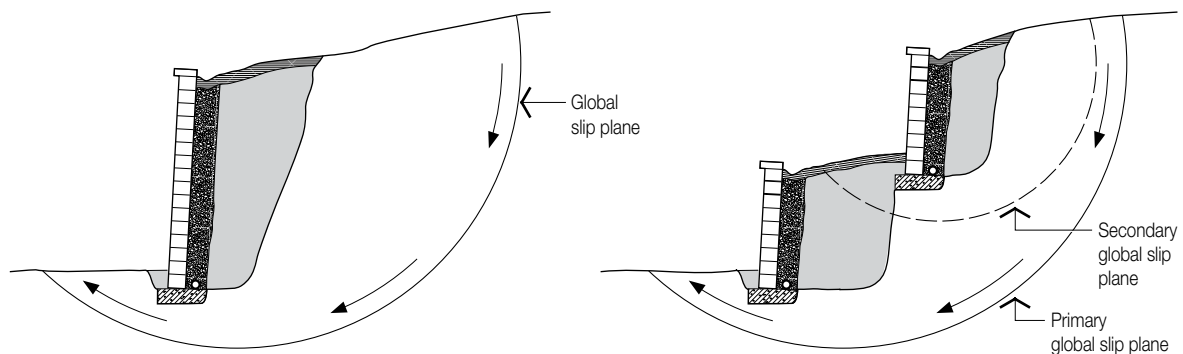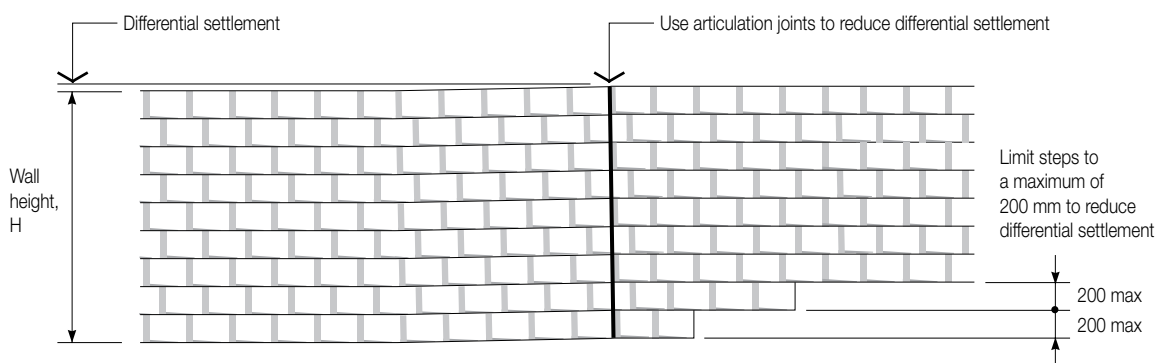

Figure 1.2   *Global Slip Failure*

Figure 1.3   *Reduction and Control of Differential Settlement*

2 Design Considerations

2.1 Scope

This guide considers retaining walls founded on undisturbed material that is firm and dry and achieves the friction angle and cohesion noted for each particular soil type. It does not cover foundations exhibiting any of the following characteristics:

- Softness
- Poor drainage
- Fill
- Organic matter
- Variable conditions
- Heavily-cracked rock
- Aggressive soils.

If these conditions are present, they must be considered by the designer.

2.2 Limit State Design

The design limit states to be considered are:

- strengths of the various components subject to ultimate factored loads;
- stability of the structure as a whole subject to ultimate factored loads; and
- serviceability of the structure and its components subject to service loads.

Important Note:
Serviceability considerations are beyond the scope of this Guide. However, the designer is strongly advised to consider closely the appropriate serviceability limits and the methods of satisfying these requirements in practical design. One common method is to limit the stresses in the foundation soil.

2.3 Partial Loading and Material Factors

Partial-loading and partial-material factors enable the designer to assign various levels of confidence to assumed or measured soil strengths, material strengths and resistance to deterioration, predictability of loads and consequence of failure of various structures.

There are several reasons for compatibility of loading factors between AS 4678-2002 and AS 1170 *SAA Loading code*, which applies to buildings[Note 3].

- Buildings are often constructed close to retaining walls, and therefore apply loads on them.
- Parts of buildings such as basement walls are often required to withstand loads imposed by earth and soil.

- The adoption of common load factors assists the rational comparison of the levels of safety and probability of failure of retaining walls and other structures.
- The design of concrete, masonry, steel and timber components of earth retaining structures is determined using Australian Standards which are based on limit state concepts and loading factors from AS 1170.
- Most structural engineers are familiar with the loading factors of AS 1170.

2.4 Load Combinations and Factors for Stability

The following load combinations and factors should be applied when checking the stability of the structure. This includes analysis for:

- Global slip
- Overturning
- Bearing capacity of the foundation under the toe of the base
- Sliding resistance of the foundation under the base[Note 4].

(i) $\quad 1.25\,G^C + 1.5\,Q^C \qquad\qquad < 0.8\,G^R + (\Phi\,R)$

(ii) $\quad 1.25\,G^C + \psi_c\,Q^C + W_u^{\ C} \qquad < 0.8\,G^R + (\Phi\,R)$

(iii) $1.25\,G^C + \psi_c\,Q^C + 1.0\,F_{eq}^{\ C} \quad < 0.8(G + \psi_c Q)^R + (\Phi\,R)$

Where:

G^C = parts of the dead load tending to cause instability.
This includes:
the weight of the retained soil, which causes horizontal pressures on the stem, thus tending to cause forward sliding, bearing failure or overturning, or the weight of the infill soil, which causes horizontal pressures on the facing, thus tending to cause stem rupture.

Q^C = parts of the live load tending to cause instability.
This includes all removable loads such as temporary loadings, live loadings applied from adjacent buildings, construction traffic and soil compaction loads and an allowance for the temporary stacking of soil of not less than 5 kPa, except for Structure Classification A.

$W_u^{\ C}$ = parts of the wind load tending to cause instability.

NOTES:
3 When published in early 2002, AS 4678 included load factors which were compatible with the load factors on the version of AS 1170 that was then current. However, changes to AS 1170 in late 2002 have meant that exact similarity of load factors no longer exists.

4 Design for bearing capacity and external sliding resistance, involve the factoring-down of the soil properties (density, friction angle and/or cohesion) which are providing the resistance to instability.

$F_{eq}{}^C$ = parts of the earthquake load tending to cause instability.

For earthquake categories A_e and B_e, design for static loads without further specific analysis is deemed adequate.
For earthquake category C_e, a dead load factor of 1.5 (instead of 1.25) should be used and specific design for earthquake may be neglected.
For earthquake categories D_e and E_e, the structures should be designed and analysed in accordance with the detailed method set out in AS 4678 Appendix I.

G^R = parts of the dead load tending to resist instability.

This includes the self weight of the structure and the weight of soil in front of the structure.

Φ^R = the factored design capacity of the structural component.

This includes calculated bearing capacity, sliding resistance, calculated pull-out strength, etc.

ψ_c = live load combination factor.

This is taken as 0.4 for parking or storage and 0.6 for other common applications on retaining walls.

Factoring of Loads

In this Guide, the following approach has been adopted.

Overturning live loads (imposed loads) include active soil loads resulting from point live loads, line live loads and uniform live load surcharges.

Overturning dead loads (permanent loads) include active soil loads resulting from the weight of the soil fill and from parts of the structure that are forward of the point of rotation.

Overturning live loads and overturning dead loads are factored generally in accordance with the principles of AS 4678, except as qualified below.

If a live load is capable of being present in a location that causes net overturning, while simultaneously being absent from another location that causes net stability, the respective parts are factored by 1.5 and zero. This corresponds to the principle described in AS/NZS 1170.0 and known as "chequer board" loading.

After an overturning load has been factored (eg by 1.5 if it is a live load or by 1.25 if it is dead load), it is *then* resolved into horizontal and vertical components. In particular, after an active soil load (tending to cause net overturning of the retaining wall) is factored, it is *then* split into a horizontal component and a vertical downwards friction component. Because the horizontal component and the vertical friction component *can only exist simultaneously*, they are multiplied by the same factor.

Designers are strongly advised to consider the validity of this approach, and modify it as considered appropriate. In particular, they should consider whether it is appropriate to:

- factor the forces first and then resolve into components, or
- to resolve the unfactored forces into components first and then apply different factors.

Note: The vertical friction is calculated from a conservative design internal friction angle, ϕ, which has been factored down from the characteristic (conservative estimate of the mean) value of internal friction angle.

2.5 Load Combinations and Factors for Strength of Components

The following load combinations and factors should be applied when checking the strength of the structure components, including strength of any associated concrete, masonry and reinforcement.

(i) $1.25\,G + 1.5\,Q$

(ii) $1.25\,G + W_u + \psi_c\,Q$

(iii) $1.25\,G + 1.0\,F_{eq} + \psi_c\,Q$

(iv) $0.8\,G + 1.5\,Q$

(v) $0.8\,G + W_u$

(vi) $0.8(G + \psi_c\,Q) + 1.0 F_{eq}$

Where:

G = dead load

Q = live load

W_u = wind load

F_{eq} = earthquake load

ψ_c = live load combination factor taken as 0.4 for parking or storage and 0.6 for other common applications on retaining walls.

2.6 Live Loads

The appropriate values for live load must be determined by the design engineer. AS 4678–2002 specifies a minimum live loading of 5 kPa for walls of any height of Structure Classifications C and B.

For walls under 1.5 metres high which are of Structure Classification A, the following minimum live loads are applicable.

Slope of retained soil less than or equal to 1:4 – 2.5 kPa

Slope of retained soil greater than 1:4 – 1.5 kPa

2.7 Earthquake Loads

The appropriate earthquake loads must be determined by the designer. If earthquake load acts on some supported structure such as a building or a fence, the effect must be considered.

2.8 Wind Loads

The load factors are such that load combination (ii) involving wind loading, will not be the governing case when the effect due to wind, W_u^C is less than $(1.5 - \psi_C)$ times the effect due to live load, Q^C. For example, for a wall that does not support another exposed structure and for a minimum live load surcharge of $Q^C = 5$ kPa, an active pressure coefficient of $K_a = 0.3$ and a live load combination factor of $\psi_C = 0.6$, a wind load on the face of the retaining wall less than 1.35 kPa will not be the governing case. However, if the wind load is applied to some supported structure such as a building or a fence, the effect must be considered.

2.9 Hydraulic Loads

The design example is based on the assumption that a properly-functioning drainage system is effective in removing hydraulic pressure.

2.10 Drained v Undrained Parameters

Based on an effective drainage system, the design example uses drained soil properties. For other situations, the designer must determine whether drained or undrained properties are appropriate.

2.11 Capacity Reduction Factors

The material strength factors from AS 4678 Table 5.1 should be used used.

2.12 Soil Analysis Model

AS 4678–2002 Appendix E provides formulae for several analysis models, but does not specify which is appropriate for any particular application.

Coulomb Method
This Guide uses the Coulomb method to analyse structures for cohesionless soils, or soils in which cohesion is assumed to be zero.

Rankine Bell Method
This Guide also uses the Rankine-Bell Method to analyse structures for cohesive soils for walls in the range 800 mm to 1200 mm maximum.

Cohesion is the property of a cohesive soil that:

- permits a cut surface to stand vertically (up to a particular height) without additional support from a wall, and

- provides significant contribution to bearing capacity.

For determining active forces on retaining walls, this Guide recommends that cohesion of retained soils should be assumed to be zero and recommends against the use of the Rankine-Bell method, except where:

- ***the retaining wall is supporting an undisturbed soil (not a fill material),***

- ***the soil exhibits considerable reliable cohesion***

- ***the retaining wall is not more than 1.2 m high (refer Appendix D),***

- ***the Structure Classification (as per AS 4678) is 'A' (where failure would result in minimal damage or loss of access), and***

- ***the structure includes an impermeable surface membrane and drainage system such that there can be no ingress of any water into the soil behind the retaining wall.***

Note: This Guide also recommends that a very conservative value of cohesion should be assumed when determining the bearing capacity. See Clause 2.16 *Bearing Failure*)

If the Rankine-Bell method is used to determine active forces on retaining walls, due consideration must be given to the following points.

- Cohesion is difficult to predict, is variable and may change over time, depending on the soil moisture content. It is important not to overestimate cohesion. AS 4678–2002 Table D4 provides a range of cohesions and corresponding range of internal friction angles for various soils.

- Surface sealing, surface drainage and subsurface drainage are critical to the correct function of the earth retaining system. The design cohesion (if used) should reflect the lowest value expected during the design life and the most pessimistic moisture conditions.

- Drained and/or undrained cohesion values should be used in the analysis, depending on effectiveness of the drainage system and the rate of loading.

- Clay soils shrink when dry and swell when saturated. Over several shrink/swell cycles, a retaining wall in clay soils will creep forward and, in extreme cases, may overturn. If forward creep is a concern, clay backfill should be replaced with a stable, cohesionless material.

2.13 Active Pressure

In response to soil pressure, the wall will move away from the soil, thus partially relieving the pressure. This reduced pressure is the active pressure.

Coulomb Method

The Coulomb equation for active pressure coefficient (K_a) can account for slope of the wall and slope of the backfill. The slope of the wall should be restricted to less than external angle of friction (δ) to ensure that there is no upward component of earth pressure which would reduce sliding resistance (ie the equation applies when wall slope is less than 15° for good-quality granular backfills in contact with concrete).

p_a = active pressure on the wall at depth of H

$\quad = K_a \gamma H$

Where:

K_a = active pressure coefficient

$$= \frac{\cos^2(\phi + \omega)}{\cos^2\omega \cos(\omega - \delta) \left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \beta)}{\cos(\omega - \delta)\cos(\omega + \beta)}}\right]^2}$$

ϕ = factored value of internal friction angle (degrees)

δ = external friction angle (degrees)

$\quad = \dfrac{2\phi}{3}$

where ϕ is the smaller of the friction angles at the particular interface

At any interface with a geotextile, the external friction angle should be taken from test data. If no data is available, it should be assumed to be zero.

ω = slope of the wall (degrees)

β = slope of the backfill (degrees)

γ = factored value of soil density (kN/m^3)

H = height of soil behind the wall (m)

Rankine-Bell Method

The Rankine-Bell equation can account for cohesion, but not for the slope of the wall. The direction of the force on the wall is assumed to be the same as the backfill slope.

p_a = active pressure on the wall at depth of H

$\quad = \gamma H \tan^2(45 - \phi/2) - 2 c \tan(45 - \phi/2)$

Where:

ϕ = factored value of internal friction angle (degrees)

H = height of soil behind the wall (m)

c = cohesion (kPa)

2.14 Pressure at Rest

If the wall is unable to move away from the soil embankment, as may be the case for a propped cantilever basement wall, there will be no relief of the pressure and the soil will exert the full pressure at rest.

p_o = soil pressure at rest

$\quad = K_o \gamma H$

Where:

K_o = coefficient for soil at rest

$\quad = 1.0$

γ = factored value of soil density (kN/m^3)

H = height of soil behind the wall (m)

2.15 Passive Pressure

If the structure pushes into the soil, as is the case at the toe of a retaining wall, the resistance by the soil is greater than the pressure at rest. This is the passive pressure, given by the following equation. If the soil in front of the toe is disturbed or loose, the full passive pressure may not be mobilised. It is strongly recommended that passive pressure in front of the wall be ignored in design.

Coulomb Method

p_p = passive soil pressure (kPa)

$\quad = K_p \gamma H_e$

Where:

K_p = passive pressure coefficient

$\quad = \dfrac{1 + \sin\phi}{1 - \sin\phi}$

ϕ = factored value of internal friction angle (degrees)

γ = factored value of soil density (kN/m^3)

H_e = depth of undisturbed soil to underside of bearing pad or wall units as appropriate (m)

Rankine-Bell Method

The Rankine-Bell equation can account for cohesion, but not for the slope of the wall. The direction of the force on the wall is assumed to be the same as the backfill slope.

p_p = passive pressure (kPa)

$\quad = \gamma H_e \tan^2(45 + \phi/2) + 2 c \tan(45 + \phi/2)$

Where:

ϕ = factored value of internal friction angle (degrees)

H_e = depth of undisturbed soil to underside of bearing pad or wall units as appropriate (m)

c = cohesion (kPa)

2.16 Bearing Failure

As soil and water pressure are applied to the rear face of the structure, it will tilt forward and the soil under the toe is subjected to high bearing pressures. Bearing is often the critical mode of failure. The following theoretical approach is used to analyse this region for bearing pressure failure and is based on the Meyerhof method. This gives consideration to footing width, footing tilt and angle of applied load and is explained in a paper by Vesic titled *Bearing Capacity of Shallow Footings* in the Foundation Engineering Handbook.

Q = Bearing capacity of the foundation (kN)

   = $q_{av} L_B$

Where:

q_{av} = average bearing capacity based on factored soil properties (kPa)

  = $c N_c \zeta_c \zeta_{ci} \zeta_{ct} + \gamma H_e N_q \zeta_q \zeta_{qi} \zeta_{qt} + 0.5 \gamma B N_\gamma \zeta_\gamma \zeta_{\gamma i} \zeta_{\gamma t}$

B = actual width of bearing pad on foundation material (m)

L_B = effective width of bearing pad on foundation material (m)

c = factored value of drained cohesion (kPa)

ϕ = factored value of friction angle (radians)

γ = factored value of soil density (kN/m^3)

H_e = depth of undisturbed soil to underside of bearing pad or wall units as appropriate (m)

$N_c = (N_q - 1)\cot\phi$

$N_q = e^{\pi\tan\phi} \tan^2[\pi/4 + \phi/2]$

$N_\gamma = 2(N_q + 1)\tan\phi$

Shape factors:

ζ_c = 1.0 for rectangular bases

ζ_q = 1.0 for rectangular bases

ζ_γ = 1.0 for rectangular bases

Factors for inclined load:

$\zeta_{ci} = \zeta_{qi} - (1 - \zeta_{qi})/(N_c \tan\phi)$

$\zeta_{qi} = [1 - P^*/(Q^* + L_B c \cot\phi)]^2$

$\zeta_{\gamma i} = [1 - P^*/(Q^* + L_B c \cot\phi)]^3$

Factors for sloping bases:

$\zeta_{ct} = \zeta_{qt} - (1 - \zeta_{qt})/(N_c \tan\phi)$ =1.0 for level base

$\zeta_{qt} = (1 - \alpha\tan\phi)^2$ =1.0 for level base

$\zeta_{\gamma t} = (1 - \alpha\tan\phi)^2$ =1.0 for level base

Q^* = vertical load based on factored loads and soil properties

P^* = horizontal load based on factored loads and soil properties

α = angle of base in radians

   = zero for level base

2.17 Sliding Failure

As soil and water pressure are applied to the rear face of the structure, the base of the wall or footing (if constructed) may slide forward. Such sliding action is resisted by the friction and adhesion between the foundation material and the base of the wall or footing, and the passive resistance of any soil in front of the toe.

Select the appropriate external friction angle, δ, that reflects the roughness of the base and its interlock with the foundation material. For a "rough" base, the external friction angle approaches the internal friction angle, ϕ, of the soil. For a normal concrete surface, the external friction angle would approach $^2/_3\,\phi$.

"Rough" bases include:

- Concrete units incorporating cores filled with crushed rock,
- Concrete units with a lip that protrudes into the crushed rock,
- No-fines concrete fill, and
- Where the base blocks are embedded in a concrete foundation.

The designer should consider the validity of these assumptions.

The NCMA (National Concrete Masonry Association [USA]) has published test results that deal with the sliding resistance of concrete blocks on various soils. For smooth bases, with an internal friction angle, $\phi = 35°$, and external friction angle, $\delta = 23.3°$ and cohesion, $c = 0$, the method used in this Guide would yield 0.97 times the resistance calculated using the NCMA method. Although the agreement is not so close for other combinations, their use is infrequent and not recommended.

When considering passive resistance, note that material can be inadvertently removed from the toe of the wall.

F = Sliding resistance based on factored characteristic soil properties

   = Friction + adhesion + passive resistance

   = $Q^* \tan\delta + c B + K_p 0.5 \gamma H_e^2$

Where:

Q^* = vertical load based on factored loads and soil properties

δ = external friction angle of the soil calculated from the factored internal friction angle.)

B = actual base width (m)

c = factored value of adhesion (kPa)

K_p = passive pressure coefficient

γ = factored value of soil density (kN/m^3)

H_e = depth of undisturbed soil to underside of bearing pad or wall units as appropriate (m)

2.18 Overturning

AS 4678-2002 does not specify an analysis method for overturning. This Guide considers overturning about a point level with the underside of the facing units and a nominated distance behind the toe of the facing units. If this nominated distance is one third of the base width and the factor against overturning is calculated as 1.0, this corresponds to the reaction being situated within the middle third of the base at *ultimate loads*.

2.19 Global slip

AS 4678-2002 Clause 3.2 requires stability (including rotation) to be checked.

The design example does not include analysis for global slip.

2.20 Foundation Material

In some cases, weak foundations with low friction angles lead to excessively-high bearing pressures or low resistance to sliding. To avoid this situation, one design option is to remove any material with a low friction angle and replace it with a more suitable material with a characteristic friction angle of at least 35°. Typically, compacted road base would be suitable in such an application. If the foundation soil does not possess suitable properties, it should be excavated and replaced with compacted road base to a depth such that sliding and bearing resistance can be achieved. In all cases, an experienced civil or geotechnical engineer should be engaged to determine the appropriate soil properties.

2.21 Lean Back

Consistent with AS 4678–2002, this guide does not cover the design of revetments with a lean back of 20° or more from vertical.

3 Appendices

The following Appendices are included:

Some Typical Examples of Segmental Concrete Gravity Retaining Walls

Appendix A   Design Example

The following example demonstrates the method used to design a typical segmental concrete masonry gravity retaining wall in accordance with AS 4678 and the design considerations set out in this Guide. It may also be used to check the suitability of commercially-available design software that is intended to be based on AS 4678 and on this Guide.

1   Wall Details

Wall slope
$\omega = 0°$

Backfill slope
$\beta = 15.0°$

Height of stem above soil in front of wall
$H' = 1.2$ m

Live load surcharge
$q_l = 1.5$ kPa

Dead load surcharge
$q_d = 0$ kPa

Height of water table from top of drainage layer
$H_w = 0$ m

NOTE:
W_u and H_u are measured in the horizontal and vertical directions. If the bedding planes are tilted, due allowance must be made for a reduction in these dimensions to values less than actual block dimensions
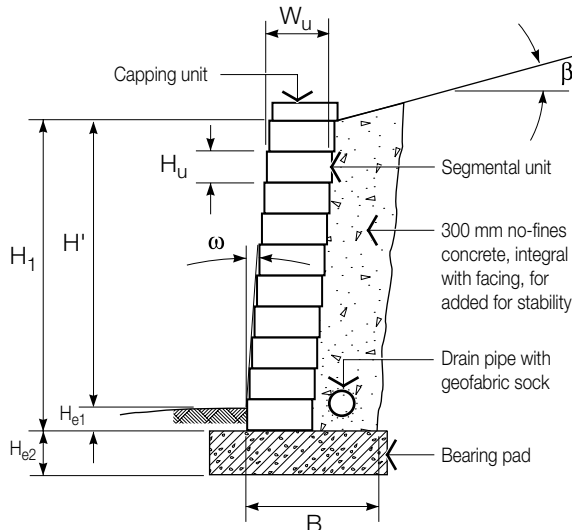
Limits for determining Structure Classification

$$\theta_{tm} = \frac{2\alpha + \phi}{3}$$

$$= \frac{(2 \times 90°) + 29°}{3}$$

$$= 70°$$

$$\theta_b = \theta_{tf}$$

$$= \frac{2\alpha + 3\phi}{5}$$

$$= \frac{(2 \times 90°) + (3 \times 29°)}{5}$$

$$= 53°$$

NOTE: Structures beyond the base limit or beyond the top limits are unlikely to be affected by, or have an affect upon, the structure clasification

There are no major structures located within the limits.
Height of structure < 1.5 m
Structure failure results in minimal damage.
Structure Classification = A

Reduction factor
$\Phi_n = 1.1$
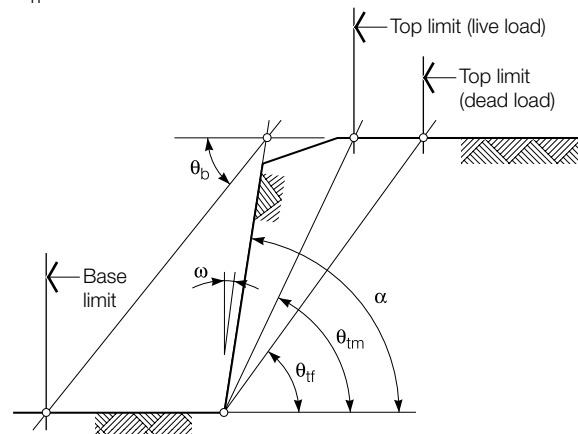
2 Earthquake Considerations

Location
  Sydney

Acceleration coefficient
  $a = 0.08$

Soil profile
  Not more than 30 m of firm clay

Site factor
  $s = 1.0$

Earthquake design category = B_{er}
  ∴ Design for static loads without further
    specific analysis

3 Load Factors

Load factor on overturning dead loads
  $G_{do} = 1.25$

Load factor on overturning live loads
  $G_{lo} = 1.5$

Load factor on resisting dead loads
  $G_{dr} = 0.8$

Load factor on resisting live loads
  $G_{lr} = 0.0$

4 Infill Properties

In order to achieve increased stability, the infill material shall be 300 mm of no-fines concrete, which adheres to the facing units forming a solid mass.

5 Retained Soil Properties

Soil description
  Stiff sandy clay
  Insitu

Characteristic internal angle of friction
  $\phi_r = 29°$

Design uncertainity factor for friction
  $\Phi_{u\phi r} = 0.85$

Design angle of friction
  $\phi_r^* = \tan^{-1}[(\tan \phi_i)\Phi_{u\phi r}]$
    $= \tan^{-1}[(\tan 29°)0.85]$
    $= 25.2°$

Characteristic cohesion
  $c_r = 5.0$ kPa

Design uncertainty factor for cohesion
  $\Phi_{ucr} = 0.70$

Design cohesion
  $c_r^* = c_r \Phi_{ucr}$
    $= 5.0 \times 0.70$
    $= 3.5$ kPa   Assume zero for design

Soil density
  $\gamma_r^* = 19.6$ kN/m^3

Design external friction angle
(soil to concrete interface)

  $\delta_r^* = \dfrac{2 \phi_r^*}{3}$
    $= 16.8°$

6 Foundation Soil Properties

Soil description
  Reconstruct the foundation to improve properties.
  Use crushed sandstone fill
  Controlled fill, Class 2

Characteristic internal angle of friction
  $\phi_f = 35°$

Design uncertainity factor for friction
  $\Phi_{u\phi f} = 0.90$

Design angle of friction
  $\phi_f^* = \tan^{-1}[(\tan \phi_f)\Phi_{u\phi f}]$
    $= \tan^{-1}[(\tan 35°)0.90]$
    $= 32.2°$

Characteristic cohesion
  $c_f = 3.0$ kPa

Design uncertainty factor for cohesion
  $\Phi_{ucf} = 0.75$

Design cohesion
  $c_f^* = c_f \Phi_{ucf}$
    $= 3.0 \times 0.75$
    $= 2.3$ kPa for bearing and zero for sliding

Soil density
  $\gamma_f^* = 18.6$ kN/m^3

7 Bearing Pad Properties

Soil description
  Cement-stabilised crushed rock
  Class 1 controlled fill

Characteristic internal angle of friction
  $\phi_b = 40°$

Design uncertainty factor for friction
  $\Phi_{u\phi b} = 0.95$

Design internal angle of friction
  $\phi_b^* = \tan^{-1}[(\tan \phi_b)\Phi_{u\phi b}]$
    $= \tan^{-1}[(\tan 40°)0.95]$
    $= 38.6°$

Characteristic cohesion/adhesion
  $c_b = 0$ kPa

Design uncertainty factor for cohesion/adhesion
  $\Phi_{ucb} = 0.90$

Design cohesion/adhesion
  $c_b^* = c_b \Phi_{ucb}$
    $= 0 \times 0.90$
    $= 0$ kPa

Density
  $\gamma_b^* = 20.0$ kN/m^3

8 Facing Details

Generic concrete block, 40% solid, filled with concrete, with 300 mm of concrete behind the facing

Height of units
$H_u = 0.2$ m

Width of units
$W_u = 0.3$ m

Width of base (units + concrete)
$B_1 = 0.3 + 0.3$
 $= 0.6$ m

Length of units
$L_u = 0.4$ m

Mass of units
$M_u = 27.0$ kg

Mass of no-fines concrete within units
$M_s = 21.1$ kg

Mass of no-fines concrete behind units
$M_c = 0.2 \times 0.4 \times 0.3 \times 1800$
 $= 43.2$ kg

Mass of units, fill and concrete
$M_{su} = 27.0 + 21.1 + 43.2$
 $= 91.3$ kg

Centre of gravity of units, fill and concrete
$G_u \approx B_1/2$
 $= 0.6/2$
 $= 0.3$ m

Spacing of units
$s_u = 0$

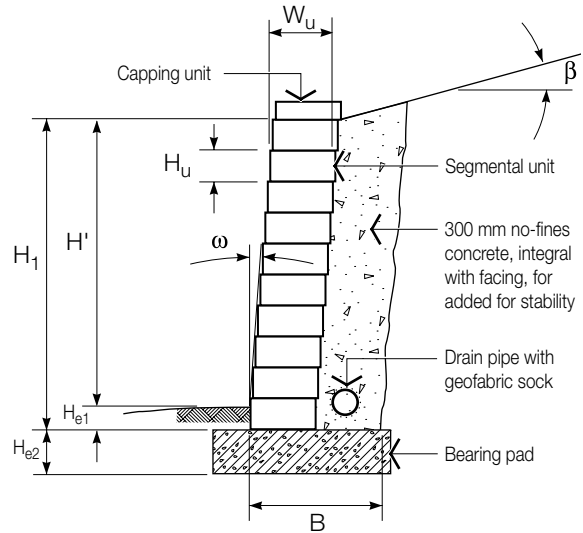Density of composite wall

$$\gamma_{su} = \frac{M_{su}\, g}{H_u\, L_u\, B_1}$$

$$= \frac{91.3 \times 9.81}{0.2 \times 0.4 \times 0.6 \times 10^3}$$
$$= 18.7 \text{ kN/m}^3$$

NOTE:
W_u and H_u are measured in the horizontal and vertical directions. If the bedding planes are tilted, due allowance must be made for a reduction in these dimensions to values less than actual block dimensions


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9 External Stability

Horizontal forces are calculated from the soil surface at the back of the facing units Because there is very little embedment, the passive force applied directly to the facing units is negligible.

For the ultimate stability limit state, provided the foundation reaction is within the base, the structure will not overturn. However, errors in the calculation of the ultimate bearing reaction, and therefore, in the position of the reaction, could lead to an overestimation of the stability. Therefore, it is prudent to nominate a minimum distance from the base of the toe to the reaction.

As a first check, one-third of the base may be used. That is, the centroid of an idealised rectangular foundation reaction is within the middle third of the base at *ultimate limit state*. This is more conservative than the same requirement imposed at *working stress* loads, and can be shown to be too conservative for this type of structure. A more reasonable distance from toe to centroid of reaction is one sixth of the base. This value has been used in these calculations.
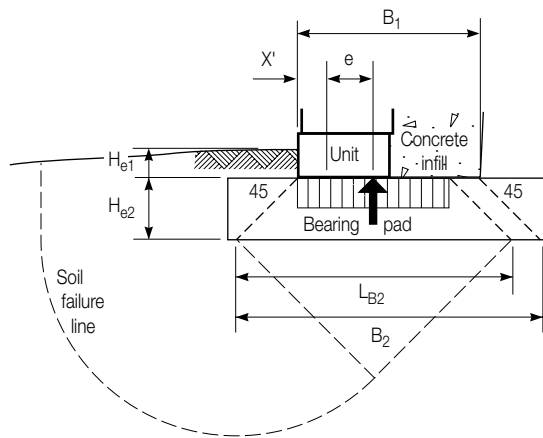
The infill material does not extend into all potential slip planes, therefore, active forces will be generated by the retained soil.

$$X' = \frac{B_1}{6}$$
$$= \frac{600}{6}$$
$$= 100 \text{ mm}$$

Depth to underside of wall units
$$H_{e1} = 0 \text{ m}$$

Depth to underside of bearing pad
$$H_{e2} = 0.15 \text{ m}$$

NOTE:
The soil analysis model used for these calculations is the Coulomb Method. Alternatively, a Rankine-Bell analysis could be used, to account for cohesion of the retained soil, provided the precautions set out in Clause 2.12 of this Guide are observed. See *Alternative Approach Using Rankine-Bell Method and Cohesion to Calculate Horizontal Forces,* on page 20.

Cohesion is difficult to predict, is variable, may change over time, and is dependent on the effectiveness of surface sealing, surface drainage and subsurface drainage. It is recommended that drained and undrained cohesion (as appropriate) should be assumed to be zero for active forces and a very conservative value for bearing capacity. Consideration must also be given to shrink/swell action of clay soils.

10 Active Pressure on Structure

Active pressure coefficient

$$K_{ar} = \frac{\cos^2(\phi_r^* + \omega)}{\cos^2(\omega)\cos(\omega - \delta_r^*)\left[1 + \sqrt{\dfrac{\sin(\phi_r^* + \delta_r^*)\sin(\phi_r^* - \beta)}{\cos(\omega - \delta_r^*)\cos(\omega + \beta)}}\right]^2}$$

$$= \frac{\cos^2(25.2° + 0°)}{\cos^2(0°)\cos(0° - 16.8°)\left[1 + \sqrt{\dfrac{\sin(25.2° + 16.8°)\sin(25.2° - 15°)}{\cos(0° - 16.8°)\cos(0° + 15°)}}\right]^2}$$

$$= 0.464$$

11 Passive Pressure on Structure

Passive pressure coefficient

$$K_{pf} = \frac{1 + \sin \phi_f^*}{1 - \sin \phi_f^*}$$

$$= \frac{1 + \sin 32.2°}{1 - \sin 32.2°}$$

$$= 3.28$$

12 Horizontal Active Force due to Surcharge

Horizontal active force due to surcharge
$$\begin{aligned}P_{qH} &= K_{ar}[(G_{do} \, q_d) + (G_{lo} \, q_l)]H_1 \cos (\delta_r^* - \omega)\\ &= 0.464[(1.25 \times 0) + (1.5 \times 1.5)] \, 1.2 \times \\ &\qquad\qquad\qquad\qquad \cos (16.8° - 0°)\\ &= 1.2 \text{ kN/m}\end{aligned}$$

Lever arm of horizontal surcharge force above base

$$y_{qH} = \frac{H_1}{2}$$
$$= \frac{1.2}{2}$$
$$= 0.6 \text{ m}$$

Overturning moment due to surcharge
$$\begin{aligned}M_{qHO} &= P_{qH} \, y_{qH}\\ &= 1.2 \times 0.6\\ &= 0.72 \text{ kN.m/m}\end{aligned}$$

13 Horizontal Active Force due to Soil

Horizontal active force due to soil
$$\begin{aligned}P_{sH} &= K_{ar} \, 0.5 \, G_{do} \, \gamma_r^* H_1^2 \cos (\delta_r^* - \omega)\\ &= 0.464 \times 0.5 \times 1.25 \times 19.6 \times 1.2^2 \times \\ &\qquad\qquad\qquad\qquad \cos (16.8° - 0°)\\ &= 7.82 \text{ kN/m}\end{aligned}$$

Lever arm of horizontal soil force above base

$$y_{sH} = \frac{H_1}{3}$$
$$= \frac{1.2}{3}$$
$$= 0.4 \text{ m}$$

Overturning moment due to soil
$$\begin{aligned}M_{sHO} &= P_{sH} \, y_{sH}\\ &= 7.82 \times 0.4\\ &= 3.13 \text{ kN.m/m}\end{aligned}$$

14   Horizontal Passive Force

Passive force on underside of wall unit

$$\begin{aligned}P_{p(k+b)H} &= K_{pf}\, 0.5\, G_{do}\, \gamma_f^*\, H_{e1}^2\\ &= 3.28 \times 0.5 \times 0.8 \times 18.6 \times 0^2\\ &= 0 \text{ kN/m}\end{aligned}$$

Lever arm of passive horizontal soil force above base

$$\begin{aligned}y_{pHR} &= \frac{H_{e1}}{3}\\ &= \frac{0}{3}\\ &= 0 \text{ m}\end{aligned}$$

Restoring moment due to passive horizontal soil force

$$\begin{aligned}M_{pHR} &= P_{p(k+b)H}\, y_{pHR}\\ &= 0 \times 0\\ &= 0 \text{ kN.m/m}\end{aligned}$$

15   Weight of Stem Units

Weight of stem units

$$\begin{aligned}P_{uV} &= G_{dR}\, \gamma_{su}\, H\, B_1\\ &= 0.8 \times 18.7 \times 1.2 \times 0.6\\ &= 10.75 \text{ kN/m}\end{aligned}$$

Lever arm on weight stem units

$$\begin{aligned}X_{uV} &= \frac{H\tan\omega}{2} + \frac{B_1}{2} - X'\\ &= \frac{1.2\tan 0°}{2} + \frac{0.6}{2} - 0.1\\ &= 0.2 \text{ m}\end{aligned}$$

Restoring moment due to weight of stem units

$$\begin{aligned}M_{uvR} &= P_{uV}\, X_{uV}\\ &= 10.75 \times 0.2\\ &= 2.15 \text{ kN.m/m}\end{aligned}$$

16   Vertical Active Force due to Surcharge

To calculate the vertical component of soil friction, the factored horizontal force component is multiplied by $\tan(\delta_r^* - \omega)$

That is, this force effectively has a load factor of 1.5 applied, even though it has a restoring effect.

Vertical component of active force due to surcharge

$$\begin{aligned}P_{qV} &= P_{qH}\tan(\delta_r^* - \omega)\\ &= 1.2 \times \tan(16.8° - 0°)\\ &= 0.36 \text{ kN/m}\end{aligned}$$

Lever arm of vertical component of active surcharge

$$\begin{aligned}X_{qV} &= \frac{H\tan\omega}{2} + B_1 - X'\\ &= \frac{1.2\tan 0°}{2} + 0.6 - 0.1\\ &= 0.5 \text{ m}\end{aligned}$$

Restoring moment due to vertical component of active surcharge

$$\begin{aligned}M_{qVR} &= P_{qV}\, X_{qV}\\ &= 0.36 \times 0.5\\ &= 0.18 \text{ kN.m/m}\end{aligned}$$

17   Vertical Active Force due to Soil Weight

To calculate the vertical component of soil friction, the factored horizontal force component is multiplied by $\tan(\delta_r^* - \omega)$

That is, this force effectively has a load factor of 1.25 applied, even though it has a restoring effect.

Vertical component of active force due to soil weight

$$\begin{aligned}P_{sV} &= P_{sH}\tan(\delta_r^* - \omega)\\ &= 7.82 \times \tan(16.8° - 0°)\\ &= 2.36 \text{ kN/m}\end{aligned}$$

Lever arm of vertical component of active soil weight

$$\begin{aligned}X_{sV} &= \frac{H\tan\omega}{2} + B_1 - X'\\ &= \frac{1.2\tan 0°}{2} + 0.6 - 0.1\\ &= 0.5 \text{ m}\end{aligned}$$

Restoring moment due to vertical component of active soil weight

$$\begin{aligned}M_{sVR} &= P_{sV}\, X_{sV}\\ &= 2.36 \times 0.5\\ &= 1.18 \text{ kN.m/m}\end{aligned}$$

18   Base Sliding

Select the appropriate external friction angle, δ, that reflects the roughness of the base and its interlock with the foundation material. For a rough base, the external friction angle approaches the internal friction angle, ϕ, of the soil. For a normal concrete surface, the external friction angle would approach $2/3\,\phi$. The designer should consider the validity of this assumption.

Friction resistance

$$\begin{aligned}P_{fr} &= \Phi_n(P_{uV} + P_{qV} + P_{sV})\tan\phi_b^*\\ &= 1.1(10.75 + 0.36 + 2.36)\tan 38.6°\\ &= 11.8 \text{ kN/m}\end{aligned}$$

Base adhesion

$$\begin{aligned}P_{ba} &= \Phi_n\, B_1\, c_b^*\\ &= 1.1 \times 0.6 \times 0\\ &= 0 \text{ kN/m}\end{aligned}$$

Passive force in front of base and key (if any)

$$\begin{aligned}P_{pH} &= \Phi_n\, P_{p(k+b)H}\\ &= 1.0 \times 0\\ &= 0 \text{ kN/m}\end{aligned}$$

Total sliding resistance

$$\begin{aligned}P_{sR} &= P_{fr} + P_{ba} + P_{pH}\\ &= 11.8 + 0 + 0\\ &= 11.8 \text{ kN/m}\end{aligned}$$

Sliding force

$$\begin{aligned}P_{bH} &= P_{qH} + P_{sH}\\ &= 1.2 + 7.82\\ &= 9.0 \text{ kN/m}\\ &< 11.8 \text{ kN/m} \qquad \text{OK}\end{aligned}$$

NOTE:
Sliding should also be checked at the interface of any soil strata beneath the structure. It is strongly recommended that passive pressure in front of the wall be ignored in design.

19 Overturning

Overturning has been checked about a point that is one-sixth of the base width from the toe and at the level of the underside of the wall units.

Resisting moments

$$M_R = \Phi_n(M_{uVR} + M_{qVR} + M_{sVR})$$
$$= 1.1(2.15 + 0.18 + 1.18)$$
$$= 3.86 \text{ kNm/m}$$

Overturning moments

$$M_O = M_{qHO} + M_{sHO}$$
$$= 0.72 + 3.13$$
$$= 3.85 \text{ kNm/m}$$
$$< 3.86 \text{ kNm/m} \quad \text{OK}$$

20 Bearing at Underside of Bearing Pad

Depth of embedment, $H_e = H_{e2} = 0.15$ m
Actual width of base, $B = B_1 + H_{e2} + H_{e2}$
$$= 0.6 + 0.15 + 0.15$$
$$= 0.9 \text{ m}$$

Ratio of horizontal loads to vertical loads

$$\frac{P_H}{P_V} = \frac{P_{qH} + P_{sH}}{P_{uV} + P_{qV} + P_{sV}}$$
$$= \frac{1.2 + 7.82}{10.75 + 0.36 + 2.36}$$
$$= 0.669$$

Eccentricity

$$e = \frac{B_1}{2} - X' - \left[\frac{M_R - M_o}{P_V}\right]$$
$$= \frac{0.6}{2} - 0.1 - \frac{3.86 - 3.85}{10.75 + 0.36 + 2.36}$$
$$= 0.199 \text{ m}$$

Bearing width of stem on bearing pad

$$L_{B1} = B_1 - 2e$$
$$= 0.6 - (2 \times 0.199)$$
$$= 0.20 \text{ m}$$

Bearing width of bearing pad on foundation

$$L_B = L_{B2} = L_{B1} + H_{e2} + H_{e2}$$
$$= 0.20 + 0.15 + 0.15$$
$$= 0.50 \text{ m}$$

Bearing capacity factors

$$N_q = e^{\pi \tan\phi_f^*} \tan^2(\pi/4 + \phi_f^*/2)$$
$$= e^{\pi \tan 32.2°} \tan^2(\pi/4 + 32.2°/2)$$
$$= 23.8$$

$$N_c = (N_q - 1)\cot \phi_f^*$$
$$= (23.8 - 1)\cot 32.2°$$
$$= 36.2$$

$$N\gamma = 2(N_q + 1)\tan \phi_f^*$$
$$= 2(23.8 + 1)\tan 32.2°$$
$$= 31.2$$

$$\zeta_q = 1.0$$

$$\zeta_{qi} = \left[1 - \frac{P_H}{P_V + L_{B2} c_f^* \cot \phi_f^*}\right]^2$$
$$= \left[1 - \frac{9.02}{13.5 + 0.50 \times 2.3 \times \cot 32.2°}\right]^2$$
$$= 0.173$$

$$\zeta_{qt} = [1 - \alpha \tan \phi_f^*]^2$$
$$= [1 - 0 \tan 32.2°]^2$$
$$= 1.0$$

$$\zeta_c = 1.0$$

$$\zeta_{ci} = \zeta_{qi} - \frac{1 - \zeta_{qi}}{N_c \tan \phi_f^*}$$
$$= 0.17 - \frac{1 - 0.17}{36.2 \times \tan 32.2°}$$
$$= 0.13$$

$$\zeta_{ct} = \zeta_{qt} - \frac{1 - \zeta_{qt}}{N_c \tan \phi_f^*}$$
$$= 1.0 - \frac{1 - 1.0}{36.2 \times \tan 32.2°}$$
$$= 1.0$$

$$\zeta_\gamma = 1.0$$

$$\zeta_{\gamma i} = \left[1 - \frac{P_H}{P_V + L_{B2} c_f^* \cot \phi_f^*}\right]^3$$
$$= \left[1 - \frac{9.02}{13.5 + 0.50 \times 2.3 \times \cot 32.2°}\right]^3$$
$$= 0.07$$

$$\zeta_{\gamma t} = [1 - \alpha \tan \phi_f^*]^2$$
$$= [1 - 0 \tan 32.2°]^2$$
$$= 1.0$$

Average bearing strength capacity*

$$P_{V_{cap}} = \Phi_n L_B[(c_f^* N_c \zeta_c \zeta_{ci} \zeta_{ct}) +$$
$$(\gamma_f^* H_e N_q \zeta_q \zeta_{qi} \zeta_{qt}) + (0.5 \gamma_f^* B' N_\gamma \zeta_\gamma \zeta_{\gamma i} \zeta_{\gamma t})]$$
$$= 19.2 \text{ kN/m}$$

Applied vertical force
$$P_V = 13.5 \text{ kN/m}$$
$$< 19.2 \text{ kN/m} \quad \text{OK}$$

* Note: In the formula for average bearing strength capacity at the underside of the bearing pad, L_B is the effective width at the bearing pad/foundation interface, accounting for eccentricity. B' is the maximum of B and L_B.

Alternative Approach Using Rankine–Bell Method and Cohesion to Calculate Horizontal Forces

Note: See limitations given in Clause 2.12 of this Guide.

Active pressure coefficient (Rankine Method – level backfill)

$$K_{ar} = \frac{1 - \sin\phi_r}{1 + \sin\phi_r}$$
$$= \frac{1 - \sin 25.2°}{1 + \sin 25.2°}$$
$$= 0.402$$

Active pressure coefficient (Rankine Method – sloping backfill)

$$K_{ar} = \frac{\cos\beta[\cos\beta - (\cos^2\beta - \cos^2\phi)^{0.5}]}{[\cos\beta + (\cos^2\beta - \cos^2\phi)^{0.5}]}$$
$$= \frac{\cos 15°[\cos 15° - (\cos^2 15° - \cos^2 25.3°)^{0.5}]}{[\cos 15° + (\cos^2 15° - \cos^2 25.3°)^{0.5}]}$$
$$= 0.464$$

Design active pressure coefficient
$$K_{ar} = 0.464$$

Net pressure at the surface considering surcharge and cohesion

$$P_{at} = K_{ar}(G_{do}q_d + G_{lo}q_l) - 2 c_r^* K_{ar}^2$$
$$= 0.464[(1.25 \times 0) + (1.5 \times 1.5)] -$$
$$2 \times 3.5 \times 0.464^2$$
$$= 0.46 \text{ kPa}$$

Net pressure at the surface considering surcharge and cohesion is negative.

If the net pressure at the surface is positive, tension cracks will not be able to form in the retained soil. This occurs when the surcharge is relatively high and/or the cohesion is relatively low.

In this example, cohesion will be ignored and the retaining wall will be designed for both soil and surcharge using the Coulomb formula.

If the net pressure at the surface is negative, tension cracks may form in the retained soil. The following expression permits the calculation of the depth of tension cracks, allowing for the combined efffects of cohesion and surcharge.

At this depth, surcharge load has been dissipated in overcoming the soil cohesion. That is, the soil (supporting both the surcharge and its own weight) may stand alone to this depth, without the aid of a retaining wall. Below this depth, there is no further need to consider surcharge. Tension cracks in the retained soil may fill with water and exert a pressure on the top part of the wall.

Depth of cracks

$$H_c = \frac{2 c'}{(\gamma K_{ar}^{0.5}) - \dfrac{(G_{do}q_d + G_{lo}q_l)}{G_{do}\gamma}}$$
$$= \frac{2 \times 3.5}{(19.6 \times 0.464^{0.5}) - \dfrac{(1.25 \times 0) + (1.5 \times 1.5)}{1.25 \times 19.6}}$$
$$= 0.432 \text{ m}$$

The active forces due to soil and surcharge act at an angle generally greater than the backfill slope.

It is commonly assumed, in the Rankine Method, that, for cohesionless soils, the direction of force is the same as the backfill slope, β.

It is commonly assumed, in the Rankine Method, that, for cohesive soils, the direction of force is horizontal. It is conservative to assume a horizontal active force.

Therefore, in this analysis, all active forces are assumed to act horizontally for all combinations of friction and cohesion.

Horizontal active force due to water in tension cracks
$$P'_{aw} = 0.5 G_w \gamma_w z_0^2$$
$$= 0.5 \times 1.0 \times 9.81 \times 0.432^2$$
$$= 3.35 \text{ kN/m}$$

Horizontal active force due to soil
$$P'_a = 0.5 K_{ar} G_{do} \gamma_r (H - z_0)^2$$
$$= 0.5 \times 0.464 \times 1.25 \times 19.6(1.200 - 0.471)^2$$
$$= 0.92 \text{ kN/m}$$

Vertical lever arm of horizontal soil load above toe
$$y_{qh} = H/3$$
$$= 1.200/3$$
$$= 0.400 \text{ m}$$

Vertical lever arm of horizontal water load above toe
$$y_{qh} = H - 0.667 H_w$$
$$= 1.200 - (0.667 \times 0.432)$$
$$= 0.400 \text{ m}$$

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Appendix B   Site Investigation

SITE INVESTIGATION

Date: ....

**Report
prepared by:** ....

Client: ....

Project: ....

Location: ....

Use for which retaining wall is intended: ....

Proximity of other structures and loads to the face of the retaining wall:

	Structure or load	*Distance (m)*
Distance of live loads from top of wall	....	....
Distance of dead loads from top of wall	....	....
Distance of other structures from base of wall	....	....

Structure classification: ....

For guidance refer AS 4678, Table 1.1

Structure Classification	*Examples*
C	Where failure would result in significant damage or risk to life
B	Where failure would result in moderate damage and loss of services
A	Where failure would result in minimal damage and loss of access

Required design life: ....

For guidance refer AS 4678, Table 3.1

Type of Structure	*Design life (years)*
Temporary site works	5
Mine structures	10
Industrial structures	30
River and marine structures	60
Residential dwellings	60
Minor public works	90
Major public works	120

Required wall type: ....

Exposed height of retaining wall stem: .... m

Slope of wall: .... 1 horizontal in .... vertical

Slope of backfill: .... 1 vertical in .... horizontal

Specified surcharge loading (if any) or other loads: .... kPa

Soil Properties

Soil	Density (kg/m^3)	Effective internal angle of friction (°)	Cohesion (kPa)	Soil type*
Insitu foundation				
Imported foundation material				
Insitu retained soil				
Infill soil				

* Please indicate the appropriate type(s) and add any other notes.
        Hard rock, sandstone, gravel, sand, silty sand, clayey sand, stiff clay, weak clay, other

Are soil strength tests required? (yes/no) ....

Is there ground water seepage present?   **Now** (yes/no) .... **After heavy rain** (yes/no) ....

                                            If yes, how much? ....

Is it practical to install subsurface drainage (yes/no) .... **and surface drainage** (yes/no)? ....

How will the drainage system affect the site? ....
....

What is the effect of excavation or filling? ....
....

Are there obvious global stability problems? (yes/no) ....

What is the effect of ground movement? ....
....

General description of site topography
(Sketch, site plan, and photographs where possible to be attached).

....
....
....
....

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APPENDIX C
Construction Specification

Supervision

The Contractor shall ensure that the work is performed and directly supervised by appropriately-experienced personnel.

Quality Assurance

Suppliers and contractors shall provide assurance of the quality of all goods, materials and services to be provided. The following are deemed to meet this requirement:

- a quality assurance system complying with AS/NZS ISO 9001, or
- a quality control system approved by the Builder.

Australian Standards

All components and installation shall comply with the Building Code of Australia (BCA) and the relevant Australian Standards, including AS 4678 and the Standards referred to therein.

Safety and Protection of Existing Structures

All excavations shall be carried out in a safe manner in accordance with the relevant regulations, to prevent collapse that may endanger life or property.

In the absence regulations to the contrary, the following may be applied, where:

- excavation is performed and remains open only in dry weather,
- there is no significant ground water seepage,
- the excavation remains open for no longer than two weeks,
- the back-slope of the natural ground does not exceed 1 vertical in 6 horizontal,
- bedding planes do not slope towards the cut, and
- there are no structures founded within a zone of influence defined by a line from the toe of the cut at 30° for cohesionless material and 45° for other material.

In all other cases, the advice of the Engineer shall be sought.

Adjacent structures must be founded either beyond or below the zone of influence. Where there is risk of global slip around a slip plane encompassing the proposed retaining wall or other structures, or where there is risk of inundation by ground water or surface water, retaining wall construction shall not proceed until remedial action has been carried out.

Foundation Material

Foundation material shall be uniform and of the type shown on the drawings.

Preparation of Foundation Material

Where there are significant variations of foundation material or compaction, soft spots or where there is ponding of ground water, the material shall be removed, replaced and compacted in layers not exceeding 150 mm at a moisture content within 2% of Optimum Moisture Content (OMC) to achieve 95% Standard Proctor density.

Trenches and footing excavations shall be dewatered and cleaned prior to placement of drainage material or footings such that no softened or loosened material remains. If necessary place and compact foundation material in layers not exceeding 150 mm to make up levels. The levels beneath the wall shall not be made up with bedding sand or other poorly-graded granular material that may permit ground water to permeate under the base of the retaining wall, except where drainage material is specified and an adequate drainage system is designed.

Drainage System

The drainage system shall consist of:

- Permeable wall facing.
- A permeable drainage layer not less than 300 mm wide adjacent to the stem of the wall.
- A 100-mm slotted PVC agricultural pipe with geofabric sock, or equivalent system, draining to the stormwater system

Natural material	Maximum height of cut (m)	Maximum permissible unpropped batter Vert : horiz
Stable rock, sandstone, firm shale etc where bedding planes do not slope towards the excavation	0 – 3.2	1 : 0.4
	Over 3.2	Seek advice of engineer
Materials with both significant cohesion and friction in its undisturbed natural compacted state	0 – 2.6	1 : 0.8
	Over 2.6	Seek advice of engineer
Cohesive soils, eg clay, silts	0 – 2.0	1 : 1.2
	Over 2.0	Seek advice of engineer
Cohesionless soils, eg Loose gravel, sand	0 – 1.4	1 : 1.6
	Over 1.4	Seek advice of engineer

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Drainage Pipe

The drainage pipe shall be a 100-mm diameter slotted PVC agricultural pipe.

Drainage Fill

Drainage fill material shall be a nominal 10-mm to 20-mm GP (poorly-graded gravel) complying with the following specification.

Specified Grading:

Sieve	Percent Passing
26.50 mm	100
19.00 mm	70–100
13.20 mm	0–100
9.52 mm	0

Installing Drainage Fill

Drainage fill shall be placed around the drainage pipe and up the wall, to a minimum thickness of 300 mm.

Installing the Drainage System

The drainage pipe shall be positioned in the drainage fill at a minimum uniform grade of 1 in 100 over a length not exceeding 15 m. It shall be connected to the storm-water system at the lower end of each run and shall drain positively away from base of the retaining wall. The drainage pipe shall be brought to the surface at the upper end of each run (to facilitate future flushing) capped and its position marked.

Concrete Blocks

Unless specified otherwise, concrete units shall comply with AS 4455 and the following requirements:

- Dimensional category DW4 except for dimensions of mating surfaces requiring tighter tolerances to achieve interlock
- General purpose salt attack resistance grade, except where exposed grade is required
- Minimum characteristic compressive strength of 10 MPa (unconfined value)
- Colour and texture shall be within an agreed range

No-fines Concrete

No-fines concrete infill placed behind retaining walls shall be free-draining allowing water to pass readily through it to the drainage system. In its unhardened state, no-fines concrete shall have low slump and shall not exert a lateral pressure in excess of 4 kPa on the retaining wall facing restraining it. No-fines concrete used to provide enhanced stability to a retaining wall shall have a bulk density not less than 1800 kg/m^3. No-fines concrete shall form a coherent mass, capable of adhering to the facing.

No-fines concrete meeting the following specification is deemed satisfactory for this application.

- Aggregate to GP cement ratio shall not be greater than 6:1 (by volume)
- Aggregate shall be GP (poorly graded) nominal 20 mm crushed rock aggregate (with all particles in the range 12 mm to 20 mm)
- Compressive strength shall be not less than 10 MPa.

Adhesive

The adhesive used to bond the capping units shall be a flexible two-part epoxy-based adhesive.

Installing Concrete Facing Units, Infill Material

Concrete facing blocks shall be installed on the levelling pad or footing such that the resulting wall has a backward slope as specified on the drawings, but not less than 1 in 40. The units of successive courses shall be stacked in stretcher bond. In high walls that are curved in plan, it may be necessary to compensate for joint creep in the upper courses (the longitudinal translation of joints along the wall and the radius of curvature increases or decreases).

Fill material shall be placed, spread and compacted layers equal to the height of the facing units, but not exceeding 200 mm in thickness, at a moisture content within 2% of Optimum Moisture Content (OMC) to achieve 95% Standard Proctor density. Fill material within 1.0 metre of the rear face of the retaining wall facing units shall be placed and compacted by at least three passes of a lightweight, mechanical-plate, tamper or roller at a moisture content within 2% of Optimum Moisture Content (OMC) to achieve 90% Standard Proctor density.

At the end of each day's construction, the fill material shall be sloped such that any rainwater is directed away from the face of the retaining wall and to a temporary (or permanent) drainage system.

The top facing unit or capping unit shall be bonded to the facing units below using an adhesive.

Bulk Fill Material

Bulk fill material shall be uniform and of the type shown on the drawings. The maximum particle size is 100 mm. It is permissible to replace material of a lower design type with properly-compacted material of a higher design category.

Installing Bulk Fill Material

Unless required otherwise to support external loads, bulk filling material shall be placed and compacted behind the drainage material in layers not exceeding 200 mm at a moisture content within 2% of Optimum Moisture Content (OMC) to achieve 85% Standard Proctor density.

Surface Sealing Material

The material used to seal the surface of the fill shall be compacted clay.

Installation of Surface Sealing Material and Catch Drain

The whole of the disturbed fill surface shall be sealed and drained by compacting a layer of surface sealing material at least 150 mm thick and incorporating a 100 mm deep catch drain which drains to the site drainage system at a minimum slope of 1 in 100.

Inspections and Tests

All new work shall remain open until it has been
inspected and approved by the Engineer. The
following inspections shall be performed.

Item or product	Inspection required	Acceptance criteria	Hold/Witness
Foundation:			
Density	Nuclear densometer	–	Hold
Friction angle	Shear box*	–	Hold
Cohesion	Shear box*	–	Hold
Levelling pad:			
Width	Spot check	+ 10%, - 2%	Hold
Depth	Spot check	+ 10%, - 2%	Hold
Density	Nuclear densometer	–	Hold
Friction angle	Shear box*	–	Hold
Cohesion	Shear box*	–	Hold
Masonry units:			
Type	Spot check	As specified	Hold
Dimensions	Spot check	As specified	Witness
Strength	Spot check dockets	As specified	Witness
Drainage system	Visual	As specified	Witness
Granular fill	Visual	As specified	Witness
Geofabric	Visual	As specified	Witness
Surface sealing and surface drains	Visual	Located per drawings	Witness

* If specified by the engineer

Appendix D   Height Table for Walls 800 to 1200 mm High

The Table may be used to determine the permissible height of retaining walls satisfying the following criteria. For retaining walls outside these criteria, the design shall be determined using engineering analysis similar to that shown in the worked example, Appendix A.

This Table is intended for use only by qualified and experienced civil or structural engineers with a comprehensive working knowledge of soil mechanics and structural analysis and design.

The limits of application of the Design Table are as follows:

- All retaining walls are designed to AS 4678 (Including Amendment 1).

- All retaining walls shall comply with AS 4678 Structure Classification A.

- Heights shall be within the range 800 mm to 1200 mm.

- All retaining walls are designed for level backfill. If the backfill has a slope greater than 1 in 8, this Table will not be applicable.

- This Table is only applicable to retaining walls that incorporate an impermeable surface membrane and drainage system such that there can be no ingress of any water into the soil behind the retaining wall.

- Structures that do not incorporate an impermeable surface membrane and drainage system such that there can be no ingress of any water into the soil behind the retaining wall are deemed to be outside the scope of this Table.

- Retained soil shall have a Plasticity Index, PI, less than 20.

- The Table is applicable to cuts in insitu soils. The Table is not applicable to cohesive fill.

- The Table applies to concrete retaining wall units of depth 280, 300 and 320 mm and a combined density of block and granular fill of 1860 kg/m^3 or more.

- All retaining walls are designed for an imposed load of 2.5 kPa. If imposed loads greater than 2.5 kPa are expected, these designs will not be appropriate.

- This Table is based on 0.8 factor on the vertical component of soil friction, for both permanent and imposed soil and surcharge loads.

- The Table applies only where footings consist of at least 5% cement-stabilised crushed rock with a minimum width of 600 mm and minimum depth of 150 mm.

- Before the bottom course is positioned, the footing should be moistened to ensure bond between the block and footing.

- The Table is based on the assumption that, within the above-mentioned limits, the active pressure coefficients, K_a, calculated using the Rankine–Bell Method are conservative for sloping walls.

- Heights in the Table are derived using the Rankine–Bell Method assuming no water in the tension cracks.

- ✖ in the Table means that the permissible height is under 800 mm. Structure Classification A retaining structures under 800 mm high are outside the scope of AS 4678 and reference should be made to Appendix E.

- Commercially-available segmental concrete retaining wall units may vary in mass, shape and dimensions from those shown in the Table. This will result in some variation in the permissible heights.

- Interpolation of the tabulated heights is permissible.

- The Table is only applicable for cohesions and internal friction angles tabulated. Extrapolation is not permitted.

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General Conditions of Permissible Height Table
(See 'Limits of Application', page 26, for full details)
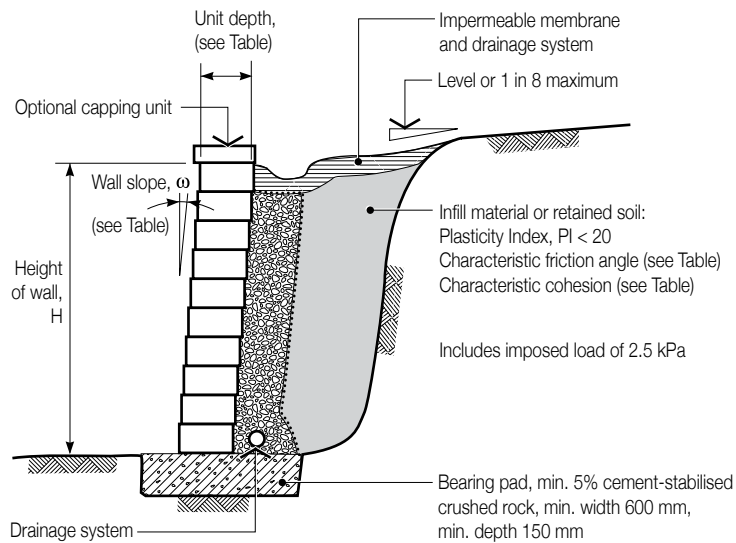
Permissible Height Table

Slope of wall, ω	Height of wall, H (mm)											
	Retained soil/infill material characteristic friction angle											
	35°	30°	25°	20°	35°	30°	25°	20°	35°	30°	25°	20°
	Characteristic cohesion (kPa) of retained soil/infill material											
	4				2				0			
Wall unit depth, 280 mm												
0°	990	890	810	✖	✖	✖	✖	✖	✖	✖	✖	✖
3°	1060	950	850	✖	800	✖	✖	✖	✖	✖	✖	✖
6°	1130	1000	900	810	850	✖	✖	✖	✖	✖	✖	✖
12°	1200	1130	1000	890	970	850	✖	✖	✖	✖	✖	✖
18°	1200	1200	1090	930	1080	890	✖	✖	✖	✖	✖	✖
Wall unit depth, 300 mm												
0°	1040	930	840	✖	✖	✖	✖	✖	✖	✖	✖	✖
3°	1110	990	890	810	850	✖	✖	✖	✖	✖	✖	✖
6°	1180	1050	940	850	910	800	✖	✖	800	✖	✖	✖
12°	1200	1180	1050	940	1040	900	✖	✖	910	800	✖	✖
18°	1200	1200	1140	980	1140	940	✖	✖	960	800	✖	✖
Wall unit depth, 320 mm												
0°	1080	980	880	800	840	✖	✖	✖	✖	✖	✖	✖
3°	1160	1040	930	840	900	800	✖	✖	800	✖	✖	✖
6°	1200	1100	990	890	960	850	✖	✖	850	✖	✖	✖
12°	1200	1200	1100	980	1100	960	830	✖	970	850	✖	✖
18°	1200	1200	1180	1020	1200	1000	820	✖	1030	850	✖	✖

✖ Refer to Appendix E.
 NOTE: Irrespective of the unit depth, cohesion or friction angle, wall heights shall not exceed 1200 mm.

Appendix E   Details for Walls up to 800 mm high

This appendix may be used to determine the permissible height of retaining walls satisfying the following criteria. For retaining walls outside these criteria, the design shall be determined using engineering analysis similar to that shown in the worked example, **Appendix A,** by qualified and experienced civil or structural engineers with a comprehensive working knowledge of soil mechanics and structural analysis and design.
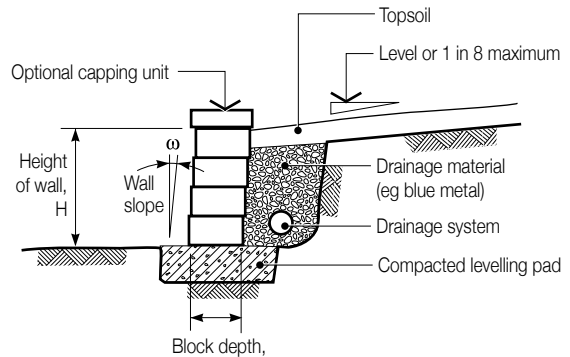
The limits of application of this Appendix are as follows:

- All retaining walls shall comply with the conditions of Structure Classification A of AS 4678.

- Heights, excluding any capping blocks, shall not exceed 800 mm.

- All retaining walls are designed for level backfill. If the backfill has a slope greater than 1 in 8, this Appendix will not be applicable.

- This Appendix does not apply to terraced retaining walls.

- This Appendix is not suitable for imposed loads. If imposed loads are expected, the retaining wall should be designed by engineering analysis similar to that in the worked example, **Appendix A**.

- This Appendix is not suitable for situations of excessive water run-off.

- This Appendix applies to retaining walls with a compacted levelling pad.

- This Appendix applies to concrete units of depth not less than 125 mm, with an interlocking mechanism and a combined density of block and granular fill of 1860 kg/m^3 or more.

Structure Classification A retaining structures under 800 mm high are outside the scope of AS 4678.

Based on industry experience, the following ratios have been adopted.

Slope of wall, ω	Wall height to block depth ratio, H:D
Vertical to 10°	3:1
Over 10°	4:1

Typical Wall Detail

CMAA

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