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CONCRETE MASONRY
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RW02

Concrete Masonry –
Reinforced Soil
Retaining Walls



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Segmental Concrete Reinforced Soil Retaining Walls – Design and Construction Guide

Concrete Masonry Association of Australia

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Preface

This guide is a revised version of MA50–2002.

Standards Australia has published AS 4678^(Ref 1) for the design and construction of earth retaining structures, including segmental concrete reinforced soil retaining walls. This standard encompasses the following features:

- Limit state design that enable separate consideration of stability, strength of components and serviceability
- Partial load factors and partial material factors that permit the uncertainty and risk associated with each of the loads and materials to be assessed and taken into account
- Compatibility with AS 1170 SAA Loading code^(Ref 2)
- Compatibility with the structures standards such as AS 3600 Concrete structures^(Ref 3) and AS 3700 Masonry structures^(Ref 4).

This guide provides a comprehensive approach to the design of segmental concrete reinforced soil retaining walls based on:

- The design and construction rules set out in AS 4678
- An analysis method developed in the United States and published by the National Concrete Masonry Association (NCMA)^(Ref 5), and modified in part by the Concrete Masonry Association of Australia (CMAA) to fit Australian practice and the Australian Standard.

The scope of this guide is limited to the design of reinforced soil structures up to 6 metres high, consisting of concrete segmental facing units and geosynthetic grids, with a maximum wall slope of 15° from vertical. This guide does not apply to seawalls, water-retaining structures, unusual ground conditions (such as soft ground, land slips, steep sides or deeply inclined gullies) or to walls subject to sustained cyclic loading.

The guide includes:

- A description of the principal features of the Australian Standard
- A description of the analysis method
- A comprehensive site investigation check list
- Design examples which demonstrates the use of the Australian Standard and analysis method.

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1 Introduction

1.1 General

For many years reinforced concrete masonry cantilever retaining walls have been constructed with reinforced concrete masonry stems (steel reinforcement grouted into hollow concrete blockwork) and reinforced concrete footings.

Segmental concrete gravity retaining structures, consisting of concrete units dry-stacked against a soil slope and resisting overturning by virtue of their own weight, were introduced into Australia in the 1980s and rapidly became popular during the early 1990's. This system provides very attractive embankment finishes, but its stability is limited by the geometry of the units and wall heights.

A revolutionary development during the 1990's has been the incorporation of geogrids into the soil mass behind the structure to create segmental concrete reinforced soil structures. Such systems can be constructed several metres high and accommodate significant loads.

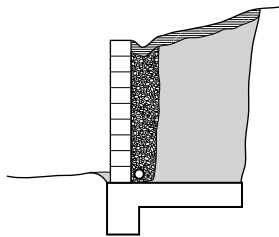


Figure 1.1 Reinforced Concrete Masonry Cantilever Retaining Walls

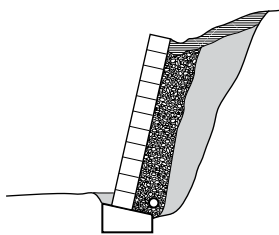


Figure 1.2 Segmental Concrete Gravity Retaining Walls

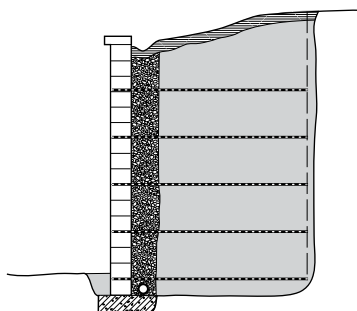


Figure 1.3 Segmental Concrete Reinforced Soil Retaining Walls

1.2 Glossary

Loads and limit states:

Design life

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

Dead load

The self-weight of the structure and the retained soil or rock.

Live load

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 facing units

Concrete blocks manufactured to provide an attractive, durable, stable face to a retaining wall. They commonly interlock or are connected by pins or connectors, and include provision for the securing of geogrids.

Geogrid

Layers of metal or plastic material, which when constructed in horizontal planes in a soil mass, strengthen the soil. The most common geogrids are open "mesh" consisting of polyester, high-density polyethylene, polypropylene or steel.

Geotextile

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

Backfill 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 behind the retaining wall facing and strengthened by the geogrids.

Drainage material

The crushed rock, gravel or similar material placed behind a retaining wall to convey groundwater 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 65% 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.

In situ 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 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 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 to 60 mm, and of a single size. This material has good drainage properties provided it is protected from infiltration by silts and clays.

1.3 Behaviour of Segmental Concrete Reinforced Soil 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. It is then restrained by the friction and adhesion between the base and soil beneath, passive soil pressures in front of the structure (usually ignored) 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.

Reinforced soil systems consist of a series of horizontal geogrids that have been positioned and pulled tight in a compacted soil mass, thus strengthening it and restricting its slump. The geogrids are strategically placed to intersect potential failure planes that are inclined from near the base of the wall, up at an angle (depending on the soil properties), to the top of the fill. The function of the geogrids is to "strengthen" the soil mass and they are "anchored" beyond the potential failure planes.

Local collapse and erosion of the front face is eliminated by fixing concrete segmental facing units to the exposed ends of the geogrids. However, the segmental concrete facing is not designed to "retain" the strengthened soil mass, which should be able to stand independently of the facing except for local effects. The connection spacing (and the geogrid spacing) must account for the local stability of the facing, including bulging and rotation above the top geogrid. The top course is normally bonded to the course below using epoxy cement.

A surface sealing layer and surface drainage system minimise the quantity of rainwater entering the soil mass. A sub-surface drainage system adjacent to the segmental concrete facing and (sometimes) beneath the wall reduce pore water pressures and thus reduce the tendency for local or global slip.

Thus, the essential features of a properly designed and constructed segmental concrete reinforced soil retaining wall are:

- Geogrids with adequate strength and anchorage
- Adequate connection to the facing to provide local stability
- A drainage system that will relieve pore water pressures for the life of the structure.

1.4 Importance of a Geotechnical Report

The design of a reinforced soil retaining wall includes two essential parts:

- Analysis of the proposed reinforced soil structure and the adjacent ground for global slip, settlement, drainage and similar global considerations; and
- Analysis and design of the reinforced soil structure itself.

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.

1.5 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.

1.6 Global slip failure

Soil retaining structures must be checked for global slip failure around all potential slip surfaces or circles.

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 a surfaces encompassing all or some of the retaining walls (Figure 1.4).

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.

1.7 Differential Settlement

Localised post-construction differential settlement should be limited to 1% of the height of the wall (Figure 1.5). However, it may be preferable to limit settlement to a lower figure, giving consideration to aesthetics (ie keeping the bedding planes level), in addition to the structural considerations

Techniques to reduce or control the effects of differential settlement include:

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

1.8 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.

1.9 Geogrid Spacing

Horizontal geogrids placed in the compacted infill soil serve to strengthen it, and should be located at centres not exceeding 600 mm (Figure 1.6).

The top section of the facing (above the top geogrid) should be stable. This can be achieved by:

- Placing the top geogrid at a depth of 300 to 400 mm below the top of the wall (excluding allowance for the capping block, if used), or
- Tying the top of the wall to some other stable structure (eg concrete pavement) placed some distance from the face of the wall.

1.10 Passive Pressure

In some circumstances, passive pressure could contribute marginally 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.

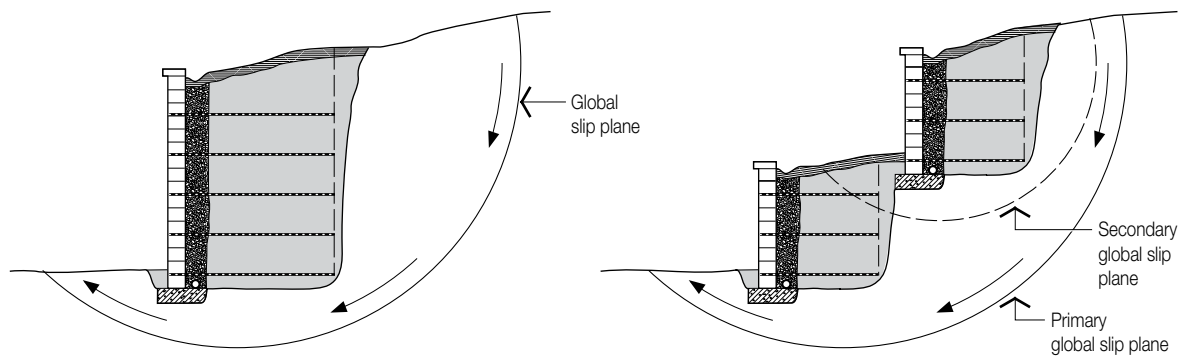


Figure 1.4 Global Slip Failure

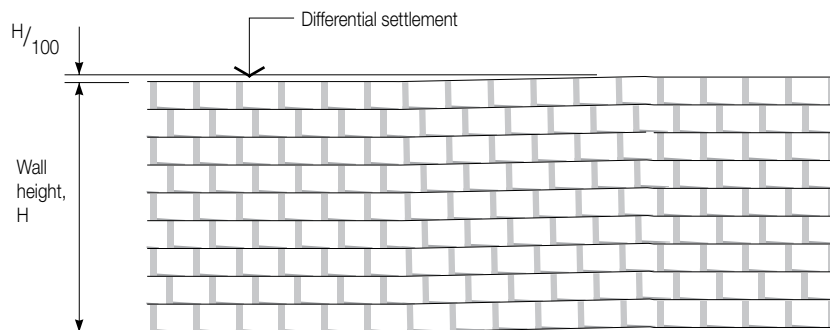


Figure 1.5 Differential Settlement

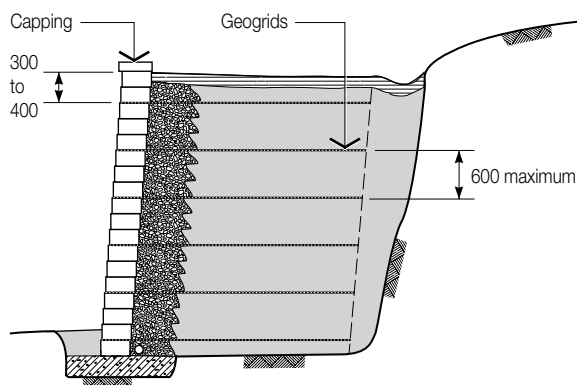


Figure 1.6 Geogrid Spacing

2 Components

A brief description of the principal components of segmental concrete reinforced soil retaining walls is set out below. The construction specification in Appendix D provides detailed specifications for each component

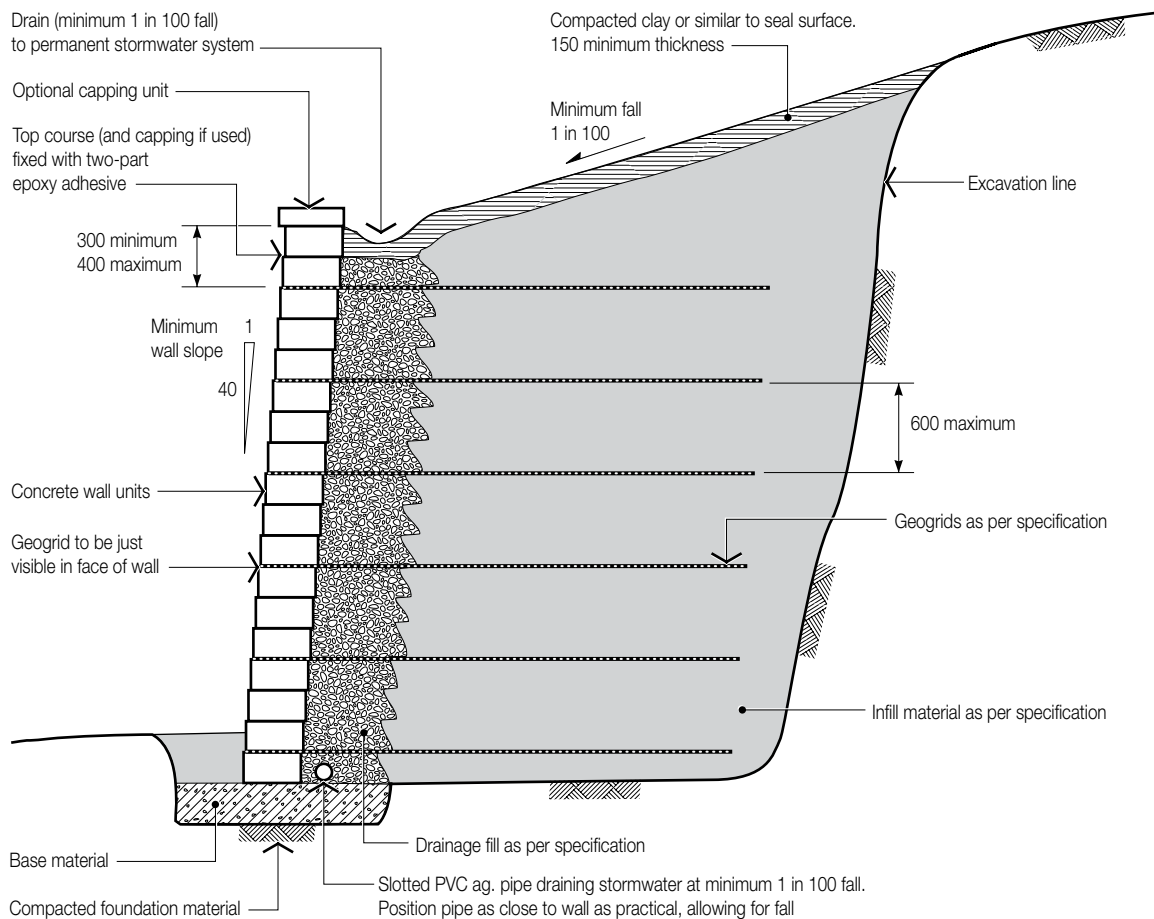


Figure 2.1 Typical Components of Reinforced Soil Retaining Wall

2.1 Drainage System

The drainage system consists of:

- A permeable wall facing system of segmental concrete units;
- A permeable drainage layer not less than 300 mm wide adjacent to the stem of the wall;
- A slotted PVC agricultural pipe, with geofabric sock if appropriate, or equivalent system, draining to the storm water system;
- A catch drain capable of removing surface water from the top of the embankment. The base of the wall must also be adequately drained; and
- A surface sealing layer that prevents the ingress of surface water into the fill behind the wall.

Drainage fill placed immediately behind the wall permits any ground water to percolate to the base of the wall where it is removed by the drainage pipe.

Drainage fill material should be:

- a single-sized gravel or crushed rock in the range of 10 to 20 mm, designated GP, or
- a well-graded gravel, designated GW, with a minimum particle size not less than 5 mm.

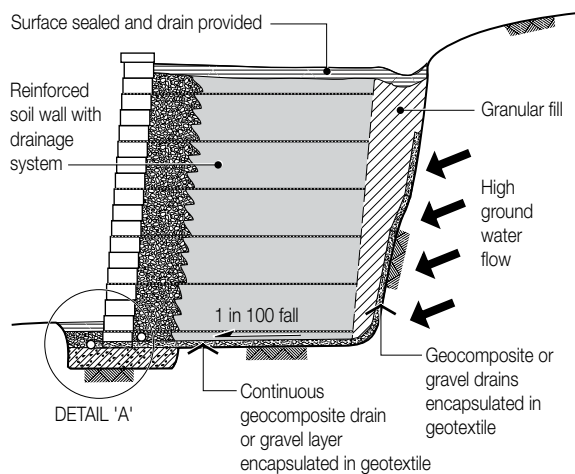
It is important that the drainage fill be free-draining, particularly in the lower parts of the wall. It should be positioned such that it delivers the water at the level of the drainage pipe, which must slope along the length of the wall.

To minimise the effect of clogging, the drainage pipe should be positioned in the drainage fill at a minimum uniform grade of 1 in 100. The pipe should be capable of removing the volume of water that may be present. The agricultural pipe should be connected to a PVC stormwater pipe and brought through the front face of the wall at intervals not exceeding 30 m. It should be connected to the storm-water system at the lower end

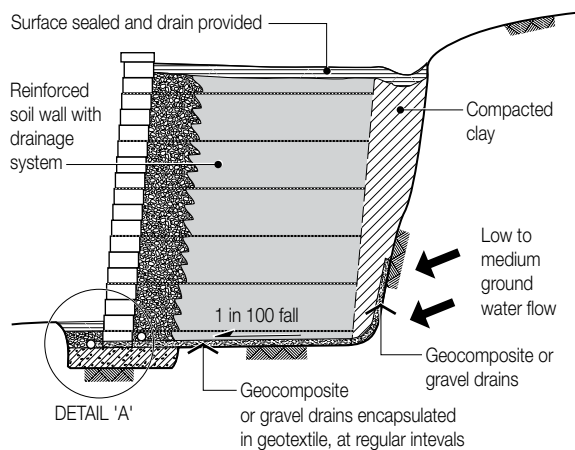
of each run, where practical, and must drain positively away from base of the retaining wall.

The drainage pipe should be brought to the surface of the backfill at the upper end of each run to facilitate future flushing. It should be capped and its position marked.

The whole of the disturbed fill surface should be sealed by at least 150-mm of compacted clay and properly drained. Alternative means, such as bentonite layers or PVC membranes may be employed, provided they do not introduce potential slip planes into the surface material.



(a) SITES WITH HIGH GROUND WATER FLOW



(b) SITES WITH LOW TO MEDIUM GROUND WATER FLOW

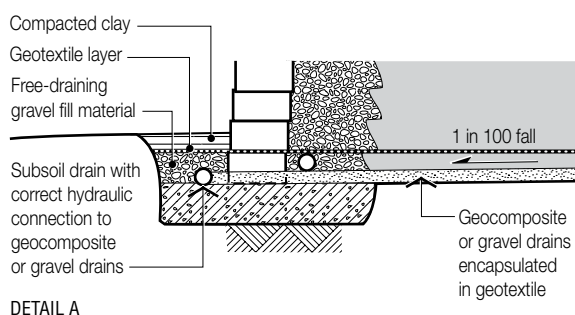


Figure 2.2 Sub-Soil Drainage Systems

2.2 Concrete Facing Blocks

Concrete facing units must be such that:

- They interlock with each other to provide a stable facing.
- They interlock with the geogrids, or alternatively incorporate pins or other means of engaging the geogrids.
- They are manufactured within tolerances such that the interlock can be achieved without distorting the face pattern.
- They have sufficient strength to resist cracking in areas of minor differential settlement.
- They are resistant to deterioration under the action of salts and ground water.
- They are of a shape, size and mass that corresponds to those tested for connection strength and interface shear.

2.3 Reinforced Infill Soil

Reinforced infill material, ie the fill that is strengthened by the geogrids, should not contain large or sharp material that will damage the geogrids. It must also be capable of being fully compacted to form a solid mass reinforced by the geogrids. Well-graded gravel (GW) or well-graded sand (SW) is recommended.

2.4 Geogrids

The long-term strength and elongation of various geogrids depends on the material type and size. The design calculations also depend on the long-term test data that is available. Therefore the geogrids must be of the type and index strength nominated by the designer, and substitutions must not be made without the approval of the designer.

Geogrids must be a single length in the direction of design tension (ie into the embankment), not lapped, making provision for connection to the facing across the whole width of the facing and providing for the specified anchorage within the in designated anchorage zone.

Geogrids must cover the whole of the plan area behind the wall for the specified anchorage length and shall be lapped with adjacent sections in accordance with the manufacturer's instructions. In the absence of manufacturer's instructions, the overlying geogrids should be separated from the geogrid below by 100 mm of infill soil to prevent them from sliding over each other.

Commercially-available geogrids are, polyester, high-density polyethylene, polypropylene or steel.

2.5 Adhesive

The adhesive used to bond the capping units and/or top-course units shall be capable of long-term adhesion in heat and wet conditions. A flexible two-part epoxy-based adhesive is recommended.

3 Design and Analysis Considerations

3.1 Limit State Design

The following design limit states should be considered:

- stability of the structure as a whole subject to ultimate factored loads,
- strengths of the various components subject to ultimate factored loads,
- serviceability of the structure and its components (including differential settlement and forward sliding and rotation) 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 geogrid, foundation soil and other components as appropriate.

3.2 Partial Loading and Material Factors

AS 4678^(Ref 1) provides partial load factors and partial material factors to be applied to characteristic loads and characteristic properties of various materials and components. These partial factors permit the uncertainty and risk associated with each of the loads and materials to be assessed and taken into account in the design.

The standard also provides rules for the combination of these factored loads and materials for separate limit states covering stability, strength of components and serviceability. These combinations are compatible with AS 1170^(Ref 2) (except where indicated otherwise)^(Note 1) and are compatible with the structures standards such as AS 3600^(Ref 3) and AS 3700^(Ref 4). However, some factors are not identical to their counterparts in AS 1170, for example, hydraulic loads and the means of combining soil properties to derive a dead load. These are discussed in more detail below.

3.3 Load Combinations and Factors for Stability of the Structure

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

External stability:

- Global slip
- Overturning
- Bearing capacity of the foundation under the toe of the base
- Sliding resistance of the foundation under the base

Internal stability:

- Internal sliding resistance within the reinforced soil mass
- Bulging resistance of the facing between the geogrids
- Anchorage of the geogrids within the soil mass beyond any potential failure plane
- Connection strength of the facing to the geogrids.
(Note 2)

- (i) $1.25 G^C + 1.5 Q^C < 0.8 G^R + (\Phi R)$
- (ii) $1.25 G^C + \psi_c Q^C + W^C_u < 0.8 G^R + (\Phi R)$
- (iii) $1.25 G^C + \psi_c Q^C + 1.0 F^C_{eq} < 0.8 (G + \psi_c Q)^R + (\Phi R)$

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 retained soil block, thus tending to cause forward sliding, bearing failure, or overturning, or the weight of the infill soil.

Q^C = parts of the live load tending to cause instability.

This includes all removable loads such as live loadings applied from adjacent buildings an allowance for the temporary stacking of soil of not less than 5 kPa. Where a live load can be applied on the retained soil, but not on the infill, the resulting active pressure will tend to cause overturning, but the gravity load will not resist overturning. For example, a road pavement may be placed on the backfill, but not fully on the infill. In this case, the appropriate factors should be applied.

W^C_u = parts of the wind load tending to cause instability. If the wind load is applied to some supported structure such as a

NOTES:

- 1 At the time of publication of this Guide, Standards Australia is preparing a revised loading standard. When that standard is published, it will be necessary to re-examine AS 4678 and this Guide for compatibility with any loads and load factors.
- 2 Design for bearing capacity, external sliding resistance, internal sliding resistance, bulging resistance and anchorage all involve the factoring down of the soil properties (density, friction angle and/or cohesion) which are providing the resistance to instability. Design for connection strength involves the factoring down of the facing material weight (and thus friction resistance) which is assumed to be the principal property resisting disengagement of the connections.

building or a fence, the effect could be significant.
However, for the case of wind on only the face of the wall, the factors are such that load combination (iii) 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 $Q^C = 5 \text{ kPa}$, an active pressure coefficient of $K_a = 0.3$ and a live load combination factor of $\psi_c = 0.6$, a wind suction load on the face of the retaining wall less than 1.35 kPa will not be the governing case.

- 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) is used and specific design for earthquake is not required.
For earthquake categories D_e and E_e , the structures should be designed and analysed in accordance with the detailed method set out in Appendix I of AS 4678
- 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. It is common to exclude consideration of passive pressure.
- Φ^R = the factored design capacity of the structural component.
This includes bearing capacity, sliding resistance, 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.

In addition to soil retained behind the structure, stacked materials, additional soil and vehicles may exert pressures on the wall. AS 4678 requires a minimum live load surcharge of 5 kPa .

The distribution of live loads and the corresponding load factors should be considered carefully by the design engineer.

Since these are matters that go to the basis of AS 4678 and AS 1170.1, it is not appropriate for this Guide to make recommendations, apart from suggesting that the following questions be considered:

- Does the live load factor for the particular load case include provision for variation in the placement of the live load?
- For external stability analysis, should a live load be placed only on the retained soil and omitted from the infill material? In such a case, are the load factors given in AS 4678 appropriate?
- Alternatively, may a uniform live load be placed across both the retained soil and infill material? If so, what are the appropriate load factors?

3.4 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. This includes analysis for:

- Tensile strength of the geogrids
 - Strength of any associated components.
- (i) $1.25G + 1.5Q$
 - (ii) $1.25G + W_U + \psi_c Q$
 - (iii) $1.25G + 1.0F_{eq} + \psi_c Q$
 - (iv) $0.8G + 1.5Q$
 - (v) $0.8G + W_U$
 - (vi) $0.8(G + \psi_c Q) + 1.0F_{eq}$

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.

See further explanation in **Clause 3.3**.

3.5 Capacity Reduction Factors

The material strength factors from AS 4678^(Ref 1) have been used, as follows.

Partial Factors on Soil Properties

Partial factors on $\tan \phi$, $\Phi_{\tan \phi}$	Strength	Service
For Class 1 controlled fill	0.95	1.00
For Class 2 controlled fill	0.90	0.95
For uncontrolled fill	0.75	0.90
For in-situ natural soil	0.85	1.00

Partial factors on cohesion, Φ_c	Strength	Service
For Class 1 controlled fill	0.90	1.00
For Class 2 controlled fill	0.75	0.85
For uncontrolled fill	0.50	0.65
For in-situ natural soil	0.70	0.85

Partial Factors on Geogrid Strength

Geogrids supplied by major manufacturers are subject to extreme testing which provides information on their long-term behaviour. In many cases, manufacturer's data may be used to determine the appropriate factors in preference to the ranges suggested in the Appendix J of AS 4678.

Product uncertainty factor, Φ_{up}	
Guaranteed minimum strength	1.00
Guaranteed characteristic strength	0.95

Creep reduction factor, Φ_{rc}	
Polyester:	
30 year service life	0.60
100 year service life	0.50
Polyethylene:	
30 year service life	0.33
100 year service life	0.30
Polypropylene:	
30 year service life	0.20
100 year service life	0.17

Extrapolation uncertainty factor, Φ_{ue}	
No extrapolation	1.00
1 log cycle of extrapolation	1.00
Polyester:	
1.5 log cycles of extrapolation	0.95
2 log cycles of extrapolation	0.90
Polyethylene:	
1.5 log cycles of extrapolation	0.80
2 log cycles of extrapolation	0.60
Polypropylene:	
1.5 log cycles of extrapolation	0.75
2 log cycles of extrapolation	0.50

Construction damage factor, Φ_{ri}	
In fine sand	0.90 to 0.80
In coarse gravel	0.90 to 0.60
(Note: For some types of geogrid in coarse gravel, a lower factor may be required. See AS 4678, Table K4)	

Thickness reduction factor, Φ_{rt}	
	1.00 to 0.90

Strength reduction factor, Φ_{rs}	
	0.90 to 0.50

Temperature reduction factor, Φ_{rst}	
	To be determined

Degradation factor, Φ_{ud}	
	0.80

Partial Factors on Soil/Geogrid Interaction

Sliding uncertainty factor, Φ_{slide}	
Controlled fill (Class 1 or 2)	0.80

Pullout uncertainty factor, Φ_{upull}	
Controlled fill (Class 1 or 2)	0.80
Natural or insitu soil	0.75

Partial Factors on Geogrid Connection

Connection uncertainty factor, Φ_{ucon}	
	0.75

Partial Factors on Structure Classification

Structure classification factor for ultimate limit state, Φ_{rl} , where failure:

C would result in significant damage or risk to life	0.9
B would result in moderate damage and loss of services	1.0
A would result in minimal damage and loss of access	1.1

3.6 Analysis Assumptions

The analysis method set out in this Guide is based generally on the method published by the National Concrete Masonry Association (USA)^(Ref 5), except were noted below. The principal differences are:

- Adoption of the limit state approach set out in AS 4678
- Adoption of the partial load factors and partial materials factors set out in AS 4678
- Modification of the bearing capacity formula to account for lateral loading due to earth pressure.

3.7 Foundation Properties and Soil Model

Segmental concrete reinforced soil retaining walls should be founded on undisturbed material which is firm and dry and achieves the friction angle and cohesion assumed in the design.

It will be necessary to carry out foundation stabilisation, drainage or other remedial work if the foundation material exhibits any of the following features:

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

Soil model.

AS 4678 does not specify an analysis method. This Guide uses the Coulomb Method to analyse the structure.

3.8 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. 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).

$$p_a = \text{active pressure on the wall at depth of } H \\ = K_a \gamma H$$

Where:

$$K_a = \text{active pressure coefficient} \\ = \frac{\cos^2(\phi + \omega)}{\cos^2 \omega \cos(\omega - \delta) \left[1 + \frac{\sin(\phi + \delta) \sin(\phi - \beta)}{\cos(\omega - \delta) \cos(\omega + \beta)} \right]^2}$$

ϕ = factored value of internal friction angle (degrees)

δ = external friction angle (degrees)

$$= \frac{2\phi}{3} \text{ at interface between soil and facing}$$

or, at interface between two soil layers, the smaller of the friction angles

or, at any interface with filter fabric, 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)

3.9 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. The common practice of assuming zero for passive pressure with reinforced soil structures has been adopted in this Guide.

$$p_p = \text{passive soil pressure (kPa)} \\ = K_p \gamma H_e$$

Where:

$$K_p = \text{passive pressure coefficient} \\ = \frac{1 + \sin \phi}{1 - \sin \phi}$$

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 base or wall units as appropriate (m)

3.10 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. The following theoretical approach is based on a Meyerhof shear failure formula. 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^(Ref 6).

$$Q = \text{Bearing capacity of the foundation (kN)} \\ = q_{av} L_B$$

Where:

$$q_{av} = \text{average bearing capacity based on factored soil properties (kPa)} \\ = c N_c \zeta_c \zeta_{ci} \zeta_{ct} + \gamma H_e N_q \zeta_q \zeta_{qi} \zeta_{qt} + 0.5 \gamma B N_\gamma \zeta_\gamma \zeta_{\gamma i} \zeta_{\gamma t}$$

B = length of reinforced soil block

e = eccentricity of vertical loading

L_B = design base width (m) based on the Meyerhof approach to account for eccentric load
 $= B - 2e$

c = factored value of drained cohesion (kPa)

ϕ = factored value of friction angle (degrees)

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

H_e = depth of undisturbed soil to the underside of the base 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:

$$\zeta_c = 1.0 \text{ for rectangular base}$$

$$\zeta_q = 1.0 \text{ for rectangular base}$$

$$\zeta_\gamma = 1.0 \text{ for rectangular base}$$

Factors for inclined load:

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

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

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

Factors for sloping bases:

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

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

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

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

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

α = angle of base tilt in radians
= zero for level base

3.11 Sliding Failure

As soil and water pressure are applied to the rear face of the structure, the structure may slide forward. Such sliding action is resisted by the friction between the foundation material and the structure and the passive resistance of any soil in front of the toe or is subjected to high bearing pressures. 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.

$$\begin{aligned} F &= \text{Sliding resistance based on factored} \\ &\quad \text{characteristic soil properties} \\ &= \text{Friction} + \text{adhesion} + \text{passive resistance} \\ &= Q^* \tan \delta + c B + K_p 0.5 \gamma H_e^2 \\ &= Q^* \tan \delta \end{aligned}$$

(In this Guide, passive resistance, base adhesion and cohesion are taken as zero. The sliding resistance of the units over the bearing pad material may differ from the sliding resistance of the infill material over the bearing pad or foundation, but the difference is generally small and its effect on total sliding resistance is usually neglected. The designer should consider the validity of this approximation.)

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, assuming a smooth base-to-soil interface (if a rough base-to-soil interface is present, a friction angle of ϕ may be used)

B = actual base width (m)

c = factored value of drained cohesion (kPa)

K_p = passive pressure coefficient

γ = factored value of soil density (kN/m³)

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

3.12 Wall Slope

This Guide does not cover the design of revetments or walls with a lean back of more than 20° from vertical.

3.13 Backfill Slope

The designer should consider the stability of the slope of any backfill placed behind and above the wall. The slope should not exceed an angle whose tangent is given by dividing the tangent of the design friction angle by an appropriate factor.

3.14 Overturning

AS 4678 does not specify an analysis method for overturning.

This guide considers overturning about the toe of the structure (which could be some distance below the finished soil level at the toe). It allows for a sloping wall and sloping fill. Because the base materials are not rigid, there can be no upward movement of the heel as so-called "rotation" about the toe commences. Therefore, there is no development of friction at the rear of the soil mass.

3.15 Tensile Strength of the Geogrids

AS 4678 does not specify an analysis method. Design is based on the force calculated using the average pressure at the midpoint of the contributory area based on measurements vertically down from the top of the wall.

3.16 Anchorage of the Geogrids Within the Soil Mass Beyond all Potential Failure Plane

AS 4678 does not specify an analysis method. Design is based on a minimum of 300-mm anchorage length beyond the failure plane, drawn from the heel of the bottom unit. An overall lower limit on the length of geogrid, measured from the front face of the wall, is set at 0.7 times wall height.

3.17 Internal Sliding Resistance Within the Reinforced Soil Mass

AS 4678 does not specify an analysis method. Design is based on the resistance to slip calculated along lengths defined within the reinforced soil mass.

3.18 Connection Strength of the Facing to the Geogrids

AS 4678 requires that the connection of the bottom geogrid to the facing be designed for 100% of the load in the corresponding geogrid and the top connection be designed for 75%.

3.19 Bulging Resistance of the Facing Between the Geogrids

AS 4678 does not specify an analysis method. Design is based on sliding due to incremental shear forces between reinforcement grids. Localised bending is not a problem, because the vertical component of soil force resists the uplift of the units necessary to allow this type of failure to occur.

3.20 Facing Unit Strength

Concrete blocks should have a characteristic unconfined compressive strength, f'_{UC} , of at least 10 MPa in accordance with AS 4456.4^(Ref 7), and calculated in accordance with AS 3700 Appendix B, to ensure that there is sufficient integrity to tolerate minor movement.

3.21 Cohesion

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. This Guide also recommends that a very conservative value of cohesion should be assumed when determining the bearing capacity.

- 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.

4 Design Procedure

Set out below is a suitable procedure for designing reinforced soils retaining walls. Appendices B and C include a worked example demonstrating typical calculations for two particular walls.

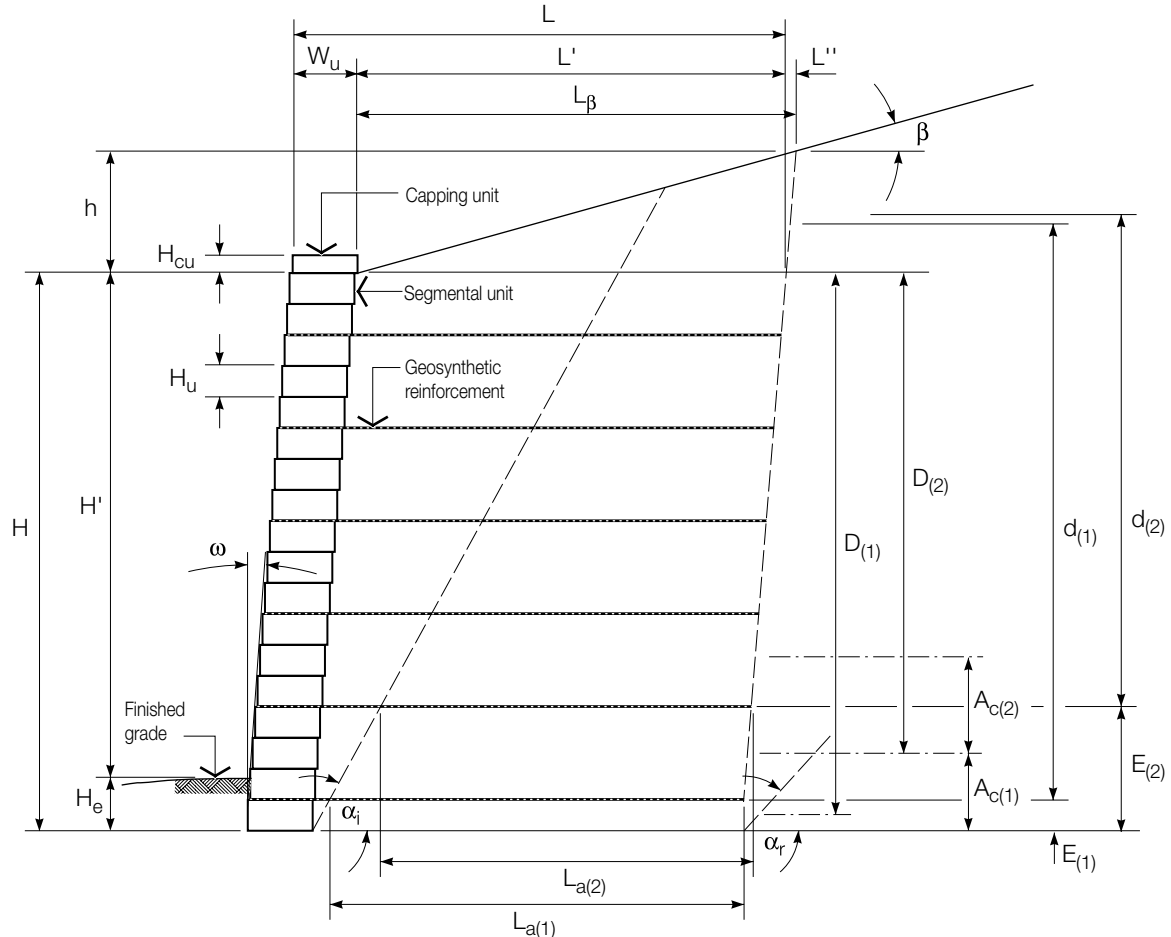


Figure 4.1 Geometric Parameters Used In Design

1 Wall Details

- Wall slope, ω
- Backfill slope, β
- Height of stem above soil in front of wall, H'
- Live load surcharge, q_l
(5 kPa minimum requirement)
- Dead load surcharge, q_d
- Height of water table from top of drainage layer, H_w

2 Earthquake Considerations

- Location
- Acceleration coefficient, a
- Soil profile
- Site factor
- Earthquake design category, B_{er}

3 Load Factors

- Load factors on overturning dead loads, G_{do}
- Load factors on overturning live loads, G_{lo}
- Load factor on resisting dead loads, G_{dr}
- Load factor on resisting live loads, G_{lr}

4 Infill Soil Properties

- Characteristic internal friction angle, ϕ_i
- Design uncertainty factor for friction, $\Phi_{U\phi i}$
- Design angle for friction, $\phi_i^* = \tan^{-1}[(\tan \phi_i) \Phi_{U\phi i}]$
- Characteristic cohesion, c_i
- Design uncertainty factor for cohesion, $\Phi_{Uc i}$
- Design cohesion, $c_i^* = c_i \Phi_{Uc i}$
(assume zero for design)
- Soil density, γ_i^*
- Design external friction angle, $\delta_i^* = 2/3 \phi_i^*$

5 Retained Soil Properties

Characteristic internal friction angle, ϕ_r
 Design uncertainty factor for friction, $\Phi_{U\phi r}$
 Design angle for friction, $\phi_r^* = \tan^{-1}[(\tan \phi_r)\Phi_{U\phi r}]$
 Characteristic cohesion, c_r
 Design uncertainty factor for cohesion, $\Phi_{Uc r}$
 Design cohesion, $c_r^* = c_r \Phi_{Uc r}$
 (assume zero for design)
 Soil density, γ_r^*
 Design external friction angle
 (soil to soil interface), δ_r^*

6 Foundation Soil properties

Characteristic internal friction angle, ϕ_f
 Design uncertainty factor for friction, $\Phi_{U\phi f}$
 Design angle for friction, $\phi_f^* = \tan^{-1}[(\tan \phi_f)\Phi_{U\phi f}]$
 Characteristic cohesion, c_f
 Design uncertainty factor for cohesion, $\Phi_{Uc f}$
 Design cohesion, $c_f^* = c_f \Phi_{Uc f}$
 Soil density, γ_f^*

7 Bearing Pad properties

Characteristic internal friction angle, ϕ_d
 Design uncertainty factor for friction, $\Phi_{U\phi d}$
 Design angle for friction, $\phi_d^* = \tan^{-1}[(\tan \phi_d)\Phi_{U\phi d}]$
 Characteristic cohesion, c_d
 Design uncertainty factor for cohesion, $\Phi_{Uc d}$
 Design cohesion, $c_d^* = c_d \Phi_{Uc d}$
 (assume zero for design)
 Soil density, γ_d^*

8 Segmental Wall Units

Height of capping unit, H_{cu}
 Height of units, H_u
 Width of units, W_u
 Length of units, L_u
 Mass of units, M_u
 Mass of soil within units, M_s
 Mass of units plus soil, M_{su}
 Centre of gravity of units plus soil from front face, G_u
 Spacing of units, S_u
 Density of units plus soil,

$$\gamma_{su} = \frac{M_{su}}{H_u L_u W_u}$$

NOTE: 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.

9 Partial Factors on Geogrid Strength

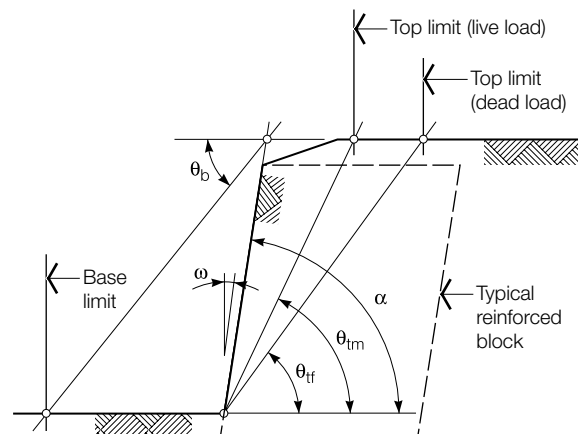
Log cycles of extrapolation
 $C_y = \log(\text{Service life} \times 365 \times 24) - \log(\text{Test duration})$
 Product uncertainty factor, Φ_{Up}
 Creep reduction factor, Φ_{rc}
 Extrapolation uncertainty factor, Φ_{ue}
 Construction damage factor, Φ_{ri}
 Thickness reduction factor, Φ_{rt}
 Strength reduction factor, Φ_{rs}
 Temperature reduction factor, Φ_{rst}
 Degradation factor, Φ_{ud}

10 Partial Factors on Soil/Geogrid Interaction and Geogrid Connection

Sliding uncertainty factor, Φ_{Uslide}
 Pullout uncertainty factor, Φ_{Upull}
 Coefficient of sliding resistance, k_{slide}
 Coefficient of pullout resistance, k_{pull}
 (The pullout resistance is based on the geogrid being sandwiched between two soil layers)
 Connection uncertainty factor, Φ_{Ucon}

11 Partial Factors on Structure Classification

Check location of adjacent structures, if any
 Structure Classification Factor
 Reduction factor, Φ_n



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

$$\begin{aligned} \theta_b &= \theta_{tf} \\ &= \frac{2\alpha + 3\phi}{5} \end{aligned}$$

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

Segmental Concrete Reinforced Soil Retaining Walls

12 Geogrid Properties

Ultimate strength, T_u

Design tensile strength of reinforcement,

$$T_d^* = T_u(\Phi_{up}) \cdot (\Phi_{rc} \Phi_{ue}) \cdot (\Phi_n) \cdot (\Phi_{rt} \Phi_{rs} \Phi_{rst}) \cdot (\Phi_{ud}) \cdot (\Phi_n)$$

13 Connection Strengths

Connection strength intercept, a_{cs}

Connection friction angle, λ_c

Maximum connection strength, S_c

14 Unit/Geogrid Interface Shear Strength

Interface strength intercept, a_u

Interface friction angle, λ_u

Maximum interface shear strength, S_u

15 External Stability

Wall embedment, H_e

Total height, $H = H' + H_e$

Triaxial geogrid length, $L = 0.7H$

Geogrid length in fill at top of wall, $L' = L - w_u$

Geogrid length increase due to backfill slope and wall slope,

$$L'' = \frac{L' \tan \beta \tan \omega}{1 - \tan \beta \tan \omega}$$

Geogrid length at top of backfill slope,

$$L_\beta = L' + L''$$

Height from top of wall to top of backfill slope,

$$h = L_\beta \tan \beta$$

Slope of drainage foundation interface, α

Active pressure coefficient,

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

16 Horizontal Forces

Horizontal active force due to surcharge,

$$P_{qH} = K_{ar}(G_{do} q_d + G_{lo} q)(H + h) \cos(\delta_r^* - \omega)$$

Horizontal active force due to soil,

$$P_{sH} = K_{ar} 0.5(G_{do} \gamma_1^2)(H + h)^2 \cos(\delta_r^* - \omega)$$

Total horizontal active force, $P_H = P_{qH} + P_{sH}$

Lever arm of horizontal surcharge load above toe,

$$y_{qH} = \frac{H + h}{2}$$

Lever arm of horizontal soil load above toe,

$$y_{sH} = \frac{H + h}{3}$$

17 Vertical Forces

Vertical weight of surcharge,

$$P_{qV} = G_r(q_d + q)L_\beta$$

or

$$P_{qV} = (G_{do} q_d + G_{lo} q)L_\beta$$

Vertical weight of soil and wall up to top of wall,

$$P_{s1V} = G_r \gamma_1^* HL$$

or

$$P_{s1V} = G_{do} \gamma_1^* HL$$

Vertical weight of soil above top of wall,

$$P_{s2V} = G_r 0.5 \gamma_1^* hL'$$

or

$$P_{s2V} = G_{do} 0.5 \gamma_1^* hL'$$

Lever arm of vertical surcharge load from toe,

$$y_{qV} = H \tan \omega + w_u + \frac{L_\beta}{2}$$

Lever arm of vertical soil weight up to top of wall from toe,

$$y_{s1V} = \frac{H \tan \omega}{2} + \frac{L}{2}$$

Lever arm of vertical soil weight above top of wall from toe

$$y_{s2V} = H \tan \omega + w_u + \frac{2L'}{3}$$

18 Base Sliding

It is strongly recommended that passive pressure in front of the wall be ignored in design.

Sliding resistance coefficient of infill material, C_{dsi}
(See page 60 of NCMA Manual ^(Ref 5))

Sliding resistance of infill soil,

$$R_{si} = \Phi_n(P_{qV} + P_{s1V} + P_{s2V})C_{dsi} \tan \phi_i^*$$

Sliding resistance coefficient of drainage soil, C_{dsd}
(See page 60 of NCMA Manual ^(Ref 5))

Sliding resistance of drainage soil

$$R_{sd} = \Phi_n(P_{qV} + P_{s1V} + P_{s2V})C_{dsd} \tan \phi_d^*$$

Sliding resistance coefficient of foundation soil, C_{dsf}
(See page 60 of NCMA Manual ^(Ref 5))

Sliding resistance of foundation soil,

$$R_{sf} = \Phi_n[(c_f^* B) + (P_{qV} + P_{s1V} + P_{s2V})]C_{dsf} \tan \phi_f^*$$

Sliding force, $P_{aH} = P_{qH} + P_{sH}$

19 Overturning

Resisting moments about toe,

$$M_R = \Phi_n[(P_{qV} y_{qV}) + (P_{s1V} y_{s1V}) + (P_{s2V} y_{s2V})]$$

Overturning moments about toe,

$$M_O = (P_{qH} y_{qH}) + (P_{sH} y_{sH})$$

20 Bearing at Underside of Infill

Depth of embedment, H_e
Actual width of base, $B = L$

Ratio of horizontal loads to vertical loads
(Check both maximum and minimum vertical loads)

$$\frac{P_H}{P_V} = \frac{P_{qH} + P_{sH}}{P_{qV} + P_{s1V} + P_{s2V}}$$

$$\text{Eccentricity, } e = \frac{B}{2} - \left[\frac{M_R - M_O}{P_V} \right]$$

Bearing width, $L_B = B - 2e$

Bearing capacity factors
 $N_q = e^{\pi \tan \phi_f^*} \tan^2(\pi/4 + \phi_f^*/2)$

$$N_c = (N_q - 1) \cot \phi_f^*$$

$$N_\gamma = 2(N_q + 1) \tan \phi_f^*$$

$$\zeta_q = 1.0$$

$$\zeta_{qi} = \left[1 - \frac{P_H}{P_V + L_B c_f^* \cot \phi_f^*} \right]^2$$

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

$$\zeta_c = 1.0$$

$$\zeta_{ci} = \zeta_{qi} - \frac{1 - \zeta_{qi}}{N_c \tan \phi_f^*}$$

$$\zeta_{ct} = \zeta_{qt} - \frac{1 - \zeta_{qt}}{N_c \tan \phi_f^*}$$

$$\zeta_\gamma = 1.0$$

$$\zeta_{\gamma i} = \left[1 - \frac{P_H}{P_V + L_B c_f^* \cot \phi_f^*} \right]^3$$

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

Average bearing strength capacity,

$$P_{Vcap} = \Phi_n L_B [c_f^* N_c \zeta_c \zeta_{ci} \zeta_{ct} + (\gamma_f^* H_e N_q \zeta_q \zeta_{qi} \zeta_{qt}) + (0.5 \gamma_f^* B N_\gamma \zeta_\gamma \zeta_{\gamma i} \zeta_{\gamma t})]$$

Applied vertical force,

$$P_V = P_{qV} + P_{s1V} + P_{s2V}$$

21 Internal Stability

Active pressure coefficient at infill soil,

$$K_{ai} = \frac{\cos^2(\phi_f^* + \omega)}{\cos^2(\omega) \cos(\omega - \delta_f^*)} \left[1 + \frac{\sin(\phi_f^* + \delta_f^*) \sin(\phi_f^* - \beta)}{\cos(\omega - \delta_f^*) \cos(\omega + \beta)} \right]^2$$

22 Horizontal Forces

Horizontal active force due to surcharge,
 $P_{qHi} = K_{ai} [(G_{do} q_d) + (G_{lo} q_l)] (H - H_u) \cos(\delta_f^* - \omega)$

Horizontal active force due to soil,
 $P_{sHi} = K_{ai} 0.5 G_{do} \gamma_f^* (H - H_u)^2 \cos(\delta_f^* - \omega)$

Total horizontal force, $P_{Hi} = P_{qHi} + P_{sHi}$

Minimum number of geogrid layers

$$N_{min} = \frac{P_{Hi}}{T_\alpha^*}$$

23 Tensile Strength

Elevation of geogrid, $E_{(1)}$

Geogrid contributory area,

$$A_{c(1)} = \frac{E_{(2)} - E_{(1)}}{2} + E_{(1)}$$

Depth to midpoint of contributory area,

$$D_{(1)} = H - \frac{A_{c(1)}}{2}$$

Elevation of geogrid, $E_{(2)}$

Geogrid contributory area,

$$A_{c(2)} = \frac{E_{(3)} - E_{(2)}}{2} + \frac{E_{(2)} - E_{(1)}}{2}$$

Depth to midpoint of contributory area,

$$D_{(2)} = D_{(1)} - \frac{A_{c(1)}}{2} - \frac{A_{c(2)}}{2}$$

Similar for remaining geogrids

Applied tensile load at geogrid,

$$F_{g(n)} = K_{ai} [(G_{do} q_d) + (G_{lo} q_l) + (G_{do} \gamma_f^* D_{(n)})] A_{c(n)} \cos(\delta_f^* - \omega)$$

Segmental Concrete Reinforced Soil Retaining Walls

24 Pullout Resistance

Angle of failure plane,

$$\alpha_i = \phi_i^* + \text{atan} \left[\frac{-\tan(\phi_i^* - \beta) + \sqrt{\tan(\phi_i^* - \beta)[\tan(\phi_i^* - \beta) + \cot(\phi_i^* + \omega)][1 + \tan(\delta_i^* - \omega)\cot(\phi_i^* + \omega)]}{1 + \tan(\delta_i^* - \omega)[\tan(\phi_i^* - \beta) + \cot(\phi_i^* + \omega)]} \right]$$

NOTE: For $\beta = 0$, $\delta = 0$ and $\omega = 0$: $\alpha = 45^\circ + \phi/2$

Geogrid length, $L_{(n)}$

Geogrid length beyond failure plane,

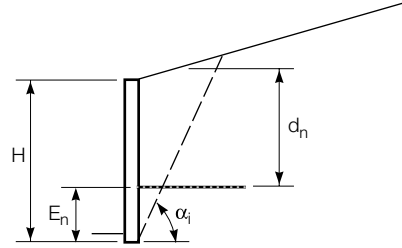
$$L_{a(n)} = L_{(n)} - W_u - E_{(n)} \tan(90^\circ - \alpha_i) + E_{(n)} \tan(\omega)$$

Average depth of overburden

$$d_{(n)} = H - E_{(n)} + \left[\frac{E_{(n)}}{\tan(\alpha_i)} - H \tan(\omega) + \frac{L_{a(n)}}{2} \right] \tan(\beta)$$

Anchorage capacity

$$AC_{(n)} = 2 k_{\text{pull}} L_{a(n)} \Phi_{\text{pull}} G_r(d_{(n)} \gamma_i^* + q_d + q) \tan(\phi_i^*) \Phi_n$$



25 Internal Sliding Resistance

Angle of failure plane,

$$\alpha_r = \phi_r^* + \text{atan} \left[\frac{-\tan(\phi_r^* - \beta) + \sqrt{\tan(\phi_r^* - \beta)[\tan(\phi_r^* - \beta) + \cot(\phi_r^* + \omega)][1 + \tan(\phi_r^* - \omega)\cot(\phi_r^* + \omega)]}{1 + \tan(\phi_r^* - \omega)[\tan(\phi_r^* - \beta) + \cot(\phi_r^* + \omega)]} \right]$$

Ineffective length of geogrid,

$$\Delta L = \frac{E_{(n+1)} - E_{(n)}}{\tan \alpha_r}$$

Effective length of geogrid, $L'_{s(n)} = L - W_u - \Delta L$

Length of slope increment above wall,

$$L''_{s(n)} = \frac{L'_{s(n)} \tan \beta \tan \omega}{1 - \tan \beta \tan \omega}$$

Length of soil acting above top of wall,

$$L_{\beta(n)} = L'_{s(n)} + L''_{s(n)}$$

Height of soil acting above top of wall,

$$h_{(n)} = L_{\beta(n)} \tan \beta$$

Weight of soil below top of wall acting on lowest geogrid, $W'_{r(n)} = G_r \gamma_i^* L'_{s(n)} (H - E_{(n)})$

Weight of soil above top of wall acting on lowest geogrid,

$$W'_{r\beta(n)} = \frac{G_r \gamma_i^* L_{\beta(n)} L'_{s(n)} \tan \beta}{2}$$

Surcharge force acting on lowest geogrid,

$$Q'_{r\beta(n)} = G_r (q_d + q) L_{\beta(n)}$$

Sliding resistance at lowest geogrid,

$$R'_{s(n)} = k_{\text{slide}} (W'_{r(n)} + W'_{r\beta(n)} + Q'_{r\beta(n)}) L_{\beta(n)} \tan(\phi_i^*) \Phi_n$$

Weight of wall acting on lowest unit/geogrid interface,

$$W_{w(1)} = G_v (H - E_{(1)}) \gamma_{sv}^* W_u$$

Shear resistance of lowest unit/geogrid interface,

$$V_{v(1)} = [a_u + (W_{w(1)} \tan \lambda_{sv})] \Phi_{\text{uslide}} \Phi_n$$

Total resistance, $R_{T(n)} = R_{s(n)} + V_{v(n)}$

Horizontal active force at lowest geogrid due to surcharge,

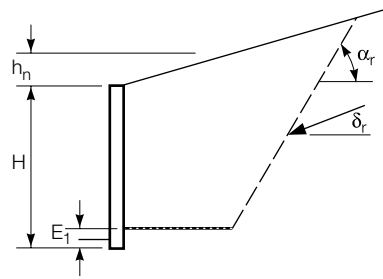
$$P_{qH(n)} = K_{ar} [(G_{do} q_d) + (G_{lo} q)] (H - E_{(n)} + h_{(n)}) \cos(\delta_i^* - \omega)$$

Horizontal active force at lowest geogrid due to soil,

$$P_{sH(n)} = K_{ar} 0.5 G_{do} \gamma_i^* (H - E_{(n)} + h_{(n)})^2 \cos(\delta_r - \omega)$$

Total horizontal active force at lowest geogrid,

$$P_{aH(n)} = P_{qH(n)} + P_{sH(n)}$$



26 Connection Strength

Weight of wall acting on each geogrid connection, $W_{w(n)} = G_v (H_{(n)} - E_{(n)}) \gamma_{sv}^* W_u$

Connection strength,

$$T_{\text{ult con}(n)} = [a_{cs} + (W_{w(n)} \tan \lambda_{cs})] \Phi_{\text{ucon}} \Phi_n$$

Force in connection,

$$P_{\text{con}} = \left[\left(\frac{H - E_{(n)}}{H} \right) 0.25 + 0.75 \right] F_{g(n)}$$

27 Bulging

NOTE: Spacing limited to 600 which should account for bulging

Active pressure coefficient at infill soil, K_{ai}

Horizontal active force due to surcharge,

$$P_{qH(n)} = K_{ai} [(G_{do} q_d) + (G_{lo} q)] (H - E_{(n)}) \cos(\delta_i^* - \omega)$$

Horizontal active force due to soil

$$P_{sH(n)} = K_{ai} 0.5 G_{do} \gamma_i^* (H - E_{(n)})^2 \cos(\delta_i^* - \omega)$$

Total horizontal force, $P_{H(n)} = P_{qH(n)} + P_{sH(n)}$

Net horizontal force at geogrid,

$$P_{H(n)} = P_{H(0)} - \sum (F_{g(n+1)} \text{ to } (n))$$

Unit/geogrid interface shear capacity, $V_{u(n)}$

5 References

- 1 AS 4678 *Earth retaining structures*, Standards Australia, 2002.
- 2 AS 1170 *Minimum design loads on structures* (known as the SAA Loading Code), Standards Australia, 1981.
- 3 AS 3600 *Concrete structures*, Standard Australia, 2000.
- 4 AS 3700 *Masonry structures*, Standards Australia, 2001.
- 5 *Design Manual for Segmental Retaining Walls*, National Concrete Masonry Association, 2nd Edition 1997, Herndon, Virginia, USA.
- 6 Vesic, A S, *Bearing Capacity of Shallow Foundations*, Foundation Engineering Handbook, Chapter 3, P121—146.
- 7 AS/NZS 4456.4 *Masonry units and segmental pavers – methods of test, Part 4 Determining compressive strength of masonry units*, Standards Australia, 1997.
- 8 Scott, C A, *Soil Mechanics and Foundations*, Third edition 1980, Applied Science Publishers Ltd, London, UK.

6 Appendices

The following Appendices are included:

Appendix A – Site Investigation	23
Appendix B – Design Example Number 1	25
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Balmain Foreshore Redevelopment, Sydney, Under Construction

Appendix A Site Investigation

The following guide may be used to gather site information needed for the design of segmental concrete, reinforced soil, retaining walls.

There should be special consideration of the following features if they are present:

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

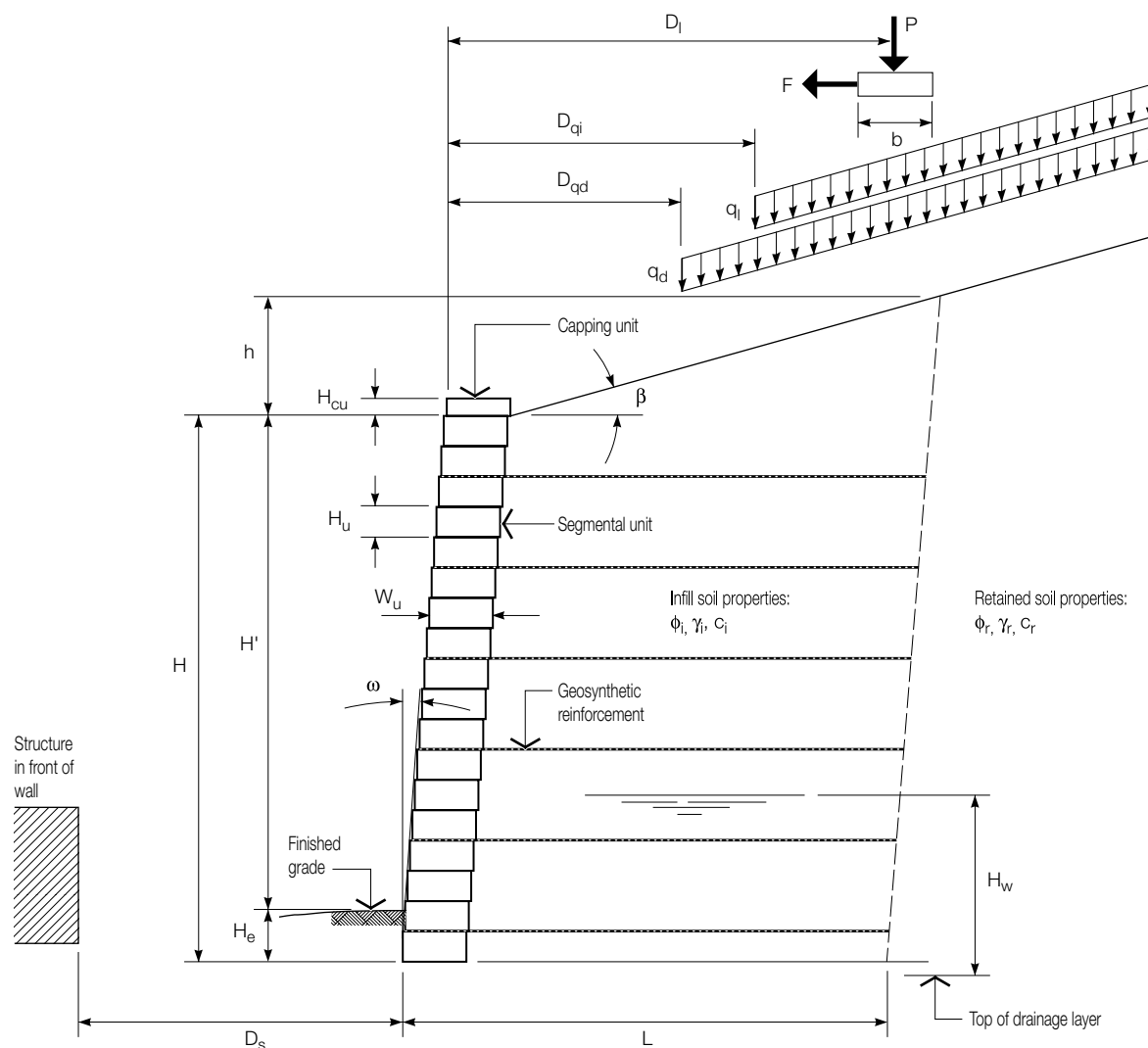


Diagram for use with suggested report

SITE INVESTIGATION

Date:

Report
prepared by:

Client:

Project:

Location:

Use for which retaining wall is intended:

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

Structure or load	Distance (m)
Distance of live loads from top of wall (D_{qi})
Distance of dead loads from top of wall (D_{qd})
Distance of line loads from top of wall (D)
Distance of other structures from base of wall (D_s)

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

Wall geometry:

Wall height above GL (H') m
Embedment depth (H_{emb}) <i>H/20 or 200 mm</i> m
Wall slope (ω) °
Angle of backfill slope (β) °
Height of backfill slope (h) m

Retained soil:

Soil density (γ_r) kN/m
Internal friction angle (ϕ'_r) °
Cohesion (c'_r) kPa

Loading data:

Dead load surcharge (q_d) kPa
Live load surcharge (q_l) kPa
Horizontal line load (F) kN/m
Vertical line load (P) kN/m
Width of bearing (b) m

Foundation soil:

Soil density (γ_f) kN/m
Internal friction angle (ϕ'_f) °
Cohesion (c'_f) kPa

Water profile:

Water table depth within wall fill m
------------------------------------	---------

NOTE: These properties are cautious estimates of the means, as defined in AS 4678.

APPENDIX B Design Example Number 1

The following example demonstrates the method used to design a typical segmental concrete reinforced soil retaining wall in accordance with AS 4678 and the stability and strength design considerations set out in this Guide. Serviceability must also be considered.

The design example considers a structure founded on undisturbed or reconstructed 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 a particular site exhibits these features, foundation treatment will be necessary before the retaining wall can be built.

1 Wall Details

Wall slope
 $\omega = 1.4^\circ$ (1 in 40)
 Use 0° for design

Backfill slope
 $\beta = 15.0^\circ$

Height of stem above soil in front of wall
 $H' = 3.6$ m

Live load surcharge
 $q_l = 5.0$ kPa (Minimum requirement)

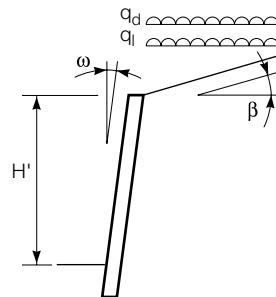
Dead load surcharge
 $q_d = 0$ kPa

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

Limits for determining structure classification

$$\begin{aligned}\theta_{tm} &= \frac{2\alpha + \phi}{3} \\ &= \frac{(2 \times 90^\circ) + 29^\circ}{3} \\ &= 70^\circ\end{aligned}$$

$$\begin{aligned}\theta_b &= \theta_{tf} \\ &= \frac{2\alpha + 3\phi}{5} \\ &= \frac{(2 \times 90^\circ) + (3 \times 29^\circ)}{5} \\ &= 53^\circ\end{aligned}$$



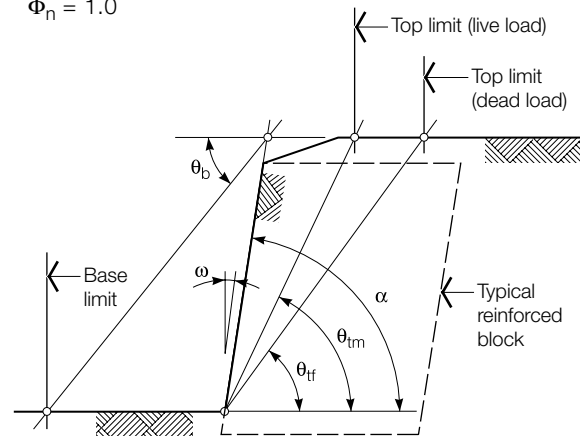
NOTE: Structures beyond the base limit or beyond the top limits are unlikely to be affected by, or have an affect upon, the structure clasification

Structure failure results in moderate damage

Structure Classification Factor = B

Reduction factor

$$\Phi_n = 1.0$$



2 Earthquake Considerations

Location
 Sydney

Acceleration coefficient
 $a = 0.08$

Soil profile
 Not more than 30 m of firm clay

Site factor = 1.0

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

3 Load Factors

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

Load factors 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 Soil Properties

Soil description
 Controlled crushed sandstone or gravel fills
 Class 2 controlled filling

Characteristic internal friction angle
 $\phi_i = 35^\circ$

Design uncertainty factor for friction
 $\Phi_{u\phi i} = 0.90$

Design angle for friction
 $\phi_i^* = \tan^{-1}[(\tan \phi_i)\Phi_{u\phi i}]$
 $= \tan^{-1}[(\tan 35^\circ)0.90]$
 $= 32.2^\circ$

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Characteristic cohesion

$$c_i = 3.0 \text{ kPa}$$

Design uncertainty factor for cohesion

$$\Phi_{uci} = 0.75$$

Design cohesion

$$\begin{aligned} c_i^* &= c_i \Phi_{uci} \\ &= 3.0 \times 0.75 \\ &= 2.3 \text{ kPa} \quad \text{Assume zero for design} \end{aligned}$$

Soil density

$$\gamma_i^* = 18.6 \text{ kN/m}^3$$

Characteristic external friction angle

$$\begin{aligned} \delta_i^* &= \frac{2}{3} \phi_i^* \\ &= \frac{2 \times 32.2}{3} \\ &= 21.5^\circ \end{aligned}$$

5 Retained Soil Properties

Soil description

Stiff sandy clay

Insitu

Characteristic internal friction angle

$$\phi_r = 29^\circ$$

Design uncertainty factor for friction

$$\Phi_{u\phi r} = 0.85$$

Design angle for friction

$$\begin{aligned} \phi_r^* &= \tan^{-1}[(\tan \phi_r) \Phi_{u\phi r}] \\ &= \tan^{-1}[(\tan 29^\circ) 0.85] \\ &= 25.2^\circ \end{aligned}$$

Characteristic cohesion

$$c_r = 5.0 \text{ kPa}$$

Design uncertainty factor for cohesion

$$\Phi_{ucr} = 0.70$$

Design cohesion

$$\begin{aligned} c_r^* &= c_r \Phi_{ucr} \\ &= 5.0 \times 0.70 \\ &= 3.5 \text{ kPa} \quad \text{Assume zero for design} \end{aligned}$$

Soil density

$$\gamma_r^* = 19.6 \text{ kN/m}^3$$

Characteristic external friction angle

(soil to soil interface)

$$\begin{aligned} \delta_r^* &= \phi_r^* \\ &= 25.2^\circ \end{aligned}$$

6 Foundation Soil Properties

Soil description

Reconstruct the foundation to improve properties.

Use crushed sandstone fill

Controlled fill, Class 2

Characteristic internal friction angle

$$\phi_f = 35^\circ$$

Design uncertainty factor for friction

$$\Phi_{u\phi f} = 0.90$$

Design angle for friction

$$\begin{aligned} \phi_f^* &= \tan^{-1}[(\tan \phi_f) \Phi_{u\phi f}] \\ &= \tan^{-1}[(\tan 35^\circ) 0.90] \\ &= 32.2^\circ \end{aligned}$$

Characteristic cohesion

$$c_f = 3.0 \text{ kPa}$$

Design uncertainty factor for cohesion

$$\Phi_{ucf} = 0.75$$

Design cohesion

$$\begin{aligned} c_f^* &= c_f \Phi_{ucf} \\ &= 3.0 \times 0.75 \\ &= 2.3 \text{ kPa for bearing and zero for sliding} \end{aligned}$$

Soil density

$$\gamma_f^* = 18.6 \text{ kN/m}^3$$

7 Bearing Pad Properties

Soil description

Crushed rock

Class 1 controlled filling

Characteristic internal friction angle

$$\phi_d = 37^\circ$$

Design uncertainty factor for friction

$$\Phi_{u\phi d} = 0.95$$

Design angle for friction

$$\begin{aligned} \phi_d^* &= \tan^{-1}[(\tan \phi_d) \Phi_{u\phi d}] \\ &= \tan^{-1}[(\tan 37^\circ) 0.95] \\ &= 35.6^\circ \end{aligned}$$

Characteristic cohesion

$$c_d = 5.0 \text{ kPa}$$

Design uncertainty factor for cohesion

$$\Phi_{ucd} = 0.90$$

Design cohesion

$$\begin{aligned} c_d^* &= c_d \Phi_{ucd} \\ &= 5.0 \times 0.90 \\ &= 4.5 \text{ kPa} \quad \text{Assume zero for design} \end{aligned}$$

Soil density

$$\gamma_d^* = 18.6 \text{ kN/m}^3$$

NOTE: 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.

8 Segmental Wall Units

Type: Generic

Height of capping unit

$$H_{cu} = 0.2 \text{ m}$$

Height of units

$$H_u = 0.2 \text{ m}$$

Width of units

$$W_u = 0.3 \text{ m}$$

Length of units

$$L_u = 0.45 \text{ m}$$

Mass of units

$$M_u = 35 \text{ kg}$$

Mass of soil within units

$$M_s = 18 \text{ kg}$$

Mass of units plus soil

$$M_{su} = 35 + 18 \\ = 53 \text{ kg}$$

Centre of gravity of units plus soil from front face

$$G_u = 0.153 \text{ m} \quad \cong W_u/2$$

Spacing of units

$$S_u = 0 \text{ m}$$

Density of units plus soil

$$\gamma_{su} = \frac{M_{su}}{H_u L_u W_u} \\ = 19.3 \text{ kN/m}^3$$

9 Partial Factors on Geogrid Strength

Service life

100 years

Geogrid type

Polyester

Specified by minimum or characteristic

Minimum

Duration of test

10,000 hours

Log cycles of extrapolation

$$C_y = \log(\text{Service life} \times 365 \times 24) - \log(\text{Test duration}) \\ = \log(100 \times 365 \times 24) - \log(10,000) \\ = 1.943$$

Backfill type (fine or coarse)

Fine

Product uncertainty factor

$$\Phi_{up} = 1.0$$

Creep reduction factor

$$\Phi_{rc} = 0.50$$

Extrapolation uncertainty factor

$$\Phi_{ue} = 0.91$$

Construction damage factor

$$\Phi_{ri} = 0.85$$

Thickness reduction factor

$$\Phi_{rt} = 0.9$$

Strength reduction factor

$$\Phi_{rs} = 0.70$$

Temperature reduction factor

$$\Phi_{rst} = 1.0$$

Degradation factor

$$\Phi_{ud} = 0.80$$

10 Partial Factors on Soil/Geogrid Interaction and Geogrid Connection

Sliding uncertainty factor

$$\Phi_{u \text{ slide}} = 0.80$$

Pullout uncertainty factor

$$\Phi_{u \text{ pull}} = 0.80$$

Connection uncertainty factor

$$\Phi_{u \text{ con}} = 0.75$$

11 Coefficients of Sliding Resistance and Pullout Resistance

Coefficient of sliding resistance

$$k_{\text{slide}} = 0.95$$

Coefficient of pullout resistance

$$k_{\text{pull}} = 0.70 \quad \text{The pullout resistance is based on the geogrid being sandwiched between two soil layers. Refer to NCMA Manual, page 107.}$$

12 Geogrid Properties

Geogrid type

Generic

Material

Polyester

Ultimate strength

$$T_u = 85.0 \text{ kN/m} \quad (\text{per metre run along grid of wall})$$

Design tensile strength of reinforcement

$$T_d^* = T_u \cdot (\Phi_{up}) \cdot (\Phi_{rc}) \cdot (\Phi_{ue}) \cdot (\Phi_{ri}) \cdot (\Phi_{rs}) \cdot (\Phi_{rst}) \cdot (\Phi_{ud}) \cdot (\Phi_{ri}) \\ = 85 \times 1.0 \times 0.5 \times 0.91 \times 0.85 \times 0.9 \times 0.7 \times 1.0 \times 0.8 \times 1.0 \\ = 16.6 \text{ kN/m}$$

13 Connection Strengths

Connection strength intercept

$$a_{cs} = 15.0 \text{ kN/m}$$

Connection friction angle

$$\lambda_c = 13.0^\circ$$

Maximum connection strength

$$S_c = 23.5 \text{ kN/m}$$

14 Unit/Geogrid Interface Shear Strength

Interface strength intercept

$$a_u = 37.0 \text{ kN/m}$$

Interface friction angle

$$\lambda_u = 31.7^\circ$$

Maximum interface shear strength

$$S_u = 37.0 \text{ kN/m}$$

15 External Stability

Wall embedment

$$H_e = 0.40 \text{ m} \\ \geq H'/20 \\ = \frac{3.60}{20} \\ = 0.18 \text{ m}$$

Total height

$$H = H' + H_e \\ = 3.60 + 0.40 \\ = 4.00 \text{ m}$$

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Trial geogrid length

$$\begin{aligned} L &= 0.7H \\ &= 0.7 \times 4.0 \\ &= 2.8 \text{ m} \end{aligned}$$

Subsequent calculations for sliding indicate that this length is too short. By iteration, a length of 3.75 m is determined and will be checked

Geogrid length in fill at top of wall

$$\begin{aligned} L' &= L - w_u \\ &= 3.75 - 0.3 \\ &= 3.45 \text{ m} \end{aligned}$$

Geogrid length increase due to backfill slope and wall slope

$$\begin{aligned} L'' &= \frac{L' \tan \beta \tan \omega}{1 - \tan \beta \tan \omega} \\ &= \frac{3.45 \tan 15^\circ \tan 0^\circ}{1 - \tan 15^\circ \tan 0^\circ} \\ &= 0.0 \text{ m} \end{aligned}$$

Geogrid length at top of backfill slope

$$\begin{aligned} L_\beta &= L' + L'' \\ &= 3.45 + 0 \\ &= 3.45 \text{ m} \end{aligned}$$

Height from top of wall to top of backfill slope

$$\begin{aligned} h &= L_\beta \tan \beta \\ &= 3.45 \tan 15^\circ \\ &= 0.924 \text{ m} \end{aligned}$$

Slope of drainage foundation interface

$$\alpha = 0^\circ$$

$$\begin{aligned} K_{ar} &= \frac{\cos^2(\phi_r^* + \omega)}{\cos^2(\omega) \cos(\omega - \delta_r^*) \left[1 + \sqrt{\frac{\sin(\phi_r^* + \delta_r^*) \sin(\phi_r^* - \beta)}{\cos(\omega - \delta_r^*) \cos(\omega + \beta)}} \right]^2} \\ &= \frac{\cos^2(25.2^\circ + 0^\circ)}{\cos^2(0^\circ) \cos(0^\circ - 25.2^\circ)} \\ &\quad \left[1 + \sqrt{\frac{\sin(25.2^\circ + 25.2^\circ) \sin(25.2^\circ - 15^\circ)}{\cos(0^\circ - 25.2^\circ) \cos(0^\circ + 15^\circ)}} \right]^2 \\ &= 0.46 \end{aligned}$$

16 Horizontal Forces

Horizontal active force due to surcharge

$$\begin{aligned} P_{qH} &= K_{ar}(G_{do} q_d + G_{lo} q_l)(H + h) \cos(\delta_r^* - \omega) \\ &= 0.46[(1.25 \times 0) + (1.5 \times 5.0)] \\ &\quad (4.0 + 0.924) \cos(25.2^\circ - 0^\circ) \\ &= 15.5 \text{ kN/m} \end{aligned}$$

Horizontal active force due to soil

$$\begin{aligned} P_{sH} &= K_{ar} 0.5(G_{do} \gamma_i^*)(H + h)^2 \cos(\delta_r^* - \omega) \\ &= 0.46 \times 0.5(1.25 \times 19.6)(4.0 + 0.924)^2 \\ &\quad \cos(25.2^\circ - 0^\circ) \\ &= 124.8 \text{ kN/m} \end{aligned}$$

Total horizontal active force

$$\begin{aligned} P_H &= P_{qH} + P_{sH} \\ &= 15.5 + 124.8 \\ &= 140.3 \text{ kN/m} \end{aligned}$$

Lever arm of horizontal surcharge load above toe

$$\begin{aligned} y_{qH} &= \frac{H + h}{2} \\ &= \frac{4.0 + 0.924}{2} \\ &= 2.462 \text{ m} \end{aligned}$$

Lever arm of horizontal soil load above toe

$$\begin{aligned} y_{sH} &= \frac{H + h}{3} \\ &= \frac{4.0 + 0.924}{3} \\ &= 1.641 \text{ m} \end{aligned}$$

17 Vertical Forces

Vertical weight of surcharge

$$\begin{aligned} P_{qV} &= (G_{dr} q_d + G_{lr} q_l) L_\beta \\ &= [(0.8 \times 0) + (0 \times 5.0)] 3.45 \\ &= 0 \text{ kN/m MIN.} \end{aligned}$$

or

$$\begin{aligned} P_{qV} &= (G_{do} q_d + G_{lo} q_l) L_\beta \\ &= [(1.25 \times 0) + (1.5 \times 5.0)] 3.45 \\ &= 25.9 \text{ kN/m MAX.} \end{aligned}$$

Vertical weight of soil and wall up to top of wall

$$\begin{aligned} P_{s1V} &= G_{dr} \gamma_i^* HL \\ &= 0.8 \times 18.6 \times 4.0 \times 3.75 \\ &= 223.2 \text{ kN/m MIN.} \end{aligned}$$

or

$$\begin{aligned} P_{s1V} &= G_{do} \gamma_i^* HL \\ &= 1.25 \times 18.6 \times 4.0 \times 3.75 \\ &= 348.8 \text{ kN/m MAX.} \end{aligned}$$

Vertical weight of soil above top of wall

$$\begin{aligned} P_{s2V} &= G_{dr} 0.5 \gamma_i^* h L' \\ &= 0.8 \times 0.5 \times 18.6 \times 0.924 \times 3.45 \\ &= 23.7 \text{ kN/m MIN.} \end{aligned}$$

or

$$\begin{aligned} P_{s2V} &= G_{do} 0.5 \gamma_i^* h L' \\ &= 1.25 \times 0.5 \times 18.6 \times 0.924 \times 3.45 \\ &= 37.1 \text{ kN/m MAX.} \end{aligned}$$

Lever arm of vertical surcharge load from toe

$$\begin{aligned} y_{qV} &= H \tan \omega + w_u + \frac{L_\beta}{2} \\ &= 4.0 \tan 0^\circ + 0.3 + \frac{3.45}{2} \\ &= 2.025 \text{ m} \end{aligned}$$

Lever arm of vertical soil weight up to top of wall from toe

$$\begin{aligned} y_{s1V} &= \frac{H \tan \omega}{2} + \frac{L}{2} \\ &= \frac{4.0 \tan 0^\circ}{2} + \frac{3.75}{2} \\ &= 1.875 \text{ m} \end{aligned}$$

Lever arm of vertical soil weight above top of wall from toe

$$\begin{aligned} y_{s2V} &= H \tan \omega + w_u + \frac{2L'}{3} \\ &= 4.0 \tan 0^\circ + 0.3 + \frac{2 \times 3.45}{3} \\ &= 2.6 \text{ m} \end{aligned}$$

18 Base Sliding

It is strongly recommended that passive pressure in front of the wall be ignored in design.

Is there geogrid or geofabric placed on the base?

No

Passive resistance, base adhesion and cohesion are taken as zero. The sliding resistance of the units over the bearing pad material may be different from the sliding resistance of the infill material over the bearing pad or foundation, but the difference is generally small and its effect on total sliding resistance is usually neglected. The designer should consider the validity of this approximation.

Sliding resistance coefficient of infill material

$$C_{dsi} = 1.0$$

Sliding resistance of infill soil

$$\begin{aligned} R_{si} &= \Phi_n [c_d^* B + (P_{qV} + P_{s1V} + P_{s2V}) c_{dsi} \tan \phi_d^*] \\ &= 1.0 [(0 + 223.2 + 23.7) 1.0 \tan 32.2^\circ] \\ &= 155.6 \text{ kN/m} \end{aligned}$$

Sliding resistance coefficient of drainage soil

$$C_{dsd} = 1.0$$

Sliding resistance of drainage soil

$$\begin{aligned} R_{sd} &= \Phi_n [c_d^* B + (P_{qV} + P_{s1V} + P_{s2V}) c_{dsd} \tan \phi_d^*] \\ &= 1.0 [(0 + 223.2 + 23.7) 1.0 \tan 35.6^\circ] \\ &= 176.8 \text{ kN/m} \end{aligned}$$

Sliding resistance coefficient of foundation soil

$$C_{dsf} = 1.0$$

Sliding resistance of foundation soil

$$\begin{aligned} R_{sf} &= \Phi_n [c_d^* B + (P_{qV} + P_{s1V} + P_{s2V}) c_{dsf} \tan \phi_f^*] \\ &= 1.0 [(0 + 223.2 + 23.7) 1.0 \tan 32.2^\circ] \\ &= 155.6 \text{ kN/m} \end{aligned}$$

Sliding force

$$\begin{aligned} P_{aH} &= P_{qH} + P_{sH} \\ &= 15.5 + 124.8 \\ &= 140.3 \text{ kN/m} \\ &< 155.6 \text{ kN/m} \quad \text{OK} \end{aligned}$$

19 Overturning

Resisting moments about toe

$$\begin{aligned} M_R &= \Phi_n [(P_{qV} Y_{qV}) + (P_{s1V} Y_{s1V}) + (P_{s2V} Y_{s2V})] \\ &= 1.0 [(0 \times 2.025) + (223.2 \times 1.875) + (23.7 \times 2.6)] \\ &= 480 \text{ kNm/m} \end{aligned}$$

Overturning moments about toe

$$\begin{aligned} M_O &= (P_{qH} Y_{qH}) + (P_{sH} Y_{sH}) \\ &= (15.5 \times 2.45) + (124.8 \times 1.641) \\ &= 243 \text{ kNm/m} \\ &< 480 \text{ kNm/m} \quad \text{OK} \end{aligned}$$

20 Bearing at Underside of Infill

Depth of embedment, $H_e = 0.4 \text{ m}$

Actual width of base, $B = L = 3.75 \text{ m}$

Ratio of horizontal loads to vertical loads

(Check both maximum and minimum vertical loads)

$$\begin{aligned} \frac{P_H}{P_V} &= \frac{P_{qH} + P_{sH}}{P_{qV} + P_{s1V} + P_{s2V}} \\ &= \frac{15.5 + 124.8}{0 + 223.2 + 23.7} \\ &= 0.568 \end{aligned}$$

or

$$\begin{aligned} \frac{P_H}{P_V} &= \frac{15.5 + 124.8}{25.9 + 348.8 + 37.1} \\ &= 0.341 \end{aligned}$$

Eccentricity

$$\begin{aligned} e &= \frac{B}{2} - \left[\frac{M_R - M_O}{P_V} \right] \\ &= \frac{3.75}{2} - \frac{(480 - 243)}{(0 + 223.2 + 23.7)} \\ &= 0.914 \end{aligned}$$

or

$$\begin{aligned} e &= \frac{3.75}{2} - \frac{(803 - 243)}{(25.9 + 348.8 + 37.1)} \\ &= 0.515 \end{aligned}$$

Bearing width

$$\begin{aligned} L_B &= B - 2e \\ &= 3.75 - (2 \times 0.914) \\ &= 1.922 \end{aligned}$$

or

$$\begin{aligned} L_B &= 3.75 - (2 \times 0.515) \\ &= 2.720 \end{aligned}$$

Bearing capacity factors

$$\begin{aligned} N_q &= e^{\pi \tan \phi_f^*} \tan^2(\pi/4 + \phi_f^*/2) \\ &= e^{\pi \tan 32.2^\circ} \tan^2(\pi/4 + 32.2^\circ/2) \\ &= 23.8 \end{aligned}$$

$$\begin{aligned} N_c &= (N_q - 1) \cot \phi_f^* \\ &= (23.8 - 1) \cot 32.2^\circ \\ &= 36.2 \end{aligned}$$

$$\begin{aligned} N_\gamma &= 2(N_q + 1) \tan \phi_f^* \\ &= 2(23.8 + 1) \tan 32.2^\circ \\ &= 31.2 \end{aligned}$$

$$\zeta_{qi} = 1.0$$

$$\begin{aligned} \zeta_{qi} &= \left[1 - \frac{P_H}{P_V + L_B c_f^* \cot \phi_f^*} \right]^2 \\ &= \left[1 - \frac{140.3}{246.9 + 1.922 \times 2.3 \times \cot 32.2^\circ} \right]^2 \\ &= 0.20 \end{aligned}$$

or

$$\begin{aligned} \zeta_{qi} &= \left[1 - \frac{140.3}{411.8 + 2.720 \times 2.3 \times \cot 32.2^\circ} \right]^2 \\ &= 0.45 \end{aligned}$$

$$\begin{aligned} \zeta_{qt} &= [1 - \alpha \tan \phi_f^*]^2 \\ &= [1 - 0 \tan 32.2^\circ]^2 \\ &= 1.0 \end{aligned}$$

$$\zeta_c = 1.0$$

$$\begin{aligned} \zeta_{ci} &= \zeta_{qi} - \frac{1 - \zeta_{qi}}{N_c \tan \phi_f^*} \\ &= 0.20 - \frac{1 - 0.20}{36.2 \times \tan 32.2^\circ} \\ &= 0.16 \end{aligned}$$

or

$$\begin{aligned} \zeta_{ci} &= 0.44 - \frac{1 - 0.44}{36.2 \times \tan 32.2^\circ} \\ &= 0.42 \end{aligned}$$

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$$\zeta_{ct} = \zeta_{qt} - \frac{1 - \zeta_{qt}}{N_c \tan \phi_i^*}$$

$$= 1.0 - \frac{1 - 1.0}{36.2 \times \tan 32.2^\circ}$$

$$= 1.0$$

$$\zeta_\gamma = 1.0$$

$$\zeta_{\gamma i} = \left[1 - \frac{P_H}{P_V + L_B c_i^* \cot \phi_i^*} \right]^3$$

$$= \left[1 - \frac{140.3}{260.7 + 1.922 \times 2.3 \times \cot 32.2^\circ} \right]^3$$

$$= 0.09$$

or

$$\zeta_{\gamma i} = \left[1 - \frac{140.3}{411.8 + 2.720 \times 2.3 \times \cot 32.2^\circ} \right]^3$$

$$= 0.30$$

$$\zeta_{\gamma t} = [1 - \alpha \tan \phi_i^*]^2$$

$$= [1 - 0 \tan 32.2^\circ]^2$$

$$= 1.0$$

Average bearing strength capacity

$$P_{Vcap} = \Phi_n L_B [c_i^* N_c \zeta_c \zeta_{ci} \zeta_{ct} + (\gamma_i^* H_e N_q \zeta_q \zeta_{qi} \zeta_{qt}) + (0.5 \gamma_i^* B N_\gamma \zeta_\gamma \zeta_{\gamma i} \zeta_{\gamma t})]$$

$$= 281 \text{ kN/m}$$

or

$$P_{Vcap} = 1187 \text{ kN/m}$$

Applied vertical force

$$P_V = P_{qV} + P_{s1V} + P_{s2V}$$

$$= 0 + 223.2 + 23.7$$

$$= 246.9 \text{ kN/m}$$

$$< 281 \text{ kN/m} \quad \text{OK}$$

or

$$P_V = 25.9 + 348.8 + 37.1$$

$$= 411.8 \text{ kN/m}$$

$$< 1187 \text{ kN/m} \quad \text{OK}$$

21 Internal Stability

Active pressure coefficient at infill soil

$$K_{ai} = \frac{\cos^2(\phi_i^* + \omega)}{\cos^2(\omega) \cos(\omega - \delta_i^*)} \left[1 + \frac{\sin(\phi_i^* + \delta_i^*) \sin(\phi_i^* - \beta)}{\cos(\omega - \delta_i^*) \cos(\omega + \beta)} \right]^2$$

$$= \frac{\cos^2(32.2^\circ + 0^\circ)}{\cos^2(0^\circ) \cos(0^\circ - 21.5^\circ)} \left[1 + \frac{\sin(32.2^\circ + 21.5^\circ) \sin(32.2^\circ - 15^\circ)}{\cos(0^\circ - 21.5^\circ) \cos(0^\circ + 15^\circ)} \right]^2$$

$$= 0.335$$

22 Horizontal Forces

Horizontal active force due to surcharge

$$P_{qHi} = K_{ai} [(G_{do} q_d) + (G_{lo} q_l)] (H - H_u) \cos(\delta_i^* - \omega)$$

$$= 0.335 [(1.25 \times 0) + (1.5 \times 5.0)] (4.0 - 0.2) \cos(21.5^\circ - 0^\circ)$$

$$= 8.9 \text{ kN/m}$$

Horizontal active force due to soil

$$P_{sHi} = K_{ai} 0.5 G_{do} \gamma_i^* (H - H_u)^2 \cos(\delta_i^* - \omega)$$

$$= 0.335 \times 0.5 \times 1.25 \times 18.6 \times (4.0 - 0.2)^2 \cos(21.5^\circ - 0^\circ)$$

$$= 52.3 \text{ kN/m}$$

Total horizontal force

$$P_{Hi} = P_{qHi} + P_{sHi}$$

$$= 8.9 + 52.3$$

$$= 61.2 \text{ kN/m}$$

Minimum number of geogrid layers

$$N_{min} = \frac{P_{Hi}}{T_d^*}$$

$$= \frac{61.2}{16.7}$$

$$= 3.7$$

∴ Minimum possible number of geogrids is 4

23 Tensile Strength

Geogrid No. 1:

Elevation of geogrid

$$E_{(1)} = 0.2 \text{ m}$$

Geogrid contributory area

$$A_{c(1)} = \frac{E_{(2)} - E_{(1)}}{2} + E_{(1)}$$

$$= \frac{0.8 - 0.2}{2} + 0.2$$

$$= 0.5 \text{ m}$$

Depth to midpoint of contributory area

$$D_{(1)} = H - \frac{A_{c(1)}}{2}$$

$$= 4.0 - \frac{0.5}{2}$$

$$= 3.75 \text{ m}$$

Geogrid No. 2:

Elevation of geogrid

$$E_{(2)} = 0.8 \text{ m}$$

Geogrid contributory area

$$A_{c(2)} = \frac{E_{(3)} - E_{(2)}}{2} + \frac{E_{(2)} - E_{(1)}}{2}$$

$$= \frac{1.4 - 0.8}{2} + \frac{0.8 - 0.2}{2}$$

$$= 0.6 \text{ m}$$

Depth to midpoint of contributory area

$$D_{(2)} = D_{(1)} - \frac{A_{c(1)}}{2} - \frac{A_{c(2)}}{2}$$

$$= 3.75 - \frac{0.5}{2} - \frac{0.6}{2}$$

$$= 3.2 \text{ m}$$

Similar for remaining geogrids

Applied tensile load at geogrid

$$F_{g(n)} = K_{al}[(G_{do} q_d) + (G_{lo} q_l) + (G_{do} \gamma_1^* D_{(n)})]A_{q(n)} \cos(\delta_1^* - \omega)$$

$$F_{g(1)} = 0.335[(1.25 \times 0) + (1.5 \times 5.0) + (1.25 \times 18.6 \times 3.75)]0.5 \cos(21.5^\circ - 0^\circ)$$

$$= 14.8 \text{ kN/m}$$

$$F_{g(2)} = 15.3 \text{ kN/m}$$

$$F_{g(3)} = 12.7 \text{ kN/m}$$

$$F_{g(4)} = 10.1 \text{ kN/m}$$

$$F_{g(5)} = 7.5 \text{ kN/m}$$

$$F_{g(6)} = 4.9 \text{ kN/m}$$

$$F_{g(7)} = 2.1 \text{ kN/m}$$

$< 16.6 \text{ kN/m} \quad \text{OK}$

24 Pullout Resistance

Angle of failure plane

$$\alpha_i = \phi_1^* + \text{atan} \left[\frac{-\tan(\phi_1^* - \beta) + \sqrt{\tan(\phi_1^* - \beta)[\tan(\phi_1^* - \beta) + \cot(\phi_1^* + \omega)][1 + \tan(\delta_1^* - \omega)\cot(\phi_1^* + \omega)]}}{1 + \tan(\delta_1^* - \omega)[\tan(\phi_1^* - \beta) + \cot(\phi_1^* + \omega)]} \right]$$

$$= 53.1^\circ$$

NOTE: For $\beta = 0$, $\delta = 0$ and $\omega = 0$
 $\alpha = 45^\circ + \phi/2$

Geogrid length

$$L_{(n)} = 3.75 \text{ m}$$

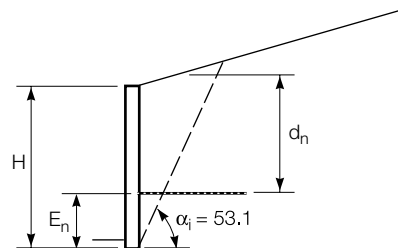
Geogrid length beyond failure plane

$$L_{a(n)} = L_{(n)} - W_u - E_{(n)}\tan(90^\circ - \alpha_i) + E_{(n)}\tan(\omega)$$

$$L_{a(1)} = 3.75 - 0.3 - 0.2\tan(90^\circ - 53.1^\circ) + 0.2\tan(0^\circ)$$

$$= 3.3 \text{ m}$$

$> 0.3 \text{ m} \quad \text{OK}$



Average depth of overburden

$$d_{(n)} = H - E_{(n)} + \left[\frac{E_{(n)}}{\tan(\alpha_i)} - H \tan(\omega) + \frac{L_{a(n)}}{2} \right] \tan(\beta)$$

$$d_{(1)} = 4.0 - 0.2 + \left[\frac{0.2}{\tan(53.1^\circ)} - 4.0 \tan(0^\circ) + \frac{3.3}{2} \right] \tan(15^\circ)$$

$$= 4.3 \text{ m}$$

Anchorage capacity

$$AC_{(n)} = 2 k_{pull} L_{a(n)} \Phi_{pull} G_r(d_{(n)} \gamma_1^* + q_d + q_l)\tan(\phi_1^*) \Phi_n$$

$$AC_{(1)} = 2.0 \times 0.7 \times 3.3 \times 0.8 \times [(4.3 \times 18.6) + 0 + 5.0]\tan(32.2^\circ) \times 1.0$$

$$= 158.3 \text{ kN/m}$$

$> 14.8 \text{ kN/m} \quad \text{OK}$

$$AC_{(2)} = 122.1 \text{ kN/m}$$

$> 15.3 \text{ kN/m} \quad \text{OK}$

Similar for remaining geogrids

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25 Internal Sliding Resistance

Check sliding at lowest geogrid

Angle of failure plane

$$\alpha_r = \phi_r^* + \text{atan} \left[\frac{-\tan(\phi_r^* - \beta) + \sqrt{\tan(\phi_r^* - \beta)[\tan(\phi_r^* - \beta) + \cot(\phi_r^* + \omega)][1 + \tan(\phi_r^* - \omega)\cot(\phi_r^* + \omega)]}{1 + \tan(\phi_r^* - \omega)[\tan(\phi_r^* - \beta) + \cot(\phi_r^* + \omega)]} \right]$$

$$= 44.6^\circ$$

Ineffective length of geogrid

$$\Delta L = \frac{E_{(n+1)} - E_{(n)}}{\tan \alpha_r}$$

$$= \frac{0.8 - 0.2}{\tan 44.6^\circ}$$

$$= 0.609 \text{ m}$$

Effective length of geogrid

$$L'_{s(n)} = L - W_U - \Delta L$$

$$= 3.75 - 0.3 - 0.609$$

$$= 2.841 \text{ m}$$

Length of slope increment above wall

$$L''_{s(n)} = \frac{L'_{s(n)} \tan \beta \tan \omega}{1 - \tan \beta \tan \omega}$$

$$= \frac{2.841 \tan 15^\circ \tan 0^\circ}{1 - \tan 15^\circ \tan 0^\circ}$$

$$= 0.0 \text{ m}$$

Length of soil acting above top of wall

$$L_{\beta(n)} = L'_{s(n)} + L''_{s(n)}$$

$$= 2.841 + 0.0$$

$$= 2.841 \text{ m}$$

Height of soil acting above top of wall

$$h_{(n)} = L_{\beta(n)} \tan \beta$$

$$= 2.841 \tan 15^\circ$$

$$= 0.761 \text{ m}$$

Weight of soil below top of wall acting on lowest geogrid

$$W'_{r(n)} = G_{dr} \gamma'_i L'_{s(n)} (H - E_{(n)})$$

$$= 0.8 \times 18.6 \times 2.841 (4.0 - 0.2)$$

$$= 160.6 \text{ kN/m}$$

Weight of soil above top of wall acting on lowest geogrid

$$W'_{r\beta(n)} = \frac{G_{dr} \gamma'_i L_{\beta(n)} L'_{s(n)} \tan \beta}{2}$$

$$= \frac{0.8 \times 18.6 \times 2.841 \times 2.841 \tan 15^\circ}{2}$$

$$= 16.1 \text{ kN/m}$$

Surcharge force acting on lowest geogrid

$$Q'_{r\beta(n)} = (G_{dr} q_d + G_{lr} q_l) L_{\beta(n)}$$

$$= [(0.8 \times 0) + (0 \times 5.0)] 2.846$$

$$= 0 \text{ kN/m}$$

Sliding resistance at lowest geogrid

$$R'_{s(n)} = \Phi_{\text{slide}} k_{\text{slide}} (W'_{r(n)} + W'_{r\beta(n)} + Q'_{r\beta(n)}) L_{\beta(n)} \tan(\phi_r^*) \Phi_n$$

$$= 0.8 \times 0.95 (160.9 + 16.1 + 0) 2.841 \times \tan 32.2^\circ \times 1.0$$

$$= 240.4 \text{ kN/m}$$

Weight of wall acting on lowest unit/geogrid interface

$$W_{w(1)} = G_v (H - E_{(1)}) \gamma_{sv}^* W_U$$

$$= 1.0 (4.0 - 0.2) 19.3 \times 0.3$$

$$= 22.0 \text{ kN/m}$$

Shear resistance of lowest unit/geogrid interface

$$V_{v(1)} = [a_u + (W_{w(1)} \tan \lambda_v)] \Phi_{\text{uslide}} \Phi_n$$

$$= [37.0 + (22.0 \tan 31.7^\circ)] 0.8 \times 1.0$$

$$= 40.4 \text{ kN/m}$$

Total resistance

$$R_{T(n)} = R_{s(n)} + V_{v(n)}$$

$$= 240.4 + 40.4$$

$$= 280.8 \text{ kN/m}$$

Horizontal active force at lowest geogrid due to surcharge

$$P_{qH(n)} = K_{ar} [(G_{do} q_d) + (G_{lo} q_l)] (H - E_{(n)} + h_{(n)}) \cos(\delta_r^* - \omega)$$

$$= 0.46 [(1.25 \times 0) + (1.5 \times 5.0)]$$

$$(4.0 - 0.2 + 0.743) \cos(25.2^\circ - 0^\circ)$$

$$= 14.4 \text{ kN/m}$$

Horizontal active force at lowest geogrid due to soil

$$P_{sH(n)} = K_{ar} 0.5 G_{do} \gamma'_i (H - E_{(n)} + h_{(n)})^2 \cos(\delta_r - \omega)$$

$$= 0.46 \times 0.5 \times 1.25 \times 19.6 \times$$

$$(4.0 - 0.2 + 0.761)^2 \cos(25.2^\circ - 0^\circ)$$

$$= 107.0 \text{ kN/m}$$

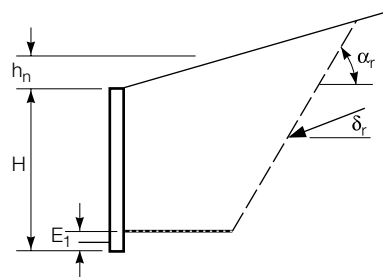
Total horizontal active force at lowest geogrid

$$P_{aH(n)} = P_{qH(n)} + P_{sH(n)}$$

$$= 14.4 + 107.0$$

$$= 121.4 \text{ kN/m}$$

$$< 280.8 \text{ kN/m} \quad \text{OK}$$



26 Connection Strength

Grid at bottom

Weight of wall acting on each geogrid connection

$$W_{w(n)} = G_v(H_{(n)} - E_{(n)})\gamma_{sv}^* W_u$$

$$W_{w(1)} = 1.0(4.0 - 0.2)19.3 \times 0.3 \\ = 22.2 \text{ kN/m}$$

Connection strength

$$T_{ult\ con(n)} = [a_{cs} + (W_{w(n)} \tan \lambda_{cs})] \Phi_{ucon} \Phi_n$$

$$T_{ult\ con(1)} = [15.0 + (22.0 \tan 13^\circ)]0.75 \times 1.0 \\ = 15.1 \text{ kN/m}$$

Force in connection

$$P_{con} = \left[\left(\frac{H - E_{(n)}}{H} \right) 0.25 + 0.75 \right] F_{g(n)} \\ = \left[\left(\frac{4.0 - 0.2}{4.0} \right) 0.25 + 0.75 \right] 14.7 \\ = 14.6 \text{ kN/m} \\ < 15.1 \text{ kN/m} \quad \text{OK}$$

27 Bulging

NOTE: Spacing limited to 600 which should account for bulging

Active pressure coefficient at infill soil

$$K_{ai} = 0.335$$

Horizontal active force due to surcharge

$$P_{qH(n)} = K_{ai}[(G_{do} q_d) + (G_{lo} q_l)](H - E_{(n)}) \cos(\delta_1^* - \omega) \\ = 0.335[(1.25 \times 0) + (1.5 \times 5.0)] \\ (4.0 - 0.2) \cos(21.5^\circ - 0^\circ) \\ = 8.9 \text{ kN/m}$$

Horizontal active force due to soil

$$P_{sH(n)} = K_{ai} 0.5 G_{do} \gamma_1^* (H - E_{(n)})^2 \cos(\delta_1^* - \omega) \\ = 0.335 \times 0.5 \times 1.25 \times 18.6 \times \\ (4.0 - 0.2)^2 \cos(21.5^\circ - 0^\circ) \\ = 52.3 \text{ kN/m}$$

Total horizontal force

$$P_{H(n)} = P_{qH(n)} + P_{sH(n)} \\ = 8.9 + 52.3 \\ = 61.2$$

Net horizontal force at geogrid (see Item 23)

$$P_{H(n)} = P_{H(0)} - \sum(F_{g(n+1)} \text{ to } (n)) \\ = 61.2 - (15.3 + 12.7 + 10.1 + 7.5 + 4.9 + 2.1) \\ = 8.6 \text{ kN/m}$$

Unit/geogrid interface shear capacity (see Item 25)

$$V_{u(n)} = 40.4 \text{ kN/m} \\ > 8.6 \text{ kN/m} \quad \text{OK}$$

APPENDIX C Design Example Number 2

The following example demonstrates the method used to design a typical segmental concrete reinforced soil retaining wall in accordance with AS 4678 and the stability and strength design considerations set out in this Guide. Serviceability must also be considered.

The design example considers a structure founded on undisturbed or reconstructed 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 a particular site exhibits these features, foundation treatment will be necessary before the retaining wall can be built.

1 Wall Details

Wall slope
 $\omega = 4^\circ$

Backfill slope
 $\beta = 0^\circ$

Height of stem above soil in front of wall
 $H' = 2.4 \text{ m}$

Live load surcharge
 $q_l = 5.0 \text{ kPa}$
(Minimum requirement)

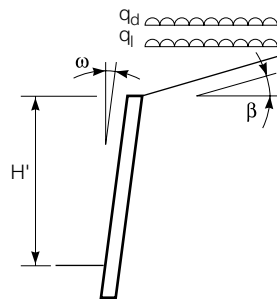
Dead load surcharge
 $q_d = 0 \text{ kPa}$

Height of water table from top of drainage layer
 $H_w = 0 \text{ m}$

Limits for determining structure classification

$$\begin{aligned}\theta_{tm} &= \frac{2\alpha + \phi}{3} \\ &= \frac{(2 \times 86^\circ) + 29^\circ}{3} \\ &= 67^\circ\end{aligned}$$

$$\begin{aligned}\theta_b = \theta_{tf} &= \frac{2\alpha + 3\phi}{5} \\ &= \frac{(2 \times 86^\circ) + (3 \times 29^\circ)}{5} \\ &= 52^\circ\end{aligned}$$



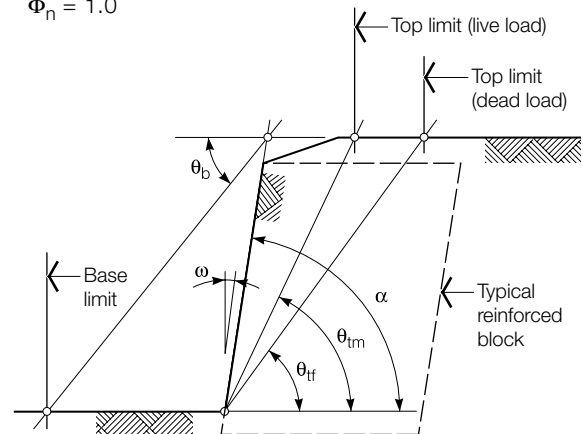
NOTE: Structures beyond the base limit or beyond the top limits are unlikely to be affected by, or have an affect upon, the structure classification

Structure failure results in moderate damage

Structure Classification Factor = B

Reduction factor

$$\Phi_n = 1.0$$



2 Earthquake Considerations

Location
Newcastle

Acceleration coefficient
 $a = 0.11$

Soil profile
Not more than 30 m of firm/stiff clay

Site factor = 1.0

Earthquake design category = C_{er}
 \therefore Design for dead loads with a factor of 1.5

3 Load Factors

Load factors on overturning dead loads

$$G_{do} = 1.5$$

Load factors 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 Soil Properties

Soil description

Class 2 controlled fill

Characteristic internal friction angle

$$\phi_i = 30^\circ \quad (\text{from Geotechnical Report})$$

Design uncertainty factor for friction

$$\Phi_{u\phi i} = 0.90$$

Design angle for friction

$$\begin{aligned} \phi_i^* &= \tan^{-1}[(\tan \phi_i)\Phi_{u\phi i}] \\ &= \tan^{-1}[(\tan 30^\circ)0.90] \\ &= 27.5^\circ \end{aligned}$$

Characteristic cohesion

$$c_i = 0 \text{ kPa} \quad \text{Assume zero for design}$$

Design uncertainty factor for cohesion

$$\Phi_{uc i} = 0.75$$

Design cohesion

$$\begin{aligned} c_i^* &= c_i \Phi_{uc i} \\ &= 0 \times 0.75 \\ &= 0 \text{ kPa} \end{aligned}$$

Soil density

$$\gamma_i^* = 18 \text{ kN/m}^3$$

Characteristic external friction angle

$$\begin{aligned} \delta_i^* &= \frac{2}{3} \phi_i^* \\ &= \frac{2 \times 27.5}{3} \\ &= 18.3^\circ \end{aligned}$$

5 Retained Soil Properties

Soil description

Stiff sandy clay
Insitu

Characteristic internal friction angle

$$\phi_r = 29^\circ \quad (\text{from Geotechnical Report})$$

Design uncertainty factor for friction

$$\Phi_{u\phi r} = 0.85$$

Design angle for friction

$$\begin{aligned} \phi_r^* &= \tan^{-1}[(\tan \phi_r)\Phi_{u\phi r}] \\ &= \tan^{-1}[(\tan 29^\circ)0.85] \\ &= 25.2^\circ \end{aligned}$$

Characteristic cohesion

$$c_r = 0 \text{ kPa} \quad \text{Assume zero for design}$$

Design uncertainty factor for cohesion

$$\Phi_{uc r} = 0.70$$

$$\begin{aligned} c_r^* &= c_r \Phi_{uc r} \\ &= 0 \times 0.70 \\ &= 0 \text{ kPa} \end{aligned}$$

Soil density

$$\gamma_r^* = 19 \text{ kN/m}^3$$

Characteristic external friction angle

(soil to soil interface)

$$\begin{aligned} \delta_r^* &= \phi_r^* \\ &= 25.2^\circ \end{aligned}$$

6 Foundation Soil Properties

Soil description

Same soil as retained

Characteristic internal friction angle

$$\phi_f = 29^\circ$$

Design uncertainty factor for friction

$$\Phi_{u\phi f} = 0.85$$

Design angle for friction

$$\begin{aligned} \phi_f^* &= \tan^{-1}[(\tan \phi_f)\Phi_{u\phi f}] \\ &= \tan^{-1}[(\tan 29^\circ)0.85] \\ &= 25.2^\circ \end{aligned}$$

Characteristic cohesion

$$c_f = 0 \text{ kPa} \quad \text{Assume zero for design}$$

Design uncertainty factor for cohesion

$$\Phi_{uc f} = 0.75$$

Design cohesion

$$\begin{aligned} c_f^* &= c_f \Phi_{uc f} \\ &= 0 \times 0.75 \\ &= 0 \text{ kPa} \end{aligned}$$

Soil density

$$\gamma_f^* = 19 \text{ kN/m}^3$$

7 Bearing Pad Properties

Soil description

Crushed rock
Class 1 controlled fill

Characteristic internal friction angle

$$\phi_d = 35^\circ$$

Design uncertainty factor for friction

$$\Phi_{u\phi d} = 0.95$$

Design angle for friction

$$\begin{aligned} \phi_d^* &= \tan^{-1}[(\tan \phi_d)\Phi_{u\phi d}] \\ &= \tan^{-1}[(\tan 35^\circ)0.95] \\ &= 33.6^\circ \end{aligned}$$

Characteristic cohesion

$$c_d = 0 \text{ kPa} \quad \text{Assume zero for design}$$

Design uncertainty factor for cohesion

$$\Phi_{uc d} = 0.90$$

Design cohesion

$$\begin{aligned} c_d^* &= c_d \Phi_{uc d} \\ &= 0 \times 0.90 \\ &= 0 \text{ kPa} \end{aligned}$$

Soil density

$$\gamma_d^* = 19 \text{ kN/m}^3$$

NOTE: 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

Segmental Concrete Reinforced Soil Retaining Walls

8 Segmental Wall Units

Type: Generic

Height of capping unit

$$H_{cu} = 0.1 \text{ m}$$

Height of units

$$H_u = 0.2 \text{ m}$$

Width of units

$$W_u = 0.315 \text{ m}$$

Length of units

$$L_u = 0.455 \text{ m}$$

Mass of units

$$M_u = 41 \text{ kg}$$

Mass of soil within units

$$M_s = 16.2 \text{ kg}$$

Mass of units plus soil

$$M_{su} = 41 + 16.2 = 57.2 \text{ kg}$$

Centre of gravity of units plus soil from front face

$$G_u = 0.158 \text{ m} \cong W_u/2$$

Spacing of units

$$S_u = 0 \text{ m}$$

Density of units plus soil

$$\begin{aligned} \gamma_{su} &= \frac{M_{su}}{H_u L_u W_u} \\ &= \frac{57.2}{0.2 \times 0.455 \times 0.315} \\ &= 19.7 \text{ kN/m}^3 \end{aligned}$$

9 Partial Factors on Geogrid Strength

Service life: 100 years

Geogrid type: Polyethylene

Specified by minimum or characteristic

Minimum

Duration of test

10,000 hours

Log cycles of extrapolation

$$\begin{aligned} C_y &= \log(\text{Service life} \times 365 \times 24) - \log(\text{Test duration}) \\ &= \log(100 \times 365 \times 24) - \log(10,000) \\ &= 1.943 \end{aligned}$$

Backfill type (fine or coarse)

Fine

Product uncertainty factor

$$\Phi_{up} = 1.0$$

Creep reduction factor

$$\Phi_{rc} = 0.30$$

Extrapolation uncertainty factor

$$\Phi_{ue} = 0.75$$

Construction damage factor

$$\Phi_{ri} = 0.85$$

Thickness reduction factor

$$\Phi_{rt} = 0.90$$

Strength reduction factor

$$\Phi_{rs} = 0.70$$

Temperature reduction factor

$$\Phi_{rst} = 1.0$$

Degradation factor

$$\Phi_{ud} = 0.80$$

10 Partial Factors on Soil/Geogrid Interaction and Geogrid Connection

Sliding uncertainty factor

Sliding uncertainty factor

$$\Phi_{u,slide} = 0.80$$

Pullout uncertainty factor

$$\Phi_{u,pull} = 0.80$$

Connection uncertainty factor

$$\Phi_{u,con} = 0.75$$

11 Coefficients of Sliding Resistance and Pullout Resistance

Coefficient of sliding resistance

$$k_{slide} = 0.95$$

Coefficient of pullout resistance

$$k_{pull} = 0.70 \quad \text{The pullout resistance is based on the geogrid being sandwiched between two soil layers. Refer to NCMA Manual, p107.}$$

12 Geogrid Properties

Geogrid type

Type 1 and Type 2

Material

Polyethylene

Ultimate strength (per metre run along grid of wall)

Type 1: $T_u = 60 \text{ kN/m}$

Type 2: $T_u = 90 \text{ kN/m}$

Design tensile strength of reinforcement

$$T_d^* = T_u (\Phi_{up}) \cdot (\Phi_{rc}) \cdot (\Phi_{ue}) \cdot (\Phi_{ri}) \cdot (\Phi_{rs}) \cdot (\Phi_{rst}) \cdot (\Phi_{ud}) \cdot (\Phi_n)$$

Type 1:

$$T_d^* = 60 \times 1.0 \times 0.3 \times 0.75 \times 0.85 \times 0.9 \times 0.7 \times 1.0 \times 0.8 \times 1.0 = 5.8 \text{ kN/m}$$

Type 2:

$$T_d^* = 90 \times 1.0 \times 0.3 \times 0.75 \times 0.85 \times 0.9 \times 0.7 \times 1.0 \times 0.8 \times 1.0 = 8.7 \text{ kN/m}$$

13 Connection Strengths

Connection strength intercept

$$a_{cs} = 9 \text{ kN/m}$$

Connection friction angle

$$\lambda_c = 30.8^\circ$$

Maximum connection strength

$$S_c = 32 \text{ kN/m}$$

14 Unit/Geogrid Interface Shear Strength

Interface strength intercept

$$a_u = 7 \text{ kN/m}$$

Interface friction angle

$$\lambda_u = 23^\circ$$

Maximum interface shear strength

$$S_u = 27 \text{ kN/m}$$

15 External Stability

Wall embedment

$$H_e = 0.30 \text{ m}$$

$$\geq H^2/20$$

$$= 2.7$$

$$= 20$$

$$= 0.135 \text{ m}$$

Total height

$$\begin{aligned} H &= H' + H_e \\ &= 2.40 + 0.30 \\ &= 2.70 \text{ m} \end{aligned}$$

Triaxial geogrid length

$$\begin{aligned} L &= 0.7H \\ &= 0.7 \times 2.7 \\ &= 1.89 \text{ m} \end{aligned}$$

Subsequent calculations indicated a length of 2.5 m is more suitable

Geogrid length in fill at top of wall

$$\begin{aligned} L' &= L - w_u \\ &= 2.5 - 0.315 \\ &= 2.185 \text{ m} \end{aligned}$$

Geogrid length increase due to backfill slope and wall slope

$$\begin{aligned} L'' &= \frac{L' \tan \beta \tan \omega}{1 - \tan \beta \tan \omega} \\ &= \frac{2.185 \tan 0^\circ \tan 4^\circ}{1 - \tan 0^\circ \tan 4^\circ} \\ &= 0 \text{ m} \end{aligned}$$

Geogrid length at top of backfill slope

$$\begin{aligned} L_\beta &= L' + L'' \\ &= 2.185 + 0 \\ &= 2.185 \text{ m} \end{aligned}$$

Height from top of wall to top of backfill slope

$$\begin{aligned} h &= L_\beta \tan \beta \\ &= 2.185 \tan 0^\circ \\ &= 0 \text{ m} \end{aligned}$$

Slope of drainage foundation interface

$$\alpha = 0^\circ$$

$$\begin{aligned} K_{ar} &= \frac{\cos^2(\phi_r^* + \omega)}{\cos^2(\omega) \cos(\omega - \delta_r^*)} \left[1 + \frac{\sqrt{\sin(\phi_r^* + \delta_r^*) \sin(\phi_r^* - \beta)}}{\cos(\omega - \delta_r^*) \cos(\omega + \beta)} \right]^2 \\ &= \frac{\cos^2(25.2^\circ + 4^\circ)}{\cos^2(4^\circ) \cos(4^\circ - 25.2^\circ)} \\ &\quad \left[1 + \frac{\sqrt{\sin(25.2^\circ + 25.2^\circ) \sin(25.2^\circ - 0^\circ)}}{\cos(4^\circ - 25.2^\circ) \cos(4^\circ + 0^\circ)} \right]^2 \\ &= 0.32 \end{aligned}$$

16 Horizontal Forces

Horizontal active force due to surcharge

$$\begin{aligned} P_{qH} &= K_{ar} (G_{do} q_d + G_{lo} q_l) (H + h) \cos(\delta_r^* - \omega) \\ &= 0.32 [(1.5 \times 0) + (1.5 \times 5.0)] \\ &\quad (2.7 + 0) \cos(25.2^\circ - 4^\circ) \\ &= 6.1 \text{ kN/m} \end{aligned}$$

Horizontal active force due to soil

$$\begin{aligned} P_{sH} &= K_{ar} 0.5 (G_{do} \gamma_r^*) (H + h)^2 \cos(\delta_r^* - \omega) \\ &= 0.32 \times 0.5 (1.5 \times 19) (2.7 + 0)^2 \cos(25.2^\circ - 4^\circ) \\ &= 31.2 \text{ kN/m} \end{aligned}$$

Total horizontal active force

$$\begin{aligned} P_H &= P_{qH} + P_{sH} \\ &= 6.1 + 31.2 \\ &= 37.3 \text{ kN/m} \end{aligned}$$

Lever arm of horizontal surcharge load above toe

$$\begin{aligned} y_{qH} &= \frac{H + h}{2} \\ &= \frac{2.7 + 0}{2} \\ &= 1.35 \text{ m} \end{aligned}$$

Lever arm of horizontal soil load above toe

$$\begin{aligned} y_{sH} &= \frac{H + h}{3} \\ &= \frac{2.7 + 0}{3} \\ &= 0.90 \text{ m} \end{aligned}$$

17 Vertical Forces

Vertical weight of surcharge

$$\begin{aligned} P_{qV} &= (G_{dr} q_d + G_{lr} q_l) L_\beta \\ &= [(0.8 \times 0) + (0 \times 5.0)] 2.815 \\ &= 0 \text{ kN/m MIN.} \end{aligned}$$

or

$$\begin{aligned} P_{qV} &= (G_{do} q_d + G_{lo} q_l) L_\beta \\ &= [(1.5 \times 0) + (1.5 \times 5.0)] 2.815 \\ &= 16.4 \text{ kN/m MAX.} \end{aligned}$$

Vertical weight of soil and wall up to top of wall

$$\begin{aligned} P_{s1V} &= G_{dr} \gamma_i^* HL \\ &= 0.8 \times 18 \times 2.7 \times 2.5 \\ &= 97.2 \text{ kN/m MIN.} \end{aligned}$$

or

$$\begin{aligned} P_{s1V} &= G_{do} \gamma_i^* HL \\ &= 1.5 \times 18 \times 2.7 \times 2.5 \\ &= 182.3 \text{ kN/m MAX.} \end{aligned}$$

Vertical weight of soil above top of wall

$$\begin{aligned} P_{s2V} &= G_{dr} 0.5 \gamma_i^* hL' \\ &= 0.8 \times 0.5 \times 18 \times 0 \times 2.5 \\ &= 0 \text{ kN/m} \end{aligned}$$

or

$$\begin{aligned} P_{s2V} &= G_{do} 0.5 \gamma_i^* hL' \\ &= 1.5 \times 0.5 \times 18 \times 0 \times 2.5 \\ &= 0 \text{ kN/m} \end{aligned}$$

Lever arm of vertical surcharge load from toe

$$\begin{aligned} y_{qV} &= H \tan \omega + w_u + \frac{L_\beta}{2} \\ &= 2.7 \tan 4^\circ + 0.315 + \frac{2.185}{2} \\ &= 1.60 \text{ m} \end{aligned}$$

Lever arm of vertical soil weight up to top of wall from toe

$$\begin{aligned} y_{s1V} &= \frac{H \tan \omega}{2} + \frac{L}{2} \\ &= \frac{2.7 \tan 4^\circ}{2} + \frac{2.5}{2} \\ &= 1.344 \text{ m} \end{aligned}$$

Lever arm of vertical soil weight above top of wall from toe

$$\begin{aligned} y_{s2V} &= H \tan \omega + w_u + \frac{2L'}{3} \\ &= 2.7 \tan 4^\circ + 0.315 + \frac{2 \times 2.185}{3} \\ &= 1.96 \text{ m} \end{aligned}$$

Segmental Concrete Reinforced Soil Retaining Walls

18 Base Sliding

It is strongly recommended that passive pressure in front of the wall be ignored in design.

Is there geogrid or geofabric placed on the base?
No

Passive resistance, base adhesion and cohesion are taken as zero. The sliding resistance of the units over the bearing pad material may be different from the sliding resistance of the infill material over the bearing pad or foundation, but the difference is generally small and its effect on total sliding resistance is usually neglected. The designer should consider the validity of this approximation.

Sliding resistance coefficient of infill material
 $C_{dsi} = 1.0$

Sliding resistance of infill soil
 $R_{si} = \Phi_n [c_i^* B + (P_{qV} + P_{s1V} + P_{s2V}) C_{dsi} \tan \phi_i^*]$
 $= 1.0 [(0 + 97.2 + 0) 1.0 \tan 27.5^\circ]$
 $= 50.5 \text{ kN/m}$

Sliding resistance coefficient of drainage soil
 $C_{dsd} = 1.0$

Sliding resistance of drainage soil
 $R_{sd} = \Phi_n [c_d^* B + (P_{qV} + P_{s1V} + P_{s2V}) C_{dsd} \tan \phi_d^*]$
 $= 1.0 [(0 + 97.2 + 0) 1.0 \tan 33.6^\circ]$
 $= 64.7 \text{ kN/m}$

Sliding resistance coefficient of foundation soil
 $C_{dsf} = 1.0$

Sliding resistance of foundation soil
 $R_{sf} = \Phi_n [c_f^* B + (P_{qV} + P_{s1V} + P_{s2V}) C_{dsf} \tan \phi_f^*]$
 $= 1.0 [0 + (0 + 97.2 + 0) 1.0 \tan 25.2^\circ]$
 $= 45.8 \text{ kN/m}$

Sliding force

$$P_{aH} = P_{qH} + P_{sH}$$

$$= 6.1 + 31.2$$

$$= 37.3 \text{ kN/m}$$

$$< 45.8 \text{ kN/m} \quad \text{OK}$$

19 Overturning

Resisting moments about toe

$$M_R = \Phi_n [(P_{qV} Y_{qV}) + (P_{s1V} Y_{s1V}) + (P_{s2V} Y_{s2V})]$$

$$= 1.0 [(0 \times 1.6) + (97.2 \times 1.344) + (0)]$$

$$= 130.7 \text{ kNm/m}$$

Overturning moments about toe

$$M_O = (P_{qH} Y_{qH}) + (P_{sH} Y_{sH})$$

$$= (6.1 \times 1.35) + (31.2 \times 0.9)$$

$$= 36.3 \text{ kNm/m}$$

$$< 130.7 \text{ kNm/m} \quad \text{OK}$$

20 Bearing at Underside of Infill

Depth of embedment, $H_e = 0.3 \text{ m}$

Actual width of base, $B = L = 2.5 \text{ m}$

Ratio of horizontal loads to vertical loads
(Check both maximum and minimum vertical loads)

$$\frac{P_H}{P_V} = \frac{P_{qH} + P_{sH}}{P_{qV} + P_{s1V} + P_{s2V}}$$

$$= \frac{6.1 + 31.2}{0 + 97.2 + 0}$$

$$= 0.384 \quad \text{MIN.}$$

or

$$\frac{P_H}{P_V} = \frac{6.1 + 31.2}{16.4 + 182.3 + 0}$$

$$= 0.188 \quad \text{MAX.}$$

Eccentricity

$$e = \frac{B}{2} - \left[\frac{M_R - M_O}{P_V} \right]$$

$$= \frac{2.5}{2} - \frac{(130.7 - 36.3)}{(0 + 97.2 + 0)}$$

$$= 0.28 \quad \text{MIN.}$$

or

$$e = \frac{2.5}{2} - \frac{(271.2 - 36.3)}{(16.4 + 182.8 + 0)}$$

$$= 0.07 \quad \text{MAX.}$$

Bearing width

$$L_B = B - 2e$$

$$= 2.5 - (2 \times 0.28)$$

$$= 1.94 \quad \text{MIN.}$$

or

$$L_B = 2.5 - (2 \times 0.07)$$

$$= 2.36 \quad \text{MAX.}$$

Bearing capacity factors

$$N_q = e^{\pi \tan \phi_i^*} \tan^2(\pi/4 + \phi_i^*/2)$$

$$= e^{\pi \tan 25.2^\circ} \tan^2(\pi/4 + 25.2^\circ/2)$$

$$= 10.9$$

$$N_c = (N_q - 1) \cot \phi_i^*$$

$$= (10.87 - 1) \cot 25.2^\circ$$

$$= 21.0$$

$$N_\gamma = 2(N_q + 1) \tan \phi_i^*$$

$$= 2(10.87 + 1) \tan 25.2^\circ$$

$$= 11.2$$

$$\zeta_q = 1.0$$

$$\zeta_{qi} = \left[1 - \frac{P_H}{P_V + L_B C_f^* \cot \phi_f^*} \right]^2$$

$$= \left[1 - \frac{37.27}{97.2 + 1.94 \times 0 \times \cot 25.2^\circ} \right]^2$$

$$= 0.38 \quad \text{MIN.}$$

or

$$\zeta_{qi} = \left[1 - \frac{37.2}{198.7 + 2.36 \times 0 \times \cot 25.2^\circ} \right]^2$$

$$= 0.66 \quad \text{MAX.}$$

$$\zeta_{qt} = [1 - \alpha \tan \phi_i^*]^2$$

$$= [1 - 0 \tan 25.2^\circ]^2$$

$$= 1.0$$

$$\zeta_c = 1.0$$

$$\zeta_{ci} = \zeta_{qi} - \frac{1 - \zeta_{qi}}{N_c \tan \phi_f^*}$$

$$= 0.38 - \frac{1 - 0.38}{20.98 \times \tan 25.2^\circ}$$

$$= 0.32 \quad \text{MIN.}$$

or

$$\zeta_{ci} = 0.66 - \frac{1 - 0.66}{20.98 \times \tan 25.2^\circ}$$

$$= 0.62 \quad \text{MAX.}$$

$$\zeta_{ct} = \zeta_{qt} - \frac{1 - \zeta_{qt}}{N_c \tan \phi_f^*}$$

$$= 1.0 - \frac{1 - 1}{21 \times \tan 25.2^\circ}$$

$$= 1.0$$

$$\zeta_\gamma = 1.0$$

$$\zeta_{\gamma i} = \left[1 - \frac{P_H}{P_V + L_B c_f^* \cot \phi_f^*} \right]^3$$

$$= \left[1 - \frac{37.2}{97.2 + 1.94 \times 0 \times \cot 25.2^\circ} \right]^3$$

$$= 0.23 \quad \text{MIN.}$$

or

$$\zeta_{\gamma i} = \left[1 - \frac{37.2}{198.7 + 2.36 \times 0 \times \cot 25.2^\circ} \right]^3$$

$$= 0.54 \quad \text{MAX.}$$

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

$$= [1 - 0 \tan 25.2^\circ]^2$$

$$= 1.0$$

Average bearing strength capacity

$$P_{Vcap} = \Phi_n L_B [c_f^* N_c \zeta_c \zeta_{ci} \zeta_{ct} + (\gamma_f^* H_e N_q \zeta_{qi} \zeta_{qt}) + (0.5 \gamma_f^* B N_\gamma \zeta_\gamma \zeta_{\gamma i} \zeta_{\gamma t})]$$

$$= 167 \text{ kN/m} \quad \text{MIN.}$$

or

$$= 435 \text{ kN/m} \quad \text{MAX.}$$

Applied vertical force

$$P_V = P_{qV} + P_{s1V} + P_{s2V}$$

$$= 0 + 97.2 + 0$$

$$= 97.2 \text{ kN/m}$$

$$< 167 \text{ kN/m} \quad \text{OK}$$

or

$$P_V = 16.4 + 182.3 + 0$$

$$= 198.7 \text{ kN/m}$$

$$< 435 \text{ kN/m} \quad \text{OK}$$

21 Internal Stability

Active pressure coefficient at infill soil

$$K_{ai} = \frac{\cos^2(\phi_i^* + \omega)}{\cos^2(\omega) \cos(\omega - \delta_i^*)} \left[1 + \frac{\sin(\phi_i^* + \delta_i^*) \sin(\phi_i^* - \beta)}{\cos(\omega - \delta_i^*) \cos(\omega + \beta)} \right]^2$$

$$= \frac{\cos^2(27.5^\circ + 4^\circ)}{\cos^2(4^\circ) \cos(4^\circ - 18.3^\circ)} \left[1 + \frac{\sin(27.5^\circ + 18.3^\circ) \sin(27.5^\circ - 0^\circ)}{\cos(4^\circ - 18.3^\circ) \cos(4^\circ + 0^\circ)} \right]^2$$

$$= 0.30$$

22 Horizontal Forces

Horizontal active force due to surcharge

$$P_{qHi} = K_{ai} [(G_{do} c_d) + (G_{lo} q)] (H - H_u) \cos(\delta_i^* - \omega)$$

$$= 0.3 [(1.5 \times 0) + (1.5 \times 5.0)] (2.7 - 0.2) \cos(18.3^\circ - 4^\circ)$$

$$= 5.5 \text{ kN/m}$$

Horizontal active force due to soil

$$P_{sHi} = K_{ai} 0.5 G_{do} \gamma_f^* (H - H_u)^2 \cos(\delta_i^* - \omega)$$

$$= 0.3 \times 0.5 \times 1.5 \times 18 \times (2.7 - 0.2)^2 \cos(18.3^\circ - 4^\circ)$$

$$= 24.5 \text{ kN/m}$$

Total horizontal force

$$P_{Hi} = P_{qHi} + P_{sHi}$$

$$= 5.5 + 24.5$$

$$= 30 \text{ kN/m}$$

Minimum number of geogrid layers

$$N_{min} = \frac{P_{Hi}}{T_d^*}$$

$$= \frac{30}{5.8} \quad (\text{Using TT060})$$

$$= 5.2$$

∴ Minimum possible number of geogrids is 6

23 Tensile Strength

Geogrid No. 1:

Elevation of geogrid

$$E_{(1)} = 0.2 \text{ m}$$

Geogrid contributory area

$$A_{c(1)} = \frac{E_{(2)} - E_{(1)}}{2} + E_{(1)}$$

$$= \frac{0.6 - 0.2}{2} + 0.2$$

$$= 0.4 \text{ m}$$

Depth to midpoint of contributory area

$$D_{(1)} = H - \frac{A_{c(1)}}{2}$$

$$= 2.7 - \frac{0.2}{2}$$

$$= 2.5 \text{ m}$$

Geogrid No. 2:

Elevation of geogrid

$$E_{(2)} = 0.8 \text{ m}$$

Geogrid contributory area

$$A_{c(2)} = \frac{E_{(3)} - E_{(2)}}{2} + \frac{E_{(2)} - E_{(1)}}{2}$$

$$= \frac{1.0 - 0.6}{2} + \frac{0.6 - 0.2}{2}$$

$$= 0.4 \text{ m}$$

Depth to midpoint of contributory area

$$D_{(2)} = D_{(1)} - \frac{A_{c(1)}}{2} - \frac{A_{c(2)}}{2}$$

$$= 2.5 - \frac{0.4}{2} - \frac{0.4}{2}$$

$$= 2.1 \text{ m}$$

Similar for remaining geogrids

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Applied tensile load at geogrid

$$F_{g(n)} = K_{al}[(G_{do} q_d) + (G_{lo} q_l) + (G_{do} \gamma_1^* D_{(n)})] A_{q(n)} \cos(\delta_1^* - \omega)$$

$$F_{g(1)} = 0.3[(1.5 \times 0) + (1.5 \times 5.0) + (1.5 \times 18 \times 2.5)] 0.4 \cos(27.5^\circ - 4^\circ)$$

$$= 8.7 \text{ kN/m}$$

$$F_{g(2)} = 7.5 \text{ kN/m}$$

$$F_{g(3)} = 6.2 \text{ kN/m}$$

$$F_{g(4)} = 5.0 \text{ kN/m}$$

$$F_{g(5)} = 4.4 \text{ kN/m}$$

$$F_{g(6)} = 2.7 \text{ kN/m}$$

For Type 1: $T_d^* = 5.8 \text{ kN/m}$ For Type 2: $T_d^* = 8.7 \text{ kN/m}$
 \therefore use Type 2 for Grids 1, 2, 3 and Type 1 for Grids 4, 5, 6

24 Pullout Resistance

Angle of failure plane

$$\alpha_i = \phi_i^* + \text{atan} \left[\frac{-\tan(\phi_i^* - \beta) + \sqrt{\tan(\phi_i^* - \beta)[\tan(\phi_i^* - \beta) + \cot(\phi_i^* + \omega)][1 + \tan(\delta_1^* - \omega)\cot(\phi_i^* + \omega)]}{1 + \tan(\delta_1^* - \omega)[\tan(\phi_i^* - \beta) + \cot(\phi_i^* + \omega)]} \right]$$

$$= 53.0^\circ \quad \text{NOTE: For } \beta = 0, \delta = 0 \text{ and } \omega = 0$$

$$\alpha = 45^\circ + \phi/2$$

Geogrid length

$$L_{(n)} = 2.5 \text{ m}$$

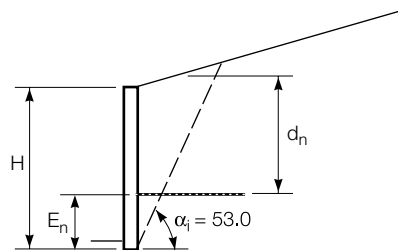
Geogrid length beyond failure plane

$$L_{a(n)} = L_{(n)} - W_u - E_{(n)} \tan(90^\circ - \alpha_i) + E_{(n)} \tan(\omega)$$

$$L_{a(1)} = 2.5 - 0.315 - 0.2 \tan(90^\circ - 53^\circ) + 0.2 \tan(4^\circ)$$

$$= 2.05 \text{ m}$$

$$> 0.315 \text{ m} \quad \text{OK}$$



Average depth of overburden

$$d_{(n)} = H - E_{(n)} + \left[\frac{E_{(n)}}{\tan(\alpha_i)} - H \tan(\omega) + \frac{L_{a(n)}}{2} \right] \tan(\beta)$$

$$d_{(1)} = 2.7 - 0.2 + \left[\frac{0.2}{\tan(53.0^\circ)} - 2.7 \tan(4^\circ) + \frac{2.05}{2} \right] \tan(0^\circ)$$

$$= 2.5 \text{ m}$$

Anchorage capacity

$$AC_{(n)} = 2.0 k_{pull} L_{a(n)} \Phi_{pull} G_r(d_{(n)} \gamma_1^* + q_d + q_l) \tan(\phi_1^*) \Phi_n$$

$$AC_{(1)} = 2.0 \times 0.7 \times 2.05 \times 0.8 \times [(2.5 \times 18) + 0 + 5.0] \tan(25.2^\circ) \times 1.0$$

$$= 47.7 \text{ kN/m}$$

$$> 8.7 \text{ kN/m} \quad \text{OK}$$

$$AC_{(2)} = 35.1 \text{ kN/m}$$

$$> 7.5 \text{ kN/m} \quad \text{OK}$$

$$AC_{(3)} = 24.1 \text{ kN/m}$$

$$> 6.2 \text{ kN/m} \quad \text{OK}$$

$$AC_{(4)} = 15.3 \text{ kN/m}$$

$$> 5.0 \text{ kN/m} \quad \text{OK}$$

$$AC_{(5)} = 8.5 \text{ kN/m}$$

$$> 4.4 \text{ kN/m} \quad \text{OK}$$

$$AC_{(6)} = 2.0 \text{ kN/m}$$

$< 2.7 \text{ kN/m}$ Problem, therefore grid 6 must be extended to increase pullout resistance

25 Internal Sliding Resistance

Check sliding at lowest geogrid

Angle of failure plane

$$\alpha_r = \phi_r^* + \operatorname{atan} \left[\frac{-\tan(\phi_r^* - \beta) + \sqrt{\tan(\phi_r^* - \beta)[\tan(\phi_r^* - \beta) + \cot(\phi_r^* + \omega)][1 + \tan(\phi_r^* - \omega)\cot(\phi_r^* + \omega)]}{1 + \tan(\phi_r^* - \omega)[\tan(\phi_r^* - \beta) + \cot(\phi_r^* + \omega)]} \right]$$

$$= 50.1^\circ$$

Ineffective length of geogrid

$$\begin{aligned} \Delta L &= \frac{E_{(n+1)} - E_{(n)}}{\tan \alpha_r} \\ &= \frac{0.6 - 0.2}{\tan 50.1^\circ} \\ &= 0.334 \text{ m} \end{aligned}$$

Effective length of geogrid

$$\begin{aligned} L'_{s(n)} &= L - W_u - \Delta L \\ &= 2.5 - 0.315 - 0.334 \\ &= 1.85 \text{ m} \end{aligned}$$

Length of slope increment above wall

$$\begin{aligned} L''_{s(n)} &= \frac{L'_{s(n)} \tan \beta \tan \omega}{1 - \tan \beta \tan \omega} \\ &= \frac{1.85 \tan 0^\circ \tan 4^\circ}{1 - \tan 0^\circ \tan 4^\circ} \\ &= 0.0 \text{ m} \end{aligned}$$

Length of soil acting above top of wall

$$\begin{aligned} L_{\beta(n)} &= L'_{s(n)} + L''_{s(n)} \\ &= 1.85 + 0.0 \\ &= 1.85 \text{ m} \end{aligned}$$

Height of soil acting above top of wall

$$\begin{aligned} h_{(n)} &= L_{\beta(n)} \tan \beta \\ &= 1.85 \tan 0^\circ \\ &= 0.0 \text{ m} \end{aligned}$$

Weight of soil below top of wall acting on lowest geogrid

$$\begin{aligned} W'_{r(n)} &= G_{dr} \gamma_i^* L'_{s(n)} (H - E_{(n)}) \\ &= 0.8 \times 18 \times 1.85 (2.7 - 0.2) \\ &= 66.6 \text{ kN/m} \end{aligned}$$

Weight of soil above top of wall acting on lowest geogrid

$$\begin{aligned} W'_{r\beta(n)} &= \frac{G_{dr} \gamma_i^* L_{\beta(n)} L'_{s(n)} \tan \beta}{2} \\ &= \frac{0.8 \times 18 \times 1.85 \times 1.85 \tan 0^\circ}{2} \\ &= 0 \text{ kN/m} \end{aligned}$$

Surcharge force acting on lowest geogrid

$$\begin{aligned} Q'_{r\beta(n)} &= (G_{dr} q_d + G_r q_i) L_{\beta(n)} \\ &= [(0.8 \times 0) + (0 \times 5.0)] 1.85 \\ &= 0 \text{ kN/m} \end{aligned}$$

Sliding resistance at lowest geogrid

$$\begin{aligned} R'_{s(n)} &= \Phi_{\text{uslide}} k_{\text{slide}} (W'_{r(n)} + W'_{r\beta(n)} + Q'_{r\beta(n)}) L_{\beta(n)} \tan(\phi_r^*) \Phi_n \\ &= 0.8 \times 0.95 (66.6 + 0 + 0) 1.85 \tan 27.5^\circ \times 1.0 \\ &= 48.7 \text{ kN/m} \end{aligned}$$

Weight of wall acting on lowest unit/geogrid interface

$$\begin{aligned} W_{w(1)} &= G_v (H - E_{(1)}) \gamma_{sv}^* W_u \\ &= 1.0 (2.7 - 0.2) 19.7 \times 0.315 \\ &= 15.5 \text{ kN/m} \end{aligned}$$

Shear resistance of lowest unit/geogrid interface

$$\begin{aligned} V_{v(1)} &= [a_u + (W_{w(1)} \tan \lambda_{sv})] \Phi_{\text{uslide}} \Phi_n \\ &= [7.0 + (15.5 \tan 23.5^\circ)] 0.8 \times 1.0 \\ &= 10.9 \text{ kN/m} \end{aligned}$$

Total resistance

$$\begin{aligned} R_{T(n)} &= R_{s(n)} + V_{v(n)} \\ &= 48.7 + 10.9 \\ &= 59.6 \text{ kN/m} \end{aligned}$$

Horizontal active force at lowest geogrid due to surcharge

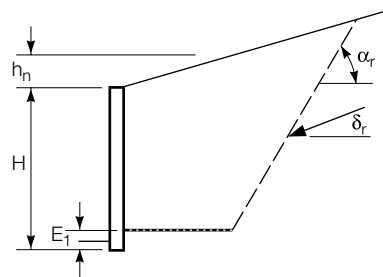
$$\begin{aligned} P_{qH(n)} &= K_{ar} [(G_{do} q_d) + (G_{lo} q_i)] (H - E_{(n)} + h_{(n)}) \cos(\delta_r^* - \omega) \\ &= 0.32 [(1.5 \times 0) + (1.5 \times 5.0)] \\ &\quad (2.7 - 0.2 + 0) \cos(18.3^\circ - 4^\circ) \\ &= 5.6 \text{ kN/m} \end{aligned}$$

Horizontal active force at lowest geogrid due to soil

$$\begin{aligned} P_{sH(n)} &= K_{ar} 0.5 G_{do} \gamma_r^* (H - E_{(n)} + h_{(n)})^2 \cos(\delta_r^* - \omega) \\ &= 0.32 \times 0.5 \times 1.5 \times 19 \times \\ &\quad (2.7 - 0.2 + 0)^2 \cos(25.2^\circ - 4^\circ) \\ &= 26.8 \text{ kN/m} \end{aligned}$$

Total horizontal active force at lowest geogrid

$$\begin{aligned} P_{aH(n)} &= P_{qH(n)} + P_{sH(n)} \\ &= 5.6 + 26.8 \\ &= 32.5 \text{ kN/m} \\ &< 59.6 \text{ kN/m} \quad \text{OK} \end{aligned}$$



Segmental Concrete Reinforced Soil Retaining Walls

26 Connection Strength

Grid at bottom

Weight of wall acting on each geogrid connection

$$W_{w(n)} = G_v(H_{(n)} - E_{(n)})\gamma_{sv}^* W_u$$

$$W_{w(1)} = 1.0(2.7 - 0.2)19.7 \times 0.315$$

$$= 15.6 \text{ kN/m}$$

Connection strength

$$T_{ult\ con(n)} = [a_{cs} + (W_{w(n)} \tan \lambda_{cs})] \Phi_{ucon} \Phi_n$$

$$T_{ult\ con(1)} = [9 + (15.6 \tan 30.8^\circ)] 0.75 \times 1.0$$

$$= 13.7 \text{ kN/m}$$

Force in connection

$$P_{con} = \left[\left(\frac{H - E_{(n)}}{H} \right) 0.25 + 0.75 \right] F_{g(n)}$$

$$= \left[\left(\frac{2.7 - 0.2}{2.7} \right) 0.25 + 0.75 \right] 8.7$$

$$= 8.5 \text{ kN/m}$$

$$< 13.7 \text{ kN/m} \quad \text{OK}$$

27 Bulging

NOTE: Spacing limited to 600 which should account for bulging

Active pressure coefficient at infill soil

$$K_{ai} = 0.30$$

Horizontal active force due to surcharge

$$P_{qH(n)} = K_{ai} [(G_{do} q_d) + (G_{lo} q_l)] (H - E_{(n)}) \cos(\delta_1^* - \omega)$$

$$= 0.30 [(1.5 \times 0) + (1.5 \times 5.0)]$$

$$(2.7 - 0.2) \cos(18.3^\circ - 4^\circ)$$

$$= 5.5 \text{ kN/m}$$

Horizontal active force due to soil

$$P_{sH(n)} = K_{ai} 0.5 G_{do} \gamma_1^* (H - E_{(n)})^2 \cos(\delta_1^* - \omega)$$

$$= 0.30 \times 0.5 \times 1.5 \times 18 \times$$

$$(2.7 - 0.2)^2 \cos(18.3^\circ - 4^\circ)$$

$$= 24.5 \text{ kN/m}$$

Total horizontal force

$$P_{H(n)} = P_{qH(n)} + P_{sH(n)}$$

$$= 5.5 + 24.5$$

$$= 30.0$$

Net horizontal force at geogrid (see Item 23)

$$P_{H(n)} = P_{H(0)} - \sum (F_{g(n+1) \text{ to } (n)})$$

$$P_{H(1)} = 30 - (7.5 + 6.2 + 5.0 + 4.4 + 2.7)$$

$$= 4.2 \text{ kN/m}$$

Unit/geogrid interface shear capacity (see Item 25)

$$V_{u(1)} = 13.5 \text{ kN/m}$$

$$> 4.2 \text{ kN/m} \quad \text{OK}$$

APPENDIX D Typical Specification

Construction Specification

Australian Standards

All components and installation shall comply with 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:

- the height of the wall does not exceed 3.2 m,
- excavation is performed and remains open only in dry weather,
- there is no significant groundwater 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 degrees for cohesionless material and 45 degrees for other material.

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 Over 3.2	1 : 0.4 Seek advice of engineer
Materials with both significant cohesion and friction in its undisturbed natural compacted state	0 – 2.6 Over 2.6	1 : 0.8 Seek advice of engineer
Cohesive soils, eg clay, silts	0 – 2.0 Over 2.0	1 : 1.2 Seek advice of engineer
Cohesionless soils, eg Loose gravel, sand	0 – 1.4 Over 1.4	1 : 1.6 Seek advice of engineer

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

If foundation material is of a type, grading or compaction that differs from that which is shown on the drawings, it shall be removed and replaced with a material that does comply.

Drainage System

The drainage system shall consist of:

- A permeable wall facing system.
- A permeable drainage layer not less than 300 mm wide adjacent to the stem of the wall.
- A 100-mm slotted PVC agricultural pipe, or equivalent system, draining to the storm-water system.
- Additional drainage layers and/or geotextiles as specified on the drawings.

Drainage Pipe

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

Drainage Fill

Drainage fill material shall be a nominal 10–20 mm GP (poorly-graded gravel) complying with the following specified grading:

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

Geosynthetic Filter Fabric

Geosynthetic filter fabrics shall be of a material which:

- is not hydrophobic
- permits water to pass freely
- does not permit fine material to enter the drainage layer
- has sufficient strength to resist tearing during the placing and back-filling operations
- has the following specified properties.

For use behind retaining walls which are retaining silt, fine sand or similar materials:

- Minimum grab tensile strength to AS 2001.2.3, 600 N
- Minimum wide-strip tensile strength to AS 3706.2, 8.0 kN/m
- Minimum trapezoidal tear test to AS 3706.3, 200 N
- Minimum CBR burst strength to AS 3706.4, 1600 N
- Maximum pore size O_{95} by dry sieving to AS 3706.7, 200 μ m (woven fabric)
- Minimum permittivity to AS 3706.9, 1.3 sec⁻¹
- Minimum coefficient of permeability to AS 3706.9, 0.003 m/sec
- Minimum flow rate under 100 mm head to AS 3706.9, 220 l/m²/sec

Segmental Concrete Reinforced Soil Retaining Walls

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.

Surface Sealing Material

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

Concrete Facing Blocks

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

- Dimensional category DW4
- General purpose salt attack resistance grade
- Minimum characteristic compressive strength of 10 MPa
- To a colour and texture agreed in writing before the supply takes place.

Broken or chipped units shall not be used. When it is necessary to cut units, they shall be cut with a saw rather than broken.

Infill Material

Infill material shall be GW (well-graded gravel) or SW (well-graded sand) complying with the following specification.

- The pH of the back filling material shall be, for polyester reinforcement, 4–9
- Plasticity Index shall not exceed 12%.
- Liquid Limit shall not exceed 30%.
- Coefficient of uniformity = D_{60}/D_{10} shall exceed 5, where D_{60} and D_{10} are the equivalent sizes, in millimetres, as interpolated from the particle size distribution curve through which 60% and 10% of the material passes, respectively.

Geogrids

The geogrids shall be of the type and index strength nominated on the drawings.

Geogrids shall be a single length in the direction of design tension, not lapped, making provision for connection to the facing across the whole width of the facing and providing for the specified anchorage within the designated anchorage zone. Geogrids shall cover the whole of the plan area behind the wall for the specified anchorage length and shall be lapped with adjacent sections in accordance with the manufacturer's instructions.

Adhesive

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

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.

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.

Installing Drainage Fill

Compact the drainage fill:

- around the drainage pipe to a minimum width of 300 mm behind the levelling fill
- behind the wall to a minimum width of 300 mm behind the wall to within 150 mm of the top

Compaction shall be by mechanical plate vibrator to a minimum of 95% of the standard proctor density. All drainage fill must be adequately drained by the drainage system.

Installing Concrete Facing Units, Infill Material and Geogrids

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).

Geogrids shall be installed under tension applied by a system of stakes that shall remain in place until the geogrids are covered by at least 150 mm of infill material.

Infill material shall be placed, spread and compacted in a manner that eliminates wrinkles in the geogrid or movement of the facing units. Infill material shall be placed and compacted in 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. Infill 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.

Tracked construction equipment shall not be operated

directly on the geogrids, which shall have a minimum of 150 mm of soil cover. In order to avoid disruption of the geogrids, tracked construction equipment shall not be turned on the infill material. Rubber tyred equipment may be used on the geogrids provided it is operated in accordance with the geogrid manufacturer's instructions, without sudden braking and turning and at speed under 6 kilometres per hour.

At the end of each day's construction, the infill 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.

Unless specified otherwise for reasons of aesthetics or by the client or architect, all construction shall be within the following tolerances:

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.

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.

Element	Vertical Position	Horizontal Position	Vertical Alignment	Horizontal Alignment
Soil surface	± 100 mm	-	-	-
Facings & wall structures	± 50 mm	± 50 mm	± 20 mm in 3.0 m	± 20 mm in 3.0 m
Footings & supports	± 50 mm	± 50 mm	± 20 mm in 3.0 m	± 20 mm in 3.0 m



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