Version 2 - November 2014

Guidance on the seismic design of retaining structures for residential sites in Greater Christchurch

Supplementary guidance to 'Guidance on repairing and rebuilding houses affected by the Canterbury earthquakes', December 2012.

Notes:

The principal users of this document will be professional geotechnical, structural engineers, designers and building control officials. The content of this document is therefore technical and written for a professional engineering and technical audience.

This guidance is issued under section 175 of the Building Act 2004.

1. Introduction

This guidance is provided for the assessment and design of retaining walls for the Greater Christchurch rebuild. Little formal guidance on the seismic design of retaining walls for residential properties is available at present. The NZTA Bridge Manual (2013) provides guidance on the earthquake-resistant design of retaining walls associated with road and highway infrastructure but these structures are generally subject to higher loadings than residential structures.

Clause B1 of the Building Code sets out basic requirements for buildings (which includes retaining walls) and site works (which includes formed batters) to ensure safety by including objectives to:

- safeguard people from injury caused by structural failure
- safeguard people from loss of amenity caused by structural behaviour
- protect other property from physical damage caused by structural failure.

Buildings, building elements, and site-works are required to have a low probability of:

- rupturing, becoming unstable, losing equilibrium or collapsing during construction, alteration, and throughout their lives
- causing loss of amenity through undue deformation, vibratory response, degradation, or other physical characteristics, during construction, alteration, when the building is in use, or throughout their lives.

Site work is required to provide stability for construction and to avoid the likelihood of damage to other property. Failing to demonstrate compliance with the above requirements because of geotechnical deficiencies would result in failure to obtain a building consent.

2. Scope

This document has been developed taking into account the performance of retaining walls following the Canterbury earthquake sequence and a review of international literature on the performance of retaining walls during earthquakes. Most of the affected walls are in the hillside suburbs and these are the focus of this document. However, the same design principles will be applicable to other residential sites in the City, although the ground conditions will be quite different.

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A total of 36 hillside areas within the Port Hills have been identified by GNS Science [GNS (2013)] as having been affected by varying scales of mass movement during the 2010/2011 series of earthquakes. Mass movement is the geomorphic process by which soil and rock material moves downhill as a semi-coherent mass. The majority of the areas identified in the report exhibit a type of mass movement that has not previously been observed in our local soils, and has been referred to in the GNS Science report as “toe slumping”. The design of retaining walls within these “toe slump” areas will require additional care especially regarding issues of global stability of sites and possible deep-seated failures of retaining walls in disturbed ground.

These guidelines are intended primarily for residential situations of normal risk. High risk walls, including very high walls, may require more detailed and specific engineering that is beyond the scope of these guidelines.

Retaining walls should be designed by qualified professional engineers under the supervision of a CPEng engineer with appropriate expertise. These guidelines are intended to assist qualified engineers to design residential retaining structures to resist seismic loading.

Earth-retaining structures should be designed to resist earthquake effects in the following situations:

1. Where failure or excessive deformation of the retaining structure might contribute to loss of life within or safe egress from a dwelling (ultimate limit state or ULS) or loss of amenity for a dwelling (serviceability limit state or SLS). (Including walls < 3m in height).

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OR

2. Where the height of the retaining structure has an effective height greater than 3m (including the height of batter above or below the retaining structure within a horizontal distance of 1.5 H, where H is the retained height).

In these cases the performance of the retaining wall under earthquake shaking needs to be considered appropriately for both SLS and ULS requirements, as recommended in this document.

Requirements for performance and design of retaining walls and formed batters affecting public thoroughfares and other specialist structures are not directly covered in this guidance and the relevant controlling authority should be consulted (eg NZTA Bridge Manual for NZTA roads and bridges (http://www.nzta.govt.nz/resources/bridge-manual/bridge-manual.html) and the pertinent local authority for retaining walls affecting facilities and roadways they control.

3. Building Code documents

Limited guidance is available within the supporting documents to the Building Code for the design of retaining walls for residential developments. NZS 1170.0:2002 specifies general procedures and criteria for the structural design of buildings including retaining walls. The standard covers combinations of actions to be considered including earth pressure and requires that earth pressure loads be determined in accordance with NZS 1170.1:2002. This states that “earth pressure actions…resulting in lateral loads on earth-retaining structures shall be determined using established methods of soil mechanics.”

NZS 1170.0:2002 requires earth pressure to be combined with factored permanent and imposed actions (dead and live loads) but no requirement to combine earth pressure and earthquake actions is stated. A load factor of 1.5 is specified for earth pressure unless it is determined using an “ultimate limit states method”, with an example of a suitable methodology being given as AS 4678-2002, *"Earth-Retaining Structures"* (recommendation given in the commentary to NZS 1170.0:2002).

This guidance is provided to meet the objectives of Clause B1 of the Building Code. Even though NZS 1170.0:2002 does not specifically require load combinations including earth pressure and earthquake actions, it will generally be necessary to consider such combinations to fulfil the objectives of Clause B1 of the Building Code.

Other documents provide more specific guidelines or rules for more specialist structures and these should, in general, take precedence over this document. Examples include the NZTA Bridge Manual (for NZTA roads and bridges).

4. Performance observations

4.1. Review of international literature

A review of international literature on the performance of retaining walls during earthquakes indicates that well-built retaining walls supporting or surrounded by soils that do not lose strength as a result of earthquake shaking perform satisfactorily during earthquake events [eg NCHRP (2008), Bray (2010), Mikola and Sitar (2013)].

4.2. General observations in the Port Hills

A number of studies of retaining wall performance have been undertaken [Dismuke (2011), Palmer *et al* (2014), Wood (2014)]. It is noted that the Palmer et al (2014) survey involved a random selection of 104 retaining walls and did not cover failed retaining walls that had been removed. In some cases it was also possible that some of the retaining walls inspected had been repaired before the inspection.

The Wood (2014) report was a review of wall damage descriptions in the SCIRT database and excluded facing walls and walls under 1.5m in height.

The following is a summary of general observations from these surveys.

- A significant number of retaining walls in residential properties suffered damage. Many of these were poorly designed and/or constructed (eg, lack of reinforcement, grouting, or low quality backfilling).
- Engineered retaining walls performed well, even though these were unlikely to have been designed to the levels of ground shaking experienced (many may not have been designed for any earthquake loading).
- Walls that retained fill often did not perform as well as those that retained undisturbed loess soil.
- Retained fill settled significantly, especially behind more flexible walls such as timber pole walls, timber crib walls and gabion walls.
- Many non-engineered rock facings, which are generally quite old structures, collapsed exposing stable, near vertical faces of undisturbed loess indicating that undisturbed, dry loess typically has high apparent cohesion under short term loading conditions.
- Several retaining wall failures appeared to be initiated by slope instability either above or below the wall.

- While there were numerous observations of outward movement of well-engineered retaining walls they were still fully functional post the earthquake sequence.

Figure 1: Failure of poorly constructed concrete block retaining wall

More specific observations following the Christchurch earthquakes for the most common types of walls were made as follows:

4.2.1. Concrete block walls

Engineered concrete block walls, whether cantilevered, buttressed, or propped generally performed well. Those that were propped or buttressed exhibited less damage than those in pure cantilever.

Where concrete block basement retaining walls were constructed integral with the dwelling little if any major structural damage resulting from ground shaking was observed. The only significant structural damage to these types of walls was observed in areas affected by land damage (predominantly in the “toe slump” areas). Observed wall rotations in these integral basement type walls were typically less than 1% from vertical, regardless of whether the walls were buttressed or not, and/or propped at the top or not. It was not possible in all cases to confirm whether these rotations were earthquake loading related.

Settlement of the drainage fill behind concrete block retaining walls was commonly observed. The settlement of fill did not necessarily coincide with excessive wall rotations. Possibly, the drainage fill had been placed loose, without adequate compaction and the resulting settlement was a “shaking down” or densification of the backfill under the earthquake loads. Drainage fill was observed to typically comprise rounded river gravel. Settlement of fill of up to 200mm was observed for a typical single storey basement retaining wall. Failure of the drainage system behind basement block retaining walls was uncommon in the walls observed.

Figure 2: Damaged concrete block basement wall

4.2.2. Timber pole walls

Engineered timber pole walls generally performed well. Failures of cantilever walls were observed where post sizes, post spacing, or embedment depths appeared inadequate and were probably not of engineered design/construction.

Localised structural failures were observed more often in tied-back walls. Undersized washers on tie-back anchors were fairly common resulting in crushing of timber. Bowed posts were common where there were tie-backs providing restraint towards the top of the wall. Vertical splits in poles were also common, but are considered to be of little structural significance.

Pull through of washers and nuts was more commonly observed than failure of the tie-back anchors themselves. However anchor failures were observed on a few walls.

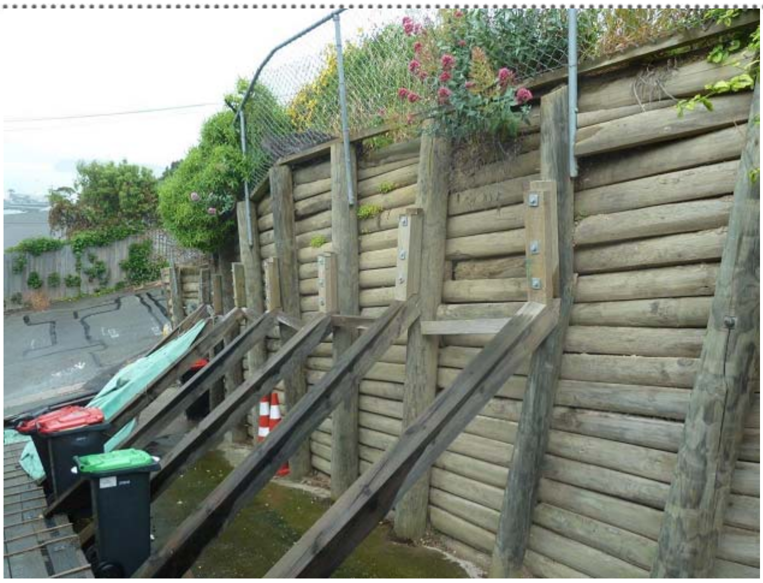


Figure 3: Damaged timber pole wall showing failure of poles at anchor location and failure of anchors

4.2.3. Timber crib walls

There was quite a wide variation in seismic performance observed for timber crib walls. It appears that this variability is much more strongly influenced by construction details and practice rather than fundamental design. A particular construction issue was the use of rounded gravel backfill within the wall units. Rounded material tends to shake out leaving voids between the block units and settlement of the ground or pavements above the wall. Certain construction practices appeared to perform better than others. For example, fixing of the header to the stretcher appears to improve wall performance by serving to minimise aggregate "shake out".

Figure 4: Damaged timber crib wall

4.2.4. Concrete crib walls

There was also a wide variation in performance observed for concrete crib walls and therefore most of the timber crib wall comments also generally apply to concrete crib walls. In some cases vegetation on the face of the wall appeared to improve performance by serving to retain the gravel backfill.

Figure 5: Concrete crib wall showing loss of rounded gravel backfill

4.2.5. Gabion walls

The use of gabions in residential settings is less common except in cases where land deformation is likely or where land slip remediation has been undertaken. They tend to be more widely employed on road reserve areas at the subdivision level of development, or for supporting heavier civil infrastructure. Quite often the uppermost one or two courses slumped outwards (>200mm) with significant cracking and settlement behind the wall in these instances. Outward movement was caused by both the stretching of the baskets, and rotation around the base of the walls. There was also evidence of a shake-down effect of the retained material in gabion walls.

Figure 6: Gabion wall showing bulging and outwards movement

5. Performance requirements for new retaining structures

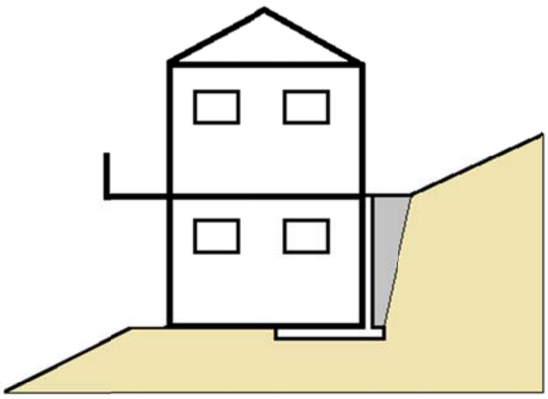
The essential performance requirements for retaining structures are given by Clause B1 of the Building Code. A recommended interpretation of these requirements is provided in Table 1 for specific cases relevant to residential situations with accompanying sketches in Figure 7.

Table 1: Performance requirements for residential retaining walls during earthquakes[1]

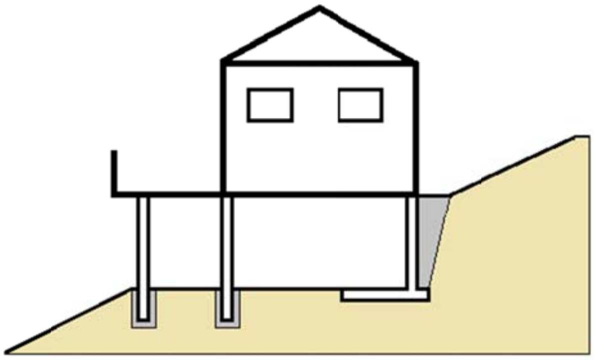
	Situation[2]	IL[3]	SLS	ULS
Case 1	Retaining wall integral to dwelling	2	No significant[4] movement	Wall movement should not be so excessive as to cause loss of structural integrity or prevent means of safe egress (eg less than 50mm for normal timber framed construction to NZS 3604).
Case 1a	Retaining wall integral to stand-alone garage	1	No requirement	Wall movement should not be so excessive as to cause collapse of the building (eg less than 150mm for normal timber framed construction to NZS 3604).
Case 2	Retaining wall supporting dwelling[5]	2	No significant[4] movement	Wall movement should not be so excessive as to cause loss of support, loss of structural integrity, or prevent means of safe egress (eg less than 100mm for normal timber framed construction to NZS 3604).
Case 3	Downslope and supporting dwelling foundations[5]	2	Minor movement, <25mm	Wall movement should not be so excessive as to cause loss of structural integrity or prevent means of safe egress (eg less than 100mm for normal timber framed construction to NZS 3604).
Case 4	Upslope and within 1.5H of dwelling	2	Minimal visual impairment for wall, <H/50	There should be a low risk of collapse of the wall. Wall deformations should not impede egress from the dwelling (noting that severe visual impairment of the wall may deter occupants from escaping the dwelling), (eg less than 100mm from vertical for typical cases).
Case 5	Facilitating access and services to dwelling (eg driveway)	1	No requirement	There should be a low risk of collapse of the wall. Wall deformations should not be so excessive as to damage services or prevent use of driveway (eg less than 150mm from vertical for typical cases).
Case 6	Other situations, H* >3m	1	No requirement	There should be a low risk of collapse of the wall.

Explanatory notes for Table 1

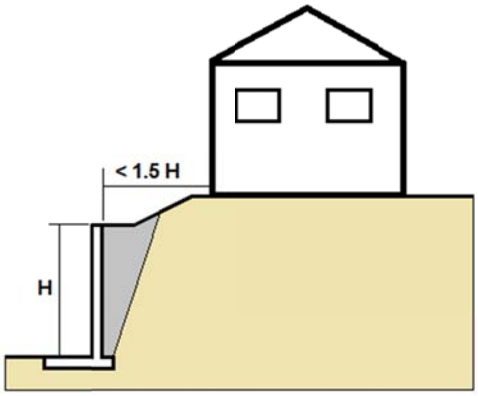
1. The intent of this table is to give guidance on selecting seismic design parameters for retaining structures. The movements indicated are for typical cases and represent permanent movement as a result of a single design earthquake for the purpose of selecting appropriate design acceleration coefficients. Instantaneous dynamic movements during an earthquake will be greater and there may be additional movements from gravity loads prior to an earthquake. Some buildings will be more sensitive to movement than others and it is the designer's responsibility to ensure that movements are able to be tolerated.
2. Refer to Figure 7.
3. Importance level from NZS 1170.0.
4. Significant movement would be movement sufficient to cause loss of amenity to the dwelling.
5. Dwelling may include existing dwelling on neighbouring property, access and services may include existing access and services to neighbouring property.



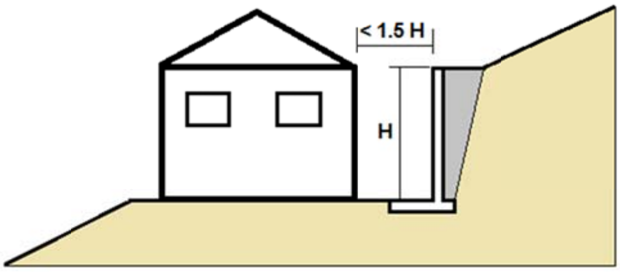
Case 1: Retaining wall integral to dwelling



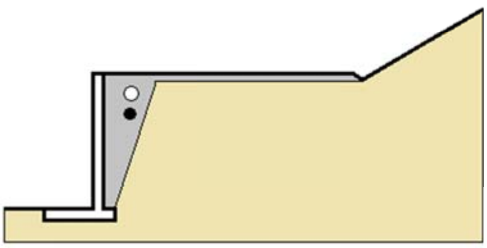
Case 2: Retaining wall supporting dwelling



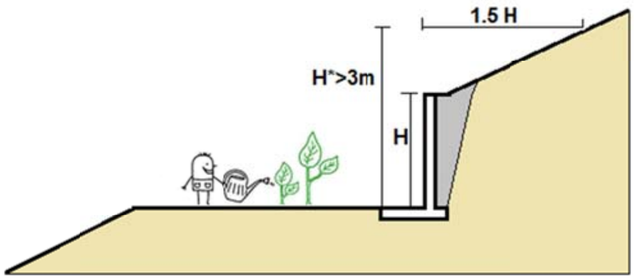
Case 3: Retaining wall supporting foundation



Case 4: Retaining wall protecting dwelling upslope



Case 5: Retaining wall facilitating access and services to dwelling



Case 6: Other situations where H* > 3 m

Figure 7: Typical situations where retaining walls are used for residential development

6. Seismic design parameters

Retaining walls are normally designed to resist earthquake loading by considering a pseudo-static horizontal acceleration applied to the wall and the retained soil. The pseudo-static design acceleration is derived from the appropriate peak ground acceleration (PGA) for the site which is a function of the location, return period, and site subsoil class.

The design acceleration for the site may be derived from the elastic site hazard spectrum for horizontal loading determined in accordance with NZS 1170.5:2004 as follows:

$$C(T) = C_h(T)ZRN(T,D) \qquad \text{(Equation 1.1)}$$

in which:

$C_h(T)$ = Spectral shape factor which may be taken as C_h (0) for retaining walls

= 1.0 for Class A and B (rock) sites

= 1.33 for Class C (Shallow soil) sites

= 1.12 for Class D (Deep soil and soft soil) and Class E (very soft soil) sites

Unless a site specific investigation has been carried out to confirm otherwise it recommended that Class C is assumed when determining the $C_h(T)$ factor for Christchurch Port Hills, refer to NZS 1170.5:2004.

The Z factors and return period factors pertaining to the Canterbury earthquake region are as follows: [B1/VM1, Amendment, 10 May 2011]

Z = 0.3 for Christchurch for ULS

R = Return period factor = 1.0 for Importance Level 2 walls, ULS

= 0.5 for Importance Level 1 walls, ULS

= 0.33 for Importance Level 2 walls, SLS
(for Canterbury earthquake region, = 0.25 elsewhere in NZ)

N(T,D) = Near fault factor which may be taken = 1.0 for residential retaining walls

6.1. Topographic amplification factor

Ground shaking in the Port Hills was found to be significantly amplified by certain topographic features including long ridges and cliff tops. The phenomenon of topographic amplification is well recognised internationally and the following simplified recommendations have been adapted from Eurocode 8, Part 5: BS EN 1998-5: 2004 (Annex A):

An amplification factor A_{topo} should be applied to the level ground design acceleration using Equation 1.2 in the following situations:

- For cliff features >30m in height, A_{topo} = 1.2 at the cliff edge and the area on top of the cliff of width equal to the height of the cliff.
- For ridge lines >30m in height with crest width significantly less than base width, and average slope angle greater than 30 degrees, A_{topo} = 1.4 at the crest diminishing to unity at the base.

- For ridge lines >30m in height with crest width significantly less than base width, and average slope angle greater than 15 degrees and less than 30 degrees, A_{topo} = 1.2 at the crest diminishing to unity at the base.
- For average slope angles of less than 15 degrees the topography effects may be neglected.

$$C(T,A_{topo}) = C(T)A_{topo} \qquad \text{(Equation 1.2)}$$

Note that *average slope angle* refers to the natural slope angle averaged over the height of the ridge, not the slope angle of the site.

6.2. Wall displacement factor

Designing flexible retaining walls to resist the full ULS peak ground acceleration (PGA) is unnecessary and uneconomic in most cases. Most residential retaining wall systems are sufficiently flexible to be able to absorb high transient ground acceleration pulses without damage because the inertia and damping of the retained soil limits deformations. Wave scattering effects also reduce the accelerations in the backfill to values less than the peak ground motions adjacent to retaining walls.

In most cases, some permanent wall deformation is acceptable for the ULS case (refer to Table 1) and the wall may be designed using a reduced value of acceleration coefficient given by Equation 1.3:

$$k_h = C(T,A_{topo})W_d \qquad \text{(Equation 1.3)}$$

in which:

k_h = horizontal acceleration coefficient for pseudo-static design

W_d = wall displacement factor, given in Table 2

The wall displacement factor, W_d, is selected according to the amount of permanent displacement that can be tolerated for the particular design case with guidance given in Table 2.

Table 2: Wall displacement factor, W_d for pseudo-static design of residential retaining walls in Christchurch Port Hills for ULS

Case (from Table 1)	IL	W_d
Case 1	2	0.7
Case 1a	1	0.5
Case 2	2	0.5
Case 3	2	0.5
Case 4	2	0.4
Case 5	1	0.3
Case 6	1	0.3

Explanatory notes for Table 2

6. International practice [eg Kramer, (1996)] is to adopt a seismic acceleration coefficient of between 0.33 to 0.5 of the peak ground acceleration for retaining structure design using pseudo-static procedures. Numerous case studies have shown that retaining structures designed in this way have performed satisfactorily during earthquakes, including observations from the Canterbury earthquakes (as discussed earlier in this document).
7. Reducing the design acceleration by W_d implies that permanent movement of the structure and retained ground is likely to occur. Several other assumptions are implied, including: a) that the retaining structure is sufficiently resilient or ductile to withstand the movement, b) that the supporting soils are not susceptible to strength loss with straining, and c) that any supported structures or services are able to tolerate the movement.
8. Analysis using "Newmark's sliding block" approach [eg Jibson, (2007)] indicates that retaining structures in the Port Hills designed using the values for W_d given in Table 2 should not exceed the movements indicated in Table 1.
9. For situations where less movement can be tolerated, a higher value of W_d should be selected. Wall movement may be estimated using the approach of Jibson (2007). As there is a high level of uncertainty in the source earthquake, the adoption of a 84th percentile displacement values is recommended.
10. Alternatively, where it is impractical to limit movements of the retaining structure sufficiently, other measures should be taken as appropriate (eg, it may be necessary to found an adjacent building on piles rather than on soil retained behind a wall (Case 3), or there should be structural separation between the retaining wall and dwelling [Case 1 and Case 2)].
11. W_d = 1.0 in all cases for SLS.

7. Design of new retaining structures

7.1. General requirements

New retaining structures should be designed for both the gravity load case and the earthquake load case using the combinations of actions as specified in NZS 1170.0:2002. For some walls the gravity load case may be more critical than the earthquake load case. For most walls both the gravity and earthquake load cases should be checked.

7.2. Serviceability Limit State

Wall movements should be checked for the SLS level earthquake for Cases 1, 2, and 3 from Table 1. Other cases have no SLS performance requirement for earthquake loading.

Wall movements should be checked using the following load combinations:

$E = [G + F_E + 0.4Q]$ gravity case (Equation 1.4)

$E = [G + F_S + 0.3Q]$ earthquake case (Equation 1.5)

in which:

E = action effect
F_E = static earth pressure
F_S = pseudo-static SLS earth pressure and wall inertia

G = self-weight (dead load)
Q = imposed action (live load)

Note that for the gravity case, F_E, the static earth pressure may be greater than the *active* earth pressure. Typically, F_E should be calculated using K_o.

7.3. Ultimate Limit State

Gravity retaining walls (including concrete cantilever walls, mass masonry walls, crib walls, gabion walls) may reach the ultimate limit state by several different modes of deformation:

- overturning
- sliding
- foundation bearing failure
- deep seated slippage
- yielding of structure (internal stability)

Embedded walls (including timber pole walls, sheet pile walls) have fewer modes of deformation:

- overturning
- deep seated slippage
- yielding of structure (internal stability)

Tied-back walls and propped walls have additional modes including:

- ground anchor pull-out
- tendon extension and failure
- prop buckling

Additional detail about the various modes of deformation is provided in the worked examples.

All relevant deformation modes need to be checked for both the gravity and earthquake load cases. Modes related to *stability* of the retaining structure should be checked using the following load combinations:

For loads that produce net stabilizing effects($E_{d,stb}$)

$E_{d,stb}$ = $[0.9G]$ (Equation 1.6)

For loads that produce net *de*stabilizing effects($E_{d,dst}$)

$E_{d,dst}$ = $[1.2G + 1.5F_E + 0.4Q]$ gravity case (Equation 1.7)

$E_{d,dst}$ = $[G + E_u + 0.3Q]$ earthquake case (Equation 1.8)

in which:

$E_{d,stb}$ = design action effect, stabilising
$E_{d,dst}$ = design action effect, destabilising
F_E = static earth pressure
E_u = ultimate earthquake action (pseudo-static earth pressure and wall inertia)

G = self-weight (dead load)
Q = imposed action (live load)

When checking stability, the self-weight of the wall and the weight of soil above any heel, is considered to be acting to stabilise the wall and should be factored by 0.9 for the gravity only load combination and 1.0 for the earthquake load combination. Surcharge loads behind the wall and acting to destabilize the wall should be factored by 1.2 (permanent, “dead”) or 0.4 (imposed, “live”) for the gravity only load combination and 1.0 or 0.3 respectively for the earthquake load combination.

Modes related to *strength* of structural elements should be checked using the following load combinations:

$E_d = [1.2G + 1.5F_E + 0.4Q]$ gravity case (Equation 1.9)

$E_d = [G + E_u + 0.3Q]$ earthquake case (Equation 1.10)

in which:

E_d = design action effect

Surcharge loads behind the wall which are acting to destabilise the wall are increasing loading on the wall and should be factored by 1.2 (permanent, “dead”) or 0.4 (imposed, “live”) for the gravity only load combination and 1.0 or 0.3 respectively for the earthquake load combination.

7.4. Resistance Factors

For ULS deformation modes related to stability of the *retaining structure*, using the load combinations and factors given above, the following resistance factors from B1/VM4 are recommended for *gravity* design of retaining walls:

UPDATE:
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Table 3: Resistance factors for gravity design of retaining walls for ULS

Deformation mode	Φ
Foundation bearing capacity	0.45 – 0.60
Sliding on base	0.80 – 0.90

For *earthquake* design using the simplified pseudo-static design procedure including the W_d factor, no resistance factors need be applied to the calculated resistance because it is implicitly assumed that soil yielding may occur during acceleration peaks.

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For modes related to stability of the *ground*, including deep seated slippage and rotation of embedded walls (global instability), the following factors of safety should be achieved:

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Table 4: Factors of safety for pseudo-static design of earth retaining structures

Load case	Factor of Safety
Static, gravity case	1.5
ULS earthquake case	1.0

Explanatory notes for Table 4

1. Surcharge loads should be included in the calculation of Factor of Safety using the load combinations and load factors given in Equations 1.6 to 1.8.
2. These values of Factor of Safety are for *moderately conservative* estimates of soil parameters, and for soils that are not subject to significant loss of strength with straining.

The *strength* design of structural elements should be carried out using the appropriate material codes including relevant strength reduction factors.

7.5. Gravity load case

For the gravity load case, *moderately conservative* soil parameters should be assumed (ie saturated and softened, highest water table where relevant). Long term drained parameters should typically be employed in analysis of the gravity load case.

For Port Hills loess, the long term drained parameters c' = 0, ϕ' = 30 degrees are often assumed for the purpose of calculating earth pressures for residential retaining walls in the Port Hills. Other parameters may be appropriate depending on the retained soil and backfill and the results of the site investigation, as determined by the geotechnical engineer. Provided that adequate drainage provisions are made, it may be assumed that there will be no water pressure acting against the wall.

Most residential walls are sufficiently flexible that the soil may be assumed to be in the active condition for the ULS and the soil pressure calculated using k_a. A certain amount of wall movement is required for the active soil condition to develop in the soil behind the wall - approximately 1% of wall height. For cases where no significant movement is acceptable at the SLS (eg, Case 1 in Figure 1) a higher value of earth pressure (typically K_0) should be assumed.

For stiffer walls (eg concrete walls buttressed by return walls) higher values of earth pressure should be assumed. The gravity load component of the pressure force on stiff walls that deflect less than 0.3% of their height can be taken as the at-rest pressure (ie K_0).

The effect of backfill slope on the at-rest pressure for stiff walls may be taken from Figure 8 for soil friction angles of ϕ = 30° to 35°. Figure 8 is based on the assumption that the increase in the at-rest gravity load component with backfill slope will be approximately the same as the increase in the gravity load active pressure.

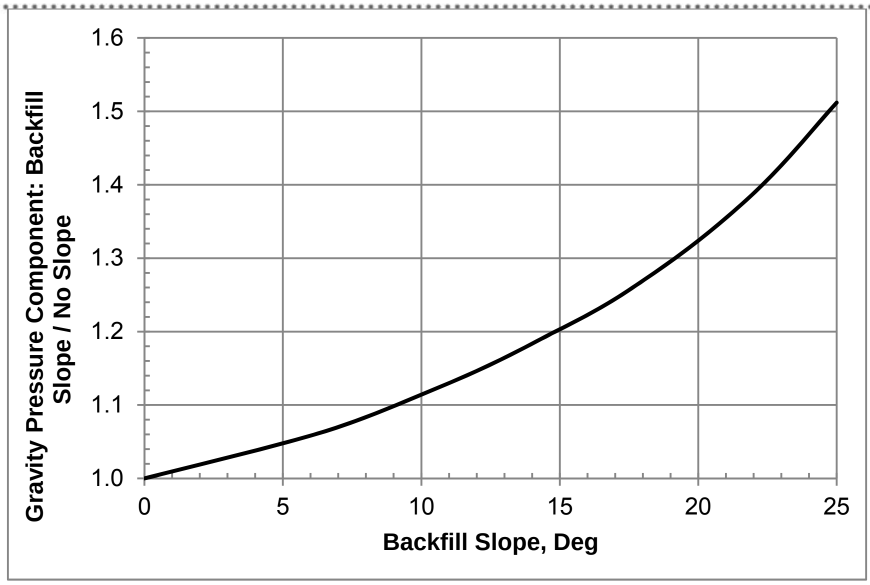

Figure 8: Increase in at-rest gravity load pressure component from backfill slope for soil friction angles $\phi = 30°$ to $35°$

The calculation of lateral earth pressure should include the effect of any surcharge applied to the retained ground (eg the weight of the dwelling in Case 3, Figure 1) and appropriate live loads (eg vehicle loads). Load factors and load combinations are given by Equations 1.4 to 1.10.

Foundations for retaining structures for the gravity load case should be designed using the methods and strength reduction factors given in B1/VM4. Wall structural elements should be designed using the methods and requirements of the relevant structural material codes.

Embedded walls (eg timber pole walls) rely on the embedment of the wall below ground level to resist overturning from earth pressure, compared to gravity walls that rely on geometry and bearing resistance to resist overturning. For embedded walls it is problematic to separate components of *load* from components of *resistance* to be able to apply appropriate load factors and resistance factors. Instead it will generally be more appropriate to assess the factor of safety in accordance with an established design procedure, such as the "Gross Pressure Method" used in the worked example (Worked Example 1). Appropriate factors of safety are given in Table 4.

Tied-back retaining walls and propped walls are typically designed using a semi-empirical procedure [eg FHWA procedure, Sabatini *et at*, (1999)].

7.6. Earthquake load case

Residential retaining walls may be designed to resist earthquake loading by considering a pseudo-static horizontal acceleration. Flexible walls are treated differently to stiff walls and tied-back or propped walls. Flexible walls are designed assuming development of active earth pressures behind the wall while stiff walls are designed using higher pressures derived from the inertia of the retained soil mass. Tied-back and propped walls are designed using a semi-empirical procedure.

7.6.1. Flexible walls

Examples of flexible walls are cantilevered concrete block walls, cantilevered timber pole walls, crib walls, and gabion walls. For the ULS load case the pseudo-static earth pressure may be calculated using K_{AE} from the Mononobe-Okabe (M-O) equations [refer NCHRP (2008) for a detailed description of the M-O method plus equations]. Charts giving values of K_{AE} for various levels of k_h, wall slope (β), wall interface friction angle (δ) and backslope angle (i) are provided in Appendix A.

UPDATE:
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For walls where no significant permanent deformation is acceptable, even for the ULS level of shaking, the full PGA should be used to calculate K_{AE} (ie, set W_d = 1 in Equation 1.3)

The inertial effect resulting from the mass of the wall under acceleration k_h, including the mass of any soil located above the heel, should be added to the calculated lateral earth pressure in all cases.

The calculation of lateral earth pressure should include the effect of any surcharge applied to the retained ground (eg, the weight of the dwelling in Case 3, Figure 1).

The seismic active earth pressure may be assumed to act at a height H/3 above the base of the wall.

7.6.2. Stiff walls

The earthquake soil pressure acting on walls that deflect less than 0.4% of their height and are restrained against permanent outward sliding displacement (eg buttressed concrete basement walls) will be greater than given by the M-O equation. The earthquake component of the pressure force on stiff walls that deflect between 0.1% to 0.2% of their height can be taken as:

$$\Delta P_E = 0.6\, k_h\, \gamma H^2 \qquad \text{(Equation 1.11)}$$

Where k_h is the earthquake acceleration design coefficient (calculated using W_d = 1), H is the wall height and γ is the unit weight of the backfill.

The earthquake pressure force component on a stiff wall reduces in an approximately linear manner to the M-O earthquake force component at a wall deflection of about 0.4% of the wall height as shown in Figure 9 [Wood, (1991)].

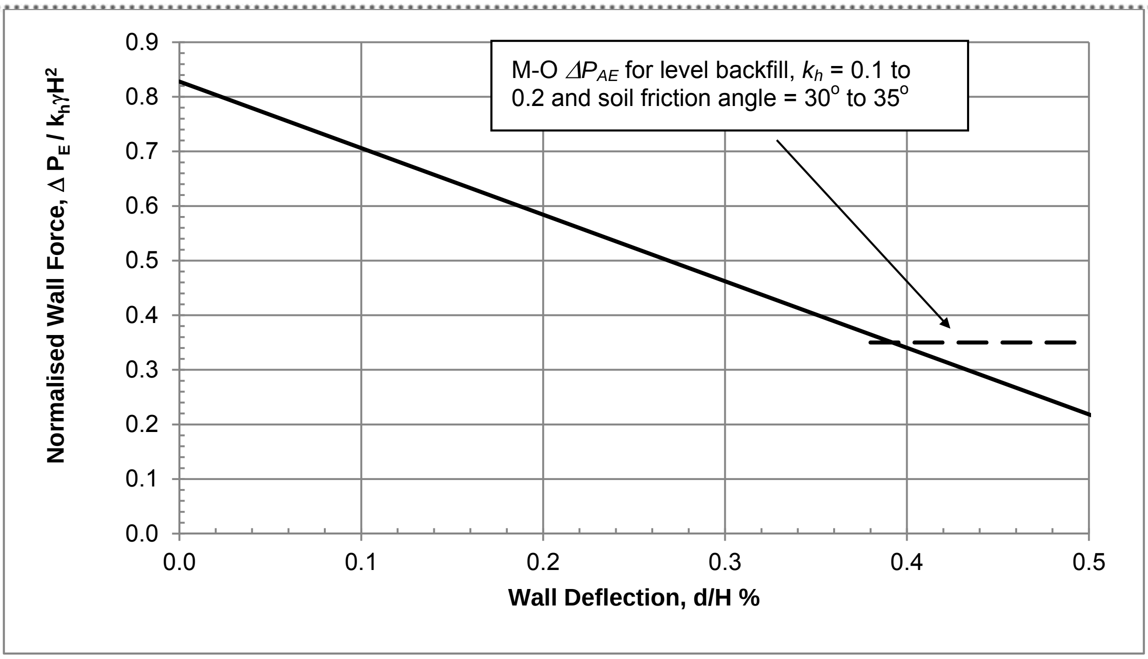

Figure 9: Earthquake pressure force component on stiff walls

The shape of the pressure distribution changes from uniform to triangular (maximum at the base of the wall) as the deflection increases from about 0.1% to 0.5% of the height. The height of the centre of pressure, h_c, for a stiff wall is shown in Figure 10.

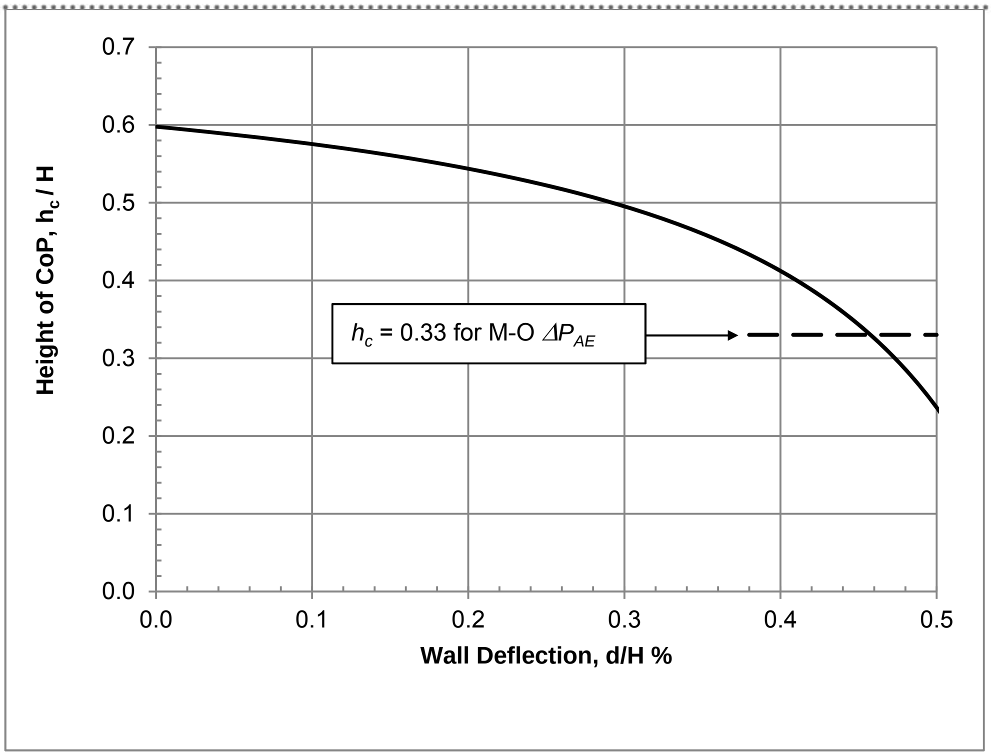

Figure 10: Centre of pressure of earthquake pressure force component on stiff walls

For stiff walls that deflect between 0.1% to 0.3% of their height the earthquake pressure component may be assumed to be uniform over the height of the wall. It will usually be necessary to carry out an iterative analysis to calculate the earthquake pressure force compatible with the deflection.

Backfill slope will result in a significant increase in the earthquake pressure component on stiff walls. Figure 11 shows the ratio of the earthquake pressure component for a backfill slope over the pressure component for horizontal backfill [Wood and Elms, (1990)].

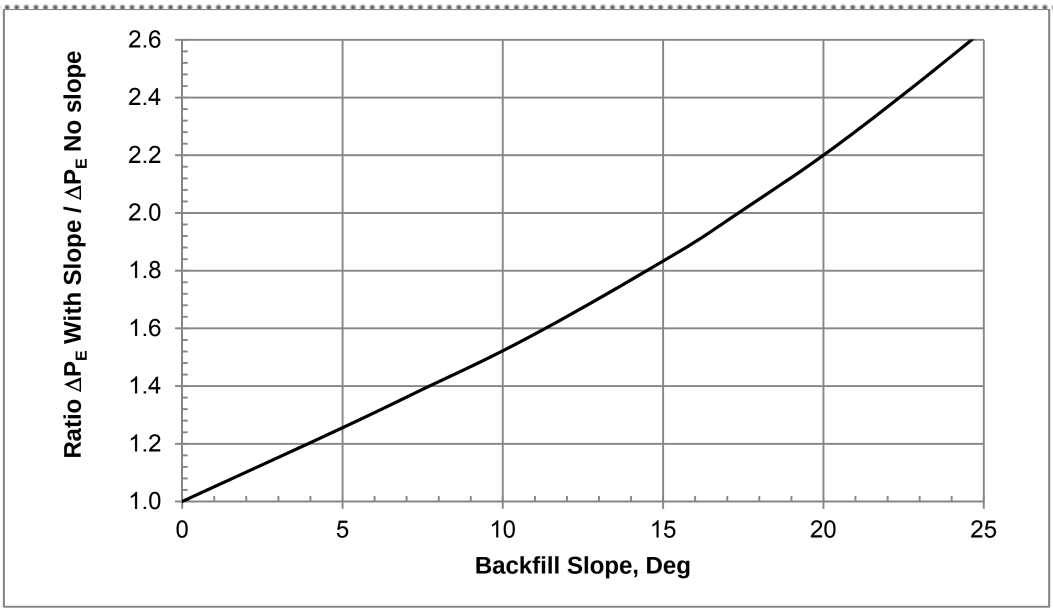

Figure 11: Increase in stiff wall earthquake pressure component from backfill slope

7.6.3. Embedded walls

Embedded walls (eg timber pole walls) rely on the embedment of the wall below ground level to resist overturning from earth pressure, compared to gravity walls that rely on geometry and bearing resistance to resist overturning. For embedded walls it is problematic to separate components of *load* from components of *resistance* to be able to apply appropriate load factors and resistance factors. Instead it will generally be more appropriate to assess the factor of safety in accordance with an established design procedure, such as the "Gross Pressure Method" used in the worked example (Worked Example 1). For the earthquake load case, K_A and K_P are replaced by K_{AE} and K_{PE} calculated using the M-O equations with the factor of safety for the earthquake case given in Table 4.

7.6.4. Tied-back and propped walls

Special design procedures are required for tied-back walls and propped walls. Guidance for calculation of earthquake induced lateral earth pressures for tied-back walls is given by McManus (2009) based on the FHWA [Sabatini *et al*, (1999)] design procedure for gravity walls, refer Worked Example 4.

7.6.5. Walls not requiring specific earthquake design

The acceleration design coefficient for Case 5 and 6 flexible walls will be $k_h \leq 0.06$ for the Christchurch Port Hills. The gravity load case for these cases will usually govern the design of retaining structures and the earthquake load case need not be considered by designers unless the backfill friction angle $\phi < 30$ degrees or the backslope angle $i > 15$ degrees.

7.6.6. Global stability

In circumstances where there is sloping ground above and/or below a retaining wall it is recommended that a global stability analysis is undertaken incorporating the effects of seismic acceleration. For such analyses seismic loads may be determined following the same approach as adopted for retaining wall design including consideration of topographic amplification (A_{topo}) and, if permanent displacement is acceptable, the use of displacement (W_d) factors. Appropriate factors of safety are given in Table 4.

Retaining walls to be constructed within the areas of mass movement in the Port Hills identified in the *Stage 1 GNS report into ground damage on the Port Hills* [GNS, (2013)] and referred to as “toe slump” areas require additional care because of the presence of highly disturbed soils and pre-existing failure planes. Global stability analysis will be required following a careful site investigation by an experienced geotechnical engineer or engineering geologist.

7.6.7. Soil parameters

For the earthquake load case, the soil parameters may be assumed for more *average* conditions than for the gravity load case (ie partially saturated, average water table). Short term, undrained parameters for cohesive soils are typically employed in analysis of the earthquake load case.

For Port Hills loess it is recommended that drained shear strength parameters (eg c = 0, ϕ = 30 degrees) be used for calculating wall loading because of the risk of shearing along pre-existing cracks or crack formation within the retained loess during strong shaking. Undrained strength parameters may be appropriate for calculating foundation bearing and passive soil resistance depending on the soil conditions, as determined by the geotechnical engineer following a site investigation. Care is required in Port Hills loess because the undrained shear strength varies significantly depending on moisture content. The strength of primary air-fall loess (intact loess) is much greater than re-worked loess (colluvium) [eg Hughes, (2002)].

7.6.8. Structural Design

Wall structural elements should be designed using the methods and requirements of the relevant structural material codes.

7.6.9. Vertical acceleration

The effect of vertical ground acceleration during earthquakes does not need to be specifically considered when designing residential retaining walls. Based on the assumption of coincident peaks in both the vertical and horizontal ground accelerations, Bathurst and Cai (1995) showed that the increase in earth pressure from vertical accelerations is less than 7% when the horizontal seismic design coefficient is less than 0.35. Whitman and Liao (1985) showed that when the peak ground acceleration is less than 0.4g vertical accelerations increase permanent outward sliding displacements by less than 10%. These two studies indicate that, at the level of design accelerations being considered in the Guidance, vertical accelerations can safely be ignored when calculating both the forces acting on the wall and the outward wall displacements.

8. General recommendations and observations

8.1. Wall backfill

Experience from the Canterbury earthquakes shows that the use of natural, river rounded drainage gravel as the backfill material behind retaining walls should be avoided where possible. During strong shaking, flexing of the wall permits the rounds to settle and prevent the wall from returning to its original position, effectively “jacking” the wall out of plane. Crushed aggregates, well compacted should be used in preference to rounded metal.

Irrespective of the backfill used, some settlement of the backfill behind retaining walls should be expected and allowance made in design.

8.2. Supervision and construction issues

8.2.1. Supervision

It was apparent that construction quality played a part in the performance of poorly performing retaining walls in the Port Hills. It is therefore recommended that:

- an appropriately skilled and experienced contractor is selected to undertake the retaining wall works
- contract specifications are carefully drafted
- the design assumptions are confirmed at key stages during the construction of the wall – this will require site supervision to be part of the designer’s scope of services to the client
- the works contract and manufacturers specifications are adhered to.

8.2.2. Health and safety and property damage

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When demolishing and rebuilding a residential retaining wall or building a new wall special care is required to avoid creating health and safety issues for construction personnel and/or damage to adjacent buildings, services and land (eg. through the collapse of a temporary works cut slope). Responsibility for the design of the construction method, including any temporary works, should be clearly identified and understood by all of the contracting parties. Excavations required for the construction of a retaining wall should be designed to ensure adequate stability. Special consideration should also be given to the short term stability of cut slopes and the possible consequences during construction both above and below the retaining wall. This is especially important where the ground conditions and/or site geometry are complex or constrained, or where the site is likely to be exposed to adverse weather conditions. Advice from a qualified professional engineer with appropriate expertise is recommended when demolishing, rebuilding or building a new residential retaining wall.

8.3. Timber-crib walls

Stretchers should be nailed to headers. Joints in stretcher units should be positively fixed using suitable timber connectors. Joints in stretchers should be avoided at the header connection as there is insufficient end distance to make a satisfactory nailed connection of the ends of the stretchers to the header.

Capping beams were found to be effective in providing restraint and robustness at the top of the wall.

Angular gravel backfill is preferred to rounded gravel.

8.4. Geometry

Where possible the face of the retaining wall should be sloped back towards the retained soil (eg by 1H:10V). This will allow some seismic induced movement to occur without giving the appearance that the wall is leaning over and at the point of failure.

9. References

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10. Worked examples

Worked Example 1: Cantilever timber pole retaining wall

Worked Example 2: Concrete cantilever retaining wall

Worked Example 3: Crib retaining wall

Worked Example 4: Tied back retaining wall

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Additional worked examples may be provided on the MBIE website as they are developed.

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Appendix A:

Application of Mononobe-Okabe equations with high acceleration and/or high back-slope angle

Common wisdom among engineers states that the M-O equations cannot be used to calculate values of K_{ae} for retaining walls with high back-slope angles. Above certain values of acceleration, k_h, the equations have no real solutions. The higher the back-slope angle relative to the friction angle of the soil, the lower the value of k_h for which a real solution is possible.

A similar situation exists for gravity only cases (i.e. k_h = 0) with no solution for K_a possible where the back-slope angle exceeds the soil friction angle. This latter case has a simple physical explanation because the slope angle for a cohesionless soil cannot exceed the angle of repose which is equal to the soil friction angle. Efforts to increase the slope angle above the angle of repose will result in a shallow slope failure, with soil sloughing to the bottom of the slope until the angle of repose is restored. For the case where the back-slope angle, *i* , is exactly equal to the soil friction angle, ϕ, the M-O equations give a real solution for K_a, for example:

$$i = 30\,\mathrm{deg}$$
$$\phi = 30\,\mathrm{deg}$$
$$\delta = 0\,\mathrm{deg}$$
$$\rho = 30\,\mathrm{deg}$$
$$K_a = 0.75$$

Where δ = interface friction angle at the back of the wall and ρ = angle of inclination of the failure plane behind the wall. The failure plane angle is equal to the slope inclination angle (both 30 degrees in this case) and the resulting value for K_a may be interpreted as the minimum soil pressure required to stabilise an "infinite slope" failure behind the wall. (An "infinite slope" failure may be defined as a shallow slope failure with a planar failure surface parallel to the ground surface, and with the depth of the failure plane being much less than the length of the failure plane.)

The value for K_a depends also on the interface friction angle between the soil and the back face of the wall. For the case where $\delta = \phi$:

$$i = 30\,\mathrm{deg}$$
$$\phi = 30\,\mathrm{deg}$$
$$\delta = 30\,\mathrm{deg}$$
$$\rho = 30\,\mathrm{deg}$$
$$K_a = 0.866$$

Now consider the case of a retaining wall with back-slope angle i = 0 (i.e. level ground) under acceleration, k_h. For moderate levels of acceleration, the M-O equations give real values for K_{ae}, becoming greater in value for greater levels of k_h. Above a certain critical acceleration, however, no real solution is possible for K_{ae}. This critical acceleration is found to be equal to $\tan(\phi)$, for which a real solution may be found by considering the limit as $k_h \rightarrow \tan(\phi)$:

$$k_h = 0.577$$

$$i = 0\ \text{deg}$$

$$\phi = 30\ \text{deg}$$

$$\delta = 0\ \text{deg}$$

$$\rho = 8.993 \times 10^{-4}\ \text{deg}$$

$$K_{ae} = 1.333$$

In the limit, $k_h \rightarrow \tan(\phi)$ and $\rho \rightarrow 0$, i.e. the M-O equations predict that the inclination of the failure surface is parallel with the ground surface, similar to the "infinite slope" failure for the case of steeply inclined backfill. The value for K_{ae} in this case may similarly be interpreted as the minimum soil pressure required to stabilise an "infinite slope" failure behind the wall, where the "infinite slope" in this case is horizontal.

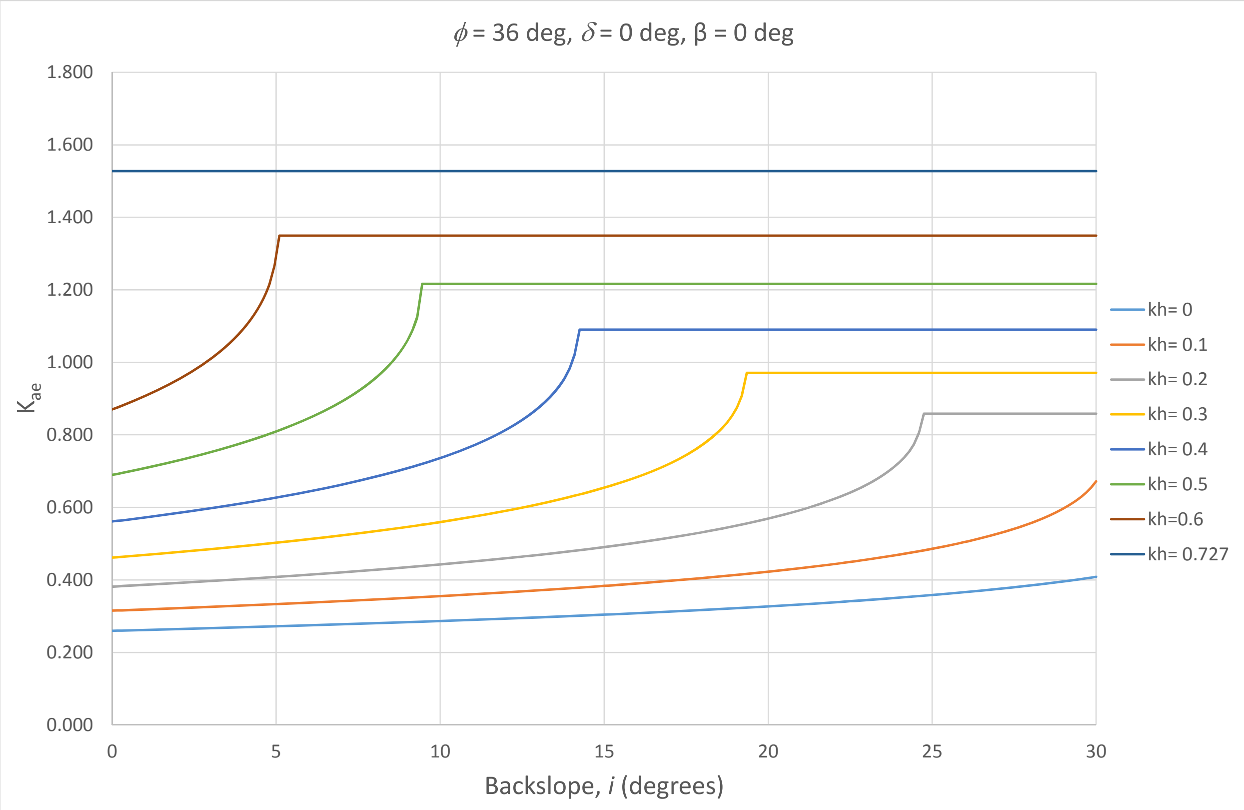
For a non-cohesive soil, the horizontal acceleration cannot be increased beyond $k_h = \tan(\phi)$ because the soil shear strength along a horizontal failure surface has already been fully mobilized ,i.e. the retained soil is effectively "base isolated" from higher horizontal ground accelerations. Therefore, the limiting value obtained for K_{ae} (1.333 in the example) might be considered the maximum possible active soil pressure (for ϕ = 30 degrees and δ = 0).

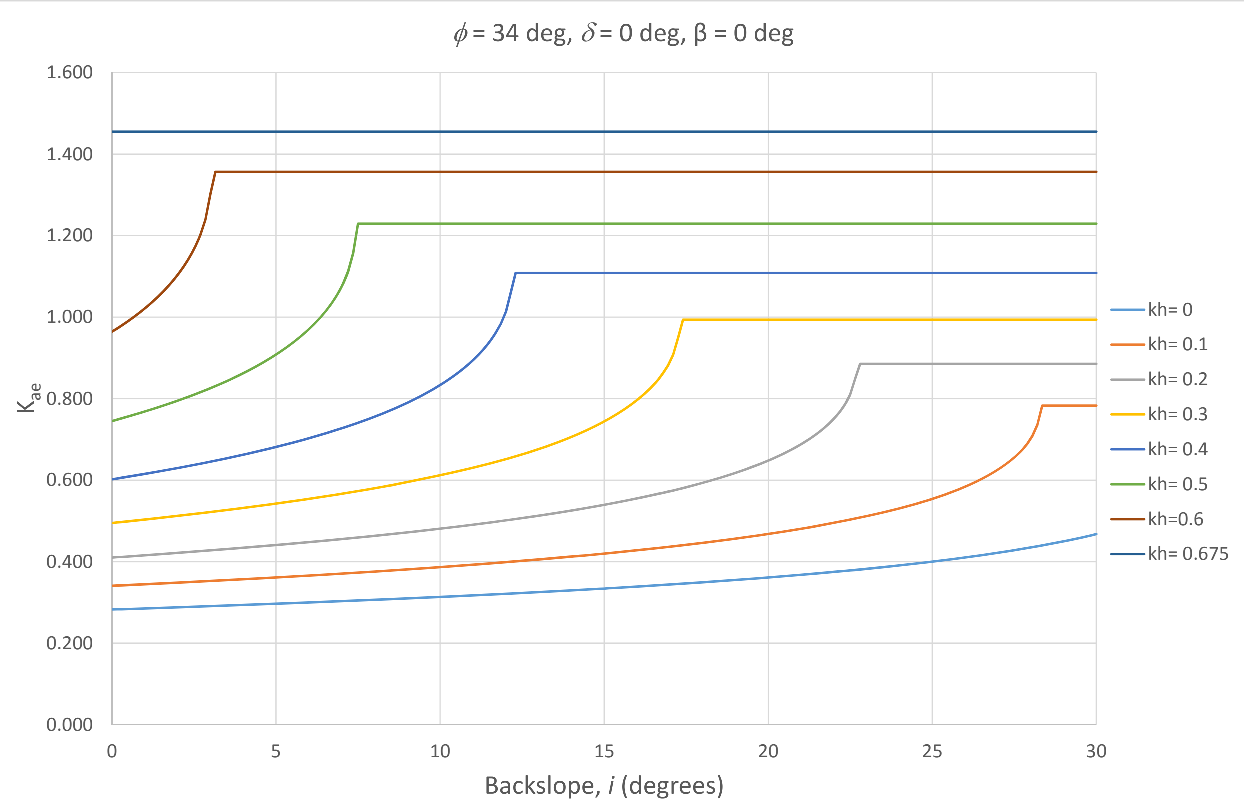
For both of the above cases, the retained soil has reached a state of "general fluidization" [Richards et. al. 1990]. Any attempt to place loads on the soil surface, for instance by placing additional soil to steepen the slope, will fail because the soil will simply "flow", very much like a viscous fluid, until the stable slope angle is restored. The minimum or "active" soil pressure required to stabilize the respective "infinite slope" will not change. Increasing the soil pressure applied by the retaining wall will not change the stability of the slope nor increase the maximum slope angle possible in either case.

For the first case (where $i = \phi$), applying any horizontal acceleration will have the effect of de-stabilizing the slope. The slope will no longer be in equilibrium and soil must flow until the slope angle is reduced to a new angle that is stable under the acceleration. The active soil pressure required to stabilise the new, stable, "infinite slope" angle is able to be calculated using the M-O equations. The wedge of soil material temporarily located above the new, stable slope angle is irrelevant to the calculation of active soil pressure for the retaining wall, just as placing soil onto the surface of a lake has no effect on the fluid pressure acting against a dam.

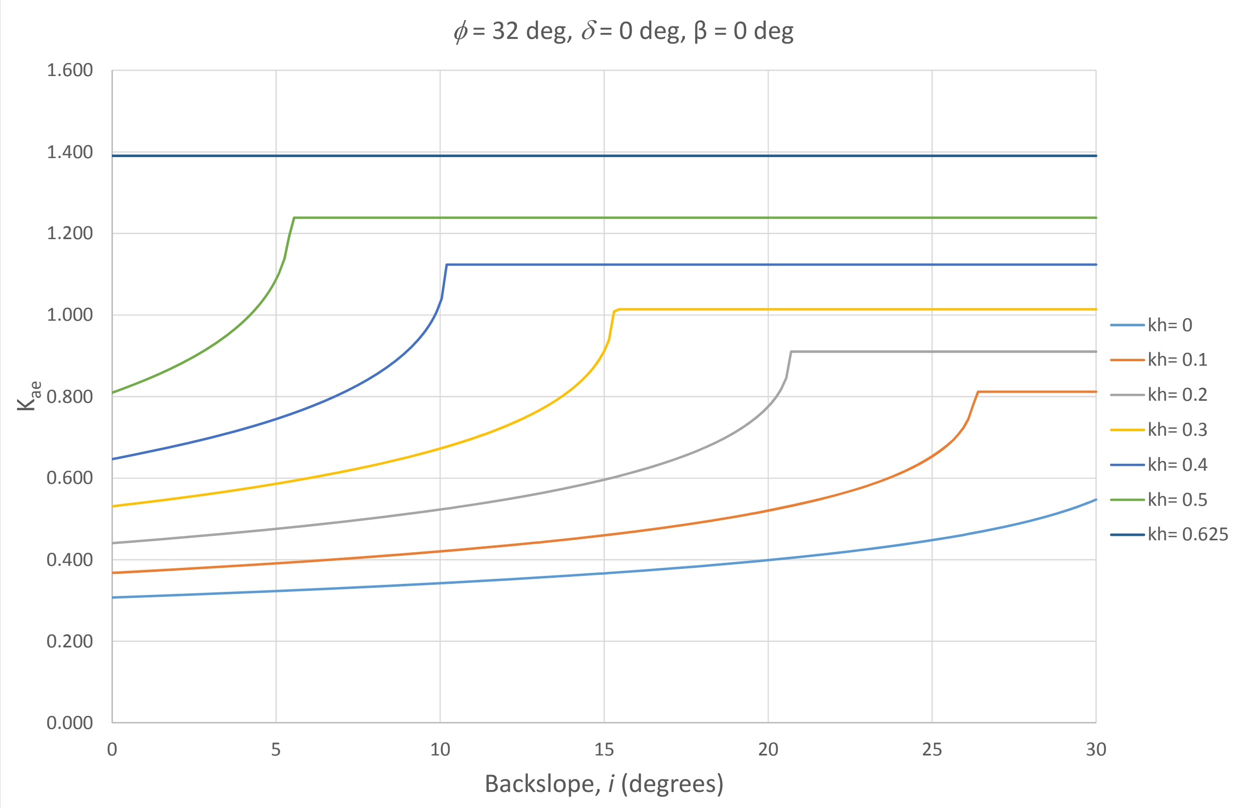
For any given horizontal acceleration k_h, the corresponding stable, "infinite slope" angle may be calculated as $i = \phi - \tan^{-1}(k_h)$. A real value for K_{ae} may be calculated for these values of k_h and i and represents the maximum value for K_{ae} for that value of k_h for all slope angles. Sample charts have been calculated and are shown below. (Note: K_{ae} collapses to K_a when k_h = 0).

Walls with vertical back-face ($\phi = 0$), no interface friction ($\delta = 0$):

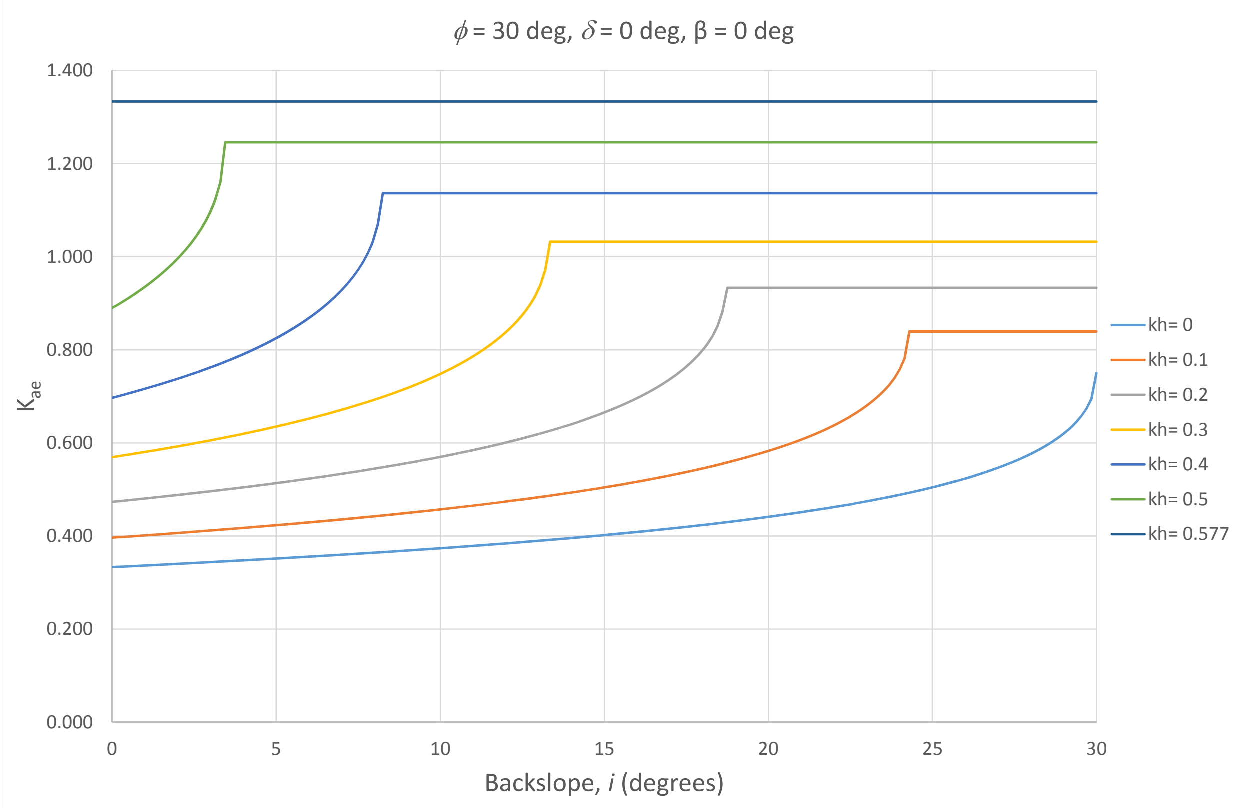


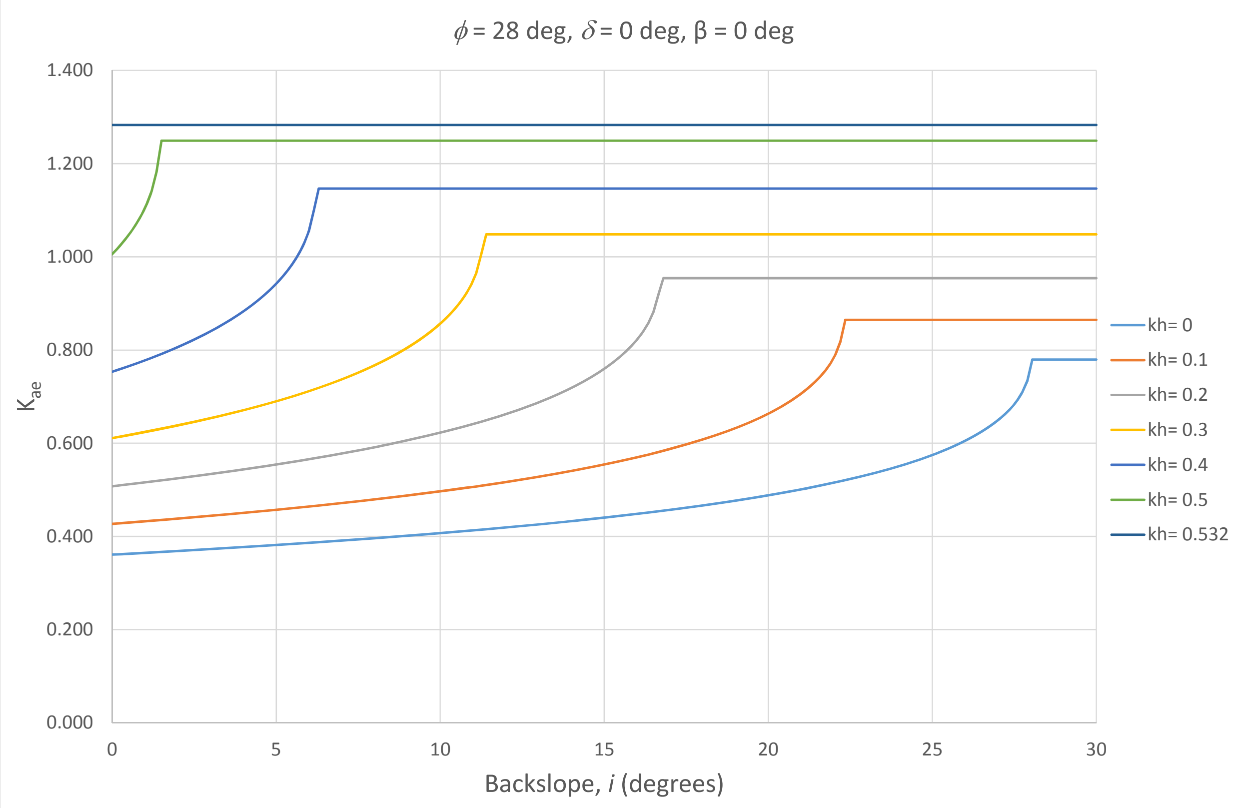


ϕ = 34 deg, δ = 0 deg, β = 0 deg
1.600
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
K_{ae}
0
5
10
15
20
25
30
Backslope, *i* (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.5
kh=0.6
kh= 0.675

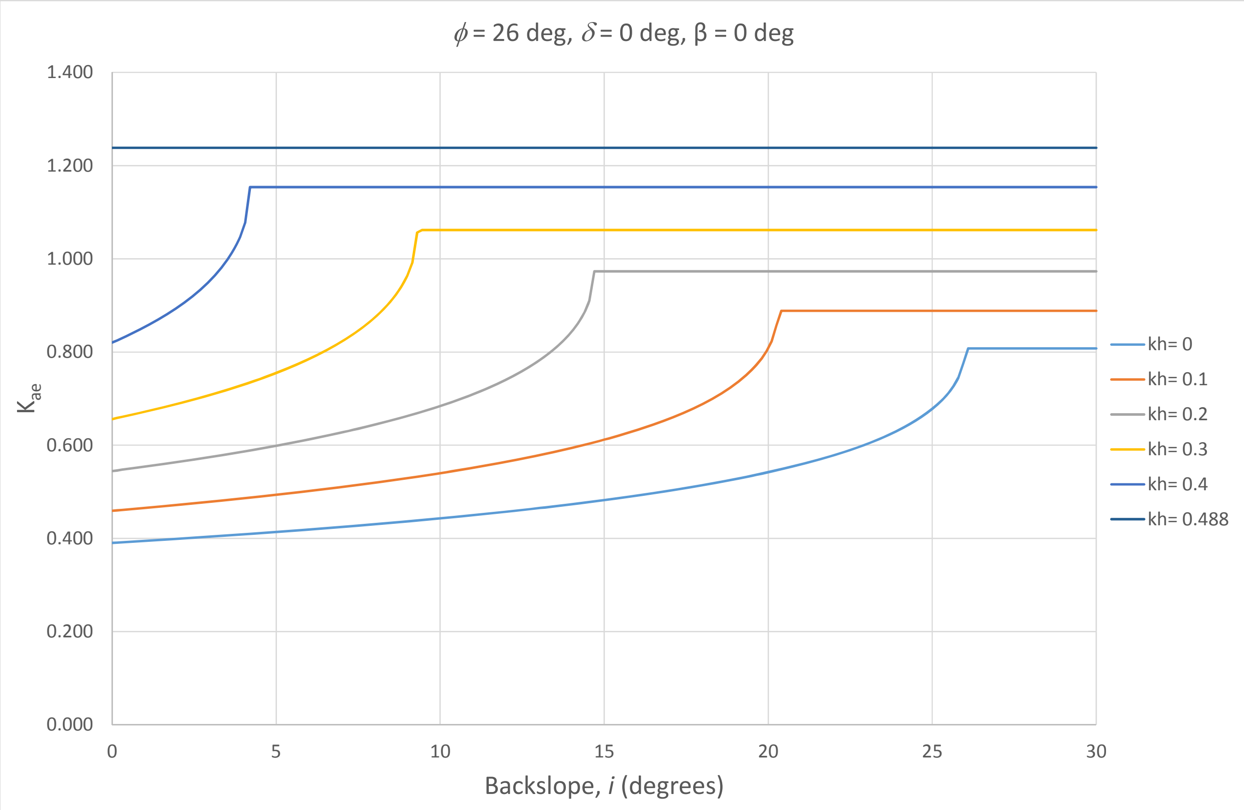


ϕ = 32 deg, δ = 0 deg, β = 0 deg
1.600
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
K_{ae}
0
5
10
15
20
25
30
Backslope, *i* (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.5
kh= 0.625


ϕ = 30 deg, δ = 0 deg, β = 0 deg
K_{ae}
Backslope, *i* (degrees)
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
0
5
10
15
20
25
30
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.5
kh= 0.577

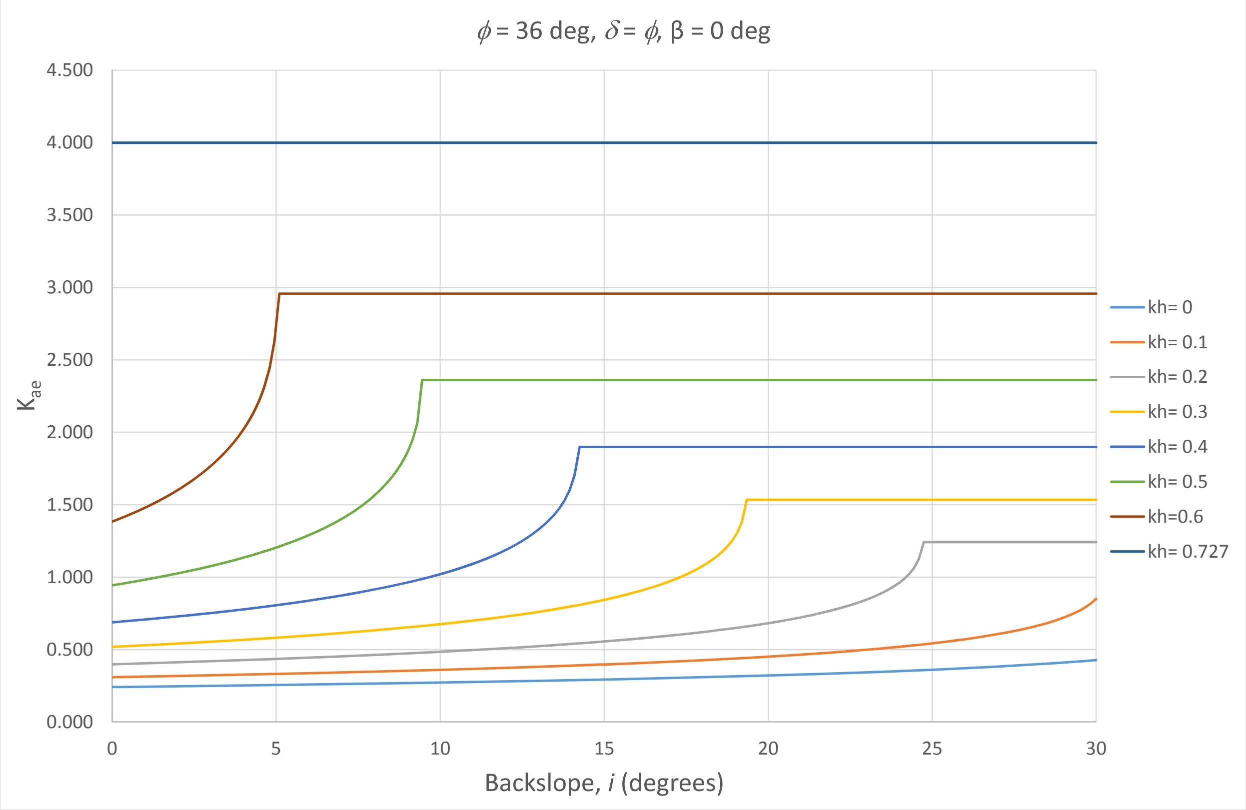


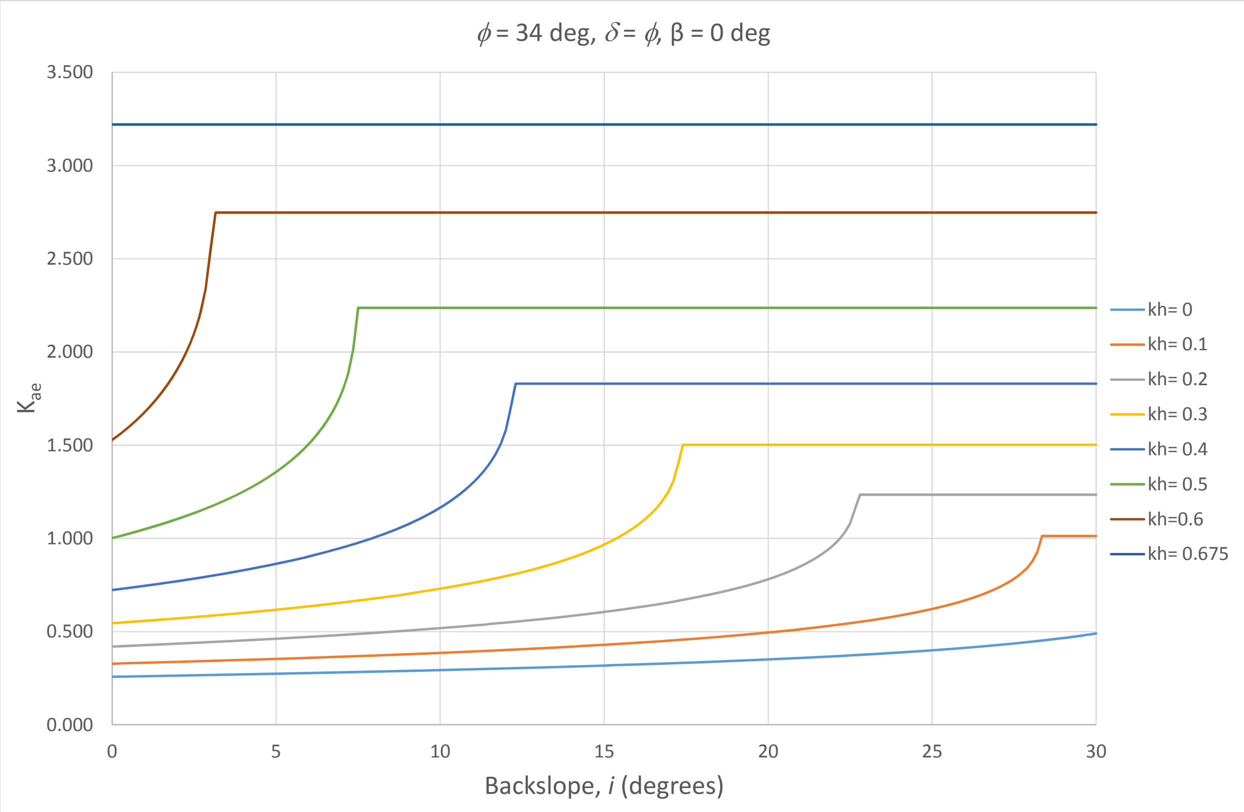
ϕ = 28 deg, δ = 0 deg, β = 0 deg
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
Kae
0
5
10
15
20
25
30
Backslope, i (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.5
kh= 0.532

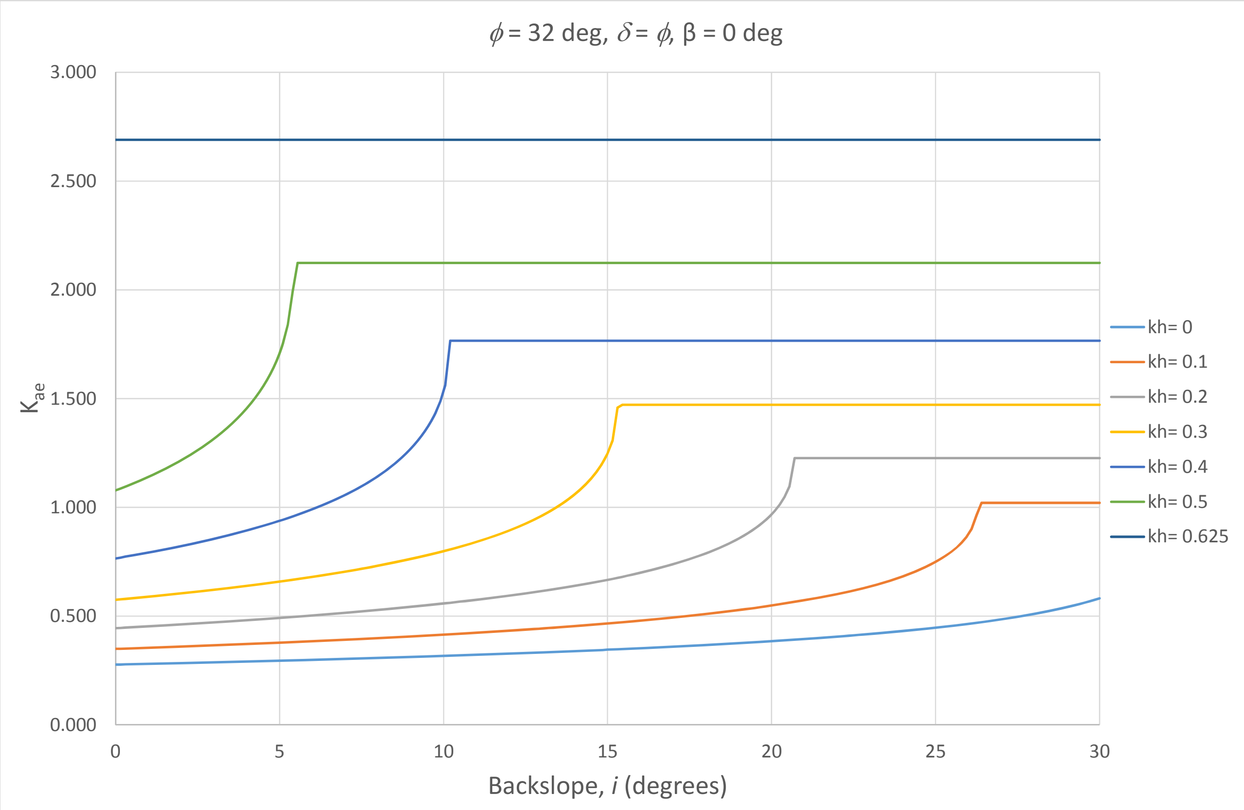


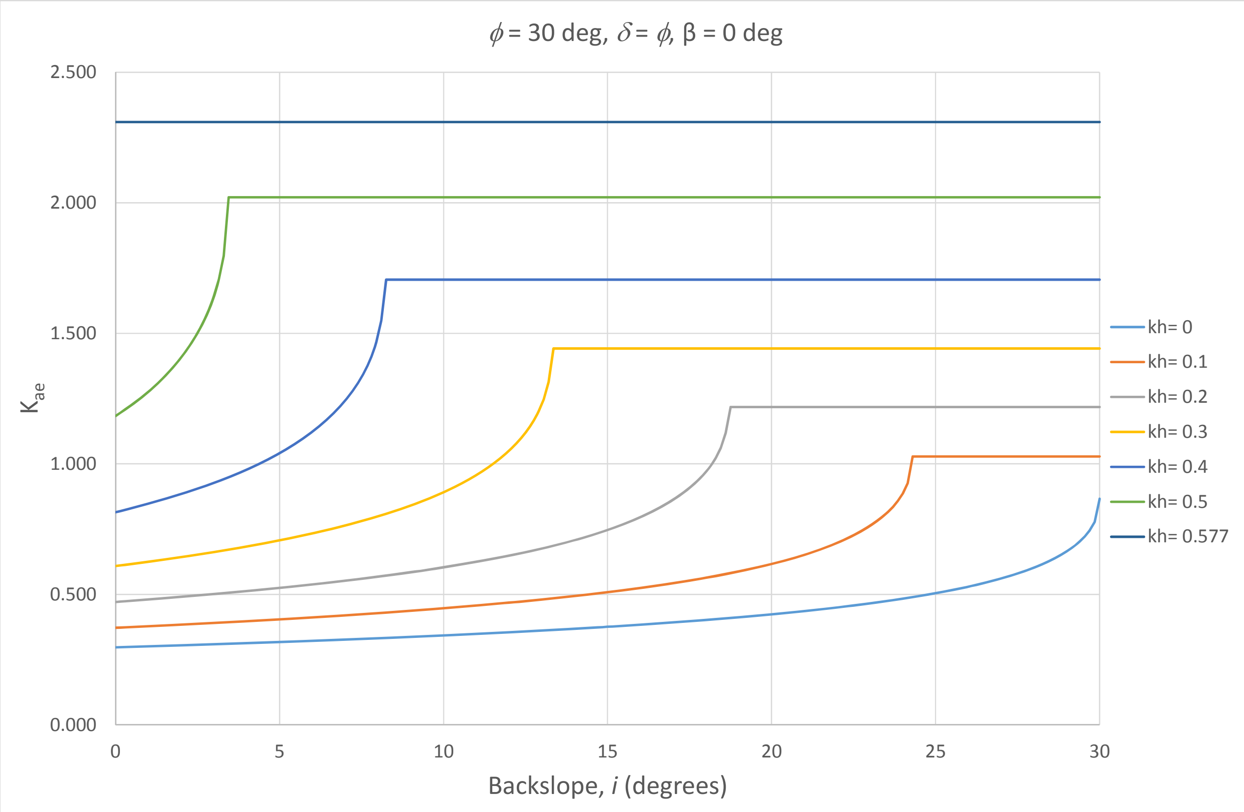
ϕ = 26 deg, δ = 0 deg, β = 0 deg
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
K_ae
0
5
10
15
20
25
30
Backslope, i (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.488

Walls with vertical back-face (ϕ = 0), full interface friction (δ = ϕ):

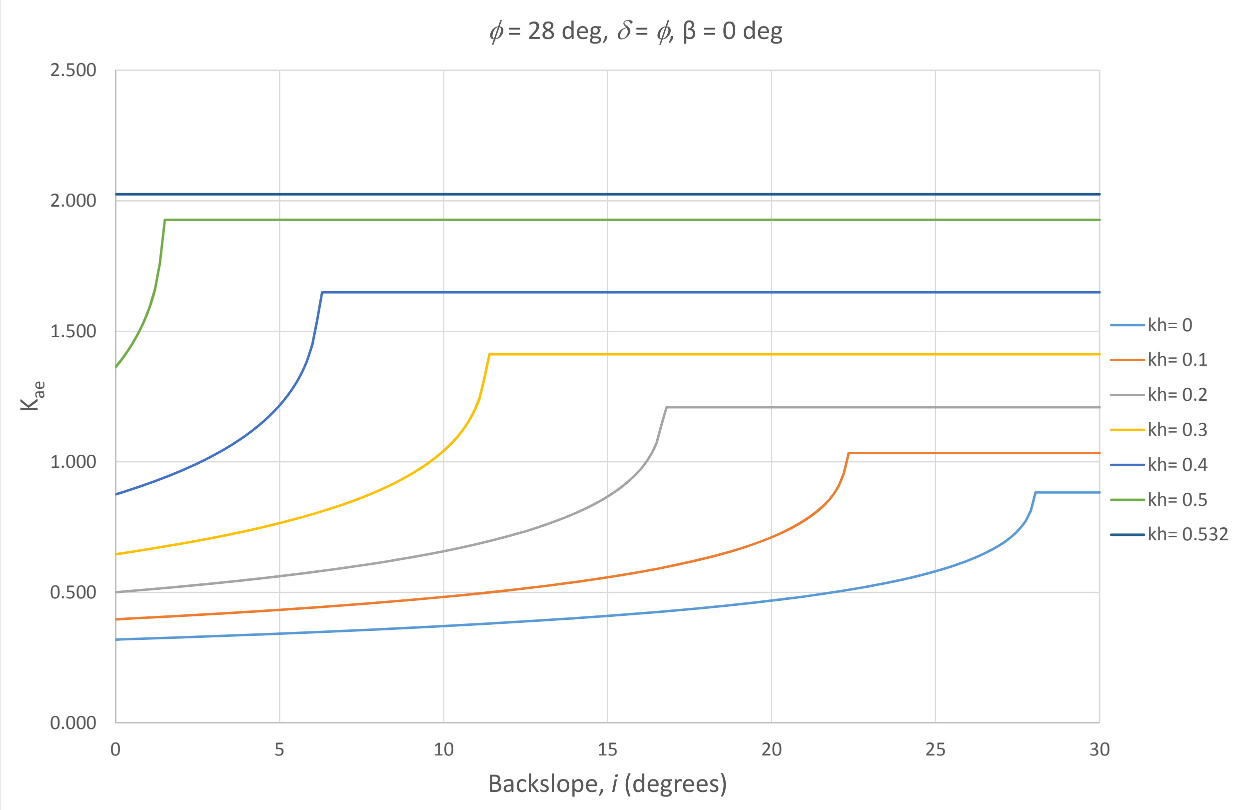



ϕ = 34 deg, δ = ϕ, β = 0 deg
3.500
3.000
2.500
2.000
1.500
1.000
0.500
0.000
K_{ae}
0
5
10
15
20
25
30
Backslope, *i* (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.5
kh=0.6
kh= 0.675

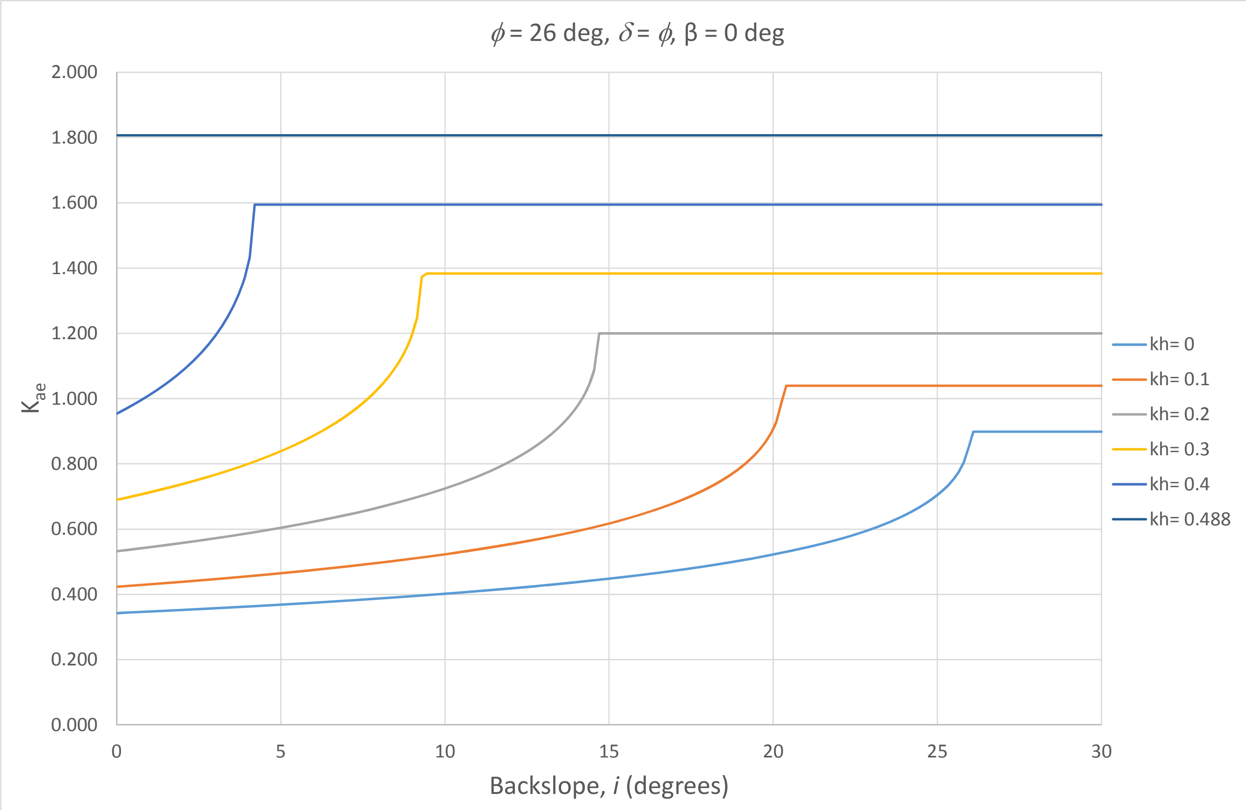

ϕ = 32 deg, δ = ϕ, β = 0 deg
3.000
2.500
2.000
1.500
1.000
0.500
0.000
K_{ae}
0
5
10
15
20
25
30
Backslope, *i* (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.5
kh= 0.625



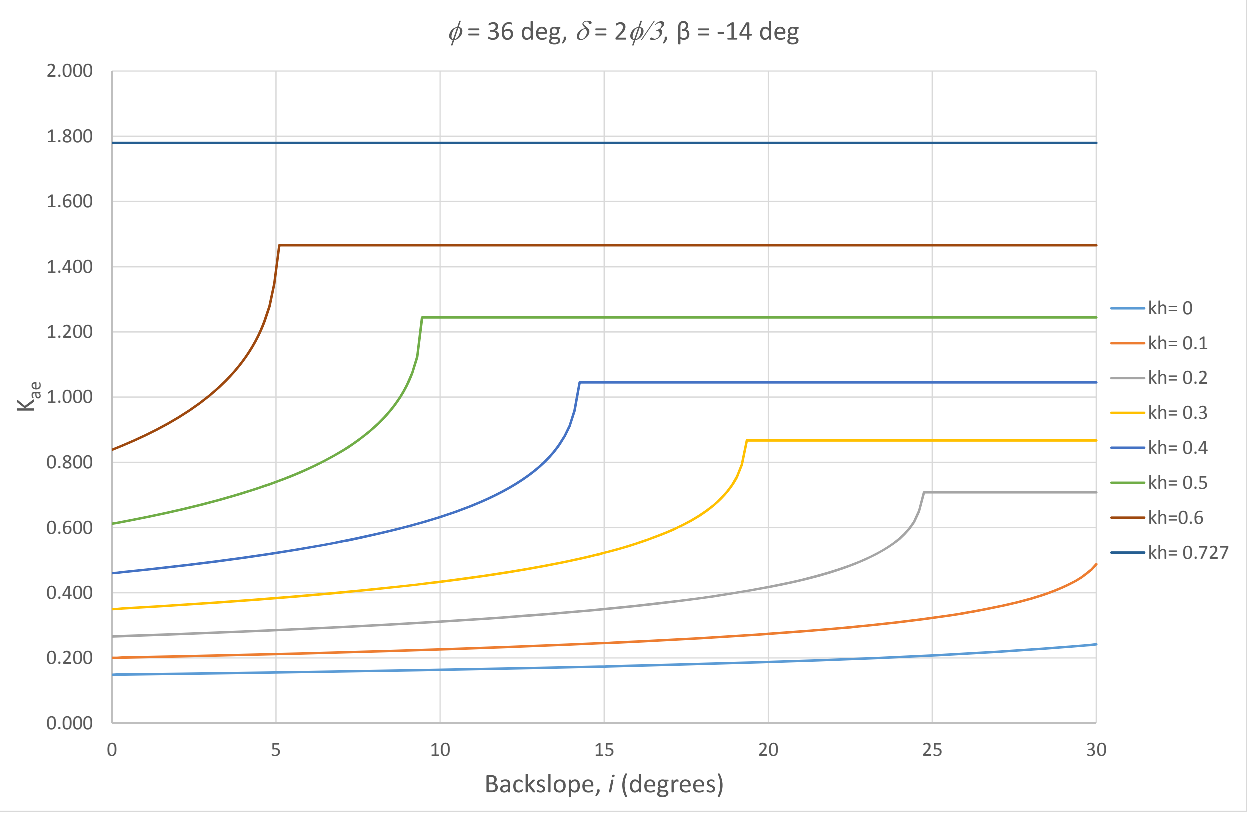
ϕ = 30 deg, δ = ϕ, β = 0 deg
2.500
2.000
1.500
1.000
0.500
0.000
K_{ae}
0
5
10
15
20
25
30
Backslope, *i* (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.5
kh= 0.577

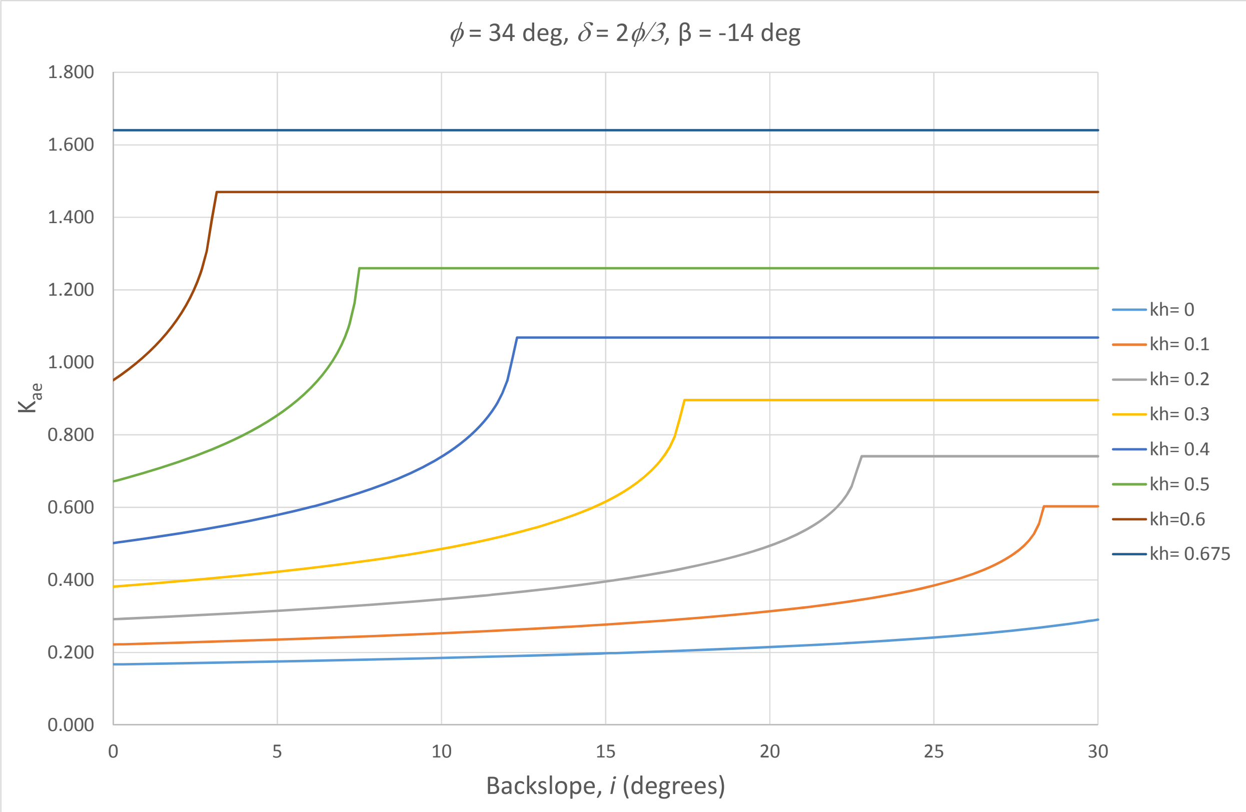


ϕ = 28 deg, δ = ϕ, β = 0 deg
2.500
2.000
1.500
1.000
0.500
0.000
K_{ae}
0
5
10
15
20
25
30
Backslope, *i* (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.5
kh= 0.532

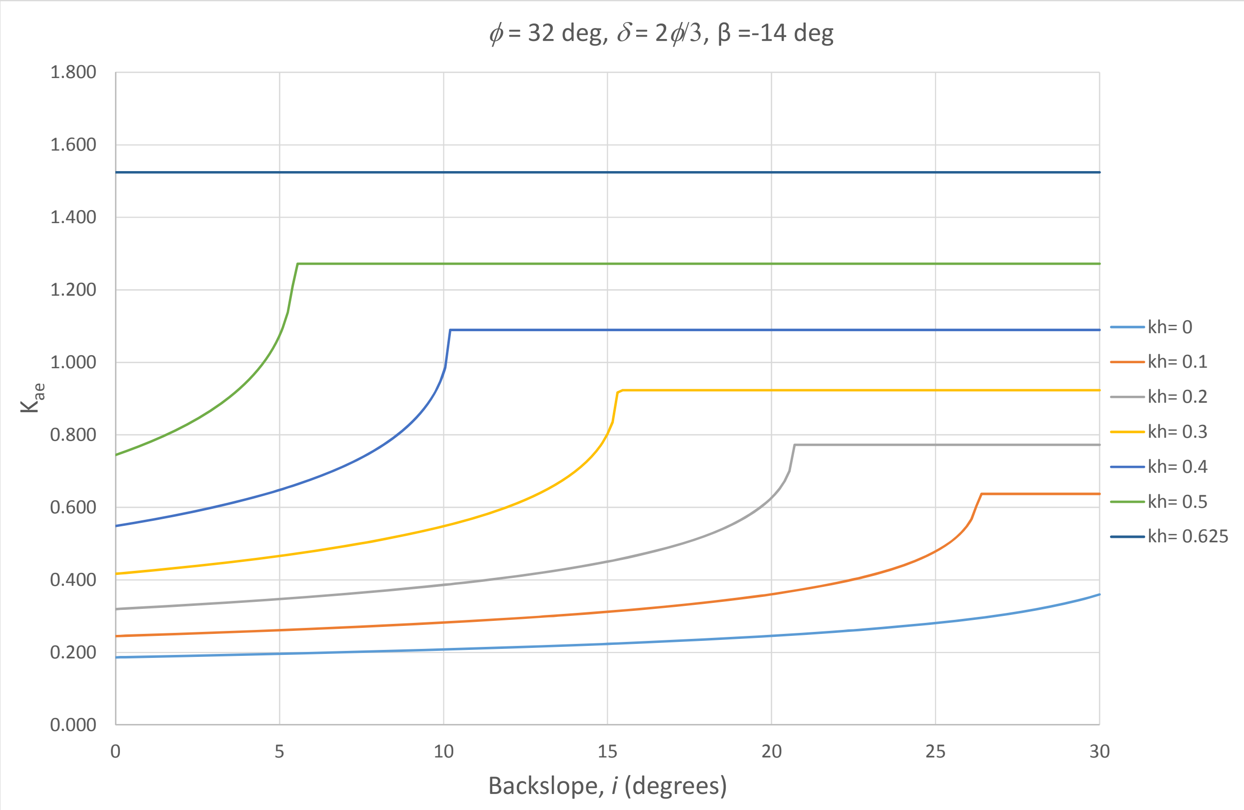

ϕ = 26 deg, δ = ϕ, β = 0 deg
2.000
1.800
1.600
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
K_{ae}
0
5
10
15
20
25
30
Backslope, *i* (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.488

Walls with backwards sloping back-face (β = -14 deg), intermediate interface friction ($\delta = 2\phi/3$):

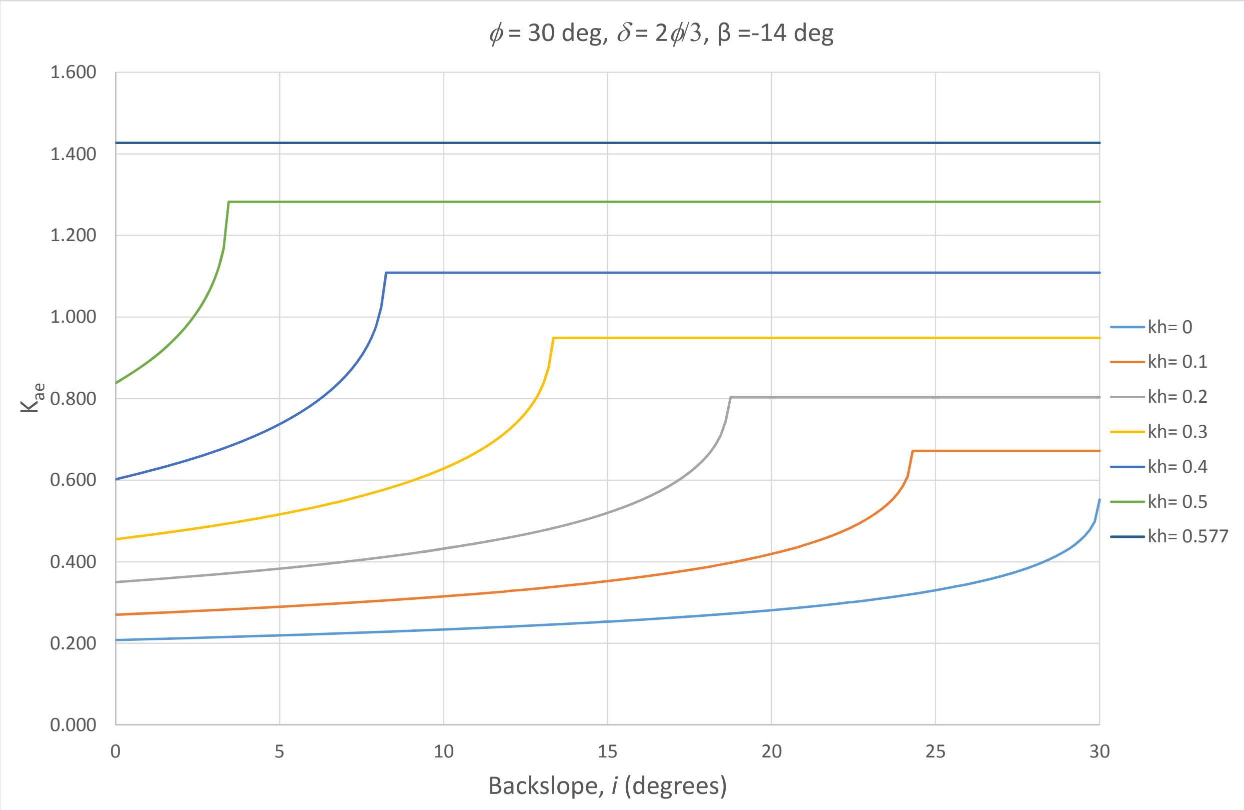


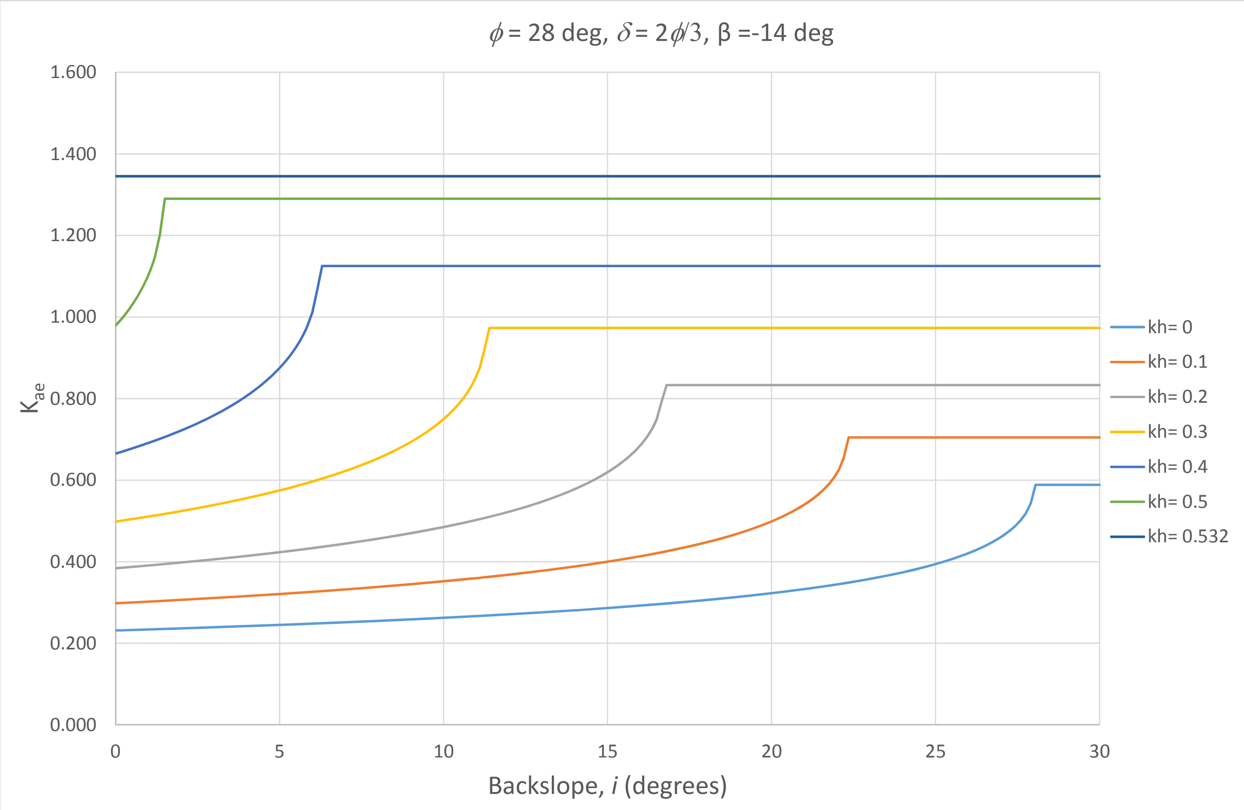


ϕ = 34 deg, δ = 2ϕ/3, β = -14 deg
1.800
1.600
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
K_{ae}
0
5
10
15
20
25
30
Backslope, *i* (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.5
kh=0.6
kh= 0.675

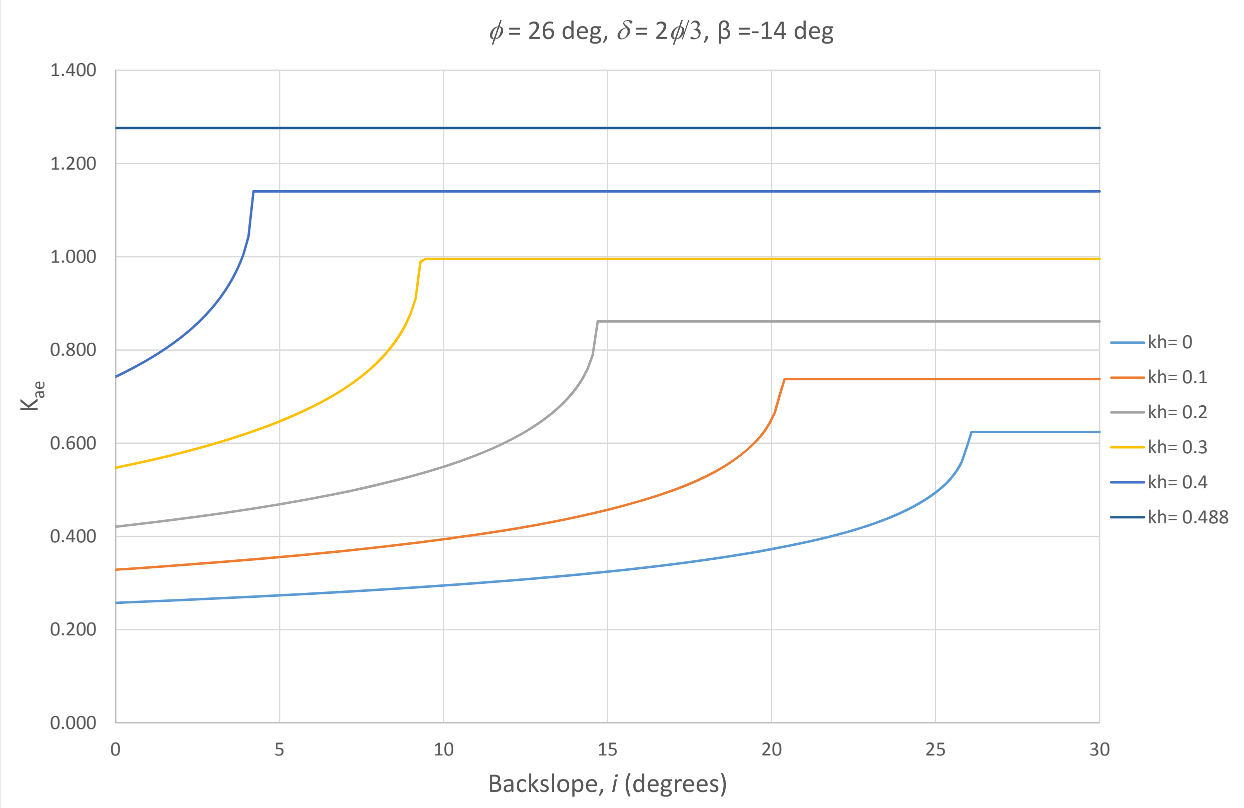


ϕ = 32 deg, δ = 2ϕ/3, β =-14 deg
1.800
1.600
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
Kae
0
5
10
15
20
25
30
Backslope, i (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.5
kh= 0.625


ϕ = 30 deg, δ = 2ϕ/3, β =-14 deg
1.600
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
Kae
0
5
10
15
20
25
30
Backslope, i (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.5
kh= 0.577



ϕ = 28 deg, δ = 2ϕ/3, β =-14 deg
1.600
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
Kae
0
5
10
15
20
25
30
Backslope, i (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.5
kh= 0.532



ϕ = 26 deg, δ = 2ϕ/3, β =-14 deg
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
K_ae
0
5
10
15
20
25
30
Backslope, i (degrees)
kh= 0
kh= 0.1
kh= 0.2
kh= 0.3
kh= 0.4
kh= 0.488

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