ROCSCIENCE

RSWall

Theory Manual

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Introduction

RSWall is a software program that can be used to perform analyses for various types of retaining walls including MSE walls, segmental walls, gravity walls, abutment walls, cantilevers. In this theory manual, an overview of the technical base associated with the engineering analysis of retaining walls is presented.

The theory covered in this manual will focus mainly on the underlying geotechnical engineering principles. It should be noted that retaining walls are typically designed to adhere to the requirements of regional design standards, which may in themselves contain minor variations, approximations and deviations from the basic theory.

1. Background

A general introduction to retaining wall systems and how to analyze them.

Figure 1.1: Illustration of a segmental block retaining wall

1.1. Retaining Walls

A retaining wall is any structure that holds back soil. Retaining walls can consist of modular units like blocks, such as in the case of the example illustrated in Figure 1.0, or baskets containing aggregate materials in the case of gabion walls. They can also consist of a single mass of concrete, such as in the case of a gravity or cantilever wall.

If a retaining wall is very tall, it may need to be reinforced by extending geosynthetic or metallic strips into the backfill soil, utilizing the self-weight of soil behind the wall to contribute to its stability. In this case, the wall is commonly referred to as a “mechanically stabilized earth” (MSE) wall.

Retaining walls are beneficial because they allow for steep changes in elevations, especially in applications such as alongside highways, on the ends of bridges, in construction, in architectural and landscaping designs, in the design of foundations and basements, among others.

1.2. Analysis

The primary responsibility of a retaining wall is to resist the driving forces exerted by the soil that it retains and transfer those loads into the underlying soil. Engineering calculations involve estimating the driving forces and comparing them against the resisting capacity.

1.2.1. Failure Modes

There are several ways in which a retaining wall system could fail and potentially lead to disastrous consequences.

Assessment of the stability of the retaining wall with respect to each type of failure is typically mandated by regional design standards distributed by governing authorities to regulate the practice of geotechnical engineering and protect public life and safety.

The following commonly occurring modes of failure can occur in retaining walls:

- Base sliding
- Overturning
- Bearing capacity failure
- Rupture of reinforcements in tension
- Pullout (or “adherence”) of reinforcements
- Connection failure between facing and reinforcements
- Internal sliding along modular units
- Crest toppling above reinforced portion of wall
- Global/overall and internal compound failure

The analysis of a wall with considering each of these failure modes is described in detail within the later sections of this manual. While other failures can occur, these are the most commonly known and checked during engineering analyses.

1.2.2. Estimation of Forces

Active Forces

The primary driving force behind the wall is typically referred to as the “active” force because the estimation of its magnitude traditionally involved assumptions about the deformation of the wall relating to Rankine’s active earth pressure theory. There are various methodologies available in existing literature, and variations are required by different design standards. These will be presented in a later section within this manual.

Resisting Forces

Resisting components against the driving forces typically consist of frictional forces or restoring moments contributed by the self-weight of the structure’s internal components, and in some cases the passive resistance of the soil at the front of the wall is permitted to be considered if the soil can be considered permanent throughout the lifetime of the structure. In the case of bearing failure, the bearing capacity of the soil below the wall is considered. The driving forces also induce tension that must be resisted by the reinforcing components.

Seismic Actions

The effect of seismic actions can also contribute adversely to the stability of a retaining wall. Seismic loading is typically represented via the addition of inertial forces and by applying forces in both the horizontal and vertical directions proportional to the structure’s self-weight. The magnitude of the active force is also affected by seismic actions.

Effect of Groundwater

Retaining walls are typically designed to drain quickly to prevent a buildup of hydrostatic pressures behind the wall. However, if the design condition requires groundwater to be considered, the groundwater is assumed to act normally to the surfaces of the wall below the water table. Both hydrostatic and hydrodynamic (in the case of seismic analysis) forces need to be considered.

1.2.3. Design Standards

The design of retaining walls is typically regulated by regional authorities via design standards. Design standards stipulate requirements and recommendations relating to the methods that engineers can use to analyze retaining walls. Examples of these include:

- AASHTO LRFD Bridge Design Specifications
- Eurocode 7: Geotechnical Design
- BS 8006-1: Code of Practice for Strengthened/Reinforced Soils
- AS 4678: Earth Retaining Structures
- CSA S6:19: Canadian Highway Bridge Design Code

The requirements in the design standards are heavily based on the theory covered in this manual.

1.2.4. Load and Resistance Factor Design

Most design standards adhere to the Load and Resistance Factors Design (LRFD) philosophy. To account for uncertainties in the magnitudes of loads and resistance parameters, this design philosophy applies design factors which increase the magnitudes of applied loads and actions based on low probabilities of exceedance and decrease the values of resistance parameters to obtain lower-bound values.

After the design factors are applied, stability is evaluated by calculating the ratio of factored resistance to factored actions, and if the ratio exceeds unity, then the design is considered to be inadequate. The procedure is analogous to computing a “safety factor”, except that the uncertainties of the actions and resistances are already considered. Depending on the design standard, this ratio can be termed “capacity demand ratio” (CDR) or “over-design ratio” (ODR).

Some design guides, such as the National Concrete and Masonry Association’s (NCMA) Design Manual for Segmental Retaining Walls, still rely on Working Stress Design (WSD) philosophy, whereby the same ratios of resistances versus actions are computed but without applying any factors to the nominal inputs of the analysis. In this case, the ratio is a true “factor of safety” (FS).

1.3. Outline

The rest of this manual focuses on various technical aspects of the analysis of a retaining wall as presented in the previous subsections.

- Methodologies for estimating forces are presented, and
- Theory relating to the assessment of each failure mode is presented

2. Estimation of Forces

A general description of retaining wall systems.

2.1. Active Forces

The estimation of active force is based on the conventional assumption that the wall will exhibit a slight outward rotation about its base (Das 2010). In this way, a wedge forms behind the wall as illustrated in Figure 2.0.

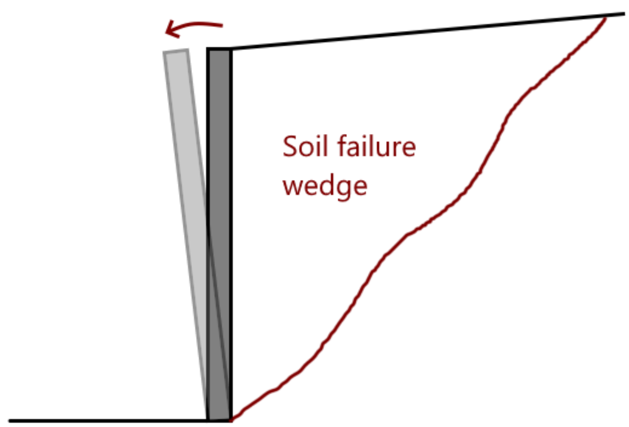

Figure 2.1: Illustration of a segmental block retaining wall

When this deformation occurs, the soil behind the wall is typically assumed to exert a linearly varying pressure distribution on the wall. Depending on whether certain factors such as soil-structure friction are considered, this distribution can vary. The following methods for estimating this pressure distribution are available in RSWall, presented in increasing order of complexity:

- Rankine (simplified)
- Rankine (generalized)
- Rankine-Bell
- Coulomb
- Generalized Wedge Method
- Kerisel-Absi (1990) Method (presented in the EN1997 Annex C1)
- Brinch Hansen Method (presented in the EN 1997 Annex C2)

In many of these methods, a value of Ka, known as the lateral earth pressure coefficient, is outputted, which relates the active stress (σ_a) to the vertical stress (σ_v) in the soil at a given depth z via the following relation:

$$\sigma_a(z) = K_a\, \sigma_V(z) \tag{2.1}$$

The active stress acts in either the lateral direction or at a slight angle from the horizontal when soil-structure interaction is considered. The magnitude of the resultant active force (P_a) is given by integrating the active stress over the depth of the analysis below ground (H), and simplifies to the following equation in the case of homogenous soil:

$$P_a = \int_0^H \sigma_a(z)\ dz = \frac{1}{2} K_a \gamma H^2 \tag{2.2}$$

Where γ is the unit weight of retained soil. For the case of homogenous soil, this resultant acts at a location one-third of the height of the analysis above the bottom.

For non-homogenous soil or soil that is partially saturated with groundwater, the effective active stress is integrated using the first part of the equation and K_a may differ for each soil layer. In some methods, the magnitude of P_a is calculated directly without computing the lateral pressure coefficient. For instance, the Generalized Wedge Method simply solves for equilibrium of the active soil wedge and outputs the resultant force.

A comparison of each method with regards to which analysis considerations it supports is presented in Table 2.0. A checkmark (✓) indicates that the analysis consideration is supported by the method, where an "X" indicates that the method neglects the effect (that is, you would get the same answer regardless of the value of the considered variable). The table also describes the shape of the failure plane assumed by the method – whether it is planar (a straight line on the 2D section) or log-spiral (a curve on the 2D section).

Table 2.0: Comparison of Active Pressure Methods

Consideration	Rankine (simplified)	Rankine (generalized)	Coulomb Method	Generalized Wedge	Kerisel & Absi	Brinch Hansen
Wall batter	X	X	✓	✓	✓	✓
Backslope angle	X	✓	✓	✓	✓	✓
Soil-structure friction	X	X	✓	✓	✓	✓
Cohesion	X	X	X	✓	X	✓
Active boundary	Planar	Planar	Planar	Planar	Log-spiral	Log-spiral
Outputs Ka value	✓	✓	✓	X	✓	✓

2.1.1. Rankine (simplified)

This is the simplest method, as it neglects all the effects of wall batter, backslope angle, soil-structure friction and cohesion. Specifically, it assumes that the wall is vertical and that the backslope topography is flat. The lateral earth pressure coefficient is estimated as follows:

$$K_a = \tan^2\left(45° - \frac{\phi}{2}\right)$$

(2.3)

Where ϕ is the friction angle of the retained soil. As this method often oversimplifies the analysis, many design standards prohibit its use in the final design of a retaining wall.

2.1.2. Rankine (generalized)

An extension of the Rankine (simplified) method to consider the slope of the backfill (β , positive sloping upwards away from the wall) is presented below.

$$K_a = \frac{cos\beta - (\cos^2 \beta - \cos^2 \phi)^{\frac{1}{2}}}{cos\beta + (\cos^2 \beta - \cos^2 \phi)^{\frac{1}{2}}} cos\beta$$

(2.4)

Note that the Rankine-Bell methodology in RSWall also adopts this equation when estimating the active forces.

2.1.3. Coulomb

The Rankine method ignores the effect of soil-structure friction and wall batter. The Coulomb equation for lateral earth pressure considers the effect of the soil-structure friction angle (δ) and wall batter (ω):

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

(2.5)

The soil-structure friction angle is typically estimated to be some fraction of the friction angle (e.g. 2/3 times the value). Depending on the design standard, requirements and restrictions for assuming this value may be stipulated.

The resultant active force calculated using this method acts act an angle δ offset above the normal to the structure's surface.

2.1.4. Generalized Wedge Method

This method involves the following procedure, which is outlined in detail by Caltrans (2011):

- Determine a trial wedge by drawing a failure plane at a trial inclination angle from the horizontal (α), extending from the base of the wall to the ground surface
- Calculate the (effective) weight of soil contained within the resulting soil wedge
- Calculate the cohesive force along the length of the failure plane
- Calculate the adhesive force along the soil-structure interface

- Add any other forces acting on the soil wedge, including surcharges and the porewater pressure resultant if applicable
- Assume directions for the reaction force acting on the failure plane ($\alpha - \phi$ from vertical) and resultant reaction from the structure active force (offset angle δ from the direction normal to the structure's surface). The latter is the equal and opposite reaction to the active resultant, P_a.
- Solve for static equilibrium to determine the magnitudes of the reaction and resultant active forces.
- Repeat the procedure for varying values of the trial inclination angle (α) and select the maximum value of P_a.

2.1.5. Kerisel-Absi Method

This is a complex method that assumes a log-spiral-shaped failure surface behind the wall. Kerisel & Absi (1990) published tables on which the value of K_a can be obtained for varying values of friction angle, soil-structure friction, wall batter, and backslope inclination angle.

One can alternatively consult the charts in Eurocode 7, Annex C1 (European Commission, 2004) to estimate the value of K_a from this method.

2.1.6. Brinch Hansen Method

This is another log-spiral method which involves estimating the tangent angles of the failure surface at the wall and at the ground. The detailed procedure is presented in the Eurocode 7, Annex C2 (European Commission, 2004).

2.2. Passive Forces

The soil in front of the wall can be considered to provide resisting forces in some cases. Where permitted, the resisting soil is assumed to exert a stress distribution based on the following relation:

$$\sigma_p(z) = K_p \, \sigma_V(z) \tag{2.6}$$

Where σ_p is the passive stress and K_p is the passive pressure coefficient. The resultant passive force is obtained by integrating the above equation along the depth of the soil.

The following methods for computing passive forces are available in RSWall. Their availability depends on the selected design standard for the analysis.

- Caquot & Kerisel (1949)
- Kerisel-Absi (1990) Method (presented in the EN1997 Annex C1)
- Brinch Hansen Method (presented in the EN 1997 Annex C2)
- Rankine-Bell method
- Use at-rest lateral pressure

The first three are complex log-spiral methods for which values of K_p can be estimated by reading charts and tables found in the respective references.

The Rankine-Bell method assumes a variation of the Generalized Rankine equation whereby the numerator and denominator terms are inverted.

The passive pressure assumption may not always be conservative, as it can result in a large resisting force at the front face of the structure. To account for this, some engineers use the at-rest lateral pressure method which conservatively assumes $K_p = K_o = 1\text{-}\sin\phi$.

2.3. Tensile Forces

The following methods for computing tensile forces in the reinforcement layers are available in RSWall. Their availability depends on the selected design standard for the analysis.

- Simplified method
- Coherent gravity method
- Tieback wedge method
- Stiffness method

Depending on the selected design standard, the actual steps for each method may vary, even if the method shares the same name across different regions. However, the general assumptions of each method are presented as follows.

2.3.1. Simplified Method

This method is presented in AASHTO (2020) and technically adopted by the NCMA (2009) with minor variations. The maximum tensile force (T_{max}) in each layer of reinforcement is calculated via the following equation:

$$T_{max} = K_r \sigma_v S_v \tag{2.7}$$

Where K_r is the internal active pressure coefficient calculated using the properties of the internal soil region, σ_v is the vertical stress on the reinforcement layer, and S_v is its tributary height. The tributary height of each reinforcement layer is assumed to be represented by the region covering half the distance to the next layer of reinforcement in either direction. For the first and last reinforcement layers, it is half the distance to the second and second-last layer, respectively.

2.3.2. Coherent Gravity Method

There are variations of this method adopted worldwide, with all having the following common assumptions and general procedure.

To determine the maximum tensile force in a reinforcement layer using the Coherent Gravity Method:

- The active earth forces acting above and behind the back of the layer are determined, along with the weights and resistances above that layer,
- A summation of driving and resisting forces and their moments about the front end of the reinforcement layer is obtained,
- The resultant vertical reaction, *R*, below reinforcement layer and its assumed location is determined by calculating the eccentricity, *e,* and magnitude of the base reaction.

- The vertical stress, σ_v, experienced by the reinforcement layer is calculated by dividing the reaction *R* over the effective bearing length of the layer, *L – 2e*.
- The maximum tensile force is then obtained using the same equation as the Simplified method.

When evaluating pullout capacity, the shape of the internal failure plane may differ across regional variations of this method. European standards use a bi-planar failure surface, whereas North American standards assume a planar failure surface.

The shape of the internal failure surface also depends on the assumptions used to compute K_r, which is typically based on simplistic methods such as Rankine (simplified) or Coulomb lateral earth pressure. RSWall will automatically select the internal lateral earth pressure method used to calculate K_r based on a combination of that which was selected for computing K_a, and the requirements of the selected design standard.

In some variations, the value of K_r is obtained as a function of depth based on the following figure.

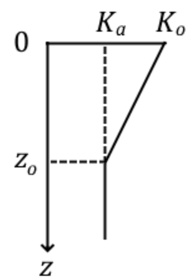

Figure 2.2: Bilinear variation of K_a with depth in the Coherent Gravity Method.

The value of reference depth z_o is typically around 6 m (20 ft), and K_o is the at-rest lateral earth pressure.

2.3.3. Tieback Wedge Method

The Tieback Wedge Method is introduced in the British Standard (BS8006-1) and is the same as the coherent gravity method except that some simplifying assumptions can be made when calculating T_{max} if the design of the wall meets the following conditions.

- the wall is vertical,
- the wall segments are uniform over the height of the wall
- the water table is not above the reinforcement layer
- uniform surcharges are assumed to start at the back of the wall and continue infinitely
- the backslope is flat

If these conditions are met, then it can be shown that T_{max}, when calculated using the Coherent Gravity Method, simplifies to the following equation:

$$T_{max} = \frac{K_{a1}(\gamma_1 h_j + q) S_v}{\left(1 - \frac{K_{a2}(\gamma_2 h_j + 3q)\left(\frac{h_j}{L}\right)^2}{3(\gamma_1 h_j + q)}\right)} \tag{2.8}$$

Where K_{a1} is the active pressure coefficient in the reinforced soil and K_{a2} is the active pressure coefficient in the backfill soil. *L* is the length of the reinforcement layer, γ_1 and γ_2 are the unit weights of the reinforced and backfill soil respectively, h_j is the depth of the reinforcement layer below the top of wall, q is the magnitude of uniform surcharge. If additional strip loads are present or cohesive soils are present, then additional terms are added to T_{max} in the British Standard.

If the wall design violates any of the above conditions, then the standard steps of solving for the vertical steps via the Coherent Gravity Method procedure are used to calculate σv in RSWall.

In the Tieback Wedge method, the internal failure plane is typically assumed to be planar using any of the Rankine or Coulomb methods. The value of K_r is obtained as a function of depth according to Figure 2.1.

The value of T_{max} is then calculated using the same equation as in the simplified method.

2.3.4. Stiffness Method

The Stiffness Method was developed by Allen & Bathurst (2001; 2015; 2018) and is applicable for extensible reinforcements (i.e. geosynthetics, geogrids and geostrips). The method assumes that the entire reinforced region deforms as a unit and that the stresses are distributed through the entire system. A contributory stiffness is provided by each reinforcement layer, and the total tensile force is distributed evenly between each layer.

The equation for T_{max} is presented in the equation below.

$$T_{max} = S_v \left[H\gamma_r D_{tmax} + \gamma_f \left(\frac{H_{ref}}{H} \right) S \right] k_{avh} \Phi \tag{2.9}$$

The coefficient Φ is applied to consider the various effects of facing stiffness, permanent cohesion, and the global and local interactions of the reinforcement layers. It is a product sum of the individual influence factors corresponding to each of these contributions. k_{avh} is the active earth pressure coefficient with assuming that the wall has a vertical face. D_{tmax} is an effective depth of soil above the layer but below the top of wall, and S is the average soil surcharge above the reinforcement computing using a reference height, H_{ref} . γ_r and γ_f are the unit weights of the reinforced soil and surcharge soil, respectively.

In this way, the equation for T_{max} is the same as with all the other methods, with the vertical stress being the term inside the square brackets, and the value of K_a being substituted as $k_{avh}\Phi$.

3. Failure Modes

Procedures relating to the assessment of each failure mode

3.1. External Stability

These failure modes reflect the overall stability of the wall as a system.

- In general, an active soil wedge forms behind the supporting structure and produces the primary driving force.
- The supporting structure is the wall section profile, and in the case of a reinforced wall, includes the soil within the reinforced region.

3.1.1. Base Sliding

This failure occurs by translational slipping along the base of the wall structure due to driving forces behind the wall acting parallel to the base of the wall.

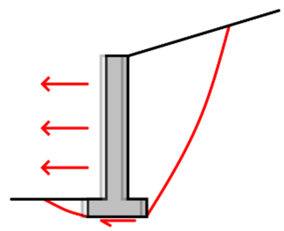

Figure 3.1: Base sliding failure mode

In this failure mode, the capacity demand ratio is calculated as follows:

$$CDR = \frac{\Sigma N \tan \phi + cL + R_p}{\Sigma F_d} \tag{3.1}$$

The sum of driving forces, ΣF_d, is resisted by up to three components of the resisting forces. The primary component is the friction caused by the internal weight of the supporting structure above the base of the wall ($\Sigma N \tan \delta$). The sum of normal forces at the base, ΣN, is a function of the internal weights and components of the active and passive forces, and δ is the soil-structure friction angle at the base.

- If available, permanent cohesion (cL) and passive resistance (R_p) from soil in the front face of the wall may contribute towards the sliding resistance, but only as permitted by the respective design standards.

If the available resisting forces are exceeded by the driving forces, then sliding failure will occur.

Note that if a levelling pad is specified in a conventional block wall, both sliding above and below the pad will be considered in the computations.

- For sliding below the levelling pad, the additional weight of the levelling pad is considered, but the active and passive zones extend to the bottom of the pad, resulting in higher active and passive forces.

3.1.2. Overturning

This failure mode occurs by rotation of the wall about its toe point due to driving forces.

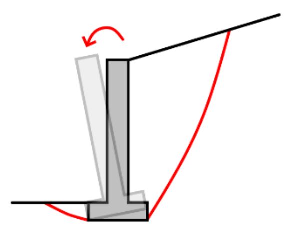

Figure 3.2: Overturning failure mode

The sum of driving moments, ΣM_d, (i.e. rotational actions which cause overturning) is compared against the sum of resisting moments, ΣM_r:

$$CDR = \frac{\Sigma M_r}{\Sigma M_d} \tag{3.2}$$

The moments are calculated by multiplying the internal and external forces by their moment arms about the toe point of the wall.

3.1.3. Bearing Capacity

This failure mode occurs when the bearing capacity of the foundation soil is exceeded due to excessive gravitational forces above it.

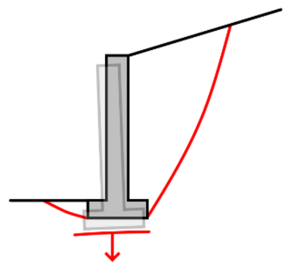

Figure 3.3: Bearing capacity failure mode

The capacity demand ratio for this failure mode is calculated by comparing the bearing capacity (q_r) against the bearing stress (σ_n):

$$CDR = \frac{q_r}{\sigma_n} \tag{3.3}$$

The bearing capacity can either be directly specified or calculated either using variations of Terzaghi's equation.

Calculating the bearing stress involves computing the eccentricity, e, of the resultant normal force at the base of the supporting structure, which reduces the effective bearing width of the supporting structure:

$$e = \frac{B}{2} - \frac{\Sigma M_r - \Sigma M_d}{\Sigma N} \tag{3.4}$$

Where B is the total base width. The distribution of bearing stress along the effective bearing width depends on the design standard and can be a function of the type of contact between the base of the wall with the foundation soil. As in many cases if the bearing stress is assumed to be uniform, then its value is given in the following equation.

$$\sigma_n = \frac{\Sigma N}{B - 2e}$$

(3.5)

Note that if the eccentricity is too high, the width of the base may not be adequate and either a Warning or Failed Check will be produced during the computations, depending on the selected design standard.

3.2. Internal Stability

These failure modes apply only for segmental and gabion walls. Note that the internal strength of concrete in the wall section profile is not within the scope of this program.

3.2.1. Internal Sliding

This failure mode occurs by translational slipping along any of the intermediate layers of the wall and is illustrated below.

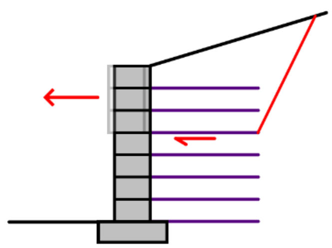

Figure 3.4: Internal sliding failure mode

RSWall checks this failure mode at every layer interface of the wall section profile that has a reinforcement layer, or in the case of unreinforced segmental walls (including gabions) it is assessed at every layer.

It is computed in the same way as external sliding mode in that the active wedge is formed behind the supporting portion of the structure above the sliding interface, and the driving forces parallel to the sliding interface are compared against the frictional resisting forces.

The capacity demand ratio is calculated in the same manner as for external sliding except that all the forces and resistances are evaluated above the sliding interface. Additionally, the shearing interface between adjacent courses of wall units may be assumed to contribute to the resisting forces via a constant, linear, or bilinearly varying function of depth.

3.2.2. Crest Toppling

This failure mode occurs by rotation of the wall about the facing point at an intermediate layer of the wall. It is computed for each layer in the wall section profile that is above the highest level of reinforcement.

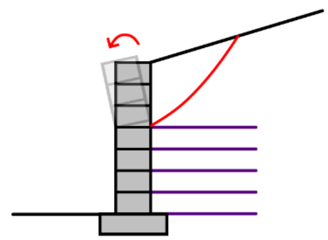

Figure 3.5: Crest toppling failure mode

It is assessed in the same way as overturning whereby the driving moments above the pivoting point are compared against the resisting moments.

3.3. Reinforcement Strength

These failure modes apply only for walls with reinforcements (i.e. segmental walls).

- They are technically also internal failure modes but are made distinct from the others because they involve the reinforcements.
- They are assessed for each layer in the wall section.

3.3.1. Tensile Strength

This failure mode occurs by rupture of the reinforcements at any layer caused by exceeding its tensile strength. An active wedge forms internally causing the wall and soil within the wedge to pull outwards, resulting in tensile stresses in the reinforcements.

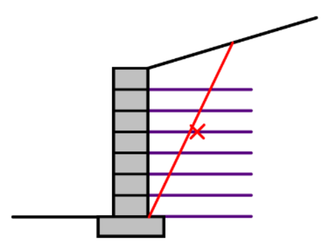

Figure 3.6: Tensile strength failure mode

The capacity demand ratio in this case is simply the tensile strength of the reinforcement layer divided by the value of T_{max} determined in the previous section.

3.3.2. Pullout strength

This failure mode occurs by detachment of the wall body along with any layer of reinforcement from the reinforced soil region. Similar to the assessment of tensile strength, an active wedge is assumed to form internally through the reinforcement layers, and the wall along with the soil inside the active wedge pulls outwards.

The resisting normal stress is computed above the length of the reinforcement outside of the failure wedge, and a frictional resistance is determined.

- There are various methods and assumptions that can be employed to calculate the frictional resistance, which can be found in the governing design standards.
- Commonly, either a friction factor or a coefficient of interaction is applied to the vertical stress above the pullout region of the layer and then multiplied by the bonding area of the reinforcement layer.

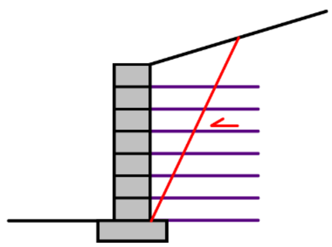

Figure 3.7: Pullout strength failure mode

The capacity demand ratio in this case is simply the pullout strength divided by the value of T_{max}. If the pullout force exceeds the frictional resistance, then failure occurs.

Note that if the length of reinforcement does not reach the internal failure plane, then the rest of the reinforcement analyses are typically considered to be invalid, because the tensile force at each reinforcement layer is assumed to be distributed based on tributary areas assuming that each layer resists some portion of the total active force. In this case, RSWall outputs a capacity demand ratio of zero.

3.3.3. Connection strength

This failure mode occurs by exceeding the connection strength at the wall facing relative to the tensile force in any layer of reinforcement.

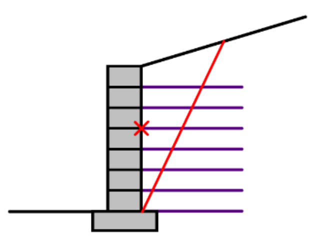

Figure 3.8: Connection strength failure mode

The connection strength may either be constant or defined to vary frictionally depending on the weight of the wall above it.

The tensile force experienced at the location of the connection is estimated based on the various procedures stipulated in the design standards. In the worst case, it is equal to the tensile force used to evaluate tensile strength.

3.4. Other Failure Modes

All the above failure modes can be checked within the RSWall program. There are some other failure modes that may need to be considered during your design.

3.4.1. Overall Stability

Overall or global stability failure occurs when the underlying soil fails as a result of the slope that is created by the retaining wall. It can be checked using limit equilibrium methods (via Export to Slide2) or finite element methods (via RS2).

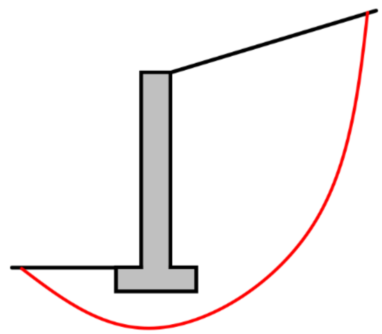

Figure 3.9: Overall stability failure mode

Note that RSWall does not in itself check for the global, overall or internal compound stability of the inputted wall.

3.4.2. Internal Compound Stability

Internal compound stability involves shear failure through the various layer interfaces in a segmental wall, with failure planes of varying shapes extending behind the facing, either through reinforcement layers or into the backfill region.

This failure mode can also be analyzed using limit equilibrium methods (via Export to Slide2).

3.4.3. Other Failures

RSWall does not check the structural integrity of the actual mass of the wall profile (e.g. structural failure of the concrete in the wall profile's structure via shear and bending moments). These failure modes typically require analysis under separate assumptions and LRFD factors under different design standards.

4. Other Effects

Additional considerations in the computations

4.1. Effect of Groundwater

Most retaining walls are detailed to provide adequate drainage which prevents porewater pressures from building up behind the wall. However, there may be some design cases where a constant or differential water table elevation needs to be considered.

In RSWall, the effect of hydrostatic and hydrodynamic pressures can be considered. Note that seepage analysis is not considered in RSWall but can be performed in a finite element program such as RS2. Porewater pressures and their resultant forces can be applied behind the wall, in front of the wall, and below the wall. These forces act normal to the structure.

In undrained conditions (ϕ = 0 with cohesion value specified), total stress analysis is typically used when comparing forces and stresses in the assessment of the failure modes. In drained conditions, effective stress analysis is required.

Except for in confined soils in seismic conditions, the earth pressure forces are typically separated from the hydrostatic force.

The presence of a water table can also adversely affect bearing capacity by reducing the effective unit weight of soil which contributes to various terms in Terzaghi's equation. The design standards handle this assumption in different ways, and details can be found within their respective texts.

4.2. Seismic Actions

The effect of seismic loads is typically considered during the analysis by adding equivalent static forces to the applied forces and by modifying K_a. In most cases, RSWall uses variations of the Mononobe-Okabe method for estimating the earth pressure force as permitted by the design standards. The analysis depends on the assumed coefficients of k_h and k_v which represent acceleration ratios in the horizontal and vertical directions respectively.

Internal inertial forces equal to a fraction of the internal weight W are applied horizontally (k_hW) and vertically ($\pm k_vW$). If k_v is non-zero then the analysis is repeated for both cases of k_v being positive and negative, as either can govern the analysis. Some design standards allow for the effects of k_v to be neglected, because the effects of peak accelerations in either direction may not always act concurrently.

During seismic analysis, the porosity of soils is also considered. If porewater is confined in the soils, then the water and soil move together, exerting a total earth pressure force on the wall. If porewater is instead considered to be unconfined, then it exerts a separate hydrostatic and hydrodynamic force on the wall. The hydrodynamic force is considered in addition to the hydrostatic force and is commonly evaluated using the Westergaard method.

5. References

- Kerisel J, Absi E, 1990, Active and Passive Earth Pressure Tables. AA Balkema: Rotterdam Netherlands.
- Caquot A, Kerisel J, 1949, Mécanique des sols, Libraire du Bureau des Longitudes, de l'Ècole Polytechnique, Paris, France.

- FHWA, 2006, NHI Course No. 132012: Soils and Foundations, Reference Manual - Volume II, Publication No FHWA NHI-06-089, US Department of Transportation - Federal Highway Administration.
- FHWA, 2023, Design and Construction of Mechanically Stabilized Earth: FHWA GEC 011, US Department of Transportation - Federal Highway Administration.
- AASHTO, 2020. AASHTO LRFD Bridge Design Specifications, 9th edition. American Association of State Highway and Transportation Officials (AASHTO).
- British Standards Institution, 2016. British Standard 8006-1+A1:2016: Code of practice for strengthened/reinforced soils and other fills (BS 8006).
- European Commission, 2004. EN1997-1:2004 : Eurocode 7 - Geotechnical design.
- Standards Australia Limited, 2008. AS 4678: Earth-retaining structures (amendment 2).
- NCMA, 2009. Design Manual for Segmental Retaining Walls, 3rd edition. National Concrete Masonry Association (NCMA).
- Allen TM, Bathurst RJ, 2001, Application of the K0-Stiffness method to reinforced soil wall limits states design. Washington State Department of Transportation, Report number WA-RD 528.1.
- Allen TM, Bathurst RJ, 2015, Improved stiffness method for prediction of loads in reinforced soil walls, J Geotech Geoenviron Eng, 141(11):04015049.
- Allen TM, Bathurst RJ, 2018, Application of the simplified stiffness method to design of reinforced soil walls, J Geotech Geoenviron Eng, 144(5):04018024.
- Caltrans, 2011, Trenching and Shoring Manual, Revision No. 01. California Department of Transportation.
- Das B, 2010. Principles of Foundation Engineering, 7th edition. Cengage Learning: Stamford CT.
- Das B, Sobhan K, 2012. Principles of Geotechnical Engineering, 8th edition. Cengage Learning: Stamford CT.
- Jumikis AR, 1962, Soil Mechanics. D Van Nostrand Company Inc: Princeton, NJ.
- Kramer, 1996, Geotechnical Earthquake Engineering. Prentice Hall: Upper Saddle River, NJ.