K-Réa v4

C. TECHNICAL MANUAL

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C.1. Introduction and main principles

C.1.1. Calculations and application field

K-Réa is intended to study the behaviour of retaining walls (internal efforts and deformations) subjected to a series of construction stages.

The calculation method used consists in the subgrade reaction calculation method (type SSIM-K[1] according to the application standards of the Eurocode 7 designated by SSIM in this document for simplification purposes). It is based on the model of a beam supported by elastic-plastic springs.

K-Réa enables the analysis of two types of projects:

- « Simple wall » projects: comprising one single plane retaining wall;

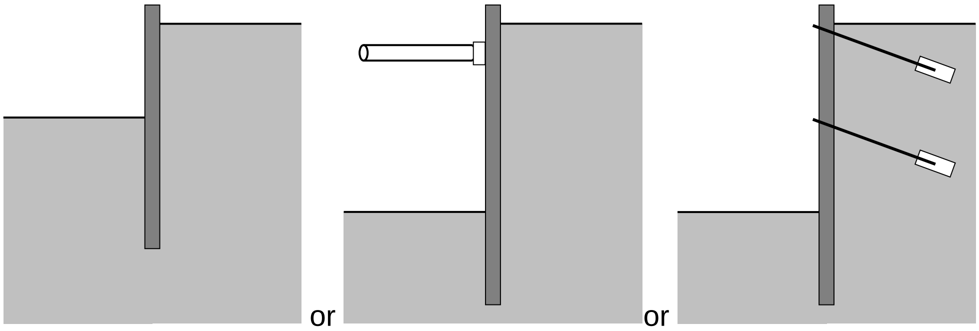


Figure C1: Examples of « simple wall » projects

- « Double-wall » projects: comprising two plane walls, linked to each other (or not) by one or more linking anchor layers.

  Note: in this manual, the term “double walls” designates projects with either 2 walls with approximately the same length (cofferdams for instance), or a main wall anchored on a smaller rear wall.

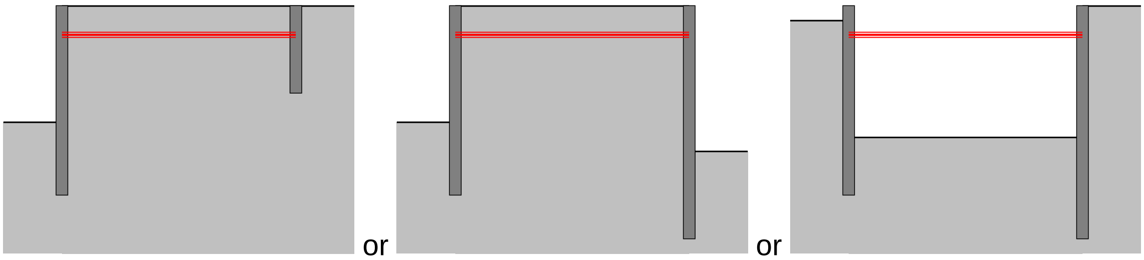


Figure C2: Examples of « double-wall » projects

The series of construction stages includes the initial stage of the wall(s) installation which is followed by different phases, each one corresponding to a set of actions, such as the implementation of struts or anchors, the modification of the water or soil level, the application of overloads or the implementation of link anchors (in the case of a double-wall project).

The SSIM calculation is presented and detailed in sections C.1.2.1 and C.2.

Moreover, in addition to the SSIM calculation, K-Réa performs 3 types of ULS checks according to the recommendations of the Eurocode 7 (cf chapters C.1.2.2 and C.4), particularly the implementation of the limit equilibrium method (LEM) for cantilever walls.

The global articulation between these calculation types and checks is displayed in an diagram in chapter C.1.2.3.

[1] SSIM-K: Model of soil-structure interaction based on the subgrade reaction method.

C.1.2. Introduction to the calculating methods and suggested verifications

C.1.2.1. Basic calculation method SSIM

The SSIM method associates a beam model representing the wall and elastic-plastic springs representing of the soil-wall interaction. The anchoring elements are modelled with equivalent elastic-plastic springs.

In K-Réa, the model is equated with an overall matrix formulation associating both walls. In this formulation, liaison elements like struts or anchors produce a coupling between the freedom degrees of both walls.

C.1.2.2. ULS checks according to Eurocode 7

Eurocode 7 (completed by its application standards) fixes the list of verifications (ULS) to carry out considering the principle risks related to retaining structures:

- Verification of passive earh pressure (1);
- Verification of retaining wall resistance and of its supports (2);
- Verification of vertical equilibrium of the wall (3);
- Verification of hydraulic stability (4);
- Verification of stability of the anchoring block (5);
- Verification of overall stability (6);

K-Réa carries out checks (1), (3) and (5) for each stage according to standard NF P 94-282 (Eucorode 7). It also provides the necessary elements to carry out check (2). Checks (4) and (6) require specific calculation programs.

In K-Réa v4, these checks can be done according to one of the three approaches of the Eurocode 7 (see §C.4.1 for a detailed description of these approaches and their implementation within K-Réa v4).

C.1.2.3. Articulation of different calculation methods

In the case of a calculation led without ULS checks, all phases are processed using the « basic » model, which is a displacements model based on the subgrade reaction coefficients method (SSIM-K, designated in this document by SSIM only), and performed without weighting factors on soil properties nor surcharges. The results obtained include wall displacements, mobilised pressures as well as shear forces and bending moments (V, M).

In the case of calculations carried out along with ULS checks, two calculations are executed for each stage:

- « SLS » calculation method based on the SSIM model without weighting on soil and surcharge properties. This calculations results are strictly identical to those of a calculation "without ULS verifications": displacements, mobilised pressures and forces (V, M);
- « ULS » calculation method which model varies depending on whether the wall is anchored or not in the considered stage: SSIM model for the case of an anchored wall, LEM model for a cantilever wall. The result of ULS calculations is completed with the following mechanisms analysis:
    - Verification of passive earh pressure;
    - Verification of vertical equilibrium of the wall;
    - Verification of stability of the anchoring block;

The figure hereunder summarises the general diagram of calculations performed by K-Réa and their articulation.

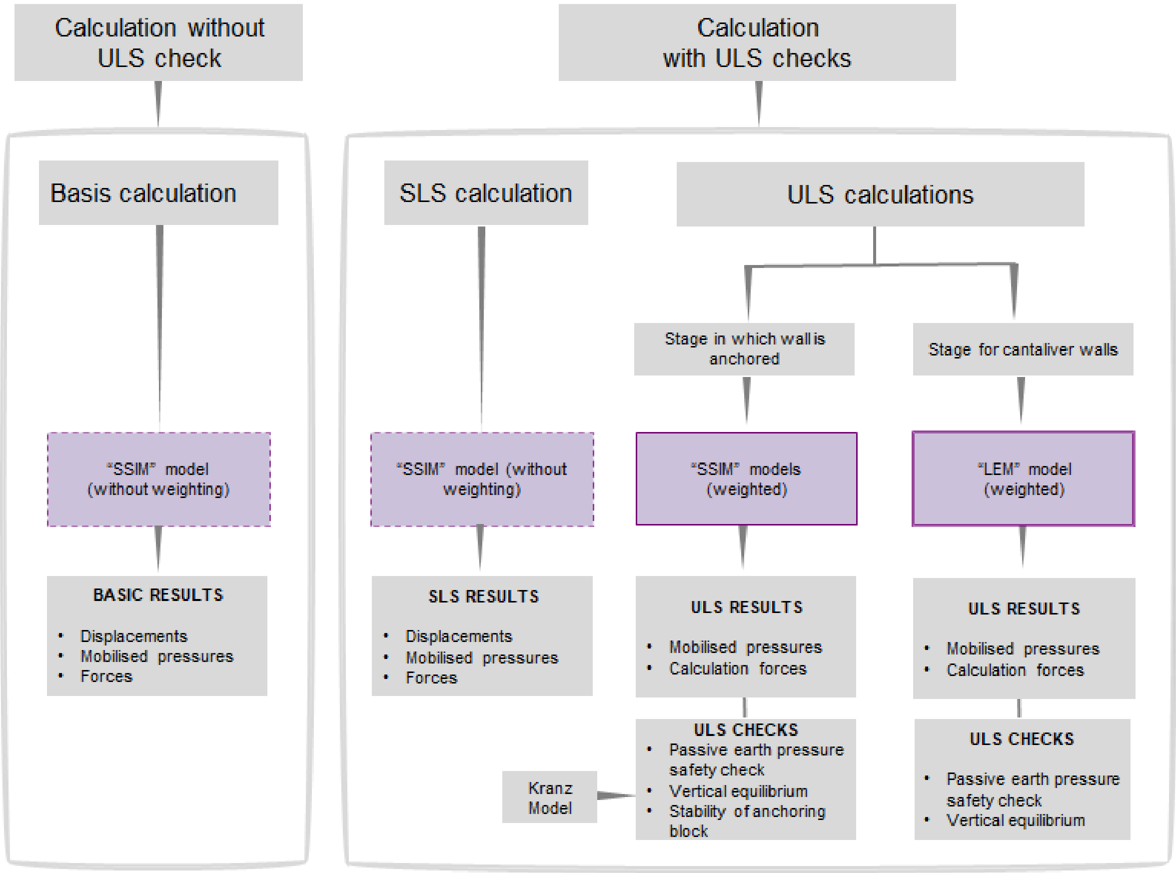


Figure C3: Calculation diagram

C.1.3. Sign convention

For each wall, the part left to the wall is called the left side; the part right to the wall is called the right side. Wall displacements and forces are positive when directed to the right (cf. **Erreur ! Source du renvoi introuvable.Erreur ! Source du renvoi introuvable.**).

Note: the « main » excavation can be either located on the left or on the right side without distinction.

The z-coordinates are either positive upwards when using the **levels**, either positive downwards when using **depths**. This option is defined in the **Menu Data**, **Titles and Options**.

As for the external forces applied onto the wall, the forces (represented by F on the figure hereunder) are positive when oriented from left to right and moments (represented by M in the figure hereunder) are positive when anti-clockwise.

Support forces are considered positive:

- In traction in the case of an anchor (grouted or linking);
- In compression in the case of a strut (single or linking).

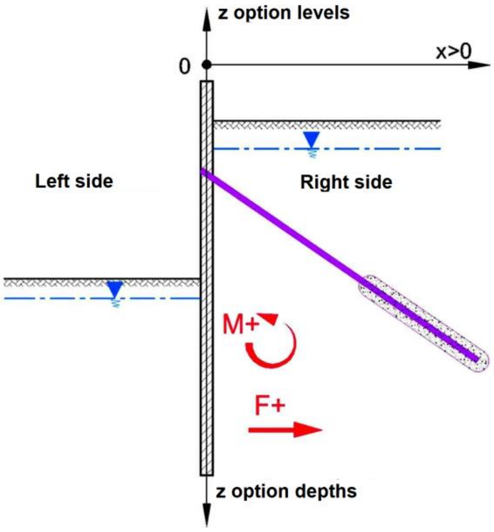


Figure C4: Sign conventions for external loads

In addition, the figure hereunder presents the sign conventions used within K-Réa for the internal forces (M, V and N). The axial force N is considered positive in compression.

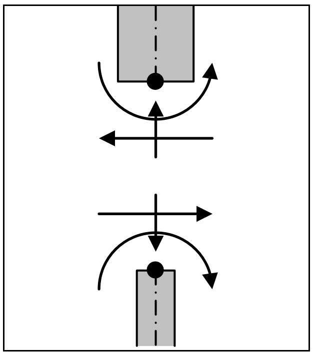


Figure C5: Sign convention for inner efforts

C.2. Theoretical aspects

C.2.1. Equation

C.2.1.1. Wall behaviour

Each wall « i » is represented by a linear elastic beam with a homogeneous section. We consider the hypothesis of a thin beam to allow neglecting the deformations caused by the shearing force.

The behaviour of the beam in bending mode, representative of wall « i », can be described with the following general equation:

$$\frac{d^2}{dz^2}\left(EI_i \frac{d^2 w_i}{dz^2}\right) + Rc_i . w_i = q_i^{ext} - \left(r_i^d - r_i^g\right) - r_i^a \quad (1)$$

In which:

- w_i bending (transversal displacement) of the wall « i » (positive towards the right);
- EI_i product of inertia of wall « i »;
- Rc_i cylindrical rigidity of wall « i »;
- r_i^d density of soil horizontal reaction on the right side of wall « i »;
- r_i^g density of soil horizontal reaction on the left side of wall « i »;
- r_i^a density of the horizontal reaction of anchors connected to wall « i »;
- q_i^{ext} horizontal density of external loads on wall « i ».

C.2.1.2. Soil/wall interaction law

The soil / wall interaction law is described, for each side and each wall, with a curve of classical active and passive earth pressure characterised by 4 parameters:

- k_h: horizontal subgrade reaction coefficient of the wall (or surface unit stiffness);
- p_a: limit horizontal active earth pressure (or active pressure);
- p_b: limit horizontal passive earth pressure (or passive pressure);
- p_0: horizontal reference pressure (also named « initial » pressure or “at rest pressure”).

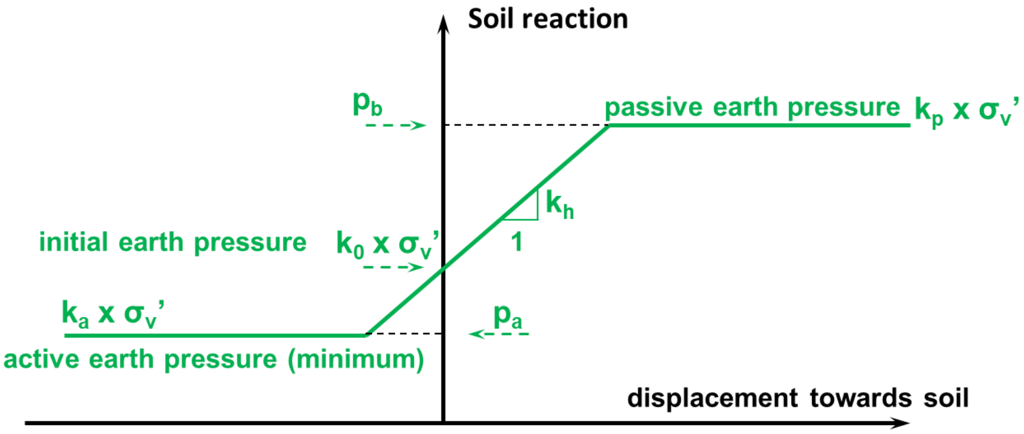


Figure C6: Soil/wall interaction law

According to the notations of the above figure, the lateral reaction of the soil on one side of the wall can be expressed as follows:

$$\begin{cases} r_i^d = +\alpha.w + \beta \\ r_i^g = -\alpha.w + \beta \end{cases} \qquad (2)$$

In which:

- Elastic stage: $\alpha = k_h$ $\beta = p_0$
- Limit state of active earth pressure: $\alpha = 0$ $\beta = p_a$
- Limit state of passive earth pressure: $\alpha = 0$ $\beta = p_b$

By default, the values of $p_a/p_b/p_0$ are automatically determined by K-Réa according to the soil characteristics and the effective vertical stress σ_v' for a given stage, wall and side (see §C.3.1).

C.2.1.3. Pore pressure

A non-zero pore-water pressure u(z) (hydrostatic or flow conditions) (cf. §C.3.1.3):

- Modifies the state of effective stress which is directly dependant on the mobilization of the soil reaction law ($p_a/p_b/p_0$ are functions of σ_v');
- Mobilizes horizontal pressure directly on the wall equal to u(z), that adds up to the external load's density on the wall $q^{ext}(z)$.

C.2.1.4. Anchors

Isolated anchors (struts, ties, circular walings, rotational springs and surface struts) should follow an elastoplastic reaction law like in the hereunder diagram.

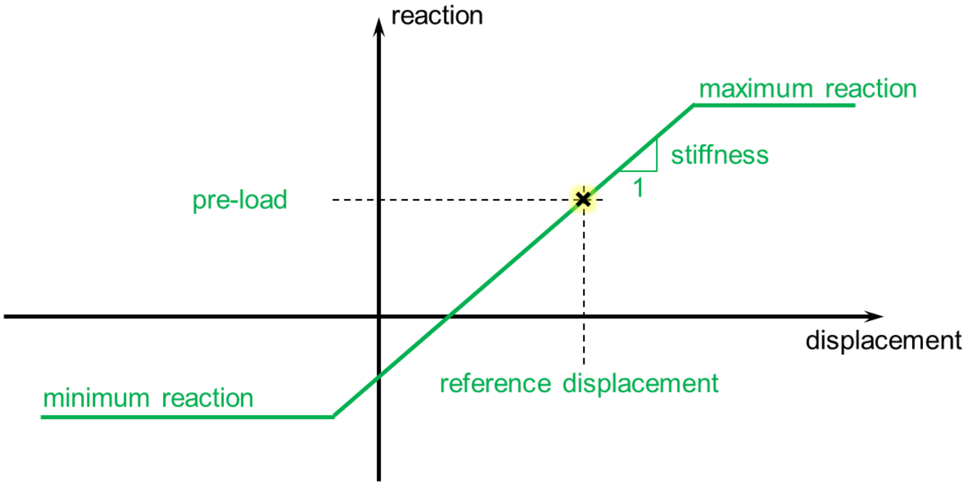


Figure C7: Mobilization law of anchor reaction

The anchor reaction mobilization law can also be expressed with the following equation:

$$r_i^a = k_i^a.w + p_i^a \qquad (3)$$

C.2.1.5. Resolution

The resolution of the equations system (1) + (2) + (3) can be conducted digitally by discretizing the representative beam of screen "i" in elements with two nodes and four degrees of freedom (two displacements and two rotations).

This discretization allows to express the elastic-plastic equilibrium of the wall in the form of a matrix system of size 2(n+1) x 2 (n + 1), where n is the total number of elements:

$$\left(\mathbf{K}_i^e+\mathbf{K}_i^s+\mathbf{K}_i^a\right)\mathbf{w}_i=\mathbf{F}_i^{ext}-\mathbf{P}_i^s-\mathbf{P}_i^a \qquad (4)$$

In which, for wall « i »:

- $\mathbf{w}_i$: equivalent displacement vector constituted by the displacements and rotations of each mesh node;

- $\mathbf{F}_i^{ext}$: load vector of external loading (+ water pressure);
- $\mathbf{P}_i^s$: reaction vector of the constant part (β) of the soil reaction;
- $\mathbf{P}_i^a$: reaction vector of the constant part (p^a) of the anchor reaction;

- $\mathbf{K}_i^e$: wall stiffness matrix (in bending mode and cylindrical);
- $\mathbf{K}_i^s$: soil stiffness matrix (elastic part α for each level);
- $\mathbf{K}_i^a$: anchor stiffness matrix (elastic part k^a for each level);

The resolution of this equation provides the displacements and the reactions for each point of each mesh element.

C.2.2. Linking anchors

We now examine the case of a double-wall with one or more linking anchor of type ties/ struts (single or surface struts). These elements follow a reaction law similar to the one of the "non-linking" anchors (cf. §**Erreur ! Source du renvoi introuvable.**).

The particularity of a linking anchor resides in the fact that its reaction is a function of the relative displacement between both walls (and not of the absolute displacement).

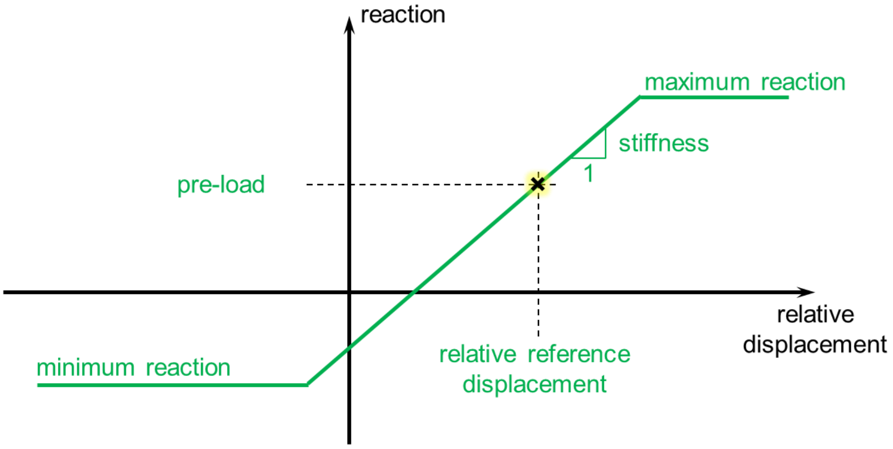


Figure C8: Mobilization law of the linking anchor reaction

Using the matrix formulation for each wall, the balance of the two walls in interaction can be determined with a unique matrix system:

$$\begin{pmatrix}\mathbf{K}_1^e+\mathbf{K}_1^s+\mathbf{K}_1^a+\mathbf{K}^L & -\mathbf{K}^L\\ -\mathbf{K}^L & \mathbf{K}_2^e+\mathbf{K}_2^s+\mathbf{K}_2^a+\mathbf{K}^L\end{pmatrix}\cdot\begin{pmatrix}\mathbf{w}_1\\ \mathbf{w}_2\end{pmatrix}=\begin{pmatrix}\mathbf{F}_1^{ext}-\mathbf{P}_1^s-\mathbf{P}_1^a-\mathbf{P}^L\\ \mathbf{F}_2^{ext}-\mathbf{P}_2^s-\mathbf{P}_2^a+\mathbf{P}^L\end{pmatrix}$$

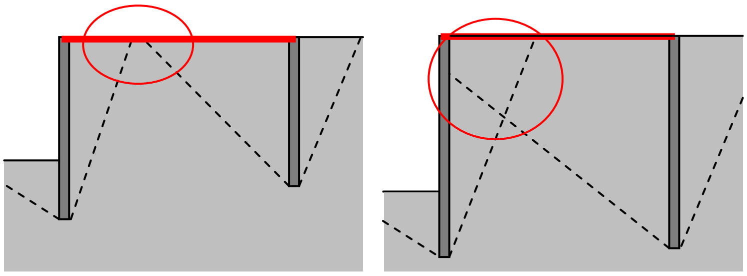
In which:

- $\mathbf{K}^L$: stiffness matrix of linking anchors (elastic part);
- $\mathbf{P}^L$: vector of the constant part of the linking anchors reaction.

For the model to be valid, it is assumed that the linking anchors if they exist are the only interaction between the two walls. K-Réa does not take into account any interaction between the two walls through the soil situated between them. In particular, K-Réa does not explicitly carry out overlapping verifications (figures herebelown) for:

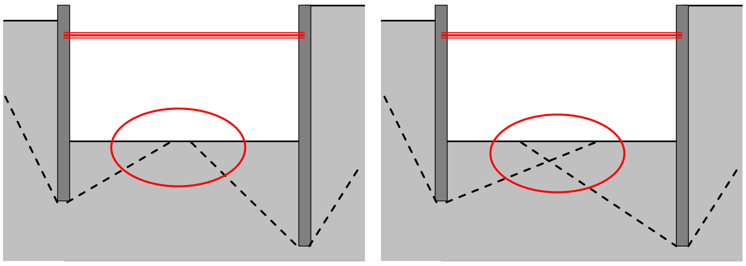
- ○ Active/ passive earth pressure corners in the case of double wall project;
- ○ Passive earth pressure corners in the case of an excavation with struts.

These interactions have to be verified by the user with other means. Nevertheless, in the case of a double wall project (head wall anchored to an anchor wall using anchors), the stability verification of the foundation block with a Kranz model (that K-Réa does automatically if ULS calculations are requested) implicitly suggests that there is enough distance between both walls not to consider any interaction between both walls through the anchoring block between both walls.



Case of non-interaction of active/passive pressure corners | Case of interaction of active/passive pressure corners together and with one of the walls

Figure C9: Case of a double wall with interaction between corners of active/passive pressure



Case of non-interaction betwee passive pressure corners | Case of interaction of active pressure corners

Figure C10: Case of an excavation with struts and interaction between passive pressure corners

C.2.3. Stress calculations

In K-Réa, internal effort calculations for each wall are performed by the integration of the reactions calculated in the preceding stage.

- Shearing force $V_i(z) = \int_0^z \left[q_i^{ext}(t) - r_i^d(t) + r_i^g(t) - r_i^a(t) - Rc_i(t).w_i(t)\right]dt + V_i(0)$
- Bending moment $M_i(z) = -\int_0^z V_i(t)dt + M_i(0)$ (+reaction of the joints)
- Orthoradial pressure $N_i^{\theta\theta}(z) = -Rc_i(z).w_i(z).R_i(z)$ (positive in compression)

Where R_i (z) indicates the radius of the excavation at z level in the case of a circular wall ($Rc_i \neq 0$).

K-Réa also calculates a vertical axial force $N_i^{zz}(z)$ taking into account the "surface" weight of the wall, the vertical component of the external load and support efforts, as well as the vertical projection of earth pressure pv. The latter is estimated from the horizontal earth pressure ph with the following equation:

$$p_v = \begin{cases} \tan\delta_a p_a \left(\dfrac{p_0 - p_h}{p_0 - p_a}\right) & \text{if } p_a \le p_h \le p_0 \\ \tan\delta_b p_b \left(\dfrac{p_h - p_0}{p_b - p_0}\right) & \text{if } p_0 \le p_h \le p_b \end{cases}$$

Where, δ_a and δ_b are the values of the inclination of the active and passive earth pressures to horizontal.

C.2.4. Effects of 2nd order

It is possible to consider second-order effects on the wall. This consists in considering the displacements and complementary forces (moments and shear) brought by the additional vertical axial force $\Delta_{ad}N$ (z). The latter is calculated by considering the vertical components of linear loads and forces in the anchors. Mathematically, this is equivalent to the application of an additional lateral load of density $\Delta q_{ad}(z)$:

$$\Delta q_{ad} = \Delta N_{ad} \frac{d^2 w}{dz^2}$$

These effects are considered iteratively until convergence of the term $\Delta\ q_{ad}(z)$. At the end of the calculation, the evaluation of additional internal efforts (ΔM_{ad}, ΔV_{ad}) due to the 2nd order effects is conducted using the following equation:

$$\frac{d\Delta\Delta_{ad}}{dz} = \Delta N_{ad} \frac{dw}{dz} \qquad \Delta V_{ad} = -\Delta N_{ad} \frac{dw}{dz}$$

C.2.5. Phasing management

C.2.5.1. Soil / wall interaction

C.2.5.1.1. Effect of a change in the effective vertical stress

The modification of the effective vertical stress in the ground σ_v' in a given phase, under the effect of an excavation ($\Delta\sigma_v' < 0$), of a backfilling ($\Delta\sigma_v' > 0$) or of the application of an overload on the ground ($\Delta\sigma_v' > 0$) results in the following double effect:

- Modification of the value of the pressure p_i with zero displacement using recompression k_r and decompression k_d coefficients:
    - $\Delta p_i = k_r.\Delta\sigma'_v$ si $\Delta\sigma'_v > 0$
    - $\Delta p_i = k_d.\Delta\sigma'_v$ si $\Delta\sigma'_v < 0$
- The update of both plastic earth pressures (active/passive) using the coefficients of passive/active pressure defined by the user for each layer:
    - $\Delta p_a = k_a.\Delta\sigma'_v$
    - $\Delta p_b = k_p.\Delta\sigma'_v$

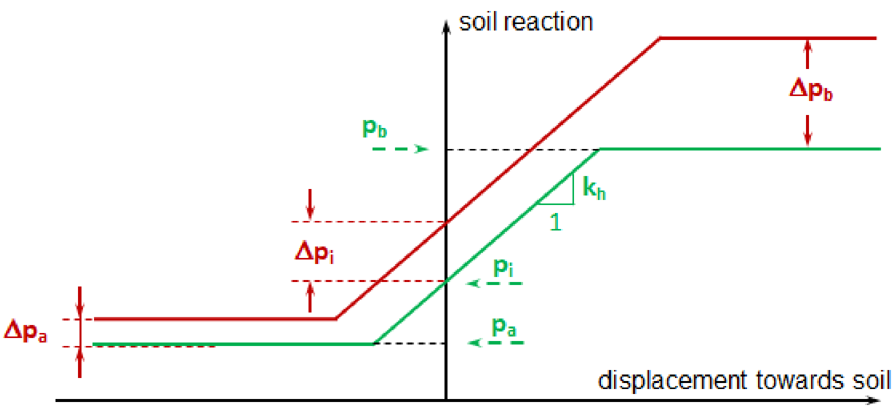


Figure C11: Effect of a modification in the effective vertical stress

C.2.5.1.2. Effect of plastification

Soil plastification in a phase has the effect, in the next phase, of horizontally shifting the soil/wall interaction curve with a residual displacement δ_r. This leads to a "fictitious" modification of the initial pressure p_i. Therefore, its value can no longer be directly connected to the vertical stress state.

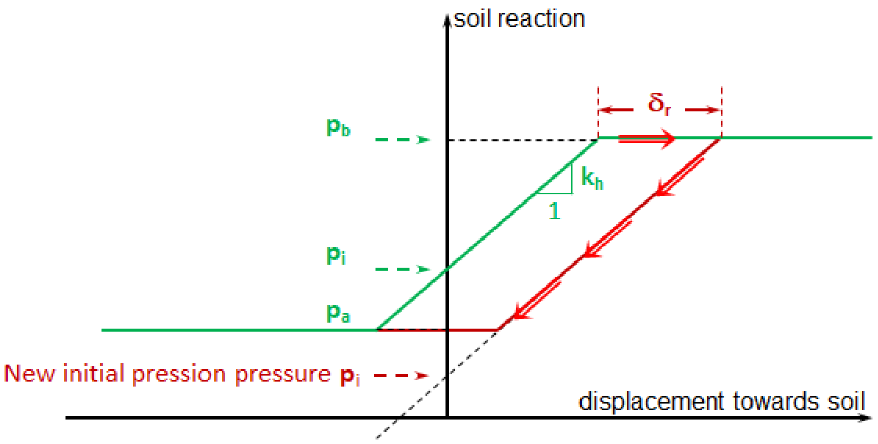


Figure C12: Effect of soil plastification – notion of residual displacement

Special case of deplastification: the line of return is unchanged and so the initial pressure is also unchanged.

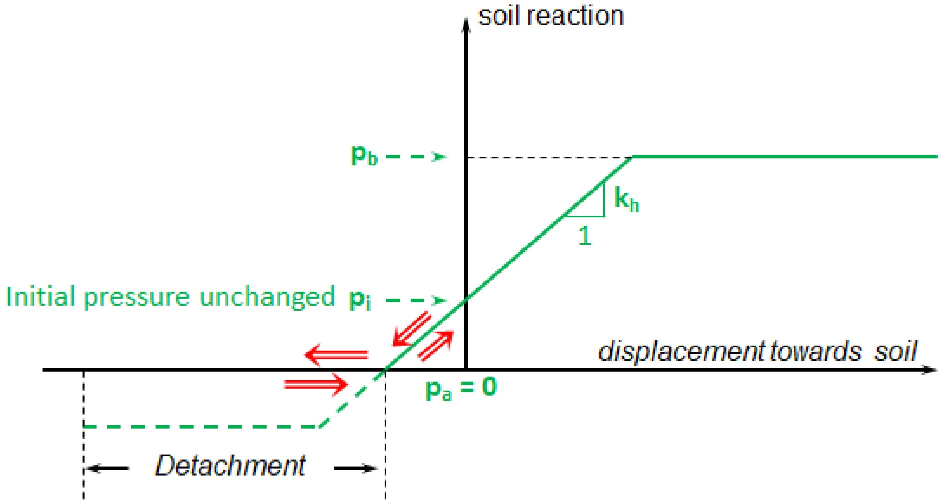


Figure C13: Particular case of soil detachment

C.2.5.1.3. Modification of the reaction coefficient

The modification of the reaction coefficient leads to a rotation of the elastic part around the point of balance achieved in the previous phase, which implies a modification of the initial apparent pressure (figure below).

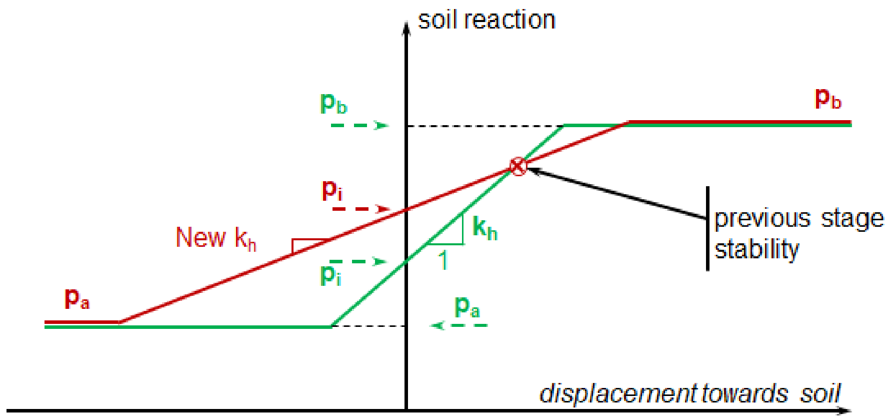


Figure C14: Effect of a change in the coefficient of soil reaction

As suggested in the figure above, the modification of the reaction coefficient does not have an impact on the previous equilibrium and no displacement is generated if there is no additional load.

C.2.5.2. Anchors

C.2.5.2.1. Creep

The modification of an anchor stiffness during phasing is treated differently depending on whether it is reduced (creep) or increased with respects to its initial value. Reducing the stiffness of an anchor (creep) leads to a regeneration of the interaction law around the reference point, thus leading to an additional displacement in the absence of any other action during the study stage.

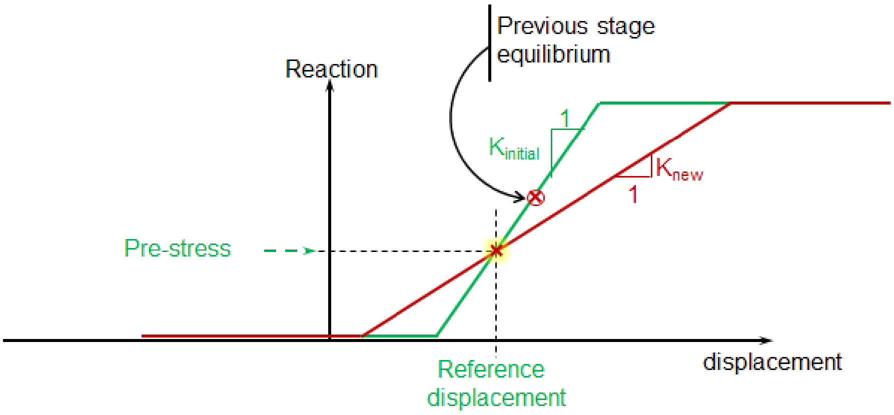


Figure C15: Creep of the anchors - modification of the mobilization law

C.2.5.2.2. Stiffening

A stiffness increment is treated by applying a rotation of the reaction law around the equilibrium point achieved in the previous phase (and not around the reference point as in the case of a creep). As a result, the previous equilibrium is not modified and no movement is generated in the absence of any other load.

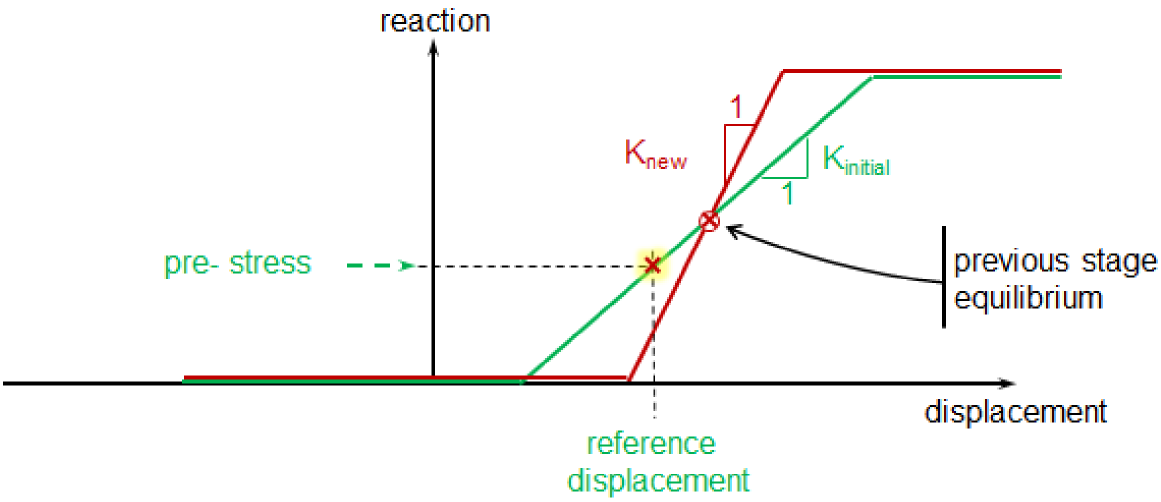


Figure C16: Increase of anchor stiffness

C.2.5.2.3. Modification of pre-loading

The modification of the pre-stress during phasing is treated as a vertical shift of the mobilization curve equal to the difference between the new pre-loading and the original one (figure below).

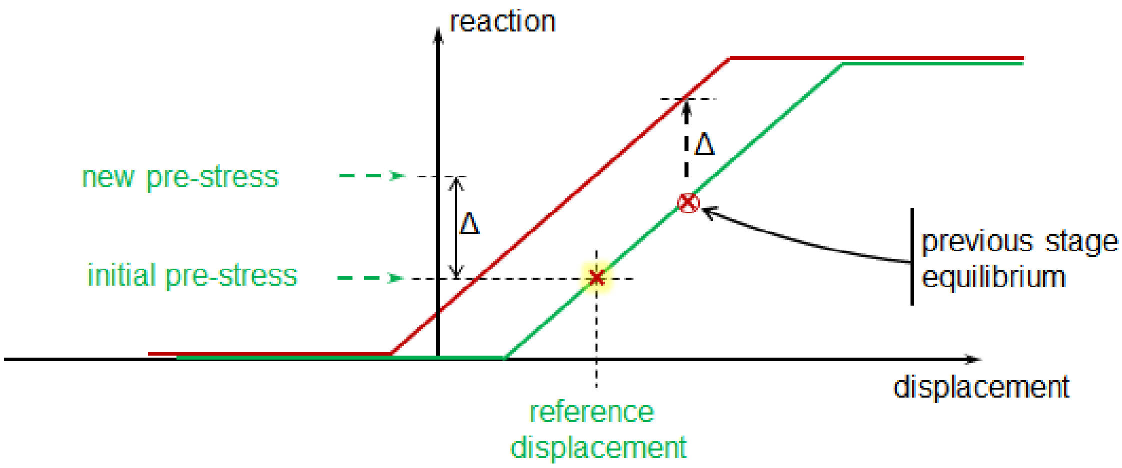


Figure C17: Effect of a change in preload during phasing

C.2.5.2.4. Detachment

The anchors working "unilaterally" follow a reaction curve that includes a 'minimum' level. The detachment/re-examination process is schematized in the figure below.

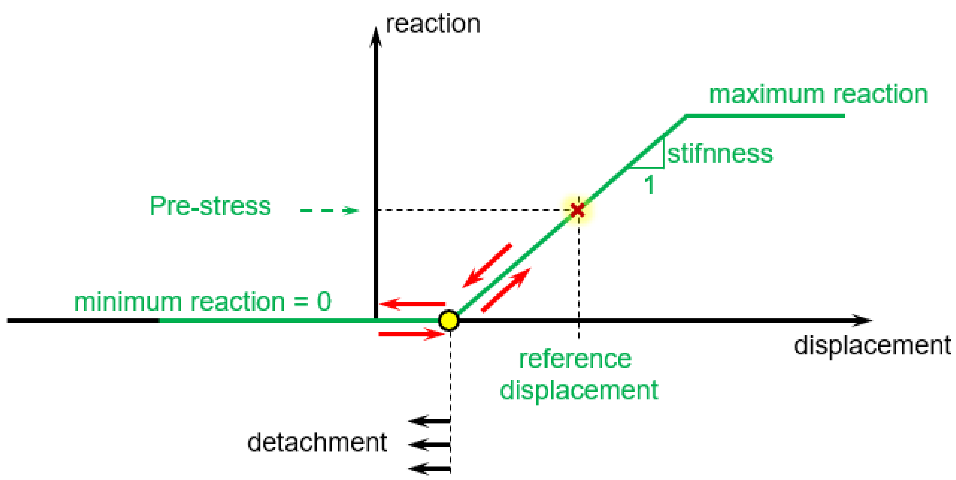


Figure C18: Process of detachement/repasting for an anchor working unilateraly

C.2.5.2.5. Plastification

In the general case, plastification management during phasing is conducted in a similar way to the law of soil/wall interaction, by updating the law of mobilization at each stage taking into account the accumulation of irreversible displacements.

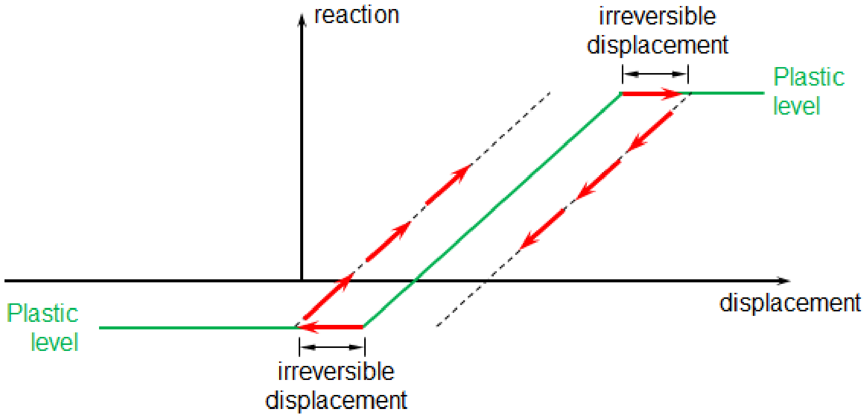


Figure C19: Plastification of the anchors during phasing - general principle diagram

C.2.5.3. Modification of wall stiffness

The modification of the wall stiffness (product of inertia EI and/or cylindrical stiffness) is treated differently depending on whether it is a creep (reduction of rigidity) or an increase in rigidity compared to the previous stage. This difference in behaviour is handled automatically by the calculation engine of K-Réa, for each section of the wall.

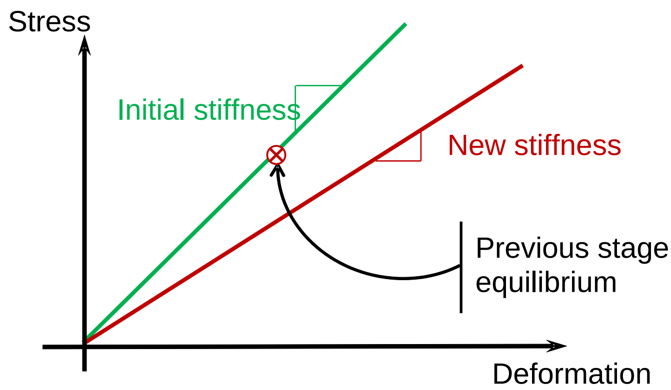


Figure C20: Modification of the rigidity of the wall - creep

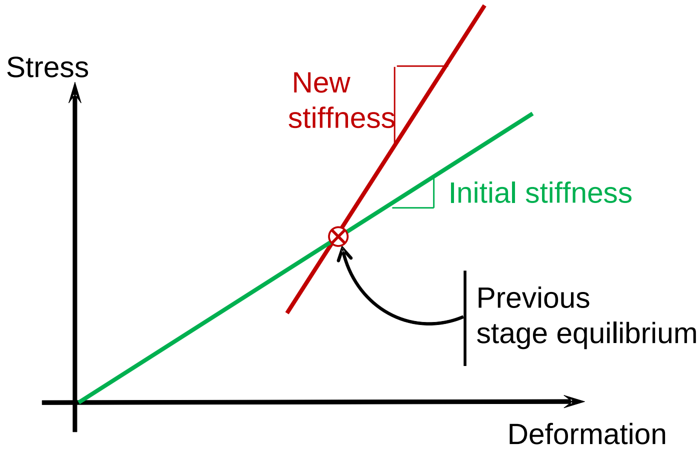


Figure C21: Modification of the rigidity of the wall - stiffening

C.3. Implementation

C.3.1. Ground and water pressure

C.3.1.1. At-rest earth pressure

The (horizontal) pressure for zero displacement uses, in the initial state of the ground, the notion of active earth pressure at rest characterized by the active earth pressure at rest coefficient, k_0, assigned to the considered soil layer, in which case:

$$p_i = p_0 = k_0 . \sigma'_{v0}$$

The value of k_0 is a function of the intergranular friction angle of the soil, of the initial ground's slope as well as of the over-consolidation state (cf. §C.5.1.1). The effective vertical stress, at rest, is evaluated as follows:

$$\sigma'_{v0} = \int_0^{z_w} \gamma dz + \int_{z_w}^{z} \gamma' dz$$

In which:

γ	total soil unit weight above water table
γ'	submerged soil unit weight below water table
z_w	preatic level

As stated in the § C.2.5.1.1, the modification of this pressure under the effect of a vertical stress increment uses the notion of decompression/recompression coefficients (k_r and k_d) according to the following equation:

- $\Delta p_i = k_r . \Delta\sigma'_v$ if $\Delta\sigma'_v > 0$
- $\Delta p_i = k_d . \Delta\sigma'_v$ if $\Delta\sigma'_v < 0$

The definition of these coefficients is detailed in §C.5.1.2.

C.3.1.2. Pressure limit

The limits of active/passive earth pressures are linked to the effective vertical stress σ_v' (at the level of the wall) through the coefficients of active/passive pressure:

- Limit active earth pressure: $p_a = \max\left(k_{a\gamma} . \sigma'_v - k_{ac} . c \,;\, k_{a\min} . \sigma'_v\right)$
- Limit passive earth pressure: $p_b = \min\left(k_{p\gamma} . \sigma'_v + k_{pc} . c \,;\, p_{max}\right)$

In which:

$k_{a\gamma}$	coefficient of active earth pressure (cf. §C.5.1.3)
k_{ac}	coefficient of active pressure related to cohesion (cf. §C.5.1.3)
$k_{a\min}$	coefficient of minimum active pressure, by default equal to 0.10 (NF P 94-282)
$k_{p\gamma}$	coefficient of passive earth pressure (cf. §C.5.1.3)
k_{pc}	coefficient of passive earth pressure related to cohesion (cf. §C.5.1.3)
p_{max}	ultimate soil pressure (applicable value for a discontinued wall)
c	soil cohesion

C.3.1.3. Pore pressure

C.3.1.3.1. Hydrostatic system

In hydrostatic state, the pore-water pressure on the screen is evaluated as follows:

$$u_w^0(z) = \gamma_w(z - z_w)$$

Where γ_w designates the unit weight of the water.

C.3.1.3.2. Hydraulic gradient

The presence of a hydraulic gradient (upward flow) means there is a hydraulic state different from the hydrostatic one. Such state can be characterized with a pore pressure diagram defined according to the following equation:

$$u_w(z) = \gamma_w[z - h_w(z)]$$

Where $h_w(z)$[2] designates the hydraulic potential at depth z.

The presence of a hydraulic gradient also implies the effective vertical stress will be modified according to the following relationship:

$$\sigma'_v = \sigma'_{v0} - \Delta\sigma(u) \text{ In which } \Delta\sigma(u) = u_w - u_w^0 = [z_w - h_w(z)]\gamma_w$$

An « ascendant » hydraulic gradient ($u_w \geq u_w^0$) reduces the effective stress, and therefore, the available resistance (reduction of the limit of the passive earth pressure).

C.3.1.4. Reduced earth pressures

The case of a discontinuous wall requires the correction of the active and passive earth pressures on each side of the wall.

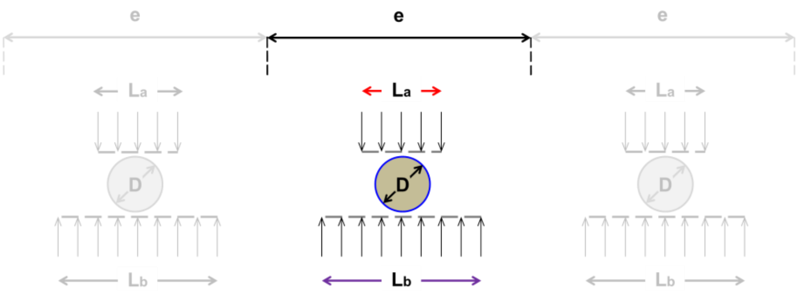


Figure C22: Conventions and notations for a discontinuous wall

For an isolated element, we have:

- mobilization of the active earth pressure over a length L_a bigger or equal to the equivalent diameter of each element D;
- mobilization of the passive earth pressure over a length L_b bigger or equal to L_a (growth effect).

The calculation is performed by considering a "wall" of equivalent stiffness with:

- An active earth pressure reduced in relation to the continuous wall:

$$p_a|_{\text{reduced}} = \frac{L_a}{e} p_a$$

- A passive earth pressure reduced in relation to the continuous wall:

$$p_b|_{\text{reduced}} = \frac{L_b}{e} p_b$$

---

[2] In the case of hydraulic system, we have: $h_w(z) = \text{Cte} = z_w$

In K-Réa, this reduction is controlled with the following two factors R and C:

$$R = \frac{L_a}{e} \qquad C = \frac{L_b}{L_a}$$

And so:

$$p_a\big|_{reduced} = R.p_a \qquad p_b\big|_{reduced} = R.C.p_b$$

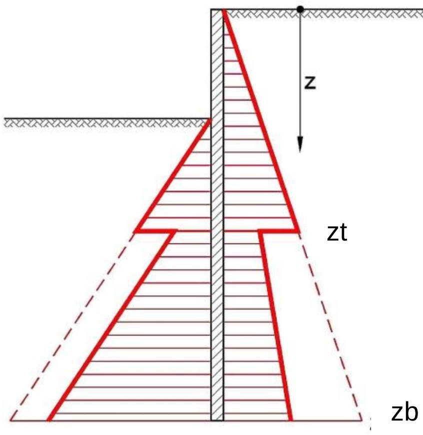


Figure C23: Definition of the reduced active pressure

Usually, the length of the active pressure is taken to equal the diameter (i.e. R = D/e), and the length of the passive pressure is equal to 2 to 3 times the diameter (i.e C = 2 to 3).

The standard NF P 94 282 recommends the following:

- L_b = 2 x D for purely cohesive soil (i.e. R x C = 2D/e), L_b = 3 x D for cohesive-frictional soil (i.e. R x C = 3D/e);
- $L_a = L_b$ (C = 1 according to the conventions used in K-Réa).

C.3.2. Soil overload

C.3.2.1. Caquot

It consists in a semi-infinite load on one side of the wall, at a depth z_0. Its application induces an increment of uniform horizontal stress below z_0:

$$\Delta\sigma_v(z) = q \quad \text{for} \quad z \geq z_0$$

C.3.2.2. Boussinesq overload

C.3.2.2.1. <u>General case</u>

It consists in a localized vertical overload, of length l and density S, applied at a depth z_0 and at a distance x from the wall. Its application induces an increment of the horizontal stress at the wall level estimated by integrating the Boussinesq solution (initially established for the case of semi-infinite homogenous soil):

$$\Delta\sigma_h = \alpha_e \frac{S}{\pi}\left(\text{atg}\left(\frac{hl}{x(x+l)+h^2}\right) + \frac{xh}{x^2+h^2} - \frac{(x+l)h}{(x+l)^2+h^2}\right) \quad \text{where} \quad h = z - z_0$$

The factor α_e designates an amplifier factor that takes into account the « mirror effect » implicitly induced by the presence of the retaining wall (by construction, this effect is not included in the Boussinesq solution). The value α_e can be approximately calculated using the following formula (NF P 94 282):

$$\alpha_e \approx \frac{x+2}{x+1}$$

In K-Réa, this horizontal stress increment is « transformed » in an equivalent increment of the vertical stress through the following association:

$$\Delta\sigma_v = \frac{1}{0{,}5}\Delta\sigma_h$$

On the base of this vertical stress increment (equivalent), the modification of the initial levels of active and passive pressure, is made according to the equation described in §C.2.5.1.1.

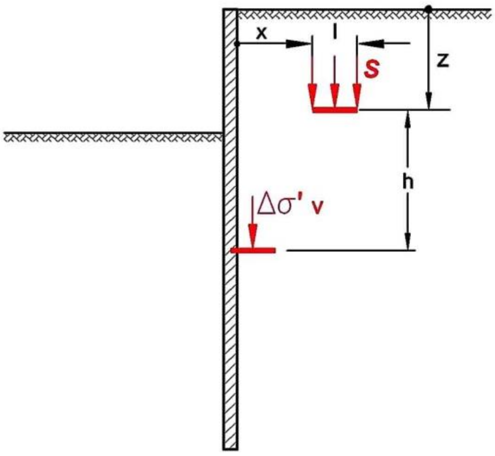


Figure C24: Simulation of an overload on the ground with the Boussinesq model

C.3.2.2.2. Case of an overload defined in the initial stage

For the overloads defined in the common calculation stages, the stress increment is only considered on the side where the overload is applied (wall effect). In the initial stage, when the wall isn't implemented yet, there is stress continuity from one side of the wall to the other and the increment that results from a declared overload in the initial stage is considered (initially) on both sides of the wall.

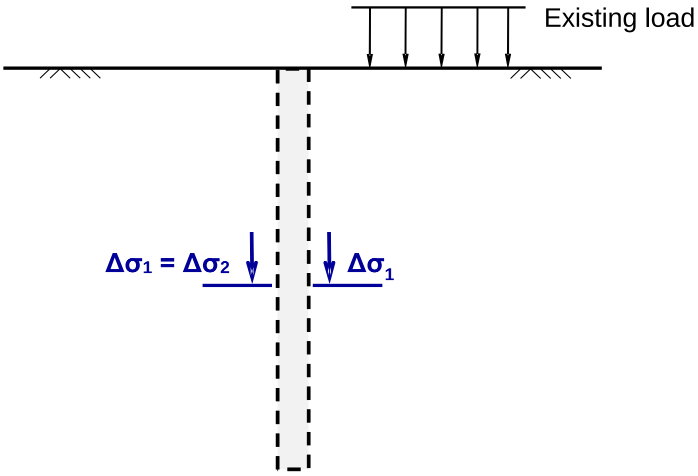


Figure C25: Treatment of a « Boussinesq » overload defined in the initial stage

Therefore, a Boussinesq overload defined in the initial stage (representative for example of an existing building) requires the following adaptations (applied automatically by the kernel):

- increment of the identical horizontal and vertical equivalent stresses on both sides of the wall;
- absence of the mirror effect ($\alpha_e = 1$).

These adaptations apply for the Boussinesq overloads and for the actions that depend on it (initial pressure related to the effects of the embankment and platform – cf. §C.3.3).

C.3.2.3. Graux overload

It consists in a localized overload on the ground whose underground diffusion is supposed to follow a "diffusion cone" linked to the shear parameters of the encountered layers, as shown in the figure below. The stress increment generated at a depth z is:

$$\Delta\sigma_v(z) = S\frac{l}{l_{diff}(z)}$$

Where $l_{diff}(z)$ designates the diffusion length at depth z. On the basis of this vertical stress increment, the modification of initial active and passive pressure levels is then conducted according to the equation described in §C.2.5.1.1.

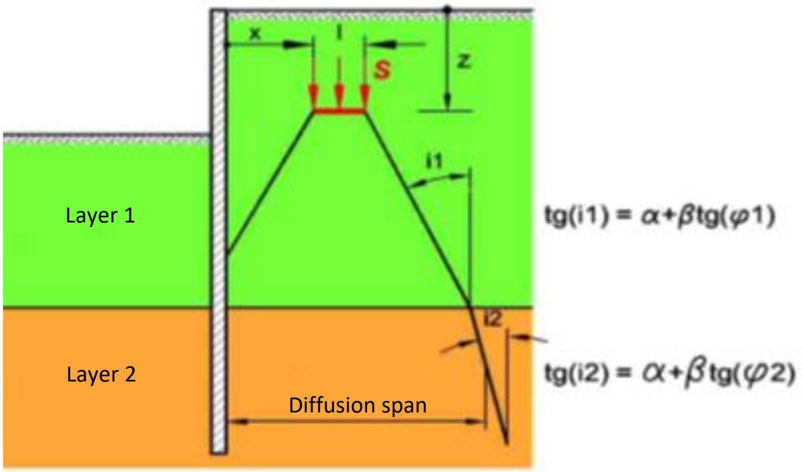


Figure C26: Diffusion principle of the vertical stress under a Graux overload

C.3.2.4. Elasto-plastic approach

Available soon.

C.3.3. Slope and berm

The simulation of slope and berm effects can be conducted with three different approaches.

C.3.3.1. Method of equivalent overloads

In the case of a slope, this method consists in assimilating the slope's weight to a superposition of Boussinesq overloads of equivalent density S(x) as shown in the figure herebelow. Active/passive pressure levels (initial and limit) are updated following the same steps as the ones described in §C.2.5.1.1 and §C.3.2.2.

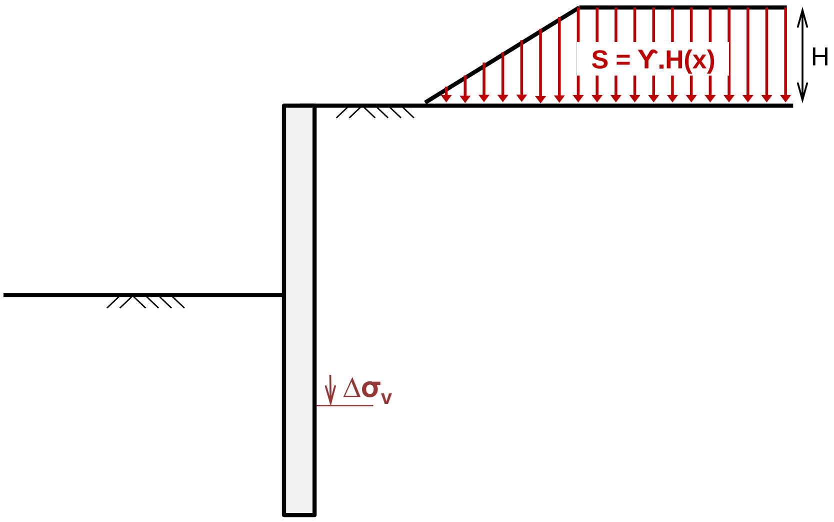


Figure C27: Simulation of the effect of a slope through Boussinesq equivalent overloads

In the case of a berm, this method consists in assimilating the berm to a fictitious horizontal layer whose weight is corrected by superposition of negative semi-infinite overloads applied at different levels on the height of the berm, as in the figure below.

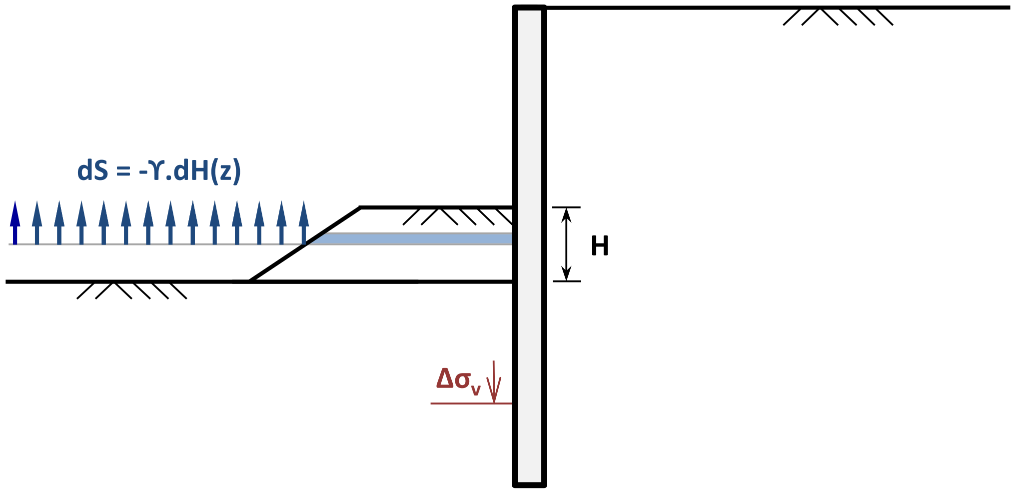


Figure C28: Simulation of the effect of a berm through equivalent Boussinesq overloads

The update of active/passive pressure levels (initial and limit) follows the same process as described in §C.2.5.1.1 and §C.3.2.2.

Attention is drawn on the fact that such an approach is likely to lead in some cases to <u>overly optimistic</u> results (cf. NF P 94 282).

C.3.3.2. Models complying with NF P 94-282

The application of the model below aims exclusively to control the diagrams of active/passive pressure <u>limits</u> in relation to the recommendations of standard NF P 94 282. The "initial" (or at rest) active pressure is, in any case, evaluated with the equivalent overloads method described previously.

C.3.3.2.1. Case of a slope

The standard NF P 94 282 recommends carrying out the evaluation of the effects of a slope in compliance with the Houy model as shown in the figure below.

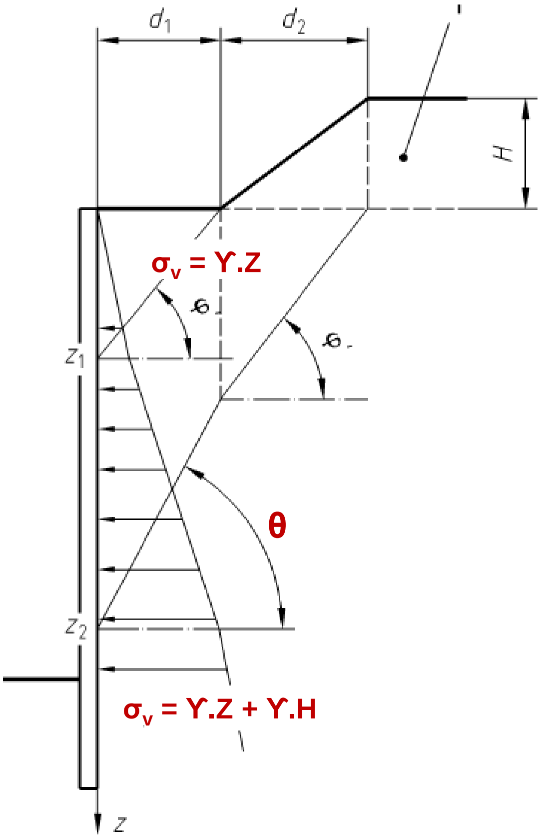


Figure C29: Effect of a slope according to the Houy model

According to the notations of the figure above:

- for $z \leq z_1$ slopes not taken into account
- for $z \geq z_2$ effect equivalent to a Caquot equivalent overload
- for $z_1 \leq z \leq z_2$ linear interpolation of active/passive pressure diagrams

The value of θ is taken to be equal to:

- $\theta = \frac{\pi}{4} + \frac{\varphi}{2}$ for the evaluation of the active pressure limit;
- $\theta = \frac{\pi}{4} - \frac{\varphi}{2}$ for the evaluation of the passive pressure limit;

The case of a multilayer requires a suitable reprocessing of the model, which is automatically managed by K-Réa (scheme incorporating a variable friction angle by layer).

C.3.3.2.2. Case of a berm

Houy's model principle previously described can be extended to the case of a berm as presented in figure below.

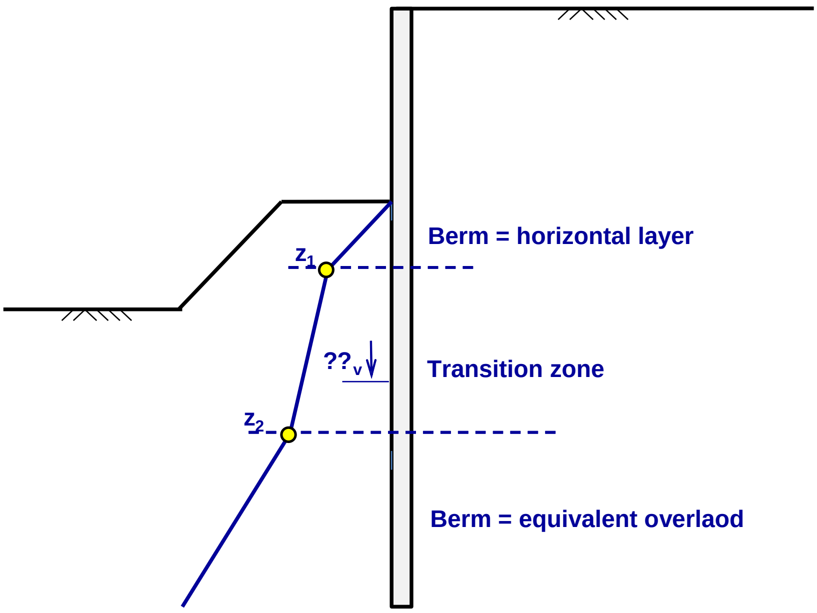


Figure C30: Effect of a berm according to the generalized Houy model

There are three areas:

- for $z \leq z_1$ — the effect of the berm is the effect of a horizontal layer
- for $z \geq z_2$ — effect equivalent to the effect of an equivalent overload
- for $z_1 \leq z \leq z_2$ — linear interpolation of active/passive pressure diagrams

Furthermore, the standard NF P 94-282 recommends, in the absence of an advanced approach, to control the limit of the passive pressure at height H of a berm by ensuring that it does not exceed the resulting shear force mobilisable at the base of the berm, according to the notations of the figure hereunder:

$$B_{max} = \frac{1}{2} k_p \gamma H^2 + k_{pc} c.H \leq W.\tan(\varphi) + c.L_r$$

It is implicitly assumed that the failure mechanism of the passive pressure is a horizontal plan which is developed preferentially at the base of the berm. Note that K-Réa applies this verification at all points at the full height of the berm.

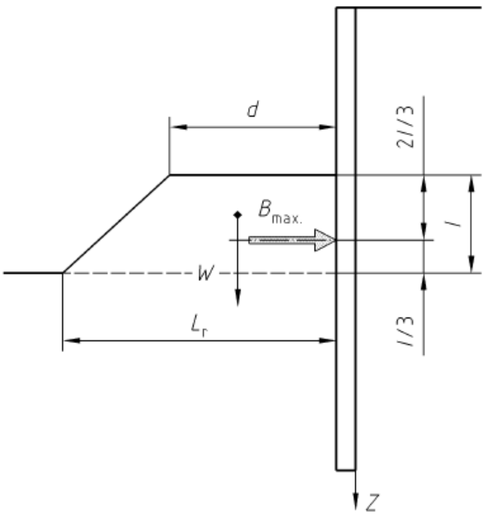


Figure C31: "Banquette approach" to control the limit passive earth pressure over the entire height of a berm

C.3.3.3. Kinematic method for failure surfaces

Using the kinematic method for failure calculations implemented in Talren v5 allows to work within a rigorous framework in which active/passive pressure diagrams can be evaluated for any type of stratigraphy as shown in the figure below (Cuira and Simon, 2016). For the active pressure limit, we need a stabilising pressure diagram that enables to obtain the limit equilibrium. For the passive pressure limit, the limit equilibrium is reached with a destabilizing diagram.

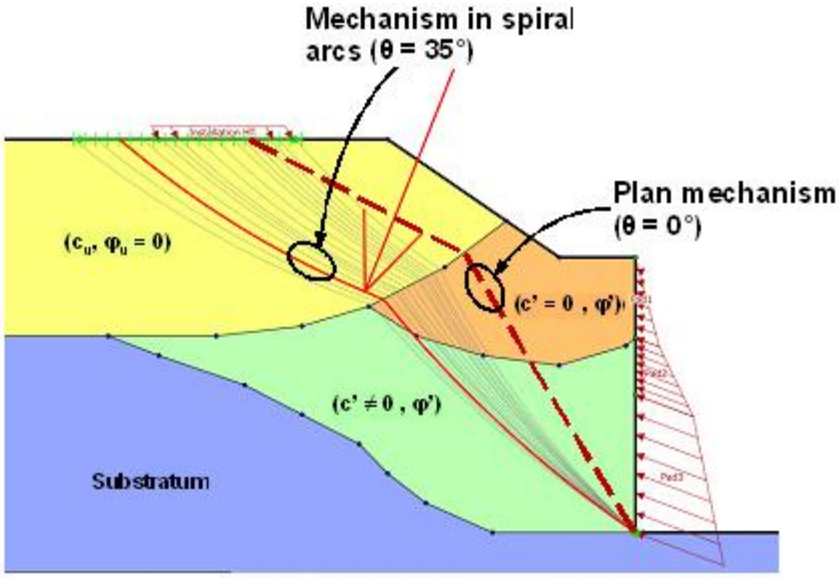


Figure C32: Failure calculations (software Talren v5) to determine the pressure limits

The active/passive diagrams obtained can be introduced directly into K-Réa using the option « imposed pressures ».

It should be noted that failure calculations also facilitate the recognition of strengthening reinforcements in the block (nails, inclusions, ballasted columns) as shown in the figure below (example of frequency of the passive pressure limit in a reinforced excavation with rigid inclusions).

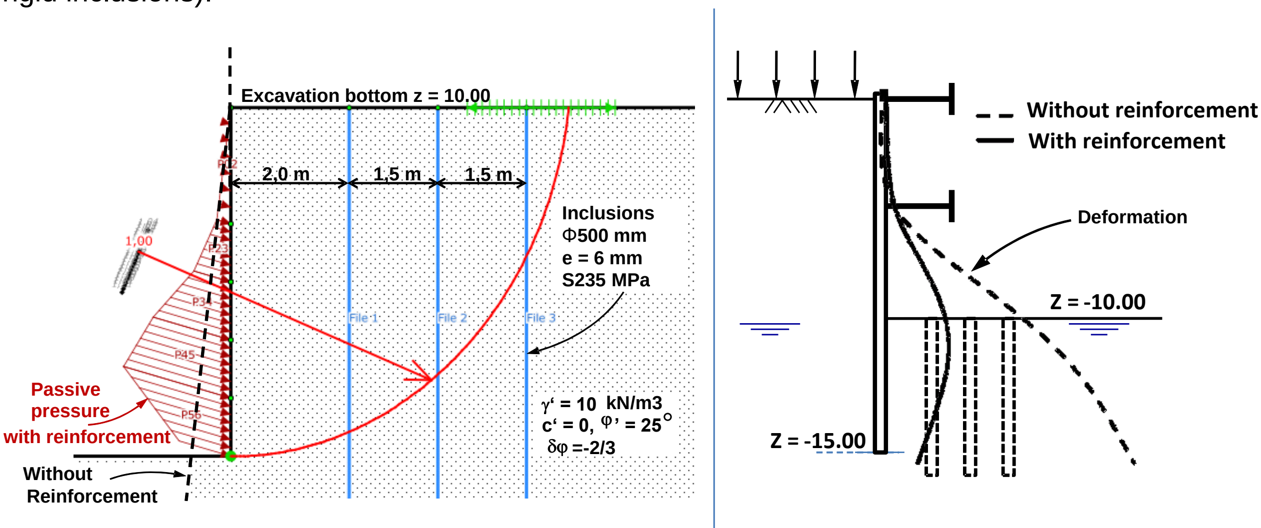


Figure C33: Use of failure calculations (software Talren v5) to determine the passive pressure limit in an excavation reinforced with inclusions

C.3.4. Treatment of load combinations

The treatment of complex projects with a large number of load cases requires an automated management study of different combinations depending on the regulatory framework applicable to the project. This concerns all applications where the wall is connected with civil engineering works (directly or indirectly through the foundation block). This also concerns the harbour structures with a high number of combinations to study and is too laborious for manual processing.

The phasing diagram usually considered for civil engineering calculations consists of treating load combinations through orphan complementary stages issued from the studied stage (stage 1 by combination). The validity of such pattern implicitly assumes “elastic linear” behaviour and the lack of any 'irreversible' displacements of the system, which is not the case for a retaining structure: in this case it is essential to ensure the consistency of the elastoplastic calculation for a given load combination. This justifies the use of the stage principle, shown below, which consists in generating a "full" phasing diagram in parallel to each of the combinations studied. Then, the interface only operates the stages for which the combination has been requested.

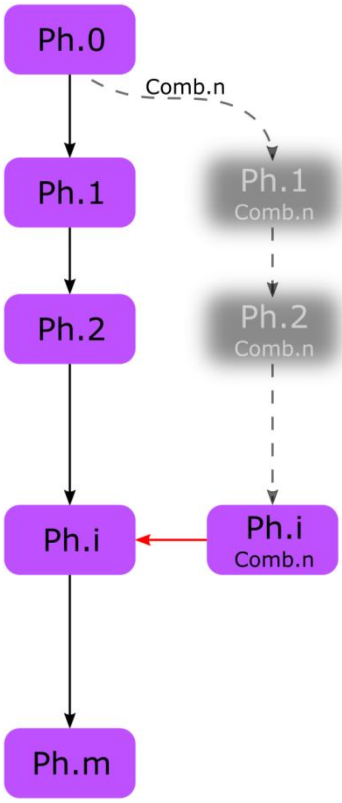


Figure C34: Phasing principle for the treatment of a load combination

Note that for ULS calculations, the defined weightings of load combinations are added to the specific ones that belong to the calculation approach of ULS checks:

- ULS Calculation without load combinations $S_{calculation} = \gamma_Q.S$
- ULS calculation with load combinations $S_{calculation} = \psi_{combination}.\gamma_Q.S$

C.3.5. Taking into account seismic conditions

C.3.5.1. Principle

The seismic effects in K-Réa are simulated using a pseudo-static approach, whose principles are the following (see figure below):

- Re-evaluation of limit levels of active pressure (p_a) and of passive pressure (p_b) on each side of the screen, taking into account the inertia forces in the soil;
- Reassessment of the water pressure on the wall taking into account hydrodynamic effects in the levels where the watertable is considered to be 'free' under earthquake conditions ('open' soil with or without earthquake);
- Taking into account the inertia forces $F_H = k_H\ P_{wall}$ x and $F_V = k_V$ x P_{wall} associated with the dead weight of the wall P_{wall};
- Revaluation of ancor stiffness;
- Zero modifications in the elastic level (k_h) and in the initial pressure p_i.

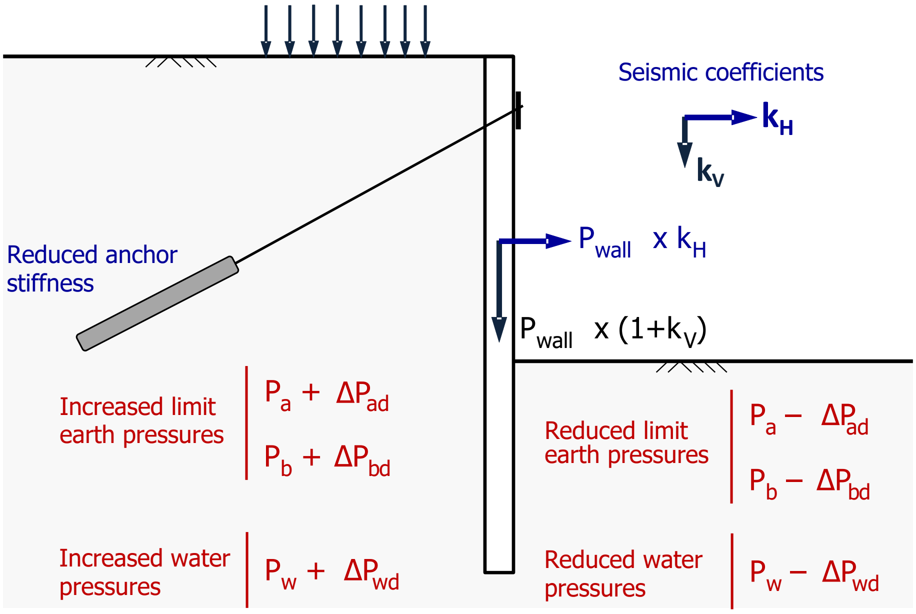


Figure C35: Taking into account seismic conditions – principles of the implemented method in K-Réa v4

C.3.5.2. Behaviour modes under seismic conditions

The implementation of the pseudo-static method for the calculation of the retaining structures under seismic conditions differentiates, in the framework of Eurocode 8 - part 5, three types (or modes) of soil behaviour under seismic stress: dry soil, 'open' soil and 'closed' soil. For each type of behaviour, the table below details the soil characteristics to take into account for the seismic calculations.

Case	Type of soil	Shear behaviour	Shear parameters	Soil weight
A	Sands and gravels above water table	Friction	Friction angle	$\gamma^* = \gamma$
B	Soil « open» below watertable = very permeable under seism	Friction	Friction angle	$\gamma^* = \gamma'$
C	Soil « closed» below watertable = « waterproof» under seism	Cohesion	Undrained cohesion	$\gamma^* = \gamma'$

Table C1: Types of behaviour under seismic conditions

C.3.5.3. Seismic coefficients

The implementation of the pseudo-static method is based on the concept of seismic coefficients defined as follows:

$$k_H = \frac{1}{r}\frac{a_N}{g} \qquad k_V = \pm\frac{1}{2}k_H$$

Where a_N refers to nominal seismic acceleration, which is a function of the seismicity zone, of the soil classification and of the type of structure.

The parameter 'r' is a dimensionless factor bigger than/or equal to 1, which is a function of the sensitivity of the stucture whose displacements have been studied. A value of r = 1 must be considered for a structure sensitive to displacements.

The concept of seismic coefficients allows to introduce the concept of equivalent seismic inclination Θ whose value depends on the type of behaviour according to the notations of the previous table:

- Case A (sands and gravels above water table) $\tan\theta = \frac{k_H}{1 \pm k_V}$
- Case B (open soil below water table) $\tan\theta = \frac{\gamma_d}{\gamma'} \cdot \frac{k_H}{1 \pm k_V}$
- Case C (closed soil below water table) $\tan\theta = \frac{\gamma}{\gamma'} \cdot \frac{k_H}{1 \pm k_V}$

Or

- γ total soil weight above water table;
- γ' submerged soil weight below water table;
- γ_d soil weight below water table (not submerged).

C.3.5.4. Increment of the active dynamic pressure (limit)

Seismic effects imply a reduction of the shear strength available and therefore an increase of the level of the ultimate active earth pressure through a "dynamic" increment Δp_{ad}, as schematized in the figure below.

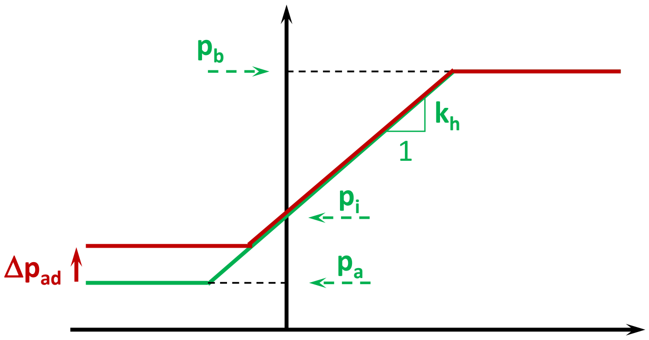


Figure C36: Taking into account a dynamic increment of the active earth pressure limit

The evaluation of this dynamic increment is conducted using a generalized form of the Mononobe-Okabe method (1924), extended to the case of a soil with a non-zero cohesion. This model consists in the generalization of the Coulomb active pressure corner by integrating to the forces equilibrium those related to the effects of inertia, which influence the mass of the block, as shown in the figure below: P refers to the « stabilizing » reaction of the wall at the limit equilibrium state (resulting in the limit of the active earth pressure).

The method simply explores the failure plan mechanisms forming an angle α with respect to the wall. For each value of α, the vertical and horizontal forces components at limit equilibrium leads to a system with two equations and two unknown values (R_f and P), which allows to estimate the value of P(α). Then, we calculate the value of α when P is at its maximum.

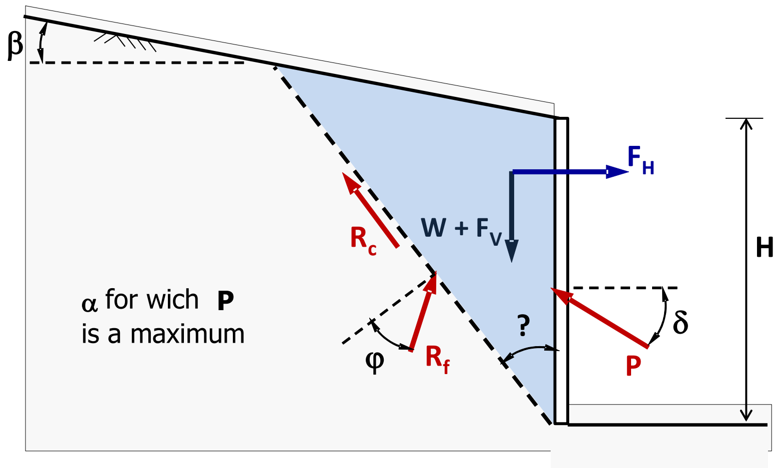


Figure C37: Mononobe-Okabe model for a non-zero cohesion soil – active pressure mechanism

The implementation of this model allows to establish the equation that results from the dynamic active pressure limit:

$$P_{ad} = K_{ad}\left[\frac{1}{2}\gamma^*(1 \pm k_V)H^2\right] - K_{acd}[cH]$$

The coefficients of dynamic active pressure K_{ad} and K_{acd} are functions of four parameters:

$$\begin{cases} K_{ad} = f_1(\varphi, \delta, \theta, \lambda) \\ K_{acd} = f_2(\varphi, \delta, \theta, \lambda) \end{cases} \quad \text{where} \quad \lambda = \frac{\gamma H}{2c}$$

Functions f_1 and f_2 are obtained through digital integration. The figure below shows the case of a horizontal active earth pressure (δ = 0).

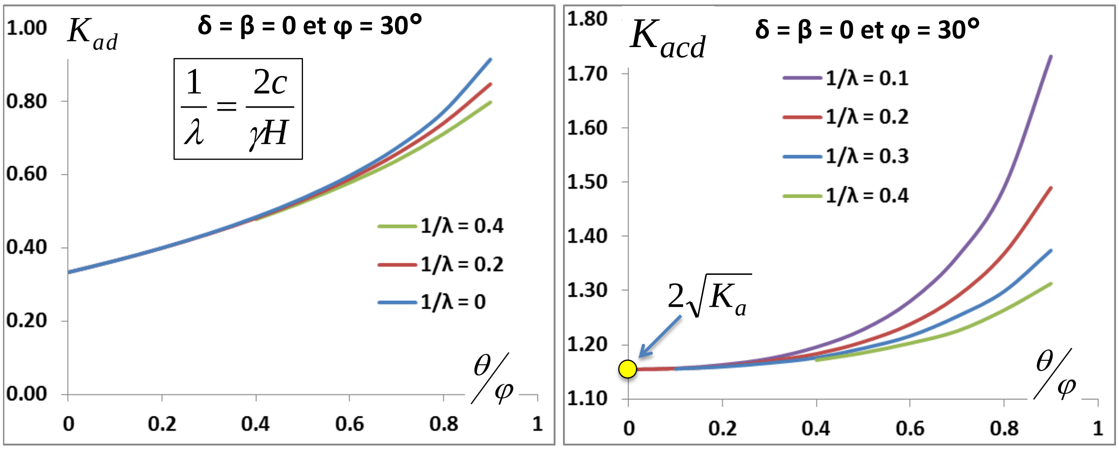


Figure C38: Mononobe-Okabe model for a non-zero cohesion - soil coefficients of dynamic active pressure

On the basis of the variation of P_{ad} with depth, we can estimate by differentiation, a dynamic active pressure density p_{ad} between depths z_{i-1} and z_i from the top of the wall:

$$p_{ad}(z_{i-1} \leq z \leq z_i) = \frac{P_{ad}(H = z_i) - P_{ad}(H = z_{i-1})}{z_i - z_{i-1}}$$

We can then deduce the 'dynamic' increment to be considered on the 'static' active pressure limit:

$$\Delta p_{ad} = p_{ad}(k_H, k_V) - p_{ad}(k_H = 0, k_V = 0)$$

The limit active pressure taken into account in the calculation can also be expressed as:

$$p_a\big|_{statique+dynamique} = p_a\big|_{statique} + \Delta p_{ad}$$

C.3.5.5. Increment of the (limit) dynamic passive pressure

Seismic effects imply a reduction in shear resistance and therefore a decrease in the level of the limit passive pressure through a "dynamic" increment Δp_{bd}, as schematized in figure below.

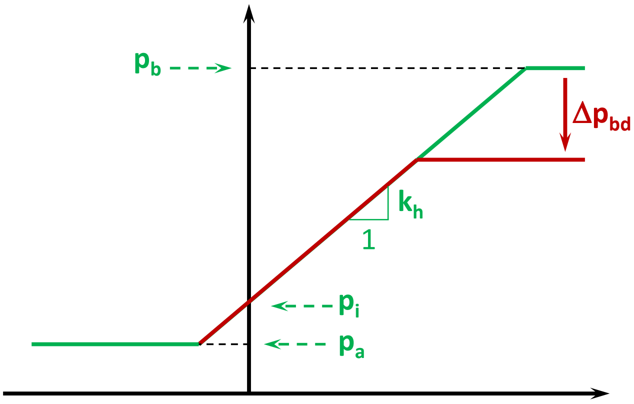


Figure C39: Taking into account a dynamic increment on the level of passive pressure limit

The evaluation of this dynamic increment is conducted using a generalized form of the Mononobe-Okabe method (1924), extended to the case of a soil with a non-zero cohesion. This model consists in the generalization of the Coulomb active pressure corner by integrating to the forces equilibrium those related to the effects of inertia, which influence the mass of the corner, as shown in the figure below: P refers to the « destabilizing » reaction of the wall at the limit equilibrium state (resulting in the limit of the passive earth pressure).

This method simply explores the failure plan mechanisms forming an angle α with respect to the wall. For each value of α, the vertical and horizontal forces components at limit equilibrium limit leads to a system to two equations and two unknown values (R_f and P), which allows to estimate the value of P(α). Then, we calculate the value of α when P is at its minimum.

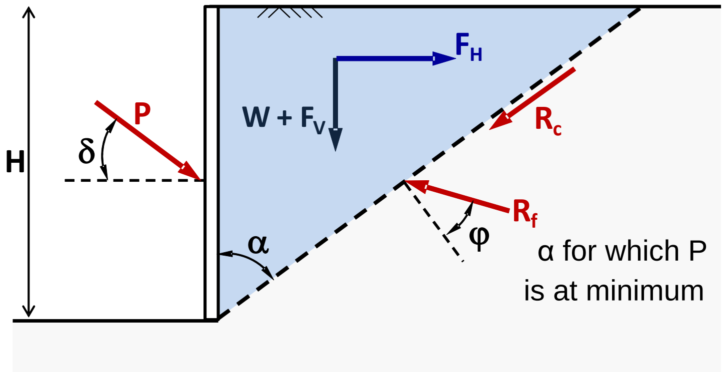


Figure C40: Mononobe-Okabe model for a non-zero cohesion soil – passive pressure mechanism

This method allows to establish the equation that results from the limit dynamic passive pressure:

$$P_{bd} = K_{pd}\left[\frac{1}{2}\gamma^{*}(1 \pm k_V)H^2\right] + K_{pcd}[cH]$$

The coefficients of dynamic passive pressure K_{pd} et K_{pcd} are functions of four parameters:

$$\begin{cases} K_{pd} = g_1(\varphi,\delta,\theta,\lambda) \\ K_{pcd} = g_2(\varphi,\delta,\theta,\lambda) \end{cases} \quad \text{where } \lambda = \frac{\gamma H}{2c}$$

The functions g_1 and g_2 are obtained by digital integration.

On the basis of the variation of P_{bd} with depth, estimated by differentiating a density of dynamic passive pressure p_{bd} between depths z_{i-1} and z_i from the top of the wall:

$$p_{bd}(z_{i-1} \le z \le z_i) = \frac{P_{bd}(H = z_i) - P_{bd}(H = z_{i-1})}{z_i - z_{i-1}}$$

We can then deduce the 'dynamic' increment to be considered on the limit "static" passive pressure:

$$\Delta p_{bd} = p_{bd}(k_H = 0, k_V = 0) - p_{bd}(k_H, k_V)$$

The limit passive pressure taken into account in the calculation is:

$$p_b\big|_{statique+dynamique} = XP.\left(p_b\big|_{statique} - \Delta p_{bd}\right)$$

Where XP is a multiplying factor (less than/or equal to 1.00) that aims at reducing the passive pressure taken into account in the calculation for structures that are sensitive to displacements (for sensitive industrial facilities XP is usually between 0.33 and 0.50).

C.3.5.6. Hydrodynamic effects

The hydrodynamic effects, which are likely to develop in the levels where the water is considered to be earthquake "free" (soil absence or 'open' soil), are simulated using the Westergaard method as shown in the figure below.

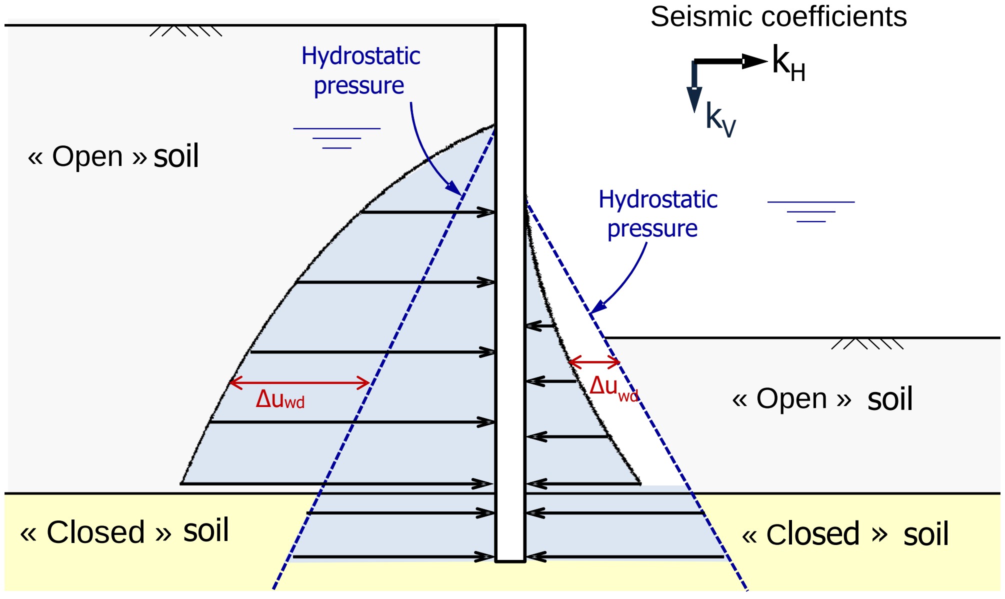


Figure C41: Principle of the Westergaard method as implemented in K-Réa

Taking into account seismic conditions implies a 'static' water pressure modification of the dynamic increment, such as (in the 'open' soil layers under the watertable):

$$u_w\big|_{static+dynamic} = u_w\big|_{static} \pm \Delta u_{wd}$$

In which:

$$\Delta u_{wd}(z) = \frac{7}{8} k_H \gamma_w \sqrt{Hz}$$

Where:

- Z designates the depth of the calculation point below the water table;
- H designates the height of the watertable from the base of the wall.

C.3.5.7. Modification of anchors stiffness

The seismic effects induce a modification of the anchors' visible stiffness, according to the following equation:

$$K_{dynamic} = \frac{1}{1+1{,}5\left|k_H\right|}\left(\frac{\cos(\alpha \pm \theta)}{\cos\alpha}\right)^2 .K_{static}$$

Where α refers to the inclination of the anchor from the horizontal axes.

C.4. ULS checks

C.4.1. Calculation approaches

C.4.1.1. Weighting principle

The weighting system of K-Réa is applied on moments (variable and permanent), moment effects (calculation results), strength parameters (shear characteristics), as well as on strenghht (passive pressure and anchors). Three calculation approaches are proposed (1, 2 and 3) according to Eurocode 7 and its application standard NF P 94-282.

C.4.1.1.1. Action weighting

Moment weighting is applied according to the following equation:

$$A_d = \gamma_A . A_k$$

In K-Réa, this concerns the following parameters:

- « Active » soil pressure — weighting of active pressure limit coefficients
- Water pressure — weighting of differential water pressure
- Soil overloads — weighting of the value of overloads characteristics
- Wall overloads — weighting of the value overloads characteristics

C.4.1.1.2. Weighting of moment effects

The weighting of the moment effects is applied according to the following equation:

$$E_d = \gamma_E . E_k$$

In K-Réa, this applies to the "results" of the calculation and aims to evaluate calculation values of loads on the wall, the anchors and on the soil:

- Loads on the wall — weighting of efforts diagram (N, V, M)
- Anchorage forces — weighting of reactions of struts and anchors
- Mobilized passive pressure — weighting of the mobilized passive pressure

The value of the partial coefficient γ_E is identical for all action effects.

C.4.1.1.3. Weighting of shear parameters

The weighting of shear parameters is applied according to the following equation:

$$\tan\varphi_d = \frac{\tan\varphi_k}{\gamma_M} \qquad c_d = \frac{c_k}{\gamma_M}$$

In K-Réa, this implies a reassessment of the (limit) active/passive pressure coefficients on the basis of the calculation value of shear parameters. It is worth noting that the pressure at rest (k0) coefficients and the reaction coefficient remain unaltered.

C.4.1.1.4. Strength weighting

The weighting of the resistances is applied according to the following equation:

$$R_d = \frac{R_k}{\gamma_R}$$

In K-Réa, this concerns the following parameters:

- Soil limit passive pressure — weighting of passive pressure (post-treatment)
- Anchorage structure — weighting of the elastic limit of anchors
- Anchor block — weighting of the disruptive strain issued from Kranz

C.4.1.2. Approach 2/2* - NF P 94 282

According to the Eurocode 7 application standard in France (NF P 94-282), the approach 2/2* offers partial coefficients which differ according to the calculation method used (SSIM or LEM) for the wall equilibrium:

- SSIM: weighting (post-processing) of the effects of moments and strength;
- LEM: weighting (at the source) of moments and strength;

In both cases, no weighting is applied to strength parameters.

The table below shows the partial coefficients proposed by default in K-Réa when this approach is used.

	Approach 2/2*		SSIM method	LEM method
Actions (γ_A)	Limit active soil pressure		1.00	1.35
	Water pressure		1.00	1.35
	Wall weight		1.00	1.35
	Loads applied on soil	Permanent	1.00	1.00
		Variable	1.11	1.11
	Loads applied on wall	Permanent favorable	1.00	1.00
		Permanent unfavorable	1.00	1.35
		Variable unfavorable	1.11	1.50
Effect of actions (γ_E)	Wall loads Anchor loads Mobilized passive pressure		1.35	1.00
Strength parameters (γ_M)	Friction angle	Drained behaviour	1.00	1.00
	Cohesion (effective)	Drained behaviour		
	Friction angle	Undrained behaviour	1.00	1.00
	Cohesion	Undrained behaviour		
Resistances (γ_R)	Mobilisable passive pressure	Permanent stage	1.40	1.40
		Transitory stage	1.10	1.10
	Support strength	Elastic limit	1.00	-
	Anchor block (Kranz)	Disruptive strain	1.10	-

*Table C2:Partial coefficients applied WITH approach 2/2 **

C.4.1.3. Approach 3

Approach 3 offers by default identical partial coefficients between SSIM and LEM methods.

Unlike approach 2/2*, this approach is characterized by the weighting application at the source on the strength parameters (c and φ), which requires a reassessment by the computation engine of the active/passive pressure coefficients considered in the ULS calculations:

$$k_{a,d} = k_a\left(\frac{\tan\varphi_k}{\gamma_M}\right) \qquad k_{p,d} = k_p\left(\frac{\tan\varphi_k}{\gamma_M}\right)$$

Then, except for the transitory overloads (weighted by 1.30), weighting isn't applied on actions (nonstructural initial permanent loads), on moment effects or on strength.

It should be noted that this approach doesn't allow (by default) to differentiate any security level between transitory and permanent stages.

The table below shows the partial coefficients proposed by default in K-Réa when this approach is used.

		Approach 3	SSIM method	LEM method
Actions (γ_A)	Limit active soil pressure		1.00	1.00
	Water pressure		1.00	1.00
	Wall weight		1.00	1.00
	Loads applied on soil	Permanent	1.00	1.00
		Variable	1.30	1.30
	Loads applied on wall	Permanent favorable	1.00	1.00
		Permanent unfavorable	1.35	1.35
		Variable unfavorable	1.50	1.50
Effect of actions (γ_E)	Wall loads Anchor loads Mobilized passive pressure		1.00	1.00
Strength parameters (γ_M)	Friction angle	Drained behaviour	1.25	1.25
	Cohesion (effective)	Drained behaviour		
	Friction angle	Undrained behaviour	1.40	1.40
	Cohesion	Undrained behaviour		
Resistances (γ_R)	Passive pressure mobilized	Permanent stage	1.00	1.00
		Transitory stage	1.00	1.00
	Support strength	Elastic limit	1.00	-
	Anchor block (Kranz)	Disruptive strain	1.00	-

Table C3: Partial coefficients applied with approach 3

C.4.1.4. Approches 1.1/1.2

Approach 1 has two "variations":

- a possible variant 1.1 similar to approach 2 (moment weighting, no weighting of the strength parameters);
- a possible variant 1.2 similar to approach 3 (weighting of strength, no weighting of moments);

In countries where this approach applies (for example in England), it is advised to examine successively both variants and retain the one leading to the worst-case scenario design.

The tables below show the partial coefficients proposed by default in K-Réa when this approach is used.

	Approach 1.1		SSIM method	LEM method
Actions (γ_A)	Limit active soil pressure		1.35	1.35
	Water pressure		1.35	1.35
	Wall weight		1.35	1.35
	Loads applied on soil	Permanent	1.00	1.00
		Variable	1.11	1.11
	Loads applied on wall	Permanent favorable	1.00	1.00
		Permanent unfavorable	1.35	1.35
		Variable unfavorable	1.50	1.50
Effect of actions (γ_E)	Wall loads Anchor loads Mobilized passive pressure		1.00	1.00
Strength parameters (γ_M)	Friction angle	Drained behaviour	1.00	1.00
	Cohesion (effective)	Drained behaviour		
	Friction angle	Undrained behaviour	1.00	1.00
	Cohesion	Undrained behaviour		
Resistances (γ_R)	Passive pressure mobilized	Permanent stage	1.00	1.00
		Transitory stage	1.00	1.00
	Support strength	Elastic limit	1.10	-
	Anchor block (Kranz)	Disruptive strain	1.00	-

Table C4: Partial coefficients applied with approach 1.1

	Approach 1.2		SSIM method	LEM method
Actions (γ_A)	Limit active soil pressure		1.00	1.00
	Water pressure		1.00	1.00
	Wall weight		1.00	1.00
	Loads applied on soil	Permanent	1.00	1.00
		Variable	1.30	1.30
	Loads applied on wall	Permanent favorable	1.00	1.00
		Permanent unfavorable	1.00	1.00
		Variable unfavorable	1.30	1.30
Effect of actions (γ_E)	Wall loads Anchor loads Mobilized passive pressure		1.00	1.00
Strength parameters (γ_M)	Friction angle	Drained behaviour	1.25	1.25
	Cohesion (effective)	Drained behaviour		
	Friction angle	Undrained behaviour	1.40	1.40
	Cohesion	Undrained behaviour		
Resistances (γ_R)	Passive pressure mobilized	Permanent stage	1.00	1.00
		Transitory stage	1.00	1.00
	Support strength	Elastic limit	1.10	-
	Anchor block (Kranz)	Disruptive strain	1.00	-

Table C5: Partial coefficients applied with approach 1.2

C.4.2. Soil levels

K-Réa offers the possibility to "weight" the soil levels considered in the calculations (SSIM and LEM methods). This "weighting " is controlled by an 'over-excavation' parameter Δa which is user-defined for each side and each stage.

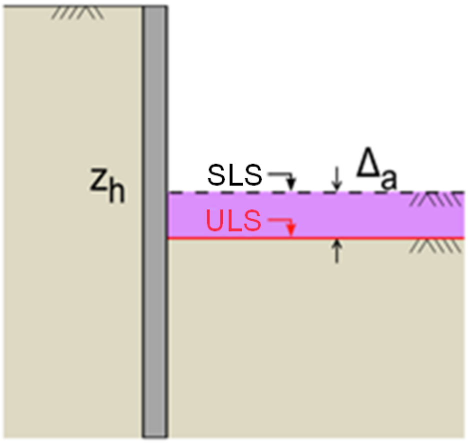


Figure C42: Soil Levels - notion of over-excavation

In the absence of control on the bottom of the excavation, standard NF P 94-282 recommends the following value:

$$\Delta a = \min(50\text{ cm}, 10\%H)$$

In which H designates the height of the effective support defined as shown in the figure below.

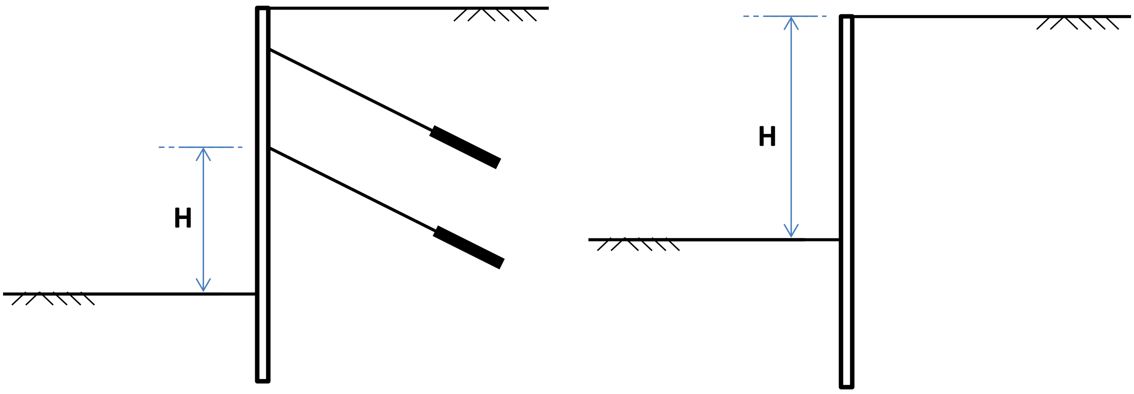


Figure C43: Notion of effective retaining height

C.4.3. Passive pressure failure check

The passive pressure failure check verifies if the embedment length available allows a high enough level of security between the mobilisable passive pressure and the level required for the wall equilibrium (ULS).

C.4.3.1. General case

For the general case of the stages where the wall has one or more anchor levels, this check is carried out on the basis of the SSIM method results, according to the following equation:

$$B_{t,d} \leq B_{m,d}$$

Where:

- $B_{t,d}$: calculation value of the result of mobilisable passive pressure (from the SSIM method);
- $B_{m,d}$: calculation value of the result of the available passive pressure (or utlimate).

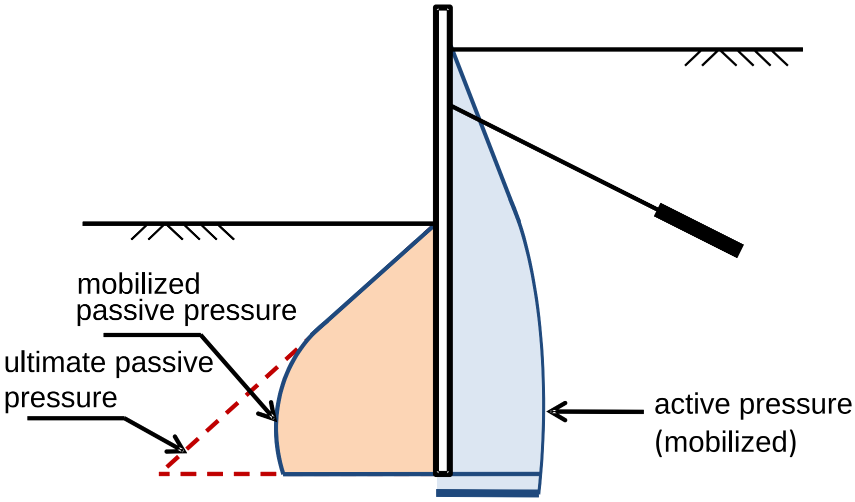


Figure C44: Mobilized and ultimate passive earth pressure for an anchored wall equilibrium

Calculation values of the mobilized and mobilisable passive pressures are defined on the basis of the following relations:

$$B_{t,d} = \gamma_E . B_{t,k} \qquad\qquad B_{m,d} = \frac{B_{m,k}}{\gamma_R}$$

The values of γ_E and γ_R are specified (for each calculation approach) in the chapter C.4.1. In particular, in the approach 2/2 * (NF P 94 282), the values considered by default in K-Réa (SSIM method) are the following:

Stage	γ_E	γ_R
Transitory	1,35	1,10
Permanent	1,35	1,40

*Table C6: Example of weighting applied in the approach 2/2**

C.4.3.2. Special case: stages where the wall is cantilever

C.4.3.2.1. Principle

The NF P 94-282 standard imposes the use of the limit equilibrium model (LEM) for ULS calculation when the retaining wall is cantilever.

As suggested by its name, this model consists in studying the retaining wall's equilibrium, (the wall being assumed perfectly rigid - the calculation does not consider the wall flexibility) by considering that the soil on both sides of the wall reaches the limiting earth pressure, down to a certain point called « transition point ». Beyond this point, the soil is assumed to reach the limiting counter active pressure on the downhill side of the wall, whereas on the uphill side, we check that the counter passive pressure necessary for horizontal equilibrium of the retaining wall is lower, with sufficient safety, than the limiting counter passive pressure available below the transition point (see **Erreur ! Source du renvoi introuvable.**).

The « transition point » is defined in paragraphs §C.4.3.2.3 and §C.4.3.2.4.

With the notations in **Erreur ! Source du renvoi introuvable.**, the equilibrium of the retaining wall involves the following force system:

- F_a: horizontal resultant of the active earth pressures diagram $p_{a,d}$
- F_b: horizontal resultant of the passive earth pressures diagram $p_{b,d}$
- Fc_a: horizontal resultant of the counter active earth pressures diagram $pc_{a,d}$
- Fc_b: horizontal resultant of the diagram of available counter passive earth pressures $pc_{b,d}$
- ΔU: horizontal resultant of the differential water pressures diagram $u_a - u_b$

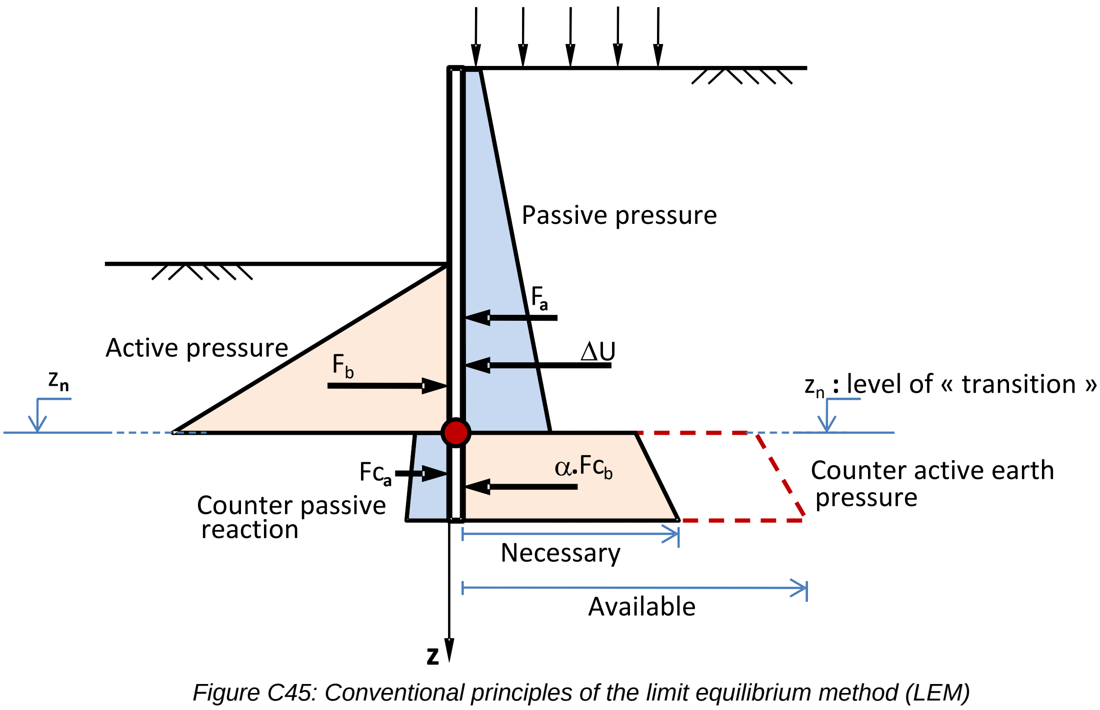


Figure C45: Conventional principles of the limit equilibrium method (LEM)

The « α » factor is called « mobilisation » factor of the counter earth resistance (passive pressures), defined as being the ratio between the counter earth resistance necessary for the horizontal equilibrium of the retaining wall and that available (or limiting). Pressure diagrams displayed above are expressed in "design values" according to the weighting system detailed in § C.4.1. The wall limit equilibrium also considers the surcharges applied directly to the retaining wall (linear force, couple, trapezoidal surcharge) also expressed in design values.

Based on this model, and according to the provisions of the NF P 94-282 standard, stability is checked with respect to passive side failure by performing the following tests:

- Check of the embedment, which consists in ensuring that the available embedment exceeds, with sufficient safety, the minimum embedment necessary for moment equilibrium.
- Check of the counter-earth resistance, which consists in verifying that the counter-earth resistance available under the transition point is sufficient to equilibrate horizontal forces. The application of this check requires determining the position of the transition point. For this, two models of calculation are available in K-Réa: approach D (applied by default) and approach F.

C.4.3.2.2. <u>Embedment check</u>

The check of the retaining wall embedment is based on the following condition (**Erreur ! Source du renvoi introuvable.**):

$$f_b \geq 1{,}20\, f_0$$

Where:

- f_b: embedment « available » below the zero differential pressure point O;
- f_0: minimum embedment, below the zero differential pressure point O, required to achieve moments equilibrium (above point C);

According to the notations of the **Erreur ! Source du renvoi introuvable.**, we have: $f_b = (z_P - z_O)$ et $f_0 = (z_C - z_O)$.

The differential pressure mentioned, noted pd, designates the resultant diagram obtained by superposing the design values of the active earth pressures, passive earth pressures, and water pressures diagrams. So, we have, by definition (for the cases where the excavation is located on the left):

$$p_d = p_{a,right} - p_{b,left} + u_{right} - u_{left}$$

Searching point C consists in writing the general equation translating the momentum equilibrium with respect to this same point:

$$M(p_d)|_C + M(S_d)|_C = 0$$

Where:

- $M(p_d)|_C$: moment relative to point C, resultant of the differential diagram pressures p_d (between the top of the wall and point C);
- $M(S_d)|_C$: moment relative to point C, resultant of the overloads applied directly on the wall between the top and the point C.

This equation is resolved by a dichotomous search process with a relative stop criterion set to 10^{-4}.

On Figure C46:, the force R_C refers to the resultant (design value) of the horizontal forces applied over the height between the top of retaining wall and point C:

$$R_C = -R(p_d)|_C - R(S_d)|_C$$

Where:

- $R(p_d)|_C$: resultant of the differential pressures diagram p_d over the height between the top of the retaining wall and point C;
- $R(S_d)|_C$: resultant of surcharges (in design values) applied directly on the wall between its top and point C.

Checking the counter earth resistance aims at ensuring that the counter earth resistance available is sufficient to take-up the R_C load.

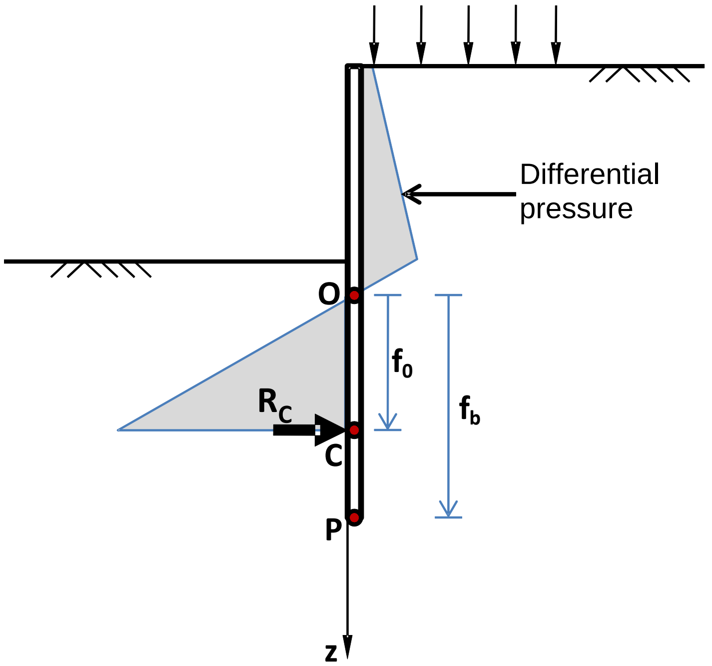


Figure C46: Notions of minimum embedment f0 and available embedment fb according to the LEM method

C.4.3.2.3. Checking failure on the passive side with approach D

This approach, applied by default in K-Réa, rigorously searches the transition point $\mathbf{z_n}$ to ensure the overall equilibrium for both forces and moments on the height of the screen (figure below).

In this method, the embedment (conventional, counted from point O) of the wall used in the calculation can be "set" according to three options (figure below):

- Option 1 calculation embedment = real wall calculation sheet (option by default);
- Option 2 calculation embedment = 1,2 x f_0 (recommended for long embedment);
- Option 3 calculation embedment = value imposed by the user.

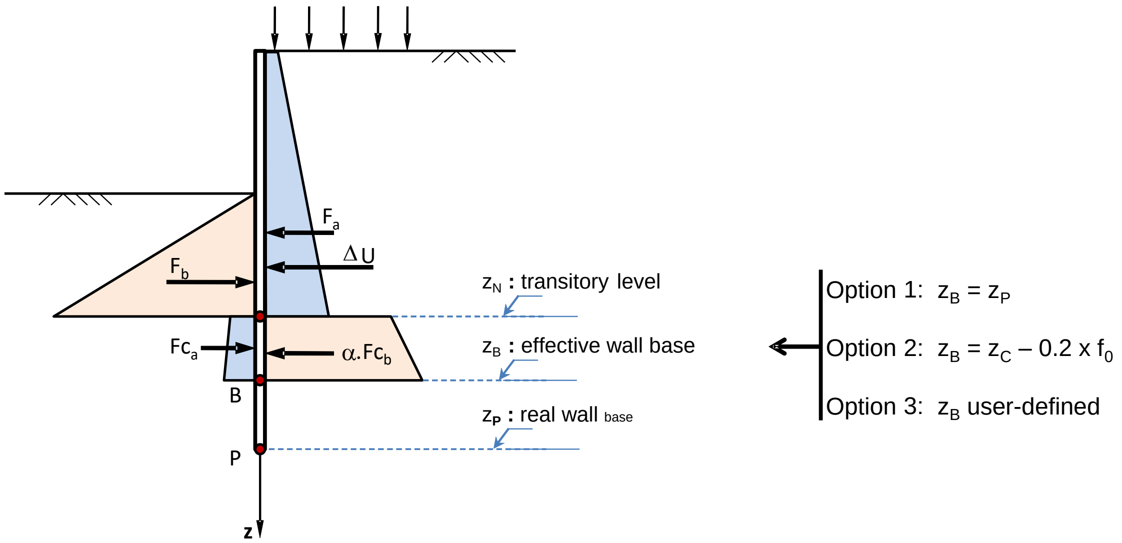


Figure C47: Counter-passive earth pressure check according to approach D

According to the notations of the previous figure, the overall equilibrium of the wall can be expressed with a system of two equations with two unknown values (**α, z_n**):

- Forces equilibrium: $F_a - F_b + \alpha.Fc_b - Fc_a + \Delta U + R(S_d) = 0$
- Moments equilibrium: $M(F_a) - M(F_b) + \alpha.M(Fc_b) - M(Fc_a) + M(\Delta U) + M(S_d) = 0$

Where:

- F_a, F_b, Fc_a, Fc_b are respectively the resultants of the diagrams of active pressure, passive pressure, counter-active pressure and counter-passive pressure. Their values are functions of the position of the transition point $\mathbf{z_n}$;
- $M(F_a)$, $M(F_b)$, $M(Fc_a)$, $M(Fc_b)$ are respectively the forces moments F_a, F_b, CF_a, CF_b in relation to the point P (bottom of the wall). Their values are also functions of the position z_n;
- ΔU and M(ΔU) are respectively the resultant of the diagram of differential water pressures and the moment that corresponds to the point P. Their values are independent of z_n;
- $R(S_d)$ and $M(S_d)$ are respectively the resultant and the moment in relation to P of potential overloads (design values) applied directly on the wall.

Solving this equation system is carried out through a process of dichotomous research with a relative stopping criterion set by default at 10^{-4}. Using this approach allows to obtain simultaneously the transition level $\mathbf{z_n}$ and **α** factor allowing to check the counter earth resistance through the condition: $\alpha \le 1$.

C.4.3.2.4. Checking failure on the passive side using approach F

The approach F is a simplified method that consists in assimilating the mobilized counter-passive earth resistance to a uniform pressure applied to a length equal to 0,2 f_0 on one side and from point C on the other, as shown in the figure below.

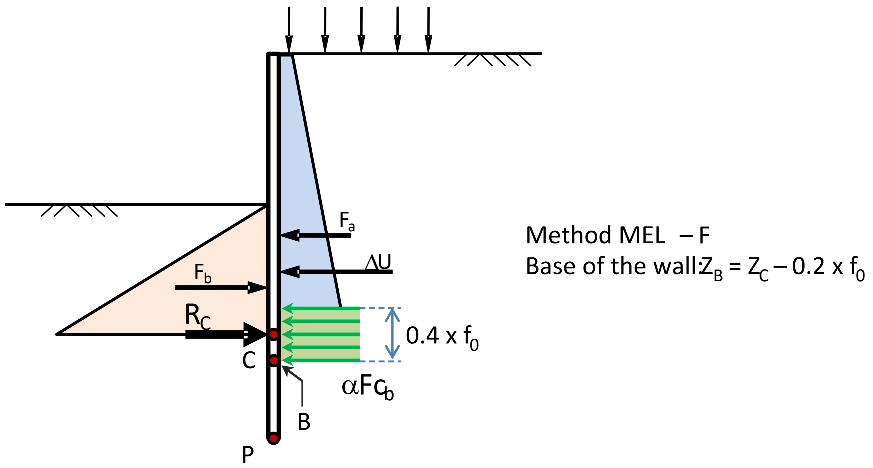


Figure C48: Counter-passive earth resistance check according to approach F

Hence, according to the notations in the figure above, the equilibrium of horizontal forces results in the equality:

$$R_C = \alpha.Fc_b - Fc_a + \Delta U_{inf} + R(S_d)\big|_P^C$$

Where:

- $R(S_d)_P^C$: is the resultant of surcharges (if any), applied directly on the retaining wall below point C;
- ΔU_{inf}: is the resultant of differential water pressures applied to the retaining wall below point C.

The mobilization factor « α » is thus obtained through the relation:

$$\alpha = \frac{R_C + Fc_a - \Delta U_{inf} - R(S_d)_P^C}{Fc_b}$$

C.4.4. Calculation of ULS loads

The calculation of ULS loads is conducted according to the same method used for the passive pressure failure: SSIM for the stages when the screen is anchored, LEM for the stages when the wall is considered cantaliver. The design value of the loads on the wall and the anchors is obtained according to the following equation:

$$E_d = \gamma_E . E_k$$

As a reminder, in the case of the approach 2/2 * (NF P 94 282), the value of γ_E is equal to:

- γ_E =1,35 for the SSIM method established by default without weighting on permanent actions and strengths;
- γ_E =1,00 for the LEM method established by default with weighting at the source of permanent actions by 1,35 and strengths by 1/1,10 or 1/1,40.

It should be noted that in the case where the approach 3 is used, we have γ_E =1,00 for SSIM and LEM methods. These are established with weighting at the source of shear parameters by 1.25.

C.4.5. Verification of vertical equilibrium

C.4.5.1. General case

Checking vertical balance consists in estimating the vertical resultant of the forces applied to the retaining wall, and checking whether this resultant is oriented upwards (negative value), or downwards (positive value). The vertical resultant of the forces, if oriented downwards, should then be used as an input parameter to check the bearing capacity of the retaining wall (using specific calculation methods not integrated into K-Réa).

This check notably allows to consider the relevance of the values considered for the obliquities of active, passive and counter passive earth pressures.

The design value of the vertical resultant Rv_d of the forces applied to the retaining wall is given by the following general expression:

$$Rv_d = P_d + Pv_d + Fv_d + Tv_d$$

Where:

- P_d: total weight of the wall;
- Pv_d: design value of the vertical resultant of the earth pressures on the height of the wall;
- Fv_d: design value of the vertical resultant of the inclined linear forces applied to the retaining wall;
- Tv_d: design value of the vertical resultant of forces due to inclined anchors connected to the wall .

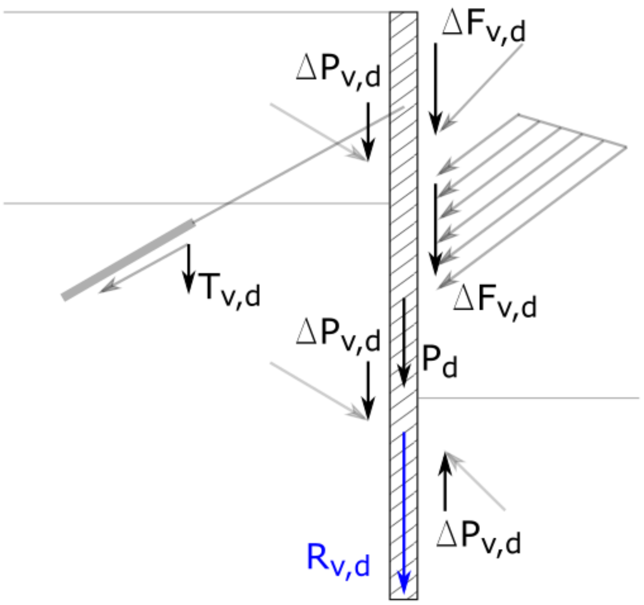


Figure C49: Vertical forces along the wall

It should be recalled that K-Réa calculates, at ULS and SLS, the axial force (vertical) at each point of the wall. The vertical resultant of the forces is none other than the value of the axial force at the base of the wall:

$$\mathrm{Rv_d} = \mathrm{N}^{zz}_{\mathrm{ULS}}\left(z = z_{base}\right)$$

C.4.5.2. Case of a wall cantaliver

In the case of a wall cantaliver, the ULS equilibrium of the wall treated with a limit equilibrium mehod, the vertical component of earth pressure is directly obtained through a projection according to the "limit" inclinations (and not intermediate) of active/passive pressures defined by the user.

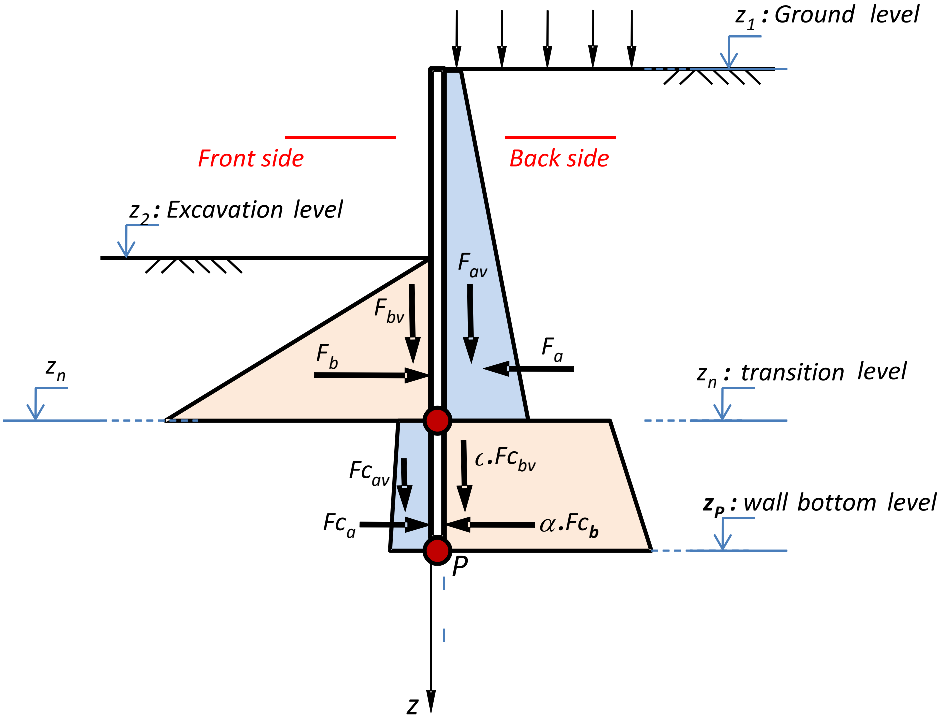


Figure C50: Assessment of the vertical forces for the limit equilibrium method (LEM)

In the case where the obtained vertical resultant is directed upwards, K-Réa offers the possibility to change, manually or automatically, the counter-passive pressures inclination in order to obtain a "relevant" vertical equilibrium (i.e. with resultant downwards). In the "automatic" mode, this adjustment is controlled by a 'Xcb' factor defined as follows:

$$(\delta/\varphi)_{\text{counter-passive earth pressure}} = Xcb \times (\delta/\varphi)_{\text{passive earth pressure}}$$

The Xcb factor has an initial value of 1.00 and then is reduced automatically (if necessary) until you obtain a downwards vertical resultant. The process stops in any case when Xcb reaches the value of -1,00.

Note that changing the inclination of the counter-passive pressure implies a recalculation of counter-passive reaction coefficients $k_{p,\ cb}$ and $k_{pc,\ cb}$, which are involved in the calculation of the counter-passive reaction available under the transition point z_n. These coefficients are re-automatically calculated by the program according to the "reference" method designated by the user ('Kerisel and Absi' by default).

C.4.6. Check of the stability of the anchoring block

C.4.6.1. General principle

The general principle of the check is to ensure that the anchor forces (for active anchors only) can be safely transferred to the ground, by checking the stability of the failure surface at the bottom of the soil block, and thus to prove that each anchor's length is sufficient.

This check is led according to the simplified « Kranz » approach mentioned in appendix G of the NF P 94 -282 standard. The method is said to be simplified as it uses a plane failure surface (CD), as shown in **Erreur ! Source du renvoi introuvable.**.

As specified in the notations of **Erreur ! Source du renvoi introuvable.**, this check consists in justifying the stability of the ABCDA block by ensuring that the anchor force remains inferior to a limit value corresponding to the ultimate equilibirum of the block, called « destabilising force ». The « Kranz » method defines an approach allowing to determine this destabilising force.

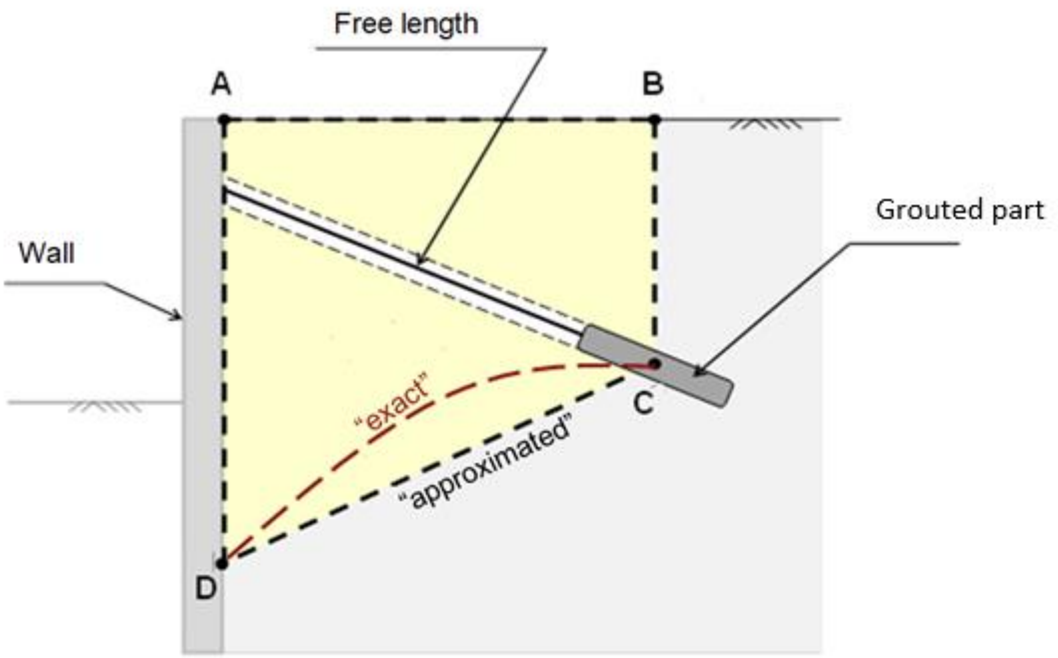


Figure C51: Simplified Kranz method - diagram

C.4.6.2. Case of a single anchor

C.4.6.2.1. Définition of the anchoring block

The anchoring block ABCDA subject of the check is bounded by the following points:

- **A:** top of the wall or intersection of the wall with the top of the first layer;
- **D**: zero shear level (taken under the bottom of the excavation);
- **C**: effective anchoring point corresponding to the effective length of the anchor $\mathbf{L_u}$;
- **B**: vertical projection of point C on the axis (AX);

C.4.6.2.2. External forces

The Figure C52: summarizes the assesment of the forces applied on the block ABCDA:

- T_u: force in the anchor;
- P1: reaction of the retaining wall, taken equal to the resultant of earth pressures on [AD];
- P_2: resultant of active pressures applied uphill the block on [BC];
- W: weight of the block (wet above the watertable, and submerged below). Groundwater level is assumed horizontal;
- F_e: resultant of external surcharges applied to or in the block;
- R_c: limit resistance due to cohesion mobilisable along [CD];
- R_f: limit resistance due to friction mobilisable along [CD].

The limit equilibrium of the block is hence converted into the vectorial equation:

$$\vec{R_c} + \vec{R_f} + \vec{W} + \vec{F_e} + \vec{P_1} + \vec{P_2} + \vec{T} = \vec{0}$$

The previous figures call for several comments:

- The friction force R_f is tilted at an angle equal to φ with respect to the normal to (CD). In the case of a homogeneous soil block, this inclination is merely equal to the soil friction angle;
- The horizontal component of P_1, noted P_{1H}, is calculated directly by integration of mobilised horizontal pressures, resulting from the horizontal equilibrium calculation of the retaining wall (SSIM method with weighting of the surcharges by 1.11). Its vertical component, noted P_{1V} is calculated with the same process as the one considered when checking vertical equilibrium of the retaining wall (see §**Erreur ! Source du renvoi introuvable.**).
- The uphill resultant of active pressures P_2 is assumed to be horizontal ($P_{2V} = 0$). Its horizontal component P_{2H} is calculated directly from the properties of the layers encountered between B and C, and considering the surcharges applied uphill the anchoring block;
- The force R_c is calculated by simple integration of soil cohesion along the [CD] segment (taking into account the potential variation with depth).

In the next sections, T_{dsb} designates the value of T allowing to reach equilibrium of the block (destabilising anchor force).

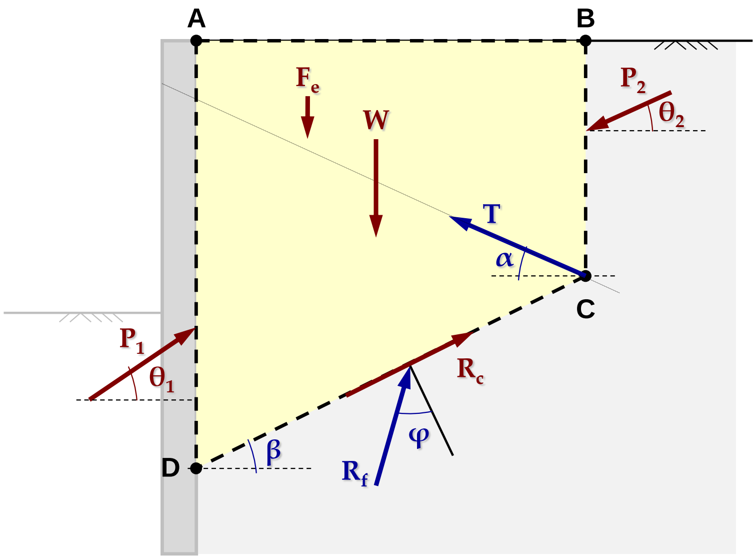


Figure C52: Schematic review of the forces exerted on the anchor

C.4.6.2.3. Discretization of the anchoring block

We consider the general case in which the assumed failure surface [CD] intersects several soil layers. In this case, the resolution of the limit equilibrium of the block requires discretising the block (ABCDA) into as many blocks as there are layers crossed, so as to ensure that the « base » of a given block is « homogeneous ». The point of this discretisation is to set the inclination of the mobilisable friction force at the base of each block (see figure below).

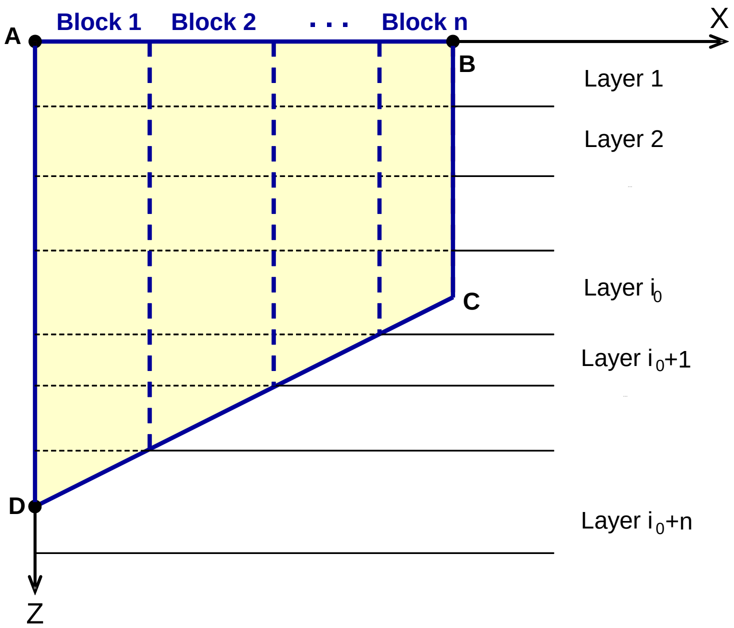


Figure C53: Discretization of the anchor in several blocks

The local equilibrium of block 'k' is governed by the system of forces that follows (figure below):

- $H_1^{(k)}$ and $V_1^{(k)}$ external reactions mobilized on the vertical left border;
- $H_2^{(k)}$ and $V_2^{(k)}$ external reactions mobilized on the vertical right border;
- $W^{(k)}$ submerged soil weight;
- $F_e^{(k)}$ resultant of the applied external overloads in block k;
- $R_c^{(k)}$ resistance due to mobilizable cohesion along segment $D^{(k)}C^{(k)}$;
- $R_f^{(k)}$ resistance due to the available friction along segment $D^{(k)}C^{(k)}$.

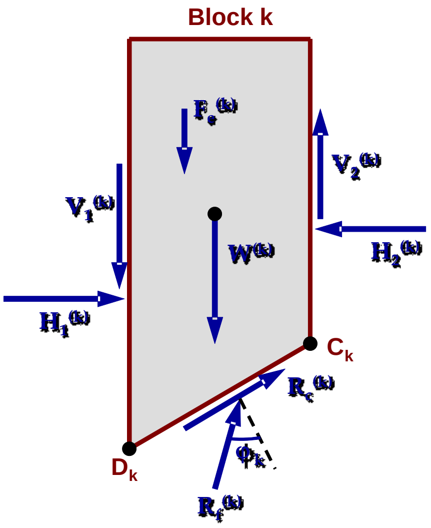


Figure C54: Local equilibrium of a block – forces assessment

In the figure above, φ_k designates the friction angle of the soil layer at the base of block « k ». For simplicity, we adopt the so-called Bishop hypothesis that consists in assuming that the reactions "interblocks" are horizontal, which means that, according to the notations of the Figure C54:

$$V_1^{(k)} = 0 \text{ and } V_2^{(k)} = 0$$

This condition is valid only along the « interblock » borders, an exception must hence be considered for the first (k = 1) and last blocks (k = n). Therefore, we end up with the general diagram of the figure below:

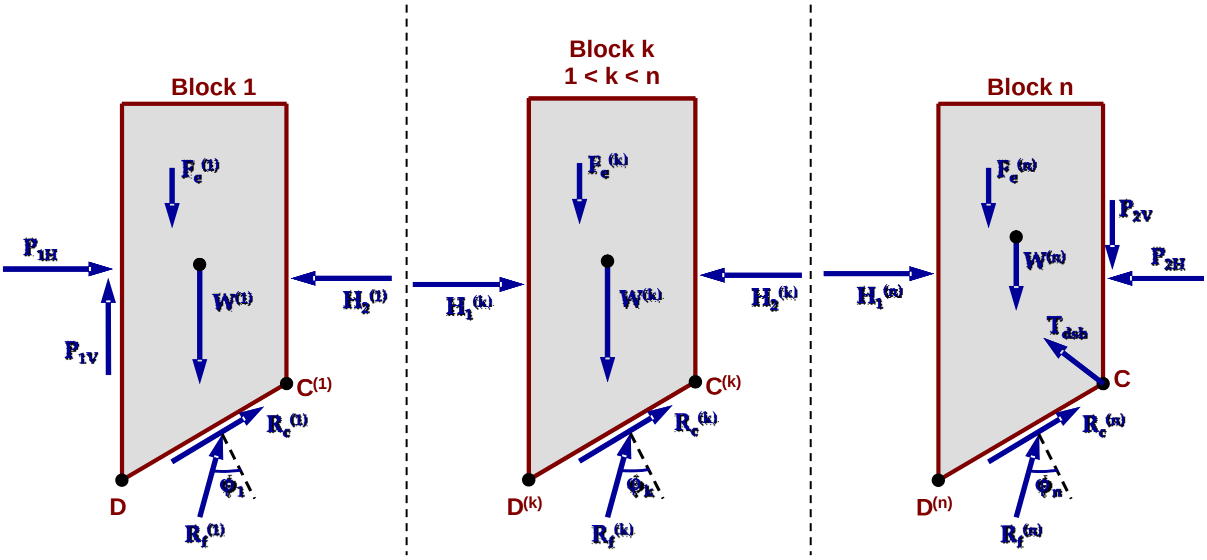


Figure C55: Local equilibrium of the blocks, taking into account the simplifying Bishop assumption

Please, note that because of successive slices, the anchor load T_{dsb} is considered only in the equilibrium of the last block (n). Actually, as the action line is unique, assigning this force to one block or the other has no impact.

C.4.6.2.4. *Resolution of overall equilibrium*

For a discretisation into « n » blocks, the local equilibriums lead to a system with 3n – 1 equations and 3n – 1 unknowns. More precisely, the equation system is obtained by projecting the local equilibrium of each block along Ox and Oz (i.e. 2 equations per block) and writing the action/reaction principle between two jointive blocks, translated by: $H_1^{(k)} = H_2^{(k-1)}$.

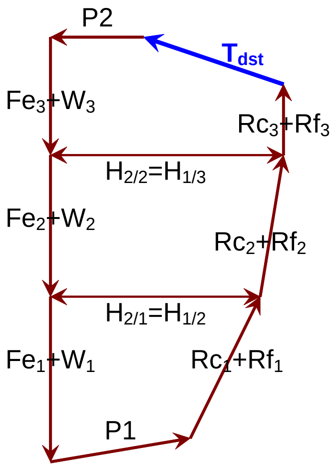


Figure C56: Example of forces for the case of 3 blocks

The resolution of this system allows to obtain the values of $H_1^{(k)}$, $H_2^{(k)}$, $R_f^{(k)}$ and $\mathbf{T_{dsb}}$.

C.4.6.2.5. *Check*

Obtaining the characteristic value of the destabilizing force T_{dsb} allows for checking the stability of a anchoring block at ULS:

$$T_{ref,d} = \gamma_E . T_{ref} \leq T_{dsb,d} = \frac{T_{dsb}}{\gamma_R}$$

In the case of the approach 2/2* according to the standard NF P 94 282: γ_R = 1,10 and γ_E = 1,35.

C.4.6.3. Case of several anchors

C.4.6.3.1. *General principle*

We consider the case of a retaining wall anchored with several levels of anchors, as shown in the figure below. The stability of the anchoring block is checked by studying successively the stability of blocks « associated » with each anchor (the way that was defined in the previous section for the case of a single anchor). Hence, for each anchor « j », we study the stability of the AB_jC_jDA block taking into account the anchor forces of all anchors located inside block « j ».

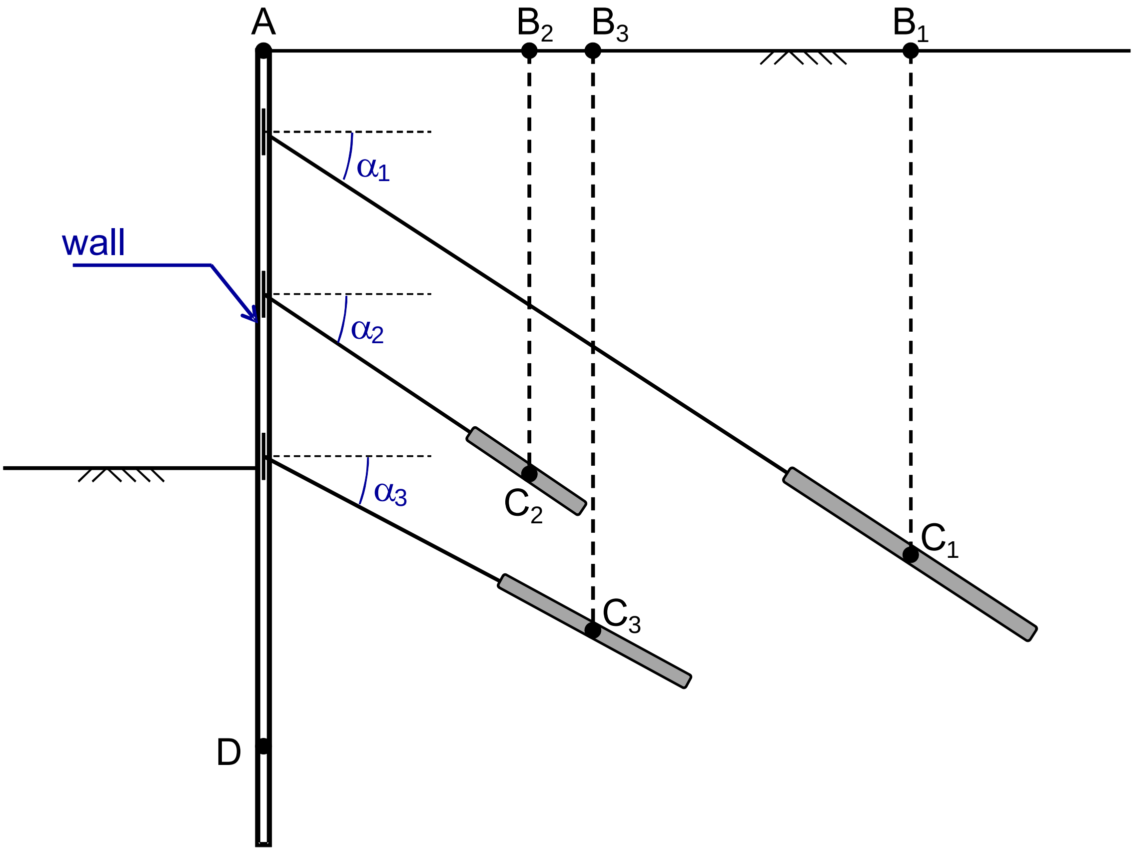


Figure C57: Generalization to the case of several layers of anchors

For example, for the case shown in the figure above, checking the stability of the anchoring block consists in examining three situations:

- Situation 1: we isolate anchoring block AB_1C_1DA associated with anchor « 1 ». The anchoring points C_2 and C_3 are located inside the block, therefore the three anchors are taken into account;

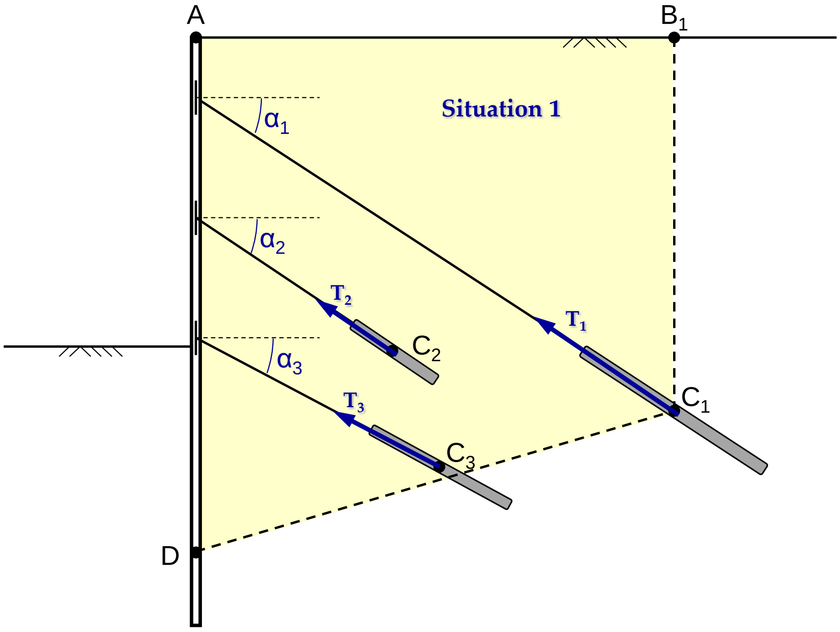


Figure C58: Sample application - Situation 01

- Situation 2: we isolate anchoring block AB_2C_2DA associated with anchor « 2 ». The anchoring points C_1 and C_3 are located outside the block, therefore only anchor « 2 » is taken into account;

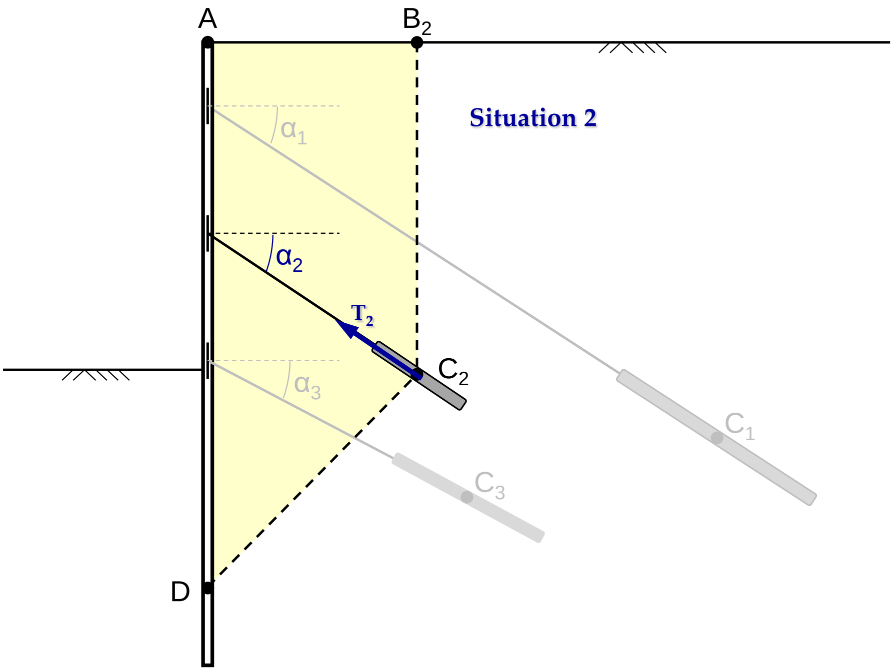


Figure C59: Application example - Situation 02

- Situation 3: we isolate anchoring block AB_3C_3DA associated with anchor « 3 ». The anchoring point C_2 is located inside the block, whereas C_3 is located outside. Anchors 2 and 3 are hence taken into account.

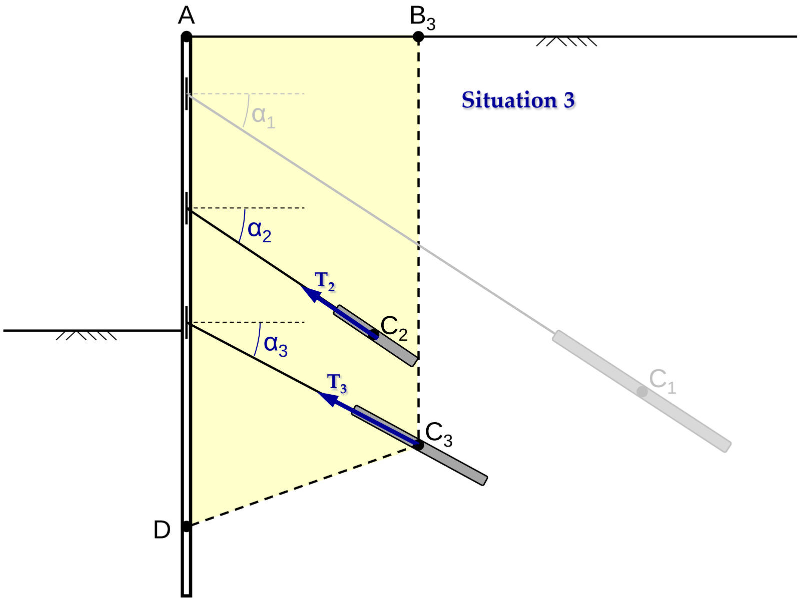


Figure C60: Application example - Situation 03

For a given situation, taking into account an anchor or not is decided depending on the relative position of its anchoring point with respect to the corresponding block boundaries. Attention is drawn to the case in which this anchoring point, although located geometrically outside the block, is close to the borders BC or CD, and in which case its influence cannot be neglected. Adapting the useful length of the anchors is necessary to allow them to be taken into account (refer to paragraph §C.4.6.3.4).

C.4.6.3.2. Efforts overview

For a given situation, we calculate the equivalent resultant T_{eq} of the forces T_i taken up by all anchors taken into account in this situation. We designate by α_{eq} the inclination of this resulting force with respect to horizontal. To study the stability of the anchoring block associated with the situation considered, we thus use an equilibrium system similar to that considered for a single anchor (figure below), with a « dummy » anchor inclined with α_{eq} with respect to horizontal and taking up a force equal to T_{eq}.

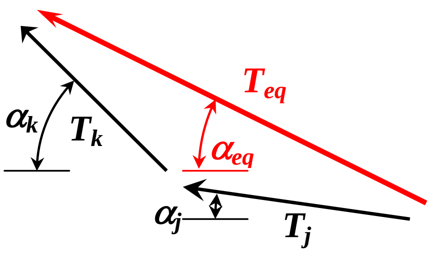


Figure C61: Result of a fictional anchor

C.4.6.3.3. Resolution

For each situation, the formulation is based on an approach similar to that followed for the case of a single draft. For a given situation, the resolution of the balance system provides the characteristic value of the destabilizing effort $T_{dsb,\,k}$ of the associated anchor. Its calculation value $T_{dsb,\,d.}$ taken equal to T_{dsb} / g_R is then compared to the design value of the anchoring effort of equivalent reference $T_{réf,d} = \gamma_E \times T_{eq}$.
The stability of the anchor is justified if for all situations, we have: $T_{réf,d} \leq T_{dsb,d}$.

C.4.6.3.4. Taking into account the bounding length

For a given anchor 'i', three configurations are distinguished (**Erreur ! Source du renvoi introuvable.**):

- Configuration 1: anchoring point C_i (= center of grouted part) is located inside the block, in this case the anchoring effort 'i' is fully taken into account;
- Configuration 2: head of the grouted part S_i is located outside of the block, in this case the anchor 'i' is not taken into account;
- Configuration 3: intermediate case, S_i inside, C_i outside of the massif. The anchoring effort 'i' is partially taken into account in proportion to the report S_iR_i/S_iC_i, where R_i means the point of intersection of the grouted part with the external border of the massif.

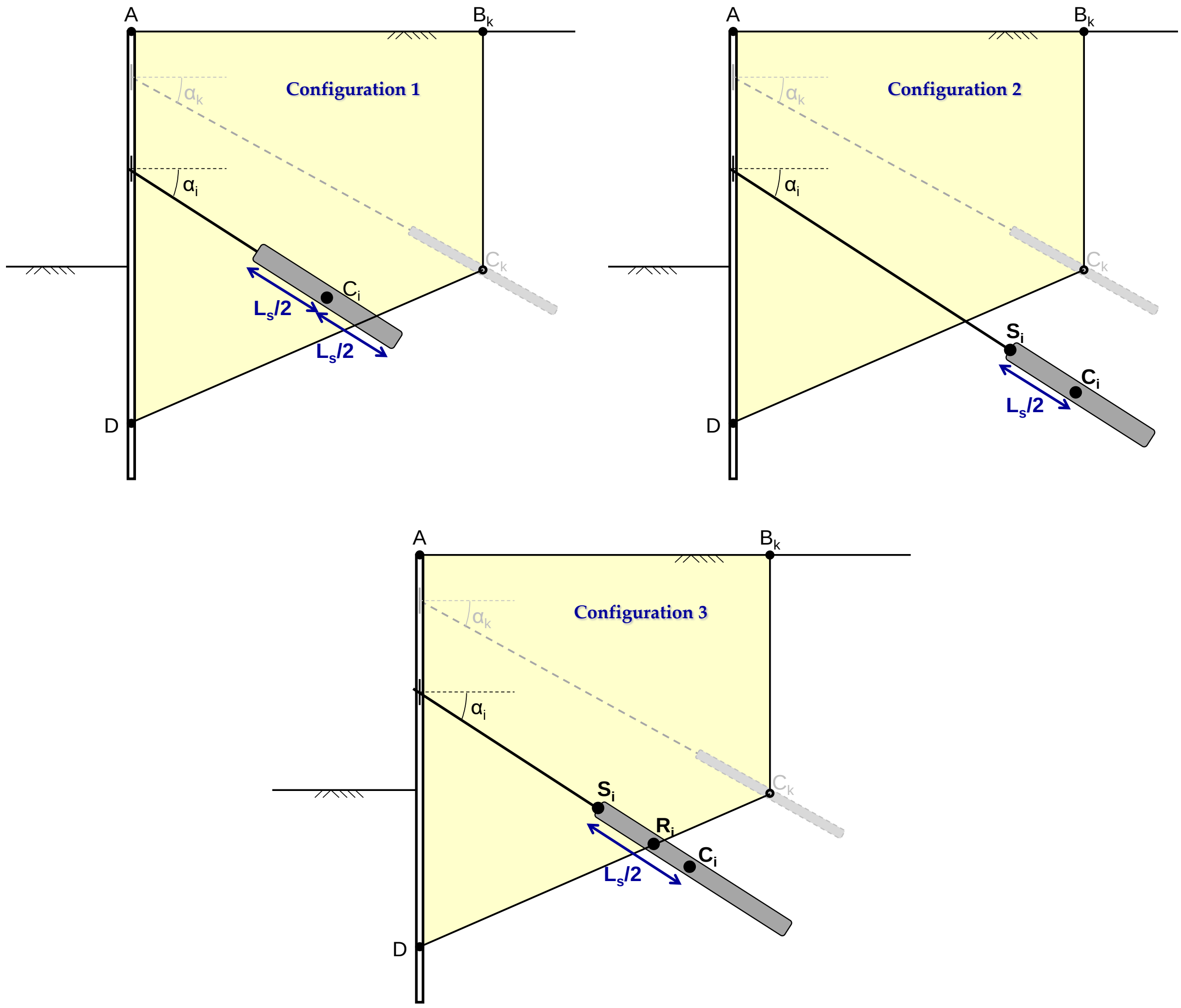


Figure C62: The 3 possible relative positions of the anchor and examinated soil block

With the notations above, the reference anchoring effort considered in a given situation is calculated according to the following formula:

$$\vec{T}_{réf} = \sum_i \min\left(\frac{S_iR_i}{S_iC_i};1\right).\vec{T}_i = \sum_i \frac{\min\left(2S_iR_i;L_s^i\right)}{L_s^i}.\vec{T}_i$$

C.4.6.4. Accounting for seismic conditions

The model described above can be easily adapted by introducing the inertia forces resulting from seismic action affecting the soil block.

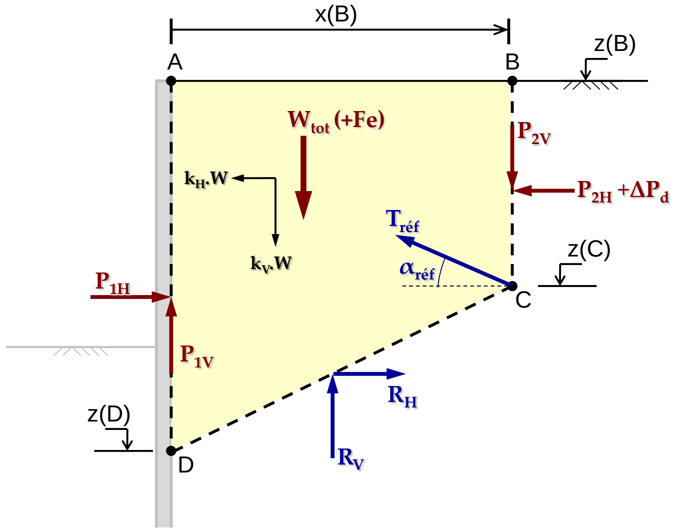


Figure C63: Kranz model – Accounting for seismic conditions

With the notations of the figure above, these forces of inertia modify the equilibrium as follows:

- A dynamic increment in the assessment of the forces of thrust upstream (P_{2H} and P_{2V}), which changes the balance of the last block;
- Introduction of the vertical and horizontal forces of inertia (proportional to the weight) in the equilibrium of each block.

Solving limit equilibrium highlights an exclusively unfavourable effect of the earthquake with a systematic reduction of the safety between the destabilizing anchoring force and that required for the balance of the wall.

C.4.6.5. Spiral arcs failure surface

The calculation model previously detailed can be improved using failure surfaces in spiral arc with concavity downwards, as shown on the figure below.

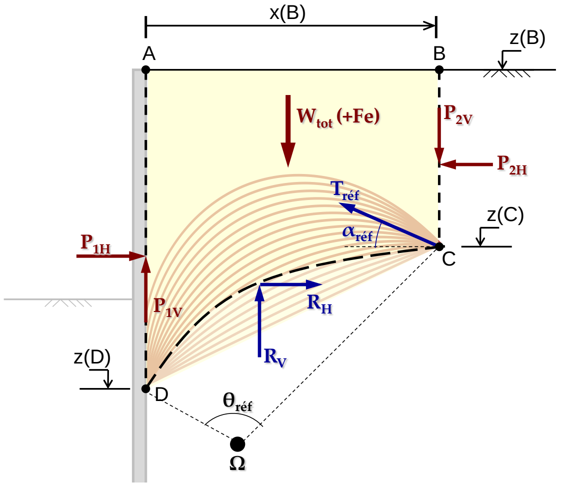


Figure C64: Stability of the anchoring block examined by a failure along an arc of spiral surface

For each failure surface, the limit equilibrium is reviewed by decomposition into vertical slices with the simplifying assumptions of Bishop. The curved shape of the failure surface imposes a fine discretization of the anchor: by default, K-Réa divides soil block into 100 blocks and analyzes fracture surfaces with a step value of $\Delta\theta réf = 5°$.

The exploratory process retains the surface (associated with an angle θref) leading to the lowest destabilizing effort. A value of θref = 0 corresponds to a planar surface.

C.4.6.6. Case of a double wall project

C.4.6.6.1. System of type « wall anchored on rear-wall »

Kranz model, such as detailed above for the case of a wall anchored by one or more anchors, can be adapted to the case of a system of a main wall anchored on a secondary wall as shown on the figure below.

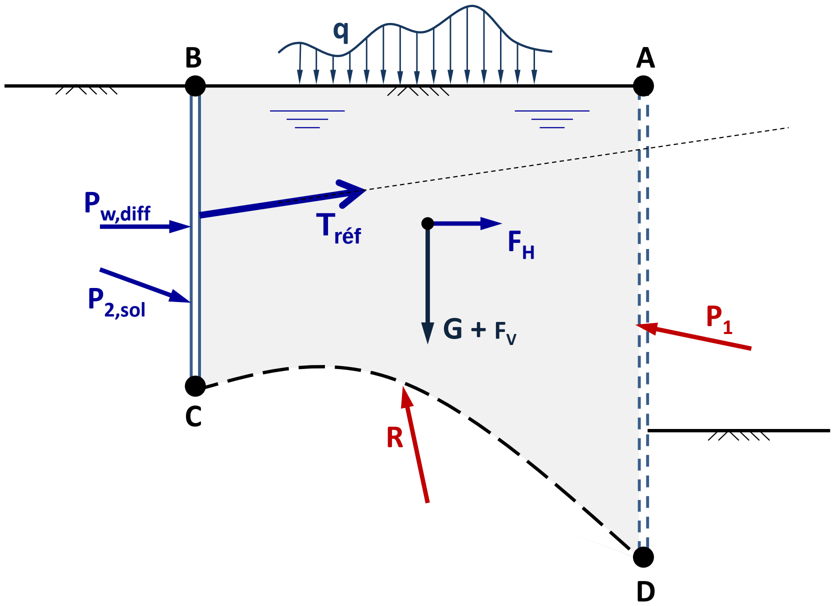


Figure C65: Limit equilibrium of the anchoring block for a double wall project

The case of a double wall requires the following adjustments:

- Geometry of the block: the upper border of the massif is in the back face of the rear-wall. Point C is confused with the foot of the rear-wall if it is short and more generally with the zero-shearing point of the rear-wall;
- The reference anchoring force (T_{ref}) corresponds to the vector sum of all the anchoring efforts mobilized in the node-to-node anchors (single or links) and those whose grout is located (at least partly) inside of the anchor ABCD;
- The upstream active pressure effort (P2) represents the result of external forces applied to the rear-wall block. This includes on the one hand the pressure from the ground to the back of the soil block and the differential pressure of water between the two sides of the screen.

$$\vec{P}_2 = \vec{P}_{2,soil} + \vec{P}_{w,diff}$$

These adjustments are applied automatically by the calculations of K-Réa's engine.

C.4.6.6.2. Case of a double anchor

K-Réa also allows to process double-walls with "dual anchors" as presented in the figure below: a (1) main wall anchored on a secondary wall (2), anchored himself by bonded anchoring ties. Points "C" and "D" are points of zero shearing force, respectively for walls 1 and 2.

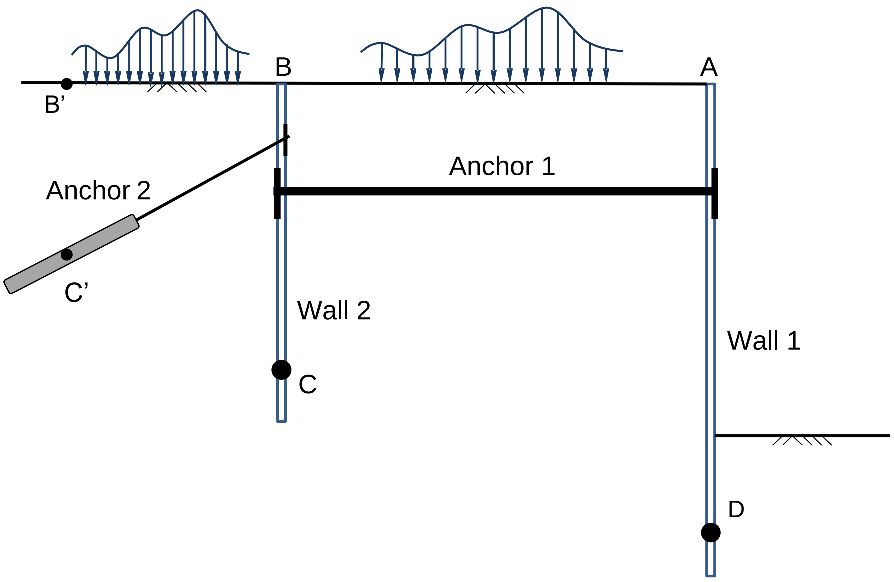


Figure C66: Limit balance of the anchor for a double screen with double anchoring

In this case, K-Réa examines (at least) two configurations that correspond to each anchoring block:

- Anchoring block ABCD associated with the main screen, whose destabilizing force is compared (weighted) to the force taken up by the linking anchor (anchor 1). For this configuration, the effort mobilized by anchor '2' is deducted from the upstream active pressure force (P2) applied to the back of the soil block;

$$\vec{P}_2 = \vec{P}_{2,soil} - \vec{T}_2$$

- Anchoring block BCC'B' associated with the secondary wall, whose destabilizing force is compared (weighted) to the effort taken up by the active anchor (2).

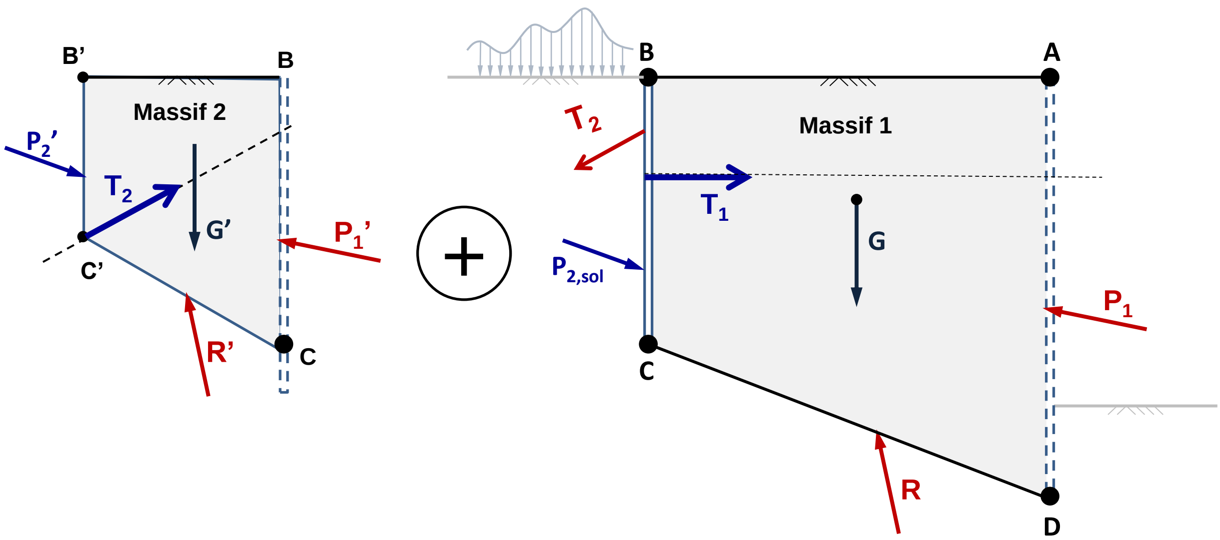


Figure C67: Soil block anchor considered in the case of a double-screen with dual anchor system

C.5. Theoretical bases used for data input wizards

This section describes the theoretical bases used for the different wizards proposed to the user. Handling of these wizards is described in part B of the manual (user manual).

BEWARE: wizards are only a help for the user, they are not a compulsory step in a project. the user is responsible for their use.

C.5.1. Wizards related to soil characteristics

C.5.1.1. Coefficient k_0

The **k0 Jaky** wizard calculates the value of k_0 using the following formula:

$$k_0 = (1+\sin\beta)(1-\sin\varphi)\sqrt{OCR}$$

In which:

β : slope inclination [°];
φ : friction angle [°];
OCR : overconsolidation ratio.

C.5.1.2. Coefficients k_d and k_r

The unloading and reloading ratios enable to account for the variations of horizontal stresses applied by the soil on the wall due to the loading and unloading of this soil, by modifying the zero displacement initial pressure and the values of plasticity thresholds.

- In the general case, for a normally consolidated soil, drained behaviour, we can take $k_d = k_r \approx k_0$.
- In the case of an overconsolidated soil, whose behaviour can be compared to that of an elastic material, can be $k_d = k_r = \dfrac{\nu_{ur}}{1-\nu_{ur}} \prec k_0$
- In the case of a normally consolidated soil with undrained behaviour, then $k_d = k_r \approx 1 \geq k_0$ (ν_{ur} ~ 0,5).

The article referenced in [6] offers a formula for the coefficient k_d value of the OCR.

Attention is drawn to the important influence of the values assigned to these parameters on the design (including in the case of very hyperstatic structures).

C.5.1.3. Coefficients $k_{a\gamma}$ and $k_{p\gamma}$

3 wizards are available in K-Réa for the determination of the coefficients $k_{a\gamma}$ and $k_{p\gamma}$.

C.5.1.3.1. Wizards « Tables of active and passive earth pressures of Kerisel and Absi »

This wizard is the accurate reproduction of the tables defined by Kerisel and Absi, published by Presses de l'Ecole Nationale des Ponts et Chaussées, under the title "Tables de poussée et butée des terres de Kerisel et Absi" [1].

C.5.1.3.2. Wizard « Active and passive earth pressures according to the Coulomb formula »

This wizard displays the result of the calculation of Coulomb formulas (from Techniques de l'ingénieur; Construction; C242; "Ouvrages de soutènement, poussée et butée" written by F. Schlosser [2]):

$$k_{a\gamma,\delta} = \frac{\cos^2(\lambda - \phi)}{\cos(\lambda + \delta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta).\sin(\phi - \beta)}{\cos(\lambda + \delta).\cos(\lambda - \beta)}}\right]^2}$$

$$k_{p\gamma,\delta} = \frac{\cos^2(\lambda + \phi)}{\cos(\lambda + \delta)\left[1 - \sqrt{\dfrac{\sin(\phi - \delta).\sin(\phi + \beta)}{\cos(\lambda + \delta).\cos(\lambda - \beta)}}\right]^2}$$

In which:

- φ friction angle [°];
- β angle between the soil surface and the horizontal axis (°);
- λ angle between the wall and the vertical axis (default value is 0) (°);
- δ/φ report of the obliquity of the constraints on the angle of friction.

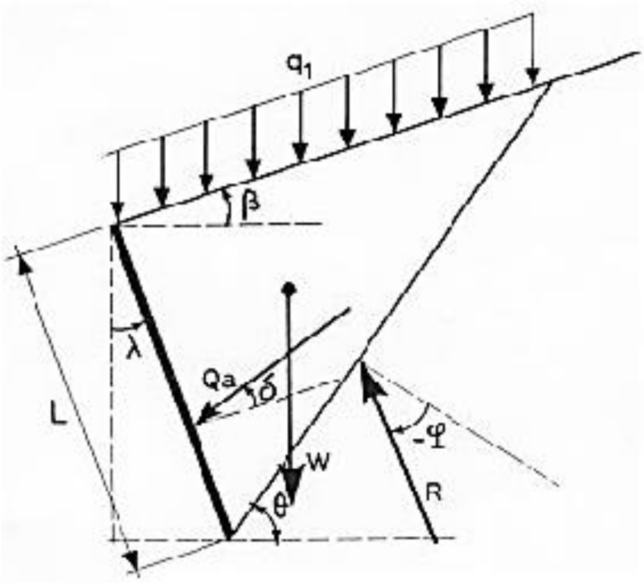


Figure C68: Data for the Coulomb formula

Coefficients $k_{a\gamma,\delta}$ and $k_{p\gamma,\delta}$ correspond to the values tilted by δ_a and δ_p. The wizard then provides the $k_{a\gamma,\delta}$ and k_p values of the horizontal active and passive ratios.

C.5.1.3.3. Wizard « Passive and active earth pressures according to the Rankine formula »

This wizard is available under 2 different forms:

- The simplified Rankine wizard corresponding to the "Rankine" button in the main soil properties dialogue box: this wizard calculates the values of $k_{a\gamma}$ and $k_{p\gamma}$ by Rankine's formula <u>with a free horizontal surface</u> and transfers automatically the values to the corresponding box, such as:

$$k_{a\gamma} = \tan^2\left(\frac{\pi}{4} - \frac{\varphi}{2}\right) \text{ and } k_{p\gamma} = \tan^2\left(\frac{\pi}{4} + \frac{\varphi}{2}\right).$$

Where φ is the friction angle (°).

- The Rankine wizard allows to consider the slope inclination. It may be reached by the "kaγ/kpγ" button in the soil properties dialogue, then the « Rankine » choice: this wizard displays the result of Rankine's formulas for a retaining wall with a inclined embankment extracted from Techniques de l'ingénieur; Construction; C242; "Ouvrages de soutènement, poussée et butée" written by F. Schlosser [2] and reminded below:

$$k_{a\gamma} = \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\varphi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\varphi}}\right]\cos\beta$$

$$k_{p\gamma} = \left[\frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\varphi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\varphi}}\right]\cos\beta$$

In which:
- φ : friction angle [°];
- β : inclination of the free surface to the horizontal [°].

C.5.1.4. Coefficients k_{ac} and k_{pc}

Following formulas provide active/passive earth pressures coefficients due to the cohesion:

- Active pressure $$k_{ac} = \frac{1}{\tan\varphi}\left[\frac{\cos\delta_a - \sin\varphi\cos\alpha}{1+\sin\varphi} e^{-(\alpha-\delta_a)\tan\varphi}\cos\delta_a - 1\right]$$
- Passive pressure $$k_{pc} = \frac{1}{\tan\varphi}\left[\frac{\cos\delta_p + \sin\varphi\cos\alpha}{1-\sin\varphi} e^{(\alpha+\delta_p)\tan\varphi}\cos\delta_p - 1\right]$$

Where $\sin\alpha = \dfrac{\sin\delta}{\sin\varphi}$.

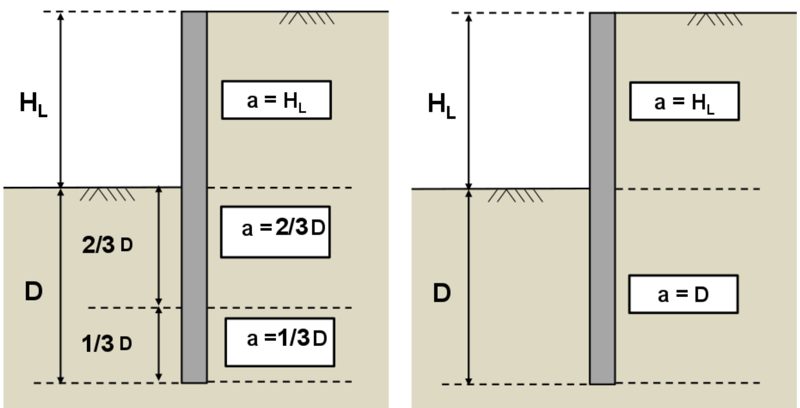
C.5.1.5. Coefficient k_h

C.5.1.5.1. Balay Formula

Balay formula [4] is based on the following formula:

$$k_h = \frac{E_m}{\left(\frac{\alpha.a}{2}\right) + 0{,}133.(9.a)^\alpha}$$

In which:
- E_m: pressuremeter modulus (kN/m² or KsF);
- α: rheological parameter (see also section C.3.2.3);
- a: dimensional parameter (m).



Cases where D < H_L   Case where D ≥ H_L

Figure C69: Parameter of the formula of Balay

C.5.1.5.2. Schmitt formula

Schmitt's calculation method [5] relies on the following formula:

$$k_h = \frac{2{,}0.\left(\frac{E_m}{\alpha}\right)^{\frac{4}{3}}}{(EI)^{\frac{1}{3}}}$$

In which:

- E_m : pressuremeter modulus (kN/m² or KsF);
- α : rheological parameter (see also chapter B.3.2.4).

C.5.1.5.3. Chadeisson abacus

Chadeisson abacus [6] provides the value of k_h according to the soil friction angle and cohesion.

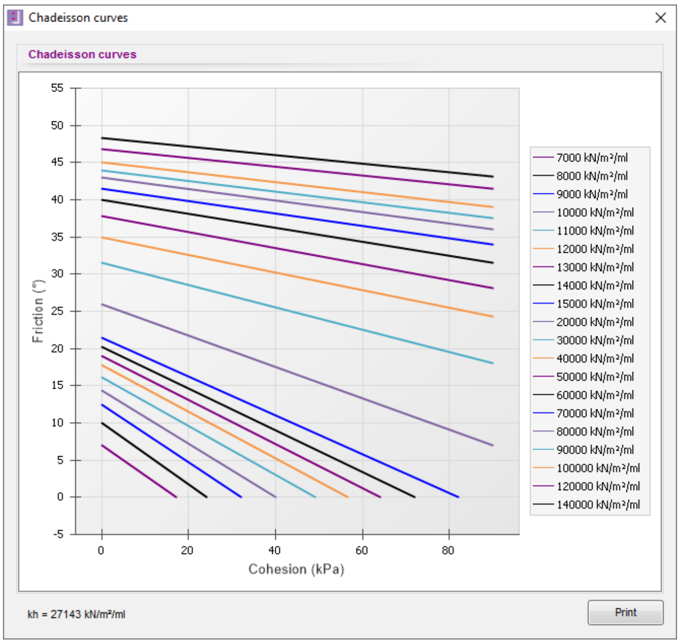


Figure C70: Chadeisson abacus

C.5.2. Wall characteristics wizards

C.5.2.1. Cylindrical rigidity wizard

Use this wizard to evaluate the cylindrical stiffness of a circular continuous wall and its possible variation with depth. It is available from the definition of the wall if the option "cylindrical enclosure" is checked. Continuous cylindrical stiffness of a circular wall is given by the following formula:

$$R_c = \frac{E.t}{R^2}$$

In which:

- E deformation modulus of the wall material [kN/m²];
- t thickness [m];
- R average radius [m].

In general, a circular continuous wall consists of a suite of vertical elements (i.e. the rectangular panels of diaphragm wall) made according to a maximum tolerance of verticality, which leaves the possibility of a relative deviation between two consecutive vertical elements. The "residual" thickness of contact between these elements can decrease with depth, which implies a reduction or cancellation of the cylindrical rigidity as shown below.

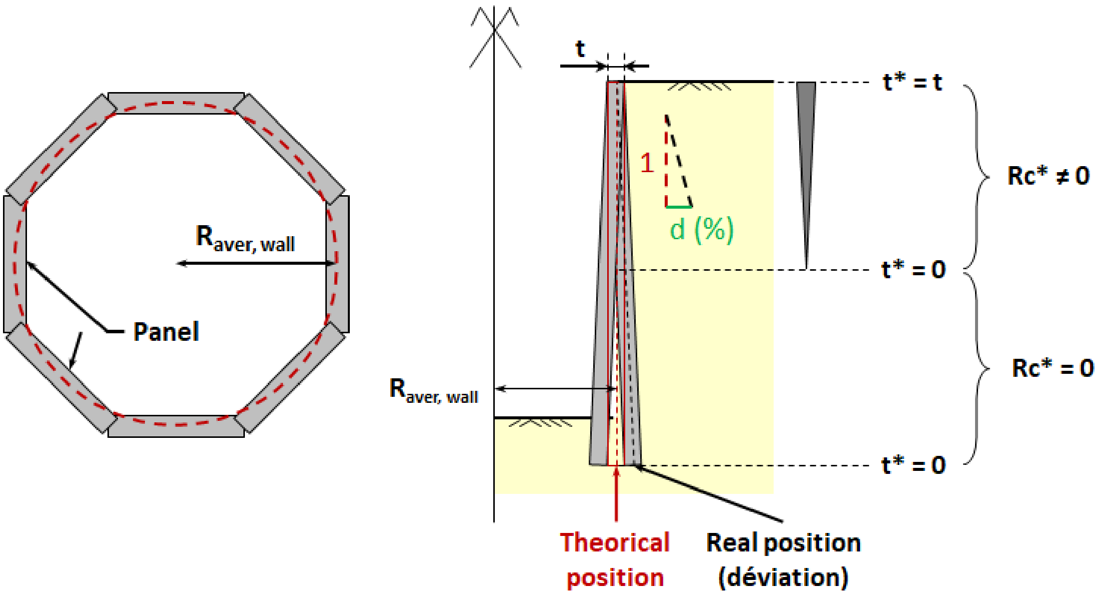


Figure C71: Diagram of deviation for a circular diaphragm wall

The screen is divided into several sections each characterized by a cylindrical "effective" rigidity equal to:

$$R_c^*(z) = \begin{cases} \dfrac{t - 2.d.z}{t} R_c & \text{if } z \leq \dfrac{t}{2.d} \\ 0 & \text{otherwise} \end{cases}$$

C.5.2.2. Composite wall wizard

This wizard offers the calculation of the product of inertia EI per linear meter (or Ft) for a composite wall made of piles and panels between the piles:

$$\mathrm{EI}_{\text{composite wall}} = \mathrm{EI}_{\text{pile}} + \mathrm{EI}_{\text{wall}}$$

This approach implicitly assumes that the panel and the piles have the same neutral plane. The wizard considers different types of piles. The calculation of the equivalent EI product of piles per linear meter (or Ft) is detailed in the following subchapters. The product of inertia EI of the panel by ml (or Ft) is calculated as follows:

$$EI_{wall} = \frac{E \times h^3}{12}$$

Where:

- E: Young modulus of the panel (kN/m², KsF);
- h: wall thickness (m, Ft).

C.5.2.2.1. Circulair piles

The product of inertia EI of the piles by ml (or Ft) is calculated as follows:

$$\mathrm{EI}_{\mathrm{pile}} = \frac{\mathrm{E}\times\mathrm{I}}{\mathrm{e_h}} \text{ where } \mathrm{I} = \frac{\pi\times D^4}{64}$$

In which:

- E: Young modulus of piles (kN/m², KsF);
- e_h: distance between main piles (m, Ft);
- D: diameter of each pile (in m, Ft).

C.5.2.2.2. Steel profiles

The product of inertia EI of the pile (profile) per ml (or Ft) is calculated as follows:

$$\mathrm{EI}_{\mathrm{pile}} = \frac{\mathrm{E_{steel}I_{profile}}}{\mathrm{e_h}}$$

In which:

- $I_{profile}$: inertia of selected profile (m^4);
- E_{steel}: Young modulus of steel (kN/m², KsF);
- e_h: distance between the axes of the profiles (m, Ft).

C.5.2.2.3. Mixed piles

The selection of the Wizard "mixed piles" allows taking into account the calculation of the EI product in both the profile and the full section around the profile. The calculation is led by with the principle of superposition:

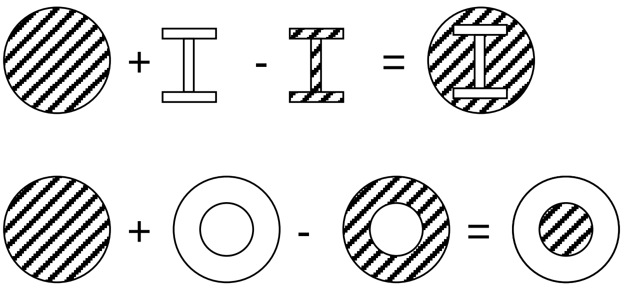


Figure C72: Configurations of mixed piles

The product of inertia EI of mixed piles by ml (or Ft) is calculated as follows:

$$\mathrm{EI}_{\mathrm{pile}} = \frac{1}{e_h}\left(\mathrm{E_{concrete}}\left(\frac{\pi \mathrm{D}^4}{64} - \mathrm{I_{profil}}\right) + \mathrm{E_{steel}I_{profil}}\right)$$

In which:

- $E_{concrete}$: Young modulus of pile (kN/m², KsF);
- e_h: distance between the axis of the piles (m, Ft);
- D: diameter of each pile (in m, Ft);
- $I_{profile}$: inertia of selected profile (m^4);
- E_{steel}: Young modulus of steel (kN/m², KsF).

C.5.2.3. Reduced active pressure wizard

Use this wizard to define R and C coefficients to be used in the case of a discontinuous wall. It is possible to set several values of R and C at different depths. To do this, simply set the higher application level (z_t) and lower level (z_b) between which you want to apply them.

Two methods are available:

- **« Classic »** Mode
- **« Standard NF P 94-282 - Annex B »** Mode

The **classic** mode requires the definition of:

- e spacing of the main elements [m];
- L_a length of the calculation of limit active pressure [m];
- L_b length of calculation of limit passive pressure [m].

From these data, we get:

- R: reducing coefficient of active pressure $R = L_a / e$
- C: growth factor applied to passive pressure $C = L_b / L_a$

The « **Standard NF P 94-282 - Annex B** » mode needs the definition of:

- Target soil type, selected between:
    - Purely cohesive soil (Sol_Type_1)
    - Cohesive-frictional soil (Sol_Type_2)
- Geometry of the main element:
    - If the key element is rectangular (EP_Type_1):
        - L1: length (m)
        - L2: width (m)
    - If the key element is circular (EP_Type_2):
        - D: diameter (m)
- e: space between main elements (m)

The wizard automatically updates the help figure depending on the choices and values defined by the user. In accordance with the standard NF P 94-282 - Appendix B, the values of the La, Lb, R and C are calculated as shown below depending on the case:

- **Case 1: Soil_Type_1 + EP_Type_1**

  La = Lb = min (L1 + L2 ; e)
  R = La / e
  C = 1

- **Case 2: Soil_Type_1 + EP_Type_2**

  La = Lb = min (2 x D ; e)
  R = La / e
  C = 1

- **Case 3: Soil_Type_2 + EP_Type_1**

  La = Lb = min (L1 + 2 x L2 ; e)
  R = La / e
  C = 1

- **Case 4: Soil_Type_2 + EP_Type_2**

  La = Lb = min (3 x D ; e)
  R = La / e
  C = 1

C.5.3. Anchor characteristics wizard

C.5.3.1. Anchor wizard

Use this wizard to define the stiffness and the preload per unit length for each grouted anchor.

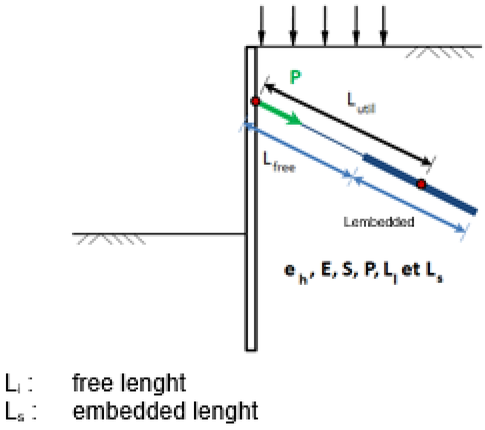


Figure C73: Schematic of grouted anchor level

An anchor layer is assimilated to an equivalent spring with:

- An axial stiffness per unit length:

$$K_{axial} = \frac{ES}{L_u e_h}$$

- A pre-load (axial) per unit length:

$$P_{axial} = \frac{P_t}{e_h}$$

In which:

- e_h : anchor horizontal spacing (m);
- E : Young modulus of anchor (kN/m²);
- S : anchor section (m²);
- P_t : preloading applied on an anchor (kN);
- L_u : useful length of anchor (m).

Conventionally the effective length of an anchor layer is defined as follows:

$$L_u = L_{free} + \frac{L_{embedded}}{2}$$

The elasto-plastic behaviour allows you define a maximum value ($F_{adm,tr}$) for the traction force mobilized in a anchor $F_{adm,tr}$. This value is linked to the maximum allowable stress of the anchor material as follows:

$$F_{adm,tr} = \frac{\sigma_{adm} S}{e_h}$$

Note that for a shooting angle of α with regards to the horizontal, the projection of the axial stiffness leads to horizontal stiffness equivalent equal to:

$$K_{horiz} = (\cos\alpha)^2 . K_{axial}$$

This is the considered stifness in the (horizontal) equilibrium of the screen.

C.5.3.2. Strut wizard

C.5.3.2.1. General case

Use this wizard to calculate the stiffness and the preloading per unit length of a strut level.

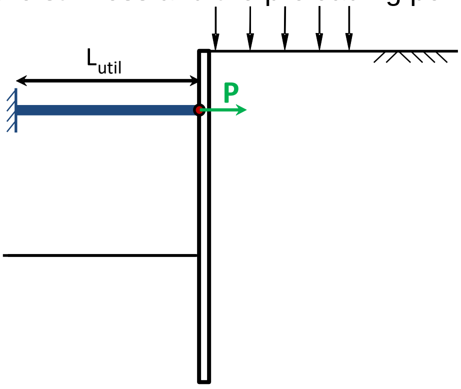


Figure C74: Schematics of level of struts

A level of struts is considered to be an equivalent spring characterized by:

- Stiffness (axial) per unit length:

$$K_b = \frac{ES}{L_u e_h}$$

- A pre-load (axial) per unit length:

$$P = \frac{P_b}{e_h}$$

In which:

- E : Young modulus of strut [kN/m²];
- S : strut section [m²];
- e_h : horizontal spacing between struts [m];
- P_b : pre stress applied on a strut [kN];
- L_u : usefull length [m].

C.5.3.2.2. Option « diagonal strut »

The option « Diagonal strut » let you consider a strut layer set on a horizontal slab that rests on the soil. Equivalent axial stiffness of the strut involves the axial stiffness of the strut and the stiffness related to the soil settlement.

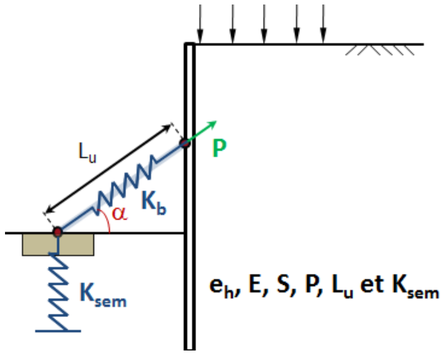


Figure C75: Schematics of a diagonal struts layer

Axial stiffness of the strut is: $$K_b = \frac{ES}{L_u e_h}$$

The vertical stiffness of the slab is assessed using the pressuremeter method for a rectangular slab:

$$K_{slab} = E_M \frac{9L}{\lambda_c \alpha + 2\frac{B_0}{B}\left(\lambda_d \frac{B}{B_0}\right)^{\alpha}}$$

In which:

- B_0: reference width = 0.60 m
- B , L: width and length where B ≤ L [m];
- E_M: pressuremeter modulus [kN/m²];
- α: rheological coefficient [-];
- λ_c , λ_d: form factor related to coefficients and deviatoric settlements.

The equivalent axial stiffness per length of study is:

$$K_{axe,eq} = \frac{1}{e_h} \frac{K_b K_{sem} (\sin\alpha)^2}{K_b + K_{sem} (\sin\alpha)^2}$$

On which:

- $K_{,b}$: axial stiffness of the strut [kN/m];
- K_{slab}: vertical stiffness of the footing [kN/m];
- α: inclination of the strut in relation to the horizontal [°];
- e_h: spacing between struts [m].

Noted that for a strut tilted of α to the horizontal, the horizontal stiffness taken into account in the balance of the screen is:

$$K_{horiz} = (\cos\alpha)^2 . K_{axe,\,eq}$$

C.5.3.3. Circular waling wizard

Use this wizard to define the apparent rigidity of a circular waling supporting a circular wall.

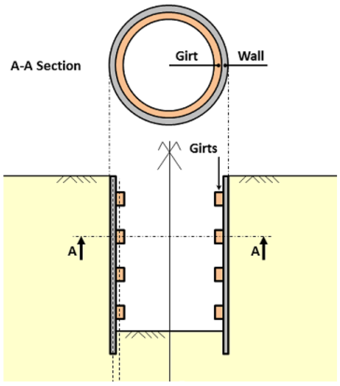


Figure C76: Schematics of a circular wall with a circular wailing

The circular waling apparent stiffness is calculated with the following expression:

$$R_c = \frac{1}{R^2} ES$$

In which

- ES: axial stiffness of the waling (Young modulus x Section);
- R: average radius of the waling [m].

C.5.4. Wizard for the determination of the limit pressure diagram (coming soon)

This wizard allows for the determination of limit active pressure (Pa) and limite passive pressure (Pb) diagrams on each side of a vertical wall. The general case is that of a layered soil with embankment or berm geometry, subject to external loads and seismic loads.

The method of calculation used is based on the general theory of the yield design method developed and formalized by J. Salençon. It is an external approach looking at the ground with a range of logarithmic spiral failure surfaces (the allure of the spiral depends on the angle of friction of the ground layer). In the case of a multilayer ground, surfaces become logarithmic multispirale type with a single pole.

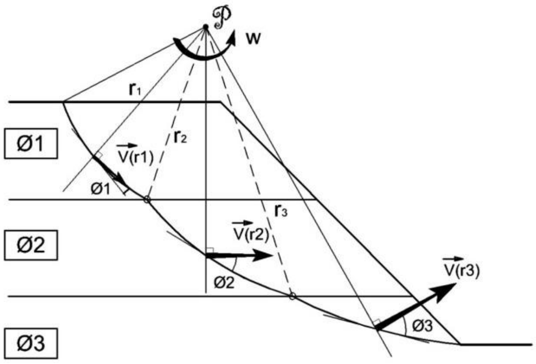


Figure C77: Multispirale failure in a multilayer soil

Principle of the method of research of the diagrams of pressure limits:

- The active pressure (Pa) diagram is determined at any point of the height of the wall, as the maximum stabilizing pressure in the state of limit equilibrium;
- The passive pressure (Pb) diagram is determined at any point of the height of the wall as the minimum destabilizing pressure in the state of limit equilibrium. limite;

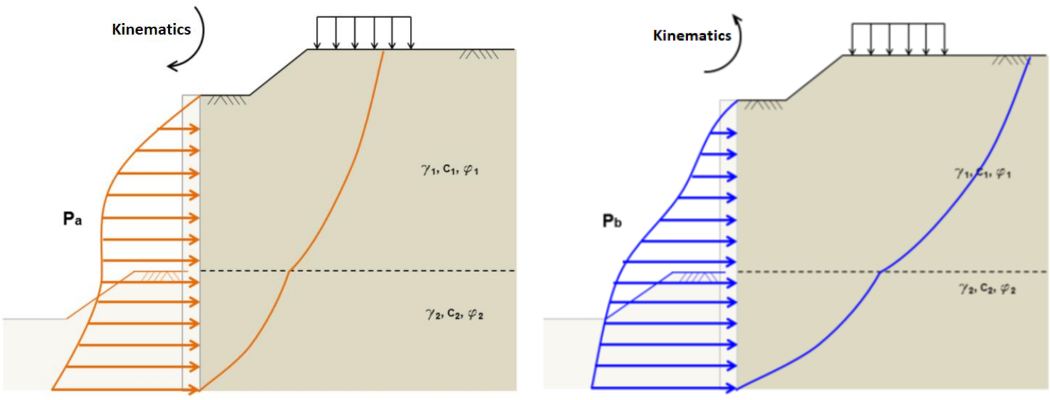


Figure C78: Kinematics of the active mechanism (left) and the passive mechanism (right)

The input data for the calculation are:

- Maximum interval of exit points in research steps n ° 1 and n ° 2;
- Exploration interval of the angle at the pole of the spirals;
- Minimum value of the angle at the pole of the spirals;
- Minimum increment of the angle at the pole of the spirals;
- Maximum number of divisions of the angle at the pole of the spirals.

The search for the most unfavourable failure surface is addressed as follows:

- Step 1: initial exploration of surfaces (step 1) by varying the entry point, exit point and angle at the pole.
- Step 2: more detailed exploration in refining the exit points intervals, for a same entry point, by varying the angle to the pole.

The diagrams of limit pressures obtained depend on the situation under review; see a few examples below:

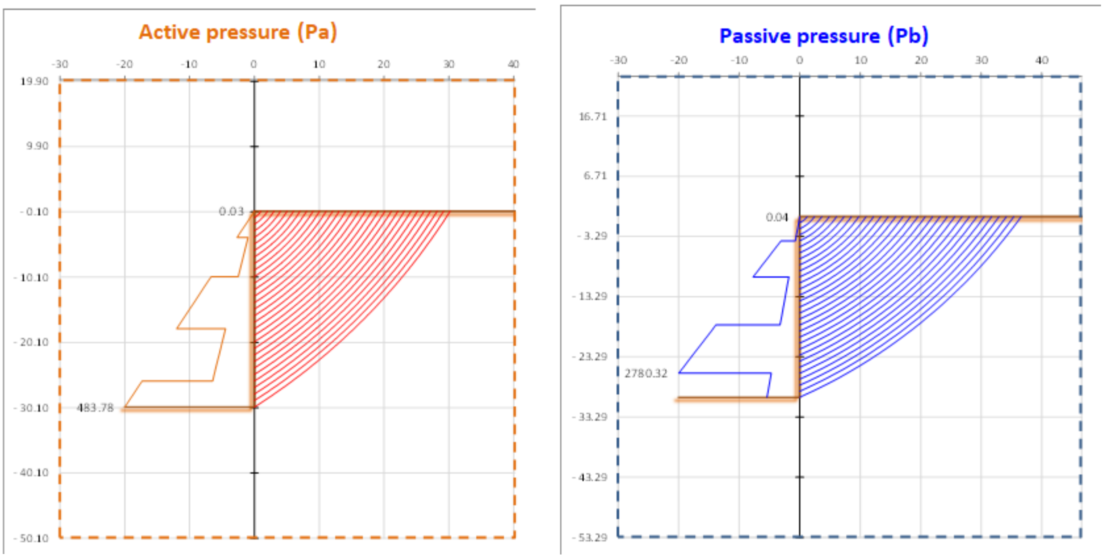


Figure C79: Diagrams of limit active pressure (left) and limit passive pressure (right) obtained for a multi-layer soil (5 layers)

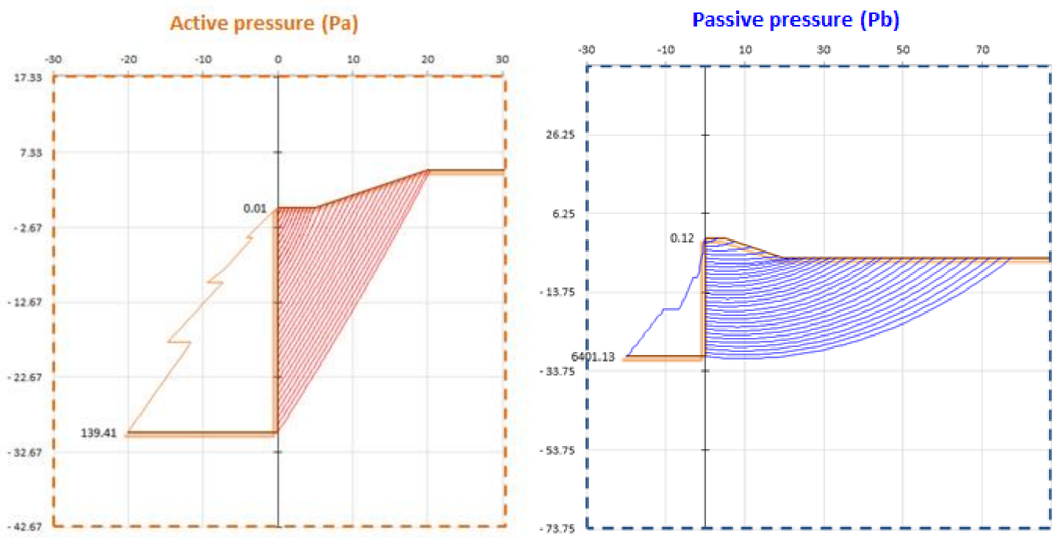


Figure C80: Diagrams of limit active pressure with embankment (left) and limit passive pressure with berm (right) obtained for a multi-layer soil

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