DEEPXCAV

A NONLINEAR FINITE ELEMENT PROGRAM
FOR THE DESIGN OF
FLEXIBLE EARTH RETAINING WALLS

NON LINEAR ANALYSIS REFERENCE MANUAL
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FOREWORD

This manual includes several hints to the PARATIE User and summarizes the assumptions, the formulas and the theories on which the program is founded.

PARATIE Users should keep in mind what follows:

- A PARATIE Users should have a good skill in Soil Mechanics;
- Using PARATIE without a proper skill could result in an unrealistic and unreliable design;
- All the suggested criteria for the estimate of the soil parameters as well as the advised design methods included in this manual should be very carefully understood and accepted by the User. PARATIE authors will have no responsibility on any consequence and loss of money, health and goodwill due to the use of the program.
1. PROGRAM HISTORY

Current is release 6.2 of PARATIE, a program whose development started in 1985 based on a work of professor Roberto Nova published in 1987 (see Becci & Nova 1987).

The central part of the program, represented by the soil model by professor Nova, is still more or less the same as in the first version, because such model proved to be effective when used in the design of very many actual retaining walls during the past 20 years.

Since the beginning, the program was designed to allow a quite wide range of modelling capabilities, as compared with some competitor programs. Such program flexibility is basically due to the fact that PARATIE is a finite element program in which the soil element just represents one of the many finite element options included.

Since 1996, a Windows based interface is available.

From 2001 version, a brand new part is included in the PARATIE interface, aiming at a support to the User during the estimation of the soil properties to be prescribed for the analysis. Such part is somehow distinguished from the main development of PARATIE and should be revised just an optional yet not mandatory tool when using the program.

PARATIE is used by hundreds engineers in Italy and worldwide, in many challenging projects, such as those witnessed by the following pictures.

Berlin wall at the Collecervo Tunnel gate near Imperia (Italy) – Courtesy of Ferrovial Agroman S.A.

R.c. anchored bulkheads in Milano – Courtesy of Quadrio Curzio S.p.A.

Sheetpile cofferdams for Cable stayed bridge construction across Po River (Italy) – Courtesy of Grandi Lavori Fincosit.

In PARATIE 6.2, special effort was made to include seismic design options and LRFD design approaches according to most recent guidelines such as those implemented in Eurocodes. Such options, however, are covered by a separate manual under preparation.
2. PARATIE CONCEPTUAL MODEL

PARATIE is a non-linear finite element code for the analysis of the mechanical behaviour of flexible earth retaining structures during all the intermediate steps of an open excavation.

The actual problem is reduced to a plane problem, in which a unit wide slice of the wall is analysed, as outlined in the figure below. Therefore PARATIE is not suitable to model excavation geometries in which three-dimensional effects may play an important role.

![Figure 2-1](image-url)

In the modelling of the soil-wall interaction, the very simple and popular Winkler approach is adopted. The retaining wall is modelled by means of beam elements with transversal bending stiffness $EJ$; the soil is modelled by means of a double array of independent elastoplastic springs; at each wall grid point, two opposite springs converge at most.

![Figure 2-2](image-url)
According to the Winkler model, it is assumed that the behaviour of every soil spring is totally uncoupled from the behaviour of adjacent elements: the actual interaction among different soil regions is totally left to the retaining wall.

The real progress of an excavation process is reproduced in all the intermediate steps, by means of a STATIC INCREMENTAL analysis. Due to the elastoplastic behaviour of the soil elements, every step in general depends on the solution at the previous steps. Corresponding with every new step, the solution is obtained by means of a Newton-Raphson iterative scheme (see Bathe (1996)).

PARATIE only computes the lateral behaviour of the retaining wall: at each nodal point only the lateral displacement and out-of-plane rotation (about the X axis) are activated as independent degrees of freedom. It is very important to remark the following main consequences of such an approach:

1. PARATIE does not compute vertical settlements in the wall and in the adjacent soil;\(^1\)
2. PARATIE does not compute the overall stability of the soil+wall+supports assembly

Moreover, within PARATIE framework, the vertical stress distribution in the soil not influenced by the lateral deformations in the soil itself. At each depth, the vertical stress is an independent variable computed by PARATIE by means of the usual assumption of geostatic distribution.

Such remarks on PARATIE should encourage the User to very carefully assess the results obtained by PARATIE.

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\(^1\) Several approximate methods are available to relate vertical movements to horizontal wall deformations; see Nova, Becci (1987) or section 7 of this manual.
3. TYPICAL STEPS OF A PARATIE ANALYSIS

Typical initial, intermediate and final steps of a PARATIE model of an opencut excavation retained by a flexible wall are reviewed in the following subsections. Each analysis step is currently different from the previous or from the forthcoming step due to different excavation, free field or water level, different active element layout and sometimes, different soil properties. In particular all the finite elements included in PARATIE can be activated and later removed during the analysis. An activated element is assumed to be strain free at the step when it is activated: only the prestressing force, if any, is transferred to the adjacent elements.

3.1 The initial step

The numerical analysis of a geotechnical problem usually starts with the restoration of some initial at rest configuration, which is assumed to exist in the soil mass prior to any subsequent modification. When using PARATIE, the recovery of such initial condition is advised as well. To do so, balanced excavation and water level should be assigned between uphill and downhill soil and no external loading or tie anchor should be activated at all.

The solution for such initial step is expected to be represented by null lateral displacements as well as null bending and shear force distribution in the wall. The soil element will display non zero stress, corresponding with the at rest lateral stress distribution, related to the vertical stresses by the at rest $K_0$ coefficient. In the initial step, lateral and vertical effects due to strip loadings are added to the geostatic stresses.

Note that PARATIE implicitly assumes that the insertion of the wall in the soil does not significantly change the in situ stress distribution in the soil itself.

PARATIE usually obtains the solution for initial step by means of two equilibrium iterations at most: if more than two iterations are necessary, the initial conditions are likely to be incorrectly specified. The User is encouraged to check if the initial configuration has been correctly reproduced.

3.2 An excavation phase

An excavation phase is simply described by lowering the excavation level. PARATIE automatically removes all the soil elements (only solid skeleton part) above the excavation or the free field level. The element removal modifies the equilibrium configuration at the previous step, because some internal forces are now missing. A new balanced configuration must be reached, by means of an iterative process, which successfully converge towards a new deformed configuration unless failure conditions are met.

In the remaining elements inside the excavation, the vertical stress is reduced, due to the removal of the weight of the excavated soil; for such elements the maximum horizontal stress limit (passive condition) is therefore reduced, thus requiring some stress redistribution to return within the plasticity boundaries.

Along with soil excavation, a water table lowering may be prescribed as well: such effect must be very carefully modelled since its effects on the overall wall stability may be very significant.

In some special situations, the improvement of the natural soil inside the excavation by means of special techniques, like jetgrouting, may be taken into account updating the stiffness and resistance properties of the soil.

3.3 Modelling a backfill

A previously removed soil layer may be reactivated, by simply rising the excavation or the free field level with respect to the previous step. The stress field inside such reactivated elements is defined, at the first iteration of the reactivation step, as follows:

- the effective vertical stress is computed assuming geostatic conditions and including the contribution of any strip loadings as reported in section 8;
- the effective horizontal component is computed by multiplying the vertical stress due to soil weight, the uniform surcharge, but not the contribution from strip loadings, by $K_0$;
- the water pressure is computed as in any other soil region.

Written by Ce.A.S Srl, Milan - Italy
Note that, at the end of the iteration process, the effective horizontal stress in the reactivated elements may differ from the at rest conditions.

Soil compaction may be approximated by simply prescribing and later removing an uniform heavy surcharge.

3.4 Anchor or strut installation

It is recommended to install active tie anchors in a step where no other modifications in the model occur.

A ground anchor installation should be preceded by an excavation step in which the dredge line is set just below the anchor level. Such step may be critical as far as deformations and bending stresses in the wall are concerned.

\[ K = E \times \frac{A}{L} \]

where \( E \) is the wire Young modulus, \( A \) is the wire area per unit depth, and \( L \) is the length of the deformable part of the anchor, to be estimate according to the next figure

\[ L = L_{\text{free}} + L_{\text{bond}} \times \eta \quad (\eta < 1) \]

For example, compute the \( A/L \) ratio for a 600 kN anchor placed at 2.5 m in horizontal direction (depth): assume
\[ \sigma_{\text{adm}} = 800 \text{ Mpa (typical working stress for ground anchors)}; \quad L = 15 \text{ m}; \]

we’ll have: \[ A = \frac{600 \times 10^3}{(2.5 \times 10^3 \times 800)} = 0.3 \text{ mm}^2/\text{mm}; \quad \text{then} \quad \frac{A}{L} = \frac{0.3}{15000} = 2 \times 10^{-5} \]

Note that a properly designed active anchor should keep its initial force almost unchanged during the subsequent steps (variations of about 10%–15% are acceptable, as a rule of thumb).

If no initial force is prescribed, a passive anchor is modelled. Similar behaviour is assumed during strut or slab installation. Struts (TRUSS elements) may have no tension behaviour.

### 3.5 Applying external loadings or restraints

It is usually not necessary to apply external loadings when modelling an excavation process of a retaining wall, because, in any step, the wall is deformed by the unbalanced internal forces between uphill and downhill soil elements. However PARATIE allows the definition of external concentrated or distributed lateral forces and moment, for very special situations, like for example, the presence of a temporary load cantilever fixed to the wall.

By the way, note once again that vertical loadings are not treated as external loadings on the wall but they simply affect the vertical stress distribution in the soil.

As for restraints, prescribed displacements or rotations can be assigned at every step in any position along the wall. Such option may be useful to model:

- rigid strut installation
- active tie anchor effects, in a preliminary design phase.
4. THE SOIL MODEL

4.1 Preliminary remarks

The interaction between a retaining wall and the soil is a complex geotechnical problem that can be solved at different degrees of approximation. Perhaps, the most commonly used approach, in the research field, is the represented by the solution of a two-dimensional plane strain finite element model. This method allows for a precise geometrical and soil parameter variation description; when using advanced finite element (or finite difference) programs, quite complex and realistic constitutive models for the soil behaviour can also be included. However, in practice, a two-dimensional (or, sometimes three-dimensional) finite element model can be reasonably used only at the final step of the design phase, when the final excavation and wall layout has been defined.

For the intermediate design phases, as well as for the design for structures with a low degree of complexity, a simpler numerical method like PARATIE can be very useful, since very many trial and error preliminary design iterations can be carried out at, practically, no cost. For this purpose, PARATIE implements a very simple numerical scheme, in which the soil is modelled by an array of Winkler springs; indeed this approach has been already adopted by many other authors (see Bowles (1988)).

What is original in PARATIE is the constitutive model of the simple linear soil springs, in which most of the observed soil behaviour aspects have been included. In particular, the soil spring stiffness should not only depend on the soil properties but also on the wall geometry as well as on the wall flexibility (see, for example, Jamiolkowski & Pasqualini (1979)). This aspect has been automatically included in PARATIE.

In spite of the quite complex and complete modelling features in the soil model of PARATIE, the input data required by PARATIE are represented by the usual resistance and flexibility parameters. Of course, the reliability of the obtained results highly depends on the accuracy in the parameter estimation.

To make the User familiar with the soil model, in this section the physical meaning of the soil parameters and their role in the constitutive model of PARATIE is briefly presented.

4.2 Soil model parameters

The constitutive soil parameters can be divided into two families: thrust parameters and compressibility (or flexibility) parameters

Thrust coefficients are the at-rest coefficient \( K_0 \), the active coefficient \( K_A \) and the passive coefficient \( K_P \).

The at rest coefficient affects the initial stress state in the soil prior to any excavation phase. \( K_0 \) ties the effective horizontal stress \( \sigma_h' \) to the effective vertical stress \( \sigma_v' \) by means of the following relationship:

\[
\sigma_h' = K_0 \sigma_v'
\]

\( K_0 \) depends on the soil resistance, through the effective friction angle \( \phi' \), and on the geological history:

\[
K_0 = K_0^{NC} (OCR)^m
\]

where:

\[
K_0^{NC} = 1 - \sin \phi'
\]

is the normal consolidated at rest coefficient (OCR=1). OCR is the overconsolidation ratio and \( m \) is an empirical parameter, usually ranging between 0.4 e 0.7. Ladd et al. (1977), Jamiolkowski et al. (1979) report \( m \) values for some Italian clays.

The active and passive thrust coefficients are given, according to Rankine, for a frictionless wall, by:

\[
K_A = \tan^2 (45^\circ - \phi' / 2)
\]

\[
K_P = \tan^2 (45^\circ + \phi' / 2)
\]
Through properly selected $K_A$ and $K_P$ values, the soil-wall friction angle $\delta$ and the soil surface sloping can be account for; selected values in NAVFAC (1986) (figure 6-6) o by Caquot & Kerisel (1948) are recommended.

Extreme effective horizontal stress limits are given by

$$\sigma_h = K_A \sigma_v - 2c' \sqrt{K_A}$$
$$\sigma_h = K_P \sigma_v + 2c' \sqrt{K_P}$$

where the first value is the minimum stress for soil in active conditions and the second the maximum stress corresponding with passive condition. $c'$ is the drained cohesion. Some modifications would be necessary to account for wall adhesion, however such parameter is not included in PARATIE.

Soil compressibility parameters enter in the spring stiffness. The stiffness $k$ per unit length of a Winkler spring array is given by:

$$k = E / L$$

$E$ is some soil stiffness modulus whereas $L$ is some scale length. In PARATIE, lumped springs are generated at finite distance $\Delta$, therefore the stiffness of each spring is:

$$K = \frac{E \Delta}{L}$$

$\Delta$ depends on the finite element mesh density; The $L$ parameter is automatically selected by the program. This parameters represents a characteristic length which is different between downhill and uphill soil regions. For uphill soil (which is very frequently in active state):

$$L_A = \frac{2}{3} \lambda_A \tan(45^\circ - \phi'/2)$$

whereas downhill (passive zone):

$$L_P = \frac{2}{3} \lambda_P \tan(45^\circ + \phi'/2)$$

$\lambda_A$ and $\lambda_P$ are:

$$\lambda_A = \min\{l, 2H\};$$
$$\lambda_P = \min\{l - H, H\}$$

where $l =$ total wall height and $H =$ current excavation depth. Details on such proposal can be found in Becci & Nova (1987).

The soil modulus $E$ depends on the stress history and on the local stress increment, as shown in Becci & Nova (1987).

$E$ may be very frequently related to the effective mean stress $p = \left(\sigma_v' + \sigma_h'\right) / 2$ by the equation

$$E = R \left(p / p_a\right)^n$$

in which $p_a$ is the atmospheric pressure, whilst $R$ and $n$ are experimental parameters. Of course, setting $n = 0$ a constant modulus is obtained, whereas, if $n$ is set to 1, typical modulus variation is obtained for normally consolidated clays.

$R$ is different between virgin compression and unloading-reloading load paths. Reference values for $R$ and $n$ are reported by Janbu (1963). Such parameters vary within a very wide range: for a sand, $n$ may be between 0.2 and 1.0 and $R$ between 8 and 200 MPa (a practical experience using PARATIE in a gravelly sand soil environment is reported by BARLA et al., (1988)). For NC clays $n = l$. $R$ values for Italian clays can be found in Jamiołkowski et al. (1979).
Since the initial stress state is not isotropic, the virgin compression soil stiffness is currently less that the measured stiffness in a drained isotropically consolidated triaxial test.

If $n=0$, $R$ modulus in virgin compression can be identified with the usual Young modulus. The unloading-reloading modulus is currently 3 to 10 times higher than virgin modulus for clays, whereas it is usually 1.5 to 3 higher for sands.

Note that, in PARATIE, the traditional subgrade modulus can be used as well, thus neglecting some of the most interesting features of the program. Typical subgrade modulus values may be found in Cestelli-Guidi (1984), Scott (1981), Bowles (1988).

The on line correlations included in PARATIE are based on the above remarks.
4.3 The soil model – general issues

It is assumed that the vertical and horizontal effective stress components, \( \sigma'_v \) and \( \sigma'_h \), the principal stresses. In a stress plane a yield function and a hardening rule are defined. (see Figure 4-1).

![Figure 4-1: the stress plane for a drained SOIL element](image)

With reference to such a plane, three phase conditions (or states) are possible; i.e.:

- **Elastic phase**: the soil element behaves elastically; this state corresponds with an unloading or reloading condition in the soil, whose stress is currently less than or equal to some previous stress level. The UL-RL mark is printed out for such state. In this conditions, the highest element stiffness is currently assigned to the element.

- **Hardening phase**: the stress state is increasing beyond maximum stress level in the previous step. A strain hardening behaviour is assigned during this phase and the element stiffness is still represented by a non zero value. In this state the V-C (Virgin Compression) mark is printed out.

- **Yielding**: the minimum (active) or maximum (passive) horizontal stress is reached and the element behaves as a plastic material.

To set up the initial stress state at the beginning of the analysis, the effective vertical stress at each depth is computed based on the free field elevation, on the surcharge an on the water level. The horizontal stress is then recovered using the at rest coefficient \( K_0 \). The contributions due to point loads are then added.

To establish the initial element phase (whether the element is in UR-LR or V-C phase), the overconsolidation ratio OCR the normally consolidated at rest coefficient \( K_{NC}^0 \) are used as follows:

Assume that maximum past stresses are:

\[
\sigma'_{v,max} = OCR \sigma'_{v,\text{step } 2} \\
\sigma'_{h,max} = K_{NC}^0 \sigma'_{v,max}
\]

If each of the two initial stress components are below the limits above, then the element is initially in UL-RL conditions, otherwise it is in virgin compression state.
In the subsequent steps, the effective stresses are computed as follows:

The vertical stress is computed based on the current excavation layout, current surcharge and current seepage conditions.

The horizontal stress is updated by computing the stress increments due to the element incremental deformation, by means of the elastic soil properties. The incremental stress is iteratively updated to meet the yield conditions based on the current vertical stress value.

The element pore pressure is then added to the effective lateral pressure to compute the total lateral pressure.

The relationship between $\sigma'_h$ and $\sigma'_v$ depends on the current element state, as follows:

- Initially, such a relationship is represented by the at rest coefficient $K_0$;
- At yielding, $\sigma'_h$ and $\sigma'_v$ are constrained to meet the yield condition
- In a stress path internal to the elastic domain, corresponding with null incremental lateral deformation, the incremental horizontal effective stress $\Delta\sigma'_h$ is related to the incremental vertical effective stress $\Delta\sigma'_v$, depending on $K_0^{\text{NC}}$. $\sigma'_v\text{max}$ and $\sigma'_h\text{max}$. With reference to Figure 4-2, some stress paths are represented corresponding to a freezer lateral deformation: if $\Delta\sigma'_v > 0$ and the stress point represents a normally consolidated state on the elastic domain bounded by the $\sigma'_v = \sigma'_v\text{max}$ and $\sigma'_h = \sigma'_h\text{max}$ lines, $\Delta\sigma'_h$ is computed by $\Delta\sigma'_h = K_0^{\text{NC}} \times \Delta\sigma'_v$; for example, such condition occurs for stress path from 0 to 2, or form 4 to 5 or from 7 to 8. For an overconsolidated soil which is reloaded (stress path from 3 to 4 or from 6 to 7), or in an unloading stress path (from 0 to 1), $\Delta\sigma'_h$ is computed by a non linear equation which, for an oedometric condition, meets the following relationship

$$\sigma'_h = K_0^{\text{NC}} (\sigma'_v\text{max} / \sigma'_v)^n \times \sigma'_v$$

or

$$\sigma'_h = K_0^{\text{NC}} (OCR)^n \times \sigma'_v$$

A mixed behaviour is finally assumed for stress paths like from 6 to 8, through 7, or from 3 to 5 through 4.

![Figure 4-2: oedometric stress paths](image)

Let’s now comment some other stress paths to further clarify the soil element behaviour.

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2 In this figure $\sigma'_v\text{max} = \sigma'_v\text{0}$ and $\sigma'_h\text{max} = \sigma'_h\text{0}$
Refer to Figure 4-3, and follow the stress path in the $\sigma'_v-\sigma'_h$ plane as well as in the $\delta-\sigma'_h$ plane, where $\delta$ = lateral soil deformation (+ve if the soil is compressed). The point 1 corresponds with at rest conditions for a granular overconsolidated soil; in fact, point 1 is internal to the boundary between the elastic and the virgin compression (hardening) region. Subsequent points track the lateral stress state evolution at $\sigma'_v$=constant: by compressing the element, the virgin compression boundary is firstly encountered (pt 2); from 1 to 2 the elastic (unloading-reloading) stiffness is used, whereas from pt 2 on, the virgin compression stiffness is adopted until the yield (passive) limit is reached. Point 4 is reached by the development of plastic strain. The path from 4 to 5 represents an unloading path (with elastic stiffness) whereas from 5 to 6 a reloading path.

In Figure 4-4 a very similar path is reported. The only difference stays in the 3-4 branch, which shows the unloading behaviour from the virgin compression branch, before reaching the yield conditions.
Figure 4-5

In Figure 4-5, point 1 represents an at rest stress state for a normally consolidated granular material: in fact such point establishes the boundary between elastic (unloading-reloading) region and the virgin compression zone. Subsequent points no. 2 and 3 represent a stress evolution at $\sigma'_v=$const. towards passive conditions (point 3); the path from 3 to 4 is related to vertical stress reduction along with a lateral strain release towards active limit conditions in 4.

Additional stress path examples can be found in the paper by Becci & Nova (1987).
4.4 The soil model for clays

The constitutive model for clays describes the limit conditions based on effective resistance parameters only, and allows the simulation of both drained and undrained conditions, as well as the transition between them.

In drained conditions, this model is very similar to the model for granular soils. The only difference lies in the apparent cohesion parameter $c'$, which is now a varying parameter with the preconsolidation level, whereas for granular soils is a fixed User’s input value.

During undrained conditions, both effective stress path (ESP) and total stress path (TSP) are computed and the previous one is monitored to check limit conditions. The ESP evolution is highly affected by the imposed constraint on the volumetric deformation which must be null in such conditions. Since both ESP and TSP can be computed, the pore pressure change within the saturated soil in undrained condition can be easily computed as well.

4.4.1 Failure condition

Let’s first analyze the limit condition in terms of effective parameters. The elastic domain is shown in the next figure: it basically depends on four thrust coefficients ($K_{A,cv}$, $K_{A,peak}$, $K_{P,cv}$, $K_{P,peak}$) and on the preconsolidation level ($\sigma_{v,max}$, $\sigma_{h,max}$).

![Figure 4-6: clay model - limit conditions](image)

Based on $\sigma_{v,max}$ or $\sigma_{h,max}$, points A and P are computed on the limit state lines $\sigma_{h} = K_{A,cv} \sigma_{v}$ and $\sigma_{h} = K_{P,cv} \sigma_{v}$. Then A’ and P’ are computed on axes ($\sigma_{v} = 0$ and $\sigma_{h} = 0$), as the intersections of two lines from A or P with slope equal to $K_{A,peak}$ or $K_{P,peak}$. The (A’A0PP’O) polygon is the elastic domain and O-A’ or O-P’ segments represent the cohesion. As long as $\sigma_{v,max}$ or $\sigma_{h,max}$ increase, A or P move and the domain expand irreversibly.

The $\sigma_{h} = K_{A,cv} \sigma_{v}$ and $\sigma_{h} = K_{P,cv} \sigma_{v}$ lines can be revised as critical state lines and represent the ultimate conditions related to large deformations. $K_{A,cv}$ and $K_{P,cv}$ are active and passive thrust coefficients depending on the critical state friction angle $\phi^{'cv}$ and on the friction between the soil and the wall.

$K_{A,peak}$ e $K_{P,peak}$ are the slope of two straight boundaries of the linearized domain near the origin and are active and passive thrust coefficients depending on a friction angle $\phi^{'p} < \phi^{'cv}$ and on the friction between the soil and the wall as shown in the next figure.
Figure 4-7: Mohr limit condition for OC clays

It’s worth noting that, based on this model, a clay resistance may be completely described by $\phi'_{cv}$ only (which currently depends on the Plasticity Index); in this case, PARATIE internally computes $\phi'_{p}$ via:

$$\tan(\phi'_{p}) = \frac{\tan(\phi'_{cv})}{1.5}$$

and computes the thrust coefficients by the Rankine formulas.

4.4.2 Drained behaviour

In case of drained behaviour, no relevant differences exist in the clay model with respect to granular soil. Just note that the apparent cohesion varies along with stress. In the next figure, some effective stress paths (ESP) are shown.

Figure 4-8: some drained stress paths (clay model)

Drained soil stiffness is computed by PARATIE, based on the effective modulus according to User’s input.
4.4.3 Undrained behaviour

In undrained conditions, the zero volume change condition is imposed, as briefly outlined in the following.

- initial drained conditions are assumed:
  \[ \sigma_{h,0} = K_0 (\sigma_{v,0} - u) + u + \text{strip foundations effects} \]
- in the subsequent steps, if the lateral deformation is freezed (at the very beginning of each step), any total vertical stress increment produces an equal total lateral stress increment, due to a water pressure increment only; i.e. the effective stress does not change and the incremental load on the soil is only supported by the water
- when incremental lateral deformations are allowed, total lateral stress increments occur (with zero total vertical stress increment); in the elastic domain it must be ensured that:
  \[ \Delta \sigma_v' + \Delta \sigma_h' = 0 \]
  to satisfy zero incremental volumetric strain constraint. Such conditions dictated the effective stress path slope within the elastic domain.
- When the elastic domain boundary is reached, different behaviours are possible, depending on the stress path. In such cases, plastic strains are computed assuming an associated flow rule. In Figure 4-9 some possible stress paths are shown (note the elastic stress path slope \( \Delta \sigma_v'/\Delta \sigma_h' = -1 \) and a different slope towards virgin conditions in which \( \Delta \sigma_v'/\Delta \sigma_h' = -\alpha \) or \(-1/\alpha\), with \( \alpha = E_{uv}/E_{vc} \))
- the water pressure is computed by subtracting the effective stress from the total stress (see Figure 4-10, in which, for example, in point C, \( u_c \) is negative, like a suction)
- soil permeability is null.

![EFFECTIVE STRESS PATHS DURING UNDRAINED CONDITIONS AT](image)

**Figure 4-9: some stress paths during undrained conditions**

It is worth noting that, in principle, no undrained shear strength \( S_u \) is necessary to model the undrained behaviour, because such parameter is implicitly derived from the effective constitutive model. It is however possible to assign an external value for \( S_u \) thus prescribing an additional check on the total shear. In this case, PARATIE will monitor both effective and total stress path and stop the load increment as long as the first one of the above reaches its relevant boundary.

With respect to, we note that stress point 0 may reach A’, as far as only effective failure condition is met by the effective stress path (ESP); however it should stop at A, because the total stress path (TSP) reaches the total failure.
domain given by $S_u$ (thick line). On the other hand, the ESP starting from 1 moves on the effective failure boundary (1-B-C) whereas the corresponding TSP is still elastic with respect to $S_u$ (of course, the stress point is plastic, because the ESP conditions prevails on the TSP one).

To remove the dependency on $S_u$, a very large value must be assigned to such parameter.

$$\Delta \sigma = 0$$

**Figure 4-10: undrained stress path examples**

Undrained soil stiffness is internally computed by PARATIE, based on the effective modulus.

**4.4.4 Switching from undrained to drained behaviour and vice versa**

**Passing from undrained to drained behaviour:**

During undrained behaviour, both total and effective stress path is computed; therefore pore pressure can be computed by subtraction. Note that currently, is such state, pore pressure varies due to soil shear deformation.

When passing into drained conditions, pore pressures are simply recalculated based on the current uncoupled water table conditions (i.e. based on the phreatic surface and eventual steady state seepage). Therefore excess pore pressure due to soil deformation is dissipated. Further, just effective stress component is included in the constitutive equations, as described above.

**WARNING:** if the simplified undrained analysis option has been activated (as explained in sect. 4.4.5) such state change is not possible.

**Passing from drained to undrained behaviour:**

at the beginning of the i-th step, at which such state change has been prescribed, PARATIE computes $\sigma_{h,i}$ by:

$$\sigma_{h,i} = \sigma_{h,i-1}$$

with

$$\sigma_{h,i} = \sigma_{h(i-1)} + \mu_{i-1}$$

where

$\sigma_{h(i-1)}$ horizontal effective stress at the end of the previous (i-1) step, when drained behaviour existed.

$\mu_{i-1}$ water pressure at the end of the previous (i-1) step

During the iterative process, $\sigma_{h,i}$ may further vary due to incremental soil deformation.
4.4.5 Simplified undrained behaviour

When using such soil model, neither undrained shear strength $S_u$ nor undrained modulus $E_u$ have to be input in spite of undrained soil behaviour modelling.

However, it is possible to perform a simplified undrained analysis giving just $S_u$ and $E_u$. In this case, PARATIE will monitor the total stress only whereas the effective stress failure conditions are simply neglected. In such case, it’s however not possible to pass into drained conditions because the pore pressure evolution is lost during the undrained phase.

4.4.6 How to initialize the clay behaviour

When using this model, it’s very important to restore as precisely as possible the initial soil conditions, because the initial (past) state highly affects both the drained and undrained behaviour.

For a NC clay, no particular strategy is needed to start the analysis, with respect to a sandy soil.

For an OC clay, a realistic OCR profile has to be restored first. For such purpose, two possibilities are given:

1. a unique OCR value greater than 1 may be assigned to each OC clay layer, or
2. an initial OCR=1 is assigned (thus obtaining a NC clay), but the true analysis has to be anticipated by the simulation of the overconsolidation process as follows:
   a. an initial step is prescribed in which drained conditions are activated and a free field surcharge is imposed to simulate the proper overconsolidation ratio
   b. a second step is included, in which such surcharge is removed, but still drained conditions are maintained.
   c. the actual initial analysis step is finally prescribed, in which the clay is either in drained or in undrained conditions depending on the expected behaviour during the excavation process.

The second approach is recommended because a more realistic OCR profile is currently recovered.

![Figure 4-11: recovering initial OCR profile in a clay layer](image)

The figure above briefly outlines such procedure.
### 4.5 Soil model summary

In the next table, a very brief summary of the relevant features of the three available soil models is included to assist the User in the selection the most appropriate model for his/her problem.

<table>
<thead>
<tr>
<th></th>
<th>granular soil model</th>
<th>clay model</th>
<th>simplified model for undrained behaviour</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>drained conditions</strong></td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td><strong>undrained conditions</strong></td>
<td>—</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td><strong>from drained to undrained</strong></td>
<td>—</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td><strong>from undrained to drained</strong></td>
<td>—</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td><strong>resistance parameters for drained conditions</strong></td>
<td>$c', \phi'$</td>
<td>$\phi'<em>{cv}$, $\phi'</em>{p}$</td>
<td>—</td>
</tr>
<tr>
<td><strong>resistance parameters for undrained conditions</strong></td>
<td>—</td>
<td>$\phi'<em>{cv}$, $\phi'</em>{p}$, $S_u$</td>
<td>$S_u$</td>
</tr>
<tr>
<td><strong>flexibility parameters for drained conditions</strong></td>
<td>$E_{ur}$, $E_{vc}$</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td><strong>flexibility parameters for undrained conditions</strong></td>
<td>—</td>
<td>$E_{ur}$, $E_{vc}$</td>
<td>$E_u$</td>
</tr>
<tr>
<td><strong>resistance parameter modification during analysis</strong></td>
<td>✓</td>
<td>—</td>
<td>✓</td>
</tr>
<tr>
<td><strong>flexibility parameter modification during analysis</strong></td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td><strong>pore pressure calculations</strong></td>
<td>✓</td>
<td>✓</td>
<td>—</td>
</tr>
<tr>
<td><strong>permeability</strong></td>
<td>✓</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td><strong>granular soil modelling (sands, gravels)</strong></td>
<td>✓</td>
<td>✓</td>
<td>—</td>
</tr>
<tr>
<td><strong>clay modelling in drained conditions</strong></td>
<td><strong>simplified</strong></td>
<td>✓</td>
<td>—</td>
</tr>
<tr>
<td><strong>clay modelling in undrained conditions</strong></td>
<td>—</td>
<td>✓</td>
<td><strong>simplified</strong></td>
</tr>
<tr>
<td><strong>cemented sand modelling</strong></td>
<td>✓</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td><strong>modelling of improved soil behaviour</strong></td>
<td><strong>recommended</strong></td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td><strong>simulation of weak rock behaviour by unconfined compressive strength</strong></td>
<td>—</td>
<td>—</td>
<td><strong>recommended</strong></td>
</tr>
</tbody>
</table>
5. THE WATER IN THE SOIL

When taking into account the water effects in the soil, PARATIE assumes that the submerged soil is fully saturated (100% degree of saturation).

It is assumed that the pore pressure distribution is not affected by any deformation and stress in the soil solid skeleton.

In a submerged soil region to which undrained behaviour has been assigned, the pore pressure is undefined, unless the clay model is used.

All time dependent effects like soil consolidation are not considered at all.

Two possible stationary conditions are foreseen:

1. hydrostatic condition, without water flow within the soil;
2. steady state seepage conditions, represented by a water flow within a porous medium.

In the first case, the pore pressure distribution is hydrostatic; in the second case, some important remarks are needed, which are included in the next section.

5.1 Water pressure in steady state seepage conditions

If a downhill water table lowering is prescribed (by setting the $DZWT$ parameter to a non zero value), steady state seepage conditions arise. The pore pressures and the hydraulic gradients in the soil mass are computer by a simplified yet conservative scheme outlined in the following.

![Figure 5-1: PARATIE seepage scheme](image)

A vertical flow path is assumed, as shown in the figure above. The overall path length $L$ is computed as the minimum path length adjacent to the wall (neglecting the wall thickness). PARATIE assumes that the lining effects of the wall finish at elevation $Z=Z_{BALANCE}$. Based on such an approximation, the flow path length is minimized (about this assumption, which is in agreement with the very simple overall scheme adopted in PARATIE, see Lancellotta (1988)).

Let:

$DH$= Overall head loss ($DH = DZWT$)

$v$= flow velocity

$K_i$ = $i$-th soil layer permeability coefficient

$L_i$ = $i$-th soil layer thickness
\( DH_i = \) partial head loss within \( i \)-th soil layer

By invoking the Darcy law, the continuity equation and assuming a constant hydraulic gradient inside each soil layer, we can say that:

\[ v = K_i \frac{DH_i}{L_i} \]

and, noting that:

\[ DH = \sum_j DH_j, \]

the \( v \) unknown can be removed, and the \( i \)-th partial head loss \( Dh_i \) is readily given by

\[ DH_i = DH \frac{L_i}{K_i} \sum_j \frac{L_j}{K_j} \]

Such sum has to be extended over all the crossed soil layers in both uphill and downhill regions.

Once each partial head loss has been computed, the pore pressure distribution is given by applying the Bernoulli theorem:

\[ u = u(Z) = \gamma_w(ZWT - Z) - \sum_k DH_k \]

in this equation, the sum must be extended over all the crossed soil layers up to current position.

If an undrained region is encountered along the assumed flow path, the flow is actually stopped. In this case, hydrostatic conditions occur, based on the assumptions reported in section 5.2.

Remarks:

- Such scheme represents a first guess yet acceptable solution of the real problem, because it is currently on the safe side. The hydraulic gradients are currently overestimated thus overestimating the seepage drag forces which actually represent a real danger as long as downhill quick conditions are concerned.

- On the other hand, it can be shown that the pore pressure distribution is slightly underestimated with respects to other more precise approaches; however such a drawback is fairly compensated by the conservativeness in the seepage forces estimate.

- When the permeability is omitted in the definition of the property of a soil layer, PARATIE assigns a very low number for such parameter, thus implicitly assuming that such layer is practically impervious. However, if such a default value is assigned all the soil layers (in other words, when the permeability is always omitted), a seepage in a homogeneous medium is actually reproduced, because PARATIE activates the flow anyway.

- It is not necessary to use consistent unit when assigning permeability with respect to other soil parameters; however it is important to preserve the representative permeability ratios among the various layers.

- The balance level \( Z\text{BALANCE} \) can be explicitly assigned by the User, thus modifying the PARATIE default assumption corresponding with the lowest wall depth. By this technique, some effects can be reproduced, as follows:

  a) By placing \( Z\text{BALANCE} \) at a very low level, an uncoupled pore pressure distribution between uphill and downhill soil is obtained (like in the case when an impervious soil exists at the wall toe):

  b) By placing \( Z\text{BALANCE} \) at a lower elevation that the wall toe, the flow path is artificially stretched thus recovering a desired pressure distribution which somehow differs from the default scheme.

- For the soil element below the \( Z\text{BALANCE} \) level, the pore pressure is computed by setting the balance level at the element position.

Some frequent situations are outlined in figure 5-2
It is mandatory to prescribe the water weight for all the soil layers. No check is made by PARATIE about the input value. If the water weight is omitted, unpredictable and unreasonable results may be obtained.

---

Here the same pressures exist up and downhill. Water pressures may be neglected as long as lateral equilibrium is concerned.

Same water pressure for uphill and downhill soil elements are computed at $Z = Z_{\text{BALANCE}}$.

A situation like this can be modelled by putting $Z_{\text{BALANCE}} = -\infty$.

If a very low permeability is assigned in a soil region, the head loss is dissipated in such an impervious area whereas hydrostatic pressures are computed elsewhere. For the stability of the impervious region, good mechanical properties are currently necessary.

**Figure 5-2**: some typical cases that can be modelled by PARATIE
5.2 Pore pressure distributions when one undrained soil layer exists

PARATIE assumes that no seepage is possible across an undrained soil region

First of all, consider the case where only one undrained soil region interferes with the possible flow path: this situation occurs when undrained behaviour is assigned to only one soil layer, in only one side of the wall.

In this situation, the undrained layer acts as an imperious boundary which hydraulically separates the soil regions which communicate with either the uphill or the downhill phreatic levels.

In any soil region, the relevant hydrostatic conditions can be easily established. In the next figure, some examples are outlined.

In case 1, uphill region 3 directly communicates with the downhill water table; in case 2, on the contrary, downhill region 2 communicates with the uphill water table; in case 3, regions 3, 4 and 5 are linked to the downhill water table; in particular, in region 3 a null pore pressure distribution is assumed. In case 4 the same hydrostatic conditions hold for any soil region.

In the undrained regions, the pore pressures are undefined.

Let’s now consider several undrained regions interfering with the potential seepage path: is this case some drained regions between undrained layers may exist, which directly communicate neither with the uphill nor with the downhill water table.
From a general point of view, for such “unconnected” regions, no hydraulic conditions may be established. In these cases, however, PARATIE operates as follows:

- at the first analysis step ($T=0$), corresponding with initial conditions, PARATIE assigns such regions a mean water table between uphill and downhill level.

**Figure 5-4: initial pore pressure when more than one undrained region exists**

- in any subsequent step, PARATIE assigns such regions the pore pressures of the previous step.

**Figure 5-5: pore pressure when more than one undrained region exists, $T>0$**
According to this approximation, it is strongly advised to prescribe initial balanced hydraulic conditions.

Let’s finally consider the case where some undrained regions exist, not interfering with the assumed flow path.

**Figure 5-6: an undrained layer not stopping the seepage**

In this case the steady state seepage is not stopped by the undrained impervious barrier.
5.3 Dredge line stability and “Lining Option” usage

To model some lining effects at the dredge line inside an opencut excavation, in order to allow dewatering operations, the following two alternatives methods can be used with PARATIE:

1. A soil improvement can be modelled which highly reduces the permeability of a soil mass inside the excavation;
2. Using the “Lining Option”

5.3.1 The Simulation of a soil improvement.

The dewatering inside an excavation is sometimes activated after improving a certain soil mass at the dredge line by cement grouting or by other techniques like jetgrouting. From the mechanical point of view, such a treated soil mass receives an improvement in terms of permeability reduction, shear strength and elastic modulus increase.

Let’s assume that, from a certain step on, such an improvement operation has been simulated by PARATIE simply changing the natural soil properties (permeability, cohesion and stiffness) in accordance with the foreseen improved properties. Assume that the natural soil is a cohesionless soil.

The dewatering operation can be modelled by simply assigning a non zero value for the head loss parameter DZWT>0

If the downhill water table is lowered under the lower surface of the improved mass (case A, in Figure 5-7), a seepage flow will be activated in the natural soil only; no water pressures will occur at the improved mass base, whose stability will be not reduced; the soil under the improved mass may contribute to the wall stability unless quick conditions are reached in such zone.

If the downhill water head is higher than the lower improved soil mass elevation (case B in Figure 5-7), the head loss will be basically dissipated inside the improved soil; in the natural soil, the uphill water head will act and the seepage forces will be negligible. Such a situation is only possible if the uplift water pressure resultant at the improved soil mass base does not exceed the total weight of the improved soil mass: assume

\[ \gamma_t \cdot h_t \geq \gamma_w \cdot z_t \]

If the condition above is not met, additional downwards load must be added, by some ballasting surcharge and/or by vertical ground anchors. Such stabilizing effect must be included in the PARATIE model by applying an equivalent surcharge \( q_s \) at the dredge line level; \( q_s \) shall be at least equal to \( (\gamma_w \cdot z_t - \gamma_t \cdot h_t) \).

\[ ZWT-DZWT \]

Figure 5-7: improving the soil for dewatering

5.3.2 The “Lining Option”
If the *Lining Option* is activated, PARATIE removes the pore pressure part of the element stress for those soil elements above the round level and neglects the water weight above the ground level when computing the total vertical stress in the remaining elements under the excavation level at the excavation side.

![Figure 5-8: the "lining option"](image)

This simulation is realistic provided an adequate surcharge is prescribed at the excavation level, that must be greater than or at least equal to the water pressure at the excavation elevation with respect to the uphill water table. Such option should be used very carefully.

### 5.4 Pore pressures by tabular values

As an alternative to the default procedure described above, uphill and downhill pore pressure profiles can be assigned by tabular values along the wall depth.

![Figure 5-9: tabular pore pressure profiles](image)

In this case, following assumptions are made by PARATIE (Figure 5-9):

- At each side, the phreatic level is assumed corresponding with maximum elevation in pore pressure profile description.
- Below such phreatic elevation, saturated soil is assumed.
- Out of the pressure definition range, zero pore pressures are assumed.

No check is made by PARATIE on input values: it’s on User’s side to assign consistent values with physical problem. Therefore such option should be used by skilled Users only.

Note that pore pressure definition by tabular values prevails on standard water description; however, when in a step description, no tabular values are given, standard water description is restored: should the User want to give the same tabular pore pressures in many steps, he/she should explicitly assign such values at each step.

Tabular description may be useful to model special situations, such as the following:

- Definition of pore pressures profiles computed by different algorithms, such as finite difference flow nets
- Modelling of special water conditions that cannot be reproduced by standard procedure (e.g. Figure 5-10 and Figure 5-11)

![Figure 5-10: a suspended aquifer](image1)

![Figure 5-11: an artesian (pressurized) aquifer](image2)
6. SOIL PARAMETER ESTIMATE

6.1 Premise

PARATIE offers a set of correlation to estimate most of the request soil parameters. These correlations are intentionally limited to soils or weak rocks which currently represent most of the situations where a flexible wall is required.

The proposed correlations have been selected amongst the great amount available in the technical literature and should revised as a tool for a first guess estimate of the soil parameters.

Most of such correlations are based on field tests: for granular soils such method of investigation currently represent the most common and feasible technology employed, however, for cohesive soils, an adequate laboratory test campaign is more advisable.

As for the proposed correlations for elastic moduli, we note that most of the available studies aim at an evaluation of suitable elastic properties for foundation problems, which obviously quite differ from the problem of a retaining wall, in particular in the fact that, in the latter case, lower deformation are currently expected with respect to the previous one. Nevertheless we believe that such correlations, yet not perfectly suitable for the estimate of elastic properties in a retaining wall problem, may as well assist the User in the evaluation of such parameters.

We finally observe that the correlations are currently presented in a graphic way, that should reveal the complexities underneath and suggest a great care in their usage.

When relevant doubts still remains in the evaluation of some parameters, a sensitivity analysis is advised, thus studying the dependency of relevant results on the variation of most doubtful parameters. By this approach, a quite reliable depiction of the problem under investigation is currently obtained.
## 6.2 List of Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c'$</td>
<td>drained cohesion</td>
<td>kPa</td>
</tr>
<tr>
<td>CPT</td>
<td>Cone Penetration Test</td>
<td></td>
</tr>
<tr>
<td>$D_r$</td>
<td>Relative density $^3$: $D_r = (e_{\text{max}} - e) / (e_{\text{max}} - e_{\text{min}})$</td>
<td>%</td>
</tr>
<tr>
<td>$D_r$</td>
<td>granular soils are usually subdivided as follows</td>
<td></td>
</tr>
<tr>
<td>$e$</td>
<td>void ratio defined as the ratio of the void volume to the solid volume</td>
<td>%</td>
</tr>
<tr>
<td>$E$, $E_{\text{vc}}$, $E_{\text{uv}}$</td>
<td>Young modulus $^4$</td>
<td>kPa</td>
</tr>
<tr>
<td>$E_a$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$e_{\text{max}}$</td>
<td>Max. void ratio, corresponding with the loosest state.</td>
<td>%</td>
</tr>
<tr>
<td>$e_{\text{min}}$</td>
<td>Min. void ratio, corresponding with the densest state.</td>
<td>%</td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus</td>
<td>kPa</td>
</tr>
<tr>
<td>$G_0$</td>
<td>Shear modulus at very small strains</td>
<td>kPa</td>
</tr>
<tr>
<td>$K_a$</td>
<td>Active thrust coefficient</td>
<td></td>
</tr>
<tr>
<td>$K_p$</td>
<td>Passive thrust coefficient</td>
<td></td>
</tr>
<tr>
<td>$N'_{60}$</td>
<td>Normalized blow representing the SPT blow count measured by a SPT device with an energy ratio=60% at an overburden pressure of 1 atm</td>
<td></td>
</tr>
<tr>
<td>$N_k$</td>
<td>Cone factor</td>
<td></td>
</tr>
<tr>
<td>$N_{\text{spt}}$</td>
<td>SPT blow count</td>
<td></td>
</tr>
<tr>
<td>OCR</td>
<td>Overconsolidation ratio</td>
<td></td>
</tr>
<tr>
<td>$p_a$</td>
<td>Atmospheric pressure</td>
<td>kPa</td>
</tr>
<tr>
<td>PI</td>
<td>Plasticity Index: $\text{PI}=w_p-w_L$</td>
<td>%</td>
</tr>
<tr>
<td>$q_c$</td>
<td>Cone penetration resistance</td>
<td>kPa</td>
</tr>
<tr>
<td>SPT</td>
<td>Standard Penetration Test</td>
<td></td>
</tr>
<tr>
<td>$S_u$</td>
<td>Undrained shear strength</td>
<td>kPa</td>
</tr>
</tbody>
</table>

$^3$ $D_r$ in PARATIE is expressed as a fraction (between 0 and 1).

$^4$ Depending on the right index, a different physical meaning is assigned, which will be easily argued from the context.
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>$u$</td>
<td>Pore pressure</td>
<td>kPa</td>
</tr>
<tr>
<td>$w$</td>
<td>water content defined as the ratio of water weight to the solid weight</td>
<td>%</td>
</tr>
<tr>
<td>$w_l$</td>
<td>liquid limit(^5)</td>
<td>%</td>
</tr>
<tr>
<td>$w_p$</td>
<td>plastic limit (note 5)</td>
<td>%</td>
</tr>
<tr>
<td>$\delta$</td>
<td>Friction angle between soil and wall</td>
<td>[°]</td>
</tr>
<tr>
<td>$\phi'$</td>
<td>drained friction angle</td>
<td>[°]</td>
</tr>
<tr>
<td>$\phi'_{cv}$</td>
<td>constant volume friction angle</td>
<td>[°]</td>
</tr>
<tr>
<td>$\phi'_{peak}$</td>
<td>peak friction angle</td>
<td>[°]</td>
</tr>
<tr>
<td>$\phi_u$</td>
<td>undrained friction angle (currently $\phi_u = 0^\circ$)</td>
<td>[°]</td>
</tr>
<tr>
<td>$\eta$</td>
<td>Energy ratio of a SPT device</td>
<td></td>
</tr>
<tr>
<td>$\sigma'_{h}$</td>
<td>effective horizontal stress</td>
<td>kPa</td>
</tr>
<tr>
<td>$\sigma'_{v}$</td>
<td>effective vertical stress</td>
<td>kPa</td>
</tr>
<tr>
<td>$\sigma_h$</td>
<td>total horizontal stress</td>
<td>kPa</td>
</tr>
<tr>
<td>$\sigma_v$</td>
<td>total vertical stress</td>
<td>kPa</td>
</tr>
</tbody>
</table>

\(^5\) See the Atterberg limits definition
6.3 Some remarks on the field tests

Most of the correlations proposed by PARATIE are based on penetrometric tests. When using dynamic penetrometric tests, explicit reference is made to the SPT test carried out according to the ASTM procedure. The normalized blow count \( N'_{60} \) requested by some correlation is defined as:

\[
N'_{60} = (\eta/60) \times C_N \times N_{spt}
\]

Where

- \( \eta \) = sampling device energy ratio (as a percentage between 0 and 100)
- \( C_N \) = overburden correction factor
- \( N_{spt} \) = sampled blow count (no. of blows to obtain 1 ft penetration)

\( N'_{60} \) is therefore the equivalent blow count that would be measured by a 60% performing equipment at a depth corresponding with an effective vertical pressure of 1 atm (in practice at a depth of 5÷5.5 m, in a dry sand).

In PARATIE, the \( N_{spt} \) value can be input to compute the \( N'_{60} \) parameter.

A very great care must be taken when entering these correlations with some blow count number obtained by a different sampling procedure than the standard one.

Several different dynamic penetration procedures which actually differ from the SPT test are available and very frequently used. However the results given by such procedures should not directly be used to enter the correlations based on the SPT results, unless information is available that can somehow correlate the used non-standard procedure to SPT. For non-standard dynamic penetration procedures used in Italy, such kind of information is available after Cestari (1990).

Along with the blow count profile, a borehole log is currently necessary as well as granulometric tests etc.

Note that using the proposed correlation without a good knowledge of the soil nature may bring to very unreliable conclusions.

As for the static cone penetration test (CPT), reference is made to the cone resistance \( q_c \) that must be measured by a standard procedure. CPT tests are usual for clays and loose sands. Literature relationship between \( q_c \) and \( N_{spt} \)
(Figure 6-2) are available to set up a cross reference between such different results. It may be worthwhile using this relationship to compare the results given by a correlation based either on $N_{spt}$ or on $q_c$.

No explicit reference is made here to other field test technology like, for example, the pressurimeter, which may provide the engineer with even more valuable information than the penetrometric tests. We feel that the User is free to select the desired testing method and directly enter the soil parameters without using the correlations proposed by PARATIE.
6.4 Correlations for granular soils

Some correlation are selected based on the following filed test methods

- Standard Penetration Test (SPT),
- Static Penetrometer (CPT) (to be used for loose sands only)

6.4.1 Resistance parameters

Drained cohesion \( c' \) is usually null for granular soils; however non zero values may be input in very special circumstances to model, for example, the apparent cohesion in a wet soil\(^6\). It is recommended to very carefully select such a value because even a very low cohesion greatly reduced the soil thrust onto a wall.

Friction angle \( \phi' \) is obtained by a correlation with \( D_r \). The latter is evaluated by means of relationships with either \( N'_{60} \) or \( q_c \). The procedure is as follows.

**STEP 1: \( D_r \) computation**

<table>
<thead>
<tr>
<th>If SPT is available</th>
<th>Correlations after Cubrinovsky &amp; Ishihara (1999) are implemented which include a dependency with the relevant grain size by means of ( e_{\text{max}}-e_{\text{min}} ).</th>
</tr>
</thead>
<tbody>
<tr>
<td>If CPT is available</td>
<td>Correlations for NC and OC sands are given according to Baldi et al. (1988).</td>
</tr>
</tbody>
</table>

**STEP 2: \( \phi'_\text{peak} \) calculation**

| \( D_r \rightarrow \phi'_\text{peak} \) | PARATIE offers correlations after Bolton (1986)\(^7\) or Schmertmann (1977). |

**STEP 3: assessment of the design value for \( \phi' \)**

The design value for the friction angle \( \phi' \) lays between \( \phi'_{\text{cv}} \) and \( \phi'_{\text{peak}} \). As a first guess assumption, a 1.5 safety factor can be applied to \( (\phi'_{\text{peak}} - \phi'_{\text{cv}}) \): i.e.

\[
\phi' = \phi'_{\text{cv}} + (\phi'_{\text{peak}} - \phi'_{\text{cv}})/1.5
\]

6.4.2 Stiffness parameters

For a first guess estimate of elastic moduli (\( E_{\text{vc}} \) and/or \( E_{\text{ur}} \)) linear correlations with either \( N_{\text{SPT}} \), or \( q_c \) are available.

The correlations with \( N_{\text{SPT}} \) are in the form:

\[
E = C_1 N_{\text{SPT}} + C_2
\]

where \( C_1 \) e \( C_2 \) are in [MPa]

The correlations with \( q_c \) are in the form:

\[
E = \alpha q_c
\]

To assess the consistency between these two families, let’s look at the following relationship:

\[
G [\text{MPa}]= N_{\text{SPT}} \quad (\text{Randolph, 1981}),
\]

assuming \( \nu=0.25 \), we have:

\[
E [\text{MPa}] = 2.5 N_{\text{SPT}}
\]

\(^6\) apparent cohesion should not be assigned to submerged soils.

\(^7\) In this case the constant volume friction angle \( \phi'_{\text{cv}} \) must be also selected. The proposed correlation refers to \( (\phi'_{\text{peak}} - \phi'_{\text{cv}}) \) values for plane strain conditions.
Let’s finally assume:

\[ q_c [\text{MPa}] = 0.4 \div 0.5 \text{ N}_{\text{SPT}} \]

therefore

\[ E = 5 \div 6 q_c \]

We recover values for \( \alpha \) in agreement with experimental data.

Hence we find that the two different approaches to compute the elastic modulus, actually produces very similar predictions for such values.

Most of the available correlations give a prediction of a **secant elastic modulus** \( E_s \) to be used in the calculation of foundations settlements. By definition, the secant modulus depends on the strain level at which it has been defined (Figure 6-3): hence the values given by such correlations are defined at typical strain level for foundations (about the 30%-50% of the strains at failure), usually higher than the typical strain values for the problems dealt with by PARATIE. From Figure 6-4, we note that soil stiffness currently decreases when the strain is increased: it can be therefore concluded that the secant moduli obtained by experimental correlations tuned for foundations currently underestimate the secant moduli to be used in a retaining wall problem. In other words, we can say that the correlations included in PARATIE just give a quite coarse approximation of the moduli and very likely such values represent an underestimate of the real values. A similar conclusion has been reached by backanalysis studies on real excavation cases (see, for example, Fenelli & Pagano, 1999).

![Figure 6-3: secant modulus definition](image)

![Figure 6-4: secant shear modulus G decay with strain increase (G₀ is the shear modulus at very low strains); this plot is given by Viggiani & Atkinson, 1995, for a clay, but the same trend is shown by sands.](image)

If stress dependent moduli are used, in the form

\[ E = R \left( \frac{p'}{p_a} \right)^n \]

PARATIE presents some first guess values of the \( R \) parameters for sand or gravels, related to \( D_r \) (values from Lancellotta, 1988) to be used taking \( p' = \sigma_{\text{hy}} \), with \( n \) varying from 0.4 to 0.6.

The \( E_{ur}/E_{vc} \) ratio is related to OCR. For low OCR values, a fixed value equal to 1.60 is suggested. In general, OCR varies along with the analysis however PARATIE assumes that \( E_{ur}/E_{vc} \) is kept constant equal to the initial value assigned by the user.

No correlations are proposed for Winkler constants.
6.5 Correlations for cohesive soils

PARATIE includes some correlations based on the CPT results as well as on laboratory tests.

6.5.1 Undrained resistance parameters

The undrained shear strength (which should be used only in a simplified undrained analysis) can be computed based on \( q_c \) by:

\[
S_u = \frac{(q_c - \sigma_v)}{N_k}
\]

The cone factor \( N_k \) is between 10 and 20; some studies show a dependency with the plasticity index PI. For a first guess estimate, it is reasonable to use \( N_k = 15 \).

For NC clays, as an alternative, a linear relationship between \( S_u \) and \( \sigma_v' \) is given.

Existing correlations between \( S_u \) and \( N'_{60} \) are not included in PARATIE.

The undrained friction angle \( \phi_u \) is zero, for a saturated soil.

6.5.2 Drained resistance parameters for clay model

A first guess estimate is available by a correlation between \( \phi'_{cv} \) and PI. \( \phi'_{p} \) may be computed by the following equation:

\[
\tan(\phi'_{p}) = \tan(\phi'_{cv}) / 1.5
\]

6.5.3 Undrained stiffness parameters

The elastic modulus \( E_u \) is currently related to \( S_u \) by a linear relationship:

\[
E_u / S_u = k
\]

The \( k \) factor decreases as long as OCR and PI increase. PARATIE includes the correlation after Duncan & Buchigani (1976), related to secant moduli at 50% of the strain at failure (referred to as \( E_{u50} \)). The comments made in section 6.4.2 are still valid here.

6.5.4 Drained stiffness parameters

If the undrained modulus \( E_u \) is known, a first approximation of the drained modulus may be represented by the 80% of \( E_u \).

PARATIE also includes some correlations with \( q_c \) (after Bowles, 1988) as well as some indications for variable moduli \( E = R (p' / p_a)^n \).

In the latter case, for NC clays, typical values of \( R \) are given to be used in a relationship like:

\[
E = R (\sigma' / p_a)
\]

which describes a linearly varying modulus with depth.
6.6 Thrust coefficients

6.6.1 At rest coefficient

For sands, the normal consolidated at rest coefficient can be computed via:
\[ K_{0NC} = 1 - \sin(\phi'_\text{peak}) \]

For clays:
\[ K_{0NC} = 1 - \sin(\phi'_c) \]

The at rest coefficient in an overconsolidated soil can be estimated by
\[ K_0 = K_{0NC} \times OCR^n \]

\( n \equiv 0.5 \) for both sands and clays.

6.6.2 Active coefficient \( K_a \)

The active thrust coefficient \( K_a \) depends on \( \phi' \), on \( \delta/\phi \) and on the uphill soil slope. PARATIE includes a program to calculate \( K_a \) based on Coulomb formulas, which give a reasonable estimate for practical purposes, with respect to other formulas, like the one by Caquot & Kerisel, 1948. The friction angle \( \delta \) between the soil and the wall slightly affects \( K_a \) and it is advised to neglect such effect. The diagram in figure 6-6 can be used to compute \( K_a \) accounting for a sloped soil; for this purposes, the diagrams in appendix G of EUROCODE 7, Part 1 can be used as well.

For undrained behaviour, where \( \phi_u = 0 \), PARATIE automatically sets \( K_a = K_p = 1 \).

6.6.3 Passive coefficient \( K_p \)

\( K_p \) is one of the parameters that mostly affect the results especially in a cantilever or a single anchor wall analysis; like \( K_a \), it is related to \( \phi' \), \( \delta \) and to the ground slope; applicable values may be computed by figure 6-6, or directly by the utility provided by PARATIE. It’s worthwhile mentioning that such values, based on logarithmic failure surfaces, show a good agreement with experimental data; in addition they are more conservative than the values predicted by the Coulomb equations based on planar slip surfaces.

Figure 6-5 proposes a comparison between the two approaches (logarithmic or planar surfaces) is for a horizontal ground surface. When the friction angle is greater than 30° and \( \delta/\phi > 0 \), Coulomb values may be significantly not conservative. Additional references for \( K_p \) calculation can also be found in appendix G of EUROCODE 7, Part 1, where the suggested procedure produces similar values to Caquot & Kerisel theory. Note that all the available

\[ \delta \] Typical values for \( \delta \) are included in Table 6-1; some suggestions are also included in EUROCODE 7, Part 1.

For a concrete wall in a granular soils, \( \delta \) can be at least one half of the friction angle.
approaches are approximate because not all the soil behaviour components are included. For this reason, a great care is recommended when selecting the value for $K_p$ for design purposes.

**Figure 6-6:** $K_a$ and $K_p$ values (after Navfac (1986) DM 7.02)
Table 6-1: Reference values for $\delta$ (after NAVFAC (1986))

### Ultimate Friction Factors and Adhesion for Dissimilar Materials

<table>
<thead>
<tr>
<th>Interface Materials</th>
<th>Friction factor, $\tan \delta$</th>
<th>Friction angle $\delta$ [degrees]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass concrete on the following foundation materials:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clean sound rock</td>
<td>0.70</td>
<td>35</td>
</tr>
<tr>
<td>Clean gravel, gravel-sand mixtures, coarse sand...</td>
<td>0.55 to 0.60</td>
<td>29 to 31</td>
</tr>
<tr>
<td>Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel...</td>
<td>0.45 to 0.55</td>
<td>24 to 29</td>
</tr>
<tr>
<td>Clean fine sand, silty or clayey fine to medium sand...</td>
<td>0.35 to 0.45</td>
<td>19 to 24</td>
</tr>
<tr>
<td>Fine sandy silt, nonplastic silt...</td>
<td>0.30 to 0.35</td>
<td>17 to 19</td>
</tr>
<tr>
<td>Very stiff and hard residual or preconsolidated clay...</td>
<td>0.40 to 0.50</td>
<td>22 to 26</td>
</tr>
<tr>
<td>Medium stiff and stiff clay and silty clay...</td>
<td>0.30 to 0.35</td>
<td>17 to 19</td>
</tr>
<tr>
<td>(Masonry on foundation materials has same friction factors.)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel sheet piles against the following soils:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls...</td>
<td>0.40</td>
<td>22</td>
</tr>
<tr>
<td>Clean sand, silty sand-gravel mixture, single size hard rock fill...</td>
<td>0.30</td>
<td>17</td>
</tr>
<tr>
<td>Silty sand, gravel or sand mixed with silt or clay...</td>
<td>0.25</td>
<td>14</td>
</tr>
<tr>
<td>Fine sandy silt, nonplastic silt...</td>
<td>0.20</td>
<td>11</td>
</tr>
<tr>
<td>Formed concrete or concrete sheet piling against the following soils:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clean gravel, gravel-sand mixture, well-graded rock fill with spalls...</td>
<td>0.40 to 0.50</td>
<td>22 to 26</td>
</tr>
<tr>
<td>Clean sand, silty sand-gravel mixture, single size hard rock fill...</td>
<td>0.30 to 0.40</td>
<td>17 to 22</td>
</tr>
<tr>
<td>Silty sand, gravel or sand mixed with silt or clay...</td>
<td>0.30</td>
<td>17</td>
</tr>
<tr>
<td>Fine sandy silt, nonplastic silt...</td>
<td>0.25</td>
<td>14</td>
</tr>
<tr>
<td>Various structural materials:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Masonry on masonry, igneous and metamorphic rocks:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dressed soft rock on dressed soft rock...</td>
<td>0.70</td>
<td>35</td>
</tr>
<tr>
<td>Dressed hard rock on dressed soft rock...</td>
<td>0.65</td>
<td>33</td>
</tr>
<tr>
<td>Dressed hard rock on dressed hard rock...</td>
<td>0.55</td>
<td>29</td>
</tr>
<tr>
<td>Masonry on wood (cross grain)...</td>
<td>0.50</td>
<td>26</td>
</tr>
<tr>
<td>Steel on steel at sheet pile interlocks...</td>
<td>0.30</td>
<td>17</td>
</tr>
</tbody>
</table>

### Interface Materials (Cohesion)

<table>
<thead>
<tr>
<th>Interface Materials (Cohesion)</th>
<th>Adhesion $c_a$ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very soft cohesive soil</td>
<td>( 0 - 250 psf)</td>
</tr>
<tr>
<td>Soft cohesive soil</td>
<td>( 250 - 500 psf)</td>
</tr>
<tr>
<td>Medium stiff cohesive soil</td>
<td>( 500 - 1000 psf)</td>
</tr>
<tr>
<td>Stiff cohesive soil</td>
<td>(1000 - 2000 psf)</td>
</tr>
<tr>
<td>Very stiff cohesive soil</td>
<td>(2000 - 4000 psf)</td>
</tr>
</tbody>
</table>
6.7 Reference permeability values

For the pore pressure calculation in PARATIE, permeability is needed for each layer. In the following table, some typical values are included for a first trial estimate:

<table>
<thead>
<tr>
<th>Soil nature</th>
<th>permeability [cm/sec]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean gravel</td>
<td>$10^1$</td>
</tr>
<tr>
<td>Sand and gravel</td>
<td>$10^{-2}$</td>
</tr>
<tr>
<td>Clean sand</td>
<td>$10^{-1}$</td>
</tr>
<tr>
<td>Silty sand</td>
<td>$10^{-4}$</td>
</tr>
<tr>
<td>Silt</td>
<td>$10^{-5}$</td>
</tr>
<tr>
<td>Clay</td>
<td>$10^{-8}$</td>
</tr>
</tbody>
</table>

It may be sometimes useful to redefine the permeability in a certain soil region to model, for example, some soil improvement by cement grouting or by the jetgrouting technique. The obtained permeability after improvement obviously depends on the technology with respect to the *in situ* soil nature; it is usually quite difficult to predict in advance the improved soil permeability, whereas an a posteriori measurement is currently needed. However, just for a very first guess estimate, it can be anticipated that a well done jetgrouting improvement in a gravelly soil like the one existing in the Milan urban area, reduces the natural soil permeability by two orders of magnitude at least.
7. Vertical settlement evaluation

PARATIE only analyses the lateral soil-wall interaction; therefore no direct information is given by the program about vertical settlements. However, if the excavation is adjacent to building foundations, an estimate of the foundation settlements induced by the opencut excavation is necessary. General speaking, in these situations, a more general analysis method should be used; however, in spite of its simplicity and high degree of approximation, also PARATIE may give some valuable information to indirectly estimate such results. Based on lateral wall deformations, an estimate of vertical displacements can be attempted using some proposed methods like those reported in Bransby & Milligan (1975), which are based on experimental results and small scale models.

In the following, an alternative very simple approach is outlined, which may be used for cantilever walls in granular soils.

With reference to symbols in figure 7-1, vertical settlements in both downhill and uphill soil region can be evaluated computing the soil volumes subject to lateral movements.

Compute the area \( A \) of the ABC triangle based on the lateral displacements known by PARATIE (linearly interpolate between zero displacement point C and max. deflection point B). Assume that the soil beyond linear boundaries CD e CE remains undeformed. If deformation developed at constant volume, the uphill wedge would preserve its original area: thus, assuming a straight deformed surface between B’ and D, constant volume condition would require that area of \( BB'D = area of ABC = A \), thus:

\[
BB' = 2A / \lambda_m
\]

Similar assumption could be made for EGC wedge in the downhill soil. However, the soil, during deformation, currently increases its volume (dilatancy). Calling \( \psi \) be the angle of dilatancy, between the volumetric strain \( \nu = (\epsilon_v + \epsilon_h) / 2 \) and maximum shear strain \( \gamma = \epsilon_v - \epsilon_h \), a simple relationship holds:

\[
\nu = |\epsilon_v - \epsilon_h| / 2 \quad \rightarrow \quad \epsilon_v = -\epsilon_h (1 - \tan \psi)/(1 + \tan \psi)
\]

Now, assuming a linear deformed shape, the average uphill lateral deformation can be computed by:

\[
\epsilon_h^M = \delta_m / \lambda_m = \text{costante}
\]

then

\[
\epsilon_v^M = -\epsilon_h^M (1 - \tan \psi)/(1 + \tan \psi)
\]

Recalling that the settlements was assumed to linearly vary between B and D, in B, we’ll have the maximum settlement:

\[
u_m = \epsilon_v^M \lambda_m
\]

whereas in D, a null settlement will be computed.

\( \psi \) value to employ in this calculations is the dilatancy at critical state. A first guess value is given by:

\[
\psi = \phi - \phi_{cv}
\]
in which $\phi_{cv}$ is the constant volume friction angle (typically 30° for a sand); $\psi$ cannot be negative.

This engineering approach cannot be generalized: when the soil-structure interaction is complex, a simple solution scheme like the one by PARATIE could be unsuitable to produce a reasonable prediction of the real behaviour; therefore and a more complete analysis tool should be selected.
8. HOW TO MODEL A STRIP FOUNDATION

The calculation of the effective vertical stress $\sigma'_v$ is simply based on the soil layer weights, on the applied surcharge at the round level and on the water table, but it does not account for any deformation of the soil mass.

Initially, the lateral effective stress is given by:

$$\sigma'_h = K_0 \sigma'_v$$ (step 1)

in which $K_0$ is the at rest thrust coefficient. In the subsequent steps, $\sigma'_h$ changes according to the lateral wall deformation.

This approximation becomes too poor if the initial soil stress cannot be reasonably described by such simple scheme, like when a relevant concentrated surcharge exists near to the wall.

![Figure 8-1: equivalent uniform surcharge](image)

In this case, both the vertical and horizontal stress distribution becomes more complex than the simple geostatic stress field.

To include such effects in a PARATIE model, two different approaches are available:

1. If the loaded area is quite far from the wall and the applied surcharge is not too high, a uniform equivalent surcharge can be prescribed, according to the simple scheme in figure 8-1.

2. Otherwise, it can be assumed that the concentrated load produces an additional stress distribution in the soil mass that can be computed by means of the Theory of Elasticity (see Timoshenko & Goodier (1970)). This additional stress field is added to the geostatic field (which depends on $K_0$), to set up the initial stress condition for the analysis.
When using the latter option, in the first step of the analysis, in all the soil elements, in both uphill and downhill side, $\sigma'_h$ is computed by:

$$\sigma'_h = K_0 \sigma'_v \text{ (step 1)} + \Delta \sigma'_{h,\text{init}}$$

where $\sigma'_v$, in this formula, does not account for increment due to the foundation load. $\Delta \sigma'_{h,\text{init}}$ is computed by the elastic formula from the half-space solution with uniform strip load at the free surface. Such value of $\sigma'_h$ is just used to establish initial stress condition for each soil element: the lateral stress may later change due to the wall movements and the $\Delta \sigma'_{h,\text{init}}$ part may vanish.

In the downhill elements, $\sigma'_v$ is not modified by the strip foundation effects, whereas is the uphill soil it is, in the following way.

With reference to figure 8-3, if $\text{ZETA} > \text{ZF}$, no additional vertical stress is considered. Otherwise, a stress increment $\Delta \sigma'_{v,1}$ is computed as follows:

if $\text{ZETA} > \text{ZF} - \text{DY} \tan(\beta)$ then

$$\Delta \sigma'_{v,1} = 0$$

if $\text{ZETA} < \text{ZF} - \text{DY} \tan(\beta)$

$$\Delta \sigma'_{v,1} = \int_{\text{DY}-\text{B}}^{\text{DY}} \frac{QF(y)}{L(y)} dy$$

where

$$QF(y) = \begin{cases} QF & \text{if } y \geq (\text{ZF} - \text{ZETA}) / \tan(\beta) \\ 0 & \text{if } y < (\text{ZF} - \text{ZETA}) / \tan(\beta) \end{cases}$$

$$L(y) = y + (\text{ZF} - \text{ZETA}) / \tan(\beta)$$

The integral is computed by subdividing the integration range into 100 segments.

Now, the obtained value is compared with $\sigma'_v$ given by formula figure 8-2 and the maximum is selected:

$$\Delta \sigma'_{v} = \max(\Delta \sigma'_{v,1}, \sigma'_v)$$

Note that this stress increment decreases with depth.
This formulation assumes a uniform pressure distribution on the loaded area, suitable for the modelling of a very flexible foundation. If the foundation is rigid, the actual pressure distribution is given in figure 8-4. In this case, such not uniform distribution may be approached by subdividing the loaded area into several uniformly loaded sub-areas, like outlined in the cited figure.

Figure 8-3

Figure 8-4: pressure distribution under a rigid foundation
9. HOW TO MODEL A BERM

The dredge line inside the excavation is sometimes modelled to provide a limited soil mass adjacent to the wall, higher than the general excavation depth. Such a soil mass is called “berm” and may be included in the calculation during some intermediate steps. For such purpose, input data to PARATIE may be given according to one of the criteria outlined here below.

Method 1 (Fleming et al., 1992)

Method 2 (Fleming et al., 1992)

Figure 9-1

An equivalent horizontal dredge line is assumed, according to Figure 9-1.

Figure 9-2

A uniform distributed load equal to the berm weight divided by the passive wedge extension $L$ is applied at the excavation level defined neglecting the berm.

Method 3

Figure 9-3

Any one of these modelling criteria can be adopted using standard input procedures included in PARATIE.
An equivalent sloped dredge line based on Figure 9-3 is assumed. Excavation level can be raised up to point A, but passive thrust coefficient must be reduced to account for unfavourable sloping.
10. RESULT ASSESSMENT

At each analysis step, the following results are both printed out on the output file and stored in the result database:

1. NODAL DISPLACEMENT COMPONENTS: i.e. total lateral y displacement and total x rotation.
2. FINITE ELEMENT RESULTS: the issued results depend on each element type
3. REACTIONS AT FIXED NODAL POINTS: lateral reaction (force per unit out-of-plane depth) and moment (moment per unit out-of-plane depth)

Element available results are listed in the following.

TRUSS ELEMENT

1. FORCE : ELEMENT FORCE PER UNIT OUT-OF-PLANE DEPTH (+VE TENSION)
2. STRESS : ELEMENT STRESS

BEAM ELEMENT (see figure 10-1)

1. VA : SHEAR AT FIRST EDGE
2. VB : SHEAR AT LAST EDGE
3. MA : MOMENT AT FIRST EDGE
4. MB : MOMENT AT LAST EDGE
   (all of them, per unit out-of-plane depth)

ELPL ELEMENT (ELASTOPLASTIC SUPPORT)

1. FORCE : ELEMENT FORCE PER UNIT OUT-OF-PLANE DEPTH (+VE TENSION)
2. PLASTIC : PLASTIC STRAIN

WIRE ELEMENT (GROUND ANCHOR)

1. FORCE : ELEMENT FORCE PER UNIT OUT-OF-PLANE DEPTH (+VE TENSION)

CELAS ELEMENT (ELASTIC SUPPORT)

1. FORCE : ELEMENT FORCE PER UNIT OUT-OF-PLANE DEPTH (+VE TENSION)
2. MOMENT : ELEMENT MOMENT PER UNIT OUT-OF-PLANE DEPTH

SLAB ELEMENT (A SLAB BETWEEN TWO WALLS)

1. VA : SHEAR AT FIRST EDGE
2. VB : SHEAR AT LAST EDGE
3. MA : MOMENT AT FIRST EDGE
4. MB : MOMENT AT LAST EDGE
5. AXIAL : AXIAL FORCE
   (all of them, per unit out-of-plane depth)

NODAL VARIABLES

Figure 10-1: BEAM element results
1. YDISPL : LATERAL DISPLACEMENT  
2. XROT : ROTATION  
3. YREACT : HORIZONTAL REACTION  
4. XMOMREAC: MOMENT REACTION

As a first check, it is suggested to carefully assess the initial step results: in this step lateral deflections should be zero; hence, in the soil elements, if no strip foundations have been prescribed, the at rest geostatic conditions should be found: i.e.:

\[
\left( \frac{\sigma'_h}{\sigma'_v} \right)_{STEP \ 1} = K_0
\]

It may be worthwhile controlling the resultant of the stresses in downhill or uphill soil elements: such resulting thrusts are automatically issued in the final report file, as follows:

With reference to figure 10-2, for each stress distribution between elevations \( Z_a \) and \( Z_b \), the following items are computed:

<table>
<thead>
<tr>
<th></th>
<th>Formula</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRUE EFFECTIVE THRUST</td>
<td>( S = \int_{Z_a}^{Z_b} \sigma'_h , dz )</td>
<td>Effective stress resultant over all the soil elements in a group</td>
</tr>
<tr>
<td>WATER THRUST</td>
<td>( W = \int_{Z_a}^{Z_b} udz )</td>
<td>Pore pressure resultant over all the soil elements in this group</td>
</tr>
<tr>
<td>TRUE TOTAL THRUST</td>
<td>( T = S + W )</td>
<td>The sum of the TRUE EFFECTIVE THRUST and WATER THRUST: it represents the overall thrust on the wall</td>
</tr>
<tr>
<td>MINIMUM ALLOWABLE THRUST</td>
<td>( A = \int_{Z_a}^{Z_b} (K_A \sigma'_v - 2c' \sqrt{K_A}) , dz )</td>
<td>It is the minimum thrust from this soil region, if active conditions are fully developed</td>
</tr>
<tr>
<td>MAXIMUM AVAILABLE THRUST</td>
<td>( P = \int_{Z_a}^{Z_b} (K_P \sigma'_v + 2c' \sqrt{K_P}) , dz )</td>
<td>It is the maximum thrust that can be resisted by this soil region, if passive conditions are fully developed</td>
</tr>
<tr>
<td>MAXIMUM/TRUE RATIO</td>
<td>( \frac{P}{S} )</td>
<td>It is the ratio between the passive thrust and current effective thrust: it can be used as a safety factor</td>
</tr>
<tr>
<td>PASSIVE THRUST PERCENTAGE</td>
<td>( = 100 \times \frac{S}{P} )</td>
<td>The actual effective thrust is represented by the percentage of the maximum available resistance</td>
</tr>
<tr>
<td>TRUE/MINIMUM RATIO</td>
<td>( \frac{S}{A} )</td>
<td>It is the ratio between the current effective thrust and minimum soil resistance</td>
</tr>
</tbody>
</table>

![Figure 10-2](image)
If failure conditions are almost reached, as in a limit analysis, soil element pressures and wall bending moments can be compared with hand calculated distributions by traditional methods (see Bowles (1988)).

Pore pressures can be easily reproduced by hand calculations too, because the simple formulas reported in this manual are used by the program as well.

Frequent checks between expected results and PARATIE results are encouraged. Note that the postprocessing features included in PARATIE allow an extensive result assessment in both graphical and numerical depictions.

At the end of each analysis, a summary report is printed out, where information about convergence is outlined. If a not converged solution is detected at a certain step, a very clear warning message is issued. A not converged solution means that, for that step, no satisfactory equilibrium condition has been reached, using the maximum number of iteration allowed (currently 10). Note that PARATIE does not stop if non convergence is reached since a converged (well balanced) solution may be recovered in the next step: however the results for the non converged solution step are unreliable and are not released to the User.

When a not converging solution is detected, the User may try the following actions:
1. increase the maximum number of iterations (note that such remedy is rarely effective, it may be successful in cases where a very flexible wall is inserted in a very stiff soil);
2. change the solution strategy by, for example, subdividing an excavation step into more intermediate excavations;
3. check relevant soil parameters (like $K_p$ etc) and eventually slightly modify them.

When clear failure conditions are reached, a non positive definite stiffness matrix is found by PARATIE. In this case the program suddenly stops.

When NON CONVERGENCE or FAILURE conditions are reached, a true collapse of the wall is very likely to have been encountered. In this case, the input data must be carefully revised since the wall design may require some modifications.

Finally, once again note that soil parameters are currently affected by great uncertainty: therefore, even very satisfactory solution results should be very carefully assessed and discussed with criticism; sensitivity analyses on most relevant and/or uncertain soil parameters are strongly advised, especially when new or non familiar soil conditions are to be dealt with.
11. A DISCUSSION ABOUT SAFETY FACTORS

PARATIE is an engineering tool in which the final design of the retaining wall arrangement is obtained by a progressive refinement of an initial assumed trial configuration. In other words, PARATIE does not directly compute the requested embedment length of a cantilever bulkhead, but provides the User with all the information for a precise assessment on the adequacy of a first guess and any refined embedment.

Provided a reasonable estimate of the soil parameters has been done, the information given by PARATIE are in general more valuable than those available by traditional design methods based on limit equilibrium concepts. Such methods, for example, do not give reliable information about the lateral movements of the wall; on the other hand, limit equilibrium methods currently issue some safety factor of the wall, a parameter that is not directly given by a method like PARATIE.

For simple retaining walls, a safety factor may be defined in several ways (see Bowles, 1988). For example, one may compute the embedment length of a cantilever wall, based on a reduced passive thrust coefficient $K_p$ divided by a safety factor and then use such length as the final length; or, one may compute the minimum length using an unreduced $K_p$ and then prescribe an increased embedment etc. Note that, in the case of walls with several supports, the safety factor in the traditional sense (the ratio of the true embedment to the minimum embedment) may be meaningless.

An in deep discussion about such issues can be found in the most popular Soil Mechanics textbooks; some guidance may be found in Eurocode 7 as well.

For very simple wall schemes (cantilever or single propped walls) a special procedure is included in PARATIE to compute a safety factor in the traditional sense: the procedure is as follows

1. The wall is analysed in all the realistic excavation phases, taking into account the real (unreduced) embedment length;
2. From the last step on, additional steps are solved in which the dredge line is kept constant at the final design depth, but the wall embedment is progressively reduced (upon request, PARATIE automatically removes all the finite elements below a prescribed depth: the User must activate the Find Safety Factor option in dialog box where analysis steps are defined and, at each step, input the cut depth in the $Z_{cut}$ field);
3. Corresponding with the minimum embedment depth, a solution is no long possible which meets both equilibrium and plasticity requirements (PARATIE may fail to converge or may compute a converged solution with unreasonably high deformations).

This way, the User may recognize the minimum wall configuration required for equilibrium.

Collapse conditions may be reached in many alternative ways like:

- increasing a surcharge
- decreasing the soil resistance parameters (increasing $K_s$ and reducing $K_p$ and cohesion).

The resulting safety factors will be different depending on the way they have been computed. It’s worth comparing the results by different methods.

When the wall has two supports at least, no failure is possible only due to limit conditions in the soil. In such cases, to use the limit equilibrium approach like the one outlined above, it would be necessary to include limit conditions for props as well.
In our option, a different approach is more meaningful for such situations. The safety of the wall may be effectively related to a stability number given by the following ratio:

\[
\frac{\text{Passive thrust resultant}}{\text{True thrust resultant}}
\]

This ratio should be computed for the downhill soil region below the excavation level. Such ratio may be revised as a safety factor with respect to limit (passive) condition in the embedded toe. If an unfactored $K_p$ coefficient has been included in the analysis, acceptable values for such ratio may fall within 2 and 3, depending on the problem. If a reduced $K_p$ value has been used in PARATIE, values between 1.5 and 2 may be fairly acceptable.

In the final report, PARATIE issues such ratio at every analysis step.

In the final judgement on the wall safety, the calculation of the safety factor above is not sufficient. A global stability check of the wall plus the soil within a potential slip surface should be performed by means of traditional methods (Bishop (1955), Fellenius (1936), Morgenstern & Price (1965), etc. or complex numerical analyses. (Cundall & Board (1988)).

![Figure 11-1: failure mechanism not detectable by PARATIE](image)

Of course a careful assessment of the ground anchor safety as well as the stability of struts, if any, should be included as well in the final stability report.

When water table lowering has to be dealt with, additional considerations must be explicitly included about:

- stability of the excavation against uplift;
- safety factor with respect to quick conditions (PARATIE gives very useful information for this aspect)
- adequacy of water lowering devices with respect of fine removal from soil composition etc.
12. REFERENCES


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