RIDO 4.20

USER MANUAL

TECHNICAL ANNEXES

PUBLICATIONS

RFL
ROBERT FAGES LOGICIELS

RIDO 4.20

AVERTISSEMENT

Ce document accompagne la version 4.20 du progiciel **RIDO.**

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PRESENTATION OF THE RIDO PROGRAM VERSION 4.20

The RIDO program calculates the elastoplastic equilibrium of retaining walls (diaphragm walls, berlin walls, sheet piles, ...) or piles in various type of soils.

The calculation follows, phase by phase, the sequence of works, because they condition the internal forces particularly due to the irreversibility of the soil behaviour and the incidence of the geometry during the operations (installation of strut and preloading ...).

The elastoplastic calculation of the whole set of elements (wall, soil, struts, anchors) is carried out based on the finite elements model. For hypothesis see (1). The WINCKLER model (2) is satisfactory for sizing a project : this is shown in (3) and other references.

RIDO calculates the forces (soil reactions, tensions in anchors, ...) that minimize the elastic energy of the wall, the struts, the anchors, the soil, with linear conditions :

- equalities for overall equilibrium, bilateral conditions,
- inequalities for unilateral links with soils, anchors, ...

RIDO uses the finite elements method (force model) for the simulation of the wall.

The algorithm of resolution is an original adaptation of the "primal-dual" method applied to quadratic programming (the elastic energy is a quadratic function of the variables), see the reference (4).

Version 4.20 of RIDO presents the following facilities :

The program :

- Simulates excavations in each of the soils limited by the wall,

- Takes into account slopes and berms by their geometrical description,

- Allows modification of soil characteristics in case of back-filling and grounting,

- Can calculate the Ka an Kp coefficients resolving directly the limit equilibrium differential equations of BOUSSINESQ-RANKINE,

- Allows directly introduction of active, static and passive soil pressures for special cases (CULMANN's method for example),

- Accepts water table variations in each soil and also confined and perched water tables,

- Automatically takes into account the hydraulic gradient effect to the apparent soil density,

- Takes into account the application or removal at any moment of surcharges type CAQUOT, BOUSSINESQ, GRAUX, of purely plastic type and user defined,

- Considers that BOUSSINESQ surcharges are linked to the soil state (active pressure, passive pressure, ...) in the same manner as for CAQUOT surcharges.

- However, on option, their additive contribution can be used for control although the principle of superimposing does not apply in the elastoplastic model

- Makes difference if the surcharges are present before or after the wall construction,

- Allows installing, preloading, removing struts or anchors with unilateral or bilateral link with the wall,

- Can calculate (optional) the buckling of sheet piles retained by inclined anchors,

- Allows application or removal of distributed or concentrated loads at any level of the wall,

- Allows definition of elastic links in displacement and rotation with a given structure (floor, ...),

- Allows various boundary conditions at top and toe of the wall,

- Allows modifications in the geometry of the wall during the works (moulding of upper part of the wall, ...),

- Can calculate Berlin walls and discontinuous toe-in walls,

- Considers long term parameters either for the soil and wall (eg. Concrete wall).

- Permits variations of the elastic modulus at any stage,

- Calculates automatically the lack of toe-in, not only in case of equilibrium failure, but also in case of wall displacement exceeding a specified value.

- A landslide can be simulated.

- At any time of the sequence of the physical phases a calculation of test in the limit states can be asked. This one will have no effect on the normal sequence of the phases.

- As well for one seismic calculation as for a conventional calculation in the limit states partial factors can be applied to loads and to geotechnical data

- In semi-automatic mode, permits the calculation of two walls in interaction.

- Allows a calculation and a check according to the Eurocodes 7 and their French adaptation NF P 94-282

From the user's point of view, advantages are :

- Data introduction in free format with simple description language (keywords and data) : the data are number but also expressions with symbolic constants, variables, functions (predefined or user defined), comments, etc ...

- Functions are supplied which allow the automatic access to values situated in EXCEL worksheets with possibility of interpolation.

- An integrated working environment is delivered : it permits to edit the data (editor mode or question/answer mode), to control the data, to evaluate the expressions, to run RIDO, to show graphically the results on screen, to preview the printouts, to control the printing and plotting outputs. It permits also to export the results in text or graphic forms to other applications and also to send it by E-Mail (ZIP compression).

- A complete yet clean presentation of the results.

- The program uses a dynamic allocation of the computer memory so that there is no limits to the amount of data (user has not to worry about number of soil layers, struts, anchors, ... nor it have to give explicitly this numbers).

- The numerical method of resolution of the equilibrium equations gives a stable calculation even for deep wall (over 50 meters).

- Typically a calculation of one equilibrium lasts 10 milliseconds on a recent PC.

- Units can be chosen independently for the data and for the results (practical units with Tonne-forces, S.I. units with Newtons, U.S. units with Pounds),

- The program comes with a graphic user interface (G.U.I.) according to the WINDOWS standard and run on PCs with WINDOWS XP/Vista/7/8

- The vector or bitmap graphics outputs are ready to import in WINDOWS applications as WORD through the "clipboard".

- The program is multilingual : Freanch/English/Spanish.

- The local network version is available.

RIDO V:4.20 requires at least :

A PENTIUM or compatible processor
512 Mo of RAM, 1024 Mo for Vista/7/8
20 Mo free on disk
One USB port
WINDOWS XP/Vista/7/8

REFERENCES

(1) R.FAGES et C.BOUYAT - Revue TRAVAUX oct. 1971, déc. 1971
Calcul de rideaux de parois moulées ou de palplanches. Modèle mathématique intégrant le comportement irréversible du sol en état élastoplastique.

(2) E.WINCKLER - H.Dominicus Prag. 1867
Die Lehre von Elastizitat und Festigkeit.

(3) R.KASTNER, F.MASROURI, J.MONNET et R.FAGES - XIème Congrès international de mécanique des sols et fondations. San Francisco 1985
Etalonnage sur modèle réduit de différentes méthodes de calcul de soutènements flexibles.

(4) G.DUPUIS et A.PROBST - Journal de mécanique Vol. 6 n°1 mars 1967
Structures élastiques avec conditions unilatérales.

(4) J.GIELLY, R.KASTNER, J.MONNET, C.BOUYAT - Colloque Franco-Polonais de Mécanique des sols, Gdansk, pp. 108-117
Calcul élastoplastique des rideaux de soutènements. Comparaison des prévisions et des mesures in-situ.

(6) R.KASTNER - Thèse de Doctorat ès Sciences - INSA LYON 1982
Evcavations profondes en site urbain. Problèmes liés à la mise hors d'eau. Dimensionnement des soutènements butonnés.

(7) F.MASROURI - Thèse de Doctorat - Laboratoire de Géotechnique de l'INSA de LYON 1986. Comportement des rideaux de soutènement semi-flexibles.

(8) F.MASROURI, R.KASTNER - Revue Française de Géotechnique n° 55 avril 1991
Essais sur modèle de rideaux de soutènement ; confrontation à diverses méthodes de calcul.

(9) R.KASTNER, J.FERRAND - Retaining Structures. Thomas Telford, London, 1993
Performance of a cast in situ retaining wall in a standy silt.

(10) F.MASROURI, R.KASTNER - Retaining Structures. Thomas Telford, London, 1993
Anchored flexible retaining walls experiments on models : calculation by the reaction modulus method.

(11) A.MONNET - Revue Française de Géotechnique, n° 66 1994.
Module de réaction, coefficient de décompression, au sujet des paramètres utilisés dans la méthode de calcul élastoplastique des soutènements.

(12) F.MASROURI, R.KASTNER - Underground Construction in Soft Ground. Fujita & Kusakabe 1995 Blakema, Rotterdam
Earth pressure on braced flexible walls - Model tests ans field investigations.

(13) A.KASDI - Thèse de Doctorat de l'Université de Lille 1994
Détermination des paramètres des modèles elastoplastiques a partir de l'essai pressiométrique.

(14) P.SCHMITT - Revue Française de Géotechnique, n° 71 1995
Méthode empirique d'évaluation du coefficient de réaction du sol, vis-à-vis des ouvrages de soutènement souples..

(15) B.SIMON - Revue Française de Géotechnique, n° 71 1995
Commentaires sur le choix des coefficients de réaction pour le calcul des écrans de soutènement.

(16) J.B.KAZMIERCZACK - Thèse de Doctorat de l'Université de Lille 1996.
Comportement et dimensionnement des parois moulées dans les argiles raides saturées.

(17) JOHN N.CERNICA – FOUNDATION DESIGN – John Wiley & Sons 1995.

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RIDO PROGRAM - VERSION 4.20

USER MANUAL

Location of files

The installation procedure permits the choice of the path to the named RIDO420 installation's folder of the RIDO program.

From this last one, the executable files are put in a named RIDO subfolder and the user manuals in a named DOC subfolder.

Also you will find the subfolder BIRIDO which contains typical examples of the calculation of two walls in interaction as well as complement to the user manual.

Example : C:\RIDO420\RIDO contains the executables files
C:\RIDO420\DOC contains

README.TXT	Information and useful advices
LISEZMOI.TXT	idem but in French language
WRIDONOTA.PDF	The working environment user manual
WRIDONOT.PDF	idem but in french language
RID4NOTA.PDF	The present RIDO user manual
RID4NOT.PDF	idem but in French language

The working files (data and results) are by default located in the ..\RIDO420\RIDO folder but they can be placed in any other folder (see the WRIDONOT user manual).

From WINDOWS 7, if RIDO420\RIDO is a subfolder of
C:\Program Files (x86)
the folder of the working files after the installation will be ..\Documents\RIDOdata.

In the particular case of a RIDO installation by copying the files without the help of the installation procedure and in case of not running, perhaps this problem can be solved by the creation of a WINDOWS environment variable named RIDO with its value equal to the path of the executable files folder (C:\RIDO420\RIDO for the above example).
It is obviously advised to use always the procedure of installation of the CD-ROM.

For more efficiency under WINDOWS, now WRIDO.EXE contains the working environment and the calculus module (previously RIDO.EXE).

However if we wish to use only the module of calculation, for example in a personalized chain of treatment, an executable named RIDO.EXE is supplied (it is a launcher which only activates in WRIDO.EXE its calculation part). This last one, in transparently mode, works as the previous RIDO.EXE.

It is supplied also the executable GRID.EXE (WINDOWS console mode application) which allows to obtain graphic HPGL, WMF or DXF files from the results of RIDO.EXE.
Its mode of use is shown on the screen running it without argument from the command line.
It will not be used while normal working with the WRIDO.EXE environment.

Setting up

The installation procedure put one shortcut on the PC desktop and also several other in the “All Programs” menu of WINDOWS.

The normal launching will be made by a double click on one shortcut, however here are useful informations for the “Line command” mode or from the WINDOWS explorer.

WRIDO <without argument or from a shortcut>	: entered WRIDO through the welcome window
double click on WRIDO.EXE	: idem
WRIDO <file name>	: direct to data (*.RIO file) or to visualizations (*.GRI for graphic results, *****.LST for ‘text’ results files)
double click on a *.RIO file	: launching WRIDO for this file
RIDO <without argument>	: entered WRIDO through the welcome window
RIDO <*.RIO file>	: not interactive direct calculation (results in *.GRI, *.LST)
GRID <*.GRI file> <options>	: Obtaining of graphic files *.PLT ,*.WMF or *.DXF
GRID <without argument>	: user manual of GRID

RIDO PROGRAM - VERSION 4.20

DATA INTRODUCTION

The novelties of the version 4.20 from the version 4.02 are written below in blue colour.

GENERALITIES

In following pages the word “line” will be used for one line of a text file treated by a text editor or best with the integrated working environment **WRIDO** which permits the assisted input of the data. See the user manual of **WRIDO : WRIDOnotA.PDF**.

Data are in free format according to the following specifications of the **RIDO** program version 4.20.:

- Data are separated by blanks (any number)

- A line can start with blanks

- Distribution of data on one line is imposed. However, a logical line can be broken in several physical lines. In this case, each continuation line must begin with the sign + followed by a space.

- If the list of data of one line is shorter than the required list, the not-defined part is taken as a sequence of zeros.

- A datum can be numerical as `5.27` or `1.02e-4` (for 1.02 10^{-4}) or an expression as `(5+2)/4.25` or also an algebraic expression with variables and functions as `level+2*tan(pi/4+phi/2).` This last point will be detailled subsequently.

It is possible to insert comment lines that will be printed at the corresponding position in the printouts. These lines must begin with * (asterisk). No limitation of comments.

If comments immediatly follows the title line, they are considered as describing the studied problem and form the subject for a particulary introduction.

A comment indicated by : (and not by *) is a comment for data only and not exists in printouts. Contrary to comments indicated by * a comment indicated by : can be in the final part of a definition line beginning by # (see RIDO-NOT-30 page)

If in the data describing the operations to simulate in a phase of calculation a comment declared by the character * contains a text between two " this one will be reproduced in the graphs of the considered phase.
For the "text" outputs it will be presented as an ordinary comment

The keywords of the example which follows will be clarified in this user manual. Note at the moment that a suite of actions followed by the keyword CAL constitutes a phase of calculation.

Example CAL
```
CAL
*    ”Installation of the first layer of struts”
STR(1) 162.20  2.5  15  -50  4500
CAL
*    ”Excavation to 159.0 m and draining”
EXC(2) 159.0
WAT(2) 158.5
CAL
…….
```

Also to comment the envelope curves concerning only a subset of phases placeits text between the characters [and].
This will be taken into account only in the description of a phase ending with one keyword CAL the second argument of which is 1

Example

```
CAL                              …….
*   ” Excavation to 159.0 m and draining ”
*   [ For the working phases ]
EXC(2) 159.0
EAU(2) 158.5
CAL(0,1)
…….
```

The dynamic allocation of memory used in this program allows to introduce with no limitation soil layers, strut or anchor levels, various sections of the wall... in any required number. However the total number of data can be limited by the computer capacity.
It is not required to enter the total number of entities as the number of struts for example.

However, when a particular number is required, as a strut’s number, it is its ordinal number of introduction in the data.

Data are given in 3 groups : A,B,C.

"A" GROUP

Basic data describing initial state of wall or pile and soil.

A1 : TITLE AND OPTIONS

- One title line (obligatory)

In this line options can be chosen.
Each option, indicated by a letter, must be written between two *. (The character * cannot be used in the title part).
The option order is of no consequence.

Example: `QUAY WALL AREA 4        *FA*`

Seven options are available in **RIDO 4.20** :

A = Boussinesq surcharges are (A)dded to soil pressures following the principle of superposition applicable to elastic states but spread out the plastic states (see B-2-2 annex)

E = The printout results are (E)xtended with the values of limits active and passive pressures. (Warning : the printed line contains 168 characters).

F = Buckling calculus of wall into account the vertical component of inclined anchors (see C-2 annex)

H = The model of (H)OUY (hypothesis of the soil in plastic state of active pressure reached) will be used for the effect of banks described by the keyword EXC.
Warning : The model of HOUY supposes that we expect that the concerned soil is in a state of active pressure (see the note for the SUA order).

L = the preceding number define the useful number of (L)ine in each page of the printout, example : 80L (default : 60L)

M = if it is preferred to use a frequent sign convention: (M)oments and curvatures are of opposite signs.

U = choice of (U)nits independently of the data and the outputs (printouts, plotted graphs,...) with the following rule :

U:xy where x is the input's units code and y is the output's units code.

This codes are :	T	→ practical units	(Tonne Force): default
	N	→ S.I. units	(Newton)
	P	→ U.S.A. units	(Pound)

Example : `WALL NB 101 *120L U:PN*`
In this case the input data are in U.S.A. units but the outputs in S.I. units.

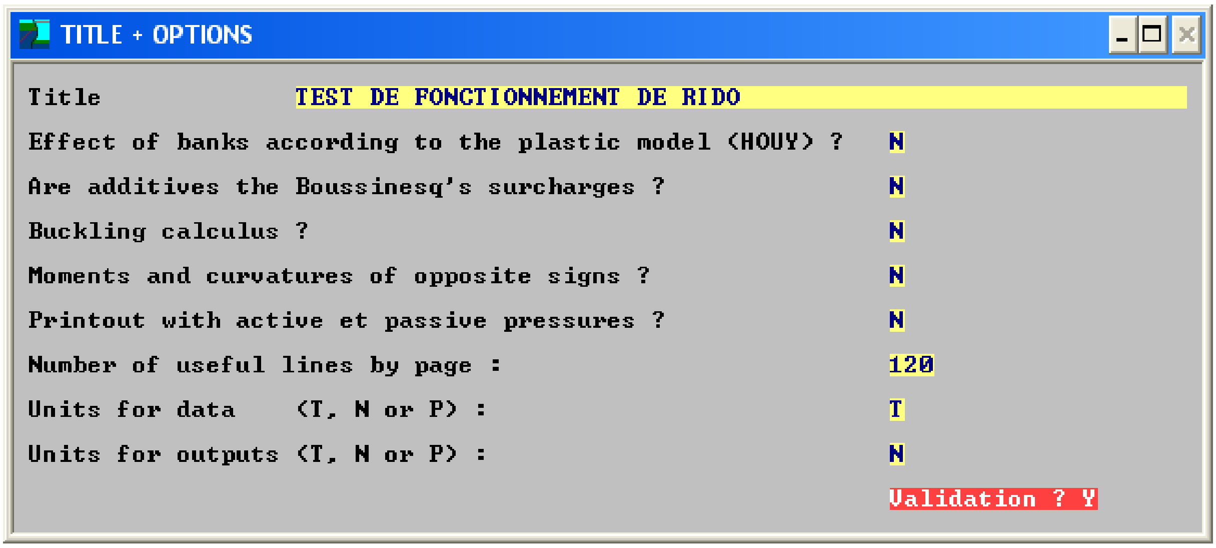
If only one units code is present it is valid for the inputs and the outputs.

Hereafter you see the correspondences of units :

T	N	P	
m	m	Ft	
mm	mm	In	
1/m	1/m	1/Ft	
T	kN	KiP	
T/m	kN/m	KiP/Ft	
T/m^2	kPa	KsF	: Pressure
T/m^3	kN/m^3	KcF	: Vol. density
T/m^3	kPa/m	KsF/Ft	: Elastic rigidity
T/m^3	kPa/m	KsF/Ft	: Cyl.rigidity
$T.m^2/m$	$kN.m^2/m$	$K.Ft^2/Ft$	: EI product
m.T	kN.m	K.Ft	
m.T/m	kN.m/m	K.Ft/Ft	: Moment

The correct output units are also used in the plotting issues and result files.

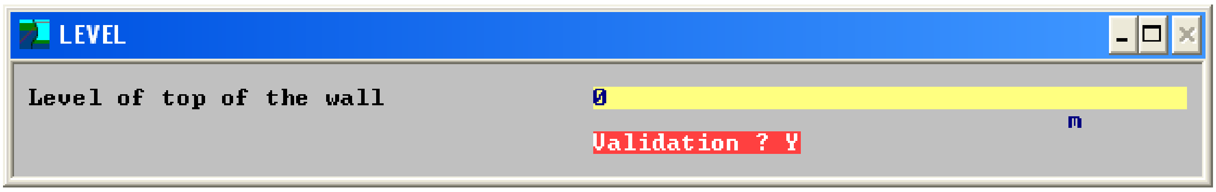
The dialog boxes as the one who follows are presented in this user manual as illustration of the assistants to the introduction of lines of data offered by the WRIDO.EXE working environment.



A2 : THE WALL

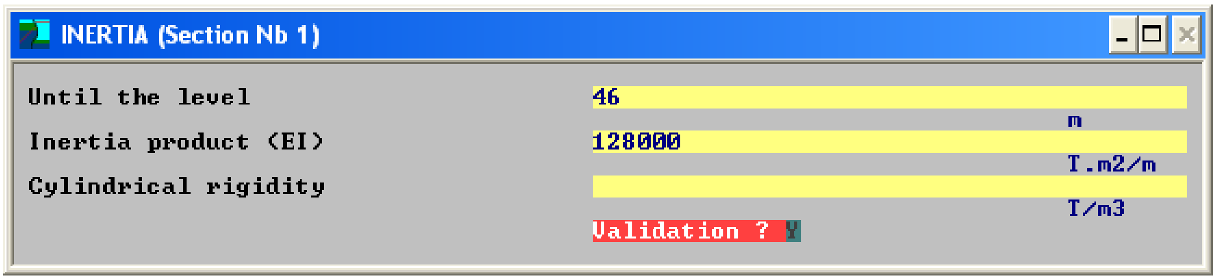
- A first line to define the level (*m*, *Ft*) of the top of the wall

Z_0



followed by a line per each section with varied inertia described from top to bottom

Z EI Rc



where

Z (*m, Ft*) is the level of the end of section
EI (*T.m²/m, kN.m²/m, K.Ft²/Ft*) is the inertia product
Rc (*T/m³, kPa/m, KsF/Ft*) is the cylindrical rigidity (for plane wall Rc=0).

It is possible to give zero inertia sections : that means these parts of wall does not exist at the beginning of the works and will be added at a given time (see A-1 annex).

It is possible to launch a calculation with for all the sections a null inertia. In this case, we calculate an initial equilibrium of the ground before the implementation of the wall : This allows the calculation of the preconstraints in the ground sought, for example, by pre-existent surcharges in the implementation of the wall.

The last line permits the calculation of the height of the wall.

The sequence of the levels Z_0 and Z fix the direction of the axis of levels toward the bottom or toward the top according to the increasing or decreasing of their values.

The calculation zone is defined between the levels Z_0 and the last one Z. Unless otherwise specified all the levels in the data have to be in this zone.

Example :
```
WALL Nb 101 *120L*
165
160.50 18744
151.00 9852
: Height of the wall = 165-151 = 14 meters
```

A3 : THE SOIL.

- One line to fix the initial level of soil (the same for each side of the wall)

Z (*m, Ft*)

- One line for each ground layer (described from top to bottom)

Zc PVw PVs Ka K_0 Kp C φ Da Dp Re Rp

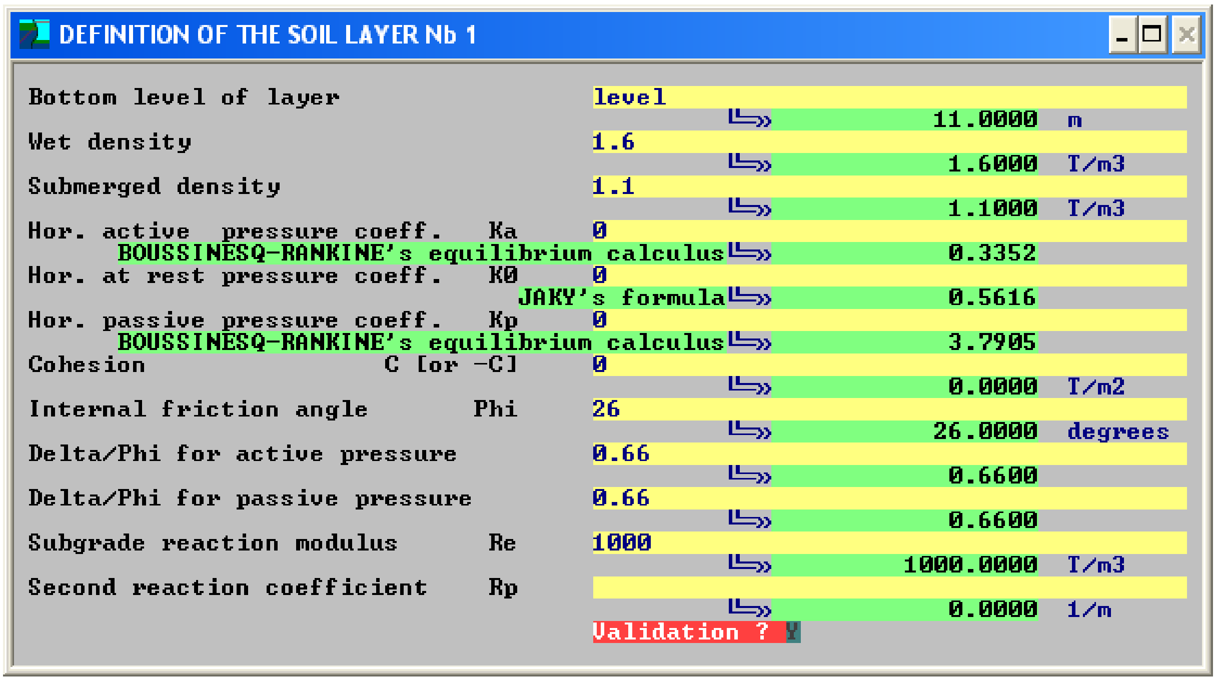
with

Zc (*m,Ft*) : bottom level of the layer

PVw (*T/m*^{3}, *kN/m*^{3}, *KcF*) : wet density
PVs (*T/m*^{3}, *kN/m*^{3}, *KcF*) : submerged density
Ka : horizontal active pressure coefficient
K$_0$: pressure coefficient at rest
Kp : horizontal passive pressure coefficient
C (*T/m*^{2}, *kPa, KsF*) : cohesion
φ (*degrees*) : internal friction angle
Da,Dp : δ/φ with δ for the inclination of stress on wall for active and passive pressure.These values already taken into account in Ka and Kp must be given here for the calculation of the cohesion terms in Caquot's formula.
Re (*T/m*^{3}, *kPa/m, KsF/Ft*) and **Rp** (*1/m, 1/Ft*) : allow the calculation of the subgrade reaction modules Kh (horizontal) at any point where the vertical pressure linked to the earth weight at level **z** is **P(z)** by

$$\mathbf{Kh = Re + Rp * P(z)}$$

If constant coefficient Kh for the ground layer is wanted, then ignore Rp.



The levels **Z** and **Zc** can be upper than the top of wall (level Z$_0$). This description can be useful in case of description of one bank with the EXC keyword.

If in the data Ka = 0 and/or Kp = 0, that indicate a wanted calculus of their values by resolution of the plastic limit equations of BOUSSINESQ-RANKINE integrated in the **RIDO** program.

If in the data K$_0$ = 0, K$_0$ is calculated with the JAKY's formula : K$_0$ = 1-sinφ.

Usually, the subtractive terms in passive pressure and additive terms in active pressure due to cohesion, are calculated by **RIDO** with the CAQUOT formula according to the technical annexes. It is possible to give the values of theses terms directly. To do it, enter the cohesion with the minus sign (which starts this special process) and replace respectively the δ/φ ratios

in passive pressure and active pressure by the subtractive and additive terms for the soil layer parameters concerned.
The printouts are consequently changed.

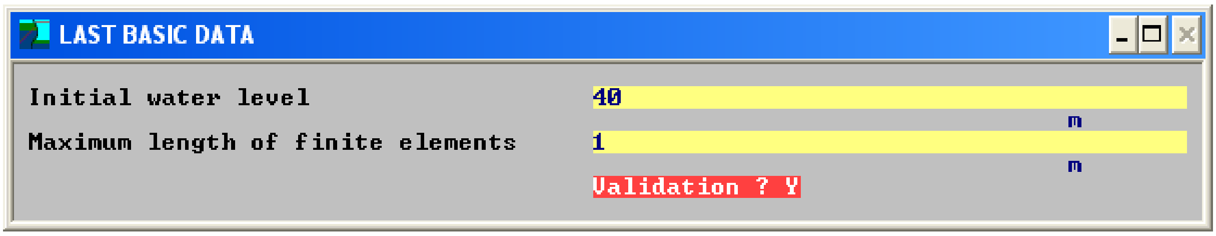
- One line with :

Zh Step

with

Zh (*m, Ft*) : initial water level (if no water table fix Zh under the bottom of the wall)
There is no condition on the value of Zh

Step (*m, Ft*) : is the upper limit specified for the length of the wall elements created by the program (if Step is too small, the maximum number of elements, typically 200, will give their maximum length). The slope (or rotation) of the wall little has to vary on a distance of Step.
Current value is Step = 0.5 m or Step = 1 Ft



"B" GROUP

These data describe work phases and results output control.

Each operation is defined by a keyword followed, eventually by brackets with 1 or 2 arguments and by a list of parameters.
Each keyword, except STOP, is exactly 3 characters long. These characters can be upper or lower case; in this way CAL, cal, Cal are the same keyword.
If the last or the two arguments into brackets are zero they can be neglected :

Ex :

```
CAL(2,0) same as CAL(2)
CAL(0,0) same as CAL
```

The following table contains the keyword's list, their brief description, and the page of this user's manual where they are explained.

To facilitate the working with the multilingual version, keywords can be used equally in their English and French forms.

Is named “phase” the sequence of operations witch terminate with CAL (request of an equilibrium calculus).

Only the useful operations are to describe

Soils are numbered 1 for the left side of the wall and 2 for the right side of the wall
Forces and displacements are positive from soil 1 to soil 2. Moments are positive clockwise (and reverse clockwise with the M option).

GLO : GLOBAL EQUILIBRIUM MODEL

In this version there is an overall equilibrium model for a better calculus : it takes into account the little displacement of all the soil and the wall toward the excavation side.
For example this model finds, in presence of struts with very big stiffness, a soil pressure concentration toward the struts level and a bigger reaction force in struts.

The usual elastoplastic model does not take into account this phenomenon and thus underestimates the forces of struts : this was stated by certain publications

The global displacement is function of the difference of the weights of the left and right soils calculated under the toe of the wall and the subgrade modulus at this same level.

By default, **RIDO 4.20** do not use this new model.
To use it, place the code **GLO** in the first phase and only in this first.
It is not possible to suppress this model on a later phase.

It is possible to modulate the effect of the model with a parameter :

GLO x

Where

x is an incidence factor.

For example **GLO 0.8**
take into account for 80% the overall displacement,

and **GLO**
or **GLO 1.0**
take it into account for 100%.

This choice is clearly indicated on the outputs. The calculated value of this overall displacement appears in each phase results.

If we prefer to model this phenomenon by a movement given of all the soil we can use the order SLI (see page RIDO-NOT-xxx)

LIM : MODIFICATION OF THE LINKS AT TOP OR TOE OF WALL

LIM(s,t)

where

s = 1 at top
s = 2 at toe

t = 0 : free
t = 1 : simple support at last displacement
t = 2 : imposed slope at last value (e.g. : pile embedded in pile cap)
t = 3 : embeddement in last position (displacement and slope)

Top and toe of wall are implicitly free.

If there is an elastic link see order CFM.

COE : ADDITIONAL COEFFICIENTS APPLIED TO THE SOIL PRESSURES

COE Z1 Z2 CO

From level Z1 to level Z2 pressures in soils 1 and 2 are multiplied by CO before being applicated in the calculation to the one meter-wide wall (or one Ft-wide wall).
This can be useful in the following cases :

- Discontinuous toe of wall, where the lower part is periodically absent, then

CO = effective width / period

- Piles for which EI was not introduced per linear-meter of wall, then

CO = effective width of file

- Berlin wall : EI is defined by linear-meter of wall in "A" group and COE is used in the first phase with :

Z1 = level (*m*, *Ft*) of the top of the piles
Z2 = level (*m*, *Ft*) of the bottom of the piles
CO = effective width of files / period

In case of it, in order to take into accept a tridimensionnel effect in the ground, must be taken a supplementary coefficient for the passive pressure state, then write

COE Z1 Z2 CO CB
and Kp is multiplied by CO * CB for the soils between Z1 and Z2.

This COE order modifies also the water pressures and the loads introduced by the LOA order.

At the levels Z1 and Z2 piles or screen are supposed present otherwise or this order will be ignored or Z1 and/or Z2 will be corrected.

PRX : DIRECTLY INTRODUCTION OF SOIL PRESSURES

If it is required to use a different equilibrium plastic limit of soil theory from the BOUSSINESQ-RANKINE theory or to use the CULLMAN'S method in case of non horizontal surface of soil (**RIDO** as a build in model for banks and berms : see the EXC order) the curves of the soil pressures can been directly introduced : for the internal coherence of the elasto-plastic equations of **RIDO** it is necessary to describe the 3 curves : active pressure, at rest pressure, passive pressure.

For each level (linear interpolation between 2 levels) put a line

PRX(n) Z Pa P_0 Pp

where

n is the soil number (1: the left, 2: the right)
Z (*m, Ft*) the level
Pa (*T/m^3, kPa, KsF*) the active pressure
P_0 (*T/m^3, kPa, KsF*) the at rest pressure
Pp (*T/m^3, kPa, KsF*) the passive pressure

In case of discontinuity, do not put two lines with the same level, but

PRX(n,1) Z Pa Po Pp Pa' Po' Pp'

where Pa', Po', Pp' are the second values for the same level.
For the levels outside of the interval defined by a PRX sequence the soils pressures are normally calculated from the weight of soil.

SUX : DIRECTLY INTRODUCTION OF SURCHARGE EFFECT PRESSURES

If surcharge's models build in **RIDO** are unusable, it is possible to enter point by point the additive contribution of surcharges to the three curves of active, at rest, passive pressures by several lines as

SUX(n) Z Pa P_0 Pp

with the same syntax as the PRX order (equally in case of discontinuities).

SUC : CAQUOT TYPE SURCHARGE

SUC(n) Q

where

n : soil number
Q : pressure (*T/m*^{2}, *kPa*, *KsF*) on the free horizontal surface of soil n

To remove a previous surcharge give Q = 0
Successive Caquot surcharges are not cumulative (i.e. a new one on the same soil replace the actual one).(see B-2-1 annex)

SUB : BOUSSINESQ TYPE SURCHARGE

SUB(n) Z A B Q

where

n : soil number
Z : level (*m*, *Ft*) of load strip
A : smallest distance (*m*, *Ft*) from strip to wall
B : largest distance (*m*, *Ft*) from strip to wall
Q : uniform loading (*T/m*^{2}, *kPa*, *KsF*) on strip parallel to wall.

This order makes all Boussinesq surcharges in soil n to be replaced by the new one.

Note : from the version 4.20 the level Z can be above the highest point of the definition of the products of inertia in the initial data ("A" group).

If one requires an addition of a new surcharge

SUB(n,1) Z A B Q

then the previous surcharges are kept in place.

To suppress one elementary surcharge Q write

SUB(n,1) Z A B Q’ with an opposite load (Q’ = -Q)

To suppress all Boussinesq surcharges write only

SUB(n)

Ii is possible to use the theory of the images with a coefficient included between 1 and 2 ::

SUB(n,1) Z A B Q CI

Where

n is the soil number
r is equal to 0 or 1 as previously
CI (1 <= CI <= 2) is a coefficient applied to the effect of Boussinesq surcharge in the soil n

If **CI** is missing or < 1 then CI=1

With Boussinesq surcharges existing before the wall is set, the soil is influenced by this surcharges on each side of the axis of the future wall.

In this situation you will have to calculate an equilibrium of the soil without the wall, and then to put it (INE order). In this case put CI=2, according to the image theory for a correct initialization of stress at zero displacement.
Reminder : an absence of screen is defined by one or several null products of inertia in initial data ("A" group).

If option A is defined (in the title line), Boussinesq surcharges are distributed loads, additives and simply superposed to the soil pressures without taking into account the its state : it is the traditional way of calculation.
In the other case, the stress on the wall due to a Boussinesq surcharge is given by

$$k * S / 0.5$$

with S is the stress given by Boussinesq formula and k = Ka,K_0 or Kp according to the soil state (plastic active, elastic or plastic passive state)
So, there is a continuity between Boussinesq type and Caquot type surcharges.
This is an innovation of **RIDO** program from its version 3. (see B-2-2 annex).

SUA : 'ACTIVE PRESSURE' TYPE SURCHARGE (norme NF P94 282 -Annexes informatives)

Note : This model of calculation supposes the state of active plasticity reached on the height of its effect as well as the constancy of the angle of internal friction PHI.
Of more the principle of superimposing which requires linear equations does not apply in plastic state.
If these conditions are not respected the calculation will however be made but with warning messages.
Finally the equilibrium of RANKINE is implicitly used in this model. If the ration DELTA / PHI is not null he can have a certain incoherence between this model and the results of RIDO .
In fact the theoretical effect will be multiplied by (Ka Boussinesq-Rankine) / (Ka Rankine).

SUA(n,r) Z A B Q

where

n : soil number
r : same parameter as for the SUB order
Z : level (*m*, *Ft*) of load strip
A : smallest distance (*m*, *Ft*) from strip to wall

B : largest distance (*m, Ft*) from strip to wall
Q : uniform loading (T/m^2, *kPa, KsF*) on strip parallel to wall.

This order makes all surcharges of this type in soil n to be replaced by the new one.

The level Z can be above the highest point of the definition of the products of inertia in the initial data ("A" group). The used PHI angle will be the one of the ground in this highest point.

SUA( ) has the same syntax as SUB ( ), particularly for the n and r parameters

SUG : SEMI-INFINITE GRAUX TYPE SURCHARGE

SUG(n,r) Z A α β Q

where

n : soil number
r : same parameter as for the SUB order
Z is th level (*m,Ft*)
A is the distance (*m, Ft*) between the wall and the beginning of the semi-infinite surcharged strip
α, β (*degrees*) are the two angles of GRAUX ($\alpha<\beta$)
Q (T/m^3, *Kpa, KsF*) is the uniform loading.

The level Z can be above the highest point of the definition of the products of inertia in the initial data ("A" group).

SUG( ) has the same syntax of SUB ( ), particularly for the n and r parameters

Warning : The model of GRAUX supposes that we expect that the soil of side n is in a state of active pressure (see the note for the SUA order).

SOI : PARAMETER REDEFINING FOR A SOIL LAYER

The line

SOI(n) Z Ki

with

n : soil number
Z : start of modified layer (*m, Ft*)
Ki : actual pressure coefficient (Ka<=Ki<=Kp), that defines the initial state of the soil followed by one line defining a layer as in "A" group, allows to redefine a layer between two levels at one side of the wall. This allows to have different soils at each side of the wall even from the start, or to take into consideration non-horizontal layers.
Grouting of one layer can also be taken into account by this line. (see B-3-1 annex)

If Ki is not indicated then $Ki=K_0$.

If reinitialisation of the active soil pressures with Ki is not needed (when taking into account long-term parameters) use keyword **PLA**.
The **ELA** order permit to modify the elastic reaction modulus only.

SOI(n,r) Z Ki OCR

See below the same meaning of **r, Ki** et **OCR** for the keyword **BAC**

BAC : BACKFILLING

The line

BAC(n) Z Ki

where

n : soil number (backfilled side)
Z : new soil level (*m, Ft*)
Ki : actual pressure coefficient (Ka <= Ki <= Kp) defining initial state of soil

This line will be followed by one line defining the parameters of the backfilled soil and indicating necessarily as end level the previous possibly excavated soil level.
(The definition of a layer of ground is in accordance with the definition of a layer of the group A).

If Ki is not mentioned, then Ki = K_0

The backfilled layer is initialised at the elastic-active pressure limit state for the existing deformation of the wall if Ki = Ka which will be the regular case.

BAC(n,r) Z Ki OCR

where

n : soil number (backfilled side)
r **=** 0 or absentee : the initialization of the pressures of the ground by Ki is for the current displacement of the wall
r = 1 : the initialization of the pressures of the ground by Ki is for the final displacement of the wall (at the end of the phase)
Z : new soil level (*m, Ft*)
Ki : actual pressure coefficient (Ka <= Ki <= Kp) defining initial state of soil will be followed by one line defining the parameters of the backfilled soil and indicating necessarily as end level the previous soil level
OCR (optional) is the over consolidation ratio..

OCR is indicative if **Ki** is not zero**.**

Present **OCR** allows the calculation of **Ki** if the value introduced for **Ki** is zero according to the model of Mayne and Kulhawi [[1]] :

$$\mathbf{Ki = K0\ (OCR)^{\sin(\varphi)}}$$

EXC : EXCAVATION - BANK - BERM

Simple excavation

EXC(n) Z

with

n : soil number on excavation side
Z: new level in this soil (*m, Ft*)

Excavation with berm

EXC(n) Z1 Z2 A B

with

n : excavated soil number
Z1 : excavation level close to wall (*m, Ft*)
Z2 : deeper excavation level (*m, Ft*)
A : width of berm at level Z1 (*m, Ft*)
B : width of berm at level Z2 (*m, Ft*)

If in a further phase there is an excavation to a level Z, with Z between Z1 and Z2, a new A value is calculated corresponding to level B. (see B-4-2 annex).
There is an automatic control of the intrinsic berm stability and an automatic calculation of a new minimal width to obtain the stability. This calculus is run only if there is the parameter 1 in second position in the EXC order.

Example `EXC(2,1) 5 6.5 2 6`
(Here the scale of the levels is supposed increasing downward)

with only EXC(2) the width of the berm is maintened but the passive pressure is recalculated at each level within the berm according to its resistance capacity .

The stability of a berm is calculated at every point of calculation on its height by supposing horizontal lines of sliding.

Note : The version 4.11.04, and only this one, made one very security hypothesis for the calculation of the limit of holding by supposing this limit reached on all the height of the berm..
The version 4.20, as version previous ones, makes this calculation for the pressures effectively mobilized in its height.

Excavation with a bank

The EXC order can be used in a symmetrical way to a berm description. Then **RIDO** automaticaly calculates the correct decomposition in surcharges (CAQUOT and BOUSSINESQ) according to the indications for making this manually. (see B-4-3 ANNEX).
RIDO takes into account of the eventually change of soil and presence of water within the bank.

Note : From the version 4.20 the level Z2 can be above the highest point of the definition of the ground layers (level Zs) in the initial data ("A" group). The layer of ground considered between the levels Z2 and Zs is the same that the one present at the level Zs. In case of multiple layers or within the height of the bank it will be necessary to create a fictitious section of wall with null inertia product so that Z_0=Zs

BER : EXCAVATION WITH PLANKS (BERLIN WALL)

BER(n) Z

where

n : excavated soil number
Z : new level after excavation (*m, Ft*)

This order assumes that, in the first phase the soil's pressures have been modified by the required coefficients with the keyword COE.
These coefficients are then reset to 1 till Z level, in order to take into account the planks just in place.
The soil behind the planks decompressed by the excavation has it state initialised at the limit active pressure elastic state for the existing deformation.
If planks are not placed till the formation level, one must write :

BER(n,r) Z1 Z2

where

n : excavated soil number
r : the degree (0, 1, 2 or 3) of the interpoling polynome for the restoration at 1 of the coefficient applied to the pressures.

Z1 : level of excavation (*m*, *Ft*)
Z2 : lower level of planks (*m*, *Ft*)

If there is also a bank or a berm in the soil number n, it is necessary to complete the BER order with an appropriate EXC order.(see A-3-1 annex).

WAT : MODIFICATION OF WATER LEVEL AND PRESSURES

with hydrostatic pressures :

WAT(n) Z

where

n is the number of the soil the water table is changed
Z (*m*, *Ft*) is the new level of water table
There is no condition on the value of Z

with hydrodynamic pressures :

In this case the resulting water pressure curve is described point by point (linear interpolation between points) by consecutive lines (from top to bottom) as

WAT(n) Z Pw

where

Z (*m*, *Ft*) is the level
Pw (T/m^3, *kPa, KsF*) is the water pressure
In the case of discontinuity, do not put two consecutive lines with the same level, but

WAT(n,1) Z Pw Pw'
where

Pw' is the second pressure.

There is no condition on the values of Z except that they have to be for monotonous progress.

Under the last level defined in this manner, the hydrostatic pression curve is the default.
A pressure Pw with the value -1 is replaced by the hydrostatic pressure for this level.
In this way it is easy to describe perched water tables and confined water.
The hydraulic gradient effect to the soil densities is automaticaly calculated. Particularly a discontinuity of water pressure produce a CAQUOT's surcharge applied at the same level (can be negative for example in the case of an uplift under an impervious layer of soil).

For the levels where the water pressure is zero, it is the wet density of soil which is used in the calculus and the submerged density, if the water pressure is not nul.

Remark
If an uplift is under an invert and not a soil layer, it is necessary to annulate the uplift effect automatically introduced by **RIDO**, with a GRAUX surcharge (SUG) at the level of the invert.

HDC : HYDRODYNAMIC CORRECTION

Obsolete, considering the description of the keyword WAT described above.

Maintained only for compatibility with older **RIDO** versions.

Water pressure is assumed hydrostatic. This assumption can be corrected by a linear distribution given by a set of points. Each point is defined by one line :

HDC Z Q

with

Z : level of angle point (*m, Ft*)
Q : the value of the correction for this point (T/m^2*, kPa, KsF*)

The various HDC lines will be placed from top to bottom.
One line with HDC alone suppresses a previous correction.
If one requires to take into account the variation of apparent weight due to hydraulic gradient, then :

HDC Z Q DG1 DG2

where

Z, **Q** same as above

DG1, **DG2** : (T/m^3, kN/m^3, *KcF*) algebraic difference to applicate to submerged density of soil 1 and 2 respectively. The differences are taken constant from Z of preceding line CHD up to Z of that line.

STR, ANC : STRUTS AND ANCHORS

These keywords are described in a common manner : there are two, only to facilitate the reading.

Placing struts or anchors

STR(k) Z E I F R (strut)
or **ANC(k) Z E I F R** (anchor)

where

k = link code :
k = 0 : bilateral link
k = 1 : unilateral link; the wall frees if it moves towards soil 1
k = 2 : unilateral link; the wall frees if it moves towards soil 2
Z : level (*m, Ft*) of the strut/anchor
E : longitudinal spacing (*m, Ft*) between struts/anchors
I : inclination of anchor (degrees)
F : preload (*T, kN, KiP*)
R : stiffness *(T/m, kN/m, KiP/Ft*)

If in the title line option "F" has been defined there will be a buckling calculation of the wall taking into account the vertical component of the loads in anchors. In this case the sign of I angle is important:
In the usual case of inclination « downwards » it will be positive if the anchor is at the left side , and negative if the anchor is at the right side.

For the phase of implementation of a preloaded strut or anchor (F not = 0) its rigidity will be taken to 0 so that its strength is effectively equal to the preload. The rigidity will be taken equal to its value R in the later phases. It will thus be advisable to define a phase for every implementation of preloaded strut or anchor (see the CAL keyword)

If, when a strut is put in place, there is no contact with the wall, because it is too short, or because it simulates the shrinkage of a floor, then :

STR(k,-1) Z E I D R

where

D is the gap (*mm, in*).

The levels of struts or anchors receive a number in the order of apparition in phases (see C-1 annex).

Modification of the preloading of a strut or anchor bed

STR(0,num) F
or **ANC(0,num) F**

where

num : level number
F : new preload (*T, kN, KiP*)

STR(1,num)
or **ANC(1,num)**

in this case the new preload is the current force in the strut or the anchor (sometime useful for an automatic iterative calculation of the interaction between two walls)

Removing of struts/anchors

STR(0,num)
or **ANC(0,num)**

where

num is the level number removed.

STI : MODIFICATION OF A BED OF STRUTS OR ANCHORS

Modification of the stiffness of the struts or anchors at any phase.
This is mainly planned for struts or floor in concrete in case of long term modification of its Young modulus E.

STI(n) stiff

Where

n = number of the layer (introduction order number)
stiff = new stiffness

It is best to make a specific phase for one or several commands STI

GAP : GAP OF THE ANCHORPOINT OF A BED OF STRUTS OR ANCHORS

Movement of the anchor point of struts or anchors in the course of works.
This is mainly planned for an iterative calculation of two walls in interaction connected by one or several beds of struts or anchors .
Another usage is the consideration of a sliding of a defective anchor point.

GAP(n) gap

où **n** = number of the layer (introduction order number)
gap = axial gap of the anchorpoint (*mm*, *In*)

or also :

GAP(n,1) gap

où **n** = number of the layer (introduction order number)
gap = horizontal gap of the anchorpoint (*mm*, *In*)

gap is signed (negative towards left, positive towards the right)

Safe in case of interactive calculation it is best to make a specific phase for one or several commands **GAP**.

CFM : APPLICATION OF A FORCE AND/OR A MOMENT ON THE WALL

CFM Z F C

where

Z : level of application (*m*, *Ft*)
F : concentrated load (*T/m*, *kN.m*, *KiP/Ft*)
C : moment (*mT/m*, *kN.m/m*, *K.Ft/Ft*)

These values do not cumulate with the existing ones at the same level but replace the lasts : to suppress one force at a given level, then apply a zero value.

If at the same level Z we had placed previously an elastic connection by **CFM (1)** this last one is preserved and only F and C are modified.

If at level Z there is an elastic link, for instance a floor slab, the connection matrix can be introduced with :

CFM(1) Z F C
CFY CFA CMY CMA

so that

DT = CFY * DY + CFA * DA + F
DM = CMY * DY + CMA * DA + C

where

DT : jump in shear force due to link
DM : jump of moment due to link
DY : variation of deflexion at level **Z** from the deflexion at the setup of the elastic connexion (current value of this deflexion if there was already no elastic connection at the same level)
DA : variation of rotation at level **Z** from the rotation at the setup of the elastic connexion (current value of this rotation if there was already no elastic connection at the same level)
F and **C** : same definition as above but often absent.

(see C-2 annex)

LOA : APPLICATION OF A TRAPEZOIDAL LOAD ON THE WALL

LOA Z1 Z2 Q1 Q2

with

Q1 : load (*T/m²*, *kPa, KsF*) at level **Z1** (*m, Ft*)
Q2 : load (*T.m²*, *kPa, KsF*) at level **Z2** (with linear interpolation between Z1 and Z2).

These values do not cumulate with existing ones at the same levels. The new ones replace the existing ones.
Annulling loads can only be done by Q1 = 0 and Q2 = 0.

At the levels Z1 and Z2 piles or screen are supposed present otherwise or this order will be ignored or Z1 and/or Z2 will be corrected.

INE : WALL INERTIA ALTERATION

INE(n) EI Rc

where

n : number of the modified section (numbering starts from top of the wall).
EI : new product of inertia (Tm^2/m, $kN.m^2/m$, KFt^2/Ft)
Rc : new cylindrical stiffness (T/m^3, kPa/m, KsF/Ft)

This order allows estimate of the effects of a Young modules variation with time (long term modules).

If sections with zero-modules have been prepared in "A" data group, we can simulate putting in place of sections of wall by given no-zero values to EI at the right moment. This is only valid for a new section immediately next to an existing section with an inertia different from zero.
So this order allows to perform calculations on a wall in which the sections are cast following the excavation, or to study the case of a wall with an additional part of wall on top during the phases of works.

In these cases the new cast sections are supposed "perfectly vertical" and there will be a discontinuity of the tangent on the neutral axis of the wall.
The wall is supposed to transmit moments, at this point, to the added part.

If a soil is present along the new section its state is initialised by a soil’s equilibrium calculation before this operation. (see A-1-5 annex).

PLA : MODIFICATION OF PLASTIC CHARACTERISTICS OF SOIL

PLA(k) Ka Kp C φ Da Dp

The following parameters of layer of soil number **k** take the new values

Ka : active horizontal pressure
Kp : passive horizontal pressure
C : cohesion (*T/m², kPa, KsF*)
φ : internal friction angle
Da,Dp : δ/φ with δ for the inclination of stress on wall for active and passive pressure.These values already taken into account in Ka and Kp must be given here for the calculation of the cohesion terms in Caquot's formula.

Usually, the substractive terms in passive pressure and additive terms in active pressure due to cohesion, are calculated by **RIDO** with the CAQUOT formula according to the technical annexes. It is possible to give directly the values of theses terms. To do it, enter the cohesion with the minus sign (which starts this special process) and replace respectively the δ/φ ratios in passive pressure and active pressure by the subtractive and additive terms for the soil layer parameters concerned.
The printouts are consequently changed.

This allows to take into account the effect of long term parameters after the end of the works (see B-3-2 annex).

Remark : the soils introduced in "A" and "B" groups with SOI or BAC are defined by a number corresponding to the sequence of their description in the input data.
If in the data Ka = 0 and/or Kp = 0, that indicate a wanted calculus of their values by resolution of the plastic limit equations of BOUSSINESQ-RANKINE integrated in the **RIDO** program.

ELA : ELASTIC REACTION MODULUS MODIFICATION

It is possible to modify the elastic modulus for any calculus phase and this separately on left and right soils.

ELA(n) Z1 Z2 c1

where

n is the soil number (1 for the left one and 2 for the right one)
Z1 et Z2 the levels (*m*, *Ft*) between the elastic modulus is modified
c1 is a multiplier applied to the original elastic coefficients modulus.

It is possible to have a soil layer changing between Z1 and Z2.

It is also possible to write :

ELA(n) Z1 Z2 c1 c2

and between Z1 and Z2 there is a linear interpolation of the multiplicative coefficient between c1 and c2.

During consecutive use of the order ELA for the same levels, it is the last coefficients which play on the module initially introduced into the description of the layers of ground.

In the laws of behavior reaction of the ground / movement the linear part "rotate" around the last point of equilibrium (or on the limit point of null pressure if there is unsticking in the presence of cohesion).

In a same phase there is priority for the ELA order. For example these two sequences give the same result :

```
ELA(2) 5 10 0.63
EXC(2) 5
CAL
```

```
EXC(2) 5
ELA(2) 5 10 0.63
CAL
```

The elastic modulus (modified or original) appears on the listing.

SLI : LANDSLIDE

This order will possibly be to complete by orders DEC if the anchorings of anchors in the soil are also concerned.

SLI Z1 Z2 D1 D2

With :

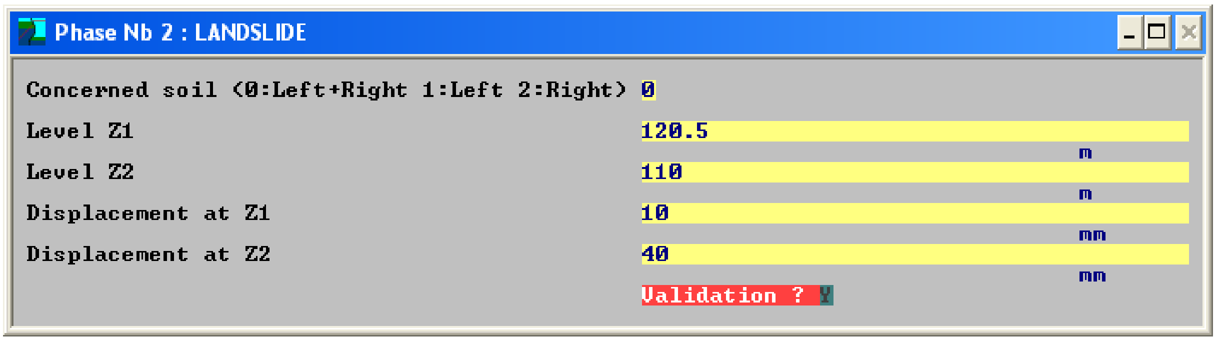
D1 : ground displacement (*mm,Ft/1000*) at level **Z1** (*m*, *Ft*)
D2 : ground displacement (*mm,Ft/1000*) at level **Z2** (Z2 below Z1)
with linear interpolation between Z1 and Z2.

This concerns the left soil and the right soil.

SLI(n) Z1 Z2 D1 D2

With n = 1 for the left soil and n = 2 for the right soil if limited to only one side of the wall.

These displacements cumulate with previous ones at the same levels and are relative to the present state of soil



These displacements mean making a translation according to the horizontal axis of the curves of elastoplastic reactions described in pages RIDO-ANN-7 and RIDO-ANN-8.

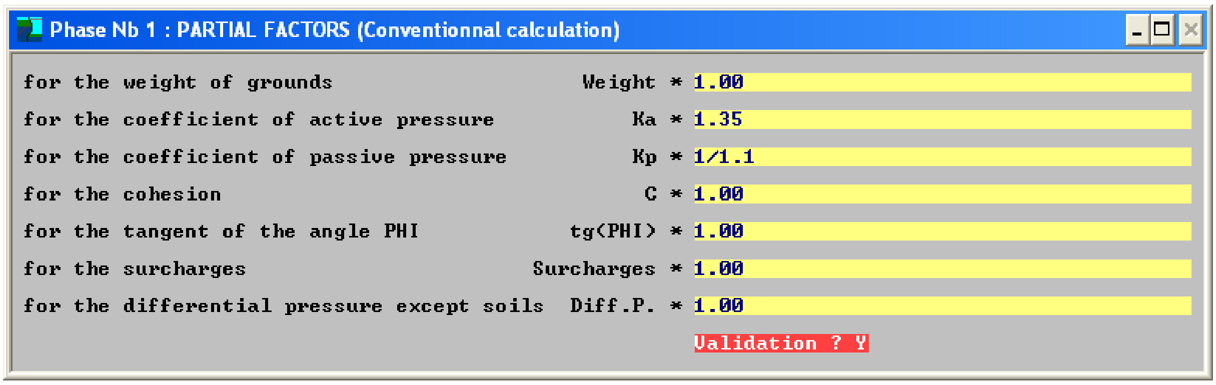
PFA : PARTIAL FACTORS

The order PFA allows to introduce from any phase the partial factors described in the NF P94 282 norm. Warning : this one does not specify if these factors are to be taken into account from the beginning or simply in final phase (the results will be very different because of the evolutionary and irreversible calculation for which the principle of superimposing(overlapping) does not apply!).

Safe for every phase of type TELi t is not possible to redefine new partial factors and this for the reasons evoked above.

PFA F1 F2 F3 F4 F5 F6 F7

F1 : Factor applied to the weight of grounds
F2 : Factor applied to the coefficients of active pressure
F3 : Factor applied to the coefficient of passive pressure
F4 : Factor applied to the eventually cohesions
F5 : Factor applied to the tangent of the angles PHI
(The new angles PHI are accordingly redefined)
F6 : Factor applied to all the surcharges (defined by SUC, SUB, SUG et SUA)
F7 : Factor applied to the differential pressures applied directly on the wall except soils (water pressures, introduced by CHA, etc …)
The concentrated efforts are not concerned.



To avoid inconsistent effects we shall not give values different from 1 simultaneously in certain subsets of factors (for example F2 and F5).

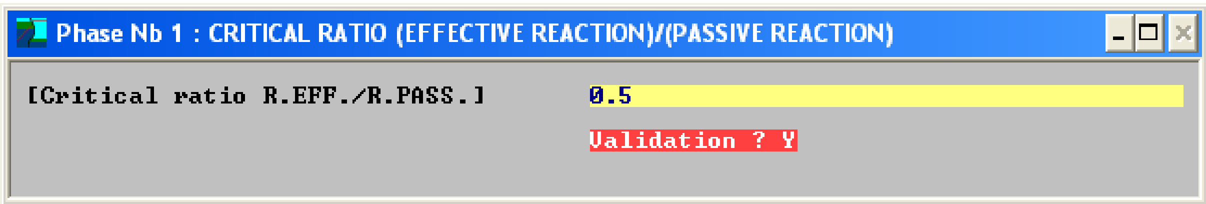
In the ouputs of type "text", if they are different from 1, these factors are systematically specified.

CRR : CRITICAL RATION (EFFECTIVE REACTION)/(PASSIVE REACTION

A critical value can be introduced by the order RRC which can appear in one or several phases:

CRR r

r : critical ratio



It will be used to show alerts in case of overtaking but also in the processing of a phase of type TEL..

Its value will be preserved in phase following ones which would not contain this order and resumption after a phase of type TEL which would have modified it locally.

From the version 4.20 the ratios in question are presented as well in the outputs of type "text" as "graph".

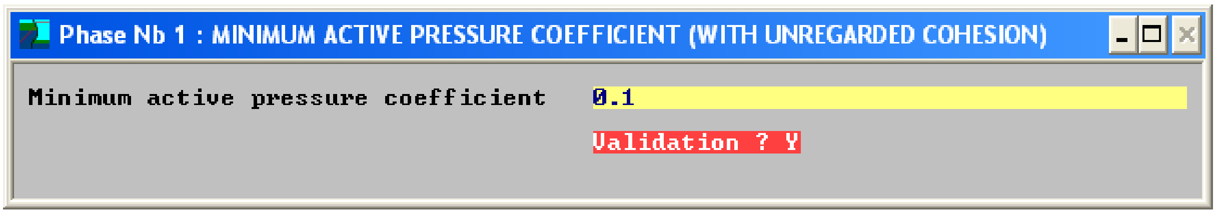
KAM : MINIMUM COEFFICIENT OF ACTIVE PRESSURE IN THE PRESENCE OF COHERENT SOILS (NF P94 282)

The norm NF P94 282 art. 5.1.3.1 (4) + Note 1 invites, by caution, to ignore the possible unsticking ground-WALL and to consider à considérer one minimum pressure with a fictive active pressure coefficient of value 0.1 for a pulverulent supposed soil.

The order **KAM** (for **Ka** **M**inimum) allows to fix any value :

KAM Kam

With **Kam** = 0.1 according to the norm



We can place it in an any phase, even give several values in various phases. Its value is preserved for the phases following one except obviously if it is fixed in one TEL phase. It is however recommended to place him(it) in a TEL phase.
Indeed this security approach can pull unrealistic results because of the evolutionary calculation between the various phases and of the irreversibility of the behavior of the soil.

CAL : CALCULATION AND OUTPUTS CONTROL

One phase can contain several elementary operations, each one being described by one of the previous defined keywords.
When one phase is described, the word "CAL" makes the calculation to be started.

All the elementary operations described in the same phase are considered to happen simultaneously. It is necessary to define enough phases to be sure that the irreversibility of the forces acting in the soil is well integrated.
For instance the introduction of struts or anchors with preloading must be the object of one specific phase (and thus a calculation order must be given).

CAL

gives way to calculation and output of results.

CAL(k)

gives way to calculations and various types of outputs according to :

k = 0 regular output
k = 1 compact output only with important data
k = 2 regular output and semi graphic output on the printer
k = 3 no output of results

CAL(k,1)

has the same meaning but moreover envelops of shear forces and moments till and from this phase will be printed (for the last phase of works this envelop is always given).
The control of the output of these results will be made in the "C" group (STA order).

If a failure is expected because of a lack of toe-in, this value can be calculated and a new calculation performed with a new length of wall.
In this case, write :

CAL(k,r) X Y Z

where

k (0, 1, 2 or 3)**, r** (missing or 1)
X : displacement (*m, Ft*) considered as the allowed limit of displacement beyond which there is a equilibrium failure
Y : upper limit of the lack (*m,* Ft) of toe-in to be calculated
Z : increment to be added to the lack (*m, Ft*) of toe-in when restarting the calculation.

If X = 0 the failure will correspond to the total plastification of soils.

If Z = 0 or not mentioned, the lack of toe-in will be calculated but the calculation will not be restarted.
If X,Y,Z parameters were specified during one phase they will be taken with the same value in the following phases. So they do not need to be repeated in the other CAL lines.

Remark :

If X > 0 the displacement of the wall may perhaps not be reduced by a longer toe-in if the wall is too flexible and in this case the calculation of the lack of toe-in useless. It will be indicated by an information that Y is insufficient.

Possible messages after a calculation

EQUILIBRIUM FAILURE

There is a complete plastification of the soils and the equilibrium cannot be reached.
The results in the form of text presents by the calculated last one deformation the mode of break (fall, uprising of the bottom of excavation,…).

NUMERICAL INSTABILITY

Les équations peuvent devenir très difficiles à résoudre numériquement en raison des approximations dues à la représentation des nombres réels dans l'ordinateur et aux erreurs inhérentes à chaque calcul élémentaire
The equations can become very difficult to resolve numerically because of the approximations due to the representation of the real numbers in the computer and to the errors inherent to every elementary calculation (rounding error). This meets when the product EI of the screen is too low in front of the subgrade reaction modulus of the soil and/or when we are close to the equilibrium failure. In the first case the problem can disappear by decreasing the step of calculation ("A" data group).
This situation, not to confuse with the mechanical instability or EQUILIBRIUM FAILURE, does not meet with realistic sizings.

THEORICAL REQUIRED PRECISION NOT REACHED

The calculation is useful but the precision of the calculation is considered insufficient towards the severe requirement of RIDO. For example the moment calculated in foot free of a wall would be 0.00562 (theoretical value 0.0).
It can mean that we are close to the break of balance because in this case the numeric calculation becomes difficult (see above : NUMERICAL INSTABILITY)..

SOIL INSTABILITY AT LEVEL z

The level **z** is 0.50 m. under the foot of the wall. At this level are calculated the active and passive pressures of soils situated to the left and to the right of the axis of the wall. If the active pressure of one soil is superior to the passive pressure of other this message appears.

It is a warning inviting in an additional check.

- Are we in a situation of global break of equilibrium of the massif ?
  A control with the circles of sliding would be to envisage.
- As it is in the presence of pumping and of the phenomenon of Renard ?
- Was it taken by the securities on the geotechnical data so that the calculation has no sense ? For example the cohesion was neglected ?

TLS : TEST AT THE LIMIT STATES

TLS or **TLS(1)**

The order TLS (for Test at Limit States) to place at the beginning of the description of a phase allows a special processing only for this one.

- The phase in question with its description of conventional loads will have no effect on the calculation of the following phases.

- If several phases of type TLS are consecutive the result of the calculation will resume as the equilibrium of the phase placed just before the first one of type TLS for this sequence.

- If there is instability in type TLS's phase, the lack of toe-in can be calculated with the usual parameters of the order CAL

- If there is stability a calculation of reduction of the height toe-in will automatically be made if we **TLS(1)**

  Two cases are planned :

  a - If the critical ratio of mobilization of the passive pressure defined by CRR is nil the calculated reduced height will correspond of loss of equilibrium

  b - If the critical ratio of mobilization of the passive pressure defined by CRR is not nil, the calculated reduced height will correspond to its reached value.

This situation can be analyzed in the outputs of type "text" where this additional calculation is detailed.

CANTILIVER walls

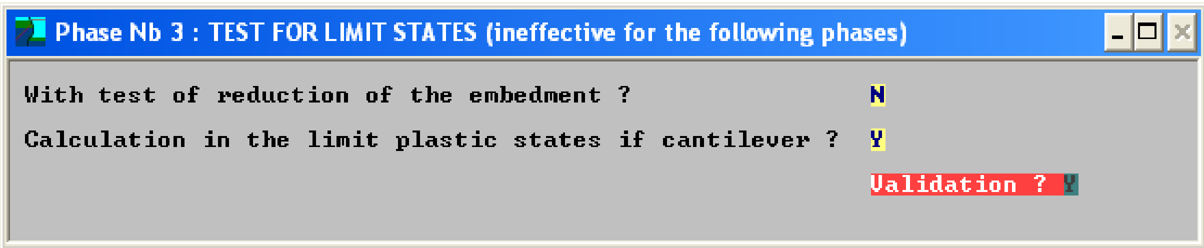
When such a situation is detected and in mind of NF P94 282 are presented the following elements :

- The highest level with nil differential pressure (in the presence of left and right soilss)
- The lowest level with nil differential pressure (the point of pivoting)
- The ratio of mobilization of the passive pressure under this point.

All this is valid in multilayer grounds, with surcharges, bank, berms and in the presence of water.

If we want to make a calculation to the limit plastic states (MEL historical method) :

Put 1 as second parameter of the order TEL : **TEL(0,1)** ou **TEL(1,1).**



In this case the displacement is not calculated and the pressures of grounds are taken as:
- Concerning land side: active pressure to the point of pivoting, partial passive pressure then
- Concerning excavation side : passive pressure up to the point of pivoting, active pressure then.

A system of two equations (moment nil and cross force in foot) is then solved with two unknowns : the level of the point of pivoting and the ratio of mobilization of the partial passive pressure.

The soil can be multilayer, with banks, berms, surchargess, applied forces, in the presence of water as far as the configuration is compatible with the method of plasticity.

The data possibly present in the order CAL will be ignored as well as a request of calculation of lack of toe-in

MEM, RES : STUDY OF A VARIANT FROM A CERTAIN PHASE

MEM is for a memorization and **RES** is for a resumption

Usage :

Put the word **MEM** at the beginning of a phase description. The environment of the last calculus will be memorized (the preceding phase or the initial state if phase number 1).

Put the word **RES** at the beginning of a phase description and this last one will resume the calculus with the previously memorized environment as if the calculations since the MEM had not taken place

Remarks :

It is possible to put several **MEM** orders but the resumption take in account only the last memorized environment.

It is possible to put several **RES** orders : each resumption is a restart to the same last memorized environment (a **RES** order not preceded by **MEM** will be ignored).

If **MEM** and **RES** are not placed at the beginning of a phase description the result will be unpredictable

Example :

<title>
<basic data>
: phase 1
<phase 1 data>
CAL
: phase 2
<phase 2 data>
CAL
: phase 3 first variant
MEM
<phase 3 data>
CAL
: phase 4 first variante
<phase 4 data>
CAL
: phase 5 second variant resumption of calculus end of phase 2
RES
<phase 5 data>
CAL
: phase 6 second variant
<phase 6 data>
CAL
: phase 7 third variant resumption of calculus end of phase 2
RES
<phase 7 data>
CAL
FIN
STOP

END : END OF CALCULATION

END

stops the calculations and outputs.

"B" group is now ending with the word END

"C" GROUP

This group eventualy missing controls with keywords (one per line) some printouts and outputs on files.

GRF : SEMI-GRAPHIC CURVES ON PRINTER

Use of characters to show on the printouts the different curves.
Useful when a graphic printer is not available. Very rare case because any WINDOWS compatible printer is graphic!
Printing only of the curves for the phases selected with CAL(2) and for all the phase if there is not any selection.

STA : GLOBAL STATISTICS OUTPUTS ON PRINTER

On the printouts:

- Envelop’s curves of moments and cross forces
- Historic of forces of struts and anchors
- Maximum displacements and moments

EVP : WRITING AN « ENVELOPES » FILE

In order to facilitate the retaking of the results for a scrapping calculation (diaphragm walls), a readable file in FORTRAN, C as in BASIC, is created with the following attributes:

- its name : <name of the RIDO date file>.EVP

- its type : text file with « , » and CR-LF as separators

- its content :
    . 1st line : the number of calculated points, the numbers of the 1st phase and the last concerned and a code according to the units (1:practical units, 2: SI units, 3:USA units).
    . one line for each point : abscissa, mini and maxi shear force, mini and maxi moment.
    . possible repetition of the above mentioned according to the **RIDO** data (duplication in the .EVP file of the envelopes given on the printer) which allow the coming out of the envelopes for several phases.

Remar*k* : as the points can be discontinued points for the envelope-curves, two following points can have the same abscissa.

In the case of the user want to run a program made for an old **RIDO** version, the RIDON.EVP file is output in the old format if it is was created a WINDOWS environment variable of name RIDONEVP ann of value OLD.

The units are always the practical units.

With value OLD+UNITS the format will be the old one and the units are the same as the printed **RIDO** outputs.

ASC : WRITING RESULTS IN AN « ASCII » FILE

Useful if it is made a data processing from the **RIDO** results.
The created file has the name <name of the RIDO data file>.ASC
In the case of the calculation of two walls in interaction this file will be even created without this ASC order.

It contains one element per line :

- the title (60 characters)
- the user's name (23 characters)
- the calculus date (8 characters)
- the code of the used units (1:practical units, 2:S.I. units, 3:USA units)
- the number N of calculus points
- the index corresponding of the top of the wall (generally 1)
- N levels (2 consecutives values can be equal for the discontinuities)
- for each phase :
    - the index of the upper level with presence of the wall
    - the index of the lower level with presence of the wall
    - the levels of excavation at left and right of the wall
    - the water table levels at left and right
    - N values of displacements (in mm or in inch)
    - N values of moments
    - N values of cross forces
    - N values of pressures other than soil pressures
    - N values of the left soil pressures
    - N values of the right soil pressures
- before the physical end of the file, two consecutive zeros.

XLS : WRITING RESULTS IN AN EXCEL FILE

The created file has the name **<name of the RIDO data file>.XLS**
We find, at the rate of one phase by sheet, the detailed numeric results identical to the printed outputs

EXP : WRITING RESULTS OF THE LAST PHASE IN AN « ASCII »

The text file named <name of the RIDO data file>.EXP is created. It contains the data and results of the last phase to chain with another application.

STOP : END OF DATA

Remark : if there is not "C" group, it is sufficient to put the line with **END** of the 'B' group.

SYNTAX OF THE EXPRESSIONS IN THE DATA

If the data file has .RIO as extension, it is possible to use expressions with the following rules :

- Each numerical datum can be replaced by an algebraic expression :

Example: `ANC(1) 5 2 0 0 2e6*(pi*4e-2**2/4)/15`

where

the anchor rigidity is calculated (diameter 4cm, length 15m)
pi is a predefined constant
** is the integral power (use ^ for real power or resulting from expressions. There is also the pow() function).

- You can use the **predefined functions** of the C language or specific to RIDO (this ones are the **internal functions** described on page RIDO-NOT-33) and also **formula functions**.

To **define a formula function** put the # character at the beginning of a line and use formal parameters :

Example:

```
# rigid(diam,len)=2.e6*(pi*diam**2/4)/len
anc(1) 5 2 20 0 rigid(4e-2,12+9/3)
```

Formal parameters are local and can have the same name as the other items with no problem.
We can use in the formula constants, variables as well as other functions provided that they are defined at the time of the call.
The actual parameters are any expression. Omitted at the end of list them will be taken for 0.0. This will be valid also for the internal and external functions.

- It is possible to use **external functions** written in C/C++ , FORTRAN or VISUAL BASIC language (and also other programming languages if they produce standard DLL dynamic link libraries)

Example: `# elast(a,b,c)=@elastx.exe`

In this case a C language program has been compiled to the executable file elastx.exe.
The @ is here to indicate a file (here ELASTX.EXE). If the file name and/or path contain(s) spaces put it within ".
The calling name in the data language will be elast(…,…,…).

On the folder ..\RIDO there is the file EXFONC.C which is an example in C/C++ language with the instructions for making external functions. Same with the file EXFONC.F90 for the FORTRAN language and EXFONC.VB for VISUAL BASIC.

Example with a dynamical library : `# elast(a,b,c)=@elasty.dll`

This supposes that a program, in a language allowing it, was compiled in a dynamic library named elasty.dll.
The @ is here to indicate a file (here ELASTY.DLL).
The calling name in the data language will be elast(…,…,…).

On the folder ..\RIDO there is the file DLFONC.C which is an example with the instructions for making external functions in DLL forms. This example is written in C/C++ language but it is useful as example for other programming languages.

See the user manual WRIDONOT.PDF NOTE 2 for the statement of an interactive method of check of the good running of these external functions.

- You can **define and modify variables** in lines beginning with #:

Example: `# level0=1 esp=2` : two defined variables
`# level0=level0+1` : one modified variable
`ANC(1) level0+3 esp 20 0 rigid(2e-2,15)`

The space is a separator but it is not one within the ( ).
After = you can put any expression.

- You can **define constants** :

```
# Es==2.e6
# rigid(diam,len)=Es*(pi*diam**2/4)/len
```

Note the double sign =.

Unlike a variable, once defined the value of a constant, we cannot modify it any more.

- Functions, variables and constants can be used in any expression in lines situated after their definitions (also within formula functions definitions).
Their identifier (or name) has to begin with a letter followed by letters or by digits.
Their names are case sensitive.
Not it's better to use as names of the keywords.

- Logical expressions

The logical operators are :

!	for the negation
?	?<expression> is 0.0 if <expression> is null and 1.0 otherwise
||	for the relation OR
!!	for the relation exclusive OR (XOR)
&&	for the relation AND

The comparison operators are :

<	less than
<=	less or equal than
>	great than
>=	great or equal than
==	equal
!=	not equal

Within a logical context one expression or one variable is taken as 0.0 if its value is nul and as 1.0 otherwise.

Examples :

```
# x=5 y=0 z=-3
# u = x<7                    : u will be worth 1.0
# u = x<=y                   : u will be worth 0.0
# u = x==y                   : u will be worth 0.0
# u = x||y                   : u will be worth 1.0
# u = x!!z                   : u will be worth 0.0
# u = x!=1                   : u will be worth 1.0
# u = (x > z) && (x < 10)    : u will be worth 1.0
# u = !(x > z)               : u will be worth 0.0
# u = !x                     : u will be worth 0.0
# u = ?x                     : u will be worth 1.0
```

Note that the space is not a separator if it is attached to the sign =

Here is an example of one formulae function using logical expression :

```
# foo(x,x0,a,b)=(x >= 0)*(a*x + (x > x0)*(b-a)*(x-x0))
```

It is a question of one continuous, linear function by pieces, null if x is negative, of slope a if 0 <= x <= x0 and of slope b if x > x0.

The conditional operator `? \`

It is about a ternary operator answering the following specifications :

```
<exp>?<exp1>\<exp2>
```

It takes the value of <exp1> if <exp> is true and the value of <exp2> otherwise

Example :

```
# ssx(x) = x==0?1\sin(x)/x
```

or in a equivalent way because of the logical interpretation of a numerical expression:

```
# ssx(x) = x?sin(x)/x\1
```

This operator is associative to the right :

`z<0?-1\z>1?1\0` is the same as `z<0?-1\(z>1?1\0)`
which value is –1 if z < 0, 1 if z > 1 and 0 otherwise.

- Automatic variables

These variables of identifiers %1,%2,%3, etc. have for values respectively the evaluations of the expressions of the line containing them (%x: expression number x, the first one is %1).
Provided that there is no cyclic reference these variables are valid in all the expressions of the current line.
For example the internal function chad (C, Phi) give the coefficient of subgrade reaction of the ground according to its cohesion C and of its angle of friction Phi.
We can write for a line of description of a soil (so we shall avoid a redundancy of the data and their coherence will be guaranteed) :

```
…
Level+3.5 1.6 1.1 0 0 0 5 26 0 -2/3 chad(%7,%8) 0
```

Equally %%1, %%2, %%3, etc…are variables which their values are equal to the automatic variables %1, %2, %3, etc… of the previous line with expressions.

Example :

```
Level+3.5 1.6 1.1 0 0 0 5 26 0 -2/3 chad(%7,%8) 0
%%1+2 1.6 1.1 0 0 0 0 35 0 -2/3 chad(%7,%8) 0
```

The automatic variable %% situated in p position is equivalent to %%p

Exemple :

```
Level+3.5 1.6 1.1 0 0 0 5 26 0 -2/3 chad(%7,%8) 0
%%+2 1.6 1.1 0 0 0 0 35 0 -2/3 chad(%7,%8) 0
```

- Putting **the ! character followed by a space at the beginning of a line** give the list (on the printouts) of the functions, variables and constants with their definition or present value. We can put a comment after !.

- If **a line of data is too large**, you can split it into several lines. To indicate a **continuation line** put a **+ followed by a space** at the beginning of this line:

Example:

```
# preload=50
ANC(1) (level0 + 3) esp 20 -preload
+ rigid(2e-2,15)
```

- **Constants in data file can be dynamically exported** within another data file.
The **constants selected to be exported** must have a name beginning with the '_' character.
When the constants are imported they have two '_' at the beginning of their new name.

Example :

In the data file WALL10.RIO

_RefLevel==57.25

In the data file WALL12.RIO

CONST=@WALL10.RIO

...

exc(2) __RefLevel-2.40

...

The directive # CONST permits the importation of all the exportable constants from a file named after @. If the file name and/or path contain(s) spaces put it within ".

The values of this constants can be calculated by the regular rules of the expressions in RIDO data files.

It is possible with a text editor to create a file with only constants.

Its file name must have the extension .CST and contain lines with constant identifier, spaces and numerical value. All other text wil be interpreted as comments.

Example :

File YOUNG.CST

Contents | Young Modulus
| E_STEEL 2.0e5 Mpa
| E_CONCRETE 0.4e5 Mpa

The import within a data file for RIDO is made by :

CONST=@YOUNG.CST

The rule of the constants names beginning with '_' is not applicable here.

A file *.CST is first searched in the working folder and then in the folder ...\RIDO (executables of RIDO installation folder).

The delivered file ARCELOR.CST contains the EI products of the sheet piles distributed by ARCELOR-MITTAL. Same for the HOESCH.CST file.

- **The predefined constant pi** equals **л**

- **The predefined constant g** equals 9.81

- **The exportable constant _U** permits to know the choice of the units in a data file with the codification :

1 : practical units
2 : S.I. units
3 : U.S.A. units

- The named variable PHASE is automatically created. Its value is the phase number in witch this variable is in use. It is usable in the data particularly when calling external functions. Its value is internally modified but is a constant for the user.

- La predefined constant Px equals 1 if the data file name begins with P1 or p1, equals 2 if this name begins with P2 or p2 and 0 otherwise. Useful for the calculation of two walls in interaction.

- The expressions and their calculated values are **showned at the top of the printouts**.

- The integrated environment WRIDO permits the complete control of validity of these expressions.

DESCRIPTION OF THE INTERNAL FUNCTIONS OF THE DATA LANGUAGE

The arguments and the results are real numbers in double precision
It does not there take place to distinguish integer and real : 5 or 5.0 are identical.

`abs(x)`

Returns the absolute value of **x**

`min(x,y)`

Returns the minimum value between **x**, **y**

`max(x,y)`

Returns the minimum value between **x**, **y**

`floor(x)`

Returns the biggest integer value less or equal to x

`ceil(x)`

Returns the lowest integer value great or equal to **x**

`sin(x)`

Returns the sinus of **x** in radian

`asin(x)`

Returns the angle in radian the sinus of which is **x**

`cos(x)`

Returns the cosinus of **x** in radian

`acos(x)`

Returns the angle in radian the cosinus of which is **x**

`tan(x)`

Returns the tangent of **x** in radian

`atan(x)`

Returns the angle in radian within **−π/2,π/2** the tangent of which is **x**

`atan2(x,y)`

Returns the angle in radian within **−π,π** the tangent of which is **x/y**

`exp(x)`

Returns **e** exponent **x**

`log(x)`

Returns the natural logarithm of **x**

`log10(x)`

Returns the base10 logarithm of **x**

`sinh(x)`

Returns the hyperbolic sinus of **x**

`cosh(x)`

Returns the hyperbolic cosinus of **x**

`tanh(x)`

Returns the hyperbolic tangent of **x**

`sqrt(x)`

Returns the square root of **x**

`hypot(x,y)`

Returns **`sqrt(x*x+y*y)`**

`pow(x,y)`

Returns **x** exponent **y**

`fmod(x,y)`

Returns the rest of the integer division of **x** by **y**

THE FOLLOWING FUNCTIONS ARE SPECIFIC TO RIDO

`d_r(x)`

Return to degrees the angle x expressed in radians

`r_d(x)`

Return to radians the angle x expressed in degrees

`bool(x)`

Returb 1 if x is not nul and 0 if x is nul (for the logical expressions)

<u>Access to the results of RIDO</u>

`result(f,p,z,r)`

Returns the result of a calculation of RIDO read in an ASCII file obtained by the option ASC (in the "C" data group)
Returns 0.0 if the file *.ASC do not exists.
Take this point in account at the first iteration of the calculus of two walls in interactions.

Arguments :

f : integer : 1 or 2 -> indicates the RIDO data file Px*.RIO, résults in Px*.ASC
integer : 0 -> name of the default data file (present calculus) or defined by the directive

```
# RESULT=@<file name>.ASC
```

(the file name can contains its path otherwise it is looked in the current data folder. If the file name and/or path contain(s) spaces put it within ").

p : integer : phase number

z : level (must be a calculated point)

r : 1 -> displacement (in meter or foot)
: 2 -> moment
: 3 -> cross force
: 4 -> differential pressure without soil pressures
: 5 -> left soil pressure
: 6 -> right soil pressure
: 7 -> concentrated moment for linear meter (at z level)
: 8 -> Strut/anchor horizontal force for linear meter (at z level)

The returned value and the level z are in the units of the current data while the results of calculation in the *.ASC file are in their own units.

This function replaces the old external function RIDORESU.EXE and can be used directly in the data expressions.

Functions of access to EXCEL worksheets

The concerned file XLS must beforehand have been declared by the directive :

```
# XLS=@<file name>.XLS
```

If the file name and/or path contain(s) spaces put it within ".

This directive can be to renew for the access to several files.

The file name can contains its path otherwise it is looked in the current data folder.
These functions Returns 0.0 in case of error of resolution. For example the value is not a number, missing or not corresponding.

`xlc(n,l,c)`

Returns the numerical value which is present for the sheet number **n** in the cell of line **l** and column **c.**

`xlx(n,li,lv,x)`

Returns for the sheet number **n** the linear interpolation linked to the argument **x.**
The line **li** contains the monotonous values (increasing or lessening) where **x** is looked and **lv** is the line of interpolated values.
The beginning of the concernedconsecutive columns is automatically looked from the column number 1.

`xly(n,ci,cv,y)`

Returns for the sheet number **n** the linear interpolation linked to the argument **y.**
The column **ci** contains the monotonous values (increasing or lessening) where **y** is

looked and **cv lv** is the column of interpolated values.
The beginning of the concerned consecutive lines is automatically looked from the line number 1.

For example :

```
# XLS=@wall5.xls     : résults of RIDO for wall5.rio
                     : written in an XLS file
…
# dis=xly(4,1,2,90.2)
```

`dis` will be the displacement of the wall in mm for the phase 4 at the level 90.2 meters.

`xlxy(n,x,y)`

Returns for the sheet number **n** the linear interpolation for a rectangular array, **x** is looked within the first line wich contains a monotonous scale and **y** within the first column wich contains a monotonous scale, the array of the values being down and to the right of these scales

Here is for example the sheet n° 2 of the present file **`Caquot Kerisel.xls`** in the folder ..\RIDO :

Exemple d'accès depuis RIDO à un tableau à double entrée						
Example of access from RIDO to a double entry table						
Extrait des tables de Caquot-Kerisel pour DELTA/PHI = 2/3 (Feuille 2)						
Extract of the tables of Caquot-Kerisel for DELTA / PHI = 2/3 (Sheet 2)						
Valeurs du coefficient de poussée active Ka (BETA est l'angle du terrain naturel)						
Values of the coefficient of active pressure Ka (BETA is the angle of the natural ground)						
	PHI -------->	25	30	35	40	
	BETA/PHI					
	0	0,364	0,3	0,247	0,202	
	0,4	0,422	0,352	0,291	0,239	
	0,6	0,468	0,395	0,329	0,271	
	0,8	0,546	0,469	0,397	0,33	
	1	0,879	0,822	0,756	0,683	

```
#XLS = @ "Caquot Kerisel.xls"
...
#Phi=32 Beta=15
#Ka=xlxy(2,Phi,Beta/Phi)
```

Note the " because the name of file contains a space.
Ka is calculated by linear interpolation on the sheet n° 2 ($\delta/\varphi=2/3$) for $\varphi=32$ and $\beta/\varphi=15/32=0.46875$. Ka will be equal to 0.3417

The values are automatically looked in the sheet without indication of numbers of line or column.

Obtaining of the numbers of sheets, lines and columns in the form of named constants

To facilitate the configuration of the data the directive CONST can be used under the following shape:

One XLS file being defined,

`CONST=&xlsWS()` : Recovery of the names of sheets as identifiers of constants with as values their order number .
Identifiers of constants : **w_ZZZ** (ZZZ -> got back text)

`CONST=&xlsLI(nf,li,co)` For the sheet of N° **`nf`**,
recovery of all the contents of type text of the cells of the line N° **`li`**, from the column N° **`co`** as identifiers of constants with as values their number of column.
Identifiers of constants : **`c_XXX`** (XXX -> got back text)

`CONST=&xlsCO(nf,li,co)` : For the sheet of N° **`nf`**,
recovery of all the contents of type text of the cells of the column N° **`co`**, from the line N° **`li`** as identifiers of constants with as values their number of line.
Identifiers of constants : **`l_YYY`** (YYY -> got back text)

If the got back texts contain incompatible characters with an identifier of constant these are replaced by '_'.
In this search the cells which are not of type text or space are ignored.

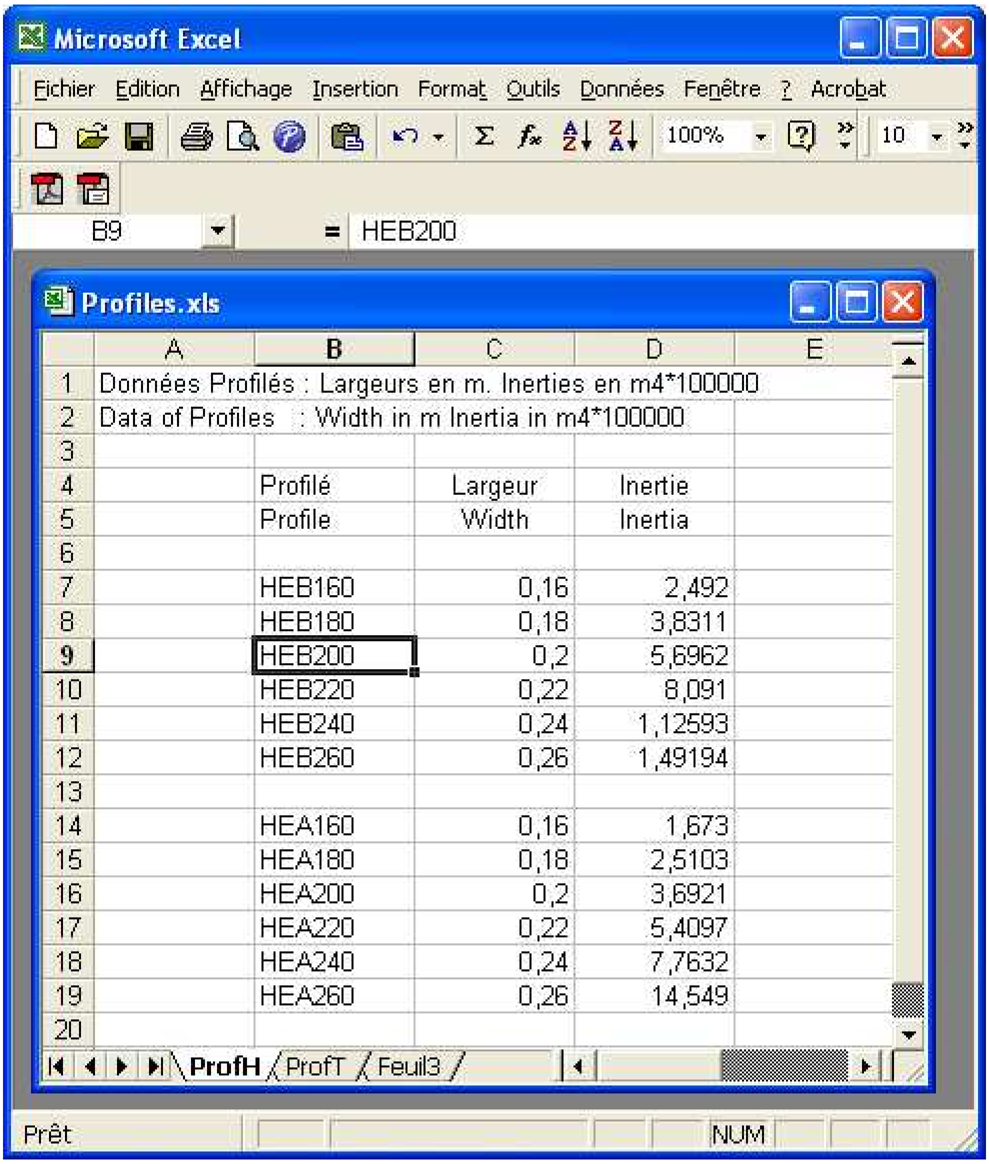
If we write :

`CONST=&xlsLI(nf,li,co,nc)` or **`CONST=&xlsCO(nf,li,co,nc)`**,

the search will limit itself in **`nc`** constants.

Example :

Profiles.xls is the following Excel file :



	A	B	C	D	E
1	Données Profilés : Largeurs en m. Inerties en m4*100000				
2	Data of Profiles : Width in m Inertia in m4*100000				
3					
4		Profilé	Largeur	Inertie	
5		Profile	Width	Inertia	
6					
7		HEB160	0,16	2,492	
8		HEB180	0,18	3,8311	
9		HEB200	0,2	5,6962	
10		HEB220	0,22	8,091	
11		HEB240	0,24	1,12593	
12		HEB260	0,26	1,49194	
13					
14		HEA160	0,16	1,673	
15		HEA180	0,18	2,5103	
16		HEA200	0,2	3,6921	
17		HEA220	0,22	5,4097	
18		HEA240	0,24	7,7632	
19		HEA260	0,26	14,549	
20					

and the following directives are :

```
…
# XLS=@Profiles.xls
# CONST=&xlsWS()
# CONST=&xlsLI(w_ProfH,5,3) CONST=&xlsCO(w_ProfH,7,2)
..
```

we obtain in particular the following constants:

`w_ProfH` de valeur **1**
`c_Largeur` de valeur **3**
`c_Inertie` de valeur **4**
`l_HEB200` de valeur **9**
`l_HEA160` de valeur **14**

and so one in the language of description of the data we shall have a coherent configuration as this one:

```
# Prof=l_HEB200 : only this parameter will be to modify
```

and use of the functions

`xlc(1,Prof,c_Width)` and **`xlc(1,Prof,c_Inertia)`**

FOR ALL THE FUNCTIONSWHICH FOLLOW IT IS AUTOMATIC ADAPTATION TO THE CURRENT UNITS OF THE DATA

Functions connected to the strength of materials

`EIpr(E,e)`

Returns the inertia product **EI** to the linear meter according to the module of Young of the material constituting the wall and of its thickness **e** in meter according to the formulae :

EI = E * e^3 / 12

`Rcyl(E,e,R)`

Returns the cylindrical rigidity of a circular wall of radius **R**, of thickness **e** and the material of which has a module of Young **E** according to the formula

Rc = E * e / R^2

`Rigid(E,d,l)`

Returns the rigidity of an anchor or a strut of diameter **d**, of useful length **l** constituted by a material of module of Young **E** according to the formula :

K = E*S/l with S = π*d^2/4

Functions connected to the soil mechanic

`Kabr(φ,Da)`

Returns the horizontal coefficient of active pressure **Ka** according to the angle of internal friction **φ** in degrees and of the ratio **Da**=**δ**/**φ** in active pressure by resolution of the equations of equilibrium of Boussinesq-Rankine

`Kpbr(φ,Dp)`

Returns the horizontal coefficient of passive pressure **Kp** according to the angle of Internal friction **φ** in degrees and of the ratio **Dp**=**δ**/**φ** in passive pressure by resolution of the equations of equilibrium of Boussinesq-Rankine

`Aac(C,φ,Da)`

Returns the subtractive term applied to the active pressure according to the formula of Caquot (formula 1 of the page RIDO-ANN-5) function of the cohesion **C**, of the angle of internal friction **φ** in degrees and of the ratio **Da**=**δ**/**φ** in active pressure.
C can take a negative value because abs(**C**) is used within this function.
If **`φ=0`** the returned value is -2*abs(**C**).

Apc(C,φ,Dp)

Returns the additive term applied to the passive pressure according to the formula of Caquot (formula 3 of the page RIDO-ANN-5) function of the cohesion **C**, of the angle of internal friction **φ** in degrees and of the ratio **Dp**=**δ/φ** in passive pressure.
C can take a negative value because abs(**C**) is used within this function.
If **φ=0** the returned value is +2*abs(**C**).

4 FUNCTIONS WHICH PRECEDE WILL BE USEFUL TO PARAMETRIZE DATA WITH SAFETY FACTORS TO VERIFY THE SPECIFICATIONS OF THE EUROCODES

Example :

```
# sC=1.0  : safety factor for the cohesion
# sKa=1.0 : safety factor for the active pressure
# sKp=1.0 : safety factor for the passive pressure
# Da=1/3  : DELTA/PHI for the active pressure
# Dp=-2/3 : DELTA/PHI for the passive pressure
12.50 18 10 sKa*Kabr(%8,Da) 0 sKp*Kpbr(%8,Dp) –sC*20 35
+ sKa*Aac(%7,%8,Da) sKp*Apc(%7,%8,Dp) chad(%7,%8)
```

for one ground layer with φ=35° and C=20 kPa and use of the function chad() for the subgrade reaction modulus (note the sign - for the datum cohesion).

If we wish that the safety factors do not concern the phases of works then we shall use one or several orders FLU in phase final as.

```
FLU(4) sKa*Kabr(%4,Da) sKp*Kpbr(%4,Dp) –sC*20 35
+ sKa*Aac(%3,%4,Da) sKp*Apc(%3,%4,Dp)
```

By modifying the values of sC, sKa and sKp we shall obtain the diverse variants of calculation required by the Eurocodes

K0jaky(φ)

Returns the coefficient of earth pressure at rest **K0** function of the internal friction angle **φ** in degrees from the first formulation of Jaky(1944) which is not empirical according to [[2]] :

K0jaky(φ) = (1-sin(φ))*(1+2/3*sin(φ))/(1+sin(φ))

K0brick(φ)

Returns the coefficient of earth pressure at rest **K0** function of the internal friction angle **φ** in degrees from the bricks model useful for stiff clays according to Simpson(1992), see [[3]]

K0brick(φ) = (√2-sin(φ))/(√2+sin(φ))

tzgC(C,φ,Da,Dp,dr) and **tzgG(Ka,Kp,dr)**

These two functions are to be collectively used to obtain the coefficients Re and Rp defining at every level the subgrade reaction modulus of a soil, according to a model inspired by Terzaghi and used in the U.S.A. See for example [[4]].

Re = tzgC(C,φ,Da,Dp,dr)

where **C** = the cohesion (in the current units)

φ = the internal friction angle in degrees
Da = δ/φ for the active pressure
Dp = δ/φ for the passive pressure
dr = the displacement in mm or in inch (if U.S.A. units) necessary to pass from active pressures to passive pressures (this one is of the order of 15 to 30 mm, of one inch according to Terzaghi, and depends on the nature of the soil).

`tgzC( )` returns 0 if C=0 can be replaced by 0 in this case.

`Rp = tzgG(Ka,Kp,dr)`

where
Ka = the horizontal active pressure coefficient
Kp = the horizontal passive pressure coefficient
dr = same definition as for tzgC( )

Reminder: the subgrade reaction modulus **Kh** will depend on the level z according to

Kh = Re + Rp * P(z)

where **P(z)** is the weight of grounds at the level z (considering the possible presences of water, surcharges, bank, berm, etc.)

Here are the expressions of these two functions:

```
TzgC() = (Cp+abs(Ca))/Dr
TzgG() = (Kp-Ka)/Dr
```

where :
- **Ca** and **Cp** are the subtractive and additive terms in the active and passive pressures due to the cohesion
- **Ka** and **Kp** are the active and passive
- **Dr** is **dr** but in meter or foot

These values are obtained from the functions Aac(), Apc(), Kabr() et Kpbr()

For this two functions, if the argument **dr** is null or absent, it will be taken for 25.4 mm or 1.0 inch.

Here is a typical usage in a line defining a soil :

```
level 18 10 0 0 0 20 35 0 -2/3 tgzC(%7,%8,%9,%10,15) tgzG(%4,%6,15)
```

or more practically using two formula functions :

```
# Re(dr)=tzgC(%7,%8,%9,%10,dr) Rp(dr)=tzgG(%4,%6,dr)
…
…
level 18 10 0 0 0 20 35 0 -2/3 Re(15) Rp(15)
```

As the values of Ka and Kp are 0, these coefficients will automatically be calculated

by resolution of the equations of Boussinesq-Rankine before their use in tgzG ()
.

chad(C,φ)

Returns the subgrade reaction modulus by not linear interpolation of the abacus of Chadeisson
For this use it is necessary to take Rp=0 because, from the model of Terzaghi, the values of the subgrade reaction modulus were averaged on the height of the part of the wall under the excavation.
Indeed for certain programs of élastoplastic calculation this coefficient must be constant.
See the paper of A. Monnet [[5]] which analyzes its genesis and the particular context.

Here is a typical usage in a line defining a soil :

```
level 18 10 0 0 0 20 35 0 -2/3 chad(%7,%8) 0
```

monC(C,φ,Dp,dr) and **monG(γ,K0,Kp,EI)**

These two functions are to be collectively used to obtain the subgrade reaction modulus Re. Rp must be 0 for the same reason that with the chad() function.
The presence of EI results from an estimation of the movement of the foot of the wall necessary to give a constant mean value to the subgrade reaction modulus.
It is the sum of these two functions that will give Re (monC () is useless if C=0 and can be omitted in this case) according to the formula of the paragraph 3.6 of the paper of A. Monnet [[5]] :

$$\text{MonC()} = \text{Apc}(C,\varphi,Dp)*\text{th}(C/Co)/Dr$$
$$\text{MonG()} = [20*EI*[Kp*\gamma*(1-K0/Kp)/Dr]^4]^{1/5}$$

avec

Apc()	the function of the additive term due to the cohesion defined previously
C	the cohesion
Φ	the internal friction angle in degrees
Dp	δ/φ for the passive pressure
th()	the hyperbolic tangent function
Co = 30 kPa	
Dr	the characteristic displacement in meter
dr	as argument **Dr** in mm or inch
EI	the inertia product of the wall
γ	the soil density
K0	the coefficient of earth pressure at rest
Kp	the horizontal coefficient of passive pressure

The characteristic displacement **dr** will be by default 15 mm if it is 0 or absentee during the call. The presence of the density **γ** limits the usage of this formula to a soil with only one layer. Furthermore it is implicitly supposed that the subgrade reaction modulus sees its effect limited to the ground situated under the excavation.
Seer Delattre, Luc (2001) [[8]]

Here is a typical usage in a line defining a soil ::

```
# EI=13486
```

```
…
level 18 10 0 0 0 20 35 0 -2/3 (monC(%7,%8,%10)+monG(%2,%5,%6,EI)) 0
```

or more practically using one formula function :

```
# EI=13486
# monnet(ei,dr)=monC(%7,%8,%10,dr)+monG(%2,%5,%6,ei)
…
level 18 10 0 0 0 20 35 0 -2/3 monnet(EI) 0
```

Note the value 0 for Rp indispensable to the coherence of the custom(usage) of this formula and the absence of the argument **dr** which means taking 15 mm.
As the values of K0 and Kp are 0, these coefficients will be calculated automatically respectively by the formula of Jaky and by the resolution of the equations of Boussinesq-Rankine before their use in monG ().

`balay(Em,α,a)`

Returns the subgrade reaction modulus Re using the formula of Balay
(see [[6]] et [[8]])
For this use it is necessary to take Rp=0.

$$\texttt{balay()} = Em / (0.5 * \alpha * a + 0.133 * (9 * a)^{\alpha})$$

where **Em is the pressuremetrics modulus of Ménard using the current units**
α is the rheologic parameter
a is the dimensional parameter (in meter or foot)

`schmitt(Em,α,EI)`

Returns the subgrade reaction modulus Re using the formula of Schmitt
(see [[7]] , [[8]] et [[9]])
For this use it is necessary to take Rp=0.

$$\texttt{schmitt()} = 2.1 * (Em / \alpha)^{4/3} / (EI)^{1/3}$$

where **Em is the pressuremetrics modulus of Ménard using the current units**
α is the rheologic parameter
EI is the inertia product of the wall in the current units

Technical references relative to the internal functions connected to the soil mechanics

[[1]] Mayne, P.W. and Kulhawy, F.H. (1982). “K0-OCR relationships in soil”. Journal of Geotechnical Engineering, Vol. 108 (GT6), 851-872.

[[2]] Radoslaw L. Michalowski (2005). "Coefficient of Earth Pressure at Rest". Journal of Geotechnical and Geoenvironmental Engineering, Vol. 131, No. 11, November 1, 2005.

[[3]] Simpson, B. (1992). "Retaining structures : displacement and design" Géotechnique, 42(4) : 541-576.

[[4]] U.S. Army Corps of Engineers (31 march 1994) "Design of sheet piles walls" Engineer Manual 1110-2-2504.

[[5]] Monnet, A. (1994). “Module de réaction, coefficient de décompression. Au sujet des paramètres utilisés dans la méthode de calcul élastoplastique des soutènements” R.F.G n° 66 – 1994

[[6]] Balay, J. (1984). "Recommandations pour le choix des paramètres de calcul des écrans de soutènement par la méthode aux modules de réaction". Note d’information technique. LCPC 1984.

[[7]] Schmitt, P. (1995) "Méthode empirique d’évaluation du coefficient de réaction du sol vis-à-vis des ouvrages de soutènement souple". Revue Française de Géotechnique n°71.

[[8]] Delattre, Luc (2001). "Un siècle de méthodes de calcul d'écrans de soutènement : L'approche par le calcul les méthodes classiques et la méthode au coefficient de réaction" Bulletin du LCPC, n°. 234

[[9]] NF P94-282 (mars 2009) "Calcul géotechnique, Ouvrages de soutènement" ICS 93-020

SIMPLE EXAMPLE OF DATA FILE WITHOUT ANY EXPRESSION

```
TEST WITH Phi=35 and C=4 FOR THE SECOND LAYER *80L*
0 4
46 128000
3
11 1.6 1.1 0.42 0.50 5.00 0. 26 0 –0.66 1000
60 1.8 1.1 0.26 0.44 8.24 4. 35 0 –0.66 10000
40 1.
*Different soil at left side of the wall
SOI(1) 11
14.5 1.6 1.1 0.42 0.50 5.00 0 26 0.75 0.75 1000
SUC(2) 4.8
CAL
 : PRELOADED ANCHORS
   ANC(2) 4 2.7 30 45 407
   CAL(2)
   EXC(1) 8
   CAL(2)
ANC 7.5 2.7 30 50 900
CAL(2)
EXC(1) 15
CAL(2)
   ANC 14.5 2.7 -30 50 900
   CAL(2)
   EXC(1) 18.5
   WAT 30
   CAL(2)
END
STA
XLS
STOP
```

RIDO 4.20

USER MANUAL

TECHNICAL ANNEXES

PUBLICATIONS

ROBERT FAGES LOGICIELS

THE MODELLING OF RETAINING WALLS OR PILES

A-1 ***ELASTICITES***

A-1-1 Case of a solid retaining wall

The program **RIDO** considers a portion of the retaining wall of 1 meter wide and supposes this sample reproductible all along the retaining wall in such a way this portion performs as an unit strip of 1 meter wide.
In the vertical direction, the retaining wall may comprise the sections in which its EI inertia products (per metre run of wall) and K_c cylindrical stiffness differ.

For a straight wall, $K_c=0$ while $K_c>0$ in the case of a cylindrical wall.

In the case of a retaining wall made of solid material (diaphram wall) of thickness e, and Young's modulus E forming a cylindrical wall of R radius:

$$Kc = \frac{Ee}{R^2} \qquad \text{if} \quad e << R$$

If the cylindrical wall is composed of sheet piles, the calculation of K_c is complex and resulted from the study of the lateral compression of a sheet pile. In the calculation, a cylindrical wall of large diameter is assimilated to a straight wall connected to fictitious elastic supports of evently distributed K_c stiffness.
This hypothesis and the numerical stability (see RIDO-NOT-25 page) will be guaranteed only if R is big enough so that Kc does not become too big with a relatively low product EI.

The normal strength n along a circumference will be deducted of the displacement y calculated by the formula :

$$n = E * y / R$$

A-1-2 Case of a wall with discontinuous embedment

For example, this case is the wall known as the "trouser legs" where the toe is distinuous. For this discontinuous embedded portion, an equivalent section with the inertia related to the per meter run of wall (average inertia between the solid parts and the null inertia of the empty parts) will be taken account of.

A-1-3 <u>Case of the wall known as « berlin walls »</u>

The soldier pile stiffness per metre run of wall will be given as

$$EI = \frac{(EI)_{pieu}}{d} \qquad \text{see figure 1}$$

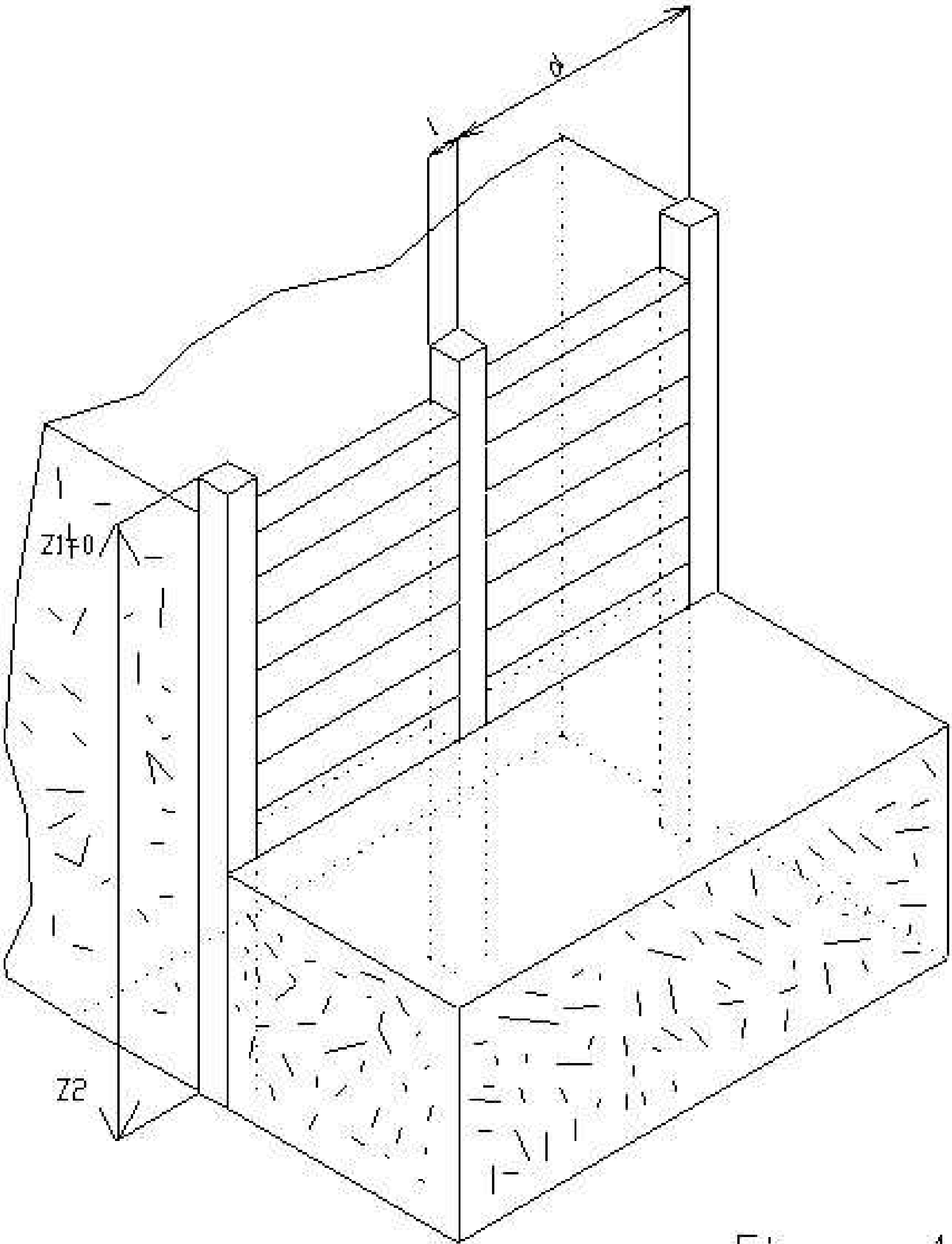


Figure 1

The laggings to be placed subsequently are supposed to transmit the trusts of the ground, yet without participating in the inertia of the retaining wall.

A-1-4 Case of the pile

The real inertia of a pile can be introduced and the keyword COE can be used to apply the reactions of the grounds on the entire width of the pile, and not on a unit strip of one meter. In the computer output the moments, the shear force etc... will concern, thus, the pile and not the width of one meter of the pile.

It is, however, possible to do the calculation for a metre of pile.

A-1-5 Null inertia

If a section with null inertia is introduced, it takes understandably a reservation for a subsequent wall and it means an absence of material in this section.

It is not allowed to place such a section between two sections with non-null inertia because the program **RIDO** is made for calculating one retaining wall and not two!

When it is certain that the redefined inertia (keyword INE) of a section with null inertia is a non-null value, this recently implemented section is supposed to be perfectly vertical and capable of transmitting moments to the rest of the retaining wall. It thus results notably in an angular point on the neutral axis if the wall has previously been deflected.

A-2 ***DISCRETISATION IN FINITE ELEMENTS***

A-2-1 The model of finite elements

The wall is discretised in the direction of its heigh in beam type finite elements.

The displacement of a finite element is described by a 5 degree polynomial in such a way that the equilibrium calculation is theoretically exact when it is sollicited by a linearly increasing load other than the concentrated loads on its extremities.

For the overall equilibrium, it is "the force" model that has been adopted for it enables the unilateral connections to be treated more efficiently than the "displacements" model.

A-2-2 The automatic generation of finite elements

To guarantee a good calculation precision, certain border points of finite elements are automatically imposed.

These are :

- the changing points of the retaining wall section
- the changing points of soil layers
- the various excavation and backfill levels
- the various water and hydrodynamic correction levels
- the levels with application of Boussinesq type of surcharges as well as the levels at which their effect is maximum
- the levels of distributed load definition
- the points of application of concentrated efforts: forces, couples, anchoring point of tiebacks.

Considering these fixed border points, the program **RIDO** optimises the distribution of elements in such a way that the longest lenght of them does not exceed the maximum specified in the data (in general, 0.50 meter or one Ft).

If the wall is of considerable height, the number of elements might exceed the limit fixed by the number of nodes authorized for the installation of the program **RIDO**.

In this case, the last condition dominates and the lenght of elements exceeds the user-specified maximum length.

If the different levels described above are close (differences below 10 cm) it will be advantageous to combine them into a single value and at the same time maintenaing an acceptable precision of calculation. However we shall never place various beds of struts or anchors in the same level.

ANNEX B

THE MODELLING OF THE SOIL

B-1 ***LAW OF THE ELASTOPLASTIC AND NON-REVERSIBLE BEHAVIOUR***

B-1-1 The parameters defining the limit states of plasticity

In order to leave all the freedom of hypothesis to the user, the coefficients of active and passive horizontal thrusts (K_a and K_p) can be directly given by their values. But they can be calculated advantageously by resolution of the differential equations connected) to the equilibrium model of BOUSSINESQ-RANKINE. (See the data group A in the RIDO user manual)

The angle of internal friction φ and the ration $\frac{\delta}{\varphi}$ in active and passive states, where δ is the angle of friction between soil and retaining wall, are supplied for three reasons:

- to allow the eventually calculation of Ka, K0, Kp
- to allow the calculation of cohesion terms if it not zero.
- to document the computer output which is a note of calculation (if not used)

At a level where the overburden pressure on a neighbouring horizontal face of the retaining wall is P (in the calculation of P, the weight of the soil in the possible presence of the ground water level, and increase due to surcharges, are considered), the active horizontal thrust is given by:

$$q_a = K_a p + \frac{C}{tg\varphi}\left[\frac{\cos\delta - \sin\varphi\cos\gamma}{1+\sin\varphi}.e^{-(\gamma-\delta)tg\varphi}\cos\delta - 1\right] + S \qquad (1)$$

or

$$q_a = K_a - 2C + S \qquad \text{if } \varphi = 0 \qquad (2)$$

and the passive horizontal thrust is worth:

$$q_p = K_p p + \frac{C}{tg\varphi}\left[\frac{\cos\delta + \sin\varphi\cos\gamma}{1-\sin\varphi}e^{(\gamma+\delta)tg\varphi}\cos\delta - 1\right] + S \qquad (3)$$

or

$$q_p = K_p p + 2C + S \qquad \text{if } \varphi = 0 \qquad (4)$$

For the RIDO versions before 4.20 : $q_x = K_x p \pm C\left[\frac{\pi}{2}+1\right] + S$ if φ = 0

In these relations:

$\gamma \in \left[0, \frac{\pi}{2}\right]$ is the solution of the equation $\sin\gamma = \frac{\sin\delta}{\sin\varphi}$

C is the cohésion.
S is the term due to the Boussinesq type of surcharges, if they are superposed (see B-2-2).

B-1-2 The elastoplastic model

The coefficient of elastic reaction w (Winckler hypothesis), variable according to the soil layers and the depth, depends on two parameters.

α: stiffness when p=0
β: increase due to effective overburden pressure

according to the relation

$$w = \alpha + \beta p$$

Using β, an increase of the soil stiffness due to the confining pressure can, thus, be taken account of.
In-stitu tests and model tests with compaction wheels showed it is better to choose non null values of β for pulverulent grounds.
For the initial position of the retaining wall (zero deplacement), the soil pressure on both sides of the retaining wall are initialized as

$$q_0 = K_0 p + S \qquad (5)$$

where p and S are calculated independently on both sides of the wall.
This corresponds to the at-rest soil pressure.

Figures 2 and 3 specify, for a given level z, the law of elastoplastic behaviour of soil 1 (on the left of the retaining wall) and of soil 2 (on the right of the retaining wall) for the first phase of calculation.

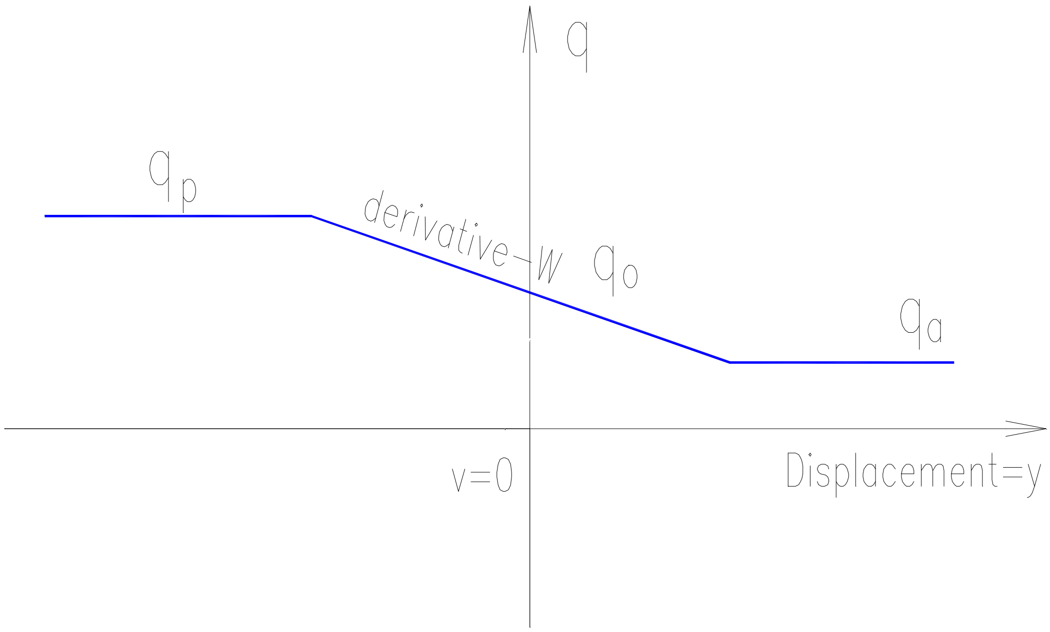


Figure 2

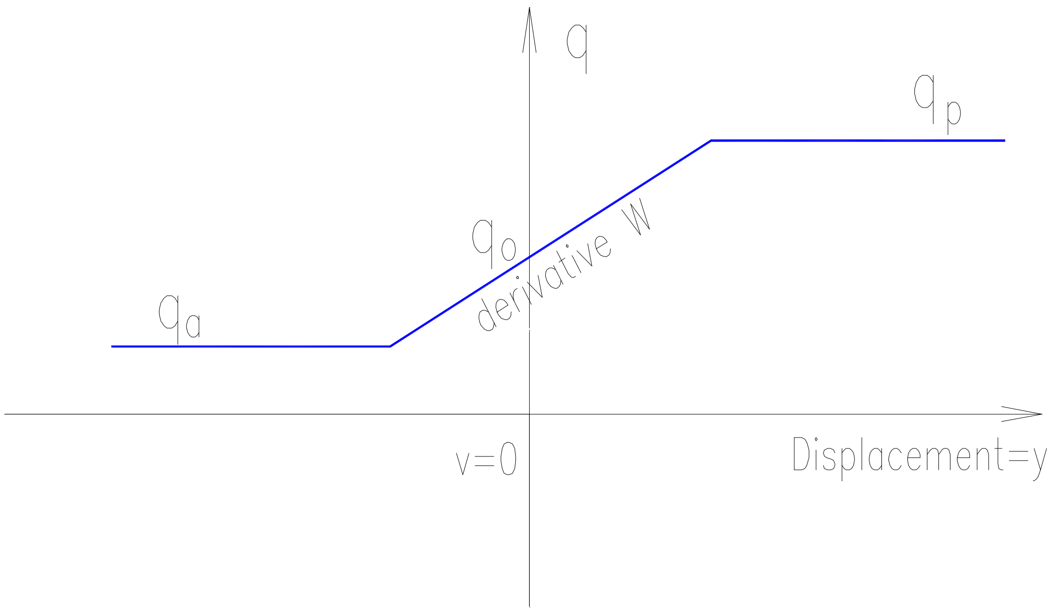


Figure 3

In a more general way, the soil pressure at level z in the elastic zone is related to the deflection of the retaining wall at the same level by the relation

$$q = K_0 p \pm w\big(y - v(z)\big) + S \qquad (6)$$

where v(z) is the value of the displacement of the soil which leads to a soil pressure q_0 (at-rest soil pressure).

Initially v(z)=0.

In the case of a cohesive soil, q_a can be negative (figure 4). The program **RIDO** admits in this case a soil-retaining wall separation, if the deflection leads to a negative pressure according to the previously defined model.

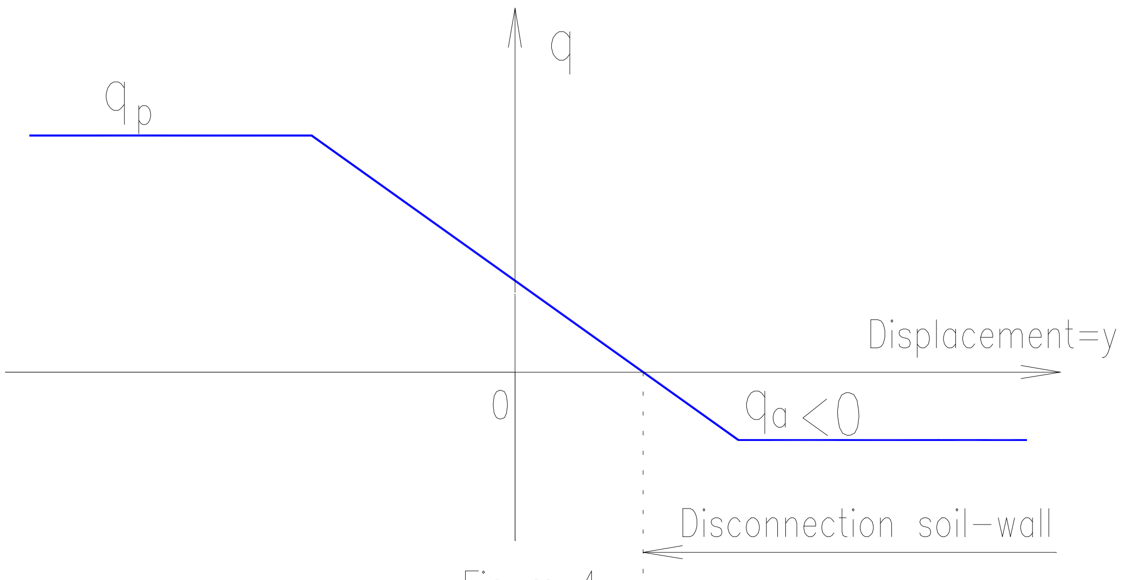


Figure 4

B-1-3 Non-reversibility

For one of the soils at a given level z, if after an equilibrium calculation, one of the plasticity limits is not achieved (active or passive), the present rheological parameters relative to this level are conserved for the following phase of calculation.
In the contrary, v(z) is recalculated conforming to the figure 5, which gives a new set of rheological parameters for the following phases of calculation.
If the retaining wall is sollicited by alternative forces from right to left and from left to right the hysterisis cycles can also be described.

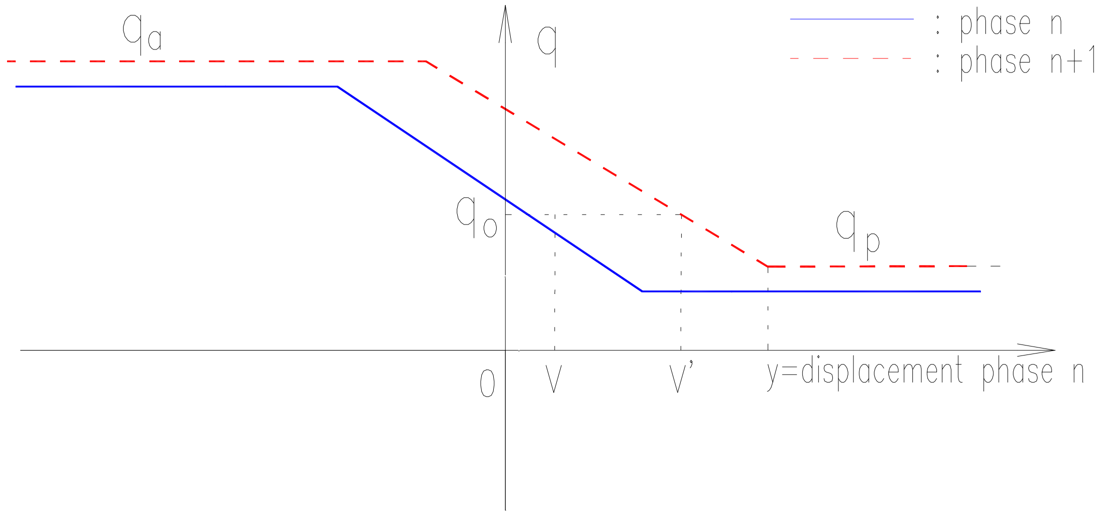


Figure 5

B-1-4 The effect of variations of the overburden pressure

Whenever excavations, drawdowns, backfilling, installation and suppression of surcharges, etc..., take place, the soil pressure p on an horizontal face varies.
The values of q_a, q_0 and q_p as well as w are recalculated to fix the position of elastic domain, the hypothesis of invariance of v(z) is adopted.
It is an hypothesis entirely coherent with the notion of equilibrium of at-rest soil pressure.
In this way, the K_0 parameter plays not only a role of definition of the initial state but it is an integral part of the theorical model and conditions the calculated sequences of equilibriums. That is why in the keywords SOIL and BAC the coefficient of initial soil pressure K_i is distingued from K_0.
Figure 6 illustrates this hypothesis.
This updating of the rheological parameters is always carried out to take into account the non-reversibility.

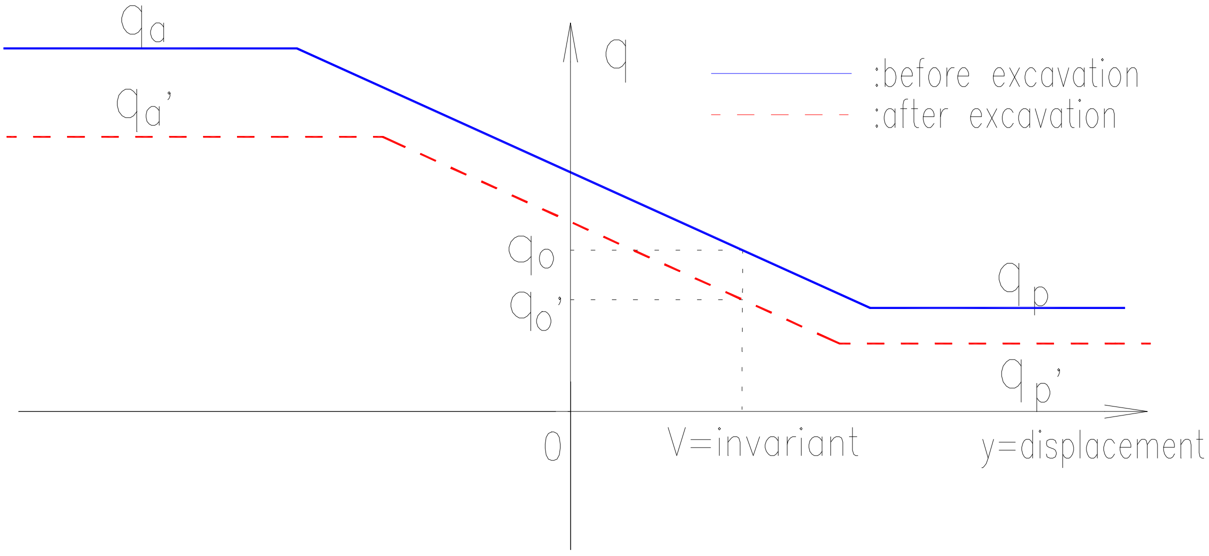


Figure 6

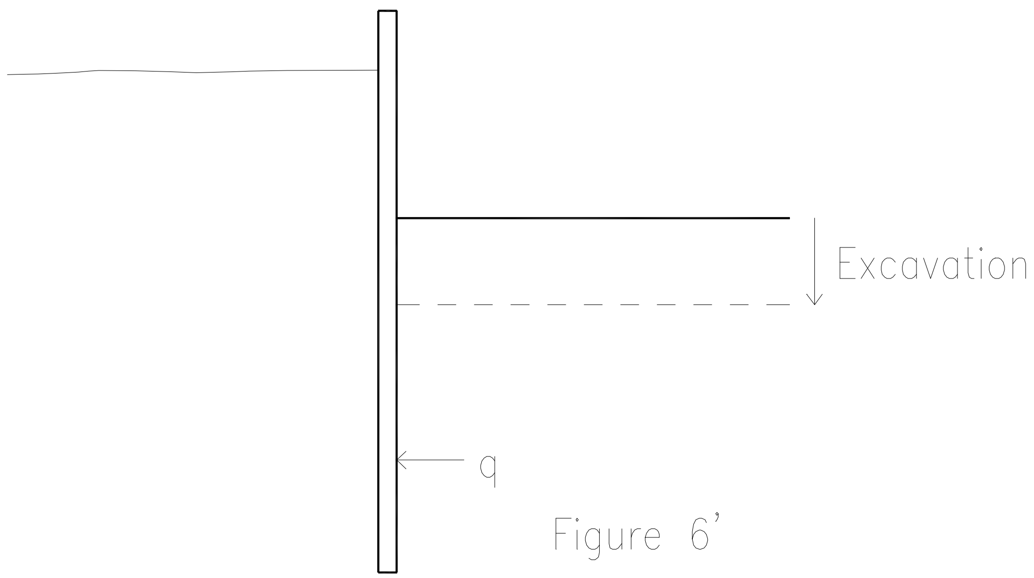


Figure 6'

B-2 ***THE EFFECT OF SURCHARGES***

B-2-1 Caquot type of surcharges

For the surcharges applied evenly on the entirety of the ground surface, the program **RIDO** uses the corresponding state principle and takes the value of the surcharge as an additive contribution in the calculation of p which conditions q_a, q_0, q_p and w.

B-2-2 Boussinesq type of surcharges

B-2-2-1 *Additive hypothesis*

In the absence of a complete mathematical model of soil behaviour, the superposition is commonly applied on a strip at level z according to the figure 7, the term S, appearing in the above expressions, takes the form

$$S(x) = \frac{Q}{\pi}\left[Arctg\frac{(b-a)x}{ab+x^2} + \frac{ax}{a^2+x^2} - \frac{bx}{b^2+x^2} \right] \quad \text{if } x>0 \qquad (7)$$

$$S(x) = 0 \qquad \text{if } x \le 0$$

If there are several of these surcharges on the same soil, there must be a running total. This hypothesis is selected by option A of the title line of data.

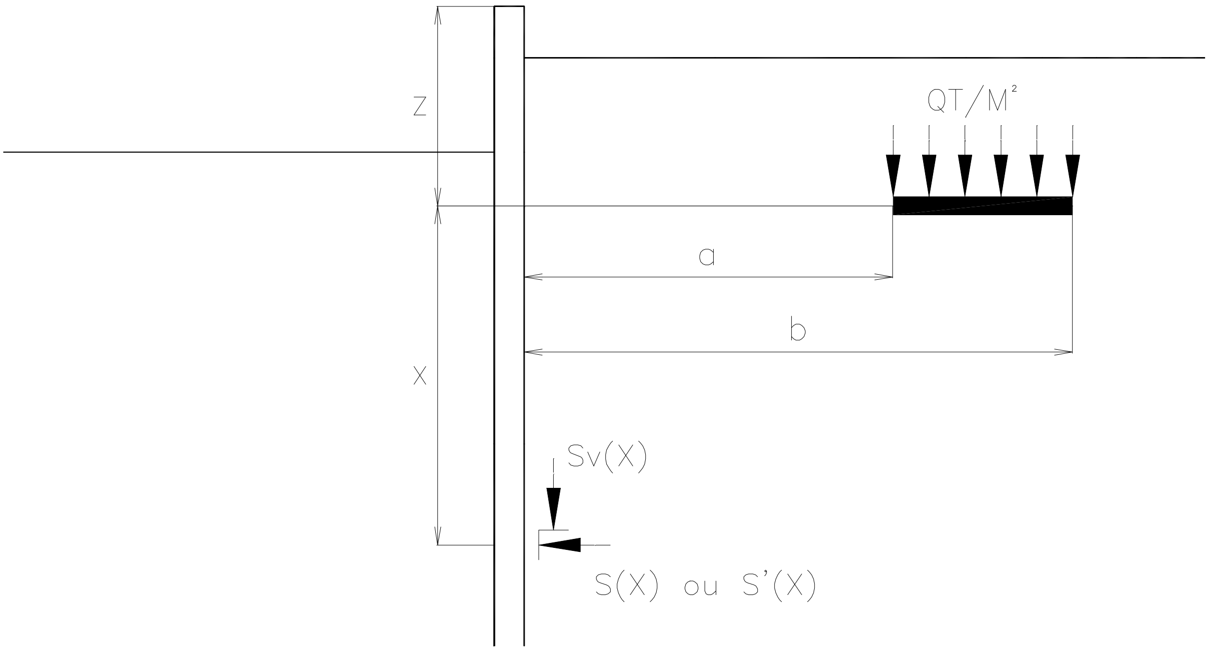


Figure 7

B-2-2-2 *Non-additive hypothesis*

The program **RIDO** version 3 and upper, allows a more elaborate, though non « classical », treatement of the Boussinesq type of surcharges.
This results from the following observation : if it is considered that the strip loaded by Q is at the ground surface, and that a=0 and $b \to \infty$, the surcharge becomes Caquot type and the formule (7) gives

$$S(x) = \frac{Q}{2}$$

In the additive hypothesis, the error committed in treating the Caquot type as the limit of the Boussinesq case is immediately seen!
Notably, the effect of the surcharge is independent from yhe ground conditions.

For Caquot, the principe of corresponding states would give :

$S(x) = K_a Q$ for the active state
$S(x) = K_0 Q$ for the elastics state
$S(x) = K_p Q$ for the passive state

Deriving thus the idea of replacing S(x) by

$$S'(x) = \frac{K}{0{,}5} S(x)$$

where $K=K_a$, K_0 or K_p according to the soil state and thus achieving the continuity between Boussinesq and Caquot types.
This hypothesis is implemented very simply in the **RIDO** version 3 and upper by cancelling the terms in the expressions (1),(2),(3),(4),(5),(6) and for each Boussinesq type surcharge, bringing the additive contribution

$$S_v(x) = \frac{S(x)}{0{,}5}$$

to the weight p relative to the level z+x.

In the order to adopt this hypothesis, not putting option A in the title line of data is enough.

Whatever is the chosen hypothesis the effect of all the surcharges of type Boussinesq and others is presented in the arrays of results. It is obtained as the difference between the calculated soil pressures at the equilibrium and that would be this pressure if the surcharges were absent in the current position of the wall.

B-2-2-3 *Theory of images*

In the presence of a strongly blinded excavation, the horizontal displacements of a neighbouring ground of the retaining wall are almost null. By leaning on the theory of the elasticity, it is advisable to cancel these horizontal displacements by placing surcharges of type Boussinesq symmetrically with regard to the retaining wall.
A coefficient 2 then is to be applied to this type of surcharge.

Since the version 4.20 of RIDO this coefficient is planned in the data and can have a value included between 1 and 2.

In the case of pre-existent surcharges in the implementation of the wall a preliminary calculation of equilibrium of the soil without wall is essential. In this case it is imperative to choose the coefficient 2 to obtain an effect of the surcharges of Boussinesq corresponding to horizontal null displacements in the axis of the future retaining wall.
In the calculation of this equilibrium displacements appear and the image effect fades in zones in elastic state, but the augmentation of pressures in the soil owed to the surcharges will be present on each side of the wall during its implementation
(see the keywords SUB and INE)

B-3 ***MODIFICATIONS OF THE SOIL CHARACTERISTICS***

B-3-1 Redefining

The keyword SOI enables the complete redefinition of a soil layer while allowing entirely a reinitialisation of the soil pressure at level z (in the interval of redefinition) for the deflection y resulting from previous equilibrium by a value K_i, introduced in the data.
This initial soil pressure q is given by

$$q_i = K_i p \qquad \text{in the absence of cohesion}$$

and by

$$q_i = q_a + \frac{K_i - K_a}{K_0 - K_a}(q_0 - q_a) \qquad \text{if } C \neq 0$$

v(z) is consequently calculated considering y to obtain an unadapted set of rheological parameters at level z.

In the case of a possibility of separation, the initialisation is performed in such a way that q_i=0, but y corresponds to the separation limit.
It is advisable to note that this redefinition does not consider the previous active/passive conditions in the soil layer concerned and it is, therefore, not adequate for the modifications of long-term soil characteristics.

The keyword BAC (process of backfilling) allows an identical initialisation in the case of a backfill, the remoulding of the ground induces one to take $K_i=K_a$.

B-3-2 Modification of the characteristics of plasticity of a ground

The keyword PLA does not perform a reinitialisation of a soil pressure, but allows the introduction of a new values for K_a, K_p, C and φ in the formules (1), (2), (3) and (4) for a given soil.

The parameters specifying the elastic domain of the model : w, K_0, v(z) are invariant.
In particular, the modification of the coefficient of elastic reaction w is not allowed because of strong risks of incoherence of the resulted model.

B-4 ***NON-COPLANER OR NON-HORIZONTAL GROUND SURFACES***

B-4-1 Straight and inclined ground

The case which goes back to an equivalent horizontal ground level may be treated by **RIDO** by introducing the adequate K_a, K_0, K_p coefficients.

If ground levels 1 and 2 are both inclined, even if their inclination is identical, the equivalent horizontal grounds do not have the same coefficients. It is advisable then to use the keyword SOIL to redefine one of them.

B-4-2 Berms

An approximative calculation of the effect of a berm is integrated to the program **RIDO** version 3 and upper.

It is an original approach to this complex problem where the coherence is sought by an unique and valid treatment irrespective of the active/passive elastic state of the soil.
Figure 8 illustrates the form of calculation.

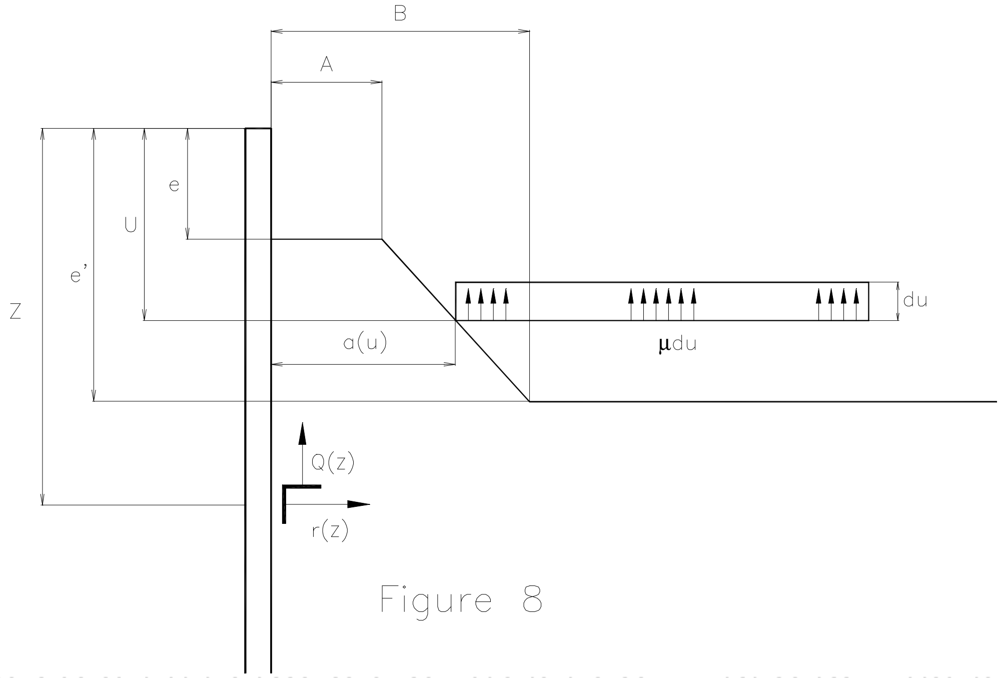


Figure 8

I

horizontal ground subject to the negative Boussinesq uniform loads, stretching to infinity and corresponding to the weigh per m^2 of the soil layer of thickness du.

Naturally, the non-additive hypothesis of the calculation of Boussinesq type of surcharges is used (see B-2-2-2) whether or not the option for the « real » surcharges of this type has been chosen.

The contribution of the soil-weigh in the neighbourhood of the wall corresponds then a decrease

$$Q(z) = \int_{e}^{e'} \frac{1}{0.5\pi}\left[arctg\frac{z-u}{a(u)} + \frac{a(u).(z-u)}{a(u)^2 + (z-u)^2}\right]\mu du$$

It should be noted that is approximate calculation even though it gives the satisfactory curves of soil pression at active and passive limit states (see figure 9), should be accompanied by a verification of the stability of the massif constituted by the berm after an equilibrium calculation of **RIDO**.

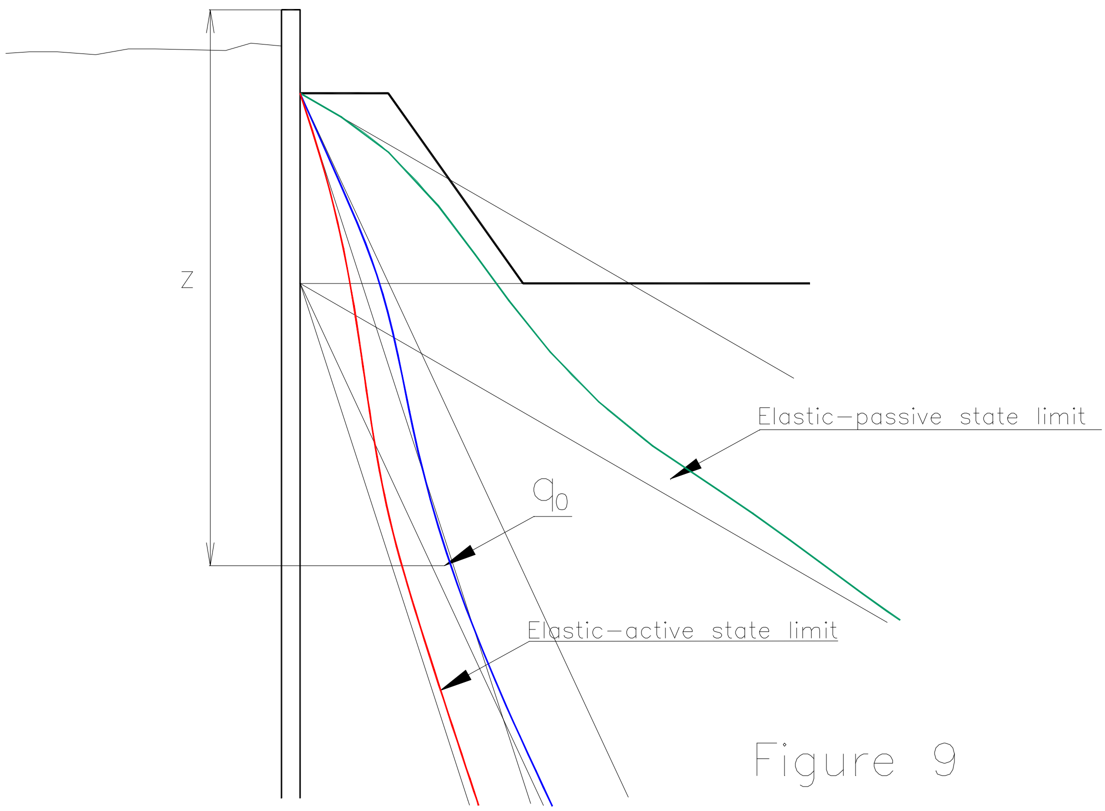


Figure 9

B-4-3 Banks

Figure 10 shows how to decompose its effect in the form of two Boussinesq type of surcharges, Q_1 et Q_2 equalling the respective weight per m^2 of the correspondant parts of the slope.
If more precision is desired, the inclined part can be decomposed in several vertical slices and the same amount of Boussinesq equivalentsurcharges can be placed to get a correct calculation, **option A in the title MUST NOT BE CHOSEN.**

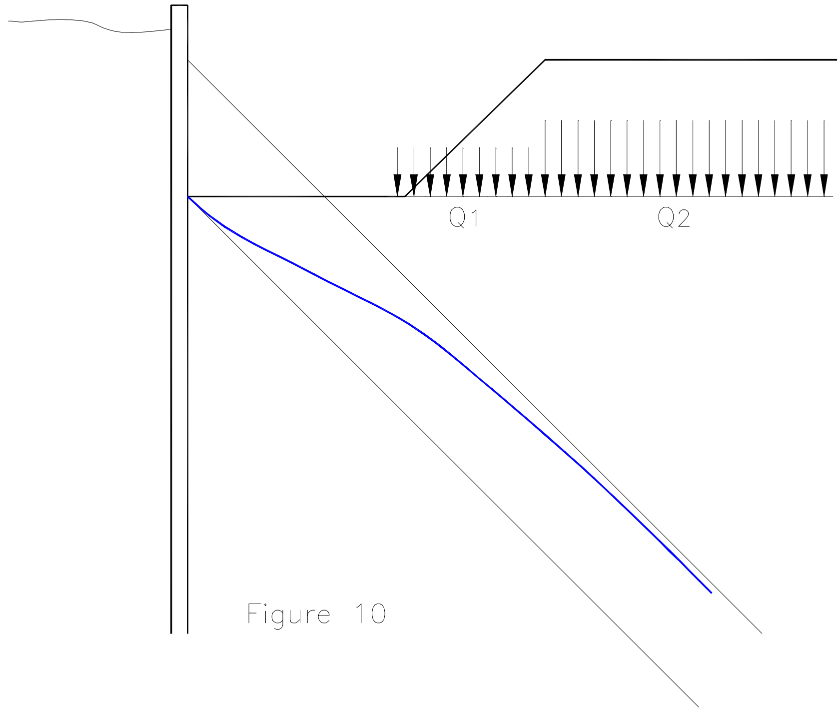


Figure 10

This decomposition is useless from the version 4.0 of RIDO. This one is made in more accurate calculation with the keyword EXC (geometrical description of the slope in symmetrical manner as a berm) while remaining compatible with the option A.
In the sloping part the decomposition is made with a step of the order of 0.5 m.

ANNEX C

SHORINGS AND LINKAGES

C-1 ***STRUTS AND ANCHORS***

A level of anchor of stiffness $K = \dfrac{E.s}{l}$

where E is the Young's modulus of the material constituting
s the section
l its free lenght
inclined at I^0 with the respect to the horizontal
spaced at a metre or Ft
preloaded at F_0 tonnes or kN or KiP

is automatically replaced by a level of equivalent horizontal anchor

with 1 metre or 1 Ft of spacing,
of $k = \dfrac{K}{a}\cos^2 I$ stiffness
and with $f_0 = \dfrac{F_0}{a}\cos I$ preload.

The load f of these fictitious anchorss in the subsequent phases after its preloading is given by the expression

$$f = k(y_0 - y) + f_0$$

where y is the deflection at the wall at the anchoring point.
y_0 is the deflection at the same point but taken at the end of the preloading or at the time of installation if there is no preloading.

In unilateral linkage, f is lower bounded by 0 if the retaining wall is free to deflect towards ground number 2, and upper bounded by 0 if the retaining wall is free to deflect towards ground number 1.

In the output, the indicated load is the effective load in an anchor given by

$$F = \frac{f.a}{\cos I}$$

In the bucking calculation, the vertical component intervenes in the calculation of bending moments with the pessimistic hypothesis that the entire loading is resisted by the point-bearing resistance of the retaining wall and not by the lateral soil-retaining wall friction.
In this case, the sign of angle I is important. Figure 11 specifies it.

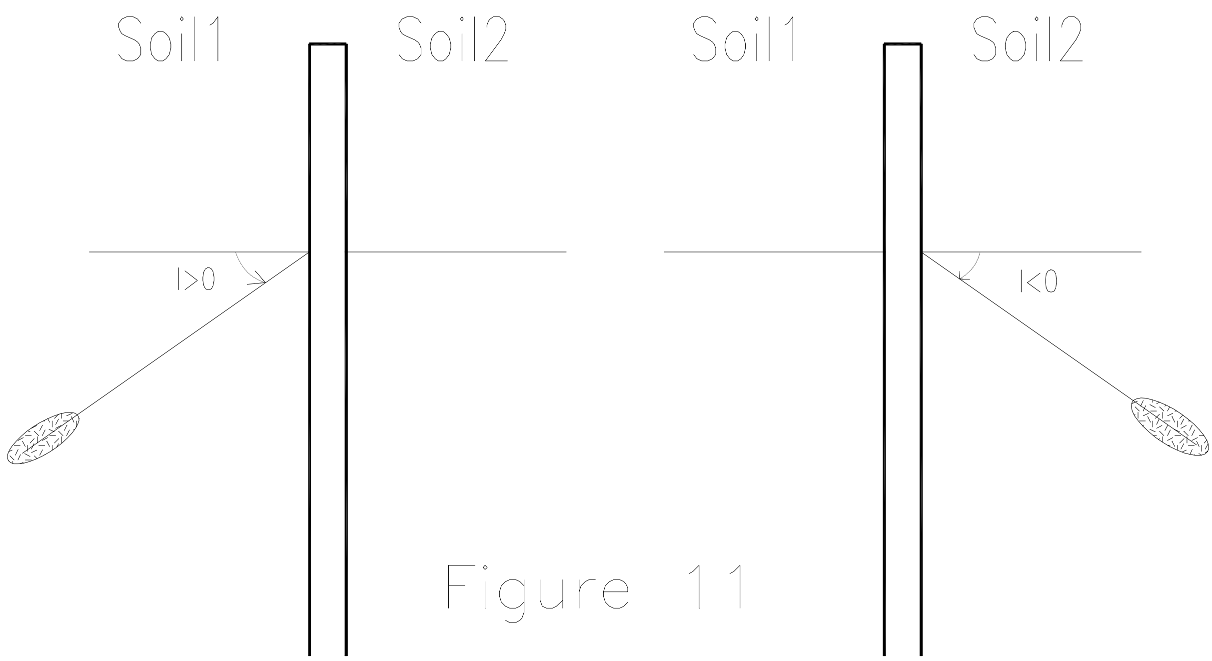


Figure 11

This option set off by option F of the title line, has been integrated to the program **RIDO** to reassure certain users and prove to them that the effects of the second order only start to be sensitive to deflections of tens of cm!...

The case of the strut is identical with I=0 and the possibility of bilateral linkages.

C-2 ***ELASTIC CONNECTIONS***

It is possible to place an elastic linkage (purely linear) at any point of the retaining wall with a given structure.
Preliminary studying of this structure and calculation of its matrix of influence in contact with the retaining wall are necessary.
For the considered level :

$$\begin{bmatrix} \Delta T \\ \Delta M \end{bmatrix} = \begin{bmatrix} CFY & CFA \\ CMY & CMA \end{bmatrix} \begin{bmatrix} \Delta Y \\ \Delta A \end{bmatrix} + \begin{bmatrix} F \\ C \end{bmatrix}$$

where ΔT is the abrupt change of shear force
ΔM is the abrupt change of moment in the retaining wall
ΔY is the variation of the deflection after installation of the elastic connection
ΔA is the variation of the angular displacement (in radians) after the installation of the connection
F is the horizontal force brought by the connection at $\Delta Y = 0$ et $\Delta A = 0$
C is the couple brought by the connection at $\Delta Y = 0$ et $\Delta A = 0$.

The sign conventions of the program **RIDO** are such that in the commun cases where the structure is a slab:

$CFY < 0$

$CFA = 0$

$CMY = 0$

$CMA > 0$

that the slab is situated on the left or right of the retaining wall.

C-3 ***TOP AND TOE CONNECTIONS***

Initially, the top and toe of the retaining wall are free. That is by far the most frequent case.

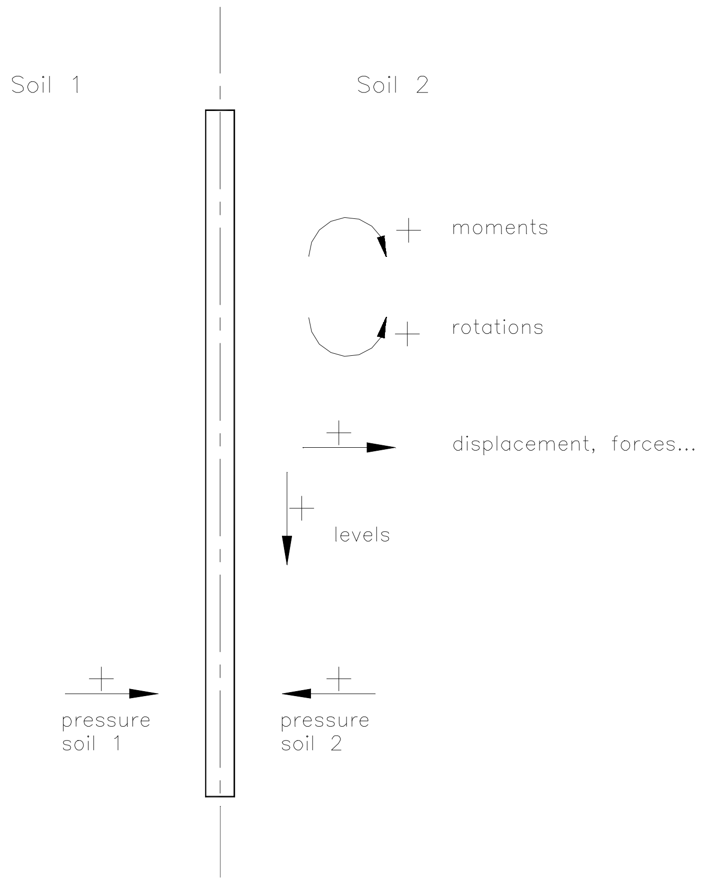
In certain circumstances, the following conditions can be chosen (with the keyword LIM) :

-simple support, for example, if the toe of the retaining wall is simply embedded in the molasse (soft tertiary sandstone).

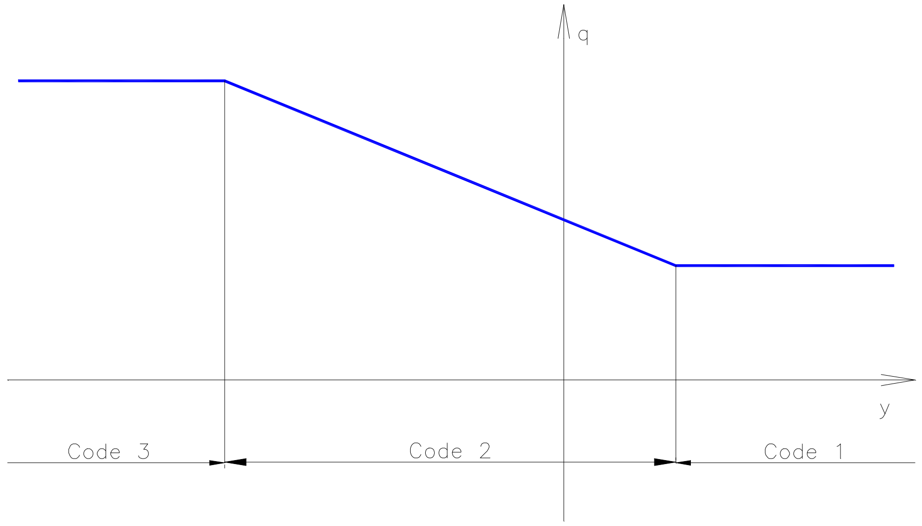
-applied inclination, but free horizontal displacement, for the heads of embedded piles in a very stiff pile cap.

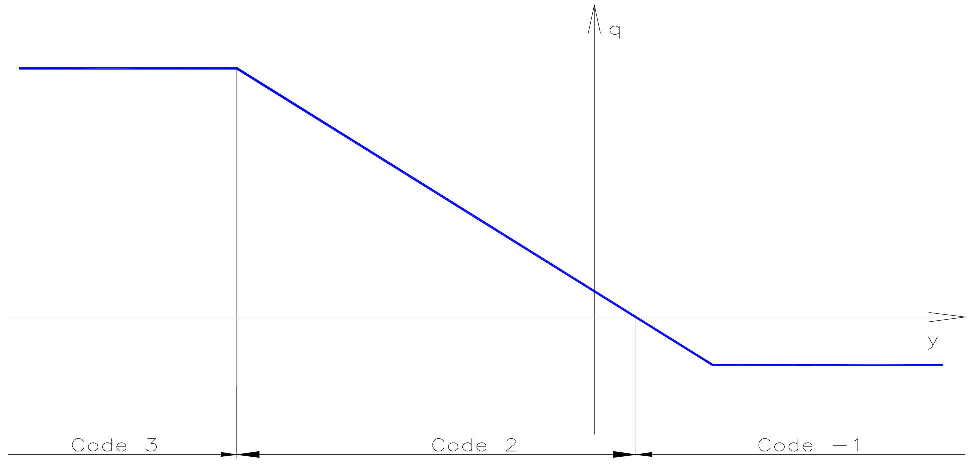
-perfectly fixed end: rare! even the molasse (soft tertiary sandstone). It is preferable to place an elastic connection (keyword CFM).

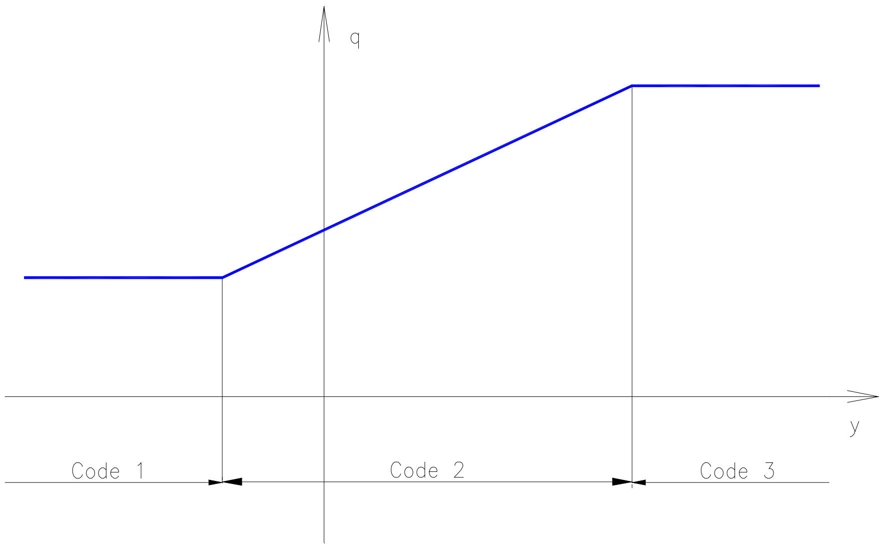
SIGN CONVENTIONS

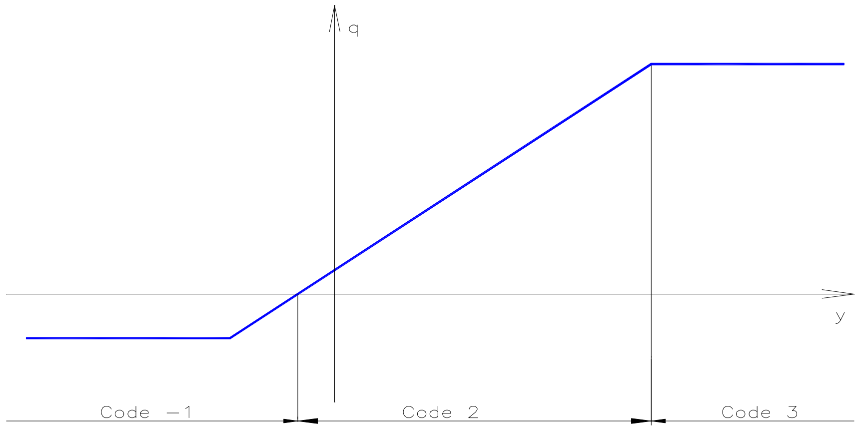


STATES OF SOIL CODES






q
y
Code 1
Code 2
Code 3


q
y
Code -1
Code 2
Code 3

RIDO 4.20

USER MANUAL

TECHNICAL ANNEXES

PUBLICATIONS

ROBERT FAGES LOGICIELS

Essais sur modèle de rideaux de soutènement ; confrontation à diverses méthodes de calcul

Experiments on retaining wall models ; confrontation with different calculation methods

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Rev. Franç. Géotech. n° 55, pp. 17-33 (avril 1991)

Résumé

On présente des essais sur modèle bidimensionnel de rideaux de soutènement en considérant l'influence des conditions de butonnage ou d'ancrage (nombre, raideur, précontrainte). Le comportement de ces rideaux est analysé et comparé aux résultats des méthodes de calcul traditionnelles en plasticité ainsi qu'à ceux obtenus par le calcul suivant l'hypothèse du module de réaction.

Abstract

We present experiments on a bidimensionnal model of flexible retaining walls and analyse the influence of anchors or struts (number, rigidity, prestressing). The behaviour of these walls is analysed and compared to the results obtained through conventional plasticity calculation methods and also to those obtained from the reaction module hypothesis.

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Underground Construction in Soft Ground, Fujita & Kusakabe (eds) © 1995 Balkema, Rotterdam. ISBN 90 5410 536 4

Earth pressure on braced flexible walls – Model tests and field investigations

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SYNOPSIS : This paper concerns the behavior of braced flexible walls. Field observations on two different sites of the subway system in Lyon are discussed. The Rido software, which utilizes the reaction modulus of the soil is used to predict the soil behavior. The measured displacements, differential earth pressures, bending moments and load in struts are compared to the calculated ones.

1 INTRODUCTION

Many kilometers of the subway tunnels in Lyon (France) have been constructed by the cut and cover method using temporary sheet piles and cast in place or precast concrete walls.These walls are supported by 2 or 3 levels of passive struts.The construction of this subway system started 20 years ago and it is still in progress. From the very begining of this construction, an extensive instrumentation and monitoring program is being carried out.

In order to study the behavior of these temporary retaining structures and to improve their design, a particular attention has been given to the prediction of the movements, bending moments, earth pressures and load in struts, by using the Rido computer program (Fages & al 1971) based on Winkler's hypothesis.

This paper outlines the discordance between predicted and measured values by analysing the results from two sites with different geological conditions. We also discuss the influence of the seepage flow at the bottom of the sheet pile on the diminuation of the passive earth pressure.

A series of experiments on a bidimensional small scale model confirm the limitations of the reaction modulus method.

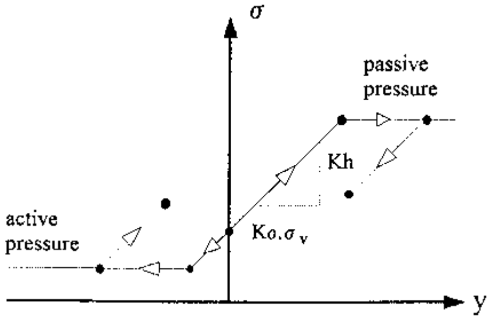

Figure 1. Load-deflection relationship of the software Rido.

2 CALCULATION USING REACTION MODULUS

The Rido program was especially developped for the construction of the first subway line and is now of general use in France. In Rido, the earth pressure is considered separately on every side of the retaining wall. The reaction modulus of the soil is limited by active and passive pressures and a hysteretical load-deflection relationship (figure 1).

3 EXPERIMENTAL SITE (COURS GAMBETTA)

3.1 Site and construction description

The excavation is situated on the east side of Rhône river. Under 4 m of recenty deposed soil we find 18 m of alluvial sandy gravel and a sandstone substratum. Figure 2 presents the excavation profile : the tunnel on the alluvial penetrates about 5 m into the water table. The high permeability of alluvial (k about 10^{-2} m/s) does not allow the lowering of the water table by pumping, therefore a water tight wall (0.6 m thick) fill in with a bentonite-cement slurry is constructed. The embedment depth of this wall in the sandstone substratum is 3 m. The sheets piles

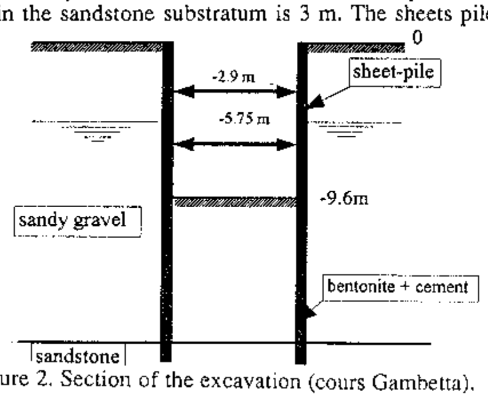

Figure 2. Section of the excavation (cours Gambetta).

are 12.70 m high and completely embedded in the slurry wall. After the excavation the sheet piles are supported by two levels of struts designed to limit the lateral movements toward the excavation.

As the sheet piles are monitored, the second level of struts are placed with 2.5 cm of slack to permit a sufficient bending of the sheet piles.

The characteristics of alluvials are measured by direct shear field tests and back analysis on similar materials (Kastner & al 1984) :

bulk density		$\gamma/\gamma_w = 2.05$
submerged density		$\gamma'/\gamma_w = 1.25$
friction angle	$\phi = 33°$	cohesion $c = 24\ kN/m^2$
reaction modulus		$k_h = 10^5\ kN/m^2$

3.2 Comparison of calculated and measured values

Inclinometers are installed on the sheet piles and displacements at the top of the piles are measured by surveying instruments. Extensometric equipment permits us to determine the distribution of bending moments of the sheet pile and also the load in the struts.

Figure 3 presents the measured displacements on the 2 last steps of construction : we observe a considerable increase of displacement when the excavation depth increases from 6.5 to 9.5 m.

A back analysis permits to adjust the set of soil parameters used in Rido program in order to fit the calculated and measured displacements at each construction stage.

We observe that the values calculated by the software Rido are highly influenced by the cohesion term while the variation of reaction modulus does not affect them.

If this adjustment seems to be acceptable for the deflection curve, but it is almost impossible to have simoultaneously correct values for the load in the struts (figure 4). The calculated load is about 50% less than the measured one.

In order to explain this discrepancy we study the calculated and the measured earth pressure values. The measured earth pressure curves are obtained by the fourth derivative of deflection curves. These curves imply a high concentration of stress just at strut levels. Such a high stress concentration can be justified by a differential displacement, between the top and the bottom of the sheet pile, which occurs at the second step of excavation. This concentration can not be calculated by Winkler's hypothesis which considers only the local displacements and not the differential ones. We note that the difference between calculated and measured load in struts equals the area between two earth pressure curves.

4 EXPERIMENTAL SITE (GORGE DE LOUP)

4.1 Site and construction description

The excavation, 9 m wide and 8 m deep, is supported by two cast in place retaining walls made of reinforced concrete 10.5 m high and 0.6 m thick (figure 5). these walls are restrained against lateral movements by two series of passive struts made of steel H-shaped sections. Before the work, the water table varied between 4 and 5 m below the ground surface.

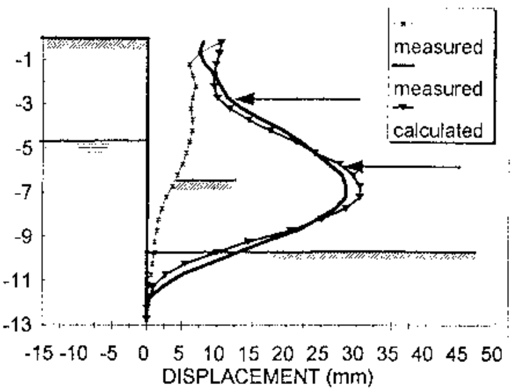

Figure 3. Comparison between calculated and measured displacements.

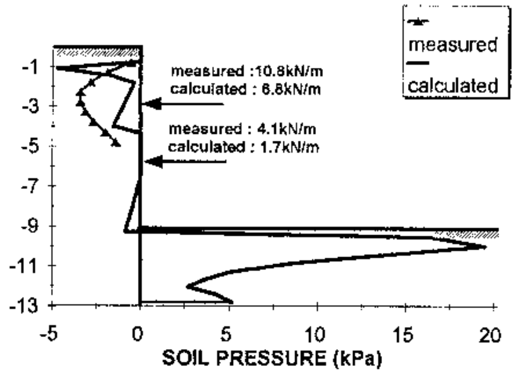

Figure 4. Comparison between calculated and measured differential earth pressures.

A preliminary hydrogeological study showed that the groundwater level might rise when intersected by these cast in situ walls. As a result, the retaining walls were designed with a reduced depth of embedment, and this in turn led to a heaving risk of the excavation floor.

The displacement of the retaining walls is monitored by inclinometers and surveying instrumentation. The load in struts is measured by extensometers. Furtheremore the water level during the excavation is surveyed by four piezometers. Four pore pressure measuring cells are placed at the foot of the pile to evaluate the uplift gradient.

On this site, the soil consisted of a succession of sandy silt layers with fairly similar identifying characteristics. The liquid limit of these silts varies between 25 and 30% with a plasticity index close to 5%. It should be noted that the natural water content of these silts is always close to their liquid limit, which explains why they are so sensitive to disturbance, especially during earthwork. Mechanical

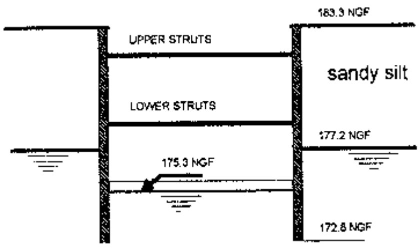

Figure 5. Section of the excavation (Gorge de Loup).

properties are homogeneous within a depth of 9 m The characteristics, deduced mainly from triaxial tests, vane tests and pressuremeter tests, are as follows :

- between 0 and 9 m :
  - undrained shear strength $cu = 70$ to 90 kPa
  - shear strength parameters with respect to effective stress $\phi' = 27°$ and $c' = 0$
  - Menard limit pressure $Pl = 0.2$ to 0.4 MPa
  - Menard elastic modulus $Ep = 1$ to 2 MPa
- beyond a depth of 9 m, the soil has slightly better characteristics :
  - $cu = 100$ to 150 kPa
  - $\phi' = 37$ to $40°$ and $c' = 0$
  - $Pl = 0.7$ to 0.9 MPa $Ep = 3.5$ to 4 MPa

4.2 Comparison of calculated and measured values

In figure 6 we present the comparison between the calculated deflection curve and the measured one during the two last steps of excavation. These curves have the same shape with a 3 mm translation. This difference could be attributed to the asymmetric load on the excavation (movement and parking of earthwork machinery on the western side very close to the excavation) which causes a general movement of the two parallel walls. This movement can not be calculated by Rido (Kastner 1992).

The first series of calculations are based on a reaction modulus value given by pressuremetric modulus. This method leads to very high displacements in comparison with those measured, especially for the first steps of the excavation. A back analysis was carried out and the modifications to the reaction modulus value that produced a calculated profile similar to the recorded profiles are described.The final modulus value should be about 10 times greater than the value obtained by pressuremetric modulus (from 2 to 4 10^4 kN/m^3). This proves that the reaction modulus value which is not an intrinsic parameter of the soil, is very difficult to estimate.

4.3 Influence of seepage flow at the toe of the excavation

When the excavation depth reaches its maximum, the pore water pressure at the bottom of the retaining walls shows that the hydraulic head decreases by 1.4 m for 2.5 m of embedded length (mean hydraulic uplift gradient = 0.56).

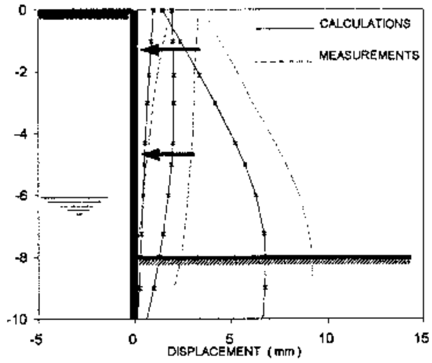

Figure 6. Comparison between calculated and measured displacements.

The safety factor for the heave of the bottom of the excavation is : $f = i_c/i = 1.7$ (i_c = critical gradient and i = actual gradient).

Seepage flow does not create any damage at the bottom of the excavation in this site, however we analysed its effect on the decrease of the passive earth pressure and on the global stability of the retaining walls. In this manner, a re design leads to decrease the passive coefficient (kp) about 40%, in order to obtain a good concordance between calculated and measured deflection curves.

In fact, the decrease of the passive earth pressure can not be detemined only by the mean gradient, and it is necessary to take into account the real flow lines shape of the seepage around the embedded length on the passive prisme.(Soubra & al 1992)

4.4 Load in the struts

For the second site (Gorge de Loup) it is also possible to obtain a correct adjustment between measured and in situ displacements by back analysis. However the calculated loads in the passive struts are basically different from those measured. The software (Rido) underestimates the load in the upper struts. The difference between these values is about 40% at the moment the struts are placed and the difference reaches 54% a few weeks after the maximal excavation.

On the deflection curves, at the upper level, there is a point without any displacement but the bottom of the wall moves towards the excavation : it seems clear that because of these differential displacements, we obtain a concentration of active earth pressure at the upper struts level. This phenomenon increases after the end of excavations because of the shrinkage of the soil at the excavation level, and because the displacement of the bottom of the sheet increases. Differential movements of the retaining walls which cause a transfer of load are not calculated by reaction modulus design.

5 MODEL TESTS

To confirm our field experiments, small scale laboratory tests on a bidimensional model, with 2 level of struts are

Table 1. Experiments on model.

test	prestress in the 1st strut kN/m ml	prestress in the 2nd strut kN/m ml	stiffness of struts kN/m ml	
A	0.38	0.33	83000	stiffs
B	0.36	0.40	816	flexibles
C	2.07	2.70	816	flexibles

investigated. The soil is simulated by the material of Taylor-Schneebeli : rustproof steel rollers of 3,4,5 mm diameter ($\gamma_d/\gamma_w = 6.1$ $\phi = 21°$ and $c = 0$)

The model pile is made of duralumin with 0.08 m width, 0.012 m thick and 0,805 m high. Its flexibility is in the range of the semi-flexible retaining walls such as the retainings to be found in the subway system in Lyon.

The horizontal movements of the wall at its top and bottom are measured with mechanical dial gauges. The deflection of this pile is measured on 20 levels by 30 strain gauges mounted on both sides of the wall. The deflection curve permits to obtain the displacement and the soil differential pressure curves by the least square method, double integration and double differentiation.Three sets of experiments are discussed (table 1).

All tests are carried out in a similar manner. First the "soil" on the outside of the pile is dredged away to a depth of 0,15 m. After the fixation of the first strut, at 0.05 m from the top of the pile, the excavation is continued in steps of 0.1 m to a depth of 0.4 m then the second strut is fixed at 0.25 m and the excavation is continued until the collapse caused by a lack of passive pressure at 0.63 m.

5.1 Comparison of model results and calculations

The reaction modulus design is based on the following values :

$k_{a\gamma} \cos\delta = 0,39$ $k_{p\gamma} \cos\delta = 3,1$ $\delta/\phi = 1$

$K_h = R_p\ \sigma_v$ and $R_p = 1500\ m^{-1}$

The soil reaction modulus (K_h) which can not be measured is determind by back analysis on two preliminary tests and then kept constant.

When the struts are passive, there is a difference between the calculated earth pressure and the measured one : the pressure concentration at strut levels (soil arching) is completely neglected by the reaction modulus design which calculates a triangular distribution. This difference disapears when the struts are active (figure 7). The concentration of pressure on the calculated earth pressure profiles for active struts is only due to their prestress, not to the soil arching which can not be calculated by the reaction modulus method (Masrouri 1986).

Figure 8 shows the measured displacements on the 2 last steps of excavation for the experiment with passive struts. We observe an important increase of displacement at the bottom of the pile. The earth pressure increases considerably at the same time. This pressure concentration is completely neglected by Rido.

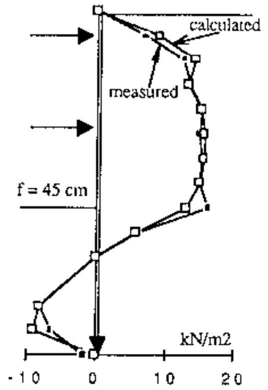

Figure 7. Comparison between calculated and measured earth-pressure.(two actives struts,test C).

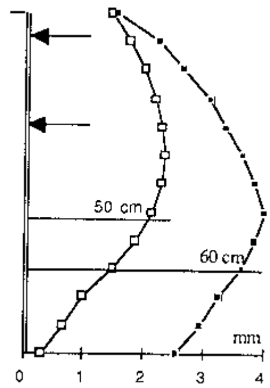

Figure 8. Increase of the displacements on the last steps of excavation.

6 CONCLUSION

Several field investigations carried out on flexible retaining walls of the subway system in Lyon, completed by laboratroy model tests, allowed an analysis of the behavior of these strutted structures.

The comparison between experimental results and back analysis by the Rido software outlined the difficulties and limits of the Winkler's hypothesis :

- the calculations highly underestimate loads in the passive struts because of the underestimation of the soil pressures on the fixed upper part of the wall. This hypothesis is not able to predict such a pressure redistribution due to a differential movement of the top and the bottom of the structure ;
- the reaction modulus value which is not an intrinsic property of the soil is difficult to estimate. In particular for silty soil, this value is very pessimistic when given by pressuremetric modulus.

Finally, the importance of the decrease of passive earth pressure due to the uplift seepage on the base of the excavation is outlined.

REFERENCES

Fages, R. & Bouyat, C. (1971). Calcul de rideaux de parois moulées ou de palplanches. Travaux,439,38-46.

Kastner, R. (1982). Excavations profondes en site urbain - Problèmes liés à la mise hors d'eau - Dimensionnement des soutènements butonnés. Thèse de docteur ès-science, INSA Lyon et université Lyon 1, 409 pp.

Kastner, R. & Ferrand, J. (1992). Performance of a cast in situ retaining wall in a sandy silt. Robinson college. Cambridge.

Masrouri, F. (1986). Comportement des rideaux de soutènement semi-flexibles: étude théorique et expérimentale Thèse de doctorat. INSA Lyon, 247pp.

Soubra, A. & Kastner, R.(1992). Influence of seepage on the passive earth pressures. Robinson college. Cambrige.

Performance of a cast in situ retaining wall in a sandy silt

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Introduction

The western terminus of the D line of the Lyon subway was constructed by means of an open-air excavation protected by strutted cast walls, in a soil horizon of silt colluvial deposits with poor characteristics. A preliminary hydrological study showed that the groundwater level might rise when intersected by these cast walls. As a result, the retaining walls were designed with a reduced depth of embedment, and this in turn led to a heaving risk of the excavation floor. As the subway might later be further extended in the same soil horizon, the SEMALY entrusted the Geotechnical Laboratory of INSA with an instrument survey with the following main objectives

- to analyze of the behaviour of the strutted cast walls
- to check the dimensioning of the retaining walls in these colluvial deposits
- to evaluate of the heaving risk by hydraulic uplift of the excavation floor.

In this Paper, we present the behaviour of this retaining wall as well as a comparison between experimental results and calculations made according to the reaction modulus hypothesis.

Site description

Geotechnical section

Within the depth of the excavation, test borings showed that the soil consisted of a succession of sandy silt layers with fairly close identifying characteristics. Grain size analysis indicated that the sandy silt was fine (0.15 mm < D_{60} < 0.2 mm) and that the percentage of grain sizes inferior to 2 microns was 10 to 20%; some places on the surface contained 10 to 30% of coarser sands. The liquid limit of these silts varies between 25 and 30% with a plasticity index close to 5%. It should be noted that the natural water content of these silts is always close to their liquid limit, which explains why they are so sensitive to disturbance, especially during earthwork.

Mechanical behaviour is homogeneous within a depth of 9m. The characteristics, deduced mainly from triaxial tests, vane tests and pressuremeter tests, are as follows

Between 0m and 9m

Undrained shear strength: Cu = 70 to 90 kPa
Shear strength parameters with respect to effective stress: $\Phi' = 27°$ and $c' = 0$
Menard limit pressure: Pl = 0.2 to 0.4 MPa
Menard elastic modulus: Ep = 1 to 2 MPa

Beyond a depth of 9m, the soil has slightly better characteristics

Undrained shear strength: Cu = 100 to 1.50 kPa
Shear strength parameters with respect to effective stress: $\Phi' = 37$ to $40°$ and $c' - 0$
Menard limit pressure: Pl = 0.7 to 0.9 MPa
Menard elastic modulus: Ep = 3.5 to 4 MPa

Excavation section

The excavation, 9 m wide and 8 m deep, is protected by two cast retaining walls of reinforced concrete 10.5 m high and 0.6 m thick. These are held by two series of passive struts made of metallic H-shaped sections. Before the

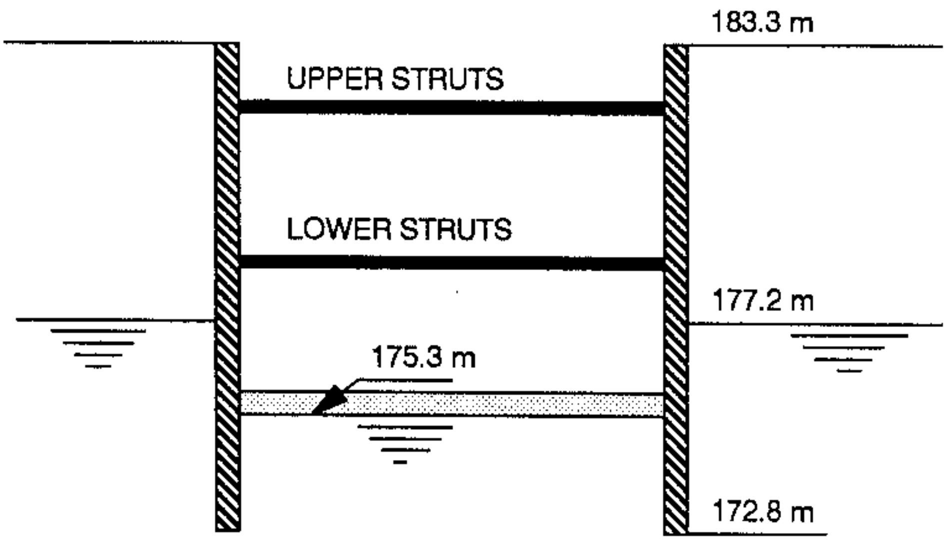

Fig. 1 . Section of the excavation

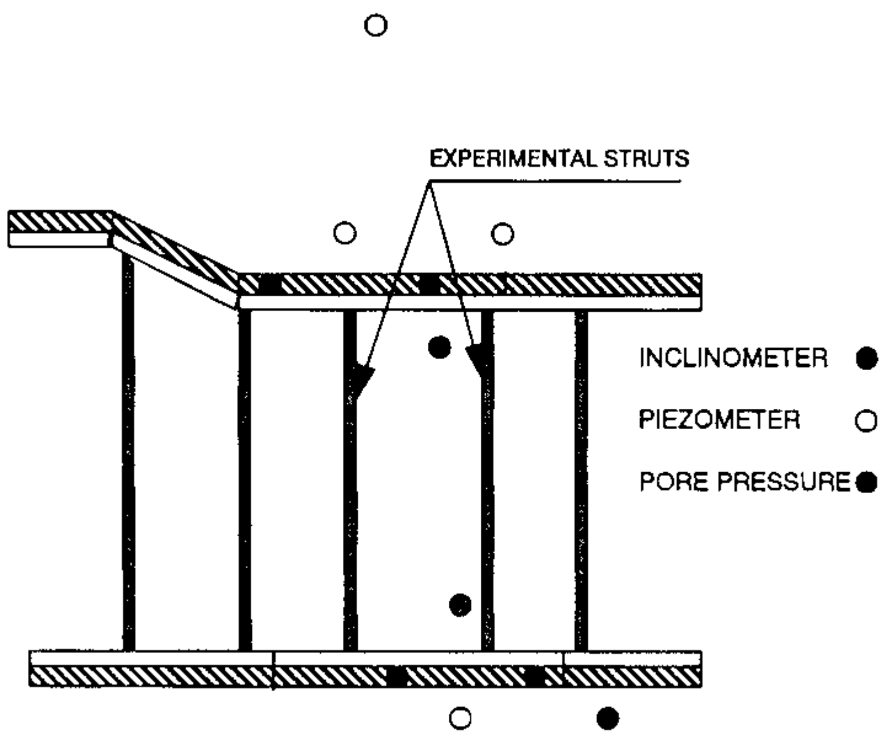

Fig. 2 . Plan view of the test zone

work, the water table varied between 4 and 5m below the natural ground level.

Experimental apparatus

The apparatus was used to measure the displacement and strain of the retaining walls, the stress in the struts, and the effect of the earthwork on the aquifer.

Displacement of the retaining walls

The deformation of the retaining wall was measured by inclinometry. Four inclinometer tubes were fixed to the reinforcing cages when they were set up in the excavation before concreting. Inclinometer readings were recorded at each stage of the work and were used to plot the displacement of the retaining walls. The curves were then readjusted: the movement at the head of the retaining walls was calculated from convergence measurements, using fixed points situated outside the zone influenced by the work.

Stress in the struts

All struts situated within the experimental section were equipped with 6 vibrating-wire strain gauges placed an the same H-shaped section, to determine the compression strain diagram. It might seem that a smaller number of gauges would have been sufficient, but compensations had to be made for the unavoidable failures taking place in situ; this apparatus gave precise measurements of stresses during the whole duration of work, while also leading to an estimate of the measurement uncertainties (ref.1).

Piezometric measurements

Changes in the groundwater level outside the excavation were followed on 4 open piezometers lined up at right angles to the excavation. To complement this, pore pressure measurements were carried out at 4 points at the level of the foot of the retaining wall to assess the foundation water pressure at the base of the excavation. Each measurement point was equipped with differential pressure cells linked to the open air by capillaries, this to compensate for variations in the atmospheric pressure.

The whole set of hydraulic pressure cells and vibrating-wire strain gauges, to which were added thermometric probes, was linked to an automatic measurement station. The analysis of the total set of measurements, read with a periodicity ranging from a few minutes to a few hours, was used in particular to assess the effect of temperature variations on the measurements, and thus correct them.

Stages of the work

After a first excavation to a depth of 1.8m, the first level of passive struts was placed 1.35m under the top the retaining wall. Excavation was then pursued in stages until a depth of 5.2m was reached; a second row of passive struts was placed at the -4.75m level.

When the excavation had reached the final depth of 8m, a geotextile was laid on the floor of the excavation, and a 30 to 40 cm gravel circulation layer was spread onto it. The excavation remained open in this state for 4 1/2 months before concreting of the lower floor, followed by construction of the tunnel.

Behaviour of the retaining wall

Displacement

Figure 3 shows the development of the retaining wall displacement as a function of the progress of the excavation and the placement of the struts.

The displacements remained small during the intermediary stages of the work; however, an important displacement of the foot of the retaining wall occurred during the last stage of the excavation: a likely hypothesis is that, at that moment, the very short depth of embedment of the retaining wall

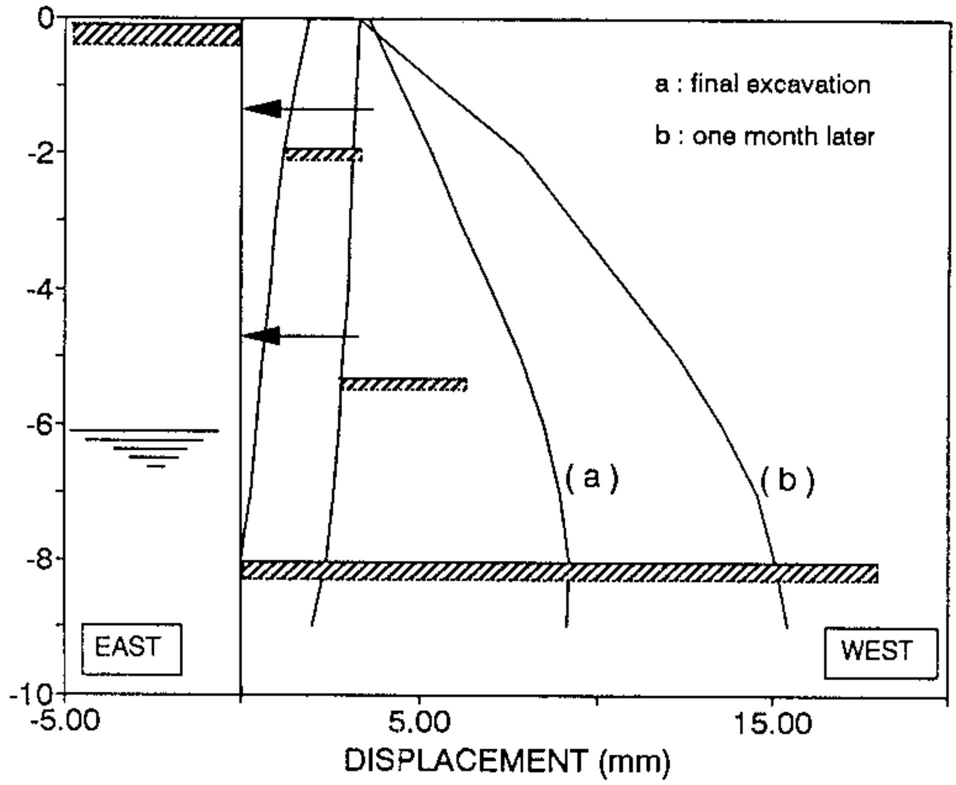

Fig. 3. Development of the displacement of the east retaining wall

produces a stress in the soil which is very close the passive pressure limit. The foot of the retaining wall is then very inefficiently held by the soil, especially as the bulk specific gravity of the soil is reduced by the vertical ascending flow between the two retaining walls, thus reducing the passive pressure available on the trench side of the embedment. A non-negligible creep of the soil appeared later, with an important evolution of the displacement during one month, varying at the base from 9mm to more than 15mm, with a subsequent stabilization.

When the east and west retaining walls are compared (Fig. 4), the asymmetry appears very clearly, with general displacement towards the west. This dissymmetry might be caused by the movement and parking of earthwork machinery, and by the construction of a 40cm thick embankment on the western side, very close to the excavation.

Stresses in the struts.

Figure 5 shows the evolution of stresses in the upper and lower struts as a function of the progress of the excavation. It appears that the stresses are not evenly spread between the two support levels, the stresses in the lower struts being inferior to those in the upper struts.

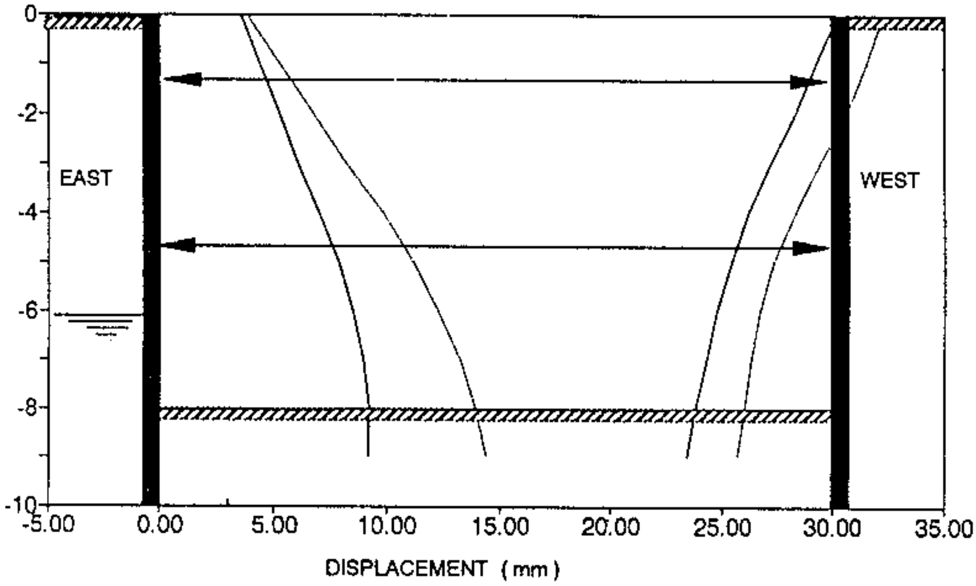

Fig. 4. Displacement of the east and west retaining walls during the final stage of excavation

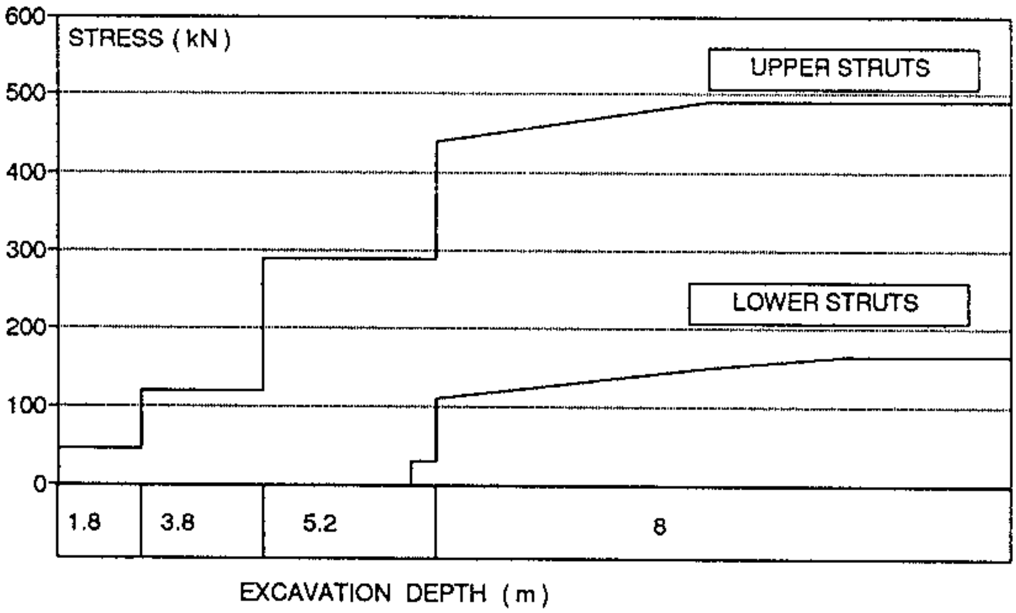

Fig. 5. Evolution of stresses in the struts

With the simultaneous measurement of stresses and displacements at the level of the struts, their real compressibility can be determined. Assuming that the solicitations are compressive, their theoretical stiffness ($\Delta F/\Delta l$) is equal to 8.10^5 kN/m. The experimental stiffness value for the upper struts is 8.10^4 kN/m and falls to 4.10^4: kN/m for the lower struts, i.e. 10 to 20 times smaller than the theoretical values. This difference has various causes

- the struts have to bear a bending as well as a compression stress
- the bearing between fender waling and the wall as well as the bearing between the strut and the fender waling introduce an additional compressibility.

When examining these results, one is led to think that the lower struts should have been prestressed to about 300 kN when being set up, which would have spread the stresses more evenly while also ensuring that the foot of the retaining wall was held more securely, thus making up for insufficient embedment depth. This prestressing might also compensate for the excessive compressibility of the strutting.

Comparison with calculations

The temporary retaining walls of the Lyon subway were dimensioned by using a method based on the reaction modulus hypothesis. The program used (RIDO) took separate account of the soil pressure on either side of the structure. The reaction law is linear, limited by the active and passive pressures, and with an irreversible behaviour once these limits have been reached (ref.2).

Calculations data

Since the drainage is fast, all calculations were made for a drained behaviour. Active and passive pressures were determined from the Caquot and Kerisel values, with the hypothesis that the soil-wall frictions are respectively equal to 2/3 and -2/3 of Φ. One of the sensitive points of these calculations is the choice of the reaction modulus: as a first step, we used the values recommended by the Laboratoire Central des Ponts et Chaussees (ref.3) calculated from the value of the pressuremetric modulus (i.e. kh varying from 1 000 to 4 000 kN/m^3 depending on the soil horizons in question). These values led immediately from the start of excavation, to displacement values considerably larger than those found in situ: for the first stage of earthwork, the movement calculated at the head of the retaining wall reached more than 12mm for a measured movement of only 2mm. It is possible that the LCPC recommended values for the determination of kh, calibrated on a few sites, are not well adapted to the soil in question. It is also possible that, with these stilts, which are very sensitive to disturbance, conventional pressuremetric tests lead to an important underestimate of the modulus, and that a self-boring pressuremeter might be better if a realistic estimate of the modulus of the undisturbed soil is wanted.

Following these first calculations, the values of the reaction modulus were modified so that the calculations would show displacement values compatible with those measured. The following characteristics were finally adopted

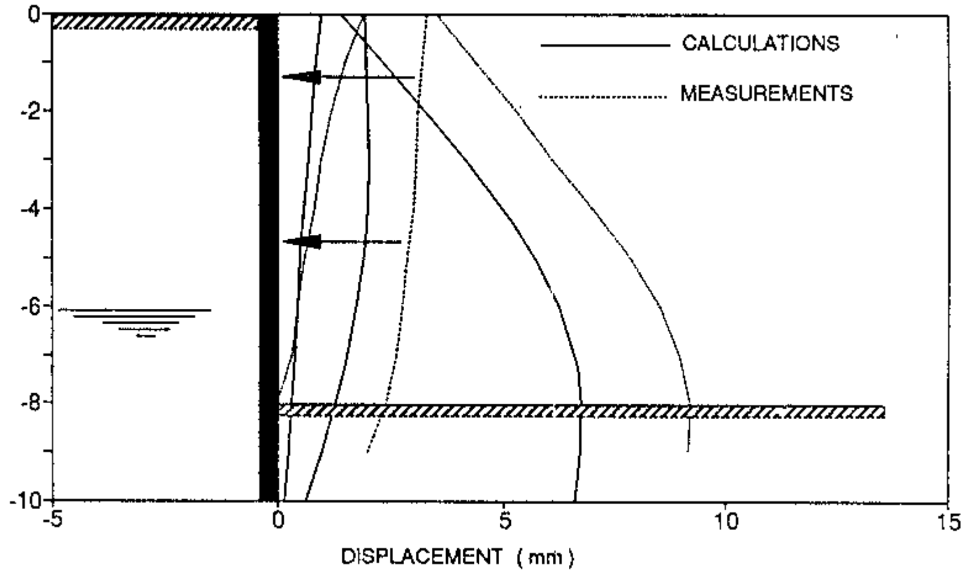

Fig. 6 . Comparison between calculated and measured displacements

Between 0 and 9 m

$\Phi = 27°$
$Ka\gamma.\cos\delta = 0.3$
$Kp\gamma.\cos\delta = 4.1$
$kh = 2.10^4\ kN/m^3$

Between 9 and 10.5 m

$\Phi = 40°$
$Ka\gamma.\cos\delta = 0.15$
$Kp\gamma.\cos\delta = 10.7$
$kh = 4.10^4\ kN/m^3$

Remark: below the aquifer, cohesion is nil; above, a capillary cohesion of 10 kPa was adopted.

In Fig. 6, the displacements calculated with those data are compared with the displacements measured on the retaining wall during the main stages of excavation. For the final stage, the value of the stress that can be summoned on the trench side of the embedment is reduced on an inclusive basis to take into account the effect of the vertical ascending flow. It should be noted that the displacement at the foot of the retaining wall is highly sensitive to this reduction, and as a result the movement of the retaining wall at this stage can only be very roughly estimated.

The results shown here correspond to a 40% reduction of the passive pressure at maximum excavation.

Table 1. Development of stresses in the struts

Depth of excavation		upper struts	lower struts
5.2 m	in situ	290 kN	
	calculation	172 kN	
8 m	in situ	440 kN	110 kN
	calculation	232 kN	112 kN
7.6 m	in situ	490 kN	165 kN
	calculation	226 kN	161 kN

It should also be noted that the calculated measured displacement curves are very similar in appearance with, however, a difference in translation reaching 3mm. This difference might be due to the general western movement of both retaining walls, of the same order in size and which cannot be simulated by the program used since the latter works under the hypothesis that the excavation is symmetrical.

Stresses in the struts

Table 1 shows the calculated and measured values of the compression stress in the struts. The values are fairly close for the lower struts; however, the calculations, to a large extent underestimate the stresses in the upper struts, while the difference increases as a function of the progress of the excavation. This phenomenon, already encountered on other experimental sites (ref.4), might be explained by the arching effect: the upper part of the retaining walls is fixed by the struts while the foot slips towards the excavation. This fixed point leads to a soil pressure concentration behind the retaining walls, pressure that has to be borne by the struts. Such pressure transfer cannot be described by the Winkler hypothesis, which does not take into account the relative displacements.

Conclusions

This experimental study allowed us to analyze the behaviour of the soil and the retaining walls, and to determine calculations hypotheses for future sites by showing in particular

- during the final stage, an important displacement of the foot of the retaining wall linked to the reduced embedment depth and to the reduction of the passive pressure that can be summoned when subjected to the vertical flow at the base of the excavation
- an effective stiffness of the strutting 10 to 20 times smaller than the theoretical stiffness, which might vindicate prestressing this strutting
- an underestimate of the reaction modulus by the traditional pressuremetric tests, which could well be replaced, in these sensitive silts, by tests with a selfboring pressuremeter
- an underestimate of stresses in the strutting when using the reaction modulus approach.

References

1. KASTNER R., PANTET H. and ONDEL C. Mesures en continu sur un soutenement du metro de lyon. *Conf. Int. Mesures et Essais en Genie Civil, Lyon* 1988, II, pp 222-230.
2. FAGES R. and BOUYAT C. Calcul des rideaux de parois moulees et de palplanches. *Travaux*, 1971, 439, pp 49-51.
3. BALAY J. *Recommandations pour le choix des paramstres de calcul des ecrans de soutenement par la methode aux modules de reaction.* Note d'information technique, 1984, LCPC, Paris.
4. KASTNER R. *Excavations profondes en site urbain. Problemes lies a la mise hors d'eau - Dimensionnement des soutenements butonnes.* These de Doctorat des Sciences, INSA Lyon et Universite Lyon 1 1982, p 409.

Anchored flexible retaining walls experiments on models: calculation by the reaction modulus method

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Introduction

The flexible and semi-flexible retaining walls which are frequently used for excavations in urban sites have to ensure the overall stability. These structures must eliminate or limit soil movements toward the excavation, because they can damage adjacent buildings and structures. The anchors, especially those which are prestressed, allow the stabilization of these movements and also stabilize the retaining structures to which they are connected.

A series of experiments on a bi-dimensional small scale model allowed us to study the influence of the anchor specifications: length, angle of inclination and prestressing force on the performance and the behaviour when in service of a retaining wall.

The design by reaction modulus method is compared to the results of experimental tests to clarify the possibilities and limitations of this method. For short anchors, the overall stability of the anchor/wall/ground is also studied and compared with the Kranz method and with french recommendations : TA 86 (ref.1).

Description of apparatus

Our experiments are realized on a model with the bi-dimensional material of Taylor-Schneebeli which is composed of 3, 4, 5mm rustproof steel rollers. ($\gamma_d/\gamma_w = 6.1$, $\phi = 21^o$, $c = 0$) .

The great density of this material will eliminate in part the simulation distortion due to the volume forces (ref.2). The model pile is made of duralumin and so could be considered smooth. It is in 3 sections (0.08 m width and 0.012 m thick by 0.805 m high) and is placed in a 1 x 2m roller mass (fig. 1). Its flexibility, as defined by Rowe (ref.3), is in the range of the semi-flexible retaining walls such as the retainings to be found in the underground system

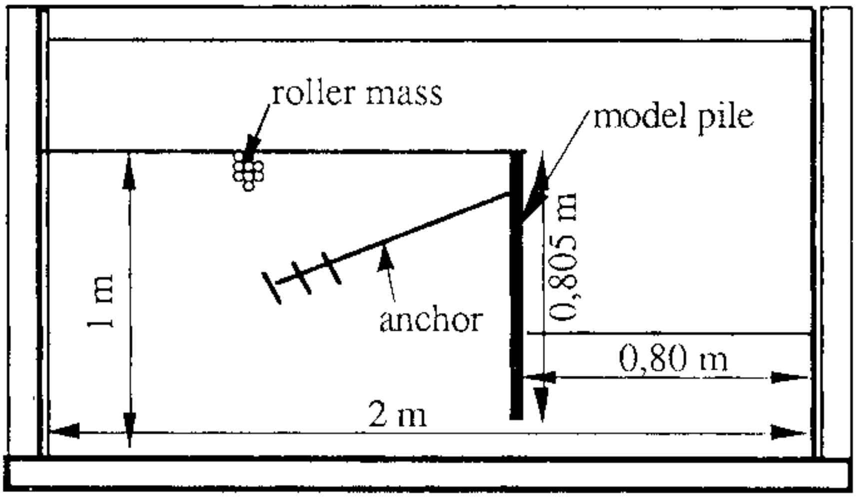

Fig. 1. Diagram of model

in Lyon.

The horizontal movements of the wall at its top and bottom are measured with mechanical dial gauges. The model pile's deflection is measured on 20 levels by 30 strain gauges mounted on both sides of the wall, and permits us to obtain the displacement and the soil differential pressure curves by the least square method, double integration and double differentiation.

The model anchor is composed of a section of steel girder embedded in the rollers (secondary grout) and attached to the model pile by 2 anchor wires equipped with load cells. At every stage of excavation, the deflection of the pile, the displacement at the top and the bottom of the pile and the force in the anchor are measured.

Programme of tests

To study the influence of the different conditions of anchorage on the behaviour and the design of the semi-flexible retainings, we realized 20 experiments. The following parameters could be varied from test to test

- anchor length from 0.55 to 1.17 m
- anchor inclination from 15 to 39°
- anchor prestressing: every test with a passive anchor (maximum anchor load at failure = A_{max}) is repeated with a prestressed one ($A_{initial}$ = 1/2 or 2/3 A_{max}).

Test procedure

All tests are carried out in a similar manner. First the "soil" on the outside of the piles is dredged away to a depth of 0.15 m. After the fixation of the

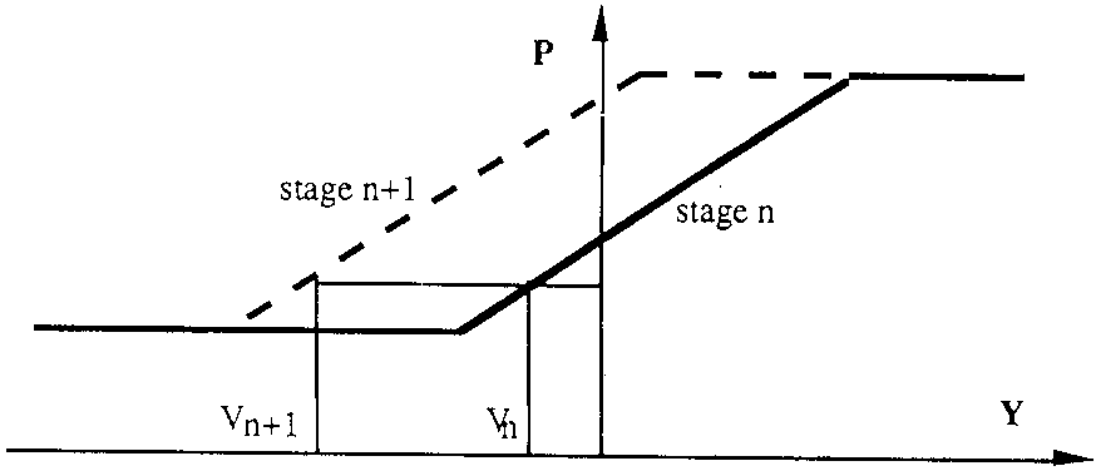

Fig 2. Load-deflection relationship of RIDO programme and its irreversibility

anchor at 0.05m from the top of the pile, the excavation is continued in stages of 0.05m until the collapse caused by a lack of passive pressure at about 0.57m of excavation. The classical design methods dictate an excavation depth of 0.45m for the optimum performance of the retaining wall (in service).

Design of anchored retaining walls

Here we study the performance of the wall in service by the Reaction Modulus Method. The overall stability of the anchor/wall/ground by the Kranz method is also verified.

The Reaction Modulus Design

The software we use is named RID03 (ref.4). It considers the ground action on the every side of the sheet pile with different load-deflection relationships by :

$P = P_0 + Kh\,(y - v)$

P_0: ground pressure distribution at rest

y: horizontal displacement of the pile at the studied point

v: the hysteresis factor

K_h: modulus of reaction defined by : $K_h = Re + Rp$

Re: constant rate of K_h

Rp: coefficient of increase of the effective vertical stress at the studied point.

P is limited by the active and passive pressures. If P overcomes these pressures the ground reaction will be irreversible, as shown in fig. 2.

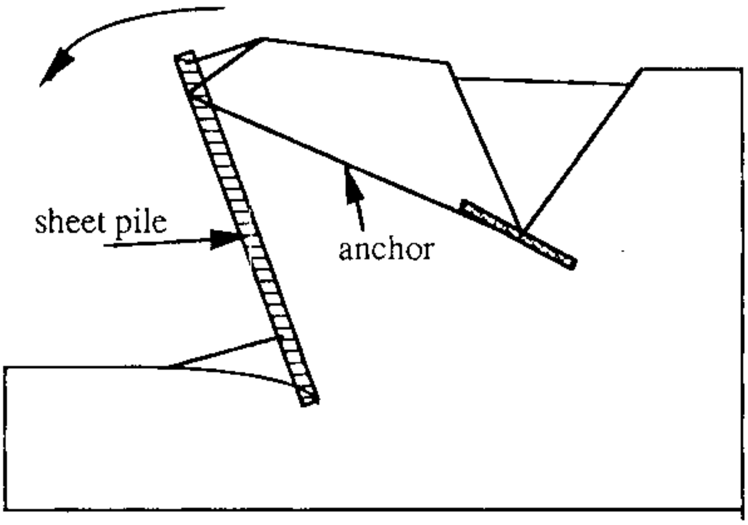

Fig. 3. Overall failure with a short anchor

The Kranz method of overall analysis

A very short anchor can cause an overall failure as shown in fig.3. Kranz (ref.5) proposed to calculate the anchor load in this case with a rectilinear failure plane which passes through the anchor secondary grout) and the assumed pivot point. The french recommendation TA 86 (ref.1) proposes to replace this linear plane by a circular one.

Comparison of calculated and measured values.

Although the failure caused by an insufficiency of embedded length can be correctly predicated by the active-passive calculations, here we are only interested by the optimum performance of the wall in service (excavation depth = 0.45 m).

The Reaction Modulus Design is done with the following values :

$K_{a\gamma} \cos \delta = 0.39$
$K_{p\gamma} \cos \delta = 3.1$
$K_h = R_p \sigma_v$
$R_p = 1500 \text{ m}^{-1}$
$\delta/\phi = 1$

The soil reaction modulus (K_h) cannot be measured, so we determined it by retro analysis of two preliminary tests. Independently of test conditions, this value is kept constant (ref.6).

Differential earth pressure

With passive anchors, there is a high pressure concentration at anchor level

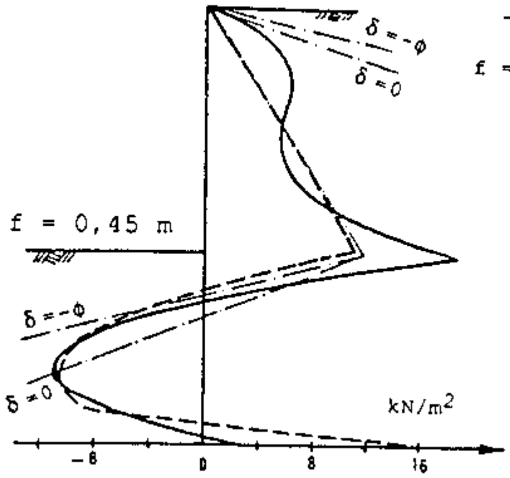

Fig 4. Differential earth pressure (passive anchor)

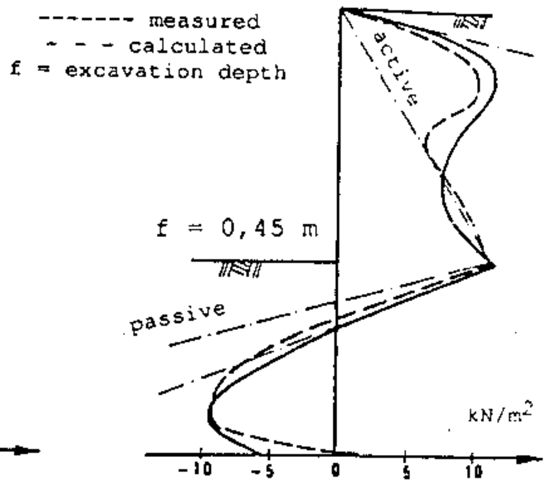

Fig 5. Differential earth pressure (prestressed anchor)

(soil arching). It is slightly less for short anchors, or very inclined ones, because of the great displacement of the model pile for such anchors. With the Reaction Modulus Method, this concentration is completely neglected and the distribution of soil pressure is triangular (fig. 4).

When a prestressed anchor is used, a considerable concentration of pressure acts at the anchor level, but it has nothing whatsoever to do with the arching on the active pressure side of the wall. This concentration is due to the passive reaction of the soil, because the anchor's prestressing directly loads it. This phenomenon is correctly calculated by RIDO to an excavation depth of 0.45 m, as shown in fig. 5, and for greater depth values, pile movements toward the excavation increase and the earth pressure decreases and becomes gradually triangular.

Anchor load

The anchor load is influenced by the earth pressure. For passive anchors, the design anchor load is highly underestimated (up to 42%). This is because the Reaction Modulus Method does not consider the effects of soil arching. The magnitude of this difference appears to be less in prestressed anchors, because of the increase of earth pressure on both the measured and calculated curves.

Bending moments

The difference between the measured and calculated curves are smaller than those we obtained for earth pressure and anchor load because of their antagonistic effect on bending moment curves (figs. 6 and 7).

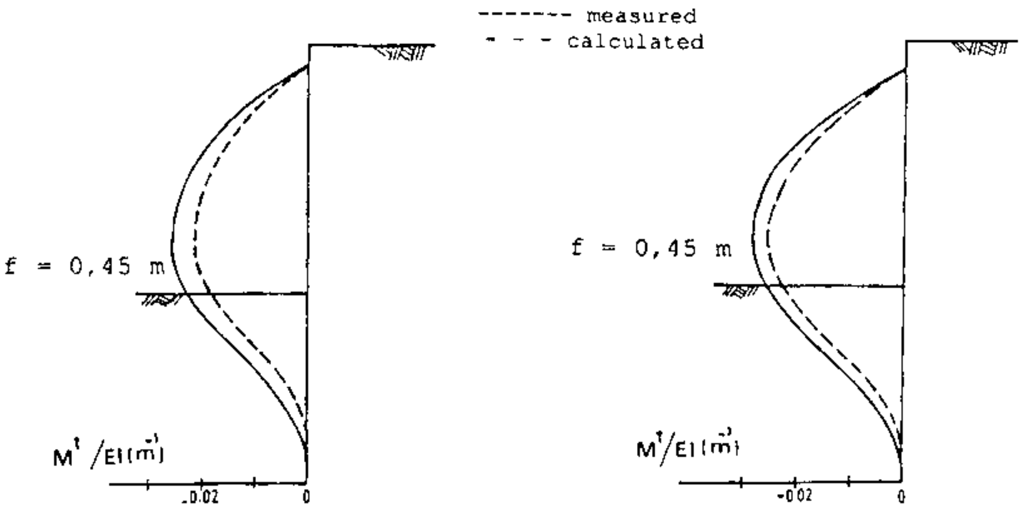

Fig 6. Bending moments (passive anchor)

Fig 7. Bending moments (prestressed anchor)

Wall displacements

Both the embedded length and the specifications of the anchor have a profound effect on the wall's movement. For long anchors, the design anchor load is underestimated and the design displacements are slightly less than those measured (fig. 8). When the anchor is short, this difference is larger, because the transposition of the anchor assembly is added to the elastic displacements (fig 9).

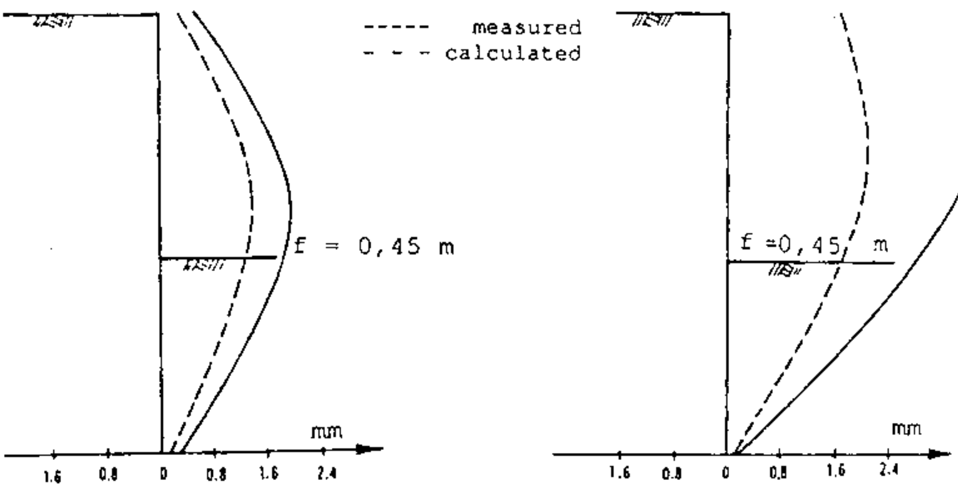

Fig. 8. Deflection curve (long anchor)

Fig.9. Deflection curve (short anchor)

Overall stability

An experiment conducted with a short anchor (length = 0.65m and angle of inclination = 22°) and continued until overall failure, is studied by stereophotogrammetry method as shown in fig. 10.

The analysis of the displacement field shows that :

- an arching pressure is developed between the secondary grout and the point of fixation of the anchor
- the slip line which limits the sliding wedge is more similar to the linear failure plane proposed by Kranz than the circular failure plane of french recommendations TA 86.

The anchor load at the point of overall failure calculated by Kranz (23 kN) and TA 86 (32 kN) are similar, but the measured load is much greater; 155 kN. To study the reason for this great difference, we analyzed the influence of the different parameters

(*a*) the active pressure distribution on the failure plane (in TA 86)
(*b*) the inclination of the active pressure on the wall (δ)
(*c*) the magnitude and distribution of this pressure
(*d*) the position of the pivot point.

It seems that the third point is the most important. The measured active pressure is only 20% greater than the calculated one, and the substitution of the theoretical triangular value by the real active pressure value decreases this difference. If we substitute all of the above values by the measured and

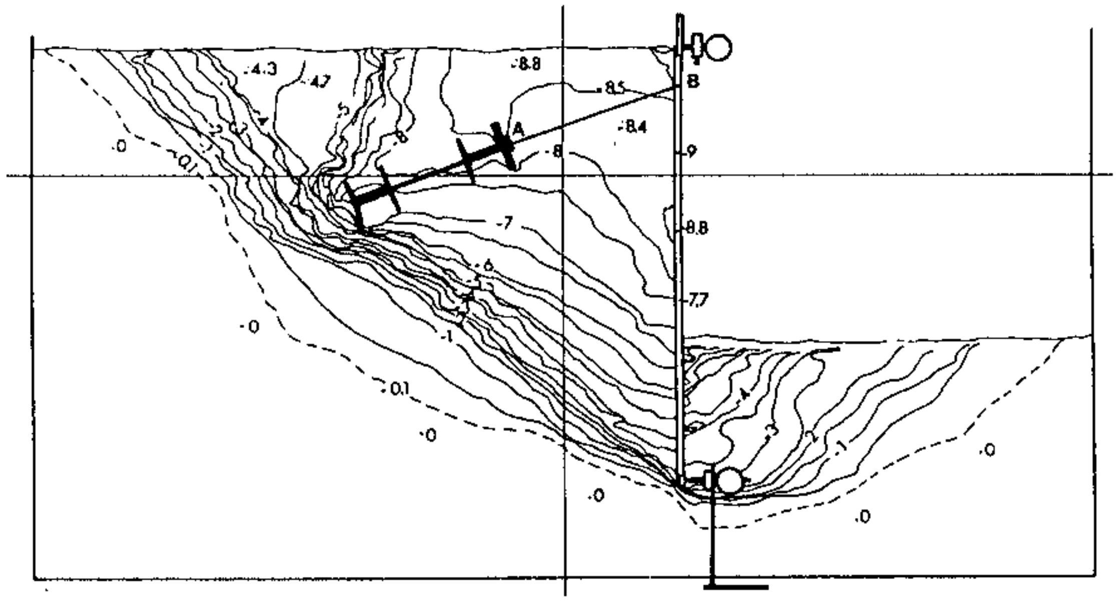

Fig. 10. Stereo photogrammetric image of overall failure (mm)

real values, the anchor loads calculated by the Kranz Method and TA 86 are very similar to the measured values. Therefore the choice of a circular failure plane, that leads to complicated calculations does not seem justified to us.

Conclusions

The bi-dimensional model we used permitted us to study the influence of different anchor specifications on the behaviour of the semi-flexible retaining walls, and also to clarify the limits of the Reaction Modulus Design as well as the overall stability.

However, we recommend that care is taken when the results are translated into full-scale structures, because this model does not take into consideration all of the similitude conditions.

Reaction Modulus Design

When there is no prestess in the anchors, this design method neglects the soil arching and highly underestimates the active earth pressure behind the anchor level. On the other hand, the calculated anchor load is smaller than the measured force. These two effects are antagonistic towards the design bending moment which is very similar to the measured curves. When there is a high prestess value, the Reaction Modulus Design correctly estimates the performance of the structure in service. The differences become important with the increase of excavation depth.

Overall stability:

The exploitation of the displacement field by stereophotogrammetry helps to visualise accurately the phenomenon of overall failure with a short anchor.

The design of the anchor load causing the failure with the Kranz Method represents a high sensitivity to the distribution of the active pressure on the retaining wall, but the choice of a circular failure plane, instead of a linear one, is not justified.

References

1. BUREAUX SECURITAS. *Recommandation concernant la conception, le calcul, l'execution et le controle des tirants d'ancrage. recommandations T.A.86, 3eme edition,* Bureaux Securitas, 1986, Paris.
2. G. SCHNEEBELI. Une analogie mecanique pour l'etude de la stabilite des ouvrages en terre à deux dimensions. *IVeme Congres International de Mecanique des Sols et des Travaux de Fondation,* 1957, vol. 2, Londre, pp 228-232.
3. P.W. ROWE. Anchored sheet pile walls. *Proc. Instn Civ. Engrs,* 1952, 1, , 27-70.

4. R. FAGES and C. BOUYAT. Calcul de rideaux de parois moulees ou de palplanches. *Travaux*, 1971, 1 , 439, pp 21-51.
5. E. KRANZ. *Über die Verankerung von Spundwande.* 2nd edition, 1953, Wm. Ernst und Sohn, Berlin.
6. F. MASROURI. *Comportement des rideaux de soutenement semi-flexibles: etude theorique et experimentale.* These de doctorat, 1986, INSA Lyon, 247pp.