THE MENARD PRESSUREMETER

Interpretation and application of Pressuremeter test Results to foundation design

GENERAL MEMORANDUM

D-60 AN

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Brochures in the same series

1. Settlement Calculation of a Vertically Loaded Pile
   A paper by M. Gambin in French, with an extended English summary and figure captions in English too (Sols Soils N° 7, 1963).

2. Calculation of Foundations Subjected to Horizontal Forces Using Pressuremeter Data (Sols Soils N° 30/31, 1979).

Contents

Introduction

This general note is a review of the technical rules adopted by specialist engineers employing pressuremeter methods. It has been edited for incorporation in soil investigation reports thus allowing the reader a full understanding of the pressuremeter analysis.

The notice first describes the practical procedures that must be respected by the geotechnicians and drilling personnel in carrying out the borings and the pressuremeter tests. The drilling equipment and test method used will vary in function of the nature of the soil encountered and the type of study undertaken.

Then, following the rules for the interpretation of the results, based on the pressuremeter theory as well as the experience gained from a large number of full-scale tests. These rules relate to the most usual cases, from the point of view of soil mechanics: apartment buildings, bridges, reservoirs, etc... Special notices are also available for studies of a particular nature: sheet pile or diaphragm walls, piles subjected to horizontal forces, stability of slopes and excavations, transmission towers, foundations subject to alternating loads or to vibrations, foundations on rock, tunnels, dams, roads, etc...

It is reminded that the pressuremeter test is essentially an in-situ load test carried out within a borehole on the actual site being investigated. Analysis of the stress/deformation diagrams obtained for each meter of penetration permits the evaluation of the mechanical properties of the soil on which are based the calculations for the foundations.

1. Performance of the pressuremeter test

1.1. Equipment

The pressuremeter (fig. 1) consists of two main elements: a radially expandable cylindrical probe which is placed inside a borehole at the desired test elevation and a monitoring unit which remains on the ground surface. The probe is made up of three independent cells and consequently exerts at the central cell level a strictly uniform pressure against the borehole walls. The pressure is increased in equal increments of time and pressure and the resulting borehole expansions recorded (fig. 2).

The instruments currently in use are the types G and GC. The monitoring device is connected to the probe by what appears to be a single flexible plastic tubing but which is in fact two tubes one inside the other. The inner tube carries the water to the central cell and the space between the two tubings allows the gas to reach the guard cells; this prevents the possible expansion of the inner tubing which would lead to erroneous readings of the amount of water injected.

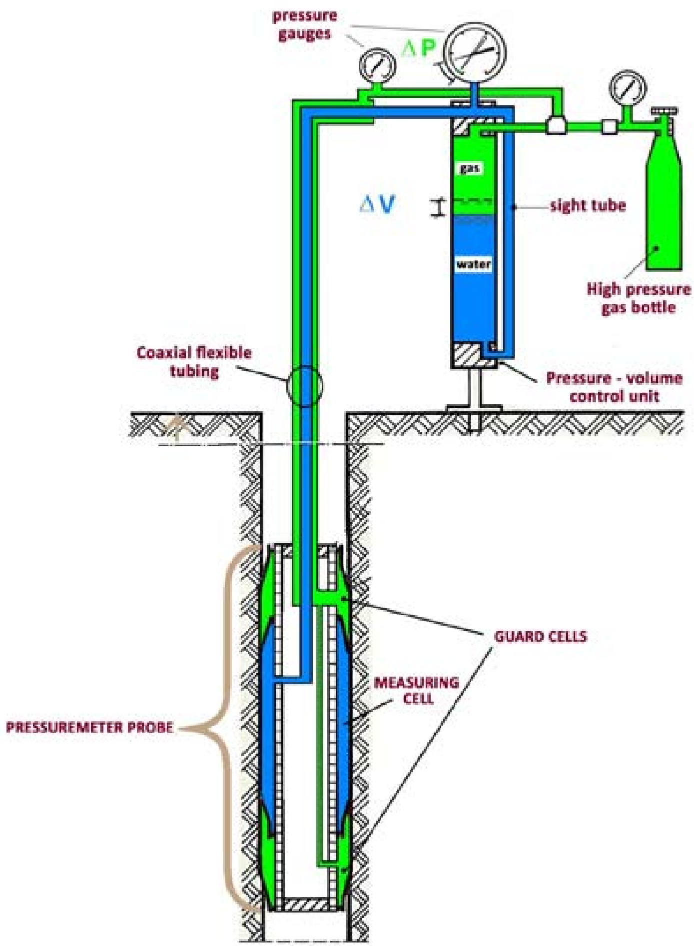
Pressure is applied to the water through pneumatic control equipment and the resulting soil deformations are indicated by volume changes which are read on a sight tube.

Each instrument can be supplied with a serie of probes which correspond to the most usual borehole dimensions.

DCDMA code	Diameter of the probe (mm)	Borehole diameter (mm)	
		mini.	maxi.
EX	32	34	38
AX	44	46	52
BX	58	60	66
NX	74	76	80

The standard metallic probe covers will allow testing the majority of soils.

GENERAL SCHEME OF A MÉNARD PRESSUREMETER



For very loose compressible soils characterized by limit pressures of less than 1.5 bar, very flexible membranes and covers with an overall inertia of 0.6 bar should be used.

On the other hand, for very high moduli (above 20.000 bars), stiff covers and membranes shall be used, previously calibrated; these exhibit a much reduced and more uniform compressibility.

When the volume variation, for a pressure increment of 1 bar, becomes less than 0.5 cm^3 (modulus above 4.000 bars), the volumetric shunt must be used, magnifying 50 times the sensitivity of the volume readings.

Finally, it is appropriate to record the recent introduction of a simpler apparatus known as the minipressuremeter which is adapted for compaction control and soil investigations at shallow depth. In this case, the probes employed are shorter, their diameters being 22 and 32 mm.

1.2. General notes on placing the pressuremeter probe

1.2.1. In submerged loose granular soils (sand, sand and gravel below water), the probe must be driven to the required elevation either by driving as in the «Standard Penetration Test», or by static pressure such as in the Dutch cone soundings, or again by vibration; in this case, the probe is protected by a casing with longitudinal slits which allow radial expansion. This method results in soil particle movement and slight compaction near the point, but as long as the soil is granular, the effects on the test results are negligible.

On the contrary, silty or clayey soils, especially below the water table, are not completely insensitive to driving, to static penetration or even to vibration; it is therefore recommended that the probe should not be placed by these methods for very precise investigations of settlements as this may result in altering the elastic properties of the soil. These methods nevertheless remain valid for the majority of foundation studies and do not alter the failure characteristics.

1.2.2. In many cases, the probe can be lowered in boreholes pre-drilled with flight augers, roller bits or rock bits. A rotary percussion wagon drill may be used provided the drilling fluid is a bentonite slurry. The traditional sounding equipment used for obtaining laboratory samples is not always suitable for pressuremeter drilling as it often leads to disturbance of the borehole walls due to the slow rate of advancement, the circulation of water under pressure between the walls of the hole and the core barrel and the vibration of the drill rods.

The tests should be carried out after each pass of the drill which itself must be limited to a length varying from 3 to 30 m according to the nature and sensitivity of the ground.

1.2.3. Very compact deposits of sand or sand and gravel require a mixed method of driving and drilling:

a) a large diameter casing is driven and the inside washed out; a small pilot hole is then drilled ahead over a short distance and an ordinary «slotted casing» driven into it. The tests are carried out as drilling advances, or

b) an open ended «slotted casing» attached to a casing of the same diameter is driven into the soil following the path of a pilot hole drilled through the casing and slightly ahead of it. The tests may then be carried out after completion of the drilling operation as the casing is being extracted.

1.3. Soil identification

As it will be seen later, the shape of the pressuremeter curves, the values of limit and creep pressures, the moduli E and the relations between moduli E and limit pressures (see para. 2.4), give precise indications of the nature of the investigated layers. These results are complemented by visual examination of soil samples whether they be augered, from a sampler or cuttings from wash boring. The rate of drilling advancement and/or the driving records supply additional information as to the layers encountered.

When particularly delicate geological conditions are encountered, an additional borehole with near continuous sampling should be provided for.

1.4. Carrying out the test

1.4.1. The standard test must be carried out less than 24 hours after the drilling operation except in the case of driven-in probes where there is no risk of alteration of the soil due to water absorption. However, tolerances of several days are admissible for boreholes carried out without water (hand augering, drilling with air scavenging) above the water table.

Whatever the investigation, the tests must be carried out systematically, meter by meter in order to record accurately the variations of strength parameters in function of depth. Discontinuities in the test spacing are not allowable; the measurements are thus practically continuous and enable complete information to be obtained concerning the various layers.

Emphasis is placed on the necessity for continuous pressuremeter testing from the ground level, whatever may be the depth previously prescribed for the foundations. This is to ensure a better understanding of the site by a complete study of the different layers and an exact determination of the «equivalent embedment» and the possible active earth pressures.

1.4.2. The test itself is standardized and should be carried out with ten equal loading increments (5 to 14 increments will be tolerated) up to the point of failure. Readings of deformations with respect to time are taken for each pressure increment at 15 seconds, 30 seconds and one minute after the application of this increment (fig. 2).

1.4.3. To obtain as complete a load deformation curve as possible, the measured volume should reach 700 cm if pl < 8 bars and 600 cm3 if 8 < pl. < 15 bars. In other cases, the test must be carried up to 20 to 25 bars pressure in soils and up to 50 to 70 bars in rocks.

2. Analysis of results obtained

2.1. Characteristics measured

From the load deformation diagrams thus obtained at each elevation, the main mechanical characteristics of the soil are calculated; these are: the deformation modulus E and the limit pressure pl at failure.

2.1.1 The pressuremeter modulus E is a distortion modulus of the soil measured in a deviatoric stress field. It characterizes the pseudo-elastic phase of the test. Obviously, it must not be confused with the oedometer modulus (measured in an isotropic or spherical stress conditions), although precise experimental relationships exist between them as will be seen later. The distortion modulus plays an essential role in the calculation of the settlement of
foundations and is generally more important than the oedometer modulus.

2.1.2. The limit pressure pl by definition corresponds to the limiting state of failure of a soil subjected to an increasing uniform pressure on the wail of a cylindrical cavity. This fundamental mechanical characteristic enters into all the analysises of foundation stability carried out in accordance with pressuremeter methods.

2.1.3. The test also permits the calculation of other characteristics of the soil: the creep pressure or elastic limit (fig. 2-c), creep coefficient, the natural «at-rest» pressure. These characteristics only enter into very special studies and do not appear in the usual reports.

2.2. Calculation of modulus E and limit pressure pL

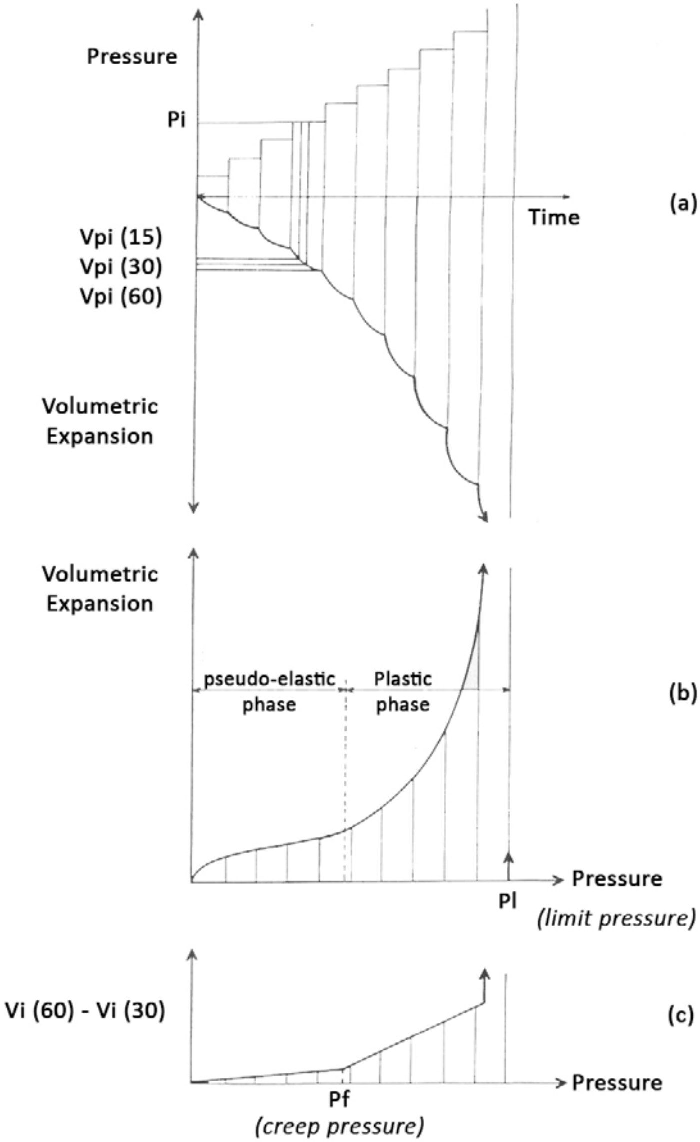
2.2.1. Computation of the modulus E

In an elastic media, the radial expansion of a cylindrical cavity is related to the pressure by the equation:

$$\frac{\Delta r}{r} = \frac{(1+\sigma)}{E}\,\Delta p \qquad (1)$$

$$\text{or} \quad E = (1+\sigma)\,\frac{\Delta p}{\Delta r} \qquad (2)$$

Where σ is Poisson’s ratio, arbitrarily assigned a value of 0.33.



(a) pressure and volumetri
expansion agains time

(b) the pressuremeter curv

(c) the creep curve

When expressed m function of volume instead of radius, equation (2) becomes

$$E = (1+\sigma)\ 2V\ \frac{\Delta p}{\Delta v}$$

Where V is the volume of the cavity at the instant when $\frac{\Delta p}{\Delta v}$ is measured, provided $\frac{\Delta p}{\Delta v}$ is measured in the pseudo-elastic phase of the test (fig. 3-a)

The modulus derived from equation (3) is referred to as the «pressuremeter modulus».

In a pressure meter test, the volume V depends on the size of the probe used; it is the sum of two components

$V = V_o + V_m$

where V_o is the initial or «at rest» volume of the measuring cell

and V_m is the mean additional volume injected (read directly on the sight tube of the instrument).

Equation (3) can therefore be expressed as

$$\mathbf{E = k\ \frac{\Delta p}{\Delta v}}$$

where K is a dimensional coefficient of the probe

$$k = (1+\sigma)\ 2\ (V_o+V_m)$$
$$= 2{,}66(V_o+V_m)$$

(The arbitrary value of 0.33 assigned to Poisson's ratio has no influence on the estimation of settlements. The same value has been assigned in the term $\frac{(1+\sigma)}{E}$ which appears in the settlement formulae).

The value of K for $v_m \approx$ 200 cm3 are as follows for standard probes (these do not apply to special BX and NX probes with longer central cells for greater accuracy).

Probe	Diameter of borehole (mm)	Vo (cm3)	K (cm3)
EX	34	535	
AX	44	535	2000
BX	60	535	
NX	76	790	2700

The nomogram found on the following page yields immediately the values of the modulus E for standard EX, AX, and BX probes.

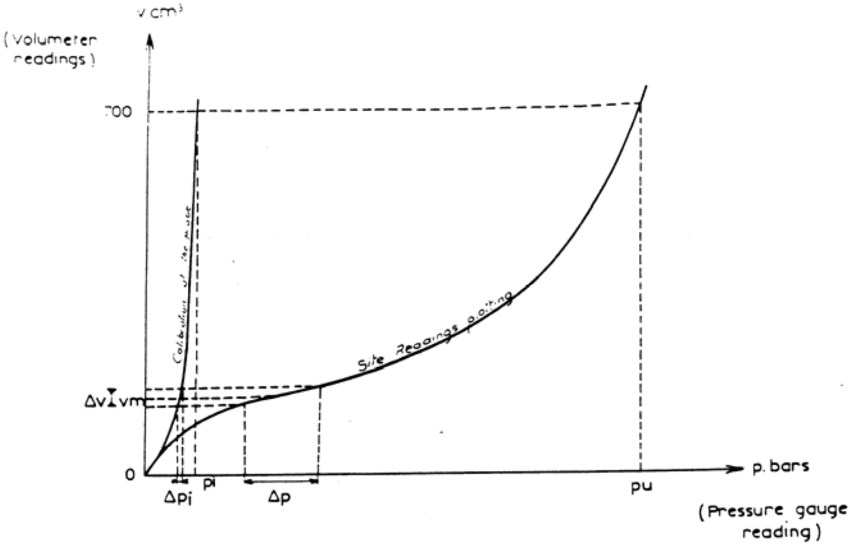
2.2.2. Computation of the limit pressure pl

By definition, the limit pressure is the abscissa of the asymptote to the pressuremeter curve. It can be determined directly from the curve but more conventionally it is taken as the pressure corresponding to a volume increase ΔV equal to the initial volume of the borehole V ($\Delta V/V = 1$).

Since the initial volume for a standard AX or BX probe is in the order of 600 cc (535 cm3 + volume injected to contact the borehole walls) $\Delta V/V$ ma-be assumed to occur for a reading of V = 700 cc

If, during a test, the volumetric increase attained is less than specified above, it is still possible to extrapolate with precision the value of the limit pressure provided the creep pressure has been exceeded during the test. This extrapolation is based on the relative or reciprocal volumes theories.

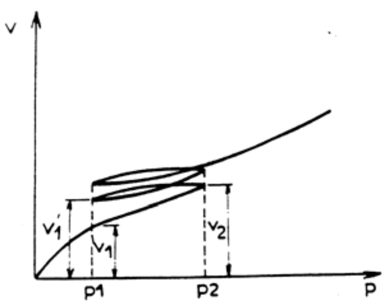
In the latter case, the last few readings corresponding to the plastic phase are plotted on a p,1/v scale paper; these should plot as part of a straight line; the point at which the extension of the straight line intersects the 700-cc ordinate corresponds to the limit pressure.



$$E = K\,(vm)\,\frac{\Delta p - \Delta p_i}{\Delta v} \simeq K\,\frac{\Delta p}{\Delta v}$$

K (v_m) can be derived from the next chart as long as water level is at 0 mark in the sight tube of the voltmeter when the probe is «at rest»

$$p_\ell = p_u - p_i + \text{water head}$$

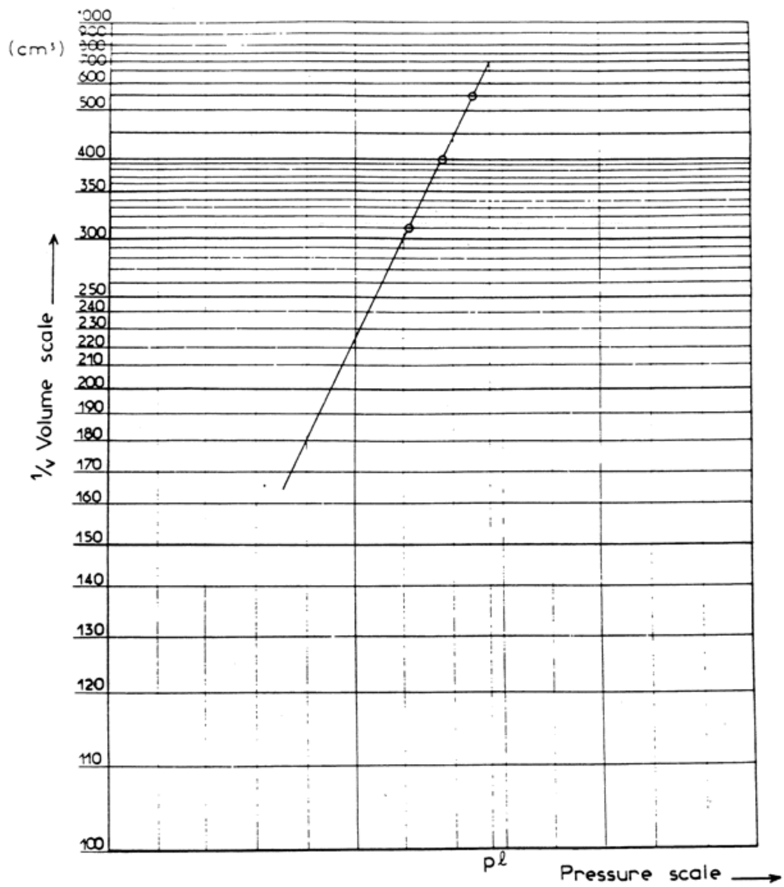


$$E = K\,\frac{p_2 - p_1}{V_2 - V_1}$$

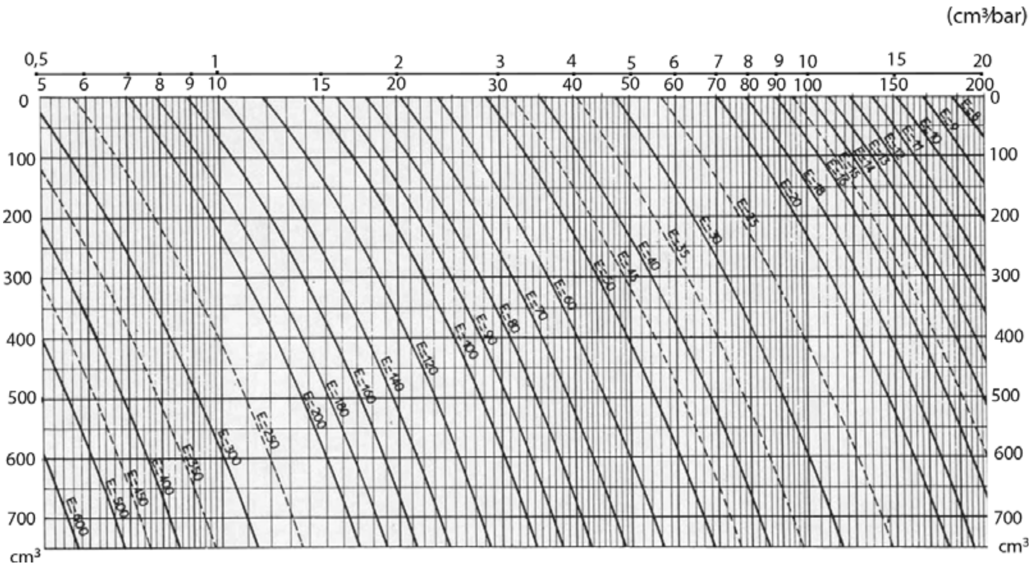
$$E_R = K\,\frac{p_2 - p_1}{V_2 - V_1'}$$

$$E_A = AVG = K\,\frac{p_2 - p_1}{V_2'' - V_1''}$$

Fig 3



How to plot the plastic phase curve on a p, 1/v graph paper to obtain the limit pressure



Value of pressuremeter modulus E as function of the average volume reading and $\frac{\Delta p}{\Delta v}$ ratio (adjustments of Δ_v and Δ_p as required)

Standard probes 60mm, 44mm, 32mm OD

When using the upper $\frac{\Delta p}{\Delta v}$ scale, multiply the E values by 10

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An approximate value of the limit pressure can also be obtained considering the following statistical results:

- The creep pressure (or the end of the elastic phase) is equal to a half or two thirds of the limit pressure.

- For every geological formation, there is a constant relationship between E and pl according to the type of soil.

2.2.3. The pressure and volume readings recorded
on the site must be adjusted to compensate for:

- The head of water in the central cell tubing;

- The inertia of the assembly consisting of the membrane + cover + possibly a slotted tube (Δ pi or pi).

- The expansion of the central cell tubing under pressure.

The first two essentially affect the value of the limit pressure and the third, which is minimal, the pressuremeter modulus.

2.2.4. When the soil investigation relates to foundations subject to cyclic loadings or vibrations, it is of interest to carry out a number of loading and unloading cycles in the pseudo-elastic phase of the test. These cycles yield:

- A rebound modulus E_R.

- A cyclic modulus E_A which is a mean of repetitive loading and rebound cycles.

These are of particular value for any study related to motorways or vibrating machinery.

2.3. Presentation of results (fig.4)

The moduli and limit pressure values are presented graphically in function of depth in parallel with the logs and driving records.

The simultaneous presentation of all these results greatly facilitates the analysis of the soil conditions.

2.4.1 It may be useful, for readers unfamiliar with pressuremeter techniques to list the usual ranges of E and p_l for the principal types of soils.

2.4. Indicative values of E and pl

Soil	E in bars	pl in bars
Mud, peat	2 to 15	0.2 to 1.5
Soft clay	5 to 30	0.5 to 3
Medium clay	30 to 80	3 to 8
Stiff clay	80 to 400	6 to 20
Marl	50 to 600	6 to 40
Loose silty sand	5 to 20	1 to 5
Silt	20 to 100	2 to 15
Sand and gravel	80 to 400	12 to 50
Sedimentary sands	75 to 400	10 to 50
limestone	800 to 200.000	30 to over 100
Recent fill	5 to 50	0.5 to 3
Old fill	40 to 150	1 to 10

2.4.2. The ratio ***E/pl*** of the modulus and the limit pressure is a characteristic of the type of soil under examination; the higher values of E/pl_m (12 to 30) are encountered amongst over-consolidated soils such as the London clay; the low values of E, *pl* (5 to 8) are more prevalent with alluvial soils (sands and gravels, silty sands under water). This ratio should be systematically studied in order to follow the soil variations with precision and bring to light any accidental remoulding during the drilling operation which could result in a decrease of 20 to 30 % of E/pl_m.

The examination of the driving records are also useful for detecting local variations although these have no absolute meaning, whatever may be the opinion of some who believe in their adequacy.

2.5. Correlation between point resistance Rp (static penetrometer) and pl_m

It is recalled that the point resistance R_p of the static penetrometer is proportional to the modulus E and the limit pressure pl the ratios R_p/pl_m are constant for a given geological layer but vary with the grain sire distribution of the soil and its water content. The following relationships have been established, deduced from the pressuremeter theory and checked experimentally.

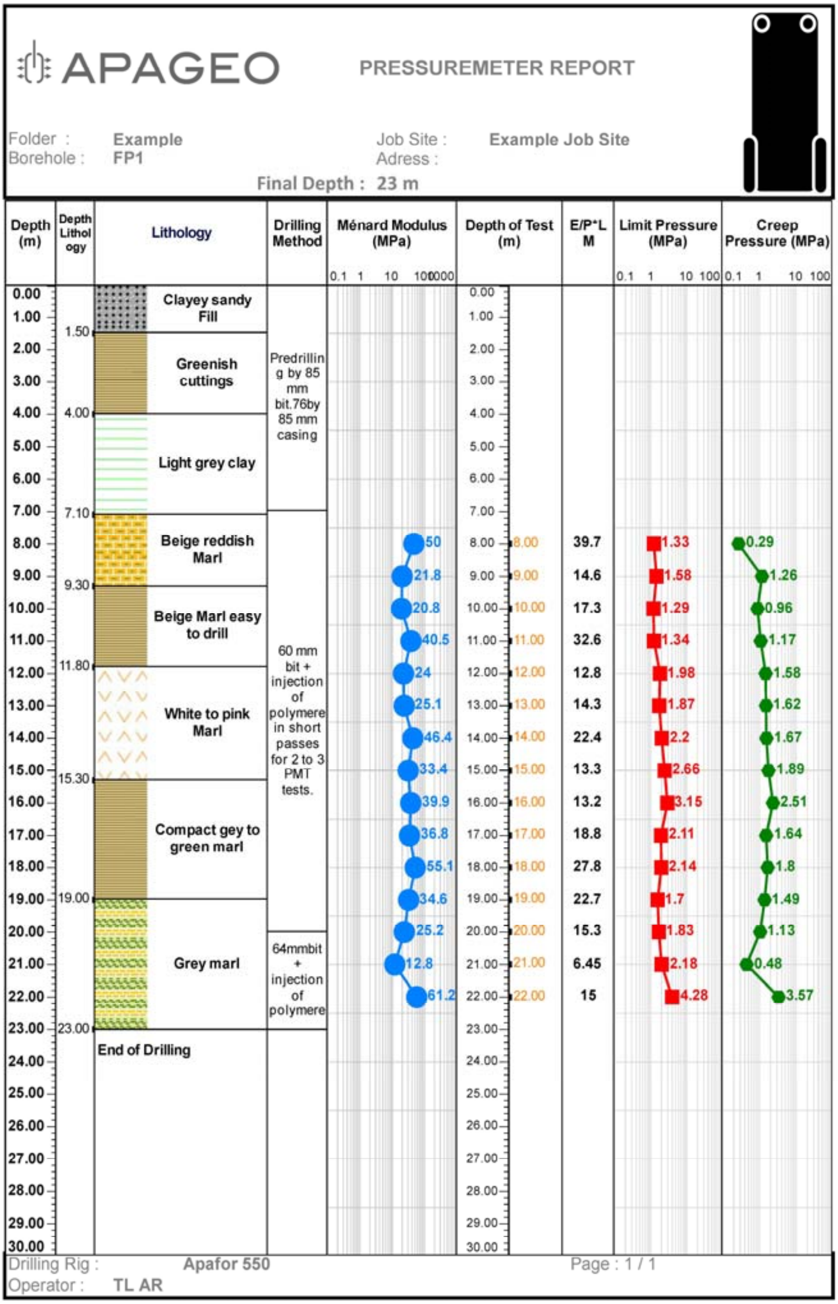

APAGEO
PRESSUREMETER REPORT
Folder : Example
Borehole : FP1
Job Site : Example Job Site
Adress :
Final Depth : 23 m
Depth (m)
Depth Lithology
Lithology
Drilling Method
Ménard Modulus (MPa)
Depth of Test (m)
E/P*LM
Limit Pressure (MPa)
Creep Pressure (MPa)
0.1 1 10 100 1000
0.1 1 10 100
0.00 1.00 2.00 3.00 4.00 5.00 6.00 7.00 8.00 9.00 10.00 11.00 12.00 13.00 14.00 15.00 16.00 17.00 18.00 19.00 20.00 21.00 22.00 23.00 24.00 25.00 26.00 27.00 28.00 29.00 30.00
1.50 4.00 7.10 9.30 11.80 15.30 19.00 23.00
Clayey sandy Fill
Greenish cuttings
Light grey clay
Beige reddish Marl
Beige Marl easy to drill
White to pink Marl
Compact gey to green marl
Grey marl
End of Drilling
Predrilling by 85 mm bit.76by 85 mm casing
60 mm bit + injection of polymere in short passes for 2 to 3 PMT tests.
64mmbit + injection of polymere
50 21.8 20.8 40.5 24 25.1 46.4 33.4 39.9 36.8 55.1 34.6 25.2 12.8 61.2
8.00 9.00 10.00 11.00 12.00 13.00 14.00 15.00 16.00 17.00 18.00 19.00 20.00 21.00 22.00
39.7 14.6 17.3 32.6 12.8 14.3 22.4 13.3 13.2 18.8 27.8 22.7 15.3 6.45 15
1.33 1.58 1.29 1.34 1.98 1.87 2.2 2.66 3.15 2.11 2.14 1.7 1.83 2.18 4.28
0.29 1.26 0.96 1.17 1.58 1.62 1.67 1.89 2.51 1.64 1.8 1.49 1.13 0.48 3.57
Drilling Rig : Apafor 550
Operator : TL AR
Page : 1 / 1

Fig 4

Type of soil	R_p/P_1
clay	2.5 to 4
silt	5 to 6
sand	7 to 9

2.6. Influence of the water-table level

The influence of the water-table level on the measured characteristics is appreciable and increases with the ratio E/pl_m of the soil. Thus, the saturation of an initially dry soil characterized by $E/pl_m = 20$ may result in a decrease of up to 40 % of the E values. This phenomenon must be taken into account when dealing with work founded on silts
situated in areas subject to flooding or large water table level variations.

2.7. General features of the site investigated

Before proceeding with the actual calculations for the foundations, it is necessary to study the general features of the site by a statistical analysis of the geotechnological results obtained. It will be seen later that the settlements (and the differential settlements in particular) rather than bearing capacity limit the loads that can be supported by a soil.

The site will therefore be defined not only by the average characteristics encountered but also, and above all, by their variation in plan and depth.

2.7.1. The study of differential settlement introduces the concept of specific settlement of the ground (the settlement of a foundation of one metre square loaded at one bar) at each sounding location and the percentage variations of the specific settlements between the different locations.

It is desirable, from the general study stage onwards, to utilize this same idea of standardized specific settlement for a typical case of loading (a case of three imaginary parallel strip footings 1 m wide at 4.5 m centres loaded at 1 bar. This settlement ws is calculated for each sounding location using the formula below (to be explained in paragraph ***4.6.1.).*** which takes into account the decrease of the stresses and the variations of moduli with depth by applying a weighting factor. The ground has been divided into layers 1m thick (except the first one which is only 50 cm) over a depth of 25 m to obtain

$$w_s = \sum_{i=0}^{i=25} \frac{n_i}{E_i}$$

where Ei is the pressuremeter modulus at depth i,

ni is the weighting factor at the same depth, the values of which are given below:

Depth (m)	0	1	2	3	4	5	6	7	8	9	10	11
Sand	11	10	5.3	3.7	2.9	2.2	1.7	1.2	0.9	0.7	0.5	0.45
Alluvium	16	13	7.8	5.8	4.6	3.5	2.8	2.2	1.7	1.4	1.1	0.9
Clay	21	18	10.3	7.5	6.0	5.0	4.0	3.3	2.8	2.3	1.9	1.6
Fill	34.5	30	18.0	15.0	13.3	11.6	10.0	8.6	7.3	6.3	5.5	4.7
Depth (m)	12	13	14	15	16	17	18	19	20	21 22	23 24	25
Sand	0.4	0.4	0.35	0.35	0.3	0.3	0.25	0.25	0.25	0.2	0.2	0.2
Alluvium	0.8	0.7	0.65	0.6	0.55	0.5	0.5	0.45	0.45	0.4	0.35	0.3
Clay	1.4	1.2	1.1	1.0	0.9	0.8	0.7	0.65	0.65	0.6	0.55	0.5
Fill	4.1	3.5	3.1	2.7	2.3	2.1	1.9	1.7	1.6	1.5	1.5	1.4

The settlement is obtained in centimeters for moduli expressed in bars.

Given that for the majority of soils, the settlement of a footing varies according to the level of the foundation, it is proper to specify the level selected for the calculation of the specific settlement (1 m depth is often chosen).

Thus, n values w_1, w_2, ... w_j, ... w_n of the specific settlement are obtained for n soundings F_1, F_2, F_l, F_n:
one sentence is lacking obviously!
«which can be analysed by the statiscal method. consequently derive»:

- The mean value of specific settlement:

$$w = \frac{\sum_{j=1}^{j=n} w_j}{n}$$

- The value of the standard deviation:

$$e = \sqrt{\frac{\sum_{j=1}^{j=n} (w_j - \bar{w})^2}{n-1}}$$

- Whence the variation index:

$$i = \frac{e}{w}$$

An analysis, in plan, of the values of the specific settlement may indicate clearly defined zones (weaker or stronger) deserving detailed consideration.

Finally, for very large investigations covering a large number of soundings, it is advisable to plot the curves of equal specific settlement: study of this «contour map» simplifies the evaluation of the site as a whole.

2.7.2. Examination of each of the pressuremeter soundings in relation to the geological conditions often makes possible a determination of the causes of local weaknesses (water table level, branch of an old river, developing cavings, underground river etc...). This is particularly valuable for estimating the possibility of extension or aggravation of any such weakness between soundings. Some complementary soundings may then appear necessary in order to clarify a local uncertainty and to eliminate risks of geotechnical nature.

3.Calculation of the bearing capacity

The pressuremeter test is a type of load test which in particular yields the limit pressure pl which corresponds to the failure of the soil. Experience and theory have shown that the ultimate bearing capacity of a foundation is proportional to pl, the factor proportionality being function of the relative depth and of the foundation shape.

This factor, called the bearing factor, has been the object of very many theoretical and experimental research works, a large number of which have been carried out at the Soils Studies Centre of Paris and published in the Sols-Soils magazine.

Whatever the type of foundations or nature of the soil involved, the direct pressuremeter method of foundation calculation presents the advantages of simplicity and greater accuracy over the conventional analysis which takes into account various parameters such as cohesion and internal angle of friction. Full scale loading tests carried out at the Soils Studies Centre, on footings, caissons and piles, have brought out the concept of critical depth of embedment : for a deep foundation (embedment greater than the critical embedment) the soil displaced by the vertical movement of the foundation is absorbed by elastic displacement of the surrounding soil whilst for a surface foundation (embedment less than the critical value), heave of the surrounding soil can be observed at the moment of failure, heave which is all the more pronounced than the embedment is the lesser.

3.1. Fundamental formula R.O

The ultimate bearing capacity of a foundation ql is related to the limit pressure pl of the soil by a linear function:

$$q_i . q_o = \boldsymbol{k} (p_i \; . p_o \;)$$

Where k is the bearing factor varying from 0.8 to 9 according to the embedment, the shape of the foundation and the nature of the soil.

q_o is the overburden pressure at the periphery of the foundation level after construction.

p_0 is the «at rest» horizontal earth pressure at the test level (at the time of the test).

The stresses po and q_o are «total stresses» (in particular po can be measured with a Geocell stress
captor).

3.2. Terminology

Often the net limit pressure pi is used which is equal to the difference between the limit pressure Pc effectively attained and the horizontal earth pressure at rest:

$$\mathbf{p}_i = \mathbf{p}_i \ \cdot \mathbf{p}_o$$

(In soil with no strength $\mathrm{p}_i = \mathrm{p}'_o \ \ i.e. \mathrm{p}_i = \mathbf{0}$

Further, it has often been the custom in dealing with pressuremeter results to tabulate the value pl - h, h being the head of water in the tubing above the probe.

The calculation and presentation of results is greatly simplified by identifying pi with pl_m - h, the error introduced being less than 2% for most dry sites.

Similarly, one can define the net ultimate bearing capacity q' of the ground as equal to the surcharge it can withstand before failure;

$$\mathbf{q'} = \mathbf{q}_i \ \cdot \mathbf{q}_o$$

(in a soil with no strength ql = qo and q' = 0) thus arriving at the simplified formula:

q' = k pl

which will be used in particular in the calculation of the end bearing resistance of piles and or caissons.

This simplification is obviously not valid for investigations in very soft soils (silt, peat) below the water table, especially if the depth is great.

When the ground level is submerged, the ultimate bearing capacity is

$$q_l \cdot \Upsilon h = k\,[(p_l \cdot h) \cdot K_O\, \Upsilon'\ h]$$

where h is the difference in elevation between the probe and the pressure gauge

Y' is the buoyant density of the soil

K
0 is the coefficient of horizontal earth pressure at rest

$[pl_m - h]$ is the limit pressure with no correction far the water held in the tubing.

Usually $K_O\ \Upsilon' = 0.55$ (when h is expressed in metre and pressures in t/m²) : experience shows that when $\Upsilon' \sim 0.55$ (very loose soil) $K_O \sim 1$, and when $\Upsilon' \sim 1.1$ (compact soil) $K_O \sim 0.5$.

3.3 Values of the bearing factor k

It has already been indicated that the value of k depends on the type of soil, the embedment and the shape of foundation.

3.3.1. Soil categories

From a practical point of view, the soils can be divided into four categories, according to the table below. It must be noted that recent fills and under-consolidated soils are not listed in this table; these will be examined in a later chapter (paragraph 4.9) dealing with the self-bearing boundary. One must keep in mind that the allowable bearing capacity is not only function of the ultimate bearing capacity of the soil, but also of the allowable settlements for the analyzed structure.

Caution = this chart was modified in 1983
in the French tender documents
(see Addendum)

BEARING FACTOR AGAINST EMBEDMENT

FOR ISOLATED FOOTINGS, PIERS AND PILES

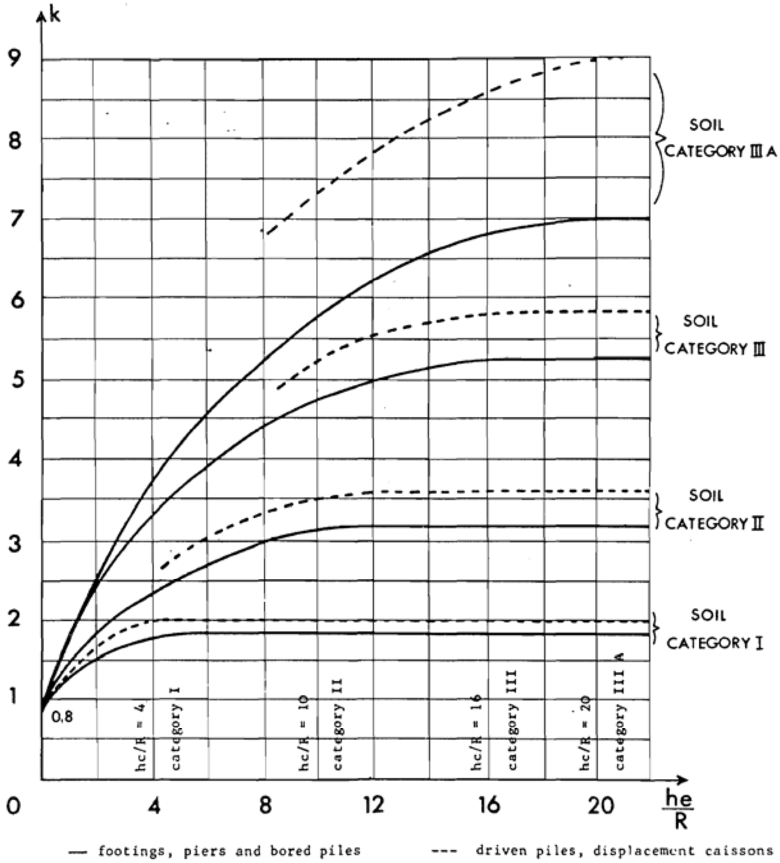


Fig 5a Bearing factor values

Caution = this chart was modified in 1983
in the French tender documents

(see Addendum)

BEARING FACTOR AGAINST EMBEDMENT

FOR STRIP FOOTINGS AND CAST-IN-SITU DIAPHRAGM WALLS

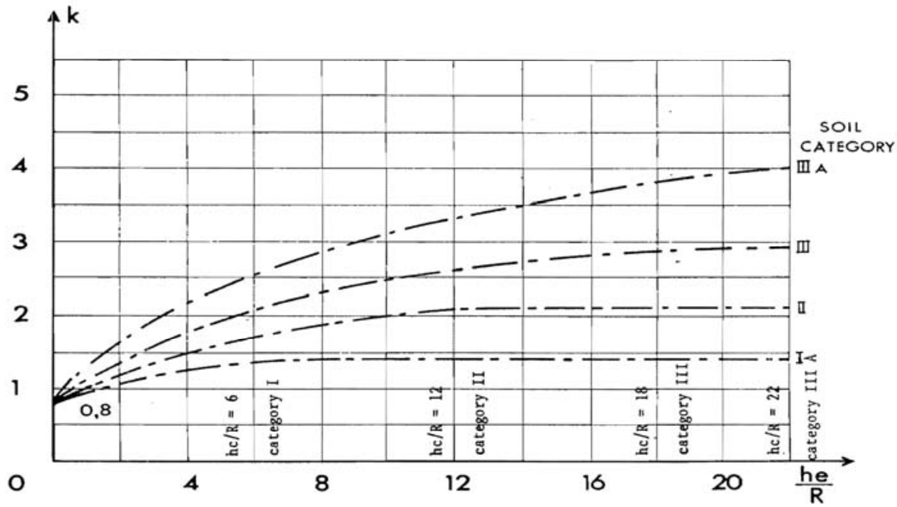


Fig 5b

Ranges of pressures limit pl	Nature of soil	Soil categories*
0. 12 bars 0. 7	Clay Silt	category I
18. 40 12. 30 4. 8 10. 30	Firm clay or marl Compact silt Compressible sand Soft or weathered rock	category II
10. 20 40. 100	Sand and gravel Rock	category III
30. 60	Very compact sand and gravel	category IIIA

3.3.2. Critical embedment hc

Below a certain depth, an embedded foundation maintains a constant ultimate bearing capacity «q1 – q0». This critical embedment is a function of the soil category: relative values (i.e. related to the half width of the foundation) are tabulated below according to the shape of the footing:

Soil	Foundation	
	Circular or square	Continuous
Category I	4	6
Category II	10	12
Category III	16	18
Category IIIA	20	22

3.3.3. Variations of k:

The nomograms on fig. 5 page 19 gives the values of k in function of the equivalent depth of embedment for the various types of soil.

The more resistant is the soil situated above the level of the foundation, the greater is the effect on the bearing capacity of the foundation (it is not just dead load as the rather too elementary theories) of plasticity would imply). The set of graphs show that, in a soil homogeneous with respect to depth, the bearing capacity increases with embedment until it reaches an asymptotic value characteristic of deep foundations.

The lowest value of k corresponds to a foundation placed on the surface:

$$K=0.8$$

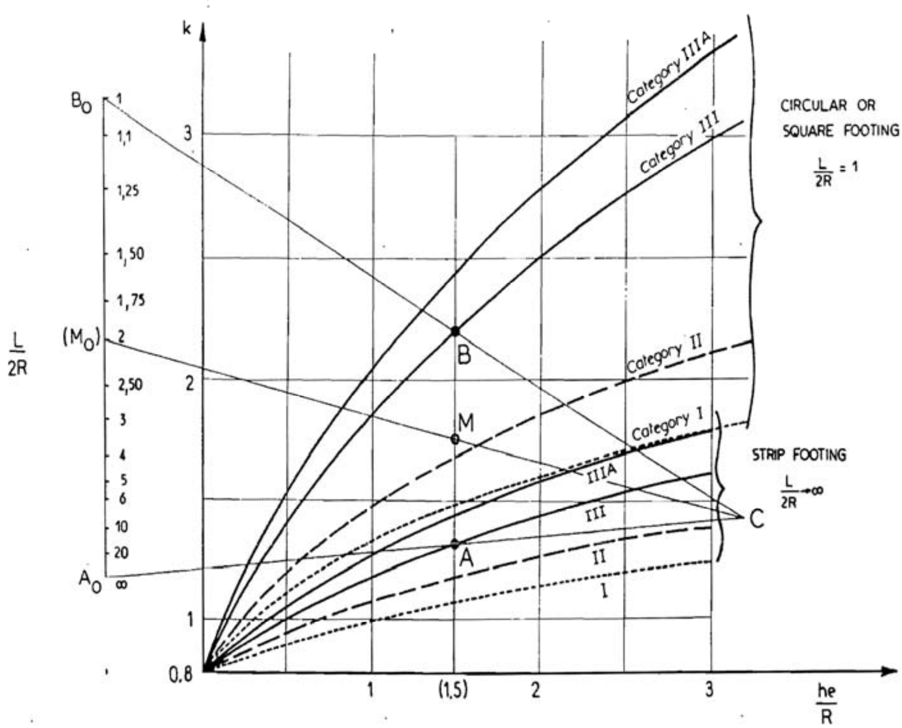
The maximum values of k which are obtained for a depth greater than the critical depth of embedment are given below and are used for calculating the end bearing capacity of foundations (rules R1 for piles, R2 for cast in-situ walls).

Soil categories	Bearing factor+		
	Drilled pile	Driven pile	Cast in-situ walls
I	1.8	2	1.4
II	3.2	3.6	2.1
III	5.2	5.8	2.9
III bis	7	9	4

The values used for shallow foundations (rule R3 will be explained in detail later.

* New categories were proposed in French tender documents (see Addendum)

+ New values were proposed in French tender documents (see Addendum)



permits graphical calculation of the bearing factor of a rectangular footing by interpolation between a strip footing and a square footing.
Example: to calculate the value of k for he/R = 1.5 L/2R = 2 (usual ratio for an isolated footing) and a soil of category III. Draw the straight line he/R = 1.5 which cuts the two category III curves at A and B.
The ordinate k of M is calculated so that M divides the segment AB in the same ratio as M_o divides the segment $A_o B_o$. Then $A_o A$ and $B_o B$ are drawn converging at C from which point a straight line is drawn passing through M_o.

3.4. Analysis for heterogeneous conditions

As a preliminary, it is appropriate to define the concepts of the equivalent limit pressure pl_e and of the equivalent depth of embedment he.

3..4.1. Equivalent limit pressure (rule 4)

When the foundation rests on strata whose strengths are variable with depth, the equivalent limit pressure pig is defined as the geometric mean of the values obtained near the level of the foundation*:*

$$p'_{le} = \sqrt[3]{p'_{l1} \times p'_{l2} \times p'_{l3}}$$

where pl'**1** geometric mean of values measured in the section from + 3R to + R above founding level

Pl'2 geometric mean of values measured in the section from+ R to - R

Pl'3 geometric mean of values measured in the section from - R to - 3R.

For a shallow foundation, the value pl'1 is not introduced, and the equivalent limit pressure becomes

$$p'_{le} = \sqrt[2]{p'_{l2} \times p'_{l3}}$$

This rule assumes that in all cases, the variations between pl'1, pl'2, pl'3 do not exceed ± 30 % of ple.

If $\left[\frac{p'_{li} - p'_{le}}{p'_{le}}\right] > 30\ \%$, it is advisable to examine

the problem in more detail. It is recommended to plot pl'e in function of the depth of the founding level and to smooth out all the peaks in the graph before use. The other concept (determination of he) will have to be considered at the same time.

When dealing with very soft soils below the water table, it is more accurate to carry out the analysis with the (pl_m - p_o) values.

3.4.2. Equivalent depth of embedment (Rule R5)

When the ground exhibits characteristics which vary with depth, it is necessary to define the equivalent depth of embedment he relative to the soil of the founding elevation. This depth he is calculated by applying the following formula (Rule R5).

$$h_e = \frac{1}{p'_{le}} \int_0^h p'_l(z)\, dz$$

where pl'e has already been defined.

Thus, in the case of a caisson founded on sandy soil with a limit pressure of 16 bars, the equivalent embedment and consequently the ultimate bearing capacity have the same values in the three following cases of overburden: 8 m of fill with pl_m = 2 bars or 2 m of sandy silt with pl_m = 8 bars or 50 cm of stabilized soil with pl_m = 32 bars. In these three cases the effective depth of embedment is 1 m (embedment of an imaginary homogeneous soil with a limit pressure of 16 bars constant from the surface).

3.5. Calculation of foundations for strip footings

3.5.1. Excavation works have a tendency, when carried out under rainy conditions or when construction is delayed for some months, to reduce the mechanical properties of sensitive soils to a depth of 0.5 to 2 m (case of fine slightly cohesive sand, clay with high liquid limit etc...). As an example, on a site where the excavations are to be a few metres deep, it is recommended, when calculating the bearing capacity, to reduce by 20 % the
values of pl_m measured before the earthworks and corresponding to the layer situated 1 or 2 m below the excavation floor.

3.5.2. The equivalent depth of embedment is calculated with the general level of excavation (basement level) taken as ground level, but it must be remembered to take into account the beneficial influence of concrete slabs resting on the soil (adopt pl_m = 100 bars over the total thickness of the slab).

On the contrary, in the case of footings poured within forms with the remainder of the excavation being backfilled, it is appropriate for the calculation of embedment to consider only the back fill and not the natural soil (adopt pl = 1 to 4 bars for the fill according to the compaction obtained).

The presence of neighbouring foundations has a generally favourable influence on the bearing capacity of surface footing (except in the case of footings very close to each other, which then behave as a continous footing or a raft).

For the calculation of k, the nomogram in figure 6 is used; this is a partial enlargement of figure 5. The method of using the nomogram is explained on the graph.

3.5.3. In the case of shallow foundations, isolated but closely grouped, the bearing k factor is limited by the following relationships:

$$K < 1 + \frac{L}{R} \qquad K < k \quad \text{nominal}$$

Where R is the half dimension of the footings in the direction of their alignment,

L is the distance between the edges of neighbouring footings. This condition is only effective for

$$\frac{L}{R} < 2$$

3.5.4. Factor af safety

The safety factor is generally taken to be 3; it should be applied to the bearing factor and consequently the net allowable bearing capacity is

$$q_a \le \frac{k}{3}\ p_{le}$$

or in a more general term:

$$q_a \le q_o + \frac{k}{3}\ (p_l.p_o)$$

3.5.5. Excentric loads

In this case, it is usual to represent the stress distribution as trapezoidal with a minimum pressure pm on one side and a maximum pressure p_M on the other.

Failure may occur either by general sinking of the footing or by localized failure in the zone of highest loading with tilting of the foundation. The two conditions must be satisfied to ensure stability:

a) the general stability is assured if

$$\frac{p_m + p_M}{2} < q_a$$

where qa is the allowable bearing capacity for the footing assumed to be uniformly loaded.

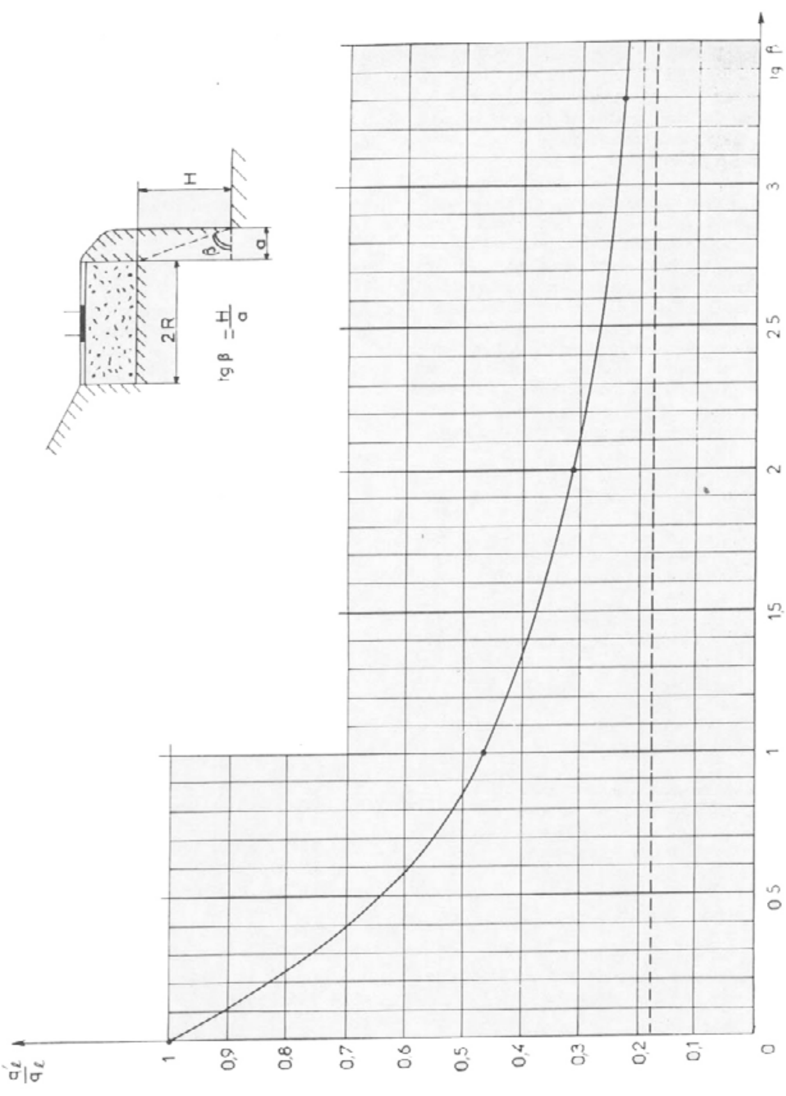
b) the stability against tilting is assured if

$$p_M < 1.5\ q_a$$

This second relationship is based on a simplifying hypothesis which is only valid for a relative embedment h/R greater than 1. If not, a new calculation is carried out for the bearing capacity of an imaginary foundation consisting of the most loaded third of the real foundation. The relative embedment of the imaginary partial foundation, as well as the corresponding coefficient k, are larger than the original ones.

3.5.6. Footings near excavations

Apart from the geometrical rule which the designer has to comply with when isolated footings are not founded at the same level (a slope of 2/3 must not be exceeded between footings), the bearing capacity of footings resting on sloping ground or near-by excavations must be reduced by a coefficient which is function of the angle β between the footing or excavation levels. The value of this coefficient is given for soils exhibiting both friction and cohesion in the diagram hereafter.



H
2R
a
β
tg β = H/a
q'ℓ/qℓ
tg β
1
0,9
0,8
0,7
0,6
0,5
0,4
0,3
0,2
0,1
0
0.5
1
1,5
2
2.5
3

3.6. Rafts

3.6.1. A raft behaves as a wide footing when it is very rigid; the rules relative to footings are applicable, taking into account the relative embedment which is generally low.

3.6.2. The shear resistance of the soil underneath the raft may vary between the outer edges and the centre of the raft, even when the site is very homogeneous in plan, but variable with depth. In the frequent case of the raft founded on a compressible layer of soil 2 to 3 m thick overlying a firm layer, the overall strength of the soil decreases progressively from the centre towards the periphery. Numerous constructions fail for this reason due to local overstressing of the soil mostly when the rigidity of the raft is insufficient to transfer the peripheral loads towards the more resistant central zones.

Thus, the allowable bearing capacity at a given point generally depends on the distance of this point from the centre of the raft. The calculation is carried out by considering imaginary footings consisting of strips resting on the border of the raft and of increasing width (1 m, 2 m, 3 m)

If the contact pressure imposed by the raft is greater than the allowable capacity calculated for one of these strips, then the raft should be stiffened at the corresponding location to allow a transfer of loads into stronger zones.

3.7. Calculation for deep foundations

The bearing capacity of a deep foundation is the sum of two terms: the point resistance and the skin friction. Although there may be interaction between these terms, it is customary to calculate them separately.

3.7.1. End bearing capacity

The values of the bearing factor k have already been given on the nomograms of figure 5 (Rule R1 for piles and R2 for diaphragm walls).

Driven piles induce compaction of the surrounding soil resulting in an increase in the value of the bearing factor; this is particularly significant in sands.

Expanded bases such as those of Franki piles must be calculated using rule A-1 (see Sols-Sons n° 5, 1964). Maximum k factor values are reduced to 1.45, 1.70 and 2.20 for categories I, II and III respectively. Under-reamed bases are calculated as described under § 3.7.5. It is usual to adopt a safety factor of 3 for the end bearing term.

3.7.2. Skin friction

The skin friction between the walls of the foundation and the soil is generally less than the shear strength of the natural soil as a result of the disturbance produced by drilling or driving.

However, loading of the pile will often result in increased confirming pressures over a height of 3 diameters from the pile point, thus locally increasing the skin friction; this is particularly true in frictional soils.

Figure 7 gives the values of skin friction in function of the limit pressure pt for the following cases:

- Traditional bored or driven piles:
  - curve A: normal skin friction
  - curve B: increased skin friction within 3 diameters of the point.

- Special piles designed for maximum skin friction (curve C).

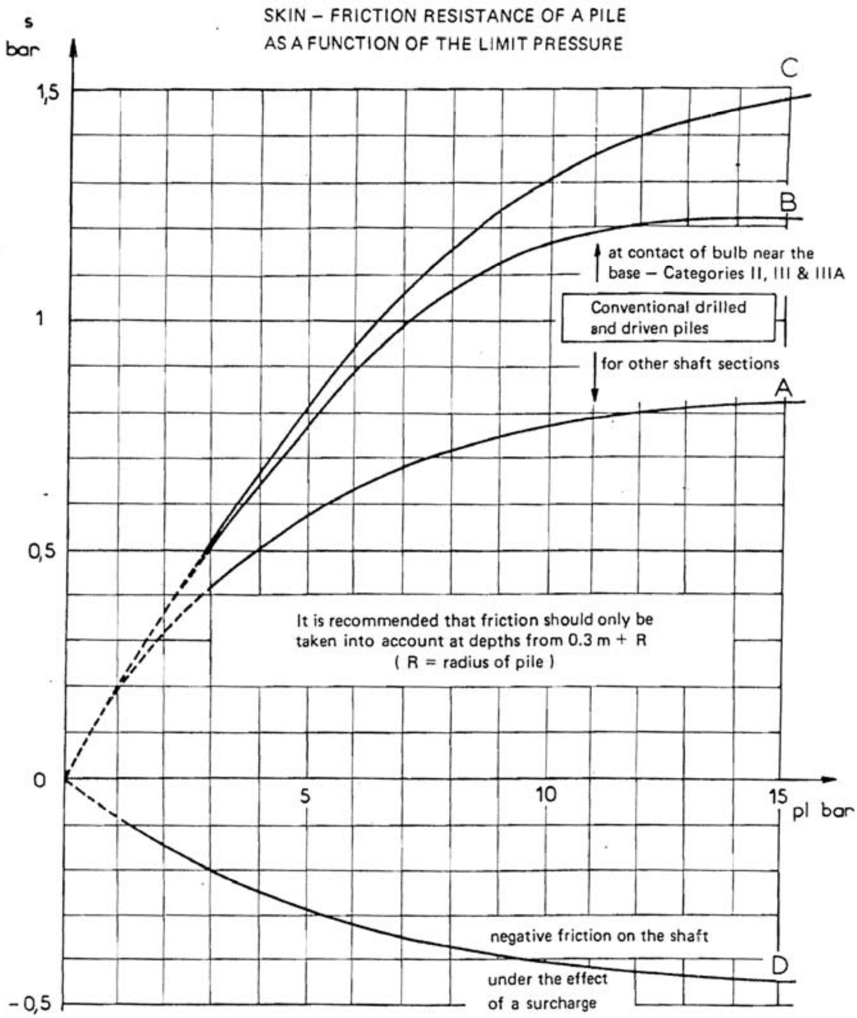
For steel piles or piles with a permanent lining, it is advisable to reduce the values of (A) and (B) by 20 % in cohesive soils and by 30 % in sands or submerged sands and gravels.

Generally, it would appear that the skin friction decreases as the pile diameter increases. The values given in figure 7 are applicable to a pile diameter of 60 cm and should be reduced by 10 % for a diameter of 80 cm and by 30 % for a diameter of 120 cm. Further reductions may be warranted in function of the difficulties encountered in the pile driving operation.

For diaphragm walls, the values of A and B should be reduced by 50 %.

For micropiles (pali-radice, pieux-aiguilles, etc...)

Chart replaced by Fig. 1 in Addendum for positive skin friction



skin friction may be increased according to experience. For piles in rock anchor piles, socketed section of drilled-in piles refers to the adequate brochure.

A safety factor of 2 is usually applied with respect to the skin friction values.

3.7.3. Negative skin friction

Two cases are considered:

a) if the soil through which piles are driven is still consolidating under its own weight (E < 15 bars); and if no new fill is to be placed on the site: hen the skin friction generated by the compressible soil may be taken as approximately 0.1 bar.

b) If a compressible soil is surcharged by a fill, such as an embankment against a piled bridge abutment, consolidation exerts downdrag forces on the pile. Negative skin friction values given by curve D of fig. 7 should then be used for any soil layer that settles 0.5 cm more than the pile at the corresponding elevation. For smaller values of relative settlement between the pile and adjacent soil, the skin friction will be proportionnally smaller.

Computation of the total downdrag force can be made by varying the end bearing pressure in a trial and error procedure. A computer program is available for this analysis.

A quick hand design method is illustrated in fig. 7b:

A) Compute the settlement of each compressible layer using the T-5 rule but taking also into account a micro-stress threshold

$$p'_\varepsilon = 0.05 - E/1.000 \text{ (bar)}$$

below which the layer can be considered as incompressible. Then draw the cumulated settlement curve (fig. 7b).

b) Assign a settlement for the pile tip

$$w_1 = \frac{p}{2E} \, 30 \, \left(\frac{\lambda R}{30}\right)^{\alpha} \text{ (T-2 rule)}$$

consistent with the allowable settlement of the structure and the maximum allowable point bearing stress (rule R1).

c) Add the elastic shortening of the pile to w1 to obtain the butt settlement wo. The largest stress in the pile section occurs at the neutral point, the intersection of the two settlement curves.

d) Derive the relative movements between the pile and the soil by comparing both settlement curves. Determine the positive and negative stresses to each element of pile.

e) Check that the stress at the pile tip obtained by adding downward algebraically the loads on each pile element corresponds to the value taken in step ib. If not, start a new process at step (b), changing the stress value at the tip and if necessary the elevation of the pile tip until a satisfactory solution is obtained. Then finally check that the stress in the pile in the neutral section (where there is no relative movement between the pile and the soil) is less or equal to the allowable stress of the pile material.

Nota - If the neutral section is taken in the first element (from the pile tip) of the soil settlement curve (computed at step a) which has a steep slope, convergence normally can be obtained by the third iteration.

3.7.4. Pile refusal

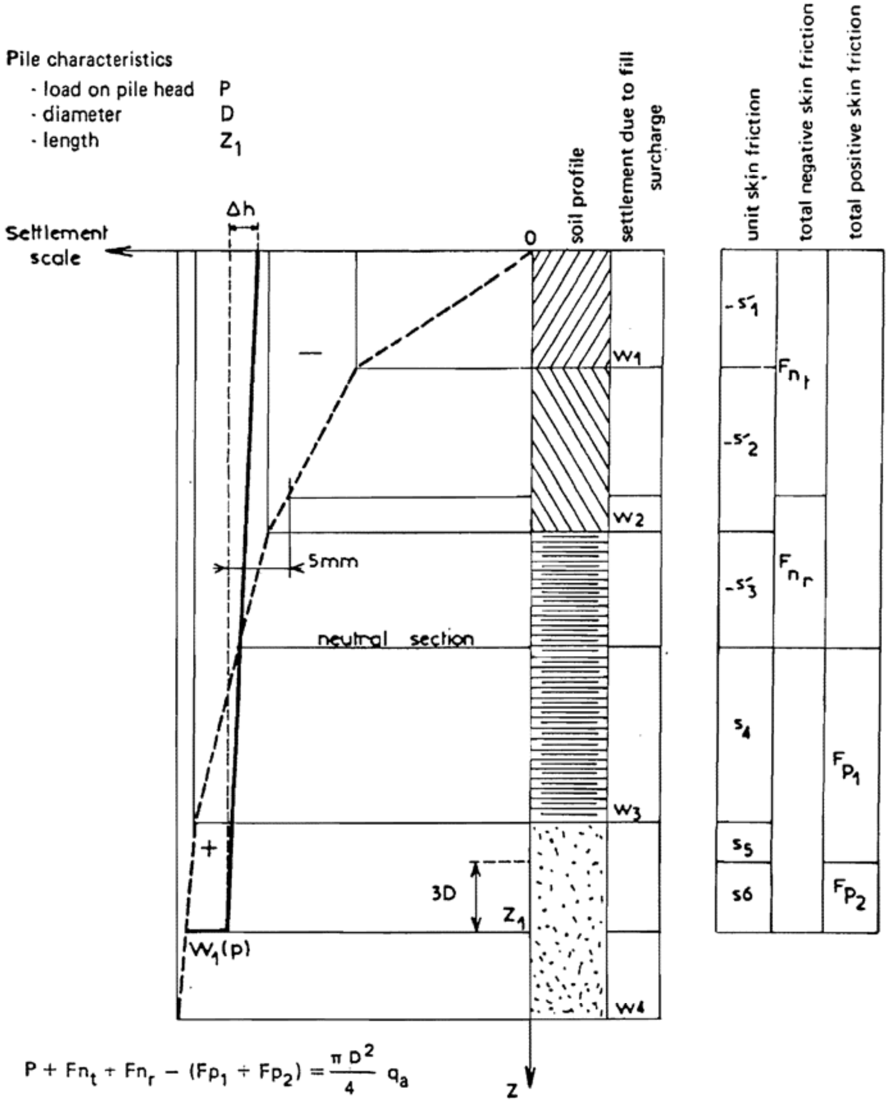
Before the bearing capacity or the pile length is computed, it is necessary to make sure that the pile can be driven as required. The possibility of pile refusal must be investigated before commencing the design.

Refusal will depend on the soil being penetrated and the pile driving parameters.

a) Influence of soil type

- In sand and gravel, experience shows that the skin friction developed under static loading has relatively little influence during the driving process. Pile refusal is primarily dependant on the soil «rigidity» near the point which is related to the pressuremeter modulus.

- On the other hand, in clay, total skin friction increases with depth and ultimately determines the pile refusal.



$$P + Fn_t + Fn_r - (Fp_1 + Fp_2) = \frac{\pi D^2}{4} q_a$$

- compare q_a and p for the further iteration
- check $P + Fn_t + Fn_r < \frac{\pi D^2}{4}$ σ max in the pile

Fig 7 bis Calculation of the negative skin friction

b) Influence of pile type

Driving efficiency depends on several pile characteristics

- For single element precast piles: the driving energy is expended to overcome skin friction and also to compress the pile.

- For West type partially, precast pile: part of the driving energy is lost at the junction between the mandrel and the point.

- For pipe piles, energy is lost in skin friction, depending on the level where the casing is driven, and part of the energy can be lost by casing compression.

Finally, the higher the hammer energy, the greater the penetration.

From these remarks and experience, the following comments are appropriate:

- Driven concrete pile, precast in one element, meet refusal at a penetration of

- 1 m in a sand and gravel layer where E > 150 bars as long as this layer is more than 4 meters thick.

- 3 m in a sand and gravel layer where E > 75 bars as long as this layer is more than 6 meters thick.

In general, overdriving is not recommended as pile rupture may take place.

Franki type piles can be driven fairly easily through sand and gravel layers 2 to 3 meters thick with E equal to 300 bars. Overdriving dos not endanger the pile.

The penetration of driven piles in clay is limited by the skin friction value derived from curve A (fig. 7) without safety factor.

3.7.5. Design of piers

The equivalent depth of embedment he/R plays an important role in selecting the allowable bearing stress of pier foundations computation.

Piers, often hand dug but sometimes also excavated with specially equipped hydraulic shovel, generally have a low equivalent depth of embedment. It to therefore important to determine he/R with rule R4 and R5 successively in order to compute the point resistance.

Lateral skin friction must be neglected or reduced depending on the he/hc value and the boring process. If a temporary wood lining is left in place, skin friction must be taken as zero.

Often, belled piers are used (Chicago method) to reduce shaft cost. Excavation of a bell causes a stress release in the soil above the bell and the bearing capacity does not increase linearly in proportion to the bearing surface. The bearing capacity is the sum of two terms:

- The bearing capacity of a pier having the diameter of the shaft (k being computed with an equivalent he/R').

- The bearing capacity of a ring foundation corresponding to the belled section and working like a footing according to rule R3 (k is obtained from fig. 6). The embedment of the equivalent footing is equal to the height of the bell.

Finally, during a further construction period, soil may be removed around portions of the shaft of the pier.

In the case of a square pier with its base below the critical depth, ki value can be determined as a function of the number of stripped faces. The excavation does not extend below the foundation grade. If the stripping is only performed to a height h' above grade the asymptote of the curve,

K= k (number of stripped faces)

is increased up to the value of k given in fig. 5 or 6.

If the stripping extends into an excavation below grade, the asymptotic value is such that

$$q = 2s$$

where 2 s is the shearing resistance (simple compression test).

Because $2\,s = (p_l - p_o)/K_B$ the result is

$$k = 1/K_B$$

Nota as a rough approximation, the order of magnitude of k, for piers stripped on n faces, is

$$k = \frac{k_{max}}{n\,\frac{k_{max} - 0.8}{3.2} + 1}$$

3.7.6. Design of small truncated conical piles

A group of short conical displacement piles, has a greater bearing capacity than a group of cylindrical piles largely spaced.

a) Influence of soil displacement

Driving a group of small conical piles increases the density of the soil. This increase depends on the type of soil and can be estimated from

$$Pl_m = Pl_o \times \beta$$

Pl limit pressure immediately after pile driving

Plo limit pressure of the natural soil

β coefficient depending on the type of soil and the ratio of concrete volume to total volume.

Densification percentage	Sand	Silt	Clay
1 %	1.3	1.2	1.1
2 %	1.5	1.4	1.2
4 %	2.0	1.6	1.3

The tabulated coefficients consider that all the short piles are driven rapidly and that only one consolidation phase takes place, which is conservative.

b) Influence of the truncated conical shape

Most of the load is applied to the soil by the one sides; the point resistance can be neglected. The vertical load is transmitted through a combination of lateral skin friction and normal reaction on the inclined pile face

The unit bearing capacity on the peripheral area $2 \pi r \Delta$ is equal to

$$s + \frac{f}{100} \times q_l$$

where:

s lateral shearing strength (totally mobilized due to the compression effect of the piles on the soil)

q_l bearing capacity of an r wide foundation at the corresponding relative depth

$\frac{f}{100}$ tangent value of half the cone angle.

In most of the cases, the usual rules are employed. Consequently the pile bearing capacity is:

- For the 0 to 1 m element

$$2\pi\,[r_1 \times p_{l1}\,(\frac{1}{6} + \frac{f.k}{100})]\;100$$

- For the 1 to 2 m element

$$2\pi\,[r_1 \times p_{l1}\,(\frac{1}{6} + \frac{f.k}{100})]\;100 \quad \text{etc ...}$$

r1 r2, etc... $(pl_m)1$, $(pl_m)2$ etc... being respectively the average radius and limit pressure (eventually increased) of the corresponding elements,

That is to say, with r in cm, pl in bar and P in metric ton:

$$P_{(t)} = 0.6\,(0.16 + \frac{f}{100} \times k)\,[r_1\,p_1 + r_2\,p_2 + r_3\,p_3 + ...]$$

when k has a constant value and

$$P_{(t)} = 0.6\,[0.16 \times \Sigma\, r_1\, p_{l1} + \frac{f}{100}\,\Sigma\, r_1\, q_{l1}]$$

when the k value changes with the element.

4.Calculation of the settlement of a foundation

4.1. The settlement beneath a foundation due to the loads it carries is the result of two completely different phenomena:

- A phenomenon of volumetric compression under the influence of the spherical component of the stress tensor. The increase of the bulk pressure causes a reduction in volume of the material in relation to the modulus of volumetric compression.

- A phenomenon of shear deformation under the influence of the deviatoric components of the stress tensor. These displacements occur without variation in volume of the material.

The representative curves of the spherical and deviatoric components of the stress tensor against depth are very different. The spherical component is maximum immediately under the base of the footing and the deviatoric component is maximum at a depth equal to the half width of the foundation.

The phenomena of shear deformation are dominant under footings, shafts or pries; the phenomena of volumetric compression predominate under a raft or an embankment, the relative importance of volumetric compression as compared to shear deformation increases as the safety factor relative to failure increases.

Field tests carried out on full scale footings (Ménard and Rousseau 1962) have shown that settlement does not increase in direct proportion to the width of a foundation as predicted by elastic theory (for a homogeneous, elastic medium). Further the relation between settlement and foundation width depends on the type and structure of the foundation soils.

This non linear behaviour of soil can be represented by using an elastic theory in which the volumetric compression modulus and the shear modulus are related by an empirical factor and which depends on the grain size and the stress history of the soil. The settlement of a foundation then becomes proportional to the foundation width raised to the $\propto$ power, e.g.

$$\mathbf{w = f\ (R)^{\propto}}$$

There is a direct relationship between the coefficient $\propto$ and the Frolich's concentration coefficient.

Furthermore, if a soil layer is not normally consolidated, the foundation will also show an extra settlement due to natural consolidation of the soil layer. This extra settlement will be analyzed later on.

The above considerations assume that the soil is self-bearing, i.e. that it does not settle under its own weight with or without influence from external phenomena. Soils which are not self-bearing are considered in a later paragraph.

4.2. General formula for settlement of an isolated foundation (T-O rule)

Consider a circular footing of diameter 2R or rectangular with dimensions 2 R x l, designating by:

E the pressuremeter modulus of the soil, assumed to be homogeneous

P the mean contact stress added to the soil by the rigid footing

R_o a reference length equal to 30 cm

$\propto$ the structure coefficient variable according to the nature of the soil and the ratio E/p obtained from the pressuremeter, derived from the following table:

Type of material	Peat		Clay		Alluvium		Sand		Sand & gravel	
	E/p_l	α	E/p_l	α	E/p_l	α	E/p_l	α	E/p_l	α
Over consolidated			> 16	1	>14	2/3	>12	1/2	>10	1/3
Normally consolidated		1	9 - 16	2/3	8 - 14	1/2	7 - 12	1/3	6 - 10	1/4
Weathered or altered			7 - 9	1/2		1/2		1/3		1/4

A soil may be altered as the result of a landslide or by the action of water (rain or seepage), especially at the bottom of an excavation. The values of $\propto$ given in this case are provisional; they are slightly conservative and will be reviewed in the light of more experimental results.

For rock, the value of $\propto$ depends primarily on the extent of fissures and structural weaknesses. The following values are representative:

$\propto$ = 1/3 for extensively fractures rock
$\propto$ = 1/2 for normal rock
$\propto$ = 2/3 for rock only slightly fractured (or very weakened, as well)

The factors λ_2, λ_3 are shape coefficients, whose values are a function of the length to width ratio of the foundation, L/2R, as given below

L/2R	1		2	3	5	20
	circle	square				
λ2	1	1.12	1.53	1.78	2.14	2.65
λ3	1	1.1	1.2	1.3	1.4	1.5

The total settlement w obtained after stabilization is expressed by the following formula in which the first term represents the influence of the deviatoric stress tensor and the second the volumetric component.

$$W = \frac{1.33}{3\,E} p\, R_o \left(\lambda_2 \frac{R}{R_o} \right)^{\propto} + \frac{\propto}{4.5\,E}\, p \lambda_3 R$$

If R>30 cm. if R<30 cm, the equation becomes

$$W = \frac{1.33}{3\,E}\, p \lambda_2^{\propto} R^{\propto} + \frac{\propto}{4.5\,E}\, p \lambda_3 R$$

These formulae are applicable to foundations embedded a depth of at least one diameter (h = 2R). Otherwise, w should be increased by 10 % for h = R and 20 % for h = 0 (strictly speaking, he should be used instead of h).

As it can be understood from the notation, a first term, corresponding to a pure elastic settlement, involving a micro-deformation modulus, is neglected in this equation.

4.3. Variable soils

The pressuremeter modulus E varies with depth in most natural soil deposits.

In theory, the problem of calculating the corresponding settlement is very complex, but provided that the variation of the pressuremeter modulus with depth is not too large, it is possible to use the formulae for the settlement, employing equivalent moduli E_A and E_B corresponding to the zones of volumetric and deviatoric influences respectively.

The calculation involves dividing the soil below the foundation into layers each having a thickness of R (fig. 8). The notation i designates the layer included between the depths (i — 1) R and R and E is the pressuremeter modulus of this layer (equal to the harmonic mean of the moduli obtained in. this layer if several moduli have been measured: The equivalent moduli are given by the following equations:

$$E_A = E_1$$

$$E_B = \frac{4}{\frac{1}{E_1} + \frac{1}{0.85\,E_2} + \frac{1}{E_{3/4/5}} + \frac{1}{2.5 E_{6/7/8}} + \frac{1}{2.5 E_{9\text{ to }16}}}$$

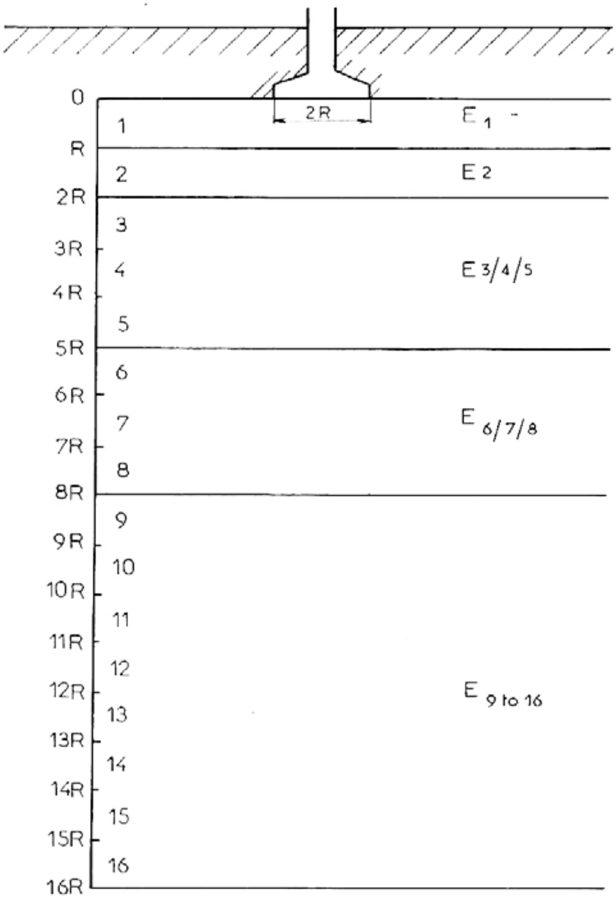
where $E_{(p\text{ to }q)}$ is the harmonic mean of the moduli of layer p to q

and $$w = \frac{1.33}{3\,E_B} p\, R_o \left(\lambda_2 \frac{R}{R_o}\right)^{\propto} + \frac{\propto}{4.5\,E_A}\, p\, \lambda_3 R$$

Note that $\propto$ can be different in the two terms according to the prevalent material in each zone of influence.

The two terms on the right-hand side of the equation are often designated by the expressions w2 and w3 (w1 corresponding to the neglected elastic settlement).

$$\frac{1}{E_i} = \frac{1}{n} \sum_{j=i}^{j=n} \frac{1}{E_{ij}}$$



Layers of soil under a footing taken into consideration
for the computation of equivalent moduli.

Note: if E_g to 16 s unknown but is assumed to be superior to the values of the upper layers, then:

$$E_B = \frac{3.6}{\frac{1}{E_1} + \frac{1}{0.85E_2} + \frac{1}{E_{3/4/5}} + \frac{1}{2.5E_{6/7/8}}}$$

if $E_{6/7/8}$ is not known either, then :

$$E_B = \frac{3.2}{\frac{1}{E_1} + \frac{1}{0.85E_2} + \frac{1}{E_{3/4/5}}}$$

4.4. Highly variable soil

When the variation in pressuremeter modulus is very large against depth, e.g. when the successive strata belong to very distinct geological formations the method described above is no longer applicable: actually, the stress distribution is no longer comparable to that of a homogeneous medium, because of the interaction between the strong and the weak layers.

4.4.1. Two Layer System (rule T5)

A two layer system often arises when a raft or embankment is founded on a relatively soft soil, underlain by a more rigid material at a depth less than the half width of the foundation.

The total settlement of the foundation after stabilization is given by the formula:

$$w = \int_0^h \frac{\alpha(z)\ \beta(F)\ p(z)}{E(z)}\, dz$$

where $p(z)$ is the change in vertical pressure at the depth z induced by the foundation load

$E(z)$ the pressuremeter modulus at depth z

$\propto(z)$ structure coefficient corresponding to the soil layer at mean depth z

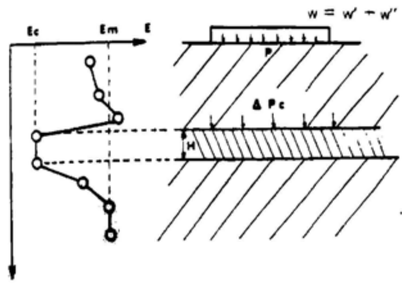
$\beta(F)$ coefficient depending on the safety factor F of the foundation, with

$$\beta(F) = \frac{2}{3} \quad \frac{F}{F.1} \quad \text{for } F < 3$$

and $\beta(F) = 1$ for $F < 3$

4.4.2. Compressible layer between more rigid layers

The existence of a weak layer of thickness H disturbs the stress distribution and settlements are increased because this layer still consolidates under the overburden weight. The best procedure is to make a calculation for the settlement of a soil assumed to be homogeneous and then add the settlement corresponding to the soft layer.



The partial settlement w' is obtained by the general formula substituting for modulus Ec of the compressible layer, a modulus Em of the same order of magnitude as the two adjacent layers.

The additional settlement w" initiated by the compressible layer is obtained from the equation

$$w'' = \alpha_c \left(\frac{1}{E_c} - \frac{1}{E_m}\right) \Delta p_c \quad H$$

where $\propto c$ and Δ pc designate the structure coefficient and the change in the vertical extra stress at the level of the soft layer.

As a first approximation Δ pc may be calculated with the aid of the Boussinesq formula.

4.5. Settlement of a deep foundation

The review Sols-Soils presented in its number 7 issue, a complete numerical method for calculating pile settlements. The reader would be well advised to refer to this paper (Rules T1 and T3). However, in some instances, more rapid methods can be used.

4.5.1. Rapid method for estimating the settlement w of an isolated semi-deep foundation, pile or pier (rule T4)

When the ratio of the equivalent embedment of a pile or pier to its radius R is less than five, i.e. $\frac{he}{R} < 5$, the following simplified formula can be employed ; as long as $R < 1$ m

$$w = C_q \frac{q'}{2E} 30 (\lambda_2 \frac{R}{30})^{\alpha} \quad \text{for } 30\,\text{cm} < R < 100\,\text{cm}$$

or

$$w = C_q \frac{q'}{2E} \lambda_2^{\alpha} R \qquad \text{for } R \leqslant 30\,\text{cm}$$

where q' is the stress applied at the butt of the pile or pier

$\lambda 2$ shape coefficient ($\lambda_2 = 1$ for a circular section, and $\lambda_2 = 1.12$ for a square section)

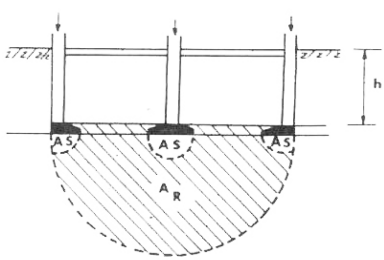
C_q a coefficient of embedment

$$C_q = \frac{1}{0.8 + 0.1 \frac{he}{R}}$$

4.6. Settlement of closely grouped foundations (flats, etc…)

When foundations are closely spaced, the stress fields overlap and a more complex method of settlement calculation is required.

For the example shown below of a building with three supporting basement walls p is the bearing stress of the footings and pm is the mean uniform stress that the building would impose if founded on a raft.



The stress distribution due to the building loads may be divided into four elements:

- A field with a spherical component of intensity pm in the zone Ar
- A field with a deviatoric component of intensity pm external to the zone Ar
- A field with a spherical component of intensity p - pm under each footing (As)
- A field with a deviatoric component of intensity p - pm external to zones As

The settlement w of the building is then given by the formulae

$$w = w_2 (p_m) + w_3 (p_m) + w_2 (p - p_m) + w_3 (p - p_m)$$

where the first two terms are calculated for a bearing area equal to that of the building
and the latter ones are calculated for a bearing area equal to that of the footings.
The «general raft» settlement or the «footing» settlement will be prevalent according to the relative distances between footings and variations in the soil moduli with depth.

4.6.1. Computation of the numerical coefficients in the specific settlement equation
In paragraph 2.7.1. the specific settlement is expressed by the equation

$$w_s = \sum_{i=0}^{i=25} \frac{n_i}{E_i}$$

Where E_j Is the pressuremeter modulus E of the layer between the depths I – 0.5 and I + 0.5 (in metre).

n_i a coefficient for the layer i tabulated in the same paragraph.

Calculation of the n_i factors is performed as follows

1) Consider a foundation system consisting of 3 strip footings 1 meter wide, 4.5 m centers apart (total width between outer sides of footings is consequently 10 meter)

2) Separate the stress tensor below each footing into isotropic and deviatoric components and combine them three by three.

As a first approximation, the six tensors can be reduced to 4 tensors the same way as the previous paragraph:

- The tensors of the equivalent raft 10 m wide, exerting a bearing stress of 0.3 times the footing contact pressure.

- The extra tensors of the footing exerting a bearing stress of 0.7 times the actual footing pressure.

3) For each layer i - 0.5/i + 0.5 m, the modulus E must be multiplied by a weighing factor taking into account the variation of the tensor intensity.

For instance, between 4.5 and 5.5 m where the modulus is E5, in the case of sand: for the raft:

the settlement due to the deviatoric tensor is

$$w = \frac{1.33}{3} \times 0.3 \times 30 \times \left(1.5\,\frac{500}{30}\right)^{1/3} \frac{1}{E_5} \times \frac{1}{20}$$

the settlement due to the isotropic tensor is

$$w = \frac{1}{4.5}\,\frac{1}{3} \times 0.3 \times 500 \times 1.2 \times \frac{1}{E_5} \times \frac{1}{9}$$

for the footing :

the settlement due to the deviatoric tensor is

$$w = \frac{1.33}{3} \times 0.7 \times 30 \times \left(2.5 \times \frac{50}{30}\right) \frac{1}{E_5} \times \frac{1}{120}$$

the settlement due to the isotropic tensor is

$$w = \frac{1}{4.5} \times \frac{1}{3} \times 0.3 \times 50 \times 1.5 \times \frac{1}{E_5} \times 0$$

and the total settlement is

$$w_5 = \frac{1}{E_5}\,(0.58 + 1.48 + 0.12) = \frac{2.2}{E_5}$$

as shown on the table, paragraph 2.7.1.

4.7. Settlement of a structure founded in deep excavation

Some structures (buildings with several underground stories) are built inside deep excavation. Excavation decreases the stresses in the underlaying soil resulting in a heave of the base of the cut. The progressive loading of the structure produces an opposing settlement, less than, greater than or equal to the heave according to the ratio between the weight of the building and the weight of the excavated earth.

The settlement is calculated in two steps:

- At the point where the bottom heave is balanced by the settlement due to a structure load equal to the weight of the soil excavated,

- At the point where the building is fully loaded and additional stresses are exerted by the completed structure.

For both steps, the general formulae are applied, but the reloading modulus E_a is used for the first step and the virgin modulus E for the second. If E_a has not been measured with the pressuremeter, the table below may be used as an approximation:

Type of soil	E_a/E
Clay	2
Silt	3
Sand et gravel	4

Actually, this conventional calculation does not take into account the two following phenomena:

- The excavation work always entails a reduction of the reloading and virgin pressuremeter moduli, this reaches 50 % in soils sensitive to rain, especially if the excavation remains open for several months.

- The distribution of stresses applied by the structure is not exactly the same as that previously imposed by the excavated soil.

This particular problem will have to be examined for each particular site. The solutions adopted will generally fall between the following limits:

- The excavation will be made with a minimum of disturbance to the foundation soil and consequently

E_A = 2 to 4 E depending on the nature of the soil

- The foundation soil will be remoulded excessively as evidenced by large heaving of the excavation bottom in which case the settlement is calculated using the total load of the structure, and the modulus E measured before excavation.

4.8. Coefficient of subgrade reaction

The coefficient of subgrade reaction k expressed in bar/cm or t/m3 was introduced by Winkler assuming a linear relationship between stresses p and vertical deformation y at the interface between a beam and an elastic soil, irrespective of the width of the beam

$$\mathbf{P = K_y}$$

Furthermore, it is assumed that the behaviour of each elementary slice of beam and soil has no influence on the behaviour of the neighbouring ones.

Because k is often used in reinforced concrete calculations, it is worthwhile to know that it can be very easily derived from the pressuremeter modulus E. If the two members of the formula T-O are divided by the stress p, then

$$\frac{1}{k} = \frac{1.33}{3\,E_B} \times 30 \left(\lambda_2 \frac{R}{30}\right)^{\alpha} + \frac{1}{4.5}\;\frac{\alpha}{E_A}\;\lambda_3\;R$$

It must be noted that in the technical literature, a particular reference is made to a k value for a square plate 1 ft wide and another formula is used to derive k for any other beam or footing dimension. When using the pressuremeter modulus E, the coefficient of subgrade reaction should be calculated for several widths of footings or influence radii (in the case of the rafts).

Last note: the use of the crude formula p = ky at several sounding locations does not permit an accurate assessment of differential settlements.

4.8.1. Subgrade reaction coefficients under vibrating machines

Design of foundations for vibrating machine involves using a subgrade de reaction coefficient applicable for cyclic loading.

Experiments have shown that this k value can be derived from the same T-O formula, E_A and E_B being replaced by $3E_A$ and $3E_B$. However, the result is only valid if the foundation block embedment is at least equal to half its width. For a shallower embedment, a reduction factor must be introduced. This factor is equal to 0.6 when the foundation block is on the ground surface.

4.9. Self-bearing condition

On a natural unconsolidated soil or on a new fill, even very lightly loaded structures will undergo large settlements with passage of time. The rate of settlement can be accelerated by wetting (such as in the case of a fill above the water table, by vibrations, or any phenomenon which tends to temporarily reduce the shearing resistance between grain contact points allowing the soil to settle into a more compact state. This settlement can also be balanced by a swelling due to gas production (such as in peat).

The self-bearing condition of a soil, i.e. the level of the soil parameters that a soil must have so as not to settle under its own weight, can be related either to physical or mechanical properties of the soil. Experience has shown that the limit pressure is
a suitable characteristic. The following table indicate the self-bearing condition of different types of soil in terms of limit pressure at a depth less than 10 meters. The table takes into account the influence of external random phenomena such as water table variation, seasonal moisture content change, road traffic vibrations, etc ...

Type of soil	Self-bearing condition in terms of pi (bar)
Clay	2.5 to 3
Silt	4
Sand	6
Sand & gravel	8

For depths greater than 10 meter the figures have to be increased.

The natural settlement of a soft clay layer, 10 m thick, where pl_m = 1 bar, will be about 1 cm per year. As a first approximation the following formula gives the one-year settlement of any soft layer as a function of its limit pressure.

The natural settlement of a soft clay layer, 10 m thick, where pl_m = 1 bar, will be about 1 cm per year. As a first approximation the following formula gives the one-year settlement of any soft layer as a function of its limit pressure

$$w \sim \frac{h\,(\mathrm{cm})}{1000} \quad \frac{1 - \alpha p_l / 2}{\alpha p_l / 2}$$

as long as $p_l < 2/\alpha$ (in bars or tsf)

α being given by the table 4.2.

This settlement rate will be accelerated under external causes as stated above. Soils with organic content will still show larger settlements. As a rule, one percent of organic content will increase the self-bearing condition value by 20 %.

5. Evaluation of differential settlement. Allowable values for structures

Distortions of the structural framework and cracking of supported elements (partitions, facades, etc...) are most frequently initiated by differential settlement of the structure and its foundations.

Due to the variability of the structural loading and non homogeneity of the soil, the settlement of the foundation varies from point to point in a building.

It would seem possible to predict differential settlement by comparing the absolute or total settlements at various points of the foundation; in fact, due to the rigidity of the structure, foundations do not settle independently of each other. The loads applied to foundations experiencing a large settlement are partially transmitted to neighbouring foundations that settle less. As a result, the calculation of the actual differential settlement requires introduction of a coefficient of rigidity, characteristic of the structure.

Structures vary in sensitivity to differential settlement; prestressed concrete is more deformable

than ordinary concrete. Plaster partitions crack with small distortions, while certain types of rain plate walls are very deformable.

Considering both the rigidity of a structure and its sensitivity to cracking, the object of a foundation design is therefore to calculate the differential settlements that will not induce damages to the structure and to determine the type of foundation and bearing capacity that satisfy this condition.

5.1. Evaluation of differential settlement (general case)

If the number of soundings were equal to or more than the number of supporting points, the evaluation of the differential settlement would be obtained by comparison of the absolute settlements calculated beforehand at each probe location. This situation is encountered in practice for bridge calculations (1 to 2 probes per support) and for numerous civil engineering structures.

For building investigations, the density of soundings is frequently quite low and differential settlements must be predicted from statistical analysis based on the variability factor for the foundation soil.

It is already assumed that due to the variability of the loads (front walls, central columns, etc...), the bearing pressures will have to be adjusted in accordance with the mean characteristics of the terrain.

The total settlement w of the foundation is calculated first by adopting the mean values obtained on the site for the pressuremeter moduli. Further, the mean value of specific settlement w, the standard deviation e and the index of variability

$= \frac{e}{w}$ are calculated as per section 2.7.1

The elementary differential settlement (defined for support points 10 m apart) has a maximum value statistically equal to w (Δw = iw); the real elementary differential settlement, taking the rigidity of the structure into account, is given by the formula

$$\Delta w_r = \frac{iw}{k_n}$$

k_n is a coefficient increasing with the rigidity of the structure with:

$k_n = 1$ for a rigidity assumed to be nil (totally independent supports)

$k_n \rightarrow \infty$ for an infinitely rigid structure.

When the number of soundings is very limited, the variability coefficient i is not sufficiently well defined statistically and must be weighted taking into account the statistical results obtained from neighbouring sites. Without the risk of appreciable error, it may also be replaced by i_m such as

$$\mathbf{i_m = 0.2\beta + (2\text{-}\beta)\ \frac{i}{2}}$$

where β is a weighting coefficient varying from0 to 2

taking

β=2 when the investigation is on a very reduced scale (1 to 2 soundings)

β=0 if the investigation is comprehensive.

5.2. Rigidity of structures

The rigidity of a bridge or a simple structure is directly calculated from the theory of structures.

With regard to a building, complete calculation is almost impossible because of the structural complexity and the presence of elements (partitions, etc ...) whose characteristics are ill-defined. By way of simplifications, the following empirical coefficients can be used when building rigidity calculations are impractical or infeasible.

In the following, the calculation of w is assumed to be carried out for a single footing such that kn may have a value less than unity.

Typical buildings:

- Typical buildings with reinforced concrete frame or supporting walls with reinforced concrete beams. The basement consisting of concrete walls (at least at the periphery and continuous footings; the stress distribution is homogeneous.

k_n =1.1 (varying between 1 and 1.2)

- Buildings with special stiffening:
comprises buildings specially reinforced to withstand differential settlement: «egg crate» design at basement level - (reinforcement increased by 50 to 100 % with respect to the usual requirements). No large openings or openings reinforced
by framework to prevent cracking - good bracing and equal distribution of loads

k_n = 1.5 (may vary from 1.2 to 1.71

- Low rigidity buildings:
industrial sheds, houses without basements or with supporting walls without bracing.

k_n = 0.8 (0.7 to 1)

The higher values for the rigidity should be used only after detained study of the structural plans

5.2. Allowable differential settlement for structures

The calculated value of differential settlement

$\Delta w_r = \frac{iw}{k_n}$ must be lower than the value allowable

for the proposed structure. Determination of allowable differential settlement has been the object of numerous studies for simple structures in concrete or steel, but no precise research has been carried out for apartment buildings.

The observation of existing structures and the results of full scale tests at the Soil Studies Centre and during underpinning work have suggested a typical range of values to be adopted. The problem is also complicated because the loading rate varies with different sites and the supported elements (internal partitions, etc...) only experience delayed settlements which occur after the frame is erected.

These general limitations in mind, the following values are recommended:

5.3.1. Habitation buildings

- Normal buildings: **Δw = 3 to 6 mm**
Δw = 5 mm (average)

-Buildings with weak supported elements partitions, facades, etc ...) and cases where prefabricated elements are fixed in the frame without flexible couplings:

Δw = 3 mm

- Buildings with flexible supported elements, or to some extent, equipped with flexible joints:

Δw = 7 mm

5.3.2. Industrial buildings

- Conventional buildings with steel or concrete frames:

Δw = 10 mm

- Conventional buildings, but with weak infilling (breeze blocks, etc...)

Δw = 8 mm

- Buildings without infilling:

Δw = 15 mm

Limiting values may be substantially higher for certain structures; in such cases, the maximum values must be determined by structural analysis and judgement.

5.3.3. Diverse structures

The tolerances for differential settlement set by structural designers are often inappropriate. For example, the allowable differential settlement of oil reservoirs with floating tops.

A similar situation often exists with respect to bridges. Certain bridge design standards introduce a theoretical allowable settlement calculated from classical methods. It is well known that the theoretical allowable settlement is often two the five times more than the actual settlement that can be
safely experienced.

When recommended values of allowable differential settlement are given it should be made clear whether the limiting values are the result of observation of structural behaviour compared to calculated or measured settlements.

5.3.4. If the differential settlement calculated using the bearing stress corresponding to a safety factor of 3 against failure is greater than the allowable differential settlement, several alternatives can be followed:

- Reduce the bearing stress,
- Increase the embedment depth,
- stiffen the structure or make it less sensitive to the differential settlement (increase the number of joints or employ flexible connections between elements, etc ...)
- Employ deep foundations, or
- Consider another type of structure better adapted to the site under consideration.

Conclusions

This paper has presented on account of the application of pressuremeter tests to the design of typical foundations.

The calculation methods and theories are based on recent fundamental research in soil mechanics and foundation behaviour.

Because these methods are in part derived from full scale experiments and the observation of the behaviour of existing structures, some refinements of these rules will undoubtedly be made in the future in the light of further experience. Nevertheless, at the present time, all the comparisons that have been made (some by State organizations or private concerns) have shown that the bearing capacity and the differential settlement predicted from pressuremeter analyses have been consistently more accurate than those deduced from conventional tests and methods of analyses.

The use of the foregoing analysis based on the results of pressuremeter tests is therefore conducive to more realistic foundation designs.

However, designers should appreciate the difference between standard embodying methods leading 1 pessimistic results and the pressuremeter analyses.

Addendum to the General Memorandum
on Menard Pressuremeter

-:-

Excerpts of DTU* N° 1877 of September 1983
and
DTU* 13-12 of March 1988

-:-

* DTU stands for "Documents Techniques Unifies", which are part of tender documents for the French private building industry.

The following recommendations will be included in the next edition of Fasicule 62 CCTG which is part of tender documents for the French public building industry.

French tender documents regarding bearing capacity of foundations assessed from (Menard) pressuremeter data exhibit a trend to oversimplifications which is not always obvious. However, it seems necessary to give the calcuation rules which are now recommended.

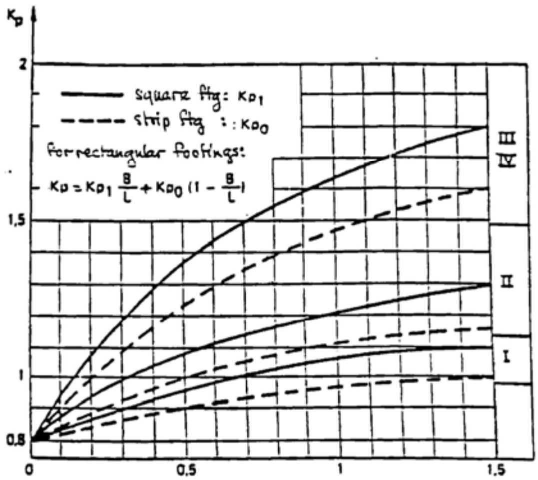
A-1. Soil categories
This oversimplification has led to the following table.

Type of soil	Soil category
Clay and silt	I
Sand and gravel	II
Chelk, marl	III
Weathered or jointed rock	IV

A-2. Bearing factor kp for shallow foundations

Chart fig. A-1 is used which give k_p for square footings k_p (1) and for infinitely long strip footings k_p (0).

Fig. A-1 Bearing factor k_p for shallow footings (subscript "p" stands for pressuremeter, as opposed to "c" in another chapter of the French tender documents for static cone)



A-3. Point load bearing factor kp for deep foundations

A deep foundation is that for which the equivalent depth De is more than 5 times Be, where B_e is the equivalent width given by

$$Be = \frac{4\ A}{P}$$

where A is the cross-section area of the foundation P is the perimeter of this cross section

Note that if for a square cross section Be = B and for a circular cross section Be = 2 R, for an infinite strip footing or diaphragm wall Be=2B.

TABLE A-I - POINT LOAD BEARING FACTOR

Type of soil	Type of pile	
	Groupe 1	Groupe 2
Clay and silt	1.2	1.8
Sand and gravel	1.1	3.2 - 4.2
Chelk, marl	1.8	2.6
Weathered or jointed rock	1.1 - 1.8	3.2

Group 1 includes all bored piles. Group 2 includes driven piles and displacement caissons. The present table for k_p is the result of the findings of the “Laboratoire Central des Ponts et Chaussées” obtained through an extensive programme of instrumented pile loading tests (more than 100 to-date).

A-4. Skin Friction
It can be derived from chart fig.A-2, which incorporates:

. type of soil
. type of pile
. workmanship level

as shown on the subsequent table A-2.

* values in brackets can only be used for very well constructed piles (no soil remoulding)

** only for soils when pl > 1.5 MPa

*** when driving is possible

TABLE A-2

Soil type	Limite pressure pl (MPa)	Pile type						
		Concrete bored	Bored and lined concrete	Bored and lined steel	Driven concrete	Driven steel	Grouted low pressure	Grouted high pressure
Soft clay and silt, loose sand, sof chelk	0 - 0.7	Abis	Abis	Abis	Abis	Abis	A	-
Medium stiff clay, silt	1.2 - 2.0	(A)* Abis	(A)* Abis	Abis	(A)* Abis	Abis	A	D**
stiff to very stiff clay	> 2.0	(A)* Abis	(A)* Abis	Abis	(A)* Abis	(A) Abis	D**	-
Medium dense sand	1 - 2	(B)* A	(A)* Abis	Abis	(B)* A	A	B	> D
Dense to very dense sand	> 2.5	(C)* B	(B)* A	A	(C)* B	B	C	> D
Weethered or factured chelk	>1	(C)* B	(B)* A	A	(C)* B	B	C	> D
Marl	1.5 - 4	(E)* C	(C)* B	B	E***	E***	E	F
Very stiff marl	> 4.5	E	-	-	-	-	F	> F
Wheathered rock	2.5 - 4	F	F	-	F***	F***	F	> F
Jointed rock	> 4.5	F	-	-	-	-	F	> F

It can be derived from chart fig.A-2, which incorporates:

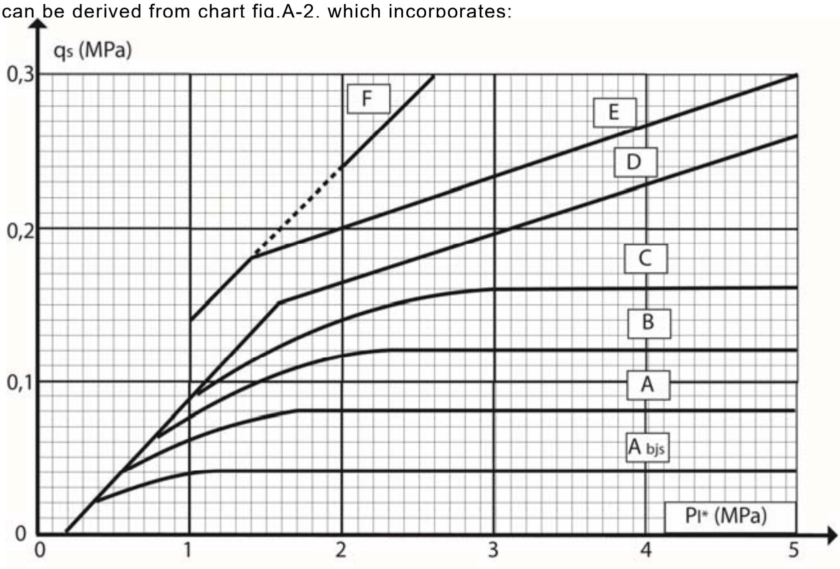


Fig. A-2.Skin friction

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Sol soils; BOOKS, ISP, Articles, refer to international committee on pressuremeter

http://icp-pressuremeter.com/pressuremeters/

STANDARD AND NORMS

- European Norm:
  ISO 22476-4

- USA Standard:
  ASTM D4719-07 Standard Test Methods
  for Prebored Pressuremeter Testing in Soils

- Russian Norm:
  GOST 20276-2012

- German norm:
  DIN ISO 22476-4

- Ménard manual pressuremeters 5 & 10 MPa with GeoSPAD®2 acquisition
- Self-Controlled Pressuremeters : GeoPAC® 5 & 10 MPa and HyperPAC® 25 MPa

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The slotted tube is placed while drilling, avoiding borehole walls remolding.

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The RotoSTAF® is a specialized drilling rig and a profitable tool for settling up all type of out-of-the-hole overburden drilling systems 76 and 90, the STAR 67/74 – NW - 104/113, and of course for pure pressuremeter works the optimal use of the so called Self-bored tube system STAF®.

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