ABSTRACT: A French engineer Louis Ménard developed in the mid 50's of 20th century a specific device for soil testing: the pressuremeter. He also proposed a useful method of foundation design on the ground of pressuremeter test results. It is possible to obtain both bearing capacity and settlement of substratum under particular loading as the pressuremeter allows to get both deformation properties (through pressuremeter modulus) and resistance failure properties (through pressuremeter limit pressure). To use the Ménard’s method of bearing capacity and settlement calculations one is obliged to possess pressuremeter data taken from a very specific zone under foundation. The author verifies here the Ménard’s rules and proposes an instruction for soil investigations on the ground of this analysis.

1. INTRODUCTION

Programming geotechnical or geological – engineering investigations for foundation design purposes, the first thing their contractor has to do is determining (usually in consultation with the investor and/or designer) the necessary reconnaissance depth. He can (and should) make use of appropriate standards and instructions being in force in his country. It is relatively easy to determine the proper drilling or penetration depth for indirect (pile) foundation. Both according to Instruction for testing the substratum of road and bridge structures (Kłosiński et al., 1998) and PN-B-02479:1998 Polish Standard Geotechnics – Soil Investigation Report – General Rules tests should reach at least 3 m below the planned piling depth. PN-B-02479:1998 Standard reminds also the recommendation connected with the record of the point 2.2.9.1 of PN-83/B-02482 Standard Bearing Capacity of Piles and Pile Foundations, as to checking five pile diameter zone under their assumed foundation depth. This depth is usually chosen deeper than the minimum bearing soil penetration (1 – 2 m) permitted by PN-83/B-02482 Standard. It may depend on piling technology or local conditions. Hence another recommendation given by Instruction... (Kłosiński et al., 1998): “boreholes should penetrate at least 6 m of bearing soil layer”. This matter can be then
easily determined between the contractor and the designer and first field test data give a chance for suitable corrections. They are usually small in percentage terms, as the reconnaissance depth for piling purposes is deep in any case, in range of more than ten, twenty or even thirty meters in typical conditions met in Poland.

Considering a direct foundation Instruction (Kłosiński et al., 1998) refers to the record of point 4.2 PN-81/B-03020 Standard Foundation Bases. It recommends to conduct both settlement calculations and the preceded investigations to such depth below the foundation level, where the additional structure loadings decrease (because of stress reduction with depth) to 30% of vertical geostatic depth value which grows with depth (overburden gravity). This zone should be extended if it ends in compressible soils of oedometer compressibility modulus at least twice smaller than in the directly underlying geotechnical layer. Also the other instruction entitled The Rules of Engineering – Geological Reporting (Bażyński et al., 1999) and PN-B-02479:1998 Standard require extending the reconnaissance depth to clarify the weak soil occurrence depth. These both publications connect the reconnaissance depth necessary for direct foundations with their width $B$, adopting the depth of $1 - 3B$ (but not less than $3 - 5m$) below foundation level as a minimum for individual or strip footing and $1B$ for raft foundation. Stricter requirements ($6 - 8m$ as a minimum investigation depth) are assumed in Instruction (op. cit.), however it deals with responsible structures (bridges). On the other hand both it and PN-B-02479:1998 Standard allow to make the required investigation zone more shallow if the top of the soils of very favourable strength parameters, specific for the given area and thick enough occurs within this zone. Borings can be terminated $0.5 - 2m$ in this hard layer. The depth of at least part of them can be reduced when soils are geotechnically homogenous and their thickness is surely bigger than the necessary minimum reconnaissance depth or the layer system is regular.

To sum up the foregoing: the necessary investigation depth under the direct foundation is usually associated with the zone of an important impact of the structure upon the substratum soils. It is estimated on the grounds of either the assumed foundation width or additional stresses calculated provisionally taking into account their declining with depth. Remarkable (to 200%) discrepancies of this estimation can be noticed when footing substrata are considered. The limitation of the reconnaissance zone under the rafts to $1B$ puzzles: it surely is not a full range of the active zone. Many designers contest this recommendation and they demand a deeper reconnaissance.

Results of more or less arbitrary choice of the investigation depth are serious considering either the economical aspect (if the reconnaissance depth is too big) or the quality of test results in the opposite case. The base of price calculation is usually one meter of drilling. Even in case of light and founded at shallow depth 2-storey houses the cost of soil investigations may grow by half if six meter deep drillings (penetrometers) are planned instead of 4 meter deep ones. Differences in prices will be higher in case of deeper drillings as usually the unit price grows with passing some investigation depths, like 10, 20 or 30 meters.

Examples below are given to convince this is a significant problem. A remarkable growth of the number of the designed and built wind turbines has taken place in Poland lately thanks to favourable political and economical atmosphere. The last generation turbines are huge, monumental structures, but still they are usually founded flat on high-dimensional (in the range of 20 m in diameter) foundations. The design supervision is usually conducted by western companies (German, Spanish etc.). They determine a necessary reconnaissance depth and the number of test points. Sometimes the depth of 10 m is sufficient (?), but usually it varies between 18 and 30 m. If 30 m depth applies to three test points surrounding the foundation and 18 m to one point in the axis the difference in costs may
reach 500%! Another example. The times when skyscrapers were built in Warsaw only have already gone. Every bigger town has had such ambitions at present. These buildings are usually founded on rafts of remarkable dimensions so the problem of a necessary drilling depth becomes really serious. Thirty meters? Fifty? Seventy five? A hundred? It is worth thinking over.

Differences in the assessment of the range of active zone, which means the zone of important influence of the structure on the substratum, seem to be the main reason of the divergence in evaluation of the necessary reconnaissance depth under foundation of various, founded flat structures. And these differences result from various design rules (methodology).

Limit pressure and expected substratum settlement are examined in every respect but in a not too complicated way by L. Ménard’s comprehensive method of foundation design on the grounds of pressuremeter test results (Ménard 1975). Analysing and verifying Ménard’s rules the author constructs directives on indispensable scope of soil investigations for the purposes of direct structure foundation design.

2. DATA REQUIRED FOR FOUNDATION DESIGN ACCORDING TO L. MÉNARD

The starting point of Ménard’s concept is the acknowledgement, that ultimate bearing capacity of the subsoil \( q_l \) (calculated from at rest state \( q_o \) to the moment of failure) is proportional to additional pressure measured from horizontal earth pressure at rest \( p_o \) to limit pressure \( p_l \) assigned during pressuremeter test:

\[
q_l - q_o = k_p (p_l - p_o)
\]  

(1)

To keep the substratum deformations caused by additional foundation loadings in the pseudo-elastic (proportional to growing stresses) phase Ménard proposed safety factor 3 and the allowable bearing capacity \( a \) formula:

\[
q_a \leq \frac{k_p}{3} (p_l - p_o) + q_o
\]  

(2)

The principle is to consider the substratum zone from foundation level to the depth of 1.5\( B \) below (reminder: \( B \) means foundation width). As pressuremeter test is made for every layer which differs from the neighbouring one or even (more orthodox approach) every meter, \( p_l \) means the result of single pressuremeter test only in the case of very narrow foundation. The 1.5\( B \) zone may be as large as a few meters or – in the case of foundation rafts - 20 – 30 m and even more. The equivalent limit pressure \( p_{le} \) is to be computed then to exchange \( p_l \) value in the formula (2). It is a geometric mean of limit pressures measured from foundation level to the depth of 1.5\( B \) below:

\[
p_{le} = \sqrt[n]{p_{l1}p_{l2}...p_{ln}}
\]  

(3)

Too differentiated \( p_{li} \) values should not be taken into account. They cannot exceed more than 1.5 times the minimum value (Amar et al., 1991). Such too high values should be deleted in the case of two layer configuration, when the foundation level is within a weaker layer and 1.5\( B \) zone reaches a stronger one (Tarnawski 2007).

The remaining variables of the formula are unimportant in typical conditions of not too deep foundation in bearing soils (high \( p_{le} \) value). On the other hand \( q_o \) and \( k_p \) are controlable to a degree. One should try to increase them in the cases when additional structure loading \( q_l \) comes close to the allowable value or exceeds it.

The bearing coefficient \( k_p \) value depends generally on:

- kind (category) of soil,
- foundation depth,
- kind and shape of the foundation (dimensions; \( L \) – length, \( B \) – width).

It can be described by the formula (line equation):

\[
k_p = a[1 + b(0.6 + 0.4 \frac{B}{L}) \frac{D}{B}]
\]  

(4)
Parameters of these equation are within the ranges $a = 0.8 \text{–} 1.3$ and $b = 0.25 \text{–} 0.80$ (Gambin and Frank 1995). Their values (which depends on the kind of soil) are collected in Table 1. Soft rocks which are not common and not tested in Poland are deleted from the list.

**Table 1. Parameters of the equation (4)**

<table>
<thead>
<tr>
<th>Kind and class of soil (state, $p_l$ [MPa])</th>
<th>Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clays, silts „A” (firm; $p_l &lt; 0.7$)</td>
<td>$a = 0.8$</td>
</tr>
<tr>
<td>Clays, silts „B” (stiff; $p_l 1.2-2.0$)</td>
<td>$a = 0.8$</td>
</tr>
<tr>
<td>Ilý, gliny, pyły „C” (hard and very hard; $p_l &gt; 2.5$)</td>
<td>$a = 0.8$</td>
</tr>
<tr>
<td>Sands and gravels „A” (loose; $p_l &lt; 0.5$)</td>
<td>$a = 1.0$</td>
</tr>
<tr>
<td>Sands and gravels „B” (medium dense; $p_l 1.0-2.0$)</td>
<td>$a = 1.0$</td>
</tr>
<tr>
<td>Sands and gravels „C” (dense; $p_l &gt; 2.5$)</td>
<td>$a = 1.0$</td>
</tr>
</tbody>
</table>

The $D_e$ variable in the formula (4) is not a real but equivalent foundation depth. It is computed as the relation of $p_l$ values measured above the foundation level (from the $0-h$ range) to the equivalent limit pressure $p_{le}$ value:

$$D_e = \frac{1}{p_{le}} \int_0^h p_l(z)dz$$

A clear distinction between the situation when a burden causes vertical displacement of compressive layer only (i.e. soil consolidation) and the one when it causes both vertical and horizontal displacements (which means not only voluminal but also shape deformation of soil) is introduced in Ménard’s settlement computation method.

According to Ménard the former case applies the situation, when a relatively thin (of the thickness $Z$ lesser than a half of foundation width $B$) compressible layer occurs under the foundation. Consolidation settlement of this layer is expected then first of all and one-dimensional method is to be used to assess it. This problem may concern raft foundation or wide road embankments for example, so the thickness of this “relatively thin” layer may be in range of a few meters or even much more. Nevertheless this situation is out of the considered problem of the required depth under the structure to be investigated, because it is obvious that the whole thickness of this weak layer must be estimated and investigated then.

Starting from the general formula of elastic half-space settlement caused by additional load $q^*$ by absolutely rigid circular foundation lying on the surface of this half-space (three-dimensional deformation method; Wilun 1976, 2000):

$$s = \frac{\pi}{4} \frac{1-v^2}{E_o} q^* B$$

L. Ménard replaced there the modulus of elasticity $E_o$ by shear modulus $G$, as the former in undrained conditions differs from effective one and the latter does not change. He got:

$$s = \frac{\pi}{8} \frac{1-v^2}{G} q^* B$$

as:

$$G = \frac{E}{2(1+v)}.$$  

Substitution of the maximum Poisson coefficient value $v = v_u = 0.5$ which corresponds to shape (undrained) deformations and deviatoric settlement $s_d$ computation was the next step:

$$s_d = \frac{\pi q^* B}{16 G}$$

Hence consolidation settlement $s_c$ was the difference between (7) and (8) and the total settlement:

$$s = s_d + s_c = \frac{\pi q^* B}{16 G} + \frac{\pi (1-2v) q^* B}{16 G}$$

Thanks to secured exclusively horizontal direction of deformations, the pseudo-elastic soil deformations which take place during pressuremeter test are permitted to be described by a parameter similar to shear modulus:

$$G_m = \frac{\Delta p}{\Delta V}$$
Pressure and volume increments connected with the straight line section of pressuremeter curve refer to $V_m$ volume: the mid-point of this section. Thanks to adaptation of a constant, average Poisson coefficient value $\nu = 0.33$ and using the formula (8) pressuremeter modulus can be defined as a parameter related to Young modulus:

$$E_M = 2(1+\nu)G_M = 2.66V_m \frac{\Delta P}{\Delta V}. \quad (12)$$

Transformations of the formula (10) applied by Ménard consist of reduction of numerical coefficient values (among the others because of the choice of foundation level at the depth of $D_e \geq B$), replacing the shear modulus $G$ by bulk compressibility modulus $K$ (= 2.66$G$ when $\nu = 0.33$) in the element of the formula related to consolidation settlement, and then the use of pressuremeter moduli $E_M$ (= 2.66$G$ or $\alpha K$; $\alpha$ is a reological coefficient) in both elements. Moreover there were introduced foundation shape (influence) coefficients $\lambda_d$ and $\lambda_c$ (similar as in the classical solution) as well as a reference (equivalent) foundation width $B_0 = 0.6$.

The situation where the substratum compliance is constant along the whole active zone may take place theoretically only. This is why L. Ménard proposed different pressuremeter modulus values for both elements of his formula: $E_d$ and $E_c$. They come from unequivocally defined substratum zones and they are averaged adequately. The described operations change the pressuremeter settlement formula for (Baquelin et al., 1984):

$$s = \frac{2}{9E_d} q B_0 (\lambda_d \frac{B}{B_0})^\alpha + \frac{\alpha}{9E_c} q^2 \lambda_c B \quad (13)$$

Following the concept of the differentiation of immediate and consolidation settlement, the averaged value of pressuremeter modulus $E_d$ applied to the first, deviatoric element of the formula is calculated in a completely different way than $E_c$ value which is to serve for conso-lidation settlement assessment. The analysed substratum is divided into five zones. The unit of this division is half width of the foundation: $0.5B$. The bottom of successive zones is found at the following depths below the foundation level: $0.5B$ ($E_1$ zone), $1B$ ($E_2$ zone), $2.5B$ ($E_{3-5}$ zone), $4B$ ($E_{6-8}$ zone) and $8B$ ($E_{9-16}$ zone). This latter level means the end of the active zone.

Shear deformations and settlement first grows fast with depth and then wanes slowly. Therefore the $E_d$ value is calculated following the appropriate formula (harmonic mean):

$$\frac{1}{E_d} = \frac{1}{4} \left[ \frac{1}{E_1} + \frac{1}{0.85E_2} + \frac{1}{E_{3-5}} + \frac{1}{2.5E_{6-8}} + \frac{1}{2.5E_{9-16}} \right]. \quad (14)$$

If more than one pressuremeter test was carried out within a zone its average modulus value is calculated with the harmonic mean as well.

The zone directly under the foundation is essential for consolidation settlement. Hence the modulus $E_c = E_1$.

The reological coefficient value may alter. It should be established separately for $E_d$ and $E_c$ elements of the formula (13). One is to choose dominating values or to average them.

3. NUMERICAL EXAMPLES

The presented above description of Ménard foundation design rules shows, that allowable bearing capacity estimation requires checking of $1.5B$ below foundation level as a minimum and structure settlement estimation - $8B$ zone. Especially this second condition is severe. This means the need of dozen or so meters of the substratum to be investigated even if both foundation depth $D$ and foundation width $B$ are typical, 1 or 2 m for example. The width of 10 – 20 m is a common case for foundation raft. Then the defined above active zone grows rapidly even to 100 m! To test it completely is often too difficult, uneconomical and groundless from engineering – geological point of view, as normally both the values of pressuremeter moduli and soil strength grow with depth.

There are two possible procedures in case of lack of $E_M$ values from deeper depths.
- If we are not sure the soils occurring below the investigation depth are characterized by more favourable $E_M$ values than the investigated ones we adopt for them $E_z$ values from the last investigated zone;
- If we can assume pressuremeter modulus values to grow with depth and $E_{9/16}$ is unknown we are permitted to calculate $E_d$ value with the formula:

$$\frac{1}{E_d} = \frac{1}{3.6} \left[ \frac{1}{E_1} + \frac{1}{0.85E_2} + \frac{1}{2.5E_{6-8}} \right],$$

and if also $E_{6/7/8}$ is unknown, the formula:

$$\frac{1}{E_d} = \frac{1}{3.2} \left[ \frac{1}{E_1} + \frac{1}{0.85E_2} + \frac{1}{E_{3-5}} \right].$$

Even the latter, simplified solution means that to evaluate correctly the settlement under 20 m width foundation raft (often used for high wind turbine towers) one is obliged to check the substratum to the depth of 50 m ($2.5B$) below foundation depth. It is rather unusual. Trying to optimise the necessary test depth range it is worth to verify these rules difficult to fulfill.

Two simple cases will be examined. The first one, rather typical, will refer to the subsoil of strength growing with depth and decreasing compressibility. The second is opposite. It will refer to foundation on relatively “strong” soils, whereas weaker ones occur deeper. The borders of geotechnical layers will agree with modulus zones ($E_1$, $E_2$, etc.) to simplify computation (Fig. 1).

Typical pressuremeter parameter values of the analysed soils have been used in computations to make the examples more realistic (Table 2).

**Table 2. Approximate, typical pressuremeter parameter values (Briaud 1992)**

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Cohesive soils</th>
<th>Non-cohesive soils</th>
</tr>
</thead>
<tbody>
<tr>
<td>State of soil</td>
<td>firm</td>
<td>stiff</td>
</tr>
<tr>
<td>$P_l$ (kPa)</td>
<td>200 – 400</td>
<td>400 – 800</td>
</tr>
<tr>
<td>$E_M$ (kPa)</td>
<td>2500 – 5000</td>
<td>5000 – 12000</td>
</tr>
</tbody>
</table>

Twelve variants of different (in an ordered way) pressuremeter modulus numerical values have been examined. They are compiled in Table 3.

The idea of the variants is as follows. Three options of soil quality in foundation level were considered in both examples. Homogenous soils are assumed to occur to the depth equal to the foundation width in “a” and “c” variants. They are either less (stiff clays, dense sands; “a”) or more compressible (firm clays and medium dense sands respectively; “c”). In “b” variant the stronger soils occur in the foundation level and the weaker ones lower, between the depths of 0.5 and $1B$. These options are combined with the stronger (I) and weaker (II) substratum (from the depth equal to $B$). The compressibility of this deeper substratum decreases from the variant “1” to the variant “4”. It is homogenous (with respect to $E_M$ value) in the variants “1” and “4” and in the variants “2” and “3” $E_M$ grows with depth.

**Fig. 1 Examplary substratum profiles on the background of the “settlement zones”**
To obtain possibly large differences of calculation settlement „soils” of distinctly contrasting pressuremeter modulus were placed in successive zones, but still these examples can be met in reality.

Table 3. Data for numerical examples

<table>
<thead>
<tr>
<th>Example I</th>
<th>Variant</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>a</td>
<td>b</td>
<td>c</td>
<td>b</td>
</tr>
<tr>
<td>$E_1$ (kPa)</td>
<td></td>
<td>7 000</td>
<td>7 000</td>
<td>7 000</td>
<td>7 000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3 500</td>
<td>3 500</td>
<td>3 500</td>
<td>3 500</td>
</tr>
<tr>
<td>$E_2$ (kPa)</td>
<td></td>
<td>7 000</td>
<td>7 000</td>
<td>7 000</td>
<td>7 000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3 500</td>
<td>3 500</td>
<td>3 500</td>
<td>3 500</td>
</tr>
<tr>
<td>$E_{3,5}$ (kPa)</td>
<td></td>
<td>12 000</td>
<td>12 000</td>
<td>12 000</td>
<td>24 000</td>
</tr>
<tr>
<td>$E_{6,8}$ (kPa)</td>
<td></td>
<td>12 000</td>
<td>24 000</td>
<td>24 000</td>
<td>24 000</td>
</tr>
<tr>
<td>$E_{9,16}$ (kPa)</td>
<td></td>
<td>12 000</td>
<td>24 000</td>
<td>24 000</td>
<td>24 000</td>
</tr>
<tr>
<td>Example II</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>a</td>
<td>b</td>
<td>c</td>
<td>b</td>
</tr>
<tr>
<td>$E_1$ (kPa)</td>
<td></td>
<td>20 000</td>
<td>20 000</td>
<td>20 000</td>
<td>20 000</td>
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<td></td>
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<tr>
<td>$E_2$ (kPa)</td>
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<tr>
<td>$E_{3,5}$ (kPa)</td>
<td></td>
<td>2 500</td>
<td>2 500</td>
<td>2 500</td>
<td>5 000</td>
</tr>
<tr>
<td>$E_{6,8}$ (kPa)</td>
<td></td>
<td>2 500</td>
<td>5 000</td>
<td>2 500</td>
<td>5 000</td>
</tr>
<tr>
<td>$E_{9,16}$ (kPa)</td>
<td></td>
<td>2 500</td>
<td>5 000</td>
<td>2 500</td>
<td>5 000</td>
</tr>
</tbody>
</table>

The remaining calculation assumptions were as follows: foundation depth $D_b = B$, $L/B$ ratio = 5, reological coefficients $\alpha = 1/3$ for sands and 1/2 for clays or silts.

The situations when one disposes full (ie. to the depth of 8B) compressibility characteristics of the substratum were considered first. Analysis of the results obtained is presented in Tables 4a and 4b.

If the reconnaissance depth is reduced to 2.5B and we use the formula (16) to determine $E_{db}$, the result in Example I will be always bigger than in the case, when the formula (14) is used. This overestimation was the biggest (9.2 – 12.2%) for the variant 3, when the soils below 2.5B were twice less compressible than in the zone above and the smallest (in the range of 4.3 – 8.1%), when the substratum was homogenous and weaker than in the remaining variants (the variant 1; $E_{3-16} = 12000$ kPa).

The use of the formula (16) to determine $E_{db}$ in Example II causes underestimation of calculation settlement. It is minimal (in the range of 0.6 – 4.6%) for variant 3 where below 2.5B there occurred twice less compressible soils than in the zone above this depth and the biggest (18.3 – 27.4%) if the substratum was homogenous and weaker than in the remaining variants (the variant 1; $E_{3-16} = 2500$ kPa).
If soil investigations do not pass the depth $B$ and the presence of much less compressible substratum is not discovered in Example I, the acceptance of the presence of stiff or firm clays also below $B$ will be a safe solution. Settlement overestimation will reach 25.1 – 64.1% then, depending on the variant. It will obviously be the bigger the more hypothetic soil (which modulus was taken into account) is more compressible than the real one.

If soil investigations do not pass the depth $B$ and the presence of very compressible substratum is not discovered (Example II), the assumption of the presence of dense or medium dense sands also below $B$ will be an extremely risky solution. Settlement underestimation will reach 55.3 – 391.6% then, the more the more compressible (than the assumed one) is the real soil which occurs deeper than $B$.

The described above settlement calculation results connected with incomplete (according to Ménard methodology) soil investigation data are shown on Fig. 1, where also the results obtained with the use of the formula (15) are presented. The latter ones require data from the zone down to $4B$ and they obviously are comparatively the most accurate ones.

5. CONCLUSIONS

The analysis carried out could not take into account every aspect of the problem of substratum reconnaissance depth indispensable for direct foundation design because of a limited size of this text. Hence the conclusions cannot be full and universal as well. However the examples described above had been selected in such a way to give a chance for establishing at least general rules one should follow to determine reconnaissance depth rational in given conditions. The presented directives do not apply to indirect foundation and weak soils, as the aspect of indispensable investigation depth is not complicated then.

Basic conclusions are as follows:

- The optimal investigation depth for the needs of direct foundation cannot be function of the assumed foundation width only, but it must result from the anticipated soil conditions too.
- When the strenght of soils grows with depth and their compressibility decreases (typical situation) it is enough to know soil conditions to the depth of $1.5B$ below foundation level, but it is desirable to extend this zone, even deeper than $4B$, in more complex cases.
- The above does not mean that drillings or in situ tests must reach such a depth. It is necessary to develop an engineering – geological model. The model, possibly similar to one of the diagrams presented on Fig. 1, must determine the trend of geotechnical parameter changes with depth.

If we know that strength and compressibility parameters grow with depth, the detailed geotechnical investigations should be focused on determining the parameters of the weakest layers recognized as still bearing ones and on a general estimation of the properties of positively bearing soils. Investigations can be terminated near the top of 1.5 – 3 times less compressible soils than the weakest ones. The detailed investigation range may be even smaller than $B$ especially in cases of large dimension foundation rafts. Why? A 100%
error in weak soils compressibility assessment may cause a similar mistake in settlement estimation (also in the dangerous direction), whereas even a big mistake in geotechnical characteristics of bearing soils which occur deeper than \( B \) below foundation level means less than 10% of error in settlement calculations. Catching the top and estimating the parameters of low compressible soils is important to avoid too conservative settlement estimation caused by lack of more certain calculation data.

The situation when lithogenic homogeneous soils of not too much differentiated geotechnical parameters occur along the while active zone is the most comfortable. It requires confirmation of this hypothesis and possibly accurate estimation of strength and compressibility parameter numerical values only.

The simulations carried out indicate that the substrata, where under a thin low compressible layer (of the thickness equal to \( B \) in the analysed Example II) there occur distinctly weaker soils, are the most risky. An error in their settlement susceptibility evaluation may mean mistake in settlement assessment from a few to 40% (see Table 4a), or even bigger as according to Ménard (1975) the model of “a weaker layer at a certain depth” should basically be used then and the results obtained following this method are always bigger than with the use of the formula (13) (Tarnawski 2006).

A similar mistake – and in the dangerous direction may take place when tests do not cover the whole active zone and approximate calculation solutions are applied. But too shallow (terminated above the top of weak layer) soil investigations may cause simply dramatic consequences. The real settlement may be even several times bigger than computational ones (Fig. 2). Furthermore, if too high unit loads are designed, weak soil can yield aside. This may even cause a building disaster. At the same time even significant inaccuracy in strong soil compressibility estimation can cause an error in computational settlement result in range of 10 – 30% only, although it regards soils occurring directly under the foundation. Now, in consideration thereof: shall we aim for obtaining data about physical and mechanical properties of soils down the whole active zone in case of the model “the deeper, the weaker”? Not necessarily. Considering the first limit state we examine the substratum zone from the foundation level to the depth of \( 1.5B \) below. Weak soils either will be found within this zone causing drastical decrease of allowable bearing capacity or they will occur so deep that (high) unit loads under foundation of minimal dimensions will be rapidly reduced with depth. Following the methodology proposed by L. Ménard, the detailed testing of \( 1.5B \) zone together with obtaining estimated data on weak soils that occur deeper should be sufficient to lead computational settlement close to the real one.

The above analysis teaches that it is possible to reduce remarkably (to the depth of \( 1.5B \) in more complex and difficult cases or to even less than \( 1B \) below foundation level in more favourable situation) the detailed geotechnical investigations. However one should dispose a reliable engineering – geological model then and understand well what data and geotechnical parameters must be determined precisely enough within the framework of detailed geotechnical investigations.

REFERENCES


Polska Norma PN-83/B-02482 Fundamenty budowlane. Nośność pali i fundamentów palowych. (Foundations – Bearing Capacity of Piles and Pile Foundations)