The right way to define the pressuremeter creep pressure

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The pressuremeter test is a positive exception among other well-known methods of *in situ* soil investigations. It provides direct data (parameters) to assess the compressibility and strength of soil. Years of experience allow to estimate the approximate values of these parameters in relation to different types and states of soil (Table 1).

A graph "Pressiorama" [2] presents a similar picture of the variability of pressuremeter parameters, but extended to the rocks. The Pressiorama (it is a registered trademark deposited by Jean-Pierre Baud at French *Registre National des Marques* in 2006) takes into account the values of net limit pressure p^*_{LM}, pressuremeter (Ménard) modulus E_M as well as E_M / p^*_{LM} ratio (Fig. 1). It can be seen that the "path of growth" of both parameters and E_M / p^*_{LM} ratio from soft or loose, unconsolidated soils through normally consolidated and overconsolidated ones to solid (cemented) rocks and a specific "come back" caused by weathering, fracturing and/or altering. The authors [3] call these paths "the genetic cycle".

Table 1. Approximate, typical values of the pressuremeter parameters [5]

Kind of soil	Cohesive soils					Non-cohesive soils			
State of soil	very soft	soft	stiff	hard	very hard	loose	medium dense	dense	very dense
Limit pressure p^*_{LM} [MPa]	0 – 0,2	0,2 – 0,4	0,4 – 0,8	0,8 – 1,6	> 1,6	0 – 0,5	0,5 – 1,5	1,5 – 2,5	> 2,5
Ménard modulus E_M [MPa]	0 – 2,5	2,5 – 5,0	5,0 – 12,0	12,0 – 25,0	> 25,0	0 – 3,5	3,5 – 12,0	12,0 – 22,5	> 22,5

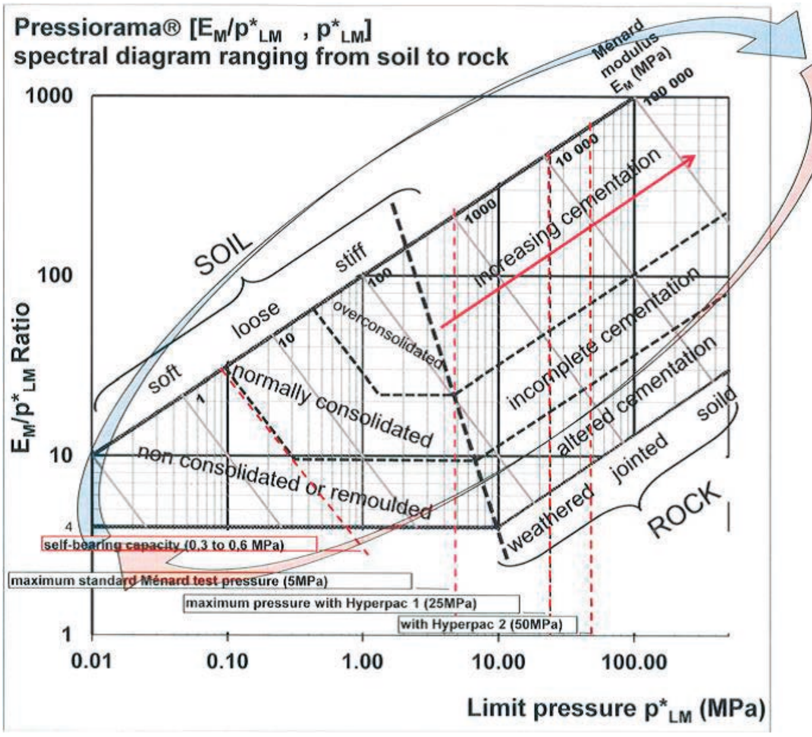


Fig. 1. Pressiorama classification [3]

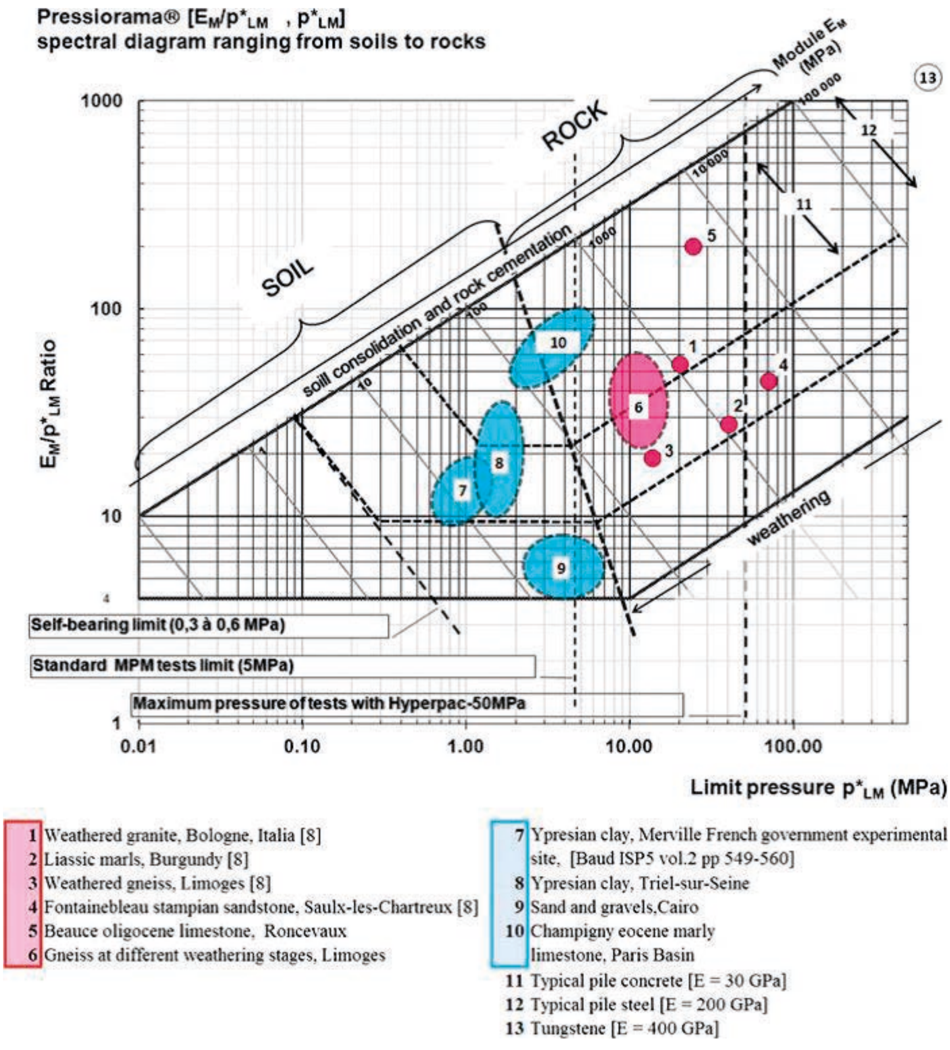


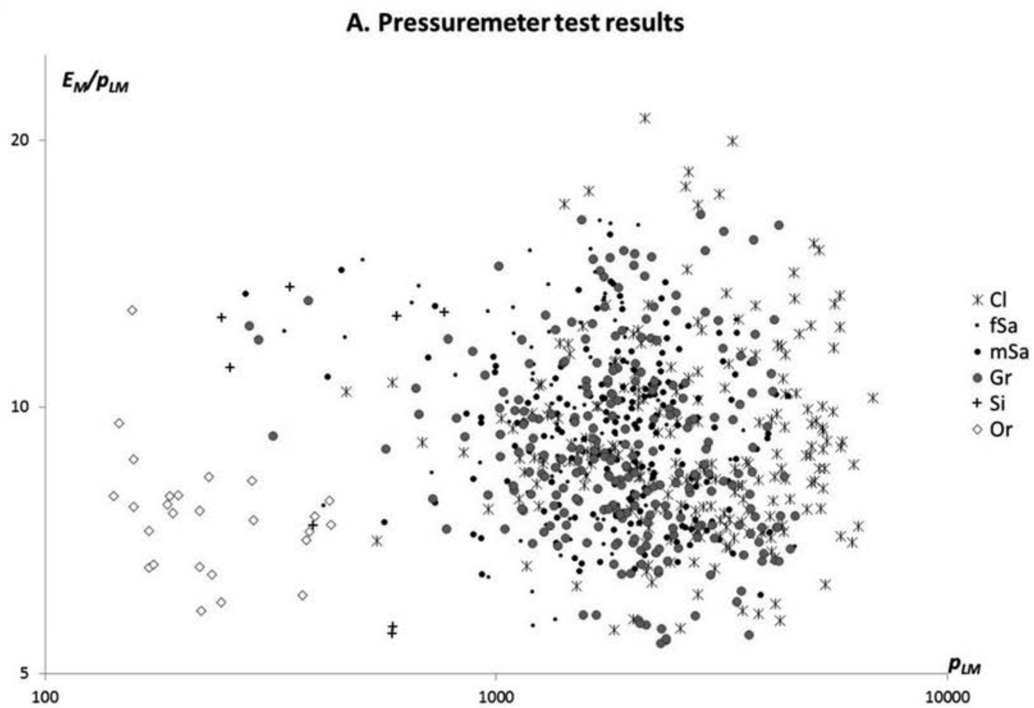
Fig. 2. Examples of soils and rocks on Pressiorama pressuremeter classification [3]

The Pressiorama seems to be a useful tool for presenting domains of various soils and rocks expressed by ranges of the above mentioned parameters, being characteristic for them (Fig. 2).

The situation complicates when we do not consider rocks or soils of different ages, origin, kind and from different places, but would expect a differentiation of the characteristics of various soils tested in one place (for a particular project). Such an expectation may fail.

CHANGEABILITY OF THE PRESSUREMETER PARAMETERS

The data sets shown in Fig. 3 refer (among the others) to the soils as diverse as Pleistocene sands and (almost pure) Miocene clays (tested in Rybnik, Poland). It can be seen there that their point clouds overlap. Why do they not show diversity? Let us try to find the reasons.



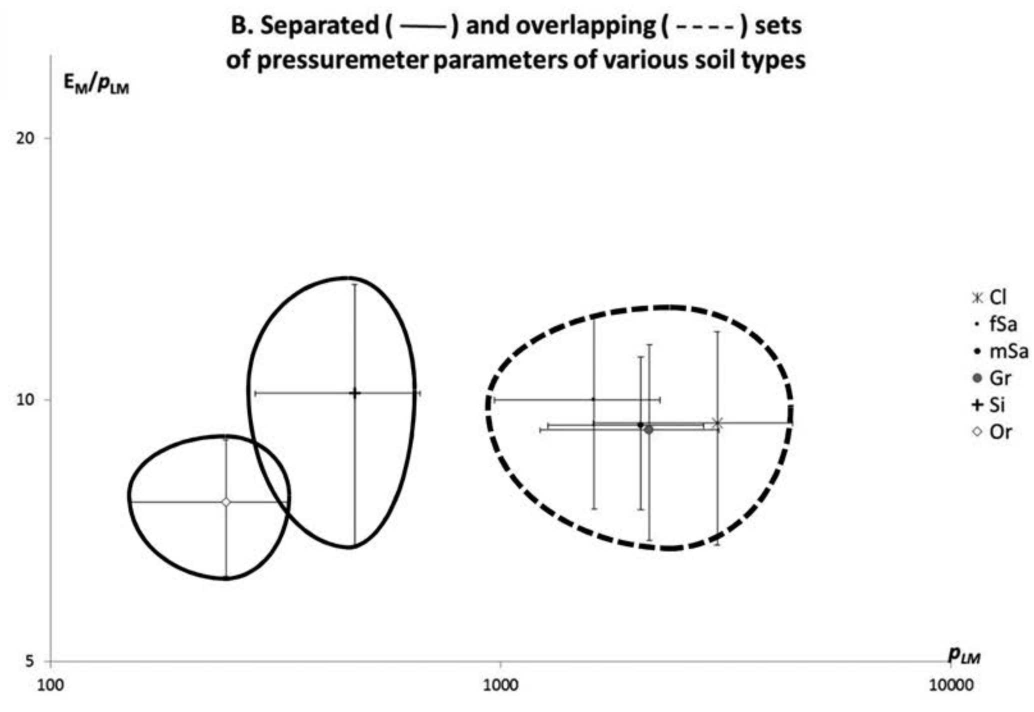


Fig. 3. Pressuremeter test results carried out in: Cl – Miocene maritime clays; fSa, mSa, Gr – Pleistocene fluvioglacial sands and gravels (Rybnik, Poland); Si – Holocene river or lake silts (muds) and Or – Holocene organic muds (Northern Poland) presented in Pressiorama manner (note: p_{LM} is used instead of p^*_{LM}). The data scatter is illustrated through the standard deviation. Weak Holocene deposits form separate sets mainly because of lower and lower pLM. Despite the varied lithology older sediments occur together (the dotted line)

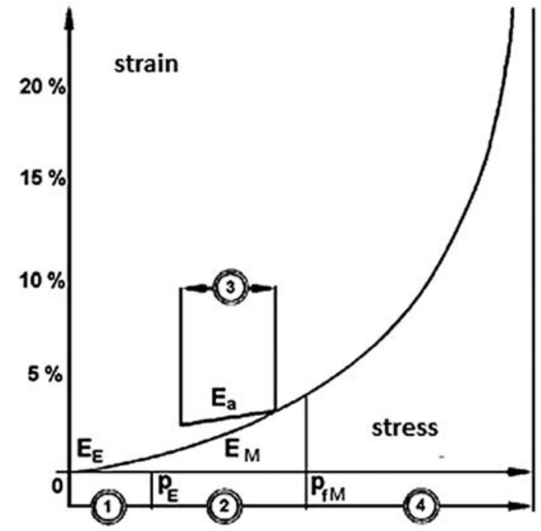


Fig. 4. Soil reaction in the field of deviatoric stress
1 – real elastic strains phase, *2* – pseudo-elastic strains phase, *3* – cyclic deformations, *4* – large strains phase, E_a – cyclic deformation modulus, E_M – pressuremeter modulus, p_E – limit pressure for elastic strains, p_{fM} – creep (critical) pressure [9; slightly changed]

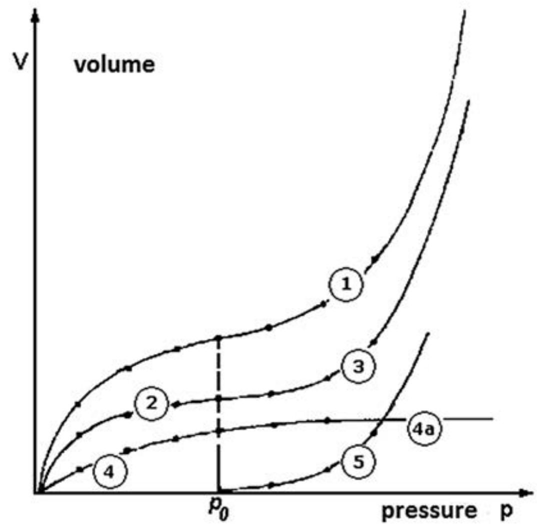


Fig. 5. Factors influencing the shape of pressuremeter curve (1): volume losses used for the prevention of relaxation (2) and resulting from the compression of the ring of disturbed soil (4). A perfect curve (2-3) will be obtained if the borehole wall is not disturbed at all. The horizontal section (4a) of the graph presenting volume losses caused by borehole wall disturbance means lack of impact of this phenomenon on the shape of the final phase of the curve. Curve (5) presents a theoretical shape of stress-strain curve [13, 14, 15]

Considering a typical investigation depth for major building projects, say 20 to 40 m we may expect (within this wide depth range) medium dense to very dense sands (but also loose ones in the superficial zone) as well as both normally consolidated and overconsolidated clays of differentiated natural moisture, which means of differentiated consistency. No wonder that limit pressure varied (see Fig. 3A) between p_{LM} = 0.3 and 4.4 MPa for sands and between p_{LM} = 0.5 and 6.8 MPa for clays. This parameter, even reduced to standard deviation zone (Fig. 3B) could not differentiate these soils.

The issue of E_M / p_{LM} ratio is even more complex. First we may expect both normally consolidated (lower E_M / p_{LM} ratio) and overconsolidated soils (both sands and – especially – clays) and second – we have to remember that E_M is very sensitive to the quality of test.

In his first significant paper Louis Ménard [9] presented a curve showing the response of a loaded soil (Fig. 4). Pressuremeter modulus is connected there with a pseudo-elastic deformation phase. In fact, the soil reaction should start from the value of primary horizontal stress p_o (see Curve 5 on Fig. 5) at the considered depth. This may happen during a perfectly performed self-boring pressuremeter test. A graph of a well-done Ménard pressuremeter test is presented on Fig. 5 as Curve 1. This figure explains the reasons of sensitivity of Ménard pressuremeter modulus zone. The entire difference between the shape of Curves 1 and 5 is due to phenomena occurring during the preparation of the test cavity: soil relaxation and disturbance of borehole wall. This means that the first (left from p_o line: see Fig. 5) part of the modulus zone (as it is chosen following the rules of ISO 22476-4:2012 Standard) is formed by rather random factors. Generally speaking it is more difficult to produce and keep a good quality test cavity in sands than in clays. E_M / p^*_{LM} ratio as low as E_M / p^*_{LM} = 4 to 8, considered usually as typical for non-cohesive soils (see the position of № 9 domain: Cairo sands and gravels on Fig. 3), means low (possibly: too low) modulus values. They may result from poor (systematically worse than in clays) quality of test cavity. Poor quality (considering the moduli) test results should obviously be excluded from data sets like the ones presented on Pressiorama. The trouble is to distinguish a proper test in relatively more compressible soil from testing in a disturbed zone.

There are two pressuremeter parameters, which depend on test quality to a limited extend only. They are limit pressure p_{LM} and creep pressure p_{fM}. The reason is simple, which is that they are read from the middle and the final part of the curve, away from the influence of test cavity disturbances. However, other factors affect the accuracy of the numerical values of these parameters.

GETTING RATIONAL VALUES OF CREEP PRESSURE

Limit pressure is often obtained indirectly and approximately by extrapolation. This can be avoided by preparing a proper test cavity (not too broad of undisturbed wall) and performing the test to a volume near 700 cm^3.

Standard definition of creep pressure calculation seems to allow to obtain its value easily. The problem is the correct interpolation of the diagonal straight line due to possibly large scatter of data points, like the one presented on the graph taken from French Standard [1]: Fig. 6.

The way to obtain creep pressure value proposed in ISO 22476-4:2012 Standard [8] seems to imply a certain helplessness: “The creep pressure value shall lay between p_{fMi} (graphically determined according to the standard procedure; “$_I$” stands for “initial”) and p_2 (the end of the pseudo-elastic phase). The closer p_{fMi} and p_2 are, the better is the quality of the test.” This is the reason that p_{fM} is not treated as an indicatory parameter.

To understand creep better, which is a slow process of change occurring in the soil under the additional load, it should be borne in mind that ground subsidence under loads is the sum of:

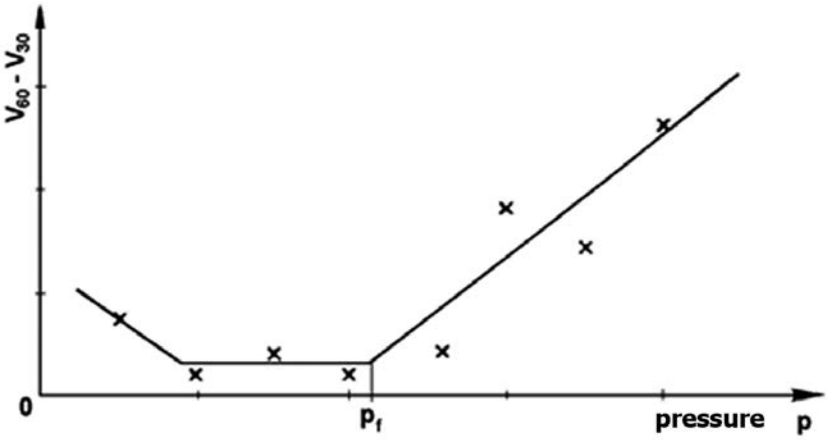


Fig. 6. Creep pressure p_f interpretation according to the French Standard

- immediate settlement resulting from shear strain and side displacements,
- proper (primary) consolidation
- secondary compressibility (Fig. 7).

The most typical engineering practice is to ensure that loadings of the structure will be smaller (in particular cases: significantly smaller) than the critical load. This way soil deformations are limited almost exclusively to immediate and consolidation settlements. The first one is a major reaction of non-cohesive soils and the second one is characteristic for cohesive and organic soils. Consolidation is a simultaneous decrease in water content and pore volume in the ground leading to elimination of excessive pore pressure generated by the applied load. Creation and persistence of excessive pore water pressure and relatively slow process of consolidation result from small permeability of fine grained soil.

The role of secondary settlement increases when approaching the critical load and especially after exceeding its value. Secondary settlement (secondary consolidation) occurs after the dispersion of excess pore water pressure caused by the load (i.e. without water outflow), and consists of reorganization (compaction) of soil particles. According to the most popular, elastic – (perfectly) plastic Coulomb – Mohr model of loaded soil behavior, this process proceeds at constant effective stress equal to the maximum shear resistance of the soil. Real soils do not meet the condition of proportionality between load and deformation in the elastic phase, and their subsequent plastic flow does not require constant value of effective stress. After reaching a peak this resistance drops to a constant residual value.

The (over)simplification of the model is only part of the problem. Associating the beginning of the creep phase with maximum shear resistance is a common misconception. Exceeding the maximum shear resistance means destruction of soil structure, which is achieving not creep but limit pressure, which lies at the end of the large strain phase (Fig. 4) and it is approximately twice the creep pressure p_{fM}.

Based on the elastic-plastic model of soil, the secondary consolidation is sometimes called "constant speed" or "not disappearing" deformation. This concept has been supplemented in recent years and third order deformations ("the accelerating ones") have been distinguished from secondary deformation [10, 7]. Finally, creep response is being divided into three stages unfolding after application of a stress: the first period of transient creep during which the strain rate decreases with time, followed by creep at nearly constant rate for some period, and then going into accelerating creep rate leading to failure or "creep rupture" [12]. These three stages are named primary, secondary, and tertiary creep (Fig. 8).

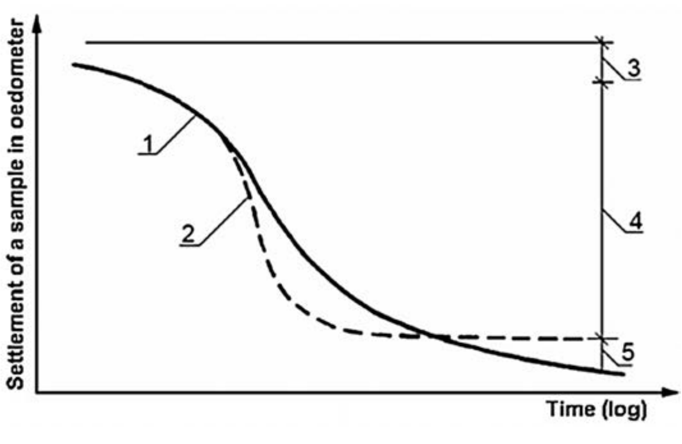


Fig. 7. Settlement of loaded soil. *1* – consolidation curve in oedometer test, *2* – settlement curve according to Terzaghi's classical approach, *3* – immediate settlement, *4* – primary consolidation, *5* – secondary compressibility (from [4]).

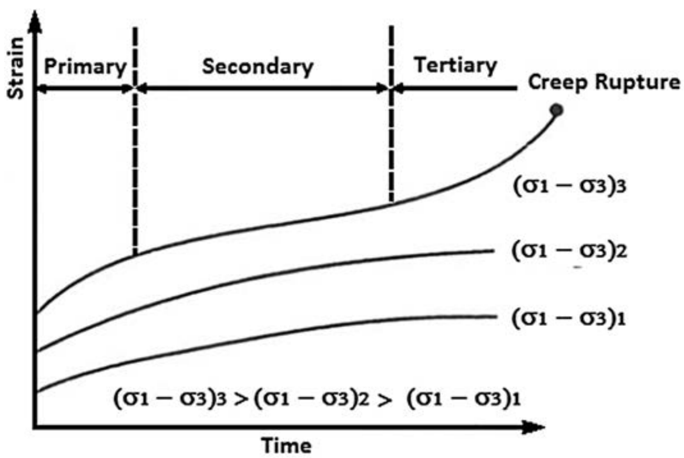


Fig. 8. Creep phases which follow growth of deviatoric stress: primary, secondary and tertiary creep and finally creep rupture [12]

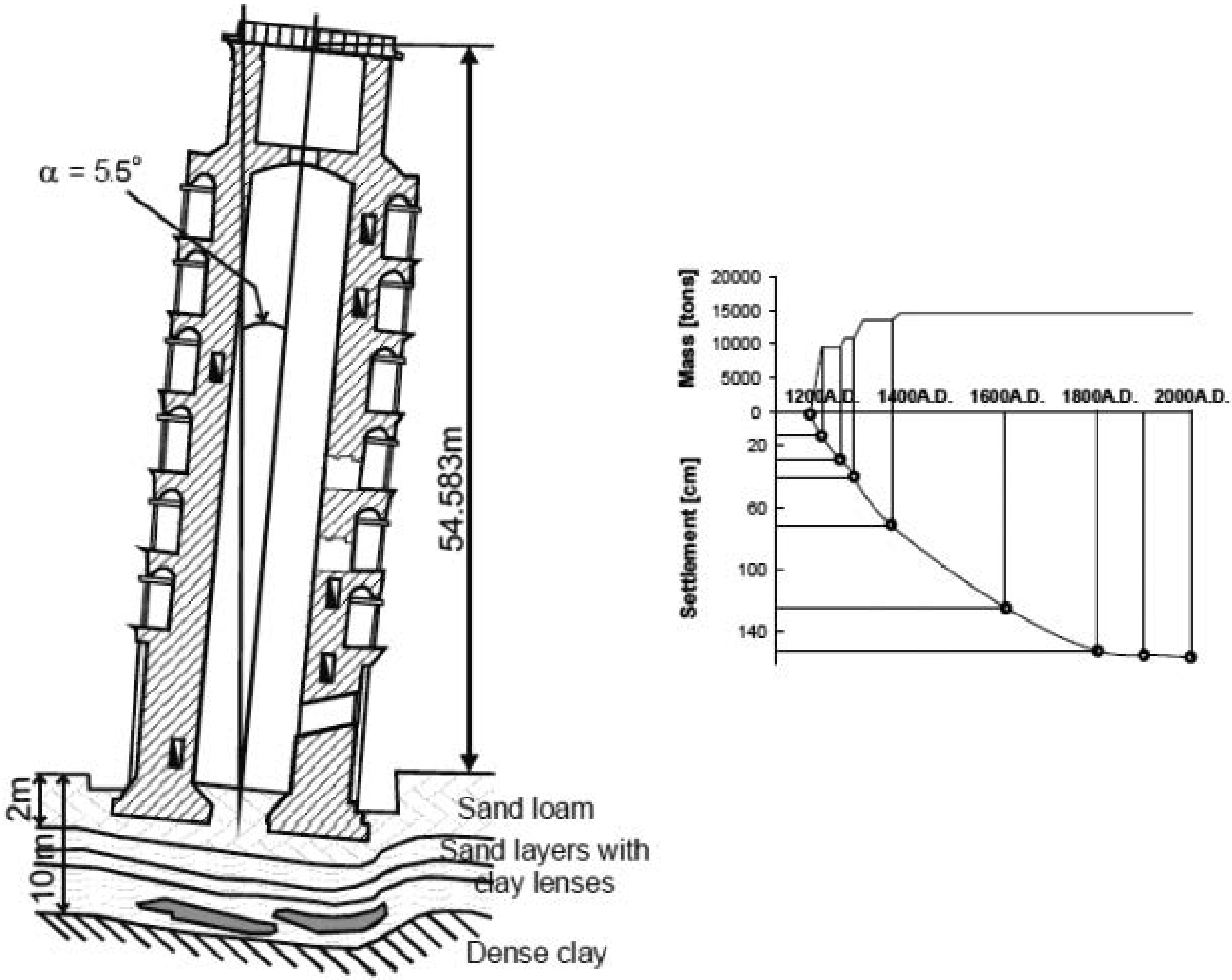


Fig. 9. Diagram of the Leaning Tower of Pisa church and its subsoil and settlement observation results [7]

The world's best known example of a structure subject to eight hundred years of secondary settlement is obviously the leaning tower of Pisa (Fig. 9). Settlement measurements carried out for centuries showed a declining tendency. This means that the process, although very extended in time, was limited to the primary creep phase (Fig. 8). However, does this mean that the tower was not in danger? Of course it was as it could have collapsed due to the increasingly eccentric loads [11]. Remedial works that ended in 2002 led to the inhibition of the tilt of the tower. More specifically – to restore the situation observed about 200 years earlier [6, 11]. Lead weights were used on the north, the least settled side of the tower as a temporary solution and removal of soil from the same side as the final remedy. Opponents of these solutions argued that they would increase the load (weights) and decrease (undercut) passive earth pressure, possibly leading to a building disaster, if the substratum had been close to its bearing capacity. Nothing like that happened. This confirms that the state before repair work exceeded the critical load, but it was far from exceeding the bearing capacity.

Having considered all that, let us return to the pressuremeter definition of creep. According to the author's experience in conducting pressuremeter tests, such a chaotic scatter of creep pressure points as shown on Fig. 6 can only be found in poor quality tests. Usually it looks different (Fig. 10). The line that should be a diagonal straight line divides into two line segments with one being steeper than the other. The working hypothesis is as follows: the line segment with the lesser slope should be connected with consolidation (primary) creep and the steeper one with shear (tertiary) creep.

Excluding the points of the latter (steeper) section (Fig. 10) we will obtain two desired effects: harmonization of p_{fM} and p_2

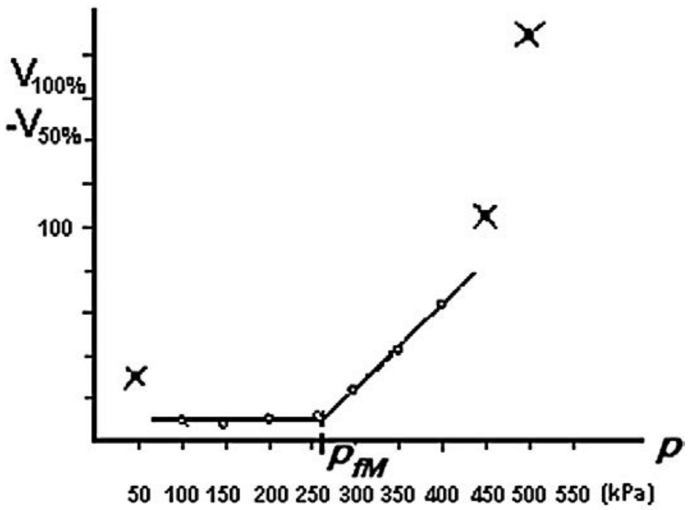


Fig. 10. Scheme for determining the creep pressure according to the author

(through systematic reduction of p_f value) as well as the parameter differentiating better different types of soil.

The proposal presented above has been incorporated into a Polish software dedicated to pressuremeter test interpretation [16]. Its use is easy. One should not use the last data points, which form the "too steep" section, the same way as it is done with the first data point(s) placed too high to be incorporated into the horizontal straight line.

To support the proposal presented in Figure 10 an exemplary set of numerical test results from Rybnik (the project analysed in Figures 3 and 11) and, for comparison, from another project have been summarized in the following Tables 2 and 3. They contain fifty examples from hundreds test results interpreted by the author.

All presented results are repetitive. Creep pressure values p_{fM} calculated the author's way are closer to p_2 than p_{fMi}, sometimes even smaller than p_2. The method fulfills the ISO Standard expectation to bring the p_{fM} closer to p_2 differentiating better different types of soil.

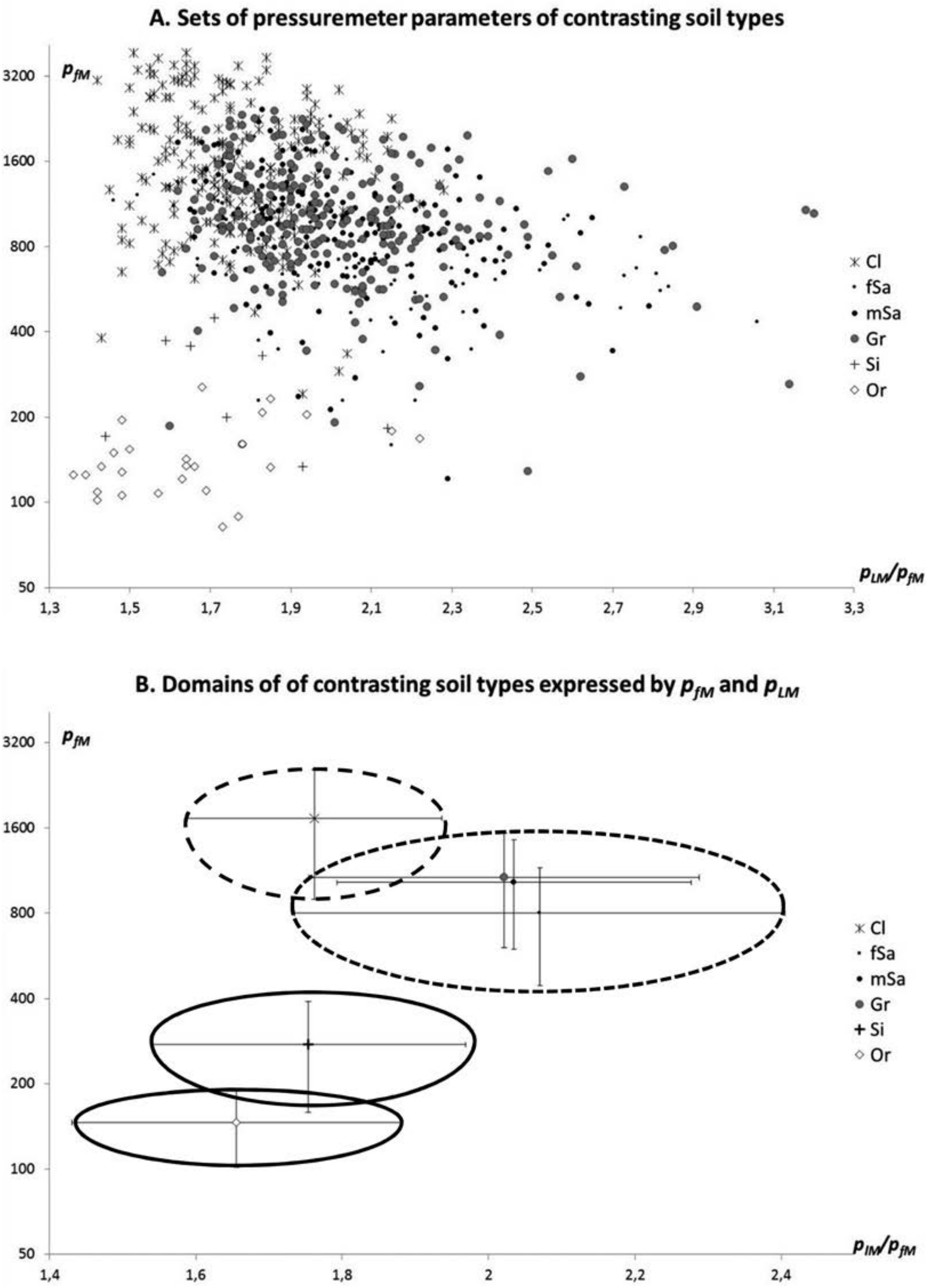


Fig. 11. Pressuremeter test results in soils described under Fig. 3. Thanks to other (than in Fig 3) set of differentiating parameters (with p_{fM} defined a new way), the bearing soils (Cl vs. Sa&Gr; dotted lines) form now separate (overlapping only slightly) domains

Table 2. Initial (p_{fMi}) and final (p_{fM}, p^*_{fM}) values of pressuremeter creep pressure (sample data from Rybnik, Poland; test points № 01 and 03)

Ord. № \ Kind of soil	Miocene clays				Pleistocene sands (1 – 10) and gravels (11 – 20)			
	p_{fMi} [kPa]	p_2 [kPa]	p_{fM} [kPa]	p^*_{fM} [kPa]	p_{fMi} [kPa]	p_2 [kPa]	p_{fM} [kPa]	p^*_{fM} [kPa]
1.	1948	1706	1725	1359	599	438	524	460
2.	1994	1622	1886	1493	434	221	347	338
3.	2289	1641	1963	1556	1171	850	1049	1015
4.	2166	1647	1776	1355	997	769	885	838
5.	2177	2146	2092	1657	1393	1095	1171	1109
6.	1898	1674	1684	1222	1162	910	1051	976
7.	2668	2170	2500	2024	982	691	756	667
8.	2005	1775	1826	1336	1102	800	952	849
9.	2396	1716	2089	1573	1033	751	878	735
10.	3483	2918	3254	2559	1978	1747	1853	1476
11.	3805	3409	3490	2781	663	473	562	484
12.	3591	2912	3072	2357	679	466	530	439
13.	3325	2940	2882	2145	713	535	672	525
14.	1035	779	975	749	795	584	741	580
15.	1126	910	1081	841	705	556	662	488
16.	2273	2121	2085	1681	1137	944	1064	877
17.	3268	3131	3054	2636	1253	1159	1133	918
18.	3038	2196	2675	2243	1196	985	1074	831
19.	2141	1741	1992	1547	1960	1552	1674	1391
20.	2806	2615	2569	2028	2071	1551	1823	1526

Table 3. Initial (p_{fMi}) and final (p_{fM}, p^*_{fM}) values of pressuremeter creep pressure (sample data from Plock, Poland)

Ord. № \ Kind of soil	Pleistocene glacial clays (test point № 01)				Pleistocene glacial sands (test points № 01 – 03)			
	p_{fMi} [kPa]	p_2 [kPa]	p_{fM} [kPa]	p^*_{fM} [kPa]	p_{fMi} [kPa]	p_2 [kPa]	p_{fM} [kPa]	p^*_{fM} [kPa]
1.	358	221	315	282	742	574	632	496
2.	547	428	499	435	1744	1247	1637	1400
3.	733	555	636	540	588	380	538	421
4.	661	602	620	498	2105	1422	2006	1767
5.	1269	995	1197	1030	390	270	332	240

SUGGESTED SOIL IDENTIFICATION METHOD

Using the test results, which have been presented on Fig. 3, the author has collected and analysed various pairs of parameters with the use of p_{LM}, E_M / p_{LM}, p_{fM} and p_{LM} / p_{fM}. Please note that the symbol p_{fM} does not mean this time the standard Ménard creep pressure but the creep pressure obtained the way proposed in the previous section of this paper. The p_{LM}/p_{fM} ratio has turned out to be the parameter that differentiates more evidently the domains of contrasting soil types, especially when it is compared with p_{fM} the way known from "Pressiorama" (Fig. 11), but this may be not a common rule [17].

CONCLUSIONS

The $E_M \big/ p^*_{LM}$ ratio compared with p^*_{LM}, with support of E_M value (ie. the Pressiorama graph) allows to present the diversity of the main pressuremeter parameters of various soil and rock types tested all over the world.

This scheme works worse when we analyse various soils tested to a considerable depth in one area. This happens because both p^*_{LM} and E_M depend first of all on the strength of soil, which is usually variable with depth or from one point to another. In addition E_M depends also on consolidation degree (and this may be changeable too) and on quality of the test.

The author observed different behaviour of different soils during the plastic deformation zone of pressuremeter test. This zone is shorter (the curve runs steeper) in the case of cohesive soils and longer for sands and gravels.

To test this differentiation the author needed a more stable parameter than the traditional p_{fM}. This corrected creep pressure value is obtained by excluding untypical data as described above.

Although both Rybnik clays and sands were characterized by similar (and high) pressuremeter limit pressure values they have appeared to be different when described by p_{LM} / p_{fM} ratio. Its average value was below 1.8 for clays and higher than 2 for sands.

As clays in Rybnik were generally "stronger" than sands the above differentiation could be reinforced by using p_{fM} on the second axis of the graph. In the opposite case or when comparing soils of distinctly different strength rather p_{LM} (p^*_{LM}) would be recommended.

The discovery of at least two different phases of soil creep encourages further research into the diagnosis of the physical diversity of this phenomenon. Ménard pressumeter seems to be a perfect tool to investigate it.

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