Foundation of a Blast Furnace constructed on Loess Soil and the Computation of Settlement

Fondations pour Haut Fourneau Construit sur un Terrain Lœssique—Calcul du Tassement

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Summary

In case of loess soil, it is advisable to use the method of deep compaction for foundations of heavy buildings. When computing the settlement, the greater rigidity of the compacted soil may be taken into account by applying the method of the representative thickness. With the computation of the settlement–time curve, having considered the axially symmetrical three-dimensional pore-water flow, the pore-water pressure at the nodal points of the network can be computed step by step. When choosing an oblong network, computation of the anisotropic permeability characteristics of the loess soil will become as simple as in case of isotropy. The measurements of the settlement have shown smaller values than computed, although all favourable circumstances have been taken into consideration.

Sommaire

Dans le cas d'un sol loessique, il est avantageux de compacter en profondeur les fondations des constructions lourdes. Lors du calcul du tassement, on ne devra pas négliger la plus grande rigidité du sol compacté en faisant intervenir une épaisseur fictive. Lorsqu'on trace la courbe tassement/temps, en tenant compte de l'écoulement spatial, symétrique par rapport à l'axe, de l'eau interstitielle, il est possible de calculer, par récurrence, la pression de l'eau interstitielle aux noeuds du réseau. En adoptant un réseau rectangulaire, le calcul de la perméabilité anisotrope du loess sera aussi simple que le calcul de la perméabilité isotrope. Les mesures de tassement ont donné des valeurs inférieures aux valeurs calculées, bien qu'on ait tenu compte de toutes les circonstances favorables.

When founding heavily loaded structures on loess soil provision must be made for additional settlement, due to soaking of the loaded ground through the rising of the ground water table or the rupture of water pipes, and for keeping these settlements within allowable limits.

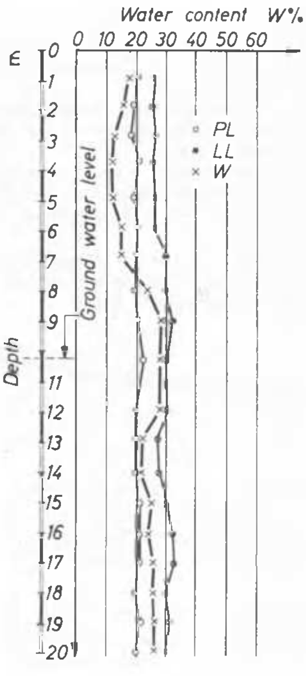

Fig. 1 Soil profile and Atterberg limits
Coupe verticale des sols et limites d'Atterberg

On the site of the blast furnace in question, the subsoil consists of loess down to a great depth (Fig. 1). The upper strata are composed, up to the limit of saturation, of genuine loess, and the underlying strata of soaked loess. As experience has shown, the rising of the ground water table must be taken into account when planning industrial establishments with a large water consumption on loess soil, since the seepage of great quantities of water, both during usage and from sewage, cannot be avoided.

In order to prevent additional settlement and reduction of the load-bearing capacity caused by soaking, the ground beneath the circular spread footing of the blast furnace was compacted to the level of the ground water, destroying the structure of the dry loess, and producing thereby a compact soil stratum insensitive to soaking.

The deep compaction was effected by driving pointed hollow steel piles, of 40 cm (approximately 16 in.) diameter in a triangular pattern, spaced 1·10 m (3 ft. 7$\frac{3}{8}$ in.) apart. After withdrawing the pipes, the bore holes were filled with compacted loess soil. The average porosity of n = 44–45 per cent in the upper soil strata could be reduced by this process to n = 33–35 per cent (KARAFIÁTH, 1954).

The practical application of deep compaction was preceded by field tests, to determine the porosity, the compressibility and the sensitivity to soaking of the samples taken both from the natural and from the compacted soil. Loading tests were also carried out using a loading platform with a surface area of 5000 cm^2 (5·4 sq. ft.) in the upper strata, and a loading plate with an area of 600 cm^2 ($\frac{2}{3}$ sq. ft.) inside bore holes in the underlying strata.

The following average moduli of compressibility were obtained by means of the loading tests:

Compacted soil above ground water level	K = 600 kg/cm^2 = 8600 lb./sq. in.
Natural soil below ground water level	K = 100 kg/cm^2 = 1425 lb./sq. in.

In the analysis of the stresses the increased rigidity of the upper, compacted strata was taken into account approximately by using the method of representative strata thicknesses.

The stresses occurring in the underlying stratum were determined by substituting an imagined stratum, of equal rigidity but thicker than the underlying one, for the upper, more rigid stratum.

This representative thickness of stratum, according to a formula by G. J. Pokrovsky is

$$h_r = h\left(\frac{E_1\gamma_2}{E_2\gamma_1}\right) \quad \ldots . \ (1)$$

h_r being the thickness of the representative stratum, E_1 the modulus of elasticity in the first stratum, E_2 the modulus of elasticity in the second stratum and $\gamma_{1,\,2}$ the bulk density.

Besides the approximate process, Newmark's influence charts can be employed as well, by means of which the stresses can also be computed at points outside the axis.

Fig. 2 shows the distribution of stresses below the so-called characteristic point when they are computed by means of the representative thickness of stratum.

In designing the settlements by the usual method, and adding

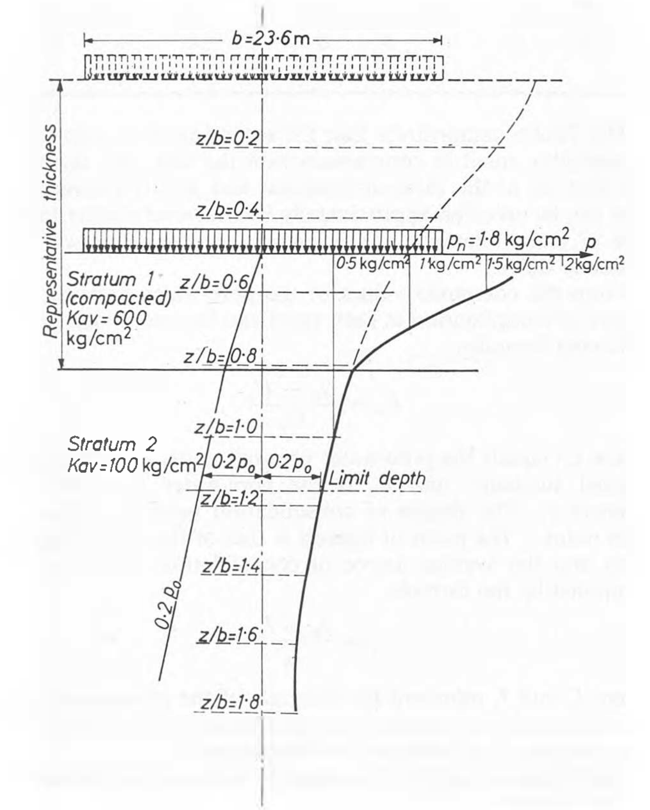

Fig. 2 Stress distribution and computation of the settlements
Répartition des efforts et calcul des tassements

the compressions produced by vertical stresses, the full settlement is $\rho = 5{\cdot}5$ cm, if the compressions are computed according to Hungarian Standards to the depth where the excess pressures on the soil attain 20 per cent of the net overburden pressure.

In the given case, the progress of the settlements must be known in relation to time, in order to evaluate their effect.

For an object symmetrical about its axis, the basic equation of consolidation is as follows (FLORIN, 1948):

$$\frac{\partial h_u}{\partial t} = \frac{1}{3\gamma_w}\frac{\partial\Theta^*}{\partial t} + \frac{k_{wr}}{m_v\gamma_w}\left(\frac{\partial^2 h_u}{\partial r^2} + \frac{1}{r}\frac{\partial h_u}{\partial r}\right) + \frac{k_{wz}}{m_v\gamma_w}\left(\frac{\partial^2 h_u}{\partial z^2}\right) \quad \ldots . \ (2)$$

where Θ^* denotes the sum of the principal stresses due to the external loading.

The equation can be solved by the method of finite differences. The hydraulic head of pore-water pressure in the nodel points of a quadratic network can be analysed in moments differing by uniform intervals of time from the following equations (FLORIN, 1948):

$$h_{t+1,\,i,\,k} = h_{t,\,i,\,k} + \frac{1}{3\gamma_w}(\Theta^*_{t+1,\,i,\,k} - \Theta^*_{t,\,i,\,k}) + \frac{k_{wr}}{m_v\gamma_w}\frac{\Delta t}{\Delta r^2}\left[h_{t,\,i+1,\,k} + h_{t,\,i-1,\,k} - 2h_{t,\,i,\,k} + \frac{\Delta r}{2r_i}(h_{t,\,i+1,\,k} - h_{t,\,i-1,\,k})\right] + \frac{k_{wz}}{m_v\gamma_w}\frac{\Delta t}{\Delta z^2}(h_{t,\,i,\,k+1} + h_{t,\,i,\,k-1} - 2h_{t,\,i,\,k}) \quad \ldots . \ (3)$$

where $h_{t,\,i,\,k}$ denotes the hydraulic head of the pore-water pressure in the time and at the point determined by the indices. If

$$\alpha_r = \frac{k_{wr}}{m_v\gamma_w}\frac{\Delta t}{\Delta r^2} \quad \ldots . \ (4)$$

and

$$\alpha_z = \frac{k_{wz}}{m_v\gamma_w}\frac{\Delta t}{\Delta z^2} \quad \ldots . \ (5)$$

prevail, and if the lateral length of the network should be chosen as

$$\alpha_r = \alpha_z = 0{\cdot}25$$

then

$$\frac{\Delta z}{\Delta r} = \left(\frac{k_{wz}}{k_{wr}}\right)^{\frac{1}{2}} \quad \ldots . \ (6)$$

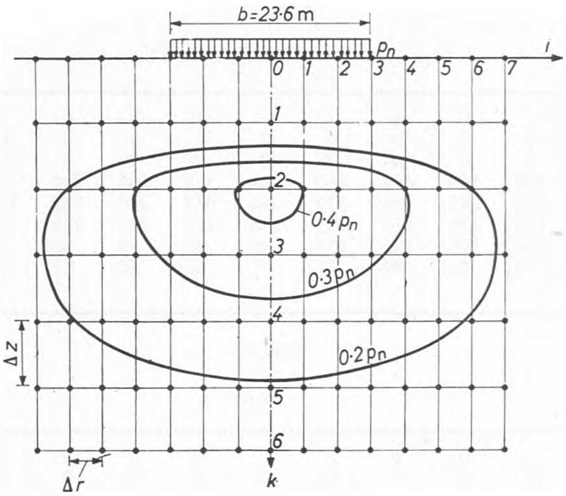

Fig. 3 Network for the computation of the pore-water pressures and the hydraulic heads at the time $t = 6\Delta t$
Réseau pour le calcul de la pression de l'eau interstitielle et des charges hydrauliques après une période $t = 6\Delta t$

The anisotropic permeability characteristics for loess soils can be taken into consideration simply by applying the calculations based on the nodal points of the quadratic network to the nodal points of the rectangular network chosen in conformity with the anisotropy of the system.

Considering this and choosing $\alpha = 0{\cdot}25$,

$$h_{t+1,\,i,\,k} = \frac{1}{3\gamma_w}(\Theta^*_{t+1,\,i,\,k} - \Theta^*_{t,\,i,\,k}) + \frac{1}{4}\Diamond_{t,\,i,\,k} + \frac{1}{8}\frac{\Delta r}{r_i}(h_{t,\,i+1,\,k} - h_{t,\,i-1,\,k}) \quad \ldots . \ (7)$$

where $\Diamond_{t,\,i,\,k} = h_{t,\,i+1,\,k} + h_{t,\,i-1,\,k} + h_{t,\,i,\,k+1} + h_{t,\,i,\,k-1}$

In the case given, the network was selected as follows (Fig. 3):

$$\Delta r = \frac{b}{6} = 3{\cdot}93\ m$$

$$\Delta z = \left(\frac{k_{wz}}{k_{wr}}\right)^{\frac{1}{2}}\Delta r \simeq 2\Delta r = 7{\cdot}86\ m$$

k_{uz}—following test results—being about four times greater than k_{wr}.

In a vertical direction, the first row of the network approximately coincides with the ground water table where pore-water pressure is zero. This yields one of the boundary conditions.

From equations 4 and 5 the value of Δt is obtained:

$$\Delta t = 0{\cdot}25\frac{\gamma_w m_v}{k_{wr}}.\Delta r^2 = 0{\cdot}25\frac{0{\cdot}001\,.\,0{\cdot}017}{10^{-7}.\,1{\cdot}7}.393^2 \cong 1{\cdot}5 \text{ months}$$

During the period of construction ($T = 18$ months $= 12\Delta t$) the net foundation pressure increased to $p_n = 1{\cdot}4$ kg/cm². After the charging of the blast furnace it rose suddenly to $p_n = 1{\cdot}8$ kg/cm². Instead of a uniform increase of the load, a step-wise augmentation is assumed; after each interval of $3\Delta t$ the load is augmented abruptly by

$$p_n = 3\frac{1{\cdot}4}{12} = 0{\cdot}35 \text{ kg/cm}^2$$

The computations are tabulated.

Pore-water pressures originating from the load increments and computed by Newmark's chart are contained in Table 1.

By the aid of this table the pore-water pressures arising at the end of a Δt interval can be computed by the equation

$$h_{t+1,i,k} = \frac{1}{4}\diamondsuit_{t,i,k} + \frac{1}{8}\frac{\Delta r}{r_2}(h_{t,i+1,k} - h_{t,i-1,k} \quad \ldots . \quad (8)$$

Table 1

$$3h_{t=0,i,k} = \frac{1}{\gamma_w}(\Theta^*_{t+1,i,k} - \Theta^*_{t=0,i,k})$$

		0	1	2	3	4	5	6	7 i
0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0
2	498	514	498	460	414	358	304	254	212
3	288	292	288	279	254	232	210	182	152
4	174	178	174	165	158	153	147	135	123
5	105	106	105	104	96	90	84	80	75
6	79	86	79	71	67	62	55	47	43
k									

Table 2

$$\frac{1}{4}\diamondsuit_{t,i,k}$$

		0	1	2	3	4	5	6	7 i
0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0
2	316	322	316	298	268	238	206	174	144
3	311	317	311	292	271	244	216	188	160
4	184	186	184	174	167	157	146	133	118
5	116	118	116	109	105	99	93	85	68
6	80	82	80	76	70	66	60	55	42
k									

As equation 8 is composed of two terms, two further tables (Tables 2 and 3) have been prepared containing the various values of these terms. Adding at each point the values of Tables 2 and 3, we obtain the hydraulic head values belonging to the time $t + \Delta t$. Fig. 3 shows the hydraulic heads of pore-water pressure after $6\Delta t$ intervals have elapsed.

Table 3

$$\frac{1}{8}\frac{\Delta r}{r_i}(h_{t,i+1,k} - h_{t,i-1,k})$$

		0	1	2	3	4	5	6	7 i
0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0
2	(7)	0	7	5	4	3	3	2	2
3	(2)	0	2	2	2	1	1	1	1
4	(2)	0	2	1	0	0	0	0	0
5	0	0	0	0	0	0	0	0	0
6	0	0	0	0	0	0	0	0	0
k									

The Tables demonstrate that the second term of equation 8 is negligibly small in comparison with the first one, therefore the analysis of the three-dimensional and axially symmetrical case can be executed approximately in a manner similar to the case of plane relations. (The stresses acting on the soil will evidently differ.)

From the computed values of the pore-water pressure, the degree of consolidation at each point can be determined by the following formula:

$$U_v = \frac{U_0 - U_t}{U_0} \quad \ldots . \quad (9)$$

where U_0 equals the pore-water pressure under the full charge applied suddenly, and U_t is the pore-water pressure at a moment t. The degree of consolidation naturally differs at each point. The point of interest is that of the average settlement and the average degree of consolidation, which can be computed by the formula

$$U_v = \frac{I_0 - I_t}{I_0} \quad \ldots . \quad (10)$$

where I_0 and I_t represent the integrals of the pore-water pressures, U_0 and U_t respectively, related to the volume of the soil mass beneath the footing of the foundation.

Fig. 4 shows a curve of settlements computed in the manner described above.

Settlements since the beginning of the construction work were measured at four points of the footing, with an exactitude

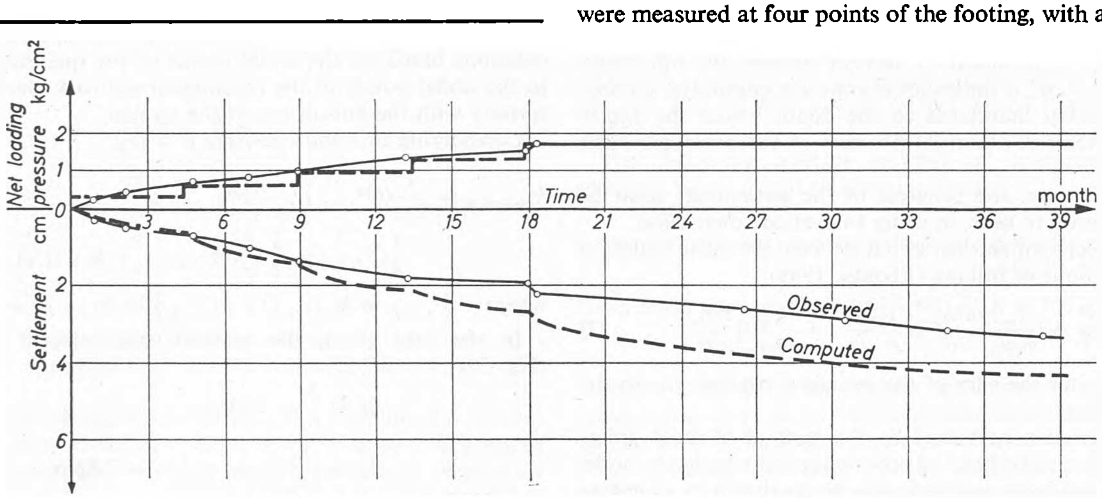

Fig. 4 Computed and observed settlements of the blast furnace
Tassement calculé et observé du haut fourneau

of 0·1 mm, and related to an underground bench mark founded at a depth of 20 m and at an adequate distance from the blast furnace, using a 'Wild N 3' levelling instrument.

The settlements measured are also shown in Fig. 4.

Conclusions

(1) The settlements observed are smaller than calculated, although all the favourable circumstances have been considered in analysing the stresses.

This may be attributed to the fact that with artificial compaction important horizontal stresses also arise in the soil.

(2) The method of designing the consolidation described above entails more computing work than the usual method, but the result corresponds well to reality.

(3) The foundation problem of the blast furnace could be solved by employing deep compaction. The result was that the settlements occurring after it was put into operation were negligibly small.

References

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