SURFACE SUBSIDENCE DUE TO OPEN-PIT COAL MINING

By

Á. KÉZDI

Department of Geotechnique, Technical University, Budapest

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Introduction

Since the 'fifties, open-pit mining has increasingly been used to extract low-calorific coal deposits located near the ground surface. This was motivated also by the possibility of complete mechanization. Utilization of machinery requires, however, to dewater soil layers above and below the coal deposit before starting the excavation. Dewatering is needed also to reduce groundwater pressure in order to eliminate the risk of hydraulic failure in the bottom. Within the range and as a function of dewatering, neutral stresses in the soil (pore-water stresses) decrease, while effective stresses increase, resulting in layer compression and surface subsidence. In a homogeneous soil mass, subsidence caused by a given water table lowering depends on the soil compressibility.

Knowledge of the numerical value of the layer compressibility is important for estimating the effect of open-pit mining on the environment; it is often necessary for deciding over legal and indemnification cases. This is why recently, the Department of Geotechnique was commissioned in several cases to make investigations under given conditions in the regions of *Visonta* and *Bükkábrány*, and to give an estimate on the subsidence value. For the analytic estimation, use may be made of the settlement calculation methods of Soil Mechanics, however, it has to be noted that these methods have been developed to determine the settlement of shallow foundations and this aim determined the mode of soil excavation and of laboratory tests. Even if these requirements are fulfilled the results of settlement calculations are rather uncertain; calculated and measured values often differ significantly. Accepting principally the possibility of calculation, we have to assume one-dimensional linear compression and consolidation, since required data are unknown. Obviously, reality is different; because of stratification, different permeabilities, water table differences, consolidation occurs in three dimensions. Compared to the usual settlement problem, another difference is due to the very slow variation of loads. All these contribute to the rather poor calculation accuracy, inferior to that of a good estimation.

Calculation of subsidence in pervious soils

The soil layer is shown in Fig. 1. It is assumed to be homogeneous and at least medium pervious. Before dewatering, water table was at depth z assumed to be constant. The distribution diagram of the total vertical stresses is line *0 1 2 3*; at most a slight inflection would appear at *1* due to the bulk density, somewhat higher below the water table because of soil saturation. This slight difference may be neglected.

Beside total stresses, also the effective and the neutral stress values have been plotted vs. depth. Compression is due to the increase of effective stresses; total vertical compression of a layer of infinitesimal thickness is, as before:

$$dy = \frac{\bar{\sigma}_z}{M} dz .$$

The total subsidence is calculated in two parts: as sum of compressions of layers having the thicknesses h_1 and h_2:

$$y = y_1 + y_2 = \int_{z_1}^{z_2} \frac{\Delta\bar{\sigma}_z}{M} dz + \int_{z_2}^{z_3} \frac{\Delta\bar{\sigma}_z}{M} dz .$$

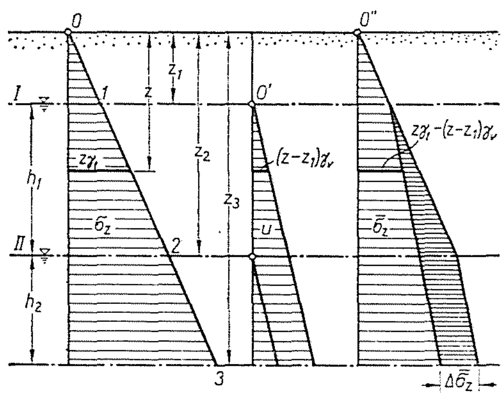


Fig. 1. Stress state variation due to water level lowering in homogeneous media

Integration — either numerical or analytical — is simple, even section-wise, divided into several layers with different M values, in knowledge of

— the increase of effective stress vs. depth;

— the variation of compression modulus vs. depth i.e. vertical stress;

— the integration limits.

With the given assumptions, the effective stress increment is simple to determine as seen in the figure. It is somewhat more difficult to assume a reliable numerical value for the compression modulus, primarily because of the scarcity and great scatter of available data. Certain considerations permit to estimate an upper and a lower limit, or even a function $M = a + bz$ can be established for the compression modulus being a linear function of depth. Determination of integration limits, especially of z_2, is, however, also rather uncertain. In a homogeneous, infinite semi-space $z_2 \rightarrow \infty$, the calculation of settlement encounters the same difficulty as in the case of the settlement of footings, a paradoxon to be solved with arbitrary assumptions: the settlement value tends to infinity even in case of a compression modulus which increases with depth. (Settlement calculation of footings generally involves the assumption of a so-called limit depth. Some procedures determine the limit depth on the basis of the threshold gradient.) A finite value is only obtained by assuming a perfectly impermeable — thus, incompressible — layer to exist at limit depth z_h confining the compressible layer. If such a layer exists, deformations can be determined according to Fig. 2 showing specific and cumulative compression values vs. depth, indicating both s_1 and s_2 values for the latter.

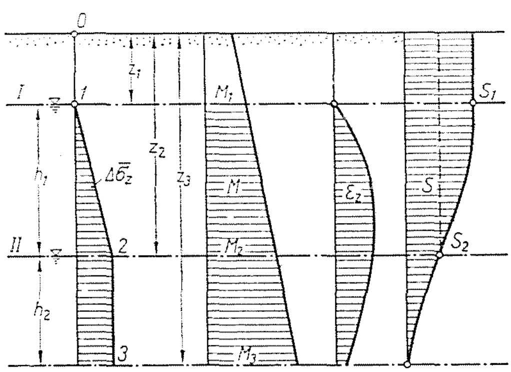


Fig. 2. Specific and summarized compression value

Determination of limit depth z_h is rather uncertain. Even if a boring indicates a rather thick, confining lower layer, to be considered as impervious and incompressible, the area is too extensive to know whether it is connected or not to a relatively close, underlying compressible layer, markedly affecting settlement.

Let us consider now the case of a stratified soil of clay and sand layers, these latter confined by clay layers containing water under artesian pressure (Fig. 3). For the sake of convenience, the piezometric level of water table in the sand layers will be assumed to be equal, the soil to be saturated throughout, and the bulk density to be constant.

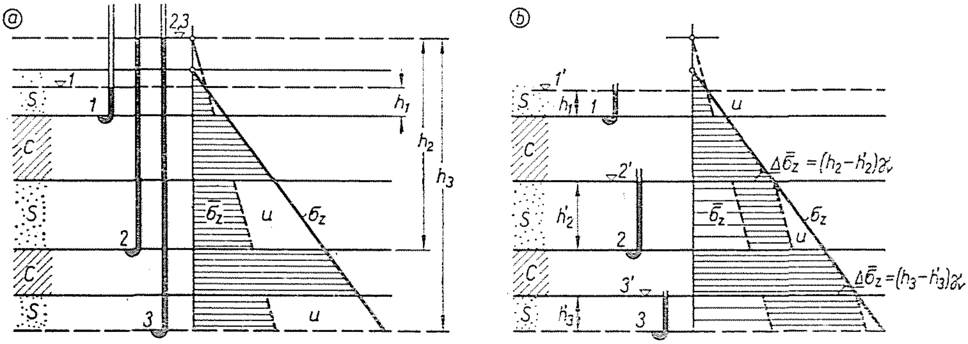


Fig. 3. Vertical stresses in a layered soil with intermediate sand layers containing piestic water. *a* — Stresses before dewatering; *b* — stresses after dewatering; $\Delta\bar{\sigma}_z$ = effective stress increment

According to this assumption, the total vertical stress increases linearly with depth. Presence of piestic water also shows the clay layers to be impermeable, crack-free enough to prevent pressures from equalizing. Part *a* in Fig. 3 is a diagram of total, effective and neutral stresses before dewatering. Dewatering is understood here as to lower the artesian pressure in a measure to make ground water conditions "regular". This is done in course of pre-dewatering. Variation of vertical stresses is seen in Fig. 3*b*. Total stress value in clay layers did not change, and there being no pore water excess during dewatering neither thereafter, effective stresses did change. In the assumed — rather ideal — case, not the clay layers but only the sand layers get compressed due to the local increment of effective stresses.

After pre-dewatering, during dewatering of the entire working site, gravitational water is removed from the sand layers, and now, a hydraulic gradient will be produced in the clay layers just as assumed in earlier calculations, starting a consolidation process in every layer. The course of consolidation will depend on the layer thickness, permeability and compressibility, as well as on the process of dewatering.

Surface subsidence in cohesive soils

Let us consider the amount and the consolidation of surface subsidence in cohesive soil layers.

We assume that the cohesive layer is horizontal and of uniform thickness, confined by two pervious layers, and the piezometer level is uniformly lowered by dewatering then we have to deal with a one-dimensional consolidation process (Fig. 4). Total vertical stresses vary linearly vs. depth; for a gravitational groundwater level which is located at the ground surface, also neutral

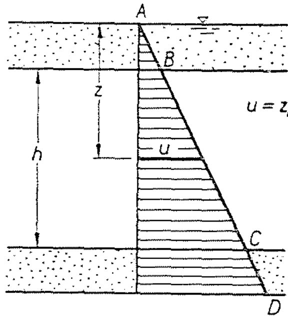


Fig. 4. Neutral stresses in the open surface layer of the infinite half-space if the water table is located in the surface level

stresses vary linearly according to $u = z\gamma_w$. Now, the only mechanical effect of the presence of water is buoyancy. Let hydrostatic pressure in the lower sand layer be reduced by Δp, e.g. by drawing off water at a constant discharge. Then water starts flowing in the clay until the hydrostatic pressure diagram reaches the line ABDE (Fig. 5). Maintaining a constant water level at depth Δh in the well, and provided the loss in the lower sand layer is uniform, the water flow will follow the hydraulic gradient $i = \Delta h/h$.

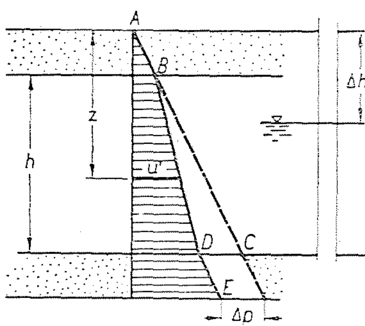


Fig. 5. Neutral stresses after groundwater lowering

The average effective stress in the clay layer is:

$$\Delta \bar{p}_{av} = \frac{\Delta p}{2} = \frac{\Delta h \gamma_w}{2} = i\gamma_w \frac{h}{2}.$$

The settlement due to this effective stress will be

$$\frac{\Delta y}{h} = \frac{\Delta \bar{p}_{av}}{M} = \frac{\gamma_w}{2\,(1+e)}\, a_v h = \frac{h\gamma_w}{2M},$$

where e is void ratio of the clay and a_v its compressibility. Assuming their value to be independent of the depth, the total settlement amounts to:

$$y = \frac{h a_v \gamma_w}{2(1+e)} \Delta h = \frac{h\gamma_w}{2M} \Delta h .$$

Let us consider now the consolidation process. The degree of consolidation at time t is given by:

$$\varkappa\% = 200 \frac{p_t}{p} = 100 \frac{\Delta y_t}{\Delta y}$$

where p_t is the mean value of effective stress at time t, and Δy_t is subsidence up to a time t. According to the consolidation theory by TERZAGHI,

$$t_{50} = \frac{0{,}2\, h^2}{c_v}$$

where t_{50} is the time needed for 50% of settlement, and c_v is the consolidation coefficient [$c_v = k(1+e)/a_v\gamma_w$.] For 50% of consolidation (see Fig. 6) approximately

$$\Delta y_t = A\sqrt{t} = \frac{\sqrt{5}}{2} \frac{k\Delta h}{c_v} \sqrt{t}$$

and

$$d\Delta y_t = \frac{\sqrt{5}}{2} \frac{k}{\sqrt{c_v}} \sqrt{t}\, dh .$$

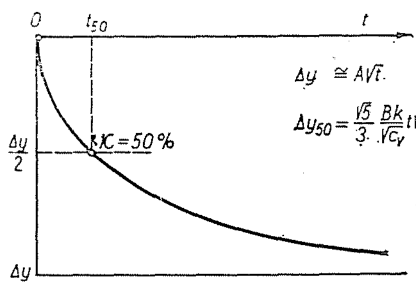


Fig. 6. Consolidation curve of the surface subsidence

We assume that the depression Δh occurs uniformly i.e. according to $\Delta h = Bt$; integration gives:

$$\boxed{\Delta y_t = \frac{\sqrt{5}}{2} \frac{Bk}{\sqrt{c_v}} t\sqrt{t}}$$

It is interesting to note that the settlement for 50% consolidation is independent of the layer thickness h.

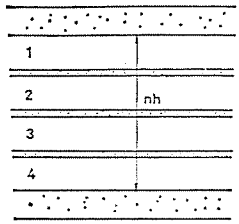


Fig. 7. Stresses in alternating cohesive and granular layers

Let us consider now the case of n pervious layers, each confined both sides by clay layers with identical properties (Fig. 7). At $t = 0$, the hydrostatic pressure drops in each pervious layer by the same value. Now, the total subsidence value after time t is:

$$\Delta y_t = \frac{2\sqrt{5}}{3\sqrt{c_v}} kt\sqrt{t}\,(B_1 + B_2 + \ldots + B_n)$$

wherein $B_1, B_2, \ldots, B_n$ are water level lowering rates in each pervious layer. For $B_1 = B_2 = \ldots = B_n$

$$\Delta y_t = \frac{2\sqrt{5}}{3\sqrt{c_v}} nk\,Bt\sqrt{t}\,.$$

In an open compressible layer, the consolidation is one-dimensional. If the soil compressibility modulus increases linearly with depth, the calculation furnishes the following result (Fig. 8):

The coefficient of compressibility varies according to:

$$M(z) = a(z - h_1) + b = az' + b\,.$$

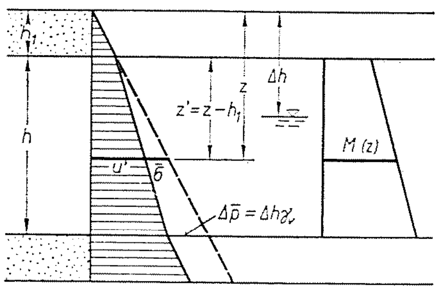


Fig. 8. Calculation in case of a compression modulus linearly increasing with depth

The total settlement amounts to:

$$\Delta y = \int_{z'=0}^{h} \frac{\bar{\sigma}_z}{M(z)} dz$$

the increment of effective stress being:

$$\bar{\sigma}_z = \frac{z'}{h} \Delta \bar{p} = \frac{z'}{h} \Delta h \gamma_w = i \gamma_w z' ,$$

integration gives:

$$\Delta y = i \gamma_w \left[\frac{h}{a} - \frac{b}{a^2} \ln \left(\frac{a}{b} h + 1 \right) \right] .$$

Effect of threshold gradient on subsidences

To now, compression due to groundwater table lowering had been calculated by assuming the water flow due to effective stress increment to be described by the original DARCY relationship $v = ki$. Water flow in clay is known, however, to be better approximated by the law:

$$v = k(i - i_0)$$

stating a so-called initial or threshold gradient i_0 to exist below that no water flow starts. The numerical value of the threshold gradient is usually small, however, the hydraulic gradients produced during pre-dewatering are also low, therefore i_0 may be of importance for the rates of both water flow and surface settlement. Namely then, a cohesive layer subject to a constant compressive stress will only undergo a partial compression.

Stress distribution in an open layer, assuming initial condition $p = $ const. is seen in Fig. 9. In plotting pressure values as lengths p/γ_w, the initial gradient is characterized by an angle

$$\tan \beta = i_0 .$$

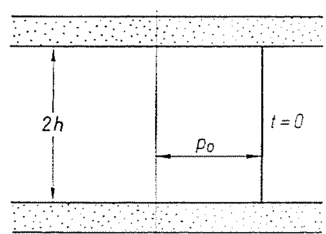


Fig. 9.

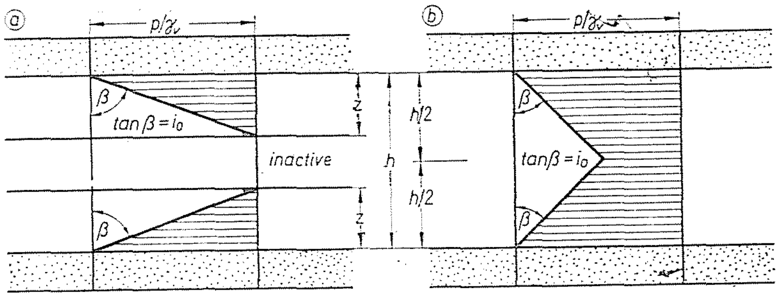


Fig. 10. Distribution of neutral stresses in an open system for non-zero initial gradient. a — $i_0 < 0$, $z < h/2$; b — $i_0 > 0$, $z > h/2$

Water seepage and thus, soil compression will start only in the range $i > i_0$. There are two possibilities (Fig. 10). Either, as seen in Fig. 10*a*,

$$z = \frac{p}{\gamma_w} \cot \beta = \frac{p}{\gamma_w} \frac{1}{\tan \beta} = \frac{p}{\gamma_w i_0};$$

depth z is less than the layer half-thickness. The total compression — settlement — value is given by the area of shaded triangles divided by the compression modulus M, i.e.

$$y = \frac{zp}{M}.$$

Substituting z:

$$y = \frac{p^2}{i_0 \gamma_w M}.$$

Note that the M value has to be determined for an interval $(p_0, p_0 + p)$ irrespective of the existence or not of a threshold gradient.

Similarly, in the second case (Fig. 10*b*), for a threshold gradient less than the former, again with respect to the soil layer compression obtained by dividing the area of the effective stress diagram by the compression modulus, the compression becomes:

$$y_0 = \frac{h}{M} \left(p - \frac{1}{4} i_0 h \gamma_w \right).$$

In the case of an open layer, the zero-isochron will follow Fig. 11; assuming dewatering to be instantaneous, stress increment due to water table lowering increases linearly with depth. Existence of a threshold gradient $i_0 > 0$ involves decrease of stresses causing compression; here also, the active part of the stress distribution diagram can be determined according to Fig. 10, its area gives the settlement value. Fig. 11*a* refers to $\tan \beta = i_0 < \tan \alpha = u_0/h\gamma_w = (z_1 - z_0)/h_1$, Fig. 11*b* to $\tan \beta > \tan \alpha$.

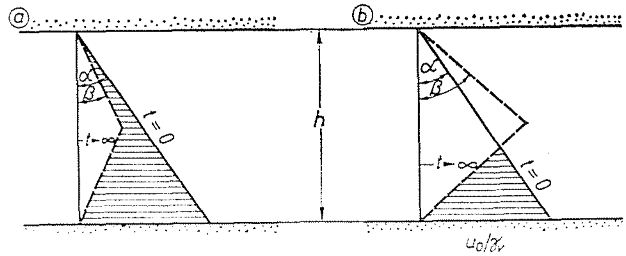


Fig. 11. Effect of initial gradient if the initial condition involves the linear increase vs. depth of the neutral stress at time $t = 0$. a — Presence of inactive layer; b — absence of inactive layer

In these cases, existence of a threshold gradient will reduce settlement. It is largely due to the threshold gradient that the moduli of compressibility determined in oedometer tests do not — cannot — truly reflect the surface subsidence value due to water table lowering. The usual method ignores the existence of inactive or partially active zones inside the layer, due to the threshold gradient.

It has to be noted that in case of a load linearly increasing in time, the less the compression, the lower the rate of loading.

Let us consider now the case of two water-bearing, pervious strata, both being subject to dewatering. Knowledge of the original (layer) piezometric levels (the upper groundwater is phreatic, the lower is piestic) and of the depression rate permits to determine the effective stress change, hence the expected surface subsidence. Assuming a soil stratification according to Fig. 12,

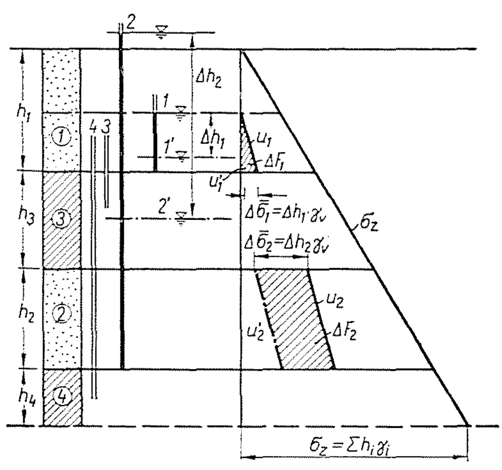


Fig. 12. Symbols in settlement calculation *1* and *2*: water bearing layers; *3* and *4*: impervious layers. 1—1′: Watertable variation in layer *1*; 2—2′: variation of the piezometric level of the water under pressure in layer *2*; u_1 and u_2 are neutral stresses in layers *1* and *2* before, u_1' and u_2' after water level lowering. F_1 and F_2 are areas representing the effective stress increments

and determining the depression due to dewatering, the subsequent surface settlement is composed of two parts, corresponding to the neutral stress decrease in both water-bearing strata. With notations in the figure, the total settlement is:

$$\Delta y = \Delta y_1 + \Delta y_2 = \frac{1}{M_1}\int_0^{h_1} \Delta\bar{\sigma}_1 dz + \frac{1}{M_2}\int_0^{h_2} \Delta\bar{\sigma}_2 dz = \frac{\Delta F_1}{M_1} + \frac{\Delta F_2}{M_2}$$

where ΔF is the area of stress diagram showing the effective stress increment.

In case of several water bearing layers:

$$\Delta y = \frac{\Sigma \Delta F_i}{M}$$

with

$$\frac{1}{M} = n \sum_{i=1}^{n} \frac{1}{M_i}.$$

These were some rather simple methods of calculating the surface subsidence due to dewatering. As a conclusion, two remarks have to be made. First, all methods assumed the stressed state in one-dimensional compression and one-dimensional consolidation as well. This is why differences in subsidence surface cannot be determined by these methods. Subsoil stratification and point-wise variation of physical characteristics cannot be determined with the reliability needed for exact calculation. Second, soil physical characteristics involved in the relationships can be determined with scattering and errors only, hence reliable data can only be obtained from values calculated from field measurements of surface subsidence.

Some numerical data

Let us present some values observed or measured in the *Visonta* open pit mining area, in particular those from the years 1955—1974—1975. Relation of surface settlements to their distance from open pit mining is seen in Fig. 13, and timely course of the motion of some typical points in Fig. 14. The long interval between measurements unfortunately discloses detailed analysis, no curves are available for the 1955 to 1972 period. Anyhow, settlement values can be stated to be much below the admissible values. (Hungarian mining practice classifies constructions according to their importance and sensitivity to surface motions into four categories. Admissible deflections in *mm/m* and the admissible curvature radiuses in km in each category have been compiled in Table 1.)

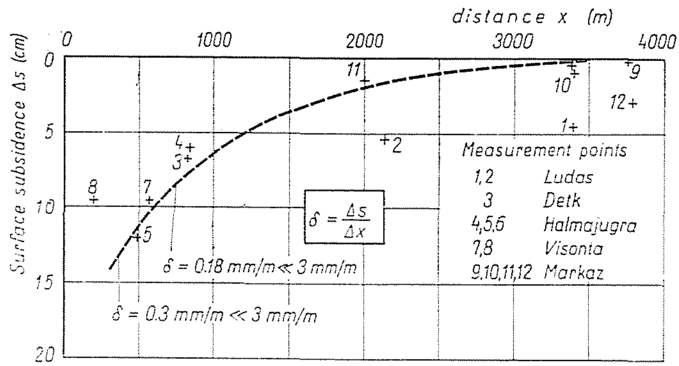


Fig. 13. Field-observed surface subsidences vs. distance from the open pit mining

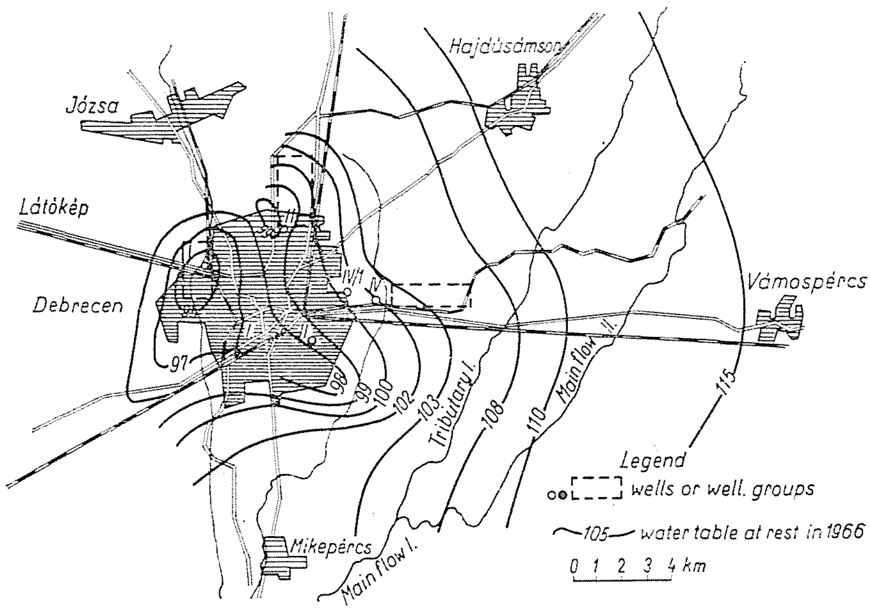


Fig. 14. Subsidence of characteristic points vs. time

Table 1

Permissible values of surface deformations

Safety class		Permissible values	
No.	Denomination	Deflection mm/m	Curvature radius km
I	Very sensitive structures	3	20
II	Medium sensitive structures	7	12
III	Slightly sensitive structures	10	6
IV	Non-sensitive structures	20	2

Further data for the analysis of the effect of dewatering were given by a survey made in the last decade in the area of *Debrecen* town.

The amount of available surface water in Debrecen is small, therefore the water demand is met from underground water.

The town and her environment are of a rather uniform geological structure. Beneath the surface, there exists a thick (100 to 150 m) layer of Pleistocene sand and silt, then there is 20 to 80 m coarse lower Pleistocene sand overlaying the upper Pannonian clay substratum.

Also Pleistocene strata include several good water-bearing layers.

Until 1913, urban water demand had been met from wells established in underground layers. Wells of water works I, constructed in 1913, have been established in the lower Pleistocene coarse sand at 110 to 170 m depth.

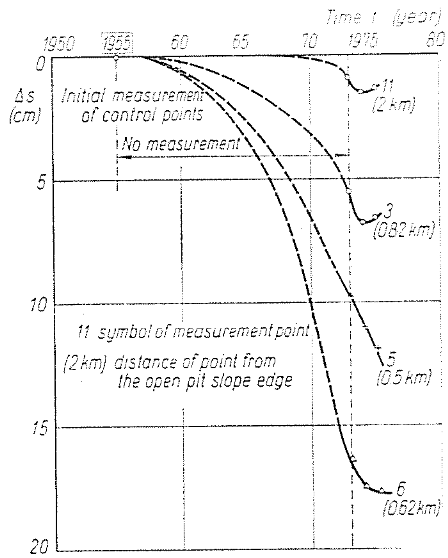


Fig. 15. Sea-level altitude of piezometer level in 1966

The 1966 depression level of the waterworks layer is seen in Fig. 15, isochrons of surface settlements due to water table lowering by the same time are seen in Fig. 16.

Figs 17 and 18 show relationships between the variation of depression level ΔH_v, the maximum subsidence Δs_{max}, the water discharge Q and time t, referring to the region of waterworks I.

Making use of Fig. 17 — where function $\Delta s = f(\Delta H_v)$ may in fact be considered as a compression curve — and assuming the pervious granular layer having an average thickness of 30 to 50 m, the compression modulus amounts to 50—230 MN/m^2.

Surface subsidence due to important and increasing water draw-off was of the order of 10 cm by 1976.

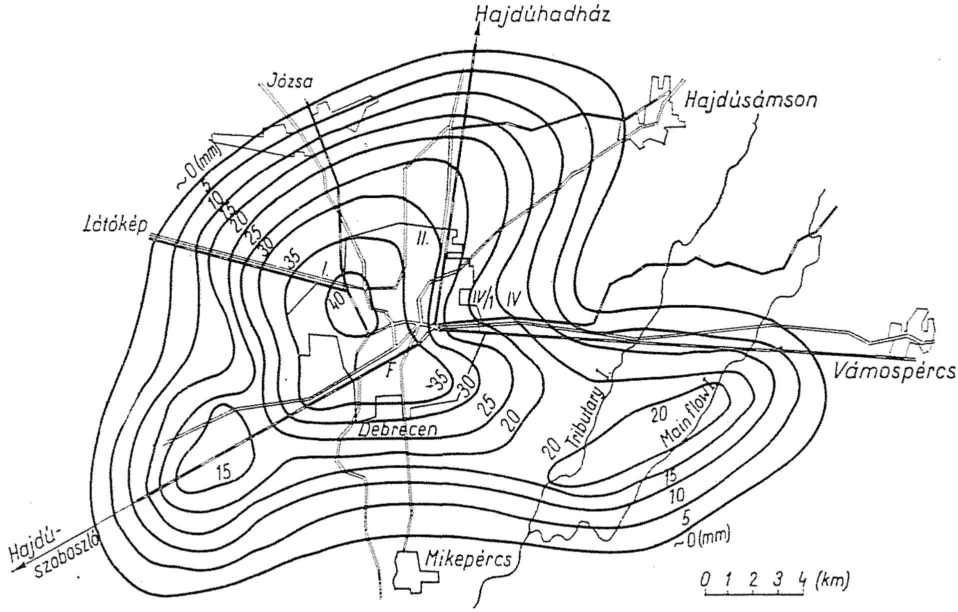


Fig. 16. Relative subsidence of the surface in the Debrecen area due to water take-off (from 1966 measurements)

The greatest surface settlements all over the world have been recorded in *Mexico City*, attributed also here to the water take-off and to the extreme compressibility of the subsoil. Major soil physical characteristics have been compiled in Table 2.

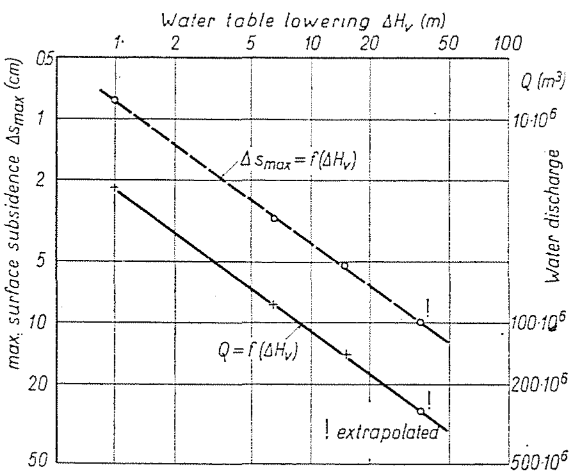


Fig. 17. Relationship between water table lowering, water discharge and surface subsidence

Table 2

Soil physical characteristics of Mexico City lacustrine clay deposits

Soil physical characteristics		Layer number 1	2	3	4
Natural water content	w %	290	78	200	220
Liquid limit	w_L %	300	85	254	590
Index of plasticity	I_p %	228	51	188	500
Specific density	p/cm³	2.55	2.60	2.61	2.31
Phase composition	solids %	12	32	16	17
	water %	88	66	84	83
	air %	0	2	0	0

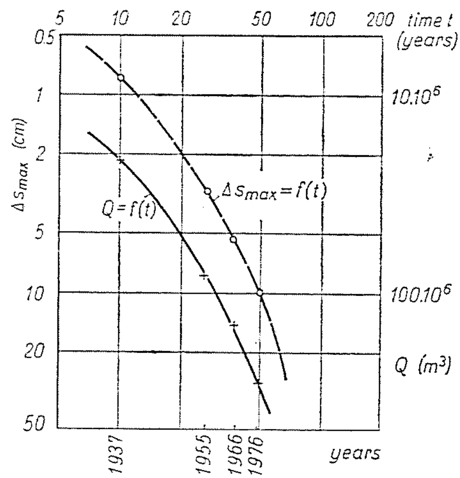


Fig. 18. Surface subsidence with increasing water take-off vs. time

The urban water supply involved wells sunk in the urban area, ending in granular, pervious layers within the clay. Pumping caused some areas of *Mexico City* to settle by more than 8 m in this century. Construction weights and street embankments are likely of having contributed to the settlement though estimated to be responsible for less than 20%. The main cause of settlement was the pore water depression in the pervious subsoil strata, starting a consolidation process in the lacustrine clay deposits. Proliferation of wells, piezometric pressure in sand layers, and surface subsidence values were found to be strictly correlated. The causal correlation was settled by NABOR CARRILLO in 1948, inducing municipalities to reduce urban water take-off, slowing down, in turn, the surface settlement.

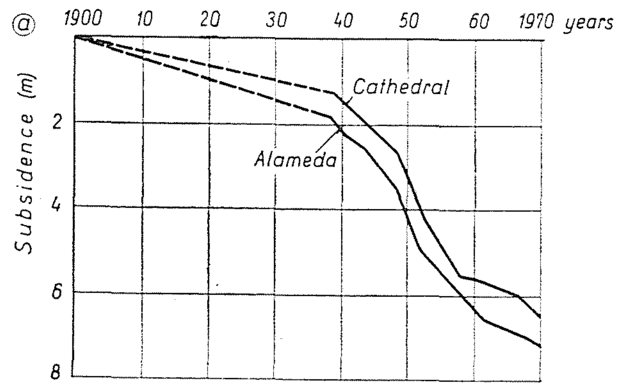
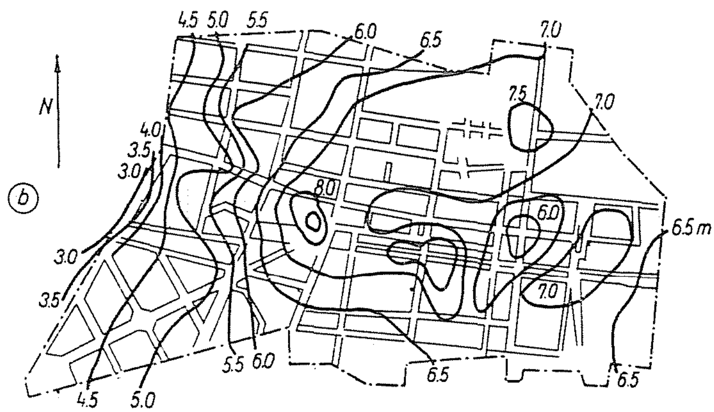


Fig. 19a. Surface subsidence variation vs. time at two characteristic points of the old town



b. Contour lines between points with identical settlement (in m)

Timely process of the subsidence of the Cathedral and of Alameda Park is seen in Fig. 19. 1900 to 1938 it amounted to 3 to 5 cm/year, to grow to 15 cm/year in the next five years, with a maximum of 50 cm/year in 1950, thereafter the moderated water take-off reduced it to 10 cm/year. Fig. 19*b* shows contour lines of identical subsidence in the town center for the 1811 to 1970 period. Actually, the greatest subsidence is over 9 m (around the equestrian statue of Charles IV), to be 5 to 7 m along the Paseo de la Reforma.

Summary

Methods are presented which may serve for the calculation of surface subsidences caused by dewatering of soil layers. Dewatering of a single layer and of layered systems are equally considered; methods to determine the consolidation are also given. The existence of a threshold gradient may greatly influence the subsidence. Some phenomena which influence the amount of subsidence are listed; these are responsible for the fact that numerical calculations are rough guesses only. The second part of the paper shows numerical data on subsidence; measurements in the area of the Visonta open-pit mining, in the area of Debrecen indicated characteristic values; finally, the tremendous subsidence values observed in *Mexico City* are discussed.

Prof. Dr. Árpád Kézdi, H-1521, Budapest