7th International IAEG Congress / 7ème Congrès International de AIGI, © 1994 Balkema, Rotterdam, ISBN 90 5410 503 8

Induced land subsidence near major river mouths: From Quaternary geology to coupled numerical models

Subsidence induite près de l'embouchure des grands fleuves: De la géologie du Quaternaire aux modèles numériques couplés

A. Dassargues
Laboratoires de Géologie de l'Ingénieur, d'Hydrogéologie, et de Prospection Géophysique (IGIH), University of Liège, Belgium

C. Baeteman
Belgian Geological Survey, Brussels, Belgium

ABSTRACT: Many of the cities located in coastal or delta plains experienced severe land subsidence induced by the groundwater withdrawal. The low consolidated recent sediments compact in response to additional stresses from load of overlying material and to the decrease of water pore pressures by intensive pumping. Usually, the engineering studies on land subsidence are limited to an overview of the case added to a computational analysis based on empirical, analytical or numerical models assuming many restricting hypothesis like homogeneity, and isotropy. Most often no calibration on historical data is presented, and constant parameters of flow and consolidation are assumed. A rigorous and complete methodology is developed to handle with more accuracy this kind of subsidence problems. In a first step, the geological, geotechnical and hydrogeological research should complement each other continuously in order to determine the three-dimensional spatial distribution of the distinguished deposits, each bearing their particular geotechnical and hydrogeological properties. Only such an approach yields the badly needed accurate quantitative information about the variables involved in land subsidence. In the second step of the study, the computer codes designed for modelling the coupled problem of groundwater flow and land subsidence , have to include the fact that the flow and the geotechnical parameters are variable and highly interdependent in time during the consolidation process leading to non linearities and coupling in the numerical procedure. It is only in these conditions, added to an adequate calibration, that a numerical model simulating the land subsidence process could be taken in consideration to proceed with prediction computations. Application of this methodology and a conceptual model is given with the case study of the land subsidence in the central zone of Shanghai.

RESUME: De nombreuses cités situées dans les plaines côtières et deltaïques ont subi des subsidences importantes induites par le pompage d'eau souterraine. Les sédiments récents et peu consolidés se compactent en réponse à l'incrément de contrainte dû au poids des matériaux surincombants et à la baisse des pressions d'eau engendrée par les pompages. Habituellement, les études des ces subsidences sont limitées à des exposés généraux basés sur des modèles analytiques, empiriques ou numériques, prenant des hypothèses très simplificatrices comme l'homogénéité et l'isotropie des terrains. Le plus souvent, aucune calibration n'est montrée et les paramètres d'écoulement et de tassement sont choisis constants. Une méthodologie rigoureuse et complète est proposée pour traiter avec plus de précision ce type de problèmes de subsidence. Dans un premier temps, les études géologique, géotechnique et hydrogéologique doivent se compléter mutuellement pour la détermination en trois dimensions des différentes entités géologiques, chacune de celles-ci ayant ses propriétés hydrogéologiques et géotechniques spécifiques. Seule une approche de ce genre peut fournir toutes les informations quantitatives demandées au sujet des variables impliquées dans le processus de subsidence. Dans un deuxième temps, le modèle numérique construit pour simuler le problème couplé des écoulements et tassements en milieu poreux , doit inclure la variation et l'interdépendance des paramètres hydrogéologiques et géotechniques au cours du processus de consolidation qui est simulé. Ce n'est que dans ces conditions, jointes à une calibration adéquate, qu'un modèle numérique simulant les subsidences peut être utilisé pour des calculs prédictifs. Une application de cette méthodologie est brièvement évoquée avec le cas d'étude de la subsidence de la ville de Shanghai.

1 QUATERNARY GEOLOGY

1.1 *Sedimentology near major river mouths*

River mouths belong to the group of highly variable depositional environments, controlled by continental and marine conditions (Einsele, 1992). The continental and marine sequences in the subsurface are gradational through estuarine and deltaic facies. One of the major forcing processes in the building up of the coastal accumulation is the change in sea level. The effect of sea-level fluctuations on river mouth deposits is overwhelming. This has been observed in particularly in the sediment record of the present geologic period, the Quaternary.

Rivers incise the lower reaches of their courses and discharge large amount of sediments increasingly further out onto the shelf. Deltas emerge and fluvial channels are cut, dissecting and eroding parts of the delta plain. At the lowest stand of sea level, estuaries seem to almost disappear and are confined to valleys (Russell,1967, Nichols & Biggs,1985).

Although the fact that every river responds differently to sea-level changes and experiences its very own evolution according to many various factors, the location of the main river channels remained relatively fixed. At least during the Pleistocene and Holocene, it seems that the rivers reoccupy the same places during various transgressions (Nichols & Biggs,1985).

The availability of sediment is another overwhelming factor. If sea level is rising sufficiently rapidly with respect to the rate of river sediment input, river mouths become estuaries. Estuaries trap not only fluvial sediments, but also sediment from the littoral system.

Waves, tides and coastal currents can also produce important modifications at river mouths.

1.2 *Natural subsidence*

In normally consolidated sediments, the fabric strength or compaction state is in equilibrium with the overburden pressure at all depths below the sediment-water interface. However, if a new increment of sediment is added on top of such a sedimentary column, this equilibrium is disturbed. In order to establish a new equilibrium between the increased vertical stress and the compaction state, some pore water must be expelled. In a fairly permeable sediment, this pore water expulsion takes place more or less simultaneously with the growth of the sedimentary column, so the sediment maintains its state of normal consolidation. Normal consolidation can be maintained in fine-grained sediments if the sedimentation rate is low and the pore water has sufficiently time to escape. If, by contrast, the sedimentation rate is high and the growing sedimentary column has a low permeability and becomes less and less permeable with depth, the pore water to be released by prograding compaction cannot escape readily to the sediment-water interface. Hence, compaction is delayed, and the sediment is in a state of underconsolidation (Einsele, 1992). Compaction affects all sediments, but are most pronounced in fine-grained sediments, such as silts and clays.

The factors that influence compaction in sands are mostly shapes and sorting of particles and depth of burial. During compaction, sand particles respond by shifting into more-dense packing arrangements, hence porosity decreases. Angular and poorly sorted sands are more compressible than rounded and well-sorted sands. Peat is the most compressible of all natural sediments because of its very high porosity and its weak skeletal framework of vegetable fibber. Not only will it compress beneath an applied load, but under certain conditions it will also compress under its own weight, a process which is called autocompaction.

Peat shows a unique behaviour with respect to consolidation. One of the reason is the more rapid reduction in permeability with change of volume, than e.g. in clay. Another reason is the loss of volume with decomposition and the complex physiochemical changes that go on all the time and that have a continuing effect on the structure and strength of the peat fabric (Kaye & Barghoorn 1964, Hawkins, 1984, Hobbs, 1986)

The sedimentary environment at the river mouth is not solely influenced by fluvial processes; coastal processes are the major factor in redistributing sediments (from the river, and eroded from subaqueous part of the delta) and hence creating the various coastal environments. The various sedimentary depositional environments occurring at the river mouths and creating their adjacent plains, and their deposits which might be found in the subsurface have been listed and described previously by Baeteman & Dassargues (1992).

1.3 *Additional sediment characteristics*

Concerning the geotechnical properties conditioning compaction, a fundamental distinction related to the genesis of the deposit is to be made. The sediments that originate as subaerial features are much more consolidated than those formed as

submerged features or subaqueous. In the fluvial system, distinction is to be made between on the one hand the flood plain and levee deposits which originate subaerially, and on the other hand the channel and patterns of deltaic surfaces which initiated as submerged features (Russell,1967) and which as a result are very compressible. Because of their deposition above the groundwater table, floodplain deposits are overconsolidated and, in the sedimentary sequence, they form stiff compact layers.

1.4 *The stratigraphic sequence*

In view of the many factors interacting and contributing to the final depositional record, it is self-evident that the resultant vertically stacked succession of the deposits is characterised by frequent lateral and vertical facies changes. Investigations aiming at the research of the characteristics of the deposits conditioning compaction, must emphasis on the three-dimensional facies geometry.

Every particular facies bears its particular geotechnical and hydrological properties, although conditions may change later on. Therefore, the spatial distribution of every facies, lateral as well as vertical, must be delimited in detail and this for the entire area under consideration. However, establishing the geometry of the various facies in the depositional body of a coastal plain requires an enormous amount of data. And it is self-evident, the more data available, the more accurate the delineation of the geometry will be.

In order to obtain a three-dimensional picture (as complete as possible) of the complex mosaic being the subsurface of the depositional body, every core is to be interpreted and this interpretation is to be put into a larger context by means of cross-sections (Cant,1986). The various distinguished and significant units from the boreholes (loggings) are to be correlated; filling the blanks between the data points remains the ever critical decision !

Correlating boreholes is not just a simple line-drawing act. It involves the understanding of the interplay of all relevant factors and processes that built the depositional body. Detailed cross-sections also implies that all available data must be used.

Literature abounds with case studies on land subsidence. Very few exhibit the detailed geologic setting of the problematic area. The main emphasis is always put on the geotechnical and hydrological conditions; the geological setting is simply recalled as kind of introductory data.

However, the deposits, with particularly their changing spatial distribution, lateral and vertical, initially account for the geotechnical and hydrogeological properties. That is why generalised cross-sections, where broad classical stratigraphical units are correlated assuming that the units remain regular and uniform between the data points, can only lead to misinterpretation and false conclusions about the geological setting in the first place, and about all other relevant characteristics conditioning compaction in the second place.

Depositional environmental interpretation and vertical sequence analysis are the most important tools in determining or delineating the geometry of coastal sedimentary facies. And only by determining the three-dimensional geometry of the individual facies, time significance of contacts between the individual facies can be interpreted (Demarest, et al., 1981). Doing so, the general record of events can be identified, and eventually the development of the coastal plain in space and time can be delineated.

2 INDUCED LAND SUBSIDENCE

Young unconsolidated or semi-consolidated elastic sediments of high porosity, laid down in alluvial or shallow marine environments form, most often, a succession of layers which can be characterised (from an hydrogeological point of view) as semiconfined or confined aquifer systems (Poland, 1984) consisting in aquifers of silty sand and sand of high permeability (hydraulic conductivity) and low compressibility, interbedded with clayey aquitards of low vertical permeability and high compressibility.

The geostatic pressure or total stress (σ) that any point undergoes in the soil is usually considered as the result of two components: the fluid pore pressure (p) and the effective stress (σ'), according to the Terzaghi (1943) principle. This principle is sufficiently accurate for the computations of the total settlements; as a matter of fact, the soil compressibility is a factor 20 to 1000 larger than the grain compressibility, so that it would be particularly uneasy and useless to choose another principle based for example on Biot (1956) theory.

One can express very simply the new stresses created by the lowering of the piezometric head in a confined aquifer (figure 1). For an initial pore pressure considered in a complete equilibrium state, the pressure is decreased in the aquifer and partially decreased in the underlying and/or overlying aquitards. During this stage, the total stress can be easily assumed constant (the layers are maintained saturated due to the recharge from the top or due to the very slow propagation of the pressure decrease through the aquitards). Consequently, the slow propagation of the pore pressure variation in the semi-permeable layers induces automatically an equivalent increase in effective stress in these compressible layers... and a drained consolidation process is started. The second stage is distinguished in the long term (figure 2), when the pore pressure decrease has reached the top of the confining layer and then, as for unconfined situations, provokes a decrease of the thickness of the saturated soils. The total stress can no more be considered as a constant except if there is an important infiltration or recharge. The transient behaviour of the phenomena is very important because the main consolidation (called usually primary consolidation) is activated by the decrease of pore pressure as long as the hydrostatic equilibrium is not restored.

Physically, the structural evolution of clays during the consolidation process is dominated by the reduction of the pore dimensions, so that the whole porosity is decreasing. Many authors have observed that the clay minerals tend to orientate their plates orthogonally to the direction of the main applied stress developing a kind of structural anisotropy (Delage & Lefebvre, 1984 and Rieke & Chilingarian, 1974).

3 RHEOLOGY OF THE SOILS

The geomechanical behaviours of the soils can be idealised in term of rheological models. The skeleton deformation under increasing effective stress is supposed to follow elastic, plastic or viscoelastic laws or any combination of them.

A non linearity of the stress strain relation can be encountered in elasticity. An elasto-plastic material can be represented by a Hookean spring and a Saint-Venant resistance in series. A viscoelastic model to describe the creep of the clayey soils consists in a spring and a dashpot taken in parallel (Kelvin model). Many other models can be imagined by combination in parallel or in series of these global or individual units, trying to reproduce the real behaviour of the low consolidated soils and recent sediments.

Clayey soils and loose sediments have a geomechanical behaviour qualified more often as non linear elasticity with progressive plasticity and viscosity. This particular behaviour leads, in practice, to choose rather models based on experimental laws than on combinations of theoretical models. Elasto-visco-plastic laws in 1D or 3D can be established from experimental results. Different loading steps, more or less elaborated, can be applied to the samples in order to parameterize an experimental law. Often these constitutive laws allow a more easy introduction of non linear and interaction effects of the parameters.

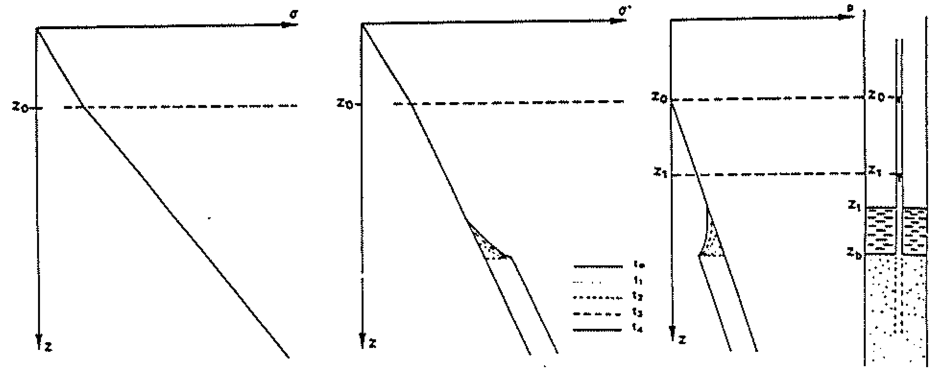

Fig. 1 Evolution of the total stress, pore pressure and effective stress in function of the depth when lowering the piezometric head in a confined aquifer. First stage: the pore pressure decrease is not yet propagated above the confining layer)

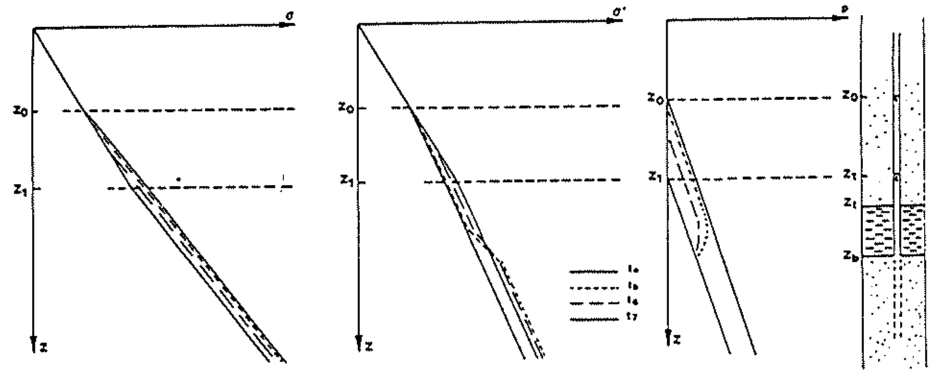

Fig. 2 Second stage: the pore pressure decrease is partially propagated.

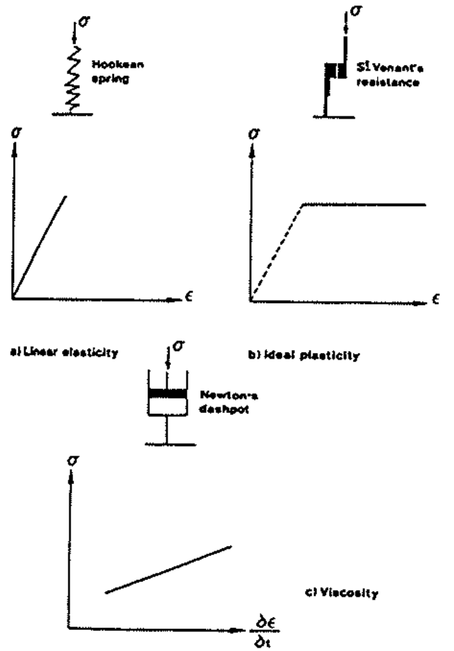

Fig. 3 Simple uniaxial rheological models

4 HYDROGEOLOGICAL AND GEOTECHNICAL PROPERTIES

The loose sediments described above can be considered as a porous medium consisting of solid skeleton (or matrix) and spaces. These spaces correspond to the pores of the sedimentary formation, or to small channels, between the grain of the solid phase. Fluid phases (i.e. air, water, oil, gas...) are occupying these spaces spread over the medium.

The flow of a fluid through these porous media is a physical process largely depending of the microscopic interface at the fluid-solid contacts.

To study the fluid flow, hydrogeologists and engineers are faced with the problem of the scale at which the processes have to be studied. At the microscopic scale, we come up rapidly against a main difficulty (Bear & Verruijt, 1987): how to represent with reliability the tortuous and non-repetitive geometry of the flow domain imposed boundaries, and how to verify the parameterization by eventual measurements ? The scale levels to use are, therefore, the macroscopic or megascopic levels with the essential and implicit assumption of the continuum approach for the different phases. Consequently, the parameters will be macroscopic and will represent some equivalent mean of the corresponding microscopic properties.

A space integration is realised to obtain macroscopic values for the porosity and for the permeability (hydraulic conductivity) of a porous medium. The volume on which the integration is made, is qualified as the Representative Elementary Volume (REV) of the porous medium. This elementary volume allows to use the porosity and permeability values defined in the REV zone (whatever the size of this zone) in the mathematical equations describing the physical process of flow.

Conventionally, the size of the REV is selected sufficiently large in regard to the pores and channels dimensions in order to get significant mean values, and sufficiently small to consider continuous parameters variations using usual macroscopic measurements.

The fluid storage and the fluid flow are the two main processes in which the aquifer porous medium is involved. They are realised by the interconnections and the dimensions of the voids. The porosity qualifies the reservoir property to release a fluid quantity; the permeability qualifies the reservoir aptitude to convey the fluid flow (Castany, 1967). The cinematic porosity called also "specific yield" is often very weak in comparison to the total porosity and is not measurable (Burger et al., 1985). That is why the effective porosity (n_e) is considered, corresponding to the fluid quantity drained by gravity from an initially saturated soil after ...a sufficiently long time. Some authors call it the drainage porosity.

4.1 *Permeability*

The hydraulic conductivity or permeability coefficient (K) may be expresses as:

$$K = \frac{k.\rho.g}{\mu} \qquad (1)$$

where k is the intrinsic permeability of the porous medium, ρ the density of water and μ the viscosity of the water.

The Darcy's law, generalised, in 3 dimensions can be written:

$$\underline{v} = -\frac{k}{\mu}(\underline{grad}p + \rho.g\ \underline{gradz}) = -\underline{K}\underline{gradh} \qquad (2)$$

where $\underline{v}$ is the Darcy velocity vector and $\underline{K}$ is the permeability coefficient tensor.

For silty and clayey semi-pervious formations, the permeability coefficient is more often measured during consolidation tests (oedometer and triaxial tests). The permeability is obtained at different stages of effective stress, leading to a relation:

$$K = f(e) \qquad (3)$$

4.2 *Specific storage coefficient and compressibility*

To describe completely the groundwater flow in a saturated porous medium, we express the continuity equation:

$$div(\rho.\underline{v}) + \rho.q = -\frac{\partial}{\partial t}(\rho.n) \qquad (4)$$

where q is a volumic flow rate (per volume unit) exchanged by the R.E.V. with the outside environment (positive if the flow is entering into the system). The right member of equation (4) is the time variation of the water stored per unit volume of soil. It characterises the aquifer capacity to store or release a volume of water in function of the pore pressure prevailing in the formation.

In transient conditions, we have henceforth to observe that :

- if the pore pressure is equal to zero, corresponding to the interface between saturated and unsaturated medium (the water table surface or free surface), the aquifer capacity of storage depends mainly on the effective porosity (defined above).
- if the pore pressure is positive, the aquifer storage depends mainly on the compressibility's of the porous medium and of the fluid (water in our case).

Seven important assumptions are needed to define the specific storage coefficient of a saturated porous medium :

. the R.E.V. concept is applied;
. isothermal conditions, so that $\rho = \rho(p,c)$ and does not depend of the temperature;
. the fluid is homogeneous, so that $\rho = \rho(p)$ and does not depend of the concentration;
. the Darcy velocity is a relative filtration velocity;
. the solid density (ρ_s) is constant, so that the compressibility of the solid grains (β) is negligible and (ρ_s) is constant in function of time and in the R.E.V.;
. the Terzaghi effective stress postulate is accepted;
. the total stress is constant.

In these conditions,

$$\frac{\partial}{\partial t}(\rho.n) = \rho.S_s.\frac{\partial h}{\partial t} \qquad (5)$$

with $S_s = \rho.g.(\alpha + n\beta) \qquad (6)$

where S_s is the specific storage coefficient of a saturated porous medium, α is the volume compressibility coefficient of the porous medium, and β is the water compressibility coefficient. The equation (6) shows the direct coupling between the transient flow and the consolidation processes, as the specific storage coefficient (S_s) is expressed in function of the compressibility coefficients of the porous medium and water.

Although a same definition, the storage coefficients of confined and unconfined aquifers correspond mainly to largely different physical processes: drainage in water table aquifer and expulsion of water in confined aquifer for any decrease in piezometric head.

For confined aquifers, the specific storage coefficient is determined, using the relation (6), on the results of drained consolidation tests. In practice, most often, oedometer tests are realised with the following assumptions : (1) the total stress is constant (drained test), (2) lateral deformations are prevented and neglected, (3) uniaxial state of stress and strain. Moreover, fluid and solid grains compressibilities are neglected in regard to α so that :

$$S_s = \rho.g.\alpha \qquad (7)$$

Generally, values obtained by pumping tests are upper than those obtained on samples by consolidation tests (Domenico & Mifflin, 1965). In the loose sedimentary layers, it is really difficult to make such comparisons because undisturbed sampling is uneasy in the silty to sandy aquifer layers and the piezometric levels are hardly measured with accuracy in clayey to silty aquitards.

5 VARIATION OF THE PERMEABILITY AND OF THE COMPRESSIBILITY

Geotechnical scientists have long been aware that during consolidation of highly compressible clays, changes in porosity due to a rearrangement of the soil skeleton may lead to decreases in both the permeability and the compressibility of the porous medium. Lambe and Whitman (1969) have presented data indicating that permeability can change by orders of magnitude and compressibility can decrease significantly as void ratio decreases. Neither of these variation relationships is linear.

5.1 *Non linearity of the specific storage coefficient linked to the compressibility*

Assuming a constant total stress (σ = cst) and a negligible compressibility of the water (β = 0), the equation (7) can be used. By the definition of the volumetric coefficient (α):

$$\alpha = \frac{d\varepsilon_v}{d\sigma'} \qquad (8)$$

where ε_v is the relative volumetric strain and σ' is the effective stress.

We obtain :

$$\alpha = -dV/(V.d\sigma') \qquad (9)$$
$$\alpha = -dn/((1-n).d\sigma') \qquad (10)$$
$$\alpha = -de/((1+e).d\sigma') \qquad (11)$$

where n and e are the porosity and the void ratio at the beginning of the $d\sigma'$ variation.

The oedometer tests are the more common 1D consolidation tests, where axial stress is applied to the sample and lateral strain is prevented. The test is drained so that the variation in void ratio or in vertical strain is obtained in function of the effective stress variation. The results are plotted on (σ', ε_v) diagrams allowing to determine α for each effective stress (figure 4).

The compressibility coefficient for sandy to clayey materials depends on the effective stress (σ') and effective preconsolidation stress (σ'_{prec}). In order to linearize the oedometer curves, ($\ln\sigma'$,ε_v) or ($\ln\sigma'$,e) diagrams can be used, so that:

$$\begin{cases} \varepsilon_v = 1/A.\ln\sigma' + cst & \sigma' < \sigma'_{prec} \\ \varepsilon_v = 1/C.\ln\sigma' + cst & \sigma' \geq \sigma'_{prec} \end{cases} \qquad (12)$$

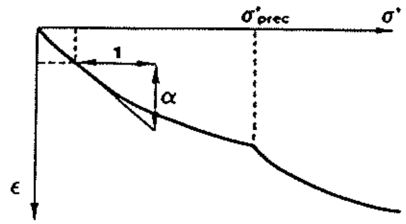

Fig. 4 Results of an oedometer test.

Then the compressibility coefficient can be expressed by:

$$\begin{cases} \alpha(\sigma') = 1/(A.\sigma') & \sigma' < \sigma'_{prec} \\ \alpha(\sigma') = 1/(C.\sigma') & \sigma' \geq \sigma'_{prec} \end{cases} \qquad (13)$$

So that the specific storage coefficient is expressed finally in function of $1/\sigma'$ values (figure 5).

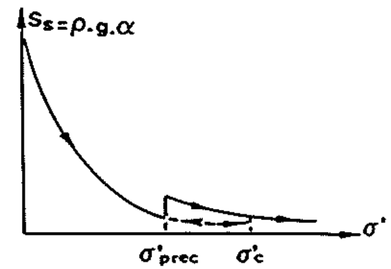

Fig. 5 Variation of the specific storage coefficient (S_s) in function of the effective stress (σ').

As the theory is not linked especially to 1D consolidation tests, other authors propose relations linking the void ratio (e) to the effective stress :

$$e = a/\sigma'^{b} \tag{14}$$

where a and b are constants experimentally determined on each material. The compressibility coefficient can be expressed, then, in function only of e :

$$\alpha(e) = c.e^{d}/(1+e) \tag{15}$$

where c and d are constants experimentally determined on each material.

5.2 *Non linearity of the permeability*

Many relations linking the permeability coefficient K to the total porosity or to the void ratio have been proposed by petroleum engineers and geologists in reservoir engineering, in order to interpret the porosity logs in terms of permeability of the reservoirs (Archer & Wall, 1987). These relations are not applicable in the consolidation and subsidence computations and, moreover, they are relative to hardened rocks and not to soil or loose sediments. In the case of estuarine recent clayey to sandy sediments, we are looking for empirical (experimentally proved) relations linking the permeability K to the void ratio (e) or the porosity (n) of saturated porous media characterised generally by high clay and peat contents, high compressibility's and low permeabilities.

The difficulties encountered when establishing this kind of relation are mainly due to the numerous parameters interacting on the permeability value of loose sediments: (1) the lithology, (2) the grains and solid particles size, (3) the shapes, orientations and specific surface of the grains, (4) the pores spatial distribution.

As mentioned above, the micro-structural evolution of clays during the consolidation, orienting the plates more orthogonally to the direction of the vertical applied effective stress, develops an increasing structural anisotropy. This evolution increases the tortuosity of the flow channels as the flow is parallel to the vertical effective stress. This statement does not rule out the decrease in K by a decrease of the total void ratio.

It is convenient, from the formalism point of view, to define equations, which are very similar to the (log σ',e) oedometer equations. We obtain (figure 6):

$$\begin{cases} e = C_{K_1} \log K + cst & K > K_{prec} \\ e = C_{K_2} \log K + cst & K \le K_{prec} \end{cases} \tag{16}$$

where K_{prec} is the permeability coefficient corresponding to the effective preconsolidation stress (σ'_{prec}), C_{K1} and C_{K2} are defined respectively as the elastic and plastic rate of K variation during the consolidation. The constant being determined experimentally, the relation could be generalised:

$$K = C/\sigma'^{n} \tag{17}$$

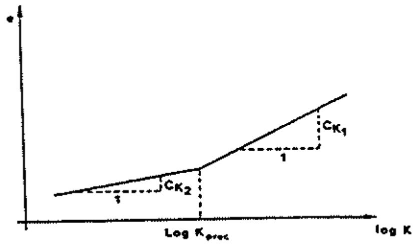

Fig. 6 (log K, e) diagram very similar to the (log σ', e) diagram of the oedometer test.

However it could be uneasy to determine a or C_{K1} and C_{K2} on practical case.

Many other relations can be experimentally fitted to the tests results (Terzaghi (1943), Rieke & Chiligarian (1974), Barends (1990), Safai & Pinder (1980), Lambe & Whitman (1969)). On basis of many test results on Quaternary coastal sediments in Japan, Nishida & Nakagawa (1969) have presented another equation linking K to e, taking into account the plasticity index (Ip) as an additional parameter.

It has been generalised and applied successfully for the computation of the subsidence in Shanghai area (Dassargues et al., 1991 and Dassargues et al.,1993) with the form :

$$K = e^{a.e+b} \tag{18}$$

where $a = \dfrac{2.3}{(c.Ip+d)}$

b, c, d are constants experimentally determined. Therefore, many relations are known, more or less well adapted to each studied case. It is important to synthesise the maximum of available geological and sedimentological data, in order to orientate the hydro-geotechnical choice (e, n, K, in function of σ') to choose a relation and to fit : its parameters, constants or exponents.

6 CONCLUSIONS: INFLUENCE ON THE COMPUTED CONSOLIDATION

The reader interested about this case history, have to see the following references: Baeteman(1989) for the Quaternary analysis, Dassargues et al.(1991) about the preparation of the hydrogeological and geotechnical data, and Dassargues & Li (1991) for a summary of the computational aspects. Moreover the Bulletin of the International Association of Engineering Geology (IAEG) has published in recently a group of 5 papers describing entirely the whole study.

Rudolph & Frind (1991), have shown that for a pore pressure variation imposed at the bottom of a clayey column, it takes more time to reach permanent flow conditions with varying parameters than with constant parameters. These conclusions are always verified if the K and S_s values are identical at the beginning of both computations. The differences in the calculated pore pressure spatial distributions induce automatically (by the Terzaghi principle) the main differences in the calculated subsidences. Moreover, even with pore pressures taken rigorously identical, it has been demonstrated (Dassargues, 1991) that the subsidences computed by the simulation with constant parameters will be systematically overestimated when compared with those calculated with varying parameters (if the initial parameters are taken identical).

In the case study of Shanghai, the final results (figure 7), show how inaccurate can be a model neglecting the variation of the parameters in the flow-compaction computations. Using the Finite Element Method (FEM), the computations are based on a detailed 3D flow model of the whole area, coupled to 32 non linear 1D flow-compaction models located where accurate measured data were available. Careful calibrations of both hydrogeological and geotechnical parameters have been made, and at last, future subsidences have been computed until year 2000, for pumping = 1.3 x recharge in the aquifers.

ACKNOWLEDGEMENTS

The Finite Element program that we have used for the case study of Shanghai, is the LAGAMINE code developed in the MSM department of the University of Liège. All our thanks to this team to have implemented in the code all the developed concepts exposed here above.

Many thanks also to IBM Belgium S.A. for the provided hardware support in the framework of a partnership.

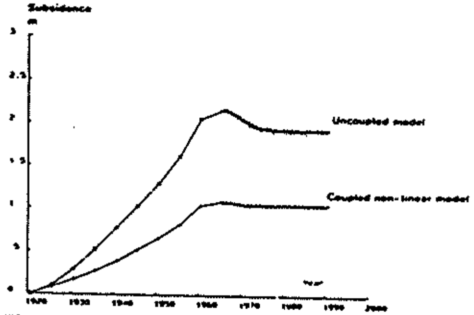

Fig. 7 Total computed subsidence since 1920 for one column of the case study of Shanghai; the computation neglecting the variation of K and S_s during the consolidation process leads to an overestimation of the subsidence of nearly 100% at this place (from Dassargues, 1991).

REFERENCES

Archer, J.S. & Wall, C.G. 1987. *Petroleum engineering, Principles and practices.* Chapter 5. Characteristics of Reservoirs. Imperial College of Science and Technology, London. Graham and Trotman.

Baeteman, C. 1989. *The Upper Quaternary Deposits of the Changjiang Coastal Plain-Shanghai area.* Belgian Geological Survey Report, 155 p.

Baeteman, C. & Dassargues, A. 1992. *Relations between the characteristics of deposits at major river mouths and their hydrogeological and geotechnical properties conditioning natural and man-induced land subsidence.* Memory prepared for the Belgian Royal Academy of the Overseas Sciences, april, Brussels.

Barends, F.B.J. 1990. The role of pore water in geological and geotechnical engineering. Proc. of the 6th IAEG Congress Rotterdam. Balkena.

Bear, J. & Verruijt, A. 1987. *Modeling groundwater flow and pollution.* Reidel, 414 p.

Biot, M.A. 1956. General solutions of the equations of elasticity and consolidation for a porous material. *J. Applied Mechanics*, Trans. ASME, 23, pp. 91-96.

Burger, A., Recordon, E., Bovet D., Cotton, L., et Saugy, B. 1985. *Thermique des nappes* *souterraines*. Presses polytechniques romandes, Lausanne, 255 p.

Cant, D. 1986. Subsurface Facies Analysis. In: R. Walker (ed.), Facies Models, Ontario, Ainsworth Press Limited, pp. 297-310.

Castany, G. 1967. *Principes et méthodes de l'hydrogéologie*, Dunod, Paris, 238 p.

Dassargues, A. 1991. *Paramétrisation et simulation des réservoirs souterrains, Couplages et non linéarités*. PhD Thesis in Applied Sciences, University of Liège, unpublished

Dassargues, A. ,Biver, P. and Monjoie, A. 1991. Geotechnical properties of the Quaternary sediments in Shanghai. *Engineering Geology*. 31, pp. 71-90.

Dassargues, A. & Li, X. L. 1991. Computing the land subsidence of Shanghai by finite element method. Proc. 4th Int. Symp. on Land Subsidence, IAHS Publ. No 200, pp.613-624.

Dassargues, A., Radu,J.P., Charlier,R., Li,X.L. and Li,Q.F. 1993. Computed subsidence of the central area of Shanghai. *Bulletin of the IAEG*. n°47,pp.27-50. Paris.

Delage, P. et Lefebvre, G. 1984. Study of the structure of a sensitive Champlain clay and of its evolution during consolidation. *Canadian Geotechnical Journal*. 21, pp. 21-35.

Demarest, J., Biggs, R. and Kraft, J. 1981. Time-stratigraphic aspects of a formation: Interpretation of surficial Pleistocene deposits by analogy with Holocene paralic deposits, southeastern Delaware. Geology, 9, pp. 360-365.

Domenico, P.A., & Mifflin,M.D. 1965. Water from low-permeability sediments and land subsidence, *Water Resources Research*. Vol. 1. n° 4, pp. 563-576.

Einsele, G. 1992. *Sedimentary Basins*. Springer-Verlag, Berlin.

Hawkins, A. 1984. Depositional characteristics of estuarine alluvium: some engineering implications. *Q. J. eng. Geol.*, 17, pp. 219-234.

Hobbs, N., 1986. Mire morphology and the properties and behaviour of some British and foreign peats. *Quarterly Journal of Engineering Geology*, 19, pp. 7-80.

Kaye, C.A. & Barghoorn, E.S. 1964. Late Quaternary sea-level change and crustal rise at Boston, Massachusetts, with notes on the autocompaction of peat. *Geological Society of America Bulletin*, v.75, pp.63-80.

Lambe, T.W. & Whitman, R.V. 1969. *Soil Mechanics*. John Wiley, New-York.

Nichols, M. & Biggs, R. 1985. Estuaries. In: R. Davis, Jr. (ed.), Coastal Sedimentary Environments, New York, Springer-Verlag, pp. 77-186.

Nishida, Y. & Nakagawa, S. 1969. Water permeability and plastic index of soils, in Land Subsidence IAHS-UNESCO,Publ. n° 89, pp. 573-578.

Poland, J.F. 1984. *Guidebook to studies of land subsidence due to ground-water withdrawal*. Studies and report in Hydrology. No.40, Unesco.

Rieke, H.H. & Chilingarian, G.V. 1974. *Compaction of argillaceous sediments*. Elsevier Amsterdam.

Rudolph, D.L. & Frind, E.O. 1991. Hydraulic response of highly compressible aquitards during consolidation. *Water Resources Research*. 27(1), pp. 17-30.

Russell, R. 1967. Origins of Estuaries. in: G.H. Lauff (ed.), *Estuaries*. Amer. Assoc. Adv. Sci., Publ. 83, Washington DC, pp. 93-99.

Safai, N.M. & Pinder, G.F. 1980. Vertical and horizontal land deformation due to fluid withdrawal, *Int. J. Numer. Anal. Meth. Geomech.* 4, pp. 131-142.

Terzaghi, K. 1943. *Theoretical soil mechanics*. Chapman and Hall, London.