Sept 91

FISOLS - 1991

ANALYSIS OF GROUNDWATER LEVEL FLUCTUATIONS AND BOREHOLE EXTENSOMETER DATA FROM THE BAYTOWN AREA, HOUSTON, TEXAS

ROLANDO BRAVO, Ph.D. candidate, Civil and Environmental Engineering Department, University of Houston, Houston, Texas, U.S.A. 77204,
JERRY R. ROGERS, Associate Professor, Department of Civil and Environmental Engineering, University of Houston, Houston, Texas, U.S.A. 77204,
THEODORE G. CLEVELAND Assistant Professor, Department of Civil and Environmental Engineering, University of Houston, Houston, Texas, U.S.A. 77204.

ABSTRACT: Former use of groundwater in the Baytown area in Houston, Texas has been about equally divided between public supply and industry. In this area piezometer wells and borehole extensometers have been constructed by the U.S. Geological Survey. The extensometer (LJ 65-16-930) was completed at a depth of 131 meters (431 feet), and another extensometer (LJ 65-16-931) was completed at a depth of 450 meters (1475 feet). A continuous record of the water levels in piezometers exists for different depths. Continuous records of consolidation (compaction) between the land surface and the depth of each extensometer also exist. This data was used to generate stress-strain diagrams. The head decline and the recovery (stress changes) plotted against compaction (strain) generates a series of open loops that represent the elastic and inelastic parts of the consolidation curve. Fourteen years of continuous records were analyzed for each of the clay layers in the Baytown area to compute important properties such as the storage coefficient for either the elastic range or the inelastic range and the vertical hydraulic conductivity.

INTRODUCTION

Use of ground water in the Baytown area, Houston Texas, was equally divided between public supply and industry. The use of ground water in this area slowly increased until 1973 when the industry started using surface water for its needs. Generally speaking, water levels in the Houston region declined from the beginning of development until 1977. Since 1977 no additional withdrawals have been allowed in this area. Nevertheless the piezometric level continues to fluctuate. This is associated with the consolidation of the compressible beds of the subsoil.

To determine small changes in elevation at specific locations, the United States Geological Survey (USGS) has placed borehole extensometers that give precise continuous records for determining the interval of consolidation that causes subsidence. The purpose of this work is to present the methodology for determining some fundamental properties of the compressible layers in the Baytown area using the relationship between the changes in the piezometric level and the recorded consolidation.

BACKGROUND

The Houston-Galveston area has two major aquifers: the Chicot and the deeper Evangeline aquifer, with the Burkeville Confining layer below. Electrical logs in the Baytown area indicate alternating sand and clay layers scattered throughout the vertical aquifer sections (Figure 1). One log interpretation has 16 layers of compressible material that are present between 131 m (431 feet) and 450 m (1475 feet) below land surface. The total clay thickness in this range is 155 m (510 feet) which represents 49% of the total thickness under study, 318 m (1044 feet).

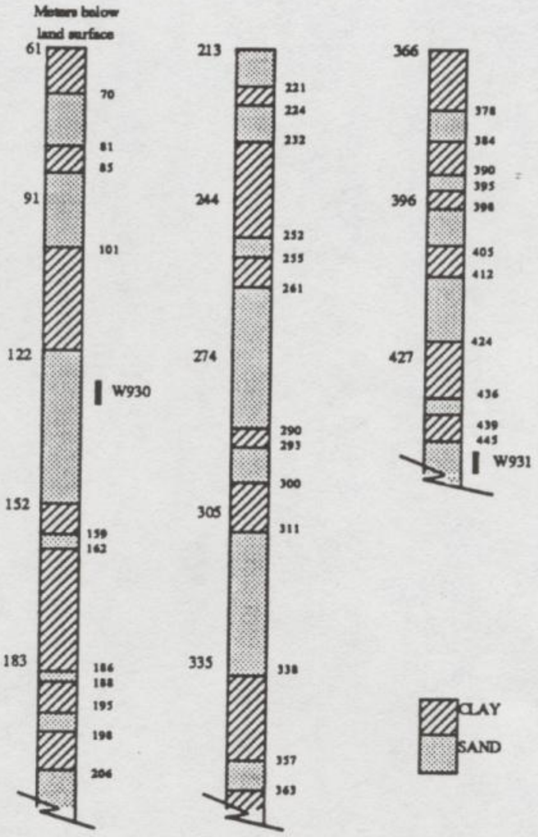


Figure 1 Soil profile: Baytown area. Solid vertical lines indicates the interval of well perforation.

Ground water pumping in the Baytown area slowly increased until 1973. During 1972, about 1.4 $m^3 s^{-1}$ (32 Mgal/day) was pumped while in 1978 a total of 0.95 $m^3 s^{-1}$ (21.6 Mgal/day) was pumped (Gaybrisch 1984). Since 1972 the ground water pumping has decreased because industry increased its use of surface water. During 1973-1977, water levels rose as much as 6.1 m (20 feet) in wells in the Baytown area. The Baytown site, established in 1972, has two extensometers installed to depths of 131 m (431 feet) and 450 m (1475 feet) to measure the consolidation in two depth intervals. In addition piezometers at different depths were also installed. Data (taken each 28 days from 1976 to 1989) is presented in Figures 2,3,4 and 5. The wells where these devices are placed are numbered as LJ-65-16-930 and LJ-65-16-931 respectively. The plot of the water level changes for both wells was made with the depth increasing downward to emphasize the correlation of this depth to the piezometric head. These graphs show that since 1976 the water has risen almost 37 m (120 feet) in the well LJ-65-16-930 and appears to remain stable since 1987 indicating that the system is reaching equilibrium. The water level in well LJ-65-16-931 has risen more than 24 m (80 feet) and the system also appears to be in equilibrium. The difference in these heads depicts the change in effective stress.

Figure 2 Water level changes: Baytown area well LJ-65-16-930, 131 m depth (1976-1982).

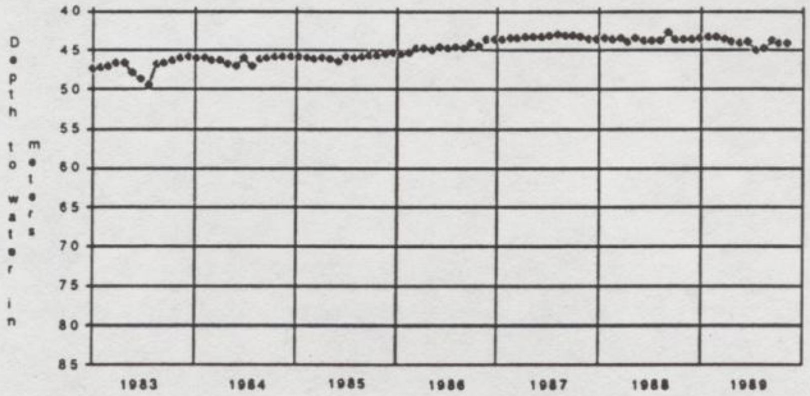


Figure 3 Water level changes: Baytown area well LJ-65-16-930, 131 m depth (1983-1989).

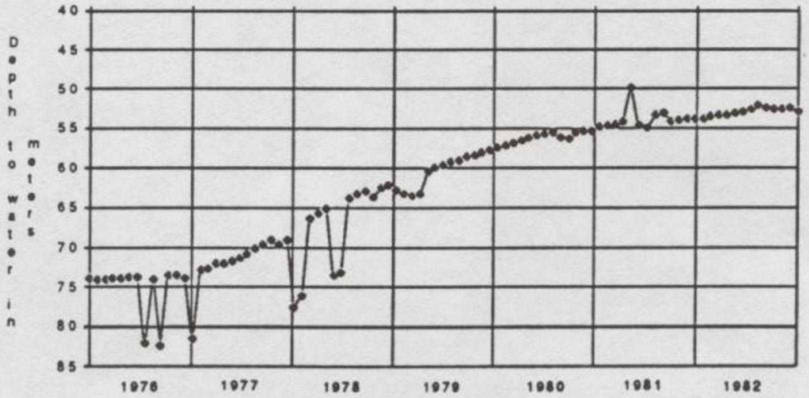


Figure 4 Water level changes: Baytown area well LJ-65-16-931, 450 m depth (1976-1982).

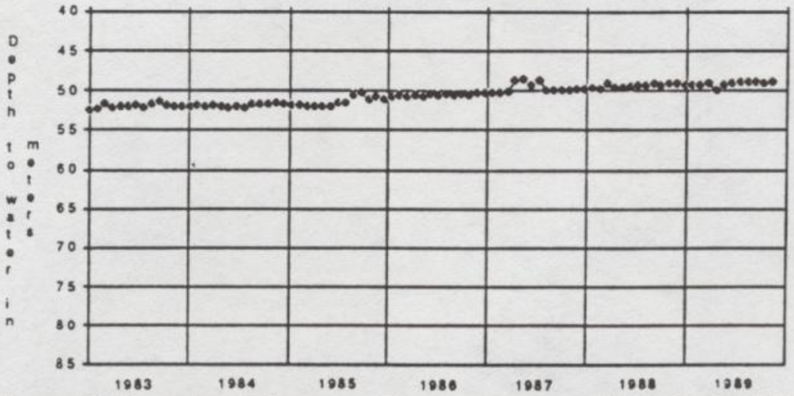


Figure 5 Water level changes: Baytown area well LJ-65-16-931, 450 m depth (1983-1989).

Recording borehole extensometers (that sometimes are called compaction monitors) are holes drilled and cased to selected depths, into which smaller diameter standpipes have been installed. The United States Geological Survey (USGS) has 11 borehole extensometers in operation at 11 different sites in the Houston-Galveston region. These borehole extensometers are able to continuously record the consolidation of the interval between the land surface and the bottom of the standpipe. Moreover, in the Houston-Galveston region, the United States Geological Survey (USGS) is collecting data on water level changes in different sands. From these data estimates of pressure change (stress change) at different depths for various time intervals may be made. Electrical logs are available for the places where the borehole extensometers are placed. These electrical logs can be used to identify the compressible and incompressible layers present either in the Chicot or the Evangeline aquifers.

BASIC RELATION BETWEEN SOIL MECHANICS AND GROUND WATER FLOW

The principle of mass conservation applied to one small elemental volume of a porous medium gives the following (Jacob, 1950)

$$-\frac{\partial(\rho_w v_x)}{\partial x}-\frac{\partial(\rho_w v_y)}{\partial y}-\frac{\partial(\rho_w v_z)}{\partial z}=\rho_w n\left(\frac{1}{E_w}+\frac{1}{E_k n}\right)\frac{\partial u}{\partial t} \qquad (1)$$

where E_w = bulk modulus of elasticity of water,

E_k = constrained modulus of elasticity (ratio of of vertical effective stress to laterally confined strain),

n = porosity (ratio of the volume of voids to the total volume).

This differential equation assumes, anisotropy of the density (density of the fluid moving through the porous media is not independent of direction), movement of the solid matrix is in the vertical direction (negligible movement in the x and y directions), the porous medium grains have no flexural strength and the load is transferred downward undiminished, and the change in volume of the soil grains is small in comparison to the change in the volume of water.

Substituting Darcy's law for the velocity terms, assuming isotropic density, and recalling that $h = z + (u/\rho_w g)$ gives the following equation that can be considered as a general differential equation for ground water flow in porous medium (Jacob 1950)

$$K_{xx}\frac{\partial^2 h}{\partial x^2}+K_{yy}\frac{\partial^2 h}{\partial y^2}+K_{zz}\frac{\partial^2 h}{\partial z^2}=n\gamma_w\left(\frac{1}{E_w}+\frac{1}{E_k n}\right)\frac{\partial h}{\partial t}\,. \tag{2}$$

The storage coefficient is the volume of water that an aquifer releases or takes into storage per unit surface area per unit change in head,i.e.,

$$S=\rho_w g b\left(\frac{n}{E_w}+\frac{1}{E_k}\right) \tag{3}$$

where b is the thickness of the aquifer,

ρ_w is the density of the water,

g is the gravity acceleration.

The specific storage (S_s) is the storage coefficient per unit width (sometimes called specific unit compaction).

In modeling the relationship of subsidence and ground water withdrawal, the aim is to determine how a change in the piezometric head deforms the soil, i.e., one needs to calculate the amount of consolidation of the compressible layers underlain in the area of study.

To achieve this goal, one important aquifer characteristic to determine is the storage coefficient . When water is pumped from confined aquifers, the ground water is removed from three sources: the expansion of water, the compression of the aquifer material, and compression of the clayey beds that are adjacent to and within the aquifer. When the compression does not result in a permanent rearrangement of the skeletal structure, the water can be replaced with an increase in pore pressure. This process is called elastic compression. Sometimes the term elastic compaction is used. However, if the pore pressure decreases beyond the interval where the soil particles compress elastically, additional water is released by rearrangement of the solid skeleton and the aquifer is permanently deformed (consolidated or compacted). This latter process is called permanent or inelastic compression (compaction) and causes the subsidence. Storage lost during the consolidation of the compressible layers is not recoverable.

The resulting compression per unit increase in effective stress in the inelastic range (virgin compression) is considerably greater than in the elastic range.In consolidation tests in soil mechanics this behavior is measured by the Compressibility Index C_c. This value of C_c is not constant and must be determined at the appropriate load. When the effective stress compressing the aquifer in the inelastic range is reduced (unloaded), the soil material again expands. In soil mechanics this behavior is measured by the Swelling Index C_s. The soil will recompact with elastic characteristics until effective stress increases beyond the new maximum effective stress (preconsolidation load).

The Compressibility Index C_c is determined by the slope of the closest straight line fit of a curve defined by a plot of void ratio values versus the logarithm of the effective stress of soil. The range of the effective stress must be in the virgin compression zone (inelastic range) (Das, 1985). Inelastic compaction is more nearly proportional to the increase in log of effective stress. C_c and consequently the corresponding storage coefficient S_{skv} is not a constant but rather is a function of effective stress. The subindex v in the notation corresponds to the virgin range. For modelling subsidence one needs to specify the corresponding value of S_{skv} when the load exceeds the preconsolidation load. Therefore it will be necessary to recalculate the value of S_{skv} for each change in the load.

The error in assuming a linear relationship between the change in the thickness of the compressible layer and the change in the effective stress can be calculated as

$$\text{error} = \frac{\Delta b^* - \Delta b}{\Delta b} \tag{4}$$

where Δb^* is the inelastic compaction assuming a constant value of S_{skv},

Δb is the actual inelastic compaction.

If the constant value of S_{skv} corresponds to an initial value of effective stress σ_o, then the error can be expressed as (Leake and Prudic, 1988)

$$\text{error} = \frac{0.434\ \Delta\sigma'}{\sigma_0 \Delta \log_{10} \sigma'} - 1\,, \tag{5}$$

where $\Delta\sigma'$ is the change in effective stress with respect to the initial value.

From equation 5, one can conclude that the percentage of error in calculating the deformation of the soil assuming a constant value of the specific inelastic storage coefficient is less than one half the percentage increase in the effective stress. Normally in most ground water flow systems, increments in effective stress are a relatively small percentage of the initial state of stress.

The coefficient of consolidation relates the effects of both storage and hydraulic conductivity by the relation

$$c_v = \frac{KE_k}{\rho_w g} \tag{6}$$

and neglecting the contribution of water elasticity to specific storage

$$c_v = \frac{K}{S_{sk}} \tag{7}$$

The time factor or dimensionless time T is

$$T = \frac{Kt}{S_s H_{dr}^2} = \frac{c_v t}{H_{dr}^2} \tag{8}$$

Finally, the specific inelastic storage coefficient was considered with the dimensionless time factor T used in soil mechanics in the solution of the partial differential equation of Terzaghi's consolidation theory. The time factor is defined as (Das, 1985) as

$$T_v = \frac{c_v t}{H_{dr}^2} \tag{9}$$

where H_{dr} is the two way drainage thickness of the layer or b/2 for this case. Sometimes the same time factor is defined in terms of a time constant denoted by τ as

$$T_v = \frac{t}{\tau} \tag{10}$$

which indicates that 100% of consolidation occurs when the time t is equal to the time constant τ. Combining equation (7) with (8) and (9) we have

$$\tau = \frac{(\frac{b}{2})^2 S_{skv}}{K_z} \tag{11}$$

This equation allows one to determine the hydraulic conductivity in the vertical direction if one knows the consolidation percentage and the specific inelastic storage coefficient.

All of the equations developed in this paper are applicable only for single layers; however, most systems studied in nature consist of many layers of material, each with its own characteristics.

Hantush (1960), studied pumping from an aquifer with scattered pervious and semipervious layers and found that the drawdown can be computed using an effective or equivalent storage coefficient equal to the sum of the storage coefficients of the individual aquifer layers. The specific storage coefficient of the system is the weighted mean of the specific storage of the individual layers

$$S_{s\ system} = \frac{S_{s1}b_1 + S_{s2}b_2 + \dots S_{sn}b_n}{B} \tag{12}$$

where S_{sn} is the specific storage coefficient of the n^{th} layer and,

b_n is the thickness of the n^{th} layer,

$B = b_1 + b_2 + \dots + b_n$.

Similarly, the horizontal hydraulic conductivity of a layered system is the weighted mean

$$K_{system} = \frac{K_1 b_1 + K_2 b_2 + \dots K_n b_n}{B} \tag{13}$$

and equation (13) can be considered to represent the maximum hydraulic conductivity of a layered system.

The vertical hydraulic conductivity of a layered system is given by

$$K_{z\ system} = \frac{B}{\frac{b_1}{K_{z1}} + \frac{b_2}{K_{z2}} + \cdots \frac{b_n}{K_{zn}}} \tag{14}$$

and can be considered as the minimum vertical hydraulic conductivity of a layered system.

THEORY

Effective stress in the compressible layers of an unconfined aquifer can change because of the change in the buoyant support associated with the water table fluctuation. In a confined aquifer the fluctuation of the piezometric head induces hydraulic gradients and seepage stresses. Roughly, the change in effective stress in a confined aquifer is equal to the change in hydraulic head (Lofgren, 1968, 1970)

The study of regional subsidence can be made by analyzing one dimensional consolidation because the extent of the study area is large enough to neglect the effects of lateral strains.

With the documented data available for the Houston area, it is possible to determine the history of the change in effective stress that will cause the consolidation for every one of the clay layers. The history of the soil deformation is also available from the recorded borehole extensometers. Therefore it is possible to plot stress-deformation graphs for each of the clay layers.

The plot of the consolidation history for the Baytown area (Figure 6 and Figure 7), shows in some periods of time a rebound of the soil and then again a recompression. This indicates an unloading followed by a loading process similar to the consolidation test performed in the laboratory where the compressibility and the swelling coefficients are determined by the slopes of the rebound and the virgin compression curves. The stress-deformation plot should show loops clearly indicating the unloading and loading process especially during the rebound periods. Because the process in each one of the clay layers is complicated, the behavior will not be the ideal, and some deviation is expected. Once the loop is found, the slope (of the unloading portion which represents the elastic expansion in response to a decreasing applied stress) will represent the corresponding storage coefficient

$$S_{ke} = \frac{\Delta b}{\Delta h} \qquad (15)$$

where Δb is the change in deformation of the clay layer and,

Δh is the effective stress change for the corresponding change in deformation measured in meters of water.

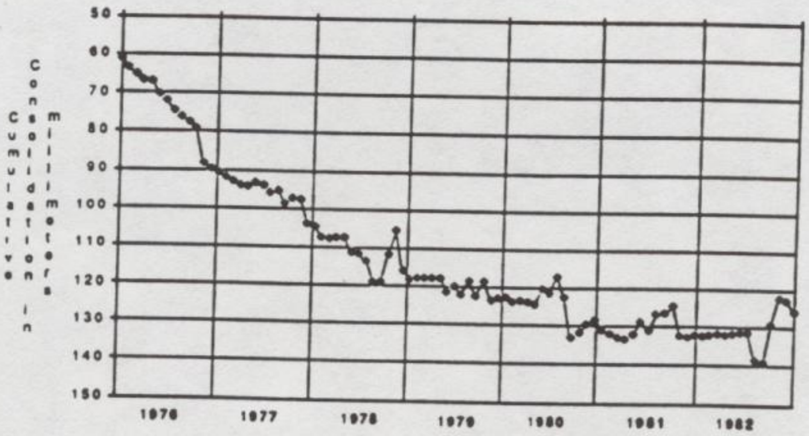


Figure 6 Consolidation history: Baytown area.

By constructing a line with a slope equal to S_{ke} from the point of minimal stress in the same loop to an interception with the deformation axis of the plot, it is possible to determine the amount of permanent deformation as the difference between these two points.

Likewise, the distance between the minimum stress and the intersection of the recompression part of the loop determines the preconsolidation load. Once the preconsolidation load is selected the slope of the ascending part of the loop will represent the inelastic storage coefficient S_{kv}.

After the inelastic storage coefficient (which is a measure of the permanent deformation) has been obtained, the proportion of ultimate consolidation can be calculated from the ratio of the change in height at time t and the ultimate change in height. This relation is expressed in terms of the inelastic storage coefficient as

$$U\% = \frac{\Delta b}{\Delta h \ S_{kv}} 100 \tag{16}$$

where Δb is the permanent deformation measured in the stress-deformation graph,

Δh represents the increase of stress beyond the preconsolidation load level.

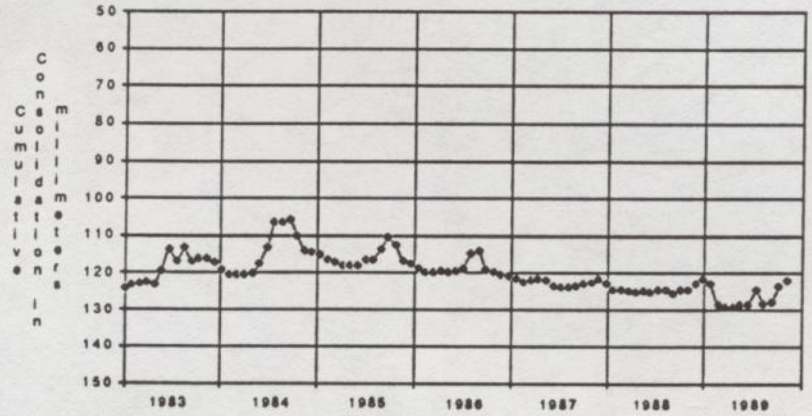


Figure 7 Consolidation history: Baytown area.

The calculation of the consolidation percentage allows the calculation of the time constant τ given the value of the time factor T_v. The time factor T_v is a function of the initial excess pore pressure distribution and the percentage of consolidation. In the present work the pore pressure distribution was simulated as a triangular increase and decrease. Thus the values of T_v can be obtained from tables such as Leonards (1962).

For calculating the time constant τ, one can assume that the consolidation at the end of the loading period is the same as that which would have resulted in half the loading period had all the load been applied at once (Terzaghi, 1950). This procedure also applied by Ryley (1969) requires the definition of the time of the loading period and the time of the maximum load. Both of these times are available from the stress history graph because the loop for the preconsolidation load determines the time of the loading period and the maximum load that corresponds to the time of the maximum stress. Mathematically the time constant using this approximation is given by the expression

$$\tau = \frac{0.5\ (t_1 - t_0)}{T_v} \tag{17}$$

By combining equation (17) with equation (11), the vertical hydraulic conductivity can be calculated.

Since the change in water level is recorded for the total thickness, one can assume that the water level change or head loss for a clay layer is proportional to the corresponding thickness. If ΔW is the total difference in water level between the piezometers LJ-65-16-930 and LJ-65-16-931, and b is the thickness of the clay layer, the head loss or hydraulic head that is determining the change in effective stress is

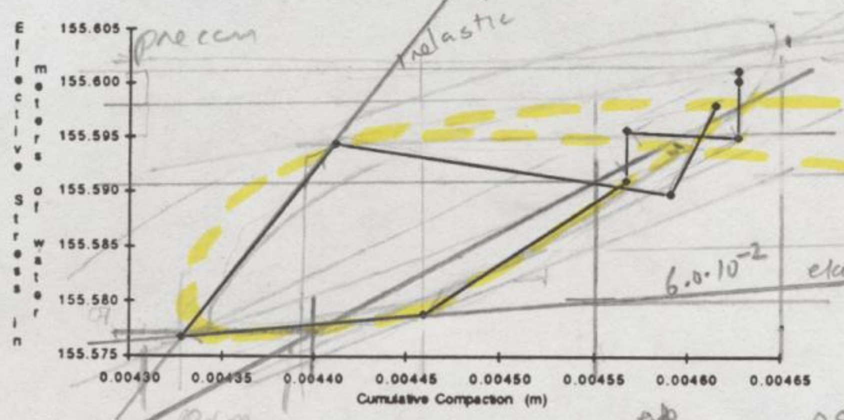
$$\Delta h = \frac{\Delta W}{L} b \qquad (18)$$

This change in effective stress could be added or subtracted from the initial depth H of the clay layer under study, depending upon the position of the water level in the piezometers. For instance when the water level measured in the deeper piezometer LJ-65-16-931, (450 m. below land surface) is less than the water level measured in the shallower piezometer LJ-65-16-930, (131 m below land surface), the amount calculated by the equation (18) is subtracted from H. The water level in the piezometers was measured with respect to the land surface. This procedure was followed to calculate the effective stress acting on each of the clay layers present in the Baytown area.

The plot of the consolidation history (Figure 6 and Figure 7) shows that even though the pumping in the Baytown area has been stopped since 1976, the soils between 131 m (431 feet) and 450 m (1475 feet) are still consolidating and during the period 1976-1989 have subsided 60.65 mm (0.199 foot). Likewise measurements from the extensometers show consolidation of about 98 mm (0.30 foot) between the land surface and 131 m (431 feet), and about 174 mm (0.53 foot) between the surface and 450 m (1475 feet). Consolidation measured by the extensometers probably does not reflect total subsidence because some additional consolidation may be occurring below the depth of 1475 feet (450 m) where the second extensometer is placed. To analyze each one of the compressible layers, the corresponding deformation is assumed to be also proportional to the thickness of the layer.

Once the effective stress and the consolidation for each one of the layers has been calculated, it is possible to plot these two variables and select one loop to determine the storage coefficients and the other soil parameters discussed above. In the Baytown area data for the year 1985 generates the loop shown in Figure 8 for the layer denoted as I. In this case a proportional variation of the stress and consolidation has been assumed. The rest of the layers generate similar loops.

The results of the calculations are summarized in Tables 1 and 2.



$\Delta\sigma = 0.0025$ 0.005

Figure 8 Stress and deformation: Baytown layer I, 1985.

$.0001$ $\frac{run}{rise} = S_{skv}$ $\frac{run}{S_{skv}} = rise$

$\frac{0.005}{0.0001}$

RESULTS AND CONCLUSIONS

$\frac{run}{rise} = \frac{1.5E-4}{0.0135} = 1.11\cdot10^{-2}$

A close relation between the change in piezometric head and soil deformation was observed from actual measurements. This fact was used to determine properties of the deformable soils in the Baytown area. In spite of the simplicity of the analytical procedure, the calculated values are consistent with other published values as shown in Table 2. This fact indicates that this simple methodology can be applied in other subsidence modeling with confidence.

TABLE 1 Baytown storage coefficients and vertical hydraulic conductivities in the compressible layers (61 to 445 m).

time

Layer	Thickness m	S_{ke}	S_{ske} m^{-1}	S_{kv}	S_{skv} m^{-1}	K_z m day^{-1}
I	6.10	8.50×10^{-3}	1.40×10^{-3}	6.10×10^{-2}	1.00×10^{-2}	2.80×10^{-4}
II	24.40	8.80×10^{-3}	1.90×10^{-3}	5.80×10^{-2}	2.40×10^{-3}	1.80×10^{-5}
III	7.60	9.00×10^{-3}	1.20×10^{-3}	5.50×10^{-2}	7.20×10^{-3}	0.40×10^{-4}
IV	7.60	9.00×10^{-3}	1.20×10^{-3}	5.50×10^{-2}	7.20×10^{-3}	0.40×10^{-4}
V	3.00	9.00×10^{-3}	2.95×10^{-3}	5.50×10^{-2}	1.80×10^{-2}	0.30×10^{-4}
VI	19.80	8.80×10^{-3}	4.40×10^{-4}	5.90×10^{-2}	3.00×10^{-3}	2.40×10^{-4}
VII	6.10	8.50×10^{-3}	1.40×10^{-3}	6.10×10^{-2}	1.00×10^{-2}	3.00×10^{-5}
VIII	3.00	9.00×10^{-3}	2.95×10^{-3}	5.50×10^{-2}	1.80×10^{-2}	0.35×10^{-4}
IX	10.70	8.80×10^{-3}	8.20×10^{-4}	5.80×10^{-2}	5.45×10^{-3}	0.70×10^{-4}
X	18.30	8.80×10^{-3}	4.80×10^{-4}	5.70×10^{-2}	3.10×10^{-3}	1.30×10^{-4}
XI	15.20	8.80×10^{-3}	5.80×10^{-4}	5.90×10^{-2}	3.90×10^{-3}	1.60×10^{-4}
XII	6.10	8.50×10^{-3}	1.40×10^{-3}	6.10×10^{-2}	1.00×10^{-2}	2.90×10^{-5}
XIII	3.00	9.00×10^{-3}	2.95×10^{-3}	5.50×10^{-2}	1.80×10^{-2}	0.30×10^{-4}
XIV	6.10	8.50×10^{-3}	1.40×10^{-3}	6.10×10^{-2}	1.00×10^{-2}	2.70×10^{-5}
XV	12.20	8.90×10^{-3}	2.23×10^{-4}	5.80×10^{-2}	4.75×10^{-3}	0.30×10^{-3}
XVI	6.10	8.50×10^{-3}	1.40×10^{-3}	6.10×10^{-2}	1.00×10^{-2}	2.90×10^{-5}

$K_{z\ system} = 4.36 \times 10^{-5}$ m day^{-1} $S_{ske\ system} = 9.00 \times 10^{-4}$ m^{-1}

$S_{skv\ system} = 5.90 \times 10^{-3}$ m^{-1}

TABLE 2 Comparison of Baytown calculated storage coefficients and vertical hydraulic conductivities with published values.

Source	S_{ke}	S_{ske} m^{-1}	S_{kv}	S_{skv} m^{-1}	K_z m day^{-1}
Bravo, et.al., (1991)	2.90×10^{-1}	9.00×10^{-4}	9.20×10^{-1}	5.90×10^{-3}	4.36×10^{-5}
Gabrysch,R.K. (1984)	4.10×10^{-2} 2.20×10^{-1}	3.95×10^{-5} 2.10×10^{-4}			
Meyer & Carr (1979)		4.90×10^{-5} 2.85×10^{-4}	3.00×10^{-4} 3.50×10^{-2}		3.65×10^{-5} 1.40×10^{-3}
Jorgensen,D.G. (1975)			5.00×10^{-3} 3.00×10^{-2}		
Riley,F.S.* (1969)	1.00×10^{-3}	2.80×10^{-6}	5.70×10^{-2}	1.40×10^{-4}	2.40×10^{-6}

* Values obtained for a cyclical load in Central California.

The data presented is valuable since the other method to determine the compressibility of the fine grained material obtaining representative undisturbed cores is difficult and expensive. Where subsidence has been well documented, such as in the Houston-Galveston region, subsidence may be coupled with historic stress changes and the amount of compressible material to determine compressibility to use for regional appraisal (Bravo, 1990; Bravo et.al.,1991).

ACKNOWLEDGEMENTS The authors wish to thank Mr. Robert K. Gabrysch, Chief of the Houston Subdistrict of the United States Geological Survey and his staff for providing much of the background data and reports.

REFERENCES

Bravo, R. (1990) Modified three dimensional finite difference model of the subsidence problem in the Houston Area, Ph.D. Dissertation, University of Houston, Houston, Texas.

Bravo, R., Rogers, J. R. & Cleveland, T. G. (1991) On the determination of the compressible soil properties required to model subsidence in the area of Houston, Texas, Proceedings of the Fourth International Symposium on Land Subsidence, Houston, Texas.

Das, Braja M. (1985) Principles of Geotechnical Engineering, PWS Publishers, Boston.

Jacob, C.E. (1950) Flow of ground water in Rouse, Hunter, Engineering Hydraulics; New York, John Wiley, pp 312-386.

Terzaghi, Karl, and Peck, R.B.(1948) Soil Mechanics in Engineering Practice, New York, John Wiley.

Riley, Francis, R.(1969) Analysis of borehole extensometer data from central California, Proceedings of the International Symposium on Land Subsidence, Tokyo , v II, pp 423-431, IASH/AIHS-UNESCO.

Leake, S.A. & Prudic, D.E.(1988) Documentation of a computer program to simulate aquifer-system compaction using the Modular Finite-Difference Ground-Water Flow Model, U.S. Geol. Survey, Open File Report 88-482.

Leonards, G.A. (1962) Foundation Engineering, McGraw-Hill Book Co., New York.

Lofgren, Ben E.(1968) Analysis of stresses causing land subsidence, U.S. Geological Survey, professional paper 600, pages B129-B225.

Lofgren, Ben E.(1970) Field measurements of aquifer-system compaction, San Joaquin Valley, California, Proceedings of the International Symposium on Land Subsidence, Tokyo , v I, pp 272-284, IASH/AIHS-UNESCO.`

Gabrysch, R.K.(1984) Ground water withdrawals and land-surface subsidence in the Houston-Galveston Region, Texas, 1906-80, U.S. Geological Survey report 287.

Jorgensen, Donald G.(1975) Analog model studies of ground water hydrology in the Houston district, Texas, U.S. Geological Survey, report 190.

Meyer, W.R., and Carr, J.E.(1979) A digital model for simulation of ground water hydrology in the Houston area, Texas: Texas Dept. Water Resources LP-103.