EVALUATING THE IMPACTS OF GROUNDWATER EXTRACTION ON LAND SUBSIDENCE IN CAI RANG DISTRICT, CAN THO CITY

Tran Van Tuan[1], Nguyen Truong Thanh[2], Nguyen Vo Ai Mi[3], Nguyen Thien Tri[3]

[1] Lecturer, Department of civil engineering, Can Tho University, Can Tho, Viet Nam

[2] Lecturer, College of Environment and Natural Resources, Can Tho University, Can Tho, Viet Nam

[3]Bachelor student, Can Tho University, Can Tho, Viet Nam

Received 09/11/2020, Revised 21/12/2020, Accepted 15/01/2021

Abstract

Groundwater is among the Nation's most important natural resources. It provides drinking water to urban and rural communities, supports irrigation and industry, sustains the flow of streams and rivers, and maintains riparian and wetland ecosystems. In many areas of the Nation, the future sustainability of groundwater resources is at risk from overuse and contamination. Groundwater extraction not only regulates the shape of the ground surface and but also has a significant influence on human life. However, the occurrence of estimating the settlement of ground surface is still not well-studied. The objective of this study is to assess relation between groundwater pumping and land subsidence in Cai Rang district, Can Tho city. It is carried out by following steps: (1) Collecting geological prospecting data;(2) Conducting the method of ground water pumping test; (3) Processing the collected data; (4) Using Riley, Lohman and Poland methods in estimating the settlement of ground surface; (5) Using Finite element method to stimulate land subsidence. The result shows that the hydro-geological parameters, K is of 18.05 m/h; S is of 0.0083; T is of 299.608 m^2/d. The cumulative land subsidence varies in the range of from 0 to 0.66 cm. Consequence of the land subsidence does not considerably affect construction work in the area. The analytical and interpretive methods described in this study will be useful to scientists involved in studies of ground-water hydraulics and aquifer-system deformation.

Keywords: *groundwater, land subsidence, pumping test, settlement*

1. Introduction

The Mekong delta one of the most important economical areas in Vietnam, has great annual rainfall and abundant surface water. However, the quality and quantity of water sources is not very well distributed in terms of time and space. As a result of the sharp increase in population and economic activities, both surface and groundwater resources in the Mekong delta are exploited at an ever-increasing rate. The market economy activities there are big requirements of the clean water for domestic and industrial sector that have promoted the exploitation of the potential resources, mainly groundwater, more than over. In view of its short tradition, there is a substantial backlog in the knowledge of groundwater extraction and drinking water treatment. There is almost no literature or documentation about these items.

In Can Tho City, flooding is becoming a more frequent and serious problem, as it happens as a result of both regular and extreme climatic events, such as tropical storms and typhoons. This problem can be seen to result mainly from: (1) the rise of sea level, and (2) the lowering of the land surface elevation, for which the rapid increase of population leads to excessive pumping of water from underground reservoirs. This resulted in the water table lowering, leading to the subsidence of some areas in the city. In response to the flooding challenges, besides climate change adaptation, knowledge of ground subsidence, such as the spatial extent and temporal evolution, is essential.

The aim of this research was thus: (1) The study was carried out following the method of groundwater pumping test in Can Tho city to determine the initial change of groundwater level in the observation wells over time, and then determine the basic hydro-geological parameters. (2) This study highlights the application of Riley, Poland and Lohman methods in estimating the settlement of ground surface caused by reducing groundwater table.(3) To make comments on the impact of decreasing groundwater table on the settlement of building constructions. (4) This study is used the ground motion prediction by Plaxis 2D software to simulate land subsidence. (5) Giving recommendations for monitoring and assessment.

2. Methodology

2.1 The method of groundwater (GW) pumping test

In 1935 C.V. Theis introduced equation (1) with the assistance of C.I. Lubin, who developed the equation for a continuous point source for the heat conduction problem. Equation (1) is a solution of equation (2) for constant discharge that involves the following assumptions, stated by Theis (1935): (1) the aquifer is homogeneous and isotropic, (2) the water body has infinite areal extent (practically its boundaries are beyond the effects of the well in the time considered), (3) the discharging well penetrates the entire thickness of the aquifer, (4) the well has an infinitesimal diameter (of no practical significance for periods of pumping longer than a few minutes) and (5) the water removed from storage is discharge instantaneously with decline in head. Thus, the assumption of a constant coefficient of storage has been added to the assumptions of homogeneity, isotropy and complete well penetration which characterize the steady state equations that have been given so far. The assumption of a constant coefficient of storage, which is used in all the transient flow equation that have been developed (there are a few exceptions where modifications of the assumption are explicitly stated), is ò doubtful validity, especially when applied to unconfined water bodies. The justification for this assumption is entirely empirical; it has been applied with some success for some decades, and deviations from it involve generally complex numerical computations.

$$S = \frac{Q}{4\pi T}\int_{\frac{r^2 S}{4Tt}}^{\infty}\left(\frac{e^{-u}}{u}\right)du \quad (1)$$

where:

s: drawdown

Q: constant discharge rate from well

T: transmissivity

r: distance from discharging well to point of observation of s

S: storage coefficient

t: time since discharge began

u: variable of integration

$$\frac{1}{r}\frac{\partial h}{\partial r} + \frac{\partial^2 h}{\partial r^2} = \frac{S}{T}\frac{\partial h}{\partial t} \quad (2)$$

Equation (1) cannot be integrated directly, but its value is given by the infinite series in the following equation:

$$s = \frac{Q}{4\pi T}\left[-0.577216 - \log_e u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - ...\right] \quad (3)$$

Where:

$$s = \frac{Q}{4\pi T}\left[-0.577216 - \log_e u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - ...\right] \quad (4)$$

Which is the lower limit of integration in equation (1); the value of the series is commonly expressed as W(u) the well function of u. Values of W(u) for values of u from 10^{-15} to 9.9 are tabulated in Wenzel (1942,pp.89), in Ferris, Knowles, Brown, and Stallman (1962, pp.96-97). For given values of u and , T may be determined from:

$$T = \left[\frac{Q}{4\pi s}\right] W(u) \quad (5)$$

And S may be determined by rewriting equation (3)

$$S = \frac{4Ttu}{r^2} \text{ or } \frac{4Tu}{\frac{r^2}{t}} \quad (6)$$

$$s = \left[\frac{Q}{4\pi T}\right] W(u) \quad (7)$$

$$\log_{10} S = \left[\log_{10}\frac{Q}{4\pi T}\right] + \log_{10} W(u) \quad (8)$$

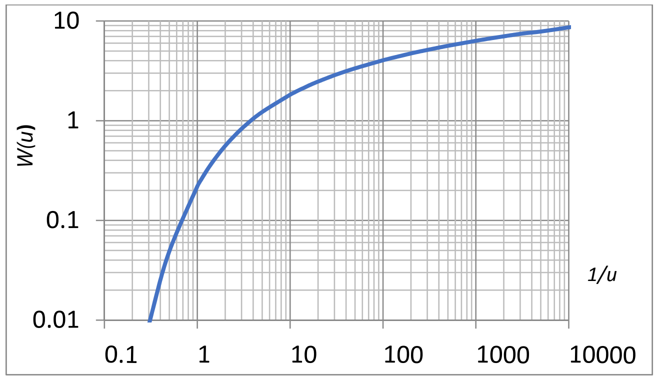

Figure 1.1. Logarithmic graph of W(u) versus u (Source: Lohman, 1972).

and:

$$\frac{r^2}{t} = \left[\frac{4T}{S}\right]u \quad (9)$$

or:

$$\log_{10}\frac{r^2}{t} = \left[\log_{10}\frac{4T}{S}\right] + \log_{10} u \quad (10)$$

The data curve is superimposed on type curve, and a fit, of near fit, is obtained, keeping the coordinate axes of the two curves parallel. An arbitrary match point is selected anywhere on the overlapping parts of the two sheets, the four values of which (two for each sheet) are then used in solving equation (5) and (6).

2.2 Riley's methods in estimating the settlement of ground surface

The application of the time-consolidation theory of soil mechanics to explain the theory of aquifer-system compaction has been summarized lucidly by Riley (1969), as follows:

Vertical compaction rates resulting from drawdown in each hydrogeologic layer were calculated according to the following expression for 1D consolidation of compressible porous media (Riley, 1969):

$$\Delta b = S_s \times b \times \Delta h \quad (11)$$

where:

Δb: is the change in layer thickness (cm)

b: is the full thickness of the layer (m)

Δh: is the change in hydraulic head, or drawdown (m)

S_s: is the layer's specific storage (m^{-1}), $S_s = \frac{s}{b}$

A metric related to the compressibility of both the sediment and water that expresses the volume of water released from storage per unit volume of water-bearing layer per unit decline in hydraulic head. This equation assumes instantaneous equilibration of hydraulic heads in low permeability units, neglecting delays in drainage. Drawdown rates in a given aquifer were applied to that layer and the overlying confining layer. Specific storage (inelastic) was assumed to be constant within each layer but decrease with layer depth (midpoint) according to an exponential function (Ingebritsen et al 2006) due to natural consolidation occurring over geologic time.

Based on 1-D water flow theory in homogenous soil to calculate the pumping rate flow over time

1. Set datum line: h_e

2. Calculate h_p, h_t at know points

$$h_p = \frac{u}{\gamma_w} \; ; \; h_t = h_p + h_e$$

(12) (13)

3. For water flow through different soil: Use "continuity equation" to solve problem.

4. For water flow through the same layer,

$k=const \quad \Rightarrow i=const$

$$i = \Delta h_t / L = (h_{t2} - h_{t2}) / L_{21} = (h_{t3} - h_{t2}) / L_{32} = const \tag{14}$$

Can use "known i" to solve h_t at any point

(13) $\Rightarrow h_p = h_t - h_e$

(12) $\Rightarrow u = h_p \gamma_w$

2.3 Poland's methods in estimating the settlement of ground surface

It is quantitatively convenient in treating complex aquifer systems to compute effective stresses and stress changes in terms of gravitational stress and the vertical normal component of seepage stress, which are algebraically additive. The following brief discussion is summarized from Poland and others (1975).

If the compaction and water-level measurements are adequate to yield stress-strain plots that define compressibility in the plastic-plus-elastic range (stress exceeds preconsolidation stress) for the full compacting interval, approximate ultimate compaction (and subsidence) for a specified stress increase can be computed by use of the equation:

$$\Delta z = m_v m \Delta p' \tag{15}$$

where:

Δz : is the computed ultimate subsidence or compaction (cm)

m_v : is the mean compressibility of the compacting beds (cm^2/kg)

m : is the aggregate thickness of the compacting beds (m)

$\Delta p'$: is the change in effective stress (KN/m^2), $\Delta p' = p_o - p_1$

The coefficient of volume compressibility, m_v in soil-mechanics terminology:

$$m_v = \frac{e_0 - e_1}{(1 + e_o)\Delta p'} \tag{16}$$

where:

e_o : Void ratio when not change effective stress

e_1 : Void ratio when the excess pore water pressure

2.4 Lohman's methods in estimating the settlement of ground surface

Lohman showed that for an elastic confined aquifers: (Lohman,1972)

$$\frac{b}{E_s} = \frac{S}{\gamma_w} - \theta b \beta \tag{17}$$

and that Hooke's Law (strain is proportional to stress, within the elastic limit) may be expressed:

$$\Delta b = \frac{b}{E_s} \Delta p \tag{18}$$

Combining equations (17) and (18):

$$\Delta b = \Delta p \left(\frac{S}{\gamma_w} - n b \beta \right) \tag{19}$$

where:

E_s : bulk modulus of elasticity of the soil skeleton of the aquifers

S: is the storage coefficient, $S = S_s \times b - S_s$: is the layer's specific storage

Δb : is compaction (cm)

Δp : is pressure caused by reducing groundwater table (N/m^2)

$\Delta p = \gamma_w \Delta h$, γ_w : is the unit weight of water (KN/m^3)

Δh : is the change in hydraulic head (m)

$\theta = n$: is the porosity (%), $n = \frac{e}{1+e}$

b: is the thickness of the aquifer system (m)

β : is the compressibility of the water (N/m^2)$^{-1}$, $\beta = \frac{1}{E_w}$

E_w : is elastic module (N/m^2), E_w = 2.1x10^9(N/m^2) = 2.1x10^6(KN/m^2)

2.5 Finite element method

2.5.1 About plaxis 2D

Plaxis 2D Foundation is a finite element program which has been written for analysis foundations of structure including piled raft foundation. It can generate a large 2D finite element meshes. The mechanical behavior of soils can be modeled by several models (e.g. Mohr-Coulomb model) for different analyses. The interactions between piles, raft and soil can be simulated via this program.

2.5.2 Mohr-coulomb model

Linear Elastic Model

This is the simplest model used for materials, which is based on the Hooke's law for isotropic linear elastic behavior. The relationship between effective stress and strain is expressed in term of the rate as below:

$$\dot{\sigma}' = D^{\varepsilon} \dot{\varepsilon} \tag{1}$$

Where D^e is the elastic material stiffness matrix. Effective Young's modulus E and effective Poisson's ratio ν are used in this model, which are attached in D^e matrix. Linear elastic model is inappropriate to model behavior of the soils which have highly non-linear behavior. This model is suitable to simulate behavior of structures (e.g. piles, raft or walls) where the strength properties of materials are very high compared with those of soil. In Plaxis, this model is usually used together with *Non-porous* type of material behavior to exclude pore pressures from these structural elements.

Mohr-Coulomb Model

The Mohr-Coulomb model is an elastic perfectly plastic model which is a constitutive model with a fixed yield surface and the behavior of points within the yield surface is purely elastic.

Based on the basic principal of elastoplasticity, equation (3.1) can be written as:

$$\dot{\sigma}' = D^e\left(\dot{\varepsilon} - \dot{\varepsilon}^p\right) \tag{2}$$

Where $\dot{\varepsilon}^p$ is the plastic strain rate component which is defined by:

$$\dot{\varepsilon}^p = \lambda \frac{\partial g}{\partial \sigma'} \tag{3}$$

Where λ is the plastic multiplier which is defined from the yield function, *f*, as below:

$$\lambda = 0 \text{ for: } f < 0 \text{ or: } \frac{\partial f^T}{\partial \sigma'} D^e \dot{\varepsilon} \le 0 \text{ (Elasticity)} \tag{4a}$$

$$\lambda > 0 \text{ for: } f = 0 \text{ and: } \frac{\partial f^T}{\partial \sigma'} D^e \dot{\varepsilon} > 0 \text{ (Plasticity)} \tag{4b}$$

g is the plastic potential function which is introduced to fix the problem of theory of associated plasticity in estimating dilatancy. Non-associated plasticity is denoted as $g \neq f$.

Therefore, the relationship between effective stress rates and strain rates can be expressed as

$$\dot{\sigma}' = \left(D^e - \frac{\alpha}{d} D^e \frac{\partial g}{\partial \sigma'} \frac{\partial f^T}{\partial \sigma'} D^e \right) \dot{\varepsilon} \tag{5a}$$

In which α = 0 (elasticity) and α = 1 (plasticity)

$$d = \frac{\partial f^T}{\partial \sigma'} D^e \frac{\partial g}{\partial \sigma'} \tag{5b}$$

For multi surface yield contour, the above equations should be extended as:

$$\dot{\varepsilon}^p = \lambda_1 \frac{\partial g_1}{\partial \sigma'} + \lambda_2 \frac{\partial g_2}{\partial \sigma'} + \lambda_3 \frac{\partial g_3}{\partial \sigma'} + \ldots \tag{6}$$

Where λ_i (i = 1, 2, 3,…) can be defined from the yield functions f_i(i = 1, 2, 3,…), respectively.

The yield condition used in Mohr-Coulomb model is an extension of Coulomb's friction law to general states of stress. In principle stress space, this condition consists of six yield functions as below:

$$f_{1a} = \frac{1}{2}(\sigma_2' - \sigma_3') + \frac{1}{2}(\sigma_2' + \sigma_3')\sin\phi - c\cos\phi \le 0 \tag{7a}$$

$$f_{1b} = \frac{1}{2}(\sigma_3' - \sigma_2') + \frac{1}{2}(\sigma_3' + \sigma_2')\sin\phi - c\cos\phi \le 0 \tag{7b}$$

$$f_{2a} = \frac{1}{2}(\sigma_3' - \sigma_1') + \frac{1}{2}(\sigma_3' + \sigma_1')\sin\phi - c\cos\phi \le 0 \tag{7c}$$

$$f_{2b} = \frac{1}{2}(\sigma_1' - \sigma_3') + \frac{1}{2}(\sigma_1' + \sigma_3')\sin\phi - c\cos\phi \le 0 \tag{7d}$$

$$f_{3a} = \frac{1}{2}(\sigma_1' - \sigma_2') + \frac{1}{2}(\sigma_1' + \sigma_2')\sin\phi - c\cos\phi \le 0 \tag{7e}$$

$$f_{3b} = \frac{1}{2}(\sigma_2' - \sigma_1') + \frac{1}{2}(\sigma_2' + \sigma_1')\sin\phi - c\cos\phi \le 0 \tag{7f}$$

Where ϕ, c are the friction angle and cohesion of the soil respectively. The condition $f_i = 0$ for all yield functions together give a hexagonal cone as shown in Figure 1.

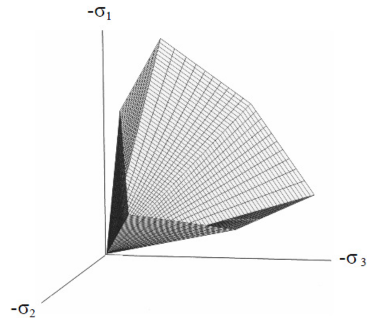

Figure 1. The Mohr-Coulomb yield surface in principal stress space for c = 0 (Source: Brinkgreve et al., 2007).

The plastic potential functions of Mohr-Coulomb model are defined as below:

$$g_{1a} = \frac{1}{2}(\sigma_2' - \sigma_3') + \frac{1}{2}(\sigma_2' + \sigma_3')\sin\psi \tag{8a}$$

$$g_{1b} = \frac{1}{2}(\sigma_3' - \sigma_2') + \frac{1}{2}(\sigma_3' + \sigma_2')\sin\psi \tag{8b}$$

$$g_{2a} = \frac{1}{2}(\sigma_3' - \sigma_1') + \frac{1}{2}(\sigma_3' + \sigma_1')\sin\psi \tag{8c}$$

$$g_{2b} = \frac{1}{2}(\sigma_1' - \sigma_3') + \frac{1}{2}(\sigma_1' + \sigma_3')\sin\psi \tag{8d}$$

$$g_{3a} = \frac{1}{2}(\sigma_1' - \sigma_2') + \frac{1}{2}(\sigma_1' + \sigma_2')\sin\psi \tag{8e}$$

$$g_{3b} = \frac{1}{2}(\sigma_2' - \sigma_1') + \frac{1}{2}(\sigma_2' + \sigma_1')\sin\psi \tag{8f}$$

Where ψ is the dilatancy angle of the soil. Hence, there are five parameters including c, ϕ and ψ for plasticity and E and ν for elasticity are required for Mohr-Coulomb model.

3. Results

Different lithofacies such as clay, silt or sand, which are originated from different sedimentary environments, have different degrees of consolidation even if they were subject to an identical load. The classification of the land subsidence area into such geological units is instrumental in analyzing the land subsidence in terms of the hydrological balance of ground water and soil mechanics. Presuming that the regional differences of the land subsidence depend on geological and hydrogeological variations.

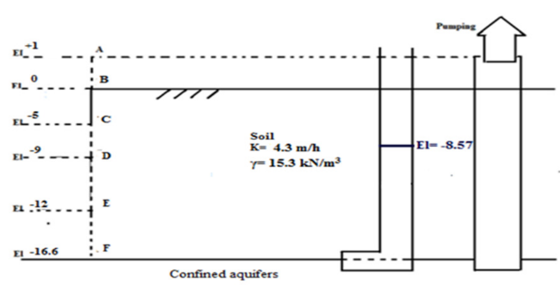

Figure 1.2. The distribution of calculated soil layers.

The examined clay layer located in Hung Phu industrial park where has a similar climatic and rainfall conditions as the study area. Hence, it is reasonable to use the soil parameters in this area as the input for the calculation in this study. The measured soil layer is divided into 4 different thickness sub-layers because geological conditions are different in dissimilar areas. The soil layers are divided into dissimilar thickness in order to be suitable with modeled values in plaxis 2D. It indicates clearly the changes of settlement parameters according to the depth. The point F (Elevation = -16.6m) is the reference point for the calculation.

3.1 The basic hydro-geological parameters

By using geological prospecting data and the basic hydro-geological parameters of the upper-middle Pleistocene (qp_{2-3}) aquifer: the land subsidence was defined and its triggers were discussed. The result of this study can be made as follows:

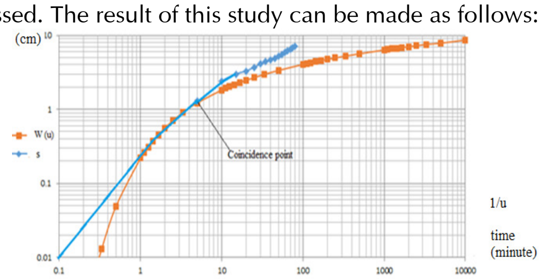

Figure 1.3. Coincidence of water drawdown in BS05 well (logarithmic scale) with logarithmic graph of W(u) versus u.

The graph shows: the points of coincidence among two graphs from Figure 3.7 and Figure 4.2 is s=1.3cm,t=5 minutes and

$$W(u) = 1.223, \frac{1}{u} = 5$$

Pumping test at BS05 pumping well, People's Committee of Tan Phu ward, Cai Rang district, Can Tho city shows a data set allows to precisely analyzed for hydrogeological parameters in the study area. Hydrogeological parameters of the upper Pleistocene aquifer (qp_{2-3}) were determined such as: permeability coefficient K = 4.3m/h, transmissivity coefficient T = 299.608m^2/h, storativity coefficient S = 0.0083, specific storage S_s = 0.0005.

3.2 Finite element method

For purposes of simulating the mechanical response (compression and expansion) of the aquifer system to groundwater level variations. Data collection related to soil, and the interpretation of such information, is fragmented. It is critical to analysis standards for subsidence-related information. Using geological parameters of this soil, the lowering groundwater are as follows:

Table 1.1. Geological parameters.

Parameters	Soil
Thickness (m)	16.6
Description	Clay
Model	Mohr-coulomb
$\gamma_{unsat}(KN/m^3)$	15.3
$\gamma_{unsat}(KN/m^3)$	16.375
$E_{ref}\ (KN/m^2)$	6.08×10^{-4}
v	0.300
c_{ref}(KN/m^2)	0.03
φ (^{0})	1.96

It was set up to match the properties specified in the geological prospecting data as closely as possible, to determine how changes in effective stress affect computed subsidence in this model. With parameters set up as described above , this study we adopted the change in head to compute compaction simulation of effective stress periods

The aquifer system is stressed by hydraulic head changes caused by pumping. Compaction caused by drawdown in the aquifer system is simulated in Figure 1.4.

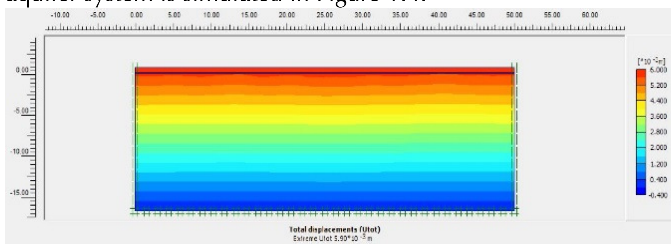

Figure 1.4. Distribution of ground settlement with depth (Drawdown = 0.75m).

Figure 1.4 presents the groundwater level decreased from -8.57m to -9.32m, the land subsidence (P) is 0.60cm, average is 0.8cm. The settlement of soil near the surface is larger than settlement of soil at larger depth

The model solves for hydraulic head and vertical displacement for specified aquifer-system properties as a function of depth and time. The model parameters were adjusted within moderate ranges and available constraints to provide the best between measured and simulated compaction. The computed subsidence volume was 0.60cm. By incorporating the resulting parameter estimates in the previously calibrated regional model of groundwater flow and land subsidence we can significantly improve the agreement between simulated and observed land subsidence both in terms of magnitude and spatial extent

3.3 Calculated Land Subsidence

Comparing four different methods at practical values with other hypothetical values:

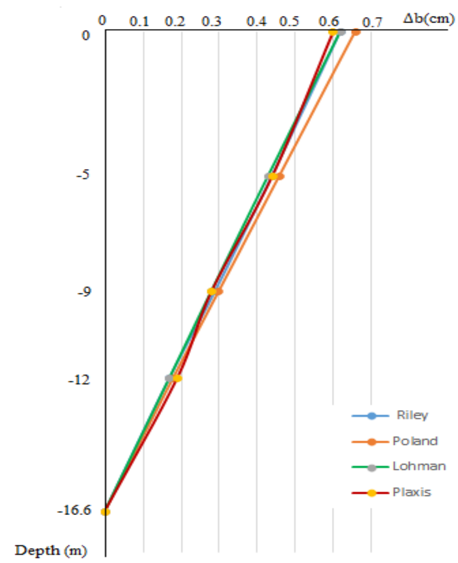

Figure 1.5. Relationship between depth and land subsidence (drawdown = 0.75m).

The value of land subsidence caused by groundwater extraction in the study area depends on geological conditions and drawdown. Using four methods (Riley, Poland, Lohman and Plaxis 2D model) the cumulative land subsidence in Cai Rang district, Can Tho city was about 0.62cm, 0.66cm, 0.61cm and 0.60cm (Figure 1.5). The land subsidence increases from 0 to 0.6cm (up to 0.66cm), the average value is 0.15cm. The calculated settlement accumulates with depth forward the ground surface.

4. Concolusions:

The results showed the appropriate methods to apply in calculating the surface subsidence due to lowering the water level below the soil. The value of land subsidence caused by groundwater extraction in the study area depends on geological conditions and drawdown. Using four methods (Riley, Poland, Lohman and Plaxis 2D model) the cumulative land subsidence in Cai Rang district, Can Tho city was about 0.62cm, 0.66cm, 0.61cm and 0.60cm. Comparison of the calculation results obtained by these four methods shown that calculation of the time-dependent settlement for the land subsidence using Lohman, Riley and Plaxis 2D (0.61cm, 0.62cm, 0.60cm) gave results relatively close to the same. Thus, Lohman, Riley and Plaxis 2D are reliable methods and can be used to predict the land subsidence caused by the groundwater extraction.

Using the Poland's method, the biggest land subsidence is 6–7% bigger than using Riley's method or Lohman's method and Plaxis 2D model. The reason for this deviation may relate to the short time of monitoring drawdowns and a small number of calculations. This long-term subsidence results from inelastic compaction of the aquifer system.

In summary, groundwater level fluctuations change effective stresses in the following two ways: 1) A rise of the water table provides buoyant support for the grains in the zone of the change, and a decline removes the buoyant support; these changes in gravitational stress are transmitted downward to all underlying deposits. 2) A change in position of either the water table or the potentiometric surface of the confined aquifer system, or both, may induce vertical hydraulic gradients across confining or semiconfining beds and thereby produce a seepage stress. The vertical normal component of this stress is algebraically additive to the gravitational stress.

References

[1] Boonchai, Ukritchon., (2009). Shallow foundation – Pile Foundation.

[2] Galloway, D.L. and F.S. Riley. 1999. San Joaquin Valley, California—Largest human alteration of the Earth's surface. In D.L. Galloway, D.R. Jones, and S.E. Ingebritsen, eds., Land Subsidence in the United States: U.S. Geological Survey Circular 1182, pp.23–34.

[3] Giao, P. H., & Thoang, T. T. (2016). Soil Characterization and Land Subsidence Prediction for the First MRT Line in HCM City. Geotechnical Engineering Journal of the SEAGS & AGSSEA, 47(1), pp.26-31.

[4] Ferris, J. G., Knowles, D. B., Brown, R. H., & Stallman, R. W. (1962). Theory of aquifer tests (pp. 69-174). Denver, Colorado: US Geological Survey.

[5] Ingebritsen, S. E., Sanford, W., & Neuzil, C. E. (2006). Groundwater in Geologic Processes, 2nd edn Cambridge University Press. Cambridge, UK.

[6] K. Terzaghi, Simplified soil test for sub-grade and their physical asignificance, Public Roads, 1925. V.7. pp.153-162.

[7] Lê Văn Phát, T. M. T., & Tỷ, T. V. (2017). Tác động của việc khai thác nước dưới đất đến biến động mực nước dưới đất tại thành phố Cần Thơ. Tạp chí Khoa học Trường Đại học Cần Thơ, pp. 22-30.

[8] Lohman, S. W., Bennett, R. R., Brown, R. H., Cooper Jr, H. H., Drescher, W. J., Ferris, J. G., ... and Stallman, R. W. (1972). Definitions of selected ground-water terms-revisions and conceptual refinements.US Government Printing Office, pp 1-15.

[9] Lohman, S. W. Compression of elastic artesian aquifers. US Geol. Surv. Prof. Pap., 424-B, 1961, pp.47-49.

[10] Minderhoud, P., Stouthamer, E., Pham, V. H, and Erkens, G., (2017). Impact of 25 years of groundwater extraction on subsidence in the Mekong delta, Vietnam, Environmental research letters, 12(6), 064006, pp. 1-13.

[11] N.M. Nguyen, Review and analysis Hanoi land subsidence monitoring data: master thesis in Engineering geology. Bangkok, 2008. 142 pp.

[12] P.H. Giao, Artificial recharge of Bangkok complex system for mitigation of land subsidence. Doctoral dissertation No. GE / 96-2. Bangkok:Asian Institute of Technology, 1997.

[13] P.H. Giao, N.A. Phien-wej, FEM Program for Land Subsidence Analysis,VNU Publisher, Hanoi, 2004

[14] Phi, T. H., & Strokova, L. A. (2015). Prediction maps of land subsidence caused by groundwater exploitation in Hanoi, Vietnam. Resource-Efficient Technologies, 1(2), pp.80-89.

[15] Poland, J. F. (1970). Status of present knowledge and needs for additional research on compaction of aquifer systems, pp. 11-20

[16] Poland, J. P.,Lofgren, B. E., Ireland, R. L., and Pugh, R. G., 1975. Land subsidence in the San Joaquin Valley, California, as of 1972. U.S. Geol. Survey Prof. Paper 437-H, pp.77.

[17] Poland, J. F. Guidebook to studies of subsidence due to groundwater withdrawal. Prepared for the International Hydrological Programme, Working Group 8.4, United States of America. Michigan: Book Crafters,Chelsea, 1984

[18] Poland, J. F., & Lofgren, B. E. (1984). Case history 9.13, San Joaquin Valley, California, USA. Guidebook to studies of land subsidence due to ground-water withdrawal: Paris, France, UNESCO Studies and Reports in Hydrology series, (40), pp.263-277.

[19] Riley, F. S., (1969), Analysis of borehole extensometer data from central California, in Tison, L. J.,ed., Land subsidence, v. 2: International Association of Scientific Hydrology Publication 89, pp. 423-431.

[20] Wenzel, L. K., & Fishel, V. C. (1942). Methods for determining permeability of water-bearing materials, with special reference to discharging-well methods, with a section on direct laboratory methods and bibliography on permeability and laminar flow. Geological Survey Water-Supply Paper (USA) eng no. 887.

Appendix

1. Calculation of land subsidence after Riley's theory

Drawdown = -0.75 m $=\Delta h$; Ground level: + 1 m

*Before pumping: Groundwater level: -8.57 m

Water pressure at point B, C, D, E, F:

- At A: $h_{eA}=1(m), h_{pA}=0(m), h_{tA}=1(m)$

- At F: $El.=-8.57m \Leftrightarrow h_{pF}=-8.57-(-16.6)=8.03(m)$

$h_{eF}=-16.6(m), h_{tF}=-8.57(m)$

$h_{tA} > h_{tF}$ $\Leftrightarrow$ Flowing A to F

-At B: $h_{eB}=0(m), h_{tB}=h_{tA}=1(m) \Leftrightarrow h_{pB}=1-0=1(m)$

$$i_{BC}=i_{BF}=\frac{h_{tB}-h_{tC}}{L_{BC}}=\frac{1-h_{tC}}{5}0.57$$

-At C: $i_{BC}=i_{BF}=\dfrac{h_{tB}-h_{tC}}{L_{BC}}=\dfrac{1-h_{tC}}{5}=0.57$

$\Leftrightarrow h_{tC}=-1.88(m), h_{eC}=-5(m) \Leftrightarrow h_{pC}=-1.88-(-5)=3.12(m)$

- At D: $i_{BD}=i_{BF}=\dfrac{h_{tB}-h_{tD}}{L_{BD}}=\dfrac{1-h_{tD}}{9}=0.57$

$\Leftrightarrow h_{tD}=-4.19(m), h_{eC}=-9(m) \Leftrightarrow h_{pD}=-4.19-(-9)=4.81(m)$

- At E $i_{BC} = i_{BE} = \frac{h_{tB} - h_{tE}}{L_{BE}} = \frac{1 - h_{tE}}{12} = 0.57$

$\Leftrightarrow h_{tE} = -5.92(m), h_{eC} = -12(m) \Leftrightarrow h_{pE} = -5.92 - (-12) = 6.08(m)$

* After pumping: Groundwater level: -9.32m

- At A: $h_{eA} = 1(m), h_{pA} = 0(m), h_A = 1(m)$

- At F: $El. = -9.32m \Leftrightarrow h_{pF} = -9.32 - (-16.6) = 7.28(m)$

$h_{eF} = -16.6(m), h_{tF} = -9.32(m)$

$h_{tA} > h_{tF}$ ⇔ Flowing A to F.

-At B: $h_{eB} = 0(m), h_{tB} = h_{tA} = 1(m) \Leftrightarrow h_{pB} = 1 - 0 = 1(m)$

$i_{BF} = \frac{h_{tB} - h_{tF}}{L_{BF}} = \frac{1 - (-9.32)}{16.6} = 0.62$

-At C: $i_{BC} = i_{BF} = \frac{h_{tB} - h_{tC}}{L_{BC}} = \frac{1 - h_{tC}}{5} = 0.62$

$\Leftrightarrow h_{tC} = -2.11(m), h_{eC} = -5(m) \Leftrightarrow h_{pE} = -2.11 - (-5) = 2.89(m)$ -At D:

$i_{BD} = i_{BF} = \frac{h_{tB} - h_{tD}}{L_{BD}} = \frac{1 - h_{tD}}{9} = 0.62$

$El = -9.32(m) \Leftrightarrow h_{pF} = -4.6 - (-9) = 4.4(m)$

-At E: $i_{BC} = i_{BE} = \frac{h_{tB} - h_{tE}}{L_{BE}} = \frac{1 - h_{tE}}{12} = 0.62$

$h_{tE} = -6.46(m), h_{eC} = -12(m) \Leftrightarrow h_{pE} = -6.46 - (-12) = 5.54(m)$

* Calculation of land subsidence after Riley's theory is computed as:

$$\Delta b = S_s \times b \times \Delta h$$

Drawdown = 0.75m $= \Delta h$

- At F: $\Delta h = h_{before} - h_{after} = -8.57 - (-9.32) = 0.75(m)$

$b_F = 0(m)$

$\Delta b_F = S_s \times b \times \Delta h = (0.0005 \times 0 \times 0.75) \times 100 = 0(cm)$

- At E: $\Delta h = h_{before} - h_{after} = -8.57 - (-9.32) = 0.75(m)$

$b_E = 4.6(m)$

$\Delta b_E = S_s \times b \times \Delta h = (0.0005 \times 4.6 \times 1.29) \times 100 + 0 = 0.17(cm)$

- At D: $\Delta h = h_{before} - h_{after} = -8.57 - (-9.32) = 0.75(m)$

$b_D = 3(m)$

$\Delta b_D = S_s \times b \times \Delta h = (0.0005 \times 3 \times 1.29) \times 100 + 0.17 = 0.29(cm)$

-At C: $\Delta h = h_{before} - h_{after} = -8.57 - (-9.32) = 0.75(m)$

$b_C = 4(m)$

$\Delta b_C = S_s \times b \times \Delta h = (0.0005 \times 4 \times 1.29) \times 100 + 0.29 = 0.44(cm)$

-At B: $\Delta h = h_{before} - h_{after} = -8.57 - (-9.32) = 0.75(m)$

$b_B = 5(m)$

$\Delta b_B = S_s \times b \times \Delta h = (0.0005 \times 5 \times 1.29) \times 100 + 0.44 = 0.62(cm)$

2.Calculation of land subsidence after Poland's theory

Drawdown = 0.75m $= \Delta h$

Saturated unit weight in soil:

$\gamma_{sat} = \frac{(G_s - e_o)\gamma_w}{1+e} = \frac{(2.6+1.51)10}{1+1.51} = 16.375 KN/m^3$

($e = e_0$, no drawdown)

Bouyant unit weight in soil:

$\gamma' = \gamma_{sat} - \gamma_w = 16.375 - 10 = 6.375 KN/m^3$

Effective stress at the middle of the clay layer when no drawdown:

$p_o = 8.57 \times 15.3 + (\frac{16.6}{2} - 8.57) \times 6.375 = 129.4 KN/m^2$

Effective stress at the middle of the clay layer when drawdown:

$p_1 = 9.32 \times 15.3 + (\frac{16.6}{2} - 8.57) \times 6.375 = 136.1 KN/m^2$ Void ratio when the excess pore water pressure:

$\gamma' = \frac{(G_s - 1)\gamma_w}{1+e_1} = \frac{(2.6-1)10}{1+e_1} = 6.375 KN/m^3 \Leftrightarrow e_1 = 1.509$

The coefficient of volume compressibility:

$m_v = \frac{e_o - e_1}{(1+e_o)\Delta p'} = \frac{1.51 - 1.509}{(1+1.51)(136.1 - 129.4)} = 0.00006$

Accordingly, the settlement of ground surface can be derived as below:

$$\Delta z = m_v m \Delta p'$$

At F: $m = 0(m)$

At E: $m = 4.6(m)$

$\Delta z_E = 0.00006 \times 4.6 \times (136.1 - 129.4) \times 100 + 0 = 0.18(cm)$

At D: $m = 3(m)$

$\Delta z_D = 0.00006 \times 3 \times (136.1 - 129.4) \times 100 + 0.18 = 0.30(cm)$

At C: $m = 4(m)$

$\Delta z_C = 0.00006 \times 4 \times (136.1 - 129.4) \times 100 + 0.30 = 0.46(cm)$

At B: $m = 5(m)$

$\Delta z_B = 0.00006 \times 5 \times (136.1 - 129.4) \times 100 + 0.42 = 0.66(cm)$

3.Calculation of land subsidence after Lohman's theory

According to the equation: $\Delta b = \Delta p (\frac{S}{\gamma_w} - n b \beta)$

Drawdown = 0.75m $= \Delta h$

The pressure caused by reducing groundwater table:

$\Delta p = \gamma_w \times \Delta h = 10 \times (-5.25 - (-6)) = 7.5(m)$

Substituting S, n, b, β, into (3), one gets:

-At F: $m = 0(m)$

$\Delta b_F = 7.5 \times (\frac{0.0005 \times 5}{10} - \frac{1.51}{1+1.51} \times 0 \times \frac{1}{2.1 \times 10^6}) = 0(cm)$

-At E: $m = 4.6(m)$

$\Delta b_E = 7.5 \times (\frac{0.0005 \times 5}{10} - \frac{1.51}{1+1.51} \times 4.6 \times \frac{1}{2.1 \times 10^6}) + 0 = 0.17(cm)$ -

At D: $m = 3(m)$

$\Delta b_D = 7.5 \times (\frac{0.0005 \times 5}{10} - \frac{1.51}{1+1.51} \times 3 \times \frac{1}{2.1 \times 10^6}) + 0.17 = 0.28(cm)$ -At C: $m = 4(m)$

$\Delta b_c = 7.5 \times (\frac{0.0005 \times 5}{10} - \frac{1.51}{1+1.51} \times 4 \times \frac{1}{2.1 \times 10^6}) + 0.28 = 0.43(cm)$ -

At E: $m = 5(m)$

$\Delta b_E = 7.5 \times (\frac{0.0005 \times 5}{10} - \frac{1.51}{1+1.51} \times 5 \times \frac{1}{2.1 \times 10^6}) + 0.39 = 0.62(cm)$