VIETNAM JOURNAL OF HYDROMETEOROLOGY

Research Article

The application of the cumulative drawdown method in designing of groundwater lowering system - Golden Hill project

Le Thi Thuy Duong[1]*, Can Thu Van[1]

[1] Ho Chi Minh University of Natural Resources and Environment;
duongltt@hcmunre.edu.vn; ctvan@hcmunre.edu.vn

*Corresponding author: duongltt@hcmunre.edu.vn; Tel.: +84–905777755

Received: 08 September 2021; Accepted: 02 November 2021; Published: 25 December 2021

Abstract: The presence of groundwater can lead to troublesome conditions when construction operations are to take place below the original groundwater level. There are several techniques or methods available for designing of groundwater lowering systems for a construction project. The selection of a technique or techniques appropriate to a particular project at a particular site or country will depend on many factors. The basic designer's "tool kit" – the formulae and concepts used in routine designs – are presented and their application discussed. By the case of Golden Hill project, the author has analyzed the details of the conceptual model, then selected the cumulative drawdown method to design of groundwater lowering system for particular engineering geological – hydrogeological at District 1, Ho Chi Minh City, Viet Nam. The paper does not merely cover the numerical aspects of design, but also discuss some of the issues over which "engineering judgement" must be exercised.

Keywords: Groundwater lowering; Dewatering; Pumped well; Cumulative drawdown method.

1. Introduction

Man has been aware of groundwater since prehistory, long before Biblical times. Over the centuries the mysteries of groundwater have been solved, and man has developed an increasing capability to manipulate it to his will. The control of groundwater is a practical problem, where theory is only part of the picture – how the theory is put into practice is vital. There are several techniques or methods available for controlling groundwater flow for a construction project. The various dewatering techniques are studied [1]. It will be seen from study "Typical applications", that only a few methods are suitable for use in all types of soils. The ranges of soils that are suitable for treatment by the various dewatering methods are shown in the traditional form of particle size distribution curves. These curves are taken from CIRIA Report 113 [2] and are based on the earlier work of Glossop and Skempton (1945) and others [3]. Similarly, the ranges of soils suitable for treatment by the various exclusion methods are shown [4]. The selection of a technique or techniques appropriate to a particular project at a particular site or country will depend on many factors. These are tentative economic and physical limits. The emphasis is on the word tentative.

In Vietnam, the use of underground space is considered an optimal solution in now, so the control of groundwater in the construction is a very important and necessary technical.

Many authors are studied to find out the best solution to controlling groundwater for construction projects. However, the philosophy and the selection of a technique methods for the design of groundwater lowering systems are too sketchy, sometimes illogical to deal simplistic at common situations, while not providing advice on the approach to more complex problems yet [5–6]. The uncertainty inherent in any ground engineering process, requires a "questioning" or "testing" approach be adopted in design, where nothing is taken for granted. Sometimes as work proceeds, the actual soil and groundwater conditions encountered may differ from what was expected and design is fail finally. As a good example at construction of Van Coc culvert, Ha Tay Province: the target drawdown is 11.6m depth, the first designer gave 105 wells with 14.6 m depth, and well spacing is 2.7 m, but after a long time of dewatering pumping, the groundwater is not gotten target drawdown be like expected. So, the contractor had to changed other design. Final output design parameters are 84 wells, 14.5 m depth and well spacing is 3.4 m [7]. It has been costly to the client.

Many engineering projects, especially major ones, entail excavations into water-bearing soils. For all such excavations, appropriate system(s) for the management and control of the groundwater, should be planned before the start of each project. In practice this can only be done with knowledge of the ground and groundwater conditions likely to be encountered by reference to site investigation data. Golden Hill project at 87 Cong Quynh Street, Nguyen Cu Trinh ward, District 1, HCMC. The plan dimensions of the well array will be 93.2 by 77.35 m, 50 floors with 4 basements. The deepest basement is located at 17.5 m depth. The target drawdown is to lower the groundwater level around to 1 m below formation level. This is 18.5m depth, or a drawdown of 14.2 m below the original piezometric level (–4.3 m). The main emphasis of the study is the basic designer's "tool kit" – the formulae and concepts used in routine designs are presented and their application discussed. Methods for estimation of steady-state discharge flow rate, and for selection of well yield and spacing are described in detail. Other design issues (such as time to achieve drawdown) are also outlined. The basic tenets of groundwater modelling are discussed in relation to more complex problems.

Figure 1. Location of project.

2. Materials and methods

2.1. Materials

2.1.1. Geological and hydrogeological of the area data

From the results of engineering geological and hydrogeological investigation to a depth of –50.0 m [8–9], based on the distribution depth, petrographic composition, water capacity, the water storage units can be classified:

a. Clay and sandy clay layers, very poor water

This layer is widely distributed throughout the survey area, it is located just below the fill soil layer, the roof layer is from 0.5–1.2 m depth, the bottom layer is from 7.0–8.0 m depth. The composition includes clay, sandy clay and clay containing laterite gravel, poor water capacity. From the hydrogeological point of view, this is an impermeable layer, and maximum piezometric level in the aquifer is –4.3 m below ground level.

b. Clayey sand, consisting of a sandy gravel

The roof floor is at the depth from 7.0 m to 8.0 m, the bottom floor is at an average depth of 45.0 m. This is the main aquifer in the entire survey area and is distributed throughout the Ho Chi Minh city. There are divided into two layers: The upper layer of clayey sand with average water capacity, the depth of the bottom layer is 26.0 m and the layer of sand with gravel lying next, with rich water storage. Analysis of field permeability test data gave an aquifer permeability k of 2.25×10^{-4} m/s (19.4 m/d) at 15 depth and a storage coefficient S of 0.005 [10–15]. This is a confined aquifer with a pressure height of 41.7 m.

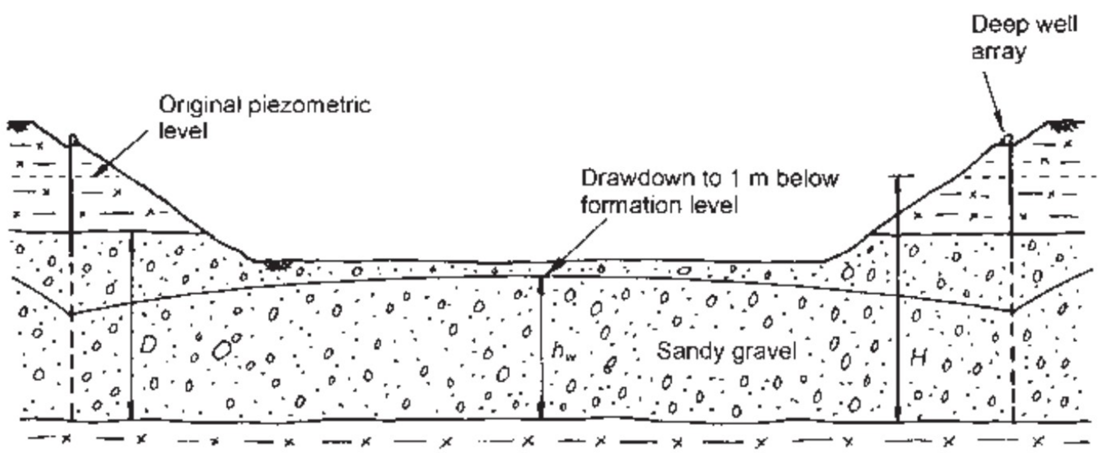

Figure 2. Conceptual model.

2.2. Methodology

2.2.1. Selection of method

The excavation extends through the stiff clay aquiclude (8 m) and into the upper 10.5 meters of the confined aquifer. The piezometric level in the confined aquifer will need to be lowered prior to excavation to prevent base heave during excavation through the clay, and then to provide a workable excavation when the excavation penetrates into the top of the aquifer.

The target drawdown is to lower the groundwater level to 1.0 m below formation level [16]. This is 18.5 m depth (B4), or a drawdown of around 14.2 m below the original piezometric level (4.3 m depth) [9–10].

For a drawdown of 14.2 m and the design permeability of 2.25×10^{-4} m/s, inspection of Figure 3 suggests that deep wells method would be suitable for this combination of drawdown and permeability [1–4]. In addition, in this case the contractor wishes to excavate rapidly to full depth [10].

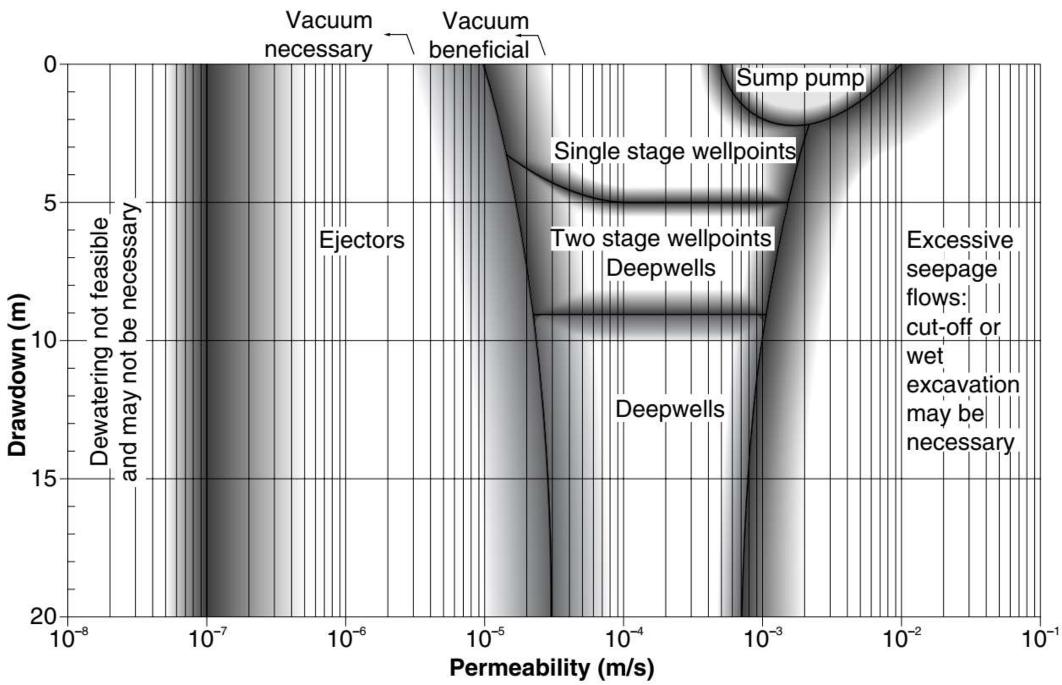

Figure 3. Range of application of pumped well groundwater control techniques – adapted from Rorberts and Preene (1994) and modified after Cashman (1994).

2.2.2. Estimation of steady-state discharge flow rate and estimation of number of wells

The cumulative drawdown method (using the Cooper–Jacob simplification) can be used in confined aquifers [16]. This method takes the advantage of the mathematical property of superposition applied to drawdowns in confined aquifers. In essence, the total (or cumulative) drawdown at a given point in the aquifer, resulting from the action of several pumped wells, is obtained by adding together (or superimposing) the drawdown from each well taken individually (Figure 4).

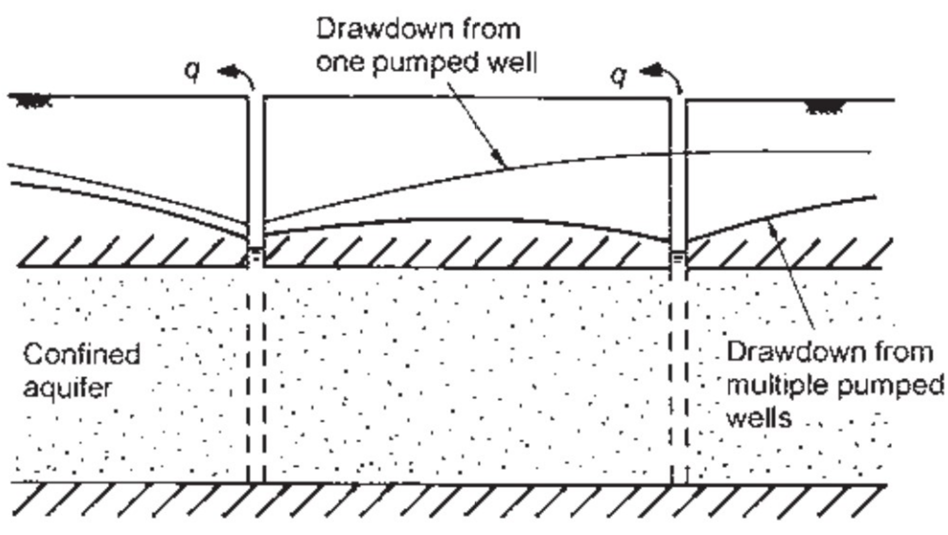

Figure 4. Superposition of drawdown from multiple wells.

This approach is theoretically correct in confined aquifers, but is invalid in unconfined aquifers where the changes in saturated thickness that occur during drawdown complicate the interaction of drawdowns. It has also been successfully applied in unconfined aquifers where the final drawdown is less than around 30 percent of the initial saturated aquifer thickness [16–17].

In this case, because drawdown is required to 18.5 m depth compared with the top of the aquifer at 8 m depth, the initially confined aquifer will become unconfined. The aquifer thickness will be reduced by 10.5 m out of 37 m, or 28 per cent. Therefore, this problem will be analyzed assuming confined behavior throughout. The cumulative drawdown is calculated using equation [17–20].

$$(H - h_w) = \sum_{i=1}^{n} (H - h_w)_i$$

$$(H - h_w) = \sum_{i=1}^{n} \frac{q_i}{4\pi kD}\left\{-0.5772 - \ln\left[\frac{r_i^2 S}{4kDt}\right]\right\} \quad (1)$$

where $(H–h_w)$ is the cumulative drawdown (at the point under consideration) resulting from n wells each pumped at constant flow rate q_i; k is the aquifer permeability: k is taken as $2.25.10^{-4}$ m/s; S is the aquifer storage coefficient: S is taken as 0.005; D is the original aquifer saturated thickness: D = 45–8 = 37m; t is the time since pumping began. In this case the target drawdown is required within fourteen days. It is always prudent to design to obtain the drawdown a little quicker than planned – this allows for minor problems during commissioning. In design, we will aim to achieve the target drawdown within ten days. T = 10×86.400 seconds will be used in calculations; r_i is the distance from each pumped well to the point where drawdown is being estimated.

The method requires that the plan layout of the well array be sketched, and the x–y co-ordinates of each well be determined [17–18]. The co-ordinates then allow the radial distances r_i (from each well to the point where drawdown is being checked) to be calculated. An initial guess is made of the number of wells and well spacing and the resulting x–y co-ordinates determined. In this case the initial guess was twenty-two wells evenly spaced at 15 m centres (Figure 5).

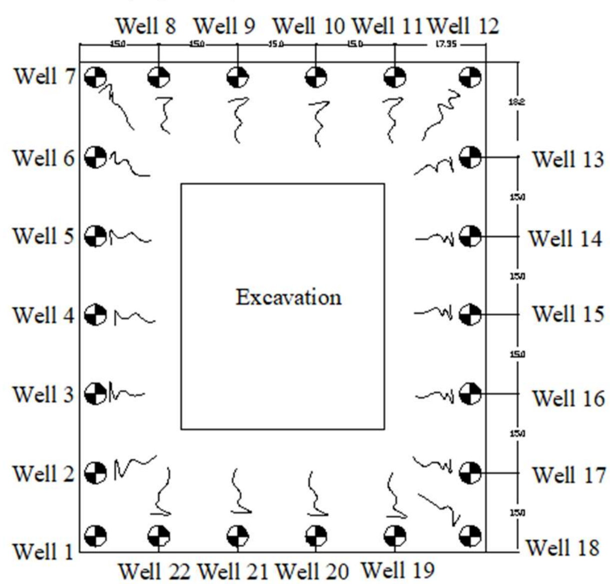

Figure 5. Schematic plan of twenty-two wells system (with xc=39, yc=47).

A spreadsheet program is then used to evaluate equation (1) for the cumulative drawdown at selected locations within the excavation [16–21]. For circular or rectangular excavations with evenly–spaced wells it is normally sufficient to determine the drawdown in the center of the excavation, because drawdown everywhere else will be greater. This is the method used here. If the well array is irregular in shape (or if the depth of excavation is not constant) it will be necessary to determine the drawdown in a number of locations, to ensure the target drawdown is achieved at all critical locations [17–18].

The results from a spreadsheet calculating the drawdown in the center of the excavation for a twenty-two well system is shown below. The radial distance r_i, from each well (at location x_i, y_i) to the location (x_c, y_c) where the drawdown is being determined [18], is calculated from:

$$r_i = \sqrt{([x_i - x_c]^2 + [y_i - y_c]^2)} \tag{2}$$

For simplicity, the flow rate q_i from each well has been assumed to be the same, but if it was intended to use pumps of different sizes in certain wells this can easily be incorporated in the calculation. In the spreadsheet different values of qi were tried until the target drawdown of 14.2 m is just achieved in the center of the excavation. The total flow rate is simply the sum of all the well flow rates.

Table 1. The total flow rate is simply the sum of all the well rates flow.

Well	x_i (m)	y_i (m)	q_i (m/s)	r_i (m)	$H-h_w$ (m)
1	0	0	8.10^{-4}	60.56	0.62
2	0	15	8.10^{-4}	49.94	0.64
3	0	30	8.10^{-4}	42.09	0.66
4	0	45	8.10^{-4}	38.71	0.67
5	0	60	8.10^{-4}	40.93	0.67
6	0	75	8.10^{-4}	47.98	0.64
7	0	93.2	8.10^{-4}	60.56	0.62
8	15	93.2	8.10^{-4}	52.27	0.64
9	30	93.2	8.10^{-4}	47.40	0.64
10	45	93.2	8.10^{-4}	47.03	0.65
11	60	93.2	8.10^{-4}	51.25	0.64
12	77.35	93.2	8.10^{-4}	60.56	0.62
13	77.35	75	8.10^{-4}	47.98	0.64
14	77.35	60	8.10^{-4}	40.93	0.68
15	77.35	45	8.10^{-4}	38.71	0.68
16	77.35	30	8.10^{-4}	42.09	0.67
17	77.35	15	8.10^{-4}	49.94	0.64
18	77.35	0	8.10^{-4}	60.56	0.62
19	60	0	8.10^{-4}	51.25	0.64
20	45	0	8.10^{-4}	47.03	0.65
21	30	0	8.10^{-4}	47.40	0.64
22	15	0	8.10^{-4}	52.27	0.64
Total			***0.176***		***14.21***

This calculation indicates that a system of twenty-two wells, each discharging 8.10^{-4} m/s (total flow rate 0.176 m/s) will achieve the target drawdown in the center of the excavation after ten days. During the pumping test could have yielded more if a larger pump has been used. It therefore makes sense to repeat the above calculations assuming fewer wells of greater discharge rate [18–22]. The results of these calculations are summarized below:

Table 2. The calculations assuming fewer wells of greater discharge rate.

No of Wells	Well spacing (m)	Well flow of rate (l/s)	Drawdown in center of excavation (m)	Total flow rate (l/s)
22	15	8	14.21	176
18	20	10	14.41	180
14	25	13	14.51	182

It is apparent that the target drawdown can be achieved by various combinations of well numbers and yields, but that the total flow rate remains approximately constant.

3. Results and discussion

The number and yield of wells chosen for the final design will depend on a number of factors, including:

The need for redundancy in a well system. Any system relying on relatively few wells is vulnerable to one or two wells suffering from damage or pump failure, leading to loss of drawdown and flooding or instability of the excavation. A system consisting of a greater number of wells will lose proportionately less drawdown if one or two wells are lost.

Each well must be able to yield the discharge flow rate qi assumed in design. However, in practice, problems can occur if the dewatering wells are not designed, installed and developed in exactly the same way as the test well – this may cause the production wells to have lower yields than the test wells. Some wells can have high yields and yet others, poorly connected into fissures, may be almost “dry”. Thus, drawdown method needs to be applied with care.

In this case it is assumed that, due to the availability of pumps of suitable capacity, the nominal system of eighteen wells, each discharging 10 l/s each will be adopted (Table 2). It is normal practice to apply an empirical superposition factor J of 0.8–0.95; the system capacity is increased by a factor of 1/J ($Q=\frac{1}{J}\sum_{i=1}^{n} q_i$) [19, 21, 22]. This empirical factor allows for interference between wells, and also provides some allowance for additional drawdown around the wells and water released from storage when the aquifer becomes unconfined. Where aquifers become unconfined, and drawdowns are small (less than 30 per cent of the initial saturated aquifer thickness), the empirical superposition factor J is normally taken as 0.8 to 0.95. In this case, because the drawdown will reduce the thickness of the aquifer by almost 30 per cent, the maximum superposition factor of 0.8 will be applied, so the system capacity (and hence the number of wells) will need to be increased by 1/0.8 = 1.25.

The final system design is, therefore for twenty-two wells (18×1.25 = 22.5), of 10 l/s capacity each. Total system capacity is 220 l/s. The pump manufacturer’s catalogue will list the minimum internal diameter of well screen necessary to accommodate the pump to be used, assuming the wells are perfectly straight and plumb. In practice, most wells deviate from the ideal alignment, and using a slightly larger screen diameter reduces the risk of a pump getting stuck down a well [19, 21, 22]. Some general guidance on well screen diameters is given in Table 3. The recommended minimum well screen diameters are generally larger than those quoted by the pump manufacturers. Even so, if a well has a large amount of deviation, even a very small pump may become jammed at the tight points in the well. Table 3 indicates that, to accommodate a pump of suitable capacity, a minimum well bore diameter of 300 mm is required. The corresponding well screen and liner diameter is 152 mm.

Table 3. Recommended well screen and casing diameters.

Maximum submersible pump discharge rate (l/s)	Recommended minimum internal diameter of well screen and casing[a] (mm)	Recommended minimum internal diameter of boring[b] (l/s)
5	125-152	250-275
10	152-203	300-325
15	165-250	300-375
20	180-250	300-375
25	203-300	325-425
44	250-350	375-475

Notes: [a] Diameter will depend on external dimensions of pump used. [b] Minimum diameter of boring in based on nominal filter pack thickness of 50 mm. Slightly smaller diameters may be feasible if natural filter can be developing in the aquifer.

4. Conclusion

Solution drawdown in the center of the excavation to dry the foundation pit by cumulative drawdown analysis from the well system arranged around the excavation is a superior solution because it has a logical theoretical approach, ensuring active lowering lower the groundwater level to the required depth, creating reverse seepage gradient overcomes the phenomenon underground erosion, flowing sand destabilizes the roof of the hole foundation, overcome the phenomenon of background flare, does not interfere with the construction of the foundation pit. For a drawdown of 14.2 m below the original piezometric level, the final system design (the methods used in combined theoretical and empirical approaches) is twenty-two wells (18×1.25 = 22.5), of 10 l/s capacity each. Total system capacity is 220 l/s, a minimum well bore diameter of 300 mm is required. The corresponding well screen and liner diameter is 152 mm.

The best design approaches incorporate elements of both the theoretical and empirical methods. The theoretical method requires a "conceptual model" of the ground and groundwater regime to be developed, following which calculations are carried out. Simple and fairly basic calculations are perfectly acceptable, and may be preferred in many cases, provided they are compared with an empirical approach. The empirical method should be used as a "sanity check" to ensure that the proposed groundwater lowering system is realistic and practicable. Any groundwater lowering system will need, to some degree, monitoring and maintenance measures to ensure effective operation. Once in operation, a groundwater lowering system is the end result of a lot of effort by a lot of people. It is a complex system dependent on a diverse range of hydrogeological, hydraulic, chemical, mechanical and human factors, but it will have a clear aim to lower groundwater levels sufficiently to allow the construction works to proceed. So that is the need for monitoring to measure the target drawdown by pumping groundwater to check back design.

Author contribution statement: D.L.T.T. devised the project, the main conceptual ideas and designed of groundwater lowering system. V.C.T. wrote the manuscript. Both authors contributed to the final version of the manuscript.

Acknowledgements: The authors would also like to thank Bach Khoa Ho Chi Minh City Science Technology Joint Stock Company (BKTECHS), Solar system foundation & Geotechnics J.S.C for providing documentation to complete the study.

Competing interest statement: The authors declare no conflict of interest.

References

1. Preene, M; Roberts, T.O.L.; Powrie, W.; Dyer, M.R. Groundwater Control – Design and Practice. *CIRIA Report C515*, London, 2000.

2. Somerville, S.H. Control of Groundwater for Temporary Works. *CIRIA Report 113*, London, 1986.
3. Glossop, R.; Skempton, A.W. Particle-size in silts and sands. *J. Inst. Eng*. **1945**, *25*, 81–105.
4. Roberts, T.O.L.; Preene, M. Range of application of construction dewatering systems. Groundwater Problems in Urban Areas (Wilkinson, W.B. Eds.). Thomas Telford, London, **1994**, pp. 415–423.
5. Anh, T.H. Research on solutions to lower groundwater in the construction of deep excavation pits in Hai Phong. Master's thesis in civil and industrial construction engineering, 2007.
6. Ba, K.N. Deep Foundation Design and Construction. Ha noi constuction publishing company, **2010**, pp. 576. (In Vietnamese)
7. Anh, L.V. Method of calculation for the ground water falling by well system. Journal of Water Resources & Evironmental Engineering. **2016**, *1*, 33-37. (In Vietnamese).
8. Research center of technology and industrial equipment (Rectie). Soil investigation report for Golden Hill project– 87 Cong Quynh, Nguyen Cu Trinh ward , Dict.1, HCMC.
9. Research center of technology and industrial equipment (Rectie). Report on hydrographic geologic investigation result to serve design lower water level borehole system during basement foundation working for Golden Hill project– 87 Cong Quynh, Nguyen Cu Trinh ward , Dict.1, HCMC.
10. Solar system foundation & Geotechnics J.S.C. Dewatering Proposal for Golden Hill project– 87 Cong Quynh, Nguyen Cu Trinh ward , Dict.1, HCMC.
11. Van, P.L.; Minh, T.T.; Van, T.T. Determination of hydro-geological parameters by the pumping test method at Tra Noc industrial zone - Can Tho city: A preliminary result. *J. Sci. Can Tho University Environ. Clim. Change.* **2017**, *1*, 31–38.
12. Ty, T.V.; Minh, H.V.T.; Ngan, L.H.B.; Nhan, D.T.; Luan, T.C. Pumping test for determinating hydrogeological parameter for goundwater flow simulation in Can Tho city, Vietnam. Vietgeo Proceedings, **2019**, 433–438.
13. Viet, K.N.; Van, N.D. Practical guide to hydrogeology. HCMC National University, **2013**, pp. 166.
14. TCVN 9903:2014 – Hydraulic structures - Requirements for design, construction and acceptance of decreasing groundwater level, 2014.
15. Viet, V.T. Geotechnical engineer's Handbook. **2010**, 409–438. (In Vietnamese)
16. Cooper, H.H.; Jacob, C.E. A generalised graphical method for evaluating formation constants and summarising well field history. *Trans. Am. Geophys. Union* **1946**, *27*, 526–534.
17. Powrie, W.; Preene, M. Equivalent well analysis of construction dewatering systems. *Géotechnique* **1992**, *42(4)*, 635–639.
18. Cashman P.M.; Preene, M. A practical guide Groundwater Lowering in Construction. Spon Press 2001, pp. 476.
19. Driscoll, F.G. Groundwater and Wells. Johnson Division. Saint Paul, Minnesota, 1986.
20. Powrie, W.; Preene, M. Time-drawdown behaviour of construction dewatering systems in fine soils. *Géotechnique.* **1994**, *44(1)*, 83–100
21. Powers, J.P. Construction Dewatering: New Methods and Applications, 2^{nd} edition. Wiley, New York, 1992. p. 11.
22. Chapman, T.G. Groundwater flow to trenches and wellpoints. *J. Inst. Eng.* **1959**, 275–280.