A STUDY ON PERFORMANCE EVALUATION OF RELIEF WELLS DOWNSTREAM OF EMBANKMENT DAMS

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ABSTRACT

A STUDY ON PERFORMANCE EVALUATION OF RELIEF WELLS DOWNSTREAM OF EMBANKMENT DAMS

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This study investigates the performances of reduction methods of the seepage problem at the foundation of a real embankment dam, Oğulcuk Dam located in Yozgat, Turkey. The steady-state seepage modeling is used in the analysis. The variation of relief wells inner diameter, location and spacing parameters are compared with each other and are also compared with different alternatives, such as cutoff walls, core trenches, and cutoff walls with blankets. Different variations of those methods i.e. locations and depths are also discussed and compared with each other and relief wells. Performances of relief wells are also compared in terms of cost. In general, it is presented that between the compared reduction methods of seepage, relief wells has a better performance and can be used as a remedial measure to protect the alluvial foundation and toe of embankment dams from the hazardous effects of foundation seepage.

Keywords: Embankment Dams, Foundation Seepage, Relief Wells, SEEP/W

ÖZ

DOLGU BARAJLARIN MANSABINDAKİ DÜŞÜRME KUYULARININ PERFORMANSLARININ DEĞERLENDİRİLMESİ ÜZERİNE BİR ÇALIŞMA

Göğüş, Doruk
Yüksek Lisans, İnşaat Mühendisliği
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Bu çalışmada, Yozgat, Türkiye'de bulunan Oğulcuk Barajının temelindeki sızıntı problemini önleme yöntemlerinin performansları araştırılmıştır. Analizde kararlı durum sızıntı modellemesi kullanılmıştır. Düşürme kuyusu çapının, yerinin ve aralık parametrelerinin etkisi birbiriyle karşılaştırılmış ve ayrıca kesme duvarları, çekirdek hendekleri ve örtülü kesme duvarları ile karşılaştırılmıştır. Bu kesme yöntemlerinin farklı varyasyonları da tartışılmış, birbirleriyle ve düşürme kuyularıyla performansları karşılaştırılmıştır. Düşürme kuyularının performansları maliyet açısından da değerlendirilmiştir. Genel olarak, karşılaştırılan sızıntı önleme yöntemleri arasında, sızıntı önleme açısından, düşürme kuyularının daha iyi bir performansa sahip olduğu ve sızıntıyı önlemek ve barajları temel sızıntısının tehlikeli etkilerinden korumak için kullanılabileceği anlaşılmıştır.

Anahtar Kelimeler: Dolgu Barajlar, Temel Sızıntısı, Düşürme Kuyusu, SEEP/W

To my family…

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I wish to express my deepest gratitude and respect to my supervisor Prof. Dr. A. Melih Yanmaz for his guidance, valuable advices, criticism and insight since the first day of the research. His knowledge and enlightening comments helped me whenever I needed and I am sincerely grateful to him for this.

I would also like to thank my co-supervisor Assoc. Prof. Dr. Melih Çalamak for his suggestions, comments and support. With his periodic meetings and encouragements, this thesis is completed in a scheduled time. His guidance had a key role for me to finish my thesis.

In closing, I would like to strongly express my eternal gratitude to Prof. Dr. Yanmaz and Dr. Çalamak. Without their inspiring guidance, I would not have been able to be where I am today in terms of my M.Sc. studies.

Last but not least, my family has always been the main reason for me to pursue my dreams, and no amount of thanks will ever make up for the fact that they have always been there for me.

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LIST OF ABBREVIATIONS

ASDSO	Association of State Dam Safety Officials
DSİ	Devlet Su İşleri
EPDK	Enerji Piyasası Düzenleme Kurumu
FEMA	Federal Emergency Management Agency
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation

LIST OF SYMBOLS

A	Summation over an area of an element
B	Gradient matrix
C	Hydraulic conductivity matrix
H	Total head
h	Pressure head
k	Hydraulic conductivity
K_b	Permeability coefficient of blanket material
K_f	Permeability coefficient of foundation material
k_s	Saturated hydraulic conductivity
k_x	Hydraulic conductivity in x-direction
k_y	Hydraulic conductivity in y-direction
i	Gradient of the total hydraulic head
L	Summation over the edge of an element
m	Curve fitting parameter
N	Vector of interpolating function
n	Curve fitting parameter
Q	Applied boundary flux
q	Specific discharge
S_p	Slope at P
t	Time

X_r Length of upstream clay blanket

Z_b Thickness of upstream horizontal blanket

Z_f Thickness of foundation

α Curve fitting parameter

θ Volumetric water content

θ_s Saturated water content

θ_r Residual water content

λ Storage term for a transient seepage

τ Thickness of the element

Ψ Required suction range

CHAPTER 1

INTRODUCTION

1.1 General

Humans have been building dams to store water for irrigation, municipal use, hydroelectric power, flood protection and navigation (Aşık, 2016). Since the human being started living in larger groups and mass producing agricultural products, they needed to store water to use it when it is not available. It is predicted that the first dam was built on River Nile around 4000 B.C. and was used for irrigation and drinking (Özgen and Karadoğan, 2013). There are various types of dams and selection of the type depends on various factors, such as topography, geology and geotechnical conditions of the dam site, and, availability of the construction material, environmental and public concerns, flooding possibility during construction, available time for construction, availability of personnel with desired skills, seismicity, the function of the reservoir, safety, standards of the country, the discharge of the river, water level difference between upstream and downstream and the effects of uplift force under the dam (Emiroğlu, 2008).

Dams can be classified in different categories based on the size, material or the purpose. Based on the type and material used for construction, they can be divided into four main group as, gravity dams, arch dams, buttress dams and embankment dams (Yanmaz, 2018). Those main groups can be divided into sub-parts. Gravity dams are divided as concrete gravity dams, prestressed concrete gravity dams, roller compacted concrete gravity dams and hardfill dams. Arch dams are categorized as constant-angle arch dams, constant-center arch dams and variable angle, variable-center arch dams. Buttress dams are divided as flat-slab buttress dams and multiple-arch buttress dams; whereas embankment dams are divided into two main categories and several sub-categories. Two main categories are earth-fill dams, which is divided into three as simple, zoned, and diaphram type embankments, and rock-fill dams which is divided into two as impermeable-face and impermeable-earth core dams (Yanmaz, 2018).

1.2 Seepage Through Foundations of Embankment Dams

Embankment dams are made of soils which are porous and susceptible to seepage when in contact with water. The seepage can take place both through and underneath the dam. The seepage at the foundation is a problem that is faced from time to time under the dams. When it is not controlled, seepage at the foundation may result in excessive hydrostatic pressures beneath an impervious top stratum on the landside, sand boils, and piping under the structure (Guyer, 2018). Piping and boiling effects are shown in Figure 1.1. Piping occurs when the water seeps through the structure or the foundation and starts to take the soil particles with it. Progressive carrying of these soil particles result in empty spaces underneath the structure. It initiates from the downstream and occurs as a cavity or a shape similar to a pipe (USBR, 2018). When this piping effect reaches upstream, a disaster may occur (ASDSO, 2018).

According to Empson (2013), 11 dam failures and 43 dam incidents happened due to foundation piping. Seepage at the foundation may result in piping or dissolve of the rock formation.

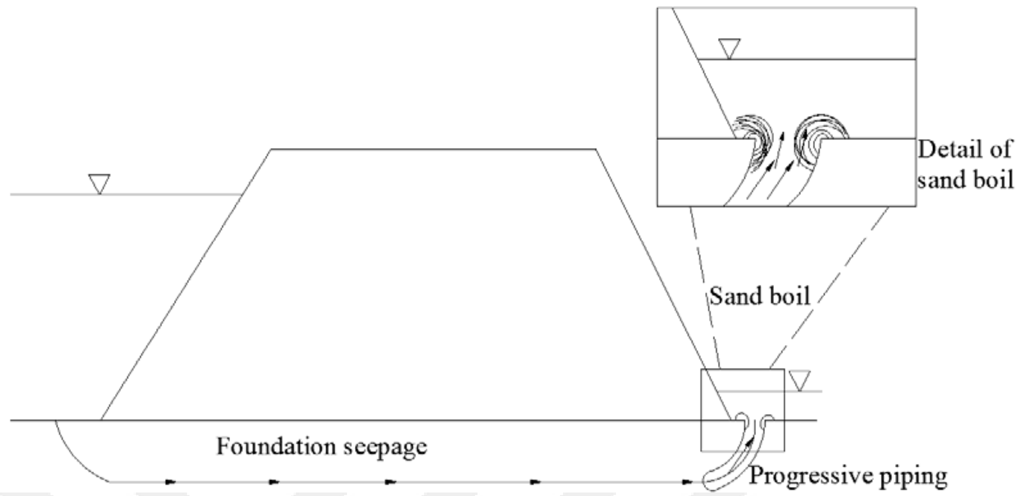

Figure 1.1. Illustration for piping and sand boiling (Garg, 2005)

Main prevention methods for seepage from the foundation are, cutoff trenches, riverside impervious blankets, landside seepage berms, pervious toe trenches and pressure relief wells (Guyer, 2018). Figure 1.2 shows the effect of the relief wells.

According to USBR (2014), a pressure relief well can be specified as a permanent or a temporary deep well specifically designed to handle and, if necessary, reduce artesian pressures below the construction excavation. However, it is not fundamentally designed to dewater the artesian aquifer, but to relieve artesian pressures and reduce hydraulic uplift pressures, which could inevitably prompt localized or widespread heaving of the floor of the excavation. A number of relief wells with certain spacing in a row would be effective in reducing this uplift pressure (Figure 1.3).

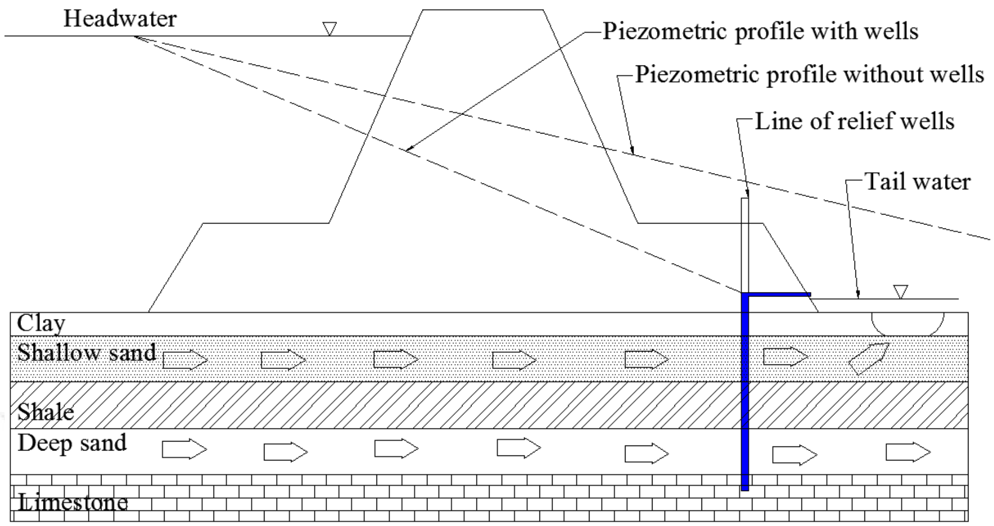

Figure 1.2. Effect of pressure relief well (Easton and Springer, 2016)

Embankment dams often require rehabilitation because of the damage they got naturally or by misusage. Some of the rehabilitations needed by those dams are due to seepage occurring at their foundation or due to the decrease of the effectiveness of the relief well networks (Guy et al., 2006). Relief wells may be installed during either the construction or rehabilitation.

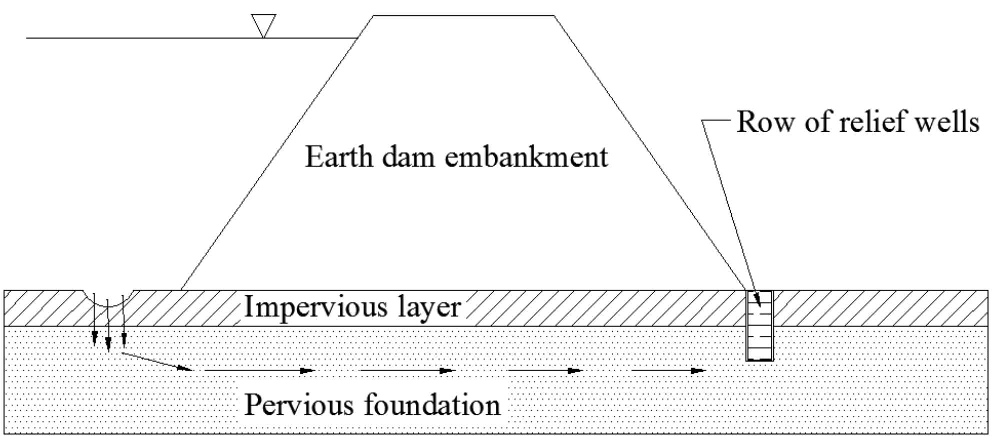

Figure 1.3. Relief wells (Garg, 2005)

1.3 Previous Works on Relief Wells

The problems related to the seepage through foundations of the embankments have been addressed by some researchers. Millmore and McNicol (1983) dealt with the Kielder Dam in Great Britain in all the aspects from investigation to performance. The design of the dam includes clay blankets and relief wells to keep the seepage under control. They used 45 cm diameter wells in the project. The project includes nine relief wells. After the completion of the dam, it was monitored that the seepage under the dam is less than expected. It is clearly highlighted that the relief wells are successful in keeping the seepage under control.

Guy et al., (2010) focus on Herbert Hoover Dike in South Central Florida. This structure is chosen because a modification was required to reduce the seepage induced instability risk. They developed an algorithm for relief well system design in accordance with USACE (1992) design guidance. The analysis conducted consist of the variation of well line positions, well losses and different levels of the tail water. The research also includes system design of a fully-penetrating well that goes into the soil until the saturated depth of the aquifer. The results show that the dike needed additional structures as a seepage barrier, a berm, combined with a system of relief wells together to solve the seepage problem.

Salmasi et al. (2015), study the effect of sizes of the relief wells and the distances in between them numerically. In the study, researchers use a finite element software, SEEP/W. They present dimensionless graphs representing the effectiveness of use of these relief wells. They conclude that when the number of relief wells increased the uplift pressures at the toe of the dam decreased. It is also highlighted that with the decrease of distance between wells the uplift pressure decreased.

Heidaezadeh et al., (2015) investigate the seepage related problems in Karkheh Dam in Iran. This dam had a serious problem about the installation of the relief wells. The artesian flow conditions needed extra attention in order to open wells safely. The dam has a height of 127 meters which made it more critical in terms of the tasks safety. The first group of relief wells were installed to the dam during the construction. Because of the decreased safety factors due to high pressures, it was planned to install new wells several years after it started to operation. After all the calculations and installations, the safety factor for uplift pressure at the downstream toe was increased to 1.3 with the help of the additional relief wells. They concluded that the main issue with the old relief wells were, insufficient number of wells and diameters with irregular well screens.

Salmasi et al., (2020-a) conducted a study with an embankment dam about relief wells by using SEEP/W software. In this study, the upstream water depth is taken as 6 meters. The effects of relief wells on uplift pressures were discussed. The results of the relocation of the relief wells to different positions and distances are analyzed and discussed.

1.4 Scope of The Study

This study examines the effects of the distance between relief wells, the sizes of relief wells and the costs of opening relief wells including also operation and maintenance costs. A comparison of performances of relief wells, cutoff walls and trenches is carried out. To the best of author's knowledge, there is no study reported in the literature, on combination of performance and cost analysis handled together. Combining the well performances with the average costs of the wells would enable a designer for cost-effective decision-making. This way, it becomes clearer to understand how the well sizes, number of wells, and distances from toe and from each other affect their performance and their operation and maintenance, and also the initial cost. In the scope of the study, seepage analysis is conducted with the finite element method software, SEEP/W. Oğulcuk Dam in Boğazlıyan, Yozgat, Turkey, being under construction, is considered for the analyses.

The aim is to investigate the effects of relief well parameters and specifications on the seepage at the dam foundation. The performance of the relief wells on decreasing the pore pressures and the seepage is also studied. The performances of the wells are compared with concrete cutoff wall and core trench applications. These applications are selected as partially penetrating cutoffs as fully penetrating cutoffs are out of scope of this study. Performances of different cutoff and trench methods are compared with pressure relief wells. Then cost analysis for the wells are conducted to find technically and economically the best option for the dam. The cost analysis include the construction of the wells with the renewal times and cost of operation of those wells. By using the findings of the cost analysis section of the study, one can easily estimate the cost of well drilling and operation for a similar sized dam.

A very long homogeneous dam (Oğulcuk Dam, Turkey), resting on an alluvial foundation has been specifically considered in this study to investigate the feasibility of various remedial measures to be taken against excessive seepage through problematic foundation conditions. In this scope, although construction of a clay core extending until the impermeable layer and use of cutoff walls with different configurations have been considered as some remedial measures, they are accepted to be inapplicable to the site-specific conditions since huge amount of clay material is not available in close proximity to the construction site. Considering cost-effective solution, implementation of a relief well system to the downstream of the dam is studied in detail. Hence the present study is specifically focused on determination of a proper configuration of spacing and location of relief wells.

CHAPTER 2

METHODS USED

The methods and tools used in this study are presented and discussed briefly in this chapter.

2.1 The Method and The Tool Used for Seepage Modeling

In this research, the finite-element method is used to model the seepage through the foundation of the dam. This method divides a region into pieces, calculates the desired parameter, and reassembles those elements to obtain the main results (Jagota et al., 2013). Since no particular difficulties are experienced when the physical domain has a complex geometry, finite-element method is generally preferred as it also has the benefit of being able to be written in the form of formulations with computer programming (Erhunmwun and Ikponmwosa, 2017).

GeoStudio SEEP/W (SEEP/W, 2012) software released by Geo-Slope Int. Ltd. is used in this study to simulate the flow through the foundation of the dam by finite element method. The program suits to the necessities of this thesis with its powerful calculating capacities about the seepage rate. This software has been widely used in similar previous research which were conducted by Çalamak et al., 2017; Çalamak et al., 2018; Çalamak and Yanmaz, 2017; Çalamak and Yanmaz, 2018; Salmasi et al., 2020-a; Salmasi et al, 2020-b; Salmasi et al., 2015; and Jaeger et al., 2017.

The SEEP/W software lets users to input the data for soil regions and material models, geometry lines depending on geometry, boundary conditions and borders of the areas of conditions, mesh size, seepage rate, and functions related to those variables. The inputted data are used for the desired calculations and the software presents the results to the user. The software is used to model injection wells, dewatering designs, hydraulic response of a dam to water level fluctuations and many more.

The finite element software of SEEP/W is used to obtain pore water pressures, total heads, and seepage rates. This software uses Darcy's Law for both saturated and unsaturated soils to obtain the results for complicated calculations related to seepage. Darcy's Law is given as (USBR, 1987);

$$q = -ki \tag{2.1}$$

In Equation 2.1, q represents specific discharge (volumetric flow rate of water per unit area), k is hydraulic conductivity and i is the gradient of total hydraulic head.

The two-dimensional seepage can be expressed as (Richards, 1931);

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial H}{\partial y}\right) + Q = \frac{\partial \theta}{\partial t} \tag{2.2}$$

In Equation 2.2, H is the total head, k_x and k_y are the hydraulic conductivities in x and y directions, respectively, Q is the applied boundary flux, θ is the volumetric water content, and t is the time. This equation determines the flow entering and leaving the unit volume of a soil in that time interval which means the rate of change of the volumetric water content with respect to time. Under steady-state conditions, the right side of Equation 2.2 becomes zero because the entering and the leaving water volumes are equal to each other. This two-dimensional seepage equation can be numerically solved using the finite element method by Galerkin approach. When this approach is applied, the resultant equation given below is obtained (Geo-Slope Int. Ltd., 2012).

$$\tau \int_A ([B]^T [C][B]) dA \, \{H\} + \tau \int_A (\lambda < N >^T \; < N >) dA \, \{H\}, t = q\tau \int_L (< N >^T) dL \tag{2.3}$$

In Equation 2.3, $[B]$ is the gradient matrix, $[C]$ is the element hydraulic conductivity matrix, $\{H\}$ is the vector of nodal heads, $<N>$ is the vector of interpolating function, q is the unit flux, τ is the thickness of the element, t is time as a component, λ is the storage term for a transient seepage, A is the designation for summation over the area of an element and L is a designation for summation over the edge of an element. This is the abbreviated finite element form of the equation of the Darcy's Law.

Equation 2.3 is the general equation for a transient seepage analysis and the terms of "$\tau \int_A (\lambda < N >^T \ < N >) dA \ \{H\}, t$" are removed as the head is not a time dependent function for steady-state analysis.

The software has also some limitations. It is able to simulate the flows that follow Darcy's Law; it cannot model the evaporation leaving a system. Detailed information about the software computation methods may be reachable at Seepage Modeling with SEEP/W (Geo-Slope International Ltd., 2012).

2.2 Basics of Modelling

The material for the foundation of the dam is chosen to be modeled as a saturated soil model only. This type of model is used to describe the soils that stay below the phreatic surface. For other locations, such as dam body regions, saturated/unsaturated model is used in order to prevent unrealistic seepage estimations. This type of modeling includes hydraulic conductivity function and volumetric water content functions for the materials.

Estimation of hydraulic conductivity function for different types of soils/materials needs a realistic approach. This software has two methods for estimating hydraulic conductivity function, which are Fredlung and Xing and Van Genuchten methods. This study used the Van Genuchten method for modeling the side view analyses, cutoff walls and trench analyses. Van Genuchten method is described as follows (Van Genuchten, 1980).

$$k_w = k_s \frac{\left(1-(a\Psi^{n-1})*(1+(a\Psi^n)^{-m})\right)^2}{((1+a\Psi)^n)^{\frac{m}{2}}} \tag{2.4}$$

In Equation 2.4, k_s is saturated hydraulic conductivity, *a, n, m* are curve fitting parameters, *n* is equal to the *1/(1-m)* and Ψ is the required suction range. The *a, n* and *m* can be estimated graphically when volumetric water content function is known for a soil. Van Genuchten (1980) claims that, the best location for estimating the curve fitting parameters halfway between residual water content and saturated water content of the slope function, which is given in Equation 2.5.

$$S_p = \frac{1}{\theta_s-\theta_r}\left|\frac{d\theta_p}{d(log\Psi_p)}\right| \tag{2.5}$$

In Equation 2.5, θ_s is saturated water content and θ_r is residual water content. The P letter indicates the point at the halfway between θ_s and θ_r.

When S_p is calculated, *m* can be found as it can be expressed as a function of S_p. The following equation is used to determine the curve fitting parameter *m.*

$$m = \begin{cases} 1-\exp(-0.8S_p) & 0 < S_p \leq 1 \\ 1-\frac{0.5755}{S_p}+\frac{0.1}{S_p^2}+\frac{0.025}{S_p^3} & S_p > 1 \end{cases} \tag{2.6}$$

The curve fitting parameter *a* is estimated from;

$$a = \frac{1}{h}\left(\theta^{-\frac{1}{m}}-1\right)^{\frac{1}{n}} \tag{2.7}$$

where h is pressure head. The given method is proposed by Van Genuchten (1980) and it is still used by this software to solve saturated/unsaturated model.

2.3 Modelling with GeoStudio

In this study, all analyses are conducted by using GeoStudio 2012 (August 2015 release version) software. SEEP/W is used for conducting seepage analyses. In all of the analyses, “steady-state” analysis type is used. Relief well analyses are conducted with plan view, alternative methods analyses are conducted with 2-dimensional view analysis. Setting correct view is important as software automatically calculates the gravity effect. View point is set from Set → Units and Scale → View as shown in Figure 2.1. View point means the location where designer sees the analysis section, a cross-section or a plan view. By using “Draw” tab, the dam is drawn as shown in Figure 2.2 and regions are assigned using “Draw Regions”.

After drawing the dam is completed, the materials are assigned with their hydraulic conductivity properties and volumetric water content function as presented in Figure 2.3. Boundary conditions are also adjusted and assigned to relevant locations as shown in Figure 2.4.

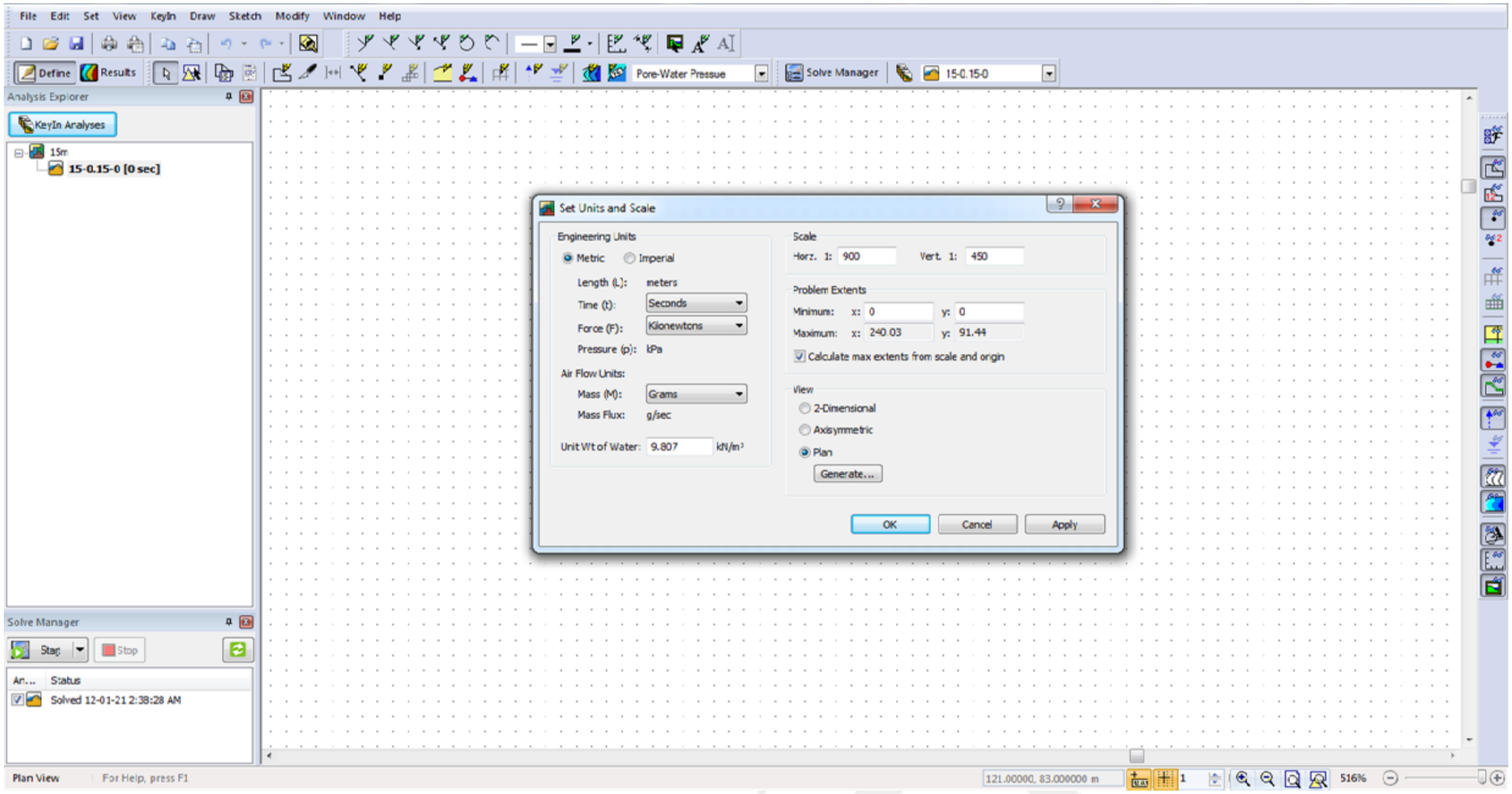

Figure 2.1. Setting view in software (GeoStudio, 2012)

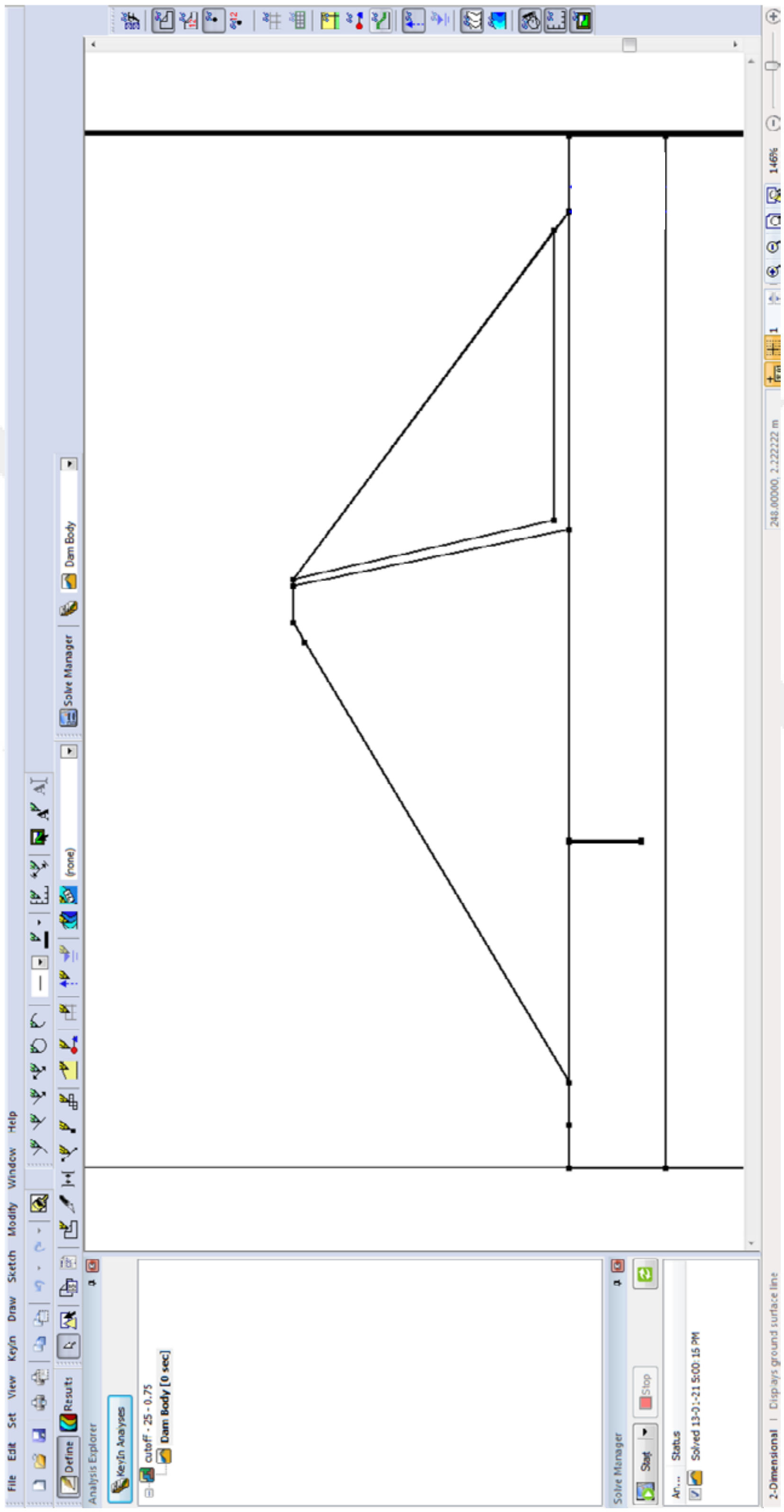

Figure 2.2. Drawing the dam (GeoStudio, 2012)

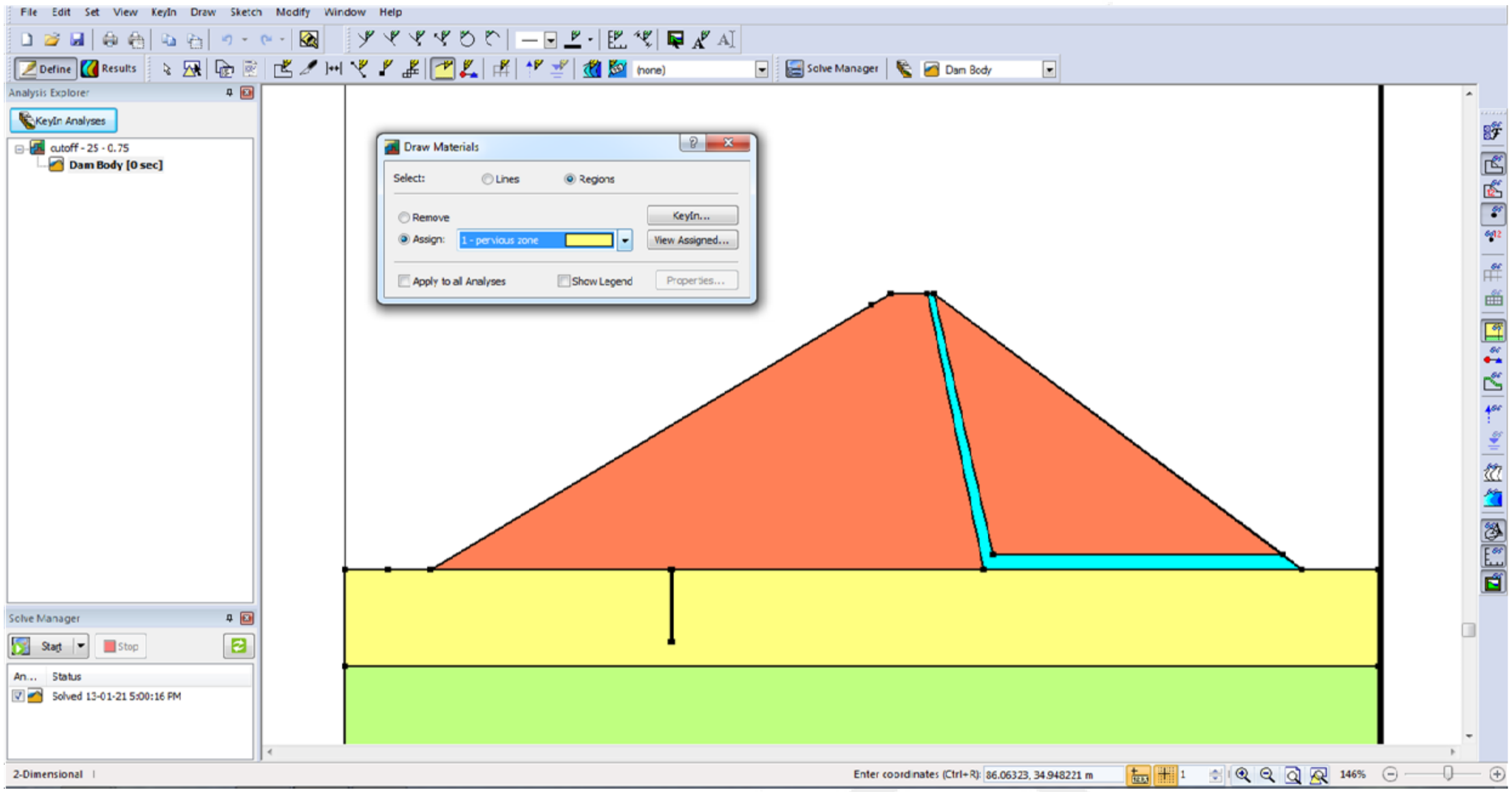

Figure 2.3. Assigning materials (GeoStudio, 2012)

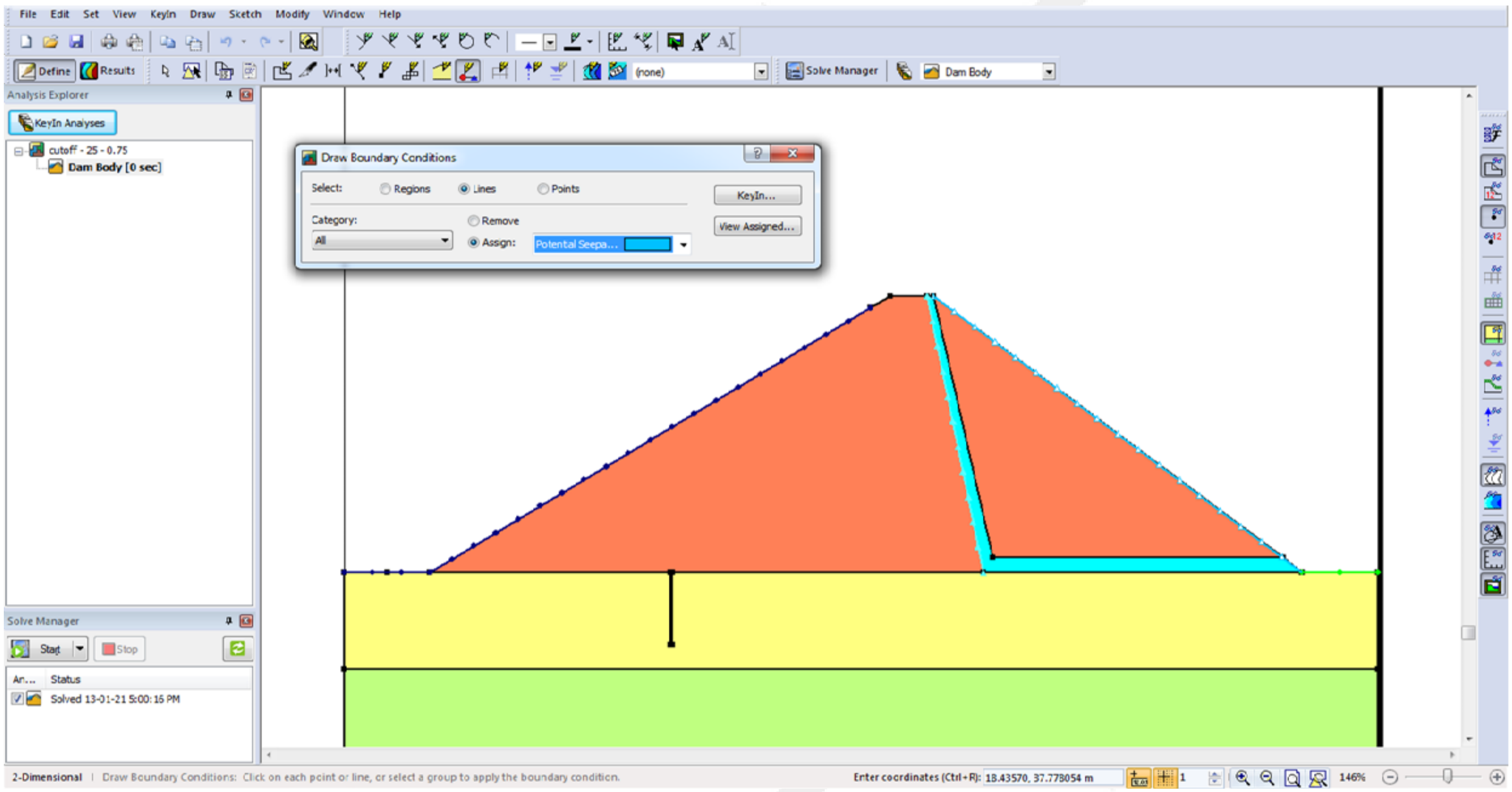

Figure 2.4. Assigning boundary conditions (GeoStudio, 2012)

The analyses are completed and results are presented. Those results consist of all output data for nodes and Gauss points as described by the program. Nodes exist at the corners or along the edges of the elements. The Gauss quadrature scheme is used by the software. This scheme involves naming the specific points as Gauss points and sampling the element characteristics at those points. Then the sampled data is added up (Geo-Slope International Ltd., 2012). These data are total head, pressure head, pore water pressure, coordinates of the points, and assigned properties as water flux, volumetric water content, etc. The results can be viewed as graphs as a function of time. These graphs can be modified in terms of fonts, legends, etc., and data can be copied to MS. Excel or another program. It is also possible to check the values for the plotted data by moving the mouse directly to the points of the graphs. Water table, contours for water head and pore water pressure are also visible. Flow paths and velocity vectors can be observed by the help of software. Also with flux section feature, seepage volume at selected section can be viewed. Part of the values that can be obtained by the analyses are used in this thesis to obtain outcomes.

CHAPTER 3

CASE STUDY

3.1 Description, Geometry and Material Properties

The study utilizes the geometry of an existing dam for the application of the proposed research. Oğulcuk Dam, an earth fill dam located in Yozgat, Turkey, has been selected for conducting relevant analyses in this study. Its construction was started as of April 2013 and the construction is still ongoing. The aim of this dam is to provide water for irrigation to the surrounding area. The dam is a homogeneous structure. Dam characteristics are outlined in Table 3.1.

Table 3.1 Dam geometry

Dam Geometry	
Dam elevation	1148.5 m
Thalweg elevation	1120.0 m
Foundation elevation	1118.0 m
Height (from thalweg)	28.5 m
Height (from foundation)	30.5 m
Crest width	10.0 m
Crest length	1529.0 m
Width at thalweg	202.375 m
Normal operating level	27.25 m
Upstream side slope	1V:3.75H
Downstream side slope	1V:3H

The foundation of the dam consists a pervious alluvial soil. Characteristic values are assigned to foundation properties to carry out related analyses. The permeability, saturated water content and residual water content of the soils of the dam body are taken from Carter and Bentley (2016). The permeability, saturated water content and residual water content of the foundation material are assigned according to Domenico and Schwartz (1998). The coefficients that are used in the analyses are given in Table 3.2, in which K is permeability, θ_s saturated water content and θ_r residual water content. Some foundation parameters, which are not specifically defined in the planning reports of the dam project, are assigned by the author. The soil is assumed to be in an isotropic soil condition and analyzed as stated.

The location of the dam is shown in Figure 3.1 and the scaled cross-section of Oğulcuk Dam is presented in Figure 3.2.

Figure 3.1. Location of Oğulcuk Dam (Google Earth, 2021)

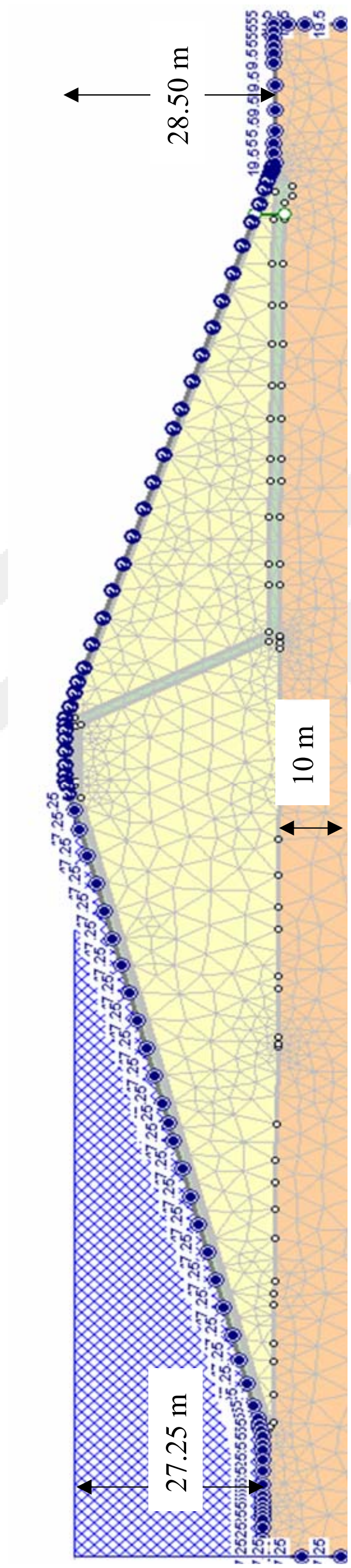

Figure 3.2. Cross-section of the dam

Table 3.2 Material properties used for the analyses

Material	Parameter		Unit
Sandy Clay	K	3.33×10^{-7}	m/s
	θ_s	0.38	-
	θ_r	0.1	-
Clay	K	5.55×10^{-7}	m/s
	θ_s	0.38	-
	θ_r	0.068	-
Sandy Loam	K	1.23×10^{-5}	m/s
	θ_s	0.41	-
	θ_r	0.065	-
Sandy Gravelly Alluvion	K	1×10^{-4}	m/s

3.2 Relief Well Design

Oğulcuk Dam has a pervious soil underneath it that may result in intolerable amount of seepage. Reduction of the seepage problem by pressure relief wells is one of the methods that is used to prevent excessive seepage. Two-dimensional models of seepage to the relief wells are established on plan view. Figure 3.3 presents an example sketch for the analysis. The model region shown in the Figure 3.3 is 225 m long and 90 m wide. The analyses are conducted over a 90 m section of the dam, which is symmetrical around the downstream river. Since the dam is relatively long, only a critical section which is subject to the maximum head difference between the upstream and the downstream is assumed to be effective for seepage reduction measures. The blue lines represent the flux sections used as control sections; whereas the brown circles represent the relief wells. The area between two orange lines are the Oğulcuk Dam's location in the analysis. In this figure, the distance between the wells are 10 meters. The example region consists of 42,656 mesh elements.

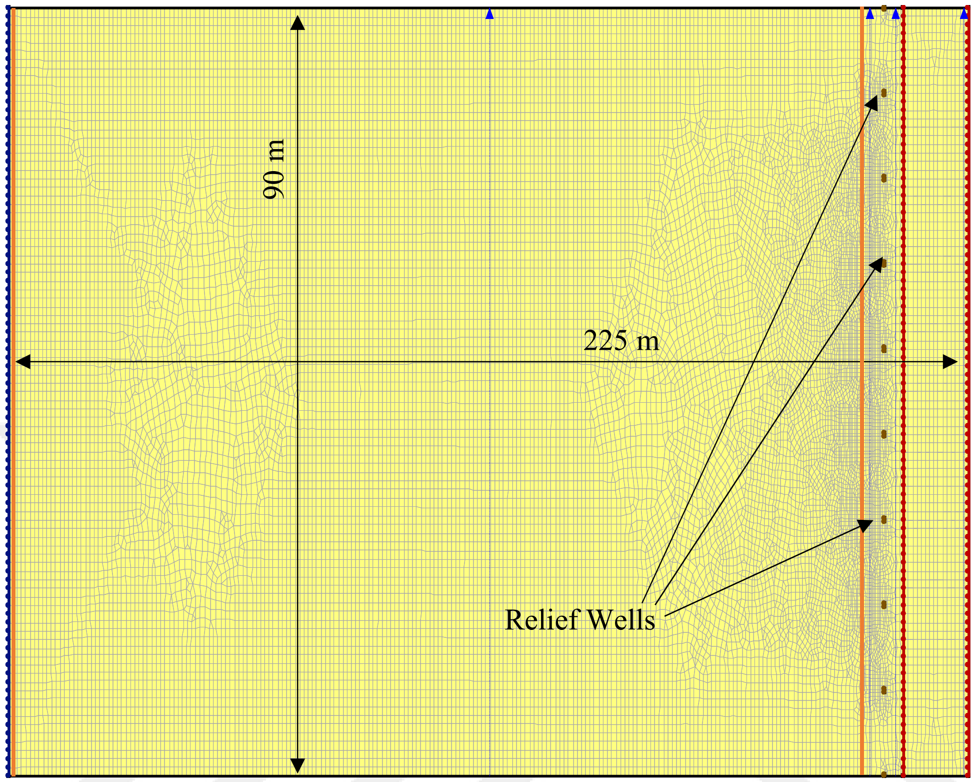

Figure 3.3. Example seepage model

The meshing in Figure 3.3 is disturbed at some points which is due to the automatic meshing feature and with this, an approximate mesh size is given to the software and it meshes the region according to this mesh size.

A total number of 216 analyses are conducted by assigning a constant head of 27.25 m, normal operating level, to the upstream water head. Table 3.3 shows the location, spacing and size of the relief wells considered for the simulations. The minimum and maximum inner diameters for the wells are chosen in accordance with USACE standards (USACE, 1992). These standards specify that the wells, including screen and riser pipe have inside diameters ranging 0.10 m and 0.46 m.

Table 3.3 Variables for well performance analysis

Distance between wells (m)	5	6	9	10	15	30
Inner diameter of wells (m)	0.15	0.2	0.25	0.3	0.4	0.46
Distance of wells from toe (m)	0	1	2	3	4	5

3.3 Blanket for Cutoff Wall Design

The study is also aimed to compare the performance of the relief well with that of the alternatives aimed to solve the foundation seepage problem. As mentioned before, Oğulcuk Dam originally does not have any preventive measure against the seepage through its foundation. Therefore, for comparison reasons, three individual seepage preventive measures are designed as an alternative to relief wells. These are an upstream blanket, a clay core, and a trench. The designs of these are performed in accordance with USACE (USACE, 1992) and FEMA (2011) standards.

The blanket thickness is chosen as 0.7 m and the length of the blanket is calculated accordingly. Bennett (1946) resolved a previously presented differential equation and proposed an equation for the effective length for blankets in different conditions.

$$\frac{d^2h}{dx^2} = \left(\frac{K_b}{Z_b K_f Z_f}\right)h \tag{3.1}$$

Equation 3.1 is the basic differential equation for limited permeability blankets. This equation was presented by Bennett (1946), whose solution is presented as follows.

$$X_r = 0.82\sqrt{\frac{Z_b K_f Z_f}{K_b}} \tag{3.2}$$

where Z_b is the thickness of upstream horizontal blanket, K_f is the permeability coefficient of the foundation material, Z_f is the thickness of foundation, K_b is the permeability coefficient of the blanket material, and X_r is the length of the upstream clay blanket. Using the relevant data for the dam, the upstream blanket length is calculated as 48.5 m.

3.4 Mesh Sensitivity Analysis

In the study, the effect of mesh size on the results are checked to ensure that its variation does not affect the results. A sensitivity analysis is conducted to determine the optimum mesh size for the models. To this end, a specific point, i.e. data of point (202, 45) as shown in Figure 3.4, is selected from the problem domain and the pore water pressure at this point is calculated for varying mesh size. This point is selected as representative because it is in the center of the toe in the analysis scheme. The mesh independence analysis is conducted at point (202, 45) on discharge flux analysis with 10 meters of distance between relief wells. The pore water pressure variation with respect to mesh size is given in Figure 3.5.

As Figure 3.5 indicates, the pore water pressure at point (202, 45) converges when the mesh size is less than 1 m. The relative error becomes 0.27% if the mesh size is 1 m or less. Therefore the global mesh size is determined as 1 m for the rest of the simulations.

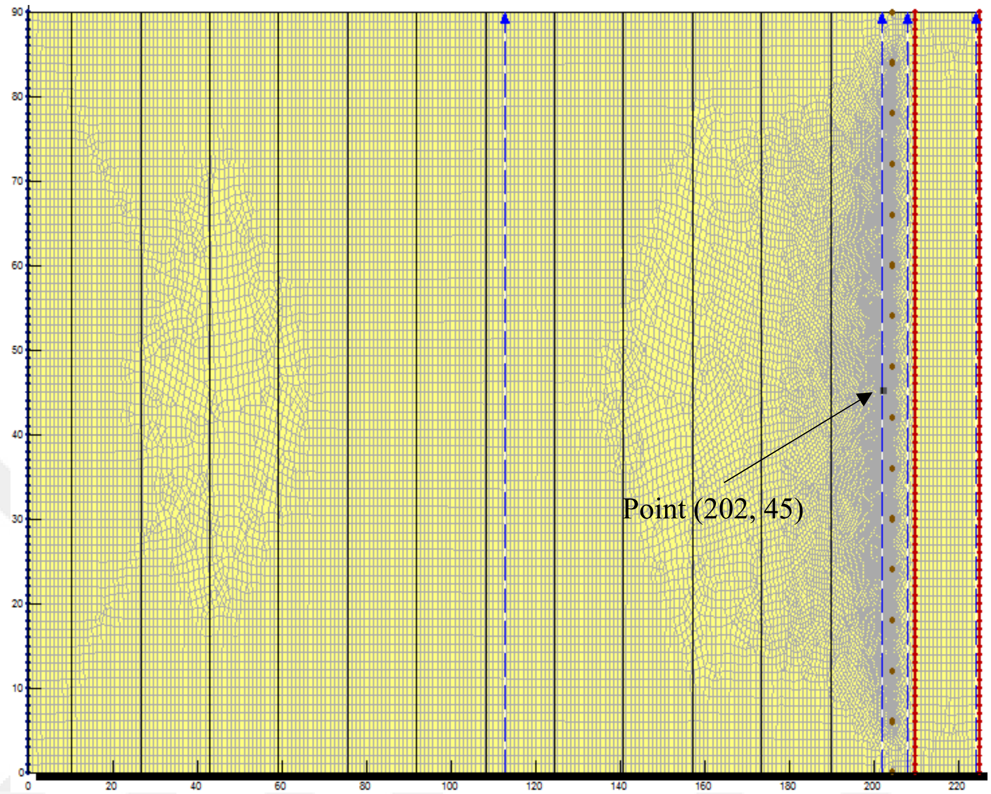

Figure 3.4. Location of point (202, 45)

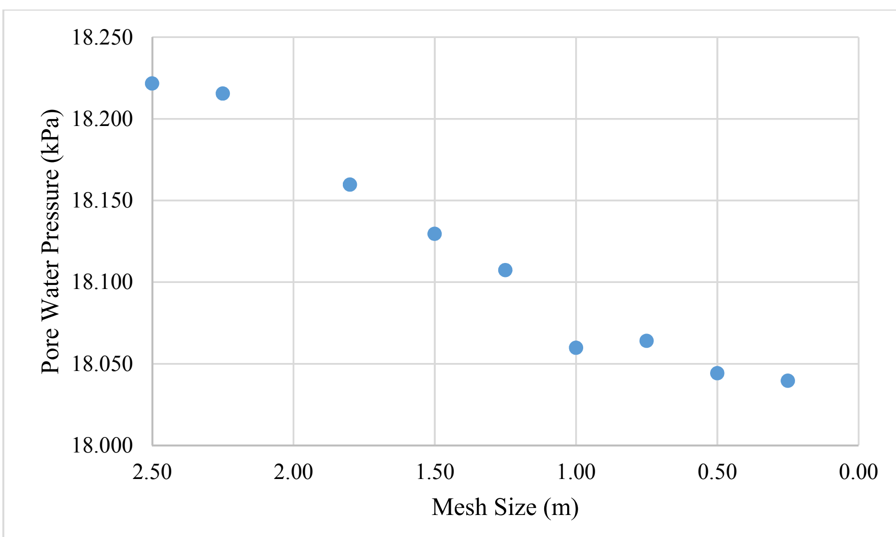

Figure 3.5. Pore water pressure and mesh size relation

3.5 Steady-State Seepage Analysis for Relief Well Configurations

The performance of the relief wells is investigated under steady-state flow conditions. The reason is that the maximum seepage occurs and the greatest pore water pressures are observed at the steady-state seepage when the reservoir is full. This is also the case in which high hydraulic gradients occur and may result in movement of fine soil particles that lead to piping failure (Yılmaz, 2017).

The analyses are conducted for the seepage through the foundation and the flow into the wells for various conditions. The analysis with the initial case (without pressure relief wells) is resulted in a seepage rate of 93.5 m^3/day. The wells are placed to reduce the effects of this seepage. Figure 3.6 presents an example relief well analysis result for 5 meter spacing, 0.46 m inner diameter and 0 m from the downstream toe. The results of analyses for the cases with 5 m spacing are reachable in the Appendix. In Figure 3.6, the colors represent a 20 kPa interval, with every change from red to blue, a 20 kPa zone is shown.

As this study focuses on relief well performances, which are used for decreasing foundation seepage as described previously, the plan view analysis doesn’t include internal seepage coming from the dam body. In fact this effect is negligibily small as previously stated.

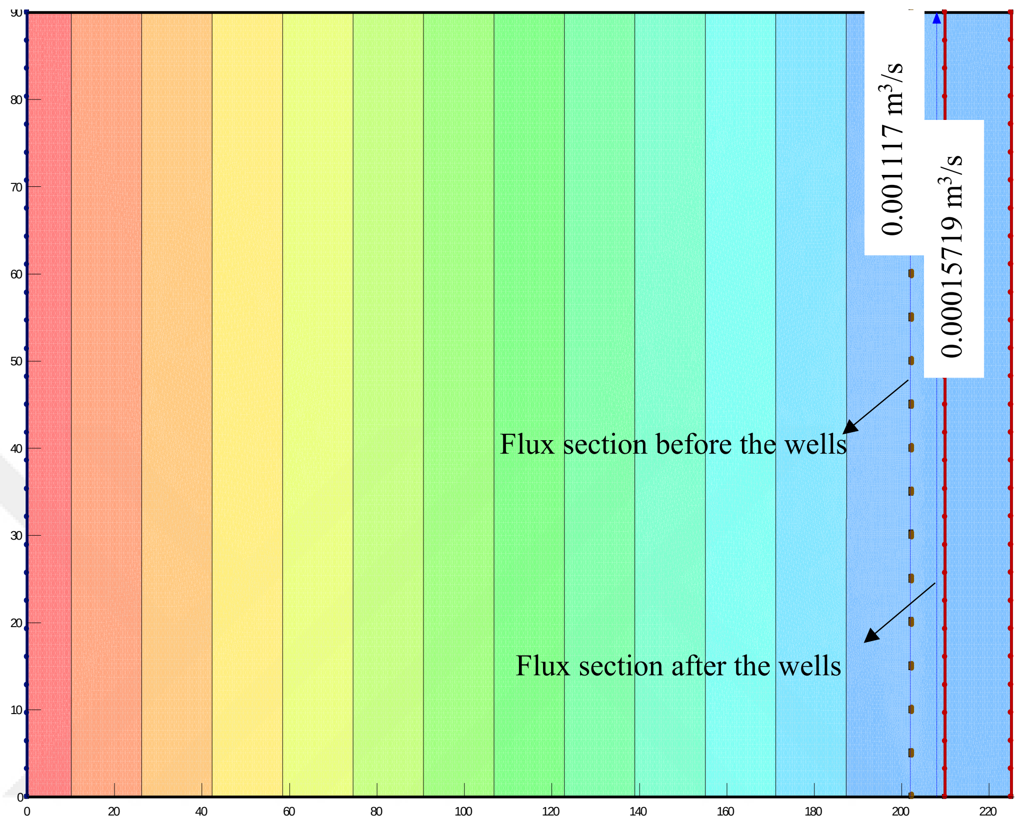

Figure 3.6. Well analysis result when spacing is 5 m, inner diameter is 0.46 m and distance is 0 m from the downstream toe

The seepage through the foundation and the collected water amount at the wells are obtained and shown from Figure 3.7 to Figure 3.30. The units of the seepage and the collected water amount are given in lt/s/m/m. This indicates the discharge per unit width of the dam and per unit depth of foundation. This unit is chosen because the analyses are conducted in a 2D environment for both top and side views. The analyses of the relief wells are conducted in plan view. The alternative solutions for seepage problem, cutoff wall with blanket, core trench, and cutoff wall given in Section 3.7, are conducted in side view. The aim of the unit is to present the findings in a comparable manner.

The difference between the flux values before and after the wells is accepted to be the seepage extracted by the wells. It is clearly seen from Figure 3.7 to Figure 3.12 that when the distance between the wells decreases, the seepage through the section decreases. This decrease of seepage is observed as much as 1/3 times of initial seepage value as the spacing/reservoir level ratio increases from 0.183 to 1.101, when the spacing changed from 5 m to 30 m. It is also shown that with the decrease of spacing, as it is inversely proportional with the seepage, the amount of collected water by wells are increased with the same ratio.

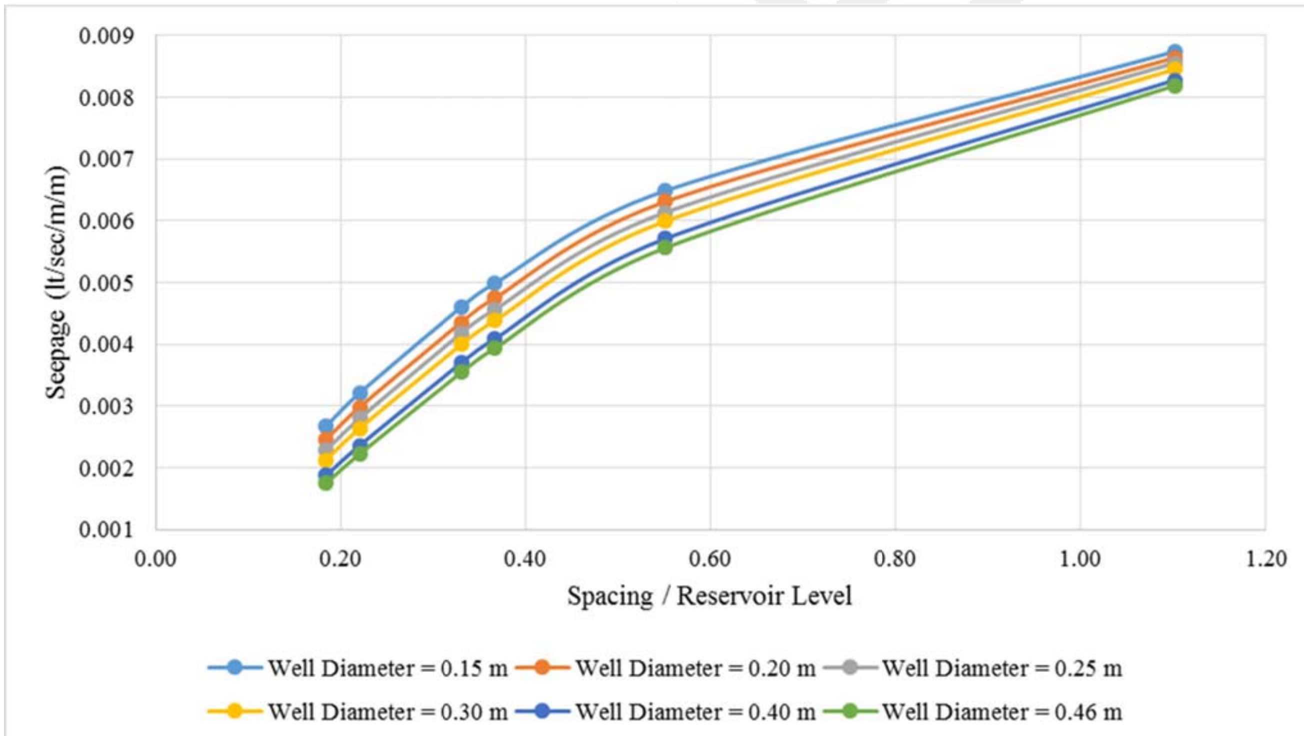

Figure 3.7. Relief well spacing and seepage amount when the relief wells are located at the toe

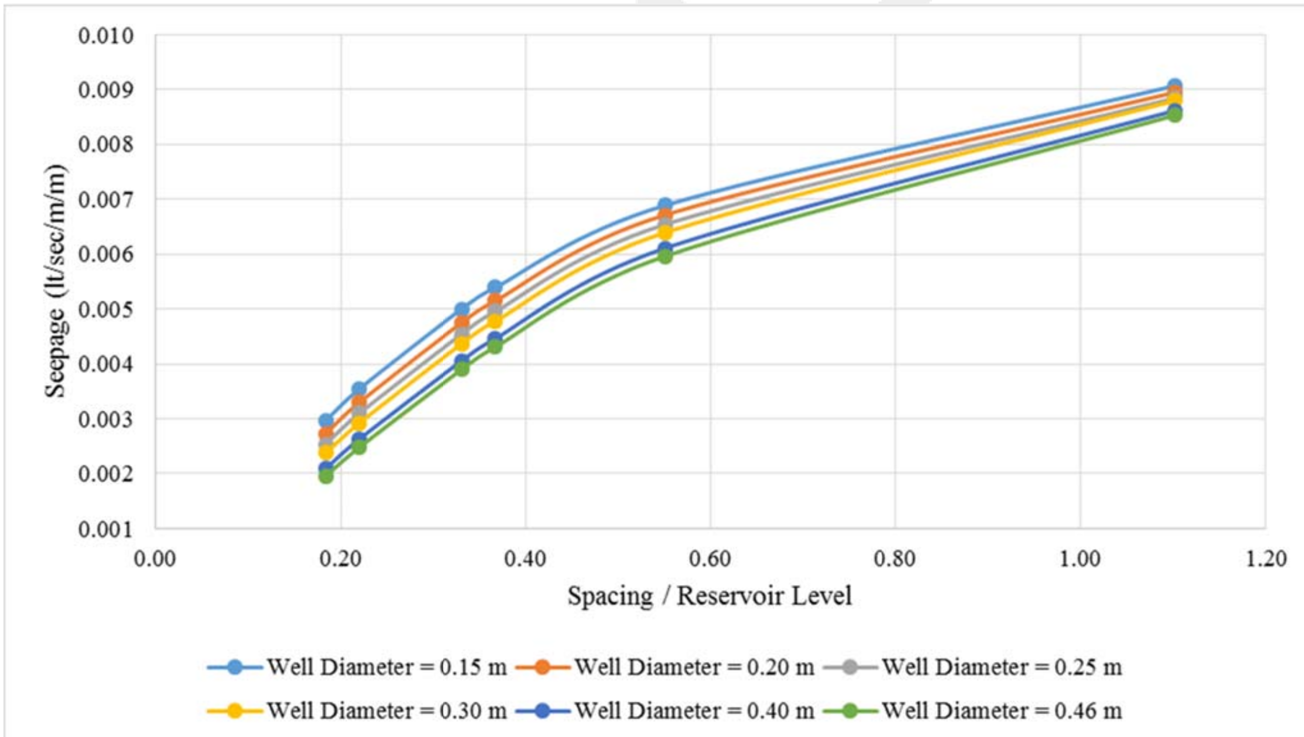

Figure 3.8. Relief well spacing and seepage amount (1 m from the toe)

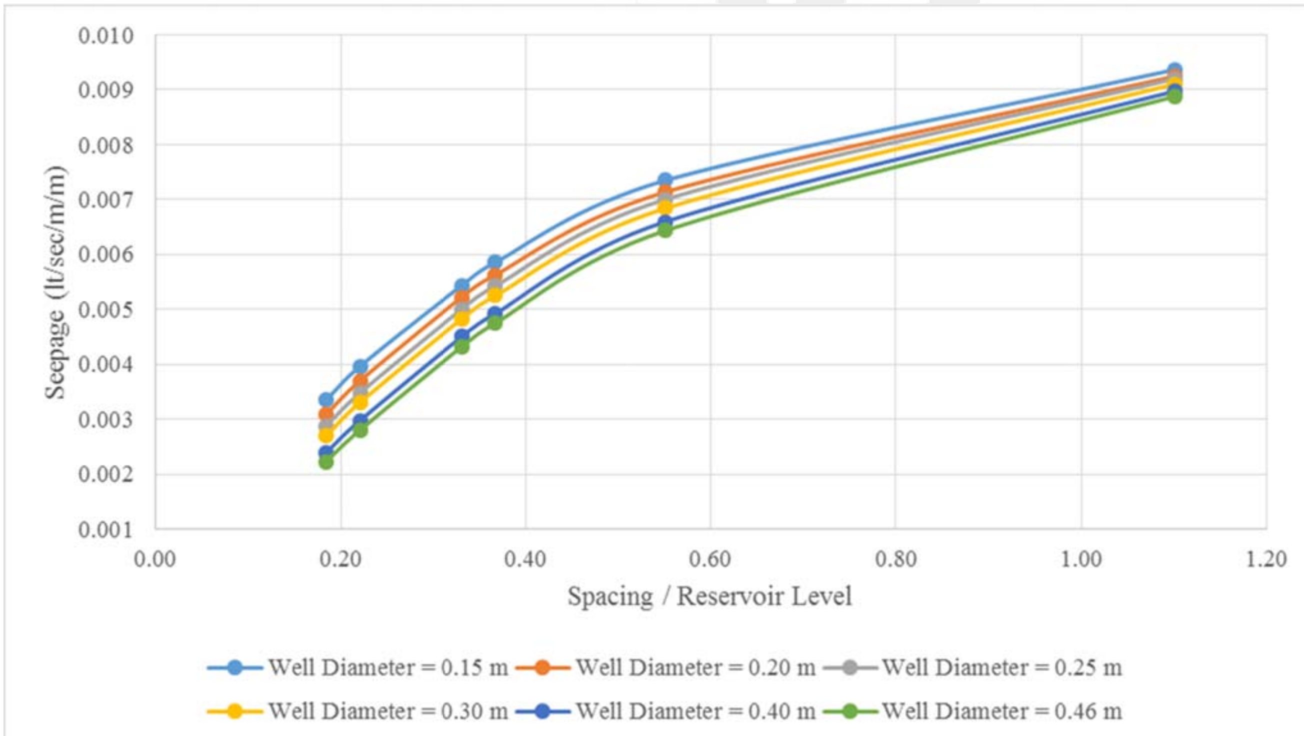

Figure 3.9. Relief well spacing and seepage amount (2 m from the toe)

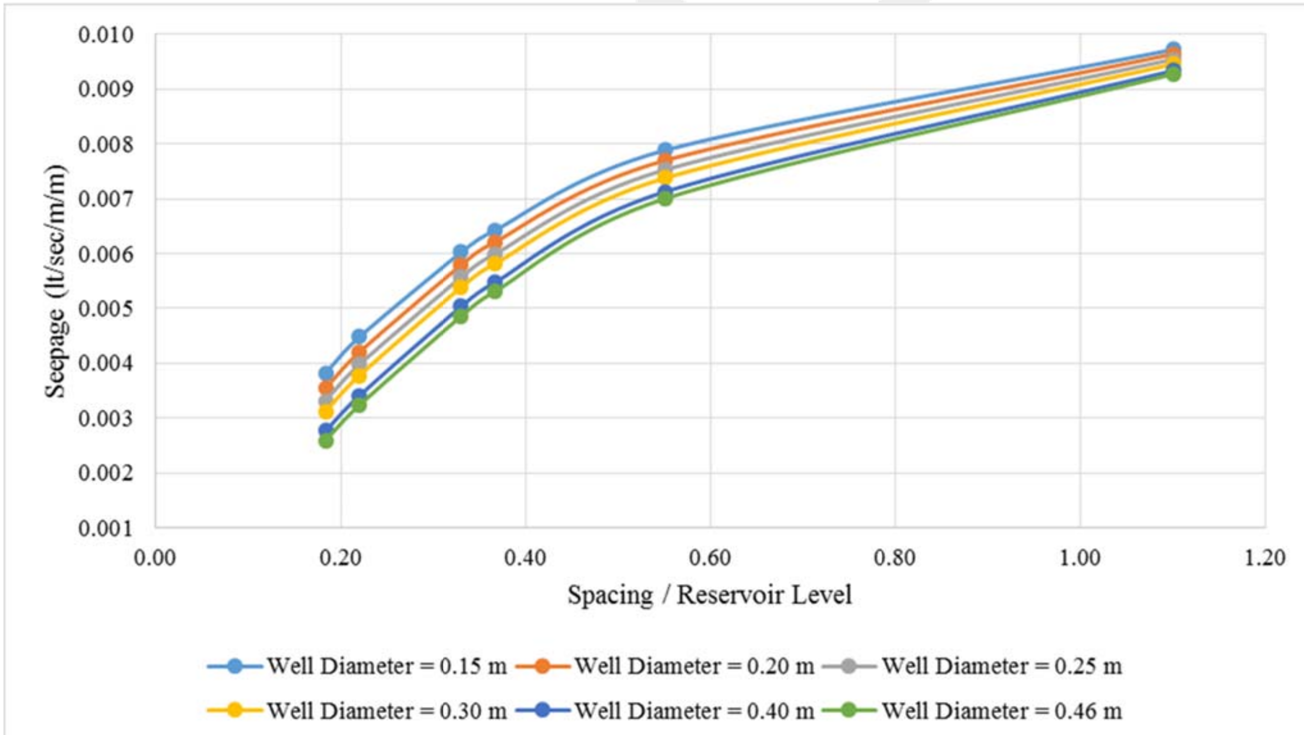

Figure 3.10. Relief well spacing and seepage amount (3 m from the toe)

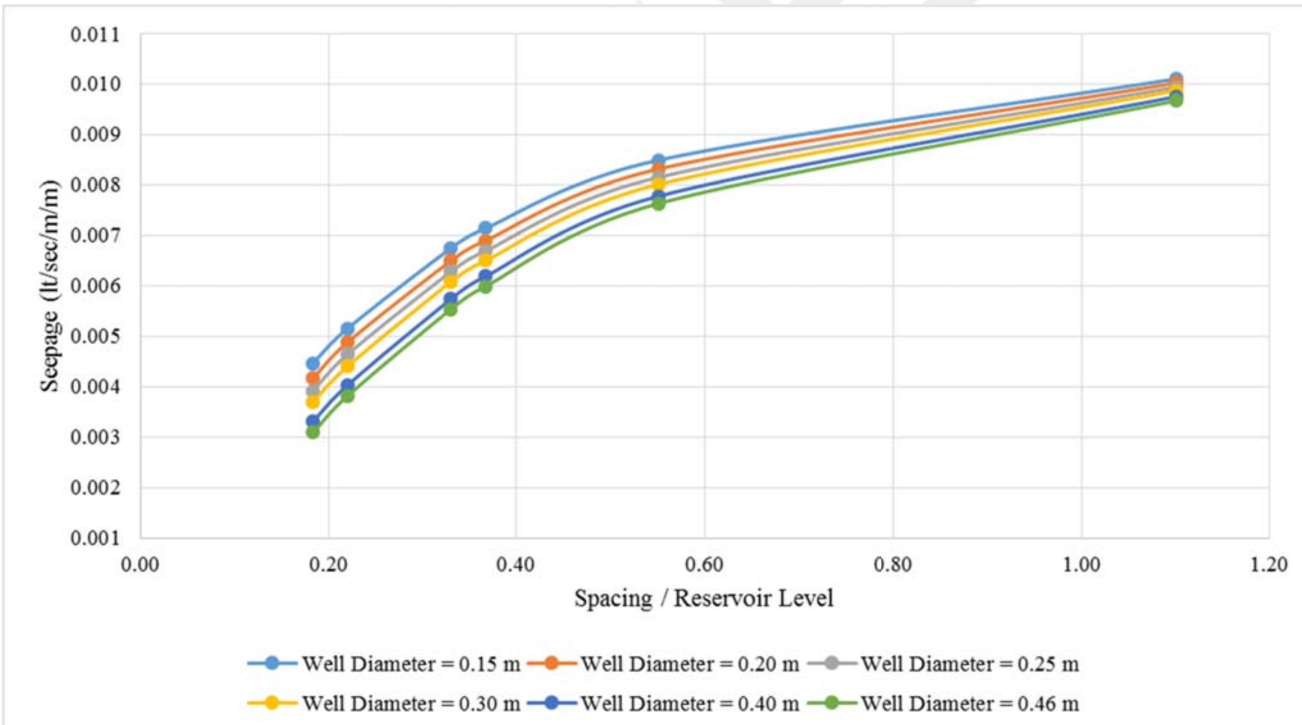

Figure 3.11. Relief well spacing and seepage amount (4 m from the toe)

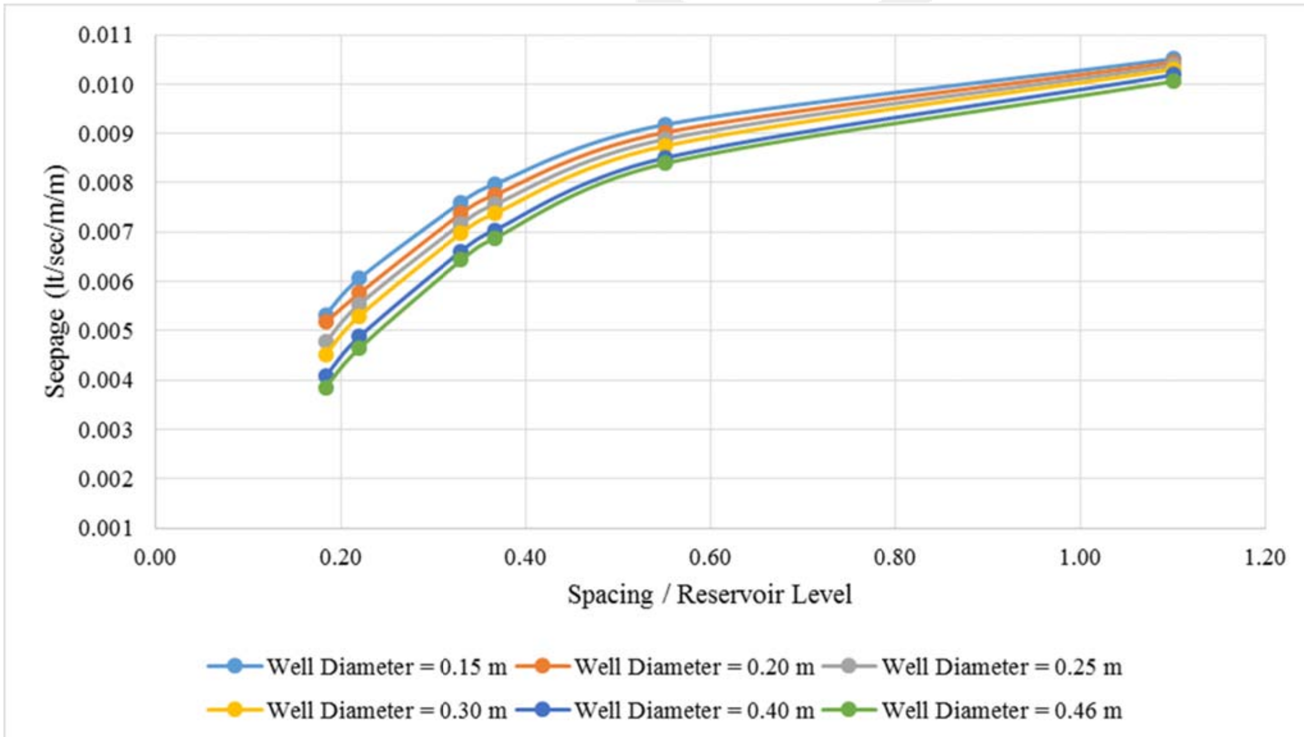

Figure 3.12. Relief well spacing and seepage amount (5 m from the toe)

The study also investigates the water collected by the wells in the same conditions with the previously presented seepage analyses. The collected water amount is estimated by using two flux sections which are placed just before and after the relief wells. The total flux passing through the section after the relief wells are subtracted from the total flux passing through the section before the relief wells. This yields to the total amount of water collected by wells. Figure 3.13 through Figure 3.18 show the results of water collected by the wells with the spacing/reservoir level ratio. The six different graphs represents six different locations for the wells. The effect of spacing is investigated in these graphs. The increase in spacing yields decrease in collected water amount in the wells. When the distance increases, number of wells also decreases.

When the inner well diameter increases, the amount of seepage decreases. As highlighted before, seepage is inversely proportional with the water collected. When the inner diameter increases, collected water amount by the wells also increases.

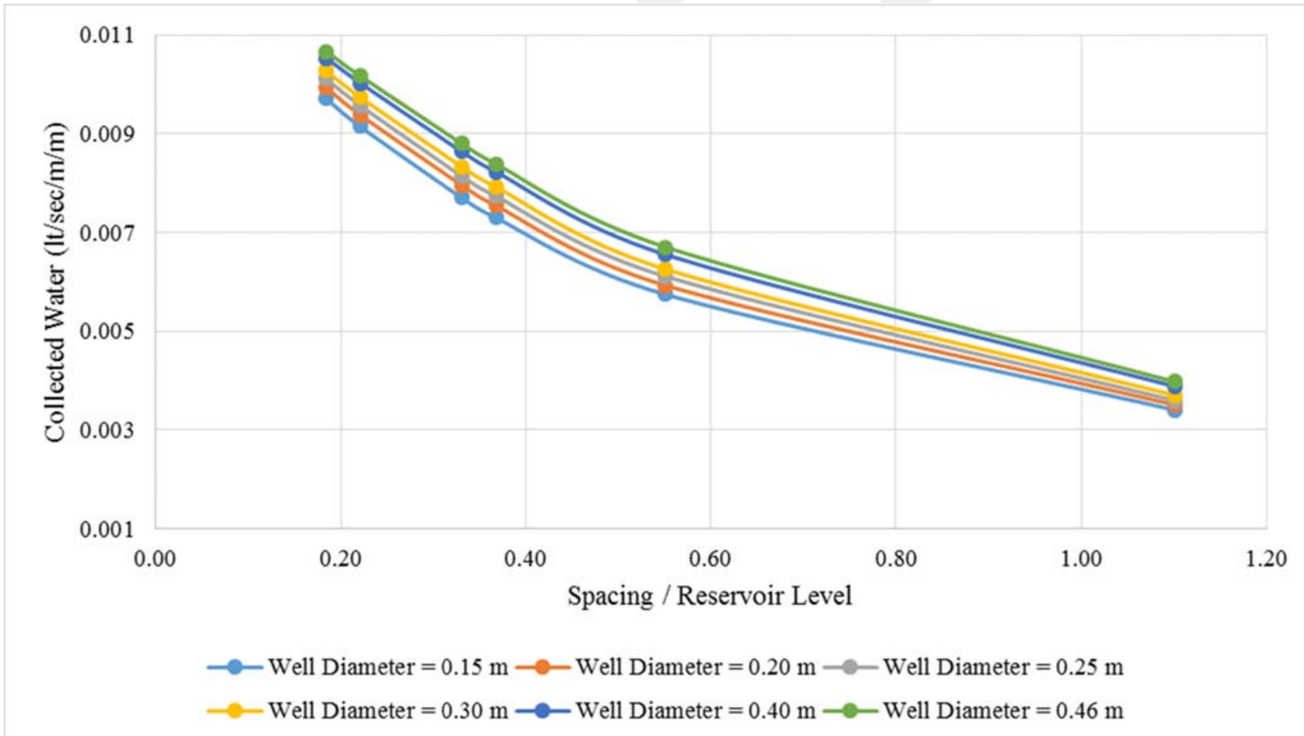

Figure 3.13. Well spacing and water collected by wells when the relief wells are located at the toe

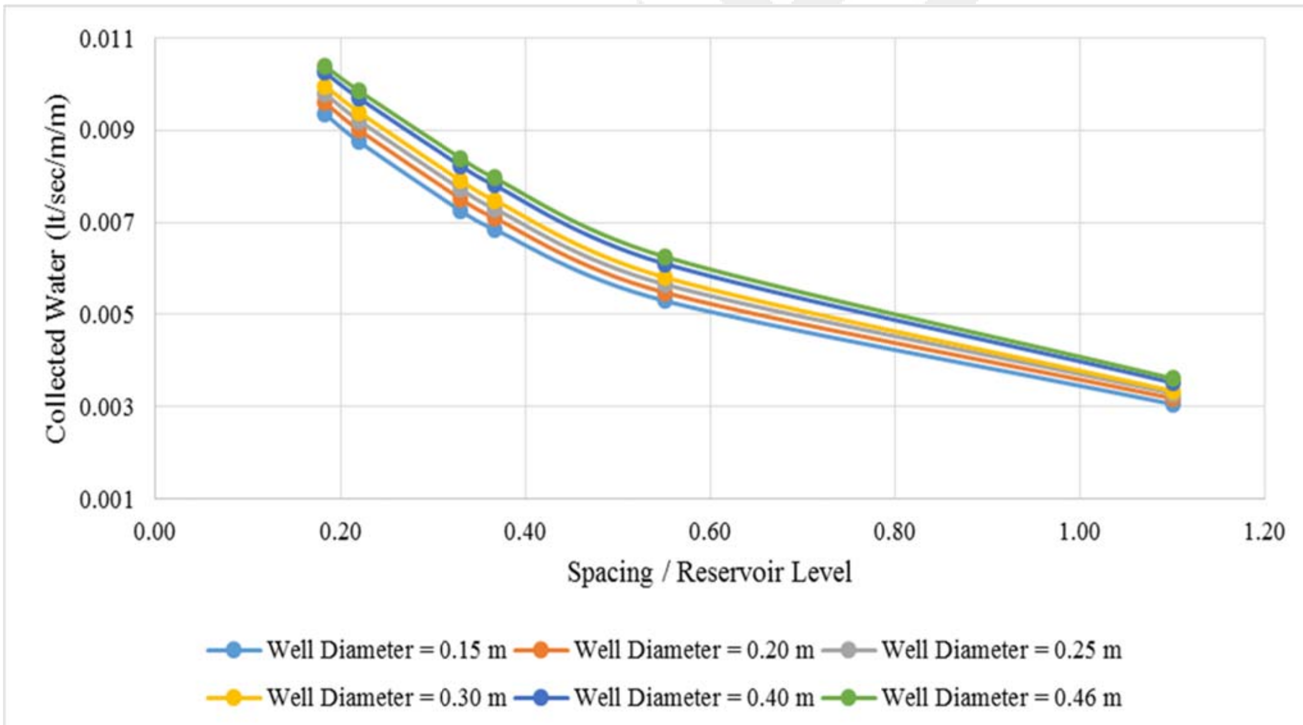

Figure 3.14. Well spacing and water collected by wells (1 m from the toe)

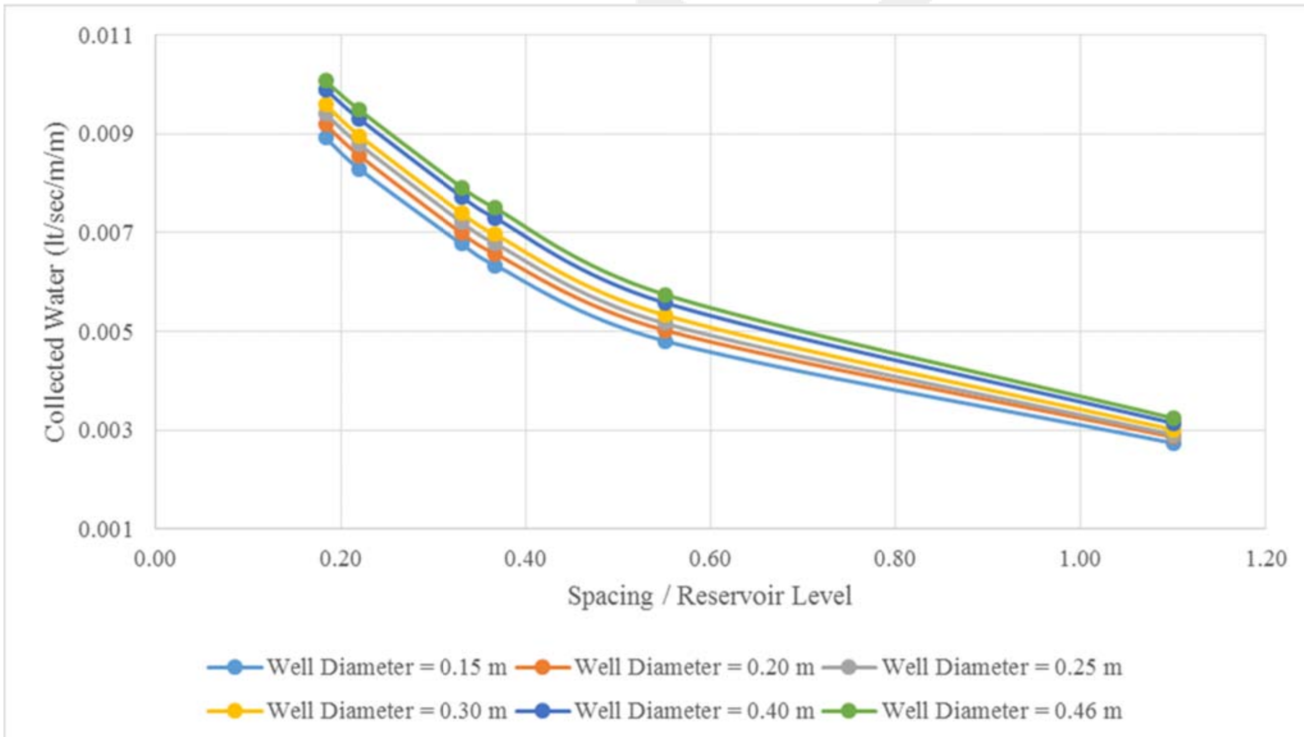

Figure 3.15. Well spacing and water collected by wells (2 m from the toe)

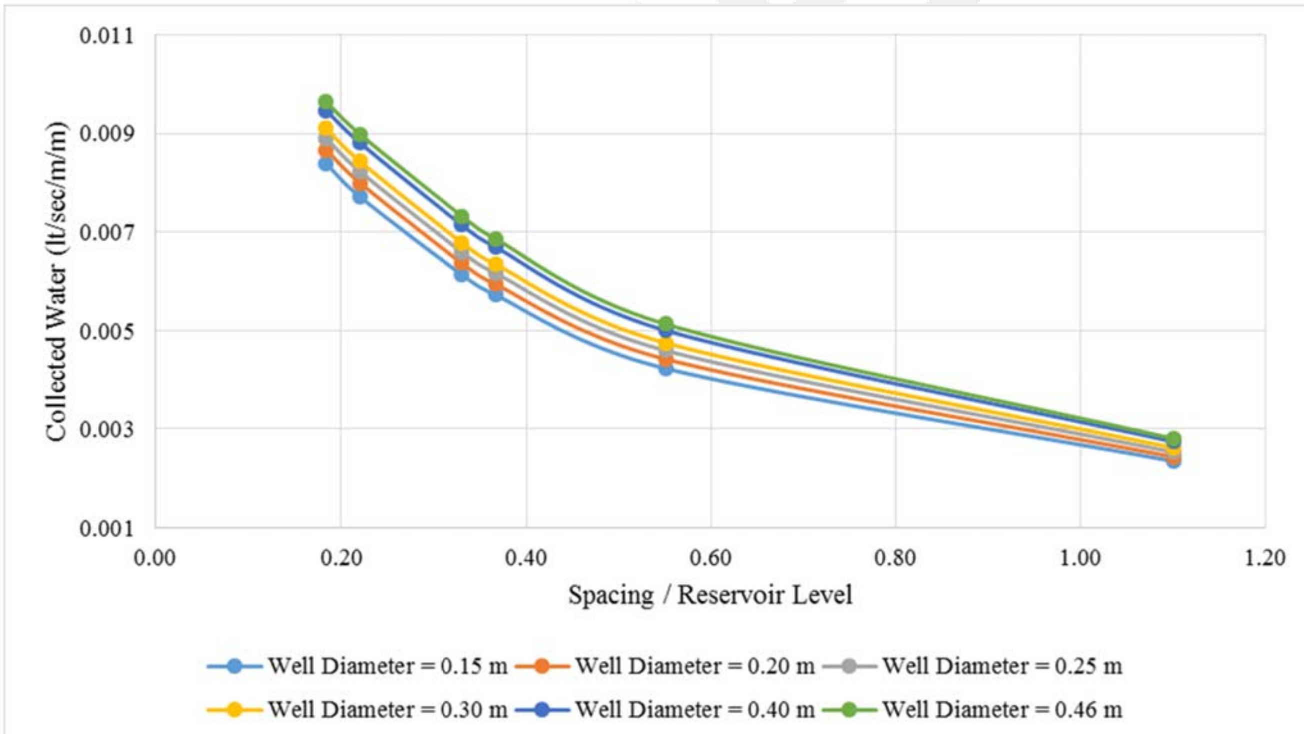

Figure 3.16. Well spacing and water collected by wells (3 m from the toe)

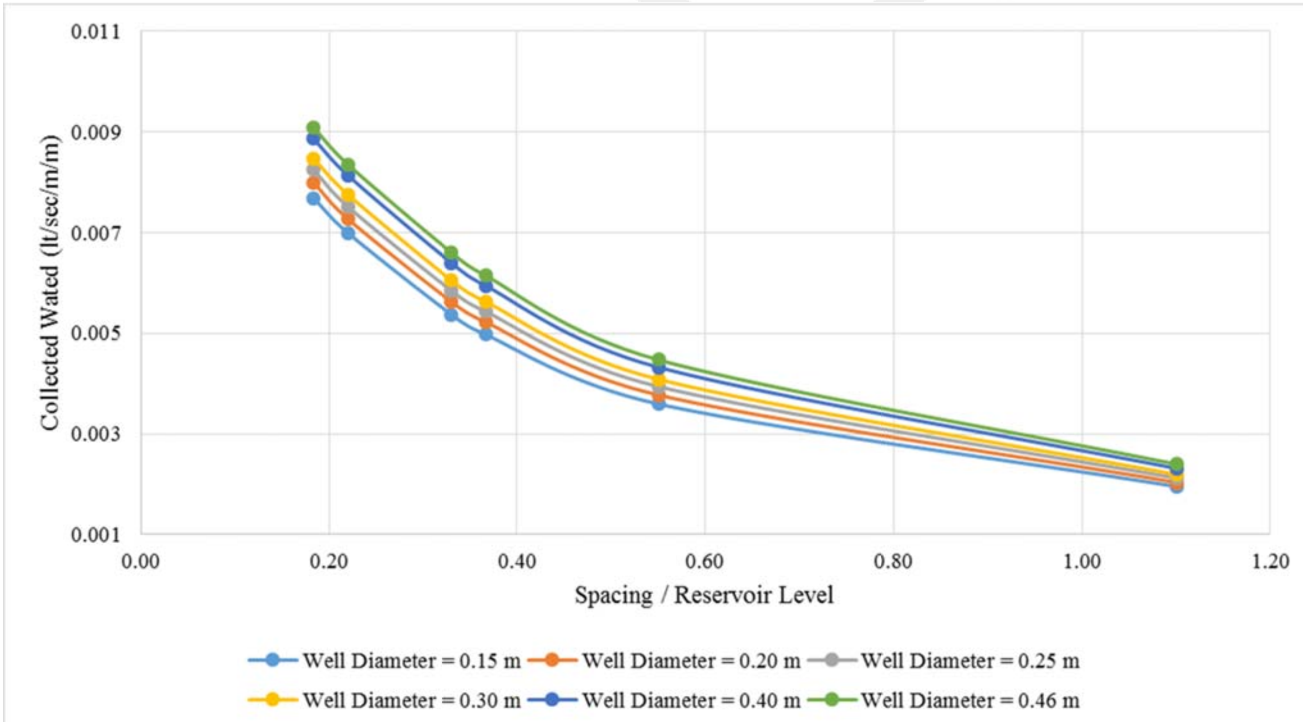

Figure 3.17. Well spacing and water collected by wells (4 m from the toe)

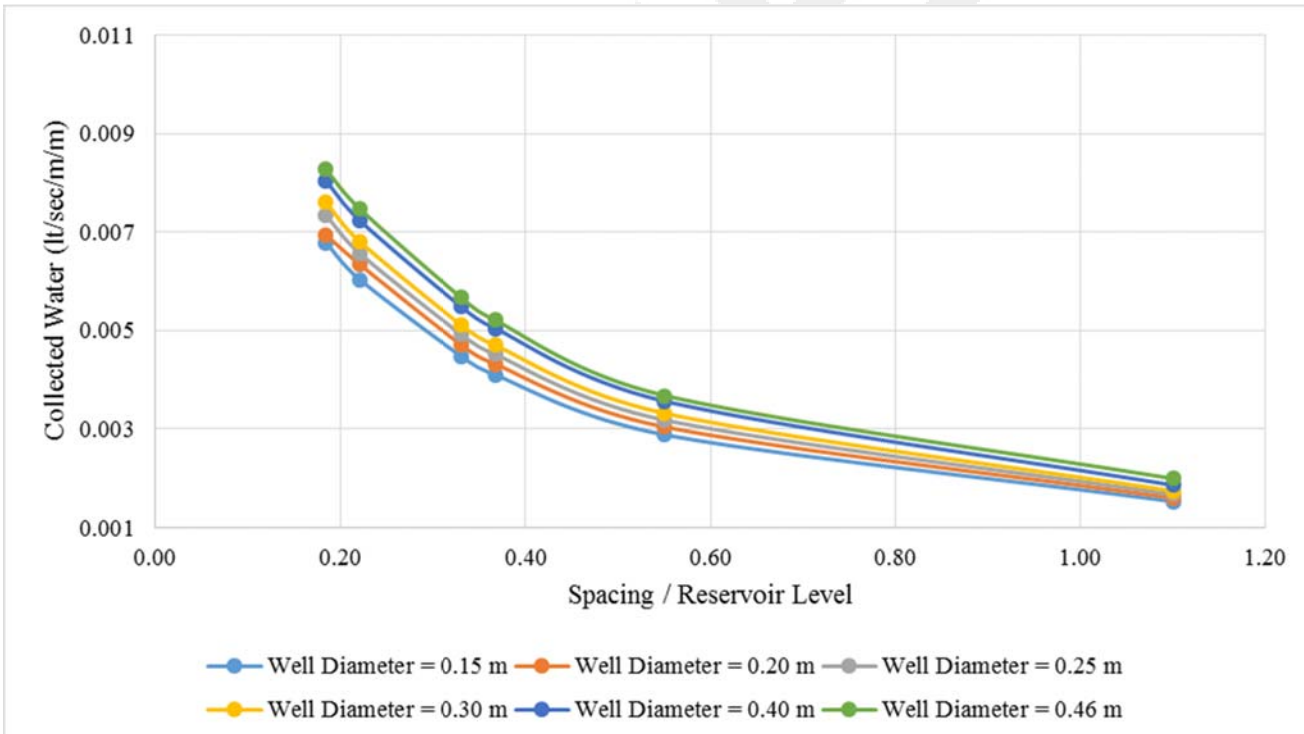

Figure 3.18. Well spacing and water collected by wells (5 m from the toe)

The effect of inner well diameter is also studied in this study. Figure 3.19, through Figure 3.24 show the change of seepage through the wells by the change of inner well diameter/reservoir level ratio. Each graph has a different spacing amount. Different lines represent different distances from the downstream toe. The increase of inner well diameter results to decrease in seepage in all of the stated figures. Figure 3.19 indicates that the increase of inner well diameter/reservoir ratio from 0.006 to 0.017 results in 34.5% decrease of the rate of seepage for the case with 5 m spacing and well at the toe (distance from the toe is 0 m). Same amount of increase of inner well diameter/reservoir ratio yields to 30.8% decrease of seepage rate for 6 m spacing as shown in Figure 3.20, 22.8% decrease of seepage rate for 9 m spacing as shown in Figure 3.21, 21% decrease of seepage rate for 10 m spacing as shown in Figure 3.22, 14.3% decrease of seepage rate for 15 m spacing as shown in Figure 3.23 and 6.3% decrease of seepage rate for 30 m spacing as shown in Figure 3.24.

It is shown that when the distance from the toe increases, the seepage amount increases and collected water amount decreases as they are inversely proportional.

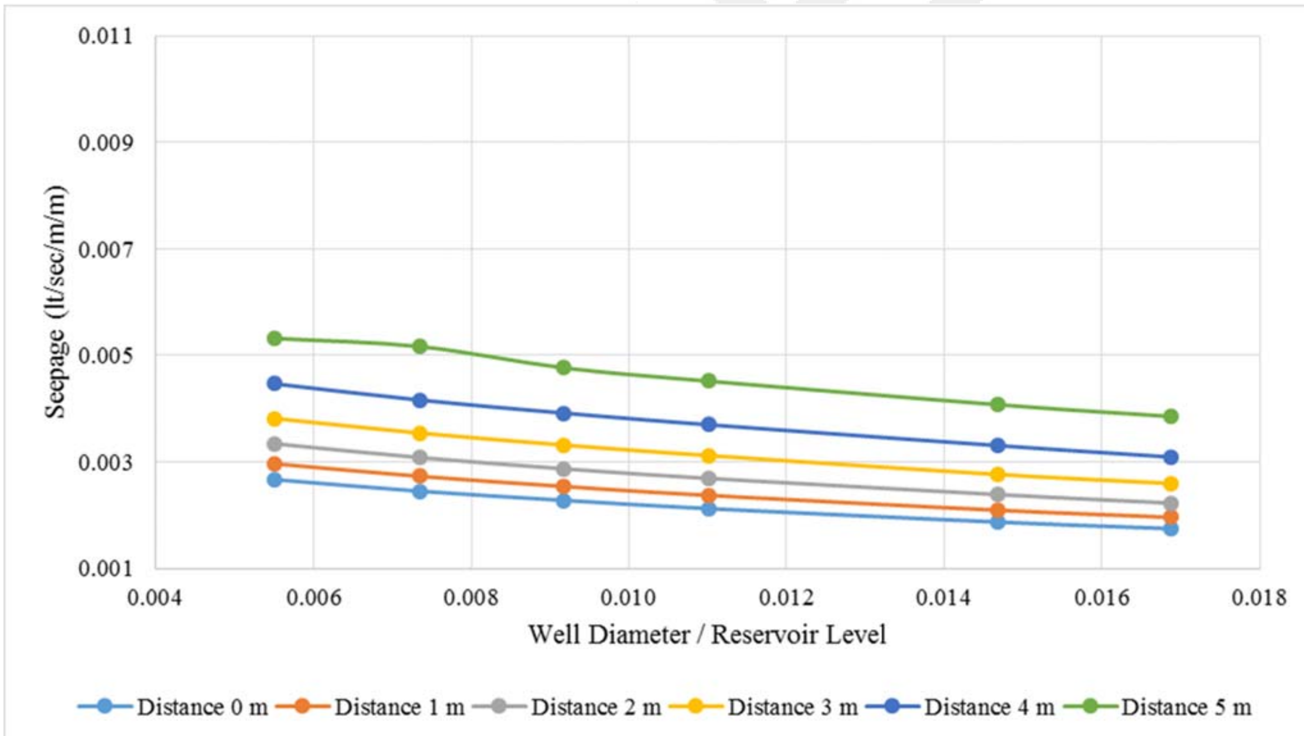

Figure 3.19. Effect of inner well diameter on seepage (5 m spacing)

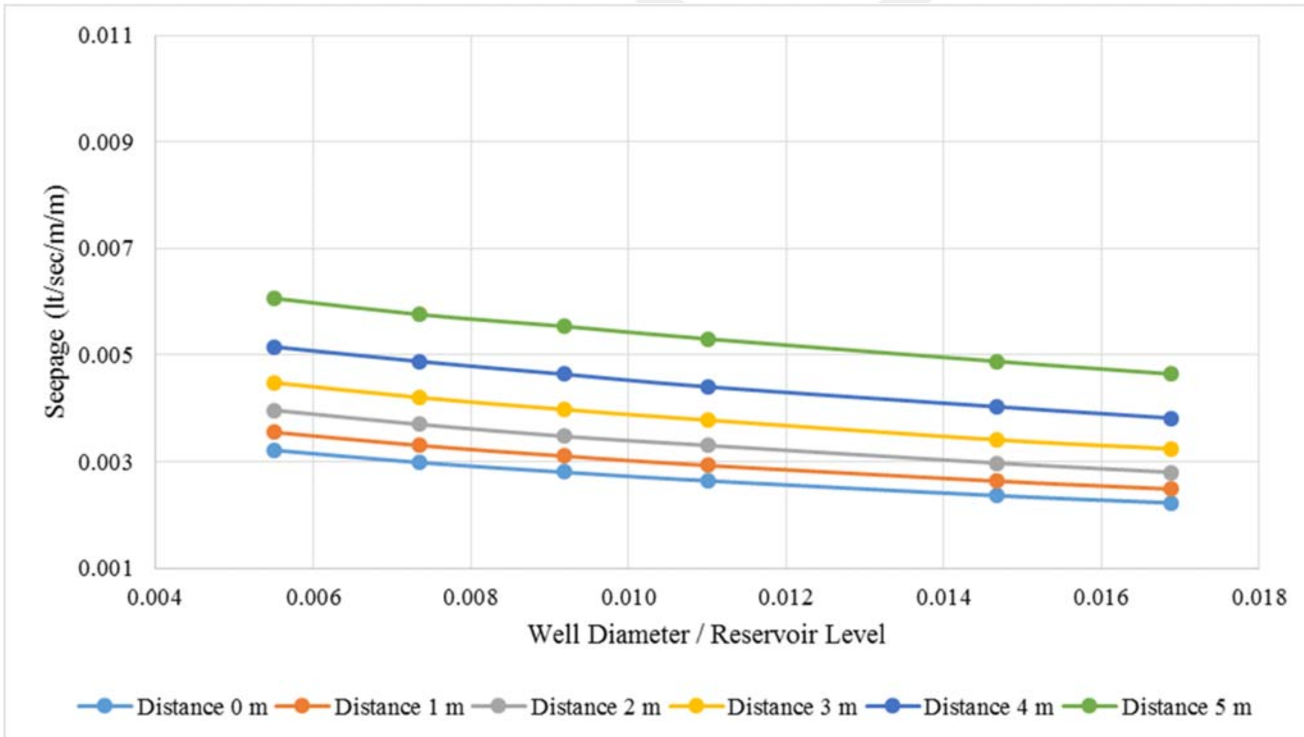

Figure 3.20. Effect of inner well diameter on seepage (6 m spacing)

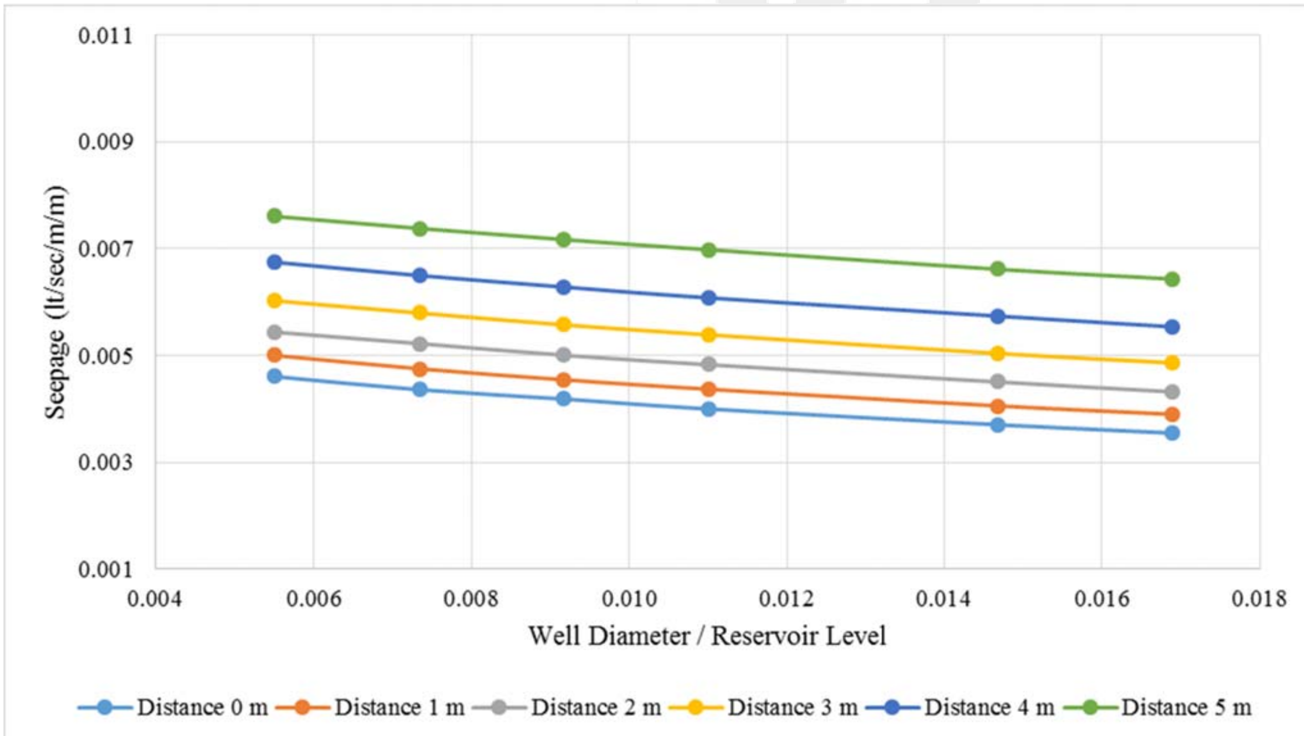

Figure 3.21. Effect of inner well diameter on seepage (9 m spacing)

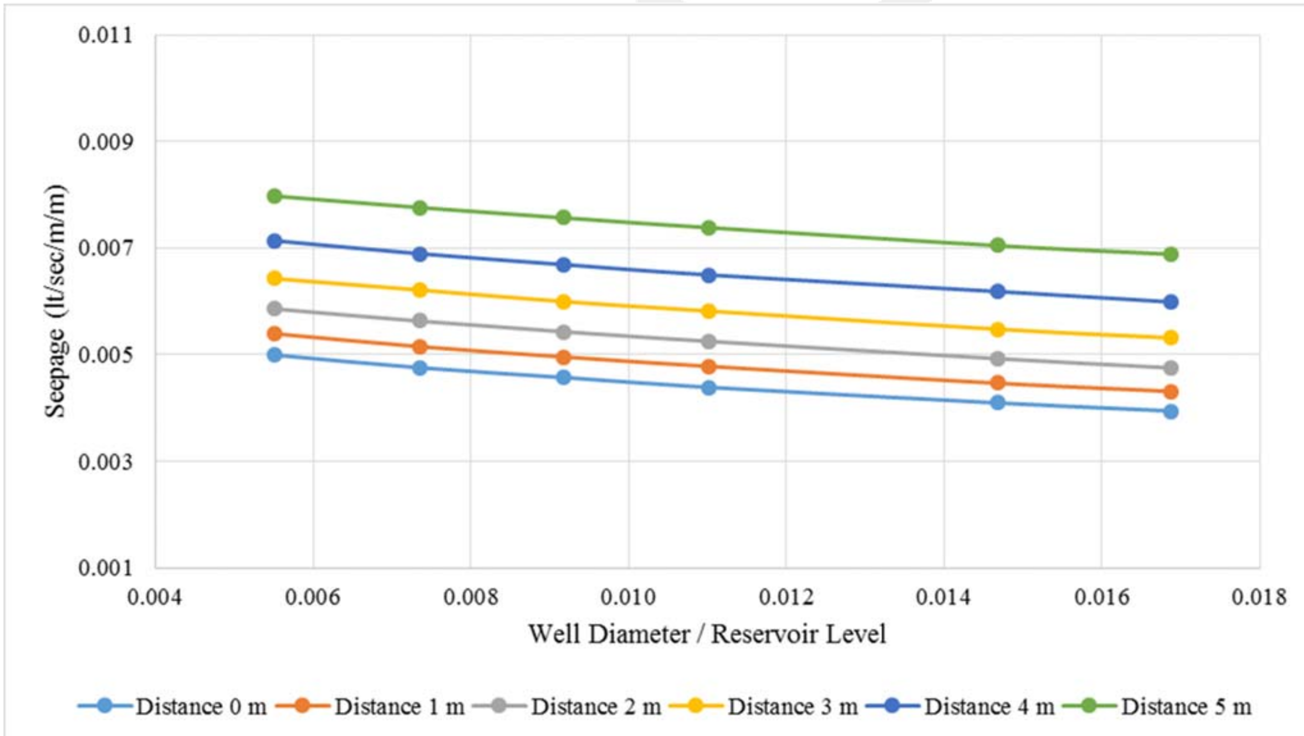

Figure 3.22. Effect of inner well diameter on seepage (10 m spacing)

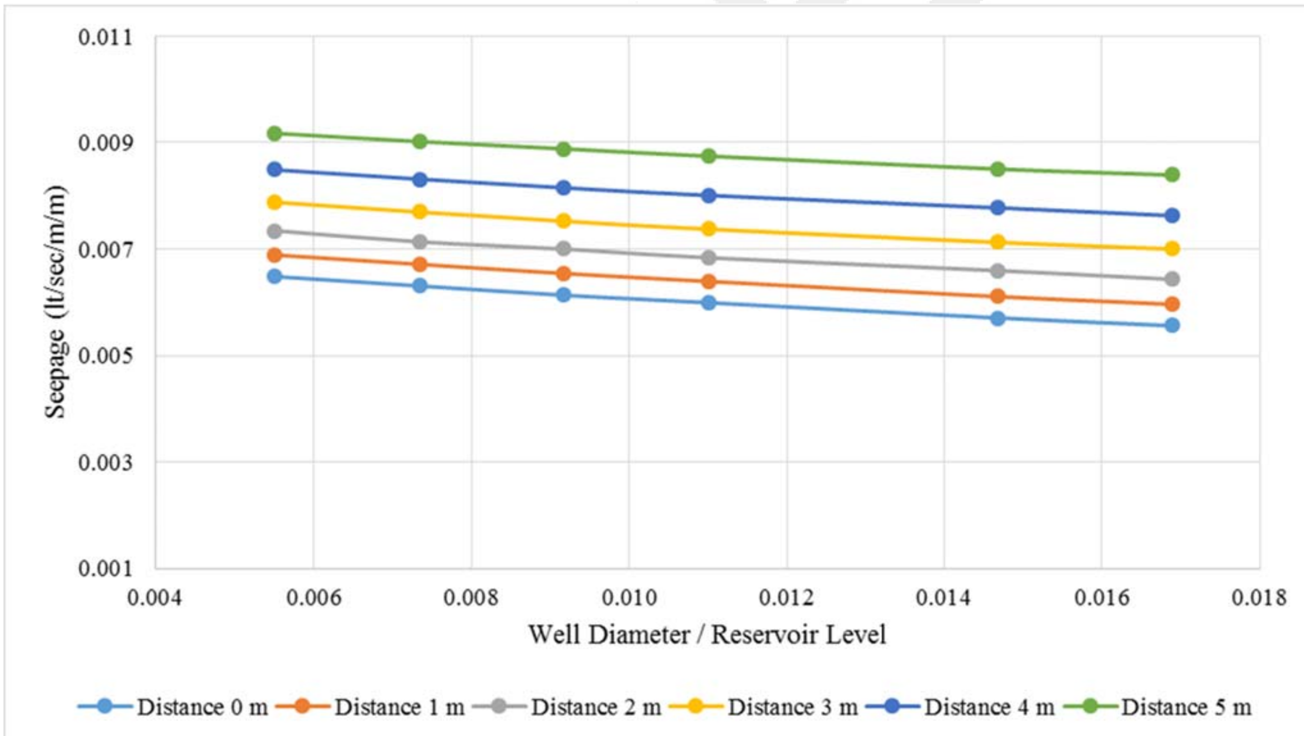

Figure 3.23. Effect of inner well diameter on seepage (15 m spacing)

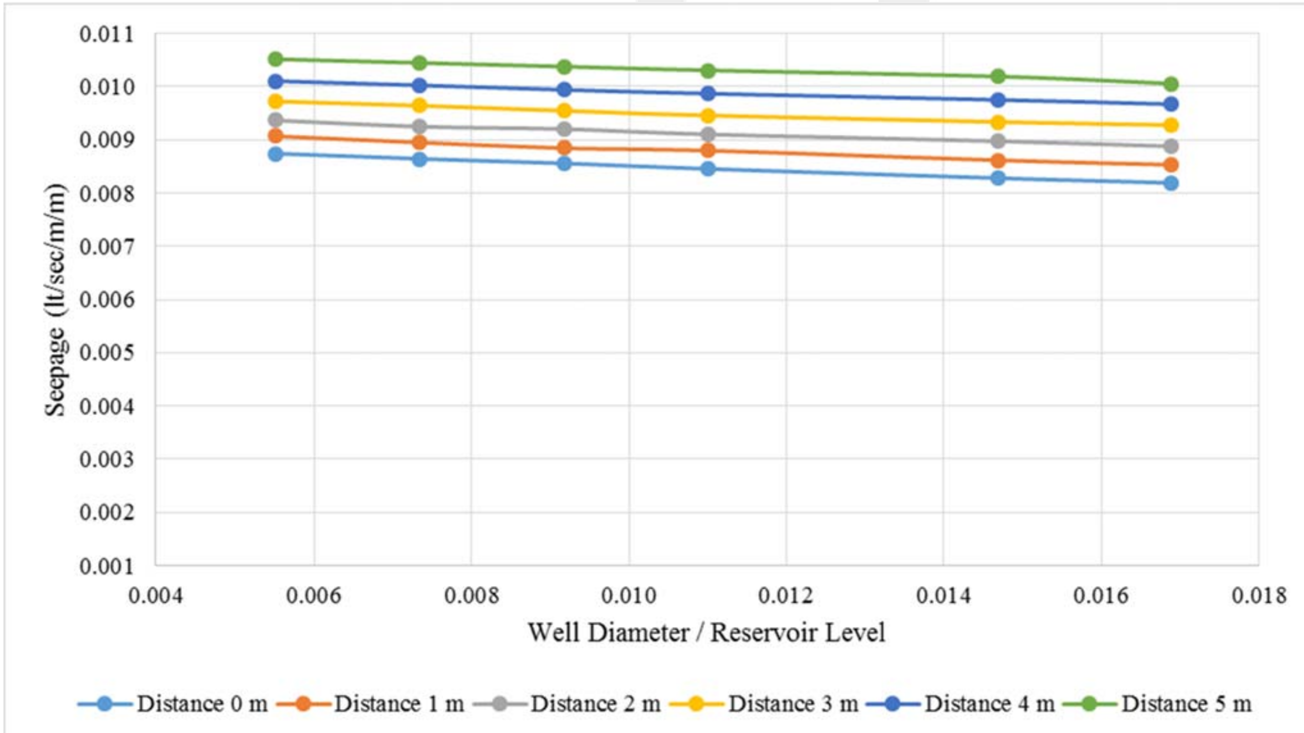

Figure 3.24. Effect of inner well diameter on seepage (30 m spacing)

The change of amount of collected water by the wells with respect to the change of inner well diameter/reservoir level ratio is shown on Figure 3.25 through Figure 3.30. The effect of inner well diameter is evaluated with the stated graphical results. These figures show that the wells closest to the toe of the dam are collected more water than others and the collected water amount decreased with the increase of the distance of the wells from the toe. The increase of inner well diameter/reservoir level ratio resulted in increase of collected water amount as 9.8% for the wells at the toe but for the case with wells 5 m from the toe, the increase of collected water amount is around 21.8% for 5 m spacing as shown in the Figure 3.25. Figure 3.30 implies that the increase of inner well diameter/reservoir level ratio resulted in increase of collected water by wells as 16.7% for the case with wells at the toe and 30.1% for the case with wells 5 m from the toe with spacing of 30 m.

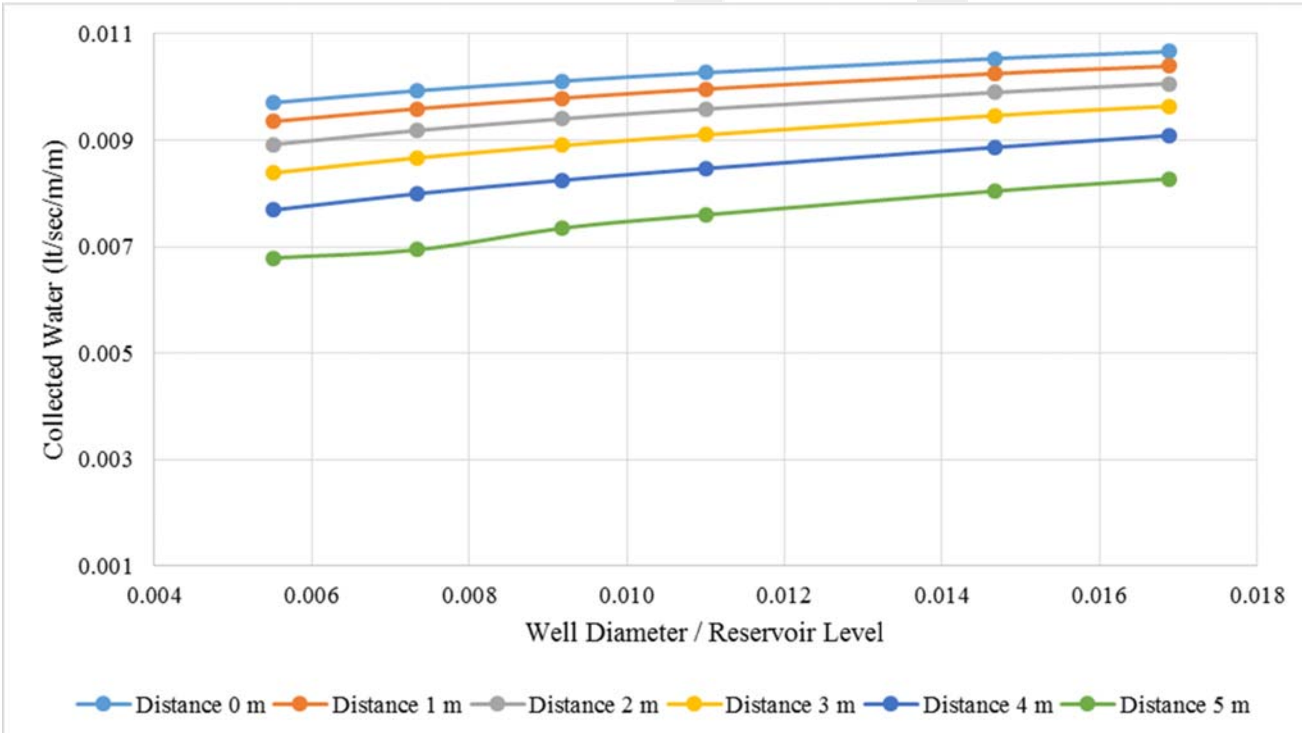

Figure 3.25. Effect of inner well diameter on collected water (5 m spacing)

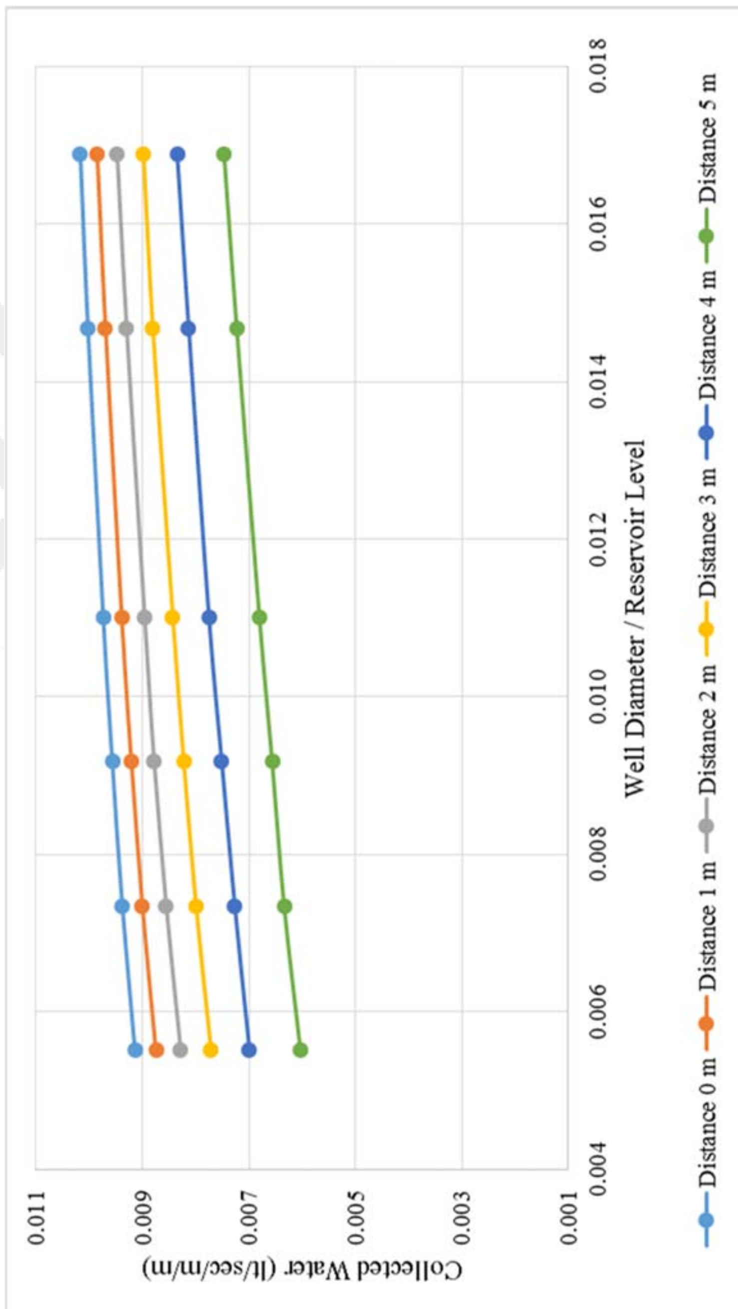

Figure 3.26. Effect of inner well diameter on collected water (6 m spacing)

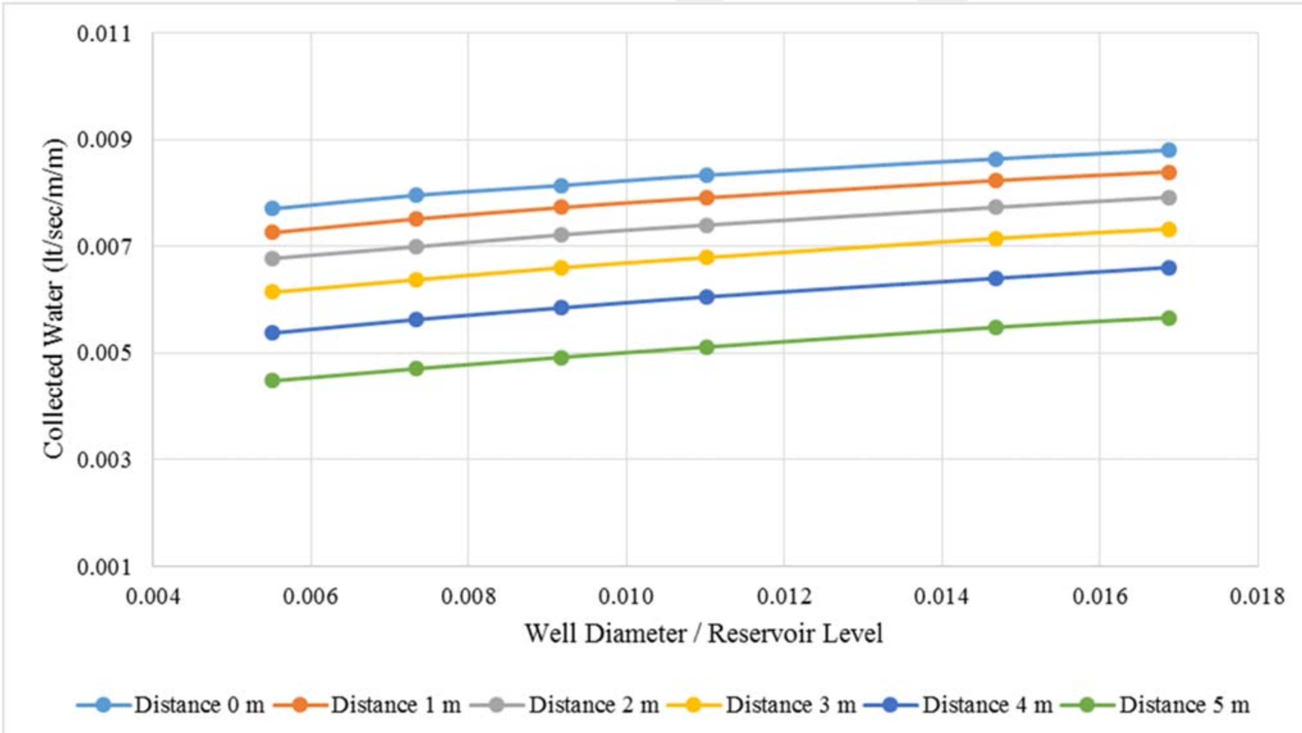

Figure 3.27. Effect of inner well diameter on collected water (9 m spacing)

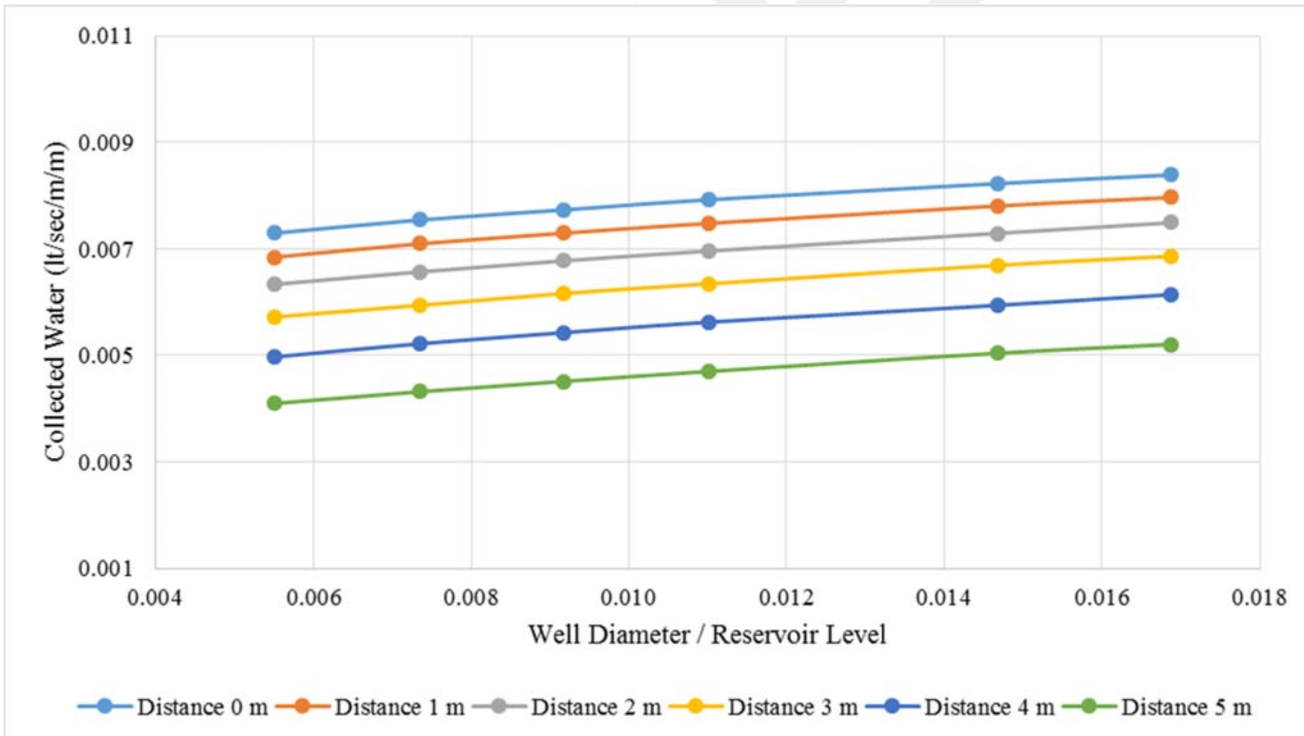

Figure 3.28. Effect of inner well diameter on collected water (10 m spacing)

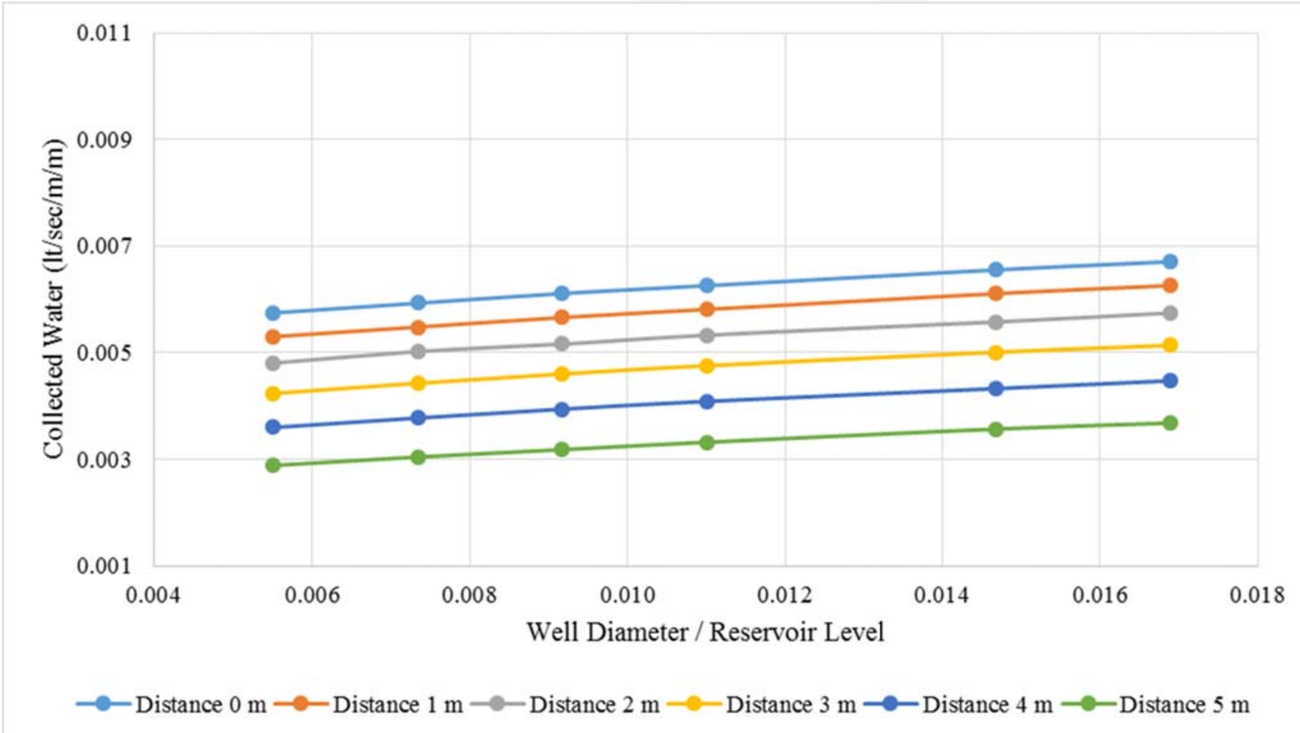

Figure 3.29. Effect of inner well diameter on collected water (15 m spacing)

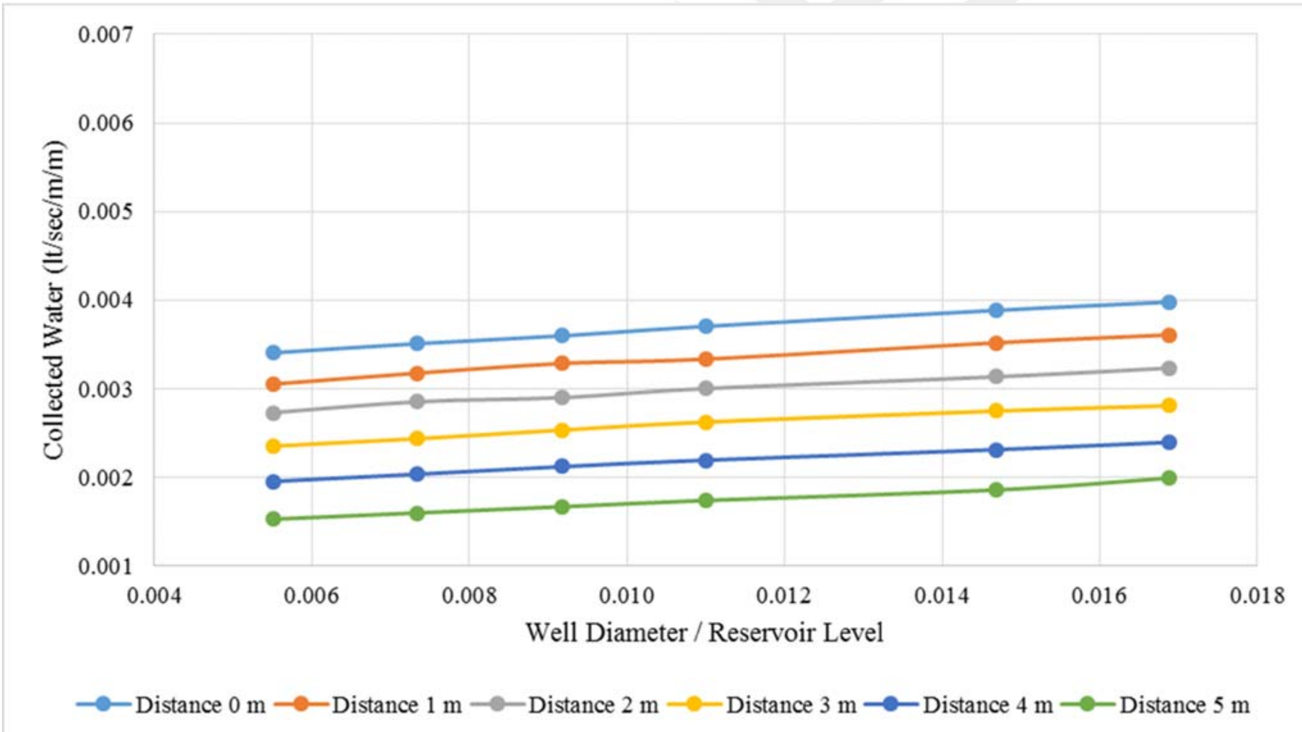

Figure 3.30. Effect of inner well diameter on collected water (30 m spacing)

3.6 Pore Water Pressure Analyses

Pore water pressures at the foundation of the downstream toe of the dam are also investigated in the scope of this study. The pore water pressure data are collected from the plan view analysis and are the pore water pressure values of selected points which are (202, 45), (205,28), and (209,70). The point (202,45) is chosen as it is at the center of the analyzed section of the dam as shown in Figure 3.31. The center is chosen to minimize the error that may occur due to location, this point is of equal distance away from two borders of the analysis section. The obtained results for the pore water pressures are presented in Figures 3.32 through 3.43. In Figures 3.32 through 3.37, change of pore water pressure is plotted with the increase of relief wells inner diameter/reservoir level ratio. It is clear that, in all of the cases, the pore water pressures decrease with the increase of inner well diameter. The rate of change varies from 4.94% at relief wells with 15 m spacing at the downstream toe to 0.25% at the relief wells with 30 m spacing located 5 m from the downstream toe of the dam. Figures 3.32, 3.33, 3.35 and 3.37 indicate that when the spacing increases, the pore water pressure data approach to higher values. Exceptionally on Figure 3.34 and Figure 3.36, the spacing value is a divider of 45, which is the point (202, 45)'s ordinate, the wells are located right after that point. This situation yields to a lower pore water pressure on highlighted point as the wells location is closer to the chosen point (202, 45). When the point is located closer to the wells, the pore water pressure decreases due to effect of pressure relief wells.

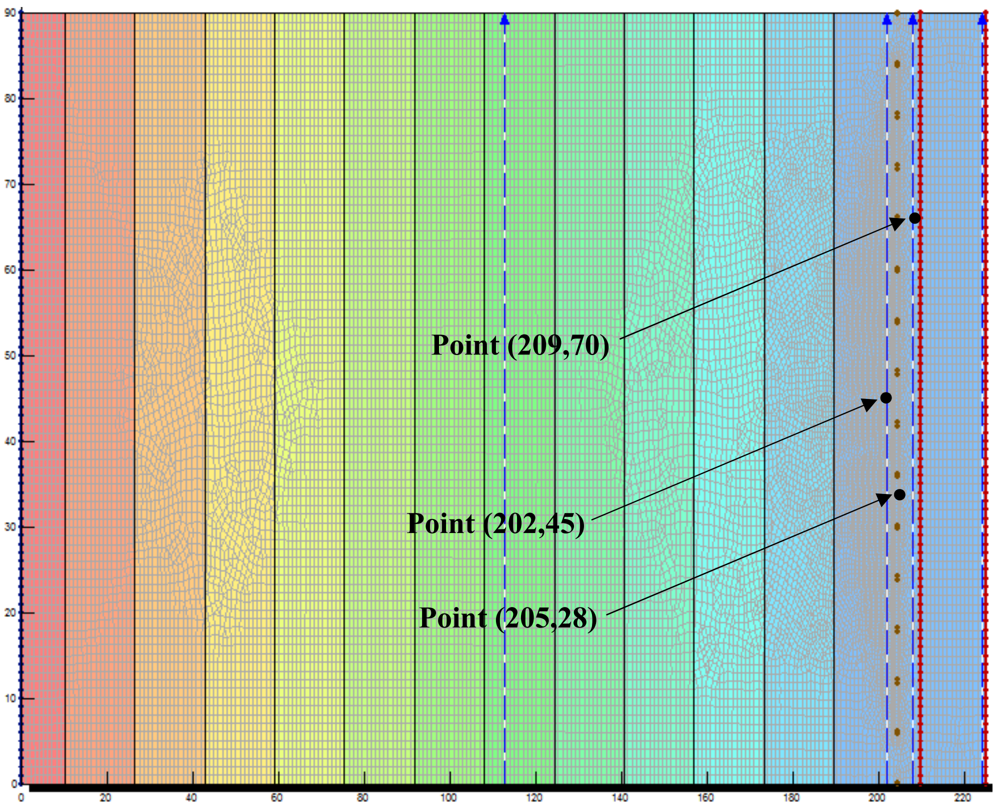

Figure 3.31. Location of the collected pore water pressure data

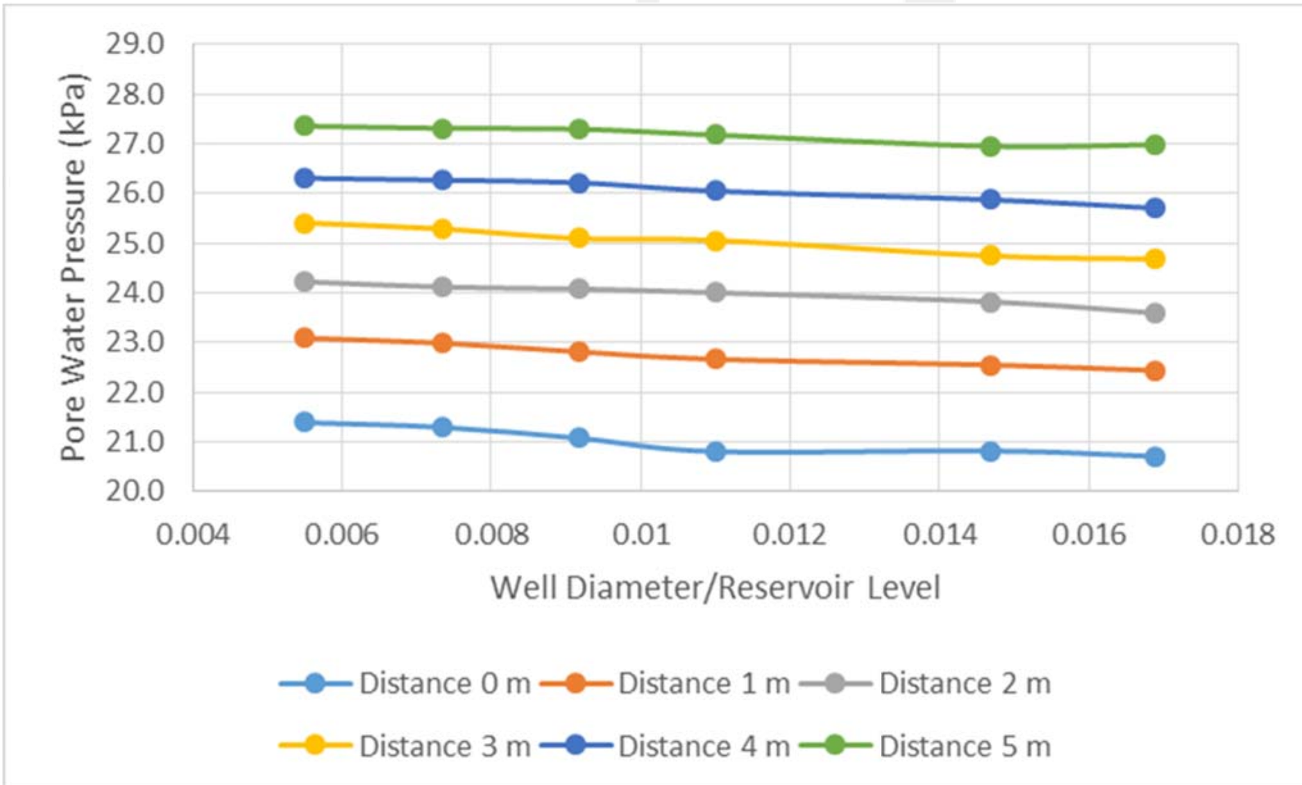

Figure 3.32. Relief well inner diameter and pore water pressure (5 m spacing)

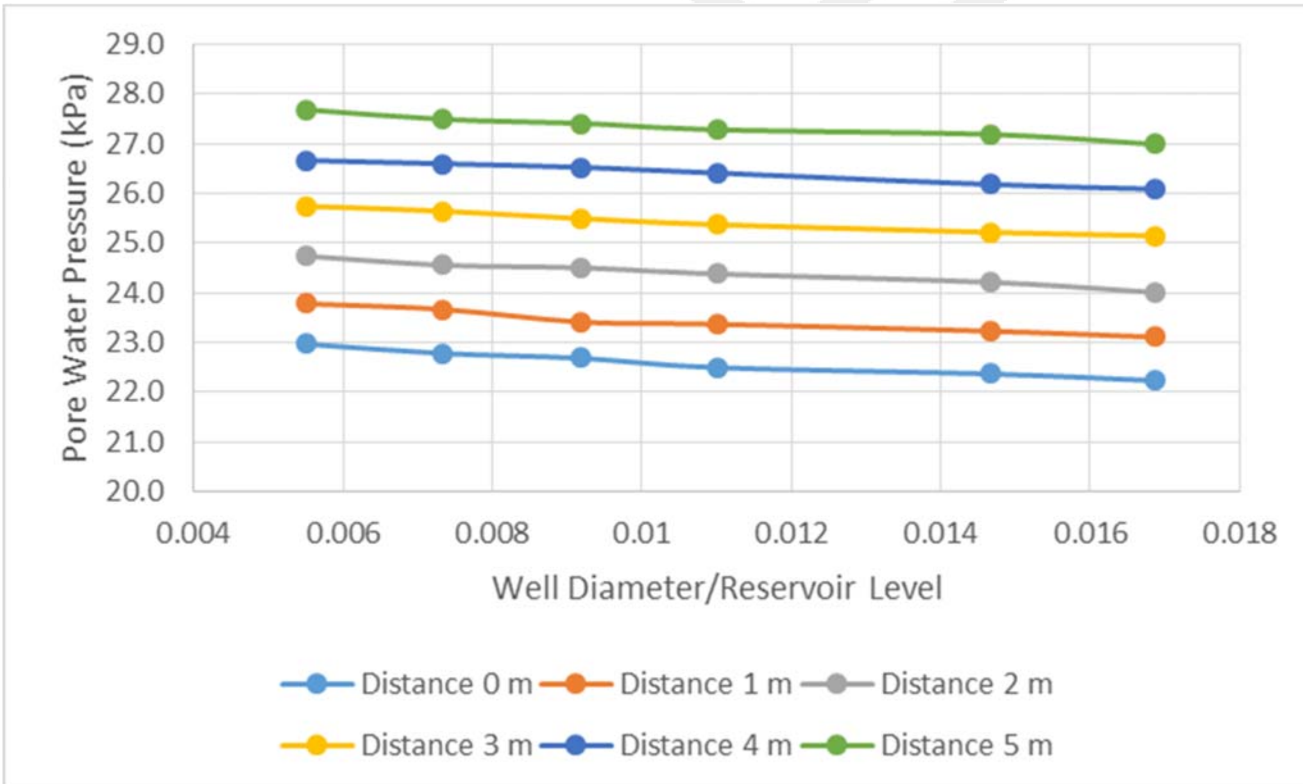

Figure 3.33. Relief well inner diameter and pore water pressure (6 m spacing)

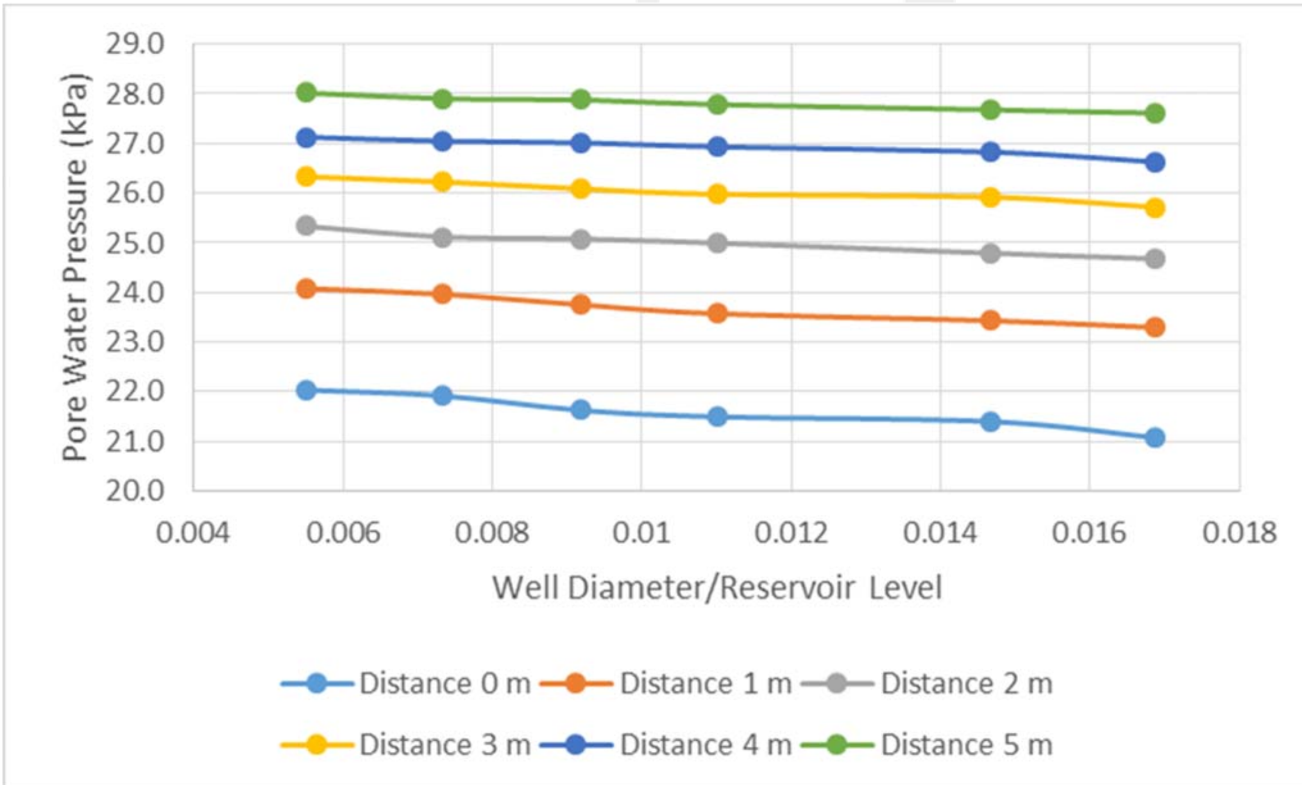

Figure 3.34. Relief well inner diameter and pore water pressure (9 m spacing)

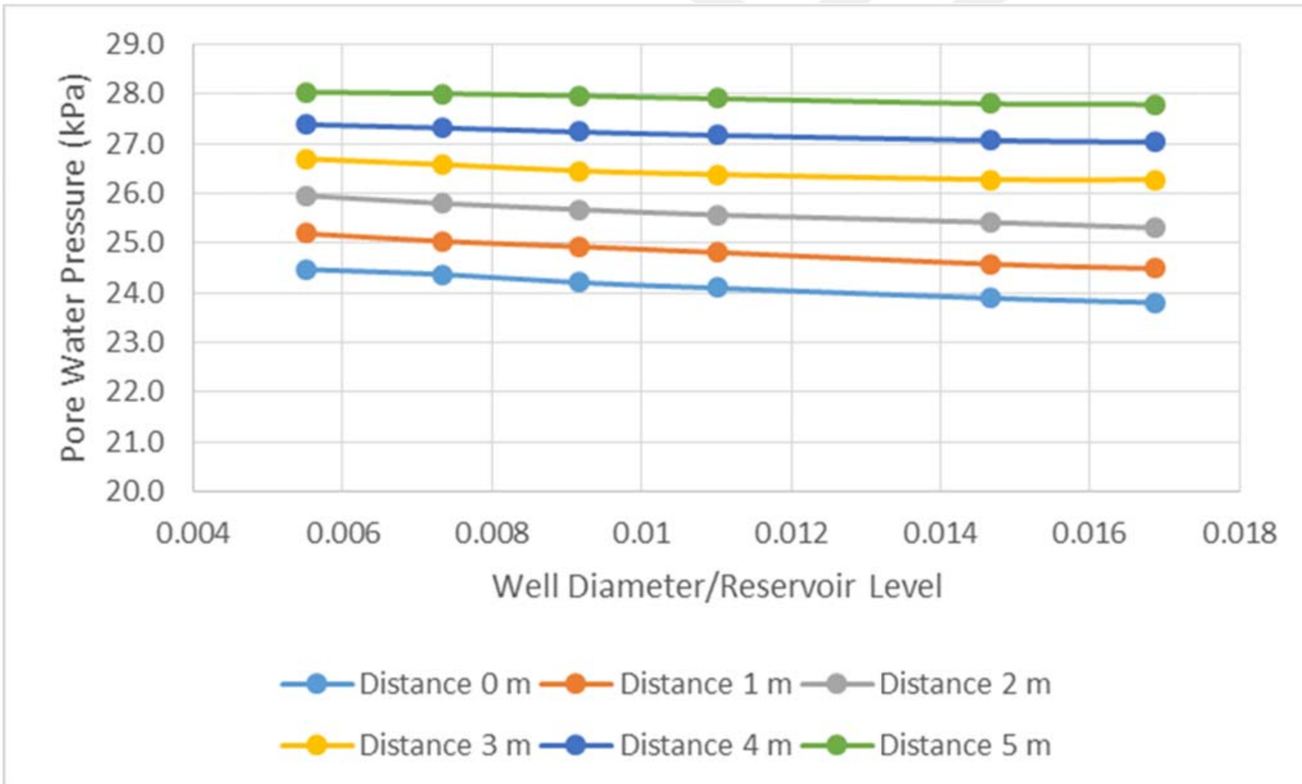

Figure 3.35. Relief well inner diameter and pore water pressure (10 m spacing)

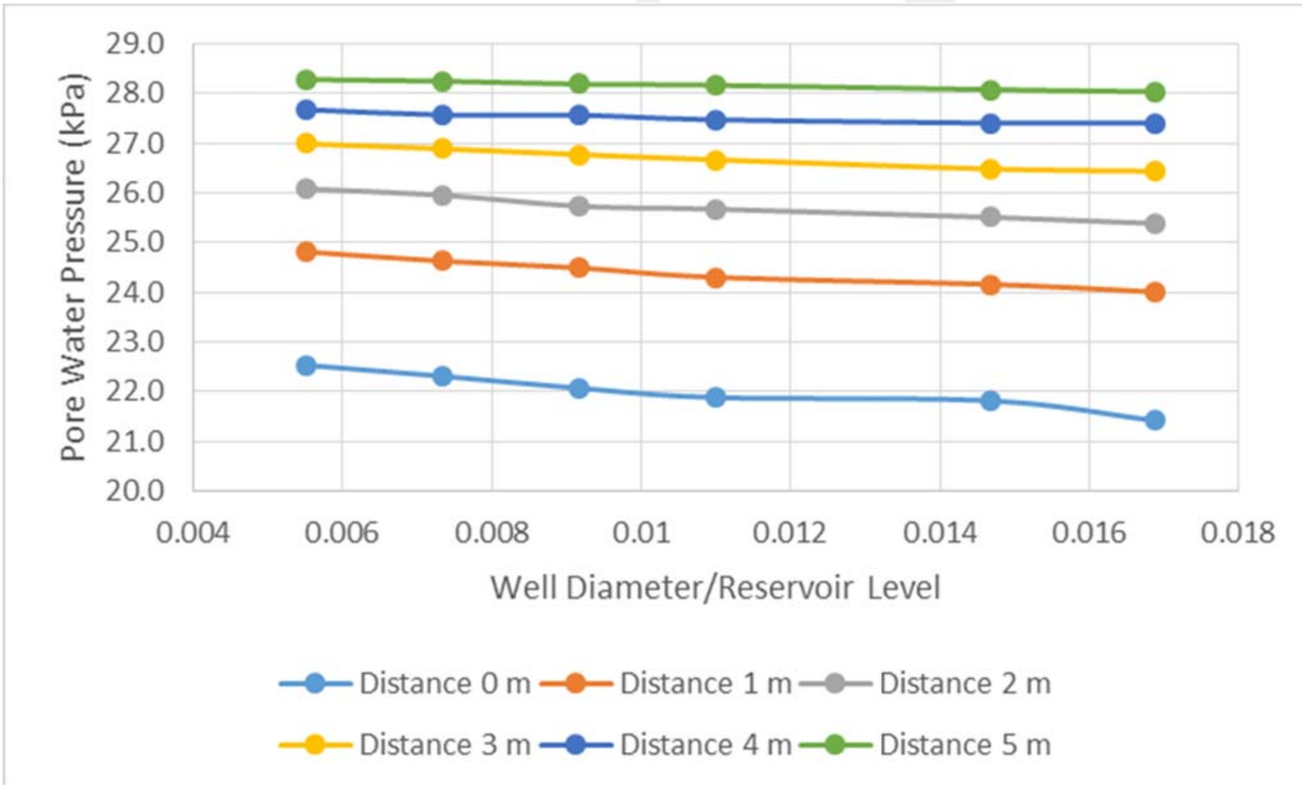

Figure 3.36. Relief well inner diameter and pore water pressure (15 m spacing)

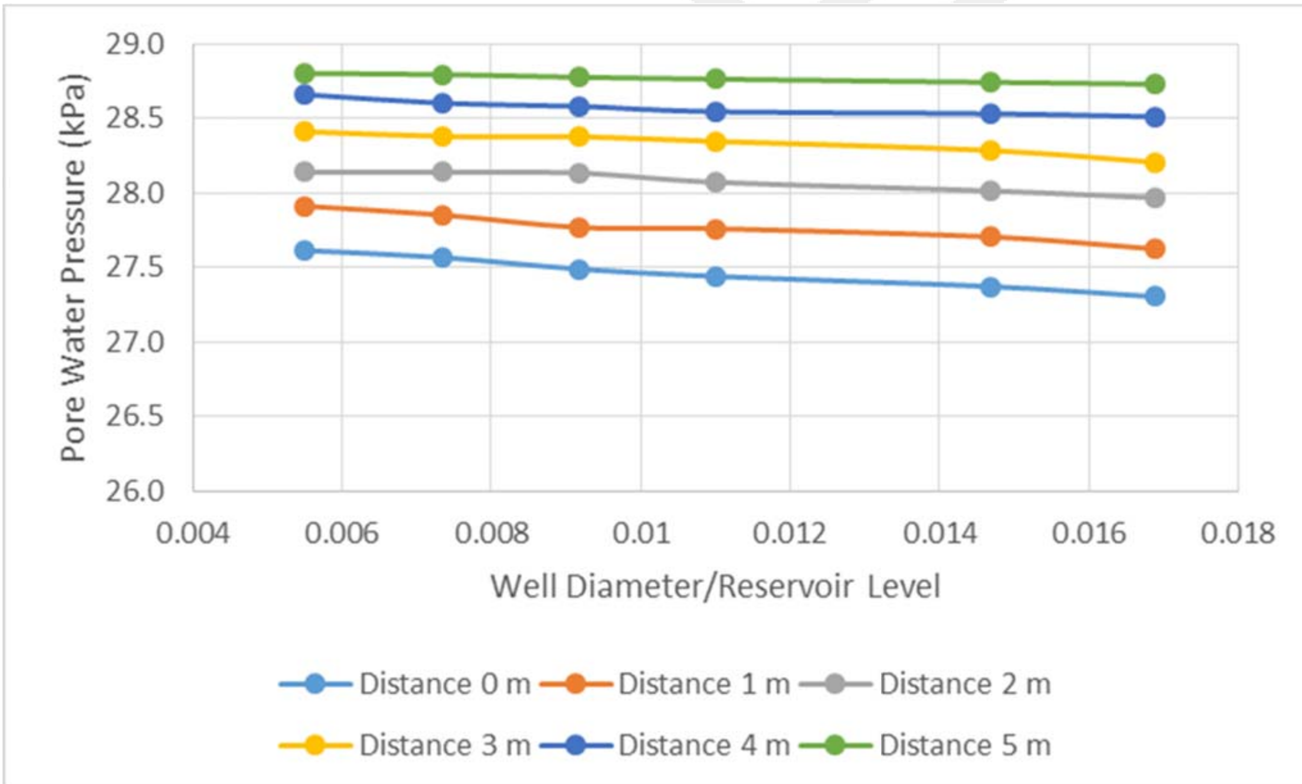

Figure 3.37. Relief well inner diameter and pore water pressure (30 m spacing)

Figures 3.38 through 3.43 show the effect of spacing/reservoir level ratio to the pore water pressure. Increase of spacing/reservoir level ratio yields to an increase of pore water pressure in all of the cases. There are exceptions again with spacing values at the dividers of 45 as stated previously, i.e. 5 m, 9 m and 15 m. Those exceptions are clearly visible on Figure 3.38 and Figure 3.39. The change of pore water pressure with the effect of spacing/reservoir level ratio varies from 5.29% on relief wells 5 m from the toe with an inner diameter of 0.15 m to 31.92% on relief wells located at the toe with an inner diameter of 0.30 m. As the distance from the toe increases, the importance of spacing/reservoir level ratio decreases at it is seen from the graphs. When the distance from the toe increases, the graphs become more linear and the change of pore water pressure decreases.

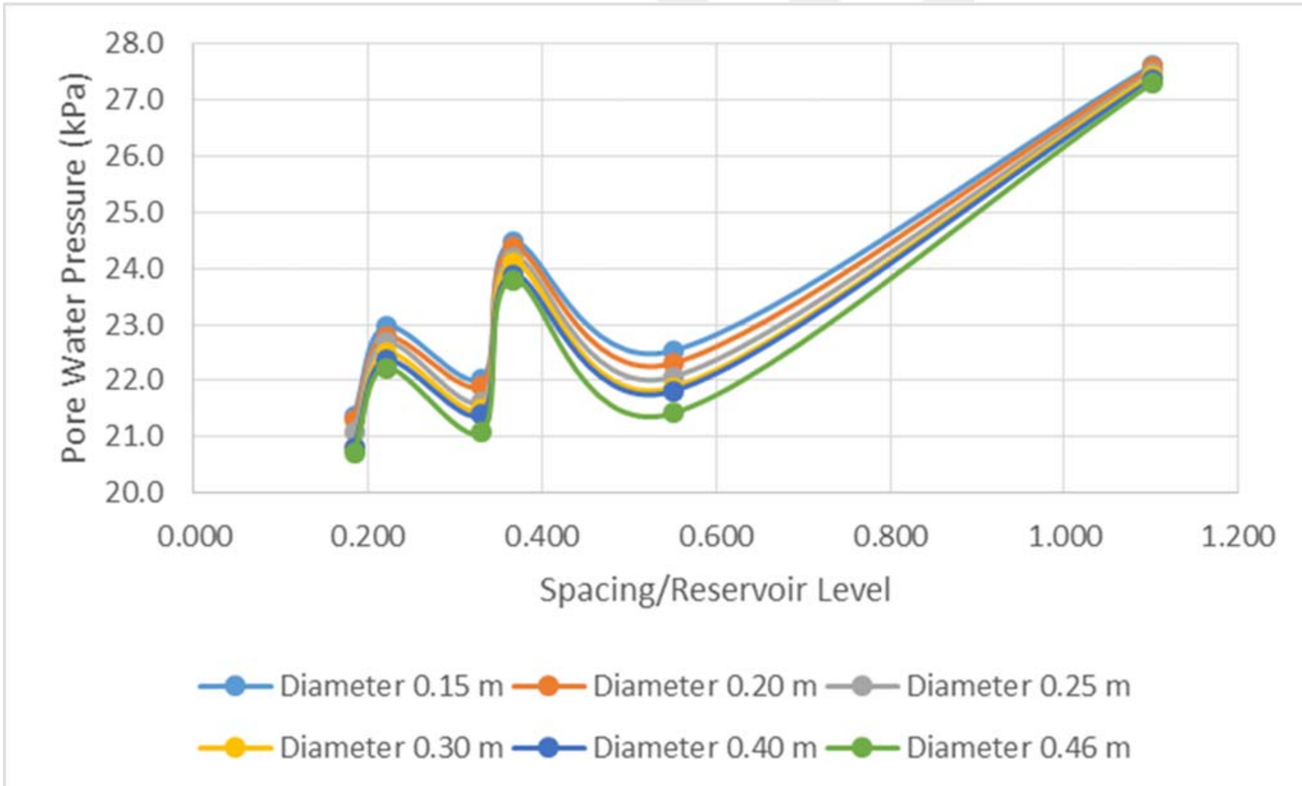

Figure 3.38. Relief well spacing and pore water pressure (wells located at the toe)

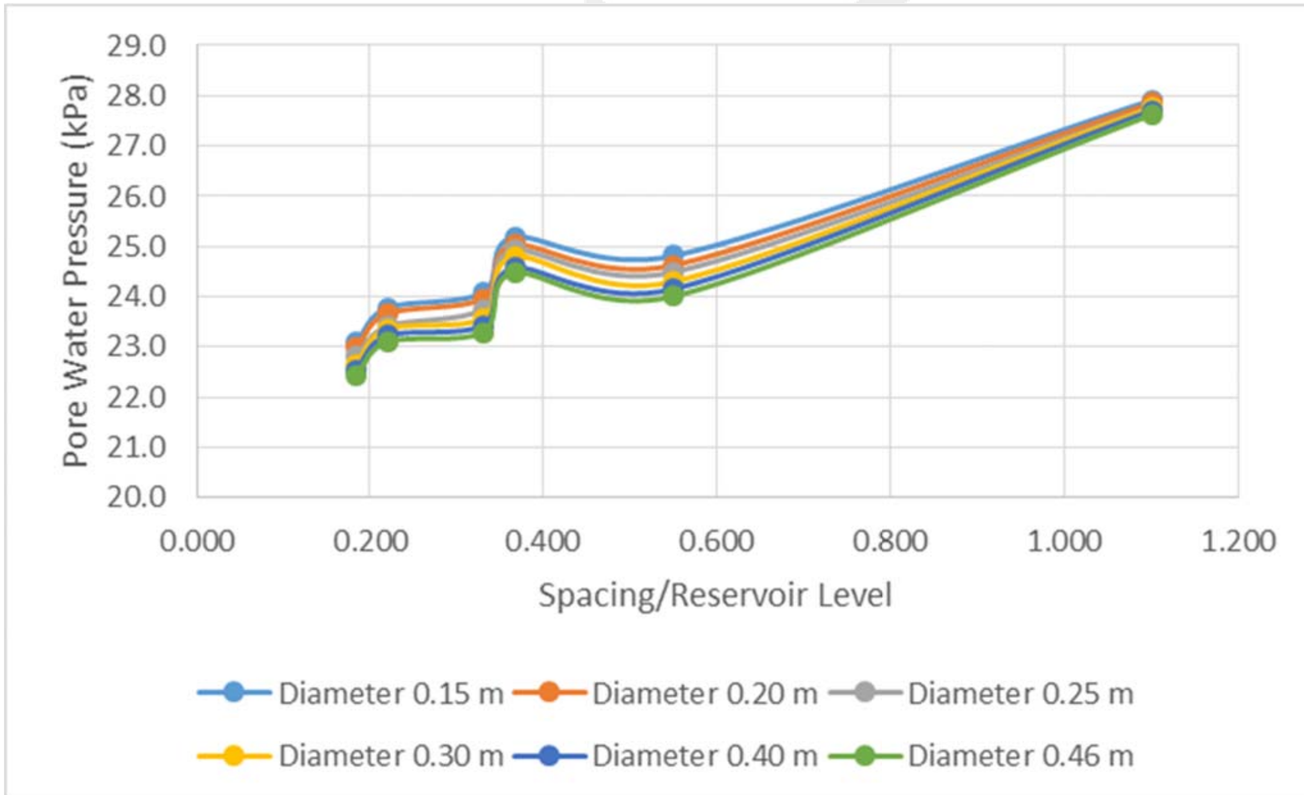

Figure 3.39. Relief well spacing and pore water pressure (wells located 1 m from the toe)

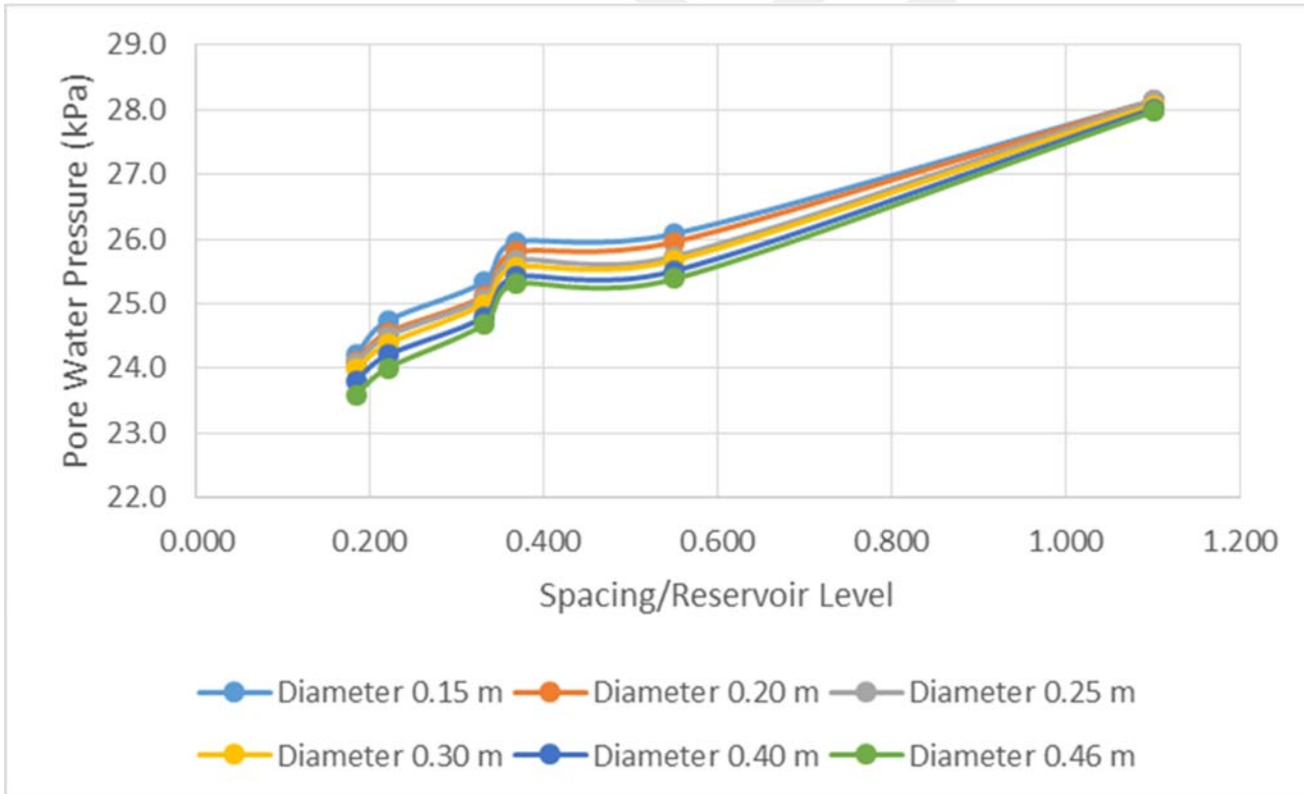

Figure 3.40. Relief well spacing and pore water pressure (wells located 2 m from the toe)

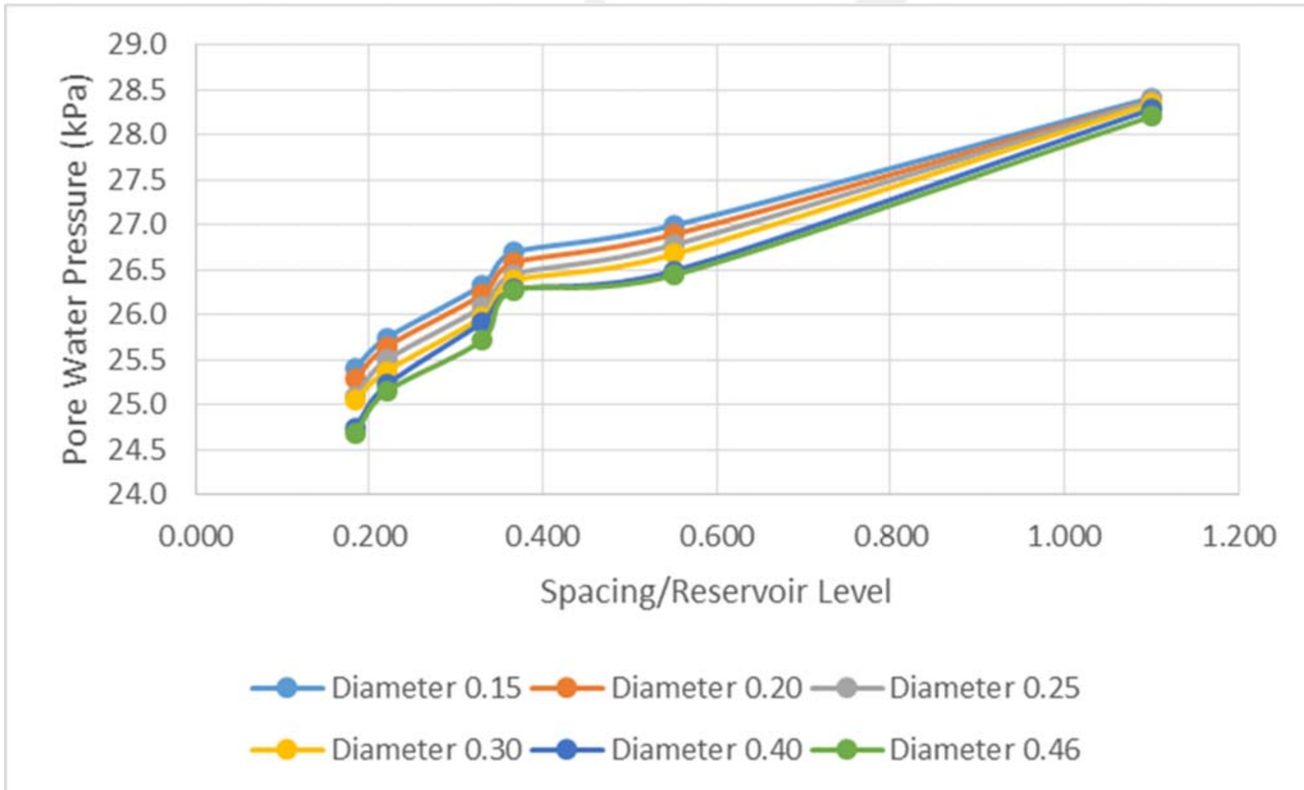

Figure 3.41. Relief well spacing and pore water pressure (wells located 3 m from the toe)

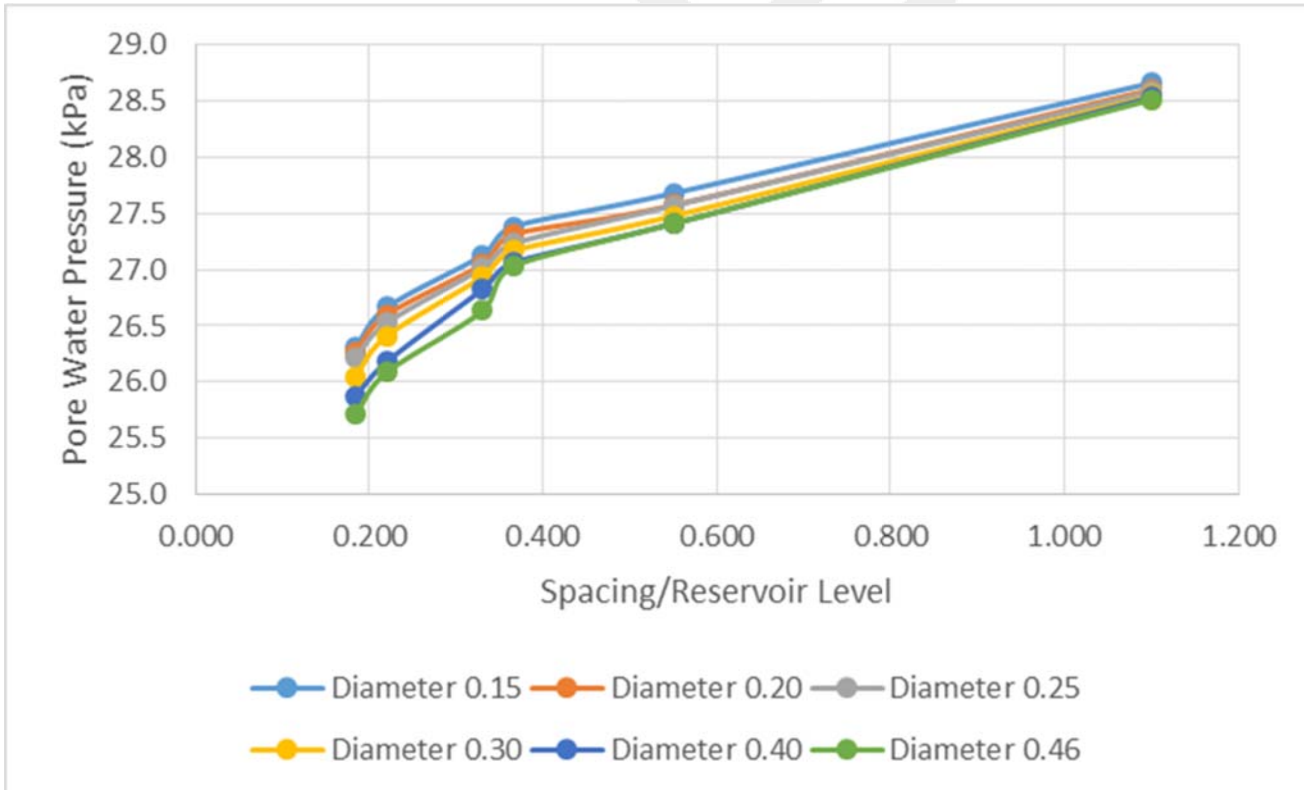

Figure 3.42. Relief well spacing and pore water pressure (wells located 4 m from the toe)

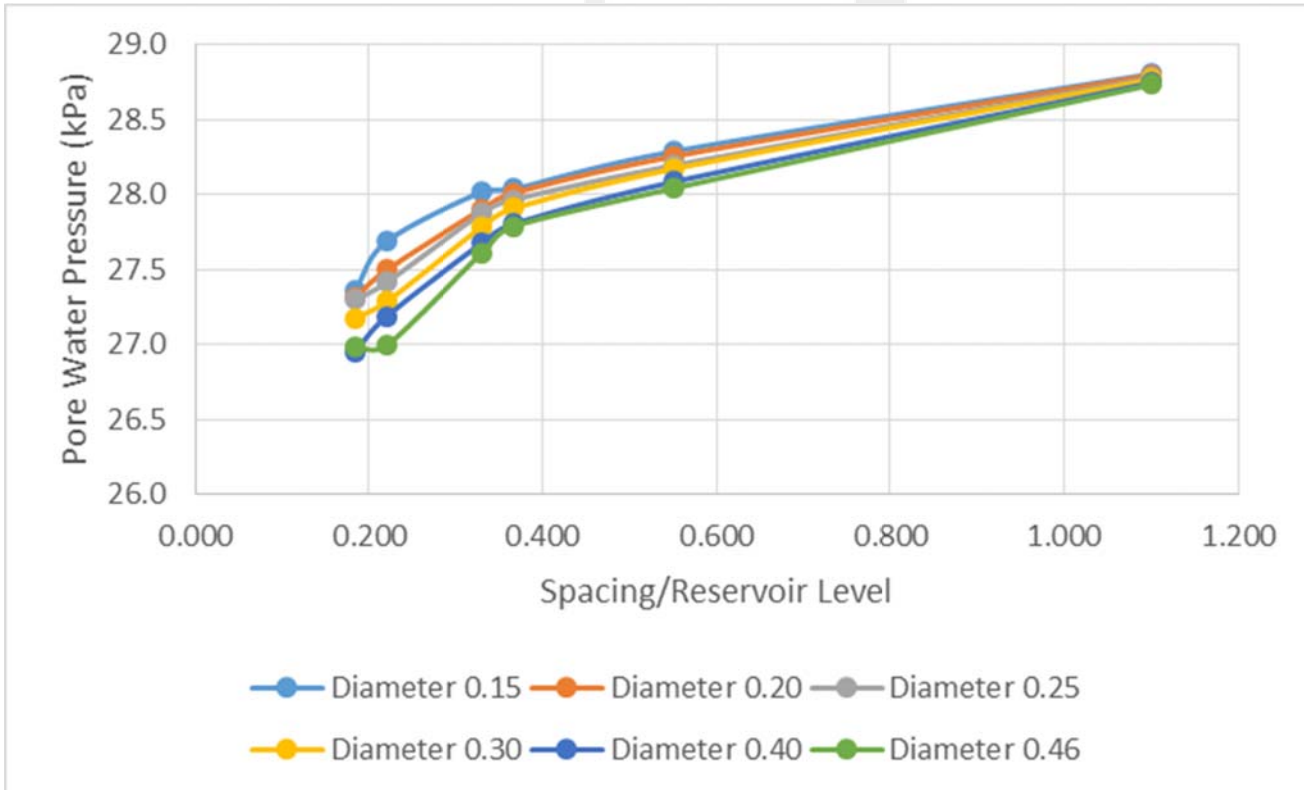

Figure 3.43. Relief well spacing and pore water pressure (wells located 5 m from the toe)

In Figures 3.44 through 3.49, the change of pore water pressure is plotted with the increase of relief wells inner diameter/reservoir level ratio at the point (205, 28). It is clear that, in all of the cases, the pore water pressures decrease with the increase of inner well diameter. The rate of change varies from 0.4% at relief wells with 30 m spacing located at 5 m from the downstream toe to 3% at the relief wells with 9 m spacing located 2 m from the downstream toe of the dam.

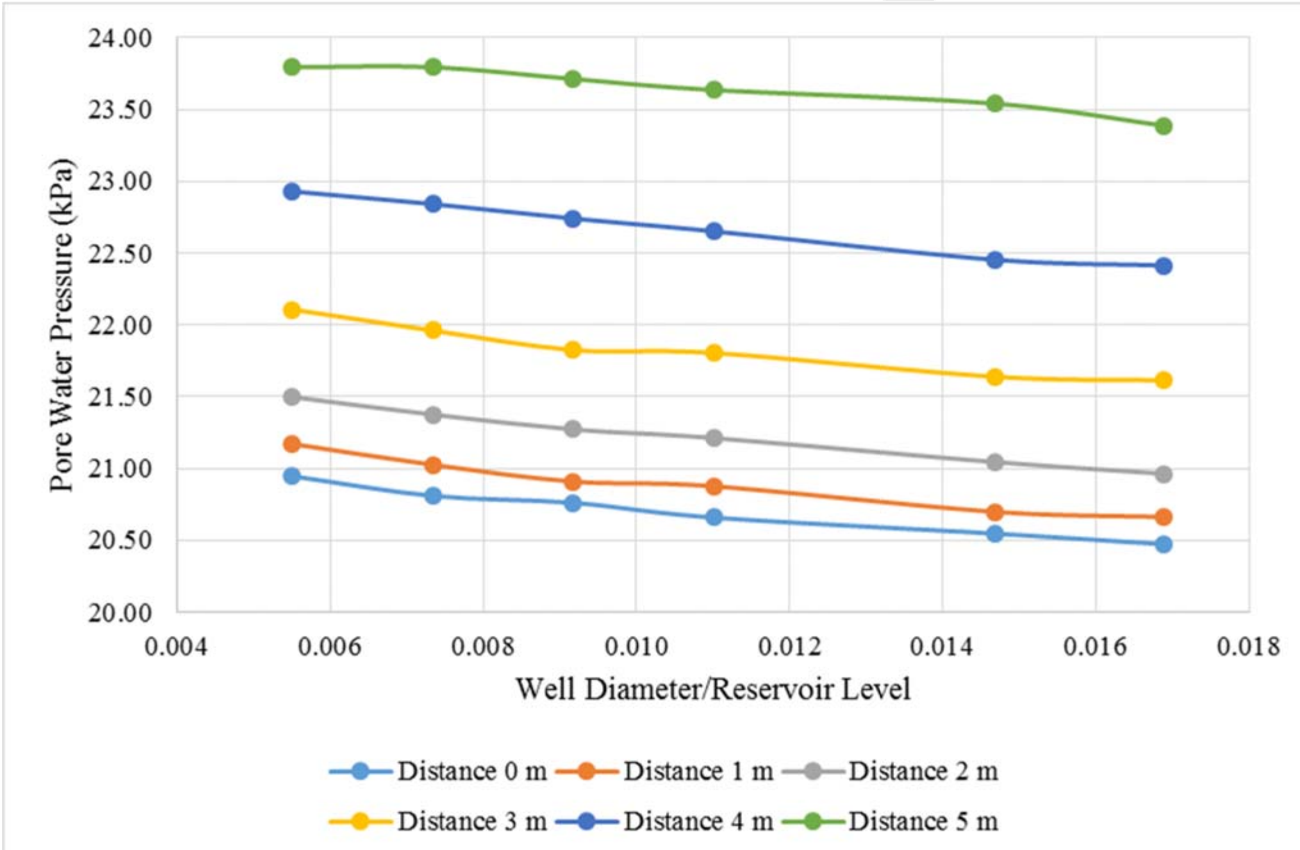

Figure 3.44. Relief well inner diameter and pore water pressure (5 m spacing)

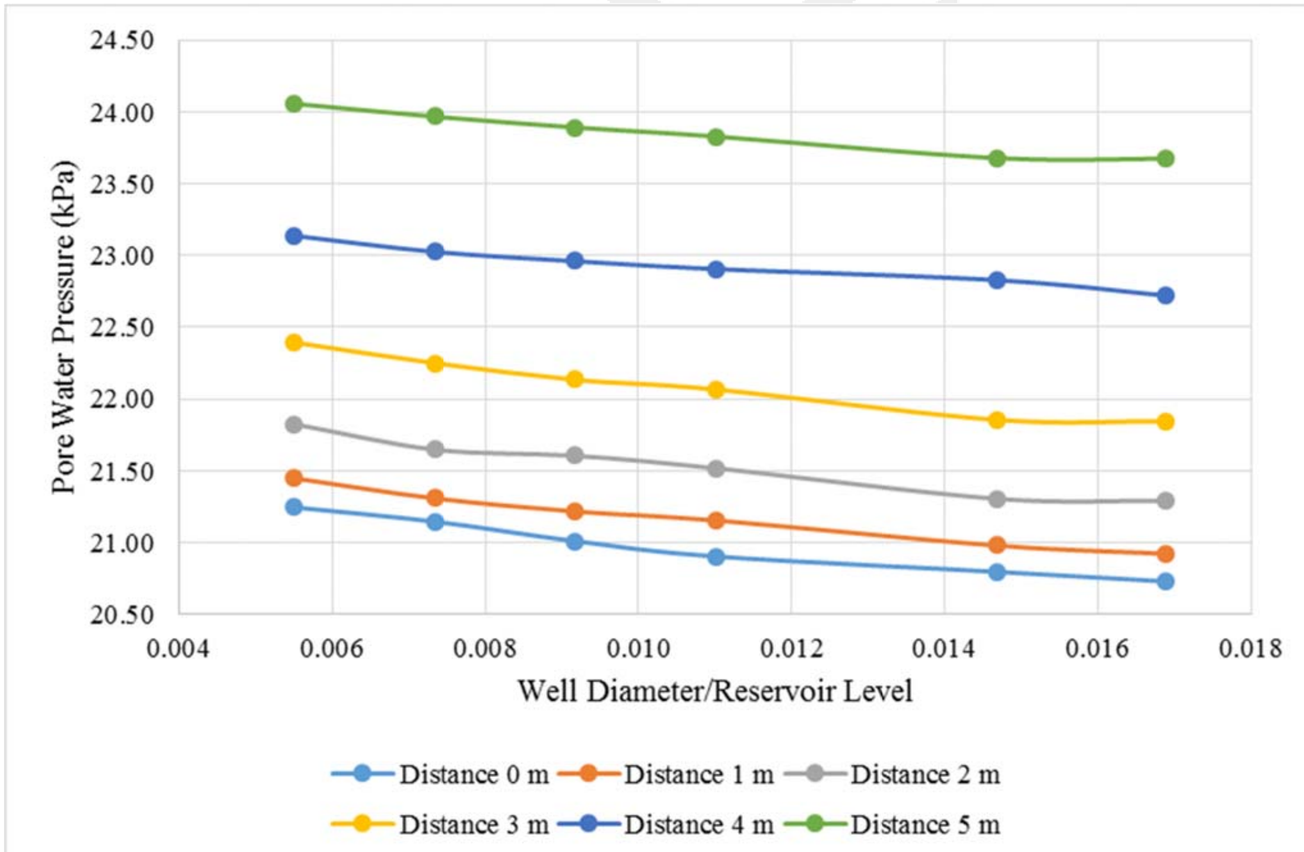

Figure 3.45. Relief well inner diameter and pore water pressure (6 m spacing)

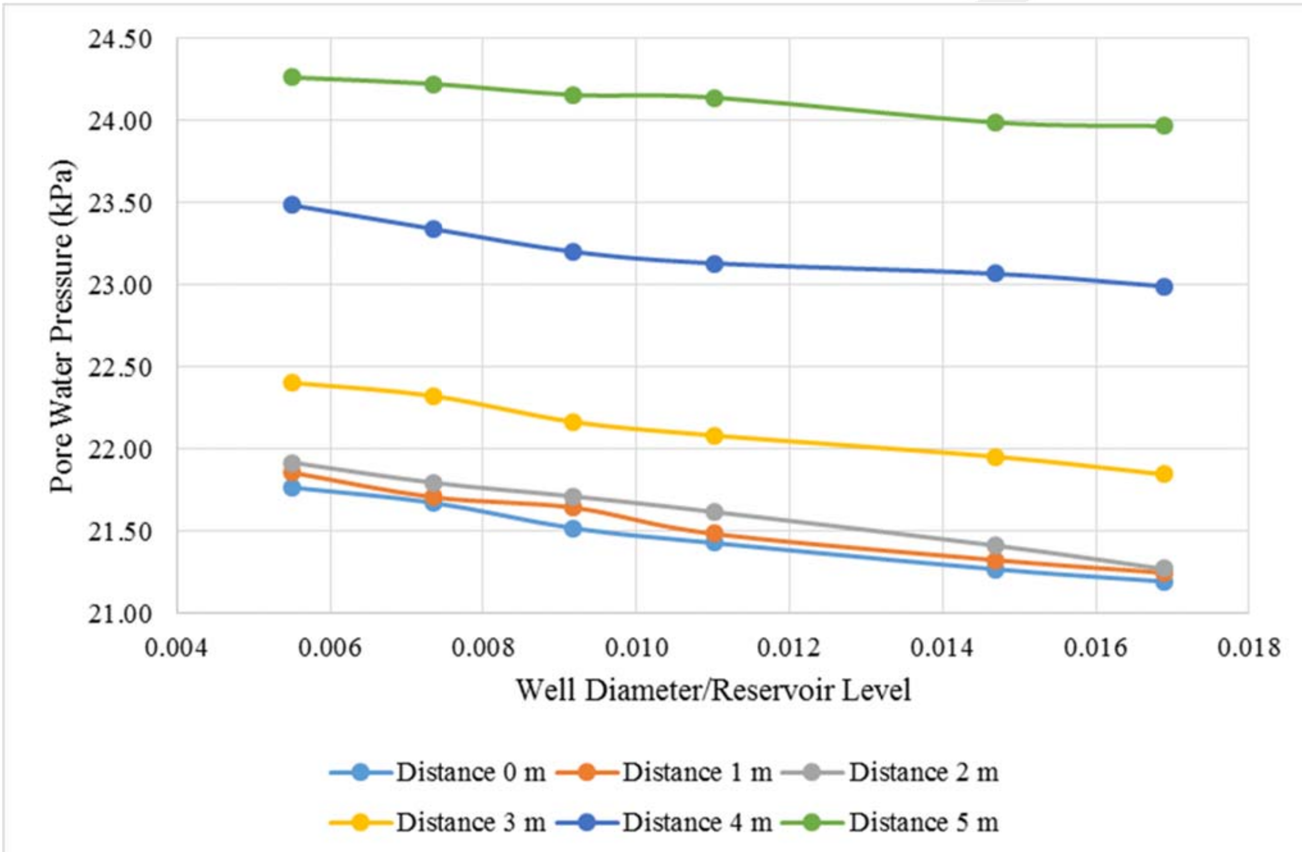

Figure 3.46. Relief well inner diameter and pore water pressure (9 m spacing)

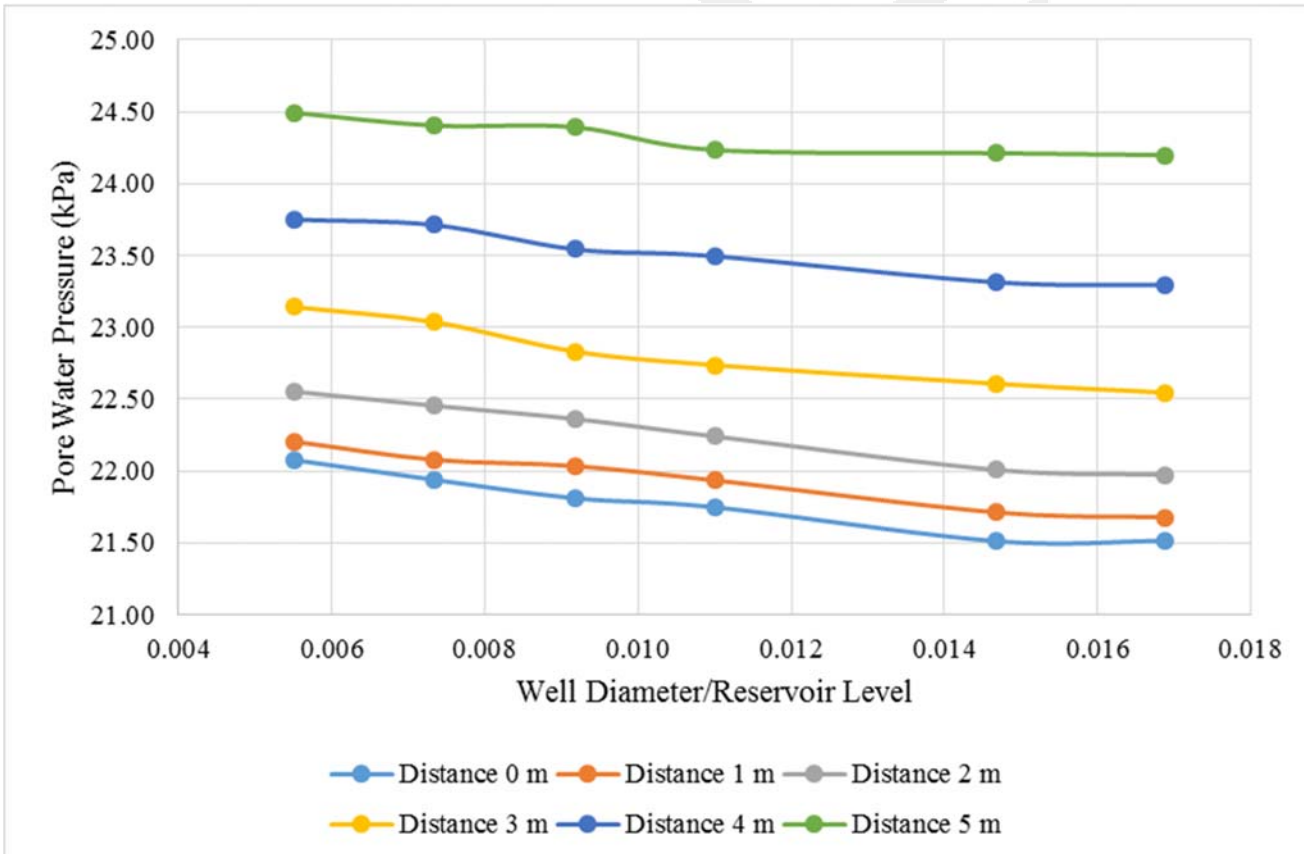

Figure 3.47. Relief well inner diameter and pore water pressure (10 m spacing)

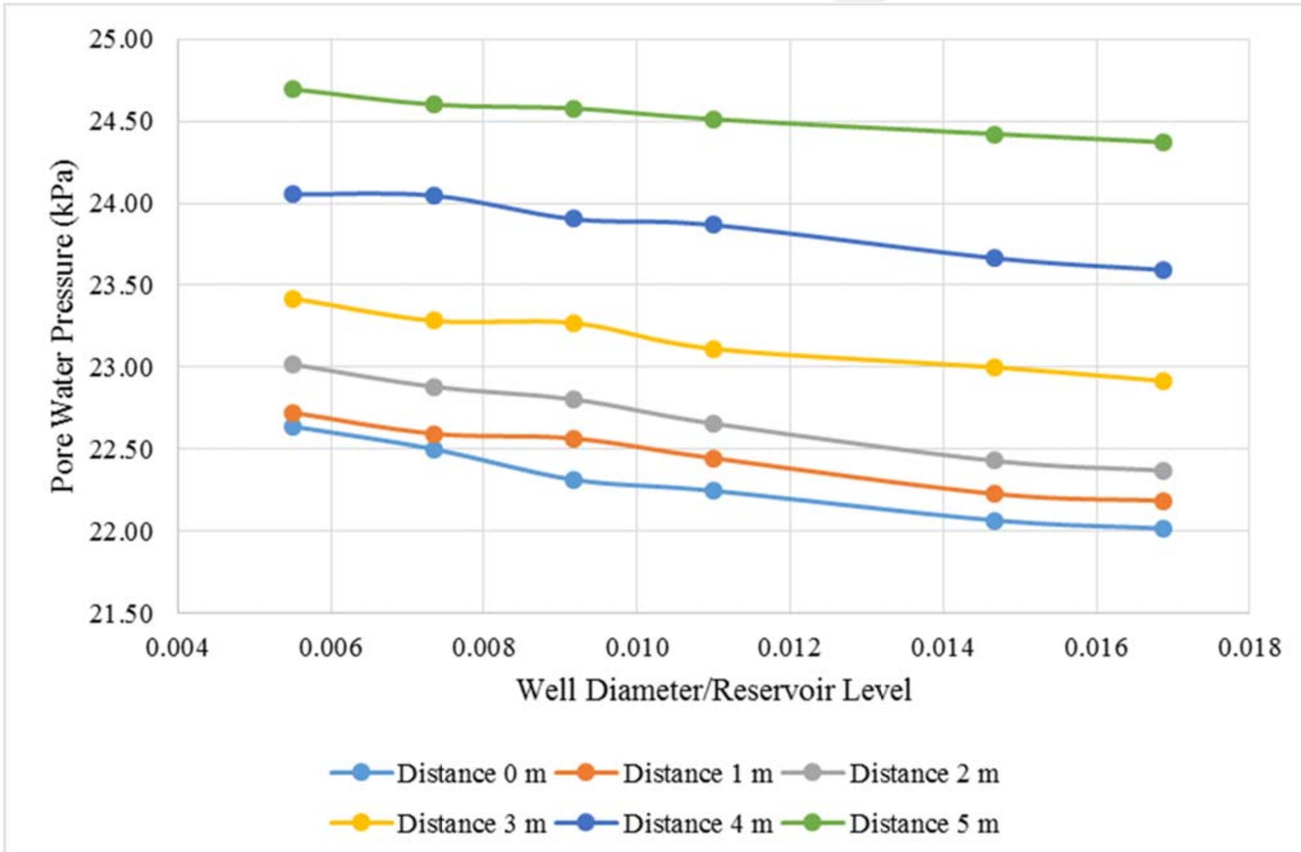

Figure 3.48. Relief well inner diameter and pore water pressure (15 m spacing)

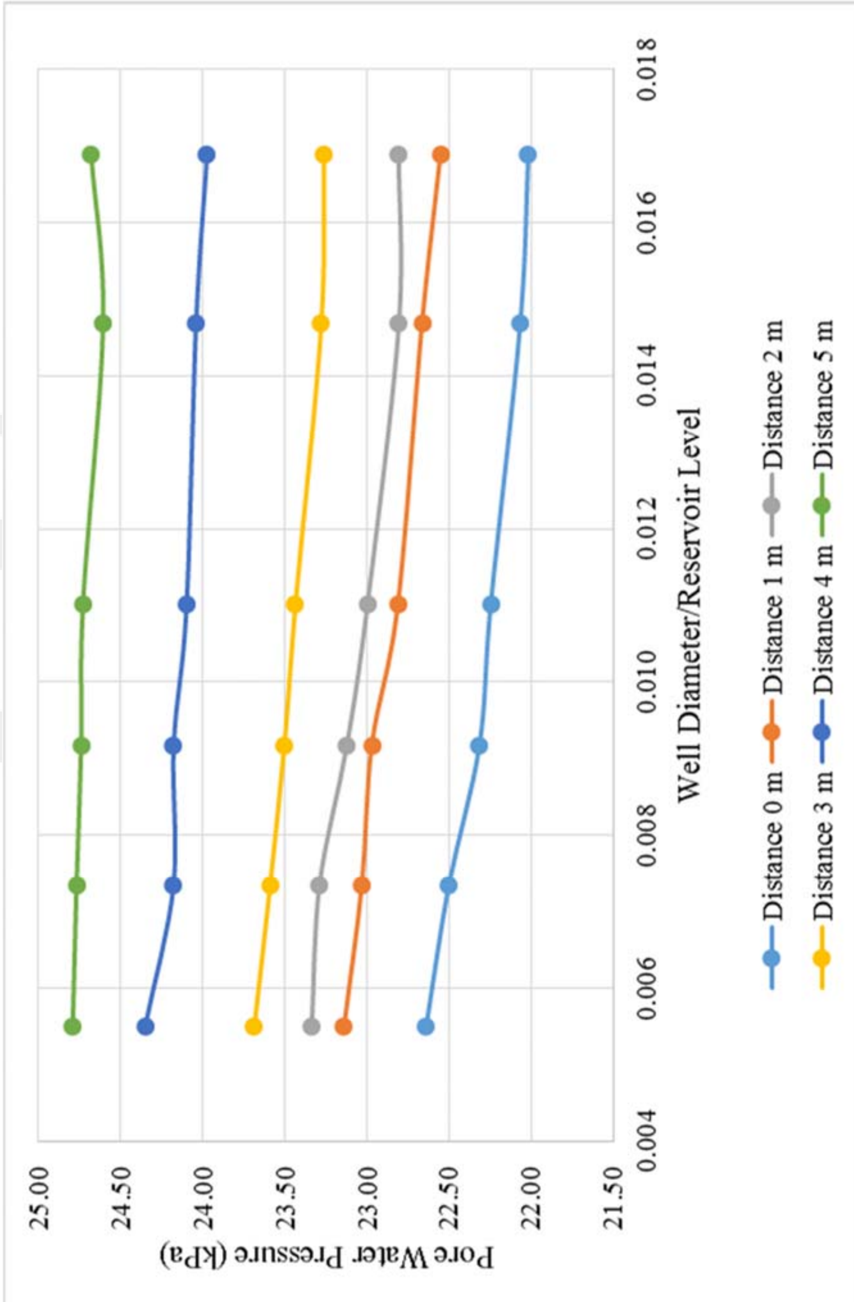

Figure 3.49. Relief well inner diameter and pore water pressure (30 m spacing)

Figures 3.50 through 3.55 show the effect of spacing/reservoir level ratio to the pore water pressure at the point (205, 28). Increase of spacing/reservoir level ratio yields to an increase of pore water pressure in all of the cases. The change of pore water pressure with the effect of spacing/reservoir level ratio varies from 3.9% on relief wells 5 m from the toe with an inner diameter of 0.20 m to 9.7% on relief wells located at the toe with an inner diameter of 0.20 m. As the distance from the toe increases, the importance of spacing/reservoir level ratio decreases as it is seen from the graphs. When the distance from the toe increases, the graphs become more linear and the change of pore water pressure decreases.

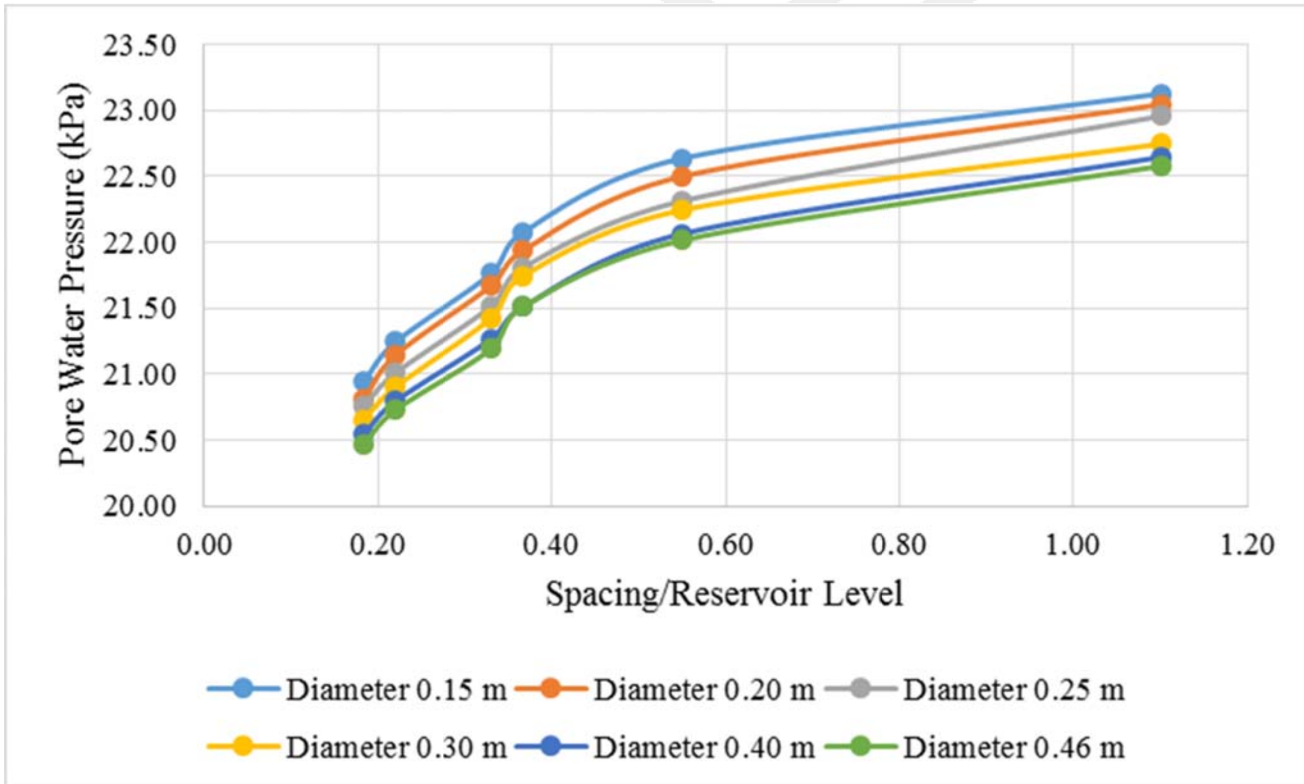

Figure 3.50. Relief well spacing and pore water pressure (wells located at the toe)

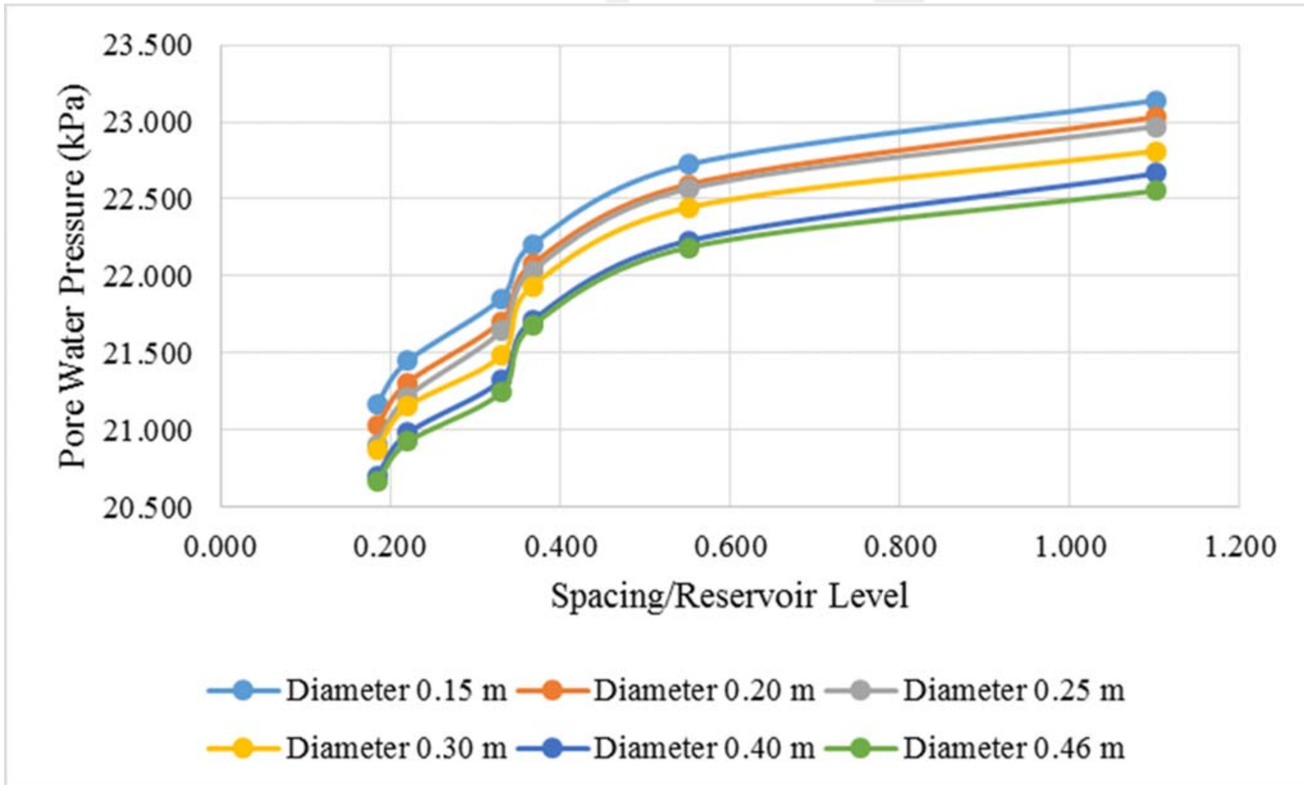

Figure 3.51. Relief well spacing and pore water pressure (wells located 1 m from the toe)

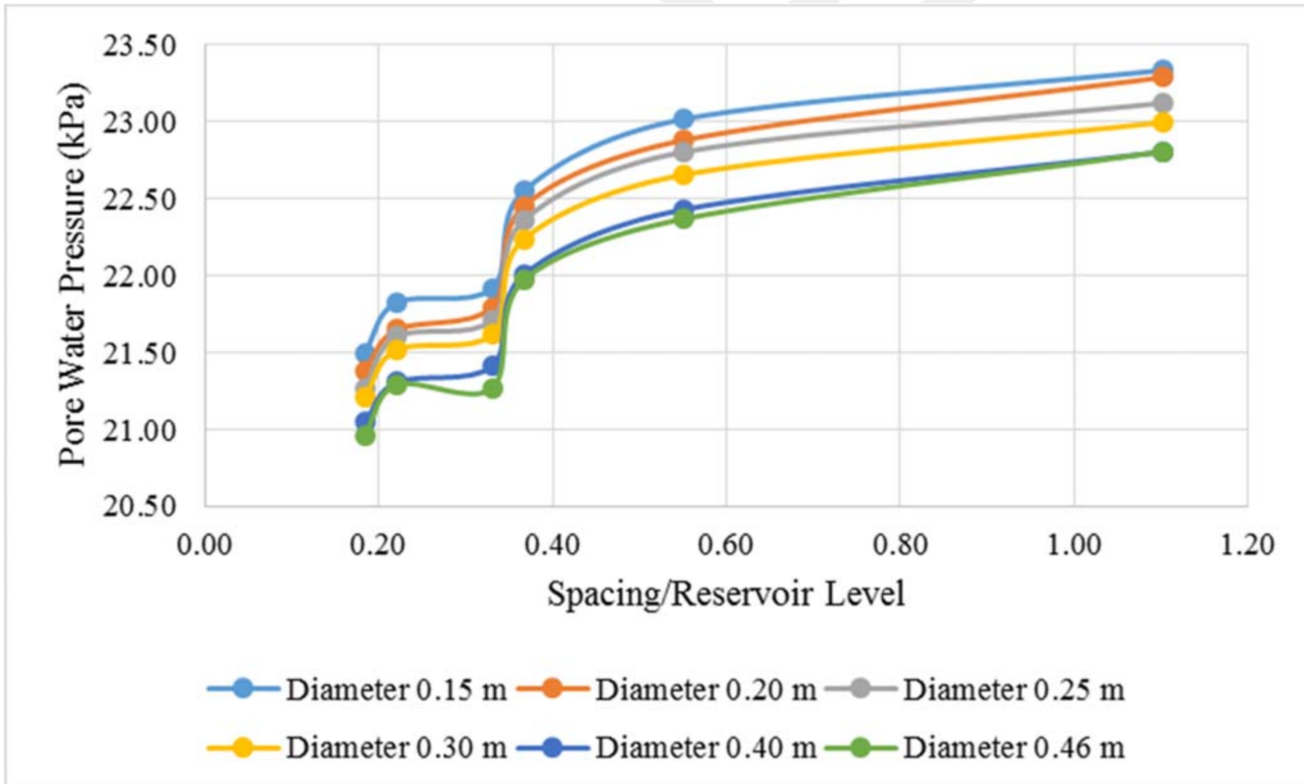

Figure 3.52. Relief well spacing and pore water pressure (wells located 2 m from the toe)

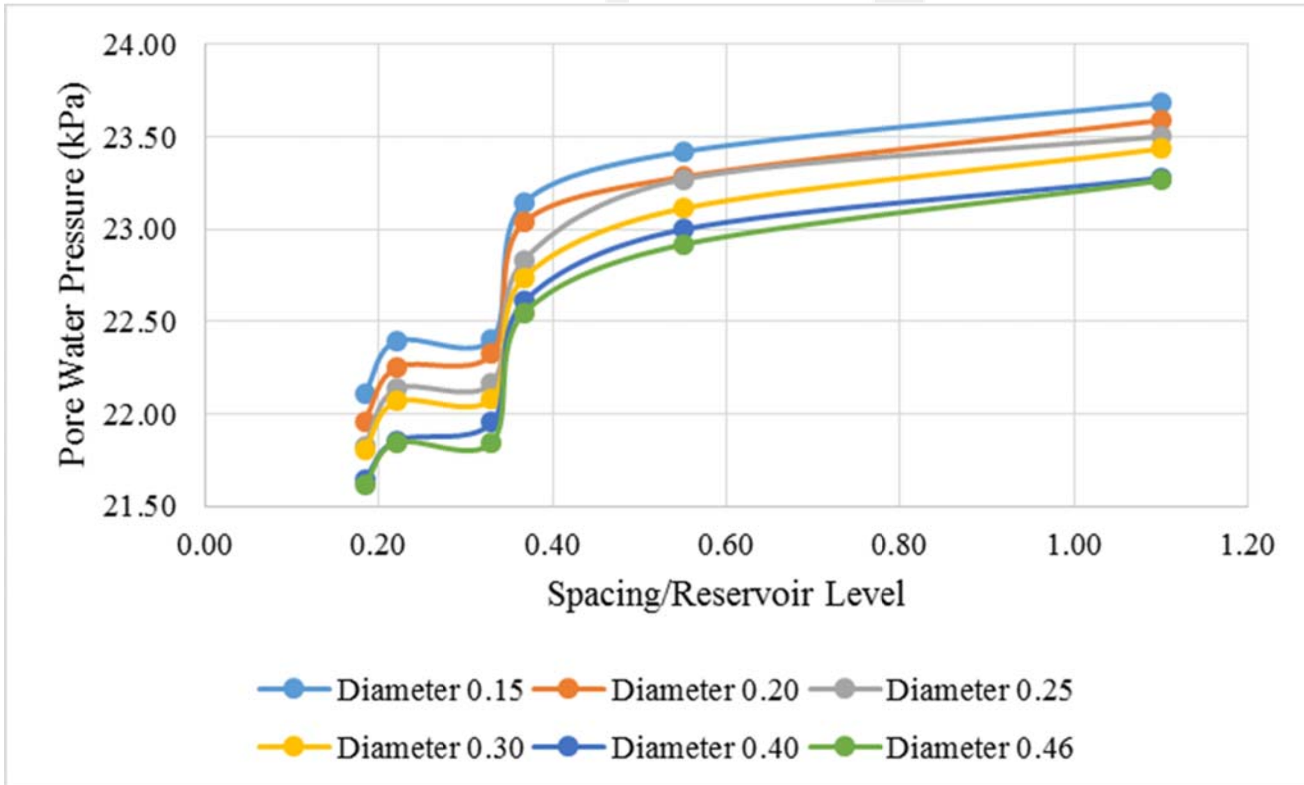

Figure 3.53. Relief well spacing and pore water pressure (wells located 3 m from the toe)

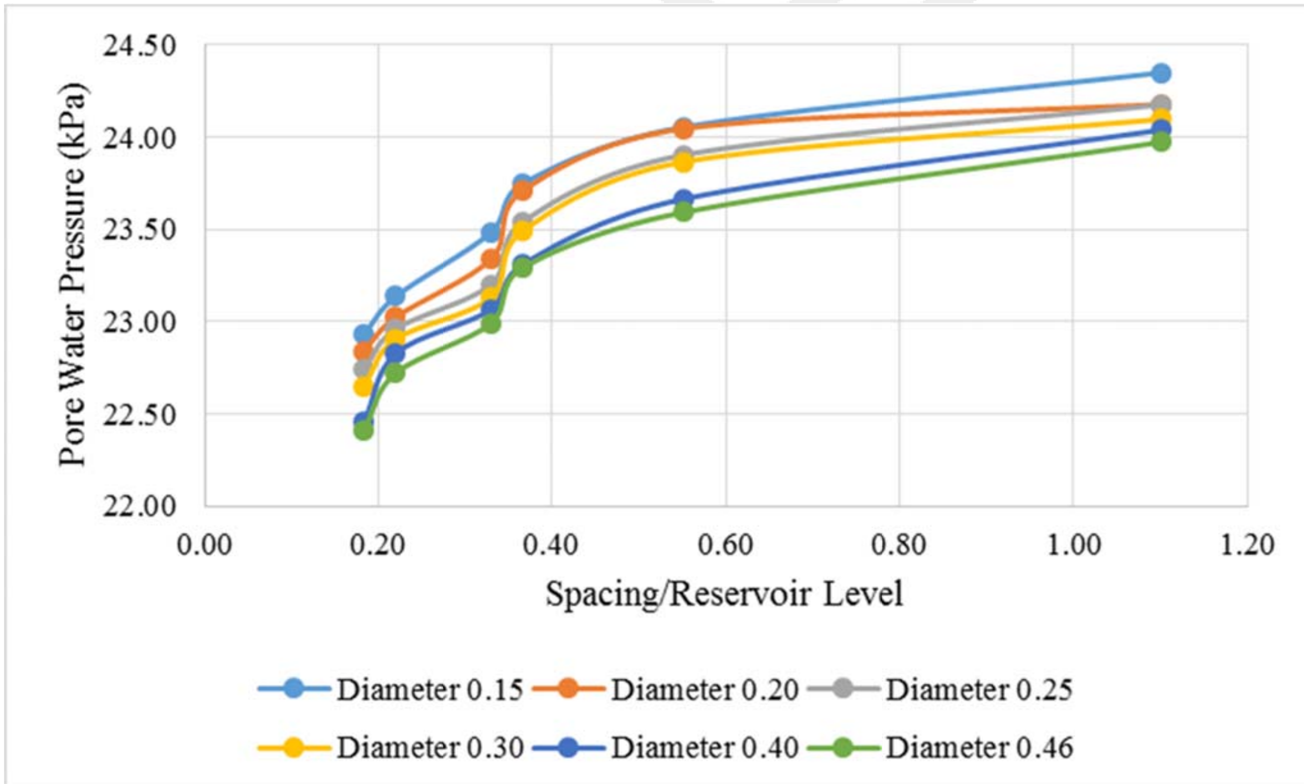

Figure 3.54. Relief well spacing and pore water pressure (wells located 4 m from the toe)

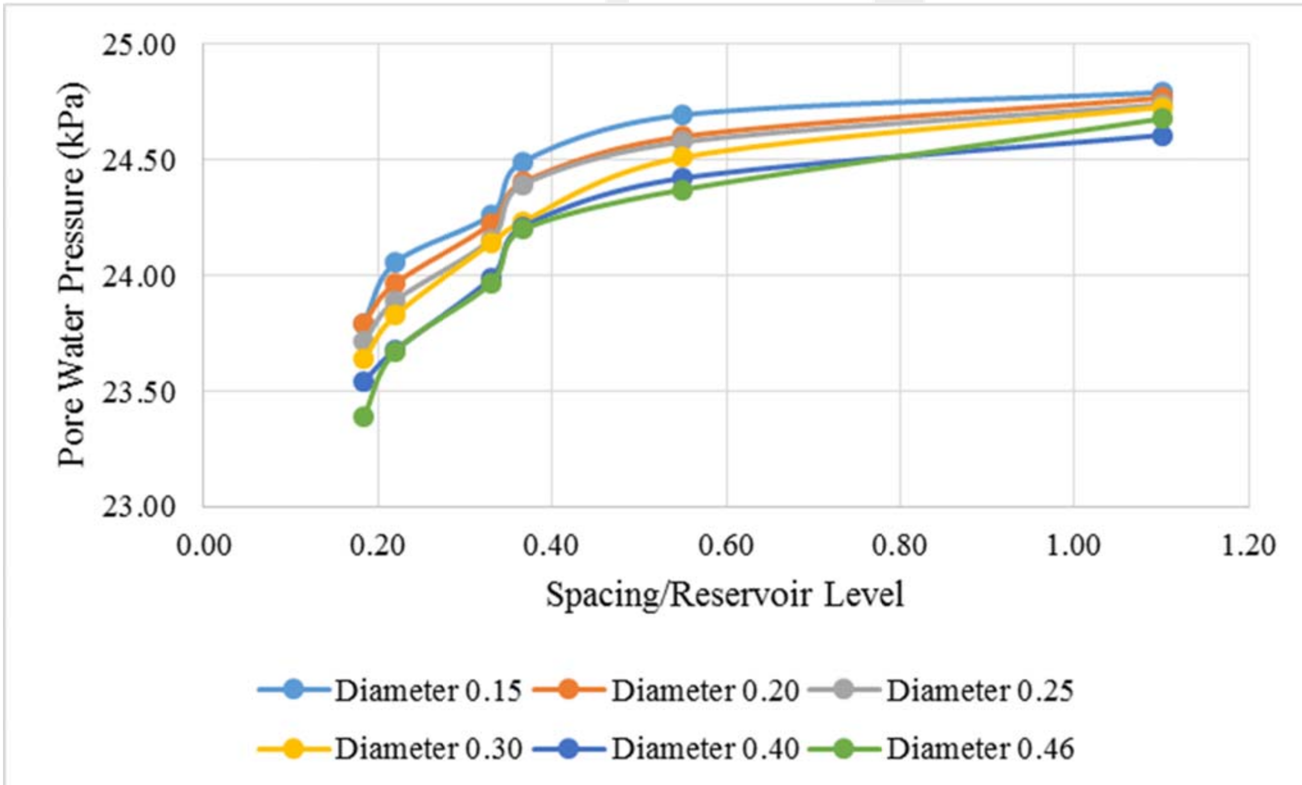

Figure 3.55. Relief well spacing and pore water pressure (wells located 5 m from the toe)

In Figures 3.56 through 3.61, the change of pore water pressure is plotted with the increase of relief wells inner diameter/reservoir level ratio at the point (209, 70). It is clear that, in all of the cases, the pore water pressures decrease with the increase of inner well diameter. The rate of change varies from 0.1% at relief wells with 30 m spacing located at 4 m from the downstream toe to 1.1% at the relief wells with 15 m spacing located 5 m from the downstream toe of the dam.

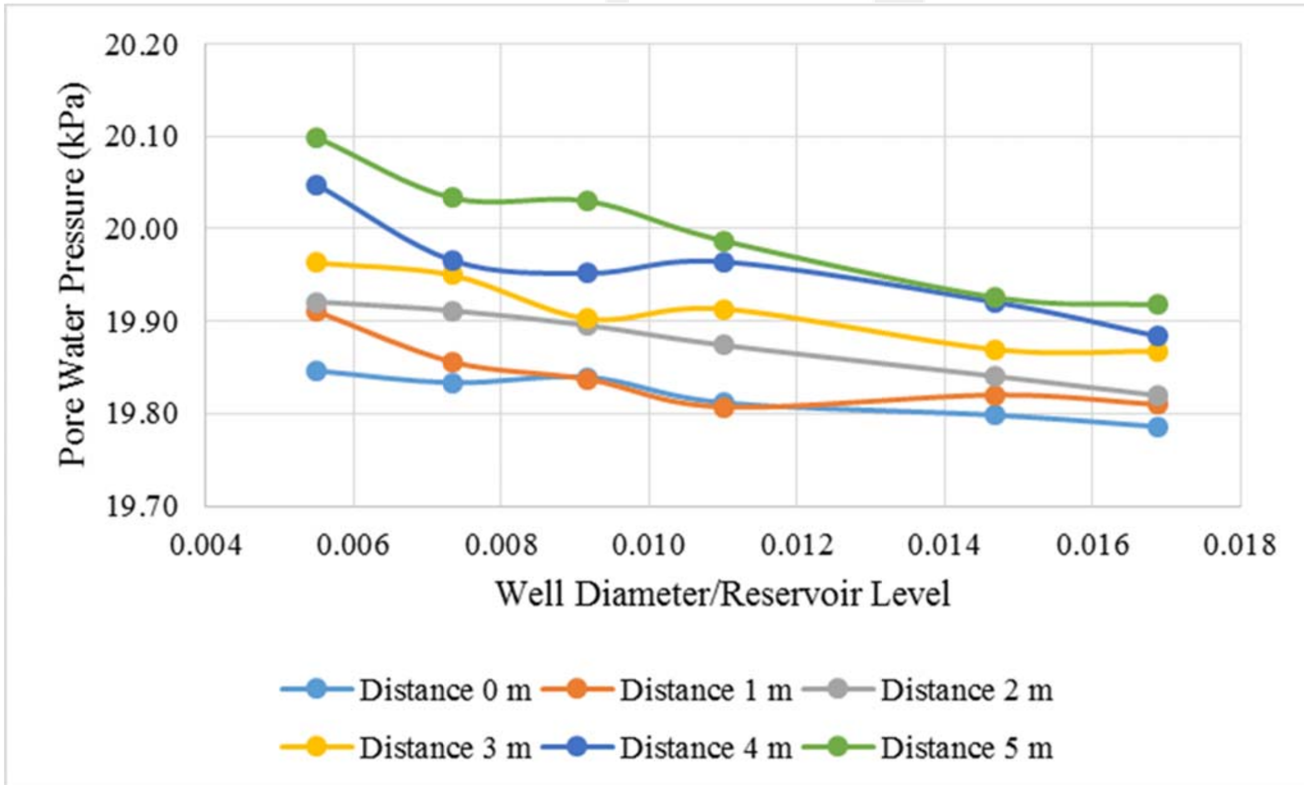

Figure 3.56. Relief well inner diameter and pore water pressure (5 m spacing)

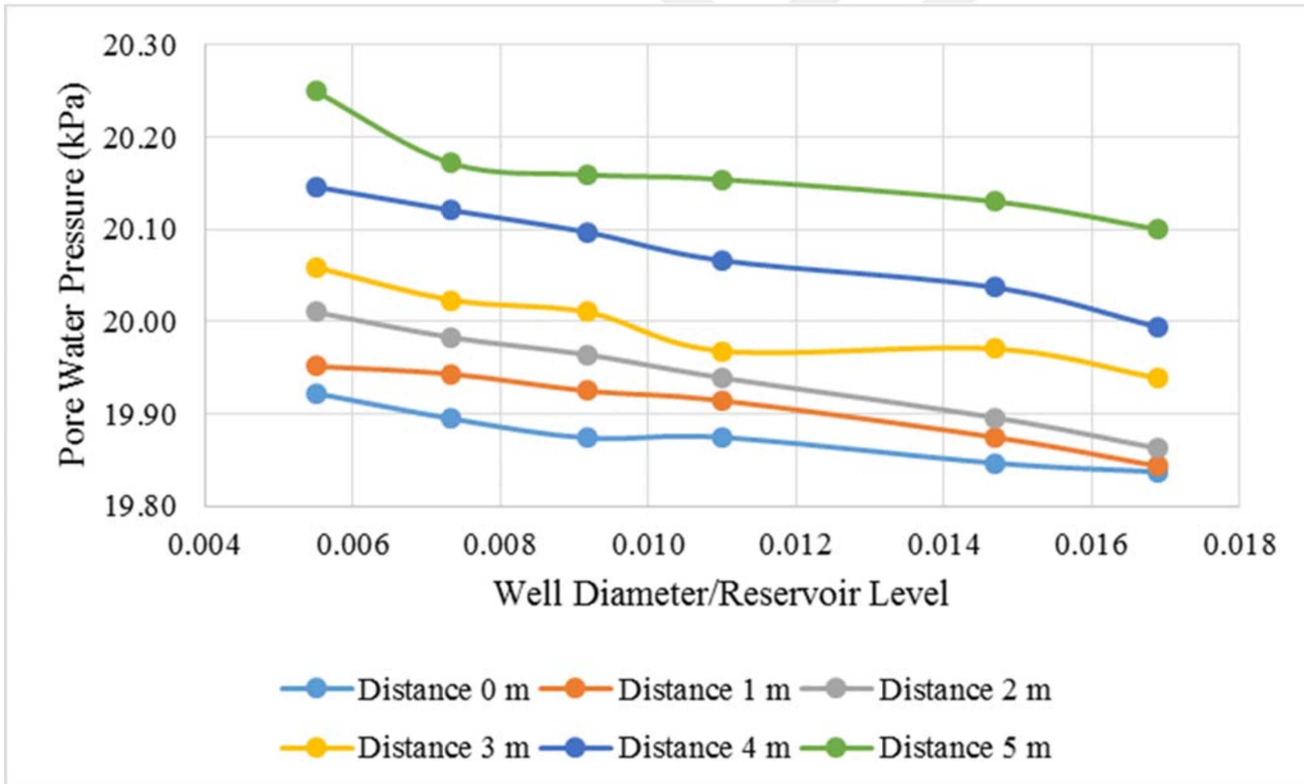

Figure 3.57. Relief well inner diameter and pore water pressure (6 m spacing)

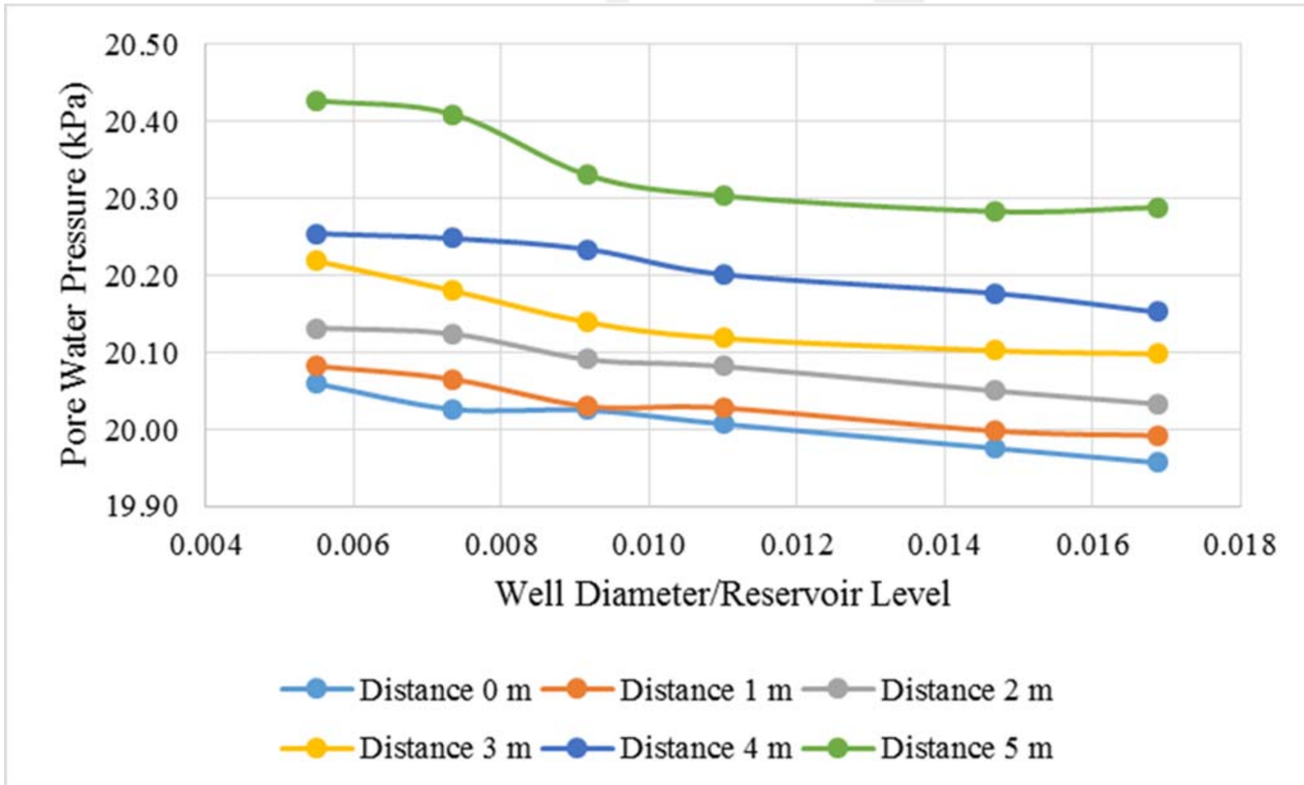

Figure 3.58. Relief well inner diameter and pore water pressure (9 m spacing)

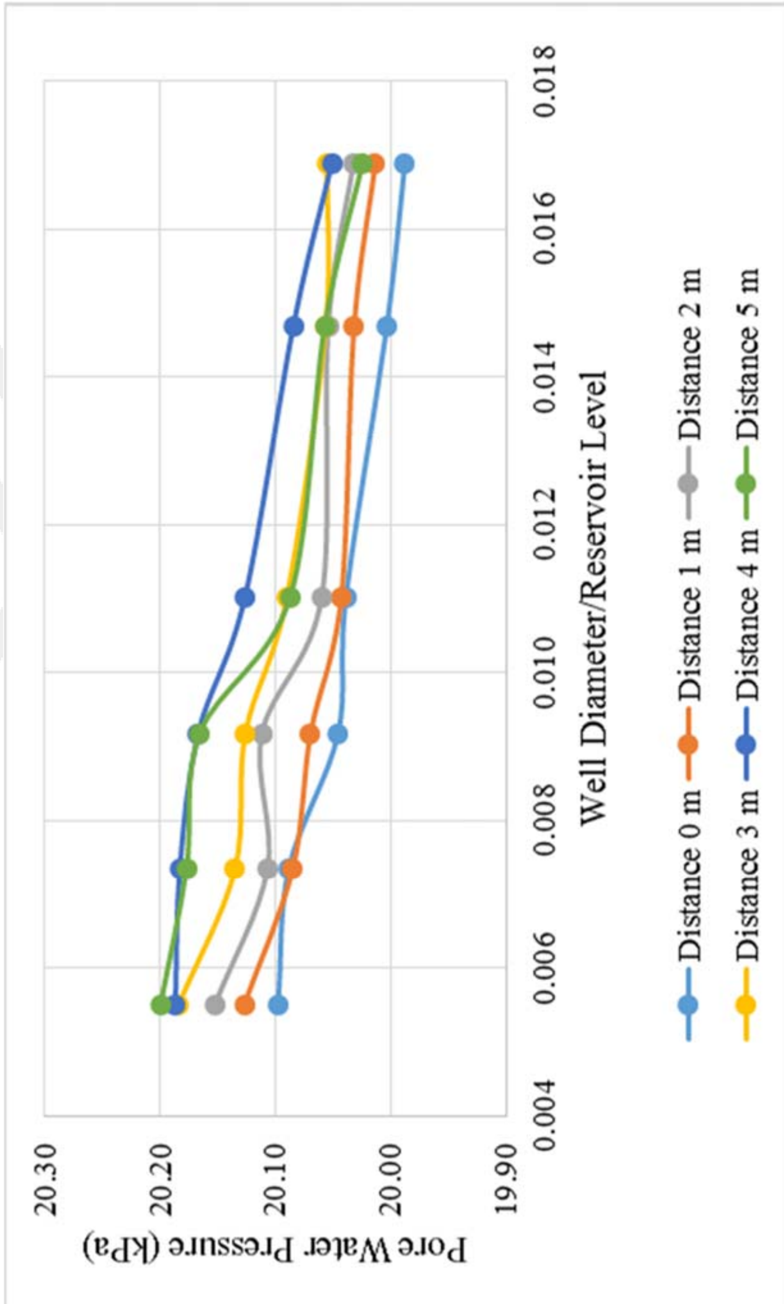

Figure 3.59. Relief well inner diameter and pore water pressure (10 m spacing)

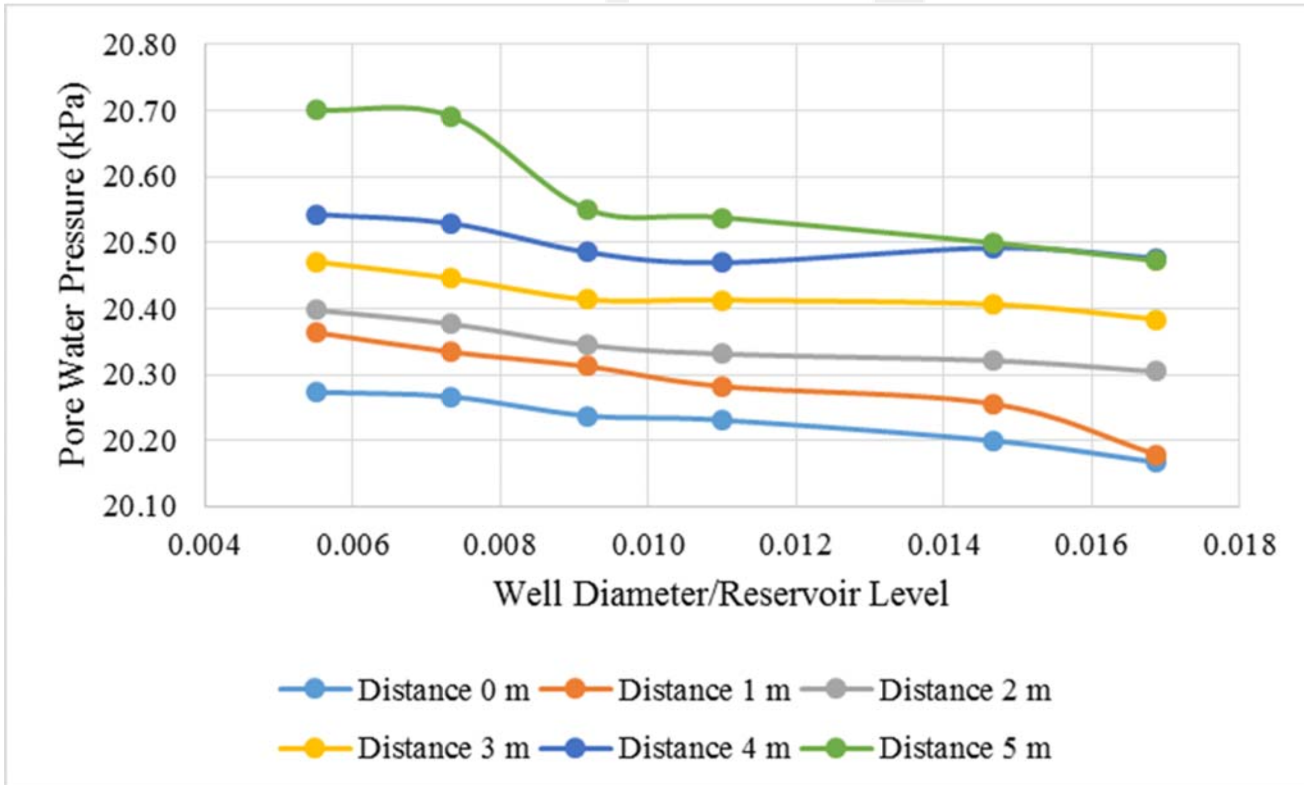

Figure 3.60. Relief well inner diameter and pore water pressure (15 m spacing)

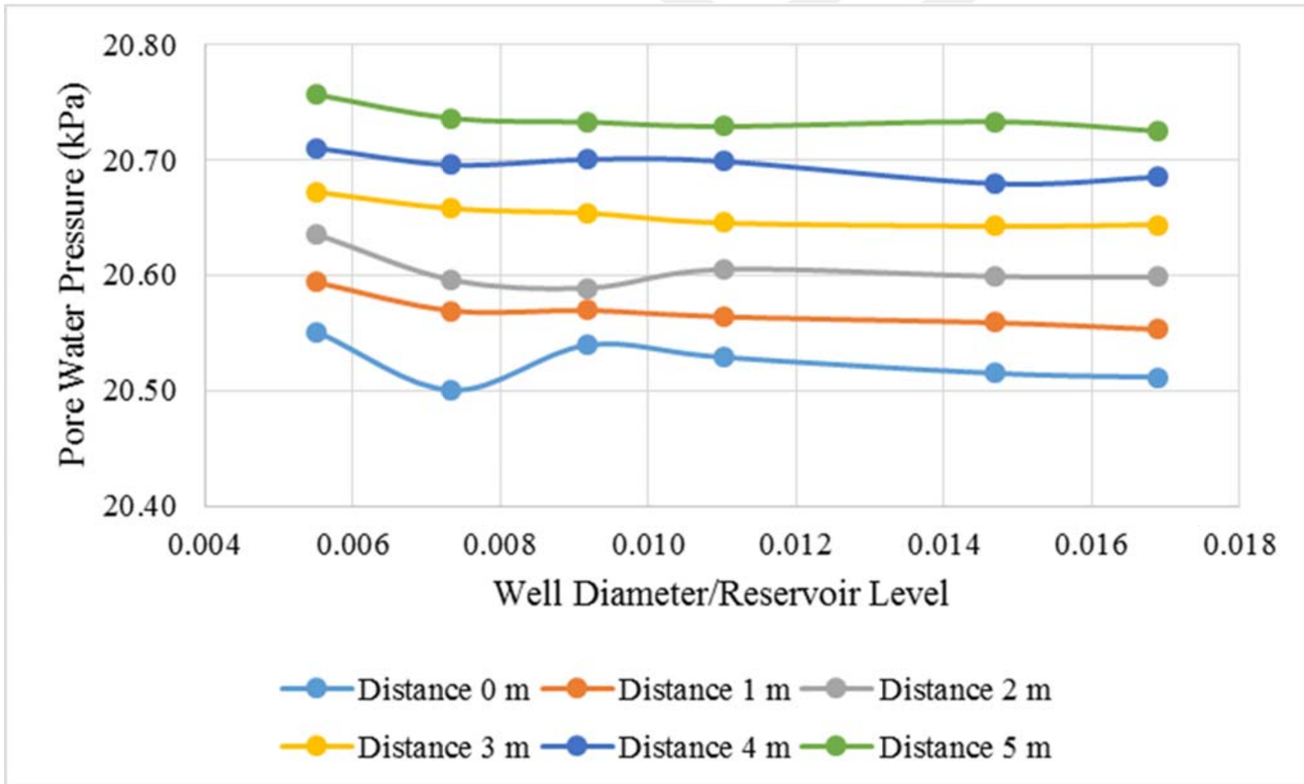

Figure 3.61. Relief well inner diameter and pore water pressure (30 m spacing)

Figures 3.62 through 3.67 show the effect of spacing/reservoir level ratio to the pore water pressure at the point (209, 70). Increase of spacing/reservoir level ratio yields to an increase of pore water pressure in all of the cases. The change of pore water pressure with the effect of spacing/reservoir level ratio varies between 3% and 4% in all cases. As the distance from the toe increases, the importance of spacing/reservoir level ratio decreases as it is seen from the graphs. When the distance from the toe increases, the graphs become more linear. As the distance from the downstream toe increases, the pressure relief wells gets closer to the selected point (209, 70). This yields to increase the effect of relief well (lower pore water pressure) at the selected point and this situation is clearly visible on Figures 3.65, 3.66, and 3.67.

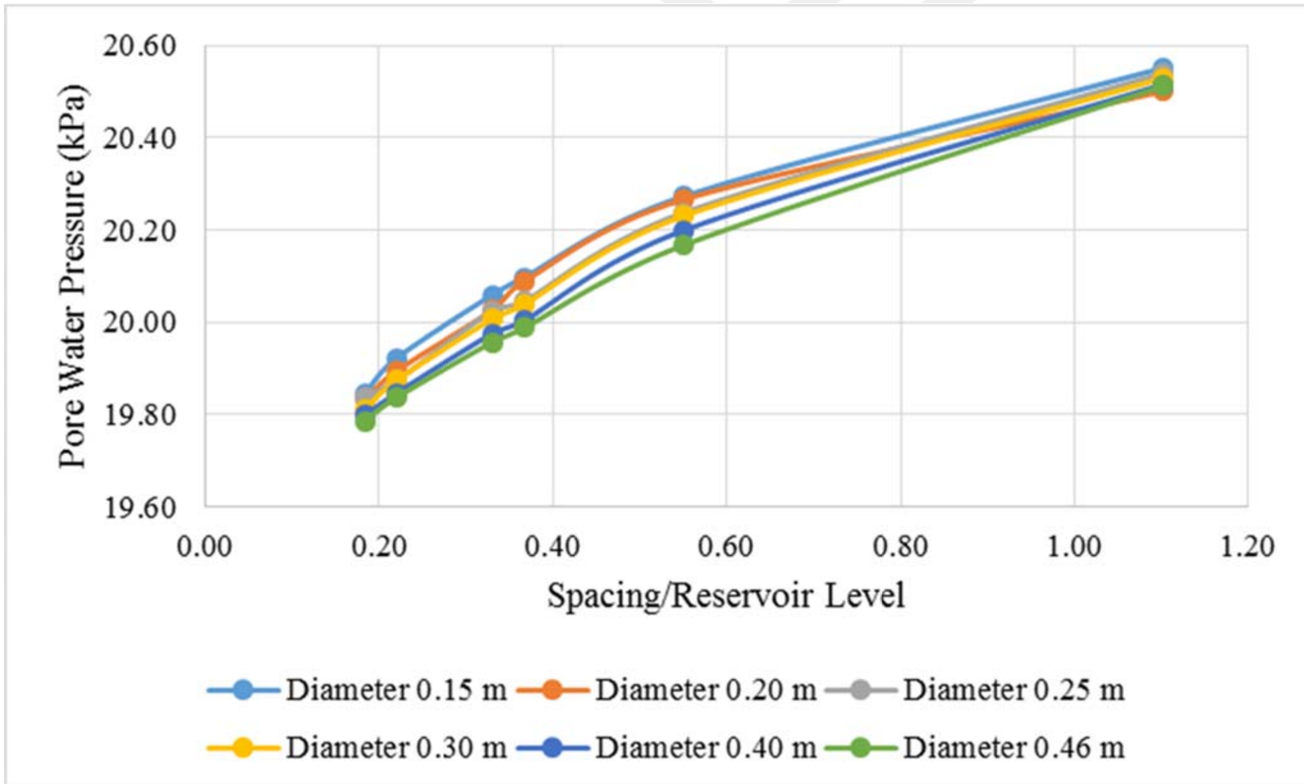

Figure 3.62. Relief well spacing and pore water pressure (wells located at the toe)

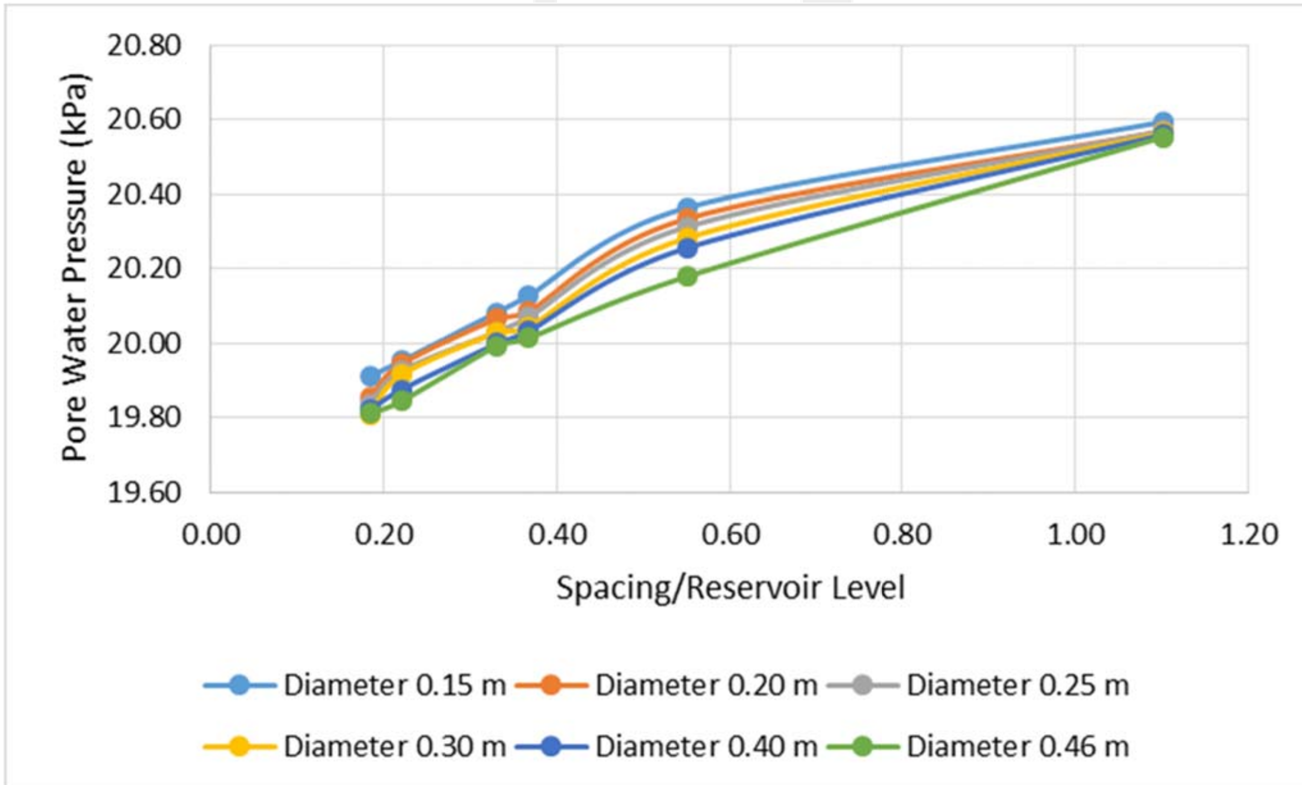

Figure 3.63. Relief well spacing and pore water pressure (wells located 1 m from the toe)

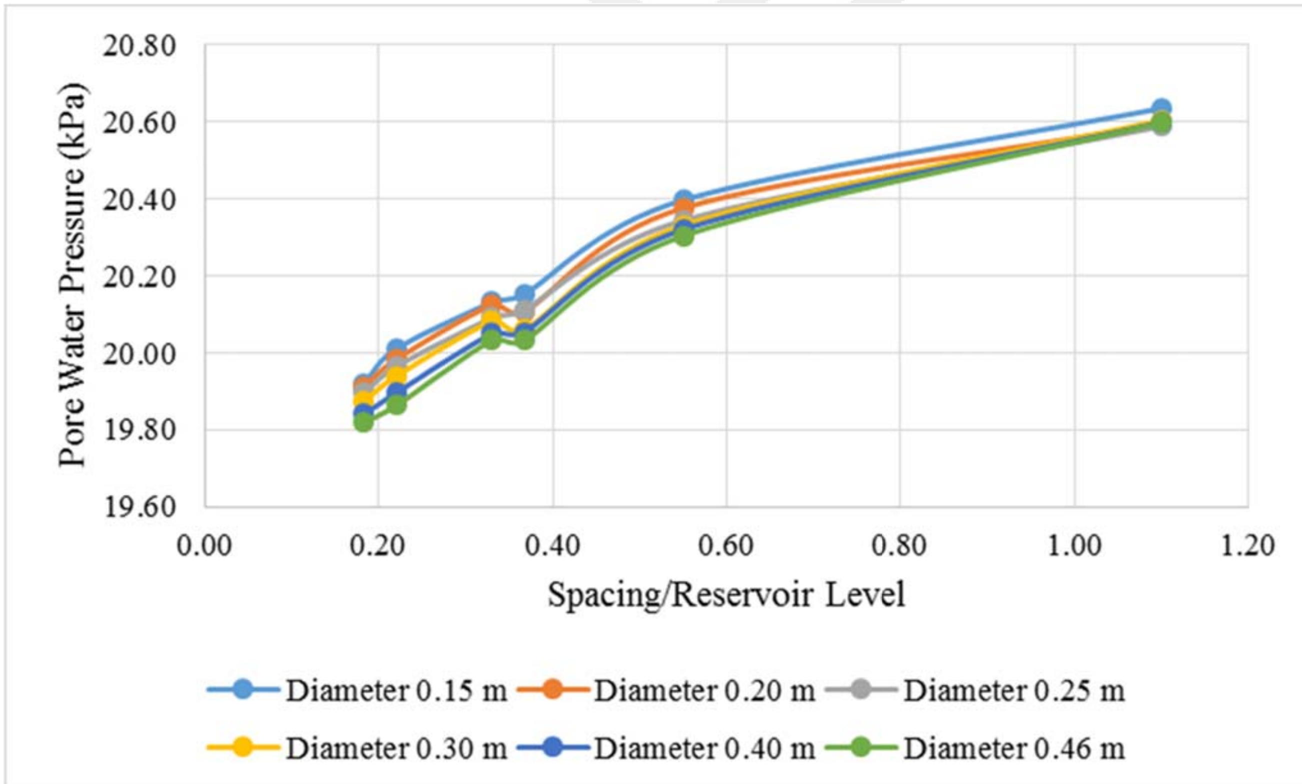

Figure 3.64. Relief well spacing and pore water pressure (wells located 2 m from the toe)

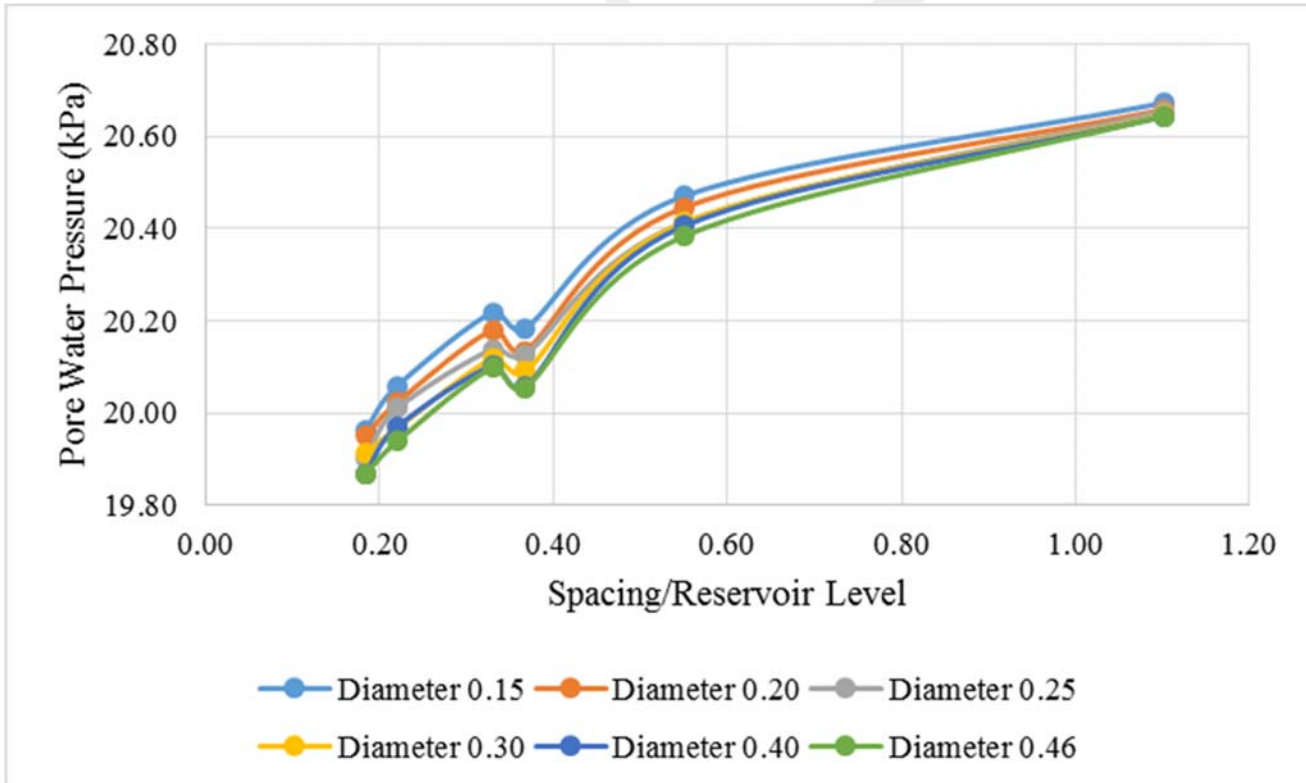

Figure 3.65. Relief well spacing and pore water pressure (wells located 3 m from the toe)

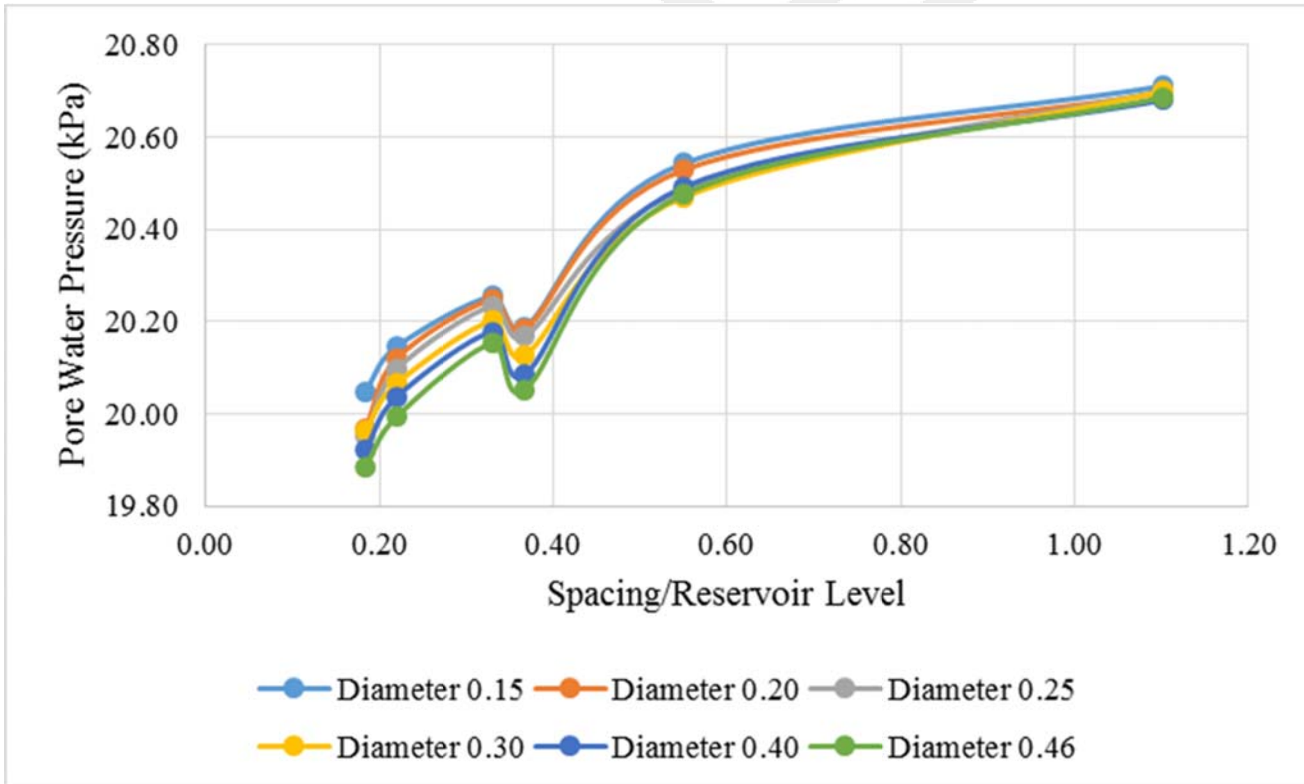

Figure 3.66. Relief well spacing and pore water pressure (wells located 4 m from the toe)

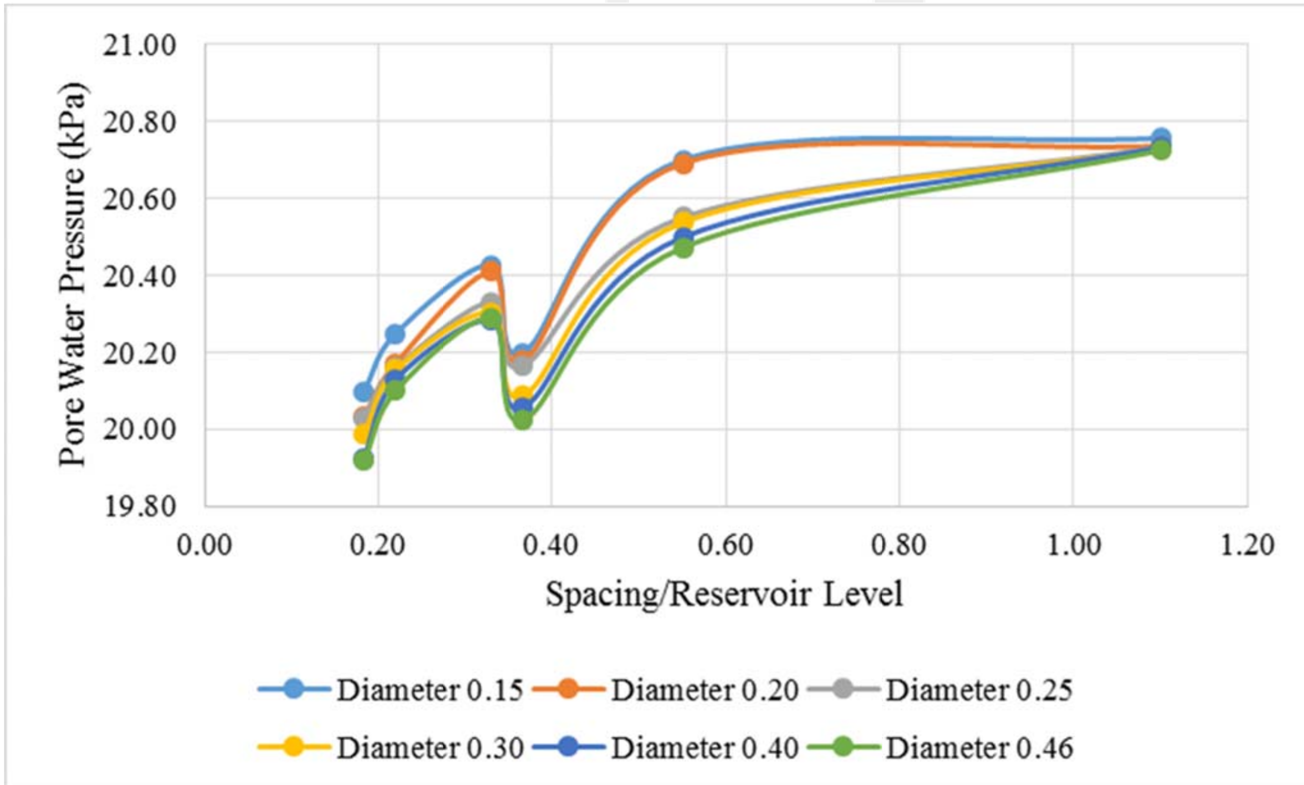

Figure 3.67. Relief well spacing and pore water pressure (wells located 5 m from the toe)

3.7 Steady-State Seepage Analysis with Cutoff Wall, Core Trenches and Cutoff Wall with an Upstream Blanket Configurations

The analysis of the cutoff walls and trenches are conducted in profile (2-D) view. Figure 3.68 represents an example sketch for those analyses. This example sketch is from the analysis file of the concrete cutoff wall with an upstream blanket. The green thin layer on the left of the figure is the clay blanket and the grey wall under the dam at the center is the concrete cutoff wall. The analyses are conducted with different depths and locations of the concrete cutoff wall. The yellow area under the dam is the pervious zone of the foundation and the light green area under it is the impervious zone. The data for the depths of these soil types are taken from the geological report of the project (B.E. Akçor, personal communication, 19th March, 2019).

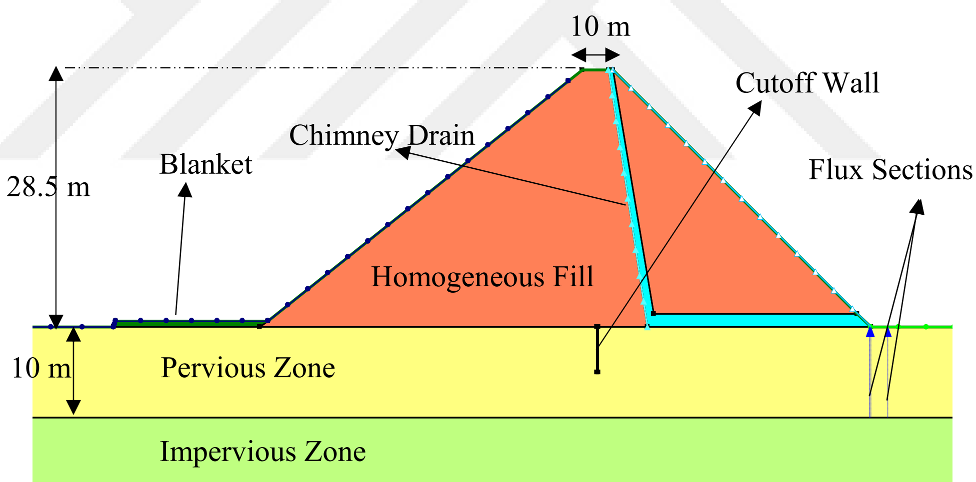

Figure 3.68. Example cutoff wall sketch

Figure 3.69 represents an example sketch for the analysis with the core trench cases. In this figure, the green core is designed and the performance of the core trench is analyzed. This analysis is performed for the case of trench height/pervious zone height ratio of 50%.

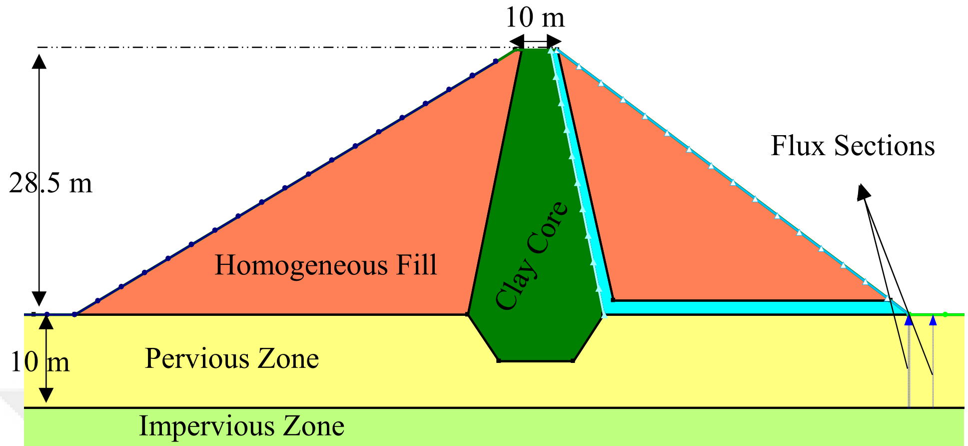

Figure 3.69. Example core trench sketch

Since the depth of pervious zone is not big, the clay core is extended until the impervious zone in a supplementary analysis. The result of the analysis is that 2.2897e-5 m^3/s, which is much smaller than the other alternatives considered for seepage reduction. However, although this application is the most effective solution against foundation seepage problem, it will not be considered in this study because the length of the dam exceeds 1500.0 m. With these dimensions the dam requires tremendous amount of core material, which cannot be obtained in close proximity to the dam site. This study is limited to the efficiency and performance of relief wells in embankment dams. A comparative investigation is done regarding other preventive alternatives, i.e., core trench, cutoff wall and the blanket, in terms of their capability in decreasing the seepage. However, the cost estimations of these are kept out of the scope.

The following Figures 3.70, 3.71 and 3.72 are the flow nets occurred with the given state of the dam. The Figures 3.73 3.74 and 3.75 are the figures for velocity vectors for the same cases in 3.70, 3.71, and 3.72, respectively.

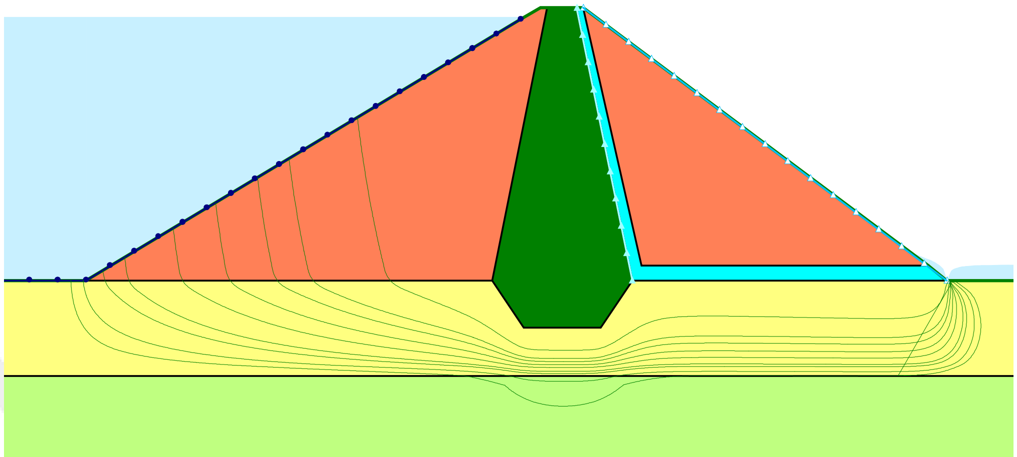

Figure 3.70. Flow nets with core trench

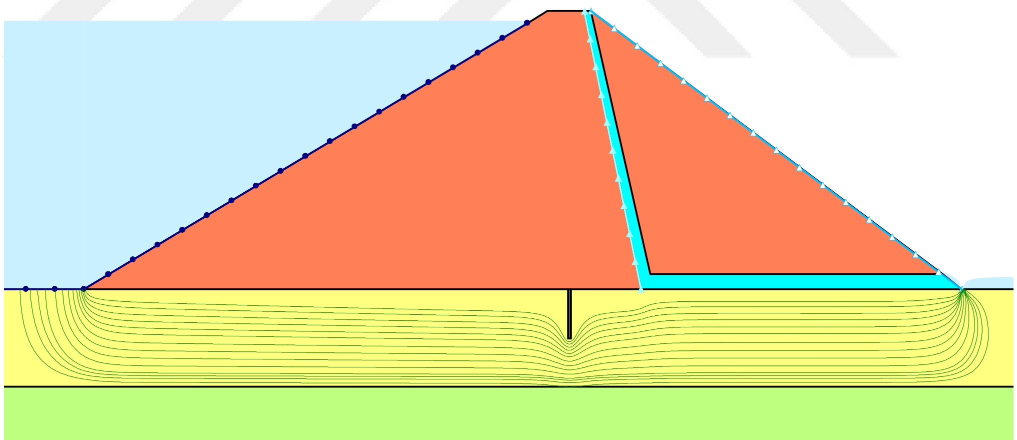

Figure 3.71. Flow nets with cutoff wall located at the center

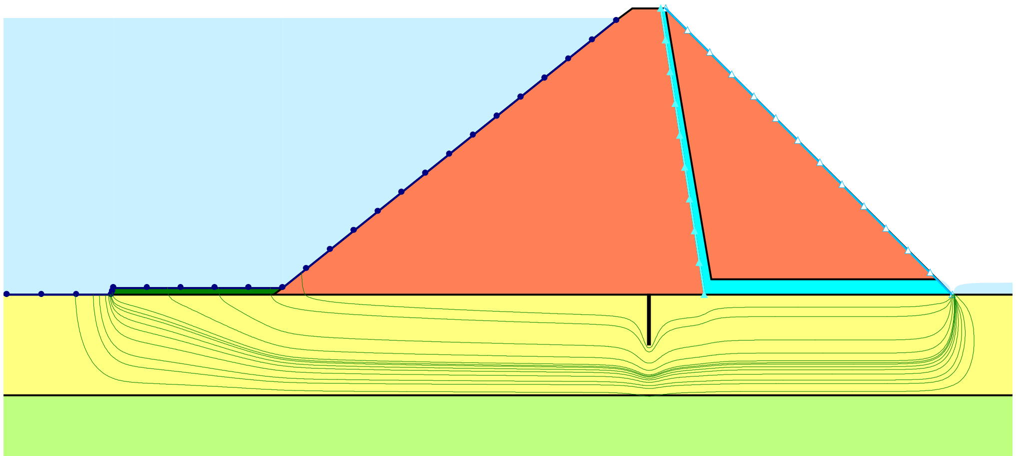

Figure 3.72. Flow nets with cutoff wall located at the center with blanket

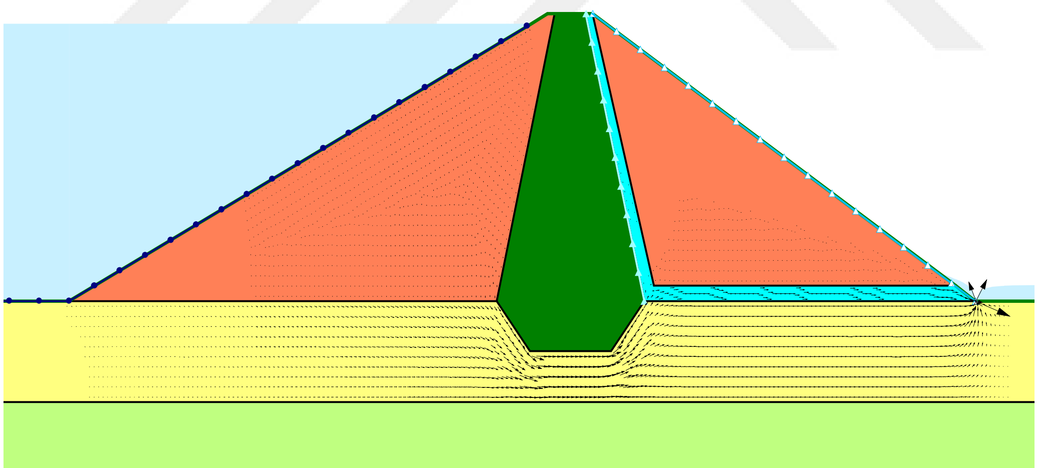

Figure 3.73. Velocity vectors with core trench

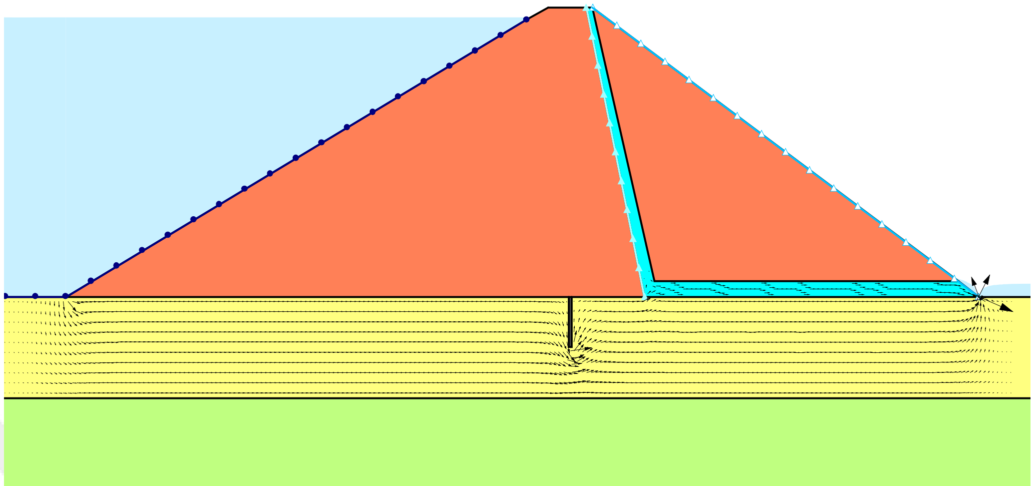

Figure 3.74. Velocity vectors with cutoff wall located at the center

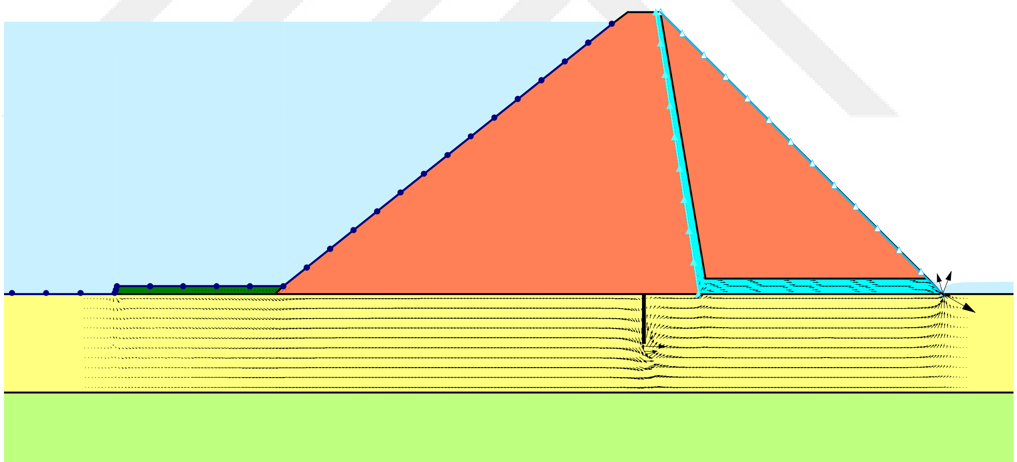

Figure 3.75. Velocity vectors with cutoff wall located at the center with blanket

Figure 3.76, Figure 3.77 and Figure 3.78 are generated with the results of the analyses presented. Figure 3.76 and Figure 3.77 do not include the core trench results because they are drawn according to the location of the trench or the cutoff wall. The increase in trench height/pervious zone height ratio resulted to decrease in seepage.

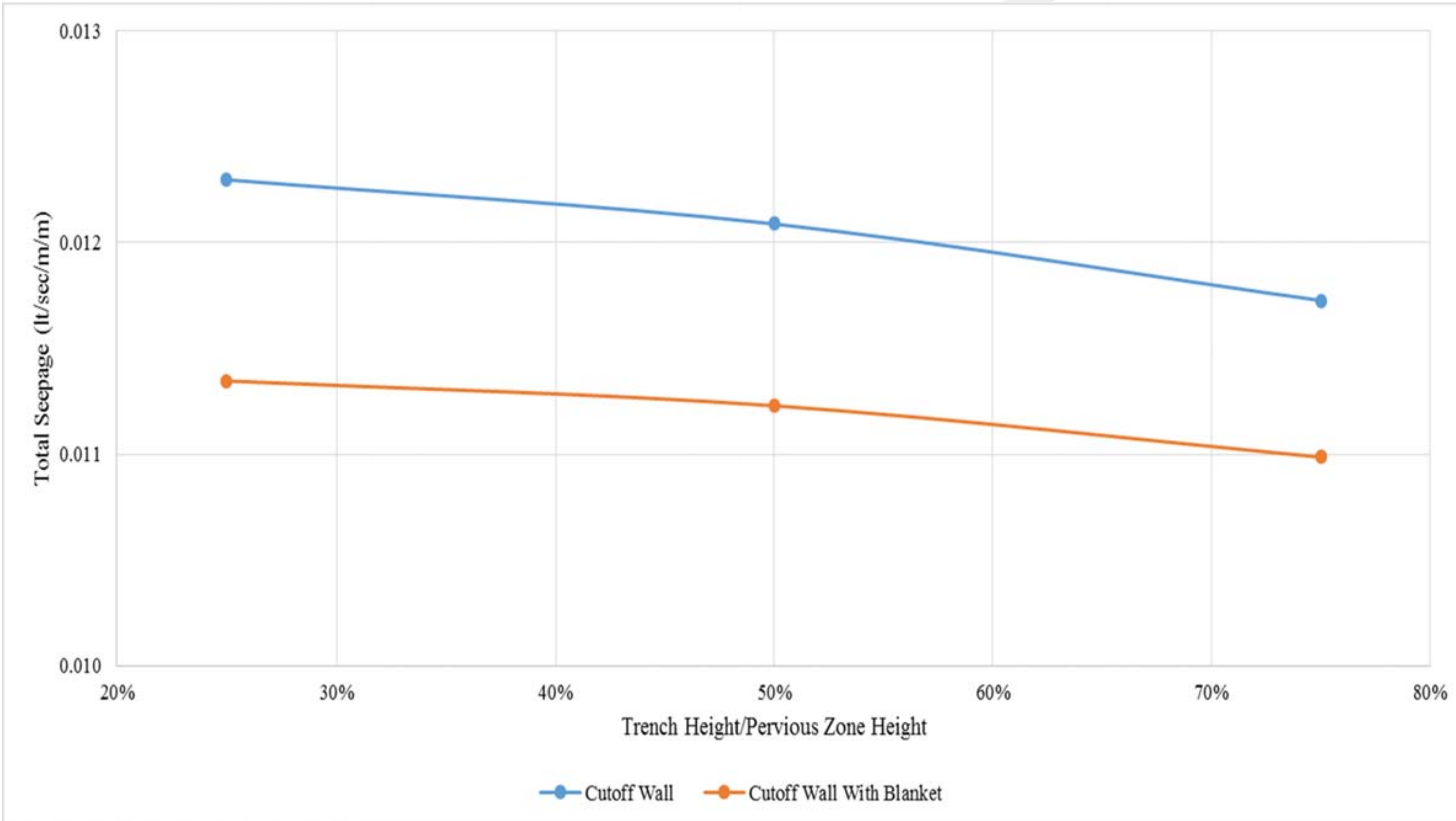

Figure 3.76. Effect of trench height/pervious zone height ratio – trench at upstream toe

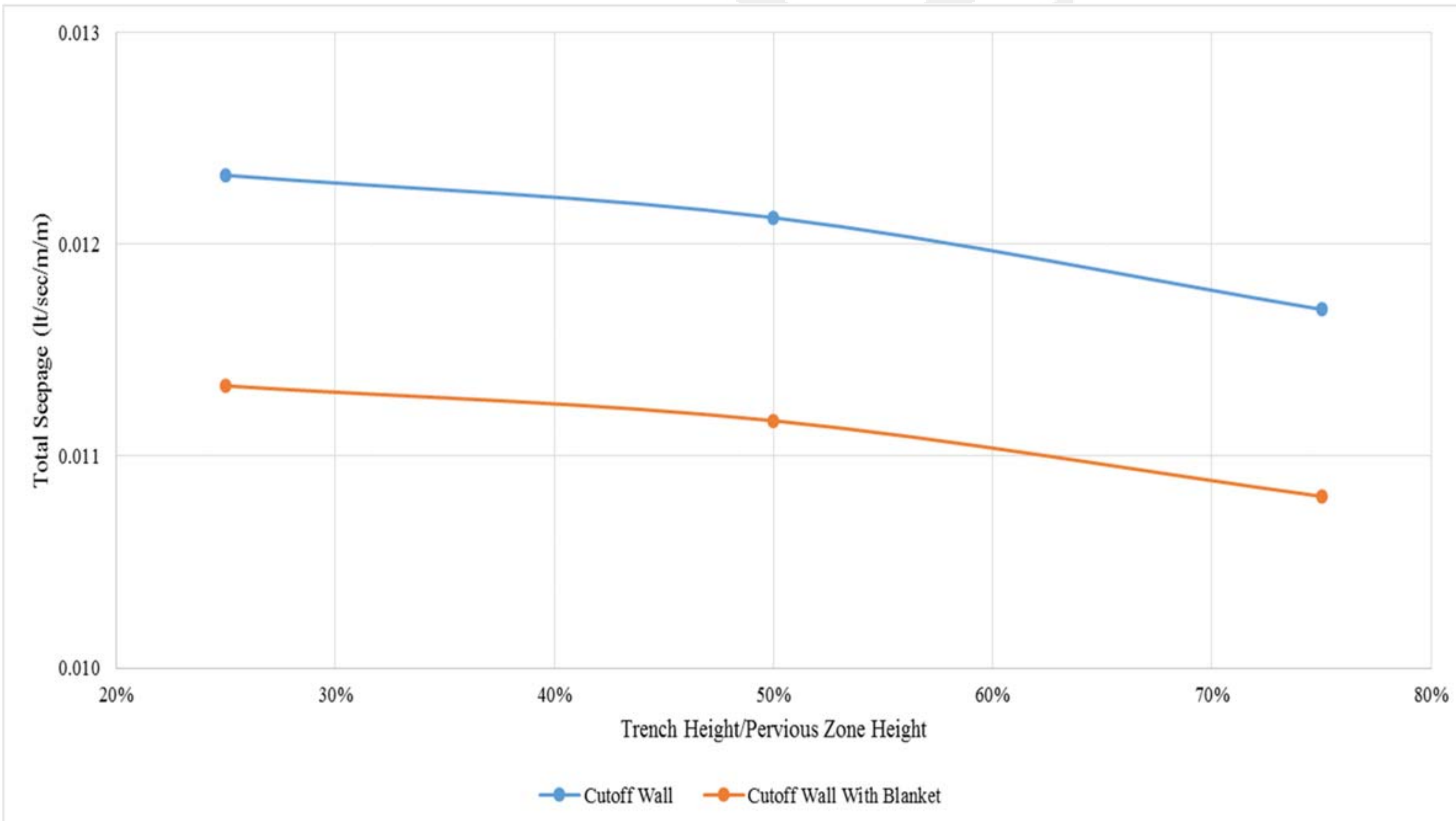

Figure 3.77. Effect of trench height/pervious zone height ratio – trench at between upstream toe

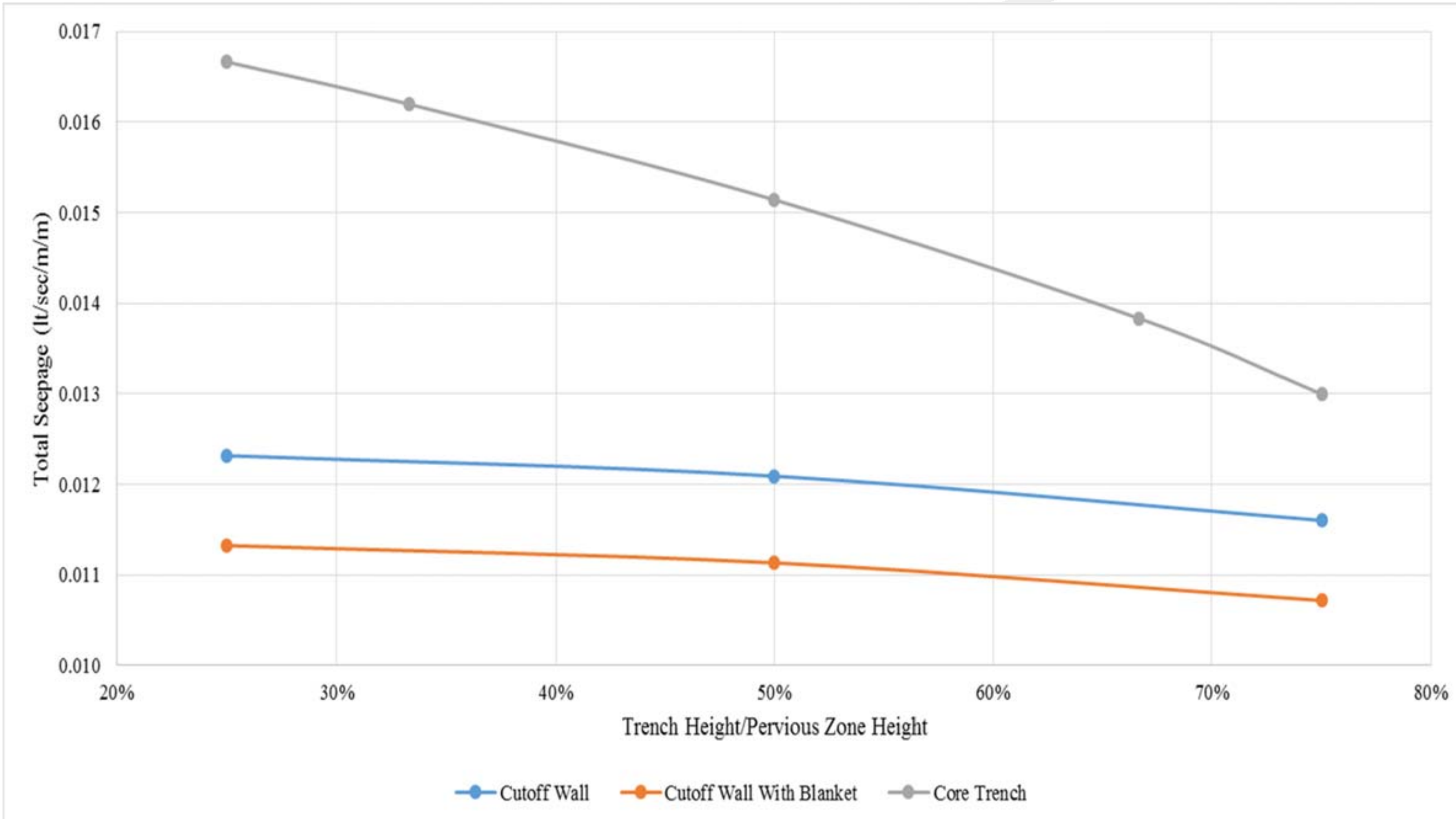

Figure 3.78. Effect of trench height/pervious zone height ratio – trench at center

Figure 3.79, Figure 3.80 and Figure 3.81 are generated with the results of the analyses according to the location of the structures. The effect of trench location is presented in the stated 3 graphs. The core trench case is only located at the center. So the results of that case is presented as the points in the stated figures. Location of cutoff wall does not affect the results of seepage when a blanket is present as there is no change seen on the figures. There is a slight increase of seepage rate (~1%) observed on the graphs of the case of cutoff wall without blanket with the change of the location.

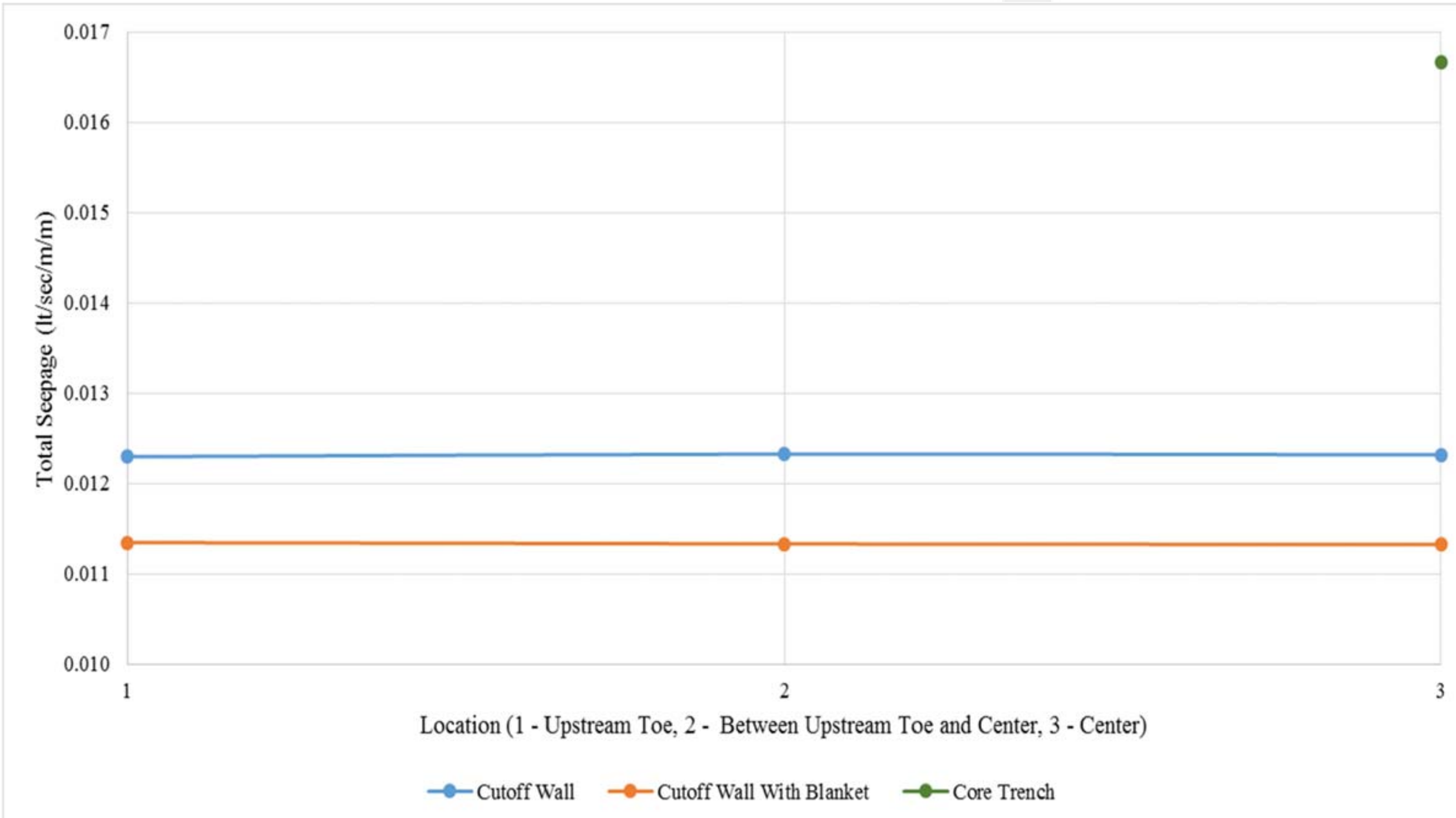

Figure 3.79. Effect of trench location – trench height/pervious zone height ratio at 25%

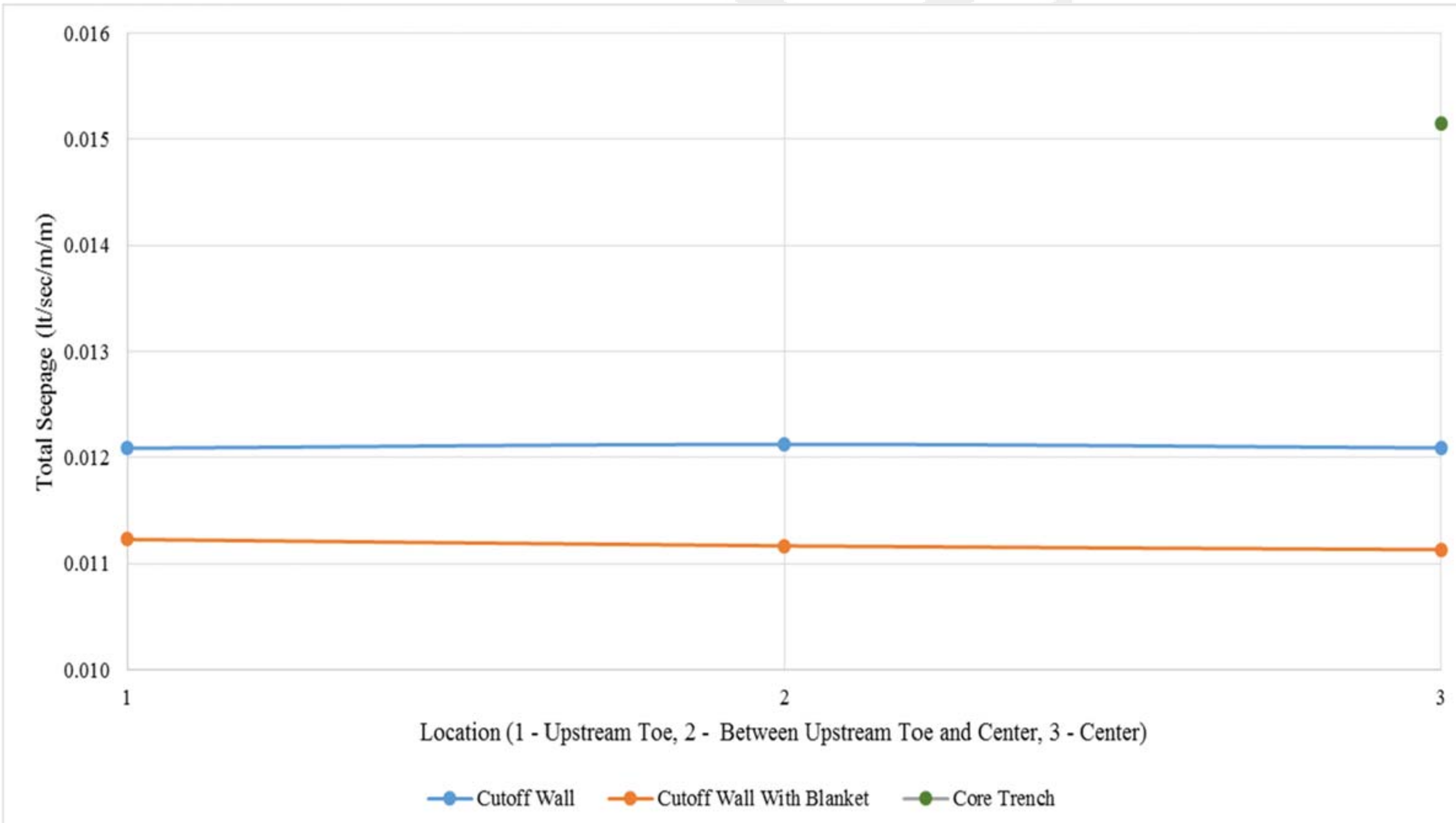

Figure 3.80. Effect of trench location – trench height/pervious zone height ratio at 50%

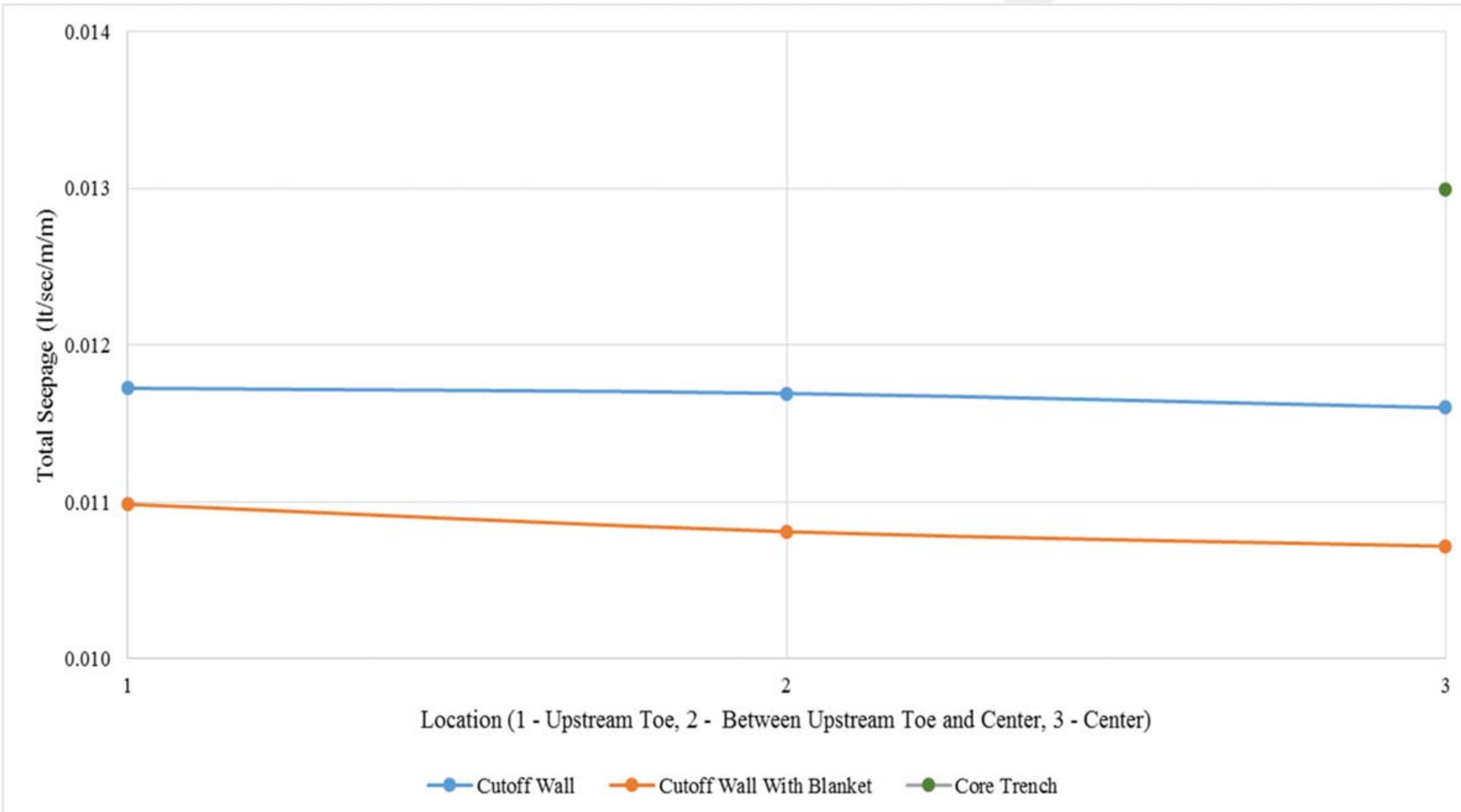

Figure 3.81. Effect of trench location – trench height/pervious zone height ratio at 75%

3.8 Cost Analysis for Relief Wells

The cost analysis is conducted based on the discharge to be pumped per well. The pumps are then selected accordingly. The total cost includes well drilling costs, initial cost of pumps, and operating and maintenance costs. The prices exclude the taxes and pipe lengths joining the wells to the tail water section. A simple pump scheme used in the analysis is shown in Figure 3.82. The pipes that extend from wells to the tail water have an inclination of 1%.

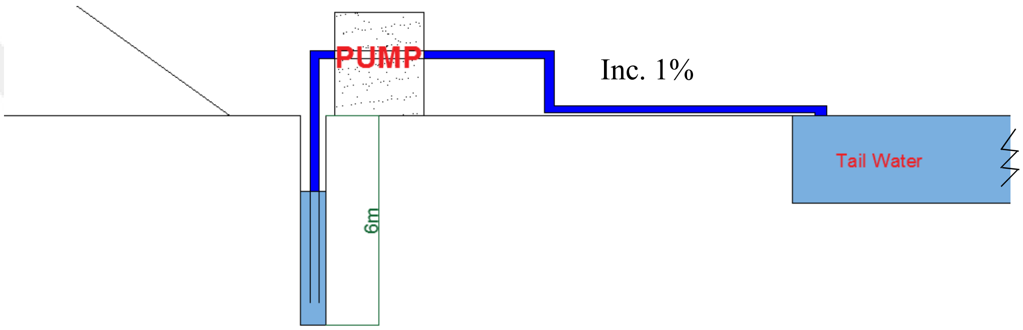

Figure 3.82. Pump scheme

The cost calculations exclude the labor cost, excavation cost and cost of transportation of materials.

In Tables 3.4, 3.5 and 3.6, pumps are chosen according to their pumping capacities and pumping elevations. Selecting the right pump depends on the discharge and pressure. Equation 3.3 is used to find the minimum pump head, H_{min}, in m:

$$H_{min} = h + h_k + h_c \tag{3.3}$$

where h is highest elevation to pump the water, h_k is pipe, elbow and valve losses and h_c is the desired outlet head. As manual indicates, h_k is taken as 20% of height of the pipe until the pump (Varan Pompa, 2019).

The pump outlet is taken equal to the tail water depth, which is accepted to be 2 m. Well are assumed to be of partially penetrating types having depths of 6 m as all other alternatives are taken as partially penetrating. Required discharge is analyzed for each case and used in selection process. Equation 3.3 yields H_{min} = 6+6*0.2+2=9.2 m.

It should be noted that, fully penetrating wells are more effective in terms of collecting water because they can collect more water with a higher volume.

The head of the chosen pump, H_m, should exceed H_{min}. The pump used is 4SD4/8. This pump has an H_m value of 10 m for 5 m^3/h discharge. This pump is an industrial drainage pump and can be used for drainage systems, irrigation and lifting ground waters. The pump can be operated at +5ºC to +40ºC. The prices are taken from the company's manual (Varan Pompa, 2019).

Pump powers in kW are taken from the aforementioned manual in accordance with the obtained results. The given 4SD4/8 pump has a power of 0.55 kW. This value is used in computation of electricity consumption of the pumps as it is the maximum power capacity of the pump. They cannot consume more than their capacity. The discharge and water pumping head are calculated and the pumps are then selected accordingly with this information. In this selection method, pump efficiency is implicitly included in the selection process.

Operating hour column in Table 3.6 is calculated as follows; the total water passing the section of the wells is subtracted from the total water approaching to wells. This difference is then divided by the number of wells in that analysis. Then the results are divided to pumping capacity of the pumps and the average operating hours are estimated. The decimals are then rounded up.

In Tables 3.4, 3.5, and 3.6, the pumps are analyzed and the total collected water amount (well performance) is plotted in Figure 3.83 to compare with the prices.

Table 3.4 Water coming to wells

		Well Locations		Total Water Coming to Wells	
Alternative	Number of Wells	Spacing (m)	Distance from Downstream Toe (m)	m^3/s	m^3/h
1	19	5	0	0.0012428	4.47
2	19	5	1	0.0012361	4.45
3	19	5	2	0.0012293	4.43
4	19	5	3	0.0012226	4.40
5	19	5	4	0.0012316	4.43
6	19	5	5	0.001224	4.41
7	16	6	0	0.0012348	4.45
8	16	6	1	0.0012284	4.42
9	16	6	2	0.0012367	4.45
10	16	6	3	0.0012296	4.43
11	16	6	4	0.0012357	4.45
12	16	6	5	0.0012279	4.42
13	11	9	0	0.0012338	4.44
14	11	9	1	0.001238	4.46
15	11	9	2	0.0012312	4.43
16	11	9	3	0.0012339	4.44
17	11	9	4	0.0012266	4.42
18	11	9	5	0.0012279	4.42
19	10	10	0	0.0012408	4.47
20	10	10	1	0.0012342	4.44
21	10	10	2	0.0012275	4.42
22	10	10	3	0.0012296	4.43
23	10	10	4	0.0012307	4.43
24	10	10	5	0.001231	4.43
25	7	15	0	0.0012449	4.48
26	7	15	1	0.001238	4.46
27	7	15	2	0.0012376	4.46
28	7	15	3	0.0012365	4.45
29	7	15	4	0.0012349	4.45
30	7	15	5	0.0012326	4.44
31	4	30	0	0.001263	4.55
32	4	30	1	0.0012592	4.53
33	4	30	2	0.001255	4.52
34	4	30	3	0.001254	4.51
35	4	30	4	0.0012498	4.50
36	4	30	5	0.0012467	4.49

Table 3.5 Water collected by wells

	Water Passing Wells		Water Collected by Wells		
Alternative	m^3/s	m^3/h	m^3/s	$m^3/h/well$	m^3/day
1	0.00028	1.00	0.00097	0.18	83.41
2	0.00028	1.02	0.00095	0.18	82.27
3	0.00029	1.05	0.00094	0.18	81.10
4	0.00030	1.07	0.00093	0.18	79.94
5	0.00048	1.72	0.00075	0.14	65.07
6	0.00049	1.75	0.00074	0.14	63.76
7	0.00021	0.74	0.00103	0.23	88.96
8	0.00021	0.76	0.00102	0.23	87.85
9	0.00036	1.31	0.00087	0.20	75.46
10	0.00037	1.33	0.00086	0.19	74.23
11	0.00052	1.89	0.00071	0.16	61.46
12	0.00053	1.92	0.00070	0.16	60.11
13	0.00020	0.71	0.00104	0.34	89.65
14	0.00030	1.09	0.00094	0.31	80.87
15	0.00031	1.11	0.00092	0.30	79.69
16	0.00042	1.50	0.00082	0.27	70.66
17	0.00042	1.52	0.00080	0.26	69.39
18	0.00053	1.92	0.00070	0.23	60.11
19	0.00026	0.93	0.00098	0.35	84.81
20	0.00027	0.96	0.00097	0.35	83.67
21	0.00027	0.98	0.00096	0.34	82.51
22	0.00037	1.33	0.00086	0.31	74.23
23	0.00047	1.69	0.00076	0.27	65.79
24	0.00057	2.05	0.00066	0.24	57.20
25	0.00030	1.06	0.00095	0.49	82.06
26	0.00030	1.09	0.00094	0.48	80.87
27	0.00037	1.34	0.00087	0.44	74.75
28	0.00044	1.60	0.00079	0.41	68.52
29	0.00052	1.85	0.00072	0.37	62.19
30	0.00059	2.11	0.00065	0.33	55.74
31	0.00046	1.65	0.00081	0.73	69.63
32	0.00050	1.80	0.00076	0.68	65.52
33	0.00055	1.96	0.00071	0.64	61.34
34	0.00063	2.25	0.00063	0.57	54.26
35	0.00067	2.42	0.00058	0.52	50.00
36	0.00075	2.71	0.00049	0.44	42.63

Table 3.6 Pump working hours per day

Alternative	Pump			Water Coming to One Well/Pump Capacity	Working Hour/Day
	Pump Type	H_m (m)	Capacity (m^3/h)		
1	4SD4/8	14	5	0.88	0.88
2	4SD4/8	14	5	0.87	0.87
3	4SD4/8	14	5	0.85	0.85
4	4SD4/8	14	5	0.84	0.84
5	4SD4/8	14	5	0.68	0.68
6	4SD4/8	14	5	0.67	0.67
7	4SD4/8	14	5	1.11	1.11
8	4SD4/8	14	5	1.10	1.10
9	4SD4/8	14	5	0.94	0.94
10	4SD4/8	14	5	0.93	0.93
11	4SD4/8	14	5	0.77	0.77
12	4SD4/8	14	5	0.75	0.75
13	4SD4/8	14	5	1.63	1.63
14	4SD4/8	14	5	1.47	1.47
15	4SD4/8	14	5	1.45	1.45
16	4SD4/8	14	5	1.28	1.28
17	4SD4/8	14	5	1.26	1.26
18	4SD4/8	14	5	1.09	1.09
19	4SD4/8	14	5	1.70	1.70
20	4SD4/8	14	5	1.67	1.67
21	4SD4/8	14	5	1.65	1.65
22	4SD4/8	14	5	1.48	1.48
23	4SD4/8	14	5	1.32	1.32
24	4SD4/8	14	5	1.14	1.14
25	4SD4/8	14	5	2.34	2.34
26	4SD4/8	14	5	2.31	2.31
27	4SD4/8	14	5	2.14	2.14
28	4SD4/8	14	5	1.96	1.96
29	4SD4/8	14	5	1.78	1.78
30	4SD4/8	14	5	1.59	1.59
31	4SD4/8	14	5	3.48	3.48
32	4SD4/8	14	5	3.28	3.28
33	4SD4/8	14	5	3.07	3.07
34	4SD4/8	14	5	2.71	2.71
35	4SD4/8	14	5	2.50	2.50
36	4SD4/8	14	5	2.13	2.13

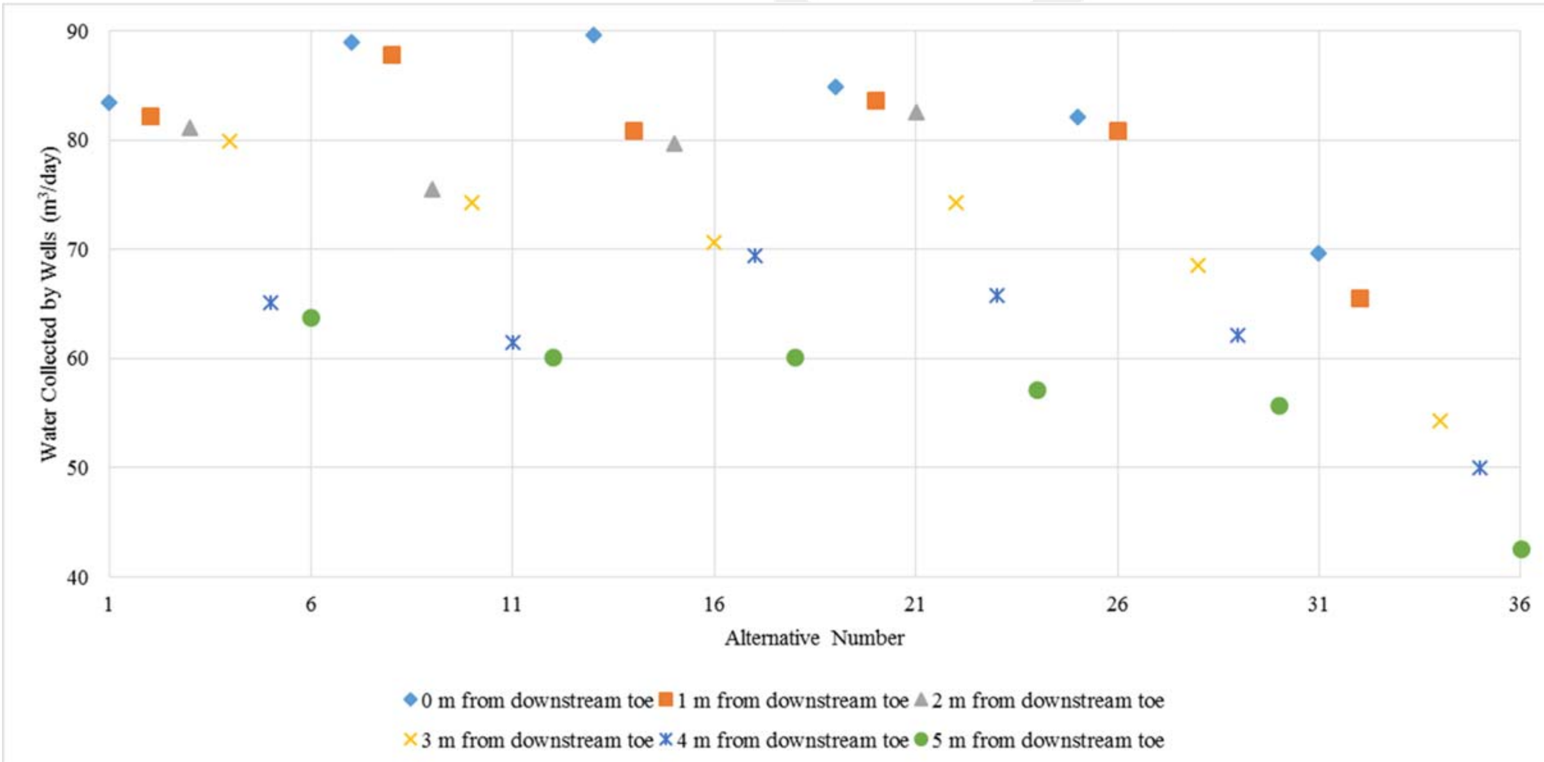

Figure 3.83. Pump performances

In Table 3.7, the minimum pumping capacities are presented. Those capacities are taken from analysis results.

In Table 3.8, operating hour column is taken from the Table 3.6.

The electricity price is calculated as an average of 3 different prices. Those prices are morning price, peak time price and night price. Average of those prices are added to the energy distribution price. The prices of electricity are taken from the EPDK (Enerji Piyasası Düzenleme Kurumu). The electricity is priced in Turkish Lira. Therefore, the prices are converted to USD by using the 01/07/2020 dated currency rate of Central Bank of Turkey. The rate was 1 TRY = 6.8507 USD on the stated date.

In Table 3.9, Annual electricity column is calculated by multiplying electricity consumption data in kW, operating hour/day, total number of days in a year and electricity price.

The price for well drilling is taken from DSİ (Devlet Su İşleri) manual (DSİ, 2020). The price for drilling is given in the manual as price/m, such that price is multiplied with well depth and number of wells.

Control panel price column is calculated with the price of materials multiplied with the number of wells.

In Table 3.10, initial cost column is calculated by adding up well drilling cost, total pump price and control panel.

Annual operating and maintenance cost column is taken as 1% of the initial cost (Karataban, 1976).

Annual total cost column is calculated by adding the yearly electricity price, annual operating and maintenance cost and initial cost multiplied by capital recovery factor. For capital recovery factor, interest rate is taken as 10% and period is taken as 50 years.

Table 3.7 Minimum required pumping capacity

				Min. Pump Discharge		
Alternative	Piece	Spacing (m)	Distance From Toe (m)	m^3/s	m^3/h	m^3/day
1	19	5	0	7.E-05	0.25	6.05
2	19		1	7.E-05	0.25	6.05
3	19		2	8.E-05	0.29	6.91
4	19		3	8.E-05	0.29	6.91
5	19		4	9.E-05	0.32	7.78
6	19		5	9.E-05	0.32	7.78
7	16	6	0	9.E-05	0.32	7.78
8	16		1	9.E-05	0.32	7.78
9	16		2	9.E-05	0.32	7.78
10	16		3	1.E-04	0.36	8.64
11	16		4	1.E-04	0.36	8.64
12	16		5	1.E-04	0.40	9.50
13	11	9	0	1.E-04	0.47	11.23
14	11		1	1.E-04	0.47	11.23
15	11		2	1.E-04	0.47	11.23
16	11		3	1.E-04	0.50	12.10
17	11		4	2.E-04	0.54	12.96
18	11		5	2.E-04	0.54	12.96
19	10	10	0	1.E-04	0.50	12.10
20	10		1	1.E-04	0.50	12.10
21	10		2	2.E-04	0.58	13.82
22	10		3	2.E-04	0.58	13.82
23	10		4	2.E-04	0.58	13.82
24	10		5	2.E-04	0.61	14.69
25	7	15	0	2.E-04	0.76	18.14
26	7		1	2.E-04	0.76	18.14
27	7		2	2.E-04	0.79	19.01
28	7		3	2.E-04	0.83	19.87
29	7		4	2.E-04	0.86	20.74
30	7		5	3.E-04	0.90	21.60
31	4	30	0	4.E-04	1.48	35.42
32	4		1	4.E-04	1.51	36.29
33	4		2	4.E-04	1.55	37.15
34	4		3	4.E-04	1.58	38.02
35	4		4	5.E-04	1.66	39.74
36	4		5	5.E-04	1.73	41.47

Table 3.8 Selected pumps and properties

	Pump				
Alternative	Pump Type	Price/Pump (USD)	Total Price(USD)	Consumption (kW)	Operating Hour/Day
1	4SD4/8	200	3,800.00	0.55	0.88
2	4SD4/8	200	3,800.00	0.55	0.87
3	4SD4/8	200	3,800.00	0.55	0.85
4	4SD4/8	200	3,800.00	0.55	0.84
5	4SD4/8	200	3,800.00	0.55	0.68
6	4SD4/8	200	3,800.00	0.55	0.67
7	4SD4/8	200	3,200.00	0.55	1.11
8	4SD4/8	200	3,200.00	0.55	1.10
9	4SD4/8	200	3,200.00	0.55	0.94
10	4SD4/8	200	3,200.00	0.55	0.93
11	4SD4/8	200	3,200.00	0.55	0.77
12	4SD4/8	200	3,200.00	0.55	0.75
13	4SD4/8	200	2,200.00	0.55	1.63
14	4SD4/8	200	2,200.00	0.55	1.47
15	4SD4/8	200	2,200.00	0.55	1.45
16	4SD4/8	200	2,200.00	0.55	1.28
17	4SD4/8	200	2,200.00	0.55	1.26
18	4SD4/8	200	2,200.00	0.55	1.09
19	4SD4/8	200	2,000.00	0.55	1.70
20	4SD4/8	200	2,000.00	0.55	1.67
21	4SD4/8	200	2,000.00	0.55	1.65
22	4SD4/8	200	2,000.00	0.55	1.48
23	4SD4/8	200	2,000.00	0.55	1.32
24	4SD4/8	200	2,000.00	0.55	1.14
25	4SD4/8	200	1,400.00	0.55	2.34
26	4SD4/8	200	1,400.00	0.55	2.31
27	4SD4/8	200	1,400.00	0.55	2.14
28	4SD4/8	200	1,400.00	0.55	1.96
29	4SD4/8	200	1,400.00	0.55	1.78
30	4SD4/8	200	1,400.00	0.55	1.59
31	4SD4/8	200	800.00	0.55	3.48
32	4SD4/8	200	800.00	0.55	3.28
33	4SD4/8	200	800.00	0.55	3.07
34	4SD4/8	200	800.00	0.55	2.71
35	4SD4/8	200	800.00	0.55	2.50
36	4SD4/8	200	800.00	0.55	2.13

Table 3.9 Additional costs

	Additional Costs		
Alternative	Annual Electricity (USD/year)	Well Drilling (USD)	Control Panel (USD)
1	294.57	12,254.62	$1,520.00
2	290.53	12,254.62	$1,520.00
3	286.39	12,254.62	$1,520.00
4	282.29	12,254.62	$1,520.00
5	229.80	12,254.62	$1,520.00
6	225.16	12,254.62	$1,520.00
7	314.15	10,319.68	$1,280.00
8	310.24	10,319.68	$1,280.00
9	266.48	10,319.68	$1,280.00
10	262.14	10,319.68	$1,280.00
11	217.06	10,319.68	$1,280.00
12	212.29	10,319.68	$1,280.00
13	316.59	7,094.78	$880.00
14	285.59	7,094.78	$880.00
15	281.43	7,094.78	$880.00
16	249.53	7,094.78	$880.00
17	245.06	7,094.78	$880.00
18	212.29	7,094.78	$880.00
19	299.51	6,449.80	$800.00
20	295.47	6,449.80	$800.00
21	291.39	6,449.80	$800.00
22	262.14	6,449.80	$800.00
23	232.32	6,449.80	$800.00
24	201.99	6,449.80	$800.00
25	289.79	4,514.86	$560.00
26	285.59	4,514.86	$560.00
27	263.98	4,514.86	$560.00
28	241.97	4,514.86	$560.00
29	219.61	4,514.86	$560.00
30	196.85	4,514.86	$560.00
31	245.91	2,579.92	$320.00
32	231.39	2,579.92	$320.00
33	216.63	2,579.92	$320.00
34	191.61	2,579.92	$320.00
35	176.58	2,579.92	$320.00
36	150.54	2,579.92	$320.00

Table 3.10 Total cost

Alternative	Initial Cost (USD/year)	Annual Operating and Maintenance Cost (USD/year)	Annual Total Cost (USD/year)
1	17,574.62	175.75	2,242.88
2	17,574.62	175.75	2,238.84
3	17,574.62	175.75	2,234.70
4	17,574.62	175.75	2,230.60
5	17,574.62	175.75	2,178.11
6	17,574.62	175.75	2,173.47
7	14,799.68	148.00	1,954.83
8	14,799.68	148.00	1,950.92
9	14,799.68	148.00	1,907.16
10	14,799.68	148.00	1,902.82
11	14,799.68	148.00	1,857.74
12	14,799.68	148.00	1,852.97
13	10,174.78	101.75	1,444.55
14	10,174.78	101.75	1,413.56
15	10,174.78	101.75	1,409.40
16	10,174.78	101.75	1,377.50
17	10,174.78	101.75	1,373.03
18	10,174.78	101.75	1,340.26
19	9,249.80	92.50	1,324.94
20	9,249.80	92.50	1,320.89
21	9,249.80	92.50	1,316.81
22	9,249.80	92.50	1,287.57
23	9,249.80	92.50	1,257.75
24	9,249.80	92.50	1,227.41
25	6,474.86	64.75	1,007.59
26	6,474.86	64.75	1,003.39
27	6,474.86	64.75	981.78
28	6,474.86	64.75	959.76
29	6,474.86	64.75	937.41
30	6,474.86	64.75	914.65
31	3,699.92	37.00	656.08
32	3,699.92	37.00	641.56
33	3,699.92	37.00	626.80
34	3,699.92	37.00	601.78
35	3,699.92	37.00	586.75
36	3,699.92	37.00	560.71

The results of calculations presented in Tables 3.7 through 3.10 are plotted on a graph given in Figure 3.84. In this figure, it is seen that between similar spacing values, alternatives with 5 m from the downstream toe are the cheapest as they are working less than other options. The reason for this is, seepage passing the wells are more in those pump locations, this yields to less operating hours which results in less annual total cost. Also between similar spacing values, the highest annual total cost belong to the ones that are 0 m from the downstream toe of the dam. A trend of decreasing cost can be seen in every 6 alternatives with similar spacing values.

When the results of Figure 3.83 and 3.84 evaluated together, it can be concluded that alternatives 1 through 6 are worst alternatives in terms of annual total cost and also those alternatives has no significant advantage on collection of water by wells.

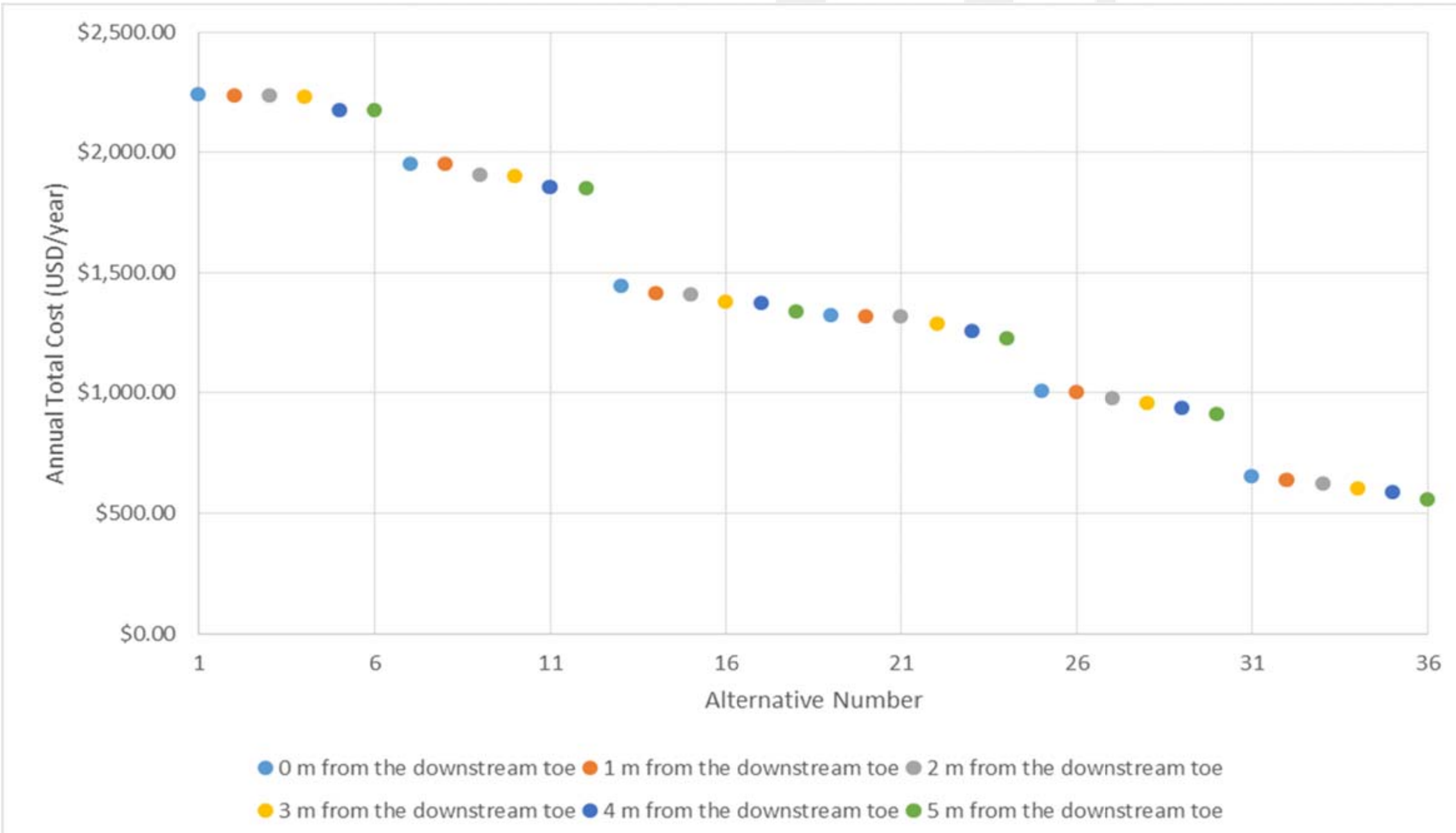

Figure 3.84. Annual total cost

3.9 Summary of the Results

As discussed in the previous parts, the seepage performances of wells with lower spacing is better than wider spacing values. The comparison in terms of seepage rate is done with the results of low-cost alternatives. This evaluation of performances does not include all of the results with high-cost and low-performance results to finalize the comparison and select the best alternative in terms of seepage. This comparison also includes 1 poorly performed result to compare the poor option with alternatives. Alternative 1 through Alternative 12 are the cases with concrete cutoff structures and Alternatives 20, 21 and 22 are the cases with core trenches. The alternatives 13 through 19 are the different well cases where the properties of the wells are defined in Table 3.11.

This subsection does not include cost comparison of alternatives as cutoff alternatives don’t have cost analyses because of the previously highlighted reasons.

Table 3.11 Comparison of alternatives

Alternative	Type	Trench height/ pervious zone height (%)	Location
Alternative 1	Cutoff wall	25	Upstream
Alternative 2	Cutoff wall	25	Center
Alternative 3	Cutoff wall with blanket	25	Upstream
Alternative 4	Cutoff wall with blanket	25	Center
Alternative 5	Cutoff wall	50	Upstream
Alternative 6	Cutoff wall	50	Center
Alternative 7	Cutoff wall with blanket	50	Upstream
Alternative 8	Cutoff wall with blanket	50	Center
Alternative 9	Cutoff wall	75	Upstream
Alternative 10	Cutoff wall	75	Center
Alternative 11	Cutoff wall with blanket	75	Upstream
Alternative 12	Cutoff wall with blanket	75	Center
Alternative 13	Well with 0.46 m diameter & 5 m spacing	-	0 m from toe
Alternative 14	Well with 0.46 m diameter & 6 m spacing	-	0 m from toe
Alternative 15	Well with 0.46 m diameter & 10 m spacing	-	0 m from toe
Alternative 16	Well with 0.15 m diameter & 5 m spacing	-	0 m from toe
Alternative 17	Well with 0.15 m diameter & 6 m spacing	-	0 m from toe
Alternative 18	Well with 0.15 m diameter & 10 m spacing	-	0 m from toe
Alternative 19	Well with 0.46 m diameter & 30 m spacing	-	5 m from toe
Alternative 20	Core Trench	25	Center
Alternative 21	Core Trench	50	Center
Alternative 22	Core Trench	75	Center

The results of the given alternatives are presented in Figures 3.85, 3.86 and 3.87 in bar charts. In those charts, lower seepage rate value represents the best option. The alternatives are aligned by seepage amounts from the left (high seepage) to the right (lower seepage).

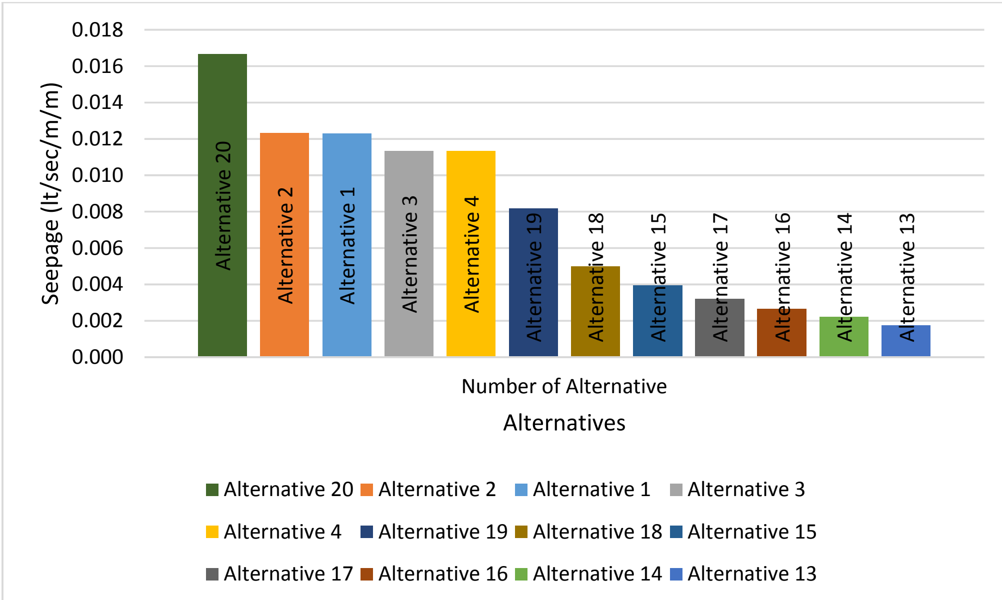

Figure 3.85. Alternative comparison – cutoff depth/pervious zone height ratio 0.25

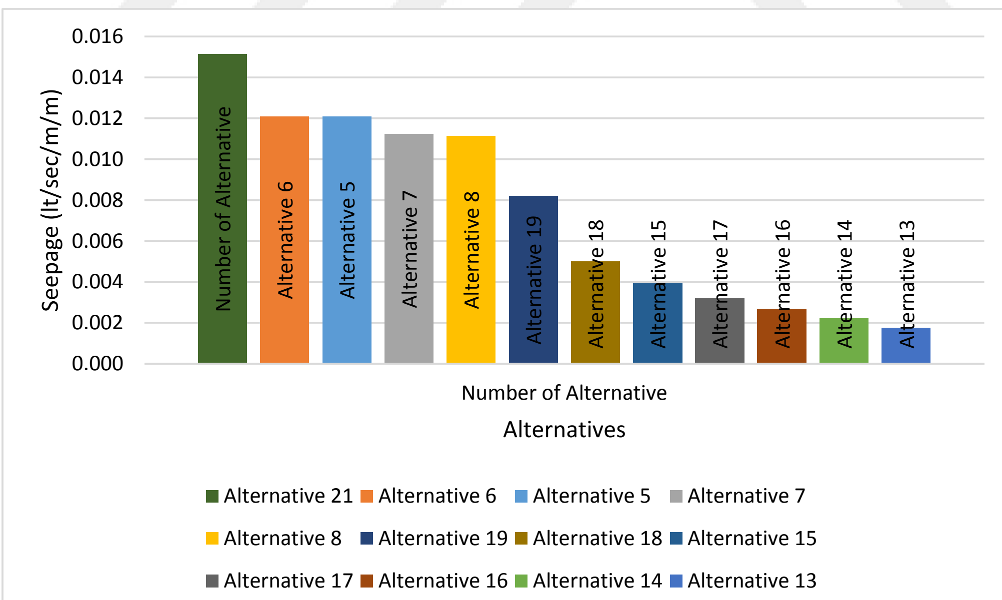

Figure 3.86. Alternative comparison– cutoff depth/pervious zone height ratio 0.50

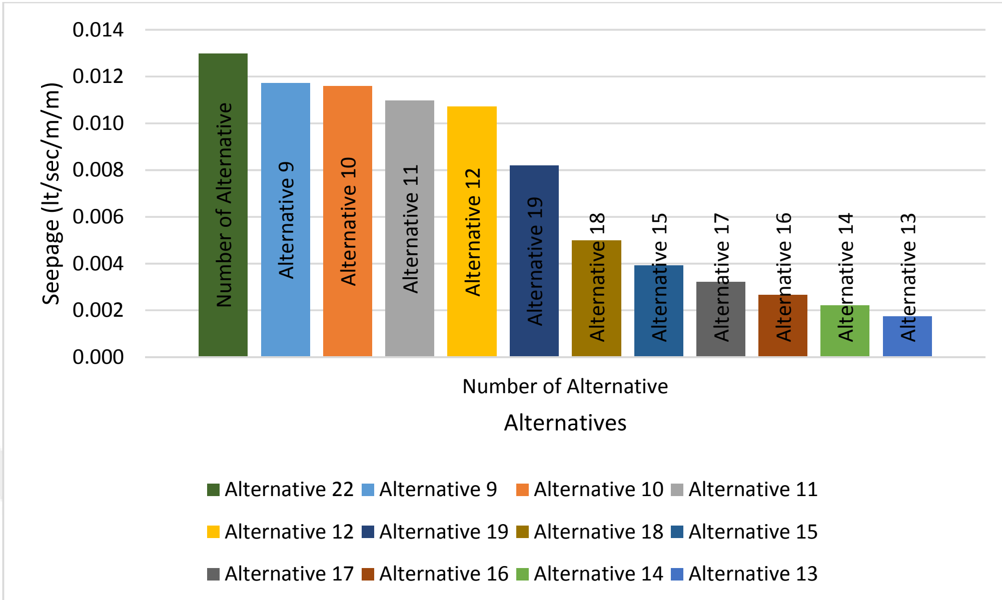

Figure 3.87. Alternative comparison– cutoff depth/pervious zone height ratio 0.75

According to Figures 3.85, 3.86 and 3.87, seepage rate results for wells are performed better than other alternatives. The figures indicate that from the ordered alternatives, from the most effective type of solution to the less effective are relief wells, cutoff walls with blanket, cutoff walls without blanket and core trench.

CHAPTER 4

DISCUSSION

4.1 Relief Wells

The design of relief wells requires calculations for characterizing the design depending on site conditions. The dam, distances between wells, locations and inner diameters of the wells should be carefully adjusted in order to find a permissible solution against intolerable seepage underneath the structure can adversely affect the foundation.

Presented results for relief wells show that the increase in distance between the wells would lead to increase in total seepage. Increase in well spacing results in decreased effective areas subjecting to seepage. When the total number of the wells increase in a section, more water is collected by the wells and less seepage occurs. The decrease in seepage with the increase in inner well diameter has the same relation between them. The increase in well area resulted in an increase in collected water amount as it could hold more water. The location of the wells has also an effect on seepage. Closer the relief wells built to the downstream toe, the more effective they are as the main role of the wells is to protect the foundation and the downstream toe of the dam.

Similar results are obtained for the pore water pressure. The decrease of total area of wells yields to higher pore water pressures. Same logic applies for the increase of spacing. When spacing between wells increase, pore water pressures increase also. So it can be said that higher number of wells with less spacing value are better performing in terms of pore water pressure. The pore water pressure is affected less from the effect of distance of wells from the downstream toe of the dam but still it can be concluded that with a small effect, constructing relief wells closer to the downstream toe is resulted better for the pore water pressure.

4.2 Cutoff Walls and Trenches

The increase in cutoff wall depth/pervious zone height ratio yields to decrease in seepage amount. Among three different types of seepage reduction methods with the location selected as center of the dam, the highest rate of decrease is observed on core trench method. At this location, it is calculated that the effect of blocked percentage yields a difference of 6.2% on seepage rate with the cutoff wall without blanket, 5.7% on seepage rate with the cutoff wall with blanket and 19.7% on seepage rate with the core trench.

It is clearly visible that the seepage rate is not affected by location of cutoff wall with blanket method. The location effect is seen on the cutoff wall without blanket but the change of seepage rate is limited to ~1%. The cutoff wall (without blanket) can be placed to the upstream side for making it slightly more efficient and effective.

Overall, placing a deeper cutoff wall or trench is more effective in terms of seepage rate. The location has a negligible effect on seepage rate for discussed prevention methods. Between discussed methods, the cutoff wall with a blanket has the best performance in terms of seepage.

4.3 Cost Comparison for Wells

In terms of cost, the pumps used for these analyses have annual costs between $2,242 and $560. It can be concluded that, by the increase in spacing value, the price range starts to increase. The alternatives 5 m from the downstream toe has lower prices as their consumption is less. The reason for less consumption is not related to the pump types but the amount they pump out from the wells. It is also presented that lowest total amount of water pumped out are wıth the wells 5 m from downstream toe.

Optimum alternative should have high amount of water collected and low annual total cost. The best data for collected water is on alternatives with spacing values of 6 m and 9 m, which are alternatives 7 through 18. Between those alternatives, ones with 9 m spacing value have less costs if costs are compared in between wells with same locations. It is previously highlighted that when the wells are placed closer to the structure, the better results are obtained so author recommends alternative 13, 14 or 15 as they are closer to the downstream toe of the dam and alternative 14 and 15 have similar amounts of collected total water.

The cases with pumps have a range of 45% to 95% better performance in terms of seepage with respect to the case with no wells.

4.4 Overall Comparison

In terms of seepage, using relief wells results in less total seepage amount in the foundation than any other alternative compared in this study. The best solution to seepage is removing the pervious alluvial soil completely if possible under economical limitations or using a cutoff wall or a trench structure until the impervious layer. Among all of the alternatives, constructing optimized number of wells to desired locations at the downstream toe is the best solution as the total seepage rate is reduced more than other presented alternatives in this study.

CHAPTER 5

CONCLUSION

This thesis is focused on the performance evaluation of pressure relief wells downstream of embankment dams, cutoff walls and trenches to decrease seepage which may lead to damage to the foundation. In this manner, Oğulcuk Dam, Turkey, with a pervious foundation is used in the analyses. GeoStudio SEEP/W software is used to handle the calculations for seepage models. Then, the results are combined on graphs and comparisons are done regarding the performances. It should be noted that the outcomes of this study are site-specific.

Main outcomes of this thesis are listed below.

1. Relief wells should be placed close to each other in order to decrease the seepage.
2. Relief wells inner diameters should be as large as possible within the limits of standards to increase the amount of water collection.
3. Relief wells should be placed as close as possible to the dam toe to decrease the seepage and its effects on the toe and the foundation.
4. Relief wells should be placed close to each other in order to decrease pore water pressures.
5. Relief wells inner diameters should be as large as possible within the limits of standards to decrease the pore water pressure at the downstream toe.
6. Relief wells should be constructed as close as possible to the downstream toe in order to maximize the efficiency in terms of pore water pressure.
7. Alternative solutions which cannot completely block the water path has less effect on seepage than relief wells.

This study investigated the cost aspect of relief wells to present readers the combined results with costs and performances. It is recommended that for more precise results and cost estimations, 3D seepage models may be used for a similar study. The novelty of this study is based on comparison of those results of relief wells, cutoff walls and trenches and including cost aspect to this comparison. Considering the outcomes of this study, readers can utilize the results obtained from this study for having an idea about relief wells performances and cutoff structures on the presented conditions and estimate results for their foundation conditions.

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APPENDIX

Analyses Results For Relief Wells With 5 m Spacing

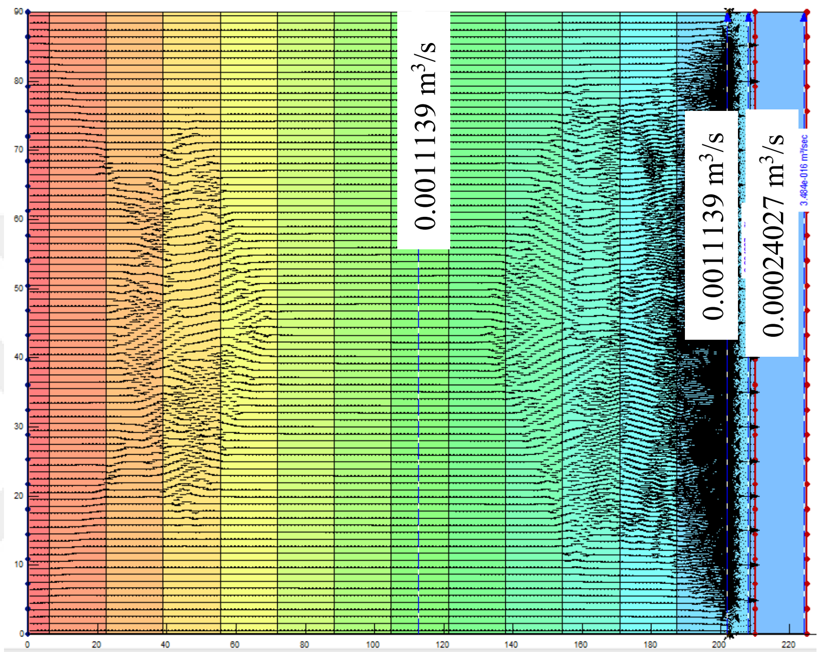

Figure A.1. Plan view analysis of relief wells with 0.15 m inner diameter, 5 m spacing and located at the toe

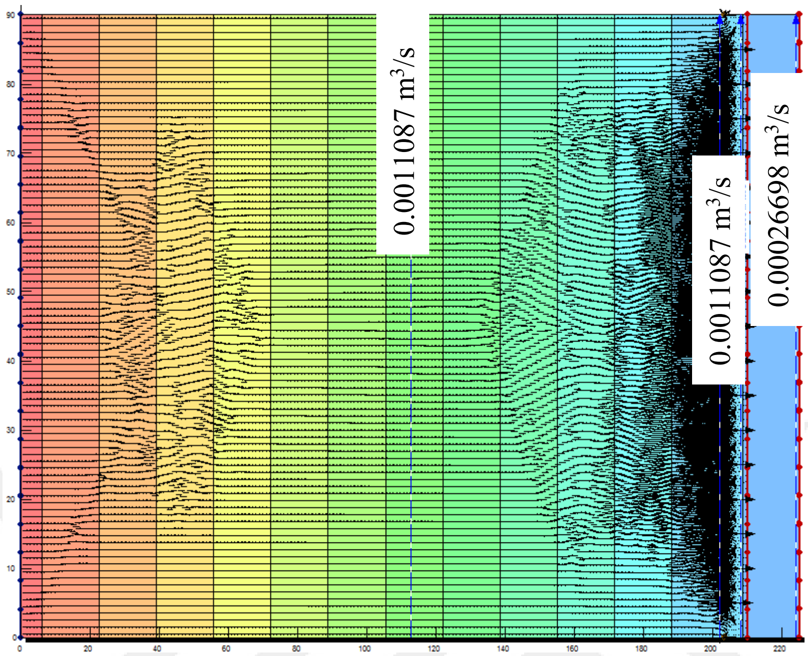

Figure A.2. Plan view analysis of relief wells with 0.15 m inner diameter, 5 m spacing and located 1 m from the toe

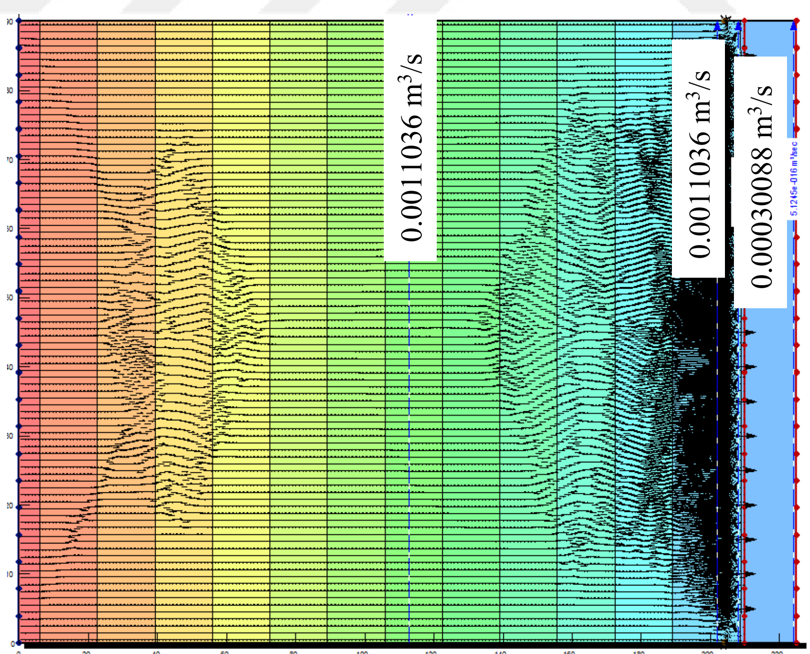

Figure A.3. Plan view analysis of relief wells with 0.15 m inner diameter, 5 m spacing and located 2 m from the toe

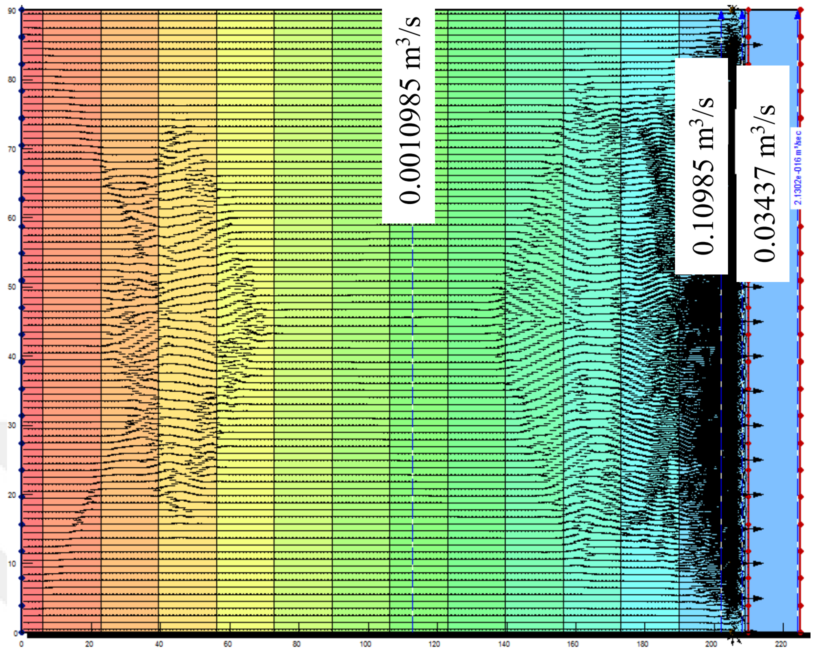

Figure A.4. Plan view analysis of relief wells with 0.15 m inner diameter, 5 m spacing and located 3 m from the toe

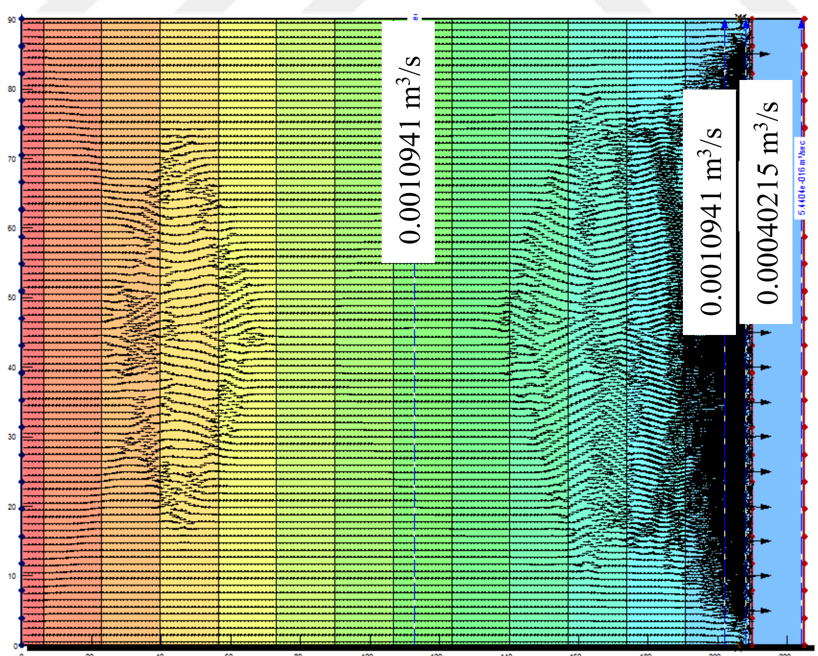

Figure A.5. Plan view analysis of relief wells with 0.15 m inner diameter, 5 m spacing and located 4 m from the toe

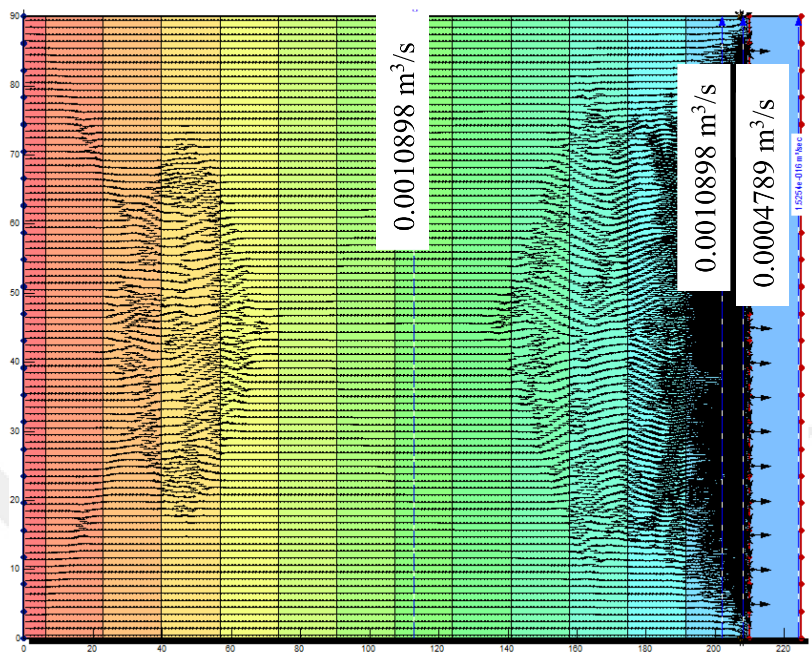

Figure A.6. Plan view analysis of relief wells with 0.15 m inner diameter, 5 m spacing and located 5 m from the toe

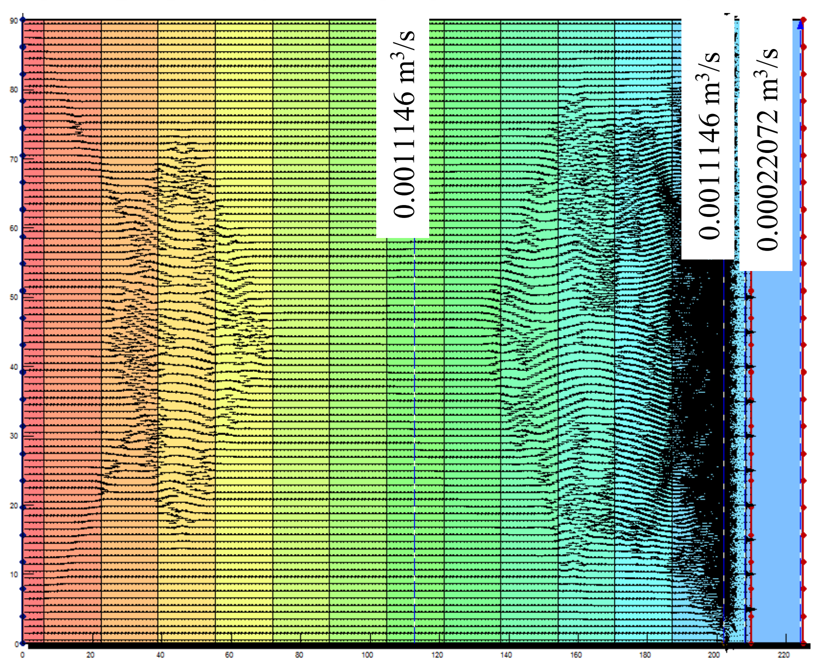

Figure A.7. Plan view analysis of relief wells with 0.20 m inner diameter, 5 m spacing and located at the toe

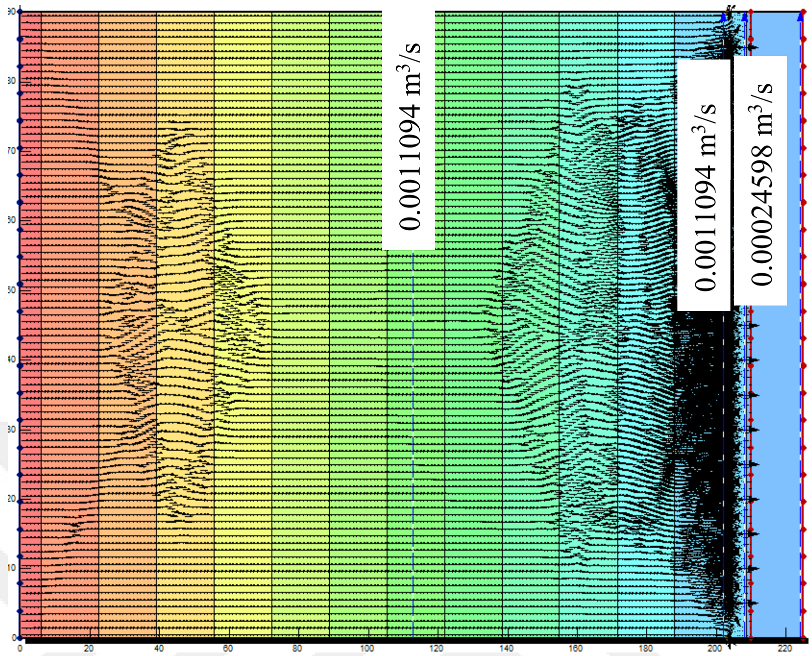

Figure A.8. Plan view analysis of relief wells with 0.20 m inner diameter, 5 m spacing and located 1 m from the toe.

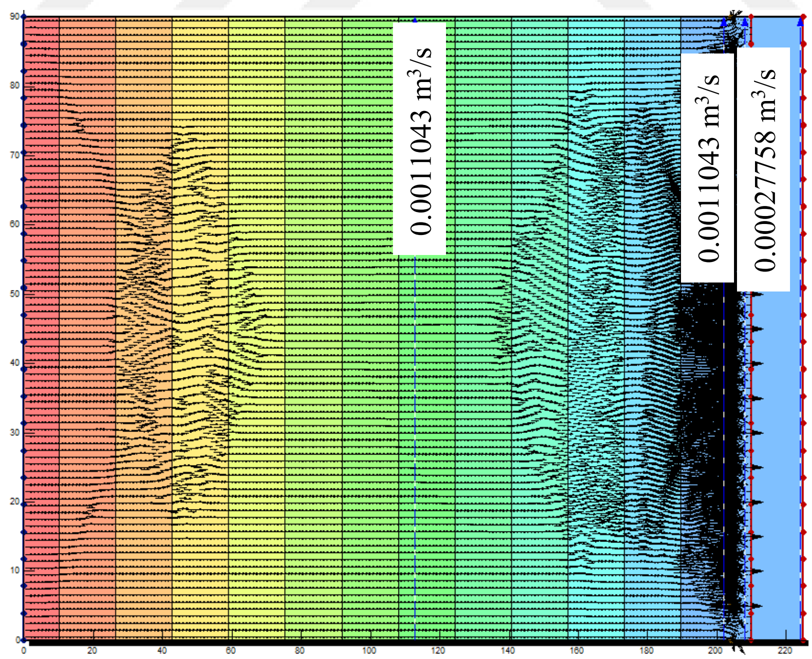

Figure A.9. Plan view analysis of relief wells with 0.20 m inner diameter, 5 m spacing and located 2 m from the toe

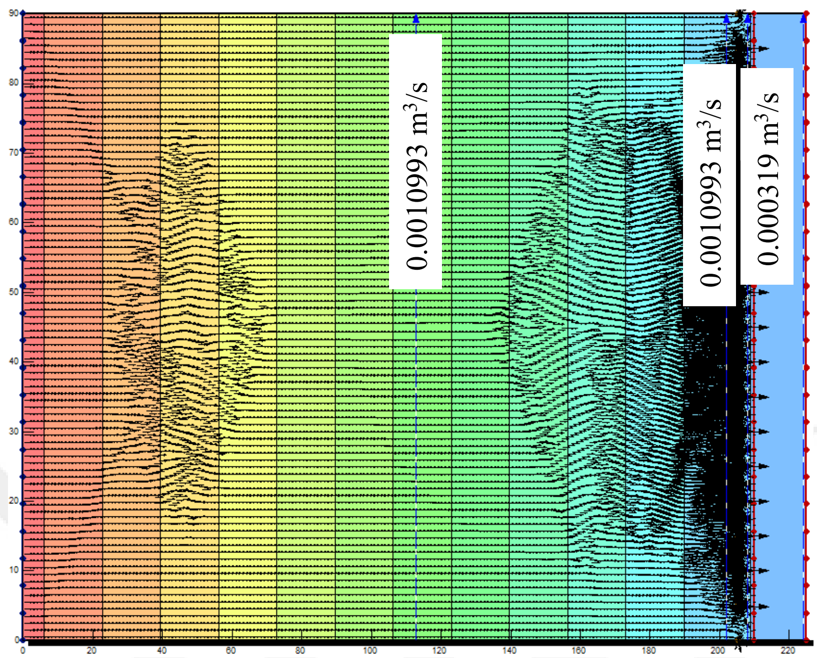

Figure A.10. Plan view analysis of relief wells with 0.20 m inner diameter, 5 m spacing and located 3 m from the toe

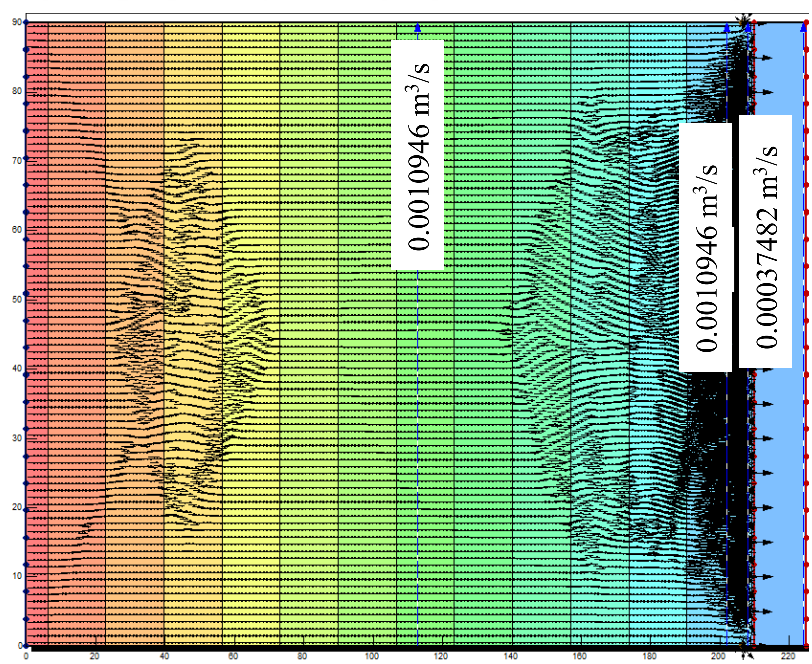

Figure A.11. Plan view analysis of relief wells with 0.20 m inner diameter, 5 m spacing and located 4 m from the toe

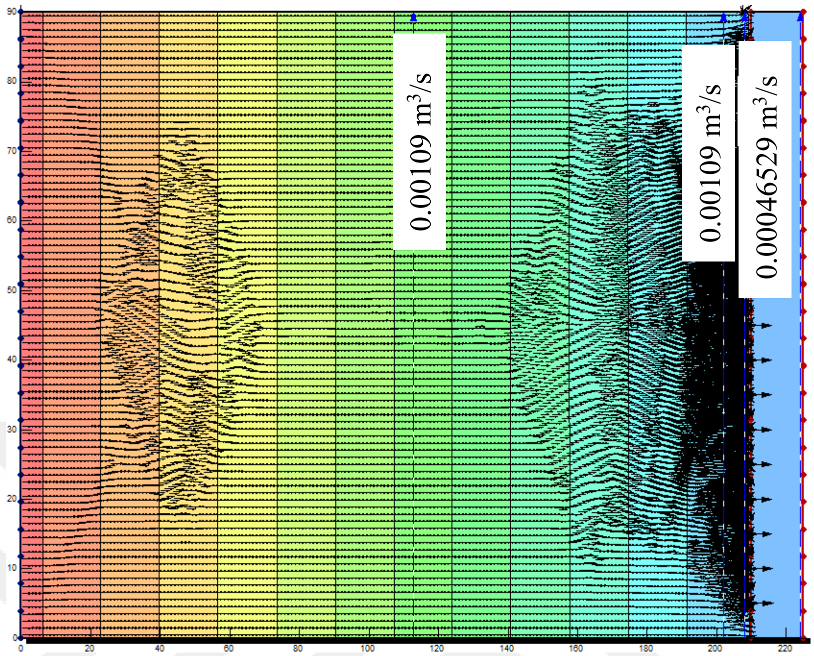

Figure A.12. Plan view analysis of relief wells with 0.20 m inner diameter, 5 m spacing and located 5 m from the toe

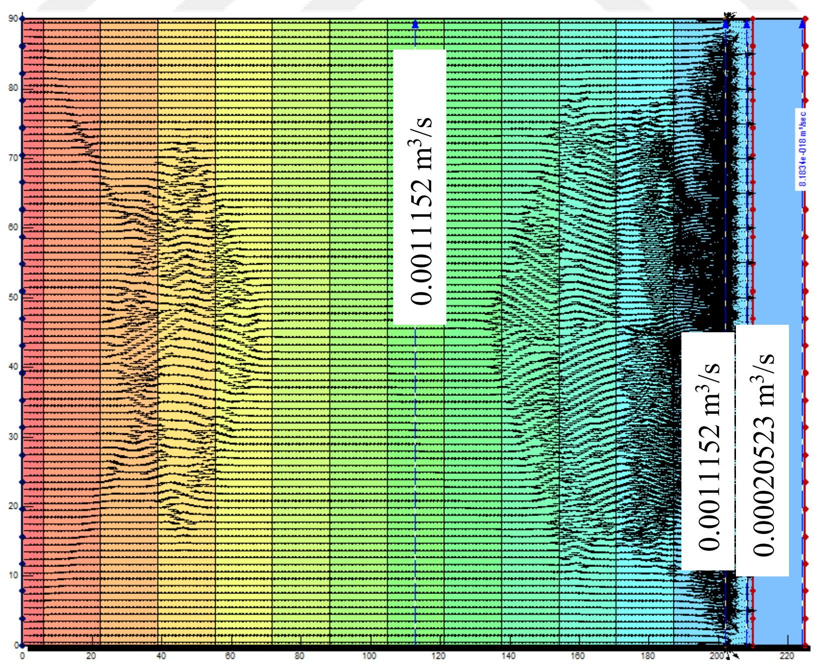

Figure A.13. Plan view analysis of relief wells with 0.25 m inner diameter, 5 m spacing and located at the toe

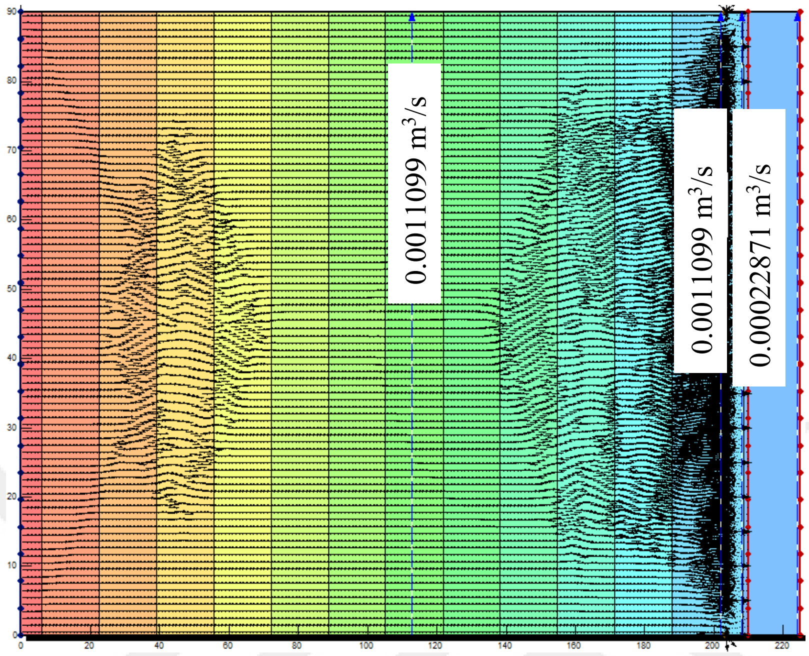

Figure A.14. Plan view analysis of relief wells with 0.25 m inner diameter, 5 m spacing and located 1 m from the toe

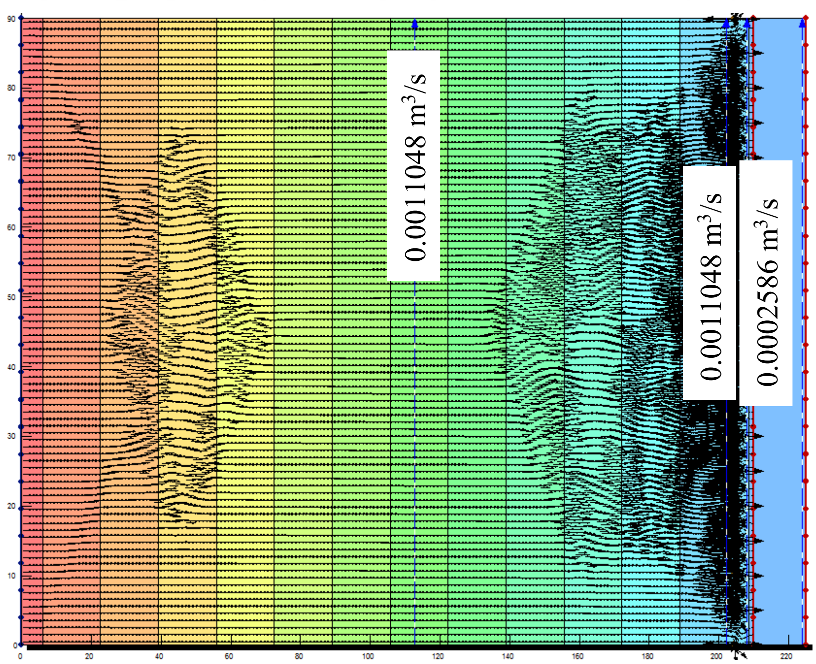

Figure A.15. Plan view analysis of relief wells with 0.25 m inner diameter, 5 m spacing and located 2 m from the toe

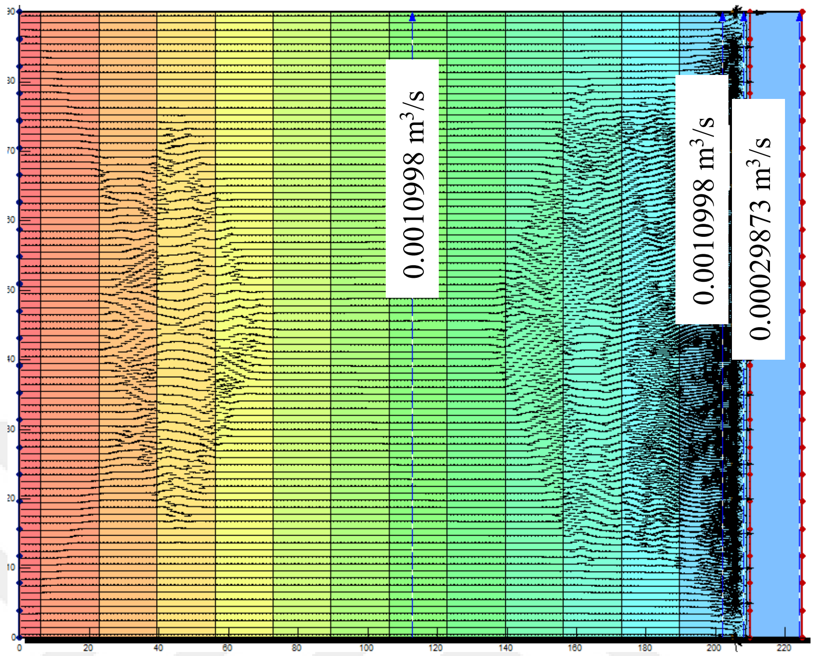

Figure A.16. Plan view analysis of relief wells with 0.25 m inner diameter, 5 m spacing and located 3 m from the toe

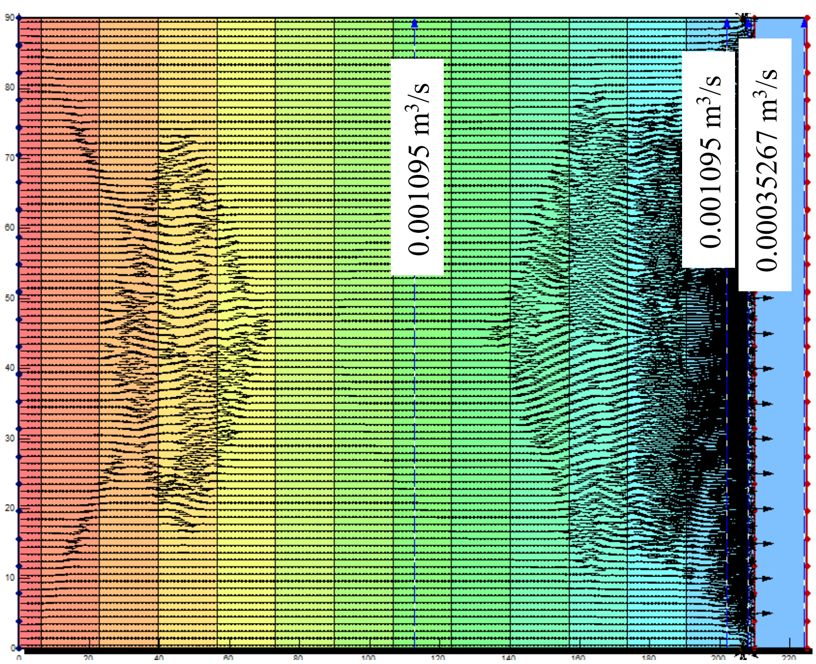

Figure A.17. Plan view analysis of relief wells with 0.25 m inner diameter, 5 m spacing and located 4 m from the toe

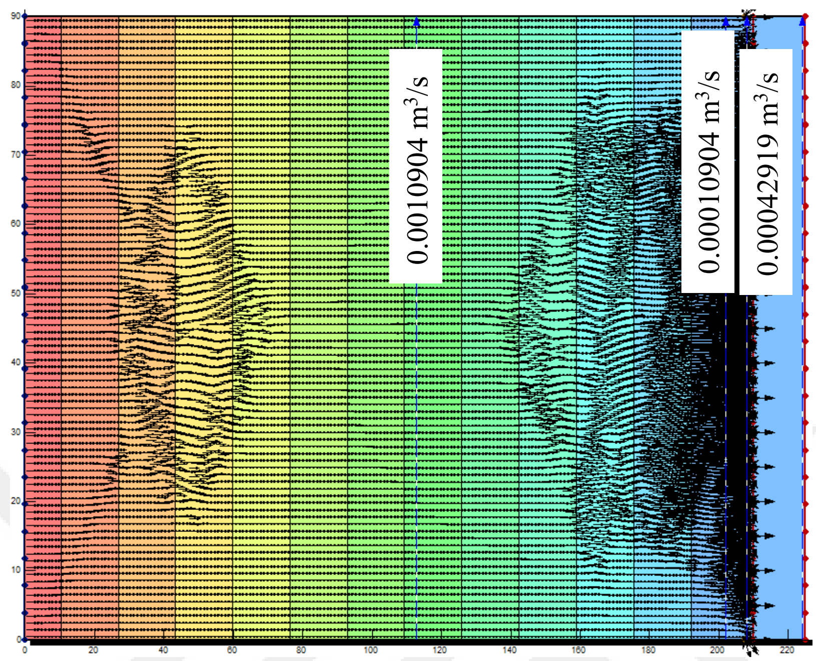

Figure A.18. Plan view analysis of relief wells with 0.25 m inner diameter, 5 m spacing and located 5 m from the toe

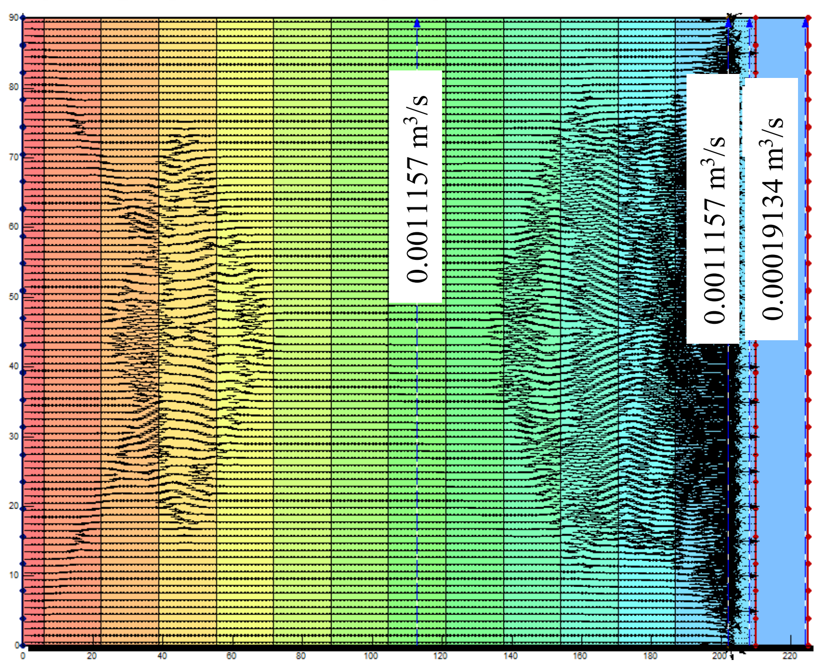

Figure A.19. Plan view analysis of relief wells with 0.30 m inner diameter, 5 m spacing and located at the toe

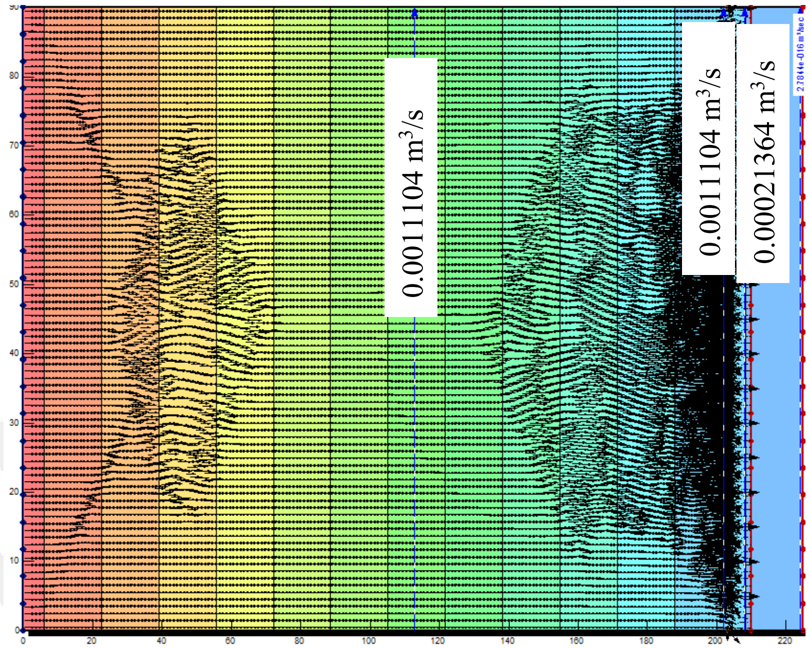

Figure A.20. Plan view analysis of relief wells with 0.30 m inner diameter, 5 m spacing and located 1 m from the toe

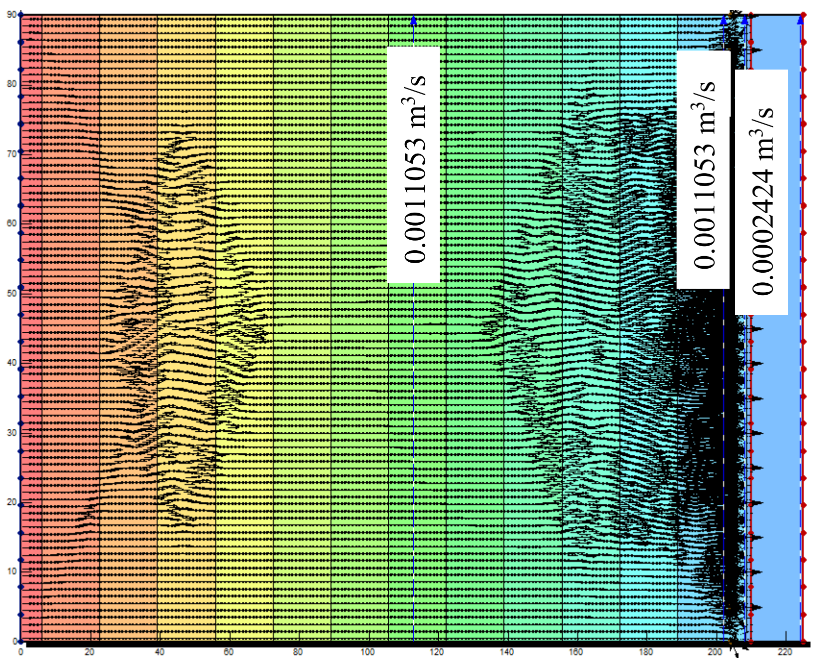

Figure A.21. Plan view analysis of relief wells with 0.30 m inner diameter, 5 m spacing and located 2 m from the toe

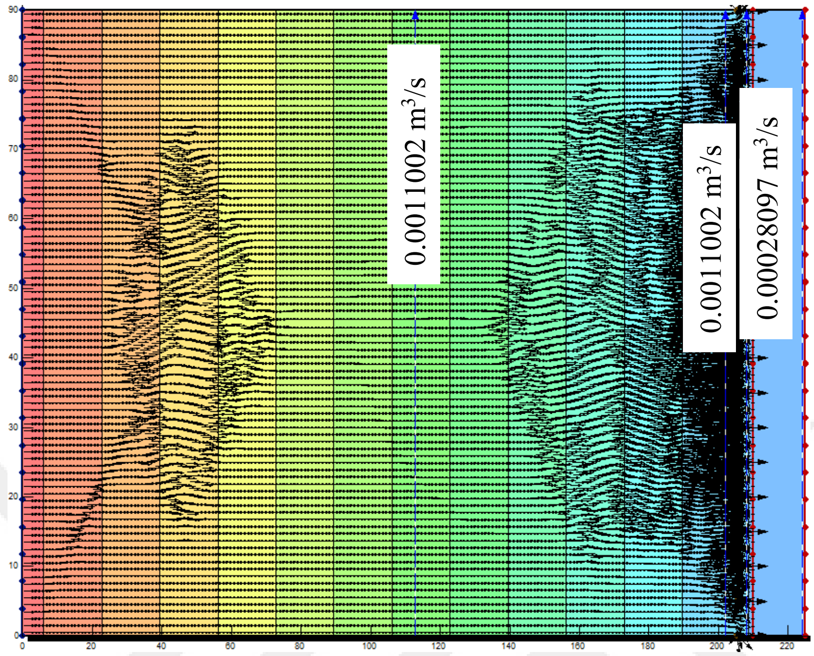

Figure A.22. Plan view analysis of relief wells with 0.30 m inner diameter, 5 m spacing and located 3 m from the toe

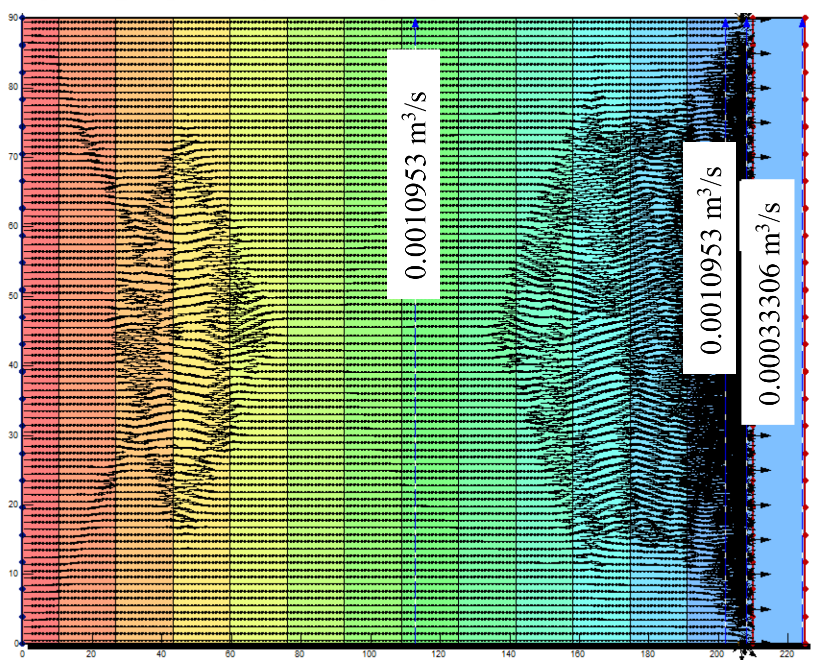

Figure A.23. Plan view analysis of relief wells with 0.30 m inner diameter, 5 m spacing and located 4 m from the toe

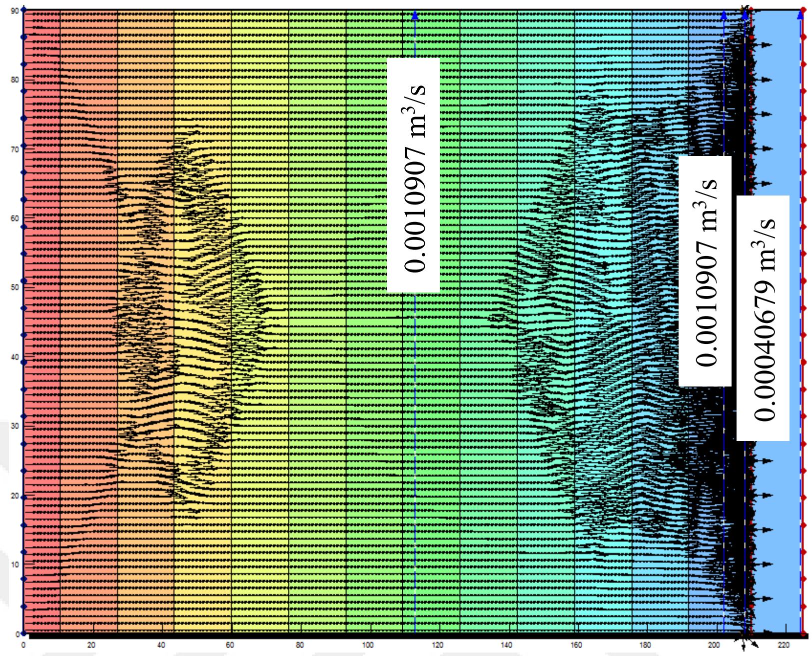

Figure A.24. Plan view analysis of relief wells with 0.30 m inner diameter, 5 m spacing and located 5 m from the toe

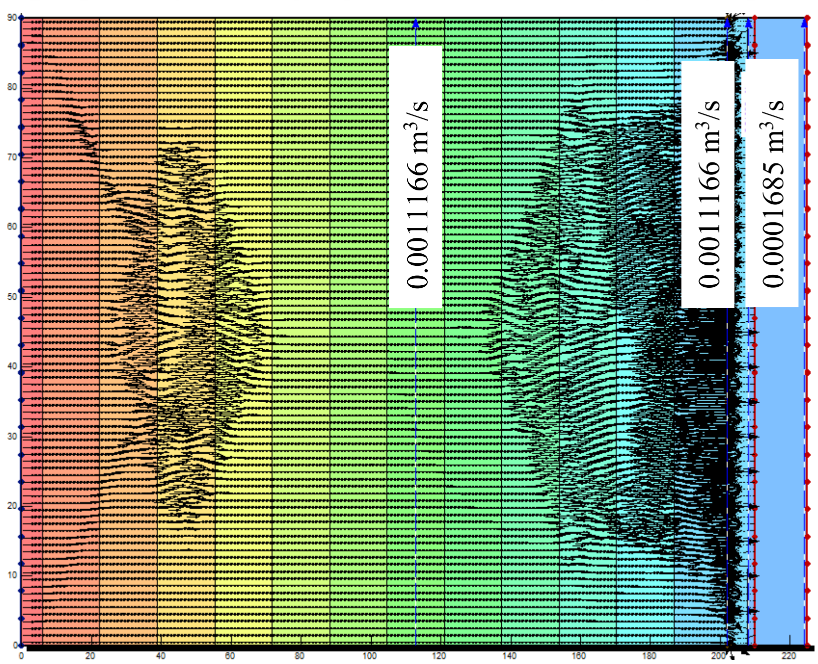

Figure A.25. Plan view analysis of relief wells with 0.40 m inner diameter, 5 m spacing and located at the toe

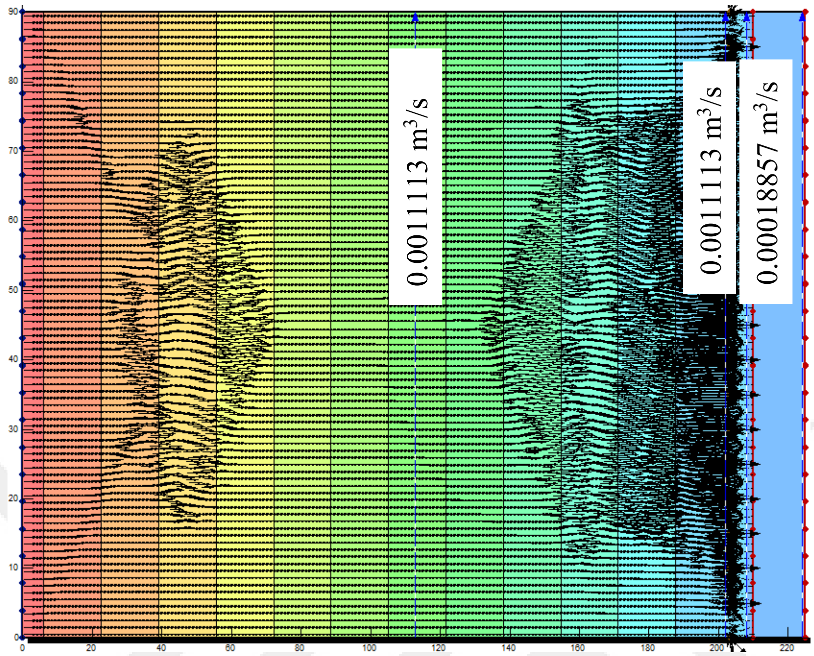

Figure A.26. Plan view analysis of relief wells with 0.40 m inner diameter, 5 m spacing and located 1 m from the toe

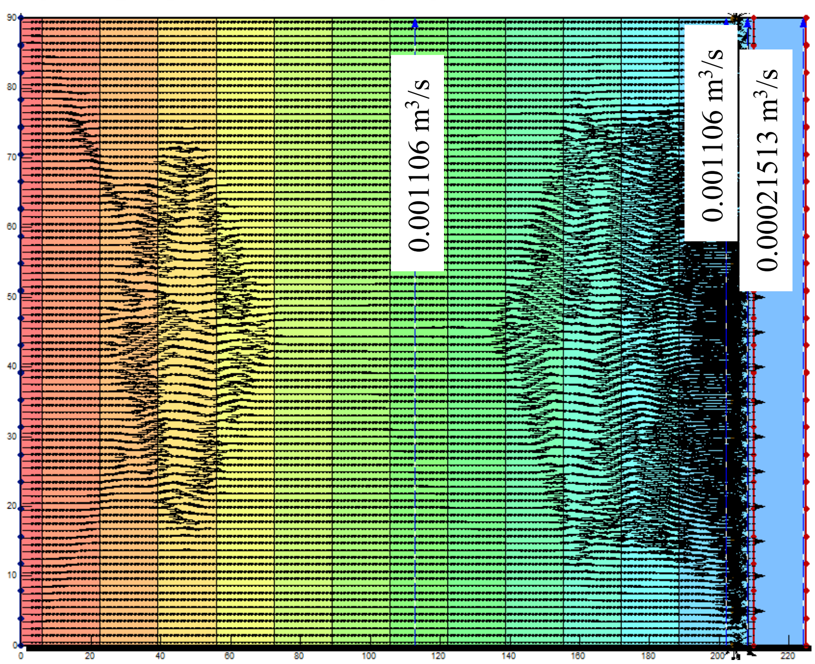

Figure A.27. Plan view analysis of relief wells with 0.40 m inner diameter, 5 m spacing and located 2 m from the toe

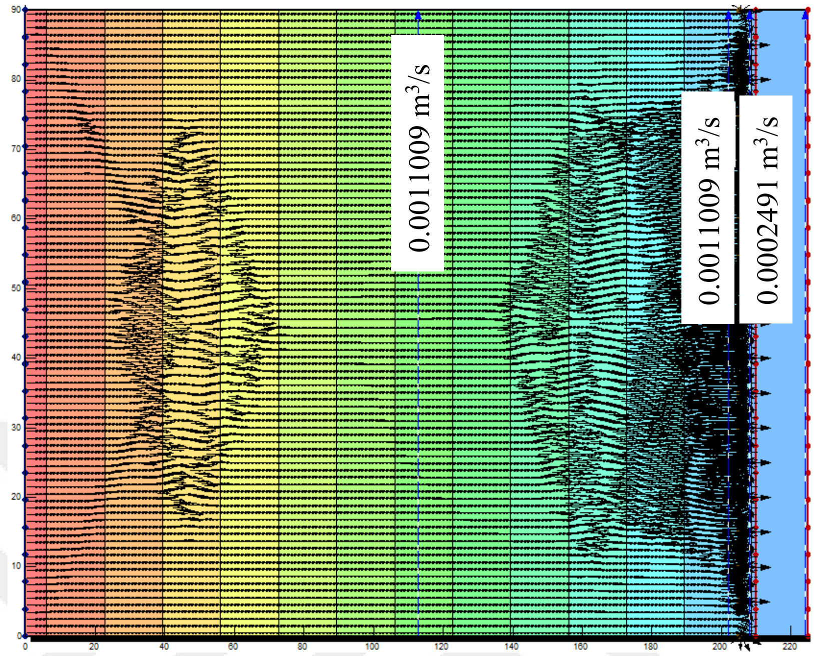

Figure A.28. Plan view analysis of relief wells with 0.40 m inner diameter, 5 m spacing and located 3 m from the toe

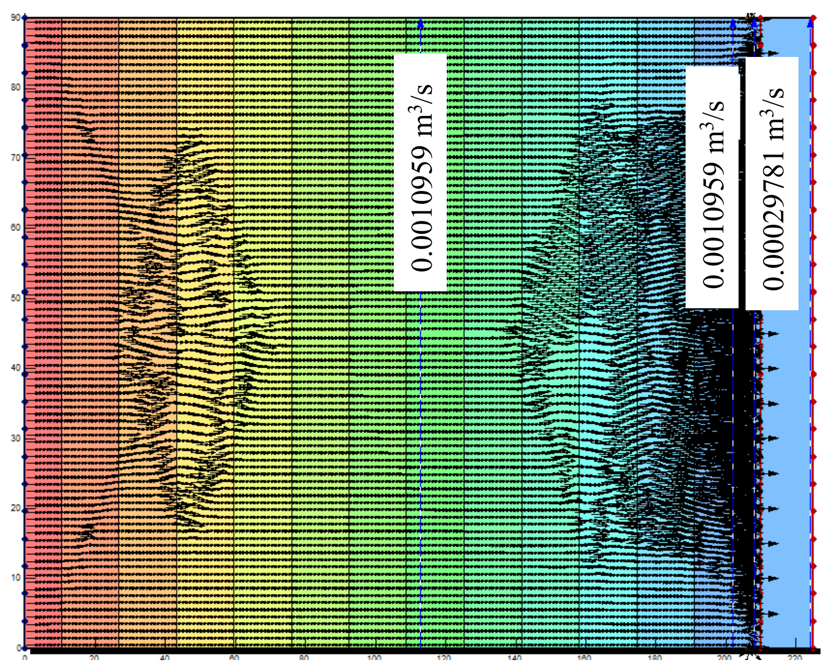

Figure A.29. Plan view analysis of relief wells with 0.40 m inner diameter, 5 m spacing and located 4 m from the toe

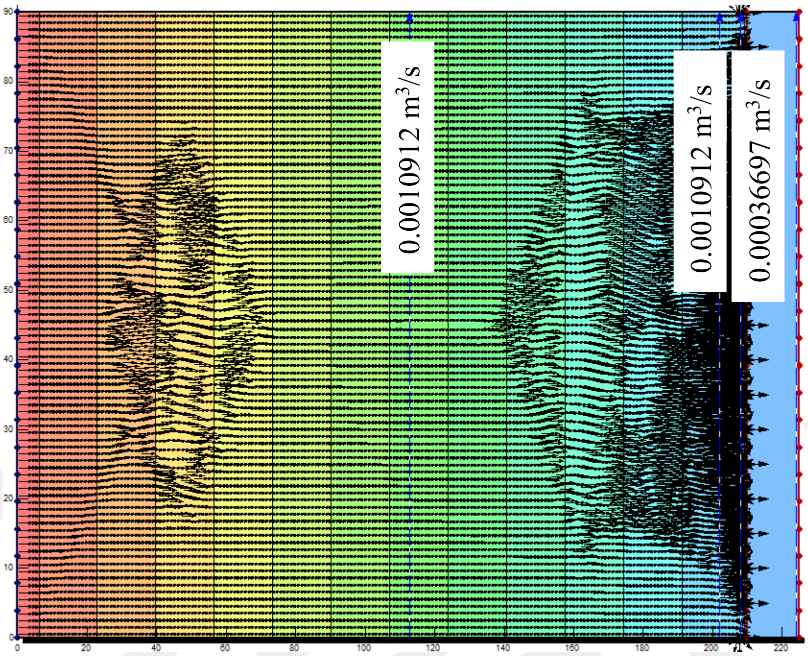

Figure A.30. Plan view analysis of relief wells with 0.40 m inner diameter, 5 m spacing and located 5 m from the toe

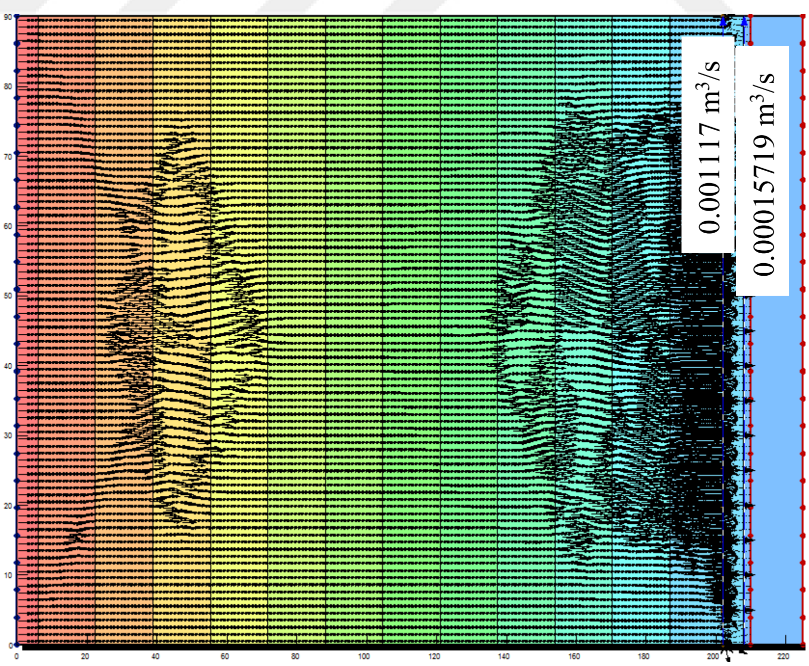

Figure A.31. Plan view analysis of relief wells with 0.46 m inner diameter, 5 m spacing and located at the toe

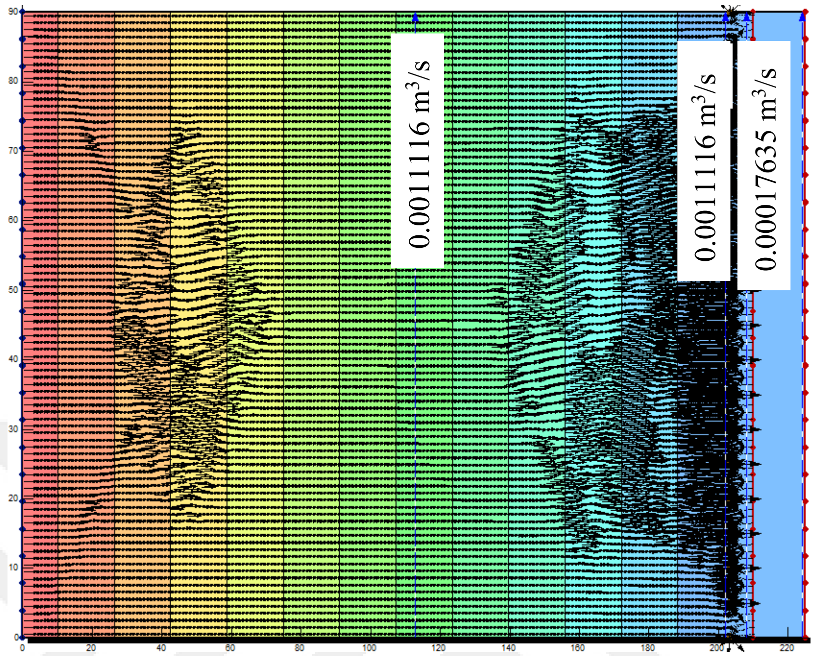

Figure A.32. Plan view analysis of relief wells with 0.46 m inner diameter, 5 m spacing and located 1 m from the toe

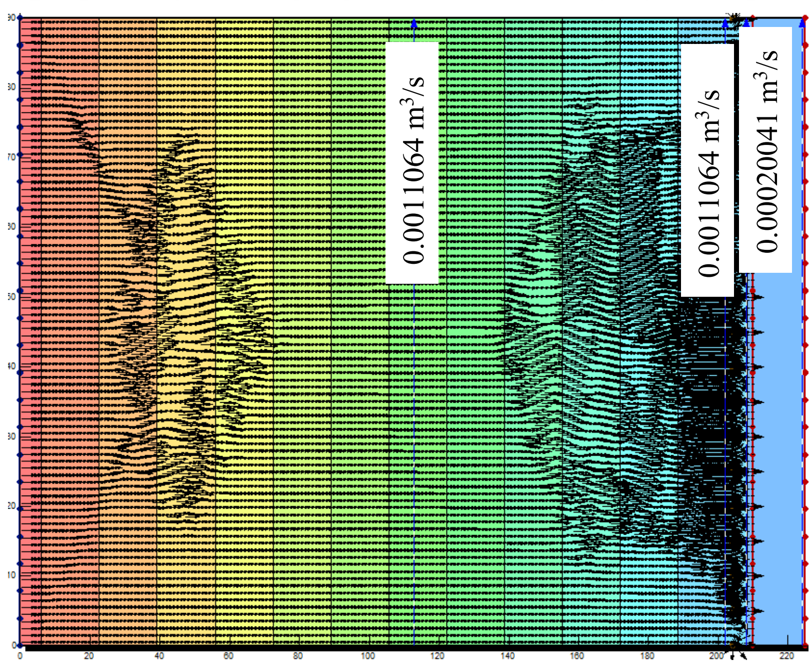

Figure A.33. Plan view analysis of relief wells with 0.46 m inner diameter, 5 m spacing and located 2 m from the toe

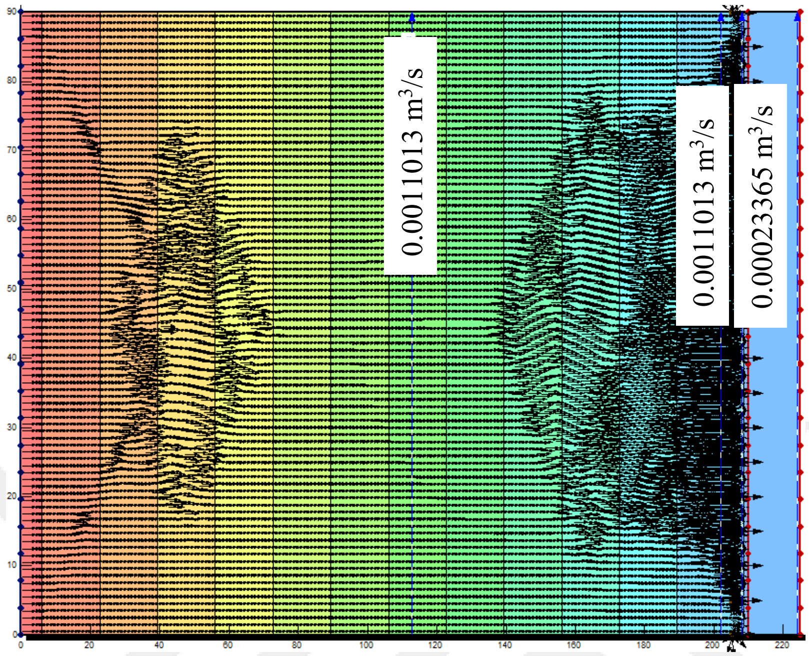

Figure A.34. Plan view analysis of relief wells with 0.46 m inner diameter, 5 m spacing and located 3 m from the toe

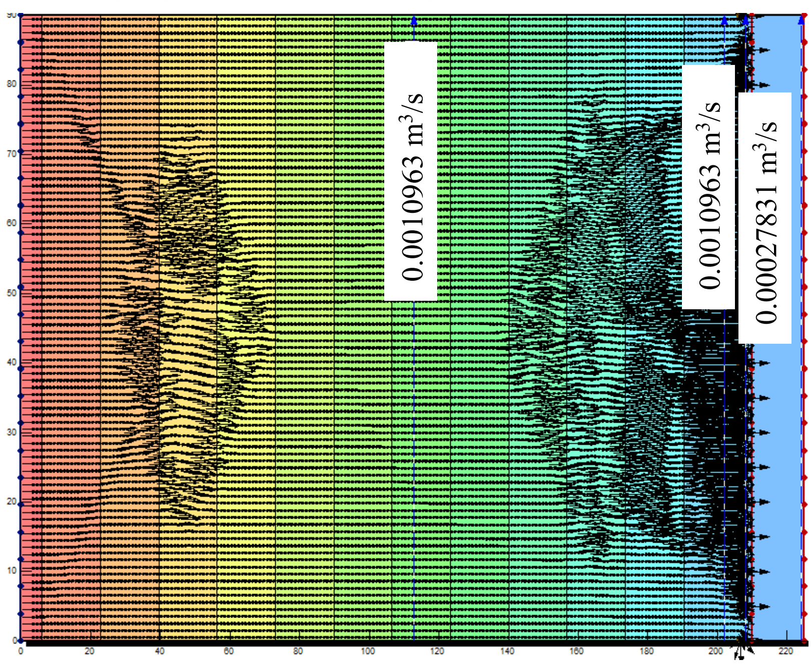

Figure A.35. Plan view analysis of relief wells with 0.46 m inner diameter, 5 m spacing and located 4 m from the toe

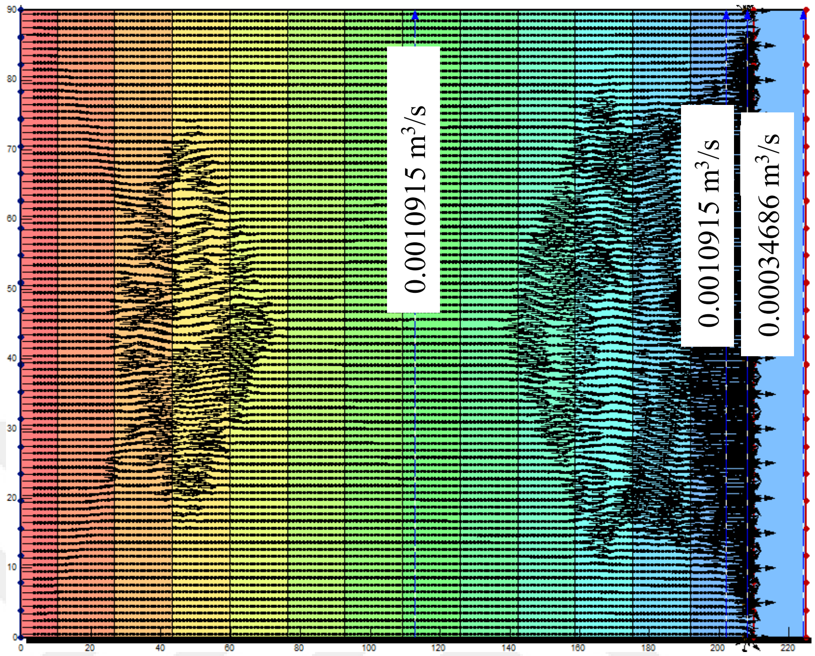

Figure A.36. Plan view analysis of relief wells with 0.46 m inner diameter, 5 m spacing and located 5 m from the toe