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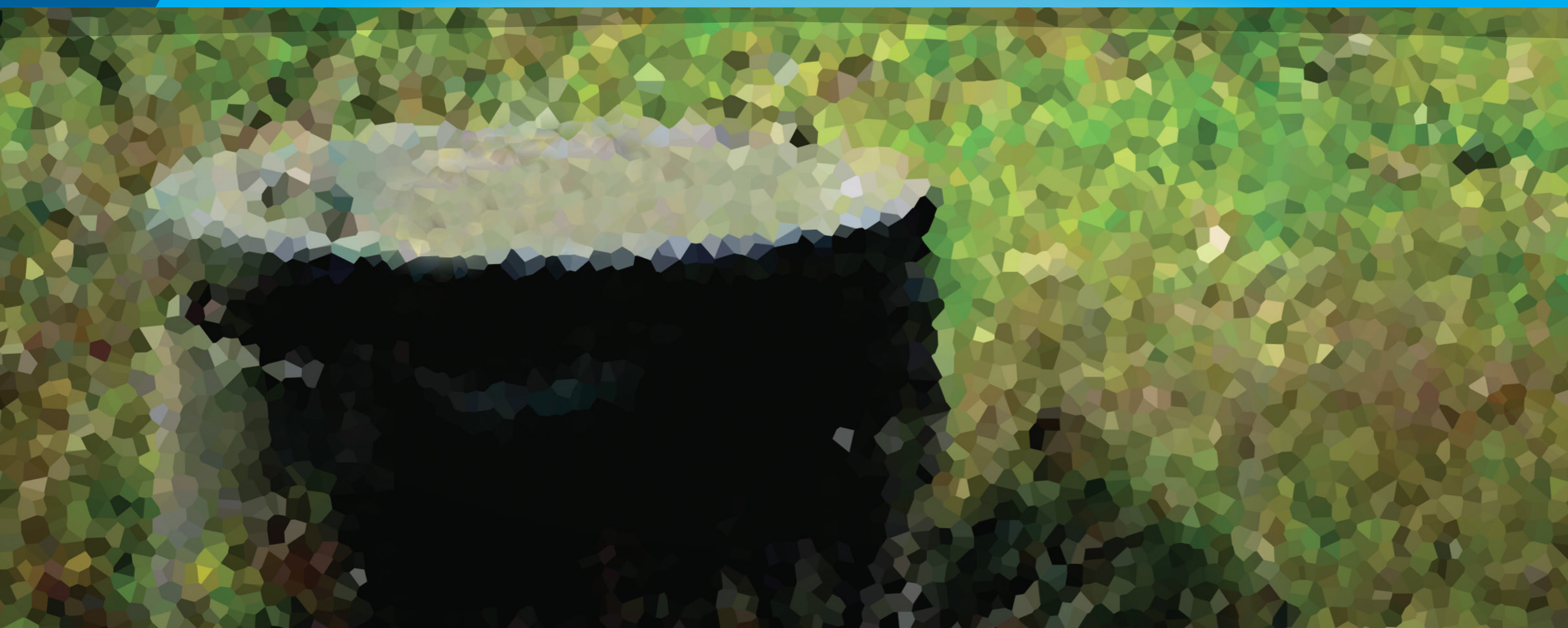
PROBABILISTIC DESIGN OF RELIEF WELLS AS PIPING MITIGATION MEASURE

CARLOS A. MIRANDA



Probabilistic Design of Relief Wells as Piping Mitigation Measure

by

C.A. Miranda Eguez

in partial fulfilment of the requirements for the degree of

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PREFACE

During my college years back in Ecuador, the mythic behavior of the water caught my attention. The mere presence of water is essential for most known forms of life. On the other hand, water can also represent a threat (flooding, tsunamis, etc). At my bachelor university, during hydraulic related lectures, the exposed examples of structures and hydraulic technology came from a distant country: The Netherlands. Since then my enthusiasm began to come to The Netherlands to study.

Some people say: "there exist no coincidences"; I love to think that way. It would take more pages than this entire thesis to tell the whole process that took me to get here, at this time at this place. Nevertheless, I would like to take this opportunity to thanks all the marvelous people I had met during this process.
Especial gratitude goes for my thesis committee. Timo Schweckendiek, my main supervisor, was my contact person and who extended me the opportunity to develop my project at Deltares. His advices and quotation of famous phrases (written on my delivered reports) played a substantial role during the development of my thesis. Besides this, I want to thank Timo for all the lessons I learned due the professional attitude he shared with me. These have been one of the most valuables things I have learned during this period, and unlikely to be learned at any educational institution. To Professor Matthijs Kok, chairman of my committee, thanks for his wise suggestions, pertinent questions and willingness to help; Patrick Arnold for his very precise comments and suggestions throughout my work; Maximiliam Huber, with whom I had the fortune to share the office, thanks for his professional advices and personal thoughts, which got me thinking more than one afternoon (lessons for life).

Thanks to the staff from Deltares, where I really enjoyed my time. Especial thanks goes to Ana Teixeira for all her help, support and high skilled tips.

My greatest appreciation to all my friends for their support during these years. Especial attention to Francoise Vogelaar who became our Dutch family.

Thanks to Pascal de Smidt, student counselor from TU Delft, for all his support during the toughest moments of my stay in the Netherlands.

I would like to thanks my parents for all their support along my life; without them nothing of this could have been possible.

To Andrea my deepest gratitude and acknowledgment. For all her support, help, love and for being my great partner. Thanks for all the moments and memories we are building in our life.

Finally I would like to thank to the scholarship program "Universidades DE Excelencia" awarded from the Ecuadorian government, for its financial support along my career.

C.A. Miranda Eguez
Delft, September 2014

SUMMARY

The traditional Dutch way to deal with piping for river levees is the implementation of piping berms. The disadvantage of such a measure is the inland space required, especially in urban areas. Relief wells, on the other hand, require less or no inland space and therefore represent an attractive solution as mitigation measure against piping. The aims of this research are first, to show how reliability analysis of relief wells systems can be carried out, and second to examine the costs required to achieve a reliability target for piping failure, as set in the Netherlands. The outcomes of this analysis will help comparing relief wells with piping berms in economic terms. To obtain these results, the statistical parameters of the influencing variables are studied using both, the collected data from existing projects in the Netherlands, and data from relevant literature.

A reliability-based design approach is followed to estimate the reliability of relief wells systems. In order to establish the limit state functions, the assessment methods recommended by the Dutch flood defence regulations are used. Applying the probabilistic axioms it is possible to resemble piping failure as a parallel system assessing uplift and heave failure mechanisms. To estimate the hydraulic head in relief wells system, the United States Corps of Engineers method is applied, as well as the latest developments in flood risk analysis, achieved by the Flood Risk Assessment (VNK) project, which are used to determine the reliability target. To estimate the probability of failure and the system reliability, Monte Carlo Simulation (MCS) and First Order Reliability Analysis (FORM) methods are utilized. A tailor-made comprehensive tool is built in Matlab to compute the hydraulic head in relief wells system and to perform the probabilistic analysis. Subsequently, a Life Cycle Cost (LCC) analysis is performed with the aim to account for the life cycle of relief wells. A comparison of the net present value of the two alternatives (relief wells and piping berms) is made. Finally, analysis of two case studies with different scenarios are performed to show the possible economic advantages of installing relief wells and sensitivity analysis is used to underpin the robustness of the conclusions.

The results show that, using USACE method, the blanket and the aquifer permeability, as well as the hydraulic losses, are the dominant variables (from the 'load' side). The sensitivity factors show high discrepancy between partially and fully penetrated wells. Even when the entrance losses cannot be accurately predicted, a total clogging scenario of the filter can be neglected. One main limitation for the applicability of relief wells system, is that there is a maximum achievable head reduction. This maximum head reduction is limited to the minimum possible well spacing (a>D/4). Results from the case studies show that relief wells are a cost-effective as piping mitigation measure, outperforming piping berms. This advantage can be upto a factor of ten regarding initial investment. This allows accounting for a shorter life cycle for relief wells in order to equate the same LCC as piping berms.

List of symbols

Roman Capital Letters

A Total area of the section

C Hanzen and Williams coefficient (will depend on the materials used)

C_{creep} Creep factor (depends on the characteristic of the material)

$C_{w,creep}$ "Weighted" (Lane) creep factor

D Thickness of the sand layer

D_h Kozeny effective diameter

G_p Flow correction factor (for partially penetrating well[USACE,1992])

H Head at the source (e.g. river)

H_{av} Average net head (in plane of wells)

H_m Net head midway (between wells)

H_e Entrance losses

H_w Well losses

L Minimum seepage length

$L_{dr,s}$ Piping or uplift (sensitive part of the dike ring under consideration)

L_e Length of sections (contribute significantly to the failure probability)

L_{berm} Berm length

L_h Total length horizontal part (seepage line)

L_v Total length vertical (the seepage line)

N Total life cycle (years)

P Principal amount

$P_{adm,loc}$ Local admissible failure probability

$P_{adm,ring}$ Admissible failure probability for the ring

P_f Probability of failure

Q Discharge(amount of water flowing through porous media, per unit of time)

Q_w Well discharge

R_z Represents the strength

S Distance from well to line source

SF	Safety factor, for deterministic calculations
S_z	The loads (solicitation)
T_f	Filter thickness
V_p	Present value
V_{p_t}	Total present value for given N
W	Depth of penetration of the well
W_f	Future worth or amount (*i.e. principal amount plus interest earned*)
X_3	Distance from landside dike toe to effective seepage exit

ROMAN SMALL LETTERS

a	Distance between wells
d	Thickness of blanket (impervious top layer)
d_{70}	70 % value of the grain distribution
d_{70m}	Reference value for d_{70}(m)
d_b	Sand diameter of the aquifer
d_{berm}	Berm thickness
d_f	Filter diameter
d_h	Diameter of filter material
g	Gravitational acceleration
h_{av}	Corr. average net head (in plane of wells)
h_m	Corr. net head midway (between wells)
h_a	Maximum allowable head
h_p	Head at point p between the well and the source
h_{po}	Head at the polder
h_w	Head at well
i	Net discount rate
i_c	Critical hydraulic gradient (for heave)
k_b	Blanket permeability
k_f	Aquifer permeability
l_{eq}	Correlation length limit state function (for piping)
n	Number of simulations
n_f	Number of simulations for which $Z < 0$
p	Porosity
p_0	1.5 for transitional flow (gravel) [Schiereck,2005]
r_w	Well radius
t_w	Thickness of well's pipe (depends on the material used as pipe riser)
v	Velocity (flow velocity, assuming there is no filter)

GREEK CAPITAL LETTERS

$\Delta\mathbf{H}$ Hydraulic head over the flood defence

ΔH_c Maximum permissible head

Δh_f Head loss (m)

ΔM Net seepage gradient toward the well

Φ^{-1} Inverse of the standard normal cumulative distribution function

Φ_p $max\{H_{av}, H_m\}$

$\Theta_{max(\frac{W}{D};\frac{D}{a})}$ Maximum value of θ_a for $\frac{a}{rw} = 1000$

$\Theta_{min(\frac{W}{D};\frac{D}{a})}$ Minimum value of θ_a for $\frac{a}{rw} = 20$

GREEK SMALL LETTERS

α FORM sensitivity factor

α_c Shape coefficient

αf Calibration factor

β Reliability index

δ_0 Correlation distance

η Dynamic viscosity

η_1 Drag force factor (coefficient of White)

γ_{cover} Specific weight of the cover layer

γ_f Specific weight of the fluid

γ_s Wet specific volume weight (of the covering ground layer)

γ_{sand} Volumetric weight of pervious foundation

γ_w Specific weight (of the water)

γ_p Apparent specific volume weight (of sand grains under water)

γ_k Specific weight of the grain

κ Intrinsic permeability

μ_z Mean value of Z

ν Kinematic viscosity of the fluid

ϕ_z Occurring potential

$\phi_{z,g}$ Head limit or potential limit

ρ_x Correlation function

σ_z Standard deviation of Z

σ_{eff} Effective stress of the soil

θ Rolling resistance angle of sand grains

θ_a Well factor

θ_m Well factor

ABREVIATIONS

FORM First Order Reliability Method

LSF Limit State Function

LCC Life Cycle Cost

MCS Monte Carlo Simulation

RBD Reliability-based Design

TAW Technical Advisory Committee on Water Defenses

USACE US Army Corp of Engineers

VNK Flood Risk Assessment(Veiligheid Nederland in Kaart)

Contents

1

Introduction

1.1. Background and Rationale

The Netherlands is historically known for its continuous battle against flooding. Nowadays, The Netherlands counts with 3600 kilometers of dikes and dunes, which fulfil the function of primary flood defense, and 15000 kilometers of defenses that provide indirect protection. With two-thirds of the country lying below sea level or at less than one meter above sea level, where around 60% of the Dutch population is located, it is imperative to have a correct control and monitoring of these structures. In order to carry out such assessment the Technical Advisory Committee on Water Defenses (TAW)(for its acronym in Dutch [1] has edited the "Guide on Safety Monitoring of Water Defenses" (Leidaad Toetsen op Veilighed). Uncertainties and gaps in the guidelines regarding piping (as far as this report concerns) are leading to qualify some dikes as "unsatisfactory"; therefore an upgrading of these structures is required, and the economic cost consequence must be assumed. Until now, the most commonly applied solution for improving piping safety in The Netherlands is to use piping berms, which has proved to be an expensive and massive solution (reducing available space for further development inland). Current efforts focus on the development of more accurate models to assess piping in order to be able to reduce the seepage length with an acceptable reliability. At present, the VNK2 project is being developed in the Netherlands. The VNK 2 project (Safety in the Netherlands, Veiligheid Nederland in Kaart), is an ongoing (expected to be finished on 2017), large scale project, which examines the probability of flooding and its consequences in the Netherlands. The aim of this project is to be able to spot areas with high probability of failure, or that does not meet the actual safety requirements, in order to prioritize interventions, propose alternatives in order to reduce risk, and to highlight the most important uncertainties with the intention of focus research in this direction. In addition to this according to the last "Assessment of Primary Flood Defenses in The Netherlands" [Inspectie Verkeer en Watersaat,2006], 680 kilometers of dikes still do not meet the safety standards, which implies that measures to improve the dikes' safety must be addressed. Considering the currently applied solution (piping berm), and the lack of space available for such a measure, this would lead to an important economical investment especially in urban areas. Hence, there is a need to investigate and assess the viability and cost-effectiveness of alternative measures.

Assuming the actions that have been used effectively in other countries, one of the most widely applied methods are the so-called relief wells, which have been successfully applied by United States of America. The USACE's acquired experience with regards to designing relief wells has been used as main source of reference, e.g. [USACE,1992].

1.2. Description of Dutch Flood Defence System

The Netherlands has always been aware of its proneness to flooding; Dutch water management is believed to have started around 800 AD. The first dikes were established as local protection for the societal developed areas. In time, the local dikes turned out to be insufficient and the need to enclose larger areas arose, which was the beginning of what is known as the dike rings (see Figure 1.1). From this point, the administrative basis of the Dutch flood defence policy developed until today. Currently, the flood defences of The Netherlands are classified in: (i) primary flood defence (3600 kilometres), and (ii) secondary flood defence (15000 kilome-

[1] Technisch Advies Westerhof

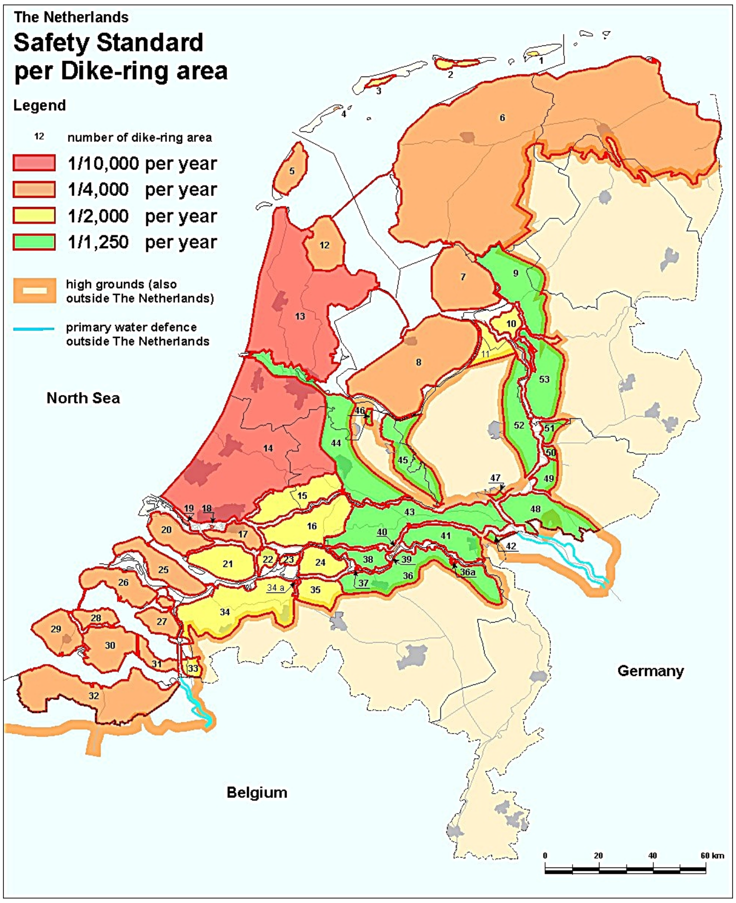


Figure 1.1: Flood defences dike rings. Source: [Recovers,2014].

tres). The primary flood defences are divided into three categories [Inspectie Verkeer en Watersaat,2006]:

1. Category a (a defences) include dikes, dunes, and hydraulic structures which provide direct protection against the sea, the major rivers, the IJsselmeer and the Markermeer lakes.

2. Category b (b defences) connects flood defences in either category a or c.

3. Category c (c defences) are defences structures which provide indirect protection against water flood.

1.3. PROBLEM DESCRIPTION

In the Netherlands over the past 20 years, sand boils (see Figure 1.2) have been spotted inland, behind dikes, after high waters levels, which is an evidence for potential risk of piping failure in dikes. Since one of the causes that can lead to dike breaching is piping, these events have drawn the attention of the authorities and the engineering community to re-evaluate the dike safet., In addition to piping berms to be an expensive solution, the current regulations do not state how to design an infrastructure against piping using drainage systems (e.g. relief wells). In the latest years, efforts have led to a better understanding of the methods for assessing piping (e.g. the new formulation of Sellmejier's rule), and the effect on the reliability of flood defences and flood risk [Jongejan et al.,2011]. Having identified the problem, it is important to mention that the purpose of this thesis is not to find the ideal solution for piping, but to develop a probabilistic approach for designing relief wells as piping mitigation measure, as well as to contribute in the research of alternative solutions for piping mitigation.

Figure 1.2: Sand boils. Source:[Rijkswaterstaat,2014].

1.4. RESEARCH QUESTIONS

The main subject of this report is the study of relief wells systems as piping mitigation measure using a probabilistic approach, based on the design method described in USACE [USACE,1992]. This thesis aims to answer the following research question:
How can relief wells be designed cost-effectively with the new Dutch safety standards, using a target probability of failure?
In order to answer this main question the following key questions have to be addressed:

- How can be piping reliability analysed for relief wells?
- Which are the dominant uncertainties involved?
- How can relief wells be designed to minimize the total life-cycle cost?
- How do relief wells compare to piping berms in terms of performance and total life-cycle cost?

1.5. OUTLINE AND METHODOLOGY

In order to answer the research questions, the Reliability-based Design (RBD) methodology is followed and the results obtained are compared, in economics terms, with the current applied alternative (piping berms), where both alternatives should accomplish the same set reliability target. The base of a RBD process is that the system should achieve an acceptable reliability target [Phoon,2008]. Since probability theory is applied, it is possible to account for uncertainties of the input variables into the analysis. This allows determining the probability of failure of the system, which will lead towards a more "rational" design. System, as addressed in this report, refers to a set of processes (failures modes) that could generate the infrastructure (dike) to failure. To measure the reliability of a system, the Limit State Function (LSF) has to be defined (refer to 4.3). Failure due to piping is treated as a parallel system, accounting for uplift and heave failures modes. The relief wells are asses using the USACE method. The methods used to perform the probabilistic analysis are MCS, and FORM. As the analysis of a well system and the probabilistic analysis are computational expensive and highly time-consuming, a computational tool is developed using the computing language `Matlab` [Matlab,1998]. In order to evaluate the costs, a unit price analysis is performed and compared later, by means of a life cycle cost analysis. Finally, a case study is performed to illustrated how to use the proposed method, and to compare the possible solutions for each alternative.

Figure 1.3 provides an overview of this thesis. Part I introduces the reference problem and the basic concepts used in this report. Part II addresses the models used and main assumptions for carrying out the probabilistic design and posterior cost analysis. In chapter 3 a description of the functionality of the relief wells, their design, the variables, and the cost analysis are presented. The same aspects are then described for piping berms. In chapter 4 the basics of the probabilistic analisys are specified. In part III the reliability-based design is performed and explained for a case study. Results from other case studies with data acquired from Deltares database are showed. In chapter 6 the main findings of this thesis are presented in the conclusions.

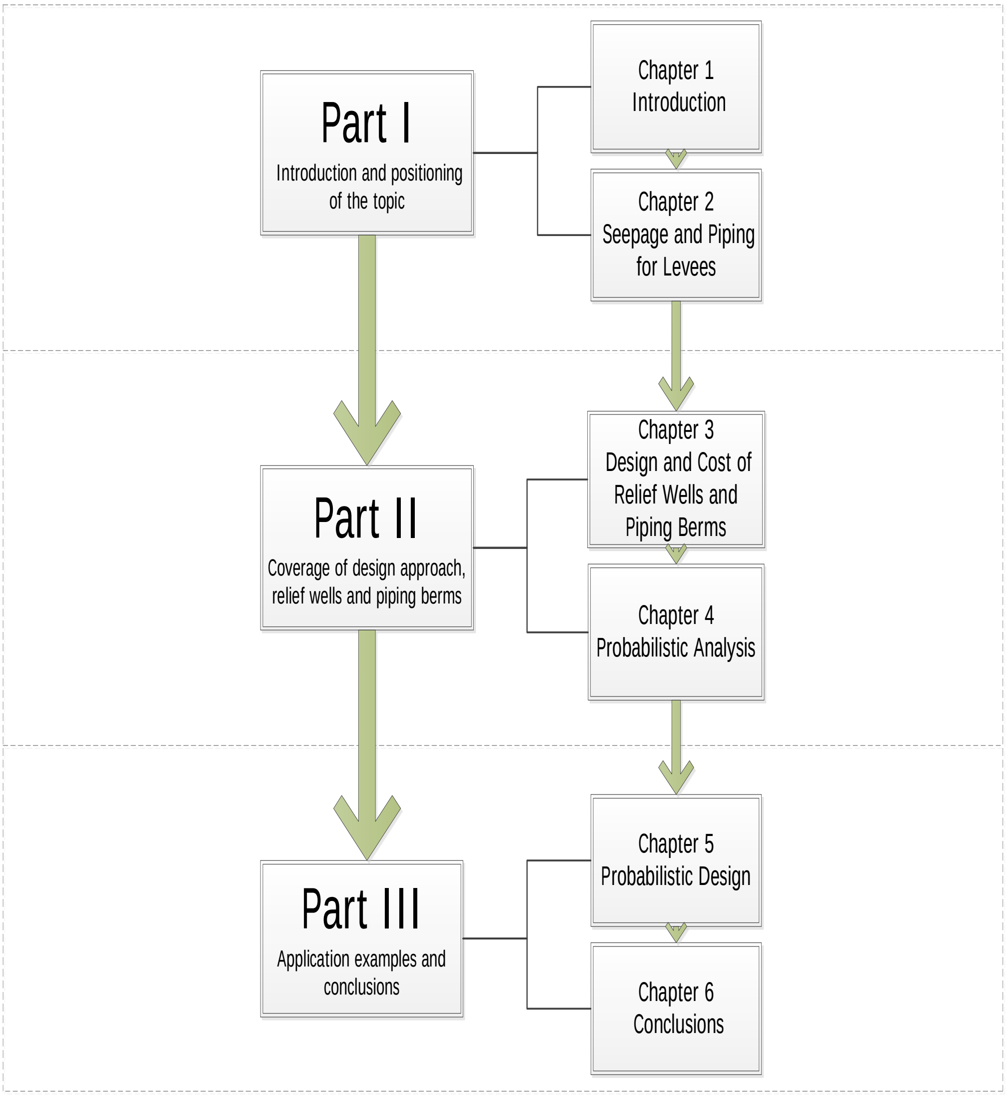


Figure 1.3: Structure of this thesis.

2

Seepage and Piping for Levees

The intention of this chapter is to introduce theory and definitions, as required for a better understanding of this report.

2.1. Seepage

Seepage being a precondition to develop piping, it is important to understand its behaviour and the rules that can describe or predict its occurrence. In a simple description, seepage corresponds to water movement in porous media [Kovács,1981]. The character of the flow is determined, basically, by the structure of the water conveying network, composed of the interconnected interstices of the layer. Apart from the structure, the instantaneous conditions of the network are also important (e.g. saturation, pressure condition). It is essential to study the driven forces for flow. The main driven forces, in the case of ground water flow, are: (i) gravity; (ii) pressure from uppers layers; (iii) vapour gases (only relevant at important depths). Few examples of seepage can be seen on Figure 2.1. In the case studied on this report, flow occurs due to difference in pressure

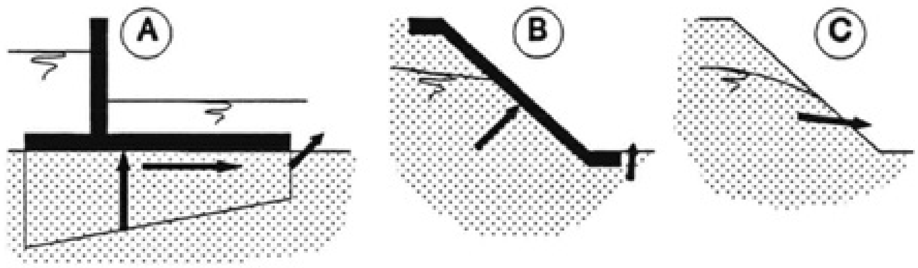


Figure 2.1: Examples of seepage. Source: [Schiereck,2005].

(defined as head difference or piezometric head) along an inpermeable structure, i.e. dikes. Refer to Figure 2.2.

Dike

Dike, or levee, is a natural or artificial wall, with a main purpose of flood protection, and usually they are placed parallel to a river. In the Netherlands, its flood prone areas are encircle by dike rings, as schematized on Figure 2.3.

Groundwater

It is the water stored, or the one that moves under the water table. It has positive pressure and can also be subdivided in:

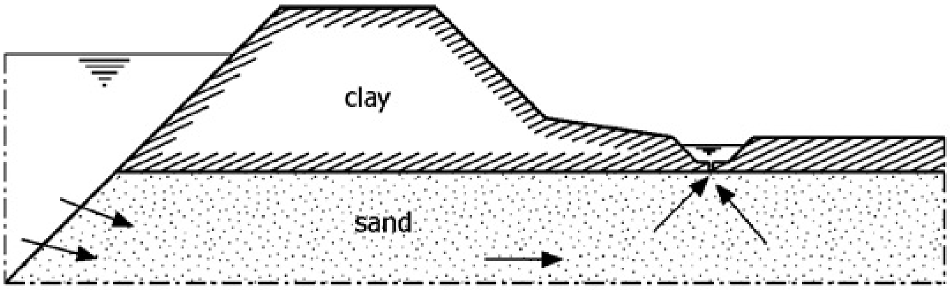


Figure 2.2: Seepageflow under dike. Source: [TAW,1998].

- Shallow groundwater: Located above the first impervious formation, near the surface, directly influence by meteorically and hydrological events.
- Deep ground water: Located below continuous impervious bed, which hinder direct contact between surface and ground water.

There is not direct recharge from precipitation or surface waters, drained only throughout shallow ground water. Kovács [Kovács,1981] define four terms in order to describe the strata where the flow takes place:

- Aquifers: Permeable geological formations which permit an appreciable quantity of water to move through them.
- Aquicludes: Impermeable strata which may content a great quantity of water.
- Aquifuges: Impermeable formations without water.
- Aquitard: Transition between aquifers and aquicludes.

UNCONFINED AQUIFER

When the upper boundary is the water table where the pressure equals to the atmospheric pressure.

CONFINED AQUIFER

Refers to an aquifer that is cover by impervious layer and the pressure at the upper layer (blanket) is higher than atmospheric pressure. This is the general case for the Netherlands, and it can be schematized as seen on Figure 2.4.

Figure 2.3: Dike ring schematization. Source: [Rijkswaterstaat,2012].

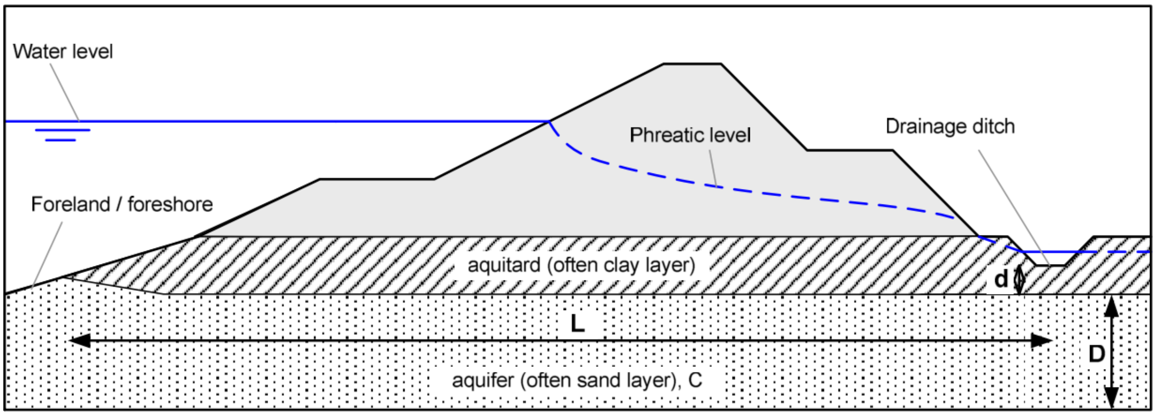


Figure 2.4: Schema for heave and piping. Source: [Schweckendiek,2014].

Hydraulic Conductivity

Hydraulic conductivity is a soil property which determines how fast a fluid moves through porous media; it is also known as permeability and is measured in m/s. The hydraulic conductivity is the proportional factor between specific discharge, amount of seepage per unit area in m/s, and seepage gradient. Hydraulic conductivity is influenced by: (i) size and shape of the grains, (ii) porosity, and (iii) irregular network of pores and channels between grains. Due to the difficulty to estimate values for the hydraulic conductivity, even with laboratory tests, sometimes representative values are used; some of these values are shown on Table 2.1.

Table 2.1: Values for permeability.

Material	$d_{50} < 63X10^{-3}m$ **or** $d_{n50}m$	**Permeability, k (m/s)**
Clay	$< 2.10^{-6}$	10^{-10} - 10^{-8}
Silt	$2X10^{-6}$ - $63X10^{-6}$	10^{-8} - 10^{-6}
Sand	$63X10^{-6}$ - $2X10^{-3}$	10^{-6} - 10^{-3}
Gravel	$2X10^{-3}$ - $63X10^{-3}$	10^{-3} -10^{-1}
Small rock	$63X10^{-3}$ - 0.4	10^{-1} - $5X10^{-1}$
Large rock	0.4 - 1	$5X10^{-1}$ - 1

Seepage Gradient

Pressure gradient describes the rate of change of hydraulic pressure. On Figure 2.5 it can be seen the sketch of Darcy's experiment [Kovács,1981]. Darcy found that seepage velocity is linearly proportional to the hydraulic gradient, and the hydraulic conductivity. This is the basic law of seepage hydraulics and it can be formulated in the next explained equation (Eq.2.1); this equation is valid in laminar flow. A distinction has to be made between laminar and turbulent flow. Laminar flow occurs when the motion of the water particles occurs ordely and in paralell lines, which is the opposite of turbulent flow that is when the flow is chaotic and the proportional linearity between seepage velocity and seepage gradient does not hold anymore.

$$\frac{Q}{A} = k_f * \left(\frac{\Delta H}{L}\right) \tag{2.1}$$

Where

Q Discharge(amount of water flowing through porous media, per unit of time)

A Total area of the cross section

In general terms, the seepage equation can be written as proposed on "Introduction to bed, bank and short protection" [Schiereck,2005]

$$\frac{Q}{A} = k_f * \left(\frac{\Delta H}{L}\right)^{\frac{1}{p_0}} \tag{2.2}$$

Where $p_0 = 1$ for laminar flow, and $p_0 = 2$ for turbulent flow ([Schiereck,2005]).

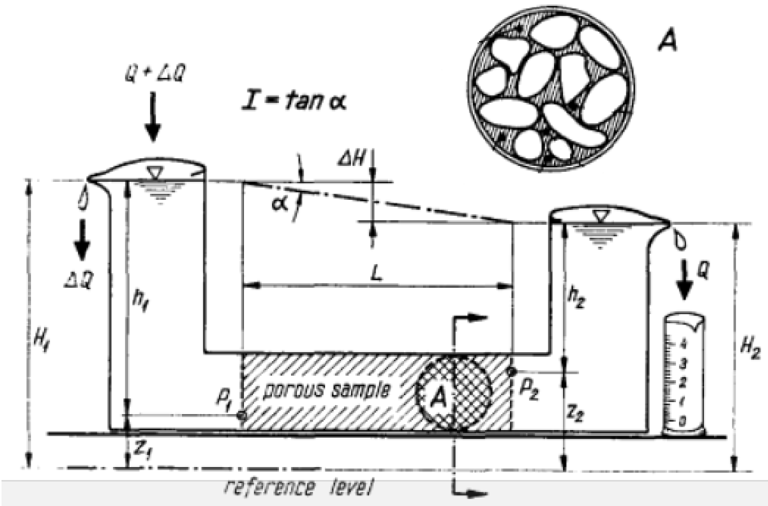


Figure 2.5: Skecth of Darcys' experiments. Source: [Kovács,1981].

2.2. PIPING

Piping is regressive of erosion from downstream to upstream, throughout pipes formed due to migration of solid particles outside of the aquifer downstream. In order to develop piping, the following phases may occur in the following order (see Figure 2.6):

1. **Uplift of impervious layer:** Water pressure underneath the impervious layer exceeds the weight of the impervious layer causing its lifting.
2. **Cracking of impervious layer:** If the water pressure is high enough, it can lead to crack the impervious layer; if this occurs, seepage starts flowing outside the aquifer through the blanket.
3. **Erosion of the sand layer:** If the velocity of the flow in the sand is high enough, this could lead to transport sand material (internal erosion).
4. **Creation of pipes:** If the flow velocity is big enough to transport sand material outside the aquifer, pipes will be formed. This material is the material that can be observed on the so called sand boils (Figure 1.2).
5. **Continuos pipe:** Sometimes the extra height due to sand boils could lead to a decrease in the flow velocity and erosion will stop. Nevertheless, if the extra height is not enough, the pipes will continue growing backwards, creating an open connection between the exit point and the outside water. The structure becomes piping sensitive, continuously increasing the size of the pipes by transporting coarse material through them.
6. **Collapse of the structure:** As result of pipes' growth, the subsidence or cracking of the structure occurs. As mentioned on the previous chapter, for the scope of this report only uplift, and heave criterion will be analysed.

FAILURE MECHANISMS

According to TAW [TAW,1998] there are three types of ground failure induce by pore-water pressure or pore water seepage, which shall be checked as relevant:

- Failure by uplift: When pore water pressure under a structure, or a low permeability grown layer, becomes larger than the mean over burden pressure (vertical stresses in the soil); it is the pressure, or stress, imposed on that layer of soil or rock by the weight of over laying material. This type of failure is characteristic on impervious cohesive layers.
- Failure by heave: Occurs when upwards seepage forces act against the weight of the soil, reducing the vertical effective stress to zero. Soil particles are then lifted away by the vertical water flow, and failure occurs (sand boiling). This type of failure is characteristic of permeable non cohesive layers.

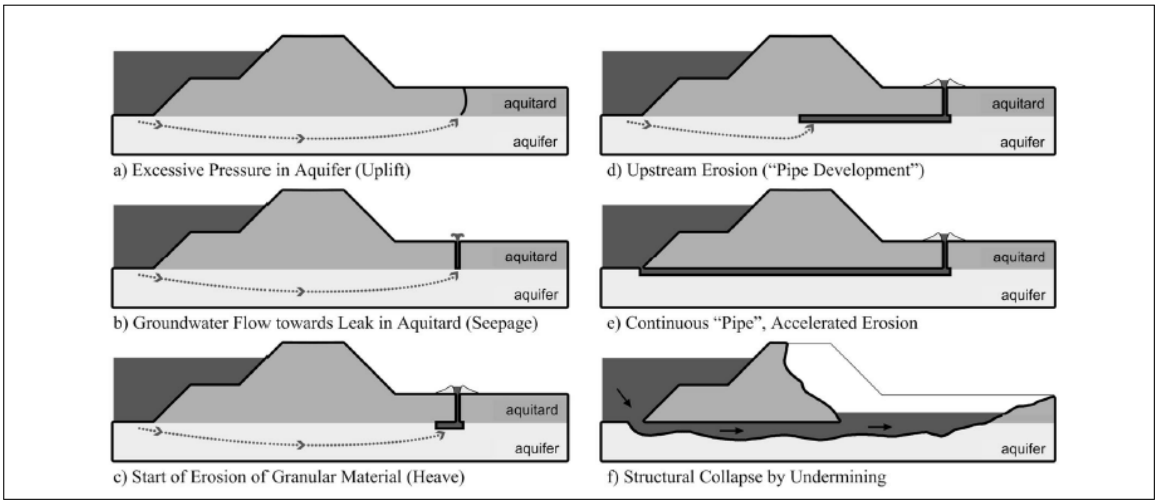


Figure 2.6: Failure mechanism for piping. Source: [Schweckendiek et al.,2013].

- Failure by piping: Special type of erosion; regressive erosion forming a pipe shaped discharge tunnel. Failure is assumed when the upstream, end of the eroded tunnel, reaches the bottom of the reservoir.

2.2.1. Uplift Evaluation Method

High water pressure in the sand layer under the previous strata (blanket) can cause uplifting of the aquitard (blanket), which can lead to the cracking of this layer. Cracking is a precondition for piping, assuming that the soil does not present previous holes or canals due to trees or any other disruption, in the impermeable cover layer. It occurs when the water pressure beneath the impervious layer exceed the weight of the cover layer. In order to verify if cracking takes place, the weight of the cover layer should be compared with the water pressure beneath it. The water pressure, which equates the weight of the covering layer, is called potential limit. The critical potential can be defined as follows:

$$\phi_{z,g} = h_{po} + d * \frac{\gamma_s + \gamma_w}{\gamma_w} \tag{2.3}$$

Where

$\phi_{z,g}$ Head limit or potential limit

h_{po} Head at the polder

d Thickness of blanket (impervious top layer)

γ_s Wet specific volume weight of the covering ground layer

γ_w Specific weight of the water

The check criterion for cracking can be drawn:

$$\phi_z \preceq \phi_{z,g} \tag{2.4}$$

Where

ϕ_z Occuring potential

2.2.2. Heave Evaluation Method

When the water pressure exceeds the effective stress of the soil, particles start to float and they are lifted away by vertical seepage. This mechanism is referred, as mentioned on section 2.2, as erosion of the sand layer; sand transport takes place in vertical direction. This transport can only occur if the vertical gradient at the exit point exceeds the critical value for heave. There are several proposed methods to obtain the critical gradient for a given soil type; nevertheless, in the scope of this report, experimental findings will be addressed. Figure

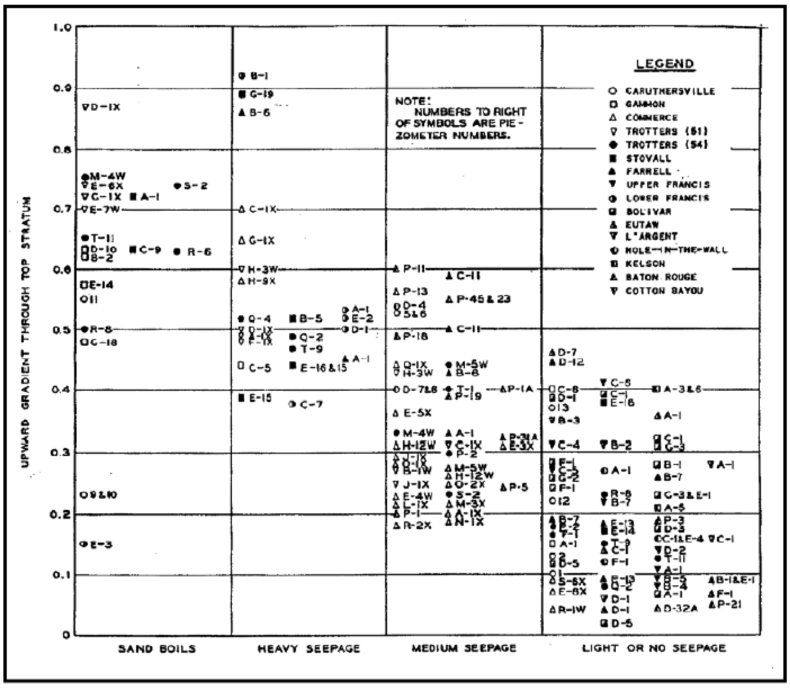


Figure 2.7: Critical gradient for seepage and internal erosion. Source: [USACE,1956].

2.7 shows the observed gradient at which sand boils were spotted. In the case of a probabilistic analysis, an educated guess is to assumed a lognormal; distribution with a mean value of 0.7 and a standard deviation of 0.1 [Schweckendiek et al.,2013]. The check criterion for heave can be written as: $i < i_c$, where i is the occurring hydraulic gradient, and i_c is the critical heave gradient.

2.2.3. PIPING EVALUATION METHOD

BLIGH

Based on his research on several dams and earth foundations in India, Bligh [Bligh,1910] developed an empirical (simple) formulation to assess piping; finally he proposed:

$$\Delta H \preceq \Delta H_c \tag{2.5}$$

$$\Delta H_c = \frac{L}{C_{creep}} \tag{2.6}$$

Where

ΔH Hydraulic head over the flood defence

ΔH_c Maximum permissible head

L Minimum seepage length

C_{creep} Creep factor, depends on the characteristic of the material

Seepage length refers to the percolation path, meaning the length that a particle of water has to travel from its entry point to the potential exit point.

LANE

Lane [Lane,1935] continued with the approach anticipated by Bligh by investigating (hundreds) dams in the United States, he suggested that vertical parts of seepage line are more efficient to reduce seepage than horizontal pats.

Grondsoort	Mediane korrel-diameter [µ m] 1)	C_{creep} (Bligh)	$C_{w,creep}$ (Lane)
Uiterst fijn zand, silt	< 105		8.5
Zeer fijn zand	105 – 150	18	
Zeer fijn zand (mica)		18	7
Matig fijn zand (kwarts)	150 – 210	15	7
Matig grof zand	210 – 300		6
Zeer/uiterst grof zand	300 – 2000	12	5
Fijn grind	2000 – 5600	9	4
Matig grof grind	5600 – 16000		3.5
Zeer grof grind	> 16000	4	3

Figure 2.8: Seepage line factors for Bligh and Lane. Source: [TAW,1999].

In order to make his suggestion accountable he proposed:

$$\Delta H \preceq \Delta H_c \tag{2.7}$$

$$\Delta H_c = \frac{\left(\frac{1}{3}L_h + L_v\right)}{C_{w,creep}} \tag{2.8}$$

Where

L_h Total length of the horizontal part of the seepage line

L_v Total length of the vertical part of the seepage line

$C_{w,creep}$ "Weighted" (Lane) creep factor

TAW [TAW,1999] shows the creep factor according to Bligh and Lane (see Figure 2.8). It is important to notice that safety is implied in these factors, so no safety factor should be applied in design or safety assessment.

SELLMEIJER'S CALCULATION MODEL

Sellmeijer formulated a mathematical calculation for modelling piping. His formulations were validated by a large-scale Delft hydraulics model in the Delta flume. In recent years, physical scale test [Beek et al.,2011] has led to a revision of the Sellmeijer formula (see also [Sellmeijer et al,2011]. Finally the new Sellmeijer formula to compute the critical head reads as follows:

$$\Delta H_c = F_1 F_2 F_3 L \tag{2.9}$$

$$F_1 = \eta_1 \left(\frac{\gamma_p}{\gamma_{water}} - 1\right) tan\theta$$

$$F_2 = \frac{d_{70m}}{\sqrt[3]{\frac{\nu\kappa L}{g}}} \left(\frac{d_{70}}{d_{70m}}\right)^{0.4}$$

$$F_3 = 0.91(D/L)^{\frac{0.28}{(D/L)^{2.8}-1}+0.04}$$

Where

γ_p Apparent volume weight of sand grains under water

θ Rolling resistance angle of sand grains

η_1 Drag force factor (coefficient of White)

κ Intrinsic permeability

d_{70} 70 % value of the grain distribution

d_{70m} Reference value for d_{70} (m)

D Thickness of the sand layer

2.3. SUMMARY

In this chapter the main definitions as they will be addressed in this report have been given. The current evaluation methods for uplift, heave and piping as set by the Dutch regulations were introduced. From these evaluation methods only uplift and heave will be referred in this research. The evaluation methods for piping described in this chapter are not suitable for relief wells systems. Piping evaluation for relief wells systems will be addressed as it will be explained in chapter 4, accordingly to the phases described in section 2.2.

3

Design and Costs of Relief Wells and Piping Berms

In this chapter the definitions, assumptions and simplifications of the methods used to carry the deterministic calculations in this research, are presented. First an introduction of relief well and relief wells system is given. In addition the USACE method to design a relief wells system is explained; the involved variables and the limitations of the method are exposed. Special attention to the hydraulic losses on the well is addressed given the lack of information regarding this variable. Additionally a brief (given that is not the main subject of this research) explanation regarding piping berms is also addressed. For both alternatives, piping berms and relief wells systems, a cost analysis is performed and the results are shown in sections 3.5 and 3.7. Finally the life cycle cost considerations are also introduced.

3.1. Relief Wells

Relief wells are drainage systems in confined aquifers consisting of a pipe drilled in the soil through the impervious strata until the pervious strata, allowing the underwater to reach the free surface, relieving the pore water pressure. Screens and filters are needed in order to avoid loss of coarse fine material and prevent clogging, which can lead to a decrease of the wells' efficiency. Sometimes, in order to obtain a reduced groundwater level and to ensure that it remains to an allowable level, is needed to have a system of wells. In this case, it is requested to find the position of such wells, in order to acquire the design requirements. In Figure 3.1 an active relief well is illustrated. Active relief wells refers to wells that make use of pumps in order to extract the groundwater and consequently reduce the phreatic level. On the other hand, passive relief wells refers to wells with artesian flow. This research focuses into passive relief wells.

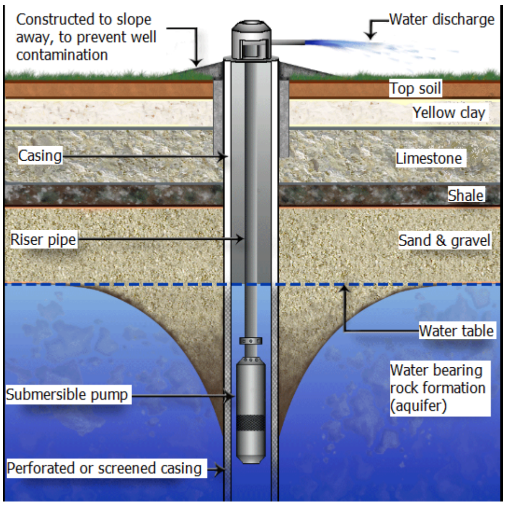


Figure 3.1: Typical relief well. Source: [Wordpress,2012].

3.1.1. SINGLE RELIEF WELL

Single relief well denotes to the case when only one relief well is addressed.Relief wells by definition, and as treated in this report, are artesian wells; hence, equations for artesian flow are applicable. In the case of confined aquifer, when a well is drilled the flow can be idealized, as shown on Figure 3.2. In order to have artesian conditions, the ground water flow pressure has to be bigger than atmospheric pressure. In case of

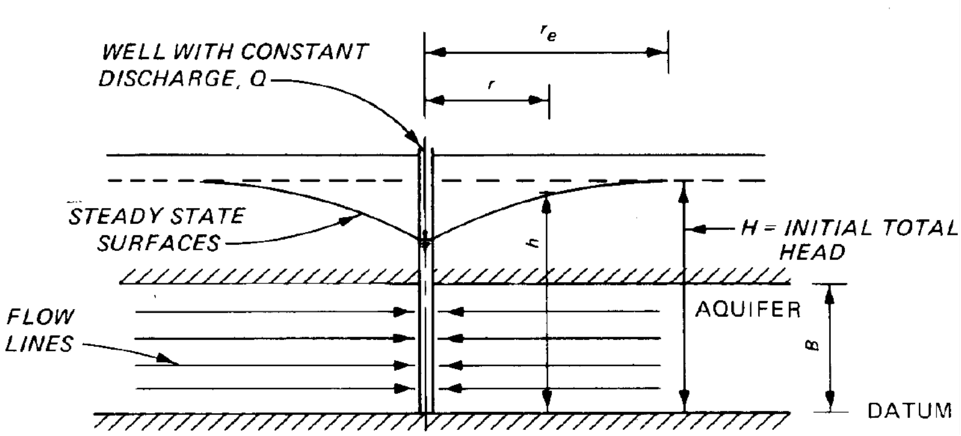


Figure 3.2: Hydraulic head in a confined aquifer. Source: [USACE,1986].

having a relief well in front of an infinite source i.e. river, the following formula [Muskat,1937] are valid:

$$\phi_z = \Delta H - \frac{Q_w}{2\pi * \kappa_D * D} ln\left(\frac{2S}{r_w}\right) \tag{3.1}$$

Where ϕ_z is the occuring potential at the well and S referes to the distance from the well to line source.On Figure 3.3 the description of the variables are shown; for more information also refer to Table 3.1.

In case of an infinite line source and infinite line sink (point where groundwater pressure equals atmospheric pressure)parallel to source, the following equations are suitable:

$$\phi_z = H\frac{S+x_3-x}{S+x_3} - \frac{Q_w}{4\pi\kappa_D} ln\left[\frac{\cosh\frac{\pi\gamma}{S+x_3} - \cos\frac{\pi(x+S)}{S+x_3}}{\cosh\frac{\pi\gamma}{S+x_3} - \cos\frac{\pi(x+S)}{S+x_3}}\right] \tag{3.2}$$

$$Q_w = \frac{4\pi\kappa_D\left(H - H\frac{S+x_3-x}{S+x_3}\right)}{ln\left[\frac{2(S+x_3)^2\left(1-\cos\frac{2\pi S}{S+x_3}\right)}{\pi^2 r_w^2}\right]} \tag{3.3}$$

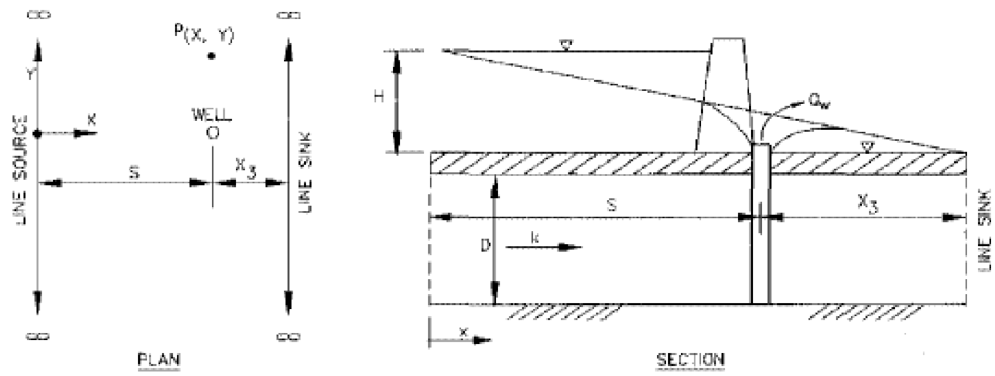


Figure 3.3: Schema for infinite line source and infinite line sink. Source: [USACE,1992].

The presented formula assumes that wells penetrate the complete previous layer; in case of partially penetrating wells, the discharge is reduced by a factor Gp:

$$G_p = \frac{W}{D}\left(1+7\left(\sqrt{\frac{r_w}{2W}}\cos\frac{\pi W}{2D}\right)\right) \tag{3.4}$$

Where

W Depth of penetration of the well

G_p Flow correction factor for partially penetrating well [USACE,1992]

3.1.2. MULTIPLE WELLS SYSTEMS

Sometimes in order to obtain a reduce ground water level and to ensure that this remains to an allowable level, it is needed to have a system of wells. In this case it is requested to find out the position of such wells in order to acquire the design requirements. Multiple wells system refers to a system of relief wells in various arrays. In Figure 3.4 a relief well system consisting of a single line of wells can be depicted. This type of system will be used for the scope of the present research.

Figure 3.4: Relief wells system. Source: [City of Chilliwack,2008].

3.1.3. DESIGN METHOD: USACE

Semi-empirical method is used to evaluate the potential[1] at the exit point in multiple well systems. This method was proposed by USACE [USACE,1992], based on the method of multiples images [Muskat,1937] as well as in the use of well factors, which are coefficients that describe the decrease in the piezometric head between partially penetrated wells. In the case of fully penetrated wells the flow into a well is only horizontal, whilst for partially penetrated wells exists also a vertical component for the seepage flowing into the well, which will generate the head in the well plane to become larger than the piezometric head between wells. This method consists of an iterative process where the designer first assumes a well spacing, then computes either the head between wells (for fully penetrated wells, and in some cases for partially penetrated wells) or the well plane's head (for partially penetrated wells), and repeat this process until the desire wells' head is found. The variables and definitions used on the USACE method are described in Figure 3.5, and Figure 3.6.

[1]Occurring potential as define on sand report[TAW,1999] page 29

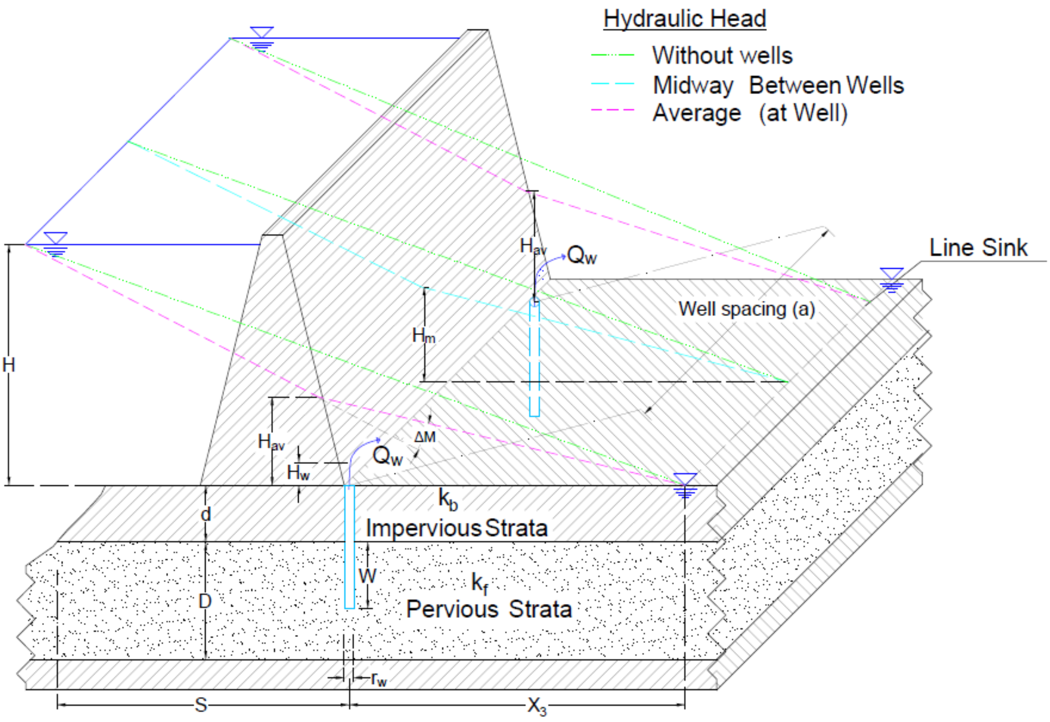


Figure 3.5: Schema of relief wells system.

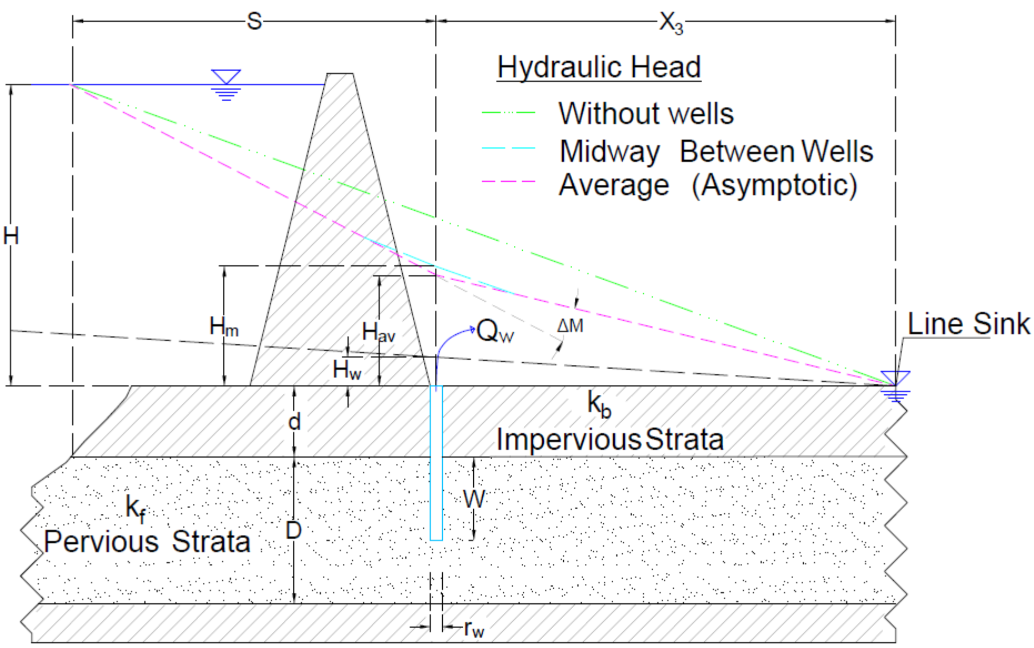


Figure 3.6: Nomenclature for relief wells system.

Where

a Distance between wells

H Head at the source (e.g. river)

h_m Corr. net head midway between wells

h_{av} Corr. average net head in plane of wells

Q_w Well discharge

S Distance from real well to line source

r_w Well radius

H_w Well losses

ΔM Net seepage gradient toward the well

H_{av} Average net head in plane of wells

H_m Net head midway between wells

In order to accomplish hydraulic head evaluation, a model for seepage analysis has to be defined when there is not piezometric data or seepage measurement. USACE developed the following method base on its experiments [USACE,1956]. This method assumes a linear hydraulic profile, in accordance to Darcy's experiments. For details regarding the method, the reader is referred to the bibliography in this report. Only the principal definitions, as they will be addressed in this thesis, are given. In Figure 3.7, L_o refers to the theoretical distance from river side of the dike to the entry point, point where water enters into the aquifer, L_1 refers to the effective entrance length; this reduction of the theoretical point is given since an impermeable blanket is assumed, which, actually, is not. L_2 refers to the base width of the dikes. In this report, S is used as distance from entry point to well, which is assumed to be located at the landside toe of the dike (most vulnerable point), and it can be definef as $S = L_1 + L_2$. For the scope of this thesis, this value is obtained from Deltares data based. L_3 is the distance from land side toe to exit point; in this thesis, the impermeable blanket is assumed to extend to infinite, for these cases USACE recommends to compute the effective exit point (assumed in this report as X_3), as follows:

$$X_3 = \sqrt{\frac{k_f dD}{k_b}} \tag{3.5}$$

Where:

k_f Aquifer permeability

k_b Blanket permeability

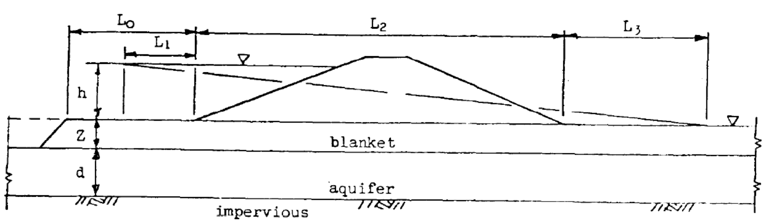


Figure 3.7: Piezometric head. Source: [USACE,1992].

The reason why this method was chosen is due to its relatively easy implementation when searching for a solution for relief wells systems; in addition, it provides an equation (Eq.3.8, Eq.3.10) which can be implemented when the LSF is derived. For details on the design process, the reader is suggested to see Appendix Process to Design a Relief Well System. For the scope of this report an infinite line of relief wells was studied. An infinite line of wells is a simplifying assumption in order to facilitate the design of the well system. This assumption contemplates the following idealized conditions:

- Wells are equally spaced, and have the same dimensions
- Pervious strata is uniform in depth and permeability, along the entire length of the system
- The effective source of flow and the landside exit, are parallel to the line of the wells

The applicable formulas for designing a system of infinite line of wells are:

$$h_{av} = \frac{h\theta_a}{\frac{S}{a} + \left(\frac{S+x_3}{x_3}\right)^{\theta_a}} \tag{3.6}$$

$$H_{av} = H_w + h_{av} \tag{3.7}$$

$$h_m = h_{av}\frac{\theta_m}{\theta_a} \tag{3.8}$$

$$H_m = H_w + h_m \tag{3.9}$$

$$\Delta M = \frac{H - H_{av}}{S} - \frac{H_{av}}{x_3} \tag{3.10}$$

$$Q_w = a * \Delta M * \kappa_f * D \tag{3.11}$$

$$h_m = a * \Delta M * \theta_m \tag{3.12}$$

$$h_{av} = a * \Delta M * \theta_a \tag{3.13}$$

Where θ_m and θ_a are the well factors (see Table 3.2) which are influenced by the distance between wells, the ratio of the wells, and well penetration. For values of $\frac{a}{r_w} = 100$, the following equations are given [USACE,1992]:

$$\theta_m = \frac{1}{2\pi} ln\left(\frac{a}{\pi * r_w}\right) \tag{3.14}$$

$$\theta_a = \frac{1}{2\pi} ln\left(\frac{a}{2\pi * r_w}\right) \tag{3.15}$$

For other values of $\frac{a}{r_w}$:

$$\theta_a = \theta_{a_{\left(\frac{a}{r_w}=100\right)}} + \Delta\theta * \left(\log\frac{a}{r_w} - 2\right) \tag{3.16}$$

$$\theta_b = \theta_{a_{\left(\frac{a}{r_w}=100\right)}} + \Delta\theta * \left(\log\frac{a}{r_w} - 2\right) \tag{3.17}$$

3.1.4. INVOLVED VARIABLES

The involved variables which play a role in the USACE method [USACE,1992] are specified on Table 3.1:

Table 3.1: Variables USACE method.

Symbol	Description	Units
γ_w	Specific weight of the water	kN/m^3
γ_{cover}	Specific weight of the cover layer	kN/m^3
γ_{sand}	Volumetric weight of pervious foundation	kN/m^3
d	Thickness impervious layer (Blanket);depth of impervious layer.	m
H	Net head difference between piezometric head at source (river) and the hydraulic head at polder or inland; in this report level (z=0) set at the groundwater level inland	m
S	Distance from effective seepage (effective source) entry to line of wells(riverside dike toe). It can be computed by any calculation model for ground water flow; in this report this value is given from previous studies (Deltares data base)	m
X_3	Distance from landside dike toe to effective seepage exit (hypothetical point where seepage flow reaches the same hydrostatic pressure at landside of the dike); can be computed by any calculation model for ground water flow.	m
k_f	Aquifer permeability (horizontal permeability of the pervious strata)	m/s
D	Thickness of the aquifer, where porous media seepage flow takes place (pervious foundation).	m
r_w	Well radius. USACE recommends min. 6 in (aprox. 15 cm) of internal diameter to prevent excessive hydraulic losses.	m
t_w	Thickness of well's pipe will depend on the material used as pipe riser.	m
g	Gravitational acceleration	m/s^2
C	Hanzen and Williams coefficient; It will depend on the materials used; generally speaking, its values range from 90 to 150.	[-]
W	Is the depth reached by the well into the aquifer; sometimes referred as percentage of the aquifer's thickness (W/D)	m
H_e	Hydraulic losses generated on the filter; values used in this report assumed that the well has already fulfil its life cycle to try to model the worst case scenario (bigger entrance losses); these are assumed to be constant.	m
H_w	Total hydraulic losses including entrance, frictional, and velocity losses in the riser pipe. Frictional and velocity losses are computed, and not treated as constant: they are dependent on the discharge.	m
SF	Safety factor, for deterministic calculations; as recommended by USACE, was set equal to 1.5 for uplift (reducing the allowable head).	[-]
h_a	Allowable head: max. pressure resistance for uplift from the covering (impermeable) layer.	m
θ_a, θ_m	Well factors	[-]

3.2. PORE PRESSURES WITH RELIEF WELLS

Since USACE method is derived from DUPUIT's equations [USACE,1992], the drawdown of the hydraulic head into relief wells is assumed to be logarithmic. Depending if the well is partially or fully penetrated, the maximum hydraulic head will be located either at the well, on the wells' plain (H_{av}), or at the midway distance between wells (H_m), respectively, as it will be addressed on the following sections. On Figure 3.8 the idealized pressure profiles are shown, as an illustration example. Here, the green line represents the hydraulic head without wells. The blue line is the idealized hydraulic head, once relief wells have been placed. The red line resembles the hydraulic head at the well.

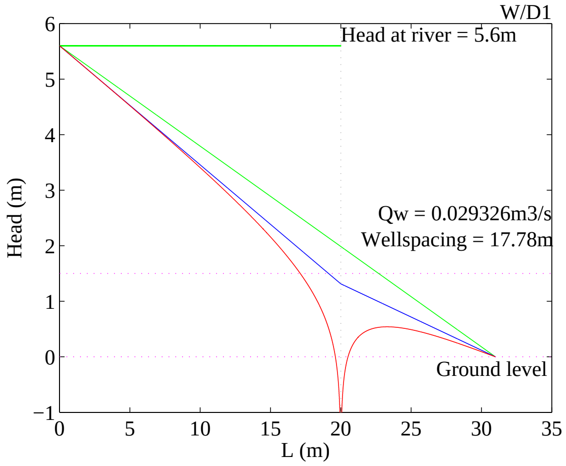
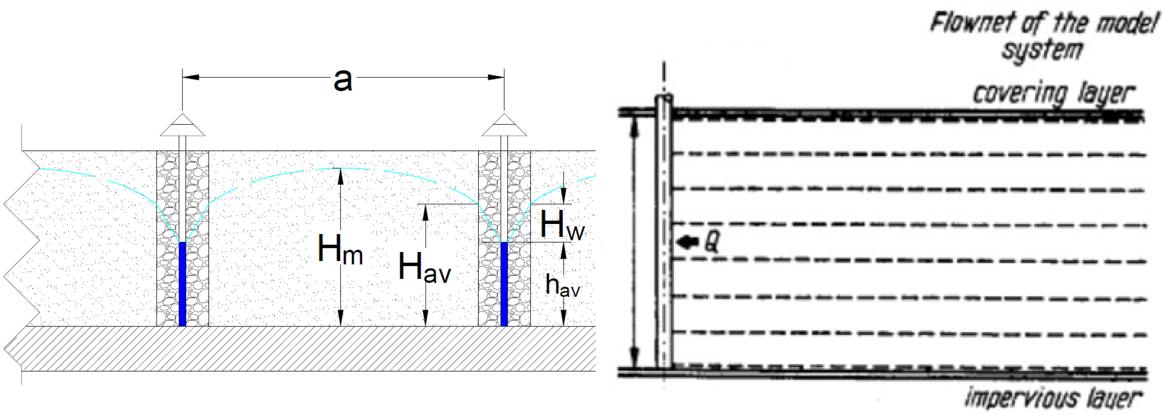


Figure 3.8: Hydraulic head profile for well penetration 100%.

3.2.1. FULLY PENETRATED WELLS

In the case of fully penetrated wells, the maximum hinterland head will always occur midway between wells (H_m refer to Figure 3.9) since the well catches the hole seepage going through the aquifer, making it more efficient on wells plane (larger catchment of discharge).

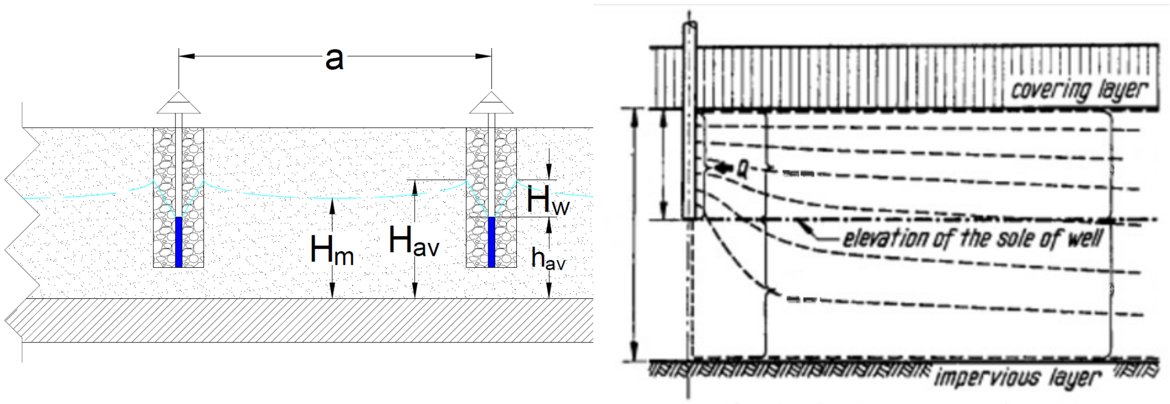


(a) (b)

Figure 3.9: Variation of hydraulic head for infinite line of fully penetrated of wells. Figure (a) schema of the hydraulic head for fully penetrated of wells. Figure (b) seepage flow into fully penetrated well [Kovács,1981].

3.2.2. PARTIALLY PENETRATED WELLS

In the case of partially penetrated wells, smaller flow discharge is moving into it, thus its efficiency is reduced. In addition, partial penetration induces a vertical flow, and increases the velocity on well's vicinity increasing therefore the head losses. This effect decreases while moving away from the well, leading the maximum head to be on wells plane (H_{av}). Refer to Figure 3.10.



(a) (b)

Figure 3.10: Variation of hydraulic head for infinite line of partially penetrated of wells. Figure (a) schema of the hydraulic head for partially penetrated wells. Figure (b) seepage flow into partially penetrated well [Kovács,1981].

3.3. RANGE OF VALIDITY FOR USACE METHOD

Given that to obtain the hydraulic head in a relief wells system is an iterative process, and that for the probabilistic analysis numerous computations are required (especially in MCS, and even in FORM), several cases for combinations of well spacing and well penetration have to be performed before finding a solution. Thus, a `Matlab` script (see Appendix Analysis Life Cycle Cost) was developed to take care of this computational task; nevertheless, when running the script, adding some random values to test it, some incoherent results were spotted, and even sometimes the script will stay running in an infinite loop (due to the need to converge the well factors). Hence, the purpose of this section is to find out the limitations of the model. The problems that arose while testing were; no convergence of well factors due to:

- There exist a range for well factors of $0.25 < D/a < 4$, and sometimes the solution (given that it is an iterative process) was searched outside its limits. For this reason, and in order to avoid breaks on the script, an analytical expression (Eq.3.21) was derived to predict the lower limit of well penetration for a given case.
- Incorrect values of well factors on data provided by USACE [USACE,1992].

More detailed information about these limitations are discussed on the following sections of this chapter.

LAMINAR FLOW

The semi-empirical method, proposed by USACE [USACE,1992], uses the equations for artesian flow and assumes that Darcy's law is applicable (i.e. seepage flow is laminar). Reynolds number is one important parameter to define the type of regime that dominates the flow; nevertheless, there has been discrepancy while determining in which value the Reynold's flow is laminar although in general terms this type of flow occurs for Reynolds < 10. It has been observed that turbulence[2] develops, on lower Reynolds numbers, on porous media rather than in sand-free vessels [Muskat,1937]. The reasons are: (i) the sharp edges of sand particles, and (ii) the actual velocities on the interstices are higher. Muskat [Muskat,1937] proposed 1 ($R < 1$) as a maximum Reynolds number value, as safe limit.

NET SEEPAGE SLOPE

Net seepage slope (into the well) is defined as the difference between the slope formed in front of the well from ΔH and H_{av}, and the hinterland slope. The minimum net seepage slope would be 0 which would mean that there is only one slope, and no head reduction is needed. A negative net seepage slope would mean that the allowable head (h_a) is larger than the actual head; in this case there would be no need for relief wells.

TOTAL HEAD LOSS

Total losses cannot be larger than the actual head at the well; this would mean that there exist another source of energy in the system. A representation of head losses can be seen on Figure 3.11, where the magenta dotted line represents the hydraulic head.

[2]Deviations of the linearity between the pressure gradient and seepage velocity

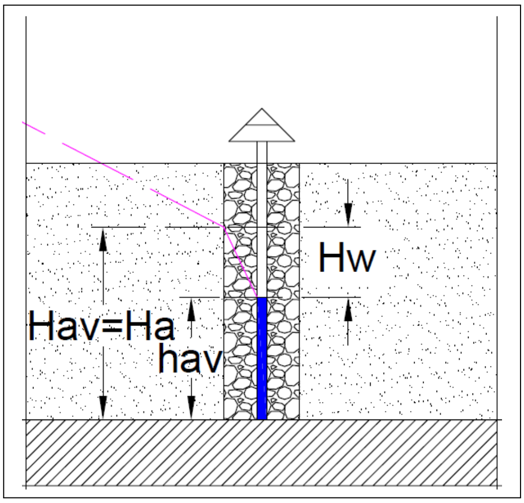


Figure 3.11: Graphic representation of head losses on relief wells' plane.

WELL FACTORS

For an infinitely long line of relief wells, the procedure requires the solution of infinite number of equations. The method used by USACE [USACE,1992] applies well factors in order to determine the head at any point within a random array of fully or partially penetrating wells, without the need of solving infinite simultaneous equations. Well factors are define as the "extra length" or uplift factor [USACE,1992]; θ_a is the average uplift factor (in well plane) and θ_m is the midwell uplift factor (between wells). On USACE's experiments [USACE,1986] it was found the following relation for the well factors:

$$\theta_a = \theta_{a\left(\frac{a}{rw}=100\right)} + \Delta\theta * \left(\log\frac{a}{r_w} - 2\right) \tag{3.18}$$

$$\theta_m = \theta_{a\left(\frac{a}{rw}=100\right)} + \Delta\theta * \left(\log\frac{a}{r_w} - 2\right) \tag{3.19}$$

The limits of application of the formula are:

$$20 < \left(\frac{a}{r_w}\right) < 1000 \tag{3.20}$$

Where $\theta_{a\left(\frac{a}{rw}=100\right)}$, $\theta_{m\left(\frac{a}{rw}=100\right)}$, and $\Delta\theta$ depend on both $\frac{W}{D}$(well penetration), and $\frac{D}{a}$ (aqcuifer depth over well spacing), and are given on Table 3.2 provided by USACE [USACE,1992].

It is worth mentioning that the value of $\Delta\theta$ for fully penetrated wells given on the USACE [USACE,1992] report was found wrong and had to be substituted by $\Delta\theta = 0.367$. This was found after plotting the well factors versus $\frac{D}{a}$, and compared those plots with the ones presented on the USACE Report [USACE,1992].

From the Table 3.2 it can be seen that data for well factors for partially penetrated wells are only available for $0.25 < \frac{D}{a} < 4$; this limits well penetration to higher values (50% >) for increasing net seepage slope. In order to predict which is the lowest limit for well penetration, for a given scenario, the following analytical expression was found (see Appendix Process to Design a Relief Well System):

$$\frac{1}{\theta_{max\left(\frac{W}{D};\frac{D}{a}\right)}} < \frac{\Delta M}{4*(H_{av}-H_w)} * D < \frac{1}{\theta_{min\left(\frac{W}{D};\frac{D}{a}\right)}} \tag{3.21}$$

Where

$\Theta_{max\left(\frac{W}{D};\frac{D}{a}\right)}$ Maximum value of θ_a for $\frac{a}{rw} = 1000$

$\Theta_{min\left(\frac{W}{D};\frac{D}{a}\right)}$ Minimum value of θ_a for $\frac{a}{rw} = 20$

A practical way to use $\frac{\Delta M}{4*(H_{av}-H_w)} * D$, is comparing it with the $\frac{1}{\theta_{min.values}}$ given on the Table 3.3; the minimum well penetration will correspond to that for which $\frac{1}{\theta_{min.values}} \approx \frac{\Delta M}{4*(H_{av}-H_w)} * D$; for example, if $\frac{\Delta M}{4*(H_{av}-H_w)} * D = 0.5$ then the minimum penetration required will be $\approx$ 30%.

Table 3.2: Well factors [USACE,1992]. The value of $\Delta\theta$ for fully penetrated wells has been corrected to 0.367.

$\frac{W}{D}$	$\frac{D}{a}$	$\frac{a}{r_w}$	θ_a	θ_m	$\Delta\theta$	$\frac{W}{D}$	$\frac{D}{a}$	$\frac{a}{r_w}$	θ_a	Θm	$\Delta\theta$
100%	0.25	100%	0.44	0.55	0.367	25%	2	100%	2.39	2.024	1.466
100%	0.5	100%	0.44	0.55	0.367	25%	3	100%	2.798	2.047	1.466
100%	1	100%	0.44	0.55	0.367	25%	4	100%	3.199	2.075	1.466
100%	2	100%	0.44	0.55	0.367	15%	0.25	100%	1.662	1.772	2.077
100%	3	100%	0.44	0.55	0.367	15%	0.5	100%	2.31	2.401	2.077
100%	4	100%	0.44	0.55	0.367	15%	1	100%	2.97	2.938	2.077
75%	0.25	100%	0.523	0.633	0.489	15%	2	100%	3.747	3.293	2.077
75%	0.5	100%	0.563	0.667	0.489	15%	3	100%	4.344	3.363	2.077
75%	1	100%	0.606	0.681	0.489	15%	4	100%	4.941	3.432	2.077
75%	2	100%	0.678	0.682	0.489	10%	0.25	100%	1.908	2.018	3.298
75%	3	100%	0.748	0.682	0.489	10%	0.5	100%	2.934	3.025	3.298
75%	4	100%	0.818	0.682	0.489	10%	1	100%	3.977	3.941	3.298
50%	0.25	100%	0.742	0.851	0.733	10%	2	100%	5.139	4.649	3.298
50%	0.5	100%	0.857	0.955	0.733	10%	3	100%	5.977	4.86	3.298
50%	1	100%	0.983	1.012	0.733	10%	4	100%	6.814	5.071	3.298
50%	2	100%	1.175	1.024	0.733	5%	0.25	100%	1.778	1.887	6.963
50%	3	100%	1.361	1.024	0.733	5%	0.5	100%	3.879	3.969	6.963
50%	4	100%	1.547	1.024	0.733	5%	1	100%	6.063	6.021	6.963
25%	0.25	100%	1.225	1.335	1.466	5%	2	100%	8.377	7.864	6.963
25%	0.5	100%	1.569	1.622	1.466	5%	3	100%	9.761	8.574	6.963
25%	1	100%	1.926	1.908	1.466	5%	4	100%	11.144	9.283	6.963

Table 3.3: Computed values of $\frac{1}{\theta_a}$.

$\frac{W}{D}$	$\theta_{a(max)}$	$\theta_{a(min)}$	$\frac{1}{\theta_{a(max.)}}$	$\frac{1}{\theta_{a(min.)}}$
1	0.81	0.18	1.24	5.44
0.75	1.01	0.18	0.99	2.1
0.5	1.48	0.23	0.68	0.97
0.25	2.69	0.2	0.37	0.46
0.15	3.74	0.21	0.27	0.29
0.1	5.21	0.39	0.19	0.22
0.05	8.74	3.08	0.11	0.16

3.4. HYDRAULIC ENTRANCE LOSSES

From the previous analyses, while developing the method, it was found that entrance losses have significant influence in our reliability system. Due to this reason a more detailed study has been performed. Entrance losses, as addressed in this report, refer to the hydraulic head losses that occur in a granular filter and in the well screen. Only the losses in the filter will be analyzed since the well screen is designed so the frictional losses are negligible; this is achieved thanks to the total of the openings area allowing an entrance velocity, into the well, smaller than 0.03 m/s. Most of the time the reduction on well's capacity is due to vandalism, back-flooding, deformation of the rising pipe, well screen, and/or possible erosion hinterland which can lead to a reduction at the exit point.

FILTER LOSSES

There is little information about how to predict filter losses in front of relief wells. Generally, this has to be found through experiments. The USACE report [USACE,1992] proposes some charts that can help when designing a relief well system without experimental data . In order to have an approximation, and be able to make an appropriate judgment about the losses, a few different approaches were addressed to predict filter losses. One difficulty, when assessing the possible losses in the filter, is to estimate the pore reduction due to clogging; this topic will be detailed on the following section.

3.4.1. MOTION OF GRAINS IN COHESIONLESS SEDIMENTS

The first step in order to analyse the motion in a porous media is to determine the acting forces. There are six types of forces that affect the particles movement: (1.) gravity, (2.) pressure of over laying layers, (3.) inertia, (4.) friction, (5.) capillarity, and (6.) adhesion. Capillarity and adhesion play an important role in the vicinity of the water table, in the unsaturated zone. Water flowing through the porous generates an additional dynamic force on the grains. When this force is greater than the forces holding a grain within the soil, grains start to move. This movement starts with the smallest particles, which can be stopped after a certain distance or removed from the layer. In the first case a structural redistribution takes place, which causes changes on the porosity and permeability of the layer. In the case that the particles are washed away, arching can compensated the lack of coarse material, but if the flow force is high enough subsidence of the layer might occur. The motion of grains can be grouped in three categories [Wit,1984]:

- Suffusion: Movement of the fine particles. If there is only a redistribution of the structure of the grains, is called internal suffusion. External suffusion, in the other hand, refers when there is loss of volume of the solid, but the stability of the skeleton remains unaffected.
- Destruction of the skeleton: If the stability of the skeleton is broken two possible scenarios may arise; subsidence of the layer or creation of channels which lead to a backward erosion, the latter is also known as piping.
- Internal liquidation: Is the loss of bearing capacity of the soil. It occurs when the hydrodynamic uplift exceeds the weight of the layer; this phenomenon is also known as heave. In order to be able to deal with the spatial and time variation of the hydraulic parameters, and to analyse seepage, the following basic assumptions had to be made:
  - Continuum approach; water move through a continuous field,
  - Homogeneity of fluid,
  - Incompressible solid matrix, and
  - Grain in porous are immobile, or at least in the range of velocities for the study otherwise erosion will take place.

CLOGGING

Clogging is defined as: the deposition of fine particles carried on percolating water in the pores of the porous media, can either be mechanical or chemical [Kovács,1981].

PARAMETERS INFLUENCING HYDRAULIC CONDUCTIVITY

In the previous section the hydraulic conductivity is considered as a constant; this assumption considers the porous media as a homogeneous and isotropic field, and that the seepage velocity is constant in space and time. Despite this simplification, in general cases, the determination of a average hydraulic conductivity provide acceptable results. There are two cases where the change of hydraulic conductivity in space and time should be considered; in unsaturated flow and when clogging occurs. Clogging is the settling of particles, moved by infiltrating water, on the original porous decreasing the hydraulic conductivity. Clogging is not only a mechanical process; small particles can also be attracted by electrostatic and electrochemical forces.

MECHANICAL CLOGGING

Mechanical clogging occurs when the smallest suspended particles can go through bigger pores in the filter, and then deposit there. Mechanical clogging is taken care of by a good filter design e.g. using geometrically closed filters.

CHEMICAL CLOGGING

Oxides and other chemical compounds can generate attraction between particles, which can be attached to the filter particles reducing its porosity. In order to establish if chemical clogging is possible, the properties of the subsurface water and soil have to be known. Nevertheless, by applying some cheap chemical agent on the wells, this issue can easily be overcome.
Total clogging of the well is unlikely to happen, due to the fact there is a maximum pore reduction after which the smallest suspended particles cannot settle down (due to the increased velocity) on the pores of the porous

media. In addition, not all the percolating water goes through the well; some of the water carrying the suspended load can deviated and do not percolate through it.
The most influenced parameter when clogging occurs, is the soil resistance (i.e. the hydraulic conductivity of the filter is decreased, which translates into the well entrance losses). In order to account for this pore reduction, the maximum amount of deposited grains will be considered as described by Kovacs [Kovács,1981].
On the reports referenced by USACE there is no record that a well has been abandoned by clogging; most of the time the reduction of the capacity of the well is due to backflooding, and the reasons exposed earlier.

FILTER DESIGN

In order to proceed with the calculation of the filter losses, its characteristics have to be known. In designing, the following filters rules are applied, the characteristic diameters of sand aquifer can be seen on Table 3.4. For more detail, refer to [USACE,1992].

Table 3.4: Characteristics sand diameter.

d_{b15}	0.00015	m
d_{b85}	0.00028	m

Filter rules:

$$\frac{d_{f15}}{d_{b85}} < 5 \tag{3.22}$$

$$\frac{d_{f15}}{d_{b15}} > 5 \tag{3.23}$$

$$\frac{d_{f60}}{d_{f10}} < 10 \tag{3.24}$$

Where

d_f Filter diameter

d_b Sand diameter of the aquifer

It is found:

Table 3.5: Filter characteristics.

d_{f15}	0.00075	m
d_{f85}	0.025	m
d_{f60}	0.005769	m
d_{f10}	0.000577	m
thickness	0.2	m

In order to prevent the filter material to enter into the well, holes on the screen should fulfill the following condition [Wit,1984]:
• For elongated slits $d_{f85} > 1.2b$, b width of the slit
• For circular holes $d_{f85} > 1.0 * d$, d diameter of the hole

3.4.2. ESTIMATION OF ENTRANCE LOSSES

Well efficiency reduction is caused by various reasons, but most of them are external factors of the system as vandalism, back flooding or deformation on the rising pipe/well screen. To be able to dismiss all these possible causes of well efficiency reduction it should be assumed that the well counts with a metal guard, a security valve, and that the pipes fulfill the design requirements. From what has been stated before, even if the determination of entrance losses can be somehow "uncertain", it can be certain that the filter cannot become completed clogged, thus the main idea is to look for the max head losses.

FILTER LOSSES (CLEAN FILTER)

In order to have an idea (approximation) of the possible head losses in the filter, the equation Carman-Kozeny will be used:

$$\Delta h_f = 180 * \frac{\nu}{g} \frac{(1-p)^2}{p^3} \frac{v}{{d_h}^2} T_f \tag{3.25}$$

Where

Δh_f Head loss (m)

ν Kinematic viscosity $1.004 * 10-6$(at 20 C)(m^2/s)

v Velocity referring to the flow velocity, assuming there is no filter

d_h Diameter of filter material

T_f Filter thickness

This equation was derived for clean filters under laminar flow with $(Re < 5)$, nevertheless this formula is applicable when filtrating water with some solid concentration. Having in mind these considerations, the computation were performed showing the following results:

Head losses clean filter = $0.025m$

For a clogged filter the following correction is proposed:

Head losses clogged filter = $0.027m$

In case of having turbulent flow, the maximum head losses on a clogged filter during back-washing can be calculated using the following formula:

$$\Delta h_{f_{max.}} = (1-p) * T_f * \left(\frac{\gamma_k - \gamma_w}{\gamma_w}\right) \tag{3.26}$$

Where

γ_k Specific weight of the grain (kN/m^3)

p Porosity

This results in:

Head loss max = $0.20cm$

Finally, a simple comparison is made by using the different permeability's, and the equation for seepage flow:

$$\frac{Q}{A} = k_f * \left(\frac{\Delta h_f}{T_f}\right)^{\frac{1}{p_0}} \tag{3.27}$$

Where

Q Well discharge

A Filters transversal area

p_0 1.5 for transitional flow (gravel) (Bed bank, and shore protection book)

Using the limits for the permeability for gravel (see Table 2.1 $(0.001 < k_f < 0.1)$ it was found that: When

$$k_f = 0.1m/s$$

$$\Delta h_f = 0.0005m$$

and when

$$k_f = 0.001m/s$$

$$\Delta h_f = 0.5m$$

USACE

USACE proposes Figure 3.12 to determine the well loses; this data has been found experimentally.

For the maximum discharge found in this example, ($0.025 m^3/s$, approx. 10 GPM per FT of screen), the well losses according to the plot are 0,5 ft (0.15 cm approx.). USACE also propose the chart seen on Figure 3.13 to estimate the losses at the end of the life cycle of the well.

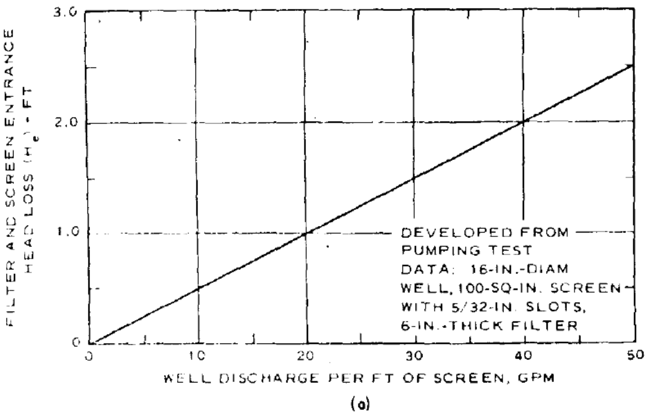


Figure 3.12: Head losses at the filter (clean). Source:[USACE,1992].

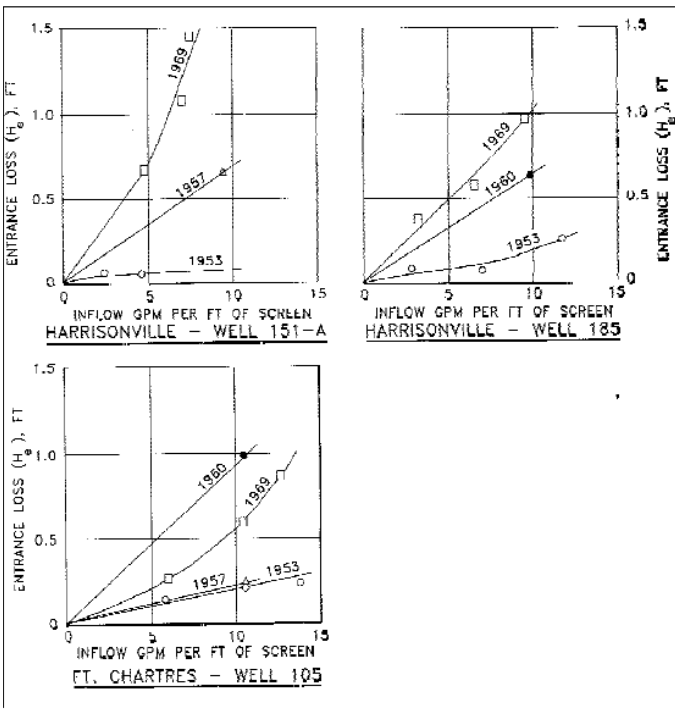


Figure 3.13: Entrance losses (total). Source: [USACE,1992].

From here it can be seen that the maximum losses found correspond to 1.5 ft (0.46 m)

CONCLUSIONS

- Given that there is not direct source of information, and as it has been stated on the reviewed literature, the filter losses are hard to asses and that experiments are required, use of the USACE seems a reasonable assumption for the present explanatory study.

- The data from USACE will be used in the probabilistic analysis using a mean value accordingly to Figure 3.13, with the respective well discharge, and a covariance of 100% (using a lognormal distribution in order to avoid negative values during the probabilistic simulation).

- Applying the analyzed formulas to find possible entrance losses, it can be depicted that these losses are highly dependent on the difference in permeability between the aquifer and the filter.

- Using the data for total entrance losses from USACE, this uncertainty is taken into consideration given that this data corresponds to the total entrance losses.

3.5. COST ANALYSIS RELIEF WELLS

In order to perform the cost analysis the prices reported by CYPE Engineers [CYPE,Ingenieros] were used as reference. These prices were updated until June 2014, and according to the local market. For unit prices, only direct costs have been taken into consideration. The unit price analysis can be seen on the Appendix Unit Price Analysis. The final intention of this cost analysis is to compare the two studied possibilities of measure for piping mitigation (relief wells, and piping berms). The goal was to obtain a cost function in order to find an optimum for each alternative, and later on compare them in terms of cost per meter length of a dike cross section. Two different costs were analyzed: cost per station, and cost per meter of well penetration. On Figure 3.14, the elements of a relief well can be visualized, for more reference.

COST PER STATION

It refers to all fixed costs that a station can generate. These costs are divided into station costs, and equipment cost (see Table 3.6).

1. **Station costs:** The most relevant costs are:

    - Metal well guard (102.16 €/unit): This metal guard is needed to avoid any possible vandalism as well as to protect the well entrance from animals around the area. It is designed with a width of 1.5 times the radius of the well diameter, and it has a height of 51 cm. It is assumed to be of black steel with galvanized painting.
    - Valve (717.76 €/unit): It is assumed that a security valve will be placed on each well station. The purposes of this valve are: (i) avoid the flow from outside the well to get into it (most of the times, clogging is caused due to dirty water), (ii) and allow the well to discharge when high water levels are reached at the well point, instead of discharging under "normal" water levels.
    - Concrete back-filling (180.22 €/unit): Its purpose is to avoid superficial infiltrated water to flow into the well. For this analysis a depth of 0.8 m was assumed.

2. **Equipment costs (727.38 €/unit):** The equipment used consist of a drainage system which will pump out the water while making the excavation; this system includes pumps, enough pipes to drain, and the necessary equipment to excavate and install the rising pipe. CYPE Engineers' archives [CYPE,Ingenieros] were used as reference to estimate this cost. Also the following expenses were considered with regarding equipment cost: (i) renting and mobilizing the equipment for drilling the wells which cover the pumping system, (ii) placing the equipment and having it ready for use, and (iii) usage of the equipment per meter of excavation (detailed on 3.5).

Table 3.6: Total cost per station.

Metal well guard	102.16	€
Valve	717.76	€
Concrete back-filling	180.22	€
Equipment	727.38	€
Total Cost	**1727.52**	**€**

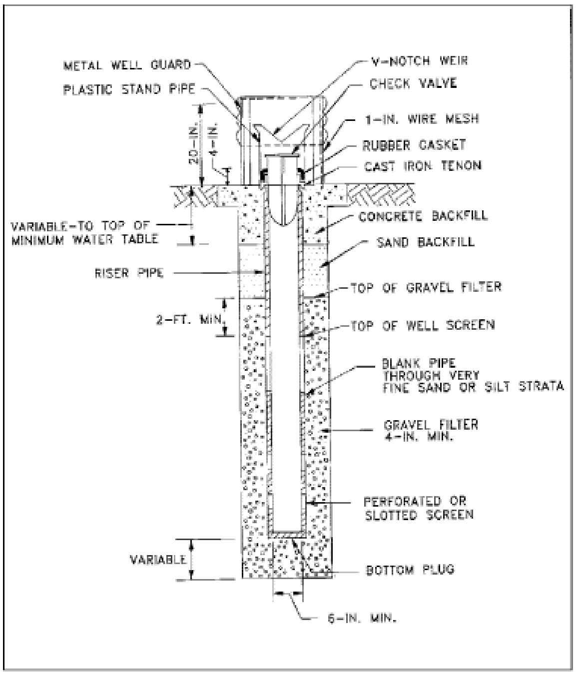


Figure 3.14: Relief wells system schema. Source: [USACE,1992].

COST PER METER OF WELL PENETRATION

To be able to calculate this cost, the expenses related to excavation, the filter, and rising piping were added up (see Table 3.7).

- Excavation (6.80 €/m): These expenses include the cost of excavation per meter depth of the relief well, using the equipment referred on the previous section.
- Filter (6.07 €/m): The material used for the filter is granular, and its cost was taken into consideration; the amount of material used was calculated based on the diameter of the relief well, and on the thickness of the filter. This thickness is as minimum as 0.15 m, following the recommendations of the USACE (min. 6 in)
- Rising piping (117.77 €/m): One of the causes for losing efficiency in the wells is due excessive deformation or corrosion on the rising pipe> In order to limit maintenance and avoid possible deformations in this cost analysis a rising pipe of galvanized steel will be addressed. In this cost are included the expenses of the rising piping which will be used as well screen (material and installation) per meter using the equipment describe on the previous section.

Table 3.7: Total cost per meter depth.

Excavation	6.80	€/m
Filter	6.07	€/m
Rising Pipe	117.77	€/m
Total cost	**130.64**	**€/m**

COST FUNCTION RELIEF WELLS

The cost function of the relief wells will be:

$$Ct_{(w)permeterdike} = \frac{C_{station} + C_{wellpenetration} * W}{a} \tag{3.28}$$

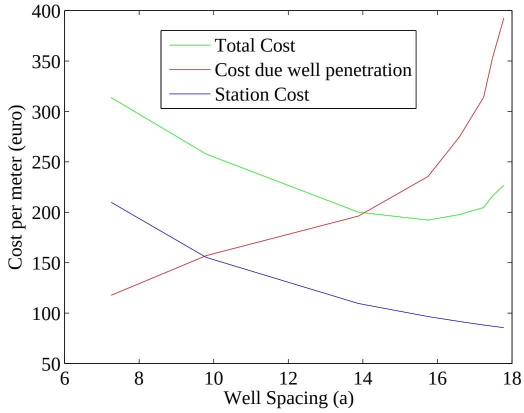
$$Ct_{(w)permeterdike} = \frac{1737.52 + 130.62 * W}{a}$$

Where

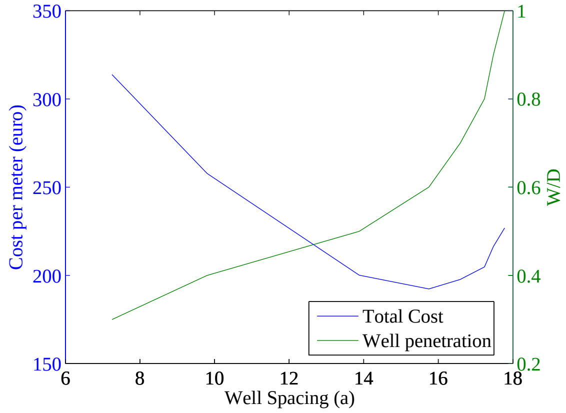
W Well penetration (m)

OPTIMAL COST OF RELIEF WELLS

From the cost function of relief wells it can be seen that it is dependent of two variables: the well spacing, and well penetration (refer to Figure 3.15).



(a)



(b)

Figure 3.15: Optimal cost of relief wells. Figure (a) cost relief wells, function of well spacing, and Figure (b)cost relief wells (double vertical axis); cost/meter on right, wells penetration left.

In a deterministic approach the optimal well penetration should correspond to that one which generates the minimum cost. When accounting for the reliability target, the optimum should be chosen among those combinations of well spacing and well penetration that fulfill the required reliability target. For example, according to the Figure 3.15b, the optimal combination would correspond to a well penetration of 60% and a well spacing of 16 m; in section 5.3 the results of the probabilistic approach will be shown.

3.6. PIPING BERMS

In the case of piping berms, the same linear piezometric profile as for relief wells is adopted. In Figure 3.7 a squematization of the grade line in a confined acquifer is depicted. The exit point, where the subsurface water reaches the same hydrostatic pressure as the phreatic level [Kovács,1981], is assumed to be equal to X_3 as mention in section 3.1.3. When designing a piping berm, its thickness will correspond to the additional required head resistance to withstand the ocurrent potential, at potential exit point (susceptible point where seepage water reaches surface). On the other hand, its length will correspond to the same longitude to where the existing potential is smaller than the maximum allowable head, always accomplishing the reliability target. The occuring potential can be computed as follows:

$$\phi_z = \left(\frac{\Delta H}{S + X_3}\right) * (X_3 - X_i) \tag{3.29}$$

Where X_i is the distance from toe lanside (start of the piping berm), to the point where the potential is evaluated.

3.7. COSTS ANALYSIS PIPING BERM

To analyze the cost of implementing piping berms the following items were considered:

1. **Cost of land (43.22 €/ m^2):** In order to estimate the cost of the land, prices of real state in the Netherlands were researched [3]. Two different locations were consulted; the first one was Amsterdam, assuming that it is one of the most expensive places to buy land, and the second one was Lelystad, which corresponds to a not so dense populated area, and the real estate prices are considerable lower compare

[3]The Netherlands Institute of Real Estate Brokers (NVM) (https://www.nvm.nl/nl-nl/over_nvm/english/real_estate_business_practices.aspx)

to the ones in Amsterdam. The average price of the construction cost per square meter in Amsterdam is 3674€/m^2, and in Lelystad 834 €/m^2 . The cost of the land was set to be 10% of the construction cost, taken in consideration the conservative case of Lelystad, given that the regions closer to the dikes are generally non-commercial zones. Additional to this value a reduction of 50% (5% of the total construction cost) is considerate assuming that these areas do not count with the basic urbanization services (i.e. they are treated as rural areas); this results in a land cost of approximately 42 €/m^2

2. **Berm material (24.41 €/m^3):** These expenses refer to the price of the material used to form the berm, and this cost is given in €/m^3. The cost per lineal meter will be computed for each scenario according to the designed thickness, and length of the berm. In this cost the expenses due to transport are not considered since it is assumed that the material is brought from the same place than the filter material, for the case of the relief wells, in order to be able to compare them.

3. **Compaction (1.70 €/m^3):** This item refers to the costs of compacting the berm material. In the same manner than for the berm material the cost per lineal meter will depend on to the designed thickness, and length of the berm.

COST FUNCTION PIPING BERMS:

The cost function for piping berms can be reduced to the sum of the cost of land plus the cost of back-filling, and compaction:

$$
\begin{aligned}
Ct_{(B)permeterdike} &= C_{backfilling+compaction} * d_{berm} * L_{berm} + C_{land} * L_{berm} \qquad (3.30)\\
Ct_{(B)permeterdike} &= 26.11 * d_{berm} * L_{berm} + 43.22 * L_{berm}
\end{aligned}
$$

Where

d_{berm} Berm thickness

L_{berm} Berm length

This function increases when the length and berm's thickness increase, as it can be seen on Figure 5.3.

3.8. LIFE CYCLE COST CONSIDERATIONS / COMPARISON

LCC of a project can be defined as "the sum of all costs incurred during its life span" [Dhillon,1947]. The importance of taking into account the possible future cost of a structure has been discussed in several literature [Dhillon,1947, Bull,1993]. In some cases, the cheapest solution regarding initial investment turns out to be more expensive due to its maintenance/replacement costs; based on these reasons, this report gives relevance to a LCC analysis in the decision-taking stage. The first step to proceed with a LCC analysis is to define the life cycle of the considered alternatives:

- For piping berms: a life cycle period of 100 years is assumed, given the importance of the structure (see Table 3.8 [Schuppener,2013]) and the durability of the material used (soil).
- For relief wells: the life cycle time considered is 20 years due to two main reasons: (i) it is the longer period of time recorded on the reports available regarding monitoring relief wells efficiency, and (ii) no maintenance or monitoring resource has been included on the cost analysis.

Table 3.8: Eurocode life cycle [Eurocode,1990].

Class	Design working life (years)	Examples
1	1-5	Temporary structures
2	25	Replacement structural parts,e.g. gantry girders, bearings
3	50	Buildings and common structures
4	100	Monumental buildings and other special or important structures
5	120	Bridges

The decision of not considering maintenance and monitoring costs, in the case of relief wells, can be questionable; nevertheless, including such costs would mean a decrease on the entrance losses which would increase the well performance, extending its life span. Additionally, it is important to mention that piping berms are also subject to monitoring, and in some cases (holes made by trees, animal digging, clay extraction, etc.) to maintenance. Accordingly to these definitions, the development of this report was carried out.

There are several methods to analyze LCC; the most common applied in an infrastructure project, and used in this report, is to calculate the net present value. The formulae to carry out this analysis is:

$$W_f = P(1+i)^{n_i} \tag{3.31}$$

Where

W_f Future worth or amount (i.e. principal amount plus interest earned)

n_i Number of interest periods (e.g. years)

P Principal amount

i Net discount rate

$$V_p = \frac{W_f}{(1+i)^n} \tag{3.32}$$

Where

V_p Present value

$$V_{p_t} = \sum_{i=0}^{N} \frac{W_f}{(1+i)^n} \tag{3.33}$$

Where

N Total life cycle (years)

V_{p_t} Total present value for given N

As an example, data from Table 5.7 has been taken. For ring 36, the cost for relief wells amounts to 695.59 €/m, and for the same cross section, alternative piping berms, has a cost of 4327.70 €/m, for rural area. The net present value has been computed (a net discount rate of 2.5% has been chosen, as reasonable value)[4] as follows:

For relief wells:

$$V_{p_t} = \frac{695.59}{(1+i)^0} + \frac{695.59}{(1+i)^{20}} + \frac{695.59}{(1+i)^{40}} + \frac{695.59}{(1+i)^{60}} + \frac{695.59}{(1+i)^{80}}$$

$V_{p_t} = 1633.73$ €/m

For piping berms:

$$V_{p_t} = 4327.70$$

$V_{p_t} = 4327.70$ €/m

It can be observed that the net present value for relief wells, will be always (accounting for the assumptions made in this thesis) lower than for piping berms despite the value of the discount rate ($i > 0$). If the discount rate is increased, the net present values will decrease; the maximum present value will occur when $i = 0$, which will correspond to 5 times the initial cost; this will be still cheaper than piping berm. For this example the relief wells could be replaced almost every 7 years in order to equate the same LCC than for piping berms.

[4] net inflation rate in the Netherlands 5.5%

3.9. SUMMARY OF DESIGN AND COST CONSIDERATIONS

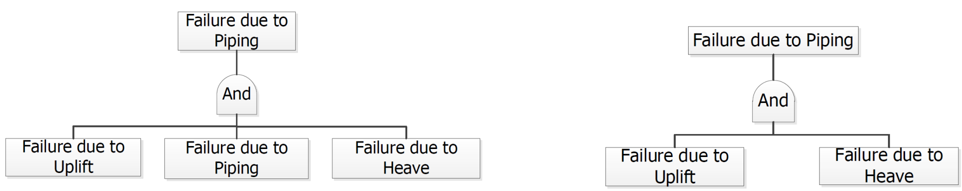
In this chapter the main assumptions and considerations for designing a relief wells system and piping berms as piping mitigation measure have been exposed. The process to design a relief wells as well as the involved variables were introduced. The limitations of the USACE method were discussed. One of these limitations is introduced by the so called well factors which were derived only for one range of well spacing $0.25 < D/a < 4$.The others limitations are subjected to the flow regimen (laminar).From equation 3.13and equation 3.13 it can be observed that the maximum head reduction is limited to the minimum possible well spacing ($a > D/4$). Special attention was paid to the hydraulic losses, from the literature it was found that total clogging is unlikely [Kovács,1981]. It was also found, accordingly to USACE [USACE,1992], that hydraulic losses vary over the time, and that sometimes well performance can improve over the time (reduction of well losses). This can be consequence that during high water levels, wells could 'clean' their self by washing away the particles retained on the filter. Given these findings it is recommended to account for a maximum well loses over their life cycle, and disregard their time dependency. Regarding the cost, a cost analyses for both alternatives was performed and the cost functions were presented on sections 3.5 and 3.7. The life cycle cost considerations were also presented and it was defined a life cycle of 100 years for piping berms, using as reference the recommendations of the [Eurocode,1990]. In the case of relief wells a life cycle of 20 years was adopted given that is the maximum period of records for well losses.

4
Probabilistic Analysis

The purpose of this chapter is to define the probabilistic framework implemented to perform the probabilistic design. The reliability systems and the limit state functions are defined, also the statistical parameters of the random variables are discussed. A comparison between the two chosen methods (MCS and FORM) used for the probabilistic analysis is performed. The main purpose of this comparison is to verify if FORM approximates the performance satisfactory and can be used as computationally cheaper replacement for MCS. The principal issue with the MCS method is that it is computational expensive; particularly for this specific case, where the USACE method itself is an iterative time consuming process. In addition, due to the high reliability target (low probability of failure), the number of Monte Carlo simulation needed increases exponentially. Finally, when the results were compared, and a good approximation of FORM was found, the subsequent analyses were performed using only FORM.

4.1. Fault Tree (Parallel System)

In order to perform the reliability analysis for piping [Schweckendiek et al.,2013] propose the fault three as described in Figure 4.1a, here the fault tree represents a parallel system with three sub-mechanism: uplift, heave and piping.Herein piping mechanism refers to the creation of pipes under the dike, and not to the failure of the system due to piping. These three sub-mechanism resemble the phases for developing piping failure, as describe in 2.2. Different to a serial system, in a parallel system an individual sub-mechanism can fail without causing the whole system to fail, only if all three mechanisms fail the system will fail. A serial system can be compared to Christmas lights, if one light fails the whole system fails, thus increasing the number of lights, increases the probability of failure of the system. In the other hand a parallel system can be compared to a candelabrum, if one light fails the others can still light the room (system does not fail). In this case adding more lights (failure modes) reduces the probability of failure of the system. For the scope of this thesis only two failure modes, due uplift and piping, will be considered (see Figure 4.1b).The reason why piping mechanism was disregarded is because there is not method for analyzing this type of failure when drainage systems are placed. The acknowledged methods of Lane and Bligh were derived for damns without drainage systems; the same occurs with Sellmeijer, the other recognized method by TAW [TAW,1999]. In addition to this, as mentioned before for the case of a parallel system, including piping failure mode will always lead to a lower probability of failure. Acknowledging this property of parallel system, allows to be on the safe side



(a) (b)

Figure 4.1: Faul trees examples. Figure (a) fault tree with three failures, and Figure (b) fault tree with two failures.

when using the fault three as depicted in Figure 4.1b.
The first step in using a RBD method is to define the limit state functions. A limit state is the condition in which a mechanism cannot longer fulfill its design requirements [Eurocode,1990]. Reliability is the probability that this limit state is not exceeded. The general form of writing the performance function is:

$$Z = R_z - S_z \tag{4.1}$$

where

R_z Represents the strength (resistance)

S_z The loads (solicitation)

The limit state is defined as $Z = 0$.
The probability of failure per mechanism $P[F_i]$ is defined as the probability of the respective LSF being exceed, i.e. $Z_i < 0$. The probability of failure of the system $P[F]$ is then:
For the fault tree depicted on Figure 4.1a:

$$P[F] = P[F_u] \cap P[F_h] \cap P[F_p] = \left\{\{Z_u(X) < 0\} \cap \{Z_h(X) < 0\} \cap \{Z_p(X) < 0\}\right\} \tag{4.2}$$

For the fault tree depicted on Figure 4.1b:

$$P[F] = P[F_u] \cap P[F_h] = \{\{Z_u(X) < 0\} \cap \{Z_h(X) < 0\}\} \tag{4.3}$$

The limit state is defined as $Z = 0$.

COMBINED FAILURE MODE

In order to account for the combined failure mode of uplift and heave, the following formulae are adopted:
For MCS

$$Z_{u+h} = max\{Z_u, Z_h\} \tag{4.4}$$

In the case of FORM, Hohenbichler [Hohenbichler et al.,1983] formulation is adopted; the reader is advice to refer to the bibliography.

4.2. RELIABILITY-BASED DESIGN

Reliability is defined as the probability that something functions as it should [CUR,1997]. The reliability index is defined as:

$$\beta = -\Phi^{-1}(P_f) \tag{4.5}$$

Where

β Reliability index

Φ^{-1} Inverse of the standard normal cumulative distribution function

P_f Probability of failure

The analysis of the decision problem entails looking for the answer to the following questions [CUR,1997]:

- From which actions the decision maker can choose
- What are the possible circumstances that influence the result
- What are the possible results as an outcome of a chosen action and given circumstances

Answering these questions results in [CUR,1997]:

- Set of all possible actions
- Set of circumstances
- Set of possible results, function of the circumstances and actions

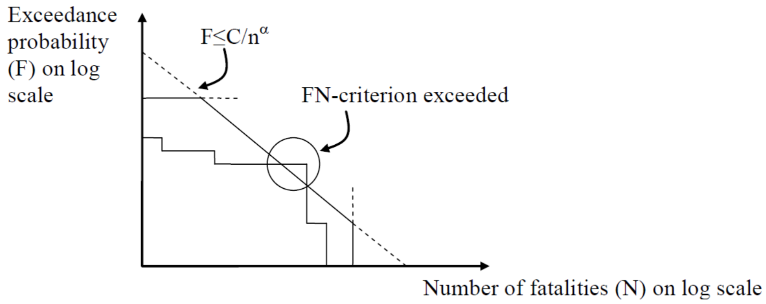


Figure 4.2: Schematic overview of FN-curve. Source: [Schweckendiek et al.,2012b].

SYSTEM TARGET RELIABILITY

Consequences of failure of flood defences are as important as the investment for increasing the reliability of such systems. For this reason it is important to determine target reliabilities. In engineering applications target reliabilities are set in accordance to risk acceptability. In order to define the acceptable risk for a flood defence there are three main widely used criteria [Schweckendiek et al.,2014]:

- Individual Risk Criteria: Individual risk criteria concerns with individual exposure, to ensure that there is a distributive justice ("equity") and no person is disproportional exposed to a safety level [CUR,1997]. It is often defines as maximum allowable probability of death. For voluntary activities 10-2, and for involuntary activities 10-6 per year [Schweckendiek et al.,2012b].
- Societal risk criteria: Societal criteria refers to the effects of large accidents can have on a vast population. The common criteria used to determine is the use of the FN-curve. FN-curve shows the exceed probabilities of different numbers of fatalities (see Figure 4.2).
- Economic Criteria: In order to get the optimal level, from an economic point of view, the cost of increasing system reliability and the expected loss (risk) should be compare. The theoretical optimal will be the point where the cost of increasing the system reliability equates the expected losses (see Figure 4.3).

4.3. LIMIT STATE FUNCTIONS

Regarding dike safety, every country manages its own regulations. As mentioned before, the current legislation for flood defences in The Netherlands is the *Flood Defences Act*. The guidelines adopted are the ones presented by TAW [TAW,1998] on its research programme "Flood risks: a study on probabilities and consequences". On its predecessor of the *Flood Defences Act, Delta Law*, dike design was focus on withstand a certain water level. The Delta committee applied risk- based economic optimization in order to develop its design philosophy. Nevertheless, at the time of its development, methods to deal with more than one stochastic variable in a risk-based optimization were not available [Ammerlaan,2007]. Currently, the TAW [TAW,1999]recommends that dike safety evaluation should be based on the probability of flooding instead of the probability of exceedance. The former approach will be used in this report. In the following section the LSF of each potential failure mechanism used on this report are defined.

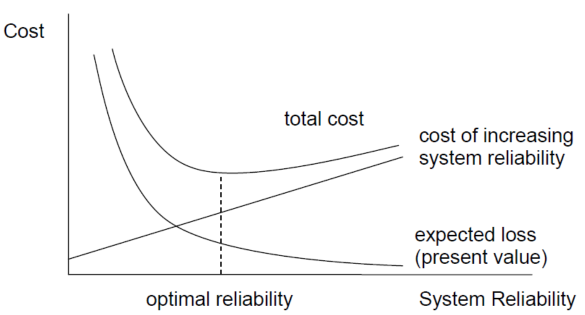


Figure 4.3: Principle of economically optimal. Source: [Schweckendiek et al.,2012b].

4.3.1. UPLIFT

The LSF for uplift is defined by the uplift resistance (R_u), is a function of the effective stress at the bottom of the cover layer, and the uplift solicitation (S_u), represented by the average head of the well (H_{av}), or by the head between wells (H_m). The larger head, H_{av} or H_m, is selected according to each case (refer to 3.2 for details).

$$Z_u = R_u - S_u \tag{4.6}$$

$$R_u = \frac{\sigma'}{\gamma_w} = h_a \tag{4.7}$$

$$\sigma' = d * (\gamma_s - \gamma_w) \tag{4.8}$$

$$S_u = \phi_z \tag{4.9}$$

where

h_a Maximum allowable head

σ' Effective stress of the soil at the bottom of the cover

In case of relief wells system $\phi_z = max\{H_{av}, H_m\}$ [1]. From equations 4.8 and 4.9 the LSF for uplift can be written as:

$$Z_u = \frac{d * (\gamma_s - \gamma_w)}{\gamma_w} - \phi_z \tag{4.10}$$

4.3.2. HEAVE

From the previous description (see 2.2.2) of the failure mechanism for heave, the logical LSF will be that one that compares the critical gradient and the existing vertical gradient on the blanket (impervious layer), where the sand transport may take place. The LSF due to heave can be written as:

$$Z_h = i_c - \frac{\phi_z}{d} \tag{4.11}$$

4.4. PARAMETERS FOR PROBABILISTIC ANALYSIS

For most of the data needed to evaluate the hydraulic head in a relief well system exist statistical information.Extensive statistical information regarding soil properties have been published [Phoon,2008] and the VNK2 [Jongejan et al.,2011] also provides information about the parameters to be used for water levels as for soil properties.For the scope of this research data from VNK2 project will be addressed and they are presented in table 4.1.

[1] According to the case for fully or partially penetrated wells

Table 4.1: Parameters of the data.

Symbol	**Unit**	**Distribution Type**	**Mean** μ_x	**COV** Vx	**SD** σ_x
γ_{cover}	kN/m^3	Normal	Nominal	0.1	
γ_w	kN/m^3	Deterministic	Nominal	NA	NA
d	m	Lognormal	Nominal	0.3	
D	m	Normal	Nominal	0.1	
k_f	m/s	Lognormal	Nominal	1	
k_b	m/s	Lognormal	Nominal	1	
h_r	m	Gumbel	Nominal	0.1	
h_p	m	Normal	Nominal		0.1
H_e	m	Lognormal	Nominal	1	
S	m	Normal	Nominal	0.05	
t_p	m	Deterministic	Nominal	NA	NA
C	[-]	Normal	130		0.13
r_w	m	Deterministic	Nominal	NA	NA
W	m	Normal	Nominal	0.05	
i_c	[-]	Lognormal	0.7		0.1

Nominal value based on data.

In the case of well loses recommendations from section 3.4 are addressed, adopting a lognormal distribution and a mean valued according to USACE advice (see figure 3.13). A few variables were assumed as deterministic due to their small variance, and small influence (deducted from preliminary evaluation, see Appendix Preliminary Evaluation) (i) the well radius r_w, (ii) well thickness t_p , (iii) and Specific weight of the water γ_w.

4.5. METHOD FOR COMPUTATION OF PROBABILITY OF FAILURE

Several methods exist to compute the probability of failure of a system in a RBD analysis. The probability of failure of a system is the driven parameter in a reliability analysis. The Joint Committee on Structural Safety suggests the following level-characterization [JCSS,2001]:
1. Level I: A probabilistic analysis is not performed but instead, the methods in this level are based on safety factors for designing purpose, according to codes and standards. Hence a failure probability can not be computed.
2. Level II: Level II encompasses approximation methods to compute the probability of failure. This approximation entails linearizing the reliability function in a selected point and the random variables are characterized by their distribution and statistical parameters. FORM falls in this category.
3. Level III: In this level a simulation based fully probabilistic analysis is carried out. The reliability of the system is computed directly from the probability of failure. These methods consider all the probabilistic characteristics of the whole random variables. The most straight forward method is MCS.
In this research level II and level III method, that is FORM and MCS respectively, are used and they will be introduced in the following section.

4.5.1. MONTE CARLO SIMULATION

A MCS relies on random sampling by following a given probability distribution function. It simulates n times the performance function ($Z_i = R_i - S_i$) generating this way random numbers of the variables according to their probability distribution function. The probability of failure can then be estimated:

$$P_f = \frac{n_f}{n} \tag{4.12}$$

where

n_f Number of simulations for which $Z_i < 0$

In Figure 4.4 an illustration of the basics of this technique is depicted. On the vertical and horizontal axes are the random variables X_2, and X_1, which represents R_z and S_z as describe in equation 4.1. The curve

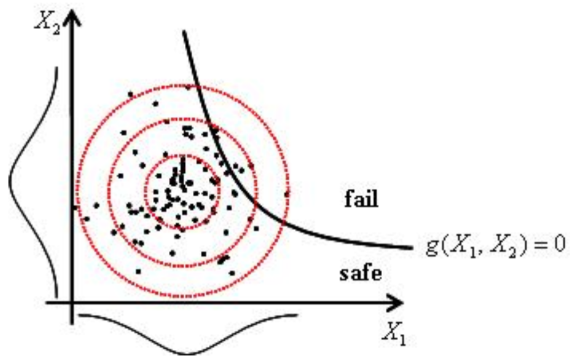


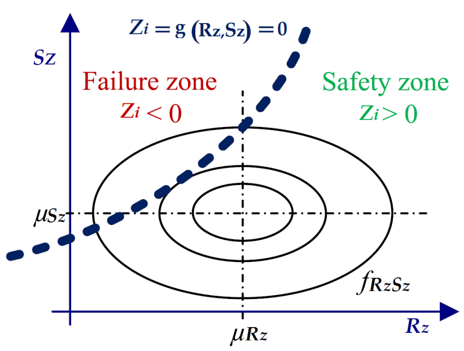
Figure 4.4: Ilustration of MCS. Source: [Yong-Kyun,2008].

black line in the X_2, X_1 plane denotes the LSF for this example. The zone above this line indicates when Z_i <0 ($g(X_1, X_2) < 0$ on the figure) which specifies failure of the system. The zone bellow specifies $Z_i > 0$, no failure. The curve lines next to the axes, represents the probability distribution function of the random variables. It can be denoted how the majority of the generated values are in the zones with higher frequency of occurrence. As mentioned previously, the main inconvenient of this method is the large number of simulations required in order to acquired a small relative error, which is function of the number of simulations and the probability of failure of the system. Roughly speaking the minimum number of simulations can be estimated using equation 4.13 [CUR,1997]. The advantage of the MCS is that it is relative easy to implement in a computer code. For this report the computational algorithm offer by OpenEarth [Den Heijer,2012] was implemented in a `Matlab` scrip. The other subroutines were developed specifically for this study and can be found in Appendix Matlab Script .

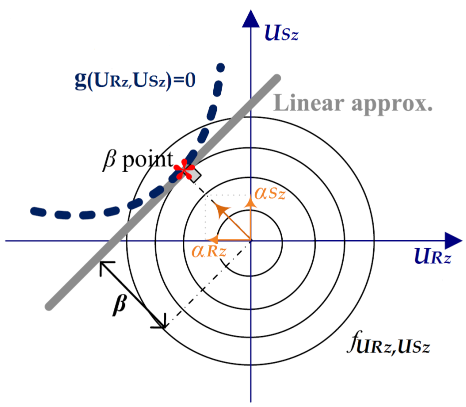
$$n \quad > \quad 400\left(\frac{1}{P_f} - 1\right) \tag{4.13}$$

4.5.2. FORM

FORM is based on the first order linearization of the LSF in the point with the greatest joint probability in the failure space (design point). The random variables are transformed to equivalent standard normally distributed variables (U-space) and theLSF is replaced by its first order Taylor approximation in the design point [Voortman,2003]. In Figure 4.5 a graphic representation of the transformation from the initial base space to the standard normal space is shown. The grey line in Figure 4.5b represents the linear approximation in the design point, which is the closest point to the origin in the standard normal space. The distance from origin to the design point is then the reliability index of the system and the unit vectors are the so called influence fac-



(a)



(b)

Figure 4.5: FORM. Source: [Teixeiraa,A. et al.,2012].

tors α_i. These factors are essential to measure how each base variable influences the performance function. In order to measure the relative influence of the variables the α_i^2 values are addressed. These values, of α_i^2, are referred in this report as sensitivity factors and will be used on the sensitivity analysis. For more information regarding these methods the reader can refer to the bibliography of this report [Faulkner,D.et al.,1991].

4.6. EXAMPLE PROBABILISTIC ANALYSIS

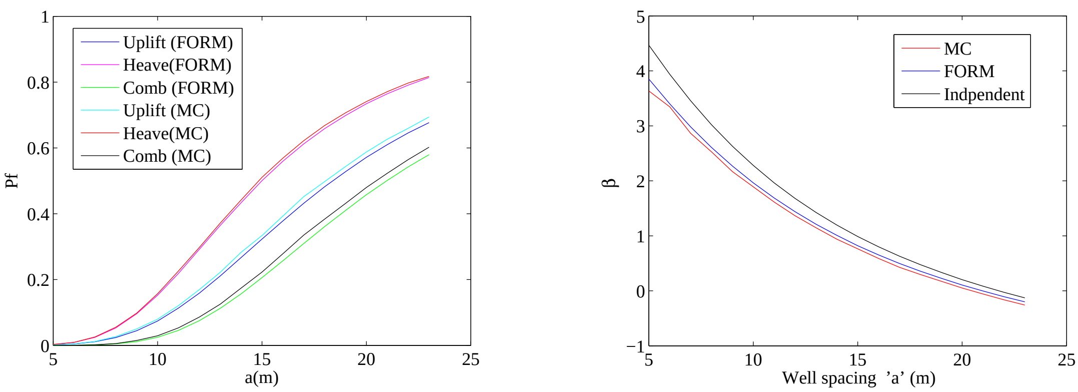
In this section, examples of the probabilistic analyses for relief wells as for piping berms are presented. The intention of these example is to illustrate how the probabilistic analysis is performed and to show the accuracy of FORM compared to MCS .

4.6.1. PROBABILISTIC EXAMPLE OF RELIEF WELLS

In order to perform the probabilistic analyses for a relief wells system as piping mitigation measure, the system is described as a fault tree as depicted on Figure 4.1b. The LSF for uplift and heave are applied at well or midwell between wells, as discussed on section 4.3. To estimate the reliability due piping, the combined failure mode of uplift and heave, the formulations describe in 4.1 are addressed. The used example, for the case of relief wells system, corresponds to a fictitious case study. This fictitious case study has been built in such a manner that falls in between the limits of application of the USACE's method. The used data is shown on table 4.2.

Table 4.2: Data for probabilistic example for relief wells.

Symbol	**Distribution Type**	**Mean** μ_x	**SD** σ_x	**Unit**	**Description**
γ_{cover}	normal	17.1	1.7	kN/m^3	Specific weight cover layer
γ_w	deterministic	10	0	kN/m^3	Specific weight water
γ_{berm}	normal	18	1.8	kN/m^3	Specific weight berm
d	lognormal	4.11	1.23	m	Blanket thickness
D	normal	9.3	3	m	Aquifer thickness
k_f	lognormal	5.7E-04	7.5E-04	m/s	Aquifer permeability
k_b	lognormal	1.1E-06	1.1E-06	m/s	Blanket permeability
h_r	gumbel maxima	-8.45	0.30	m	Head at the source
ϕ_{po}	normal	8.7	0.1	m	Head at the polder
S	normal	22.86	2.28	m	Distance entry point-well
i_c	lognormal	0.7	0.1	[-]	Critical hydraulic gradient
H_e	lognormal	0.05	0.05	m	Hydraulic losses (filter)
C	normal	125	10	[-]	Hanzen & Williams coefficient
r_w	deterministic	0.15	0	m	Well radius



(a) (b)

Figure 4.6: Ilustration example of probability failure and reliability index for relief wells systems.

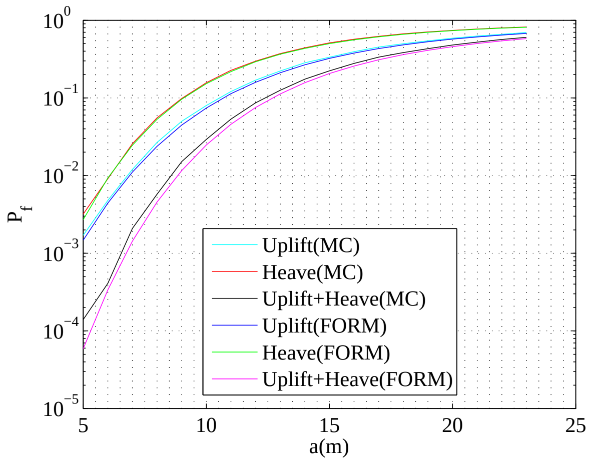


Figure 4.7: Probability of failure vs well spacing.

Figure 4.7 shows the probability of failure in linear scale. In order to be able to compare both methods for lower probabilities, in Figure 4.7 the same results are plotted in a semi-logarithmic scale (vertical axis in logarithmic scale) and in Figure 4.6b, in terms of the reliability index. A relative small difference between the two methods can be observed. Given these satisfactory results and based on the findings and previous researches [Jongejan et al.,2013, Schweckendiek,2013], from this point on the report, only FORM is used to perform the complete probabilistic design.

The results are presented on Figure 4.6. The fragility curve shown on Figure 4.6a have been cut at $a = 23$ for sake of simplicity (the attention of this investigation is in the lower probabilities of failures). One inconvenient when finding the fragility curve of a relief wells system, is that the domain (values of a) can be as large as four times the aquifer's thickness (see section 3.3). This leads to an extensive computation time. For this motive is preferred to carry out the probability analysis performing FORM. On Figure 4.6 the results obtained with FORM and MCS are shown.

4.6.2. PROBABILISTIC EXAMPLE OF PIPING BERMS

As discussed in section 3.6, for piping berms to fail, piping should occur at dikes inland toe, or at the end of the berm. This could be resemble with a serial system, as the fault tree shown on Figure 4.8. In this report is assumed that both elements (dike toe and berm end) are fully dependent; the failure of one element implies also the failure of other element. For this case, the probability of failure of the system is given by

$$P_f = max(P(E_1), P(E_2)) \tag{4.14}$$

As for relief wells, the method applied to execute probabilistic analysis for piping berms was FORM; relying

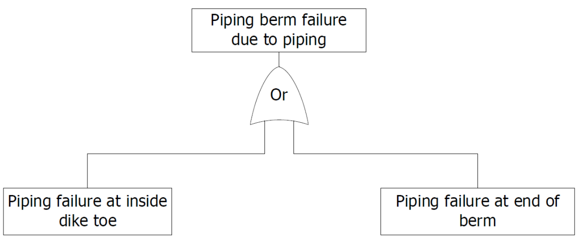


Figure 4.8: Serial system for piping berm failure due to piping.

on the research done by [Schweckendiek,2013] where the same type computations (LSF) were executed and the performance of FORM and Hohenbichler technique has been proven. For this example of probabilistic analysis for piping berms, a different example than the one for relief wells has been used. The reason why, is due to the small hinterland distance (X_3), from the example for relief wells, which will limit the length of the piping berm. The data used in this example is shown on table 4.3. The purpose of this example is to show

Table 4.3: Data used for piping berm design, dike ring 36.

Symbol	Distribution Type	Mean	SD	Unit
		μ_x	σ_x	
γ_{cover}	normal	16	1.6	kN/m^3
γ_{water}	deterministic	10	0	kN/m^3
γ_{berm}	normal	18	1.8	kN/m^3
d	lognormal	3	0.15	m
h_r	gumbel maxima	-3.793	0.304	m
h_p	normal	4.3	0.25	m
S	normal	28.5	3.42	m
k_f	lognormal	1.74E-04	3.29E-04	m/s
k_b	lognormal	1.16E-06	1.16E-06	m/s
D	normal	26.3	5.05	m
i_c	lognormal	0.7	0.1	[-]

how the reliability index vary in function of berm thickness (d_{berm})and berm length (L_{berm}). The results are the ones shown on Figure 4.9a, and Figure 4.9b, where the results obtained for the failures modes of uplift and heave, as well as the combine failure mode (H+U) are plotted in terms of the reliability index. The depen-

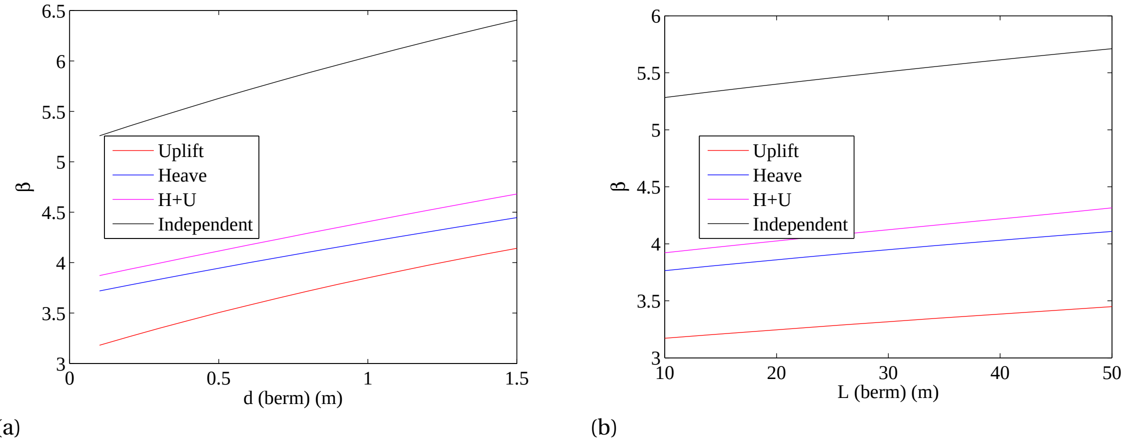


Figure 4.9: Reliability index for piping berms. Figure(a) d_{berm}, and Figure (b)L_{berm}.

dence between uplift and heave can be observed by comparing the combine probability of failure computed using Hohenbichler , (H+U) in Figure 4.9), with the combined probability of failure assuming independence of the elements (Independent in Figure 4.9). Another observation is the mild slope of the reliability index, for both cases, berm thickness and berm length. This could have repercussions on the cost when high reliability target is demanded, due to the larger thickness and length required. Finally, the optimal design will correspond to the berm thickness and length, which fulfill the set reliability target.

4.7. PROBABILISTIC SENSITIVITY ANALYSIS

The sensitivity analysis is intended to provide an overview of the dominant uncertainties affecting the performance functions. During the development of the model for relief wells, a mechanical sensitivity analysis was performed (see Appendix Process to Design a Relief Well System) by changing each variable, and measuring the outcome variation. These realizations showed that "geometrical" variables were the ones that contribute the most. Nevertheless, these variables (i.e. seepage length, entrance losses, hydraulic head at polder side) are obtained from soil properties (i.e. soil permeability). This was evident when using the base variables and performing the reliability analysis. In figure 4.10 the sensitivity factors for the two cases base study are depicted. The sensitivity factors have been grouped by well penetration, and the results of two different D/a ratio for each case, are presented.

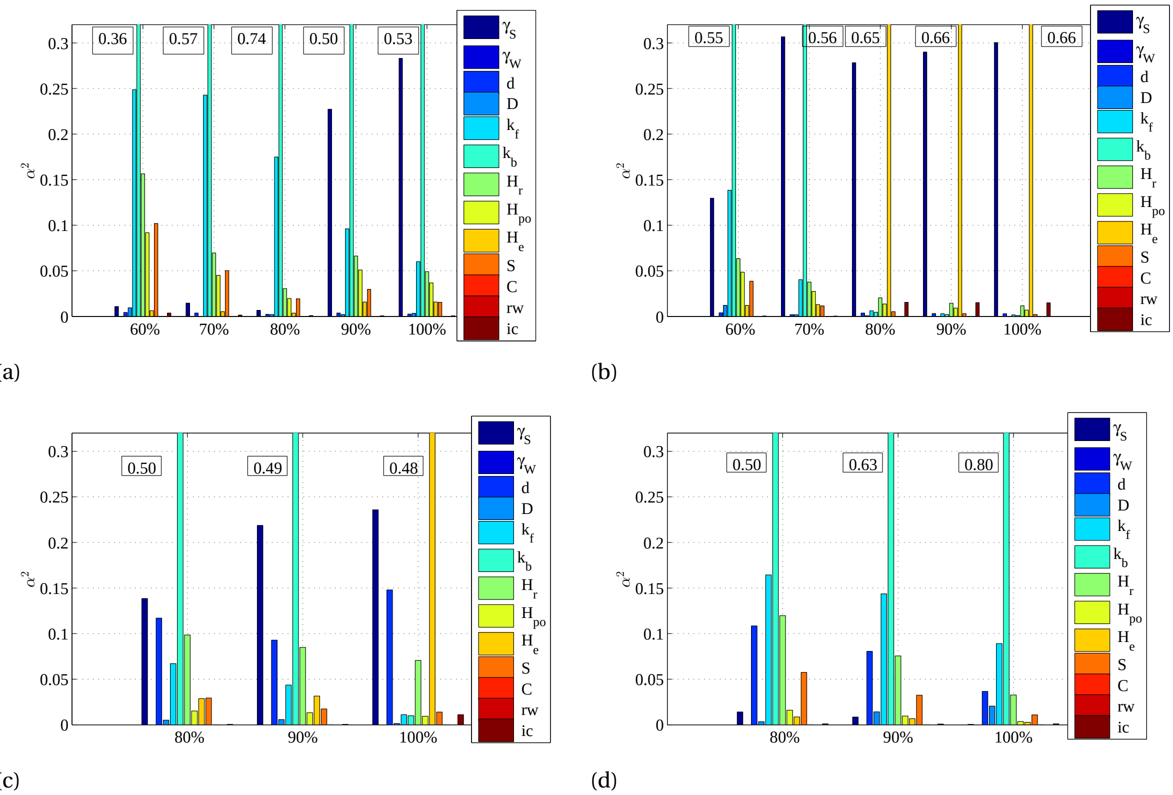


Figure 4.10: Sensitivity factors for the studied cases, grouped by well penetration. On the horizontal axis the values of well penetration are shown, and on the vertical axis the values of the sensitivity factors. The values on the boxes are the maximum values corresponding to the dominant uncertainty. Figure (a) dike ring 36, D/a =1.6, Figure (b) dike ring 36, D/a =2.6, Figure (c) dike ring 52, D/a =0.6, and Figure (d)dike ring 52, D/a =0.46.

It can be observed that there is a significant scatter among the sensitivity factors. This variance leads to the mean values (see Figure 4.11) to do not resemble any of the specific scenarios. Nevertheless some deductions can be drawn. In Figure 4.10, a) and d), corresponds to larger values of well spacing. For these cases, it can be observed that blanket permeability is the driven variable. For fully penetration, and smaller well spacing (Figure 4.10b, and Figure 4.10c), entrance losses are the driven variable. On the other hand, in case d), larger well spacing, blanket and aquifer permeability's are the driven variables despite well penetration. It can also be mentioned that in case of having small well spacing, Figure 4.10 b), the influence of the specific weight of the blanket is larger than when having larger well spacing.

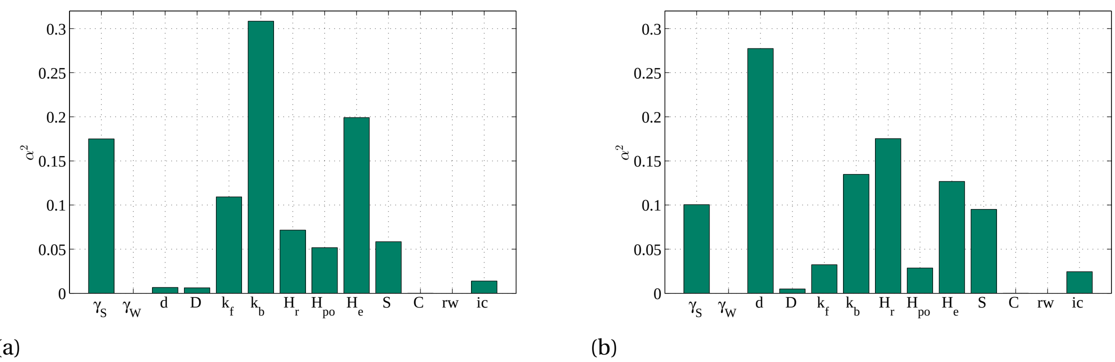


Figure 4.11: Average sensitivity factors. Figure (a) dike ring 36, Figure (b) dike ring 52.

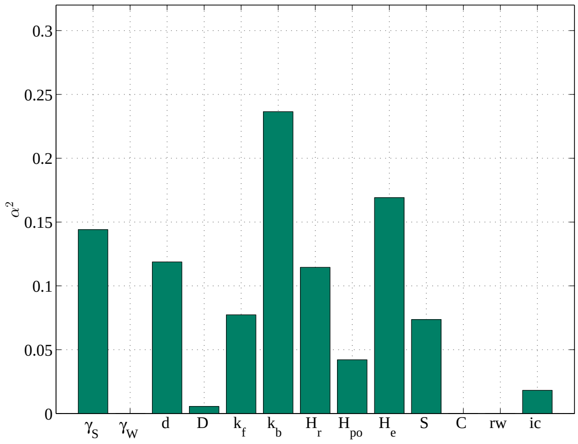
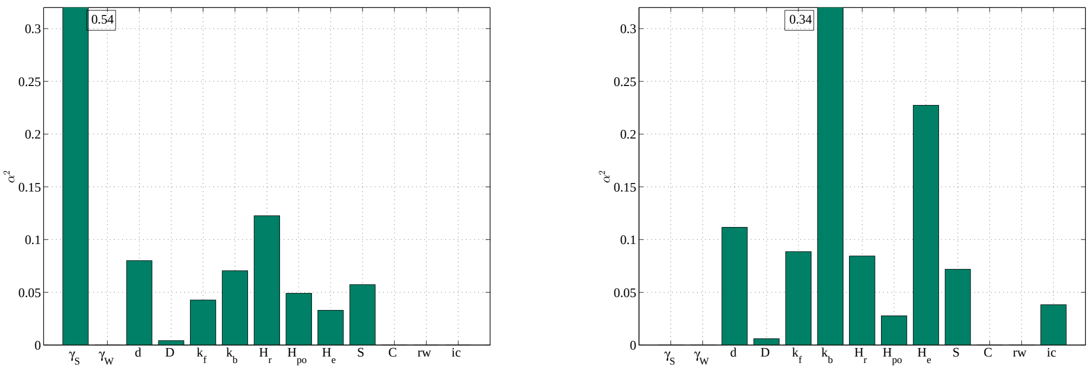


Figure 4.12: Sensitivity factors for piping average over all the studied cases.



(a)

(b)

Figure 4.13: Sensitivity factors average over all the cases. Figure (a) uplift, Figure (b) heave.

Contrary to what it was expected, even when they show relative importance, entrance losses do not stand over the other variables, but permeability does. This is due to the permeability of both, blanket and aquifer, are part of the formulae to determine the exit point and their large uncertainty (100%). In the case of uplift, the dominant uncertainty comes from the resistant part: the specific weigh of the blanket turns out to be the most important uncertainty, among all others (see Figure 4.13a). For the case of heave, blanket permeability is the dominant(see Figure 4.13b).

4.8. CONCLUSIONS OF PROBABILISTIC ANALYSIS

In this chapter the considerations, i.e. fault tree, limit state functions, statistical parameters of the involved variables and the methods for computing the probability of failure, have been presented. In addition a preliminary analysis using two examples (one for relief wells system and another one for piping berms) was performed. The purpose of this preliminary analysis was to be able to compare the computations methods (MCS and FORM). This helped to test the performance (compared to MCS) of FORM, which have shown good results. Given the good results of FORM, the probabilistic analysis of the case studies was performed using this technique.It is worth mentioning that describing piping failure as parallel system, consisting of the sub-mechanism of uplift and heave, allowed to evaluate relief wells as piping mitigation measure, which was not possible with the current evaluation methods. In the reliability system it can be observed how the reliability of the system (for combined failure mode, i.e.uplift and heave) is increased by defining piping failure as a parallel system. It should be also mentioned that the use of Hohenbichler [Hohenbichler et al.,1983] allowed taking into consideration the correlation between the two failure modes and do not overestimate the system reliability by assuming both failure modes independent. The results of the sensitivity analysis showed that there is an important variation in the influence of the variables in the reliability system. When averaging the influence factors, it was found that, from the load side, the dominant uncertainties are from the variables which defines the net seepage slope (k_b,H_e, H_r,k_f, h_{po} and S). From these variables k_b and H_e are the ones that showed higher influence. In the next chapter the probabilistic design for the case studies will be performed using the considerations described on this section.

5

Probabilistic Design

In chapter 1 the methodology of this research was described. It was stated how a RBD can lead to a more rational design by including all the uncertainties of the involved variables. In this chapter the probabilistic (reliability based) design of two case studies are shown. The purpose of this is to compare the two piping mitigation measures considered in this thesis, as well as to demonstrate how the proposed method performs. In the first part of this chapter is described how to obtain the reliability target for piping accordingly to the Dutch regulations. The definition of the optimal design as it is addressed on this research can be found in section 5.2. From the two case studies seven different scenarios (for each location) are reproduced in order to check the robustness of the results. Finally the data used as well as the results obtained are presented and discussed.

5.1. Safety Requirements

5.1.1. Dutch Regulations and Guidelines

In The Netherlands, the first measuring program for water levels date from the 17th century [Voortman,2003]. After the catastrophic flooding episode of 1953, the well known "Delta Works Committee" was form; as result, the new guide-lines for designing water defences were established on the "Delta Law" of 1956. This law was later replaced by the "Flood Defences Act" which was adopted in 1996. This document set the safety standards for primary flood defences, in terms of return periods of the design load events (see Figure 1.1).
In the Netherlands target reliability are based on the probability of exceedance of the loads for the whole system, taking into account all its failure mechanisms. In order to have the target reliability for piping, the target reliability of the system has to be broken down in to the different failure mechanisms.

5.1.2. Length Effect

Dikes usually are long structures that can be influenced by spatial variations. This spatial variation is considered in the so called length effect. Length effect can be define as the increase of the failure probability with the length of the dike due to imperfect correlations and/or independence between different cross sections and/or elements [Schweckendiek et al.,2012b]. In other words: the reliability of a system decreases with increasing its length.

5.1.3. Reliability Target for Piping

The current law for designing and assessing primary flood defences is based on a probability of exceedance of the load event. On the other hand, the present tendency is to move towards a RBD as it has been stated on the available reports of the VNK 2 [Jongejan et al.,2011, Jongejan et al.,2013]. In order to be able to perform RBD, the reliability target has to be defined. Deltares, on its report (SBW Hervalidatie piping), arrived to this formulation for translating dike ring related to safety requirements ("dijkringnorm") into local safety requirements for dike cross section for piping and uplift respectively.

$$P_{adm,loc} = \frac{0.1 P_{adm,ring}}{1 + \frac{\alpha}{l_{eq}} * L_{dr,s}} \tag{5.1}$$

Where

$P_{adm,loc}$ Local admissible failure probability

$P_{adm,ring}$ Admissible failure probability for the ring

α_c Calibration factor sub c added to difference from α Shape coefficient

l_{eq} Correlation length of the limit state function for piping

$L_{dr,s}$ Piping or uplift sensitive part of the dike ring under consideration

Table 5.1: Calibrated factor $\frac{\alpha}{l_{eq}}$ for piping and uplift (Length Effect).

	$\frac{\alpha}{l_{eq}}$
Piping	0.0028
Uplift	0.0045

5.2. DEFINITION OF OPTIMAL, RELIABILITY BASED

To be able to take a decision between alternatives it is imperative to define optimal criteria. In this report reliability-based design is addressed, and the optimal criteria adopted was the economic one, as it is described on 4.2, with the slightly difference that the considered costs are the construction cost of the alternatives, instead of the risk costs (defined as: probability of failure times consequences). In summary, the optimal alternative is the one that, while applying the minimum costs, achieves the maximum reliability. It can be formulated as follows:

$$minimize C = f(W/D, a) \tag{5.2}$$

subject to:

$$P\left(Z(\underline{X}|W, a) < 0\right) < \Phi\left(-\beta_t\right) \tag{5.3}$$

Herein C is the cost, and β is the reliability target.

5.3. PROBABILISTIC DESIGN FOR RELIEF WELLS

In the probabilistic analysis, the reliability of relief wells system was studied by looking into its probability of failure, where the only variable was the well spacing (a). In order to generate alternatives, a greater number of computations were carried out; this time not only well spacing, but also well penetration, changed. At the end, to visualize the results, they were plotted as contour lines with the same reliability target. Afterwards the cost analysis among them was performed, choosing the alternative with the lowest cost and the maximum reliability index. To design relief wells system, as mentioned on chapter 3 there are several combinations of well spacing and well penetration that could fulfill the design requirements. Acknowledging this fact, the costs of those combinations were analyzed and compared. In case of choosing small well penetration, well spacing has to decrease. This dense configuration of wells will lead to a large number of wells, and will increase the cost of the alternative. On the other hand, even when a large penetration could lead to an increase of the well spacing, it will also increase drilling and piping riser costs. As an example, the results from the reliability analysis from cross section ring 36, case 1-7 (detail in 5.5), are presented in Figure 5.1. The reliability index, as explained before, is plotted as contour lines among the possible combinations of well spacing (a) and well penetration (W/D), delimiting zones with equal reliability target. As it can be predicted, the zones with higher reliability index are located on the areas of large well penetration and small well spacing. For this example it can be seen that, in case of a target reliability higher than 4.5, almost fully penetration is needed; nevertheless,

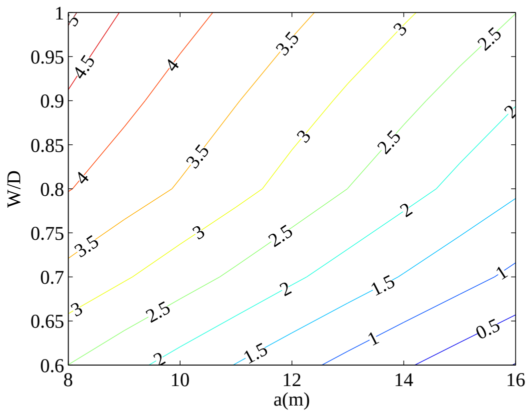
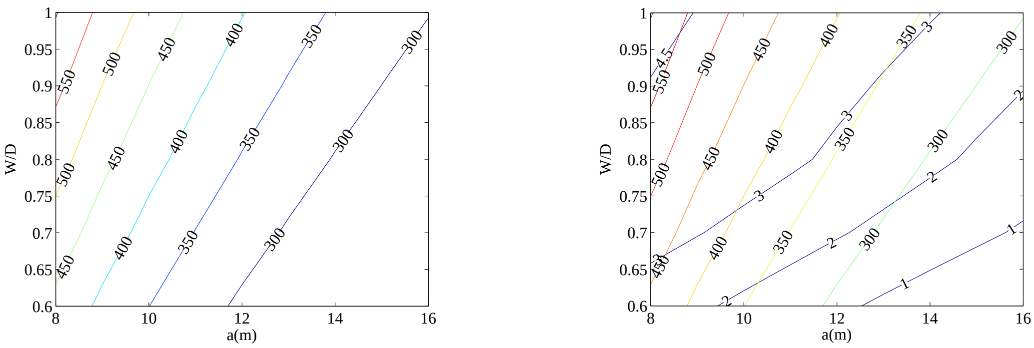


Figure 5.1: Reliability analysis case 1-7. See Table 5.7.

the cost analysis have to be performed in order to find the optimal well spacing and well penetration. The cost estimation was performed in accordance to the equations described before (refer to 3.5), assuming well penetration and well spacing as deterministic variables. The uncertainty of the cost is addressed on the life cycle of the infrastructure. In Figure 5.2a the estimated costs are shown as contour lines, in analogy with the reliability index, in order to visualize the optimal solution. The optimal solution can be found by integrating the two figures (see Figure 5.2b). A more detailed explanation of the decision-taking process will be addressed in section 5.5.



(a) (b)

Figure 5.2: Estimated cost (Figure a) and reliability index (Figure b) as contour lines for different combinations of well penetration (W/D) and well spacing (a), case 1-7. See Table 5.7.

5.4. PROBABILISTIC DESIGN PIPING BERMS

For piping berms design, the proper solution is an alternative in which the thickness and the length of the piping berm ensure the low probability of failure at the potential exit point, and at the end of the piping berm, respectively. In order to perform the probabilistic design, piping berm was modeled as a serial system with two failures modes; one at the toe (potential exit point) and the other one at the end of the piping berm. In case of piping berms, the reliability index increases by increasing the dimensions of the berm (thickness and length see Figure 4.9) which results in a monotonically increasing cost function (of the reliability,see Figure 5.3). This simplifies the optimization problem, leading the optimal to be the berm dimensions with the minimum required reliability target

5.5. APPLICATION EXAMPLES

Several cases were analysed with data acquired from the VNK project from Deltares. In this report, for illustration purposes, the cross section number 36001091 (Deltares data base), corresponding to ring 36, and cross

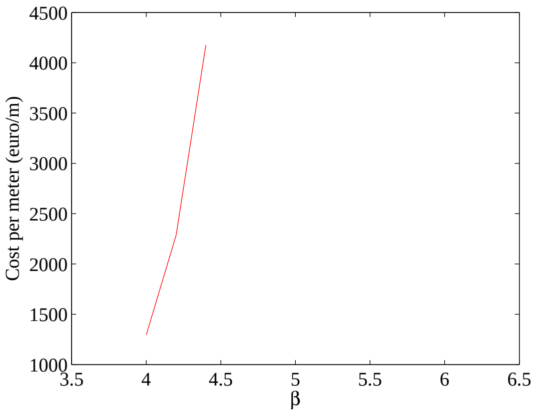


Figure 5.3: Ilustration example cost function piping berms.

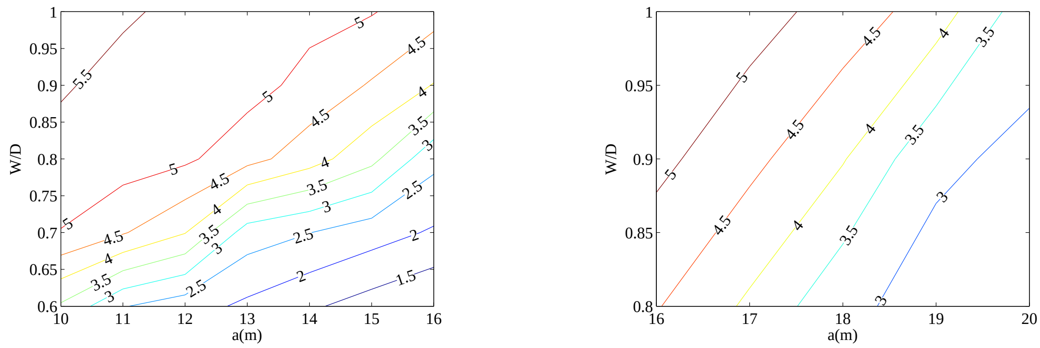
section 52001004 corresponding to ring 52 are used. Some of the cases resulted not to be piping sensitive, and in other cases the reliability target was not achieved given the max possible head reduction with relief wells. The rest of results are attached in Appendix Results Case Studies. The data used is shown on Table 5.2.On the data tables, the parameters for the head at the river (h_r) correspond to μ and β of the Gumbel distribution. Using the formulae described on 5.1.3, and assuming a 10 km dike to be piping sensitive, it was

Table 5.2: Data used for relief wells and piping berm design. Table (a) for dike ring 36, and Table (b) for dike ring 52.

		Dike ring 36		Dike ring 52			
Symbol	**Distribution Type**	**Mean** μ_x	**SD** σ_x	**Mean** μ_x	**SD** σ_x	**Unit**	**Description**
γ_{cover}	normal	16	1.6	17.1	1.7	kN/m^3	Specific weight cover layer
γ_w	deterministic	10	0	10	0	kN/m^3	Specific weight water
γ_{berm}	normal	18	1.8	18	1.8	kN/m^3	Specific weight berm
d	lognormal	3	0.15	4.11	1.23	m	Blanket thickness
D	normal	26.3	5.05	9.3	3	m	Aquifer thickness
k_f	lognormal	1.7E-04	3.2E-04	5.7E-04	7.5E-04	m/s	Aquifer permeability
k_b	lognormal	1.1E-06	1.1E-06	1.1E-06	1.1E-06	m/s	Blanket permeability
h_r	gumbel maxima	-3.79	0.30	-8.45	0.30	m	Head at the source
ϕ_{po}	normal	4.3	0.25	8.7	0.1	m	Head at the polder
S	normal	28.5	3.42	22.86	2.28	m	Distance entry point-well
i_c	lognormal	0.7	0.1	0.7	0.1	[-]	Critical hydraulic gradient
H_e	lognormal	0.05	0.05	0.05	0.05	m	Hydraulic losses (filter)
C	normal	125	10	125	10	[-]	Hanzen & Williams coefficient
r_w	deterministic	0.15	0	0.15	0	m	Well radius

found $\beta_{target} = 4.5$. In order to estimate well losses, Eq.3.12 should be applied to estimate the well discharge, and considering that value, refer to Figure 3.13. As at first hand the values of well spacing and net seepage slope are unknown, a good approximation is to assume a maximum hydraulic slope ($\Delta H/S$) and estimate a well spacing. This can be an iterative process but for general terms (for the examples that have been reviewed on this report) the well discharge is around 3 gpm per foot of well screen. For this value on Figure 3.13 the entrance losses can be estimated to be around 0.05 m (0.15 ft). In order to investigated its influence (of well losses), in the cost analisys, also a maximum value of 0.5 meters was addressed, but for this (maximum entrance losses of 0.5 m) case a covariance of 10 % is used; this reduction on the covariance is to avoid the well losses to take value larger than the actual hydraulic head (head difference for this case is 1.16 meters). Once the data and the reliability target have been set, the probabilistic analysis was performed for different scenario with well penetration between 60% and 100% (for practical reasons). In figure 5.4 the results of the reliability analysis for both case studies are shown. There it can be depicted the reliability index, as contour lines, for each combination of well penetration and well spacing.

For sake of simplicity, the optimal is searched among discrete values of well penetration and well spacing, comparing its construction cost (initial investment). In Table 5.3 the result are shown. From the Table 5.3 can



(a) (b)

Figure 5.4: Reliability analysis for different combinations of well spacing and well penetration. Figure (a)case 1-1, and Figure (b) case 2-1.

be seen that the alternative with lowest cost corresponds to a well spacing of 16 meters with fully penetration, at an initial investment of 347.79 €/m. In the same manner, the probabilistic design for piping berms was performed. It was found: $\beta = 4.5$; $L_{berm} = 50m$; $d_{berm} = 1.4m$; $Cost_{berm} = 4327.7$ €/m. In this case the land value was assumed as rural area. It is worth mentioning that in case of principal flood defences it is not required to pay land acquisition; nevertheless, in this section it was assumed a land value in order to account the added value to real states. In the following section an analysis without considering this land cost is addressed.

Table 5.3: Combination that fulfill safety requirements dike ring 36. Table (a) reliability index, and Table (b) cost.

(a)

	Reliability Index						
a (m)	10	11	12	13	14	15	16
W/D							
70%	4.98	4.54					
80%	5.41	5.25	5.10	4.68			
90%	5.53	5.42	5.28	5.19	4.84		
100%	5.63	5.53	5.44	5.30	5.16	5.03	4.7

(b)

	Cost (€/m)						
a (m)	10	11	12	13	14	15	16
W/D							
70%	453	412					
80%	540	443	406	375			
90%	522	474	435	401	372		
100%	556	505	463	428	397	370	347

Table 5.4: Combination that fulfill safety requirements dike ring 52.

a (m)	**Reliability Index**			**a (m)**	**Cost ($/m)**		
W/D							
	16	17	18	**W/D**	16	17	18
80%	4.53			80%	169.33		
90%	5.13	4.63		90%	176.93	166.52	
100%	5.04	5.21	4.79	100%	184.52	173.66	164.02

5.6. COMPARISION OF LIFE CYCLE COST

In section 3.8 was stated the method used to analyze the LCC for relief wells and piping berm. There was depicted (with the illustration example) that relief wells show better results concerning LCC despite the value of the discount rate. Nevertheless they are highly sensitive to their life cycle. This section will be addressed in such a way, to determine which could be the shorter life cycle that could be assigned to relief wells in order to equate the same LCC for piping berms. For illustration purpose the results, from case 1-3 and 1-7 taken from section 5.7 will be used. Case 1-3 corresponds to the cheapest alternative for piping berms, it was found:

$Cost_{berm} = 913.85$ €/m, when:

$$\beta = 4.5$$

$$L_{berm} = 35m$$

$$d_{berm} = 1m$$

For the same case (1-3), for relief wells was found:$Cost_{wells-system} = 281.36$ €/m, when:

$$\frac{W}{D} = 100\%$$

$$a = 16m$$

Using the LCC considerations, the results shown in Table 5.5 were found.

Table 5.5: Life cycle cost analysis for relief wells system(case 1-3).

Life cycle	5 years	10 years	13 years
NPV	2217.42	1177.06	946.11

It can be depicted, that for the cheapest case of piping berms (no land value assumed), the alternative for relief well allows changing the relief well system every 13 years in order to equate the LCC of the piping berm. In case of assuming extreme well losses (0.5 m), the following results for case (1-7), for relief wells it was found: $Cost_{wells-system} = 695.59$ €/m, when:

$$\frac{W}{D} = 100\%$$

$$a = 8m$$

For piping berm it was found: $Cost_{berm} = 1827.70$ €/m

$$\beta = 4.5$$

$$L_{berm} = 50m$$

$$d_{berm} = 1.4m$$

Table 5.6: Life cycle cost relief wells (case 1-7).

Life cycle	5 years	10 years	17 years
NPV	5481.99	2909.99	1865.63

In table 5.6 the results of the LCC for the most expensive alternative for relief wells system, assuming a life cycle of 5, 10 and 17 years are shown. It can be denoted that relief wells could be replace every 17 years in order to equate the same LCC for piping berms. It can be easily predicted that for cases where land acquisition (payment) is required, relief wells will score better. In the following section a cost sensitivity analysis is perform and the minimum life cycle which relief wells should accomplish to match the LCC of the alternative using piping berms is shown in Table 5.7.

5.7. COST SENSITIVITY ANALYSIS

In section 4.7 was depicted the most relevant uncertainties affecting the model's outcome. In this section, an analysis in order to see how these variables influence the cost is performed.In this analysis the cross sections described on section 5.5 (table 5.2) are be addressed. Three main variables are considered in this sensitivity analysis: i) Aquifer's thickness (D):given that this is the main variable in the cost function for relief wells (W is a fraction of D); ii) Aquifer's permeability(k_f):given its influence (see section 4.7), and that it is one of the main variables in order to estimate the well discharge; iii) Entrance losses (H_e):given that is the variable with the second highest sensitivity factor (see section 4.7). For the case of assuming land as urban area, a value of 100 €/m^2 was assumed, as mentioned on 3.7. The used data, as the results for two ring cases (36 and 52), are shown on Table 5.7. Is it important to mention that, on this table, all case number 1 refer to ring 36, and the number 2 refers to ring 52. The second number, for referring the cases studies, denotes the specific scenario according to the following categorization; sub case 1: base case study; in sub cases 2 and 3, the mean value of aquifers' thickness is changed; sub-cases 4 and 5 the variance of aquifers permeability is changed; sub-case 6 the covariance of the entrance losses is reduced finally in sub-case 7: the mean values of the entrance losses is increased (by a factor of 10) and its covariance is reduced to 10%. From this analysis

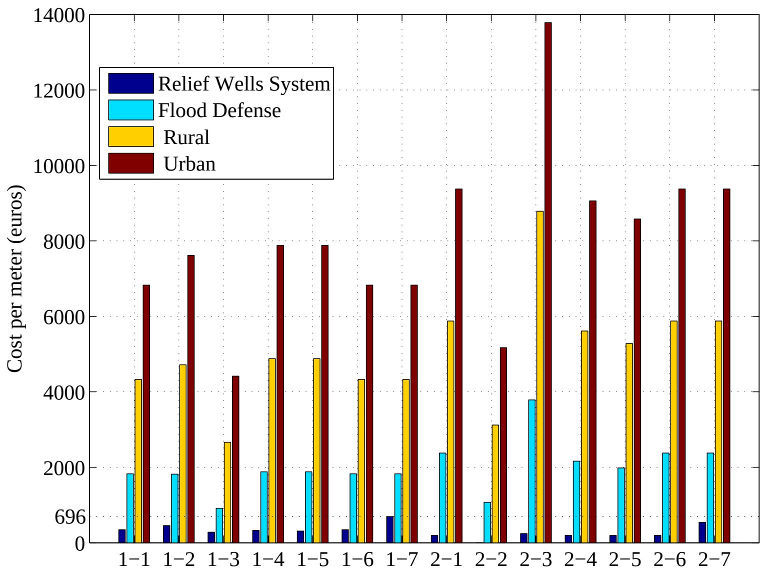


Figure 5.5: Results of the cost sensitivity analysis.

it can be seen that both alternatives (piping berms and relief wells) show better results in case of shallow aquifer. It is also important to notice that even without assigning land value , relief wells perform better. From the results, it can be seen that in the case of changing aquifer's permeability, there is a small variance on the cost. These influences decrease with the decrease of the thickness, seen on Table 5.7, cases 1-4, 1-5, 2-3 and 2-4. In case of ring 52 (refer to Table 5.7), cases 2-3, and 2-4, the influence in the cost is negligent; one of the reasons why this occurs is due to the solution is searched among discrete values of well penetration and well spacing. In this case, its influence can be depicted on the reliability index; even though the cost is the same when decreasing permeability's uncertainty, the reliability index is slightly increased (from 4.7 to 4.8). Nevertheless, this small increment of the reliability index translates into a reduction of 40% of the probability of failure ($\beta = 4.7$, $pf = 1.3E-6$, and with $\beta = 4.8$ $pf = 7.93E-7$). The reason why there is a small influence of the aquifer permeability, in the case of small well sapcing and large well penetration, can be explained by looking at the equation given by USACE to evaluate the occurring potential in relief wells system. In order to visualize this, the LSF (for uplift for relief wells) can be rewritten as (see Eq.3.11, Eq.3.17, and Eq.4.10 for reference):

$$Z_u = \frac{d*(\gamma_s-\gamma_w)}{\gamma_w} - a*\Delta M*\theta_a - H_w \tag{5.4}$$

As showed on chapter 3, wells factor for fully penetrated wells are constant, therefore the LSF becomes only dependent on well spacing, seepage slope and well losses (from the load part). Well spacing in this analysis

Table 5.7: Results case study.

Case	Parameters				Relief well System					Piping berms				Minimum Life cycle				
	μD	Cov k_f	μH_e	Cov H_e	β	W/D	a(m)	Cost per Well €	Cost per meter	d	L	β	Flood defense	Rural	Urban	years	$Q_w(m^3/day)$	$Q_w(m^3/day/m)$
1-1	26.3	189%	0.05	100%	4.65	100%	16	5564.69	347.79	1.4	50	4.5	1827.7	4327.7	6827.7	8	11.21	1.64
1-2	40	189%	0.05	100%	4.6	90%	15	6831.7	455.45	1.2	58	4.5	1817.26	4717.26	7617.26	10	16.67	2.67
1-3	13	189%	0.05	100%	4.9	90%	13	3657.63	281.36	1	35	4.5	913.85	2663.85	4413.85	13	4.50	1.00
1-4	26.3	75%	0.05	100%	4.55	90%	16	5221.16	326.32	1.2	60	4.55	1879.92	4879.92	7879.92	8	22.49	1.64
1-5	26.3	10%	0.05	100%	4.68	100%	18	5564.69	309.15	1.2	60	4.68	1879.92	4879.92	7879.92	7	15.62	1.46
1-6	26.3	189%	0.05	20%	4.7	100%	16	5564.69	347.79	1.4	50	4.5	1827.7	4327.7	6827.7	8	8.44	1.64
1-7	26.3	189%	0.5	10%	5.1	100%	8	5564.69	695.59	1.4	50	4.5	1827.7	4327.7	6827.7	17	16.98	3.29
2-1	9.3	130%	0.05	100%	4.7	100%	18	3489.13	193.84	1.3	70	4.5	2376.01	5876.01	9376.01	3	17.32	0.52
2-2	4	130%	0.05	100%	NA	NA	NA	NA	NA	1	41	4.6	1070.51	3120.51	5170.51	NA	NA	NA
2-3	18	130%	0.05	100%	4.65	100%	19	4625.53	243.45	1.45	100	4.5	3785.95	8785.95	13785.95	3	33.19	0.95
2-4	9.3	75%	0.05	100%	4.7	100%	18	3489.13	193.84	1.2	69	4.5	2161.91	5611.91	9061.91	3	19.37	0.52
2-5	9.3	10%	0.05	100%	4.8	100%	18	3489.13	193.84	1.15	66	4.5	1981.75	5281.75	8581.75	4	7.34	0.52
2-6	9.3	130%	0.05	20%	4.8	100%	18	3489.13	193.84	1.3	70	4.5	2376.01	5876.01	9376.01	3	34.20	0.52
2-7	9.3	130%	0.5	10%	4.5	80%	6	3246.18	541.03	1.3	70	4.5	2376.01	5876.01	9376.01	10	88.13	1.55

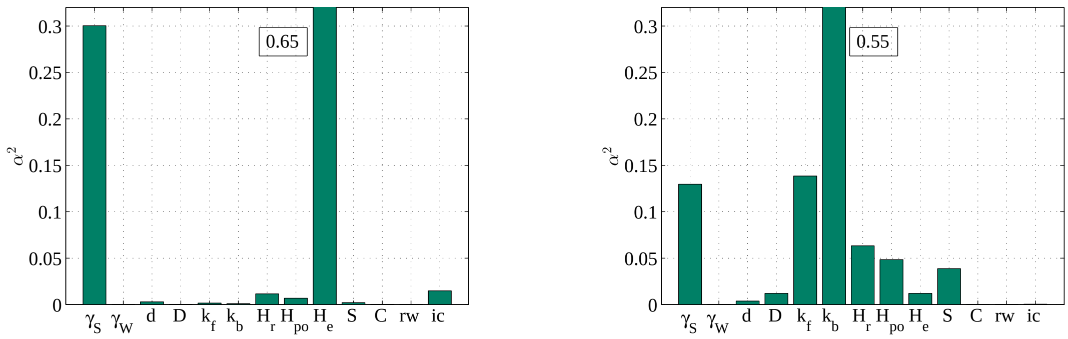
is treated as deterministic variable; then the only variable that involves uncertainties, besides the one from entrance losses from the load part, is the net seepage slope, showed on chapter 3, Eq.3.13; it is formulated as follows:

$$\Delta M \quad = \quad \frac{h_r - h_{po} - H_{av}}{S} - \frac{H_{av}}{X_3} \tag{5.5}$$

Where

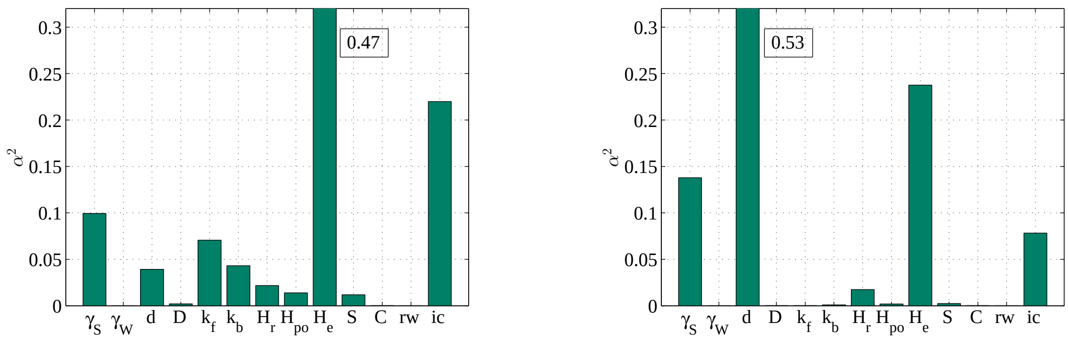
$$X_3 = \sqrt{\frac{k_z d_{bL} D}{k_b}}$$

From here, it can be seen the small influence in the formulation of the permeability, which is consistent with the sensitivity factors for the case of fully penetration (see Figure 5.6a). On the other hand, when wells are partially penetrated, wells factors prove to be sensitive to well spacing, which in turns influence the discharge, which is largely influenced by the aquifer's permeability.This causes a redistribution on the contribution of the involved variables, as is showed in Figure 5.6b. In the case of ring 36, influence of the permeability is reflected on the cost; anyhow, it is still a small variance (12% of the cost per meter is reduced, see cases 1-4 and 1-1). The reason why in the case of ring 36 the influence in the cost can be observed, is due to the aquifer's thickness is larger than the one in case of ring 52 (almost three times bigger).



(a) (b)

Figure 5.6: Sensitivity factors for piping (combined failure mode of uplift and heave). Figure (a) for fully penetrated wells case 1-1(D/a = 2.6), and Figure (b) partially penetrated wells case 1-1(D/a = 2.6, W/D = 60%).



(a) (b)

Figure 5.7: Sensitivity factors for piping (combined failure mode of uplift and heave). Figure (a) case 1-7, W/D = 100%, D/a = 2.6, and Figure (b) case 2-7, W/D = 80%, D/a = 1.6 .

Regarding entrance losses, from the sensitivity factors for fully penetrated wells (see Figure 5.6a), its large influence can be predicted. In Table 5.7 case 1-7, when the mean value of the hydraulic losses is increased to approx. 50% of the head at the well, a much conservative design is required. This leads to a reduction of almost 50% of well spacing, in case 1-7, and one third in case 2-7. The sensitivity factors found for case 2-7 are shown in Figure 5.7b. In Figure 5.5 the costs for each scenario are shown. Concerning the economic aspects it can be seen that relief wells perform better. Both alternatives showed better performance (economically speaking) in case of having thin aquifer. However, the ratio of cost per meter between relief wells and piping berms, is in the order of 10 for cases with thin aquifer (dike ring 52), and in the order of 6 for a median aquifer thickness (case of ring 36). Finally, a computation is done with the purpose of estimate the minimum life cycle of relief wells, in order to compete with piping berms LCC; the results are shown in Table 5.7. It can be noticed that in all cases for relief wells, the minimum life cycle of wells is smaller than the assumed (20 years). Though, in the case of assuming extreme high well losses, the life cycle approaches to the assumed one.

5.8. CONCLUSIONS OF PROBABILISTIC DESIGN

In this chapter two case studies, taken from VNK2 project data base were analyzed. The results showed that, when applicable, relief wells systems represent an interesting solution as piping mitigation measure. These examples were chosen in such a way that relief wells system could fulfil the target reliability. During the stage of testing this method, different cross sections were analyzed. It was found that sometimes the reliability target was not achieved; for this reason such sections were disregarded. This is mentioned with the intention of highlight that relief wells are not always applicable. Nevertheless for the two selected case studies, it was found that relief wells are a cheaper alternative than piping berms. In order to test the robustness of the results a sensitivity analysis was performed. It was found that in all the depicted scenarios, relief wells performed better despite the land value. It was also found that, for the selected case studies, almost full penetration is required to achieve the desirable reliability target. In addition to this, the well discharge was computed which for these cases turns out to be relative small. It is important to mention that the computed discharge does not account for the seepage in the hinterland blanket. Finally a backward computation was performed in order to compute the minimum life cycle that relief wells system should accomplish in order to equate the net present value of the piping berms. It was found that in average, over the case studies, the relief wells system could be replace roughly every seven years in order to have the same net present value than piping berms.

6

Conclusions and Recomendations

6.1. Introduction

In this thesis an approach to perform a probabilistic design of a relief wells systems has been presented. This approach has its grounds on the basis of reliability based design and the evaluation criteria for piping failure as envisaged in the upcoming safety assessment of flood defences in the Netherlands. This evaluation criterion assumes piping failure as a parallel system in which uplift and heave are the preconditions in order to develop piping. The chosen method to evaluate the occurring potential in a relief wells system was the USACE method, given its relative easy implementation in a computer code and the extensive documentation about the topic.The basic assumptions of this thesis are: (i) infinite line of wells, (ii) well spacing is treated as uniform and deterministic, (iii) well penetration is taken as a fraction of aquifer's thickness, related uncertainties are account for in the aquifer's thickness, (iv) steady flow, (v) piping occurs in a confined aquifer, (vi) the point where the groundwater pressure equals the atmospheric pressure is assumed to be located at the hinterland leakage length, (vii) hydraulic losses are assumed to be constant over the time. Besides achieving probabilistic design, the idea was to check if relief wells systems suppose an economic attractive solution compared to the current used alternative: piping berms. For such purpose a cost analysis was perform taking in to consideration the life cycle of each alternative. The results from the case studies showed that, when applicable (given their limitations), relief wells are economically competitive compared to piping berms. Findings and conclusions derived from this research have been already mentioned in the respective chapters. This chapter emphasizes the main conclusions and recommendations.

6.2. Final conclusions and recommendations

The goal of this thesis was to develop a method for reliability-based design of relief wells minimizing the life cycle cost. The applied USACE method allowed to estimate the piezometric head in relief wells systems, while the developments in flood risk and dike safety assessment in the Netherlands allowed to determine local safety requirements for a given dike cross section, considering two main aspects: the probability of flooding of a specific dike ring section, and its length effect. The main conclusions and recommendations found in this thesis are summarized below.

Relief wells can, in fact, be designed optimizing the life cycle cost accounting for the dominant uncertainties. To obtain an optimal design a cost analysis was performed, and in this analysis all possible combinations of well spacing (a) and well penetration (W/D), with reliability index lager or equal to the target reliability, were compared in terms of their cost. Also, from the sensitivity analysis the dominant uncertainties were depicted. From the case studies it was found that blanket permeability, aquifer permeability and entrance losses from the load part, and blanket thickness from the resistance part, were the dominant uncertainties. From the examples relief wells appear to be cheaper when considering short terms costs. On the other hand, for long term costs, relief wells suffer from their relatively short life cycle; nevertheless, they are still attractive if compared to piping berms, especially when land needs to be acquired for accommodate the reinforcement.

Since there exist possible causes of well efficiency loss, special attention was paid to this topic. It was found that although maximum clogging is likely to happen, total clogging is unlikely to occur (as detailed in 3.4). Most of the causes of losing wells' efficiency are due to backflooding of muddy water, vandalism, deformation of wells' screen, or ground movement. To avoid these sorts of problems, protective measures have been assumed (valve, metal guard, stainless rising pipe, refer to 3.5). Also, the fact that the wells filter is not working continuously will prevent wells from being clogged. In the probabilistic analysis of the relief wells, the head losses were assumed to be the maximum head losses at the end of the wells' design life time. From this analysis was found that the permeability of the aquifer and the blanket, play an important roll, as well losses, given that they have important influence on the piezometric head.

The life cycle cost analysis allowed to account for the differences in life cycle time between relief wells and piping berms, assuming a life cycle period for relief wells of 20 years, based on the data available for well losses (USACE observations), and assuming the maximum observed entrance losses and a coefficient of variation of 100%(conservative approach given the limited data). The net discount rate used was 2.5%. Using these assumptions it would be possible to replace the relief wells every 7 years (in case of not assuming land value), in order to equate the cost (Net Present Value) for piping berms.

Relief wells systems have limitations in terms of the maximum head reduction. In chapter 3.3 the limits of application were discussed. This could be enhanced by extracting water from the well actively (pumping) instead of using artesian passive wells as studied in this thesis, which would lead to an increase of the cost of the system. Hence, when large head reduction is required, which cannot be achieved with relief wells only, a combination of relief wells and piping berms may be considered as an attractive option; inclusion of relief wells will reduce the piezometric head, leading to a reduction of the berm length and height, reducing land acquisition.

Another potential drawback of relief wells is the amount of water they can catch. In order to perform a detailed analysis, the flow duration curve should be known to predict when and how much water will discharge the well. The cost of extracting this water has not been taken into account, assuming that the protected area has sufficient pumping station capacity, and that the extra cost is negligible, especially considering the low frequency of extremely high water levels. Although, it is recommended to study, in more detail, the well discharge and required pumping rates, considering the respective frequencies.

From the probabilistic analysis of piping as a parallel system, it can be seen how this approach can increase the estimation of the probability of failure. With the current model to assess piping [TAW,1999], this feature (parallel system) is under-estimated and could lead to wrongly classify as piping sensitive, sections where, for example, given their blanket thickness, heave is improbable [1].

Future research should entail the following topics, which could not be covered in this thesis extensively: (i) number of abandoned wells, (ii) model error: given the limited data available, no model error was assumed; the assumption of an arbitrary model error would increase uncertainties, (iii) possibility of adding more lines of wells, (iv) how relief wells systems affects macro stability, in order to obtain an integral design (if berms are required for macro stability, it can be sufficient to extend it), (v) time dependency, in this thesis, well hydraulic loses were addressed as constant over its life cycle.

An important remark to be made is the lack of Dutch guidelines to assess or design relief wells systems. By means of the assumptions and methods used in this thesis, it has been showed that relief wells, when applicable, are a potential economically attractive solution and an open opportunity for future research.

[1] Heave will be considered in the envisaged new safety assessment rules

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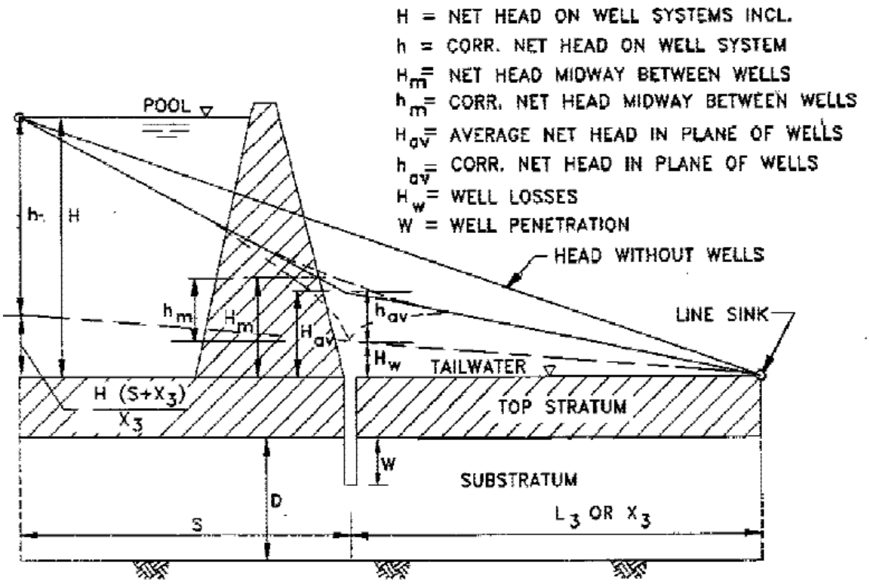
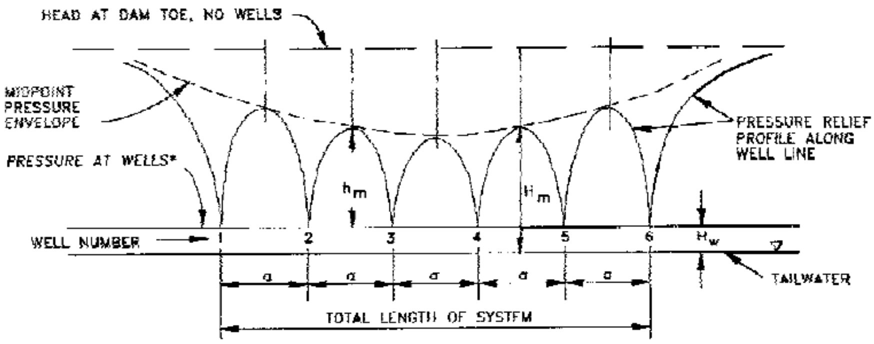
APPENDICES

Process to Design a Relief Well System

Method for evaluate Hav/Hm

U.S. Army Corps of Engineers, USACE (1992),





Basic equations:

$$h_w = H - \frac{Q_w}{2\pi * K_f * D} \ln\left(\frac{2S}{r_w}\right)$$

$$h_w = H - \frac{a * \Delta M *}{2\pi} \ln\left(\frac{2S}{r_w}\right)$$

For a relief well system (infinite line of wells):

$$Q_w = a * \Delta M * K_f * D$$

$$h_m = a * \Delta M * \theta_m$$

$$h_{av} = a * \Delta M * \theta_a$$

$$\theta_a = \frac{h_{av}}{a * \Delta M}$$

$$H_{av} = H_w + h_{av}$$

$$H_m = H_w + h_m$$

$$\Delta M = \frac{H - H_{av}}{S} - \frac{H_{av}}{x_3}$$

$$\Delta M = \frac{H - H_w - h_{av}}{S} - \frac{H_w + h_{av}}{x_3}$$

$$Q_w = \frac{h * k_f * D}{\frac{S}{a} + \left(\frac{S + X3}{X3}\right)^{\theta_a}}$$

$$h = H - H_w \left(\frac{S + x_3}{x_3}\right)$$

$a =$ distance between wells

$D =$ thickness of pervious foundation

$H =$ head at the source

$h_w =$ head at well

$Q_w =$ well discharge

$S =$ distance from real well to line source

$r_w =$ radius of well

$H_w =$ Well loses

$\Delta M =$ Net seepage gradient toward the well

H_{av} = average net head in plane of wells

H_m = Net head midway between wells

θ_a and θ_m are the well factors obtained from the following table

W/D	D/a	a/r_w	θ_a	θ_m	$\Delta\theta$
100%	All values	100	0.440	0.550	1.00
75%	0.25	100	0.523	0.633	0.489
	0.50		0.563	0.667	
	1.0		0.606	0.681	
	2.0		0.678	0.682	
	3.0		0.748	0.682	
	4.0		0.818	0.682	
50%	0.25	100	0.742	0.851	0.733
	0.40		0.857	0.955	
	1.0		0.983	1.012	
	2.0		1.175	1.024	
	3.0		1.361	1.024	
	4.0		1.547	1.024	
25%	0.25	100	1.225	1.335	1.466
	0.50		1.569	1.622	
	1.0		1.926	1.908	
	2.0		2.390	2.024	
	3.0		2.798	2.047	
	4.0		3.199	2.075	
15%	0.25	100	1.662	1.772	2.077
	0.50		2.310	2.401	
	1.0		2.970	2.938	
	2.0		3.747	3.293	
	4.0		4.941	3.432	
10%	0.25	100	1.908	2.018	3.298
	0.50		2.934	3.025	
	1.0		3.977	3.941	
	2.0		5.139	4.649	
	4.0		6.814	5.071	
5%	0.25	100	1.778	1.887	6.963
	0.50		3.879	3.969	
	1.0		6.063	6.021	
	2.0		8.377	7.864	
	4.0		11.144	9.283	

Process to design a relief wells

1) Define h allowable,

$$h_a = d * \frac{\gamma_{wetsoil} - \gamma_{water}}{\gamma_{water}}$$

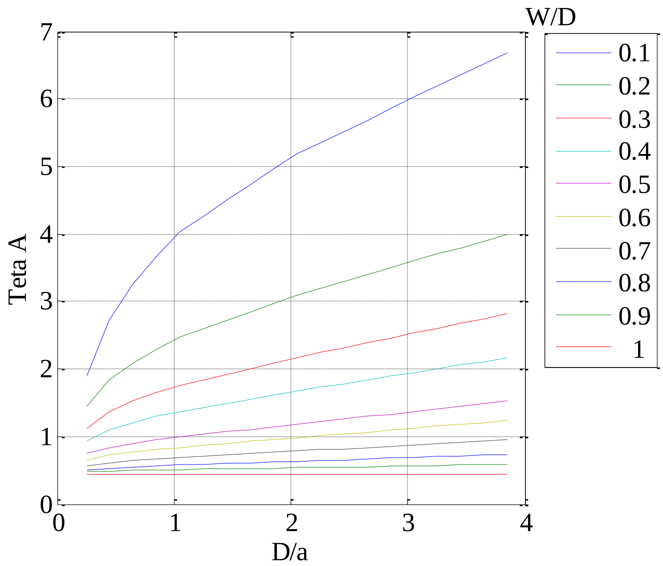
2) Set $\mathrm{H_{av}} = h_a$
3) Compute as; $\Delta\mathrm{M_3} = \frac{\mathrm{H - H_{av}}}{\mathrm{S}} - \frac{\mathrm{H_{av}}}{\mathrm{x_3}}$
4) Assume ***a*** and compute $\mathrm{Q_w} = \mathrm{a} * \Delta\mathrm{M} * \mathrm{K_D} * \mathrm{D}$
5) Compute frictional losses; applying Hazen–Williams equation;
   a. q = gal/min; $\mathrm{q}_{\mathrm{w}\left(gal/_{\mathrm{min}}\right)} = \mathrm{q}_{(m^3/s)} * 15850$
   b. $dh_{inch} = 2 * (rw - thickness)_m * 39.3701$
   c. C, varies (90-120)
   d. $f = 0.2083 * \left(\frac{100}{C}\right)^{1.852} * q^{1.852}/d_h^{4.8655}$, ft/100 ft
   e. Total frictional loses in meters; $Hf_m = f_{ft} * W_m * .01$

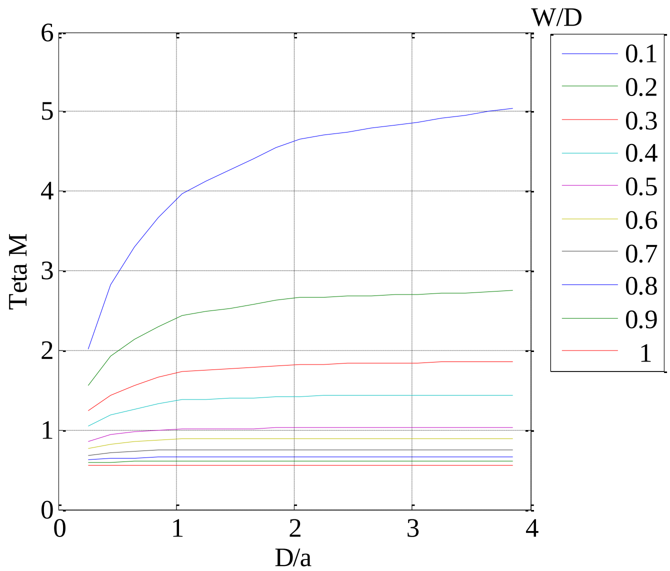
6) Compute Velocity losses
   a. $v = \frac{Q}{A} = \frac{Q}{\pi*\left(\frac{d_h}{2}\right)^2}$
   b. $H_v = \frac{V^2}{2*g}$
7) Well losses assume $H_e = 0.05$ m, $H_{w7} = H_e + H_v + H_f$
8) Compute $\mathrm{h_{av}} = \mathrm{H_{av2}} - \mathrm{H_{w7}}$
9) Compute $\theta_{\mathrm{a9}} = \frac{\mathrm{h_{av}}}{\mathrm{a4}*\Delta\mathrm{M3}}$
10) Compute $\theta_{\mathrm{a10}} = \frac{1}{2\pi} ln\left(\frac{a}{2*\pi*r_w}\right) + \Delta\theta * (log\ ^a/_{r_w} - 2)$, first term for a/rw=100
11) Change ***a₁₁*** till the values of θ_{a11} converge
12) Compute $\theta_{\mathrm{m12}} = \frac{1}{2\pi} ln\left(\frac{a}{\pi*r_w}\right) + \Delta\theta * (log\ ^a/_{r_w} - 2)$, first term for a/rw=100
13) If $\theta_\mathrm{a} > \theta_\mathrm{m}$ go back to 4.) with the value of $\boldsymbol{a_{11}}$)

If $\theta_\mathrm{a} < \theta_\mathrm{m}$

14) Compute $\mathrm{Q_{w14}} = \mathrm{a_{11}} * \Delta\mathrm{M_3} * \mathrm{K_D} * \mathrm{D}$
15) Compute Hw for $\mathrm{Q_{w14}}$
16) Assume $H_m = h_a$, and compute $\mathrm{h_m} = \mathrm{H_m} - \mathrm{H_w}$
17) Compute $\mathrm{h_{av17}} = \frac{\theta_{\mathrm{a11}}}{\theta_{\mathrm{m12}}}\mathrm{h_m}$
18) Compute $\mathrm{H_{av18}} = \mathrm{H_{w15}} + \mathrm{h_{av17}}$
19) $\Delta\mathrm{M_{19}} = \frac{\mathrm{H - H_{av18}}}{\mathrm{S}} - \frac{\mathrm{H_{av18}}}{\mathrm{x_3}}$
20) Compute $\theta_{\mathrm{m20}} = \frac{\mathrm{h_m}}{\mathrm{a}*\Delta\mathrm{M}}$
21) $\theta_{\mathrm{m21}} = \frac{1}{2\pi} ln\left(\frac{a}{\pi*r_w}\right) + \Delta\theta * (log\ ^a/_{r_w} - 2)$
22) Find $\boldsymbol{a_{22}}$ till the values of θ_{m22} converge

23) Compute $\theta_{a23} = \frac{1}{2\pi} ln\left(\frac{a}{2*\pi*r_w}\right) + \Delta\theta * (log\ ^{a22}/_{r_w} - 2)$
24) Go back to 14) using $\boldsymbol{a_{22}}$
25) Repeat till a22+1=a22
26) Repeat from 10 to 24 for various relationship W and a





Analitical deduction of range of validity of the USACE method.

From the tables for θ_a the minimum value of θ_a is 0,44;

$h_{av} = a * \Delta M * \theta_a$

$$\theta_a = \frac{h_{av}}{a * \Delta M}$$

$\theta_a > 0{,}44$

$$\frac{h_{av}}{a * \Delta M} > 0{,}44$$

$$a < \frac{h_{av}}{0{,}44 * \Delta M}$$

We also know, from by Barron (1982), that θ_a max occurs when $D/a > 4$. In order to θ_a to converge is required that $D/a < 4$;

$D/4 < a$

So therefore in order to θ_a to converge

$$D/4 < a < \frac{h_{av}}{0{,}44 * \Delta M}$$

Then a condition to converge is:

$$D/4 < \frac{h_{av}}{0{,}44 * \Delta M}$$

Substituting $h_{av;}$

$$D/4 < \frac{H_{av} - H_w}{0{,}44 * \Delta M}$$

From here it can be observed that if the losses or the net seepage slope are too big we won't find solution for our design.

Analogously there is an upper limit for θ_a for $D/a = 4$ which depends on well penetration, if we called X to the max value of θ_a for a given well penetration (W/D) and we make $a = D/4$;

$$\frac{h_{av}}{a * \Delta M} < X_{\left(\frac{W}{D}\right)}$$

$$\Delta M > \frac{4 * h_{av}}{X_{\left(\frac{W}{D}\right)} * D}$$

Preliminary Evaluation

Purpose of the meeting: Introduction of the project and its latest progress.

Problem description: Can relief wells be designed cost effectively as alternative piping mitigation measure?

Analyse:

- Relief wells systems reliability.
- How costly it would be to implement the needed measures in order to achieve the current reliability target for piping, as set in the Netherlands.
- Comparison with piping berms by means of a risk decision based analysis.

Main questions:

- How can piping safety/reliability be analysed for measures which included relief wells?
- Which are the dominant uncertainties involved?
- How can relief wells be designed optimizing the (life-cycle) cost accounting for the dominant uncertainties?
- How does relief wells compare to piping berms in terms of robustness and cost effectiveness?

Tasks:
Analysis of relief wells systems reliability

Method of analysis:
Probabilistic approach (by Monte Carlo simulation)

Probabilistic study:

The idea is to study the probability that the limit state function is smaller than zero (Zu<0). For this document only the failure due to uplift will be treated.

The Z function is defined by the resistance (Ru), represented by the effective stress at the bottom of the cover layer, and the solicitation (Su), represented by the average head of the well (Hav) or the average head between wells (Hm), the bigger one according to each case, for details see annex.

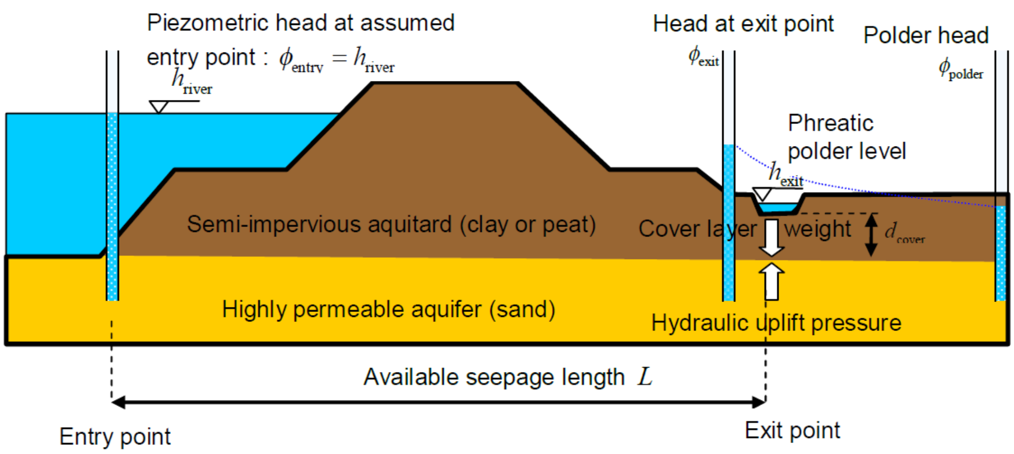


Figure 1 Definitions of geometrical properties, phreatic and piezometric levels for uplift, heave and piping

$$Z_u = \Delta\theta_{c,u} - \theta_{exit}$$
$$Z_u = R_u - S_u$$
$$R_u = \frac{\sigma_{eff}}{\gamma_{water}}$$

$$S_u = \mathrm{Max}(Hav, Hm)$$

Method to evaluate Hav,Hm: U.S. Army Corps of Engineers, USACE (1992), the objective of this design is to find the well spacing (a) and well penetration (w) in order to reduce the piezometric head to the desire, allowable head (ha).

Assumptions:

- Infinitive line of wells, impervious top stratus of finite length.
- Constant water level (steady state).
- Equal discharge and "losses" for all the wells.
- Homogenous soil.
- Entrance losses, head loss in the screen and filter, are assumed to be constant.
- Friction and velocity losses are computed using Hazen-Williams.

Using the computer code it was spotted that for some values the program was not giving any results. After debugging the program and making an analytical examination (see annex) by using the equations presented on the annex, it was found two conditions for the application of this method:

$$D/_4 < \frac{H_{av} - H_w}{0,44 * \Delta M}$$

$$\frac{4 * (H_{av} - H_w)}{11.144 * D} < \Delta M$$

There we can conclude;

$$\frac{4 * (H_{av} - H_w)}{11.144 * D} < \Delta M < \frac{4 * (H_{av} - H_w)}{0,44 * D}$$

$$0.3589 * \frac{(H_{av} - H_w)}{D} < \Delta M < 9.09 * \frac{(H_{av} - H_w)}{D}$$

$$\frac{4 * (H_{av} - H_w)}{\theta_{max\left(\frac{W}{D};\frac{D}{a}\right)} * D} < \Delta M < \frac{4 * (H_{av} - H_w)}{\theta_{min\left(\frac{W}{D};\frac{D}{a}\right)} * D}$$

$$\frac{1}{\theta_{max\left(\frac{W}{D};\frac{D}{a}\right)}} < \frac{\Delta M}{4 * (H_{av} - H_w)} * D < \frac{1}{\theta_{min\left(\frac{W}{D};\frac{D}{a}\right)}}$$

This analytical finding only applies for W/D>10%, because for W/D < 10 % tetas might get negative values

other restriction a/rw>20

In order to study the influence of the variables two types of calculation were performed.

Design. - In this type of calculation the goal is to find the well spacing for given a well penetration(W/D), different computations were performed; some of the results are shown on the next figures:

DATA		
ρsoil	1500	Kg/m3
ρwater	1000	Kg/m3
d	1,2	m
H =	6,5	m
S =	55,6	m
X3 =	64,8	m
kf =	5,00E-04	m/s
D =	66	m
rw =	0,2	m
thickness (pipe) =	0,01	m
g	9,81	m/s²
C =	125	Hazen-Williams coefficient
W =	10	m
He =	0,1	m
W/D =	15%	

Figure 2 General data used on the computations

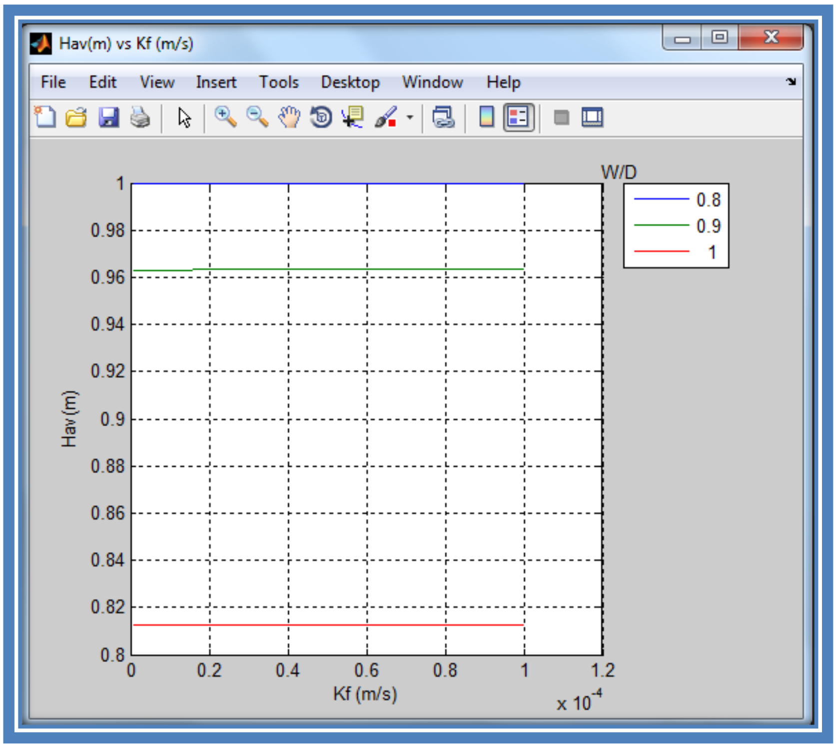


Figure 3 Net average head at well vs. Permeability (design)

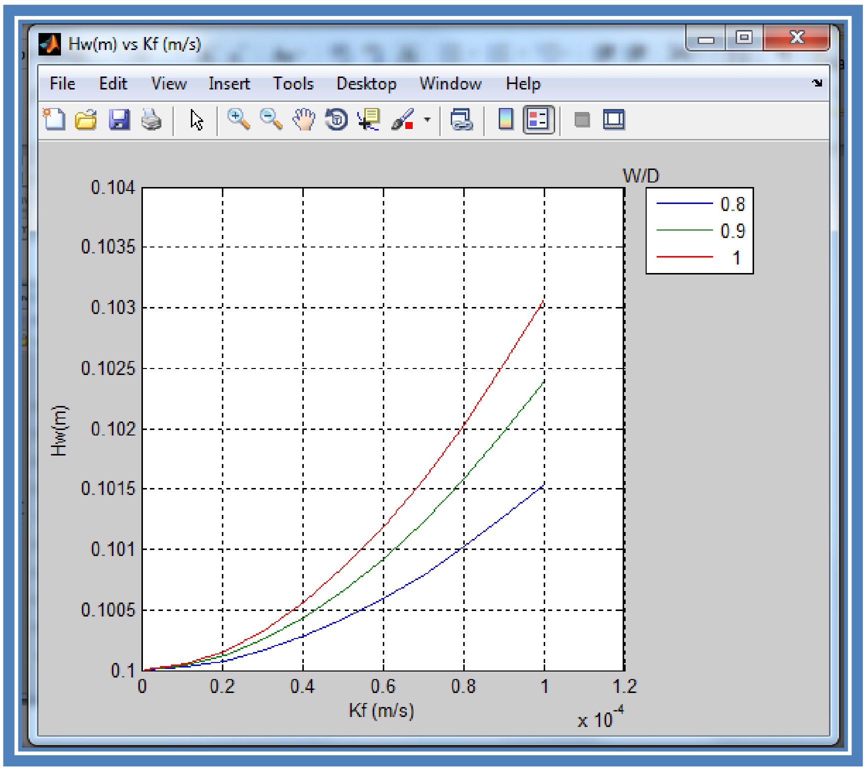


Figure 4 Head losses vs. Permeability (design)

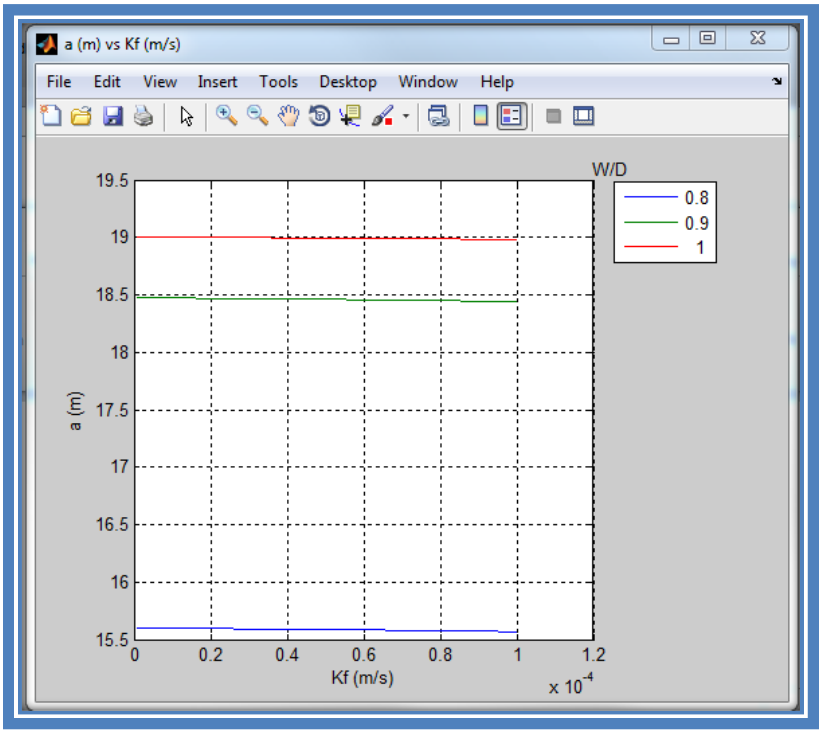


Figure 5 Design well spacing allowable head (ha) = 1 m

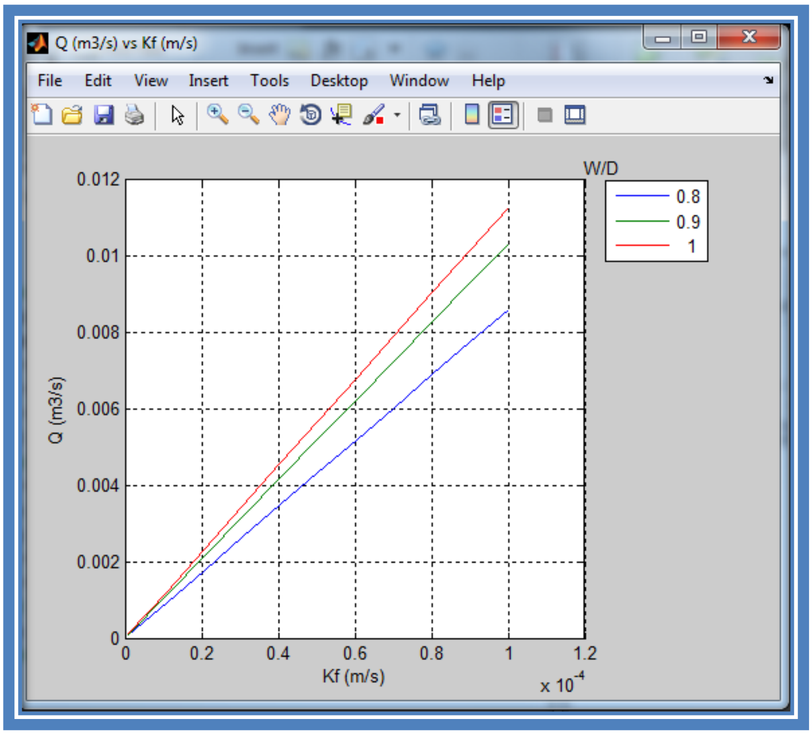


Figure 6 Well discharge (design)

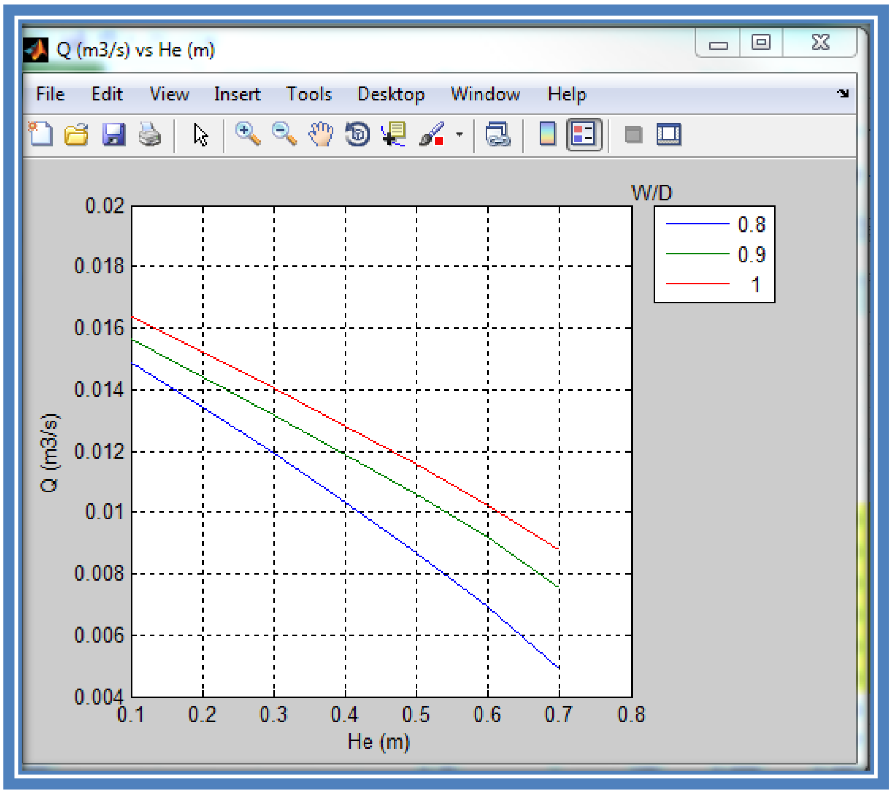


Figure 7 Well Discharge (design)

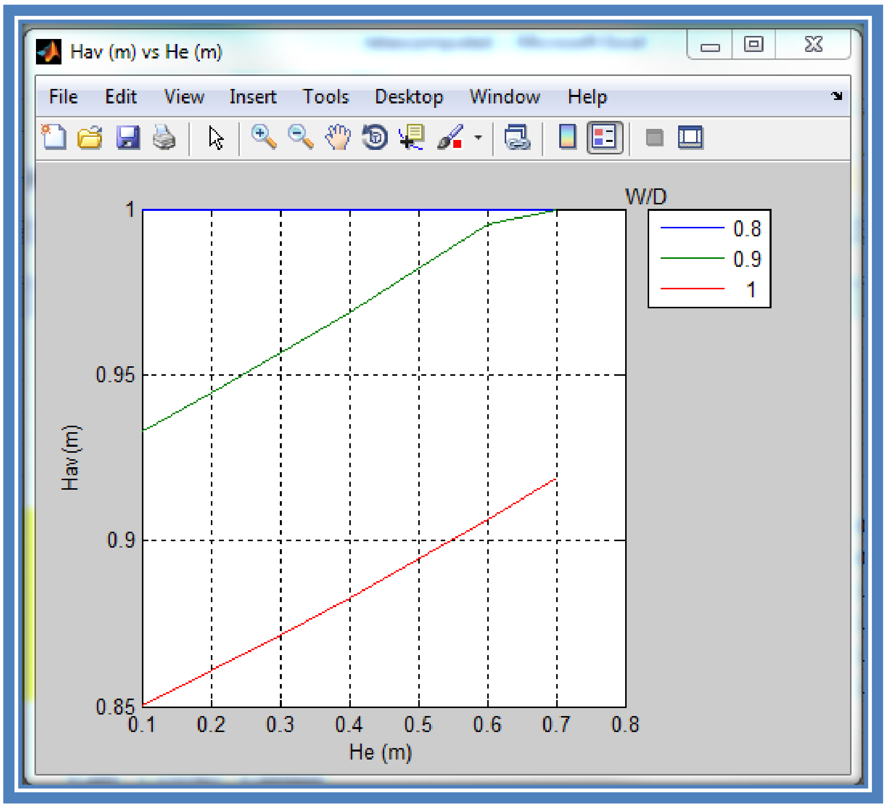


Figure 8 Net average head at well (design)

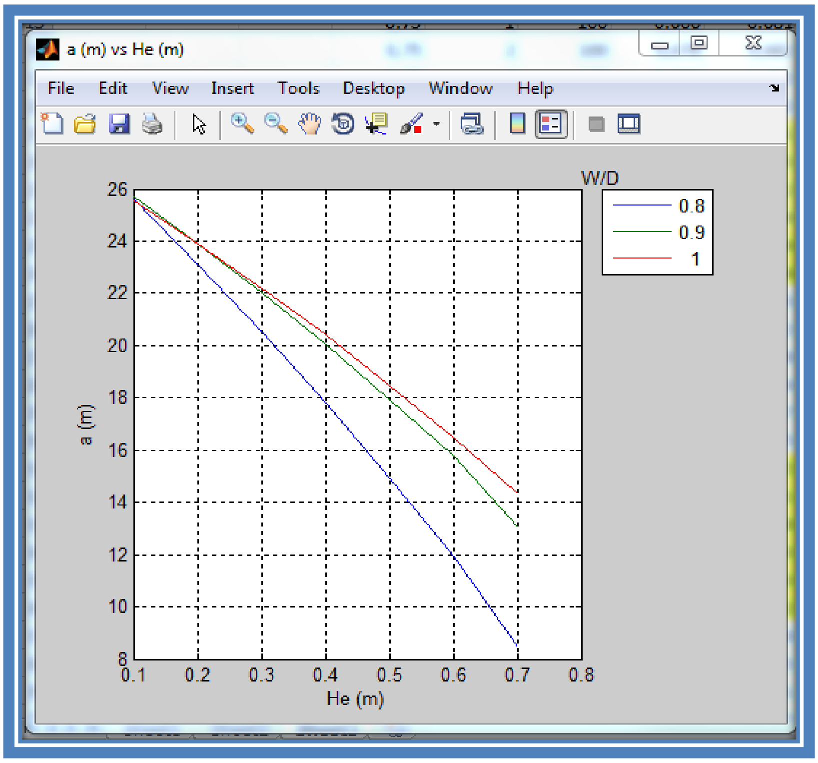


Figure 9 Design well spacing (m)

Checking: For this type of calculation the well spacing (a = 16.5 m) is given and from there all the others parameters are computed. This type of calculacion is adressed in order to check the net average head at well/ between wells (Hav or Hm , agree appropiate).

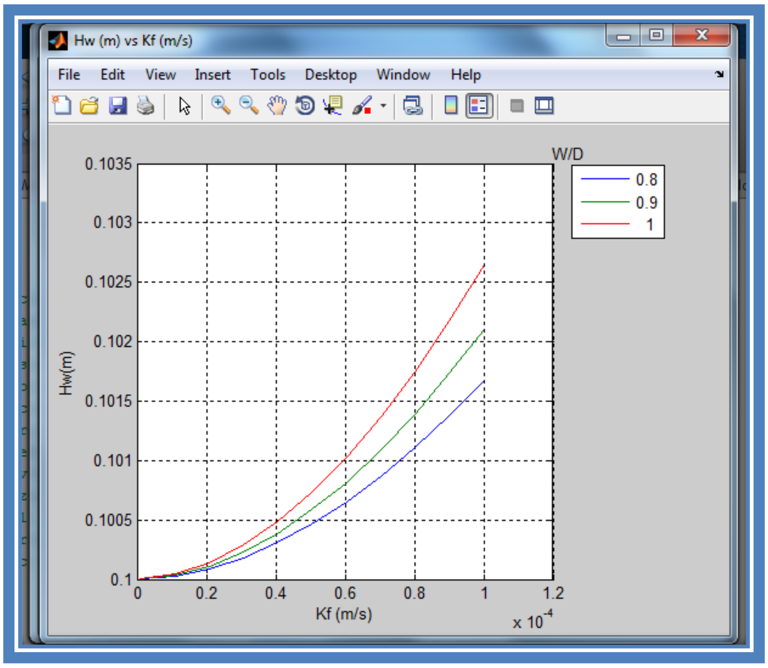


Figure 10 Head Losses (check)

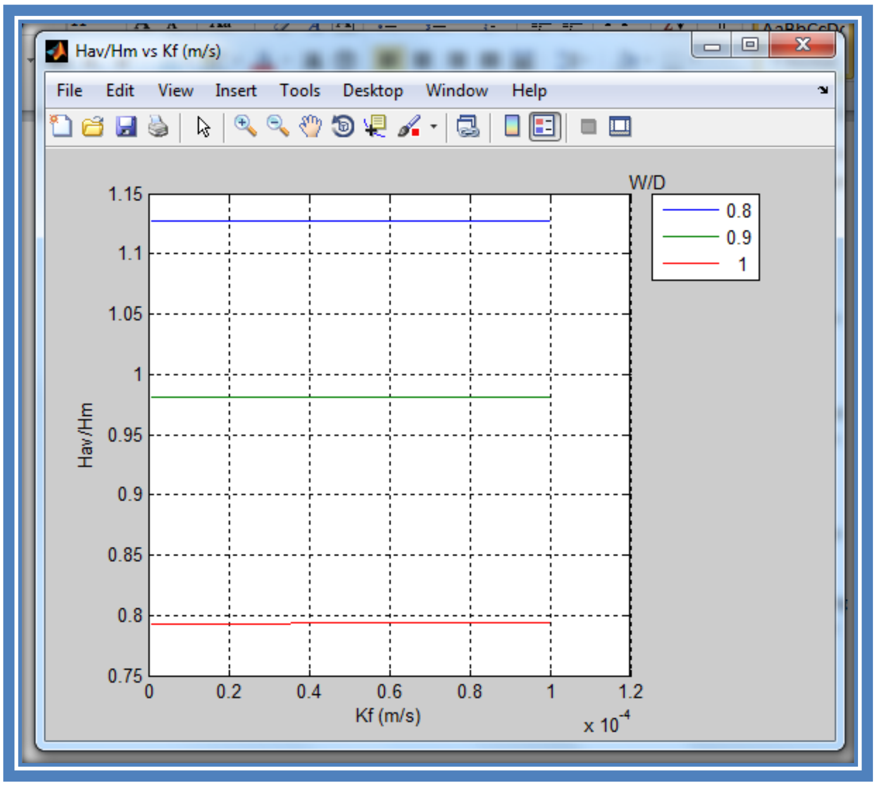


Figure 11 Hav/Hm ratio (check)

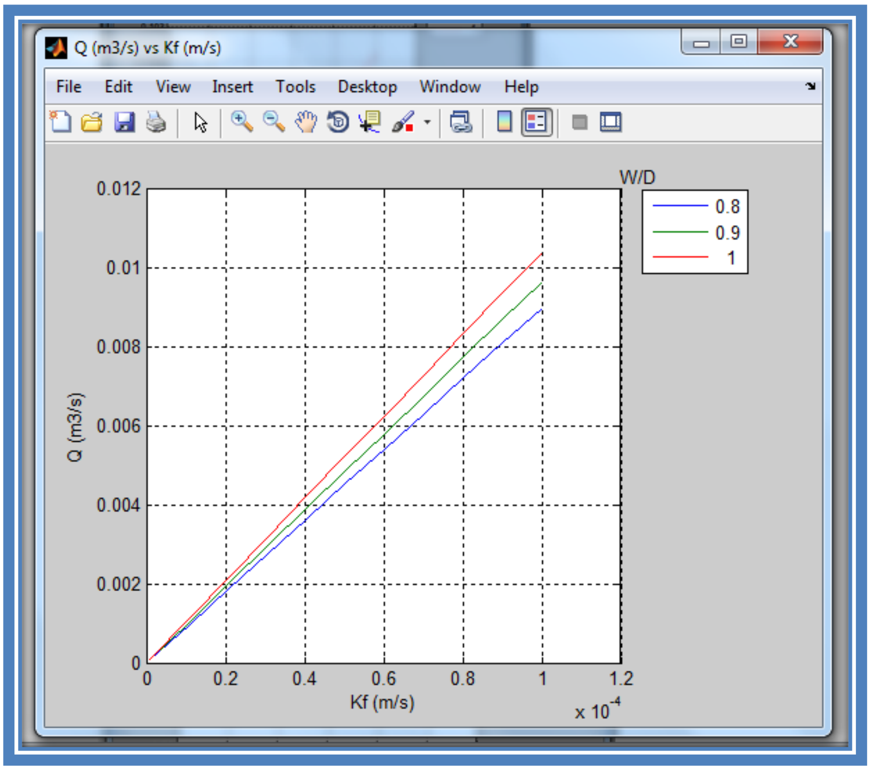


Figure 12 Discharge (check)

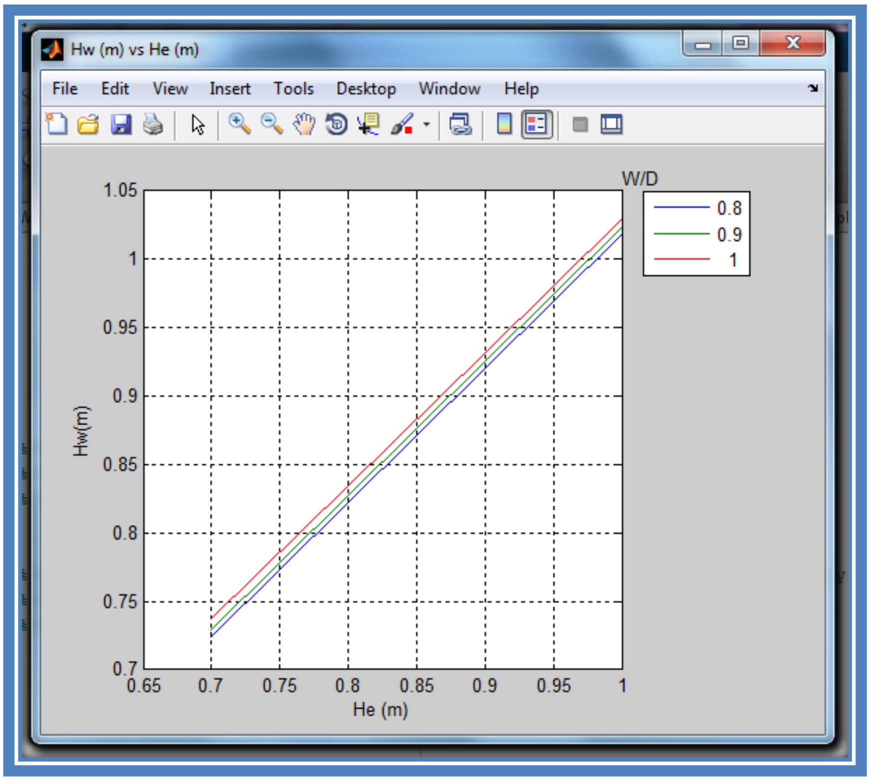


Figure 13 Head losses (check)

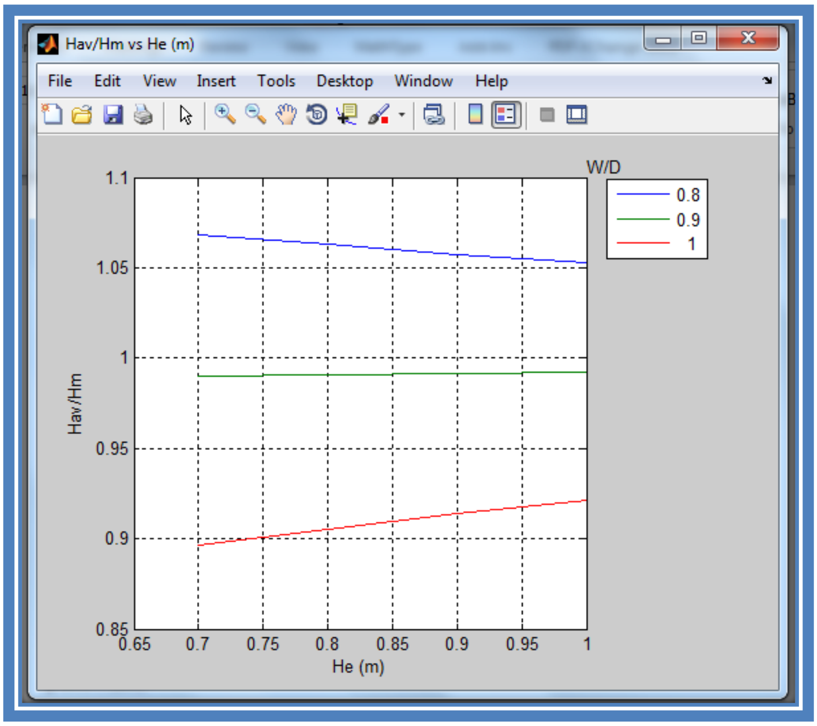


Figure 14 Hav/Hm ratio (check)

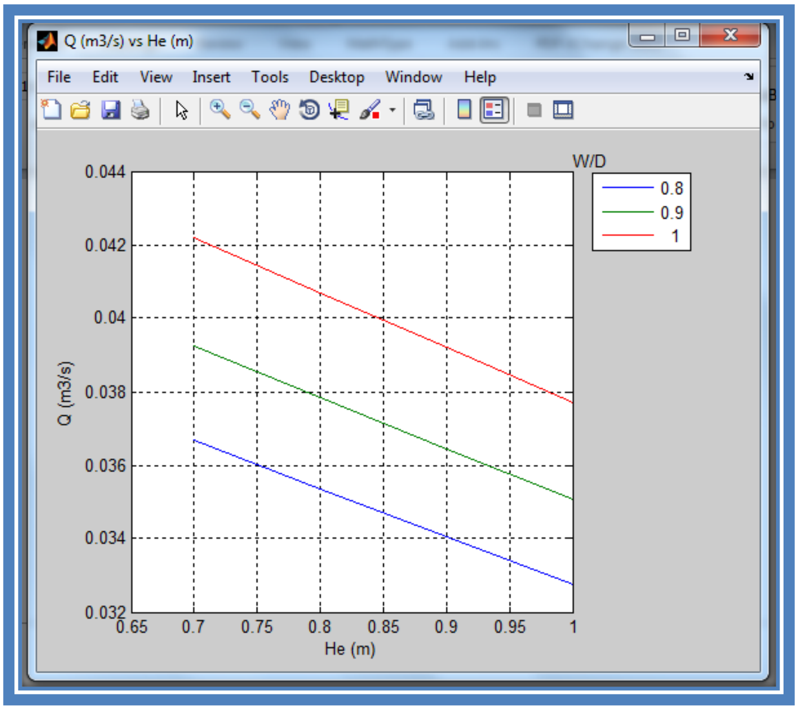


Figure 15 Discharge (check)

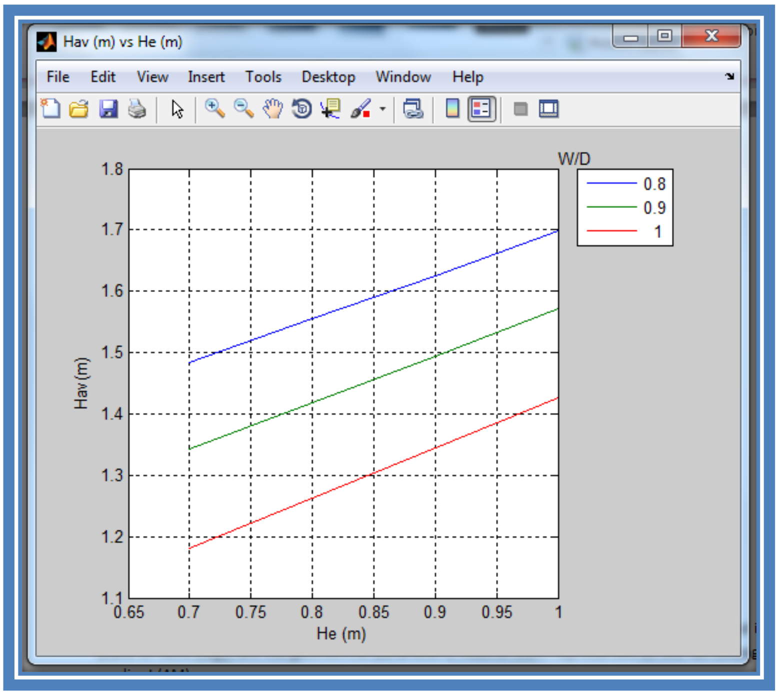


Figure 16 Net average Head (check)

Conclusions

From the results it can be concluded that the driven parameters for reducing the average net head in plane of wells (H_{av}) are the geometrical parameters, most of them represented on the net seepage gradient (ΔM).

In the other hand the permeability factor mostly influence the discharge, and its influence on the variance of the average net head (H_{av}) is neglectable.

When designing, the permeability factor does not influence the distance between wells, this is due to its little influence over the head reduction.

In order to model a possible clogging filter the well losses were incremented. It's worth mentioning that they have been assumed constant, independent of the discharge, trying to model the worst case scenario at the end of the life cycle of the relief well system. By incrementing the losses it can be observed an increment on of the average net head; as a consequence, a reduction of well spacing is necessary.

The increased of the entrance losses affected also the discharge; this can be explain by the change on the net seepage slope which is dependent on the well loses.

$$\Delta M = \frac{H - H_w - h_{av}}{S} - \frac{H_w + h_{av}}{x_3}$$

From the results is observed that a bigger well penetration is always beneficial to our system. By incrementing well penetration the net head is reduced, allowing a bigger well spacing. In the other hand a bigger penetration also catch more discharge increasing the amount of water inside the polder.

From the plots Hav/Hm it can be observed that for values of W/D > 0.9 the design parameter is Hm (Hm>Hav).

Discussion:

- Is there any source for the statistical values of the variables (He)?
- How to address clogging?

Results of preliminary designs:
In order to test the method and to check the results, different calculations were made by using data from the report "Evaluatie geohydrologische maatregelen als oplossing voor piping"(E.G.M.O.P.) corresponds to the Opijnen location. The results shown on Table 1 correspond to a schematization (Figure 17) of the input data used on the report.

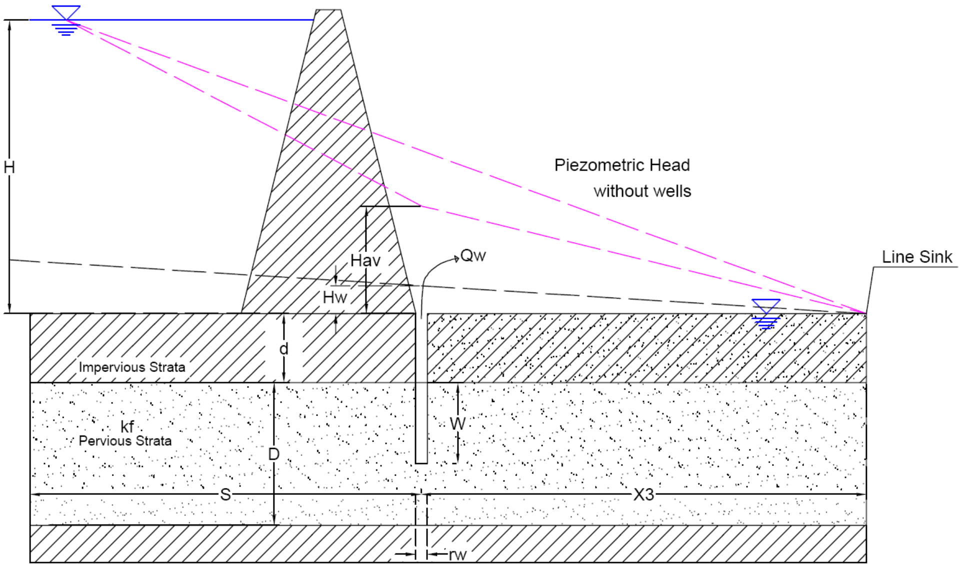


Figure 17 Schematization of relief well system

Table 1 Data for first computation

DATA		
ρsoil	1500	Kg/m3
ρwater	1000	Kg/m3
d	1,2	m
H =	6,5	m
S =	55,6	m
X3 =	64,8	m
kf =	5,00E-04	m/s
D =	66	m
rw =	0,2	m
thickness (pipe) =	0,01	m
g	9,81	m/s²
C =	125	Hazen-Williams coefficient
W =	10	m
He =	0,1	m
W/D =	15%	

Table 2 Results of first computation

Results		
ha =	0,6	m
a =	12,50	m
Qw =	0,01622	m3/s
hav =	2,220	m
hm =	1,478	m
Hw =	0,102	m
ΔM =	0,039	m/m
Θa =	4,517	
Θm =	3,008	
Θm/Θa =	0,67	
Hav	**2,32**	**m**
Hm =	1,58	m
Qw/m =	0,0013	m3/s/m
Qw/m/d =	112,10	m3/d/m
Qw =	**1401,27**	**m3/d**
a/rw =	62,50	
He/Hw =	98%	
head loss	4,18	m
a/D =	5,28	

Six different cases were studied, in order to check their variability; the two more important parameters were plotted (average head at well & well discharge).

On Figure 18 the legend can be read in the following manner: 100%D50kf(5e-4), it means 100 % well penetration (W/D), D50 fifty meters of pervious foundation, kf(5e-4) permeability equals to 5x10-4 m/s.

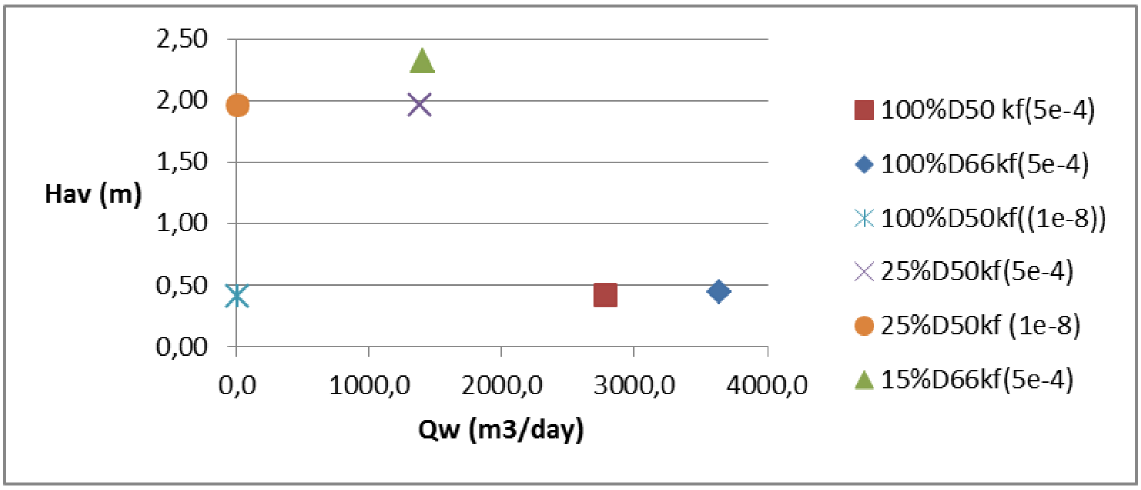


Figure 18 Average Head at well vs. Well discharge

Conclusions of deterministic analysis:
The results (from this schematization and from the report) are difficult to compare due to the fact that the calculations give different parameters (Q=m3/day/m vs. Qw= m3/day), and some parameters are unknown or assumed, e.g. distance between wells (which was assumed from the report where is mentioned 40 wells in 500 meters), entrance loses etc. Nevertheless a few conclusions can be drawn:

- The discharge found per well is close to the average proposed by Arcadis (1300 m3/day).
- When observing Figure 19 it can be spotted that the most influencing parameter in reducing the head is the well penetration; and that the conductance, represented by kfxD, is the driven parameter for the well discharge.

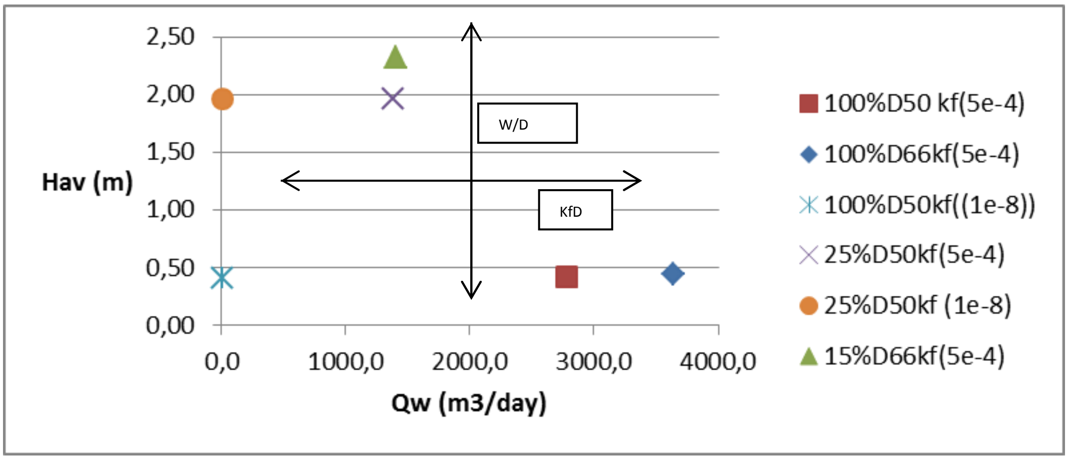


Figure 19

Probabilistic approach:

Even though from the previous section it was highlighted that the parameter that influence the most to reduce the head is the well penetration (W/D), some others parameters weren't modified to check its sensibility, as for example the well loses. Entrance losses were assumed constant and equal for every case, although they influence the well discharge by reducing the net seepage gradient (see annex for details). In order to make a sensibility analysis to check which variables influence the most to uplift failure, a FORM analysis was performed. For the sake of simplicity only fully penetrating wells were taking into account. The results are shown on the following graphs:

Data used on probabilistic approach				
Variable	**Type of distribution**	**Mean**	**Standard deviation**	**units**
Sediment density	normal	1750	100	kg/m3
Water density	deterministic	1000	-	kg/m3
Thickness impervious layer (d)	normal	2	0.2	m
Thickness pervious layer (D)	normal	20	0.5	m
Permeability (kf)	lognormal	0.00005	0.00005	m/s
Water level river side (H)	normal	6	0.1	m
Entrance loses (He)	normal	0.05	0.01	m
Foreland distance (S)	normal	30	0.1	m
Hinterland distance (X3)	normal	50	0.11	m
Well thickness	deterministic	0.01	-	m
Hanzen and Williams constant (C)	normal	125	10	-
Well radius (rw)	deterministic	0.125	-	m
Well penetration (W)	normal	20	0.02	m

Table 3 Data used on probabilistic approach

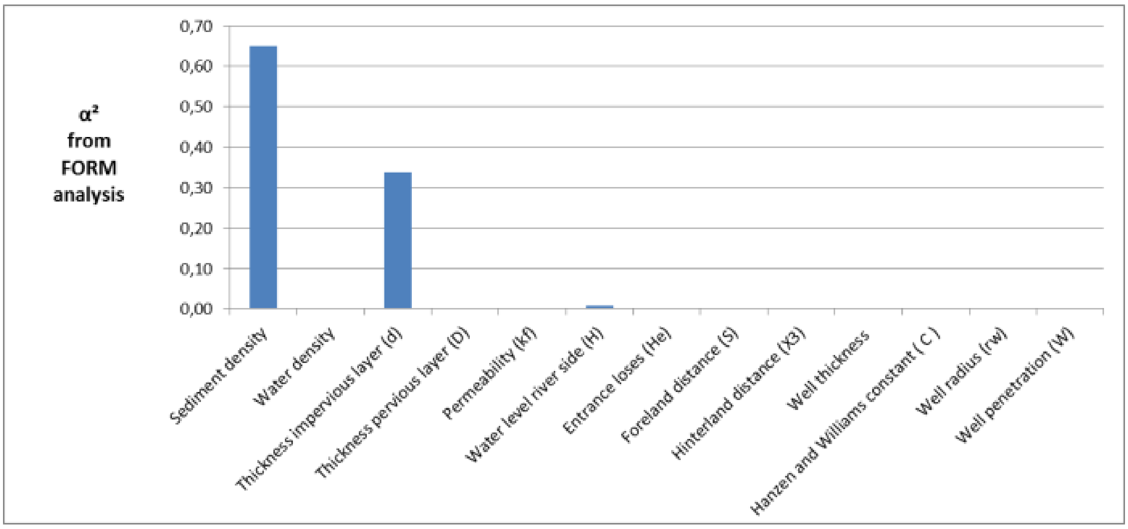


Figure 20 Sensibility analysis for relief well systems (uplift)

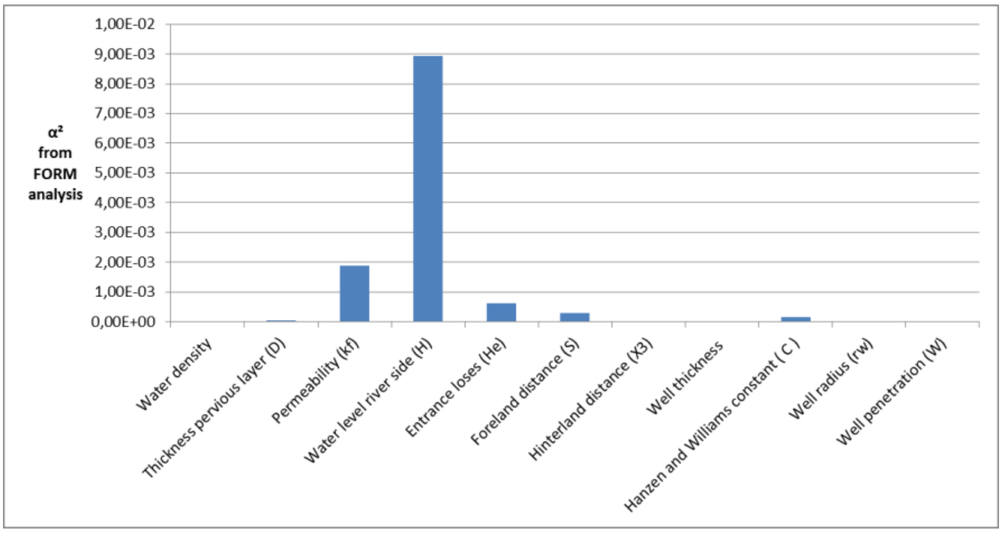


Figure 21 Sensibility analysis disregarding 'resistance' parameters (sediment density and thickness impervious layer)

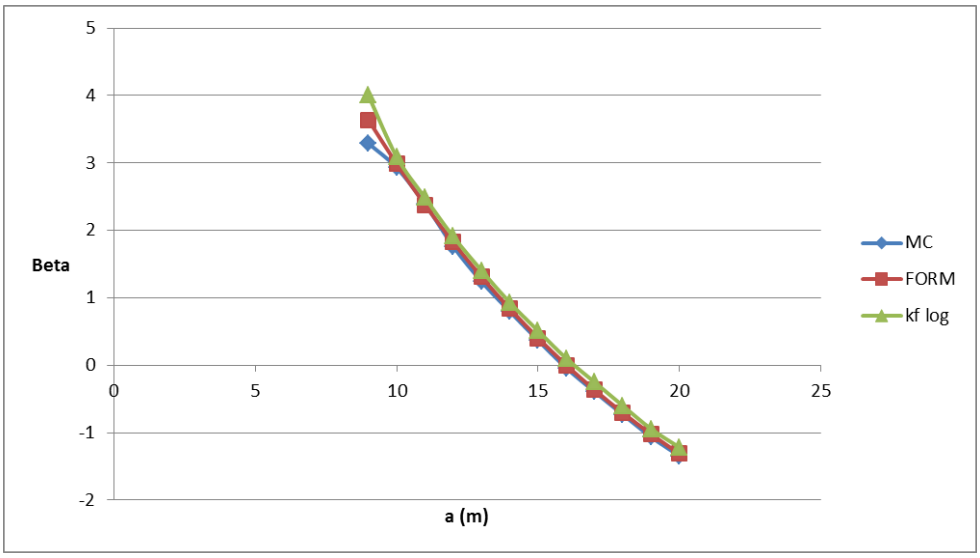


Figure 22 Variation of Reliability Index by changing the distance between wells

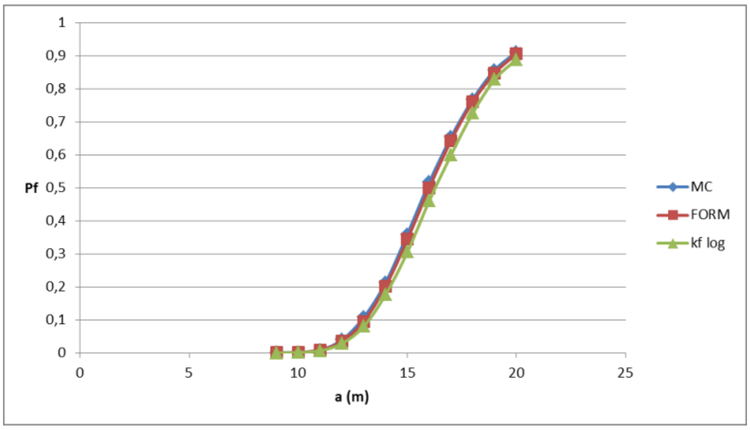


Figure 23 Probability distribution function (distance between wells)

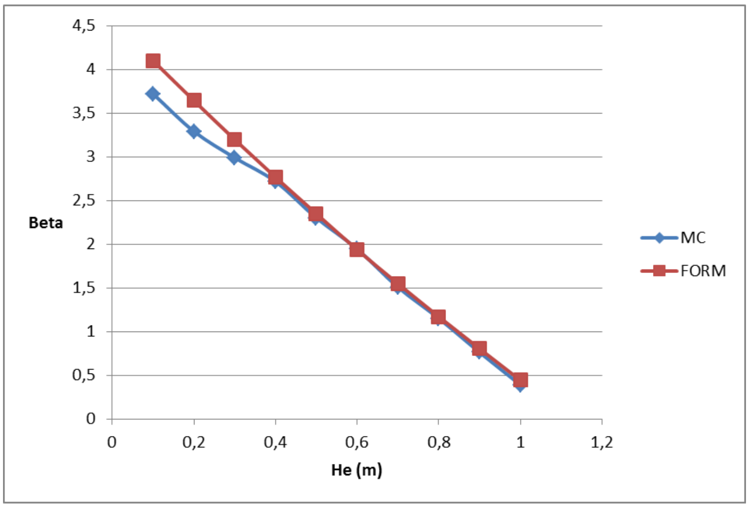


Figure 24 Variation of Reliability Index by changing entrance losses (He)

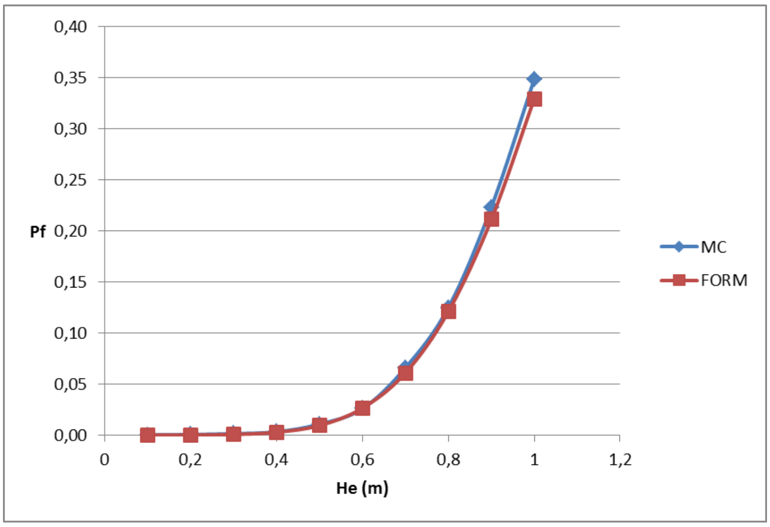


Figure 25 Probability distribution function (He)

Remarks:
Using the mean values of the variables, without using any safety factor, a value of 14.40 m for a (distance between wells) was found which correspond to a probability of failure approx. of 30%. This value of a (14.40) was used when analysing the probability of failure by changing the losses (He).

Conclusions from probabilistic analysis:

- After the head at the riverside, the variables that influence the most are permeability and entrance loses.
- When analyzing the probability of failure by incrementing the entrance losses, is remarkable that even increasing them by 1000 % (from 0,1 to 1 m), which could mean clogging, the system is still functional and its probability of failure is less than 35 %.
- In disagreement with the findings from the deterministic analysis, well penetration does not show high influence. This could be explain by the fact that the well factors (see annex) are constant for this case, due full well penetration is assumed.

ANALYSIS LIFE CYCLE COST

Purpose of the meeting: Define proper approach to find the present value.

Problem description: To find the proper way to asses these costs has generated some discussion among the experts and committee members.

Basically, three different approaches have been mentioned, and are detailed below. The base of all approaches is to find the present value of each alternative; nevertheless the processes applied have been discussed, and the three proposals are:

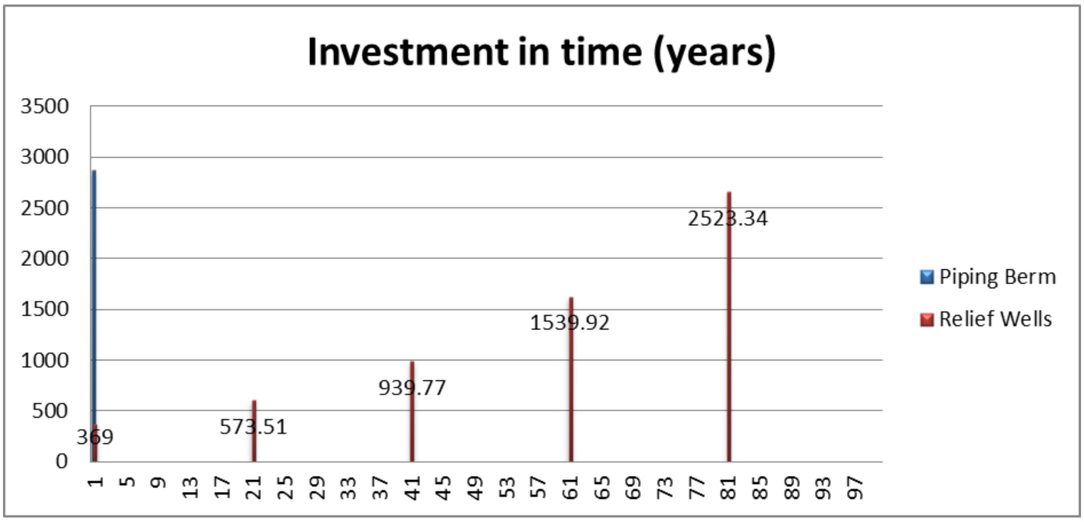
1) First proposal: It is based on future worth, assuming that it is required to have the entire budget needed to be able to develop the alternatives in the future. This would mean that if today the cost of implementing relief wells system is 369 €/m, and this implementation has to be done every 20 years, then the present value would be the sum of the present costs plus the future costs. Using the present value equation:

$$FC = Pv * (1+i)^n$$
$$FC_0 = 369 * (1+i)^0$$
$$FC_1 = 369 * (1+i)^{20}$$
$$FC_2 = 369 * (1+i)^{40}$$
$$FC_4 = 369 * (1+i)^{60}$$
$$FC_5 = 369 * (1+i)^{80}$$

And for the case of piping berm, the total cost would be only the initial investment (for example: 2866 €/m).

According to this approach the result would be:

	Initial cost	Life cycle (years)	Inflation rate (%)	Total Cost
Piping berm	2866	100	2.5	2866
Relief wells	369	20	2.5	6248



2) Second proposal: Look at the net present cost, assuming the investments along the life cycle of the infrastructure (cash flows), and including the discount rate as follows:
For piping berms:

$$PV = \frac{C_1}{(1+i)^0} = 2866$$

For relief wells:

$$PV = \frac{369}{(1+i)^0} + \frac{369}{(1+i)^{20}} + \frac{369}{(1+i)^{40}} + \frac{369}{(1+i)^{60}} + \frac{369}{(1+i)^{80}} = 866$$

PV= present value

3) Third proposal: States that having the full budget needed ahead on time and not investing it, is not a proper decision. Also, taking into account a discount rate for the future (constant and not decreasing) is speculation due to market fluctuations, plus high discount rates for long term projects will devalue the project too soon, and somehow it means trespassing the (financial) problem to future generations. For these reasons, it is better to account the initial cost and multiply it n times by the costs of rebuilding the infrastructure during its life cycle;
For piping berms:
Total Costs = $2866 * 1 = 2866$
For relief wells:
Total Costs = $369 * 5 = 1845$

MATLAB SCRIPT

```
Function z = uplift2_x2z(varargin)


%% create samples-structure based on input arguments
global  samples

samples = setproperty(samples, varargin{:});


%% calculate z-values
% pre-allocate z
%z = nan(size(samples.ros));

 [g_u, g_he,g_uHe] = LSF_uplift_heave20(varargin);

 save('Zetas.mat','g_u', 'g_he','g_uHe');


z = g_u  ;   % [-] block equation
%    p=0

end
```

```
Funcion uplift_heave

function [resultFORMUP,resultFORMhe] = uplift_heave3(...A,stochast,w)



%AA = A;

for j= 1:length(w)

for i = 1:length(A)



  % run the calculation using FORM

  %uplift
    resultFORMUP(i,j) = FORM(...
        'stochast', stochast,...
        'x2zFunction', @uplift2_x2z,...
        'x2zVariables', {'a', A(i),'W',w(j)});

    %Heave
    resultFORMhe(i,j) = FORM(...
        'stochast', stochast,...
        'x2zFunction', @heave2F_x2z,...
        'x2zVariables', {'a', A(i),'W',w(j)});

end
end
```

```
Limit State Function for Probabilistic Design

function [g_u, g_he,g_uHe ] = LSF_uplift_heave20(varargin)
%% create samples-structure based on input arguments

global  samples
samples = setproperty(samples, varargin{:});
%% calculate z-values
% pre-allocate z
%z = nan(size(samples.ros));

g_u= nan(size(samples.ros));
g_he  = nan(size(samples.ros));
g_uHe  = nan(size(samples.ros));
inc = 0.001;
% pre-allocate other functions
Wellpene = [0.05;0.1;0.15;0.25;0.5;0.75;1];
Dovera = [0.25;0.5;1;2;3;4];

%Values of teta A
Te_A = [1.778   1.908   1.662   1.225   0.742   0.523   0.44
3.879   2.934   2.31    1.569   0.857   0.563   0.44
6.063   3.977   2.97    1.926   0.983   0.606   0.44
8.377   5.139   3.747   2.39    1.175   0.678   0.44
9.761   5.977   4.344   2.798   1.361   0.748   0.44
11.144  6.814   4.941   3.199   1.547   0.818   0.44
];
%Values of teta M
Te_M = [1.887   2.018   1.772   1.335   0.851   0.633   0.55
3.969   3.025   2.401   1.622   0.955   0.667   0.55
6.021   3.941   2.938   1.908   1.012   0.681   0.55
7.864   4.649   3.293   2.024   1.024   0.682   0.55
8.574   4.86    3.363   2.047   1.024   0.682   0.55
9.283   5.071   3.432   2.075   1.024   0.682   0.55];
%Values of Dteta
Dtet = [6.963   3.298   2.077   1.466   0.733   0.489   0.367
6.963   3.298   2.077   1.466   0.733   0.489   0.367
6.963   3.298   2.077   1.466   0.733   0.489   0.367
6.963   3.298   2.077   1.466   0.733   0.489   0.367
6.963   3.298   2.077   1.466   0.733   0.489   0.367
6.963   3.298   2.077   1.466   0.733   0.489   0.367];
for ii = 1:length(samples.ros)
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

    X3 = sqrt(samples. kf (ii)*samples.d (ii)*samples.D (ii)/ samples.kb
(ii));
    W1 = samples.W *samples.D (ii);
    H=  (samples.Hr (ii))-samples.Hpo (ii);
     ha = samples.d (ii)*(samples.ros (ii)- samples.row (ii))/samples.row
(ii);     %    [m] Allowable head
     Hav = 0;
     e = 1;

     input1=W1/samples. D(ii) ; % 100% - 5%(W/D)
     input2=samples. D (ii)/samples.a     ; % 0.25 - 4.0
```

```
%%%%%%To find well factors%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

if input1>1
    input1=1;
end

if input1==1
            deltateta= 0.367 ;
            tetaA = 0.440 ;
            tetaM = 0.550;
            pl=1;
else
    tetaA  = interp2(Wellpene,Dovera,Te_A,input1,input2);
    tetaM = interp2(Wellpene,Dovera,Te_M,input1,input2);
    deltateta  = interp2(Wellpene,Dovera,Dtet,input1,input2);
end
%%%%%%Assigning tetas%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

                tetaa2 = abs(tetaA+ deltateta * (log10
(samples.a/samples.rw (ii))-2));
                tetam0 = abs(tetaM+ deltateta * (log10
(samples.a/samples.rw (ii))-2));
 %Calculation of HAV
while  e > 0.01

%%%%%%Calculations%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

Hav = Hav + inc ;                    %       first assumption

DM = abs(((H-Hav)/samples.S (ii) )-Hav/ X3);

Qw = samples.a*DM*samples. kf (ii)*samples.D (ii);

%%%%%%Frictional Losses%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

q = Qw * 15850  ;            %Converting to [gal/min] to use Hanzen
Williams formula
dh = 2 * (samples. rw (ii))  ;  % dh inch
dhi = dh*39.3701;    % dh inch
f = 0.2083 * ((100/samples. C (ii))^(1.852))*(q^(1.852))/(dhi^(4.8655));
Hf = f * W1* 0.01 ;

%%%%%%VelocityLosses%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

v = Qw/(pi*((dh/2)^2));
Hv = (v^2)/(2*9.81);

%%%%%%Total Losses%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

%Hw = samples.He (ii);
Hw = samples.He (ii) + Hv + Hf;

%hav = samples.a*DM*tetaa;

hav = Hav - Hw;
tetaa1 = hav/(samples.a * DM );

e = abs(tetaa1-tetaa2);
```

```
end

 rel1 = tetam0/ tetaa2;
hm = hav*rel1;

Hm   = hm + Hw;

        if Hm>Hav
            Hp = Hm;
        else
            Hp = Hav;
        end

        io=(Hp/samples.d (ii));
g_u(ii,:)= ha-Hp ;
g_he (ii,:) = samples.ic (ii)-io;
g_uHe (ii,:) = max (g_u(ii,:),g_he(ii,:));
end
```

```
Main for probabilistic design of wells

%% define the stochasts
% create a structure with fields  and 'propertyName'
clear;
global A stochast samples w Dc

%%%%%%For saving%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
prompt={'Enter the file name'};
% Create all your text fields with the questions specified by the variable
prompt.
title='Save File';
% The main title of your input dialog interface.
answer=inputdlg(prompt,title);
name1 = answer{1};
% Convert these values to a number using str2num.
%}
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
d = 4.11;
Dc = 9.3; %meaqn value of aquifer depth for cost calculation
w =0.6:0.1:1;
A = 5:1:10;
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

btarget = 4.5;
v=[1 2 3  btarget   ]; % for contour lines, for plotting beta
v1=[200  300  400  500  600  700  800  900 1000 ]; % for contour lines, for
plotting costs
Fixcost = 1737.52;
Variablecost= 130.62;
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

%% import data
disp '>> Define data...'

file = inputdlg('Enter the name of the data CSV file : ');
 dummy = cell2str(file);
name=dummy(3:end-3);

filename=[name '.csv'];

fid = fopen(filename, 'r');
tline = fgetl(fid);

% Split header
data(1,:) = regexp(tline, '\;', 'split');

% Parse and read rest of file
ctr = 1;
while(~feof(fid))
if ischar(tline)
ctr = ctr + 1;
tline = fgetl(fid);
data(ctr,:) = regexp(tline, '\;', 'split');
else
break;
end

end
```

```
fclose(fid);

clear ctr tline ans
fprintf('   data imported from %s.csv \n',name);
%data variable is CELL - class(data)
%need transformation - str2num(cell2mat(data(1,3)))
%% check size of Data
        if size(data,1)==13 & size(data,2)==4
        fprintf('    %s.csv , size is OK\n',name);
        else
        warning('Wrong data size, check it and Run again');
        break
        end
%% create stochast
%care on mu and sd of the Gumbel and Lognormal
stochast = struct();
%number of variables
tam=size(data);
num=tam(1);
clear tam fid

% allocate data in stochast
%(Name is char, Distr is function_handle, Params is cell)
indx=1;
  while indx<num+1,
    stochast(indx).Name    = char(data(indx,1));
      di=sprintf('%s',cell2mat(data(indx,2)));
    stochast(indx).Distr   = str2func(di);
      par1 = str2num(cell2mat(data(indx,3)));
      par2 = str2num(cell2mat(data(indx,4)));
    stochast(indx).Params = {par1 par2 };
     indx=indx+1;
   end
clear indx par1 par2 di ans

%%
%change parameters mu sigma of LN distribution

isit=strcmp(data(:,2),'logn_inv'); %lognormal variables index
isLN=find(isit);
num2 = length(isLN);

ii=1;
while ii<num2+1,

    index = isLN(ii);

[lambda, zeta]=lognpar(stochast(index).Params{1},
stochast(index).Params{2});
stochast(index).Params={lambda, zeta};

     ii=ii+1;
   clear lambda zeta
end
clear isLN index ii num2
```

```
%Create new data table

data_used=cell(num,4);
indx=1;
  while indx<num+1,
    data_used(indx,1) = cellstr(stochast(indx).Name);
    data_used(indx,2) = cellstr(func2str(stochast(indx).Distr));
    data_used(indx,3) = stochast(indx).Params(1);
    data_used(indx,4) = stochast(indx).Params(2);

     indx=indx+1;
   end
clear indx
disp '>> OK'
% load sample values
samples = struct(...
    'ros', [],...        % [kg/m3] RhoS density sediment
    'row', [],...        % [kg/m3] RhoW density water
    'd', [],...          % [m]     thickness impervious layer
    'D', [],...          % [m]     thickness pervious layer
    'kf', [],...         % [m/s]   permeability
    'kb', [],...         % [m/s]   blanket permeability
    'Hr', [],...          % [m]     water level river side
    'Hpo', [],...          % [m]     water level polder side
    'He', [],...         % [m]     Entrance loses
    'S', [],...         % [m]     Foreland distance
    'C', [],...          % [-]     Hanzen and Williams constant
    'rw', [],...         % [m]     Well radius
    'ic', [],...         % [-]     Critical gradient
    'W', [],...          % [m]     Well penetration
    'a', []);            % [m]     distance between wells


%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%% main matter: running the calculation

 [resultFORMUP,resultFORMhe] = uplift_heave3(...
    A,stochast,w);
%%%%%%%%%%%%%%%
%%%%%%%FORMCOMB
clear alphaF alphaFH betaFORM betaFORMHe pform pformHe rho12 alpha0 alpha2
rho1 CFPF BFC
num=size(stochast,2);

rho12=zeros(1,num);
for j= 1:size(resultFORMUP,2)
for i=1:size(resultFORMUP,1)

alphaF (i,:) = resultFORMUP(i,j).Output.alpha;

alphaFH (i,:)= resultFORMhe(i,j).Output.alpha;

betaFORM (i,j)= resultFORMUP(i,j).Output.Beta;

betaFORMHe (i,j)= resultFORMhe(i,j).Output.Beta;

pform (i,j)=resultFORMUP(i,j).Output.P_f;

pformHe (i,j)= resultFORMhe(i,j).Output.P_f;
```

```
if pformHe (i,j) > 0.99999999999

    pformHe (i,j) = 0.99999999999;

end

if betaFORMHe (i,j) > 6.7

    betaFORMHe (i,j) = 6.7;

end

   for jj=1:num
        if alphaF(i,jj)~=0 & alphaFH(i,jj)~=0
         rho12(jj,i)=1;
         %rho31(jj,i)=rho12';

        end
    end

    for ih= 1: num
    alpha1 (ih) = alphaF(i,ih);
    alpha12 (ih) = alphaFH (i,ih);
    rho11(ih) = rho12 (ih);
    end
    alpha0 = alpha1';
    alpha2 = alpha12';
    rho1 = rho11';

 [CombPf Combbeta alpha rho] = Combine(...

pform(i,j),betaFORM(i,j),alpha0,pformHe(i,j),betaFORMHe(i,j),alpha2,rho1,'p
arallel');
CFPF(i,j) = CombPf;
BFC (i,j)= Combbeta;
end
alphaFm(i,:)= mean (alphaF,1);
alphaFHm (i,:)= mean (alphaFH,1);

end

        figure('Name','Reliability index (FORM)','NumberTitle','off')
        x= A;
        y = w;
        Z = BFC';
         [x,y]=contour (x,y,Z);
         clabel(x,y)
         xlabel('a(m)')
         ylabel('W/D')

 %%making the BIG matrix WW, AA, EU($)
 %%making the BIG matrix WW, AA, EU($)
 %%making the BIG matrix WW, AA, EU($)
 ni= length (A);
nj = length (w);
nn = ni*nj;
W1 (1:nn)= zeros;
```

```
        for ik = 1: nj
            W(1:ni) = w(ik);
            WW (ik,:) = W;
        end
        W2= WW';
        W1 = W2(:);

        aw = repmat(A,1,nj)';
        betw =  BFC(:);
 %%making the BIG matrix WW, AA, EU($)

        Cost (i,j) = (Fixcost + Variablecost * ((w (j)*Dc)+d))/(A (i));

         for ir = 1: nn


CT (ir)= (Fixcost + Variablecost * ((W1(ir)*Dc)+d))/(aw (ir));



         end
    Ct =CT';
%%%%%%Cost as matrix%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

     for ir = 1: nn
     bb (ir) =  betw (ir);
     if bb (ir) >= btarget
         if bb (ir) < 8
         ix = 1;
         else
           ix = 0;
         end
     else
         ix = 0;
     end
     wv (ir)= W1 (ir)*ix;
         av (ir) = aw (ir)*ix;
         betv (ir) = betw (ir)*ix;
         Ctv (ir) = Ct (ir)*ix;
         inx (ir)= (CT (ir)/ bb(ir))*ix;
     end
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

      bv= bb';
         inxv = inx';
         TMatrix = [W1,aw, betw ,Ct,inxv]; %the complete matrix Well
penetration, Distance between wells, Cost, ratio Cost/Beta
         wv = wv(wv~=0); av = av(av~=0);betv = betv(betv~=0);Ctv =
Ctv(Ctv~=0);inx = inx(inx~=0); %Removing zeros for the matrices
         inxV= inx'; %transposing
         Ctv1= Ctv'; %transposing
 %%%%%%%%cost analysis for contour
 for j= 1:size(resultFORMUP,2)
for i=1:size(resultFORMUP,1)

Cost (i,j) = (Fixcost + Variablecost *((W1(ir)*Dc)+d))/(A (i));
Rcb (i,j) = BFC (i,j)/Cost (i,j);
end

end
```

```
        [rmi,cmi]=find(Ctv1==min(min(Ctv1)));
         TMaV = [wv',av',betv',Ctv1,inxV]; %Removing zeros
         %minimun cost given the reliability target
         Wop = TMaV (rmi,1);
         aop = TMaV (rmi,2);
         Cop = TMaV (rmi,4);
         Bop = TMaV (rmi,3);
         display (Wop)
         display (aop)
         display (Cop)
         display (Bop)

    J = Cost';
  %{
%%%%%%Two contours at the same time%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
 %}
  figure('Name','Reliability and Cost','NumberTitle','off');
        x= A;
        y = w;
        Z = BFC';
         C = Cost';

         [x,y]=contour (x,y,Z,v);
        hold on
        x1= A;
        y1 = w;
         [x1,y1]=contour (x1,y1,J,v1);
        hold on
        scatter(aop,Wop)
        hold on
        ylimits = get(gca, 'YLim');
        hold on;
        % plot a vertical line
        plot([aop aop], [0 Wop], 'k');
        hold on
         % plot a horizontal line
        plot([0 aop], [Wop Wop], 'k');
        hold on
        %%%%to get the position of the textbox
        yp = w (nj)- w (1);
        yp1 = Wop - w (1);
        yop = yp1/yp;
        xp = A (ni)- A(1);
        xp1 = aop- A(1) ;
        xop = xp1/xp;
       annotation('textbox',[xop 0.3 0.3
0.15],'FitBoxToText','on','String',{['a =' num2str(aop)],['W/D ='
num2str(Wop)],['\beta =' num2str(Bop,2)],['C ='
num2str(Cop,4)]},'BackgroundColor',[0.9  0.9 0.9]);
        clabel(x,y)
         clabel(x1,y1)
        xlabel('a(m)')
        ylabel('W/D')
        hold off

        savdir =
'D:\miran_cs\Documents\thesis\01_greenlight\Matlab\Chapter5';
save(fullfile(savdir,name1),'resultFORMUP','resultFORMhe','TMatrix',
'TMaV','A','w');
```

```
Script to estimate relief well head

clear;
%%%%%%Input Data%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
RoSoil = 1600;       %    [kg/m3]
RoWater = 1000;      %    [kg/m3]
d = 3  ;             %    [m]          thickness of impervious layer
H = 1.66;               %    [m]          Water level riverside
S =28.5;               %    [m]          Distance Foreland
X3 =108;             %    [m]          Distance Hinterland]
kf =1.74E-4 ;     %    [m/s]        permeability
D = 26.3;               %    [m]          Thickness permeable layer
rw = .15;            %    [m]          Well radius
tp = .004 ;            %    [m]          Pipe thickness
g = 9.81;             %    [m/s2]       Gravity constant
C = 130;              %    [-]          Hanzen and Williams constant
%W = 50;               %    [m]          Well penetration
He = 0.05;            %    [m]          Entrance losses
%a = 16.5;          %    [m]          Distance between wells, previously
compute with well
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
w = 0.7:.1:1;     %in % of D  0<w<=1
A = 8: 1: 16;
Var = A;
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
Mi = length (w);
Ni = length(Var);

jhav                 = zeros(Mi,Ni);
jHw                  = zeros(Mi,Ni);
jHav                 = zeros(Mi,Ni);
jDM                  = zeros(Mi,Ni);
jQw                  = zeros(Mi,Ni);
jhm                  = zeros(Mi,Ni);
jHm                  = zeros(Mi,Ni);
jratioHavoverHm      =  zeros(Mi,Ni);
tetaa                = zeros(Mi,Ni);
tetam                = zeros(Mi,Ni);
rel1                 =  zeros(Mi,Ni);
Dovera               = zeros(Mi,Ni);
WoverD               = zeros(Mi,Ni);
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
for   j = 1:length(Var)
    %j
 a= Var(j);
for i = 1:length(w)

    W  = w (i);

%%%%%%Finding values of teta%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

%Dteta = 1;                          % for fully penetrating well%%%
for other cases [100%-1, 75%- 0.489, 50%-0.733, 25% - 1.466, 15%- 2,077]

e = 1;

ha = d*(RoSoil-RoWater)/RoWater;     %    [m] Allowable head


%%%assumptions for firsttrial;
```

```
Hav = 0.2* ha;
input1=W;  % 100% - 5%(W/D)
input2=D/a;     % 0.25 - 4.0

%%%%%%To find well factors%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
Tetas; %script to find well factors
Dteta = deltateta ;
tetaao = tetaA  ;
tetamo = tetaM ;
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
                tetaa2 = abs(tetaao+ Dteta * (log10 (a/rw )-2));
                tetam0 = abs(tetamo+ Dteta * (log10 (a/rw )-2));
                tetaa = tetaa2;
                tetam = tetam0;
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%Main matter finding Hav%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
while e>0.01 ;
Hav = Hav + 0.001 ;
DM = abs(((H-Hav)/S)-Hav/X3) ;
Qw = a*DM*kf*D ;
%%%%%Frictional Losses%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
q = Qw * 15850;             %Converting to [gal/min] to use Hanzen Williams
formula
dh = 2 * (rw-tp) ;    % dh inch
dhi = dh*39.3701 ;    % dh inch
f = 0.2083 * ((100/C)^(1.852))*(q^(1.852))/(dhi^(4.8655));
Hf = f * W * 0.01*D ;
%%%%%%Velocity Losses%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
v = Qw/(pi*((dh/2)^2));
Hv = (v^2)/(2*g);
%%%%%%Total Losses%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
Hw = He + Hv + Hf ;
hav = Hav - Hw ;
tetaa1 = hav/(a * DM ) ;
e = abs(tetaa1-tetaa2); %%%%%%checking well factors convergence
%%%%%%Storing results%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
jtetaa (i,j)= tetaao;
jhav(i,j)=hav;
jHw(i,j)=Hw;
jHav (i,j)= Hav;
jDM(i,j)=DM;
jQw(i,j)=Qw;
jhm(i,j)=hav*tetam0/tetaa2;
jHm (i,j)= hav*tetam0/tetaa2 + Hw ;
jratioHavoverHm(i,j)= Hav/((hav*tetam0/tetaa2)+Hw);
jtetaao (i,j) = tetaao;
jtetaA (i,j) = tetaa2;
jtetaM (i,j) = tetam0;
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
end
%%%%%%Storing results well factors%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
Dovera(i,j)=D/a;
JDteta (i,j)= deltateta ;
Jtetaao (i,j)= tetaA  ;
Jtetamo(i,j) = tetaM ;
end
end
```

```
Script for deterministic design relief wells

clear;
%%%%%%%%%Input Data%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
RoSoil = 1420;      %   [kg/m3]
RoWater = 1000;     %   [kg/m3]
Roberm = 1600 ;     %   [kg/m3]
d =11.03 ;          %   [m]        thickness of impervious layer
H = 4.16;           %   [m]        Water level riverside
%S = 20;            %   [m]        Distance Foreland
%X3 = 8.15;         %   [m]        Distance Hinterland]
kb = 9.64E-9;
kf = .00087;        %   [m/s]      permeability
D = 83.45;          %   [m]        Thickness permeable layer
rw = .225;          %   [m]        Well radious
tp = .004 ;         %   [m]        Pipe thickness
g = 9.81;           %   [m/s2]     Gravity constant
C = 125;            %   [-]        Hanzen and Williams constant
%W = 30;            %   [m]        Well penetration
He = 0.5;           %   [m]        Entrance losses
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
Rosand = 1600; %[kg/m3]

%For checking flow regime
udy= 1.002E-3; %[kg/(m*s)
np= 0.4; %sand pososity
D10 = 0.000147; %(m)
D60 = 0.00027; %(m)
D70 = 0.00031; %(m)

SF = 2; %safety factor
Line = 'X3 (m)'; %For plotting purpose
L = 199.48; %this is useful when X3 and S can be designed

e0=0.001 ; %defining error accuracy
da = 0.01 ; %defining increment of a%%%
VX =134:1:134;
w = 0.1:.1:1;                              %in % of D  0<w<=1
Var = VX; %determine the variable for designing

%max net seepage slope regarding max Reynolds
DMmax = udy*np/(D60*kf*RoWater);
%%%%%%P real locating%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
Mi = length (w);
Ni = length(Var);
Fha  =zeros(Mi,Ni);
Fa   =zeros(Mi,Ni);
FQw  =zeros(Mi,Ni);
FQw1 =zeros(Mi,Ni);%corrected
Fhav = zeros(Mi,Ni);
FHav = zeros(Mi,Ni);
Fhm  = zeros(Mi,Ni);
FHw  = zeros(Mi,Ni);
FHm  = zeros(Mi,Ni);
Frel1 =zeros(Mi,Ni);
Frel = zeros(Mi,Ni);
FDovera = zeros(Mi,Ni);
FWoverD = zeros(Mi,Ni);
FtetaA = zeros(Mi,Ni);
FtetaM = zeros(Mi,Ni);
```

```
FHoverD = zeros(Mi,Ni);
FDM = zeros(Mi,Ni);
ratioHavoverHm= zeros(Mi,Ni);

%FteataA =zeros(Mi,Ni);
%FtetaM  = zeros(Mi,Ni);
%FDtetaT = zeros(Mi,Ni);
%cu = 0;
p= 0; %counter just for head reduction check
for j= 1:length(Var)
   X3 = Var(j);
   S= L-X3; %When S is in function of X3 and L
    for i = 1:length(w)
        Hamax = (H*X3-DMmax*S*X3)/(X3+S);
        DMmax1 = (((H-Hamax)/S)-Hamax/X3);
    W  = w(i);
    Lo=1000;
    a = D/Lo;                          % min a
e1 = 4;
e2 = 4;
dis=0;
ha = (d*(RoSoil-RoWater)/RoWater)/SF;     %    [m] Allowable head

xi = (H)/(S+X3)*X3; %existing head at well point
Cl = (kb/(kf*d*D))^(1/2);
 Xc = 1/Cl;                                    %leakage length= X3
 Fc = tanh(Cl)/Cl;

                if X3<Xc
                X3 = X3*Fc;
                else
                X3 = Xc;
                disp('X3 to big check the value, computation is carried
with Xc')
                end
S = S*Fc;
%%%%%%Defining max number of iterations a max=4D%%%%%%%%%%%%%%%%%%%%%%%%%
    amax=  110*D;
    maxiter = (amax)/da;

     n=0;
     iter = 0;



            if p==0
            if ha>xi %checking if head reduction is needed
                disp('Actual head is smaller than MAx alloawble head, no
need for head reduction')
                disp('Actual head is smaller than MAx alloawble head, no
need for head reduction')
                pa = num2str (xi);
                po = num2str (ha);
                Warn = ['Head ' , pa ,' Ha = ', po];
                disp (Warn);

                break
            else
                headreduction = 1- ha/xi;
                pri = num2str (headreduction);
```

```
                Warn3 = ['Head Reduction' , pri ];
                disp (Warn3);
                p = 1;
            end
            end
%%%%%%assumptions for first trial%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
Hav = ha;
DM = abs(((H-Hav)/S)-Hav/X3);
input1= W ; % 100% - 5%(W/D)
       while e1 >e0


            while dis==0

                while  e2 >e0

                    while dis==0

  if n < maxiter
%%%%%%Calculations%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

a = a + da;                      %       first assumption
Qw = a*DM*kf*D;

input2=D/a ;    % 0.25 - 4.0

input2op=[0.25,0.5,1,2,3,4];

%%%%%%To find well factors%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
Tetas; %script to find well factors

%%%%%%Finding teta for different a/rw relation%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

                tetaa2 = abs(tetaao+ Dteta * (log10 (a/rw )-2));
                tetam1 = abs(tetamo+ Dteta * (log10 (a/rw )-2));
                tetaa = tetaa2;
                tetam = tetam1;

%%%%%%Frictional Losses%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

q = Qw * 15850  ;             %Converting to [gal/min] to use Hanzen
Williams formula
dh = 2 * (rw-tp) ;   % dh m
dhi = dh*39.3701  ;  % dh inch
f = 0.2083 * ((100/C)^(1.852))*(q^(1.852))/(dhi^(4.8655));
Hf = f * W * 0.01*D ;
%%%%%%%Velocity Losses%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

v = Qw/(pi*((dh/2)^2));
vi = v*3.28084; %velocity in feet
di = dh*3.28084; % dh feet
Qwf = Qw*3.28084; %Q in ft3/s
Hf10 = 303*(vi^(1.85))/((C^1.85)*(di^1.67)); %Losses Ft/100 Ft of pipe
Hf1 = Hf10* (W*D*3.28084/100)*((100/C)^1.85);
Hv = (v^2)/(2*g);
```

```
%%%%%%Total Losses%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
Hw = He + Hv + Hf;

hav = Hav - Hw;
tetaa1 = hav/(a * DM );
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
rel= tetaa / tetam;
%%%%%%Checking well factors_in order to define critical head max{hav,Hm%%%%
        if rel < 1 %(tetaa/tetam)
            %%%step 16%%%%%

            Hm = ha  ;                                          %%%%16
            hm = Hm - Hw     ;                                  %%%%16
            hav1 = rel * hm ;                                   %%%17
            Hav1 = hav1 + Hw ;                                  %%%%18
            DM1 = abs(((H-Hav1)/S)-Hav1/X3 )       ;            %%%%%19
            tetam2= hm/(a*DM1);
            %tetam3 = abs(tetamo + Dteta * (log10 (a/rw)-2));
            e1 = abs(tetam2-tetam);
            DM=DM1;
            Qw1= a*DM1*kf*D;
            iter = iter + 1;
            %e11 = e
        else

            hav1=hav;
            Hav1 = Hav;
            DM1 = abs(((H-Hav1)/S)-Hav1/X3 );
            hm =tetam1*a*DM1;
            Hm = hm + Hw;
            DM=DM1;

            e2 = abs(tetaa1-tetaa2);
            Qw1= a*DM1*kf*D;
            iter = iter + 1;
            %e12=e
        end

                if e1<e0

                    dis=1;

                end

                if e2<e0

                    dis=1;

                end

                    if dis==1
                        e1=0.0001;
                        e2=0.0001;
                    end
Dover=D/a;
  n= iter;
  else
      pa = num2str (w(i));
      po = num2str (Var(j));
```

```
      disp('No convergence')
      Warn = ['For ' , Line , po ,' and W/D = ', pa];

disp (Warn);
                        e1=0.0001;
                        e2=0.0001;
                        dis=1;
  end   %%% stop for iterations

                    end
                end %end of while e2>0.01
            end
        end %end of while e1>0.01
  %{
 if v> 0.03 %checking velocities on rising pipe the limit of 0.03 m/s is
 just for theflow on the well sceen
  disp('Change well diameter velocities are too high')

      dhmin = ((Qw/(0.03*pi))^(1/2))*2;
  pa = num2str (Qw);
      po = num2str (rw);
      pl =  num2str (dhmin);


      Warn = ['For Qw = ' ,pa,'(m3/s)' ,' and rw = ', po,'m'];
      Warn1 = ['dhmin = ' ,pl];
      disp (Warn);
      disp (Warn1);

 end
%}
   %n
tetal=tetaA;
vf = kf*DM;
vs = vf/np;
reD10= RoWater*vs*D10/udy;
reD60= RoWater*vs*D60/udy;
reD70= RoWater*vs*D70/udy;
wf = DM*D/(4*(Hav1-Hw)); %well factor to check min well penetration
Fha(i,j)=ha;
Fa (i,j)= a;
FQw(i,j)= Qw;
FQw1(i,j)=Qw1 ;%corrected
Fhav(i,j) = hav1;
FHav (i,j)= Hav1;
Fhm (i,j) = hm;
FHw (i,j)= Hw;
FHwf (i,j)= Hf;
FHwf1 (i,j)= Hf1;
FHwV (i,j)= Hv;
FHm(i,j)= Hm;
Frel1(i,j)=rel;
Frel(i,j)=1/rel;
FDovera(i,j)=D/a;
FWoverD(i,j)=W/D;
FtetaA(i,j) = tetaa;
FtetaM(i,j)= tetam;
FHoverD(i,j)= H/D;
FDM(i,j)= DM;
ratioHavoverHm(i,j)= Hav1/Hm;
```

```
FS (i,j) = S;
FH (i,j)  = H;

Fqwft (i,j) = Qwf;
Vf (i,j)= vf;
Vs (i,j)= vs;
ReD10(i,j)= reD10;
ReD60(i,j)= reD60;
ReD70(i,j)= reD70;
Xi (i,j)= xi ;
WF (i,j) = wf;
Fvwell (i,j) = v; %velocity on rising piping
    end
end
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
Hai= (d*(RoSoil-RoWater)/RoWater);
%PlotWelldesing1; for plotting purpose well design
%plotheads015WD; for plotting purpose well piezometric head
```

```
Script to find well factors
in1= input1;
in2= input2 ;

%D = 66;
%rw = .2;
Wellpene = [0.05;0.1;0.15;0.25;0.5;0.75;1];
Dovera = [0.25;0.5;1;2;3;4];
Te_A = [1.778   1.908   1.662   1.225   0.742   0.523   0.44
3.879   2.934   2.31    1.569   0.857   0.563   0.44
6.063   3.977   2.97    1.926   0.983   0.606   0.44
8.377   5.139   3.747   2.39    1.175   0.678   0.44
9.761   5.977   4.344   2.798   1.361   0.748   0.44
11.144  6.814   4.941   3.199   1.547   0.818   0.44
];
Te_M = [1.887   2.018   1.772   1.335   0.851   0.633   0.55
3.969   3.025   2.401   1.622   0.955   0.667   0.55
6.021   3.941   2.938   1.908   1.012   0.681   0.55
7.864   4.649   3.293   2.024   1.024   0.682   0.55
8.574   4.86    3.363   2.047   1.024   0.682   0.55
9.283   5.071   3.432   2.075   1.024   0.682   0.55];

Dtet = [6.963   3.298   2.077   1.466   0.733   0.489   0.367
6.963   3.298   2.077   1.466   0.733   0.489   0.367
6.963   3.298   2.077   1.466   0.733   0.489   0.367
6.963   3.298   2.077   1.466   0.733   0.489   0.367
6.963   3.298   2.077   1.466   0.733   0.489   0.367
6.963   3.298   2.077   1.466   0.733   0.489   0.367];

pl=0;
 if input1>1
    input1=1;
end

if input1==1
            deltateta= 0.367 ;
            tetaA = 0.440 ;
            tetaM = 0.550;
            pl=1;
else
    tetaA  = interp2(Wellpene,Dovera,Te_A,input1,input2);
    tetaM = interp2(Wellpene,Dovera,Te_M,input1,input2);
    deltateta  = interp2(Wellpene,Dovera,Dtet,input1,input2);
end

%%%%%%%Assigning tetas
Dteta = deltateta ;
tetaao = tetaA  ;
tetamo = tetaM ;
```

Results Case Studies

W/D	a	Beta	Cost/m
Ring36_1			
0.6	10	3.426116	379.8704
0.6	11	2.529018	345.3367
0.6	12	2.222566	316.5586
0.6	13	1.893614	292.208
0.6	14	1.573235	271.336
0.6	15	1.282598	253.2469
0.6	16	1.009179	237.419
0.7	10	4.976214	414.2234
0.7	11	4.542864	376.5667
0.7	12	4.021078	345.1862
0.7	13	2.759308	318.6334
0.7	14	2.509364	295.8739
0.7	15	2.224581	276.1489
0.7	16	1.936868	258.8896
0.8	10	5.408366	448.5765
0.8	11	5.252329	407.7968
0.8	12	5.091595	373.8137
0.8	13	4.677174	345.0588
0.8	14	4.216979	320.4118
0.8	15	3.638057	299.051
0.8	16	2.648083	280.3603
0.9	10	5.5273	482.9295
0.9	11	5.420341	439.0269
0.9	12	5.281961	402.4413
0.9	13	5.191996	371.4843
0.9	14	4.83973	344.9497
0.9	15	4.451932	321.953
0.9	16	3.976398	301.831
1	10	5.629503	517.2826
1	11	5.532674	470.2569
1	12	5.442408	431.0688
1	13	5.303561	397.9097
1	14	5.156845	369.4876
1	15	5.032892	344.8551
1	16	4.693652	323.3016

W/D	a	Beta	Cost/m
Ring36_2			
0.6	12	1.809102	316.5586
0.6	13	1.512969	292.208
0.6	14	1.234879	271.336
0.6	15	0.960986	253.2469
0.6	16	0.692808	237.419
0.7	12	3.480699	345.1862
0.7	13	2.951797	318.6334
0.7	14	2.485134	295.8739
0.7	15	2.115376	276.1489
0.7	16	1.802972	258.8896
0.8	12	4.968578	373.8137
0.8	13	4.595728	345.0588
0.8	14	4.171014	320.4118
0.8	15	3.70034	299.051
0.8	16	3.176164	280.3603
0.9	12	5.310299	402.4413
0.9	13	5.154798	371.4843
0.9	14	4.991563	344.9497
0.9	15	4.605427	321.953
0.9	16	4.188372	301.831
1	12	5.469569	431.0688
1	13	5.35845	397.9097
1	14	5.230718	369.4876
1	15	5.089082	344.8551
1	16	4.95433	323.3016

W/D	a	Beta	Cost/m
Ring36_3			
0.8	9	5.515621	343.9964
0.8	10	5.353098	309.5968
0.8	11	5.246339	281.4516
0.8	12	4.841072	257.9973
0.8	13	4.380557	238.1514
0.9	9	5.611392	362.8638
0.9	10	5.493366	326.5774
0.9	11	5.310093	296.8885
0.9	12	5.219935	272.1478
0.9	13	4.870101	251.2134
1	9	5.669236	381.7311
1	10	5.56882	343.558
1	11	5.473382	312.3255
1	12	5.324175	286.2983
1	13	5.262691	264.2754

W/D	a	Beta	Cost/m
Ring36_4			
0.6	10	5.147922	275.6356
0.6	11	4.643303	250.5778
0.6	12	4.01065	229.6963
0.6	13	3.325522	212.0274
0.6	14	2.69384	196.8826
0.6	15	2.188179	183.7571
0.6	16	1.765258	172.2723
0.7	10	5.29691	292.6162
0.7	11	5.104253	266.0147
0.7	12	5.056196	243.8468
0.7	13	4.56911	225.0894
0.7	14	4.032078	209.0116
0.7	15	3.459557	195.0775
0.7	16	2.892187	182.8851
0.8	10	5.466391	309.5968
0.8	11	5.333725	281.4516
0.8	12	5.182051	257.9973
0.8	13	4.998702	238.1514
0.8	14	4.893796	221.1406
0.8	15	4.44357	206.3979
0.8	16	3.949952	193.498
0.9	10	5.549969	326.5774
0.9	11	5.43665	296.8885
0.9	12	5.311145	272.1478
0.9	13	5.169761	251.2134
0.9	14	4.99136	233.2696
0.9	15	4.946859	217.7183
0.9	16	4.550034	204.1109
1	10	5.627869	343.558
1	11	5.531557	312.3255
1	12	5.442432	286.2983
1	13	5.306668	264.2754
1	14	5.177725	245.3986
1	15	5.030159	229.0387
1	16	4.846103	214.7238

W/D	a	Beta	Cost/m
Ring36_5			
0.6	13	2.885493	212.0274
0.6	14	2.457859	196.8826
0.6	15	2.055393	183.7571
0.6	16	1.674338	172.2723
0.6	17	1.309315	162.1386
0.6	18	0.962692	153.1309
0.7	13	4.291948	225.0894
0.7	14	3.847922	209.0116
0.7	15	3.414108	195.0775
0.7	16	2.995459	182.8851
0.7	17	2.578696	172.1272
0.7	18	2.180875	162.5646
0.8	13	5.070962	238.1514
0.8	14	4.885924	221.1406
0.8	15	4.507169	206.3979
0.8	16	4.051448	193.498
0.8	17	3.620972	182.1158
0.8	18	3.21199	171.9982
0.9	13	5.247963	251.2134
0.9	14	5.104891	233.2696
0.9	15	4.94275	217.7183
0.9	16	4.754339	204.1109
0.9	17	4.312931	192.1044
0.9	18	3.907544	181.4319
1	13	5.402447	264.2754
1	14	5.287457	245.3986
1	15	5.161858	229.0387
1	16	5.021343	214.7238
1	17	4.861483	202.0929
1	18	4.68429	190.8656

W/D	a	Beta	Cost/m
Ring36_6			
0.6	13	1.85711	212.0274
0.6	14	1.543574	196.8826
0.6	15	1.25551	183.7571
0.6	16	0.983916	172.2723
0.6	17	0.714416	162.1386
0.6	18	0.449035	153.1309
0.7	13	2.710817	225.0894
0.7	14	2.477576	209.0116
0.7	15	2.189826	195.0775
0.7	16	1.899089	182.8851
0.7	17	1.602712	172.1272
0.7	18	1.332184	162.5646
0.8	13	4.637053	238.1514
0.8	14	4.165006	221.1406
0.8	15	3.57349	206.3979
0.8	16	2.600071	193.498
0.8	17	2.338542	182.1158
0.8	18	2.076428	171.9982
0.9	13	5.175776	251.2134
0.9	14	4.795665	233.2696
0.9	15	4.393778	217.7183
0.9	16	3.935625	204.1109
0.9	17	3.391958	192.1044
0.9	18	2.565068	181.4319
1	13	5.678201	264.2754
1	14	5.356121	245.3986
1	15	5.016978	229.0387
1	16	4.658537	214.7238
1	17	4.276992	202.0929
1	18	3.863086	190.8656

W/D	a	Beta	Cost/m
Ring36_7			
0.6	8	2.493829	523.8205
0.6	9	2.148695	465.6182
0.6	10	1.809951	419.0564
0.6	11	1.483545	380.9603
0.6	12	1.164836	349.2136
0.6	13	0.855794	322.351
0.6	14	0.556424	299.326
0.7	8	3.362145	566.7618
0.7	9	3.048096	503.7882
0.7	10	2.731618	453.4094
0.7	11	2.405418	412.1904
0.7	12	2.081068	377.8412
0.7	13	1.768332	348.7765
0.7	14	1.469054	323.8639
0.8	8	4.014979	609.7031
0.8	9	3.74271	541.9583
0.8	10	3.457937	487.7625
0.8	11	3.157732	443.4204
0.8	12	2.826263	406.4687
0.8	13	2.498879	375.2019
0.8	14	2.179285	348.4018
0.9	8	4.414439	652.6444
0.9	9	4.108743	580.1284
0.9	10	3.812751	522.1155
0.9	11	3.516557	474.6505
0.9	12	3.220935	435.0963
0.9	13	2.91946	401.6273
0.9	14	2.618929	372.9397
1	8	5.102578	695.5858
1	9	4.443253	618.2984
1	10	4.160069	556.4686
1	11	3.879179	505.8805
1	12	3.611962	463.7238
1	13	3.333011	428.0528
1	14	3.060938	397.4776

W/D	a	Beta	Cost/m
Ring52_1			
0.8	16	4.535324	169.3333
0.8	17	3.910975	159.3725
0.8	18	3.109239	150.5185
0.8	19	2.813673	142.5965
0.8	20	2.498493	135.4666
0.9	16	5.136499	176.9256
0.9	17	4.636573	166.5182
0.9	18	4.036432	157.2672
0.9	19	3.080496	148.99
0.9	20	2.898377	141.5405
1	16	5.044774	184.5179
1	17	5.21536	173.6639
1	18	4.790149	164.0159
1	19	4.250758	155.3835
1	20	3.193137	147.6143

W/D	a	Beta	Cost/m
Ring52_2			
0.6	16	2.066377	230.3165
0.6	17	1.662455	216.7685
0.6	18	1.266875	204.7258
0.6	19	0.881911	193.9507
0.6	20	0.519949	184.2532
0.7	16	3.235289	245.0113
0.7	17	2.800525	230.5988
0.7	18	2.411935	217.7878
0.7	19	2.044743	206.3253
0.7	20	1.698792	196.009
0.8	16	4.658844	259.706
0.8	17	4.138416	244.4292
0.8	18	3.37214	230.8498
0.8	19	2.976386	218.6998
0.8	20	2.621999	207.7648
0.9	16	4.856151	274.4008
0.9	17	4.384927	258.2595
0.9	18	4.392187	243.9118
0.9	19	3.649	231.0743
0.9	20	3.245212	219.5206
1	16	5.203753	289.0955
1	17	4.975307	272.0899
1	18	4.742669	256.9738
1	19	4.671173	243.4489
1	20	4.265483	231.2764

W/D	a	Beta	Cost/m
Ring52_3			
0.6	16	2.603725	187.7017
0.6	17	2.17408	176.6605
0.6	18	1.76318	166.846
0.6	19	1.364638	158.0646
0.6	20	0.997655	150.1614
0.7	16	3.840099	195.294
0.7	17	3.288804	183.8061
0.7	18	2.758983	173.5947
0.7	19	2.369461	164.4581
0.7	20	2.01728	156.2352
0.8	16	4.513643	202.8863
0.8	17	3.959743	190.9518
0.8	18	3.754569	180.3434
0.8	19	3.236295	170.8516
0.8	20	2.814871	162.3091
0.9	16	4.857867	210.4786
0.9	17	4.571705	198.0975
0.9	18	4.052916	187.0921
0.9	19	3.687808	177.2451
0.9	20	3.313357	168.3829
1	16	5.146236	218.0709
1	17	4.922057	205.2432
1	18	4.665951	193.8408
1	19	4.200951	183.6386
1	20	3.829078	174.4567

W/D	a	Beta	Cost/m
Ring52_4			
0.6	16	2.800869	187.7017
0.6	17	2.361994	176.6605
0.6	18	1.934569	166.846
0.6	19	1.516143	158.0646
0.6	20	1.123254	150.1614
0.7	16	3.777214	195.294
0.7	17	3.357702	183.8061
0.7	18	2.951961	173.5947
0.7	19	2.557311	164.4581
0.7	20	2.187996	156.2352
0.8	16	4.636752	202.8863
0.8	17	4.265137	190.9518
0.8	18	3.719604	180.3434
0.8	19	3.351145	170.8516
0.8	20	2.996101	162.3091
0.9	16	4.927642	210.4786
0.9	17	4.67072	198.0975
0.9	18	4.378734	187.0921
0.9	19	3.841973	177.2451
0.9	20	3.493463	168.3829
1	16	5.198504	218.0709
1	17	4.979694	205.2432
1	18	4.754722	193.8408
1	19	4.495063	183.6386
1	20	3.995741	174.4567

W/D	a	Beta	Cost/m
Ring52_5			
0.6	16	2.771704	187.7017
0.6	17	2.33033	176.6605
0.6	18	1.901311	166.846
0.6	19	1.482586	158.0646
0.6	20	1.090273	150.1614
0.7	16	3.777385	195.294
0.7	17	3.338621	183.8061
0.7	18	2.924934	173.5947
0.7	19	2.526105	164.4581
0.7	20	2.154825	156.2352
0.8	16	4.577724	202.8863
0.8	17	4.128795	190.9518
0.8	18	3.718799	180.3434
0.8	19	3.332904	170.8516
0.8	20	2.96851	162.3091
0.9	16	5.065939	210.4786
0.9	17	4.636099	198.0975
0.9	18	4.23269	187.0921
0.9	19	3.840813	177.2451
0.9	20	3.477892	168.3829
1	16	5.552344	218.0709
1	17	5.147342	205.2432
1	18	4.762202	193.8408
1	19	4.374992	183.6386
1	20	4.003927	174.4567

W/D	a	Beta	Cost/m
Ring52_6			
0.6	5	4.363122	600.6456
0.6	6	4.182763	500.538
0.6	7	3.982998	429.0325
0.6	8	3.736254	375.4035
0.6	9	3.461288	333.692
0.6	10	3.181074	300.3228
0.7	5	4.559679	624.9409
0.7	6	4.402843	520.7841
0.7	7	4.208807	446.3863
0.7	8	3.999121	390.5881
0.7	9	3.774065	347.1894
0.7	10	3.54371	312.4704
0.8	5	4.65716	649.2362
0.8	6	4.510533	541.0302
0.8	7	4.346702	463.7401
0.8	8	4.170192	405.7726
0.8	9	3.983802	360.6868
0.8	10	3.785842	324.6181
0.9	5	4.699454	673.5315
0.9	6	4.564356	561.2763
0.9	7	4.417121	481.0939
0.9	8	4.259931	420.9572
0.9	9	4.087596	374.1842
0.9	10	3.918097	336.7658
1	5	4.74239	697.8268
1	6	4.617175	581.5224
1	7	4.493867	498.4477
1	8	4.346126	436.1418
1	9	4.198773	387.6816
1	10	4.035457	348.9134

Unit Price Analysis

Table 1: UNIT PRICES ANALYSIS

Item	**Concrete back-filling (u)**			
Unit	Description	Quantity	Unit Price	Price
kg	Worked steel includes accessories, painting (galvanizing), labour	20	2.6	52.16
%	Indirect costs			
			Total:	52.16

Table 2: UNIT PRICES ANALYSIS

Item	**Concrete backfilling (u)**			
Unit	Description	Quantity	Unit Price	Price
m^3	Concrete class	0.07	276.00	20.14
global	Labour	1.00	4.09	4.09
%	Indirect costs			
			Total:	24.22

Table 3: UNIT PRICES ANALYSIS

Item	**Excavation (m)**			
Unit	Description	Quantity	Unit Price	Price
h	Drilling equipment	0.08	80.00	6.40
global	Labour	0.10	4.00	0.40
%	Indirect costs			
			Total:	6.80

Table 4: UNIT PRICES ANALYSIS

Item	Filter Material (m)			
Unit	Description	Quantity	Unit Price	Price
m^3	Granular material special gradation	0.03	21.67	0.67
global	Labour	0.10	4.00	0.40
%	Indirect costs			
			Total:	1.07

Table 5: UNIT PRICES ANALYSIS

Item	Rising Pipe PVC (m)			
Unit	Description	Quantity	Unit Price	Price
m	PVC pipe (160 mm diameter)	1.05	10.89	11.43
global	Labour	0.08	4.00	0.32
%	Indirect costs			
			Total:	11.75

Table 6: UNIT PRICES ANALYSIS

Item	Berm material (m3)			
Unit	Description	Quantity	Unit Price	Price
m^3	Berm material	1.400	16.50	23.10
global	Labour	0.080	4.00	0.32
h	Dumper	0.060	16.50	0.99
%	Indirect costs			
			Total:	24.41

Table 7: UNIT PRICES ANALYSIS

Item	**Compaction (m3)**			
Unit	Description	Quantity	Unit Price	Price
h	tanker truck	0.006	45.02	0.27
h	Vibratory roller	0.03	35.45	1.03
global	Labour	0.100	4.00	0.40
%	Indirect costs			
			Total:	1.70