Levees and levee evaluation
The Dutch and US practice compared
Msc. Thesis

Fugro Ingenieursbureau BV
Delft University of Technology

P.R.M. Ammerlaan
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Picture front page: Levees along the San Joaquin River California at Reclamation District 17; 01-29-2007
P.R.M. Ammerlaan

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Preface

This report is the final version of my master thesis for Hydraulic Engineering at Delft University of Technology with the title: "Levees and levee evaluation, the Dutch and US practice compared". An immediate question that will rise is probably: "What is a levee?" In the Netherlands we are familiar with the word dike as a translation of the Dutch 'dijk'. The word dike is sometimes used in the US, but in California as well as Louisiana levees is the preferred word, derived from the French word levée and introduced in New Orleans by the French in the 17th century. I will therefore only speak of levees in this report and will not use the word dike. This levee subject was suggested to me by Fugro Ingenieursbureau BV in the Netherlands and they supported me during this thesis and offered me the chance to learn about levees and levee evaluation in the US. I have spent 9 weeks in the Fugro West inc. office in Oakland, California and have been to Houston, Baton Rouge and New Orleans as well to talk about and see levees. The final result of the past 10 months is a report which is not only theoretical (chapter 7 is a theoretical/empirical study of the mechanism piping), but is also a very broad introduction to levees and levee evaluation in the Netherlands as well as in the US, especially California and gives an overview of the differences and similarities. I hope that for that reason people who are interested in an exchange of knowledge between the US and the Dutch levees will use my report as a first start. The more theoretical part of this thesis is hopefully triggering people to study the mechanism piping more closely and critically look at the current design and evaluation criteria in the Netherlands as well as the USA.

I would like to thank the people who supported me during my thesis: Professor Han Vrijling and Pieter van Gelder from my university for their comments and support, Martin van der Meer from Fugro for giving directions and critical comments and Job Nijman from Fugro for his support before and during my visit to the US.

Patricia Ammerlaan

August, 2007

Summary

The Central Valley and Sacramento-San Joaquin Delta in California are identified as extremely vulnerable to major flood disasters of the size as the 2005 New Orleans flood. With its low-lying polders and rivers flowing into a delta this part of California shows some remarkable similarities with the Netherlands. The first goal of this report was to identify the vulnerabilities of the Central Valley and Delta flood protection system and to compare them to the Dutch water defense system. A second goal was then to focus on important weaknesses or differences, try to find out their background and/or come up with recommendations on how to improve them.

This Central Valley receives runoff from the Sierra Nevada Mountains, which is drained mainly in the Sacramento River and the San Joaquin River, passing densely populated areas with the cities Stockton and Sacramento. These rivers drain in the Delta, which is the center of a large north-south water delivery system More than 22 million people (2/3 of the Californian population) partly rely on drinking water and irrigation water from the Delta. More than 3,800 km of levees protect the Central Valley and Delta, against floods. The Central Valley and Delta levees are supposed to provide a 100-year flood protection and in current evaluation program this is raised to a 200-year level. There is no federal flood protection standard. This 100-year protection is a requirement from the Federal Emergency Management Agency (FEMA), who carries out the National Flood Insurance Program (NFIP).

The River levees in the Central Valley are subject to seasonal floods in spring, when melting snow in the Sierra Nevada increases runoff, while the Delta levees hold back water the entire year. Although the Delta is situated in a relatively protected area, sheltered from the ocean, tidal influences and wind wave actions from the San Francisco Bay can still harm the area. From none of the 162 delta levee breaches of last century an indication was found that it was caused by a seismic event. However, there are people that believe that one of the most important threats to the delta is an earthquake, especially in combination with high water levels.

If the levees fail, or maybe better: when the levees fail in the Delta and/or along the rivers, the consequences are enormous. River levee failures will mainly be destructive to urban areas. Sacramento alone has already more than 450,000 inhabitants. When levees in the Delta fail, salt water will be drawn into the area. Not only people, species and infrastructure within the Delta will be harmed by the salt water, but also the people that rely on drinking and irrigation water from the Delta.

The levees are degrading from erosion and subsidence. The changing climate and growing population will ask more from those already vulnerable levees. Plans are developed to improve the flood protection in the Central Valley and Delta. Most of these plans are now gathered under “FloodSAFE California”, an initiative of the DWR (Department of Water Resources) of California. Evaluation of the urban levees is a currently running initiative financed with state bond debts.

25% of the Netherlands is situated below mean sea level. And in total 60% of the Dutch land area would be flooded daily without levees, dunes and barriers. Most of the economic activity and urbanization is in this part of the country. The Dutch water defenses are divided in primary water defenses and regional water defenses. The water levels against those primary water defenses, protecting 53 dike ring areas, are influenced by the tide, waves, storm surges and/or river discharges from the North Sea and the Rivers Meuse and Rhine. They have to be able to resist a water level with an occurrence of 1:10,000 per year to 1:1,250 per year, depending on economic

consequences within the dike ring area. Regional water defenses lie within these dike ring areas and often encircle polders with a regulated water level. Water levels at the regional levees are kept relatively constant. Large infrastructure works have been built to protect the Dutch polders. Storm surge barriers as in the Western Scheldt and the Nieuwe Waterweg are examples of this. Another example is the closure dam between the Wadden Sea and Lake IJssel, which was built to protect people living along the former Zuyder Sea against storm surges.

But the climate is changing and floods seem to occur more often than they did before. Main drivers of an increased flood risk are the changing climate, which leads to sea level rise, increasing river discharges and increasing wet and dry periods. As in the rest of the world population growth and economic growth make that the damage floods cause is increasing.

One of the initiatives to deal with flood risks in the future is the FLORIS project, Flood Risks and Safety in the Netherlands (or in Dutch: VNK). It intends to get more insight in the chances of flooding and the consequences of a flood. Another initiative is the Room for the River project, established after the 1993 and 1995 extreme river water levels; goal is to find new solutions for a better protection against the water from the large rivers.

In the Flood Protection Act is stated that each 5 years levee authorities have to report on the conditions of the primary water defenses following the prescriptions from the ministry, the 'Voorschift toetsen op veiligheid' (VTV). There is no legislation (yet) on the regional water defenses. How a levee is evaluated in the Netherlands depends on the expected failure mechanisms in an area. Macro instability of the levee, piping, overtopping and micro instability are the main mechanisms considered in a levee evaluation. Other mechanisms, which are evaluated depending on the local levee conditions, are instability of the foreland, instability of the revetment, instability by infiltration and erosion at overtopping, heave and horizontal sliding at foundation. For each of the mechanisms a process from simple to advanced is used, based on a ground model and evaluation methods. From the basic soil research that is prescribed, borings combined with soundings and lab tests, a ground model is developed and the levee is divided into sections with equal characteristics. When a levee section is expected to be vulnerable to a certain failure mechanism with a simple evaluation, more extensive soil research on the specific location and more detailed or advanced models are used. When after several steps a levee still seems vulnerable, that levee section is rejected. Improvements are necessary.

The levee is then evaluated under normative conditions. Often this is the condition with normative high water, combined with wave and wind setup. But precipitation is also sometimes a normative condition or a situation of rapid drawdown after high water. The latter two conditions are important in the evaluation of the macro stability of the levee. Loads caused by traffic on the levee are also taken into account. The stability of the levee is evaluated with computer program MStab, with which Bishop is applied, a method of slices for circular slide planes. If a weak top layer is present behind the levee, which is often the case in the Netherlands, an uplift calculation is also made using for example Uplift Van in MStab.

Piping, which is the forming of a pipe under a levee caused by (concentrated) seepage flow, is evaluated first by doing an uplift check. The weight of the blanket is then compared to the uplifting pressure of the seepage flow under normative high water conditions. If the pressures in the seepage carrying sand layer are able to lift the blanket, rupture of the blanket is possible, resulting in a concentrated seepage flow. If a (critical) pipe will form is then determined with the Bligh formula and/or the Sellmeijer formula.

The US has no federal established guidance for levee evaluation. The methods that are used in levee evaluation are partially withdrawn from the Levee Design Manual of the Army Corps of Engineers (USACE, 2000) and often combined in some sort of Standard Operating Procedure (SOP). The design and analysis procedures for levees in the United States are closely related to procedures for earth dams.

The principal causes of levee failure in California are levee through-seepage and/or under-seepage, wave-induced erosion, flood-induced erosion, current-induced erosion, static instability, levee instability due to sudden drawdown and seismic induced failures. A ground model is made from standard levee investigation, as prescribed, and cross-sections are developed to model the failure mechanisms that are important for the specific levee. When a first evaluation is finished it can be followed by more soil research and again evaluation. Loads important in California levee evaluation are earthquake loads, normative high water levels combined with waves and a situation of rapid drawdown. A stability evaluation is performed in UTEXAS4 or Slope/W using Spencer's method of slices. Different loading conditions of earthquakes combined with certain water levels are considered.

The vulnerability to sand boils and piping is determined by calculating the maximum exit gradient at the toe of the levee and compare it to the critical exit gradient of 0.5. This critical exit gradient was determined from an underseepage research along the Mississippi in the 1950s. The exit gradient is defined as the phreatic head in the seepage carrying sand layer divided by the thickness of the (impermeable) top layer. The phreatic head is determined using blanket equations.

From the above some similarities and differences between the Dutch and US water defense systems and levee evaluation methods can be found. They are shortly mentioned here. Both have a flat low-lying Delta and land below mean sea level is characteristic. Elevations reach until almost 8 m below mean sea level. Systems of rivers flowing into a Delta are somewhat the same: the San Joaquin River and Sacramento River in the Central Valley and the Rivers Meuse and Rhine in the Netherlands. If we compare recent floods again similarities are found as for example the Central Valley River flood of 1997 and the 1993/1995 Rhine and Meuse river floods which were both caused by flood waves from the rivers and where piping was one of the main problems. But there is a large difference in the level of protection that is prescribed in the Netherlands and in the US. A water level with a probability of exceedance of 1/100 or 1/200 per year is the current design level in California, while in the Netherlands the design water level has a probability of exceedance of 1/10,000 to 1/1,250 per year, for primary water defenses. The accepted probability of exceedance in the US was an arbitrary chosen value. Flood insurance, which is obligatory in areas with a less than 1/100 protection, is related to this safety level. In the Netherlands people cannot buy flood insurance. Another difference is that the economic damage in the Netherlands would be mainly limited to the flooded area itself, while in the Delta in California a flood does not only directly affect people in the Delta itself, but it also indirectly influences the rest of California, that depends on fresh water from the Delta.

On the level of levee evaluation an interesting difference is that in the US design documents are currently used for evaluation, while the Dutch have special evaluation documents. A difference in the stability evaluation within the DWR levee geotechnical evaluations compared to the Dutch methods is that uplift is not mentioned in the evaluation. Another one is that in the US the situation of rapid drawdown is performed with partly undrained parameters, while in the Dutch stability evaluation always drained parameters are used. While there is not such a large risk to seismic shaking in the Netherlands as in for example California, there is no seismic evaluation in the Netherlands, while it is performed in California. But the most interesting seems the difference between how piping is evaluated in the US and the Netherlands and this was therefore chosen as a subject for further research.

Piping can become a problem at locations where a thick sand layer is overlain by an impermeable blanket. To compare the formulas used in the US and Dutch piping evaluation, two steps in the piping process are distinguished. The first step is uplift and possible rupture of the blanket. This step is modeled in both evaluations, though conclusions are different. If uplift/rupture is possible a levee in the US is immediately rejected, while in the Netherlands then step 2 is applied. Step 2 is about movement of the sand particles. A levee in the Netherlands is only rejected if with formulas from Bligh or Sellmeijer is found that a critical pipe can develop which forms a threat to stability of the levee. In the US the exit gradient was chosen in such a way that it includes heaving of the sand particles and thus the formation of sand boils.

In both piping evaluations safety is implemented. This is done in two ways: in the parameter choice, with a 20% difference between the Netherlands and the US, and an overall safety factor applied within the formula, which in the uplift and Sellmeijer formula in the Netherlands is 1.2. In the US a 1.6 safety is applied on the theoretical exit gradient of 0.8, resulting in a critical exit gradient of 0.5. The 0.8 was based on a constant blanket thickness, which limits the application of the 0.5 criterion to areas that have a blanket layer with a volume weight above 17.6 kN/m^3.

Cases from the Mississippi research were used to quantify the differences between the criteria. From this became clear that L/H values (the available seepage length divided by the water level difference at the landside and waterside of the levee) at which boils occur at the Mississippi River

(L/H≈43) do not match the values of L/H at which problems are expected in the Netherlands with the current piping evaluation methods (L/H≈max.18). This is mainly caused by a different definition of the critical situation in both countries. The critical situation in the Netherlands is failure of the levee because of excessive growth of the pipe, while the critical situation in the US is occurrence of sand boils, which is bounded by a critical exit gradient of 0.5. In the 1956 research situations were described where piping really was becoming critical. L/H values of these locations are closer to the Dutch criterion, but are not all regarded as unsafe with the Dutch method. This could be caused by the uncertainty in the parameters used, while not enough data from these cases was available, but it also leads to questions about the Dutch criteria. Sellmeijer can identify critical situations above L/H=18, but within the Dutch evaluation rules this is not allowed: Bligh is then assumed normative.

Other causes of differences between the Dutch and US answers to the case studies can be related to the fact that conditions along the Mississippi River are different than conditions along the Dutch rivers, while all Mississippi cases are further upstream than the border of the Netherlands is. Limitations of the Dutch methods are not described together with the methods.

An interesting result from the cases is that there is a band width of about 3.5 between the L/H(≈43) where sand boils occur in the Mississippi and the L/H(≈18) where the pipe is becoming critical for the stability of the levee. The L/H of about 43 seems like a reasonable criterion for levee design: no sand boils are allowed. The L/H of about 18 is more appropriate for levee evaluation only: a critical pipe is not allowed. The result of implementing this would be that seepage berms in the Netherlands would have to become far larger than they currently are.

An overall conclusion is that the discussion on how to best model piping in the Netherlands as well as the US is not solved yet. Cautiousness is recommended as well as further research. Further on an exchange of knowledge between the Dutch and US levee specialists on various subjects concerning water defenses could be useful for both the American levees as the Dutch levees! Cooperation between the two countries should be stimulated and welcomed.

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List of abbreviations

DRMS	Delta Risk Management Strategy
DWR	Department of Water Resources
FEMA	Federal Emergency Management Agency
IPO	Inter Provincial Consultation (in Dutch: Inter Provinciaal Overleg)
Min of V&W	Ministry of Transport, Public Works and Transport (in Dutch: Ministerie van Verkeer en Waterstaat)
NAP	Amsterdam Reference Water Level (in Dutch: Normaal Amsterdams Peil)
NHW	Normative High Water or project flood (in Dutch: MHW, Maatgevend Hoog Water)
NFIP	National Flood Insurance Program
SOP	Standard Operating Procedure
TAW	Technical Advisory Committee for the Flood Defenses (in Dutch: Technische Adviescommissie voor de Waterkeringen)
USACE	United States Army Corps of Engineers
VTV	Prescribed Levee Safety Evaluation (in Dutch: Voorschrift Toetsen Veiligheid)

List of symbols

c= Factor [-]

C_{creep}= Bligh's creep factor [-]

C_v= Variation coefficient [-]

d= Thickness of pervious substratum [m]

d_{sand}= Thickness of the sand layer [m]

D= Thickness of blanket [m]

D_{70}= 70 percent value of the grain distribution of the sand [m]

g= Gravity [m/s^2]

h_p= The level of the top of the sand layer [m + NAP]

h_x= Head Beneath top stratum at distance x from landside levee toe [m]

h_0= Head beneath top stratum at landside levee toe [m]

H= Net head on levee [m]

H_c= Critical head on levee [m]

ΔH= Head difference on levee [m]

ΔH_c= Critical head difference [m]

i= Upward gradient [-]

i_0= Exit gradient at toe of the levee [-]

k_{bl}= Vertical permeability of riverside top stratum [m/day]

k_f= Horizontal permeability of pervious substratum [m/day]

L= Seepage length horizontal [m]

L_{Bligh}= Critical seepage length according to Bligh [m]

L_2= Length of levee at base [m]

L_3= Length of blanket at landside of the levee [m]

n= Number of years [-]

p= Water pressure [kN/m2]

P= Probability of exceedance of a certain water level in n years [1/year]

P_i= Probability of inundation in year i [1/year]

r= Reduced interest rate, which is the interest reduced by inflation and increased with economic growth [-]

R_i= Risk of flooding in year i [euros/year]

S_i= Damage in year i [euros]

T= Flood protection in years [years]

TR= Total Risk [euros]

x_1= Distance from landside levee toe to effective seepage entrance [m]

x_3= Distance from landside levee toe to effective seepage exit [m]

z_b= Thickness of landside top stratum/blanket [m]

z_t= Critical thickness of the blanket [m]

γ= Safety factor [-]

γ_{tot}= Total safety factor; combination of parameter safety and overall safety [-]

γ_p= Saturated weight of the sand [kN/m3]

$\gamma_{w,s}$= The wet volume weight of the top layer [kN/m^3]

γ_w= Volume weight of water [kN/m3]=10 kN/m3

γ'= Submerged unit weight of the blanket soil [kN/m3]

θ= Friction angle of the sand grains [°]

κ= Intrinsic permeability of the sand layer [m2]

μ= Mean of a dataset

η= Drag factor (coefficient of White) [-]

σ= Standard deviation

σ'= Effective stress [kN/m2]

ϕ_s= Hydraulic head in the water-bearing layer [m + NAP]

$\phi_{s,c}$= Phreatic head in the top layer [m + NAP]

ν= Kinematic viscosity [m^2/s]

1 Introduction

After the 2005 disaster in New Orleans, California was identified as America's next area to suffer from a major flood disaster. (Reid R.L, 2005)

Situated between the Sierra Nevada mountain range and the Pacific Ocean coastal mountains the Central Valley and the Sacramento-San Joaquin Delta (referred to as the Delta) were already attacked by floods several times during the last decades.

This chapter is an introduction to the Master thesis levees and levee evaluation and will present the objectives and outline of the research.

1.1 Research subject

Area description

California's Central Valley, situated between the Sierra Nevada and the coastal mountain ranges, covers 111,300 km² of land. More than 3,800 km of levees protect the urban areas along the Sacramento and San Joaquin River and the agricultural areas in the Delta (Figure 1.1). (Reid R.L, 2005)

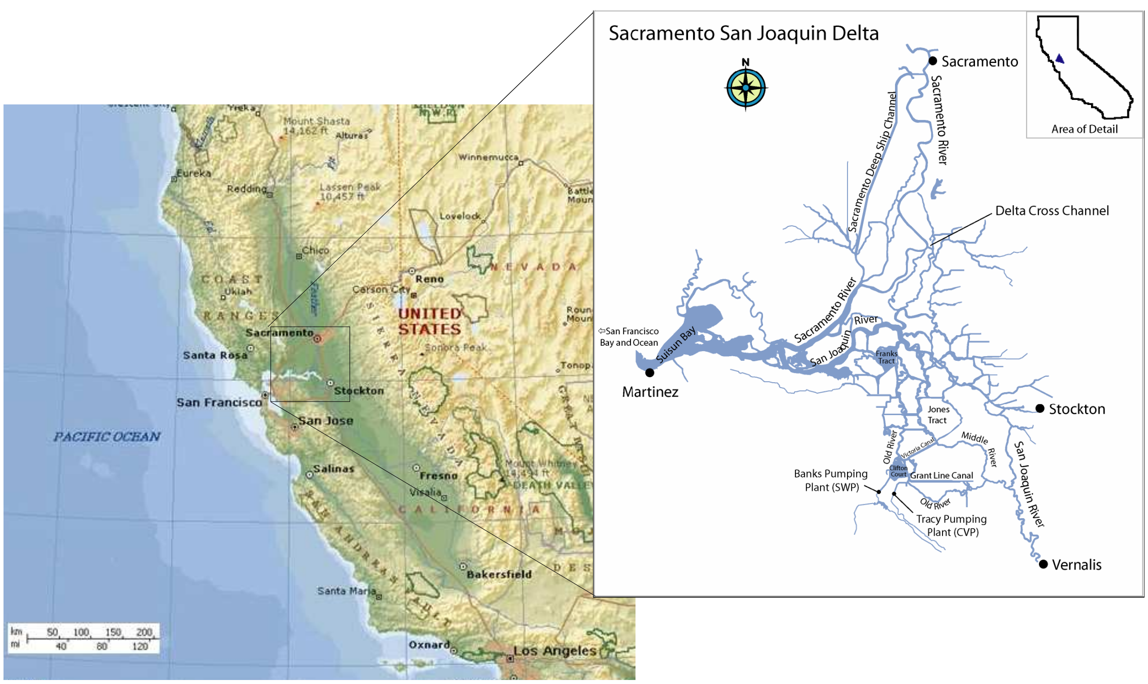


Figure 1.1 California Central Valley (green area) with Sacramento-San Joaquin Delta (DWR, 2005)

River levees

The river levees suffer most from the flood season, starting each November, caused by heavy rains and melting snow from the Sierra Nevada. In January 1997 a major flood caused three dead people. 120,000 people had to abandon their homes. Such major floods occur approximately once in a decade, caused by numerous failing levees; 30 in 1997. The levees are supposed to provide protection against a water level with a probability of exceedance of 1/100 per year.

The Delta

In the Delta the problems are different. Those levees have to hold back water during the whole year: salt water from the San Francisco Bay. The Delta is an area of around 60 islands, situated below sea level. The San Joaquin and Sacramento River carry their huge quantities of water through this Delta to the San Francisco Bay. Even during dry weather floods can occur in this area, like the 2004 dry weather levee failure in Upper Jones Tract, which in total cost about $100 million. This is just one of the approximately 160 breaches of last century.

Other problems

But the flooding of urban areas and farm land is not the only problem. 23 million people in southern California rely on fresh water supplied by a huge north-south transporting system. The Delta protects this system from saltwater intrusion, caused by tidal action from the San Francisco Bay. In winter and spring the high river flow will prevent salt intrusion during floods. But when a flood occurs during low river flow the results could be terrible for the water transfer system. Water will become much to saline to be used for drinking water and irrigation. (Ingebritsen, 2000)

1.2 Problem definition

In one sentence the problem in California can be defined:

California's Central Valley and Delta are extremely vulnerable to a major flood disaster, which will not only affect urban areas but will also endanger the drinking water supply for the whole of southern California

1.3 Research objectives

It seems as if the Central Valley with her rivers and Delta system with its low lying polders and levees founded on peat soils has some remarkable similarities with the Dutch polder system. It would therefore be very interesting to compare these systems and see if there are ways in which these systems could 'help each other' in finding solutions. Because water defenses never provide a 100% safety, but we definitely want them to be safe enough.

The first objective of this research is:

Find out the vulnerabilities of the California Valley and Delta water system and compare them to the Dutch situation in a qualitative and a quantitative way.

And the second objective:

Focus on important weaknesses or differences of the California and Dutch water defense system, find out about their background and give recommendations on how to improve them.

1.4 Outline of thesis

To get an impression of the similarities and differences between the Dutch and Californian water defense systems they are first described separately in chapters 2 and 3. How is dealt with flood risks and levees is discussed, together with the historical background and important floods. Because of the current attention in California for the levees and levee evaluation programs, a more detailed description of levee evaluation is given in chapters 4 and 5.

These descriptions are supported by case studies, which have two functions. The first function is to support the description and the second is to give examples of computations. The DWR project is case example for California. The focus of this project is currently on evaluation of the urban levees, which protect urban areas from flooding. Case examples for the Dutch methods are an evaluation of the Lake Marken levees, Eems Canal levees and Island of Dordrecht. All of which were evaluated by Fugro Ingenieursbureau BV.

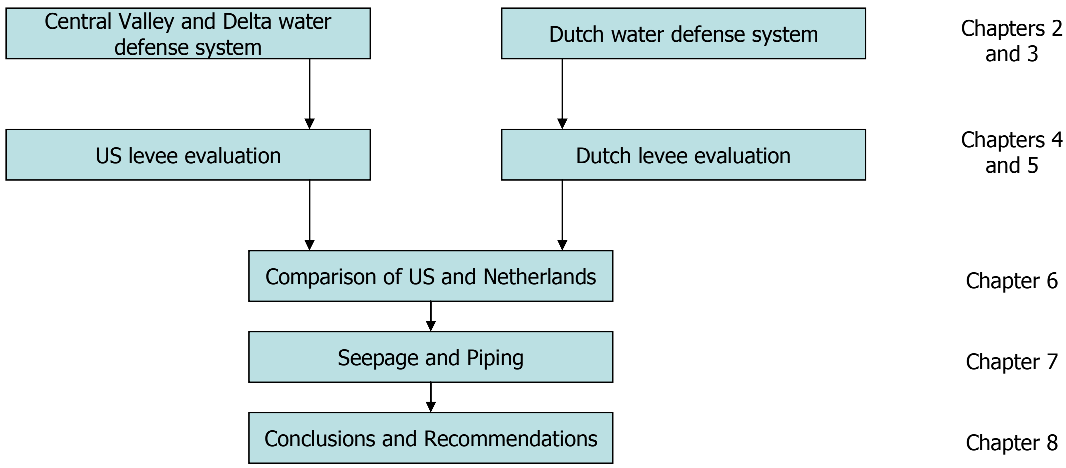


Figure 1.2 Schematized work approach

The computations are supportive to the comparison between the levees and levee evaluation systems in chapter 6. One of the most interesting differences is the evaluation of piping. Seepage and piping evaluation is therefore more thoroughly compared and discussed in chapter 7, supported by a seepage study from 1956 along the Mississippi River in the south east of the US.

Chapter 8 contains conclusive remarks on the differences and similarities of the water defense systems and levee evaluation methods and gives recommendations for further research and for evaluation as well in the Netherlands as in the US.

2 Description Central Valley and Delta

Not many people will immediately think of levees when talking about the United States of America (US) or California. This chapter provides in insight in why California has levees (2.1). What level of protection and against which threats are they supposed to provide? Organizations involved in flood policy and initiatives to prevent future disasters are mentioned as well (2.2) The last paragraph is a description of river floods and storm surges that influenced flood protection in the United States, logically including the New Orleans flood of 2005.

2.1 Increased attention for flood protection

The state of California is the most populous of the US. 36 million inhabitants live on a land area of 400,000 m^2. The Central Valley is a low lying flat area stretching for 600 km from north to south, bordered in the east and west by the Sierra Nevada and the coastal mountain ranges. This Central Valley receives runoff from the Sierra Nevada, which is drained mainly in the Sacramento River and the San Joaquin River, passing densely populated areas with the cities Stockton and Sacramento. These rivers join in the 738,000 acres (2,952 km^2) large Sacramento-San Joaquin Delta, referred to as the Delta.

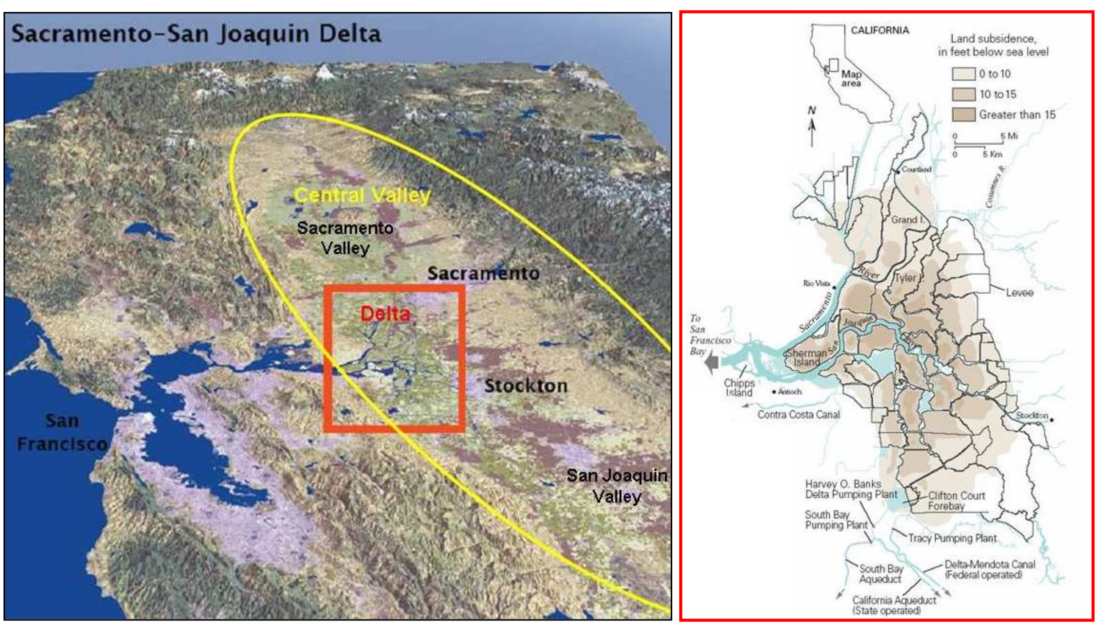


Figure 2.1 Map of the Central Valley and Sacramento-San Joaquin Delta below mean sea level (DWR, 2006-3; Ingebritsen, 2000)

The Delta is the center of a large north-south water delivery system. It receives Runoff from the whole Central Valley, which is 40% of California's State area. More than 22 million people (2/3 of the Californian population) partly rely on drinking water from the Delta and it supplies irrigation water for 7,000,000 acres of agriculture.

The Delta itself consists of nearly 60 islands and is the largest estuary system of the American West Coast. 1,100 miles (1770 km) of levees, surround the islands lying 4-6.5 meters below sea level. Locally the elevations even reach 8 m below mean sea level. The levees should protect the Delta land area and its 500,000 inhabitants against river floods and storm surges. The land use in the Delta is mainly agricultural: 538,000 acres are farmland. Open water covers about 60,000 acres and urban and commercial properties comprise roughly 64,000 acres. The remainder of the Delta consists of undeveloped natural vegetation. (DWR, 2005-2)

Currently there is an increased attention for the Central Valley and Delta levees. There are several reasons for this. One of them is that court decisions stated that the State is liable for flood-related damage caused by a levee failure. Further on Katrina has created the awareness that an equal tragedy could easily come to California (DWR, 2005-2; Harder, 2006).

If the levees fail, or maybe better: when the levees fail in the Delta and/or along the rivers, the consequences are enormous. River levee failures will mainly be destructive to urban areas. Sacramento alone has already more than 450,000 inhabitants. And urbanization along the rivers is rapidly increasing. People will drown, houses and infrastructure will be destroyed.

When levees in the Delta fail, salt water will be drawn into the area (Figure 2.2). Not only people, species and infrastructure within the Delta will be harmed by the salt water, but also the people that rely on drinking and irrigation water from the Delta. In 2004 one levee breach already caused a temporary shut down of the water supply infrastructure (see 2.3.12) The expectation is that more levee failures, as a result of the first breach or of an earthquake, will cause a shut down of all water facilities for a year or longer. The 250 species living in the Delta will be threatened, because of islands that will stay flooded and cause a change in the tidal prism. More than 3,000 homes will be destroyed, together with 2 ports, 2 major highways, a railroad and gas and oil pipelines. (UCDavis, 2006)

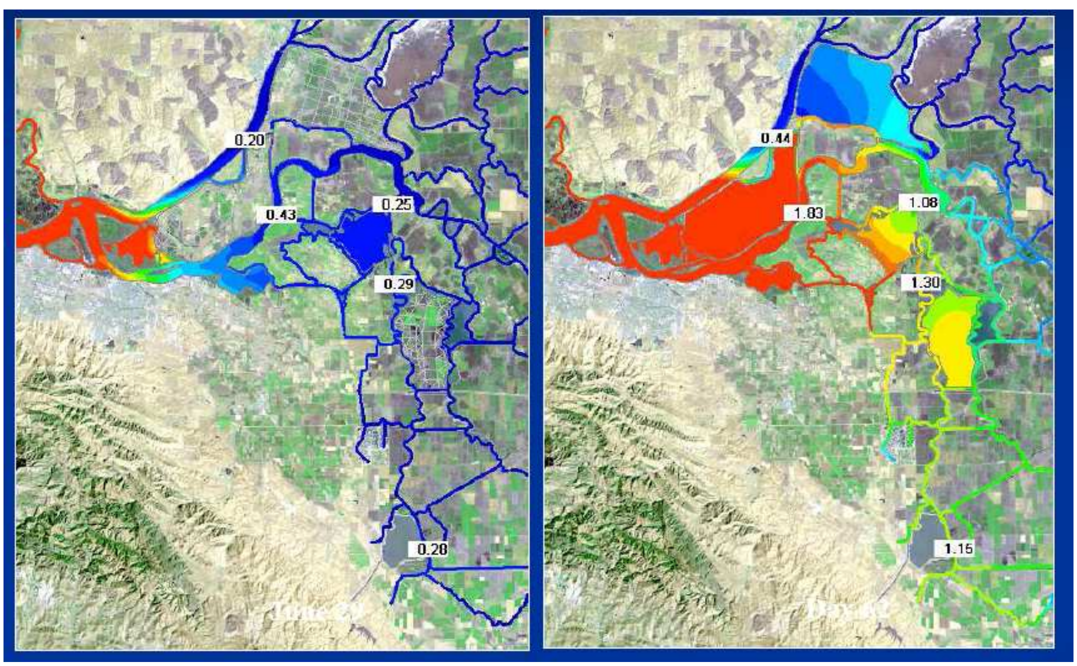


Figure 2.2 Salt intrusion model after magnitude 6.5 earthquake (UCDavis, 2006)

2.2 Central Valley and Delta flood protection system

2.2.1 Levees

More than 3,800 km of levees protect the Central Valley and Delta areas against flooding. Approximately 2,600 km of these levees are federal project levees along the Sacramento and San Joaquin River systems. They are part of the Central Valley flood control system, which also includes reservoirs, overflow weirs and bypass channels, all for which the State Department of Water Resources (DWR) is responsible. (DWR, 2005) (See Figure 2.3 and Figure 2.4)

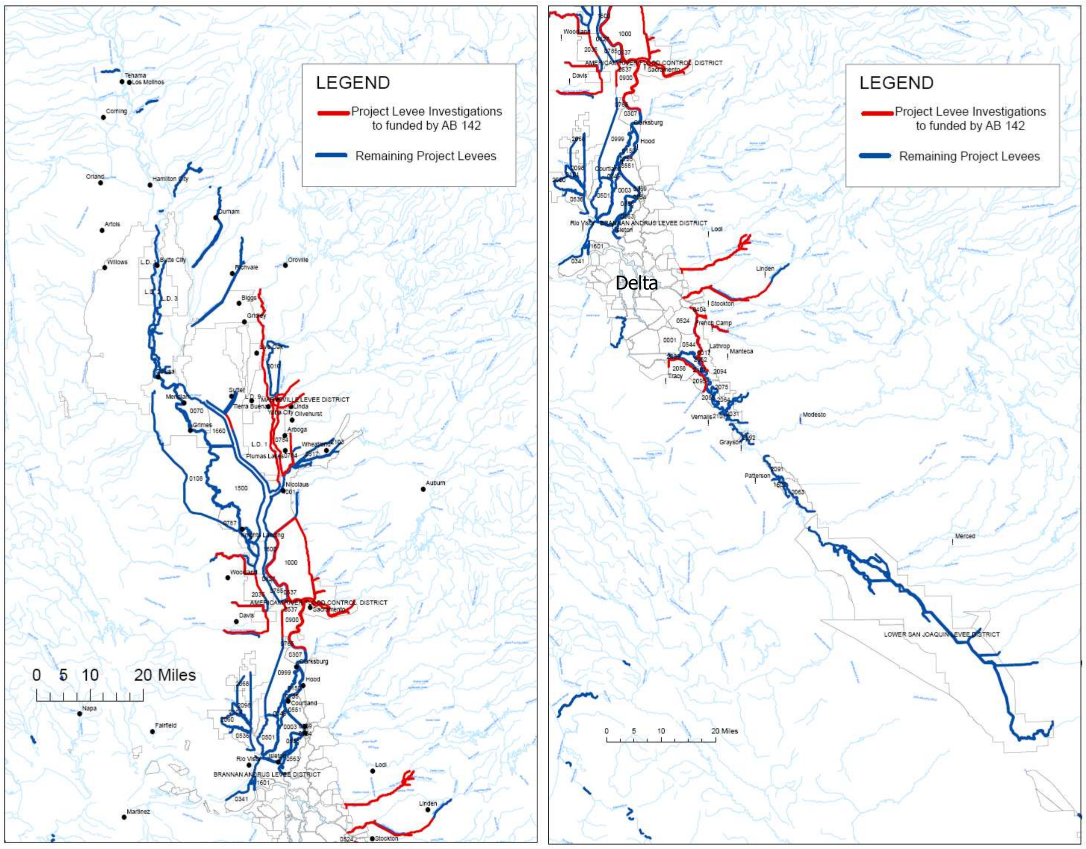


Figure 2.3 Project levees in the Central Valley and Delta (DWR, 2007)

The Delta islands are protected by 1,700 km of earthen levees. Only about 440 km, one fourth, is part of the 2,600 km of project levees. The maintenance of these levees is done by local Reclamation Districts and they are inspected and evaluated by the Department of Water Resources. Three-fourths of the levees in the Delta are non-project levees, for which local maintenance districts are responsible and which are locally or privately owned levees. There is however no distinction to what outside water the levees are subject. Both project and non-project levees within the Delta have to deal with fluctuating river discharges and tidal water levels. (DWR, 1995)

The project levees can again be divided in urban levees and rural levees. Urban levees are defined as levees that protect more than 10,000 people. They protect for example the city of Sacramento with 450,000 inhabitants. Of the 2,600 km of project levees 2,050 km are rural levees and 550 km are urban levees. (Fugro, 2007)

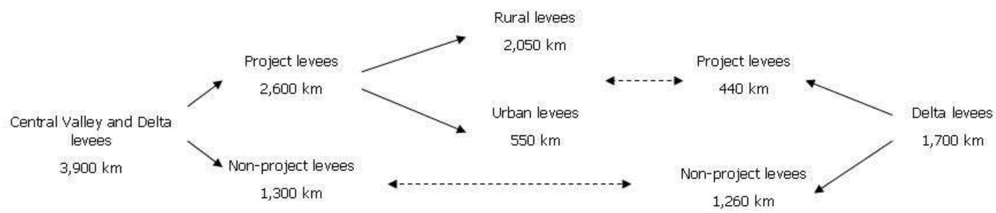


Figure 2.4 Approximate length of levees and their subdivision in the Central Valley and Delta

2.2.2 Risk based flood protection and flood insurance

The Central Valley and Delta levees are supposed to provide a 100-year flood protection. This is a protection against a flood with a probability of 0.01 to appear in a year. With P=Probability, T=flood protection in years and n=number of years the following formula can be used:

$$P = 1 - \left[1 - \frac{1}{T}\right]^n \tag{2-1}$$

It means that during a 30-year period there is a 26% chance to a flood larger than the 100-year flood level. (Mount, 2005)

Flood level standards are not uniform over the whole United States. As can be seen in Figure 2.5 the protection of America's major river cities varies from 500-year protection to 100-year protection. Sacramento, situated along the Sacramento River, has the lowest protection of all these cities. There is no federal flood protection standard. A 100-year protection is the general protection level in the USA for rivers as well as the coast. (RIVM, 2004)

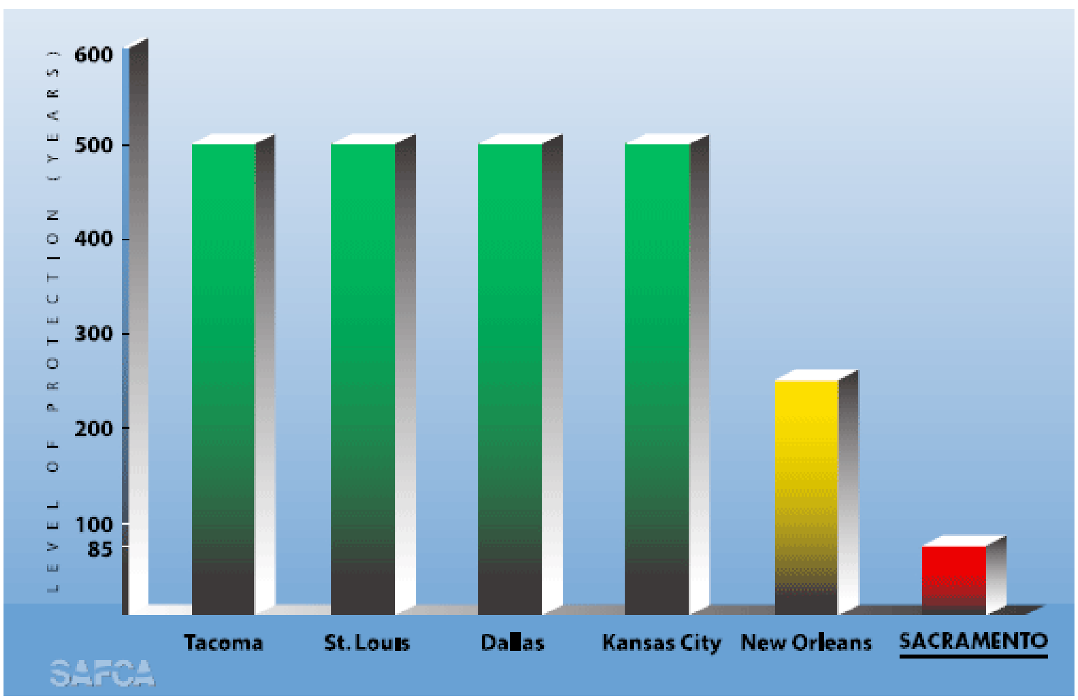


Figure 2.5 Flood protection levels of America's major river cities (SAFCA, 2007)

This 100-year protection is a requirement from the Federal Emergency Management Agency (FEMA), who carries out the National Flood Insurance Program (NFIP). This 100-year level is a national standard, arbitrary defined after the 1968 Flood Insurance Act was established. The FEMA does not build, develop or design levees, but has developed criteria to become or stay a NFIP approved levee following the chart of Figure 2.6. People living behind a rejected levee, in the 100-year flood plain have to insure themselves against floods. Flood Hazard Boundary Maps give the boundaries of the 1-percent-annual-chance floodplain within it is obligatory to purchase flood insurance. To approve or reject levees the FEMA uses the following criteria:

Design criteria: with a minimum freeboard above design level (100-year level) and requirements on embankment protection, embankment foundation and stability, settlement and interior drainage. (All data that proves that the levee fulfills the criteria has to be certified by a registered professional engineer).

Operations plan and criteria: operations of closure and drainage systems must be under supervision of an approved (federal) agency.

Maintenance plans and criteria: an extensive maintenance plan under supervision of an approved (federal) agency has to be present. (FEMA, 2002)

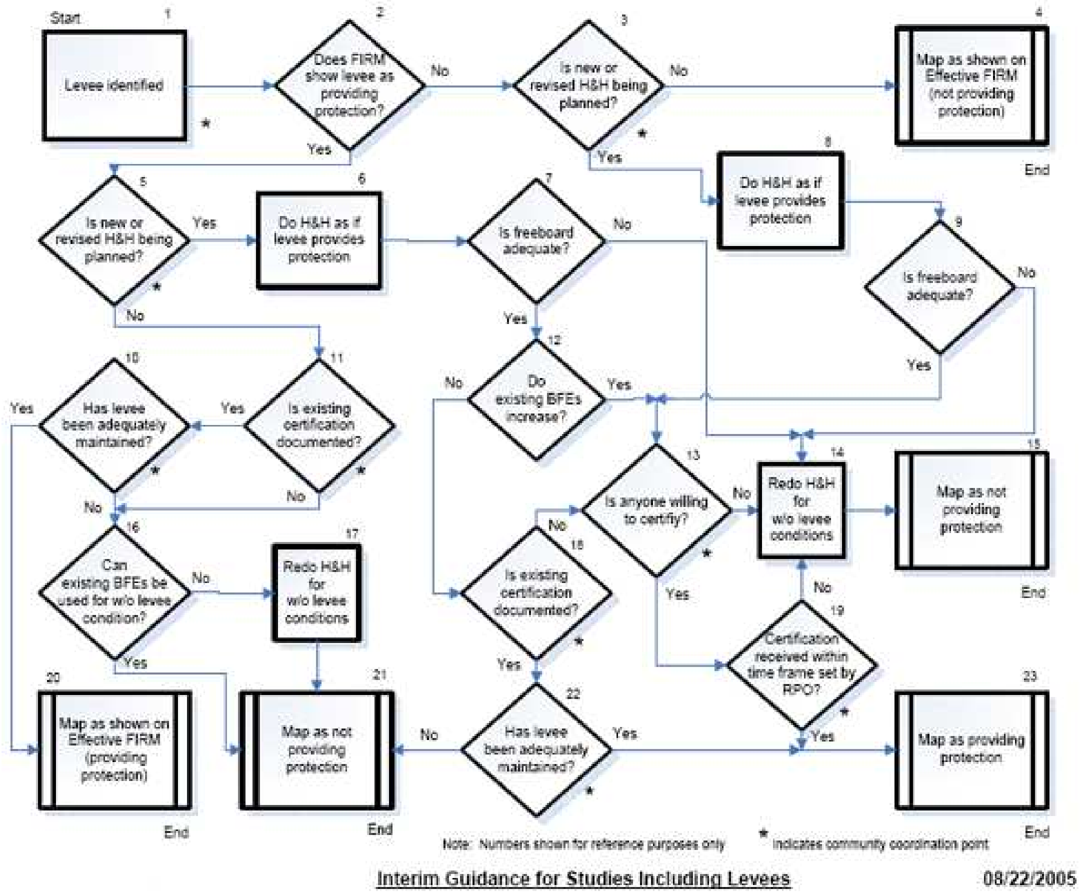


Figure 2.6 FEMA certification process (Fugro, 2007)

Not all levees in the Central Valley and Delta are certified at this moment. Most of the levees within the Flood Control Projects are, but most of the non-project levees in the Delta are not. Levees that are not certified are reasonably expected not to protect the communities against a 100-year water level. Once a levee is certified, proving good maintenance is enough to keep the levee certified. With new, more stringent rules FEMA uses today, the greater part of the levees are expected to be rejected. And FEMA is considering a regular recertification of levees. (DWR, 1995)

2.2.3 Stress events

Climate

The Central Valley of California is situated around latitude 40° north and has a hot Mediterranean climate. Summers are hot and dry, with temperatures in the mid and upper 30°s and occasional heat waves up until 48° C. Winters are cool and foggy, but it only freezes very occasionally. Rain is most typical for the winter and spring seasons. The northern Sacramento Valley gets more rain than the southern San Joaquin Valley. The average annual precipitation in the Sacramento Valley is about 15 to 30 inches, which is 380 to 760 mm a year. The San Joaquin Valley gets about 5 to 15 inches, 125 to 380 mm a year. The Delta gets something in between those two, as can be seen in Figure 2.7. (OCS, 2005)

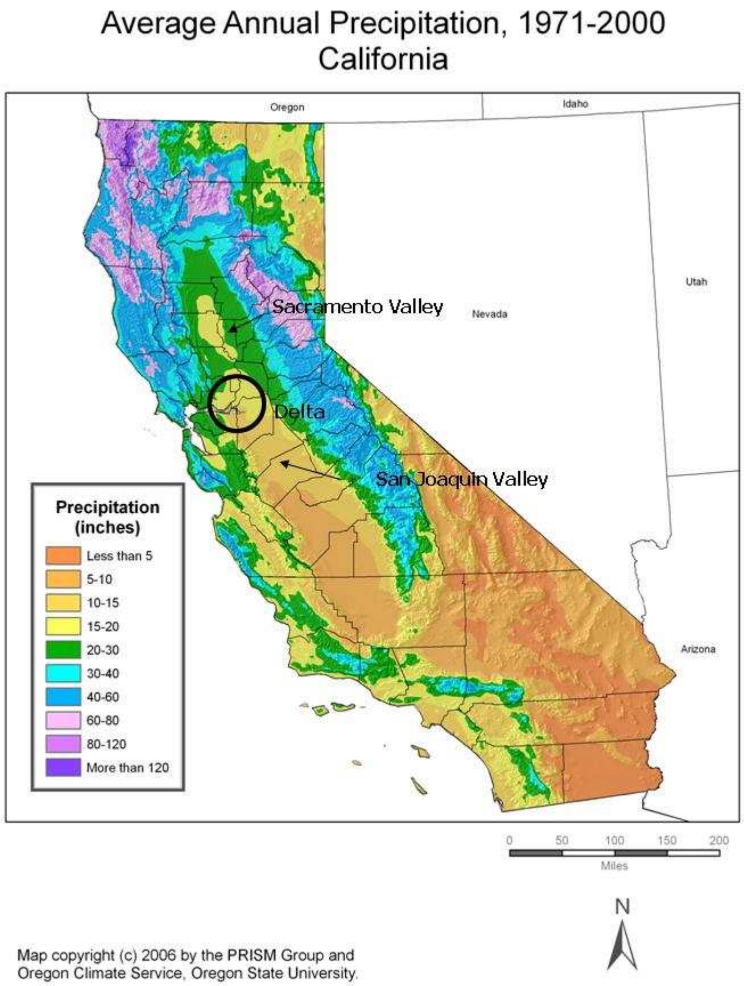


Figure 2.7 Average Annual Precipitation California and Delta (OCS, 2005)

Hydraulic boundary conditions

Levees are designed to protect against outside water. The most important function of a levee is therefore that it has to be able to withstand hydraulic boundary conditions with an acceptable chance of appearance.

All Central Valley and Delta levees deal with flood discharges from the rivers. Figure 2.8 and Table 2.1 summarize the most important mean and maximum measured Delta flows as of 2006. The USACE estimated 100-year flood elevations for the Delta in 1986. New studies are currently performed to get a better estimation of flows and water surface elevations. The Sacramento River flood control system has a design flow of 17,000 m^3/s, of which 80% is directed to the Yolo Bypass at periods of high water. Part of the water passes the Delta on its way to the San Francisco Bay and Pacific Ocean and part is transported to southern California as household water and irrigation water. When the snow in the Sierra Nevada melts, from January to June, the river discharges peak. Warm spring storms can speed up this process and cause extreme river discharges. Winter and spring floods are therefore often induced by storms. The Sacramento River is most dangerous at heavy winter rains and warm winter weather causing rapid snowmelt. The San Joaquin River also peaks from rainfall and snowmelt, but often later than the Sacramento River, as can be read from Table 2.1. (Reid, 2005; DWR, 2005-2)

Although the Delta is situated in a relatively protected area, sheltered from the ocean, tidal influences and wind wave actions from the San Francisco Bay can still harm the area. A 2004 flood in the Delta (see 2.3.12) for example took place at low river discharges and spring tide, which proves that influences from the ocean and bay can be normative.

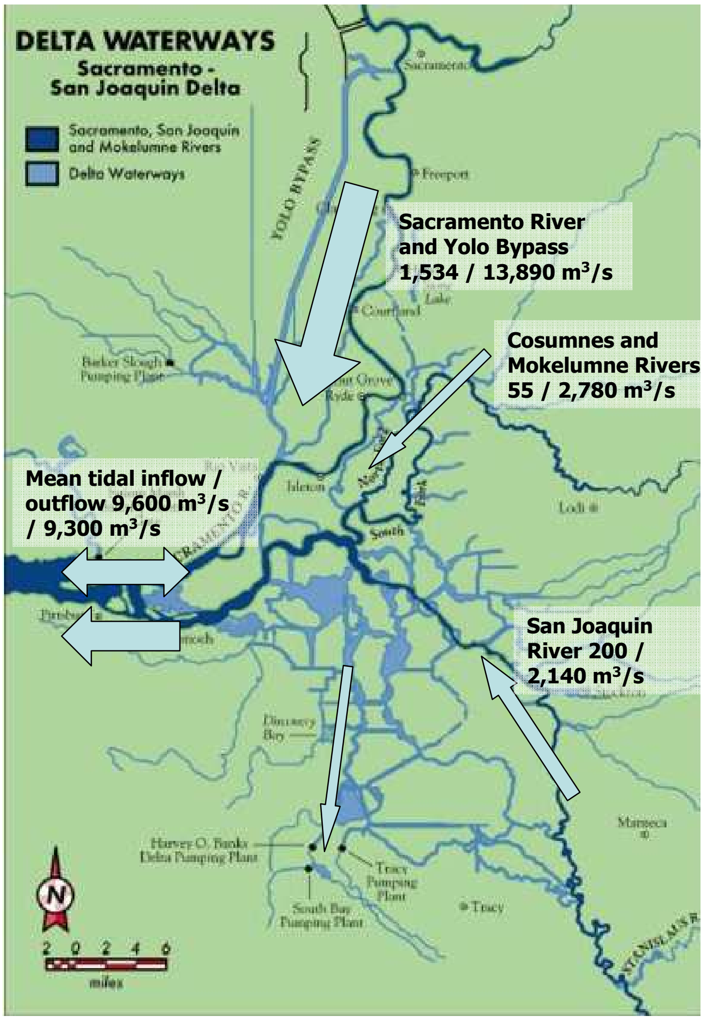


Figure 2.8 Delta major inflows; mean and maximum measured river flows (modified from UCDavis, 2006)

Table 2.1 Summary of flows on major inflows to Delta (DWR, 2005-2)

Station	High Flow Months1	Mean Flow during High Flow Months (Standard deviation) (m3/s)	Peak Flow of Record2 (second highest) (m3/s)	Date of Peak Flow of Record
Sacramento at Freeport	January – March	1,073 (7%)	3,300 (3,250)	Feb 19, 1986 (Jan 3, 1997)
San Joaquin River nr Vernalis	February - June	200 (6%)	2,140 (1,277)	Jan 5, 1997 (Mar 7, 1983)
Mokelumne River at Woodbridge	January – June	24 (12%)	150 (144)	Mar 8, 1986 (Jan 22, 1997)
Cosumnes River at Michigan Bar	January – April	31 (11%)	2,630 (1,280)	Jan 2, 1997 Feb 17, 1986)
Yolo Bypass nr Woodland	January - February	461 (5%)	10,590 (10,110)	Feb 20, 1986 (Jan 3, 1997)

Seismic events

From none of the 162 delta levee breaches of last century an indication was found that it was caused by a seismic event. However, there are people that believe that one of the most important threats to the delta is an earthquake, especially in combination with high water levels. No one knows exactly what the effects of an earthquake will be; the levees have never significantly been tested to that. The Delta lies in the vicinity of earthquake faults that are capable of producing significant ground shaking. To estimate the risk of failure as a consequence of an earthquake, damage potential zones were identified by Torres in 2000, based on local knowledge and geotechnical information. Earthquakes can induce settlements and liquefaction. Tall levees on unstable soils are the most vulnerable for these mechanisms. Areas with these characteristics therefore have the highest damage potential, as shown in Figure 2.9. (Mount 2005)

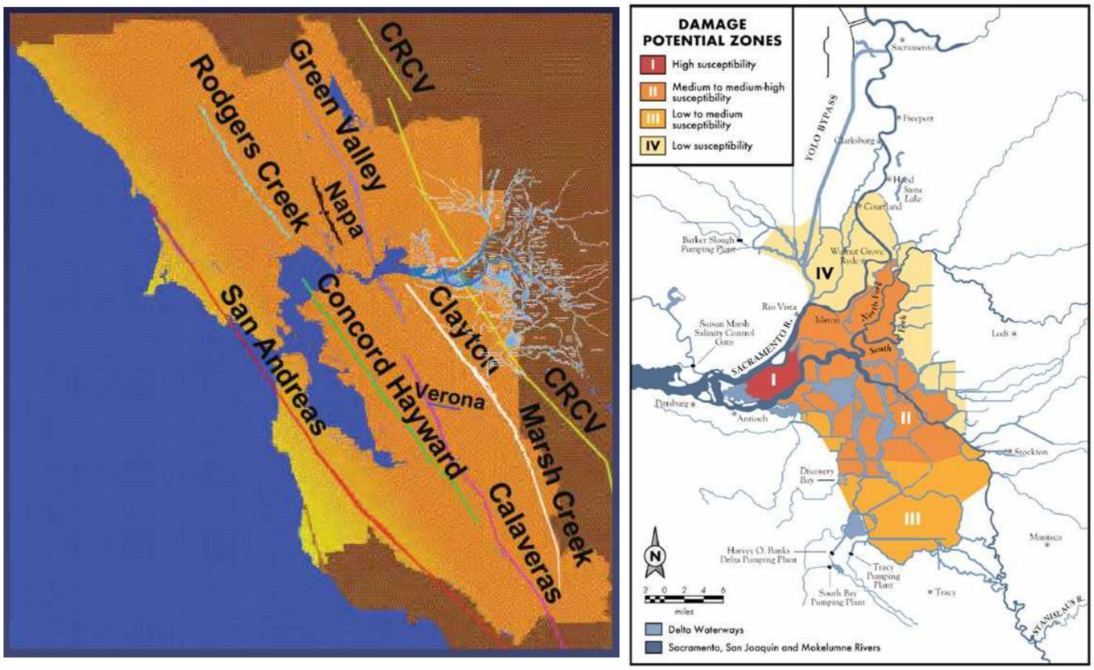


Figure 2.9 Earthquake faults near the Central Valley and damage potential zones in the Delta (Mount, 2005)

2.2.4 Strength: failure mechanisms and levee design

Levee degrading

The river levees were built in the period of the mid 1800s until the 1960s. The material used was hydraulic fill dredged from the rivers, which originated from upstream mining activities. This material is highly pervious and badly compacted. Not much is known about the construction and nature of the clay and sandy foundation material. Poor construction, seepage, erosion and deferred maintenance make these levees very vulnerable nowadays.

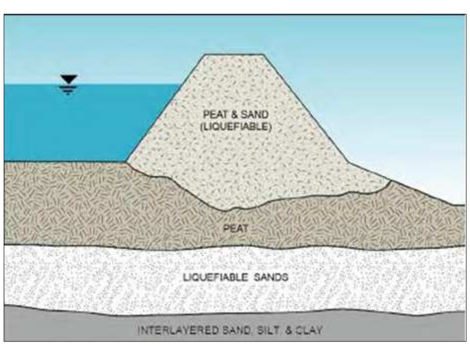


Figure 2.10 Typical cross-section of Delta levees (UCDavis, 2006)

Getting into the Delta one finds 100 years old levees, founded on weak peat soils, which are underlain by liquefiable sands (Figure 2.10). As well as the upstream river levees, material used for levee construction is often badly compacted hydraulic fill, but it is combined with local peat material. Decomposition and consolidation of the peat material degrades these levees. (Reid, 2005)

Failure mechanisms

The mechanisms that can cause levee failure and that are mentioned in reports about the Central Valley and Delta levees are: (DRMS, 2006; USACE, 2002)

- Levee through-seepage and/or under-seepage (piping)
- Piping through cracks or animal burrow in the levees
- Wave-induced erosion on both water and landside slopes
- Flood-induced overtopping
- Current-induced erosion
- Static instability
- Levee instability due to sudden drawdown
- Seismic-induced failures (deformation due to liquefaction)

A standard operating procedure (SOP) of the Sacramento district categorizes the most important failure mechanisms as in Figure 2.11. Overtopping takes place when the water level exceeds the levee height or when waves overtop the levee. A standard freeboard of 3 ft (0.9 m) above the normative water level is used as a standard to evaluate the crest height. Surface erosion is a regular observed mechanism, especially along the Central Valley rivers. During the mid-19th century the river profiles were adjusted to flush the hydraulic mining sediment, which clocked the rivers and caused floods. Nowadays the mining sediment is gone and the rivers erode the embankments. Internal erosion (piping) is divided in seepage through the embankment and under-seepage. Slides within the levee embankment or the foundation soils can be induced by water pressures, but also by earthquakes. The earthquake induced failures are only shortly mentioned in the Sacramento SOP, but it is one of the most important threats, especially for the Delta levees. (USACE, 2003)

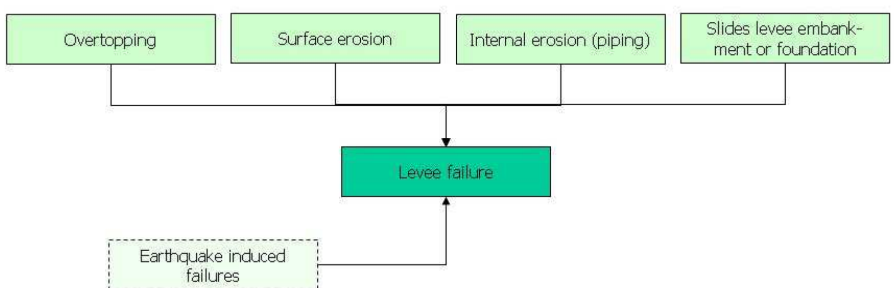


Figure 2.11 Levee failure mechanisms important for the Central Valley

Levee design

A levee design manual from the US Army Corps of Engineers (USACE, 2000) gives steps that can be followed when designing a levee. There is no standard design, because of the difference in foundation conditions, property values and available soils per region. What is prescribed is the preliminary research that should be done and how to prevent levee seepage and stability problems. A 1V on 2H slope is for example the steepest slope allowed for construction, while for a sand levee 1V on 5H is considered flat enough to prevent damage from seepage. The crown width should be at least 3.06 to 3.66 m (10 to 12 ft), but depends again on the circumstances. Compacted fills are preferred above hydraulic fill, although the latter could be used for agricultural levees, where failure will not endanger that many lives. Figure 2.12 shows levee standards as used by different organizations, as for example FEMA. The PL-99 standard refers to a Public Law, in which a minimum standard for repair assistance after damage was established. (Reid, 2005)

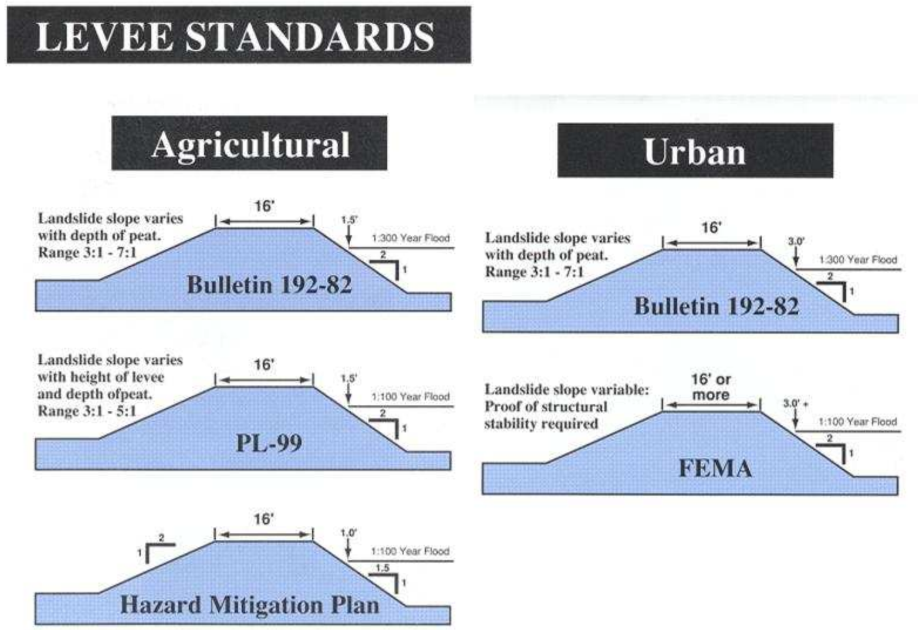


Figure 2.12 Examples of levee standards

2.2.5 Inspection, maintenance and levee evaluation

Methods to evaluate the performance of the levees in the United States are set by the USACE and described in several documents. Seepage and macro stability are the main issues dealt with in this context, including field evaluation and lab test methods. The FEMA criteria for levee certification are likely to defer to these guidelines, while the USACE from origin certificated the levees under FEMA NFIP regulations. To get levee certification it is therefore advisable to use the the USACE methods. (Fugro, 2007)

Inspection, maintenance and emergency response preparations are daily DWR flood management activities. As stated in USACE's Standard Operation and Maintenance Manual, each maintaining district is required to perform a detailed inspection every 90 days, including prior to and just after the flood season. The results of these inspections have to be reported to the DWR, who combines them and hands a quarterly and yearly report to the Reclamation Board. (Reid, 2005)

Construction of levees is normally paid for by the USACE. While the Water Resources Development Act of 1986 requires them to share these costs with local nonfederal sponsors, the Reclamation Board and local levee districts also pay their share. But maintenance is entirely paid for by local organizations. Within the Delta there is a Delta Levee Maintenance Program, because it has a statewide benefit. But since summer 2006 funding of the maintenance of these levees is left to the locals as well.

There is a Public Law which requires minimum standards for assistance (see Figure 2.12). If project levees meet those minimum standards, the USACE will help repair a levee in the event of damage. But the non-project levees are not inspected by the USACE or DWR. And it is not easy to get financial support from the USACE for these levees, unless a levee district is formed. (Reid, 2005)

2.2.6 Drivers of increasing flood risks

In the future an increased potential for levee failure can be expected if no initiatives are started. This potential is caused both by drivers of change, which cause a change in load on the levees and strength of the levees. Climate change, a still increasing population, economic growth and degradation of levees due to subsidence and erosion are the main factors, which not only cause an increasing failure risk but will also lead to more damage if a flood occurs.

Load

Climate changes: an expected temperature rise of 3 to 10.6 degrees Fahrenheit towards the end of the century, depending on emissions and ore frequent and more severe storms are expected. These changes will lead to:

Sea level rise: Last century the sea level along California's coast rose with 18 cm. If no action is undertaken the sea level will rise with an additional 55 to 88 cm by the end of this century. Flood stages in the Delta will rise and during low runoff seasons the salt intrusion will increase. Backwater effects will also increase water levels upstream of the Delta and put more pressure on the river levees.

Changes in runoff conditions: The timing and intensity of precipitation is expected to change together with increased storm intensity. The trend for flood flows is to be higher than anticipated and strong winds in open water can cause higher water levels referred to as fetch. The Sierra Nevada snow pack is expected to reduce 30% to 90% this century, decreasing the April to July runoff of the rivers pouring in the Delta. Extremely wet and extremely dry periods will lead to more flood protection problems.

Strength

Lack of maintenance: as mentioned before maintenance budgets from the State and federal sponsors have been cut down. If no local money is raised to maintain the already vulnerable levees, deterioration, especially due to erosion, will increase rapidly and a disaster is just a matter of time.

Subsidence: oxidation and consolidation of organic-rich soils cause subsidence. At this moment some islands are already 8 m below mean sea level and they subside with approximately 3 to 5 cm/year. Subsidence of the islands reduces the stability of the levees as can be seen in Figure 2.13. (Mount, 2005)

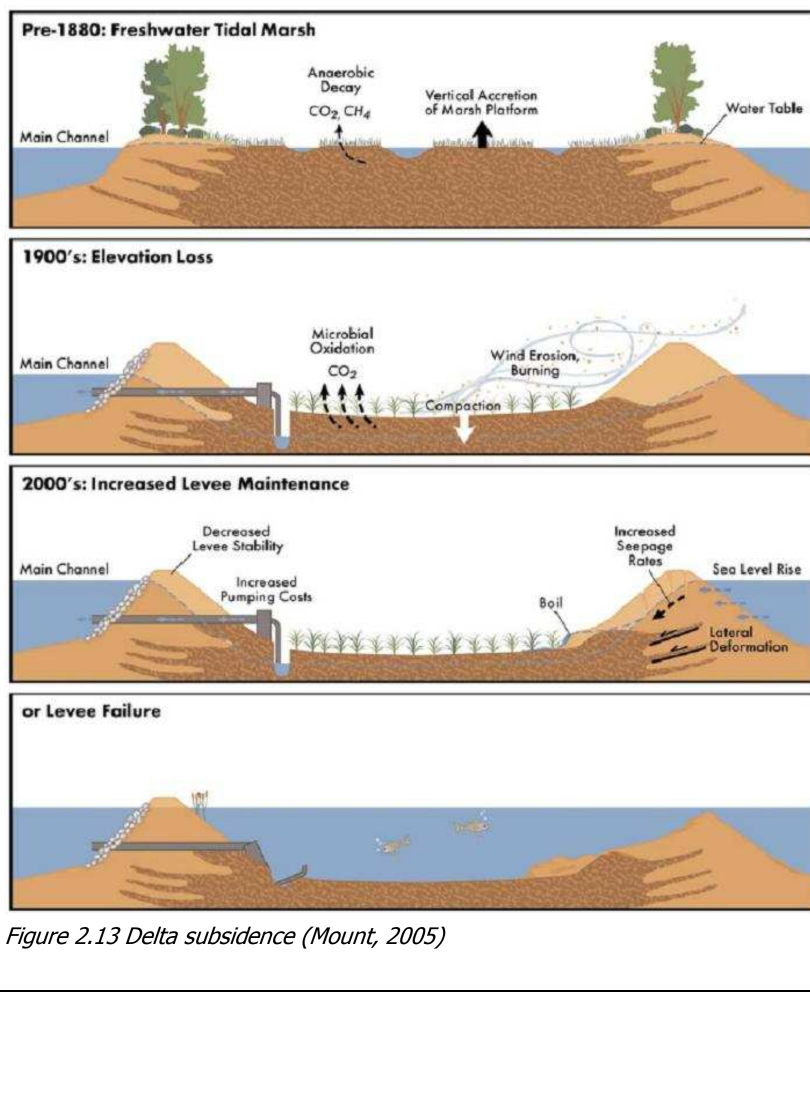


Figure 2.13 Delta subsidence (Mount, 2005)

Damage

California's population is already approaching 37 million. 44 million inhabitants are expected in 2020 and 55 million in 2050 according to the California Department of Finance. This means that the urban water demand will grow, increasing pressure on the Delta water delivery system. If something happens to the transport system, more people will be affected by this.

The population of the Delta itself is also still increasing. Urbanization along the rivers and development in the delta will cause more and more victims and more damage in case of a flood.

Environmental concerns: ecosystem needs are rising, partly because of the increasing population.

2.2.7 Dealing with increasing flood risks: Initiatives

The damage hurricane Katrina caused in New Orleans has accelerated the development of initiatives in California. Plans are developed to improve the flood protection in the Central Valley and Delta. Most of these plans are now gathered under "FloodSAFE California", an initiative of the DWR of California. The main goals of this initiative are (DWR, 2007):

- Reduce flood risk to Californians, their homes and properties;
- Develop a sustainable flood management system;
- Reduce the consequences of floods when they do occur.

The most important initiatives in the above context are:

Emergency Levee Erosion Repair Project: This project was started in early 2006 after Governor Arnold Schwarzenegger declared a state of emergency for California's levee system. The DWR initially identified 29 critical erosion sites in the Sacramento River flood control system, which had to be repaired. The DWR assisted by the USACE repaired these sites in 2006, funded by a State Assembly Bill and some money from the USACE. Another 4 sites were added in late 2006. Repairs consist of placing soil and rock and natural vegetation and wood. (DWR, 2006-2)

Delta Risk Management Strategy (DRMS): The tasks within this project are to evaluate current and future delta risks, identify consequences, identify risk reduction measures, including levee upgrades and land use changes and to evaluate alternative strategies to reduce the risk. An introduction to this project was written in 2005 and the Initial Technical Framework describing the methodology to analyze the fragility of the Delta levees is now in its second phase. (DRMS, 2006)

Task order 17: Geotechnical investigation and evaluation of the urban levees. Geotechnical firms are involved in this project, overseen by the DWR. California has recognized the urgent need to upgrade the deteriorated levee systems in the Sacramento and San Joaquin valleys and is starting now with the levees that are of the highest priority: the urban levees, which protect highly populated areas as Sacramento, Stockton, Marysville and Yuba.

State bond debts of November 2006: Two bonds that have been rewarded and make it possible to evaluate and improve levees in the Central Valley and Delta. More than 4 billion US dollars are available for flood control and levees.

Delta Vision Process: Initiated to encompass and integrate many separate planning efforts and to develop long-term strategies for a sustainable Delta

2.2.8 Organizations involved in flood policy

There are numerous instances involved in the US flood policy from federal institutes to local organizations. In Figure 2.14 the most important organizations are linked with their supervising organization, sorted per level. Most of them are also shortly described in this paragraph.

Federal Emergency Management Agency (FEMA): is a former independent agency, which became part of the Department of Homeland Security in 2003. It is a federal agency which is tasked with responding to, recovering from and mitigating against disasters. FEMA for example provides Flood Insurance Rate Maps, which give an indication about the areas where flood insurance is obligatory.

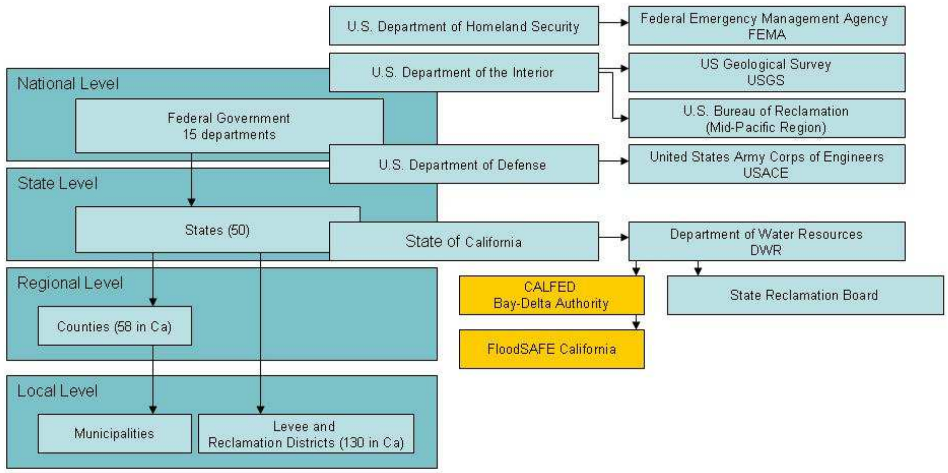


Figure 2.14 Organizations involved in flood defense policy

United States Geological Survey (USGS): USGS is a scientific federal agency. Their main tasks are measuring, analyzing and mapping of natural resource conditions. They for example monitor streamflows and provide geological maps and data.

Bureau of Reclamation (Federal); U.S. Department of the Interior: Established in 1902, the Bureau of Reclamation is best known for the dams, powerplants, and canals it constructed in the 17 western states. Today, Reclamation is a contemporary water management agency with a Strategic Plan outlining numerous programs, initiatives and activities that will help the Western States, Native American Tribes and others meet new water needs and balance the multitude of competing uses of water in the West.

United States Army Corps of Engineers (USACE): USACE is the engineering branch of the army, which supports in 5 areas: water resources, environment, infrastructure, homeland security and war fighting. Related to floods the USACE inspects project levees. They provide engineering manuals for levee design and related engineering and has the authority to fight floods to save lives or protect property whenever the district commander issues a declaration of emergency.

Department of Water Resources (DWR): Each state has its own DWR. The DWR of California operates and maintains the State Water Project, including the California Aqueduct. The department also provides dam safety and flood control services, assists local water districts in water management and conservation activities, promotes recreational opportunities, and plans for future statewide water needs. The DWR inspects and evaluates maintenance of the state's federally designed project levees.

The State Reclamation Board, under Section 8609 of the Water Code, has the authority to designate floodways in the Central Valley. It was established in 1911 to develop and oversee a single flood control plan for the Central Valley. The Board is administratively part of California's DWR, but it maintains separate and independent decision making powers. In partnership with the Army Corps, the State Reclamation Board repaired river levee erosion sites on a regular basis through the early 1980s using the Sacramento River Bank Protection Project. (Reid, 2005)

Reclamation and Levee Districts: maintain the 1,100 non-project miles in the Delta and some project levees. (DWR is responsible for channel maintenance of the Sacramento River Flood Control Project; local agencies are responsible for maintenance of the channels of the San Joaquin River system)

CALFED Bay-Delta Authority: partnership between state and federal agencies involved in protecting the ecosystem encompassing San Francisco Bay and the Delta.

Delta Protection Commission: To develop a long-term resource management plan for an area designated as the Delta primary zone; Established by the Delta Protection Act in 1992 (California Water Code Section 12220); has land use planning jurisdiction over the primary zone.

California Water Commission: Serves as a policy advisory body to the director of Water Resources on all California water resources matters.

2.3 History

2.3.1 Introduction

Because this report mainly deals with the flood control in the Central Valley and Sacramento-San Joaquin Delta in the state of California, a view on the American history of flood protection is described around the history this part of the United States. Flood control activism originated in the Sacramento and Mississippi River valleys (O'Neill, 2006). Therefore, to give an idea of the development of flood protection methods and federal initiatives, floods in the lower Mississippi River valleys will also be mentioned, for example the 2005 flood of New Orleans. This disaster again focused attention on the importance of flood protection, not only in the United States, but also in the Netherlands. Table 2.2 lists the most important and river floods and storm surges for the development of the current flood policy and exposes the danger to floods in the Central Valley and Delta today.

Table 2.2 Important and recent floods in the US and California

Important and recent floods	Impact on US or California flood policy
Mississippi floods 1949	Swamp and Overflow Land Acts of 1949 and 1950
Sacramento Valley River floods 1860s	Court decision to outlaw dumping of mining debris
Mississippi River floods 1870s to 1890s	Establishment of the Mississippi River commission and MRC standard levee design
Nationwide floods 1935 and 1936	Flood Control Act 1936
1986 River floods	State liable for flood damage
1997 Central Valley floods	
Upper Jones Tract levee failure 2004	
New Orleans flood 2005 hurricane Katrina	Increased attention on levees and levee evaluation

Figure 2.15 Map of the United States with the Mississippi Delta and California Central Valley (Welt-Atlas.de, 2006)

2.3.2 First flood protection along the Mississippi River 1719

Before the European colonists arrived in the United States, the rivers had its natural course and were bounded by natural levees. During great floods the water could easily overflow the large floodplains. But soon after European settlement, wetlands in the eastern parts of America were reclaimed. At that time the swamplands were regarded as annoying: they were a source of diseases and a restriction to overland traveling. Hand-dug ditches drained the wetlands. As the population grew, more and more wetlands were converted. (USGS, 1997)

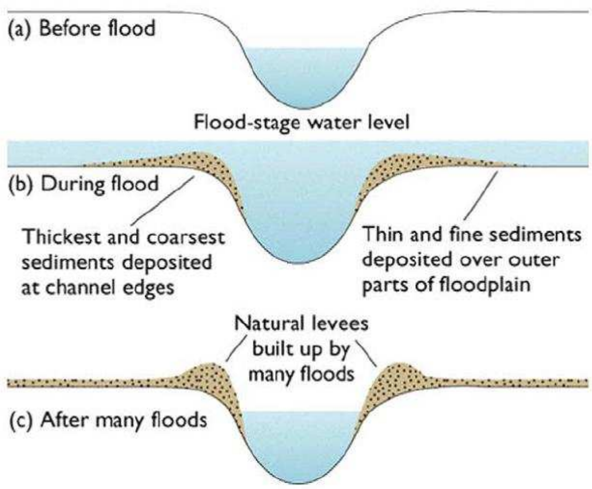


Figure 2.16 Natural levees (Berkeley, 2006)

The first known flood protection works of the United States date from just after 1717, when the French settled in south east America. It was Jean Baptiste LeMoyne who moved the capital of his colony from sterile lands to the fertile grounds along the lower Mississippi River and thus created New Orleans. He had to protect his town in the Mississippi Delta wetlands against river floods and therefore he constructed levees on top of the natural river levees (Figure 2.16). By 1727 the levee was already longer than 1.5 km and had a height of almost a meter. Landholders were responsible for their maintenance. The 1735 flood, which lasted six months, destroyed most of the levees and showed that not all landholders maintained their piece of levee properly. Only slowly, new levees were built by the slaves of the landowners. Weak levees, built to no standard, were more a rule than exception. (Cowdrey, 1977) While the city lay less than a meter above sea level periodic flooding from the Mississippi river between April and August dominated the area, together with flooding and wind damage caused by hurricanes from June until October. The deltaic plane was also subject to settlements of 0.5 to 3 meters per century. (Berkeley, 2006)

From 1803, when the state of Louisiana was purchased from the French to the United States, the United States Army Corps of Engineers (USACE) played an important role in forming the Mississippi Delta to the desires of the inhabitants. The USACE was established in 1775, when engineers were needed to support the army in the American Revolution. The first tasks were to fortify key infrastructure during wars, such as harbors and to build defenses against the British along the seacoast. Constructing of seacoast fortifications continued as the engineers' primary responsibility. With an Act of the Congress the Army Corps was permanently established in 1802. Soon the USACE was authorized by the government to improve navigation on the rivers.

2.3.3 Lower Mississippi Valley floods 1849

The Mississippi drains ¼ of the United States water from a watershed area of approximately 3,237,500 km^2. It is the third largest river watershed in the world and by far the largest river of the United States. No wonder that most flood related policy was established after a Mississippi River flood or after hurricanes that initiated east American floods. (Berkeley, 2006)

Together with land expansion the population grew from 7.2 million in 1810 to 12.8 million in 1830. The Mississippi river valley had 1.4 million white settlers by 1810 and 2.6 million by 1820. Settlers were moving westward and created a further large-scale conversion of wetlands. In the meanwhile landowners organized themselves in levee boards to coordinate flood control. The states or local governments supported these boards by creating local levee districts. Riverside and backland landowners had to pay taxes, but it took decades before all who benefited from the levees paid those taxes. But levee quality often remained poor, as no law concerning the shape and size of rivers was enforced. To prevent sabotage individual landowners or levee district officials patrolled their levees during times of high water. (O'Neill, 2006)

After the steam boat invention regular traffic was using the Mississippi River. The Army Corps recommended navigational improvements and flood protection works. The 1824 Rivers and Harbor Act supplied in this manner, but only granted the navigational part. But during the late 1820s and in the 1830s the funding for river projects was reduced and left over hundred federal water projects unfinished. The legislators were more interested in temporary dredging operations than in expensive structural projects. River activists kept on demanding for channel and harbor improvements, while those from the Mississippi Delta kept on asking for flood control.

In 1849 and 1850 severe flooding from the Mississippi River inundated large parts of the river basin and the Mississippi Delta (27 million acres). A levee 25 km upstream from New Orleans had broke and flooded 220 city blocks with 2.7 m of water. 12,000 citizens had to be evacuated. (Berkeley, 2006) This flood finally led to the first Swamp and Overflow Land Act, which conveyed the ownership of the Delta marshes from the federal government to the State, which from then was granted to reclaim wetlands and built levees. In 1950 and 1951 this Act was extended to other states, including California. (O'Neill, 2006)

2.3.4 First flood protection in the Central Valley 1850

In the late 18th century the first European colonists reached the Central Valley. Before that the Native Americans, Californian Indians, had already lived there for about 15,000 years. The total population of what is now called the State of California counted about 300,000 before colonization. People in the Valley mainly lived from fishing and hunting. Sacred missions caused the Spanish to settle themselves in what they called New Spain in the 1770s. In this period a decline of about 75% of the number of native people took place due to diseases. From 1800 expeditions to the Central Valley led to the discovery and naming of the Sacramento River and the San Joaquin River.

Soon cities were founded in Spanish California of which Los Angeles was one of the first ones. The inhabitants consisted of Indians, Africans, Spaniards and mixes of those. To provide food agricultural areas were expanded and cattle grazed the large areas surrounding the cities. In 1848 California was seized by the United States and officially entered the Union in 1850. Americans discovered California in the following decades. After gold was found in 1848, extensive gold mining activities attracted foreigners. But Eastern Americans also discovered the agricultural potential of the Central Valley. Rich soils and dry summers made the region ideal for wheat and grain production.

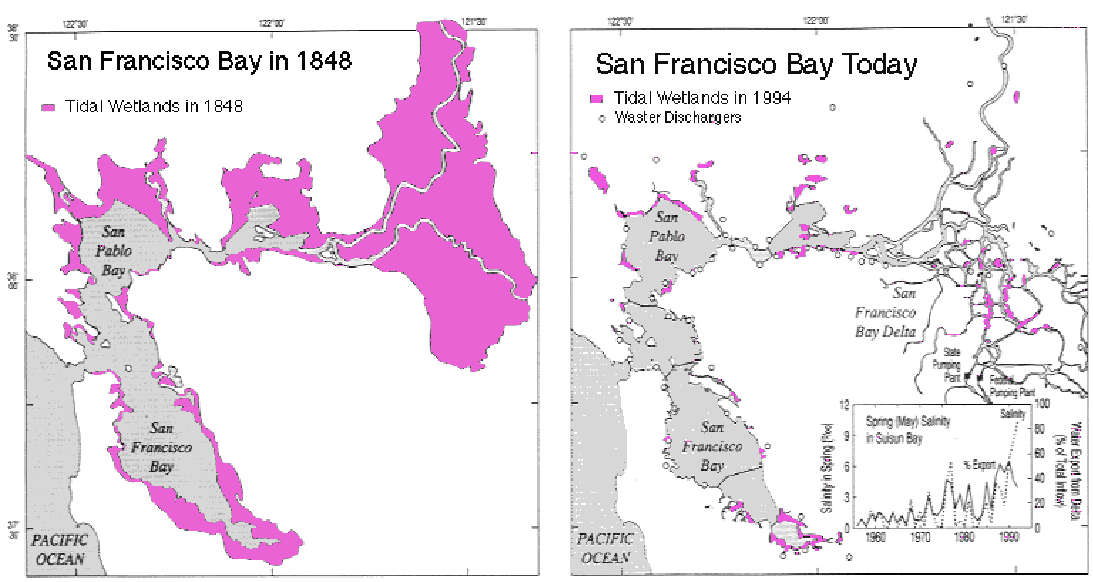


Figure 2.17 The Delta and Bay wetlands in 1848 and 1994 (USGS, 2006)

Before the Delta had its current performance the area consisted of tidal marshes with a network of channels and islands. Of the Central Valley about 4 million out of the 13 million acres were estimated to be tidal wetlands before 1850 (see Figure 2.17). Sediment of these marsh platforms consisted of inorganic material from the watershed and organic material from tules (plants) from the marshes itself. With the Swamp and Overflow land Act of 1850 development of the Central Valley started. Farmers first began diking and draining of the river flood plain areas. Development in the Delta began in the late 18th century. The tidal marshes were also reclaimed to be used as agricultural land. To prevent the agricultural land from frequent flooding, levees were built, mostly by Chinese laborers and farmers. (DWR, 1995) (USGS, 1997)

2.3.5 Sacramento Valley River floods 1862

Because of the gold mining, settling in the Central Valley developed so quickly that the settlers had often no idea of the devastating power of the Sacramento River, which had the largest fast-rising floods of the United States at that time. Soon floods became even worse when upstream in the American and Feather Rivers, in the Sacramento River valley, gold mining was expanding. Techniques evolved and led to development of nozzles to blast away deposits with water pressure. Dumped mining debris silted up rivers and caused river floods that made the farmers angry. In January 1862 a great river flood put Sacramento in 3 meters of water. Late 1861 heavy rains combined with the choked riverbeds by mining debris caused this flood. More floods had to follow this one, for example in Marysville in 1875, before government action was taken. A court decision that outlawed the dumping in 1884 had to improve the situation. (O'Neill, 2006) By that time most of California's swampland was in private ownership and steam-powered dredges were used to improve the levees with alluvial material from the river, which originated from the upstream mining activities.

2.3.6 Mississippi River floods 1870s to 1890s

In the meanwhile floods along the Mississippi River were occurring almost yearly. Finally, after a flood in 1874, the Mississippi River Commission was established by an act of congress in 1879. The task of this commission was to improve safe navigation through the Mississippi River and prevent destructive floods. They did this by building flood control structures in the upstream part of the Mississippi River and by redirecting and narrowing the river.

But a flood in 1890 proved that the changes along and upstream of the river, probably improved navigation, but were certainly not preventing floods. They had only made the situation worse. More than 80 km of levee was destroyed during this flood and the River Commission had to change its course. The 1890 flood was adopted as the design level for levees. Most of the levees had to be raised and therefore a levee standard was developed. Levees were enlarged and raised with widely available hydraulic fill (see Figure 2.18). A crown width of 2.4 m and a slope of 1:3 were prescribed. (Rogers, 2006)

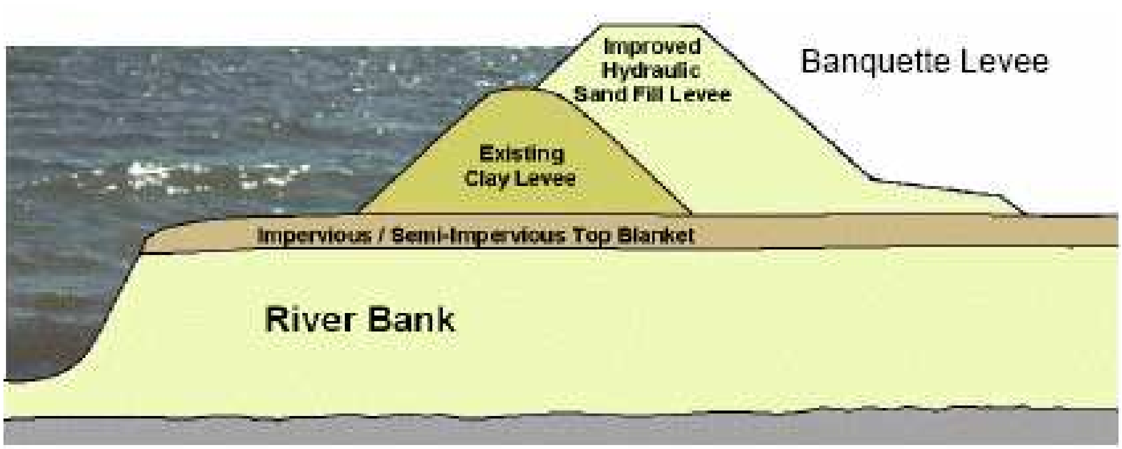


Figure 2.18 Mississippi River Commission levee design (Rogers, 2006)

2.3.7 Mississippi flood 1927

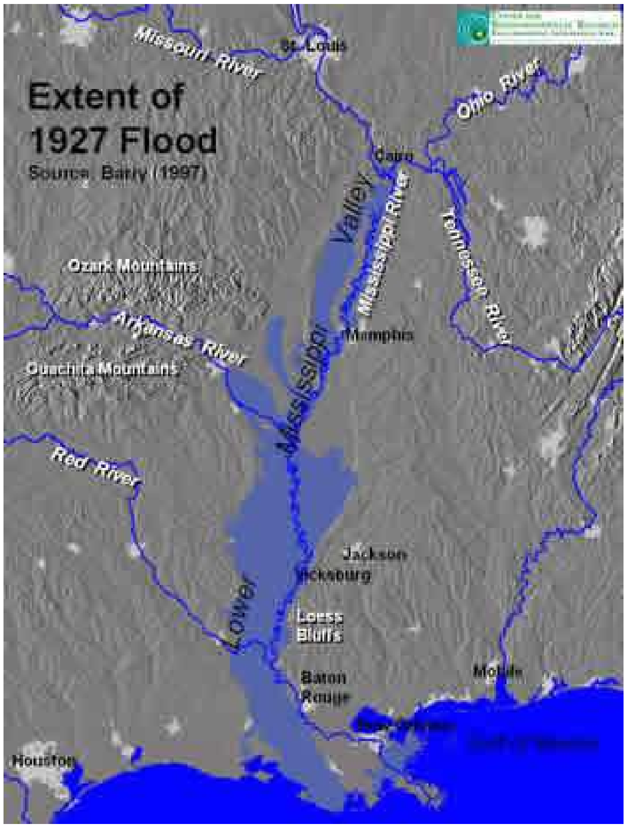


Figure 2.19 Extent of the 1927 Mississippi flood (Barry, 2002)

In early 1927 the Mississippi Valley flooded again after extensive rainfall: 45 cm in 48 hours. It created the largest destruction ever from a river flood in the United States. 246 levee breaches caused the inundation of 70,000 km^2 with a depth of about 10 m (Figure 2.19). At least 246 people were killed, while others reports mention more than 1,000 drowned people. A levee just upstream of New Orleans was dynamited to protect the city New Orleans and prevent drowning of even more people. A levee breach further upstream later that day made the blow up unnecessary. Unfortunately villages upstream had already been destroyed. It took six months for the Mississippi to return to its original size. The US had no federal disaster-response agency at that moment. Instead the Red Cross helped the government with trained volunteers, supported by donations. The US Coast Guard, Navy and Army quickly responded and with their help. One person was established by the president to be in charge of the rescue operation, which made the response very quick and efficient, but gave that one person maybe too much power. (Kosar, 2005)

2.3.8 Nationwide floods 1935/1936

Costly floods in 1907 and 1913 had already led to establishment of the House Committee on Flood Control in 1916 and the 1917 Flood Control Act. This Act was first only aimed to control floods, but after the 1927 floods it was expanded extensively. Nationwide series of floods finally lead to the 1936 Flood Control Act, the first nationwide flood control program.

In the meanwhile the Central Valley Project was started in California. It consisted of construction of large dam and aqueduct constructions to transport massive quantities of water from north to south, started in 1937. It was also a benefit to farmers in the Central Valley who could irrigate from that water.

In 1968 the Flood Insurance Act was established, which enables persons to purchase an insurance against physical damage or loss of property caused by floods in the United States. It is the only natural hazard for which the federal government provides insurance.

2.3.9 1972 failure of the Brannan Island levee, California

In June 1972 a dry season levee failure caused inundation of the 7,500 acres (3,040 ha) Brannan Island in the Sacramento San Joaquin Delta. Salt water was drawn into the Delta system and contaminated the drinking water supply and irrigation water. Water export had to be stopped for several weeks. 500,000 acre-feet of fresh water were used to flush the system. After this failure it was recognized that a Delta Levee Program was necessary. With the 1973 Flood Disaster Protection Act California's Delta Levee Program was a fact. Levee maintenance could from then be supported by the State. In the course of time this program evolved into the Delta Levees Maintenance Subventions Program and the Special Flood Control Projects, prior to some other components. From the Subventions Program reclamation districts can apply yearly for grant funds, based on their own maintenance plan. The DWR reviews these plans and with approval from the Reclamation Board agrees with the Reclamation Districts on the reimbursement with a maximum of 65% of the total costs. The Special Projects program was established to accomplish projects of specific interest to the State. Grants from this program are for example used for levee improvements, emergency preparedness and studies supportive to Delta flood control. (DWR, 2006-3; DWR, 2005)

2.3.10 Floods of 1986

After the 1986 flood event, caused by warm winter storms, the State legislature developed target elevations and cross sections for levees throughout the Delta (Mount, 2005). The Sacramento River Flood Control Project was started to evaluate 1,059 miles of levees along the Sacramento River. This project, carried out by the Corps of Engineers, lasted until 2003. 89 miles needed significant repairs, of which most have been completed. But the criteria used in those evaluations are outdated and the Corps has recently developed new seepage design criteria. (DWR, 2005; USACE, 2005)

Another result from this flood was the awareness that the State can often be held responsible for flood damage. A case "Paterno vs State of California" was started after the 1986 flood and lasted until 2003 when court decision stated that when the State had accepted and operated the flood defense systems, even when they did not construct it themselves, they were still responsible for the structural integrity. (DWR, 2005)

2.3.11 California River floods 1997

In 1997 the river levees along the San Joaquin and Sacramento Rivers proved, just like in 1986, not to be sustainable against extreme river discharges. In January of that year warm tropical storms caused melting of the Sierra Nevada snow, which was accompanied by heavy rainfall. The runoff posed a heavy load on the river levees. The 30 levee breaches that were the result were mainly seepage failures. The flood forced more than 120,000 people from their homes and damaged or destroyed about 30,000 residential and 2,000 business properties. Six people were killed. In response to these damaging floods in California's Central Valley, the Corps of Engineers convened a Levee Seepage Task Force. Results from their study recently led to new, sharpened design criteria for under-seepage. Other new plans and funds that were made available after this flood were again reduced in the following dry years when the State faced a fiscal crisis. (USACE, 2005) (UCDavis, 2006) (DWR, 2005)

2.3.12 Jones Tract 2004

At June 3rd, a dry weather Delta levee failure at Jones Tract surprised local authorities. The levee, founded on peat, suddenly failed of a reason which is still not known. The evidence got washed away with the upcoming spring tide. It caused nearly $ 100 million in emergency response and water pumping costs. The water supply infrastructure was shut down for several days. 11,000 acres were flooded by only one levee breach. Several other levees almost failed.

Figure 2.20 2004 Upper Jones Tract levee failure (Reid, 2005)

Last century more than 140 levee failures caused inundation of the islands, most of them during flood season. The most recent flood fights in January 2006 (Figure 2.21) are an example of how fragile the Delta system is today.

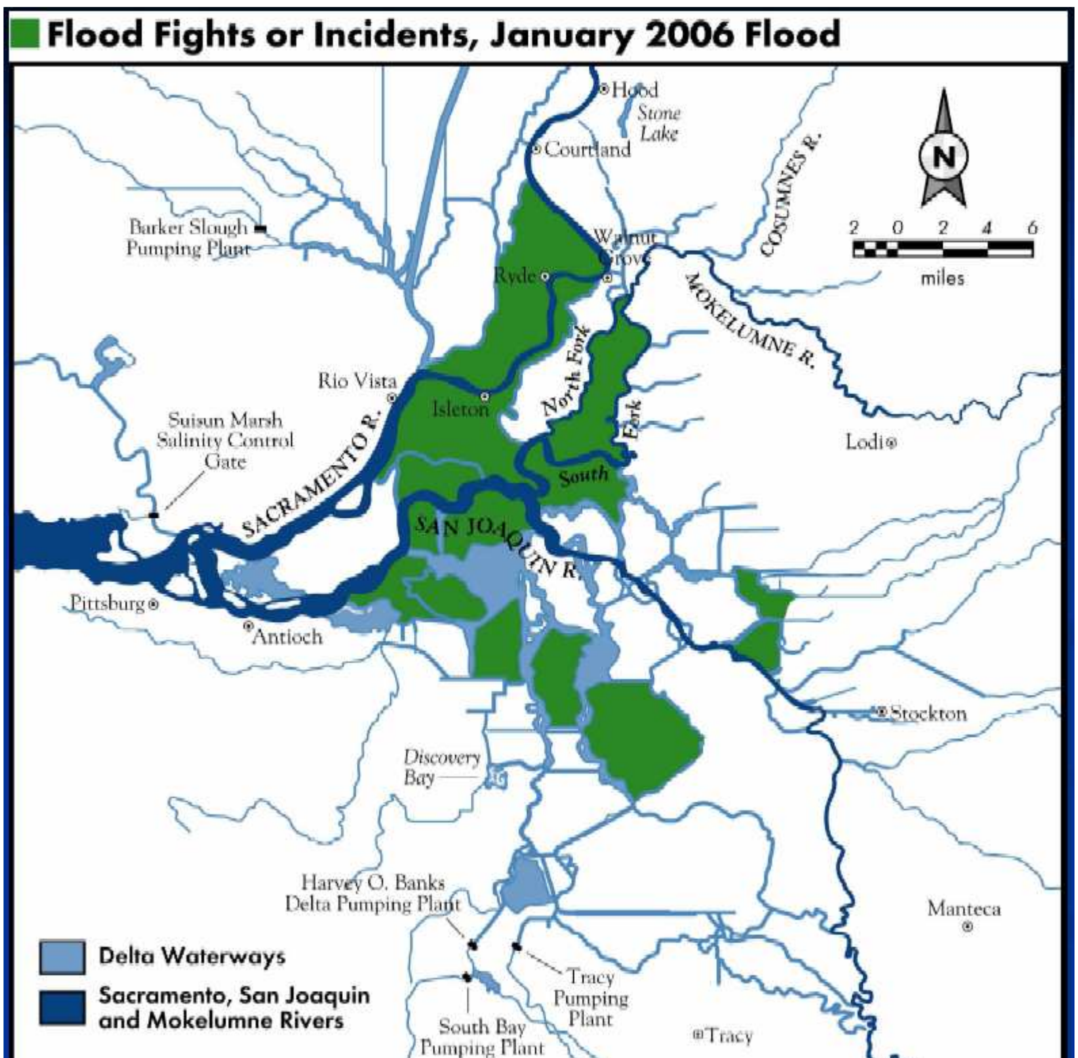


Figure 2.21 Flood fights January 2006 (UCDavis, 2006)

2.3.13 New Orleans 2005

The previous paragraphs focused on floods in the California Central Valley and Delta, but in the meanwhile the Mississippi River valley had suffered from "The Great Flood" in 1993, which was the worst since the 1927 flood and inundated 840,000 km^2. It was a nationwide flood also affecting the Midwest of the USA and the Missouri River tributaries. It was at that moment the most costly nationwide flood: damage was estimated to be about 15 billion US dollars. (DWR, 2006-3)

But better remembered today and more influencing to the current flood policies is the 2005 New Orleans flood. On the 29th of August, 2005, a storm surge caused by hurricane Katrina led to numerous levee failures, the flooding of 75% of the metropolitan areas of New Orleans and the death of more than 1,300 people. The damage is in the order of $ 200 billion, which is the most costly engineering catastrophe in history. More than 450,000 people had to evacuate. (Berkeley, 2006)

The levees mainly failed because of overtopping, which led to erosion and failing of the flood walls within and on top of the levees. Flood walls were often too shallow and could not prevent under-seepage problems as well. Some other levees were only constructed out of locally dredged material which was highly erodable. These levees were easily washed away. There were also problems with transitional structures between different flood defense systems. The Berkeley investigation team concluded that, although the storm surge was quite strong, the levees failed because of shortcomings in the levee systems. Background for these shortcomings is formed by organizational and institutional problems between governmental and local instances which were jointly responsible for design, construction, operation and maintenance of the flood protection systems. (Berkeley, 2006)

The disaster in New Orleans has again focused the attention on the importance of a well designed and maintained flood defense system and on the response to such a disaster. New Orleans had already been flooded after a hurricane before in 1915, 1940, 1947, 1965 and 1969. After the 1965 storm surge, caused by hurricane Betsy, a new flood protection system was authorized. This project was not finished yet when Katrina arrived. Projects to prevent similar disasters in the future have now been started.

It was this catastrophic event that also created the awareness in California that an equal disaster could happen to them and to call out a state of emergency.

3 Description Netherlands

The Netherlands is famous for its water and water defenses. There is admiration for structures like the Eastern Scheldt storm surge barrier, but at the same time people do not understand why the Dutch are living below sea level. This chapter will give some insight in the how and why of the Dutch water defenses (3.1 and 3.2). Similar as to previous chapter aimed flood risks, assessed threats that the flood defenses are exposed to and initiatives to guarantee safety in the future are described. Organizations related to flood defenses are also mentioned. In the last part the most interesting or devastating floods of the past are discussed.

3.1 Why flood protection?

Figure 3.1 Netherlands above and below mean sea level (Deltawerken.com, 2006)

The Netherlands, in western Europe, is 41,526 km^2 large, including Lake IJssel and the Wadden Sea (Figure 3.1). Of the total land area, approximately 34,000 km^2, 25% is situated below mean sea level, with a maximum of 6.7 m. 65% of the country would be flooded daily when there were no levees and dunes. About half of the 16.3 million inhabitants of the Netherlands live below sea level surrounded by large infrastructure works and prosperous economic areas such as the main ports Schiphol airport and the Port of Rotterdam.

To protect the country against floods from the rivers Meuse, Waal and Rhine and from storm surges from the North Sea, Wadden Sea and Lake IJssel, water defenses are built. (Huisman, 1999)

3.2 Dutch water defense system

3.2.1 Primary water defenses and regional water defenses

The Dutch water defenses are divided in primary water defenses and regional water defenses. The primary water defenses have a direct water retaining function to outside water, for example the Sea, Lake IJssel and the large rivers. They protect 53 dike ring areas and have a total length of 3,600 km (Min V&W, 2006). They include the coastal dunes, the river levees, sea levees and the Lake IJssel closure dam and storm surge barriers in the southwestern part of the country. The water level against those water defenses are influenced by the tide, waves, storm surges and/or river discharges. Regional water defenses are all water defenses that are not primary defenses and have a total length of 14,000 km. Examples of regional water defenses are canal levees and 'boezemkaden', which surround polders. Most of the dike ring areas as in Figure 3.3 enclose polder systems with a fixed polder water level. Those polders with polder ditches drain on the regional waters, which in their turn drain on lakes, rivers or directly on the sea. Therefore the regional water levels do not fluctuated much. Regional water defenses constantly hold back water, in contrast to the primary water defenses. (Figure 3.2) (RIVM, 2004)

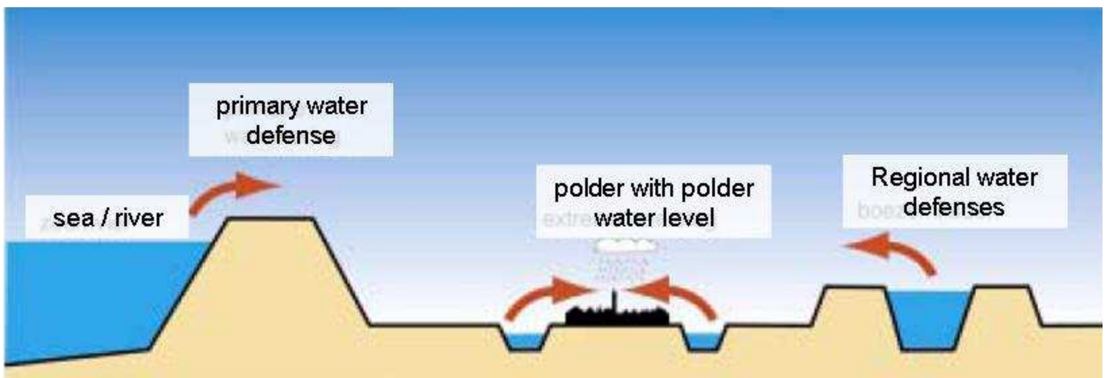


Figure 3.2 Difference primary water defenses and regional water defenses (STOWA, 2004)

3.2.2 Risk based levee design

The safety against flooding is based on chance of flooding, due to overtopping or levee breaches, and on damage determined by loss of human life, if quantifiable, and economic consequences:

Risk = Chance of flooding x Damage

The capitalized risk can be derived as: (Vrijling, 2002)

$$R_0 = P_0 S_0 \tag{3-1}$$

With:
R_0= Risk of flooding in year 0 [Euros/year]
P_0= Probability of inundation in year 0 [1/year]
S_0= Damage in year 0 [Euros]

This means that the risk in year zero is the probability of flooding in year zero multiplied by the damage in year zero. The risk in year 1 is then:

$$R_1 = \frac{P_1 S_1}{(1+r)} \tag{3-2}$$

Where:
r= the reduced interest rate, which is the interest reduced by inflation and increased with the economical growth [-]

Or in general:

$$R_i = \frac{P_i S_i}{(1+r_1)(1+r_2)....(1+r_i)} \tag{3-3}$$

With:
r_i= The reduced interest rate of year i [-]
R_i= Risk in year i [euros/year]
P_i= Probability of inundation in year i [1/year]
S_i= Damage in year i [euros}

Or with r=constant, P_i=constant and S_i=constant:

$$R_i = \frac{PS}{(1+r)^i} \tag{3-4}$$

The Total Risk (TR) is then:

$$TR = \sum_{i=0}^{\infty} R_i = \sum_{i=0}^{\infty} \frac{PS}{(1+r)^i} = \frac{PS}{r} \tag{3-5}$$

In the late 1950s the probability of flooding was set to 1:125,000 years, which was converted to a water level with a 1:10,000 chance of appearance per year for the sea water defenses along the North Sea coast: the normative high water (NHW). The other primary water defenses have a lower safety level, because they protect an area with less economic value. The safety standards for all primary water defenses are defined in the Flood Protection Act (1996) and vary from 1:10,000 per year to 1:1,250 per year. (Figure 3.3) (Table 3.1) (RIVM, 2004)

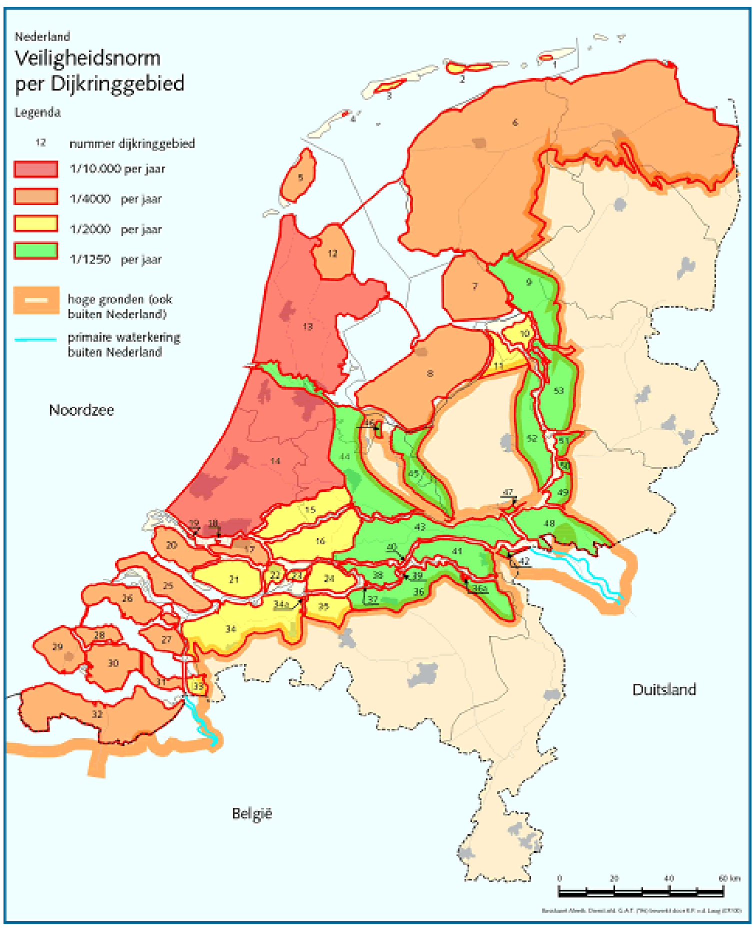


Figure 3.3 The 53 Dutch 'dike ring areas' with their aimed safety level (VNK, 2005)

Table 3.1 Safety classes for the primary water defenses

Primary water defenses	Probability of exceedance per year
River defenses upstream	1/1,250
River defenses downstream	1/2,000
Sea defenses Zeeland, Lake IJssel and North of Netherlands	1/4,000
Sea defenses along the coast of Holland	1/10,000

In the Flood Protection Act, the regional water defenses are not mentioned. For these water defenses the IPO-standards are used, which is an inter-provincial assembly, written in 1993. The IPO-standards do not give uniform rules on which safety is required. Water boards can propose a required safety from these standards, which then has to be established by the Province. To determine if a levee is safe enough, each polder has to be classified in one of the 5 safety classes (Table 3.2). The safety level is the probability of exceedance of the 'boezem' water level per year. The probability of failure (chance of flooding) is smaller than the probability of exceedance of the normative water level. For the regional water levels this difference is about a factor 0.2. For the primary water defenses that the probability of failure is approximately 0.1-0.01 times the probability of exceedance of the normative high water. (STOWA, 2004) (TAW, 1993)

Table 3.2 Safety classes for regional water defenses (STOWA, 2004)

Safety class Regional water defenses	Probability of exceedance per year
I	1/10
II	1/30
III	1/100
IV	1/300
V	1/1,000

3.2.3 Loads: Hydraulic boundary conditions

Climate

The Netherlands is bordered by the North Sea in the west and Wadden Sea in the north, crossed by large rivers as the Rhine, Meuse, IJssel and Scheldt and filled with lakes. Lake IJssel, by far the largest lake was created by closing of the Zuiderzee with a closure dam. With a mean temperature of 9.4^{o}C and a mean precipitation of 750 mm/year, the country has a humid, temperate climate with warm summers. The wind is dominantly west with a mean wind speed of 3.5 m/s.

Hydraulic boundary conditions

The most important loads on the levees are hydraulic boundary conditions. These hydraulic boundary conditions are used to design new levees and to evaluate the safety of existing levees. They vary for river levees, sea defenses and lake levees. For sea levees tide and wind set-up are combined. Measurements are extrapolated to the desired chance of appearance to determine the Normative High Water. Models like WAQUA are used to determine the NHW between two measurement stations. The Dutch coast is subject to a maximum tidal range of about 4 m and wind set up of maximum about 3 m. Wind waves, seiches, local water rise due to wind gusts and oscillations and expected sea level rise are also taken into account. The significant wave height for wave run-up can be up to nearly 10 m.

At the (upstream) river water levels are mainly determined by incoming discharges. Two rivers are mentioned here: the rivers Meuse and Rhine. The Meuse is mainly a precipitation river, with extreme discharges in winter (Figure 3.4). The maximum discharge measured in the Meuse is 3,120 m^3/s and dates from 1993. Levees along the Meuse should be able to withstand a discharge of 3,800 m^3/s at Borgharen, where the Meuse has just entered the country. (RWS, 2001)

The River Rhine is a mixed river. Precipitation and snowmelt from the Alps influence the discharges in the River Rhine, which therefore peaks in winter and spring (Figure 3.4). The maximum discharge ever measured in the Rhine is 12,600 m^3/s in 1926. Currently the levees should be able to retain 16,000 m^3/s, measured at Lobith, where the Rhine enters the Netherlands. (RWS, 2001)

Again extrapolated measurements combined with model results determine the Normative Discharge and therefore the NHW. Sometimes wind waves are taken into account. Downstream river levees will be subject to tide and wind set-up. Wind waves, local water rise and oscillations can again become important here.

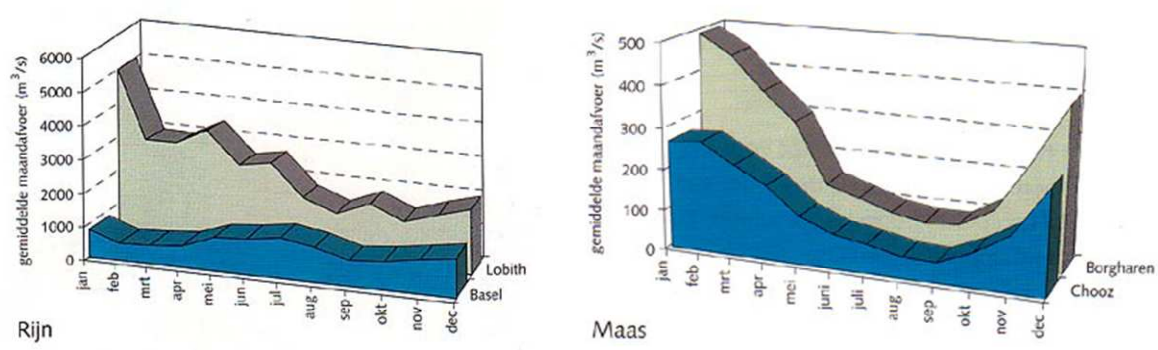


Figure 3.4 Mean monthly discharges Rivers Rhine and Meuse (natuurdichtbij.nl, 2006)

Other loads:

But there are other loads that can affect a levee for example ice load, collision from a ship, vandalism, trees or animals that cause holes and traffic on top of the levee. These loads are often not specifically taken into account in design and levee evaluations.

3.2.4 Strength: failure mechanisms and levee design

Figure 3.5 Geological map of the Netherlands (TNO-NITG, 2006)

How resistant a levee is depends on its shape and dimensions, for example the levee width, slope and height. It also depends on the soil characteristics of the levee and the levee base. Whether the levee is constructed with highly pervious bad compacted sand or from clay influences the resistance against for example seepage. In the Netherlands most levees contain clay or a combination of sand and clay. Clay forms a waterproof layer on top of the levee or within the heart of the soil structure. Regional levees were often not designed and can therefore contain undesirable materials as peat. Beneath the levee construction a clay layer is often present with below that a sand layer, which as will later be explained, influence the resistance of a levee against piping. Figure 3.5 shows a simplified geological map of the Netherlands with the most important deposits.

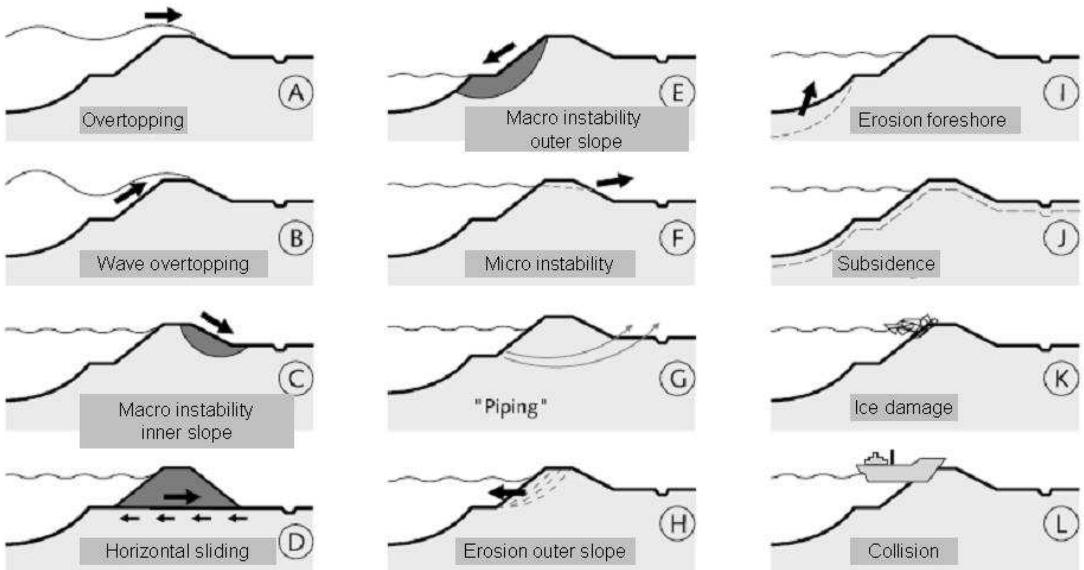


Figure 3.6 Failure mechanisms soil structures (TAW, 1998)

But against which forces or mechanisms does a levee have to be resistant? Forces have already been mentioned and the most important failure mechanisms that can be initiated by these forces are shown in Figure 3.6. These failure mechanisms represent situations where the levee is no longer able to perform its main function: retaining water. This could already be the case when the levee is not yet breached, for example when the levee is overtopped.

For the design of new levees as well as for existing levees the most important failure mechanisms are used to evaluate whether no unexpected failures could occur during extreme conditions. The process for a new levee or levee improvement is explained in Figure 3.7. This process is iterative. First a standard levee design will be used (Figure 3.8). This design will be optimized using the expected failure mechanisms for levees.

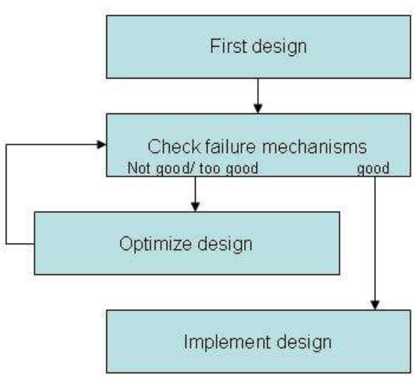


Figure 3.7 Levee design process (TAW, 1999)

The height of a primary soil structure depends on the Normative High Water level (NHW) for that specific location, which, for the primary water defenses is revised every 5 years. The construction height of a levee is NHW level with a freeboard of at least 0.5 m and expected settlements of the levee and local subsidence in the plan period (normally 50 years) added to that. The freeboard consists of wave run-up, expected local water level rise due to for example wind gusts and expected sea level rise during the plan period. The outer slope has to be 1:5 for a sea levee and

1:4 for a lake levee. The inner slope is at least 1:3. The top width of a levee should be at least 2 m, but it should be more if a road is constructed on top. (TAW, 1999)

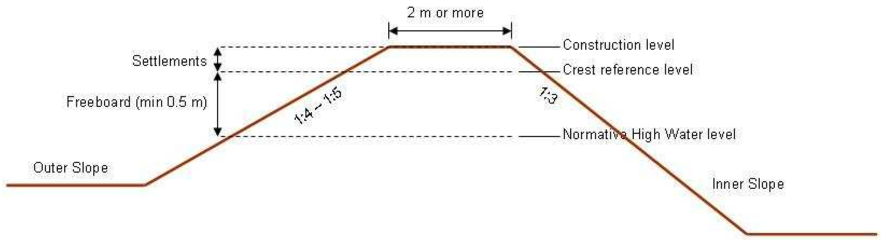


Figure 3.8 First levee design

When designing a levee, not only dimensions are important. One has to think about what soils are used and which protection is necessary. In the Netherlands clay is preferred for levees, or sand in combination with clay. Grass and stone revetments are commonly used to protect the levee against erosion and vandalism by animals and people.

3.2.5 Inspection and maintenance levees

To guard the safety of the water defenses, it is necessary to regularly evaluate their conditions. In the Flood Protection Act is stated that each 5 years levee authorities have to report on the conditions of the primary water defenses following the prescriptions from the ministry, the 'Voorschift toetsen op veiligheid' (VTV). Levee authorities are responsible for levee evaluations in their region, for which they often hire engineering companies. Levee authorities in this context are often the Water Boards (90%), but for water defenses not bordering land (for example the Closure Dam) the State, often a Rijkswaterstaat department (see 3.3.3), is responsible (10%). (Huisman, 1998)

The first two evaluation reports were finished in 2001 and 2006. According to the 2006 report 24% of the primary levees and dunes are not safe and of 32% no opinion could be given. Less than half the primary water defenses, 44%, are safe according to the 2006 evaluations. Based on results from the reports a flood protection plan is established, in which improvements for the weak sections are proposed. (Min.V&W, 2006)

In the meantime the levee authorities are supposed to have a management plan or register. In this plan instruments are given to the levee manager, who is responsible for the day-to-day management of the water defenses. A yearly visual inspection is part of this plan.

As mentioned before, there is no legislation (yet) on the regional water defenses. The IPO-standards, written in 1993 can be used to determine the safety of regional levees against flooding. The safety of the regional water defenses is a responsibility of the water boards. Methods used are likely to resemble the methods used for the primary water defenses. The first and last systematic evaluation on part of the regional water defenses dates from 1993. Only for 1,730 km out of the 14,000 km sufficient data was available to give a reliable indication of the levee conditions. About 20% of the 1,730 km did not meet the requirements. (TAW, 1993; RIVM, 2004)

Management and maintenance of the regional water defenses is a responsibility of the water boards. Periodic visual inspection is used to determine the current state of the levees. This visual inspection focuses on changing dimensions of the levees (reduction in height, deformations) and on levee damages as cracks, wet spots and animal and tree holes. Based on these inspections maintenance or repair activities will take place or further investigation is done. (TAW, 1993)

3.3 Threats and initiatives

3.3.1 Threats: drivers of change

The climate is changing and floods seem to occur more often than they did before. But what effect does the climate have on the occurrence of floods? And is it only that the water levels become more extreme, or are the levees getting more fragile as well? At the same time one flood causes more damage nowadays than it used to. But which factors drive this change? Some facts:

Loads

The changing climate is the main driver. Last century the mean temperature in the Netherlands has increased with 1°C. Continuation of this trend will expose in: (MNP, 2005)

- Sea level rise, predicted between 20 cm and 110 cm for this century, will pose an extra pressure on the Dutch sea defenses. Last century the sea level rise was 20 cm.
- Increasing river discharges; dryer summers and wetter winters will cause a change in mean discharges and peak discharge (see Figure 3.9). Design discharges will have to be adjusted to that: the Rhine to 18,000 m^3/s and the Meuse to 4,600 m^3/s.
- Increasing wet periods; the nature and frequency of storms and heavy rains will change, which will induce for example larger wave heights or a change in the timing of peak discharges at the rivers.

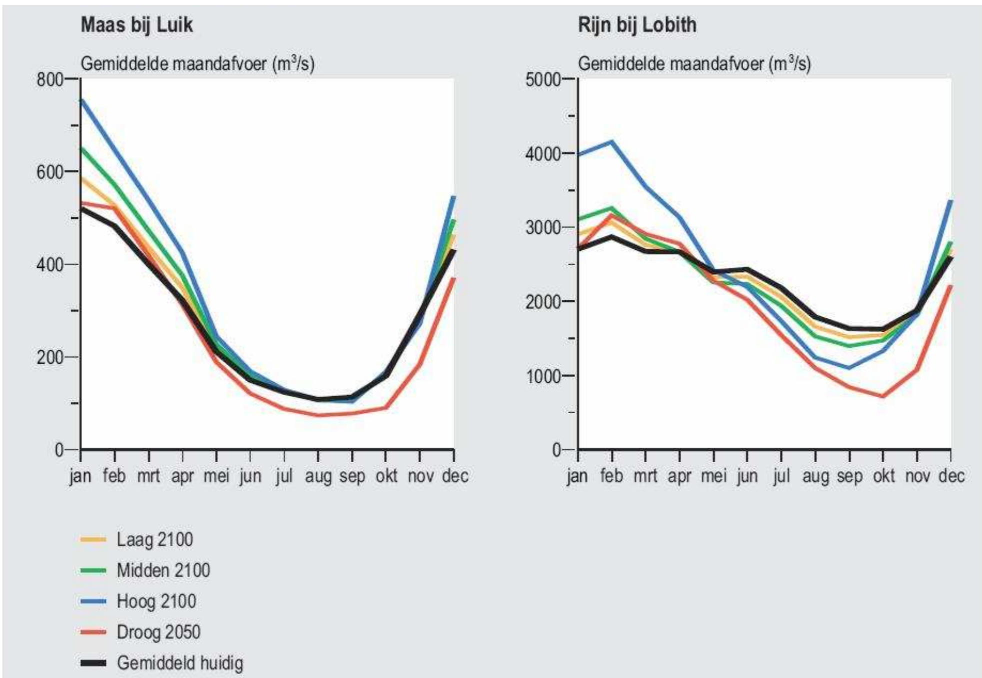


Figure 3.9 Expected future mean monthly discharges Rivers Meuse and Rhine. The black line represents the current mean, the blue line gives the highest estimate for 2100 (MNP, 2005)

Strength

But the changing climate also has an influence on the strength of the levees for example:

- Increasing periods of drought; dried out soils can make levees more vulnerable; especially peat is vulnerable, while its volume weight can even get below the weight of water
- Ongoing subsidence; will get worse in dry periods. The last 1000 years the peat subsoils have subsided about 2-3 m. Due to dewatering in dry periods the oxidation of peat can increase to 1 cm a year.
- Extremely wet periods cause the soils in the levee to become saturated, which makes them heavier and more vulnerable to instabilities.

Damage

The damage that floods cause is also increasing, this as a result of:

- Population growth; the Dutch population still increases, especially in flood prone areas. A flood will take more lives and destroy more property.

- Economic growth; the still ongoing economic growth will result in higher vulnerability to economic and social disaster.

3.3.2 Dealing with increasing flood risks: initiatives

The Economic damages and loss of life, mentioned in last paragraph, are not scaled to the current situation. 1960 values are still used to estimate the consequences of a flood. New methods are developed to take this into account. The FLORIS project, Flood Risks and Safety in the Netherlands (or in Dutch: VNK) intends to get more insight in the chances of flooding and the consequences of a flood. The FLORIS project was initiated by Rijkswaterstaat and executed under the auspices of DWW. The economic optimum is still of the 1960s standards, but does not suffice anymore. FLORIS was set up to come up with new computational methods to determine the risk of flooding instead of probability of exceedance of a water level, to supplement the 1960 knowledge. Future changes in economic value, population growth and climate changes will be taken into account. From each dike ring the probability of flooding and the weak links are identified. The consequences of flooding are estimated and put in a GIS map to give a better impression of the costs and benefits of investments in safety against floods. (VNK, 2005)

Another initiative is the Room for the River project, established after the 1993 and 1995 extreme river water levels; goal is to find new solutions for a better protection against the water from the large rivers. The project was started in 2000 and initiated by the Ministry of Transport and Public Works. It works with a new policy: instead of raising levees other methods are applied, for example flood plain lowering, river widening, creation of secondary channels, moving levees further from the river bed and creation of retention polders (Figure 3.10). The Room for the River project will be finished in 2015.

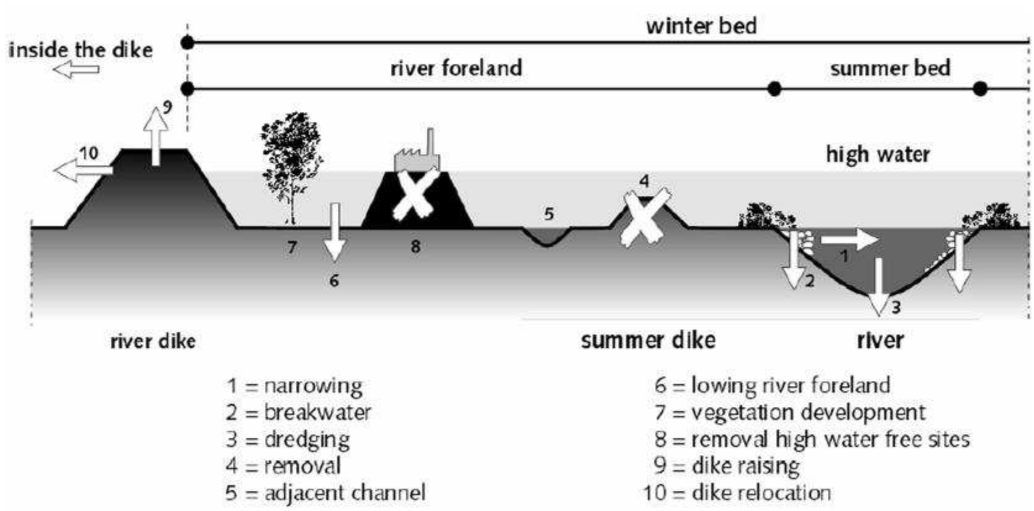


Figure 3.10 Examples of creating 'room for the river' (TAW, 1998)

Other examples of currently running programs are for example: ComCoast, a European project to developt new innovative solutions for flood protection in the coastal areas, IJkdijk, a levee test facility, Grensmaas, to improve the safety against flooding along the Meuse and let it more return to its natural appearance and "Nederland leeft met water" (The Dutch live with water), which was set up to make the Dutch more aware of the (new) water policy in the Netherlands with commercials a website and others.

3.3.3 Organizations involved in water defense policy

There are three public layers with authorities responsible for the water defense system. First there is the government on national level, then the provinces and then the water boards and municipalities on regional and local level. The most important authorities concerning water defenses are mentioned in this paragraph. (Figure 3.11)

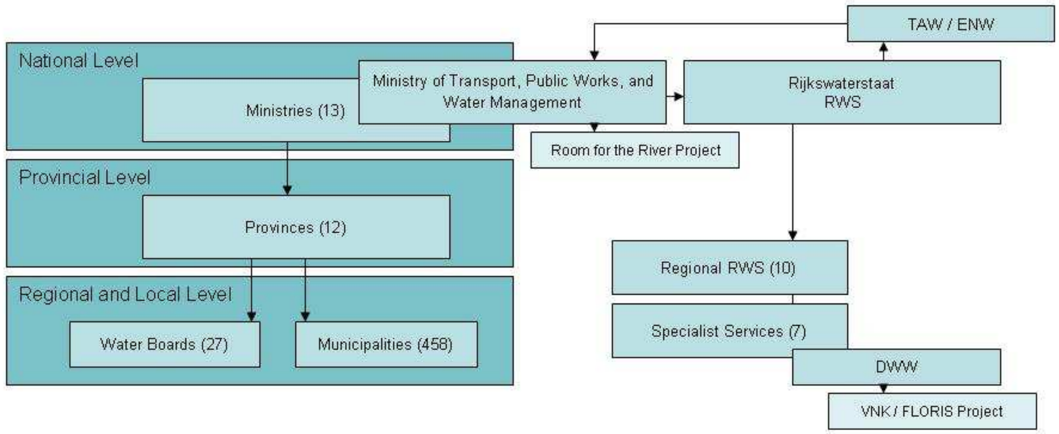


Figure 3.11 Public sector involved in flood policy and protection

Ministry of Transport, Public Works and Water Management (Min V&W): The Min V&W is of the 13 ministries the one most involved in flood protection. The Minister of the department V&W is supported by the State-Secretary for V&W, who is responsible for aviation, person transport by rail and for water. One of the governmental involvements on water policy is the 5-yearly report on the safety of the primary water defenses. The ministry will provide the hydraulic boundary conditions for this safety assessment and prescribe evaluation methods.

Rijkswaterstaat (RWS) or Directorate General of Public Works and Water Management: Was established in 1798 to deal with water related affairs on national level. RWS is nowadays the executing body of the Min V&W responsible for flood protection, water quality and quantity and for traffic control, shipping and public transport. RWS also supervises the provinces and water boards. Rijkswaterstaat has 10 regional service bodies, which mainly deal with infrastructure projects and 7 specialist services, for example the National Institute for Coastal and Marine Management (RIKZ) and the Road and Hydraulic Engineering Institute (DWW).

Provinces (Figure 3.12): The Provinces are decentralized bodies concerned with all different disciplines of government. They supervise the Water Boards on two aspects defined in the Flood Protection Act: technical quality water management and on agreement between Water Board policy and Municipality policy. The Provinces define the Water Board Tasks and set standards for the regional water defenses. (TAW, 1998)

Water boards (Figure 3.12): decentralized public bodies which are responsible for local and regional flood control, water quantity and water quality. They have their own financial system, supported by the taxpayers within the water board area. At this moment there are 27 water boards in the Netherlands. (waterschappen.nl, 2006)

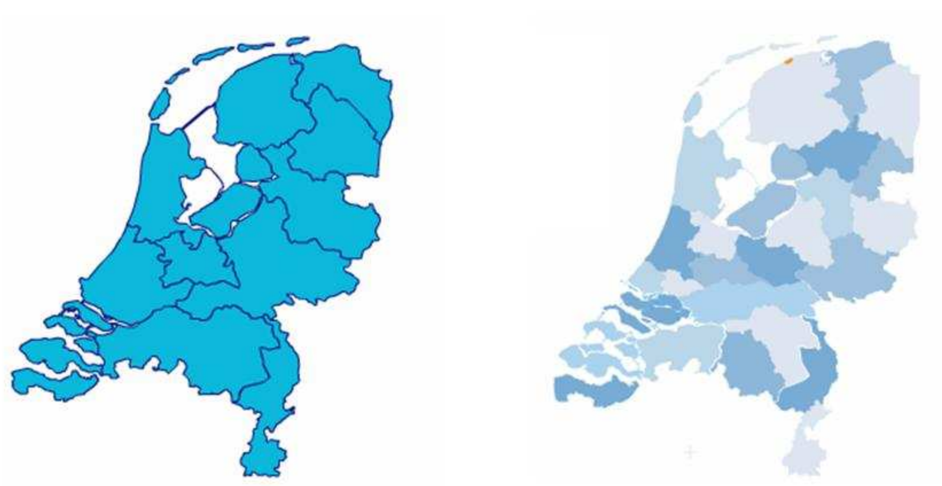


Figure 3.12 The Dutch 12 Provinces (left) and 27 Water Boards (right) (Provincies.nl, 2006; Waterschappen.nl, 2006)

Municipalities: The involvement of Municipalities in flood protection is limited to the development plans, in which flood defenses have to be adapted. Besides this, the Municipality is responsible for contingency and evacuation plans. (TAW, 1998)

Technical Advisory committee for Flood Defenses (TAW): Was since 1965 an independent advice committee for the minister of Transport, Public Works and Water management. In 2005 this committee was replaced by the ENW. They published technical reports concerning water defenses.

Expertise Network for Flood Protection (ENW): Replaced the TAW in 2005. Is still an independent advisory committee, not only for governmental authorities. ENW will also be a platform for exchange of knowledge. The ENW (like TAW) contains four work groups: safety, technique, coast and rivers. ENW will not only focus on technical problems, but also on policies and social interests. The ENW works under the responsibility of DWW. (ENWinfo.nl, 2006)

The Road and Hydraulic Engineering Institute (DWW): Is one of the seven specialist services of Rijkswaterstaat and is specialized in road and hydraulic engineering.

Besides the public sector, the private sector is also involved in flood protection. Consulting engineers, contractors and research institutes perform a substantial part of the research, advisory work and construction on flood protection. They often get their assignments from Rijkswaterstaat or water boards, but foreign governments and companies also show an increasing interest in this Dutch expertise.

3.3.4 Disaster Management

Rijkswaterstaat services RIZA en RIKZ provide information on extreme water levels. Emergency water levels are defined by the minister and are revised every 5 years. In case emergency water levels are expected, the minister should warn the involved levee managers. The levee managers can install extra levee surveillance and will inform the municipal executives, who are responsible for a disaster response plan. This response plan contains a scheme of relevant facilities, institutions and organizations and gives insight in whose in charge and what are the responsibilities. The union of water boards has provided a framework: water board and the disaster prevention. With this framework water boards can write a contingency plan, which should support the municipal response and contingency plans. The provincial board is suppose to coordinate and regulate these activities, but will have a more executive role as the disaster has more than local significance. The provincial Queen’s commissioner can ask for help from the minister of internal affairs and from governmental instances such as the army. (TAW, 1998)

3.3.5 After a flood

In the Netherlands there is no insurance to compensate for flood damage. If inhabitants suffer from failure of a water defense, they can try to get compensation from the Government, following the Compensation Act on Disaster Damage and Accidents. But it is hard to get such compensation and often only part of the damage is reimbursed.

A common believe in the Netherlands is that a flood is an act of god that no insurance company is able to bear the risks of. In the meanwhile inhabitants have no insight in to what risks they are exposed to and feel no responsibility what so ever. They trust the State to protect them against floods and blame the Government if otherwise. Living in risk areas, such as flood plains and in the deepest polders, is still not experienced as risky.

Early 2006, the Advisory Committee Water renewed the discussion on the possibility to insure flood damage. This committee is a personal advice organ established by and used by the States secretary for V&W. Based on an introductory research on the possibility to insure water damage, the committee advises to make water damage insurable. If flood insurance in the Netherlands is possible and desirable is still an ongoing discussion. (AcW, 2006)

3.4 History of Dutch flood control

3.4.1 Overview

Floods disasters often lead to new ideas, researches, legislation and prestigious constructions. In other words: floods are needed to get the attention for water defenses and investments in water defenses. In this paragraph is described how floods formed the Dutch water defense policy and system. The most important flood events of the Dutch history are summarized in Table 3.3.

Table 3.3 Selection of Dutch floods that had an impact on water defense policy

Important or major floods	Impact on Dutch flood policy
Floods of 838 and 1014 AD	First polders
Zuyderzee floods 1170 and 1196	1225 first Water Boards
St. Felix flood and Allerheiligen flood 1530 and 1570	First levee design and reclamation rules in 22 articles
River floods 1861	River canalization / normalization
Christmas flood 1717 and storm surge 1916	Construction of Closure Dam
Storm surge 1953	Delta Act 1958; Levee design based on risk analysis; Construction of Delta Works
River floods 1993 and 1995	Delta Act Large Rivers; Room for the River project
Wilnis 2003	Safety standards regional water defenses

3.4.2 First protection against floods 300-100 BC

The Dutch shoreline has transformed a lot over the course of time. In the last Glacial Era the Position of the Dutch coastline was situated 200 km west of its current position and the sea level was about 60 meters below today's mean sea level. In the Holocene, the sea level rose and reached the stage of where the coastline is currently situated some 2000 years ago. At that time the Netherlands was a large swampy delta. People that already dwelt in these areas, which regularly were flooded, lived from hunting and fishing. They build their huts on natural levees along the rivers.

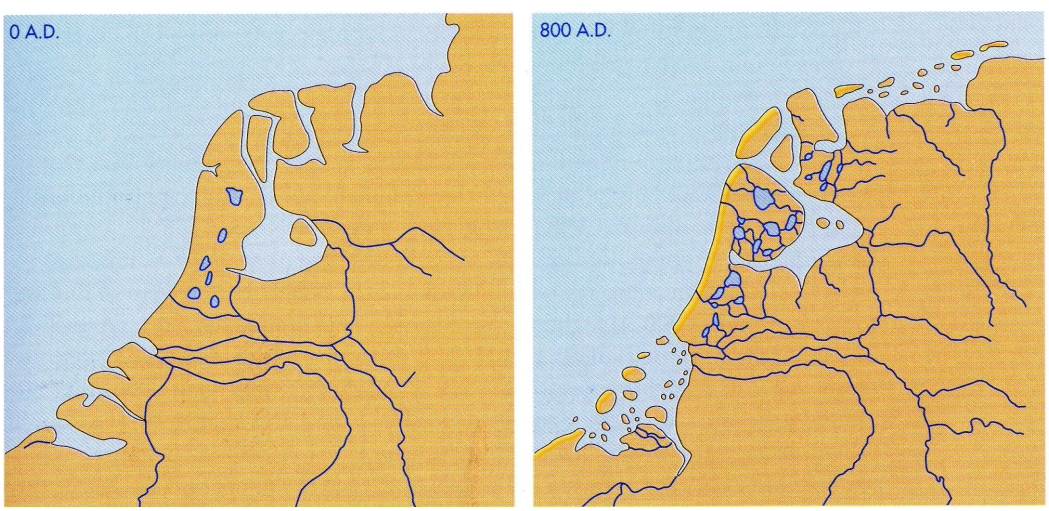


Figure 3.13 The shape of the Netherlands around 0 and 800 AD (Huisman, 1998)

The first protection works against the rising sea water were built 300-100 BC, under influence of the Romans. Artificial dwelling mounds and small river levees were constructed to protect the houses from the water and to construct roads. People started to excavate peat that was dried and used as fuel, and thus created small lakes. Other areas were drained and used for agriculture. (Dubbelman, 1999)

Between the year 0 and 1000 AD sea level rise, subsidence by drainage and occupation of the country led to an increasing influence of the sea and large areas were again lost (Figure 3.13). At about 800 AD the population of the Netherlands reached some 0.5 million inhabitants. The excavated peat lakes were also subject to the tidal influence of the sea and easily eroded to large tidal inlets. (Huisman, 1998) It was in this period that coastal dunes started to form.

3.4.3 Floods of 838 and 1014 AD

The first known floods with numerous victims are the floods of 838 AD and 1014 AD. The 838 storm surge inundated part of the northwest of the Netherlands, and caused about 2437 death. The 1014 flood also caused thousands of deaths after a break in the western coastline.

It was only after 1000 AD that the population significantly increased and started to regain influence on the sea. They created appropriate conditions for agriculture by building small dikes. Thus polders were created, where the water-regime was disconnected from the surrounding land such that high tide couldn't flood the land anymore. First these polders could be drained easily by gravity. At low tides little sluices were opened which released the surplus water. To lower the water table drains and ditches were dug. Agriculture and a drop of the water table caused the peat and clay soils to subside. Peat also oxidized, which increased subsidence even more. Therefore the ground water table had to be lowered again and an ongoing process was started. (Dubbelman, 1999)

3.4.4 Zuyderzee floods 1170 and 1196

Figure 3.14 Dutch Windmills (Huisman, 1998)

After 1100 AD subsidence had increased to such an extent that natural drainage was not possible anymore and the sea was again on the winning hand. As a result of storm disasters large parts of the country were again lost to the sea. The floods of 1170 and 1196 created the Zuyderzee, a large inland sea. But the sea was not the only threat. River discharges were increasing, and the levees people built along the downstream rivers, preventing the river to widen, enlarged and replaced problems. Upstream farmers in the eastern part of the country therefore also had to protect their land against water and had to build levees along the river. The levees were at first only 1 or 2 meter high, but some design technique was already implemented. New levees had a symmetrical performance and subsoil was excavated before construction. A new technique was implemented: building dams. The dams had to prevent intrusion of salt water during high tides and closed off tidal creeks. Behind the dikes and closure dams smaller areas were created, surrounded by intermediate storage areas. This was the start of the polder system that is typically Dutch. Artificial methods were invented to drain the excess water: hand and horse driven mills and later, from 1440, windmills (Figure 3.14).

At the same time an organizational switch took place. First the landowners themselves inspected and maintained their piece of levee, but it was soon recognized that the dammed areas became too large for that. Regional meetings were organized and representatives were chosen. The first water board was a fact in 1255: Water Board Rijnland, around the former tidal branches of the Rhine. Soon more water boards were initiated by the Count of Holland, Floris V, or by the communities themselves. The water boards were one of the earliest forms of government in the Netherlands. (Dubbelman, 1999)

3.4.5 St. Elisabeth floods 1404/1421

Poor water board organization and a lack of maintenance of the sea and river levees led to new disasters. First in 1404 and later in 1421 a storm surge from the North Sea destroyed the sea defenses as well as river levees and flooded the south western part of the country. In 1421 more than 2000 people died. This flood resulted, during the following years, in an inland tidal marsh: the Biesbosch (Figure 3.16). This area was permanently lost to the sea after a new sea flood in 1424 (Figure 3.15). These floods lead to increased attention for the water boards from the Count of Holland, Karel V, who established a research committee and provided money for rehabilitation works.

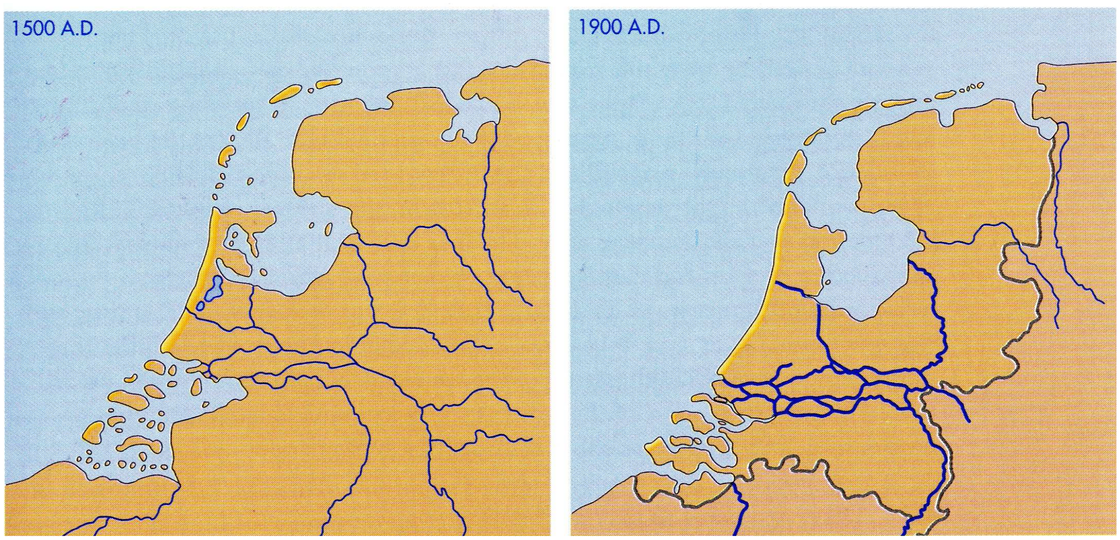


Figure 3.15 The shape of the Netherlands around 1500 and 1900 AD (Huisman, 1998)

3.4.6 St. Felix flood and Allerheiligen flood 1530 and 1570

After other floods as the St. Felix floods in 1530 and the Allerheiligen flood in 1570 Calamity management was established: the army could be mobilized in case of emergencies. To become a surveyor, one had to follow an education. After Simon Stevins mathematic ideas, new levee designs were made with flatter slopes and more solid material. A plan was made to reclaim land in the southwestern part of the country, with rules determined in 22 articles. The construction height of the levees was 4.27 meters, from which 0.61 meters was accounted for settlements. With new techniques, including the windmills it was possible to reclaim low lying areas, first in the Zeeland Delta (Figure 3.16) and later also in the (north) western part of the country. Large areas were reclaimed in the following centuries, to create agricultural land and fulfill the needs of the growing population. With the industrialization, in the 19th century, new steam driven pumping stations were introduced and made it possible to reclaim far larger areas in a shorter time period. (Dubbelman, 1999)

3.4.7 River floods 1861

Interventions in the winter beds of the rivers together with the forming of ice sheets in winter led to numerous problems in the river basins. The construction of canals, mainly to improve navigation, only made safety worse. For example the "Pannerdens Canal", constructed between 1701 and 1709. The attention now shifted to the River Meuse, Rhine and Waal. Large scale river floods in 1861 led to a river evaluation report in which improvements were suggested. A national water authority, Rijkswaterstaat, which was founded in 1798, realized many hydraulic works: bends were cut off, summer levees were lowered, river beds were normalized and objects were removed. These works resulted in fewer calamities and improved navigability and the discharge of water and ice. (Dubbelman, 1999)

3.4.8 Christmas flood 1717, Storm surge of 1916

During the 17^{th}-19^{th} century the Northern provinces also faced flood disasters. First there was a storm surge in 1675, after which the authorities introduced yearly inspection of the levees. A northwestern storm at Christmas night of 1717 attacked Germany, Scandinavia and the Netherlands, taking 14,000 lives. In the Netherlands the Northern provinces and land along the Zuyderzee suffered most from the storm surge, which led to thousands of death and left whole cities and villages flooded. (Deltawerken.com, 2006; RWS, 1998)

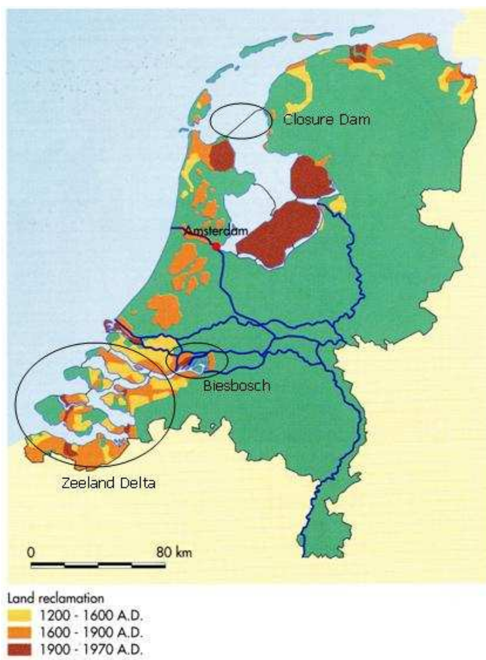


Figure 3.16 Reclaimed areas 1200-1970 and the closure dam, Zeeland Delta and Biesbosch area (modified from Huisman, 1998)

Storm surges in 1808 and 1809 led to new legislation. In the 1810 Levee Act levee maintenance and funding was unified for the whole country, which was split up in 17 dike ring areas. The water boards were responsible for maintenance in those dike ring areas. Unfortunately this law only led to conflicts and did not improve levee conditions. Yearly inspection was also part of the Levee Act, but was already abolished in 1814. In 1835 the whole Levee Act was cancelled, not to return until 1953. (RWS, 1998)

While almost all large inland lakes were reclaimed, new ideas arose. The closure of the inland Zuyderzee and reclamation of this area were already discussed since the 1850s and there were some reasons to do it. One reason was that the area was then better protected against sea floods, such as the 1717 disaster. And the other one was that salinity problems in the surrounding polders could be reduced. Two other advantages were the availability of fresh water and creation of agricultural land.

It was only after the 1916 storm surge, which flooded areas along the Zuyderzee that these ideas were taken serious by the government. The 32 km closure dam with discharge sluices was completed in 1932 and thus created Lake IJssel, a fresh water reservoir of 500 m^3. 170,000 ha were reclaimed and turned into farmland. (Huisman, 1998) (Figure 3.16)

3.4.9 Zeeland Sea Flood 1953

When plans were made to reclaim the Biesbosch area, the levees supporting this area did not seem sufficient against storm surges, when all storm actors were taken into account. In 1939 the government established a storm surge committee. Their task was to examine the safety of the Dutch water defenses at the coast and along the rivers. Up until that moment the design height of levees was based on the highest known water level. But the committee concluded that that was not sufficient anymore. Levees had to be raised and had to be qualitatively more sufficient. A statistical method was developed to determine the design height of the levees. After that more than one committee/person concluded that the water defenses in the Delta (see Figure 3.16) were in a terrible state. Van Veen made two design plans for the closure of the Zeeland Delta as a protection against storm surges. But water boards in the southwestern part of the Netherlands were too small at that moment and did not cooperate. And Rijkswaterstaat had other priorities. By only slowly starting to strengthen the sea defenses some of the recommendations of the storm surge committee were implemented. (Dubbelman, 1999; RIVM, 2004)

And then there was the 1953 storm surge. The night of 31 January to 1 February 1953 67 embankments breaches caused the flooding of an area of 500,000 ha. A water level of 3.85 m above NAP (Dutch reference water level) was far higher than the 3.28 m above NAP, which was the highest known water level. 1,836 people died together with about 200,000 livestock. Half of the 1,000 km of levee in that area was (partly) damaged. (Figure 3.17) (Dubbelman, 1999)

A few weeks later the Delta committee was established by the minister of Transport, Public works and Water management to come up with a plan to prevent future disasters and reduce salt intrusion, which was presented in 1954. The plan included:

- Closure of the tidal inlets, except the Rotterdam Waterway (Dutch: Nieuwe Waterweg) and Western Scheldt (because they have are important harbor entrance)
- Construction of dams and sluices behind the primary closure dams
- Strengthening of the sea defenses along the coast and along the Rotterdam Waterway and the Western Scheldt (Dubbeldam, 1999)

This plan resulted in the Delta Law in 1958 and contained the first standards for safety against floods. They were based on the plans and calculations already made by dr. ir. Van Veen. These standards first only focused on sea floods and did not include the rivers.

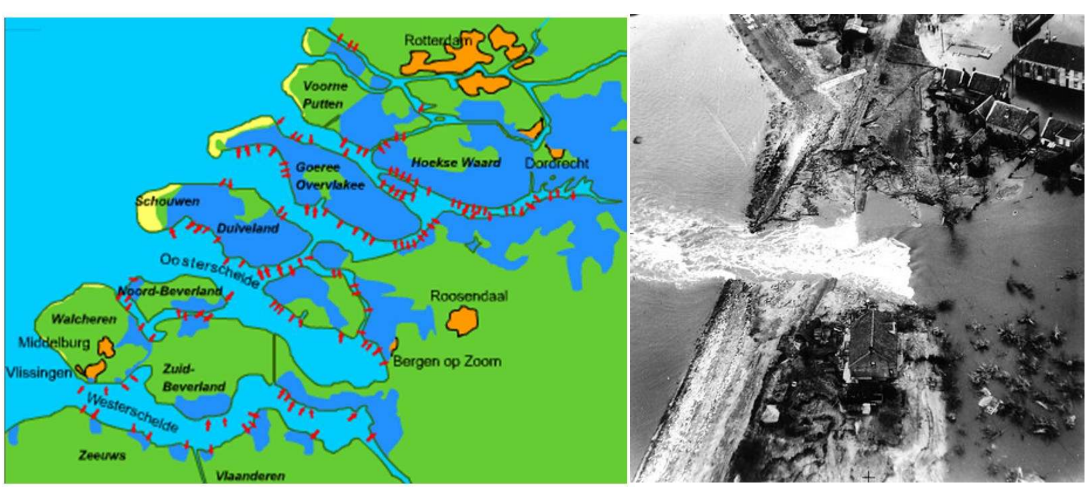


Figure 3.17 Levee breaches and inundated areas; picture of embankment breach Zeeland 1953 (Deltawerken.com, 2006)

The Delta Plan itself led to the Delta works (Figure 3.18). Six primary dams, which closed of the large sea arms, were supported by secondary dams to make construction possible. It took more than thirty years to design and build the Delta works, which reduced the total levee length with 700 km. The most impressive and expensive works are the Eastern Scheldt storm surge barrier and the Maeslant barrier, which both are open barriers which can be closed during heavy storms with extreme water levels. The Eastern Scheldt barrier was first planned as a closed dam, but of environmental reasons gates were made that can be closed in case of emergency. The Maeslandt barrier at the entrance of the Rotterdam Waterway had to be an open barrier because of the economic importance of the port of Rotterdam. Two storm surge doors which both have a length of 240 m can be closed at a storm surge with water levels of 3 m above NAP (the Dutch reference water level). (Deltawerken.com, 2006)

Figure 3.18 The Delta Project with two pictures of the Eastern Scheldt Storm Surge Barrier and a picture of the Measlandt Barrier (Huisman, 1998; Deltawerken.com, 2006)

3.4.10 Meuse and Rhine river floods 1993 and 1995

The 1953 storm surge also focused attention on the safety of the river levees. The highest known water level dated from 1926, when the River Waal caused calamities, and since that time no real problems had occurred. Additional to the plans of the Delta committee the Minister of Transport and Public Works and water board of Gelderland suggested an improvement of the levees to a 1:3,000 water level with a maximum discharge of 18,000 m^3/s at Lobith (where the River Rhine enters the country). The 1:3,000 was lower than the safety of the sea defenses (1:10,000 and 1:4,000), because a flood from the river is less devastating, partly because the water is fresh, than a flood from the sea. (RIVM, 2004; TAW, 1998)

The consequence of the new safety rule was that all levees had to be raised and strengthened. But because of large public resistance the minister established the committee on river levees (committee Becht) in 1975, to evaluate the 1:3,000 rule. Only 70 km, out of 1,800 km, of river levees was finished at that moment. Committee Becht came to the conclusion that protection against a water level of 1:1,250 was acceptable with a discharge of 16,500 m^3/s at Lobith. But still the public resistance prevented fast and extensive levee enforcement plans. As a result committee Boertien was established in 1992, who advised the same design frequency as committee Becht, but with an acceptable discharge of 15,000 m^3/s. In addition they advised to implement natural, landscape and cultural values into levee design. (RIVM, 2004)

But just after the committee Boertien published their advice the first river calamities since 1926 took place. In 1993 and 1995 flood waves overtopped and together with piping threatened levees to break. In 1995 100,000 acres were flooded and 240,000 people had to be evacuated. It turned out that most of the river levees were only protected against a 1:100 year flood level.

As an answer to these calamities the Delta Plan Large Rivers was carried out, which had the scope to speed up river levee improvements of weak spots that were identified in 1993 and 1995. A Delta law Large Rivers was installed, additional to the 1958 Delta Law. Levees had to be raised to the desired protection level in the year 2000.

To prevent the necessity of raising levees again within the next years, the project "Room for the River" was started in 2000, foreseeing a design discharge of 18,000 m^3/s for the Rhine branches and 4,600 m^3/s for the River Meuse.

In 1996 the Flood Protection Act ('Wet op de Waterkeringen') was established, which contains the ideas of the Delta Laws, with minor changes. The rules in this law are only valid for the primary water defenses.

3.4.11 Wilnis 2003

In the Flood Protection Act, as mentioned before, nothing is said about the regional water defenses. In 1960 a regional levee in Tuindorp-Oostzaan failed. 5 years later the Technical Advisory Committee (TAW) was established and started a research on the strength of the regional levees. This research took more than 25 years and was finished in 1993. Of only 323 km out of 14,000 km of regional water defenses an opinion was given. The advice of TAW was to set safety standards for the regional water defenses. As a reply the minister of Transport, Public Works and Water Management sent a letter to the Provinces. In this letter was stated that the Provinces were responsible for the determination of safety standards for the regional water defenses. This led to a Provincial assembly where the IPO-standards for the regional water defenses were written in 1993. These standards still have no legal effect. (RIVM, 2004)

At the end of August in the extreme dry 2003 summer a polder levee in Wilnis failed and caused the inundation of about 600 houses at approximately 5.9 m below NAP. 2000 people had to be evacuated, but could return the same evening after the water was pumped away. Thanks to an alert local contractor the canal was closed off quite soon and no further damage was caused. A 60 m levee compartment had shifted horizontally (Figure 3.19). Similar levee failures, or threatened failures, occurred in Terbregge and in 2004 in Stein. Several levees which contained peat soils showed bursts and cracks.

Figure 3.19 Dry weather levee failure at Wilnis (RIVM, 2004)

To determine the cause of the dry weather levee failure at Wilnis, a research was carried out by GeoDelft. In January 2004 this research was finished and concluded that a failure mechanism that could normally be neglected was now dominant: horizontal sliding. ‘Normally’ the normative load event was at extreme water levels and/or at an extreme rain event. The failure mechanisms piping, macro instability, overtopping and micro instability were then often dominant. But extremely dry weather also seems to be a normative event, where horizontal sliding can become dominant. What most probably happened is that during the driest summer in 50 years, the peaty soils of the levee lost most of its water content. This led to deformations of the peat and to a decrease in weight. The horizontal friction stresses therefore reduced until the water pressures in the aquifer and the water from the canal lifted the levee and shifted it backwards. But not everyone agrees on this theory and the possibility that a leaking pipe line caused the levee failure is sometimes mentioned.

The results of the Wilnis research led to a renewed interest in the strength of the regional water defenses, and especially those levees that contained peat soils. STOWA carried out a research on peat soils which was finished in 2005. In the meanwhile attention is given to the safety standards for regional water defenses. STOWA has written the “Leidraad toetsing veiligheid regionale keringen”, which in its definite version could become a standard for regional water defenses. (STOWA, 2006)

4 Dutch levee evaluation

This chapter gives an overview of levee evaluation as prescribed by the Dutch government and applied by Fugro Ingenieursbureau BV in the Netherlands. It starts with background information on the Dutch legislation and a short introduction to the supporting case studies used in this chapter. Paragraph 4.2 and 4.3 go into the evaluation process concerning which failure mechanisms are modeled and how to get a ground model and boundary conditions to start the evaluation with. The three case levee evaluations support this description and will also be used in the last two paragraphs (4.4 and 0) where the modeling of piping and stability, often the two main failure mechanisms, are further explained.

4.1 Background

4.1.1 Legislation

The Dutch primary water defenses are evaluated every 5 years according to the 'Voorschrift Toetsen op Veiligheid', prescribed by the Dutch Ministry of V&W (Min.V&W, 2004). As mentioned before in paragraph 3.2.2, there is no prescribed evaluation standard (yet) for the regional water defenses. To evaluate the performance of the regional water defenses, a technical report on the evaluation of 'boezemkaden', the TRB, (TAW, 1993) is used, combined with the IPO safety norms for 'boezemkaden' from 1999 (IPO, 1999). A new manual to evaluate the safety of regional water defenses is developed at this moment. New findings are implemented in this manual, which for example resulted from the levee failure in Wilnis. (3.4.11) (STOWA, 2006)

4.1.2 Cases

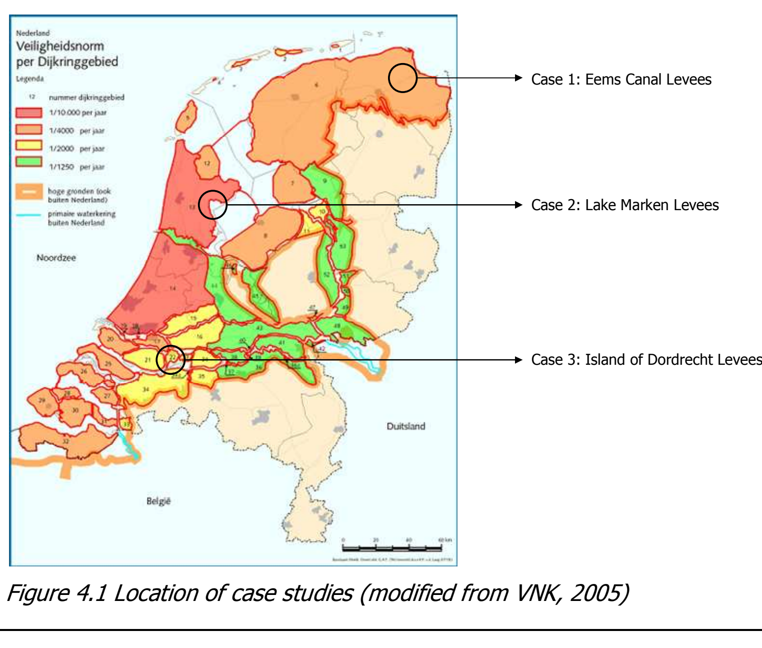


Figure 4.1 Location of case studies (modified from VNK, 2005)

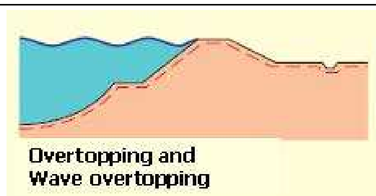
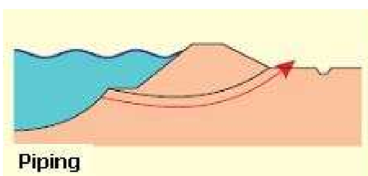
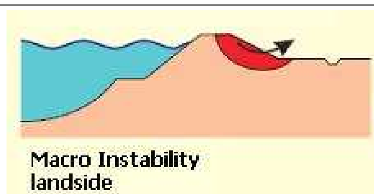
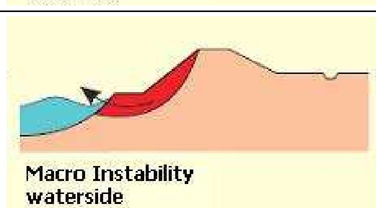
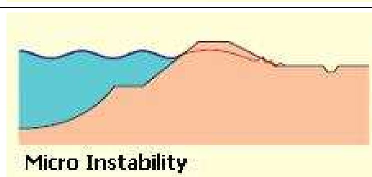
Three case studies (see Figure 4.1) are chosen to support the description of Dutch levee evaluation, based on their location and character. The Eems Canal levees, case 1, are regional water defenses and evaluated according to the TRB. The Lake Marken levees and Island of Dordrecht levees are part of the primary water defenses, surrounding the colored areas in Figure 4.1. The difference between the two is that the Island of Dordrecht levees are river levees influenced by the tide and river runoff fluctuations and that the Lake Marken levees are situated along Lake Marken, not influenced by the tide. Wind setup and waves are the most important hydraulic loads for the Lake Marken levees. All three case studies are based on evaluation reports from Fugro Ingenieursbureau in the Netherlands. (Fugro, 1998; Fugro, 2004; Fugro, 2006)

4.2 Levee evaluation process

4.2.1 Failure mechanisms

To make sure that a levee will not fail during extreme conditions, every levee failure mechanism that could form a threat to the levee is investigated. All relevant failure mechanisms were already mentioned in Figure 3.6, but not all these mechanisms are always evaluated. Table 4.1 summarizes the most important mechanisms with a short explanation of the failure mechanism, how evaluated and for which levees they are important. Failure mechanisms that need further explanation are repeated after the table. Stability and piping are the most common failure mechanisms and are discussed in separate paragraphs, 4.4 and 0.

Table 4.1 Failure mechanisms important in the Dutch evaluation (modified from Min.V&W, 2004)

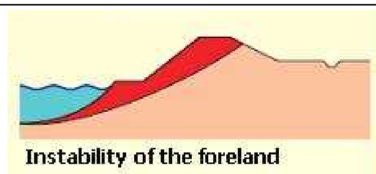
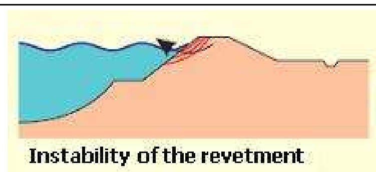
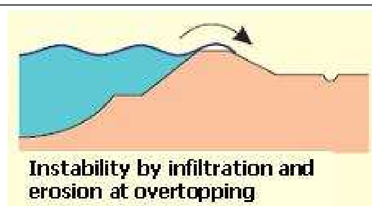
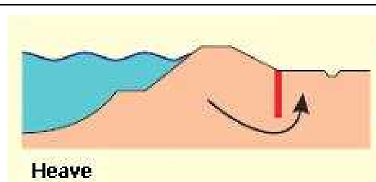
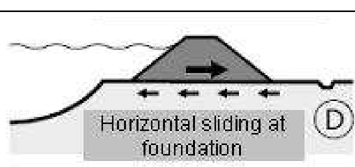
Failure Mechanism	Why / How / When evaluated?
	• The levee height should be sufficient to prevent overtopping and wave overtopping • A minimal crest height margin above the still water level should be present • All levees should be evaluated on their crest height
	• Undermining of the levee by piping should be prevented, because it could cause levee settlements (and therefore overtopping) or even levee instability • The available seepage length or path should be longer than the critical seepage length • All levees with a potential piping vulnerable soil profile should be evaluated
	• Instability of the levee body could threaten the levees water retaining function • The levee should fulfill the established stability requirement at normative conditions, especially wet conditions • This mechanism is important for all levees
	• Instability of the levee body could threaten the levees water retaining function • The levee should fulfill the established stability requirement at normative conditions, especially rapid drawdown • This mechanism is important for levees which deal with extreme water level fluctuations
	• Levee through seepage can damage the landside slope of the levee • The stability of the sand particles at the landside slope is evaluated • Important for levees that contain sand

The levee **height** should be sufficient to prevent overtopping and wave overtopping. An opinion should be given on the crest height margin (or minimum freeboard) above still water level. Still water level is the Normative High Water level, NHW, raised with wave run-up, wind set-up, seiches and shower-oscillations. The crest height margin should be at least 0.5 m and accounts for settlements and subsidence. The overtopping discharge should be less than the levee revetment can withstand and should not limit access to the levee.

Micro stability concerns the damage on the landside slope caused by phreatic water. A simple assessment on the stability of the grains on the landside slope and the ability of the toe to drain is often sufficient to determine the vulnerability of the slope. A detailed investigation on the safety against rupturing of the clay blanket on the landside slope is sometimes necessary.

Except for the major failure mechanisms now treated there are mechanisms that also require attention, often in more specific situations. They are summarized in Table 4.2.

Table 4.2 Other failure mechanisms that need attention (modified from Min.V&W, 2004)

Failure mechanism	Why / How / When evaluated?
	• Instability of the foreland imposes a threat to the stability of the whole levee body • The liquefaction potential and stability of the foreland are determined • Only evaluated when levees have a foreland
	• Especially waves can damage the waterside slope and eventually lead to failure • An assessment of the stability of the revetment is made • For levees vulnerable to wave erosion, which have a slope protection
	• Erosion by overtopping water can damage the inland levee slope and eventually lead to failure • The erosion susceptibility of the landside slope is evaluated • All levees, often in combination with the evaluation of overtopping or stability of the revetment
	• Vertical flows behind a wall can induce liquefiable sand to erode and undermine the wall and eventually the whole levee • Evaluation of the exit gradient or an estimation if the available seepage length is sufficient with Lane's formula • At situations with vertical sheet piles in the levee base
	• The whole levee can shift horizontally and lose its water retaining function • In critical situations the weight of the levee should be sufficient to prevent uplift of the levee body and horizontal sliding induced by the water pressure • When peat forms a substantial part of the levee; in combination with landside stability evaluation

When evaluating the stability of the **foreland** two mechanisms are involved: the liquefaction potential of the foreland and the resistance against horizontal sliding of the foreland. This report will not further deal with stability of the foreland.

Revetments are stone slope protection or other protections as clay and grass. Especially on large levees, along the coast or rivers, protection is necessary to reduce wave run-up and prevent erosion of the waterside slope. To assess whether the revetment provides sufficient protection is often dealt with separately from all the other failure mechanisms. Empirical formulas based on model tests with stones slope protections or grasses are used. **Infiltration and erosion** of the landside slope are integrated in the evaluation of the revetment or levee height. (Min.V&W, 2004)

The **Heave** mechanism is important for situations with a vertical seepage wall in the levee base. An estimation of the exit gradient with a ground water flow model or, more conservative the critical seepage length with Lane can both be used. Generally, the critical exit gradient for heave is 0.5 and the critical seepage length of Lane should be shorter than the available seepage length. (TAW, 1999_2)

Horizontal sliding at foundation is a mechanism which was at the background until a regional levee in Wilnis failed in 2003 (3.4.11). Researchers of that levee failure have different opinions on what happened there, but on one thing they agree: the weight of the levee was exceeded by the water pressures at the waterside of the levee and at the base of the levee. Peat levees are vulnerable to this mechanism, because of the low volume weight of, especially dry peat. An assessment of the vulnerability of a regional water defense for this mechanism should be included in the stability evaluation, as is suggested in levee evaluation documents for regional water defenses. (Van Baars, 2004, Geodelft, 2004, STOWA, 2006)

4.2.2 From basic investigation to advanced modeling

One single evaluation method based on limited information will not lead to rejection of a levee. Only if after thorough investigation is concluded that (part of) the levee is vulnerable to a certain failure mechanism the levee section will be rejected and improvements are necessary.

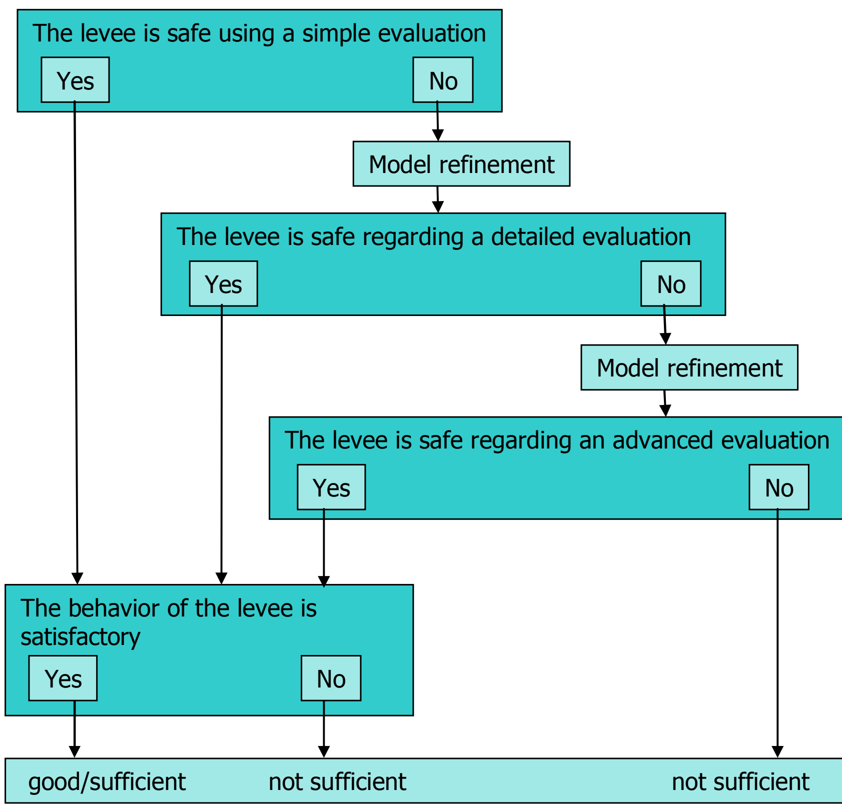


Figure 4.2 Evaluation chart (modified from Min.V&W, 2004)

As schematized in Figure 4.2 a levee is evaluated in steps. Each following step in the process means a refinement of the model and/or a more detailed or advanced method to model the failure mechanism. The following steps can often be distinguished:

1. Pre-investigation: considers the gathering of old information, tests and boundary conditions as well as interpretation and verification of old evaluations with new boundary conditions.

2. Global analysis or simple evaluation: Is performed from a global bottom profile, geometry and soil characteristics. The levee profile is compared to the safe profile. The surveyor of the levee or an expert performs this analysis. A global analysis is often only done if nothing is yet known about the levee.
3. Detailed analysis: Is performed by experts, according to technical reports and manuals from the TAW. Field measurements and laboratory tests support this analysis. The levee is divided in sections with comparable characteristics and cross-sections.
4. Advanced analysis: If still no final verdict can be given on a levee section, often more field measurements, laboratory tests and/ or more advanced calculation methods are used to assess the levee safety. These analyses are always performed by an expert.

These steps are taken for all failure mechanisms which for that particular levee or levee section are important. An opinion as 'good', 'sufficient' or 'not sufficient' is given for each section and each mechanism or sometimes 'no opinion' when not enough data is available to draw conclusions. Results of levee inspections have to be added to the whole analysis and could influence the final conclusion. In the next paragraph is described which failure mechanisms and into what detail they were assessed in the case studies. Paragraph 4.3 will discuss what boundary conditions are used in an evaluation and how a ground model is set up for different sections of a levee.

4.2.3 As applied in cases

Without going into detail, Table 4.3 summarizes which failure mechanisms were evaluated in the case studies, introduced in 4.1.2, and what steps were taken. The Eems Canal levees are regional levees. Not only were they evaluated on piping and macro stability, but also on the effect of trees in the embankment. A falling tree could leave a gap that endangers the levee its water retaining function. Most, often not really 'designed', regional levees have trees on the landside slope. (Fugro, 2004)

Table 4.3 Failure mechanisms assessed and steps taken in case studies

	Case 1: Eems Canal	**Case 2: Lake Marken**	**Case 3: Island of Dordrecht**
Global evaluation	Macro stability (land- and waterside) Piping Non-water retaining objects (trees)	Macro stability (land- and waterside) Piping Micro stability	Macro stability (land- and waterside) Piping Micro stability Stability of the foreland Connection between levee and 'hard structure'
Detailed evaluation	Macro stability Piping	Macro stability (land- and waterside) Piping Micro stability	Macro stability (land- and waterside) Piping Micro stability Stability of the foreland Connection between levee and 'hard structure'
Advanced evaluation	Piping	Macro stability (land- and waterside) Piping Micro stability	
Separately evaluated		Geometry Levee height (by levee owner)	Stability of the revetment Non-water retaining objects

In the Lake Marken levee evaluation the macro stability, piping and micro stability were assessed. The first step, phase 1, was a re-evaluation and verification of an earlier study to new boundary conditions. In earlier investigations the geometry was also checked. The levee owner himself had evaluated the crown height. Phase 2a was a re-calculation on disapproved levee reaches, based on more extensive field measurements and laboratory tests. Phase 2b concerned some advanced research to sharpen some of the results. There was also a phase 3 in this investigation, where designs were made for some principal solutions on rejected levee reaches. (Fugro, 2006)

Two phases were distinguished in the Island of Dordrecht evaluation, which was done according to the VTV (Min.V&W, 2004). In phase 1a levee evaluation from global to detailed level was performed on the following mechanisms/aspects: piping, macro stability landside and waterside, micro stability, stability of the foreland, revetments, non water-retaining objects and the connection between soil structure and special water retaining structure. The latter two were investigated and reported separately, partly by another company. Phase 2 is a further investigation on levee parts of which in phase 1 no opinion could be given. That does not necessarily mean that advanced methods are directly applied. (Fugro, 2004_2)

4.3 Loads and Ground model

4.3.1 Introduction

All that evaluation is about is to compare the loads on the levee with the resisting forces of the levee and its foundation against the failure mechanisms. The difficulty in levee evaluation lies often not in how to evaluate a levee, but more on which parameters for the strength and loads are used in the evaluation.

4.3.2 Loads

Table 4.4 Loads included in the Dutch levee evaluation (modified from TAW, 2000)

Loads included in evaluation:	**Loads not included:**
Permanent loads: • Dead weight of the levee and foundation • Extractions from subsoil (i.e. water, salt, gas) resulting in settlements • Non-water retaining objects, such as trees and pipelines Hydraulic loads: • Normative water levels • Water level changes (i.e. rapid drawdown) • Precipitation • Wind waves (significant wave heights, peak periods) • Wind set-up Other loads: • Traffic	Hydraulic loads: • Ship waves Other loads: • Ice load • Collision • Earthquakes • Explosions • i.e. damage from vermin • Vandalism/terrorism

Table 4.4 is an enumeration of the loads that are accounted for when a levee is evaluated. When levees fail it is often during extreme hydraulic conditions: extreme high water levels, rapid drawdown after extreme high water or when a levee is soaked by extensive rainfall. Wind waves and wind set-up in combination with extreme high water levels even make it worse. Therefore normative water levels and waves are established, which only have an acceptable low probability of exceedance (see 3.2.2). The hydraulic boundary conditions for the primary water defenses are prescribed by the government, based on historic water levels and flow models. These boundary

conditions are updated every five years and put together in one hydraulic boundaries book (Min V&W, 2001).

The normative hydraulic conditions for the regional water defenses and are often determined from information from the water boards. Because these levees are not subject to large fluctuations, as the river and sea defenses, the normative water level is easier to determine. Except extreme wet conditions, extremely dry conditions are also important for these levees, while peat levees, which were not designed but simply just resulted from peat excavations, are very vulnerable to dry conditions. Dry peat has a low volume weight and therefore a decreased resistance.

Other loads are for example those caused by traffic. The weight of cars and trucks but also vibrations caused by moving traffic imposes a pressure on the levee that cannot be ignored and are always taken into account in a stability evaluation of a levee with a road on top. There are also other important loads that certainly could result in levee failure, but that are not taken into account in the evaluation. Of part of those loads it is not easy to quantify them, for example damage by vermin or vandalism. Others have such a low probability of occurrence that they are not taken into account, as in the Netherlands is the case for earthquake loads and damage from ice sheets. (TAW, 2001)

4.3.3 Ground model

The preparation of a ground model is an important step in the levee evaluation process, which should precede and again follow the levee evaluation itself. Without a proper ground model, with legal assumptions on the important layer thickness and soil parameters, a detailed levee evaluation is not possible. Which ground model parts are important is determined by the failure mechanisms that are evaluated.

Table 4.5 Global method to prepare a ground model (modified from Fugro, 1998)

Phase	Gathering information	Information sources	Range of investigation (indication)	Intention	Parameters
Pre-inves-tigation and Global levee evaluation	Archive	Maps, old field investigations	Whole levee length profile	Global ground model	-
	Geological advance knowledge; experience; area knowledge	Expert	Whole levee length profile		
Detailed levee evaluation	Basic field investigation	Geophysical Soundings Borings Gauges Classification	Range Per 50 to 150 m Per 50 m Per 50 to 100 m 8 per boring	Basic ground model	Based on classificatio n
Advanced levee evaluation	Detailed / advanced field investigation	Extra field measurements: -borings -gauges (continuous) Lab-tests: - triaxial tests - compression tests - sieving	Situation dependent 3 per boring/4 per layer 2 per boring/3 per layer	Adjustme nt of ground model on specific locations	Based on field- and lab-tests

An approach from simple to advanced is used, for which the levee is divided into sections. The schematized cross sections should be representative for the whole section. First a simple levee evaluation can be performed to see if there are levee sections from which it is quite clear that they are safe without calculations or field measurements. For the sections that cannot be assessed that easy field investigation is done and calculations are used to determine the safety. Of parts where still no positive verdict can be stated more detailed and even advanced methods can be used to determine the soil properties and model the failure mechanisms. Often only one or two failure mechanisms have to be studied in more detail. Eventually there will often be a part that is still rejected. Levee improvement is then advisable.

The assessment of the levee safety is based on the model made of the levee and its base. These soil layers, in cooperation with the dimensions of the levee and the geohydrological boundary conditions, determine the resistance of the levee against the loads. But they also form a load themselves, for example in sliding planes. To know the levee and its base in detail extensive field measurements and laboratory tests are needed, which is not always necessary to give an estimation of the safety of a levee. The trick is to get a maximum reliable verdict on the levee safety with the available measurements and tests. Depending on the normative failure mechanisms, more measurements and tests can be done on critical levee parts. For example a levee part that is first rejected, because it is piping sensitive, can again be assessed with a more detailed thickness of the waterproof layer and a better indication of the permeability of the pervious layer. Thus more (undisturbed) borings at the toe of the levee can be taken, to find better estimates of these parameters.

Table 4.5 gives a global method to develop a ground model and an indication of the measurements and tests that are done in the Netherlands. Steps in the definition of the ground model are in practice often not as strict as in the table. When already some weak spots are for example known, extra field measurements and tests will already be taken in the basic investigation.

4.3.4 As applied in cases

In the table on next page is summarized on which information the ground model in the case studies from 4.1.2 was based. Hydraulic boundary conditions are also mentioned.

To develop a ground model for the Eems Canal levees the 51 km were divided in parts of 250 m. Borings and soundings were used for a first ground model and the normative high water in the canal, as provided by the water board was the most important load.

The study area of Lake Marken was divided in 36 levee reaches of 4 different levees, mostly based on geometry. The boundary conditions used are the hydraulic boundary conditions from 2001 (HRV, 2001). 3 of the 4 levees use the same soil characteristics; the other levee has others, based on historic existence. A traffic load is applied in the stability research for the levees, where the load has a negative influence on the stability.

The 37.1 km of levee of the Island of Dordrecht was divided in 22 sections with a representative cross section. This division was based on previous improvement works. An example of this are the improvements works carried out for the Delta Plan after the 1953 flood. The hydraulic boundary conditions of 2001 were used in this research and on parts of the levee a traffic road makes it necessary to use a traffic load in the stability evaluation.

Table 4.6 Development of ground model in cases

Case 1 Eems Canal	Case 2 Lake Marken	Case 3 Island of Dordrecht
Archive study		
Digital maps Eemskanaal with each 250 m a cross section; Archive soundings and laboratory tests from 1991: Consolidated triaxial tests on clay and peat monsters Polder water levels as defined by the water boards;	Digital cross sections from the water board Levee evaluation from 1988 Old piezometer data Soil characteristics form 1991	Maps with cross sections and aerial photographs from the water board Levee evaluation from the '70s and '80s, followed by levee improvements; Dutch cell tests on monsters from km 0-14.8 and km 32.0-37.1; Hydraulic boundary conditions from 2001, normative water levels, waves, storm conditions and polder water levels from water board
Basic investigation		
Deep soundings with friction, each 250 m (185 deep soundings until +/- -14 m NAP and 5 mini soundings +/- -4 m NAP) from the crest 17 borings until approx. -6 m NAP Piezometer data from season 2001/2002 (phreatic ground water level and Pleistocene rise Soil parameters based on experience and from the 1991 lab tests	Hydraulic boundary conditions from 2001 More cross-sections Borings? Lab tests: cel tests and triaxial tests	Piezometer data from season 2002/2003 Triaxial tests on samples from km 14.8-32.0
Detailed Investigation		
15 borings at landside toe (until approx. -6 m NAP) and 13 behind the bank protection (from +1.5 until -0.5 m NAP) Consolidated undrained triaxial tests on samples from toe borings from clay and peat layers Visual inspection bank protection	TNO archive- data 31 toe borings, slope borings (landside) and crown borings 58 Soundings 29 piezometers, measurements compared with old piezometer data Determination of grain size distributions of the deep Pleistocene sand layer Dissipation tests Volume weight determination Triaxial tests Atterberg tests	Extra soil research on whole area: Triaxial tests on undisturbed samples Determination volume weight 13-hour measurements piezometers to determine response of phreatic level to water level changes
Advanced Investigation		
10 deep soundings (until -45 m NAP) with local friction measurements from the crest 10 hand borings at same location as soundings (until -20 m NAP) Constant head tests on sand samples; determination of grain-size distribution 9 piezometers at locations of borings	Tests on undisturbed monsters: 15 direct simple shear tests on peat 20 isotropic, single stage triaxial tests on clay to determine the shear strength of clay	-

4.4 Macro stability evaluation

4.4.1 Why evaluation of the levee stability?

A slip surface forms when the weight of a portion of the embankment causes a driving moment that exceeds the resisting shear stresses within the levee. A large slip surface decreases the width of the levee and could immediately cause the levee to fail entirely, because it is not able to resist the water pressures anymore. Flooding of the land behind the levee is an inevitable result. Especially levees with a steep slope in wet conditions are vulnerable to this mechanism. Every slope therefore has to be evaluated for conditions that could be critical.

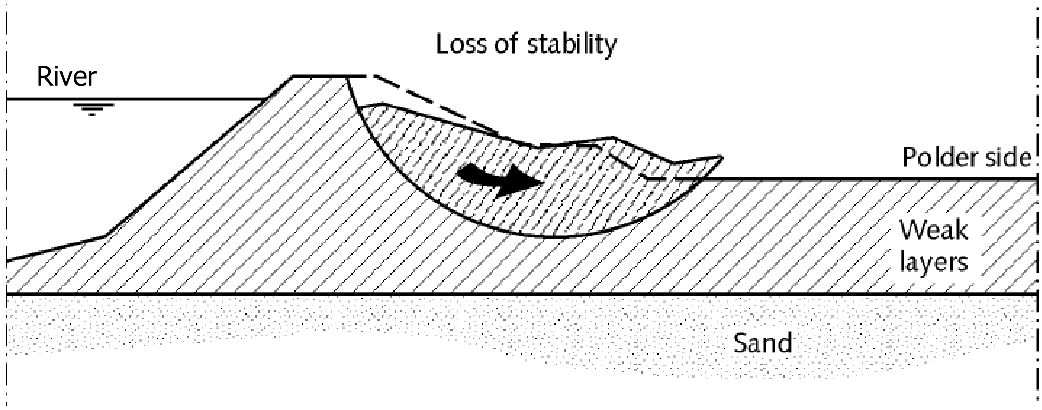


Figure 4.3 Loss of stability in a circular plane (TAW, 2000)

4.4.2 How to model the levee?

To evaluate whether the levee is stable enough under normative conditions two methods can be applied: a finite elements method or slip surface calculations. Plaxis is a software package which provides a range of finite element methods to perform 2D as well as 3D geotechnical analyses. To get a realistic result with such finite elements methods, extensive knowledge of the soil characteristics is necessary. While in levee evaluation often only limited knowledge is available, a simpler method as slip surface calculations often provides faster and equally reliable results and is normally used in practice. The software that is used in the Netherlands to perform slip surface calculations is MStab. Because there are many ways, forms and locations where a slip surface could occur, several 2D slip surfaces are assumed and for each of them the factor of safety is computed. For each of those slip surfaces the method of slices is applied, often assuming a circular slip surface (Figure 4.4). The soil above the slip surface is then divided into vertical slices. A summation of the effective stresses and water pressures acting on each individual slice results in a safety factor. This safety factor is an expression of the driving forces or moments divided by the resisting forces or moments.

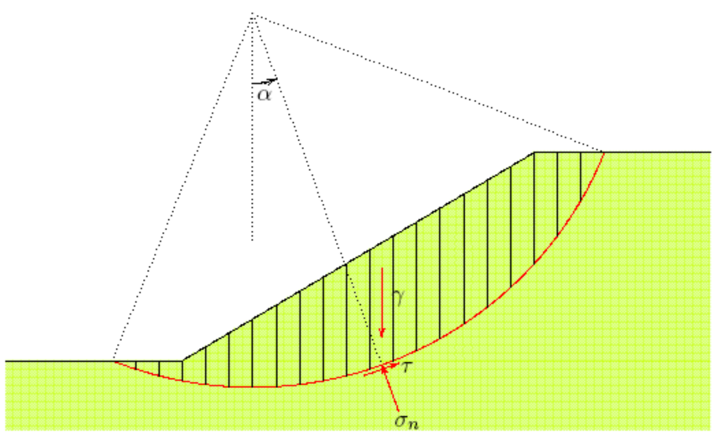


Figure 4.4 Method of slices with a circular slip surface (Verruijt, 2001)

But not only circular slip surfaces through the levee body itself can threat the levee stability. Especially in the Dutch situation, where behind the levees a thick aquifer is blanketed with a relatively thin, impermeable layer, the water pressures in the aquifer are sometimes able to lift the weak blanket layers. This phenomenon is called uplift and can induce a far larger slide plane, because during uplift the resisting forces between the aquifer and blanket layer suddenly disappear. This mechanism can be modeled in MStab with the Uplift Van module, which then divides the slide plane in a pressure bar bounded by two circular planes, as in Figure 4.5.

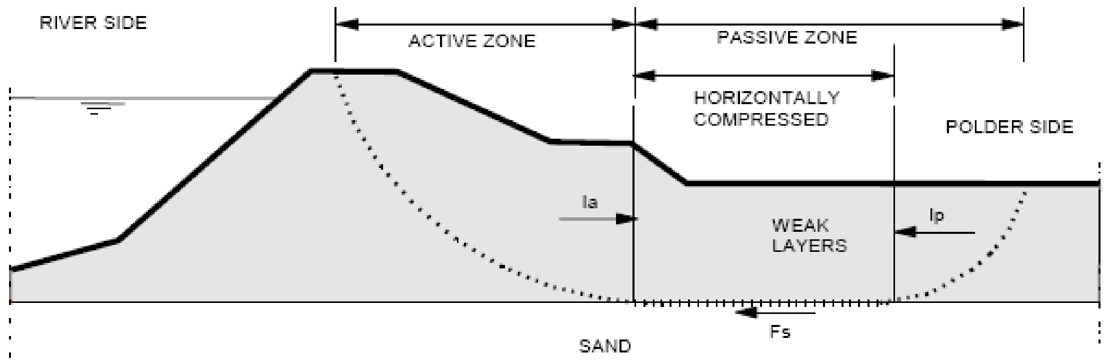


Figure 4.5 Uplift mechanism (Geodelft, 2006)

In Table 4.7 some characteristics of the methods that are usually applied in MStab are summarized. The slip surfaces with Fellenius and Bishop are for example always circular, while with Spencer also other shapes can be defined. Uplift Van has a pressure bar with two circle planes as in Figure 4.5, while Uplift Spencer only has a circle plane through the embankment. With Fellenius and Bishop MStab automatically computes all possible slip planes, within the user defined grid, and returns the slip surface with the lowest safety factor. For Spencer the safety factor is determined, for the user defined slip surface, while in the Uplift modules again different the safety factor of different slip surfaces is determined with a single tangent line for the circles and a moving grid. Fellenius is only based on moment balance, while the other methods also include vertical and/or horizontal force equilibrium.

Table 4.7 Summary of the most important methods in MStab (Geodelft, 2006)

Module	**Shape slip surface**	**Definition of slip surface**	**Stability definition**
Fellenius	Circular	Automatically	Moment balance
Bishop	Circular	Automatically	Moment balance and vertical equilibrium
Spencer	Arbitrary	User defined	Moment balance, vertical and horizontal equilibrium
Uplift Van	Horizontal plane with two circles	Partly automatic	Moment balance, vertical and horizontal equilibrium
Uplift Spencer	Horizontal plane with one circle	Partly automatic	Moment balance, vertical and horizontal equilibrium

4.4.3 Stability evaluation

Simple evaluation of the levee stability is often just a check on the levee geometry and the above methods are not yet applied. How to perform a more detailed stability evaluation is schematized in Figure 4.6. Methods for detailed or advanced evaluation do often not differ, but the ground model is refined for every step. Stability of the landside slope as well as the riverside slope is evaluated. For the landside slope the situation with normative high water and/or extreme precipitation are examined are often the most critical situations. Extreme precipitation causes saturation of the soils, which results in a less stable situation, especially for levees which contain clay. If there is a chance of uplift of the blanket landside of the levee a pressure bar calculation has to be made for the situation with an uplifted blanket. Where cracking of the top layer is possible the strength of the blanket will be assumed equal to zero in the stability calculation.

Stability of the waterside slope is examined for the situation rapid drawdown, when the water level is decreasing so fast that water pressures within the levee are not able to follow the drawdown. Again a situation with extreme precipitation could also be normative and has to be investigated.

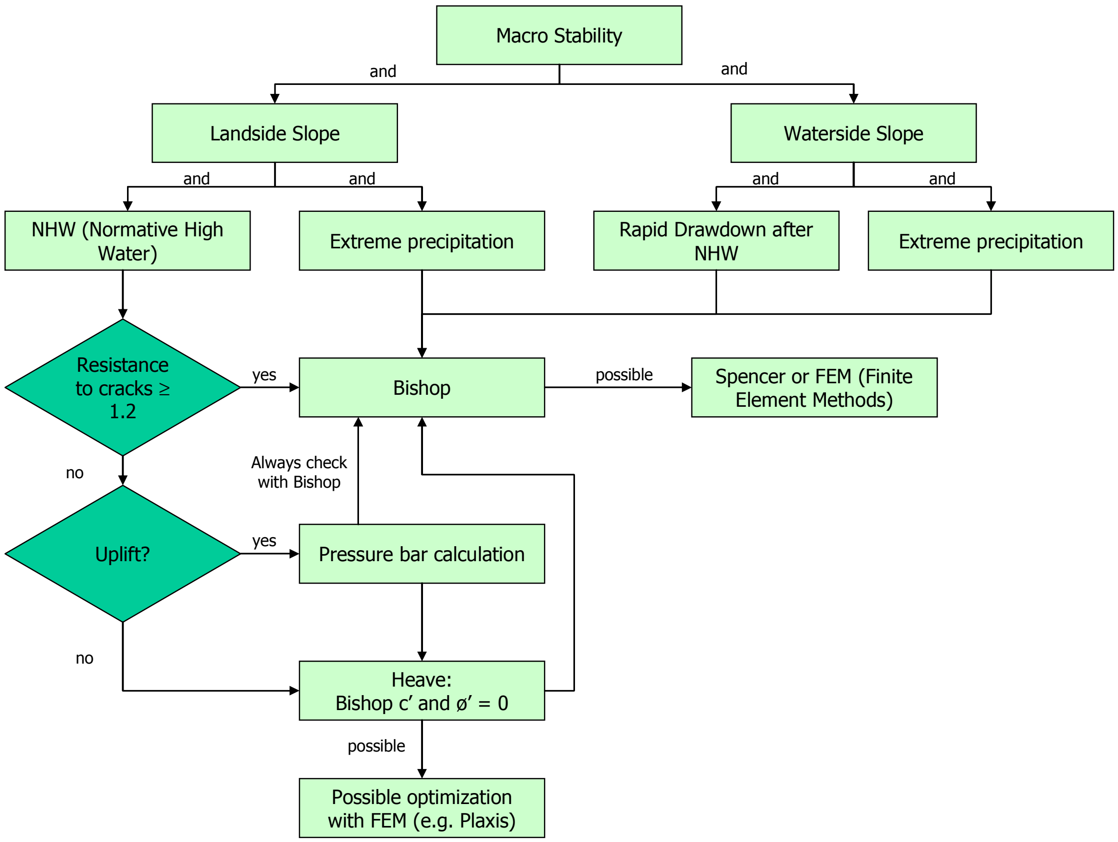


Figure 4.6 Macro stability chart Dutch levee evaluation (modified from TAW, 2001)

Necessary input parameters to model stability in MStab are the levee geometry, phreatic line(s) and volume weight and strength characteristics of the soils. The phreatic line in the levee for normative conditions is often determined from available piezometer data, extended to situations with normative high water and/or extreme precipitation. If no piezometer data is available conservative assumptions are made. For each aquifer it is possible to define a phreatic line. The wet and/or dry volume weight of the soils and the shear strength are determined from lab tests as described in previous paragraph. The strength characteristics can be defined as a 'sigma-tau-curve' where the normal effective stress (sigma) and the shear stress (tau) are related, but a combination of the cohesion and internal friction is also possible amongst others. Which safety factors are allowed is variable and depends on how the soil characteristics are determined and which methods were used to measure the strength parameters. Nowadays a probabilistic approach to determine the parameters is preferred. This means that not the mean value of for example the shear strength is taken, but a more conservative value: 95% of the measured data has to be lower/higher than this value, depending on which is conservative. This is the characteristic value. The values inserted in the computation are the characteristic values corrected with a material factor to compensate for insecurities.

In the VTV (Min.V&W, 2004) allowed safety factors ranging from 1.2 to 1.6 are mentioned. More information about safety factors and material factors can be found in the Technical Report on Soil Structures. (TAW, 2001)

4.4.4 Case studies

In the case studies is demonstrated how a stability evaluation is then applied in reality. In all cases the stability was modeled with MStab and Bishop. Further on the stability of the Eems Canal levees is only evaluated for the landside slope. No rapid drawdown is expected here, because water levels do not fluctuate much and is therefore not evaluated. The situation with NHW is assumed normative. The phreatic line for this situation was based on available piezometer data extrapolated to the normative water level. For the cohesion and friction mean values were used and a minimum safety factor of 1.0 was accepted.

In the stability evaluation of the Lake Marken levees all situations mentioned in Figure 4.6 are evaluated. The phreatic line is again determined from piezometer data and raised with 0.5 m for extreme precipitation. A minimum safety factor of 1.0 was accepted with characteristic values for the soil strength. The Island of Dordrecht levees were also evaluated for the landside as well as the waterside stability at NHW and for rapid drawdown conditions as well as extreme precipitation. Undrained characteristic values were used. Material factors varied from 1.0 to 1.3 depending on the test methods (triaxial tests or cel tests).

4.5 Evaluation of the piping mechanism

4.5.1 Why piping evaluation?

One of the first conclusions of the FLORIS (or VNK research, see 3.3.2) was that the estimated probability of failure of the Dutch levees due to piping is larger than the probability of failure due to overtopping. Part of this probability correlates with the uncertainties and variability of the characteristics of the levee foundation soils, but from the first part of the FLORIS research it is clear that piping forms a realistic threat for the stability of the Dutch levees. (FLORIS, 2005)

The 1993 and 1995 river floods revealed the vulnerability of the Dutch river levees to piping. No levees actually failed due to piping, but at several locations sand boils were observed, a clear indication of pipe forming. Sacking was necessary to prevent calamities.

4.5.2 Dutch explanation of the piping mechanism

A typical piping sensitive situation is where a pervious, sandy, layer is overlain by an impervious or semi-pervious layer, for example clay. This situation is often found along rivers. Figure 4.7 displays the steps in the development of a through pipe. Each step is explained below.

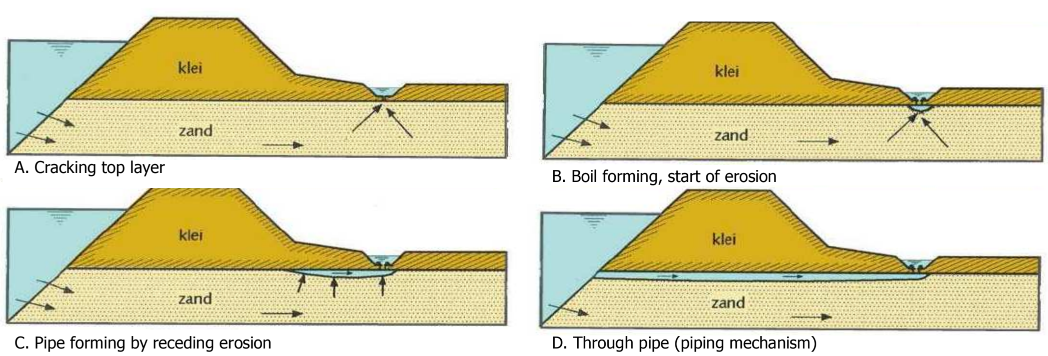


Figure 4.7 Steps in piping process (TAW, 1999_2)

A. Cracking of the top layer; High water levels can cause subsurface pressures in deep sand layers that are able to lift relatively impervious blanket layers landside of the levee. When the pressure exceeds the weight of the covering layer up-lift is the result. In weak areas, for example in a ditch, cracks can occur in the blanket, releasing the water pressures and causing under-seepage.

B. Boil forming, start of erosion; if the heavily concentrated seepage flow through the crack is strong enough it can induce the movement of sand particles from the pervious layer. The water and sand will form a sand boil and deposit the sand into a cone surrounding the boil.

C. Pipe forming by receding erosion; ongoing erosion from the sand layer will start a pipe, which can eventually become a through pipe at the interface of the pervious layer and the non-pervious layer.

D. Through pipe; a through pipe undermines the levee and causes the levee to settle or even fail. (TAW, 1999_2)

4.5.3 Piping evaluation

The evaluation of piping is, based on the description of the piping mechanism, divided in two separate processes. Uplift with possible rupture of the blanket is the first step and erosion of the sand, revealing as sand boils is the second. These mechanisms therefore return in the piping evaluation chart of Figure 4.8.

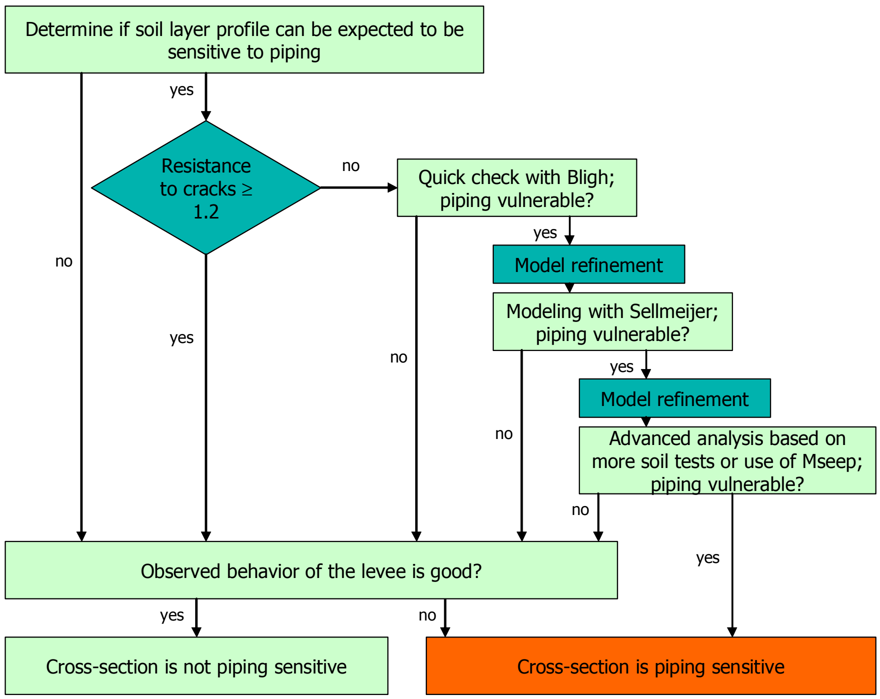


Figure 4.8 Piping evaluation chart

As was already explained in 4.2.2 the ground model that is used for, in this case, the piping evaluation is refined in every step of the process. First very conservative values are used from basic soil research. Further on in the process more detailed exploration results in better knowledge of the soil conditions and more detailed ground models.

But within the piping evaluation the modeling method is also refined. Bligh, an empirical relation published in 1910, is a fast and easy method to give a first indication of the piping vulnerability of a levee. More detailed modeling is done with the Sellmeijer formula. This formula was developed in the Netherlands in the 1980's and is the solution of differential equations for movement of a particle in a slit, supported by model tests (Figure 4.10). The outcome of the Sellmeijer formula as well as the Bligh formula is a critical head on the levee, or, which is often used in practice, a critical seepage length or path from which piping can be expected.

The evaluation chart also mentions an advanced analysis, which often consists of extra measurements and/or laboratory tests to refine the characteristics and dimensions of the sand and blanket layers and modeling with Sellmeijer. Another possibility is using Sellmeijer supported by ground-water flow models, as MSeep, to get a better impression of the permeability of the sand layer. MSeep also has a special piping module. This module is not used in the current evaluation standards.

As a result of the above described process a levee section is never immediately rejected, unless observations clearly indicate problems. Only after several refinement steps, each indicating that the levee is piping sensitive at the normative water level, the levee can be rejected. If from the analysis is determined that the levee is sufficiently safe against piping and there were no piping problems (sand boils) diagnosed in past extreme conditions, the opinion is positive regarding the piping criterion. In the following paragraphs uplift, Bligh and Sellmeijer are further explained. (TAW, 1999_2)

4.5.4 Uplift

To determine whether uplift of the blanket is possible the uplift criterion is used. The critical situation is when the pressure of the water beneath the blanket equals or exceeds the weight of the blanket, which is described with the formula:

$$\left(\phi_s - h_p\right) \leq \frac{1}{\gamma}\left(\phi_{s,c} - h_p\right) \tag{4-1}$$

With:

$$\phi_{s,c} = h_p + D\frac{\gamma_{w,s} - \gamma_w}{\gamma_w} \tag{4-2}$$

D = thickness of the blanket [m]
h_p = the level of the top of the sand layer [m + NAP]
$\gamma_{w,s}$ = the wet volume weight of the top layer [kN/m3]
γ_w = the volume weight of water [kN/m3]
ϕ_s = the hydraulic head in the water-bearing layer [m + NAP]
$\phi_{s,c}$ = phreatic head in the top layer [m + NAP]
γ = safety factor [-] (=1.2)

4.5.5 Bligh

The Bligh formula uses a critical seepage length or path. If the actual piping length is shorter than the critical piping length the levee is assumed vulnerable to piping.

$$L_{Bligh} = \Delta H C_{creep} \tag{4-3}$$

$$L_{Bligh} \leq L \tag{4-4}$$

With:
L_{Bligh}= critical seepage length [m]
L= present seepage length
ΔH= head difference [m]
D= thickness of blanket (at critical point) [m]
C_{creep}= creep factor [-] = 15 for moderately fine sand

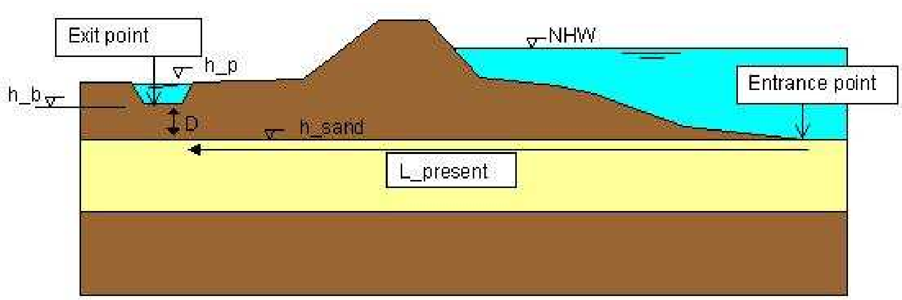


Figure 4.9 Schematized levee profile Bligh (modified from Fugro, 2004)

In the Dutch evaluation this formula was later changed to:

$$L_{Bligh} = \left(\Delta H - 0.3D\right)C_{creep} \tag{4-5}$$

The head difference on the blanket is then partly taken into account. The thickness of the cracked blanket, or in other words: the cracked canal, also offers resistance of the blanket to piping. Theoretically this factor, here 0.3, is somewhere between 0 and 1, but in practice more or less between 0 and 0.6. The 0.3 value was once found in practice. It is not a constant factor, but more sort of an expected value and a reasonable first estimation. (Sellmeijer, 2007)

The creep factor depends on the median grain diamer of the sand. In Table 4.8 the creep factors are related to the grain size.

Table 4.8 Creep factors using Bligh (modified from TAW, 1999_2)

Soil	**Median grain diameter [µm]**	**Ccreep Bligh**
Very fine sand	105-150	18
Moderately fine sand	150-210	15
Coarse sand	300-2000	12
Fine gravel	2000-5600	9
Course gravel	>16,000	4

4.5.6 Sellmeijer

The Sellmeijer formula contains more variables than the Bligh formula and has more theoretical background:

$$\left(\Delta H - 0.3D\right) \leq \frac{1}{\gamma}\Delta H_c \qquad (4\text{-}6)$$

$$\Delta H_c = \alpha c \frac{\gamma_p}{\gamma_w}\tan\left(\theta\right)\left(0.68 - 0.10\ln\left(c\right)\right)L \qquad (4\text{-}7)$$

With:

$$\alpha = \left(\frac{d_{sand}}{L}\right)^{\left(\frac{0.28}{\left(\frac{d_{sand}}{L}\right)^{2.8} - 1}\right)}, \quad c = \eta d_{70}\left(\frac{1}{\kappa L}\right)^{\frac{1}{3}} \qquad (4\text{-}8,\ 4\text{-}9)$$

ΔH = Head difference (=NHW - polder water level) [m]
ΔH_c = Critical head difference
D = Thickness of blanket at crack [m]
γ = Safety factor = 1.2
γ_p = Saturated weight of the sand [kN/m3]
γ_w = Volumic weight of the water [kN/m3]
θ = Friction angle of the sand grains [°] =41°
L = Seepage length horizontal [m]
d_{sand} = Thickness of the sand layer [m]
η = Drag factor (coefficient of White) [-] =0.25
D_{70} = 70 percent value of the grain distribution of the sand [m]
κ = intrinsic permeability of the sand layer [m2]; $\kappa = \frac{v}{g}k$
ν= Kinematic viscosity (≈1.33×10^{-6} m^2/s)
g= gravity (≈9.81 m/s^2)
k_f= permeability [m/s]

The model tests that form the background of the formulas were performed without a blanket layer (Figure 4.10) and were later transformed to a situation with a blanket layer. The 0.3D in the formula is again, as with the Bligh formula, a factor to compensate for the resistance of the crack in the blanket.

The critical head is the point where the slit under the levee starts to grow explosively. As can be seen in the diagram of Figure 4.10 the slit starts to develop at a certain head difference on the levee, but will find equilibrium at a relatively low l/L value, which is the length of the slit divided by the available seepage length. At the critical head there is no equilibrium situation anymore and l starts to grow until it equals L. (TAW, 1999_2)

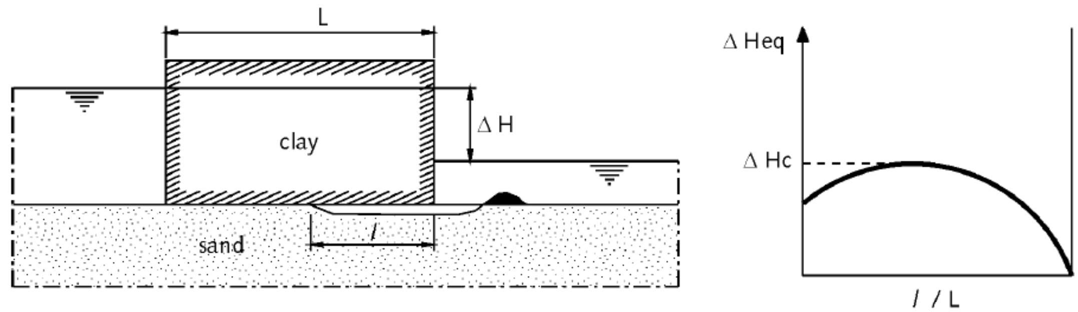


Figure 4.10 The Sellmeijer model tests (TAW, 1999_2)

4.5.7 How to determine the hydraulic head beneath the top stratum?

An important parameter for uplift and therefore piping is the head below the top stratum. There are various ways to determine the excess hydrostatic head in the pervious substratum at possible critical uplift locations. In the Dutch engineering practice a first, very conservative assumption is that the hydrostatic head is equal to the normative water level. A more realistic hydraulic grade line can be drawn from either piezometer data or from a geohydrologic ground water model. Piezometer data is obtained during some months or preferably years to get an impression of the variability of the hydraulic pressures. While it is not very likely that a flood occurs during the measuring period (piezometers are often installed when the levee evaluation is started), the piezometer data is extrapolated to the normative conditions. The result from this extrapolation is often that the water level fluctuations are not 100% followed by the hydrostatic head, but that, with a small time lag, the phreatic head follows for approximately 70%(TAW, 1985). Analytical solutions for groundwater flow or numerical models are less often used. MSeep is a numerical program which simulates stationary ground water flow.

4.5.8 As applied in case studies

In all cases an assessment of the uplift/cracking vulnerability was made. Not only from the deep Pleistocene sand layer, but also, when present, from intermediate sand layers. Further on both Bligh as well as Sellmeijer were used in the evaluation. The head beneath the blanket was estimated from piezometer data, often related to the water levels. For for example the Lake Marken levees first an 80% response was assumed, which was very conservative. This meant that 80% of the difference between the (measured) winter water level and the NHW was added to the head beneath the ditch or at the toe at winter water level conditions. Later was found that the response was 38%-54%. In the advanced piping evaluation of the Eems Canal levees MSeep was used to get a better indication of the permeability values of the sand layer. The sand layer consisted of more or less two separate layers: fine sand on top and course sand below that. With the results from the MSeep calculations most levee sections were approved.

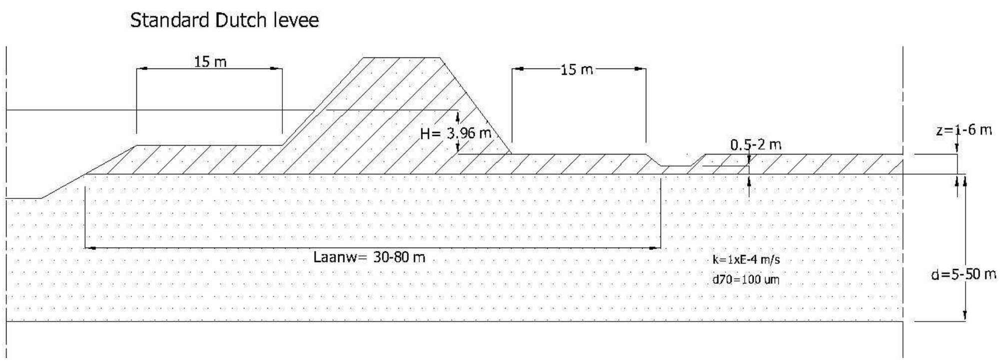


Figure 4.11 Standard Dutch levee from case studies with parameters important for piping

From the three case studies a typical Dutch levee was drawn in Figure 4.11. In all three case studies a situation was found where the Pleistocene sand layer up to 50 m thick is overlain by a clay layer of about 1-6 m. But not only the Pleistocene sand, also intermediate sand layers are sometimes thick enough to cause a seepage flow strong enough to lift the blanket. The permeability of the sand is around $1x10^{-4}$ m/s with a d_{70} diameter of about 100-200 μm. Sometimes a foreland is present of about 15 m and very often ditches can be observed some 0-15 m from the landside levee toe, with a blanket thickness of 0.5-2 m. The total available seepage length ranges from 30-80 m and the head difference on the levee (the difference between the water level on the waterside of the levee and at the landside of the levee) can be about 4 m.

5 US levee evaluation

After Katrina hit New Orleans, there is now an urgent need to evaluate the levees surrounding New Orleans and levees in California. This chapter gives an overview of how levees are evaluated in the US, focused on the currently running Department of Water Resources (DWR) project in California for evaluation of the urban levees. Case studies, as far as available, are supporting the description. After some background information about evaluation documents and an introduction to the cases the process of evaluation is described. Paragraph 5.3 is about loads, followed by development of a ground model in 5.3. Modeling of the levee stability and levee under-seepage are discussed in the last two paragraphs.

5.1 Background

5.1.1 Evaluation guidance

The US has no federal established guidance for levee evaluation. The methods that are used in levee evaluation are partially withdrawn from the Levee Design Manual of the Army Corps of Engineers (USACE, 2000) and often combined in some sort of Standard Operating Procedure. Other documents used are for example the Slope Stability Manual from the Army Corps of Engineers (USACE, 2003) and the Design Guidance for Levee Under-seepage (USACE, 2005). The Slope Stability Manual provides guidance for earth and rock-fill dams, as well as natural slopes and levees. To get levee certification from FEMA these manuals need to be followed.

The design and analysis procedures for levees in the United States are closely related to earth dams. A dam and a levee are distinguished on several aspects. Most levees are only subject to extreme water loading for a few days or weeks a year, while a dam is permanently loaded. Another aspect is that the material within a levee is often far from homogeneous because they were often constructed long time ago on poor foundations and with locally available material, while dams were often already constructed with a real engineering background and more applicable materials (SOP, 2004). Another interesting difference is that a dam, although it is a very large structure, has only a limited length of about a few hundred meters, while levees stretch over a length of many kilometers. For a dam it is therefore for example easier to monitor and evaluate than for a levee.

This chapter uses the Standard Operating Procedure of the Sacramento district (SOP, 2004) to describe the levee evaluation practice in the United States, supported by the methods used in the DWR levee evaluation applied on levees in the Central Valley and Sacramento-San Joaquin Delta. It only discusses the geotechnical levee evaluation; erosion, waves and levee height will for example not (extensively) be dealt with or only qualitatively.

5.1.2 Cases

DWR Levee Geotechnical Evaluations:

In late 2006 a state bond made it possible for the Department of Water Resources (DWR) to evaluate part of the Central Valley and Delta levees. Fugro West inc. is part of the team that was rewarded the assignment to explore en evaluate the 350 miles of urban levees in this part of

California and find hidden deficiencies (see 2.2.1). Urban levees are levees that protect more than 10,000 people. In project team workshops analysis protocols for the geotechnical analysis were developed based on the USACE manuals (URS, 2007)). The stakeholders in this project: DWR, USACE, FEMA and local agencies, review the analyses.

Reclamation District 17 (or RD 17) is the first area that is explored and evaluated by Fugro and is located along the San Joaquin River in the southern part of the Delta. This area is subject to tidal influences and has 16 miles (25 km) of levees that need to be analyzed. (Figure 5.1)

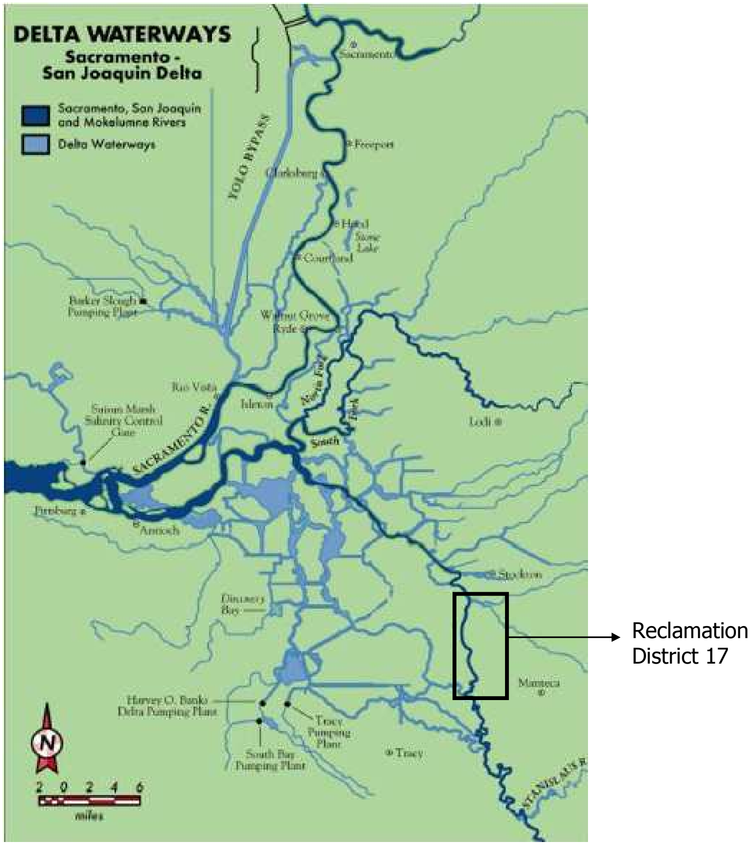


Figure 5.1 Location of Reclamation District 17 (DWR, 2006)

Reclamation District 17 levee evaluation

This evaluation from Engeo Incorporated was performed on the northern part of the RD 17 levees in 2006, in conjunction with residential development planned in that area. Client was the Keenan Land Company, a real estate developer.

Mississippi under-seepage research

A case that is only used in the last paragraph of this chapter is an under-seepage research that was performed on levees along the Mississippi in the 1940s and 1950s. See Figure 2.15 for the location of the Mississippi. After under-seepage problems during the 1937 high water 16 locations, shown in Figure 5.2, were chosen for an extensive research. Main goals of this research were to develop a better understanding of the phenomena of seepage beneath levees and of factors influencing under-seepage. The purpose was also to obtain information that would make possible a rational analysis of under-seepage and to study control methods. The results of this study were formulas and criteria for design of a levee sustainable to under-seepage.

Figure 5.2 Locations Mississippi under-seepage research (googleearth, 2007)

5.2 Levee evaluation process

5.2.1 Failure mechanisms and modeling

The principal causes of levee failure in the US according to the levee design manual are overtopping, surface erosion, internal erosion (piping) and slides within the levee embankment or the foundation soils (USACE, 2000). In the Initial Technical Framework (ITF) (DRMS, 2006) (see 2.2.7) they are more specific. In Figure 5.3 the seven failure modes mentioned in the ITF are related to levee failure.

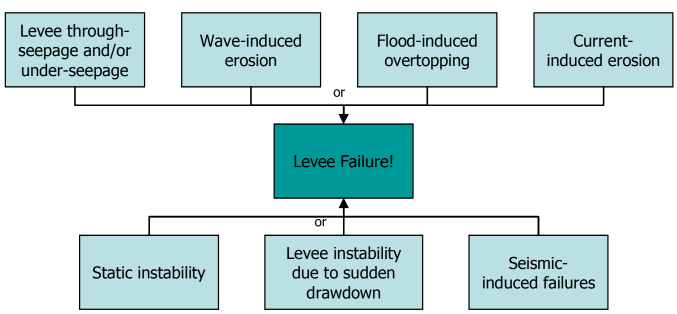


Figure 5.3 Levee failure mechanisms important for Central Valley

Internal erosion is divided in seepage through the embankment and under-seepage (piping) and is explained further in paragraph 5.5. Wave induced erosion can damage the levee on the waterside slope, especially where a large water body is connected to the levee, for example a lake, where a wind fetch can induce waves up to a few meters. But in the event of an island flooding the landside slope can also be harmed by waves and eventually cause levee failure. How to prevent wave-induced erosion or how to evaluate the levee vulnerability to this erosion is not mentioned in the design guides.

Flood-induced overtopping occurs when the water level exceeds the levee height. Evaluation of the available crown height is necessary to prevent this. The crown height loss because of settlements, which influences the available freeboard, is mentioned in the USACE manual, although it does not mention a minimum freeboard for evaluation. The settlement potential can be estimated with a detailed settlement analysis, which is not limited to design of levees, but also includes performance (USACE, 1990).

Current induced erosion is a regular observed mechanism, especially along the Central Valley Rivers. During the mid-19th century the river profiles were adjusted to flush the hydraulic mining sediment, which clocked the rivers and caused floods. Nowadays the mining sediment is gone and the rivers erode the embankments. To evaluate the erosion susceptibility of the levees a qualitative process based on inspection can be used to determine if the profile is within the safe profile. A partly quantitative process using numerical data and analyses is another option (USACE, 1994).

Slides within the levee embankment or the foundation soils are treated in 5.4. Static instability, levee instability and dynamic instability are treated. Dynamic instability is caused by earthquake movements. Another aspect of earthquakes induced failures is liquefaction. Liquefied sand has a reduced strength and stiffness and is not able to support structures anymore. Especially the saturated sand, often found under levees, are vulnerable to liquefaction. Although there are some simple standards to perform a dynamic stability analysis, using earthquake accelerations and estimating the liquefaction susceptibility of the levee foundation, new and better procedures to evaluate the seismic vulnerability of a levee are still under development. (USACE, 2003; Athanasopoulos, 2007)

5.2.2 As applied in cases

The DWR project only comprises geotechnical analyses. This means that the evaluation of the levee height is not included in this program. The erosion susceptibility of the levees is expected to go no further than just a qualitative estimation. The idea is to identify levee areas that exhibit erosion or are expected to be at risk for future erosion. Seepage and static as well as dynamic stability are the main topics in this project and are treated separately in the last two paragraphs of this chapter.

In the 2006 RD 17 evaluation the liquefaction potential of the silts and sands, levee static and dynamic instability and seepage susceptibility were included.

5.3 Loads and ground model

5.3.1 Loads

All loading conditions that could be critical should be assessed. Rules on which loads have to be used are not uniform in the whole US. While not all states deal with the same environment, boundary conditions vary widely. California does not have hurricanes to deal with and Louisiana is not afraid of an earthquake to demolish their levees. Table 5.1 gives an overview of the loads that are globally involved in the US evaluations.

Three stressing events are distinguished in the USACE manual: sudden drawdown, full flood stage and earthquake. The ITF changed them somewhat: normal 'sunny weather' conditions, flooding and seismic loading. Traffic loads and the weight of the levee itself have to be considered in each of these events. In Table 5.1 loads which are and which are not taken into account in the levee evaluation are summarized.

Normative flood levels in the US differ per State or even city, as was explained in 2.2.2. In California the normative flood level is often the water level with a probability of exceedance of 1/100 per year, which is also a FEMA criterion. While the US still has a short data history of flood levels, the normative water levels used in levee evaluation are very sensitive to changes caused by new extreme water levels.

Table 5.1 Loads which are and which are not involved in levee evaluation

Which loads are considered in the DWR levee evaluation:	And which for example not:
Permanent loads: • Dead weight of the levee • Non-water retaining objects (i.e. pipelines) Hydraulic loads: • Normative flood levels • Sudden drawdown conditions • Waves Other loads: • Traffic • Earthquakes	Permanent loads: • Extraction from subsoil (e.g. water, salt, gas) resulting in settlements Hydraulic loads: • Precipitation Other loads: • Ice load • Collision • Explosions • Damage from vermin ao • Vandalism / terrorism

5.3.2 Ground model

The USACE manual (USACE, 2000) provides a guideline with field investigation and laboratory tests on which a ground model for evaluation or design should be based. Table 5.2 summarizes the USACE guidelines. The investigation starts with an office review followed by a field survey. The office study involves a search for available information, such as topographic and geological maps, old field investigations, performance history and aerial photographs. Combined with the field survey, which includes observation of physical features and interviewing of local people or organizations, this results in a report on which further field investigations can be based.

Table 5.2 Proposed investigation to prepare a ground model

Phase	Information sources	Range of investigation (indication)	Intention
Office Study	Maps, old field investigations and inspections	Whole levee length profile	Advise for field investigation
Field survey	Experts / representatives of levee-related agencies; physical features	Whole levee length profile	
Phase 1 exploration	-Borings -Soundings -Classification on disturbed samples -May include geophysical exploration	Borings from waterside toe, landside toe and a deep exploration at the levee crest every 30 to 600 m; depth at least height of levee and not less than 3 m.	Basic ground model
Phase 2 exploration	-More borings -Piezometers -Lab-tests on undisturbed samples: triaxial tests; compression tests; sieving -May include geophysical exploration	Borings from waterside toe and landside toe, if not performed in phase 1; Piezometers should always be installed in potential under-seepage areas	Adjustment of ground model on specific locations

The subsurface exploration can be divided in a phase 1 and phase 2 explorations, of which the first phase mainly consists of soil identification and lab tests on disturbed sample borings. The spacing between borings and/or soundings is 300m maximum. The second phase is to get more detailed information about specific areas and consists mainly of extra borings and/or soundings, installation of piezometers (if not already done in the first phase) and lab tests on undisturbed samples. Use of geophysical explorations such as ground penetrating radar or electrical resistivity measurements should be considered to provide at least some level of insight regarding conditions between boreholes and CPT probes. Details on the laboratory test methods and correlations can be found in the USACE manual. (USACE, 2000)

5.3.3 As applied in case studies

DWR project:

Flood levels to be used in the evaluation are the 200-year flood level, which is requested by the DWR and is possibly adapted in the future by FEMA. But also the 100-year flood level, which is the current FEMA criterion, and the 1957 design profile water elevation. When for example the 200-year water elevation is above the crest height, a water level equal to the crest height will be used for evaluation. Besides normative flood levels rapid drawdown is also used as extreme hydraulic load on the levees. To estimate the influence of erosion high water flow, wave action and long fetch are also taken into account. The flood levels are provided by the DWR or local agencies.

The chart from Figure 5.4 will be followed in the DWR project. Following the USACE documents, each 1,000 feet (300 m) a boring is done, combined with CPTs. The phase 1 explorations are only taken through the levee crest. In phase 2 toe borings and borings in the hinterland will be taken. Fugro has the policy to do borings to a depth of 4 times the levee height, which in the case of the DWR project means a depth of about 35 meters.

Based on U.S. Army Corps recommendations for levee design, soil exploration is required every 1,000 feet (300 m) along the crown of the levee, the waterside toe and the landside toe.

To come to a final Geotechnical Evaluation Report (GER) the work flow chart of Figure 5.4 is followed. It shows that, based on a phase 1 (P1) of geotechnical exploration, a ground model is developed and evaluated, resulting in a preliminary GER. P2 explorations and model refinement, with evaluation using the same models, leads to the final GER.

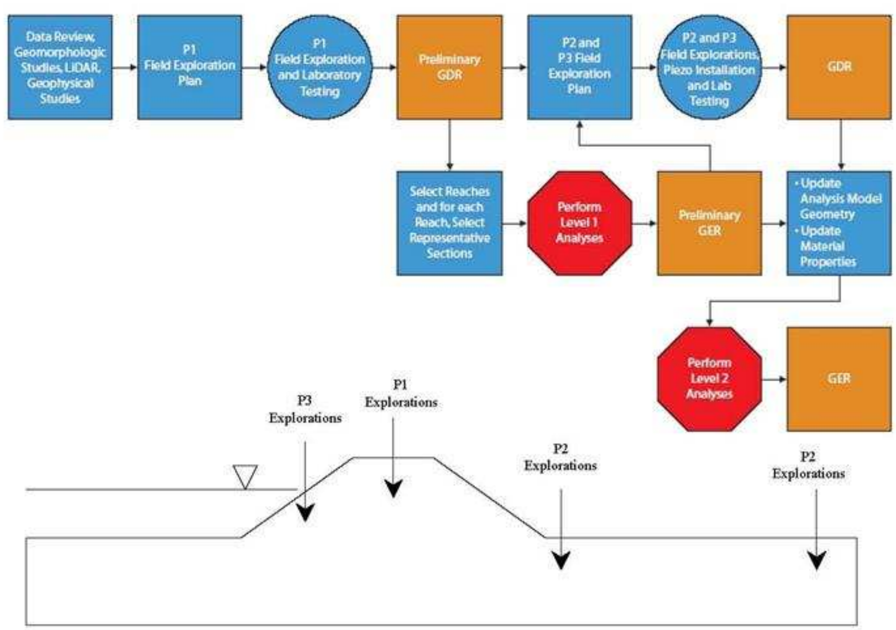


Figure 5.4 Overall work flow chart DWR project (UGF, 2007)

2006 RD17 evaluation:

The ground model was based on existing borings from a research from 1989 and 14 new borings and 40 new cpts both to a depth of about 18 m . Cpts, from the crown and landside, were spaced every 1,000 ft (about 300 m) with every fifth cpt a boring. Various lab tests supported the field research, such as permeability tests, triaxial tests (isotropically consolidated undrained tests and unconsolidated undrained tests), determination of the plasticity index and particle size distribution. The ground water level used in the evaluation was based on ground water levels found during the borings and cpts and was established at 1.5 m below ground level. River stages at normal low-flow stage, monitor stage and project-flood stage (1/200 year flood) provided by the Data Exchange Center from the Department of Water Resources were used. The loading conditions are long-term conditions, sudden drawdown and earthquake loading. (Engeo, 2006)

5.4 Stability evaluation

5.4.1 Methods

Conditions that could induce instability of the levee are extreme water levels, a rapid drawdown of the water level or an earthquake. How possible slides can be traced and which safety factor they have during normative conditions was already explained in 4.4 and is equally applicable to the US situation. The USACE has a special slope stability manual (USACE, 2003) which explains the methods used for dams as well as levees.

To perform slope stability analysis again PLAXIS is used. PLAXIS is a finite element program and was for example used to verify the mechanisms that caused levee failure in New Orleans. Detailed information about the geometry and soil characteristics is necessary to get a reasonable result. While in New Orleans the exact location of the breaches was known, extensive soil research could be done on those specific locations. PLAXIS as a result worked excellent for the New Orleans cases. But for whole levee stretches this is not the case. Therefore again the method of slices is applied. The most convenient software in the US stability evaluation using the method of slices are SLOPE/W and UTEXAS4. With both programs the most convenient methods as Bishop, Spencer and Janbu can be performed. Other methods are The Corps of Engineers method and the Lowe and Karafiath's procedure, both able to perform rapid drawdown analyses with partly undrained parameters. A difference between the two programs is that SLOPE/W has a graphic user interface, which makes the program quite accessible. The input in UTEXAS4 is a data file from a text editor. SLOPE/W is of the same series as SEEP/W, with which phreatic lines are generated and can easily be exported to SLOPE/W. The main reason that UTEXAS4 is used very often and is also prescribed in the DWR project is that it can perform the three-stage rapid drawdown analyses by Duncan, Wright and Wong (Duncan, 1990) easily.

5.4.2 Stability evaluation

The stability evaluation is explained with the chart of Figure 5.5. This is how the static stability, without earthquakes, is evaluated in the DWR project. The landside stability as well as the waterside stability are evaluated, both for a 200-year flood level. The stability evaluation is performed with UTEXAS4 using the Spencer method for both circular and non-circular slides. A check is performed with SLOPE/W. The minimum safety factor of 1.5 is required for the long-term, 200-year flood. When the safety factor is less than 1.5 the 100-year flood and the 1957 flood level (a standard set by the USACE and DWR which at some locations can even exceed the 200-year flood) have to be evaluated. The 100-year flood is a requirement from FEMA for flood insurance. When the 200-year protection is not reached, but the 100-year is, at least the FEMA criterion is reached.

For the waterside slope the rapid drawdown condition is also evaluated. Rapid drawdown occurs when the water level drops so quickly that the water level within the slope cannot follow this drop, because impermeable soils do not have sufficient time to drain. This situation is often critical for the waterside stability. There are different methods to estimate the stability of a slope after drawdown. The USACE prefers a total stress method recommended by Duncan, Wright and Wong (Duncan et al, 1990) (Duncan et al, 2005). This method uses a three-stage analysis in which in the last stage the lowest of the drained and undrained strength parameters is used. This three-stage method can easily be performed with UTEXAS4, which is why this program is currently subscribed for stability evaluation. The safety factor required for rapid drawdown is 1.0-1.2, depending on how long the extreme water level lasts and how rapid the drawdown takes place.

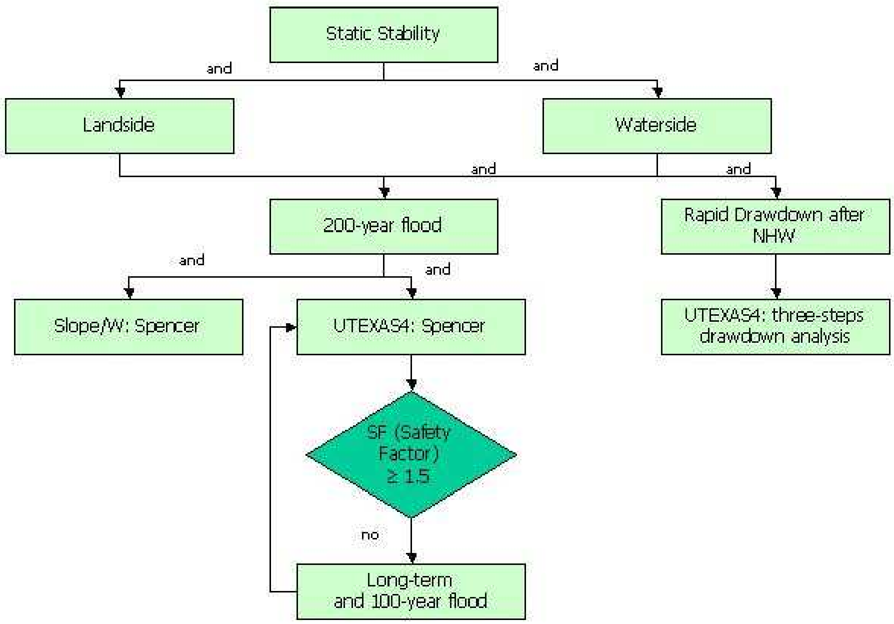


Figure 5.5 Macro stability chart DWR levee evaluation, without dynamic stability

But Figure 5.5 is only about the static stability. As mentioned before methods to indicate the levee vulnerability to earthquakes are subject of present research. Within the DWR project there is a sort of work group trying to figure out methods to do this. The idea is to develop a basic method to evaluate the seismic vulnerability of the levees and a more detailed method (Athanasopoulos, 2007). A combination of a seismic event and flood event has to be made with a reasonable probability of occurrence, while the probability that a 200-year flood coincides with a 200-year earthquake is very small. Therefore several runs will have to be made with UTEXAS4 to combine the static and dynamic stability. (URS, 2007)

5.4.3 As applied in case studies

DWR project:

How the stability is evaluated in the DWR project is already discussed above.

RD 17 research:

The stability was evaluated with SLOPEW. Both landside and waterside macro stability and combinations of earthquake and flood events led to 16 runs for each cross section. The 200-year flood stage was combined with earthquake event with return period of 72 years which yields a PGA (acceleration) of 0.12 g; seismic coefficient 0.06 for both land- and waterside. Another condition was rapid drawdown after monitor stage and 200-year flood stage at static and dynamic condition, land- and waterside. Also a post-liquefaction condition at cross-section no.3, with post-liquefaction soil strengths for static condition, normal and monitoring stage both land- and waterside; a residual strength of 300 psf was used and a factor of safety of 1.1. Calculations were done with the drained shear strength; except for the dynamic evaluations, they use undrained shear strength for clay and silt.

5.5 Seepage and piping evaluation

5.5.1 Why seepage evaluation?

Figure 5.6 Sand boils in California, 1997 (UCDavis, 2006)

Figure 5.7 Sand boil which caused a levee to fail in 1993 (Mansur, 2000)

The 1997 River floods in the Central Valley were mainly caused by seepage related levee failures. More than 30 breaches failed due to piping, as mentioned in chapter 2.

Piping has caused problems in the Mississippi River basin as well. The 1937 Mississippi high water is important in this context, because of the enormous amount of heavy seepage and sand boils that occurred along numerous reaches of the levees. After the 1937 flood in the Mississippi River basin, US authorities recognized the threat of piping for the stability of levees and started an extensive research. Although no levees actually failed in 1937, at least 6 of the about 60 major levee failures between 1890 and 1927 were caused by sand boils. More of them could have been caused by piping, but were only registered as blowouts or unknown cause of failure.

But levees at the Mississippi River have failed again because of piping during the 1993 high water, when the stage of the Mississippi River equaled or exceeded the design stage, the highest river level to which the levees had ever been subjected. (Mansur, 2000)

Piping was also one of the observed problems at levees that failed in New Orleans in 2005. (Kanning, 2006)

5.5.2 Under-seepage and piping

In the US seepage can be divided in under-seepage, below a levee, and through-seepage. Through-seepage is water seeping through the levee body itself. All levees, especially the Central Valley levees built with course materials, seep a little bit. When the phreatic line already exits the levee at the levee slope erosion problems can be expected. Therefore an evaluation of the position of the phreatic line is necessary. But this chapter is about under-seepage.

Seepage flow beneath a levee is a natural phenomenon in an alluvial valley where the river level is higher than the adjacent land. With rising river level the seepage flow increases and the hydraulic head beneath the levee and landside blanket therefore increases as well. Figure 5.8 is an illustration of the US interpretation of seepage and piping under a levee. Under-seepage, creates a hydraulic gradient in the pervious stratum. With rising water level the hydrostatic pressure in this stratum rises. When this pressure exceeds the weight of the top stratum, this pressure will cause heaving of this layer. At weak spots this can cause rupture of the top stratum. A concentrated seepage flow at these rupture points may cause sand boils. Sand boils can also be induced at places where an open channel in the top layer already exists, in bore holes or cracks. Rupture of the blanket is therefore not a necessary first step. Active erosion from under the levee as a result of concentration of seepage in localized channels is known as piping. These problems are most acute where a pervious layer underlies a levee with on top of it a thin impervious or semi-pervious blanket layer. (Mansur, 2000)

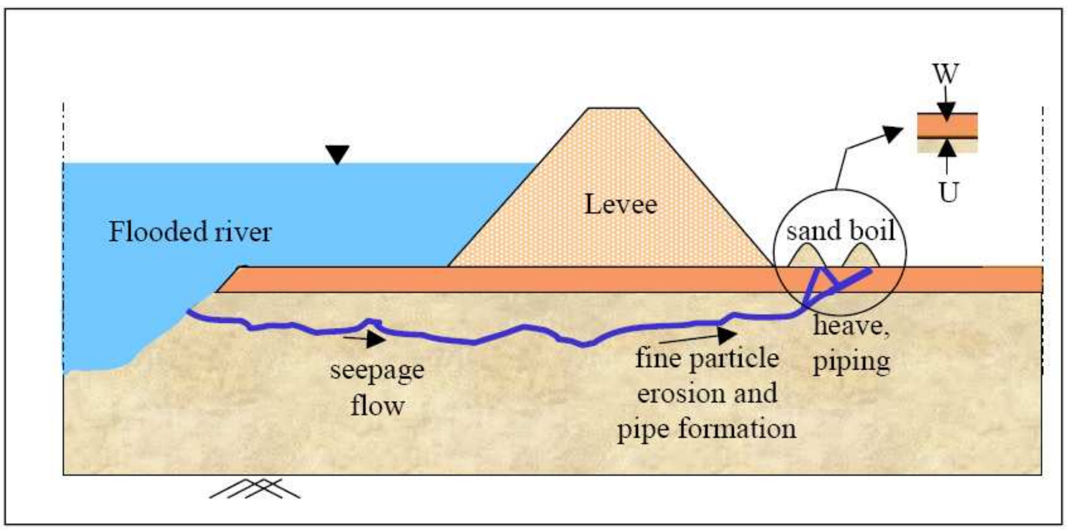


Figure 5.8 US interpretation of piping (Ozkan, 2003)

5.5.3 Evaluation

The approach that is used in the US to estimate if there is a chance piping will occur, is using the hydraulic exit gradient at the landside blanket of a levee, which is a heave criterion. The DWR project follows the flow chart of Figure 5.9 to evaluate the piping sensitivity of chosen cross-sections.

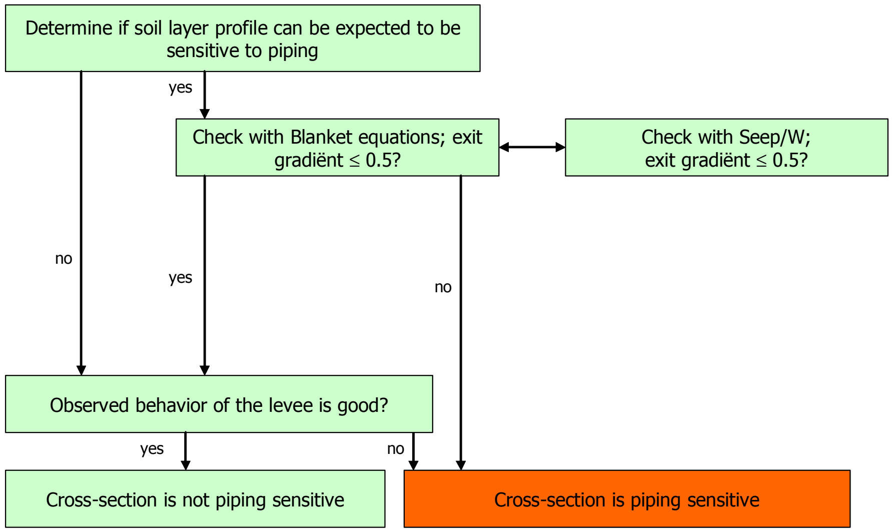


Figure 5.9 Flow chart piping modeling first GER DWR project

After cross-sections have been developed for piping evaluation, the first step is to look at the conditions: is there any reason to believe that piping could take place at this levee section. The presence of a pervious substratum with a semi-pervious or non-pervious blanket and/or historic seepage problems are such reasons. The next step is to look at the exit gradient at the toe of the levee. A maximum exit gradient of 0.5 is to be used for design and evaluation. Theoretically the critical exit gradient is defined as the gradient required to cause boils or heaving of the landside top stratum.

The gradient in the blanket layer is defined as:

$$i = \frac{h_x}{z_t} \leq 0.5 \qquad (5\text{-}1)$$

With:
i = Upward gradient [-]
h_x = Hydrostatic head in the pervious layer, above ground level at x from levee toe [m]
z_t = Critical thickness of the blanket [m]

To determine the hydrostatic head in the pervious layer the USACE subscribes two methods in her documents: the blanket equations or the use of a finite element program. These methods are presented in the design and construction of levees manual (USACE, 2000). For very simple cases the blanket equations are recommended. Computer programs as LEVSEEP and LEVEEMSU are mentioned in the technical letter as supporting software. They apply the blanket equations and can be helpful for berm design, for multiple blanket layers and when ditches and borrow pits are present. For more complicated problems finite element programs are recommended, like CSEEP or Seep/W.

5.5.4 Critical situation: background of i_c=0.5

The background of the critical exit gradient i_c, which is used as a criterion for piping, originates from Mississippi River valley research in the 1940s and 1950s. The original observations of Figure 5.10 are reproduced in Figure 5.11. The dots in the most left column are locations where sand boils were observed. Caruthersville is the most upstream location and is about 800 km from Baton Rouge, the most downstream location (see Figure 5.2). All locations are along the Mississippi River, except for Cotton Bayou, which is along the Red River. No measurements are available from Cotton Bayou. From Figure 5.11 can be read that sand boils mainly occur at the more upstream locations, with the green and blue colors. This data was used to develop the trend, shown in Table 5.3. In 2005 the USACE published a technical letter where they defined a critical exit gradient of 0.5, based on the figure and table. (USACE, 2005)

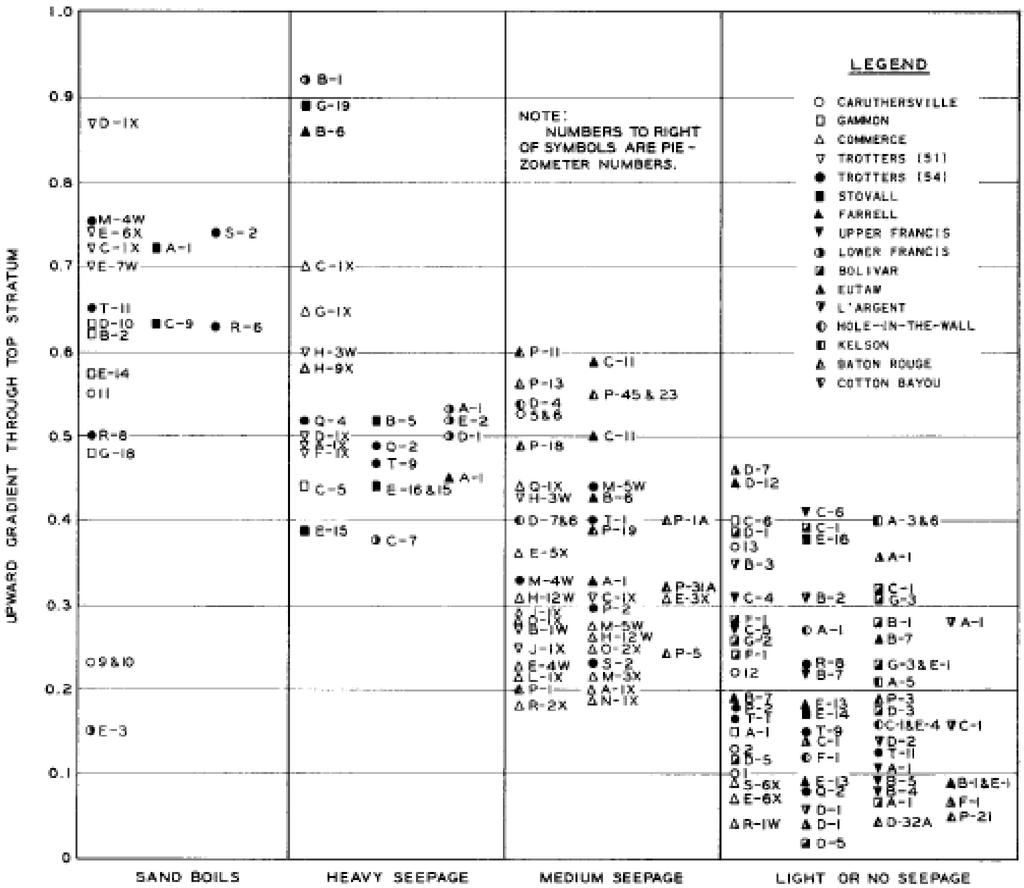


Figure 5.10 Upward gradient related to severity of seepage in 1950; Cases Mississippi at 16 locations from Caruthersville, Missouri to Baton Rouge, Louisiana (USACE, 1956)

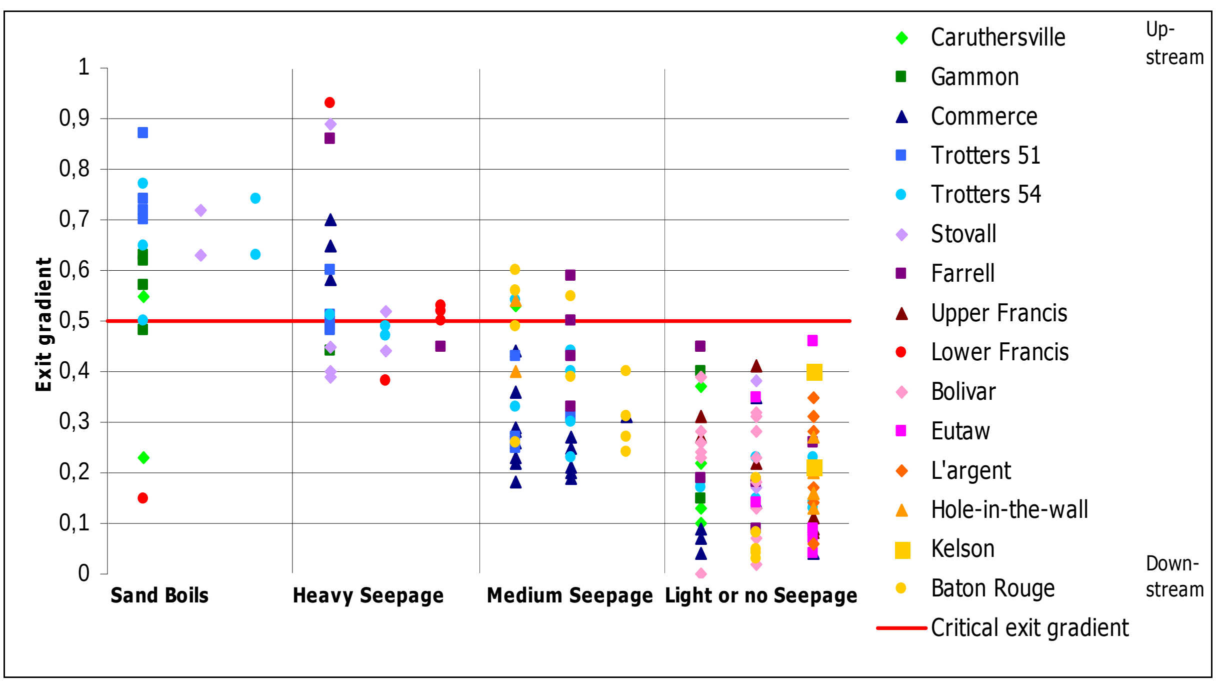


Figure 5.11 Reproduction of Figure 5.10

Table 5.3 Exit gradient vs. seepage condition trends (USACE, 2005)

Exit gradient	Seepage condition
0 to 0.5	Light / no seepage
0.2 to 0.6	Medium seepage
0.4 to 0.7	Heavy seepage
0.5 to 0.8	Sand boils

With the critical exit gradient of 0.5 a safety of about 1.6 is achieved, which can be found from the following theoretical derivation:

Upward flow in the blanket, initiated by a rise in hydrostatic head, will cause a change in the water pressure p and in the effective stress σ′. The total stress σ is constant. Then:

$$\Delta\sigma' = -\Delta p = -h_x\gamma_w = -iz_t\gamma_w \tag{5-2}$$

$$\sigma' = z_t\gamma' - iz_t\gamma_w = z_t\left(\gamma' - i\gamma_w\right) \tag{5-3}$$

With:
σ'= Effective stress [kN/m^2]
p = Water pressure [kN/m^2]
γ_w= Volume weight of water [kN/m3]=10 kN/m^3
γ' = Submerged unit weight of the blanket soil [kN/m3] $\gamma' = \dfrac{\gamma_{w,s} - \gamma_w}{\gamma_w}$
$\gamma_{w,s}$= Volume weight of the wet blanket)kN/m^3

Heave occurs when the effective stress in the blanket becomes zero:

$$\sigma' = 0 \quad if \quad \gamma' = i\gamma_w$$

The critical gradient which starts heaving of the blanket is then:

$$i_c = \frac{\gamma'}{\gamma_w} \tag{5-4}$$

The theoretical exit gradient to start heaving is about 0.8, based on a volume weight of the soil of 18 Kn/m^3. The critical exit gradient if 0.5 should therefore give a safety of 1.6 for first time boils.

5.5.5 Blanket equations

The blanket equations can be used to calculate the residual head landside of the levee below the blanket layer. They were published by Bennett in 1946 and are solutions for steady-state seepage through a two-layer system composed of a semi-pervious top blanket overlying a pervious substratum. Later analyses are based on these equations. In the 1940s and 1950s a seepage research was performed based on cases from the Mississippi river, where numerous sand boils were observed during the 1937 floods. At that moment little was known about the relation between geology features and under-seepage. Purpose of the study was partly to get a better understanding of the piping phenomena and to develop formulas and criteria for design.

The seepage research done along the Mississippi, published in 1956 (USACE, 1956) uses these blanket equations and they are still mentioned in the current levee design manual (USACE, 2000). Different blanket equations were developed depending on:

- Permeability of the blanket: non-pervious or semi-pervious;
- The presence of a landside and or riverside blanket;
- The existence of a seepage block, open seepage exit or an infinite blanket landside of the levee

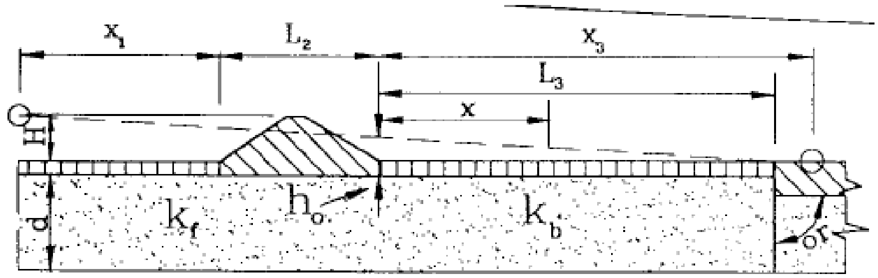


Figure 5.12 Regularly observed blanket case along Mississippi River (USACE, 1956)

A situation that is regularly observed along the Mississippi River is a levee with a semi-pervious blanket on the landside until a thick clay swale (Figure 5.12), which functions as a seepage block. The equations used for this case are:

$$h_0 = \frac{Hx_3}{x_1 + L_2 + x_3} \qquad (5\text{-}5)$$

With:

$$x_3 = \frac{1}{c\tanh(cL_3)}, c = \sqrt{\frac{k_{bl}}{k_f z_b d}} \qquad (5\text{-}6, 5\text{-}7)$$

h_0 = Head beneath top stratum at landside levee toe [m]
h_x = Head Beneath top stratum at distance x from landside levee toe [m]
H_c= Net head on levee [m]
z_b = Thickness of blanket [m]
d = thickness of pervious substratum [m]
i_0 = exit gradient at toe of the levee [-]
L_2 = Length of levee at base [m]
L_3 = Length of blanket at landside of the levee [m]
x_1= Distance from landside levee toe to effective seepage entrance [m]
x_3 = Distance from landside levee toe to effective seepage exit [m]
c = Factor [1/m]
k_{bl} = Vertical permeability of riverside top stratum [m/day]
k_f = Horizontal permeability of pervious substratum [m/day]

5.5.6 In case studies:

DWR project: In the first step of the DWR project the blanket equations are being used and a check is performed with the finite element program Seep/W.

RD 17 research: The seepage evaluation in this research was performed with SEEPW. A distinction was made between steady seepage (monitor stage) and transient seepage flow (200-year flood stage). The transient seepage flow calculated with four weeks of 200-year flood stage followed by monitoring stage.

Mississippi research:

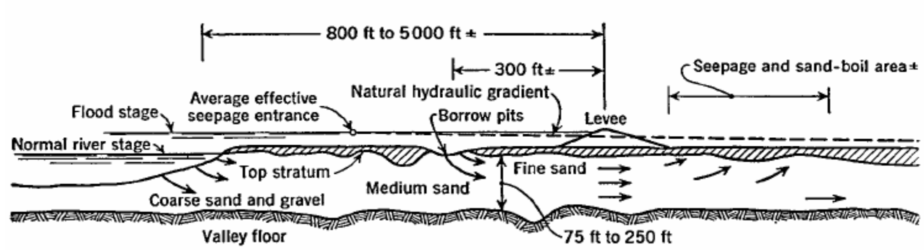


Figure 5.13 Typical cross section of Mississippi levee (Mansur, 2000)

In Figure 5.13 the typical situation along the Mississippi River is illustrated. The distance from the River to the center of the levee can be 250-1500 m. What makes these levees still vulnerable to underseepage is the riverside borrowpits that were probably dug to extract clay for levee improvements. A sand layer of about 20 to 75 m thick overlain by a 1-10 m clay layer is a situation where piping could occur. Table 5.4 gives the soil characteristics of the top stratum en sub stratum as they were found in the Mississippi research.

Table 5.4 Conditions on which the Mississippi investigation was founded (modified from USACE, 2002)

	Thickness	**Material**	**Permeability (average)**
Riverside top stratum	0-5 ft ; 10-15 ft 15-20 ft	Clay: Silt: Silty sand:	1x10-4m/s (Zbr<=5ft) 0 (Zbr>=15ft) 2.5x10-4cm/s 6x10-4cm/s
Landside top stratum	4-30 ft	Clay Silt	0.06x10-4 to 10x10-4 cm/s
Pervious sub stratum	70-165 ft	Sand	400x10-4 to 2500x10-4 cm/s

No software as SEEP/W was available yet at the moment of the underseepage research. With piezometer data, water level data, geometry data and soil data exit gradients were calculated and soil data was again recalculated from the observed exit gradients. The blanket equations were used for these forward and backward calculations.

6 Netherlands versus Central Valley, California

The goal of this chapter is to compare the water defense system of the Netherlands with the system in the Central Valley in California, USA. The descriptions from all previous chapters are used to give insight in those differences and partly try to explain them. A comparison on the level of the whole water defense system is made first. After that the focus will be on levee evaluation. In the last paragraph, 6.2.1, is concluded what the most interesting differences are for further research.

6.1 Comparison water defense systems

6.1.1 Similarities

There need to be enough similarities between two systems to make an honest comparison of their differences and try to bring them closer to each other. From the previous chapters the most interesting similarities are:

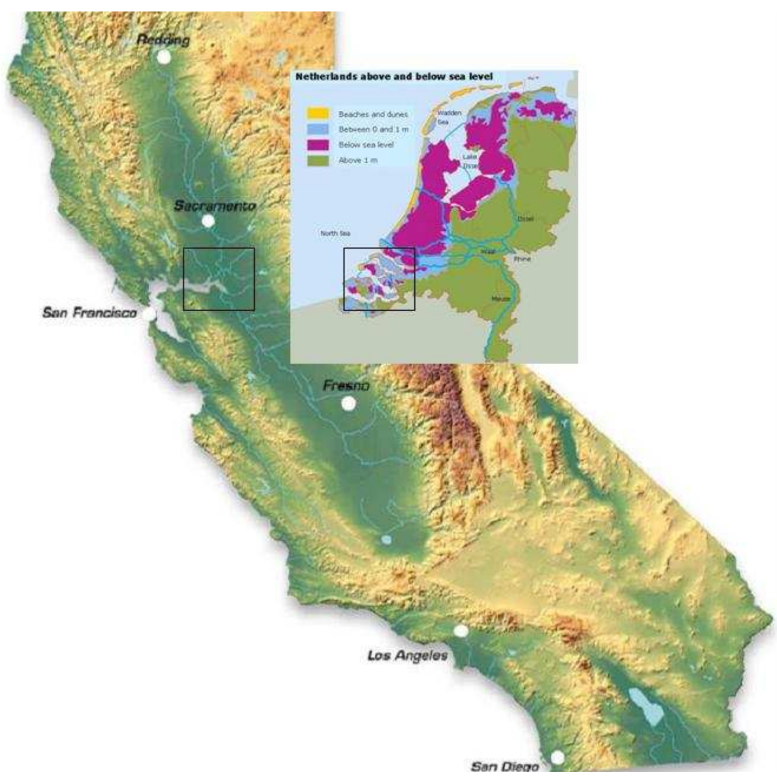


Figure 6.1 Size of the Netherlands compared to the size of California

- **Flat low-lying Delta**

When maps of the Netherlands and California Delta are compared it is interesting to see that the Netherlands is only a small country compared to the State of California. But the size of the study area: the flat Central Valley with the Sacramento San Joaquin Delta is comparable to the size of the Netherlands. The Delta has the size of about one Dutch Province. (Figure 6.1)

Both areas are characterized by its flatness. The Central Valley is a flat area situated between the Sierra Nevada and coastal mountain ranges. The Dutch River Rhine originates in the Swiss Alps and the Meuse in mountainous areas near Dijon in France, while the Netherlands no mountains but only hills can be found. When looking at Figure 6.2 land below mean sea level is characteristic for both. Elevations reach until almost 8 m below mean sea level.

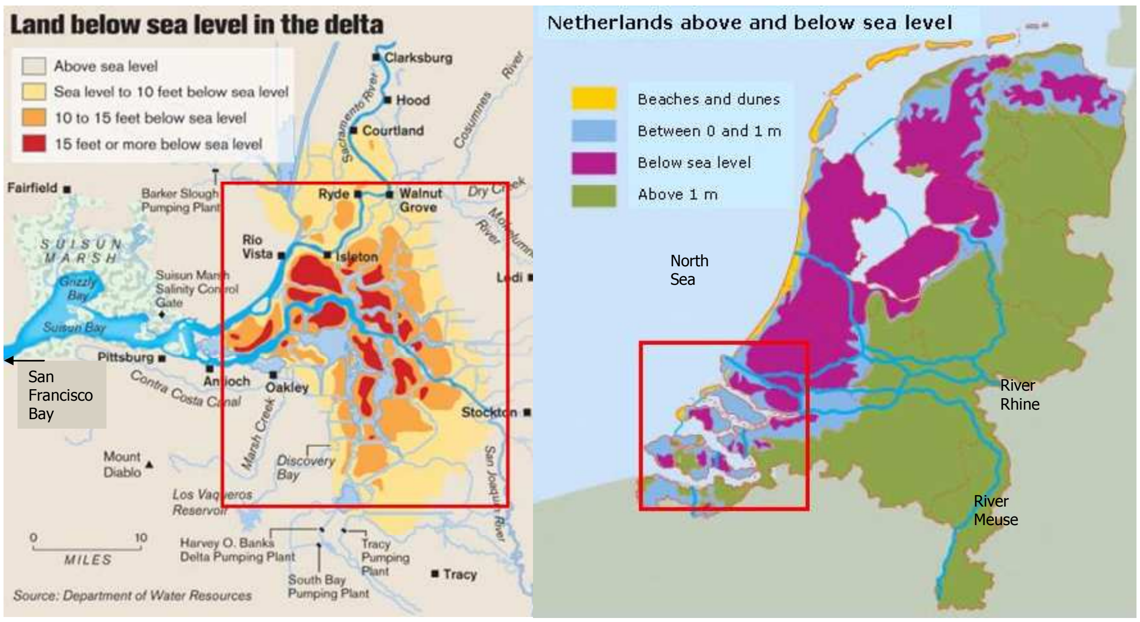


Figure 6.2 Delta California and Netherlands below sea level (red squares represent the same surface area)

Because the systems of rivers flowing into a Delta are somewhat the same: the San Joaquin River and Sacramento River in the Central Valley and the Rivers Meuse and Rhine in the Netherlands, the geology also shows similarities. Clay and sandy materials characterize the upstream river embankments, while the areas below sea level have peat subsoil which is subject to subsidence. Although the Delta in California is more sheltered from the sea/ocean than the Netherlands is, both have tidal influences in their rivers and are vulnerable to storm surges. Flood waves, caused by snow melt and heavy rainfall have caused troubles in the Central Valley as well as in the Netherlands. While both areas are influenced by salt water bodies and the tide from the North Sea in the Netherlands and the San Francisco Bay in California, salt intrusion in dry periods or during flooding is a possible threat.

- **Similar floods**

If we compare recent floods again similarities are found. The Central Valley River flood of 1997 and the 1993/1995 Rhine and Meuse river floods were both caused by flood waves from the rivers. (Figure 6.3)

In both situations piping was one of the most important failure mechanisms. In the Central Valley 30 levees failed because of piping. In the Netherlands, no levees actually failed due to piping, while large scale sacking prevented this, but levees overtopped and in that way inundated areas. In the Netherlands 240,000 people were evacuated, while in the Central Valley 120,000 people were forced from their homes and 6 people died.

Figure 6.3 1997 Central Valley River flood (left) and River Meuse flood 1995 (Reid, 2005)

Other floods that show similarities are the 2004 Jones Tract levee failure in the Sacramento-San Joaquin Delta and the 2003 Wilnis failure in the Netherlands. (Figure 6.4) Both of these two levee failures were dry weather failures. The Wilnis levee failure is most probably caused by uplift and horizontal sliding of the peat levee. Because the canal was closed off quickly the evidence was preserved and the damage limited. The cause of the Jones Tract levee failure is not known, because all evidence got washed away. What is known is that at least part of the levee and its foundation consisted of peat. The water supply infrastructure had to be shut down for several days, because the levee breach caused salt water intrusion.

Figure 6.4 Jones Tract levee failure 2004 (Reid, 2005) and Wilnis levee failure (Geodelft, 2004)

But another interesting comparison is the one between the 2005 New Orleans flood and the Dutch 1953 Zeeland flood. The impact of the flood in New Orleans and the Zeeland flood were probably quite equal. More than 1,300 people died because of hurricane Katrina, while about 1,800 people died of the storm surge that caused the Zeeland flood. The response of the Dutch people was a Delta Plan and Delta Law to prevent that such a large disaster could ever happen again. Large infrastructure works as the Eastern Scheldt Storm Surge Barrier and the Maeslandt Barrier were built and safety standards related to damage were defined. The primary water defenses should provide protection against a flood with a probability of occurrence of 1:10,000 per year to 1:1,250 per year. New Orleans is still recovering from hurricane Katrina. The New Orleans disaster created the awareness that it is necessary to invest in and pay attention to the water defenses protecting urbanized areas. As a response California currently invests millions in levee geotechnical investigations and improvements. A Delta Plan as carried out in the Netherlands is not developed for New Orleans and California. Not yet. It is clear that disasters create awareness and initiate changes.

Figure 6.5 New Orleans levee breach and 1953 Zeeland levee breach (Fas.org, 2007; Deltawerken.com, 2006)

6.1.2 Differences

Although there are similarities between the two systems, there are even more differences. The most interesting differences are:

- **Difference in risk acceptance; flood insurance**

There is an enormous difference in the level of protection that is prescribed in the Netherlands and in the US. A water level with a probability of exceedance of 1/100 or 1/200 per year is the current design level in California, while in the Netherlands the design water level has a probability of exceedance of 1/10,000 to 1/1,250 per year, for primary water defenses. Other, regional water defenses have a lower safety level: 1/10-1/1,000 per year. The regional water levels do not fluctuate that much and can be regulated by pumps, and when they breach the damage is often limited to a small area, so called polders, with a fixed polder water level. The safety levels in the Netherlands are based on the expected flood damage and were determined after the 1953 flood. The accepted probability of exceedance in the US was an arbitrary chosen value. Flood insurance, which is obligatory in areas with a less than 1/100 protection, is related to this safety levels. In the Netherlands people cannot buy flood insurance.

- **Delta risk partly outside the area**

Another very interesting difference is the economic damage that could be caused by a flood. In the Netherlands, with its 16.3 million inhabitants and major infrastructure works, a flood could lead to enormous economic damage within the country itself, although not quantified here. In the Central Valley not only the people living in large cities as Sacramento, or below sea level in the Delta are affected, but outside the Delta 33 million people rely on fresh water from the Delta. In other words: a flood does not only directly affect people in the Central Valley and Delta, but also indirectly influences the rest of California. Therefore solutions for the flood problems in California could be different than in the Netherlands. A bypass to secure the availability of fresh water is already discussed since long, but has always been resisted, off course mainly by people living in the Delta.

- **Different loading conditions on and strength of levees**

In New Orleans they have hurricanes and in California there are earthquakes. The Netherlands does not encounter such heavy earthquakes and definitely no hurricanes. But heavy storms from the North Sea have caused similar damage in the past as hurricanes did in New Orleans.

The levee strength is also different, mainly caused by historical events. Two examples of that are given. Levees in the Central Valley in California often mainly consist of highly permeable, badly compacted materials from upstream mining activities. These levees are therefore vulnerable to erosion and seepage. In the Netherlands there are levees which mainly consist of peat, remaining from peat excavations. Because peat is vulnerable to dry circumstances and subsidence it is not preferred for levee construction.

- **Levee evaluation**

How to determine if the levees are sufficiently safe is in the Netherlands documented in special levee evaluation documents. In the US engineers are familiar with designing and evaluating large earth dams. For levee evaluation levee design documents are used, which are closely related to dam design manuals. The evaluation also depends on criteria from FEMA. FEMA carries out the US flood insurance policy and is responsible for floodplain maps and the rejection or approval of water defenses. After New Orleans more focus is now on levees and on how to design and evaluate them. A desire to increase the safety levels is not only a matter of politics, but how to deal with flood insurance is also part of this. Currently there is an increased interest from the US on how the Dutch deal with levees and safety.

6.2 Comparison levee evaluation methods

From the broad description and comparison of the Dutch and US/California water defense systems levee evaluation seems one of the topics worth studying. Because levee evaluation is a hot topic at this moment, especially in the US where after hurricane Katrina many miles of levees have to be evaluated, this subject was chosen for further research.

The similarities and differences in levee evaluation as described in chapter 4 and 5 are described and partly explained in this paragraph.

- **Uplift is not modeled in stability calculations DWR project**

Uplift is an important part of the Dutch stability evaluation, which can easily be performed with an uplift modulus in MStab. In the DWR project in California the uplift mechanism is not included in the stability evaluation, while in other parts of the US it is included. The USACE has a special uplift program to perform uplift calculations. Especially at locations with sand overlain by weak, non-permeable blankets, which is definitely the case along the Central Valley rivers and in the Delta, there is a chance that the pressures in the sand layers will exceed the weight of the blanket. If this happens a far larger circular slide or a lateral translation of the whole levee could develop with disastrous results. Including the uplift mechanism in stability evaluation will often result in an immediate decrease of the safety factor at the water level where uplift is expected. But also will it lead to a far larger slide. It is therefore important to include uplift in the stability evaluation.

- **Drained versus undrained parameters**

In rapid drawdown calculations the Americans partly use undrained parameters, following Duncan's method (Duncan, 1990; Duncan, 2005). For materials that drain very easily, like course sand, drained parameters are used. In the Netherlands drained parameters are used for rapid drawdown evaluation, in combination with a realistic phreatic line for the situation just after drawdown. It would be interesting to see what differences in safety factor the Dutch and US method would deliver for various situations.

- **Piping is evaluated with different equations and criteria**

In the US a critical exit gradient, which is a heave criterion, is used for evaluation and design, where the Dutch use a critical seepage length as a criterion. The Dutch criterion is based on solutions of differential equations combined with model tests, while the US criterion is based on real cases from the Mississippi River. It would be very interesting to compare those two and come up with recommendations for both methods.

- **Same amount of prescribed soil research, different interpretation in cases**

About the same amount of soil research and lab tests are prescribed in the Dutch and US documents. But the DWR project only takes crown borings and CPTs in the first phase and then landside toe and hinterland borings in the second phase.

- **Guidelines for evaluation:**

In the US design guidelines are used also for the evaluation. In the Netherlands special evaluation manuals are available, in which the procedures differ from design guidelines. Each 5 years the safety of the primary water defenses has to be reported to the Dutch government. In the US there is no such legislation yet, but FEMA is thinking about a regular levee evaluation.

- **Simple to advanced evaluation**

No (real) process from simple to advanced evaluation is implemented in the US evaluation. With little information running an advanced model will not lead to a better estimation of the real safety than using a simple model. This knowledge is applied in the Dutch evaluation, but not yet that much in the US evaluation. One explanation for that is that there is not a separate evaluation guide.

- **Normative conditions for evaluation**

In the Netherlands the project flood, which can vary from a 1/1,250 year water level to a 1/10,000 water level, is assumed the normative condition. In some cases, where clay is dominantly present in the levee crown, extreme precipitation is normative. Rapid drawdown is also modeled as a possible normative condition, in the Netherlands as well as the US. In the US precipitation is not assumed as a normative condition. Not only the project flood is modeled, in the DWR project 1/200 year water level, but also the 1/100 year water level, which is a requirement of FEMA for levee certification.

- **Dynamic evaluation**

The failure mechanisms evaluated are more or less the same. The differences are that instability of the foreland is not mentioned in the US evaluation and that the Dutch are not doing a dynamic levee evaluation This is because there is not such a large risk to seismic shaking in the Netherlands as in for example California.

- **Programs used for stability evaluation are quite similar**

The simplest slide plane calculations, such as Janbu or Bishop can easily be calculated by hand or with a spreadsheet. But more rigorous methods as for example Spencer ask for more specialized software. Today the most common way is to use modern limit equilibrium software.

Table 6.1 Comparison of computer programs used in the US and Dutch stability calculations (Pockoski, 2000)

	UTEXAS4	**Slope/W**	**MSTAB**
Models	Bishop Spencer Janbu Corps of Engineers method Lowe and Karafiath's procedure Schwedisch procedure	Bishop Spencer Janbu Corps of Engineers method Lowe and Karafiath's procedure Ordinary method of slices Morgenstern Price General Limit Equilibrium	Bishop Spencer Fellenius Uplift Van Uplift Spencer Bishop probabilistic random field
Data input	Data file from text editor	Graphic user interface	Graphic user interface
Pore water pressure	Piezometric line(s)	Piezometric line(s)	Piezometric line(s)
Extra	Can perform multistage stability computations for rapid drawdown and earthquake loads	Can perform probabilistic stability analysis using the Mont Carlo technique; can deal with earthquake loads	Can do uplift calculations and deal with earthquake loads

The Dutch MSTAB, American UTEXAS4 and Canadian Slope/W are computer programs applied in the Dutch and US stability evaluations. Slope/W and MStab are very much alike. Not only do they have most of the same features, they also have the same kind of graphic user interface as can be read from Table 6.1 and seen in Appendix I.Appendix 3. The results from MSTAB and Slope/W logically do not differ that much, while the same models are used.

- **Semi-pervious or non-pervious blankets**

In the Dutch schematization the top layers, or blankets, are assumed to be non-pervious. In the US schematization the blanket can be schematized as a semi-pervious or non-pervious layer, which is implemented in the blanket equations. A blanket of less than 15 ft thick (≈4.5 m) is assumed semi-pervious and a blanket of more than 15 ft non-pervious.

- **Schematization of the blanket**

In the US the blanket layer often consists of more than one soil. On top of the non-pervious clay it has a sandy or silty layer, or just below the clay layer there is silty material. To convert the permeability and thickness of these separate layers to one layer they use a transformed thickness for seepage calculations and a critical thickness for uplift calculations.

6.2.1 Conclusion

The two most interesting differences that require further study are the difference in drained and undrained stability calculations and the difference in piping evaluation. The effect of using drained or undrained parameters in the stability calculation is subject of a current research in the Netherlands and is therefore not explored here further. Piping models are subject of discussion in the VNK project (or FLORIS project, see 3.3.2). While the US evaluation method is based on case studies from the Mississippi River and the Dutch methods are validated with laboratory tests, but not with real life cases, this subject is chosen for further investigation. The following paragraphs will reveal some interesting details about piping modeling.

7 Sand boils and piping

The goal of this chapter is to have a closer look on the differences between the piping evaluation methods of the US and the Netherlands, while was concluded in the previous chapter that this seems one of the most interesting differences between the US and Dutch evaluation. The 1956 seepage research from the Mississippi, on which the current under-seepage criterion for the US is mainly based, supports this elaboration (USACE, 1956). The theories and methods about sand boils and piping, presented in chapters 4 and 5, are further explained in 7.1 and more background information on safety is given in 7.2. In 7.3 the Mississippi cases are used to show the differences in the Dutch and US evaluation and in 7.4 possible explanations for those differences are discussed.

7.1 Theories US and Dutch piping criteria

The formulas and criteria for piping presented in 4.5 and 5.5 look different. But what exactly is different and what do they have in common? Which processes are important and at what point in the process is a levee rejected? Those questions will be answered in this paragraph.

7.1.1 Problem schematization

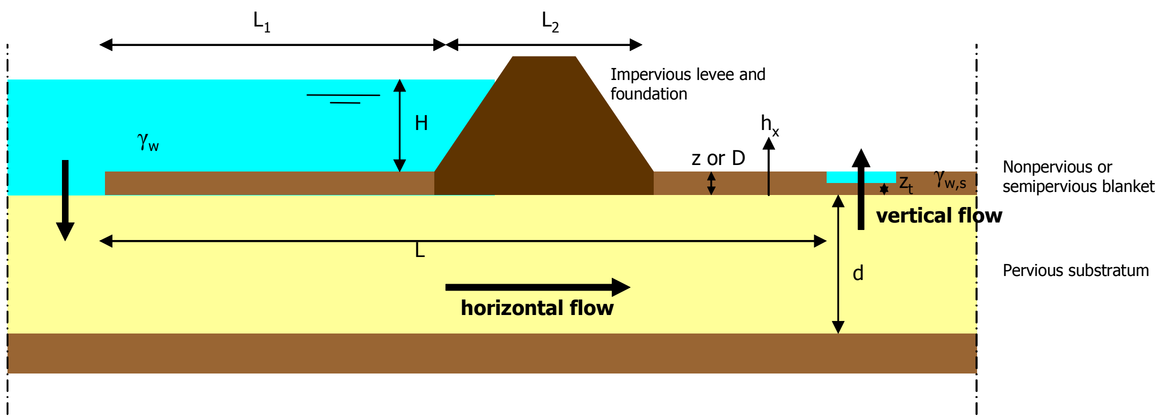


Figure 7.1 Schematized profile seepage analysis

In chapters 4 and 5 only the formula's or methods used in seepage/piping evaluation and the process (how to use those formulas) were explained. But to be able to use the formulas the levee first has to be modeled. The following simplifications for under-seepage are used, drawn in Figure 7.1 in the as well as in the Netherlands:

- A stationary, laminar flow situation is assumed.
- The blanket layer is relatively non-permeable compared to the pervious substratum and the levee itself is assumed impermeable.
- Flow in the blanket is assumed to be vertical and flow through the pervious substratum horizontal.

- Seepage may enter the pervious stratum either at the river bank, through riverside borrow pits, and/or through the semi-pervious top stratum riverside of the levee.
- The foundation was generalized into a pervious sand stratum with a specific thickness and permeability and a non-/semi-pervious top stratum with a uniform thickness and permeability.

7.1.2 Formulas and processes

The formulas, criteria and processes to assess the piping vulnerability of a levee are different. But because the same assumptions are used, a comparison can be made. The Dutch evaluation was divided in two processes: uplift and piping. These processes are separated in two steps to compare the formulas.

Step 1:

Table 7.1 contains the, rewritten, formulas that describe the first step in the piping process: uplift and possible rupture of the blanket.

Table 7.1 Dutch and US criterion for uplift

Dutch	**US**
Formula 4-1 can be rewritten to: (when the phreatic head in the blanket is equal to ground level) $$\left(\phi_z - h_p\right) \leq \frac{1}{\gamma} D \frac{\gamma_{w,s} - \gamma_w}{\gamma_w} \qquad (7\text{-}1)$$ Then this formula is rewritten to compare formulas: $$\frac{h_x}{z_t} \leq \frac{1}{\gamma} \frac{\gamma_{w,s} - \gamma_w}{\gamma_w} = \frac{1}{\gamma}\gamma' = \frac{1}{1.2}\gamma' \qquad (7\text{-}2)$$ A safety factor γ of 1.2 is normally applied combined with characteristic values for the parameters (see paragraph 7.2). A safety factor of 1.5 with mean values used to be common. Above equation is often written as: $$h_x \gamma_w \leq \frac{1}{\gamma} z_t \left(\gamma_{w,s} - \gamma_w\right) \qquad (7\text{-}3)$$ The part left of the equal sign represents the upward pressure, the part right stands for the downward pressure with a safety factor.	The criterion is again (formula 5-1): $$i = \frac{h_x}{z_t} \leq i_c = 0.5 \qquad (7\text{-}4)$$ The 0.5 can be split up, just like the Dutch criterion: $$\frac{h_x}{z_t} \leq \frac{1}{\gamma} \frac{\gamma_{w,s} - \gamma_w}{\gamma_w} = \frac{1}{\gamma}\gamma' = 0.5 \qquad (7\text{-}5)$$ As described in chapter the γ′ is theoretically about 0.8 and the safety factor γ therefore about 1.6, using expected or characteristic values (see next paragraph). The saturated unit weight of the landside blanket soils must be at or above about 17.6 kN/m^3 for this criterion to be valid (USACE, 2005).

Although written somewhat different, the above first step answers the question: can we safely expect that the blanket is able to resist the water pressures underneath it? The difference in the first step is that in US evaluation, the levee is immediately rejected when the answer to this question is no. In the Dutch method the levee is not immediately rejected, but a second step is applied. Or concluding: the process is the same, but the conclusion different (see Figure 7.2). Another difference is the factor of safety that is used. In equation 7-2 a fixed factor of safety of 1.2 is applied, where in equation 7-5 the safety factor is combined with the wet volume weight of the blanket soil, which actually results in a variable safety factor. These overall safety factors cannot be compared directly, because within the choice of parameters there is a hidden safety as well. This is explained further in paragraph 7.2.

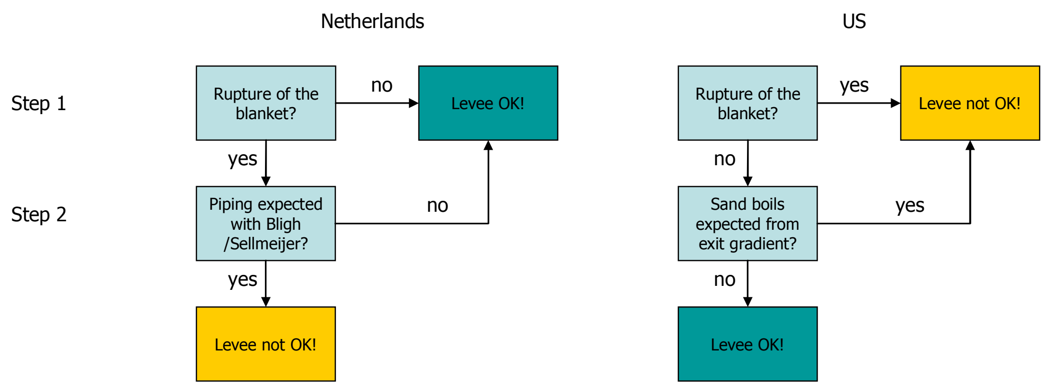


Figure 7.2 Dutch vs. US vulnerability to sand boils or piping

Step 2:

In the second steps there are even more differences. In the Netherlands, when there is no indication that the blanket is going to rupture, the levee is regarded safe against piping. Often no further calculations are done. The case where the blanket has already ruptured before, or is damaged for whatever reason, is denied here. The situations uplift and actual sand transport are separated, while is assumed that sand boils not always occur after the blanket has ruptured. If sand transport actually takes place and forms a threat to the levee stability is related to the head difference on the levee and is predicted with Bligh and/or Sellmeijer.

Table 7.2 Step 2 in formulas

Dutch	**US**
The Bligh formula (4-5) can be rewritten to: $$(H-0.3z_t)\le H_c=\frac{L}{C_{creep}} \qquad (7\text{-}6)$$ The Sellmeijer criterion can be rewritten as: $$(H-0.3z_t)\le \frac{1}{\gamma}H_c \qquad (7\text{-}7)$$ A safety factor γ of 1.2 is usually applied combined with characteristic values for the parameters. 0.3 is assumed to be a mean value as explained in chapter 4. Or both rewritten to a H_{crit}: $$H_{crit,Bligh}=H_c+0.3z_t \qquad (7\text{-}8)$$ with $H_{crit}=F(L, d_{50}, z_t)$ $$H_{crit,Sellmeijer}=\frac{1}{\gamma}H_c+0.3z_t \qquad (7\text{-}9)$$ with $H_{crit}=F(L, d_{70}, z_t, d, k_s, \gamma_{wet}, \gamma_w, \eta, v, g, \theta)$ d_{70}, η, v and θ are particle characteristics which influence particle movement. The η, v, and θ are assumed constant.	The criterion in step 2 is still (formula 5-1): $$i=\frac{h_0}{z_t}\le 0.5 \qquad (7\text{-}10)$$ The 0.5 was chosen with a safety of 1.6, not because rupture is still expected at 0.5, but because then also boils at locations where a seepage exit resulted from earlier floods or for example poorly back-filled bore holes can be detected with the criterion. The formula then becomes the same as the Dutch heave criterion for sandy materials. For case 7 (Appendix 2) with $$h_0=\frac{Hx_3}{(x_1+L_2+x_3)} \qquad (7\text{-}11)$$ This formula can be rewritten to: $$H_{crit}=\frac{0.5z_t(x_1+L_2+x_3)}{x_3} \qquad (7\text{-}12)$$ Where x_1 and x_3 $=F(k_{bl}, k_s, z_b, d, L_1, L_3)$ Thus $H_{crit}=F(L_1, L_2, L_3, k_{bl}, k_s, z_b, z_t, d)$

In the US the critical gradient of 0.5 will 'catch' all cases where the blanket is expected to rupture. When the blanket has already cracked or when there is no blanket the critical exit gradient of 0.5 is a heave criterion for the sand particles, which is also used in the Netherlands in absence of an impermeable blanket. (USACE, 2005)

In the second part of the calculation the US and Dutch formulas and criteria differ much more. Now the formulas as well as the mechanism differ. To start with the formulas: the Dutch critical head depends on particle characteristics, where the US critical head depends on groundwater flow characteristics. The US method is based on a critical exit gradient, a heave criterion, which indicates when the effective stresses are zero. The Dutch method is based on the transport of the sand particles in a horizontal slit, using a critical seepage length.

Table 7.3 Criterion for levee rejection/approval

Dutch opinion	**Uplift ($H>H_{c,uplift}$)**	**No uplift ($H\leq H_{c,uplift}$)**
Piping ($H>H_{crit,Bligh,Sellm.}$)	Rejected	Approved
No piping ($H\leq H_{crit,Bligh,Sellm}$)	Approved	Approved
US opinion	**Uplift ($i>\approx 0.7$)**	**No uplift ($i\leq\approx 0.7$)**
Sand boils ($i>0.5$)	Rejected	Rejected
No sand boils ($i\leq 0.5$)	Rejected	Approved

The different conclusions at the end of step 1 and step 2 are summarized in Table 7.3. An interesting conclusion is that in the Netherlands a levee is approved more often than is the case in the US, where levees are easily rejected. To get an idea of the band width between Dutch rejection/approval and US rejection/approval it is important to look at the differences in safety and try to show some examples, which is done in the next two paragraphs.

7.2 Safety

As already explained in the previous paragraph, not only the method, but also the applied safety is important to make a realistic model of reality and to cope with uncertainties. To compare the previously mentioned evaluation methods and especially the outcomes, a discussion and explanation of applied safety factors and parameter choice is inevitable.

How was dealt with safety in the 1956 research?

There is an interesting difference in how was dealt with safety in the 1956 research and how that is currently done in the Netherlands. The current practice in the US is different, but in the 1956 research was worked with 'best guess' from measured parameters and an overall safety of 1.6 on the theoretical critical gradient for uplift of 0.8.

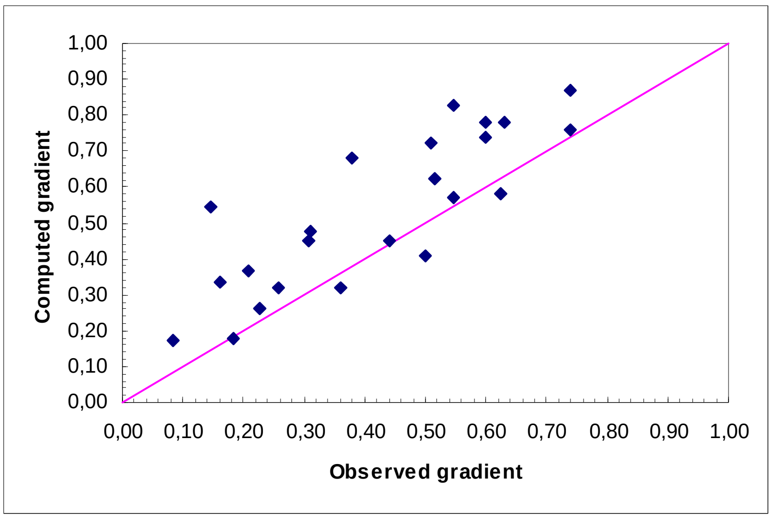


Figure 7.3 Correlation between the computed gradient and observed gradient

The 1956 WES research came up with observed values for the hydrostatic head at piezometer locations landward of the levees, for the 1950 high water and/or occasionally for other years. For each of the 16 sites that were studied about two cross-sections were developed from borings and lab tests, at locations of piezometer lines. For this research locations were chosen of which was quite certain that piezometer locations at the toe of the levee or in a landside ditch matched the developed cross-sections. Using blanket formula's and the best guess values from the 1956 research a re-calculation was made, to see if these formula's would give the same results.

The results are displayed in the chart above. A positive correlation of 0.83 was found between the measurements and calculations, which is satisfying. But it should be noted that sometimes blanket formulas were used to estimate unknown parameters. The re-calculations of these cases naturally matched better to the measurements. Unfortunately it is not exactly known for which parameters and cases this is true. A conclusion that can be drawn from Figure 7.3 is that the best guess parameters and formulas used for the calculations are not mean values, but are slightly conservative parameters. These values led to a safe result: no locations were regarded as safe from the calculations (<0.5), when the observed gradient gave indications that the situation was not safe (>0.5).

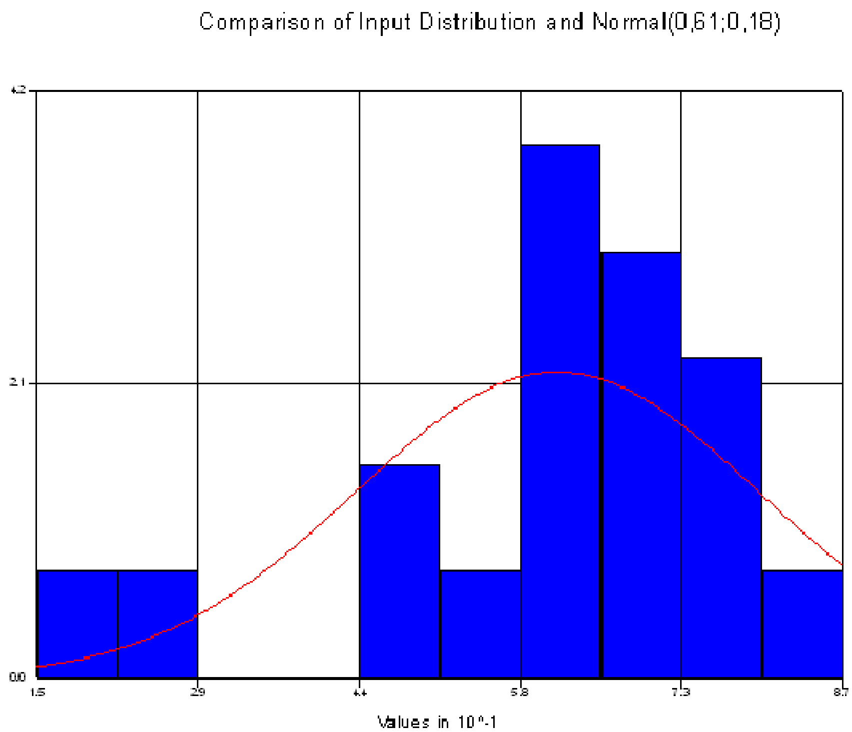


Figure 7.4 Observed gradients at sand boil locations fitted with a normal distribution

The critical exit gradient of 0.5 was based on the sand boil locations from the 1956 research. But does this 0.5 really give such a safe situation? When a normal distribution is fitted on the 1950 sand boil points of the research a mean exit gradient of 0.61 was found with a standard deviation of 0.18 (Figure 7.4). 0.5 is then 0.6 times the standard deviation from the mean. Which means that when 0.5 is chosen as a critical point about 28% of the sand boils have a lower exit gradient than 0.5 and would not be detected. Does this mean that 0.5 is not a safe criterion? Probably not. An explanation for the low exit gradients at sand boil locations, which is mentioned in USACE documents, is that part of the sand boil points were not first time boils or were located at a bore hole or another weakness in the blanket. The measured hydraulic head at those points is much lower than would be expected from a calculation. An estimation of the exit gradient with blanket equations often leads to a higher exit gradient for this point, see example 2 in next paragraph.

How is currently dealt with safety?

USACE practice nowadays is to use the 'one third/two thirds rule', which means that the design strength is chosen such that it is less than two thirds of the measured values (USACE, 2006). The safety factor of 1.6 on the relatively constant assumed relative weight of the blanket soil is still used, while 0.5 is still used as a criterion.

In the Netherlands a safety factor of 1.2 is currently used in the formulas for uplift and piping and characteristic values for the strength. A 5% (or 95%) value is prescribed, which is at about

1.64 times the standard deviation (σ) from the mean (μ) in a normal distributions. How to get these values depends on engineering insight and is not always straight forward. Before the Dutch used this characteristic value a safety of about 1.5 to 2 combined with mean values. Both characteristic values are drawn in Figure 7.5.

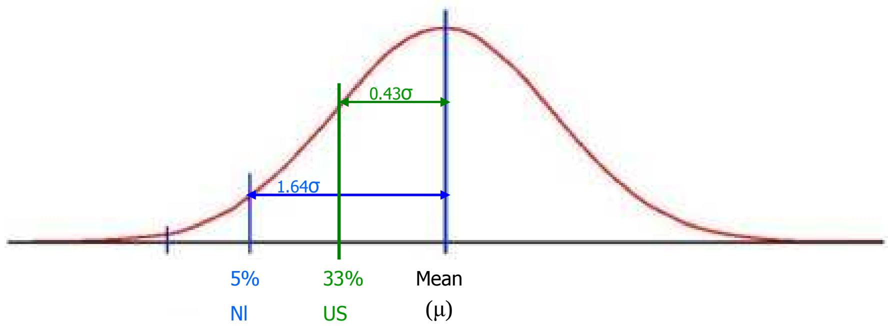


Figure 7.5 The US 33% percentile vs. the Dutch 5% percentile, for a safe parameter choice

But what effect does the above chosen safety factor have on the final parameter choice? What is then the approximate difference between the US and the Netherlands? To quantify this we can use the coefficient of variation, which is a measure of the dispersion of a distribution:

$$C_v = \frac{\sigma}{\mu}$$

With:
C_v= Variation coefficient [-]
μ= Mean [depends on parameter]
σ = Standard deviation [depends on parameter]

C_v=0.1-0.25 is rather normal in levee evaluation practice. We can use 0.1 and 0.25 to give an indication of the difference in parameter choice between the US and the Netherlands and insert it in formulas to give an indication of the difference in overall safety. In Table 7.4 the differences are quantified. The conclusion is that the lower characteristic value of the Netherlands is on average about 0.8 times the US value. In other words: the difference between the Dutch and US parameter choice is 20%. The same can be done for the high characteristic value, then a difference Netherlands divided by US is about 1.2. For for example the permeability of the sand layer the high characteristic value is chosen, because a high permeability is less safe. But in the 1956 research mean values were probably used. A difference between the 5% value and the mean value is a factor of about 0.7.

Table 7.4 The estimated effect of the difference between US and Dutch parameter choice

$C_v = \sigma / \mu$ [-]	$X_{5\%}=\mu-1.64\ \sigma$ (NL)	$X_{33\%}=\mu-0.43\ \sigma$ (US)	Difference NL/US
0.1	0.84μ	0.96μ	0.88
0.25	0.59μ	0.89μ	0.66
Mean difference μ and $X_{5\%}$: 0.72 (≈0.7)		Mean difference NL/US: 0.77 (≈0.8)	

Figure 7.6 is an illustration of step one of the seepage evaluation. It shows the safe areas for both methods. The assumption of a relatively constant volume weight of the blanket limits the area of application of the sand boil criterion. The exit gradient of 0.5 is only valid above a volume weight of the blanket above 17.6 kN/m^3 (right of the black line). In areas with less heavy clay or with relatively impermeable peat, which could even have a weight below 10 kN/m^3, the US criterion is therefore not valid! When applying the 0.5 criterion this would result in very unsafe situations. The blue line in Figure 7.6 is equation 7-5, when varying the submerged weight of the blanket. For a volume weight of 18 kN/m^3 the factor of safety is 1.6. When keeping the exit gradient at a constant value of 0.5 this will result in a safety of less than one, which is the case for blanket soils with a volume weight less than 15 kN/m^3, which is not inappropriate. This means that the uplifting pressures will exceed the weight of the blanket (while the safety factor is the

resisting forces divided by the inducing forces). The Dutch safe are is represented with the green area.

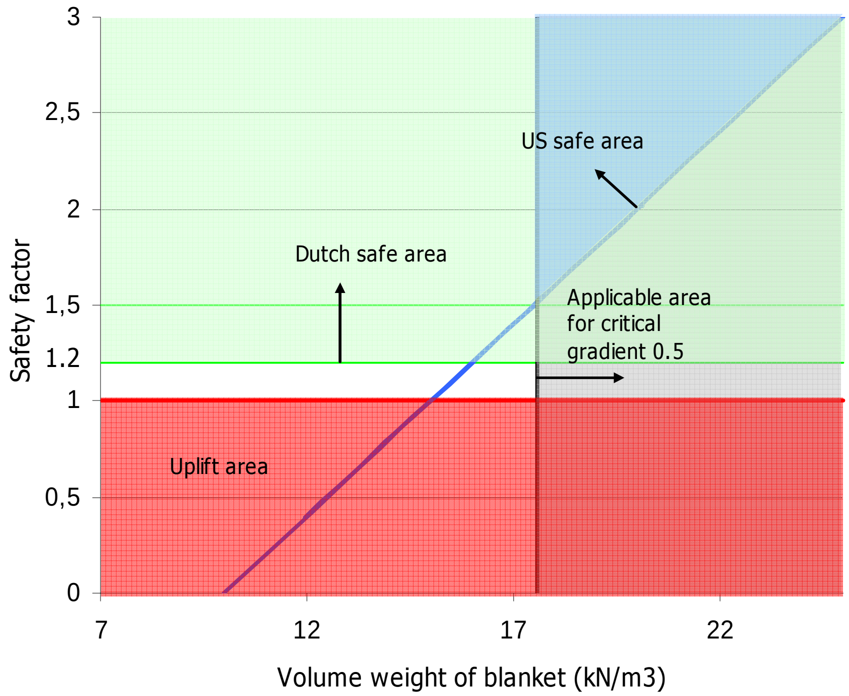


Figure 7.6 Comparison of uplift safety Netherlands and US

7.3 Mississippi cases vs. US and Dutch rules

Earlier in this chapter we concluded that there is a large difference between the US and Dutch criterion. To quantify the band width between those criteria the 1950 Mississippi cases, on which the US criterion is founded, can be used.

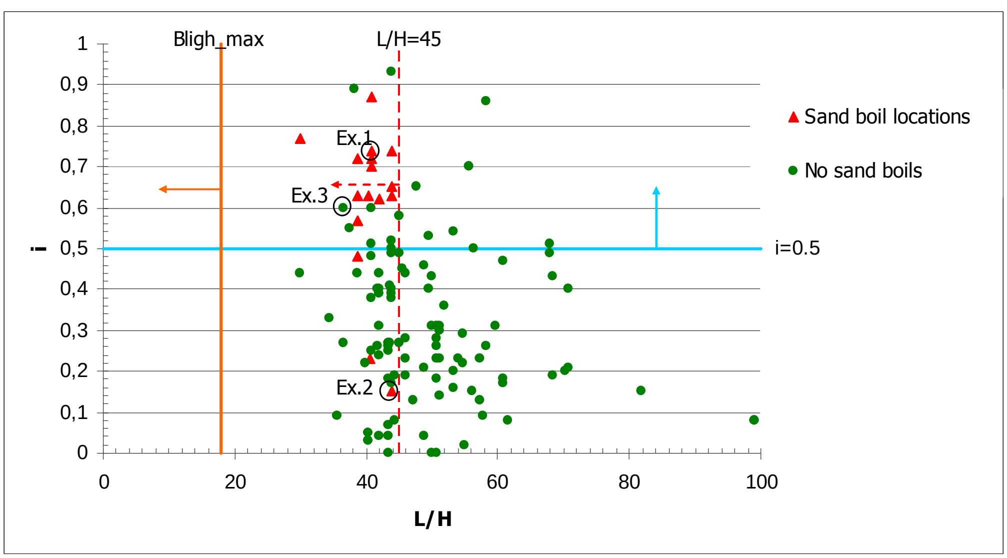


Figure 7.7 Observed L/H in Mississippi River cases from 1950

In Figure 7.7 all sand boil points (red triangles) and points where no sand boils were observed (green dots) from the Mississippi research are displayed. It shows the exit gradient on the vertical axis and the seepage length divided by the head difference on the levee on the horizontal axis (see 7.1.2). The US criterion (i=0.5) is drawn with the blue line. The arrow points in the direction of the area where the levee would be rejected. The maximum using Bligh is drawn with the orange line, again with the arrow directing to the unsafe area.

It is interesting to see that the red points are all safe according to Bligh, while in the US these points are non-preferred situations. Bligh found an empirical relation between the seepage length and the critical head of maximum 18. This is said to be a conservative value. Sellmeijer is sometimes somewhat less conservative up to an L/H of about 24, but within the Dutch rules Bligh is then chosen as being normative. When the minimum seepage length that was available at the Mississippi sites, which was often only the base width of the levee or the base width and the berm length together, boils already start to develop at values of L/H of approximately 43. The L and H of all these cases were observed values along the Mississippi. While these parameters can be measured very well, no difference between the measured values and characteristic values is assumed.

To quantify the differences between the US and Dutch interpretation three cases from Figure 7.7 are examined further on the following pages. But some assumptions have to be made to be able to compare them. While the distribution of the parameters used in the 1956 research is not known a relation between the mean values of the 1956 research and the desired characteristic values has to be made. This can be done by using the assumptions from previous paragraph. A factor of 0.7 of the 5% characteristic value divided by the mean value was found. To translate this a total safety γ_{tot} is applied instead of a safety γ within the formula combined with a safety hidden in the parameter choice γ_p. This results in the allowed approximate total safety factors as displayed in Table 7.5. However these are very rough estimates! A further study to give a better definition of the characteristic values would lead to more reliable results. The 3 examples are therefore just illustrative examples!

Table 7.5 The total safety factor to translate the Mississippi cases to Dutch criteria

Method	Safety factor γ	Mean factor on parameter safety γ_p ($X_{5\%}/\mu$)	Total safety factor γ_{tot}
Uplift	1.2	0.7	1.5 – 2.0
Bligh	1	0.7	1.2 – 1.6
Sellmeijer	1.2	0.7	1.5 – 2.0

The three Mississippi cases on the following pages, circled in Figure 7.7, show that the Dutch evaluation methods would give other conclusions than the US methods and/or observations. The first two examples are levees with a borrow pit where most of the blanket was excavated. Water can enter the pervious substratum through these borrow pits. After serious problems in 1937 a seepage berm was constructed at the Trotters 51 location. This berm reduced the sand boil problems at this location, but in 1950 still undesirable sand boils were observed at this location. The piezometer data indicates that the observed gradient was larger than the exit gradient, but according to the Dutch criteria nothing is wrong. The difference with example 2 is that the piezometer data indicated no problems, as well as the Dutch formulas, while a calculation with blanket formulas indicate a vulnerability to sand boils. Sand boils were observed. The last case is a bit different. This situation, the most downstream location of the Mississippi research is the most similar to the Dutch situation, with relatively fine sand and a permeability of about $5x10^{-4}$ m/s. Although there is a riverside borrow pit, the blanket in this borrow pit is thick enough to prevent water from entering the sand layer. No sand boils were found at this specific location and problems in this are were only minor. The piezometer data indicates a vulnerability to sand boils, while with the Dutch methods is concluded that piping does not form a threat for this levee.

These examples confirm that levees are rejected more often with the US criterion than with the Dutch criteria, with the assumptions as mentioned above.

Example 1: Trotters 51, Mississippi USA

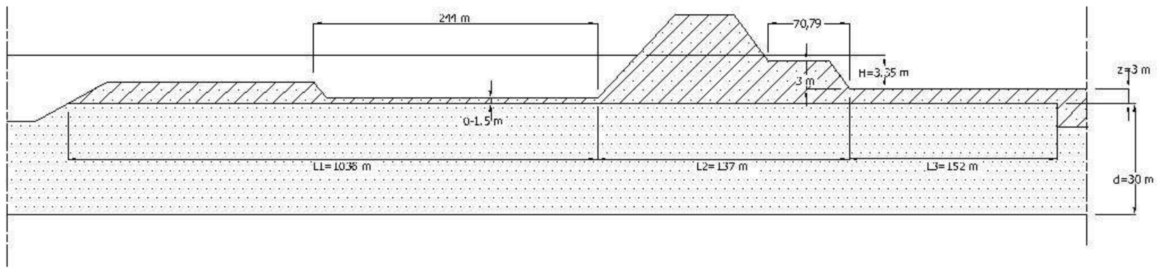


Figure 7.8 Cross-section Trotters 51 levee representative for station 50/36+50

Parameters:
z_t=z_b= 3.05 m
d=30 m
L_1= 1036 m
L_2= 137 m
L_3= 152 m to seepage block
k_{bl}=0.5x10^{-6} m/s
k_s=10x10^{-4} m/s

Measured 1950:
H = 3.35 m
h_0 = 2.25 m

Additional estimated parameters
L=L_2=137 m
γ_w=10 kN/m^3
$\gamma_{w,s}$=18 kN/m^3
d_{50}=0.4 mm (fig.17 Appendix 1)
d_{70}=0.44 mm
C_{creep}=12

Dutch interpretation:

Uplift (eq. 7-3)	$F_{up} < \frac{1}{\gamma_{tot}} F_{down}$	γ_{tot} from calculation smaller than allowed γ_{tot} from Table 7.5?
With h_0=2.25 m from piezometers	$22.5kN/m^3 < \frac{1}{\gamma_{tot}} 24.4kN/m^3$	1.08 < 1.5-2; not safe!
Bligh (eq. 7-6)	$H - 0.3z_t < \frac{1}{\gamma_{tot}} H_{crit} \left(= \frac{L_2}{C_{creep}} \right)$	
	$2.44m < \frac{1}{\gamma_{tot}} 11.42m$	3.68 > 1.2-1.6; safe
Sellmeijer (eq. 7-7)	$H - 0.3z_t < \frac{1}{\gamma_{tot}} H_{crit}$	
	$2.44m < \frac{1}{\gamma_{tot}} 12.49m$	5.12 >> 1.5-2; safe
Conclusion:		**Safe**

US interpretation:

Exit gradient at toe (eq. 7-4)	$i < i_c$	
With h_0 from piezometers	0.74 > 0.5	Not safe
Conclusion		**Not Safe**

Damage 1950, at H=3.35 m:

8 sand boils of 10-20 cm in diameter observed between station 50/5 and 50/40, which is about 1 km.

Example 2: Lower Francis, Mississippi, USA

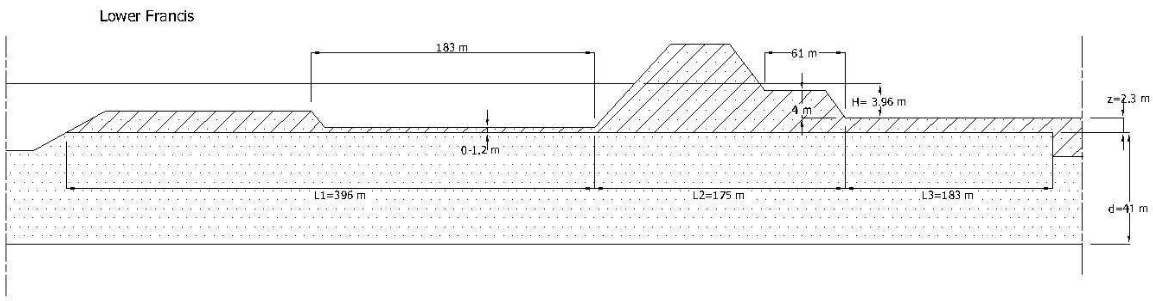


Figure 7.9 Cross-section Lower Francis levee representative for station 145

Parameters:		Additional estimated parameters
z_t=z_b = 2.29 m	measured 1950:	L=L_2=175 m
d=41 m	H = 3.96 m	γ_w=10 kN/m^3
L_1= 396 m	h_0 = 0.33 m	$\gamma_{w,s}$=18 kN/m^3
L_2= 175 m		d_{50}=0.4 mm (fig. 17 Appendix 1)
L_3= 183 m to seepage block		d_{70}=0.56 mm
k_{bl}=0.1x10^{-4} m/s		C_{creep}=12
k_s=16x10^{-4} m/s		

Dutch interpretation:

Uplift (eq. 7-3)	$F_{up} < \frac{1}{\gamma_{tot}} F_{down}$	γ_{tot} from calculation smaller than allowed γ_{tot} from Table 7.5?
With h_0 from piezometers	$3.3 kN/m^3 < \frac{1}{\gamma_{tot}} 18.32 kN/m^3$	5.55 >> 1.5-2; Safe
With h_0 from geohydr. mod.	$16.4 kN/m^3 < \frac{1}{\gamma_{tot}} 18.32 kN/m^3$	1.12 < 1.5-2; Not safe
Bligh (eq. 7-6)	$H - 0.3 z_t < \frac{1}{\gamma_{tot}} H_{crit} \left(= \frac{L_2}{C_{creep}} \right)$	
	$3.27 m < \frac{1}{\gamma_{tot}} 14.58 m$	4.46 > 1.2-1.6; Safe
Sellmeijer (eq. 7-7)	$H - 0.3 z_t < \frac{1}{\gamma_{tot}} H_{crit}$	
	$3.37 m < \frac{1}{\gamma_{tot}} 15.75 m$	4.67 > 1.5-2; Safe
Conclusion:		**Safe**

US interpretation:

Exit gradient at toe (eq. 7-4)	i < i_c	
With h_0 from piezometers	0.14 < 0.5	Safe
With h_0 from geohydr. mod.	0.71 > 0.5	Not Safe
Conclusion		**Not Safe**

Damage 1950, at H=3.96 m:

Medium to heavy under-seepage and numerous sand boils from station 141 to 147 (9.6 km stretch). Some boils discharged as much as 1 cu yd of sand (0.76 m^3).

Example 3: Baton Rouge, Mississippi USA

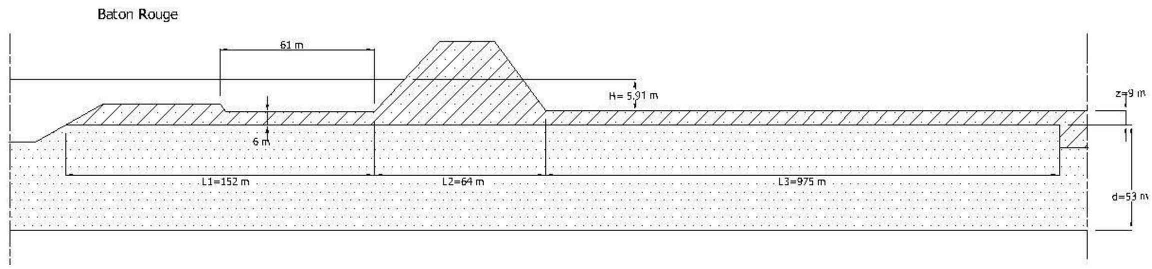


Figure 7.10 Cross-section Baton Rouge levee representative for station 79-106

Parameters:

z_t= 7.62 m
z_b = 9.14 m
d=53 m
L_1= 152 m
L_2= 64 m
L_3= 975 m to seepage block
k_{bl}=6x10^{-8} m/s (From blanket formulas and piezometric data)
k_s=5x10^{-4} m/s (Mean of fig.17 and lab tests, Appendix 1)

Additional estimated parameters
$L=L_1+L_2$=216 m
γ_w=10 kN/m^3
$\gamma_{w,s}$=18 kN/m^3
d_{50}=0.2 mm (fig. 17 Appendix 1)
d_{70}=0.3 mm
C_{creep}=15

Measured 1950:
H = 5.91 m
h_0 = 4.57 m

<u>Dutch interpretation</u>:

Uplift (eq. 7-3)

$$F_{up} < \frac{1}{\gamma_{tot}} F_{down}$$

γ_{tot} from calculation smaller than allowed γ_{tot} from Table 7.5?

With h_0 from piezometers

$$45.7kN/m^3 < \frac{1}{\gamma_{tot}} 60.96kN/m^3$$

1.33 < 1.5-2; not safe

Bligh (eq. 7-6)

$$H - 0.3z_t < \frac{1}{\gamma_{tot}} H_{crit} \left(= \frac{L_2}{C_{creep}} \right)$$

$$3.62m < \frac{1}{\gamma_{tot}} 14.40m$$

3.98 > 1.2-1.6; safe

Sellmeijer (eq. 7-7)

$$H - 0.3z_t < \frac{1}{\gamma_{tot}} H_{crit}$$

$$3.62m < \frac{1}{\gamma_{tot}} 14.71m$$

4.06 > 1.5-2; safe

Conclusion: **Safe**

<u>US interpretation</u>:

Exit gradient at toe (eq. 7-4)	$i < i_c$	
With h_0 from piezometers	0.60 > 0.5	Not Safe
Conclusion		**Not Safe**

<u>Damage 1950</u>, at H=5.9 m:

Four sand boils, comparatively small; no sand boils at location of measurements!

7.4 Possible sources for differences

But why is there such a large difference in the Dutch and US critical water level for piping? And how does this affect the applicability of the different methods? Possible sources for differences are discussed in this paragraph.

1) Situation which is assumed critical is different

This is a very logical explanation. In the US sand boils are not accepted. The criterion of a critical exit gradient of 0.5 was based on when sand boils started to form, how big or small they were was not discussed. This is different from the Dutch criterion, where small sand boils are accepted and the critical situation is much closer to failure. The Bligh criterion is based on dam failures and the Sellmeijer formula is a tool to find the water level at which a pipe, which already started to form, is starting to grow rapidly. The critical water level for the critical exit gradient and for Bligh or Sellmeijer is illustrated in Figure 7.11. On the horizontal axis we have the length of the pipe divided by the total seepage length. The head difference on the levee is drawn on the vertical axis. At a certain critical water level boils are observed, which is an indication that a pipe starts to form. This is the critical situation in the US. With rising water level the pipe grows and I/L is therefore increasing, but the situation is stable (line a in Figure 7.11). From a certain critical pipe length, the Dutch critical head difference (red line) the situation will become instable and the length of the pipe starts to grow explosively until it is equal to the total seepage length and the levee will collapse (line b).

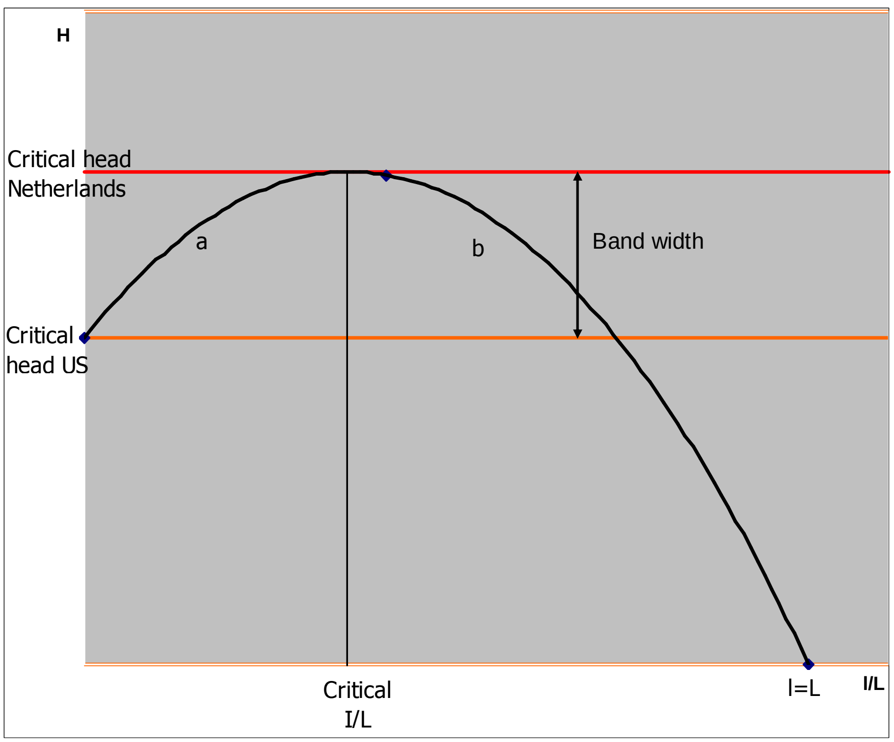


Figure 7.11 Difference critical situation US and Netherlands (modified from TAW, 1999_2)

While no Mississippi levees actually failed in 1950 the Bligh and Sellmeijer criterion cannot be compared with those Mississippi cases to indicate if Bligh and Sellmeijer would detect really critical situation. But there is limited documentation of 1937 problems available, which was the reason to start the under-seepage research and construction of seepage berms at critical boil locations. Some of these locations really were critical: levee banquets settled and pipes formed already over a length of almost 60m! These situations are therefore also implemented, resulting in Figure 7.12.

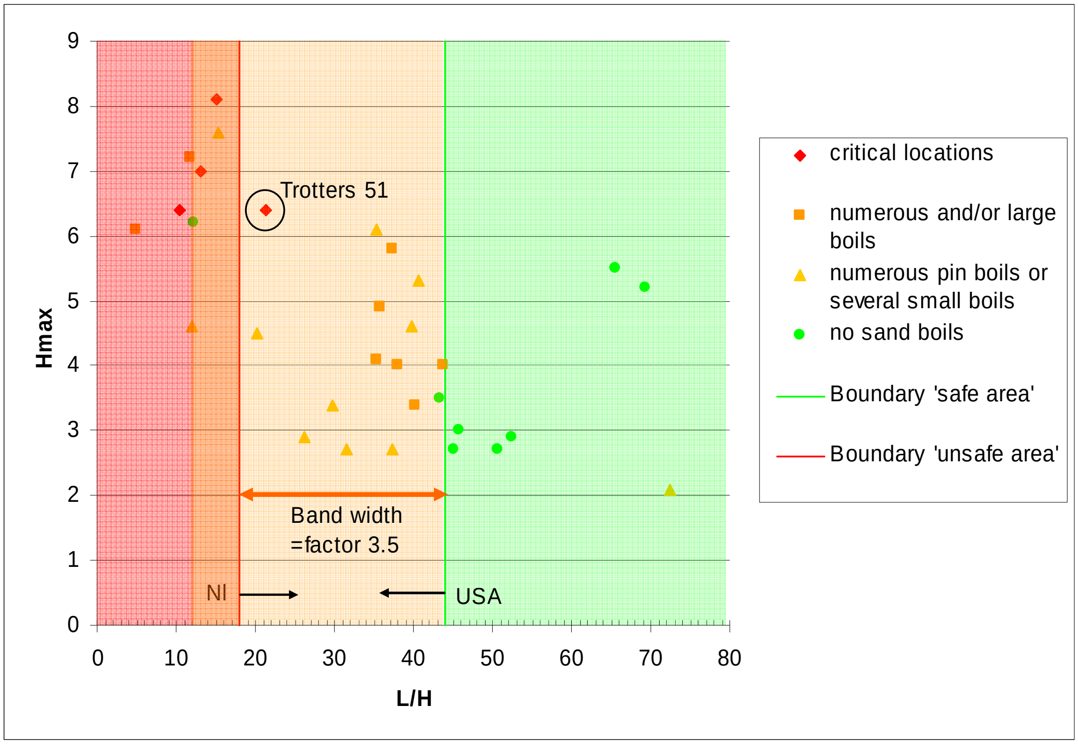


Figure 7.12 Critical boil locations from 1937 combined with L/H estimates and observations from other locations in 1937, 1945 and 1950 (data from USACE, 1956)

The red dots in Figure 7.12 represent critical piping locations of 1937. At these locations the levee (almost) failed. In the 1956 Army Corps document the history of under-seepage of the locations before 1950 was described (USACE, 1956). At Trotters 51, the circled point in Figure 7.12, a boil that occurred about 60 m from the levee toe discharged considerable material as it moved across a road, causing it to cave into a depth of 5 m to within 7 m of the levee toe. The levee did not fail, but it is clear that the water level at which the pipe explosively started to grow was already reached. Approximately 160,000 sacks and up to 500 men were necessary to construct sublevees around this boil and around other large boils in the area. While other boils continued to break out beyond the limits of these sack levees, finally one large-sized sublevee was constructed. Another example of where the critical point that Bligh and Sellmeijer try to find is reached is at Farrell, where the levee banquette settled in the vicinity of 11 large boils (within 30 m).

Areas which are assumed safe and not safe are colored in Figure 7.12. For a levee to be rejected, the L/H-points at least have to be situated left of the red line, in the dark orange or red area! This red line bounds the Bligh maximum of 18. For locations along the Mississippi a maximum L/H of 12-15 should be closer to reality, because of the soil characteristics. The difference between 12 and 18 is the dark yellow area. But the Trotters 51 point is clearly situated outside this area. Bligh would therefore not have rejected that levee! And Sellmeijer, if it rejected the levee would not have been normative, because above a value of L/H of 18 Bligh is assumed to be normative, although Sellmeijer can become up to about 24.

The Trotters 51 point is quantified in Table 7.6. These calculations are based on the information that was available and partly on reasonable estimates also used in example 1 in 7.3. A calculation of the expected exit gradient is not made here, because the exit gradient is very sensitive to the blanket thickness. The blanket thickness at this location is known to be very irregular. But a qualitative estimation is possible. When exit gradients in 1950 exceeded 0.5, one can reasonable expect that exit gradients of >>0.5 would be found in 1937 at this location, because of the larger water level (6.4 in 1937 and 3.35 in 1937) and the construction of a berm after the 1937 to improve the vulnerability against piping. As can be seen in the table Bligh wouldn't have traced this point and Sellmeijer also not, although values are getting close to a critical point. Better knowledge of the parameters would maybe lead to rejection with Sellmeijer.

Table 7.6 Quantifying Trotters 51 critical situation

Location	Trotters 51
Observed problems 1937	Sand boil, discharged considerable material; pipe started 60 m from toe of levee and moved across road towards levee for about 53 m
Observed water level 1937	6.4 m
Observed seepage length	76 m levee + 60 m behind levee = 136 m
L/H	21 > 18 and far larger than 12-15 which is applicable for this case; Conclusion: safe according to this criterion
Bligh $H - 0.3z_t < \frac{1}{\gamma_{tot}} H_{crit} \left(= \frac{L_2}{C_{creep}} \right)$	5.49 < 11.33; with a total safety of 2.06 > 1.2-1.6 allowed; conclusion: safe situation expected!
$H_{c,Sellmeijer}$ $H - 0.3z_t < \frac{1}{\gamma_{tot}} H_{crit}$	H_{crit} ≈13 m; a total safety of 2.36 > 1.5-2 . Sellmeijers is getting close to a critical situation. A really safe situation is not guaranteed.

Concluding: A very important and logical explanation of the difference in denying/approving a levee lies within the fact that sand boils are not accepted in the US, while in the Netherlands sand boils are accepted until the point where the levee is pipe starts to grow explosively. But when looking at really critical situations along the Mississippi, of which only limited data is available, it seems that Bligh as well as Sellmeijer would not expect problems, while in fact a very critical situation was present! A more detailed investigation is necessary to prove if Sellmeijer and Bligh are not sufficient for the above locations. The following explanations are therefore also focussed on why Sellmeijer and Bligh do not detect these locations as critical.

2) Conditions are different; are Sellmeijer and Bligh valid?

Conditions that were found:

The conditions for the Mississippi River levees, related to underseepage, are different from the conditions at Dutch levees. This was already shown in the different levee cross-sections in Chapters 4 and 5. To quantify these differences in Table 7.7 conditions from Mississippi cases used before and of levees along the Dutch rivers are summarized.

Table 7.7 Comparison (seepage) conditions Mississippi / Dutch river levees

Location	Trotters 51 Mississippi, USA (USACE, 1956)	Baton Rouge, Mississippi, USA (USACE, 1956)	Netherlands (example from TAW, 1999-2)	Netherlands; Island of Dordrecht (Fugro, 2004)
Position from coast	850 km	200 km	>150 km	50 km
L [m]	137 m	216 m	39 m	30-80 m
Ditch? Distance from toe?	no	no	no	Yes; 0-10 m from toe
d_{sand} [m]	30 m	53 m	40 m	1-5 m
z_t [m]	3 m	7.6 m	2,8 m	1-6 m
k_{bl} [m/s]	$5x10^{-7}$ m/s	$6x10^{-8}$ m/s	$1x10^{-7}$ m/s	-
K_s [m/s]	$10x10^{-4}$ m/s	$5x10^{-4}$ m/s	$9x10^{-4}$ m/s	$1x10^{-5}$m/s
d_{50} [mm] (mean)	0.4 mm	0.2 mm	0.17 mm	-
d_{70} [mm]	0.44 mm (mean)	0.3 mm (mean)	0.2 mm (char.)	0.1 mm (char.)
H [m] (project flood)	7.83 m	7 m	3.35 m	3-5 m
γ_b [kN/m^3]	-	-	17 kN/m^3	15 kN/m^3
C_{creep} [-]	12	15	17	17

While in the Netherlands characteristic values are used and in the Mississippi research expected values, the values in the table shouldn't be taken too strict. For the d_{70} values is mentioned if it is a mean or characteristic value. What can be seen from this table is that the Mississippi River cases are more upstream than any case in the Netherlands would be. Baton Rouge is already further from the coast than a location at the border with Germany would be along the Rhine. Differences in permeability, grain size and layer thicknesses seem to be related to the location, upstream of downstream. These differences could lead to not fitting of certain formulas.

What is mentioned in the 1956 document is that the sand layer often seems to consist of two layers: fine sand in the upper part and courser sand below. This situation is also often encountered in the Netherlands, for example at the Eems Canal levees, which were used as a case study in Chapter 4. For a situation with 2 sand layers with different permeability and grain size more advanced evaluation methods are used in the Netherlands, while just Bligh and Sellmeijer are not sufficient. In the Eems Canal levee evaluation the permeability of the total sand layer was estimated with MSeep, while another method to evaluate piping of 2-layer sand is developed by GeoDelft and has not been tested to real cases yet. Using the latter two methods on the Mississippi cases would be interesting.

Limitations of models/equations:

- Of the critical exit gradient of 0.5 is known that it is only valid when the blanket has a volume weight above 17.6 kN/m^3. For the Dutch cases in the above table this criterion can therefore not be applied. It also raises questions about the applicability at locations along the Mississippi and in the Central Valley. A variable safety factor, as mentioned in previous paragraphs, could offer a (temporary) solution.
- Bligh is used all over the world. In documentation in the Netherlands where the formula is mentioned no restrictions are added. The question of the applicability along the Mississippi River can therefore not be answered directly here. It is important though, to present the limitations of the equation together with the equation.
- Sellmeijer was developed in the Netherlands and is, at this moment, only used in the Netherlands. As with the Bligh formula, no restrictions are presented together with the formula, although they could be expected, because the model is partly fitted on model tests.

3) Design criterion of evaluation criterion?

The Bligh and Sellmeijer criteria are both used for design and evaluation. The US critical exit gradient was meant for design purpose and is now also applied for evaluation. Why are those criteria the same? It would be more convenient to put the design point at the boundary where sand boils start to form and evaluation where levees start to fail. If Sellmeijer and Bligh would be chosen as an evaluation criterion and the L/H=43 as a design value, what would then be the consequences? From Figure 7.12 a factor of approximately 3.5 between critical water level for sand boils according to the Mississippi cases and for failure according to the Dutch criteria can be found (43/18=3.5). Applying this L/H=43 instead of L/H=18 would result in a far larger berm. For example a levee with an H of 3 m, a width of 40 m a berm of 15 m is sufficient for L/H=18, but for L/H=43 that berm needs to be extended to 90 m! It is questionable if this is possible and maybe not too much, also because the room for that is not available, villages will have to be removed?!

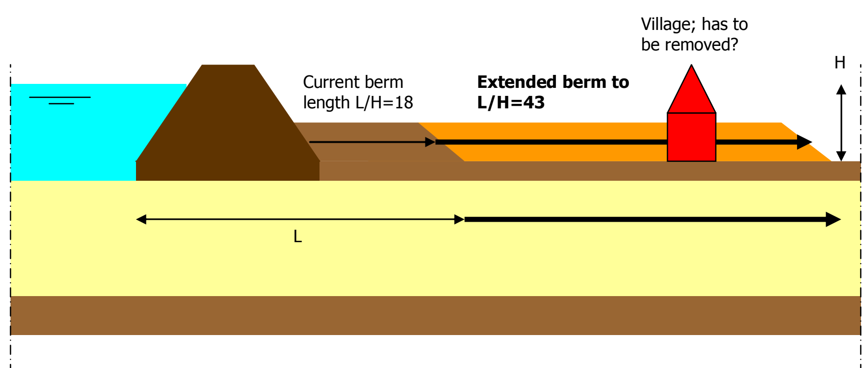


Figure 7.13 Result of using L/H=43 for design instead of L/H=18

Concluding: it is a reasonable explanation that the US criterion, the occurrence of sand boils, would be satisfying as a design point and the Dutch criteria, levee failure, as an evaluation point. But the Dutch and US criteria are used for design and evaluation. A critical review of what criterion is necessary for design and what criterion for evaluation could be interesting. It is at least recommended for as well the Dutch as the US method to do some further research and establish different criteria for design and evaluation. But if applying those criteria is possible in practice is always questionable.

4) Or other possible sources?

There could be more sources for the differences. Some examples:

- Measurements are not reliable; need to be verified from other background documents, which were not available in this research.
- Observed mechanism is something else than piping. Sink holes and piping are sometimes mixed up. But from the description of for example the Trotters 51 site it seems clear that sand boils and thus piping was the case.

8 Conclusions and Recommendations

From all gathered information in previous chapters and the comparison between the US and Dutch methods conclusions can be drawn on which subjects are interesting to exchange between the Netherlands and the US. The most important conclusions are summarized in 8.1. Recommendations for exchange of knowledge, on current levee evaluation practice and for future research are given in 8.2.

8.1 Conclusions

8.1.1 Conclusions comparison water defense systems Netherlands / California Central Valley

- The Central Valley water defense system and the Dutch water defense system have things in common; for example their flatness, areas below sea level, peat subsoils and vulnerability to flooding from rivers as well as from a sea or bay, although there are also differences: the Dutch levees are not threatened by earthquakes, while the Central Valley levees are.
- The Delta in the Central Valley is, as well as the Netherlands, partly situated below sea level. But the economic consequences of a flood mainly lie outside the Delta, because of the vulnerability of the drinking and irrigation water facilities. Where the economic consequences of the Netherlands mainly lie in the area itself. Solutions for flood related problems could therefore be different.
- Flood insurance in the United States is closely related to levee evaluation. Levees in California should currently be able to resist a flood with a 1:100 probability of exceedance per year and in the near future 1:200 per year. 1:100 is also the limit for flood insurance: in areas with less than a 1:100 protection people are obliged to buy flood insurance. In the Netherlands the intended protection is much higher, up to a 1/10,000 water level. People cannot insure themselves against flood related damage.

The overall conclusion is that there are enough similarities to learn from the differences.

8.1.2 Conclusions comparison levee evaluation methods

- In the US design documents are used for the evaluation of levees, while there are no separate evaluation documents. In the Netherlands separate evaluation documents and even documents in which hydraulic boundary conditions are defined are available and prescribed for levee evaluation.
- The stability of levees is evaluated practically the same, with the same kind of software and methods. But an important difference is that rapid drawdown in the US is evaluated with partly undrained parameters, while in the Netherlands only drained parameters are used. Another difference Uplift was not included in the most recent

version of the DWR stability evaluation program in California, while it is modeled in other part of the US and in the Netherlands.

- Underseepage is evaluated very different in the US and the Netherlands. A critical exit gradient is used in the US, based on experience, while the formulas of Bligh, also from experience, and Sellmeijer, from theoretical background combined with model tests, are used in the Netherlands.
- Evaluation in the Netherlands starts with a rough ground model and rough methods. More detailed information from weak areas is combined with a more detailed model. In the US this stepwise evaluation is applied for the ground model, but not yet for evaluation models.

Overall conclusion: There are interesting differences. The difference between drained/undrained parameters in stability evaluation is a topic currently studied in the Netherlands. In this research was chosen to focus on piping evaluation, because of the importance of this mechanism in the Netherlands and the US and the difference in how this mechanism is modeled.

8.1.3 Conclusions piping evaluation methods

- The US criterion, a critical exit gradient, was based on a Mississippi underseepage research in the 1950s. This criterion differs from the Dutch methods, where an uplift evaluation is followed by an evaluation of the critical seepage length at which a pipe can grow explosively with the formula of Bligh or Sellmeijer.
- L/H values at which boils occur at the Mississippi River (L/H≈43) do not match the values of L/H at which problems are expected in the Netherlands with the current piping evaluation methods (L/H≈max.18), which is caused by a different definition of the critical situation:
    - Critical situation Netherlands: failure of levee because of excessive growth of pipe.
    - Critical situation US: occurrence of sand boils.
- But situations which were critical were not critical according to the Dutch method, which could be caused by
    - Different circumstances; the Mississippi locations are situated more upstream along the Mississippi than the Dutch levees are in the Netherlands. While not exactly is known to what conditions the Dutch methods are restricted misuse of the formulas could lead to problems.
    - The reliability of the data and its variability is not clear.
- Evaluation and design is not distinguished. The Dutch and US criteria are used for both design and evaluation:
    - The Dutch criteria are based on levee failure and would be appropriate for levee evaluation.
    - The US criterion is based on the existence of sand boils, which would be and appropriate criterion for design.

  The result of applying a criterion where boils start as a design criterion in the Netherlands (i=0.5 or L/H=43) would result in a berm that is far larger than the current berm. The seepage length will have to be 3.5 times the current seepage length. This would lead to practical problems.
- The US criterion: i=0.5 can not randomly be applied. The area of application is restricted to blankets with a volume weight above 17.6 kN/m.
- In the Netherlands a 20% more conservative value for the strength of the levee is chosen than in the US.

Overall conclusion: The discussion on how to best model piping in the Netherlands as well as the US is not solved yet. Cautiousness is recommended as well as further research.

8.2 Recommendations for further research

8.2.1 For now

- A critical note about the applicability of the critical exit gradient in seepage evaluation is necessary, to prevent unsafe situations, especially in Delta areas as in California. A constant factor of safety should be considered.
- Model restrictions for formulas as Bligh or Sellmeijer as well as the critical exit gradient of 0.5 should be researched, documented and mentioned together with the formulas to prevent misuse and misunderstandings.
- Separate evaluation documents, preferably consistent over the whole US would be an improvement and save time in levee evaluations, as well as modeling in steps, while with little information and a simple model the results equal or even better than an advanced model with little information.
- It should be wise to investigate if uplift is in the current DWR stability evaluation standard and if not to consider implementing it.

8.2.2 Future research:

- Recommended is to start a similar seepage investigation in the Central Valley and Delta in California, as was done along the Mississippi in the 1940s. These levees have more similarities with the Dutch levees, when comparing their dimensions. Also these levees are now said to be really vulnerable. A large research is started, but it will take some years until the levees are safe. Piping problems, as they dealt with in 1997, could be expected in the following years in that area and could deliver some interesting data, for the US as well as the Netherlands.
- There is more information available on the Mississippi research, for example maps and soil data. To get a complete and honest comparison a research on all those data would be interesting, preferably combined with data from other seepage researches all over the world. We could use the already available data from the US and other data to verify and improve the Dutch methods. No real proof is given in this report that the current criteria are not safe, but what it does proof is that caution is necessary and that piping is still not a completely solved problem.
- If (obligatory) flood insurance is considered in the Netherlands, it is important to include the influence of the aimed safety level on the willingness of people to buy flood insurance in the investigation, while in the US this seems quite important now that they want to raise the safety level.

Concluding:

More exchange of knowledge between the Dutch and US levee specialists could be useful for both the American levees as the Dutch levees! Cooperation between the two countries should be stimulated and welcomed.

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Appendices

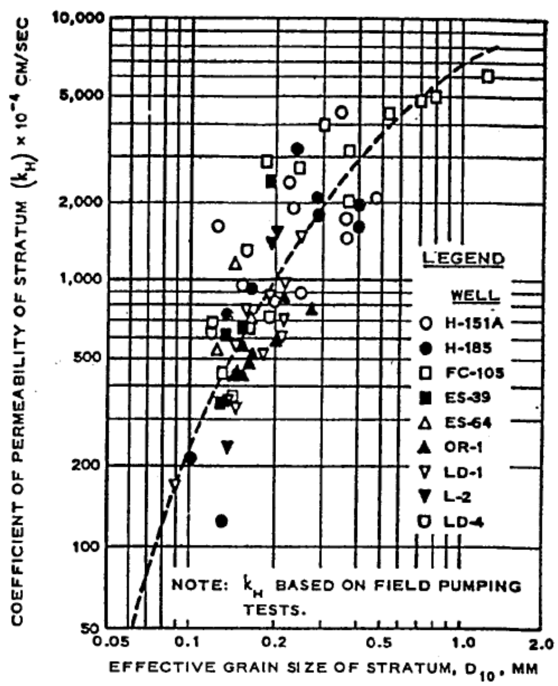
Appendix 1. Determination of D70 for calculations

To estimate the d_{70} of the pervious substratum in examples 1-3, a grain size curve is necessary. Because only Volume I of the 1956 investigation was available in this research, where in Volume II grain size curves were published, an alternative approach was necessary to estimate a reasonable d_{70} value. In the 1956 research the horizontal permeability of the substratum was determined from field pump tests or from a relation between d_{10} and the permeability, as given in Figure 1. While we do not have the grain size curves, but do know the estimated permeabilities the backward calculation is done. From the permeability a d_{10} value is estimated with Figure 1. For the Dutch Sellmeijer formula we need the d_{70}. The following formula is used to determine the d_{70} from the d_{10}:

$$d_{10,char} = \alpha' \frac{d_{70,char}}{U_{char}}$$

To get to a d70 value that is close to the characteristic value of d70 a low estimate of U is taken, which is 2, and a low estimate of the d10 is taken. U is d60/d10 and α'=0.9 is a correction factor to compensate for the d60 instead of d70.

Using d10-kh curve:



b. Effective grain size, D_{10} versus coefficient of permeability, k_h (from WES TM No. 3-424, ref. A-1)

Figure 1 Effective grain size, d_{10} versus coefficient of permeability k_h (USACE, 1956))

The curve in the above figure presents an approximate relationship between d10 and k_h, which is the horizontal permeability of the pervious stratum. This relationship was derived from pumping tests done at the Mississippi River test sites and laboratory tests on samples from these locations.

Appendix 2. Damage Mississippi cases

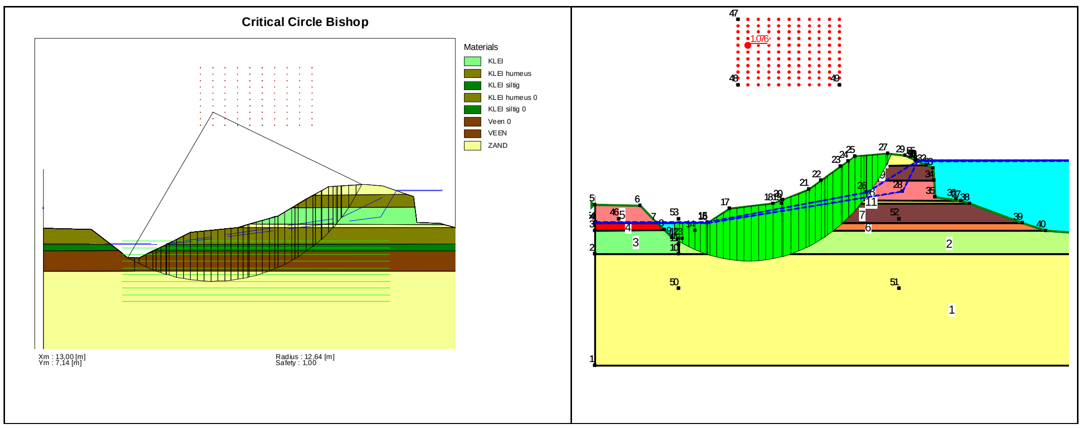
In 1956 an underseepage research from the Mississippi River was published (USACE, 1956). Below an indication of the maximum water level, the available seepage length and the seepage length divided by the water level is given, together with the damage that occurred at that site. This information is retrieved from the 1956 report.

1937		Hmax	Lmin	L/H	damage?!
	Caruthersville	4,6	55	12	numerous pin boils
	Gammon	6,1	30	5	numerous sand boils from pin size to 0.3 m
	Commerce	6,4	67	10	quite large sand boils developed and numerous boils required sacking
	Trotters 51	6,4	136	21	numerous sand boils from 0.1 to 0.3 m in diameter; another boil (60 m from levee at L/H=21.4) discharged considerable material as it moved across the road, causing the orad to cave in to a depth of 4.5 m to within 7.6 m of the levee toe; several sack levees were constructed around the active sand boils, but other boils continued to break out beyond the limits of these sack levees; finally one large-sized sack sublevee was constructed; this location had the most serious sand boils of its district
	Trotters 54	7,2	84	12	about 300 to 500 relatively small sand boils
	Stovall	8,1	122	15	five sand boils; another large sand boil and three smaller boils; they were all discharging considerable material; they were surrounded with large sack sublevees but continued to discharge very fine sand for more than 15 days; one of the worst boils in this area is shown in fig. 9;
	Farrell	7	91	13	heavy sand boils; levee banquette settled at location of at least 12 sand boils, which discharged considerable sand
	Upper Francis	6,2	76	12	no boils!
	L'Argent	7,6	116	15	one 6-inch sand boil (at location where H was 7.62 m)
	Hole-in-the-Wal	4,5	91	20	numerous small sand boils
	Kelson	5,5	360	65	no boils!
	Baton Rouge	5,79	216	37	8 large sand boils (at seismic shot point locations)
1945					
	Lower Francis	4,9	175	36	numerous sand boils; six large sand boils and 54 smaller ones; toe of the berm was unstable
	Bolivar	3,4	101	30	numerous pin boils
	Eutaw	2,9	76	26	numerous pin boils
	L'Argent	5,4	878	163	no sand boils
	Hole-in-the-Wal	3,5	152	43	no sand boils
	Baton Rouge	6,1	216	35	4 sand boils
1950					
	Caruthersville	2,7	85	31	some sand boils
	Gammon	4	152	38	approximately 40 small sand boils from 0.07 to 0.3 m
	Commerce	2,7	122	45	no sand boils
	Trotters 51	3,4	137	40	8 sand boils from 0.07 m to 0.2 m were discharging considerable material; numerous pin boils in landside drainage ditch
	Trotters 54	4,1	145	35	numerous sand boils in landside drainage ditch up to 0.3 m diameter and numerous pin boils
	Stovall	4,6	183	40	Ten sand boils from 0.05 to 0.1 m diameter
	Farrell	2,1	152	72	several pin boils in drainage ditch 30 m from berm toe
	Upper Francis	3	137	46	no boils
	Lower Francis	4	175	44	numerous sand boils; some boils discharged about 0.75 m3 of sand
	Bolivar	2,7	101	37	very heavy seepage, which made it impossible to determine whether any sand boils developed
	Eutaw	2,7	137	51	no sand boils
	L'Argent	4,7	878	187	no sand boils
	Hole-in-the-Wal	2,9	152	52	no sand boils
	Kelson	5,2	360	69	no sand boils
	Baton Rouge	5,3	216	41	4 relatively small sand boils

Appendix 3. Slope/W versus MStab

In this Appendix some results are presented of stability evaluations performed with MStab and Slope/W for the Lake Marken levees. From this data it becomes clear that there are not major differences between these programs.

Markermeer	Mstab	Slope/W	change	% change
NHW ls	3.523	3.447	0.076	2.16
NHW ws	1.345	1.326	0.019	1.41
Rapid drawdown ws	1.194	1.194	0	0.00
rain ws	1.258	1.264	0.006	0.48
rain ls	3.523	3.447	0.076	2.16



There are only small differences in the output from Slope/W and MStab. These small differences could be caused by for example:

- Round off errors
- Small differences in model input?
- Definition of piezometric lines
- Difference in input of material properties: tables vs. bilinear of functions