Investigation of the blue spots in the Netherlands National Highway Network

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1205568-000

Deltares

Title
Investigation of the blue spots in the Netherlands National Highway Network

Client	Project	Reference	Pages
Dienst Verkeer en Scheepvaart	1205568-000	1205568-000-GEO-0007-jvm	70

Headwords
Climate Change, Netherlands National Highway Network, Flooding, Risk Management

Version	Date	Author	Initials	Review	Initials	Approval	Initials
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1 Introduction

1.1 Background

1.1.1 Climate change in relation to roads

At present it is generally accepted that climate change in Europe will lead to warmer and drier summers, with more intense rainfall and showers; and warmer, wetter winters. It is difficult though to precisely quantify the changes in terms of for example location, magnitude and frequency.

Flooding is (and has been already) one of the results of climate change / change of weather patterns. Flooding poses an important threat to roads, it may lead to massive obstruction of traffic and damage to the road structures themselves, with possible long term effects. Forthcoming is the possible economic impact.

Design guidelines with a relation to the magnitude and intensity of showers have already been changed by Rijkswaterstaat Centre for Transport and Navigation. This is to anticipate to future climate change, acknowledging that the intensity of rain showers is changing substantially. However, changing the entire existing road network, including drainage systems and surrounding water systems, on a short term is very costly, unnecessary and impossible. To draw good conclusions and to be able to take good decisions, a further analysis is necessary.

1.1.2 Mission and responsibilities of Rijkswaterstaat

Important background for this project are the mission and the responsibilities of Rijkswaterstaat. With respect to current report the relevant parts of the mission are written below[1].

Rijkswaterstaat is the executive organization that develops and manages the national infrastructure networks on behalf of the Minister and State Secretary for Infrastructure and the Environment.

Rijkswaterstaat manages the country's main road network, main waterway network and main water systems. It is responsible not only for the technical condition of the infrastructure but also, and especially, for its user friendliness. It facilitates the smooth and safe flow of traffic, keeps the national water system safe, clean and user-friendly and protects the country against flooding.

With respect to the main highway network Rijkswaterstaat's mission is to ensure that road traffic moves smoothly and safely and to provide reliable and useful information. As road authority and traffic manager, Rijkswaterstaat is responsible for keeping vehicles moving, dealing with incidents quickly and informing road users.

From this responsibility follows the necessity to identify possible obstructions for the mission and to find solutions to mitigate or adapt to these obstructions.

1.1.3 Identification of blue spots

Rijkswaterstaat Centre for Transport and Navigation has taken up the responsibility and has initiated this investigation of spots in the Dutch National Highway Network vulnerable to flooding.

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[1] *more information on www.rijkswaterstaat.nl*

Furthermore the Dutch Parliament (Tweede Kamer) regularly asks questions regarding the vulnerability of our national highways to climate change and extreme weather events, for example due to flooding. The Director General of Rijkswaterstaat has asked the Centre for Transport and Navigation to investigate these spots.

To answer this question, the Centre for Transport and Navigation has commissioned Deltares to identify the vulnerable spots due to flooding in the Dutch National Infrastructure Network.
Rijkswaterstaat has worked together with Danish Road Institute (DRI) in accompanying this project. Before DRI was first responsible for the SWAMP project within the ERANET ‘Getting to grips with climate change’ call. The SWAMP approach has been used as a basis in the current Deltares investigation and has been customized to the Dutch situation.

1.2 Objective and delimitation

The objectives of the presented study are:

- To identify the vulnerable spots to flooding on the Dutch National Highway Network. In this report, a vulnerable spot to flooding is called a blue spot, with the following definition: A blue spot is a location on the Dutch National Highway Network that can be flooded in certain circumstances. A blue spot only refers to the probable cause of flooding and not to the consequences and therefore the identification of a blue spot does not per definition mean that the risk of flooding on that location is unacceptable.
- To analyse the probability of flooding. Both now and in 2050, based on the worst case KNMI climate change scenario for each type of flooding.

In chapter 3 the methodology used to identify the blue spots is described. A basic principle in this method is the fact that we worked from a more rough perspective towards a more detailed analysis. Consequently, we introduced the term potential blue spot. A potential blue spot is probably a blue spot, based on a rough analysis. However, not all potential blue spots in reality will be a blue spot. Therefore, a more detailed analysis is necessary. This analysis including a risk assessment will be the objective of further studies.

1.3 Different types of flooding

In the study, different types of flooding are taken into account with a division in three main types of flooding. In Table 1.1 one can see the types of flooding that are taken into account. In the table, one can also see which climate parameters are taken into account in the analysis.

1.3.1 Effects of flooding

The physical effects of flooding on the road are taken into account to this level that the vulnerable spots can be identified with use of the results. The consequences of the flooding (e.g. availability, safety, etc.) are not taken into account.

Water on the road due to failure of flood defences obviously leads to traffic stagnation or from a certain water depth to traffic stoppage. High water on the road or on the sides of the road construction can also lead to loss of bearing capacity for the short and long term after flooding. Deep lying sections, tunnels as well as roads with a light weight foundation can be prone to uplift and heave.

Possible effects of intense rainfall are pluvial flooding and instability of the road foundation. Snowfall in relation to pluvial flooding is not taken into account in spite of the fact that snowfall (and thaw afterwards) can influence the water management. However, the Dutch highway

network is not situated in a hilly country and consequences from snowfall in relation to pluvial flooding are expected to be very low.
Possible effects of excess groundwater levels are uplift and heave of roads in excavation, loss of bearing capacity, uplift of roads with a light weight foundation and leaching of pollution. Possible effects of excess hydraulic heads, in the aquifer[2] directly below the (mainly holocene) cover deposits, are uplift and heave of roads in excavation and in deep-lying polders.

The appearance of water on roads during heavy rain can lead to problems with availability of the road and safety for vehicles. The endangered safety aspect is caused by the development of spray behind cars with resulting poor visibility and in the worst case by aquaplaning.

Type of flooding			Influence parameter	Physical effects
A	Failure of flood defences	Flooding from sea and large rivers	Sea and river levels at certain frequencies	Flooding of the road Uplift / Heave Instability
		Flooding from small rivers/canals		
B	Water system in the area around the road is not capable for drainage / discharge of water	Pluvial flooding (overland flow after precipitation)	Intense rainfall Long period of rain	Flooding of the road Uplift / Heave Instability
		Increase of groundwater levels	Intense rainfall / long periods of rain	Uplift / Heave Instability
		Increase of aquifer hydraulic heads	Intense rainfall / long periods of rain / sea level rise	Uplift / Heave
C	Road surface not capable for enough drainage / discharge of water	Run-off on the road	Intense rainfall	Flooding (waterfilm) of the road
		Flooding of the storm water drainage system		

Table 1.1 Types of flooding, as investigated for the Dutch national highway network

1.3.2 Impact of flooding in terms of duration and area

Not every type of flooding has the same impact on the road infrastructure. It is not the scope of the current study to analyze the effects of flooding. Nevertheless in this chapter a small insight in the differences between the different types of flooding is provided.

Table 1.2 shows again the different types of flooding, but now combined with a general and qualitative description of the impact in terms of duration and area. In general it can be stated that the impact of flooding decreases from flooding type A to C. However, the probability of flooding increases from flooding type A to C, as will be shown in the report in chapters 5 to 7.

The impact of flooding is very relevant when a risk assessment is performed. Especially when measures are being identified and implemented. The accent of measures for flooding with a high impact probably lies more on evacuation and emergency planning, where measures for flooding with a low impact probably deal more with traffic management and regulation. This assessment is something that should be performed in a following research, in which the RIMAROCC framework [2] can be of help (see also chapter 9.6).

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[2] *An underground layer of sand or gravel that yields water.*

<table>
<tr><th colspan="3" rowspan="2">Type of flooding</th><th colspan="2">Impact</th></tr>
<tr><th>Duration</th><th>Area</th></tr>
<tr><td rowspan="2">A</td><td rowspan="2">Failure of flood defences</td><td>Flooding from sea and large rivers</td><td>Weeks to months</td><td>Dike ring areas (numerous square kilometers)</td></tr>
<tr><td>Flooding from small rivers/canals</td><td>Days to weeks</td><td>Polder areas (several square kilometers)</td></tr>
<tr><td rowspan="3">B</td><td rowspan="3">Water system in the area around the road is not capable for drainage / discharge of water</td><td>Pluvial flooding (overland flow after precipitation)</td><td>Hours to a day</td><td>Locally</td></tr>
<tr><td>Increase of groundwater levels</td><td>Weeks to months</td><td>Regional</td></tr>
<tr><td>Increase of aquifer hydraulic heads</td><td>Weeks to months</td><td>Regional</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Road surface not capable for enough drainage / discharge of water</td><td>Run-off on the road</td><td rowspan="2">Minutes to hours</td><td rowspan="2">Locally</td></tr>
<tr><td>Flooding of the storm water drainage system</td></tr>
</table>

Table 1.2 Impact of different types of flooding in terms of duration and area affected

1.4 Introduction to the report

This report is the final report of the study on the identification of blue spots in the Dutch National Highway Network. Chapter 2provides general information of this network as well as the source of the data that are used for the study. In chapter 3 description of the used methodology is provided. Chapter 4gives a brief introduction to climate change in the Netherlands and explains which existing knowledge on climate change is used for the performed analyses. It is also explained how the existing knowledge has been used.
In the chapters 5 to 7 the actual analyses on the different types of flooding have been reported. Table 1.3 provides a specific reference for each type of flooding. Chapter 8 is a summary of the results of the interviews with five road administrators. Finally in chapter 9 a general conclusion of the acquired results is presented.
In the chapters specific references to the appendices are made. Worth to mention here is appendix C in which a list of the produced maps (appendix D) is presented together with the source information that is used for the different maps.

<table>
<tr><th colspan="3">Type of flooding</th><th>Chapter</th></tr>
<tr><td rowspan="2">A</td><td rowspan="2">Failure of flood defences</td><td>Flooding from sea and large rivers</td><td>5.1</td></tr>
<tr><td>Flooding from small rivers/canals</td><td>5.2</td></tr>
<tr><td rowspan="3">B</td><td rowspan="3">Water system in the area around the road is not capable for drainage / discharge of water</td><td>Pluvial flooding (overland flow after precipitation)</td><td>6.1</td></tr>
<tr><td>Increase of groundwater levels</td><td>6.2</td></tr>
<tr><td>Increase of aquifer hydraulic heads</td><td>6.3</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Road surface not capable for enough drainage / discharge of water</td><td>Run-off on the road</td><td>7.1</td></tr>
<tr><td>Flooding of the storm water drainage system</td><td>7.2</td></tr>
</table>

Table 1.3 Analysis of the different types of flooding

2 Road information

2.1 General road information

The following description of the national highway network is originating from the documents Basic Maintenance Level (Basisonderhoudsniveau, BON 2011) and the Frame of Reference Control and Maintenance highway network (Referentiekader beheer en onderhoud Hoofdwegennet, RBO 2011). Details such as numbers and lengths can differ, but the description provides a good outline scope.

2.1.1 The Dutch National Highway Network

The highway network forms the connection between approximately forty economical and governmental centers in the Netherlands and connects with the main infrastructure of surrounding countries. A big part of the national highway network is part of the Trans European Road Network (TERN). The network is coarse meshed and exists of corridors, city ring-roads and interurban connections. Around 2,2 percent of all paved roads in the Netherlands belong to the main network.
The major part of the main network are motorways: 2150 kilometers of motorways with two roadways of each two traffic lanes and 250 kilometers with more than two roadways and/or traffic lanes per roadway. The other part of the main network not being a motorway exist of almost 670 kilometers with one roadway and approximately 200 kilometers with two roadways.

2.1.2 Road age

The major part of motorways in the Netherlands has been realized in the period between late sixties and early eighties. Nevertheless, in the eighties and nineties, as well as recently, substantial investments have been done in the main highway network. However, the emphasis shifted toward broadening and improving the existing roads. This age structure has distinct consequences for the control and maintenance of the main network. Above all this leads to peaks in large maintenance on structural works and pavements.

2.1.3 Pavements

The pavements of motorways are designed and constructed in such a way that the main network can fulfill its function. Pavements should (1) be sufficiently flat, (2) be sufficiently rough, (3) be sufficiently resistant for traffic in curves (a combination of roughness and slope), (4) have sufficiently bearing capacity for transmitting traffic loads to the subsoil without damage, (5) have sufficiently transverse slope to drain the water and (6) be designed such that not to many noise develops

The pavements that are maintained by Rijkswaterstaat have a total surface of around 88 square kilometers. Current replacement value of the pavements is estimated for 6,2 billion euros, excluding ground work and purchase.

In the meantime, around 83 percent of all pavements in the motorways have porous top layers with a majority of so called Open Graded Porous Asphalt (OGPA). OGPA contains hollow spaces that absorb noise and can store/drain the water (less spray and splash). Double layered OGPA has been released for use on motorways in 2005. Sometimes other innovative pavements are used.

2.1.4 Structural works

Structural Works are civil engineering structures such as bridges, viaducts, tunnels and aqua ducts. They are build at places where a road embankment is not possible (e.g. crossings with roads, waters and railways). Besides larger structural works also small structural works exist. Examples are culverts and crossing bicycle and pedestrian tunnels. These small structural works are seldom visible for road users since they are hidden beneath the road surface.

2.2 Provided specific road information by Rijkswaterstaat

Information about the National Highway Road infrastructure was provided by Rijkswaterstaat in three GIS files:

1. KernGis20110115.
2. Digital Topografic Data (in Dutch 'DTB').
3. BPS_banen.shp.

More information about how the data in these GIS files are used in the analyses can be found in appendix A.

Information on the presence of special objects (Expanded polystyrene (EPS), MSW slags, piled embankments and foamed concrete) vulnerable to excess water tables was not available in KernGIS or other systems of RWS. An inquiry by phone with the following seven specialist consultants, RWS technical specialists and commercial parties provided most information:

- Mr. Henkjan Beukema, RWS Centre for Navigation and Traffic (EPS, foamed concrete).
- Mr. Piet van Dijk, RWS Zuid-Holland (EPS, foamed concrete).
- Mr. Harrie van den Top, RWS Oost-Nederland (EPS, foamed concrete).
- Mr. John Gieltjes, former GeoBlock (EPS).
- Mr. Milan Duskov, InfraDelft (EPS).
- Mrs. Suzanne van Eekelen, Deltares (piled embankments).
- Mr. Jacco Booster, former Deltares (MSW slags).

Additional information on exact locations was derived from a brief archive study. In map B-11 the location of all special objects is presented.

Information concerning the drainage capacity of the road surface during periods of intense rainfall was derived from the database IVON, the RWS information system for pavement maintenance planning. For every 100 m of the main and auxiliary lanes of the network, IVON contains data on pavement condition obtained from biannual inspection and measurements. This present study uses the 2011 IVON data on transverse and longitudinal slope and type of wearing course.

It is important to notice that all analyses depend on the correctness of the provided information.

3 Methodology

Following are the general steps that are carried out for all types of flooding. In the specific chapters 5 to 7, the applied methodology for the different types of flooding is elaborated. In Figure 3.1, one can see a graphical presentation of this methodology.

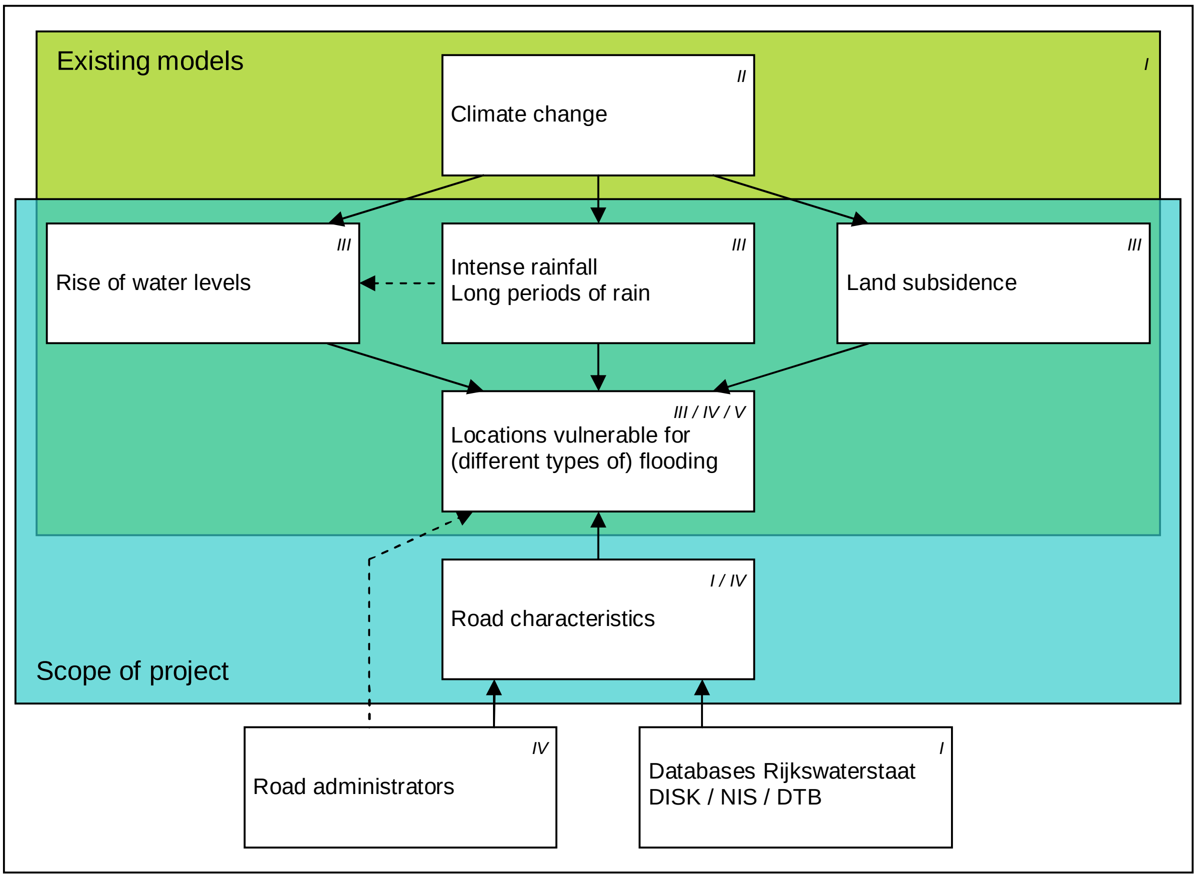


Figure 3.1 Methodology

I. Collecting data and existing models
A lot of data are collected to identify the blue spots. These data deal with the road, climate change and the existing modeling results for the types of flooding A and B.

II. Determination of climate change for the different analyses
To anticipate for future climate change it is determined what climate change needs to be dealt with in the project. It is agreed with Rijkswaterstaat that climate change will be taken into account for the relevant worst case scenario for the different types of climate change in 2050. Chapter 4 deals with climate change and the outcomes of this step.

III. Phase 1: determination of locations on road network, vulnerable to flooding
For the analyses, we used as much as possible existing knowledge and modeling results. In phase 1 of the study we combined this knowledge with the road information and climate change in order to gain a first insight in potential blue spots.
Based on location and height of the road we were able to identify locations where water heights are higher than road heights. Subsequently we used information about the

construction of the roads to identify other vulnerable spots, also for locations where water heights do not exceed road heights. In chapters 5 to 7 the results of this phase are presented.

IV. Phase 2: calibration of results with road administrators of road districts
The results of phase 1 are based on existing approximate model calculations, and sometimes assumptions and general information of the road. Especially for the types of flooding B and C a calibration of the results was thought to be necessary. The calibration was performed by comparing the results of phase 1 to the experience of road administrators by interviewing the road administrators of different districts. The calibration included as well the verification of identified potential blue spots in phase 1 as the identification of still unidentified blue spots.

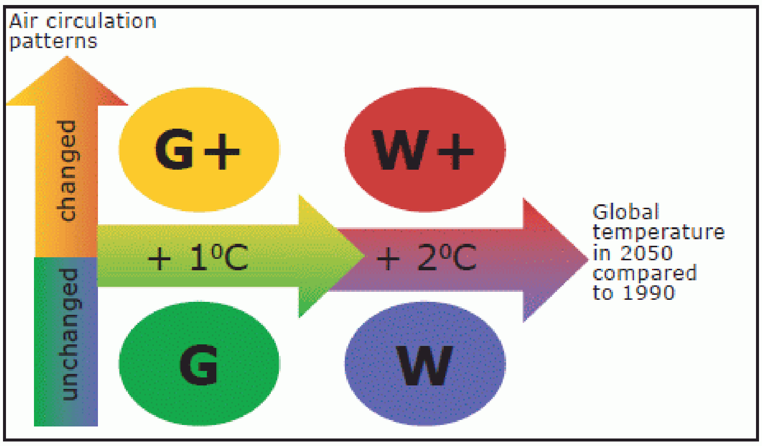
V. Phase 3: analysis of the identified potential blue spots
The identified potential blue spots are not necessarily the actual vulnerable spots in the road sections. There can for instance be facilities that prevent flooding, or the design of the road can be very robust. Phase 3 zooms in on the identified potential blue spots from phase 2 in order to filter not vulnerable spots from the potential blue spots. A list of more likely, vulnerable blue spots is the result of phase 3. These more likely blue spots can later (outside the scope of this project) be analyzed to be sure whether it will be a blue spot or not. Within the scope of the current project it was not possible to analyze all types of flooding in phase 3. In collaboration with Rijkswaterstaat phase 3 zoomed in on specific types of flooding. The results are processed in the chapters 5 to 7.

4 Climate Change

In this chapter at first a rough insight in climate change will be provided. Secondly it is reported how climate change is taken into account for the three types of flooding.
In appendix C the climate scenario's applied for the different analyses have been listed.

4.1 Introduction

At present it is generally accepted that climate change in Europe will lead to warmer and drier summers, with more intense rainfall and showers; and warmer, wetter winters. Since there are many uncertainties in both climate research itself and in the way global development will take place, it is not possible to make one single forecast for the effects of climate change on weather patterns. Therefore scenarios are used in order to gain insight in different possible changes of climate. The Royal Dutch Meteorological Institute (KNMI) has presented four possible climate change scenarios in 2006 with an update in 2009 [12] for the Dutch situation. These scenarios discern in one and two degrees rise of global temperature in 2050 on the one hand and changing or unchanging air circulation patterns on the other hand (see Figure 4.1).



G	Moderate*	1°C temperature rise on earth in 2050 compared to 1990 no change in air circulation patterns in Western Europe
G+	Moderate +	1°C temperature rise on earth in 2050 compared to 1990 + milder and wetter winters due to more westerly winds + warmer and drier summers due to more easterly winds
W	Warm	2°C temperature rise on earth in 2050 compared to 1990 no change in air circulation patterns in Western Europe
W+	Warm +	2°C temperature rise on earth in 2050 compared to 1990 + milder and wetter winters due to more westerly winds + warmer and drier summers due to more easterly winds

Figure 4.1 KNMI Climate scenarios

For those four scenarios general predictions are presented by KNMI for the weather parameters that can be present in 2050. All four scenarios are equally likely and have different effects on the parameters to be used for the different types of flooding that are addressed in this report. Therefore it is not appropriate to use only one scenario for all analyses. Together with Rijkswaterstaat it is decided to use the most pessimistic scenario for each specific type of flooding.

4.2 High water

The impact of climate change on the probability of flooding due to failure of the defence structures will be negligible because the strength and height of levees (dikes) and dunes will be adjusted to increasing loads by national flood protection programmes (in Dutch: Hoogwaterbeschermingsprogramma,(n)HWBP). For the same reason possible consequences of land subsidence on the strength and height of the defence structures will be negligible. Therefore the maps are made with the present failure risks as a starting point.
The impact of climate change and land subsidence on the consequences of flooding, i.e. an increase of the water depths, has only been reported in terms of an increase of total damage per dike ring area, without local differentiation or differentiation in cause. As a consequence we were not able within the scope of this project to analyze these changing consequences due to climate change and land subsidence.

4.3 Groundwater levels

NHI is the Dutch national hydrological instrument (www.nhi.nu) and provides scenario results for phreatic groundwater levels in two “Delta scenarios”:

- GGE: a combination of the meteorological “G”-scenario with the socio-economic scenario “Global Economy” [28].
- WPRC: a combination of the meteorological “W+”-scenario with the socio-economic scenario “Regional Communities” [28].

The characteristics of the scenarios, in terms of gross and net precipitation changes, are shown in Table 4.1. Changes in precipitation are the main driver for changes in groundwater levels under climate change.

	1976-2005	G 2050	G+ 2050	W 2050	W+ 2050
gross annual precipitation (nationwide average, in mm)	804	831	794	856	784
net annual precipitation (nationwide average, in mm)	241	254	206	266	170

Table 4.1 Changes in gross and net precipitation according to the KNMI 2006 climate scenarios for 2050. Source: www.knmi.nl

Given the focus on excess groundwater levels, the W–scenario seems the worst case, because both gross and net precipitation increase are highest. However, W is not included in the NHI-scenarios for groundwater levels. Therefore, the climate effect on phreatic groundwater levels was simulated by doubling the effect (change in groundwater level with regard to current situation) calculated for GGE. The doubling is motivated by the fact that the increases in precipitation in the W-scenario are twice those in the G-scenario (e.g. respectively 52 mm (=856-804 in Table 4.1) and 27 mm for gross annual precipitation). It must be realized that this provides an overestimate of the effect, because groundwater behaves highly non-linear: the higher the groundwater rise, the more drainage ditches and tiles will start leveling out groundwater peaks. However, the available input data do not allow for a more sophisticated approach to estimate groundwater levels under the W-scenario. The GE-component represents local changes in the groundwater system caused by land use developments. By doubling GGE, also the local effects of this GE component are doubled.

4.4 Intense rainfall

There is a large uncertainty in the estimation of the effects of climate change on intense rainfall. Maximum daily precipitation is fairly reliable to predict, but for shorter periods than a day (intense and relatively short showers) this is quite difficult. After consultation with the KNMI it is decided together with Rijkswaterstaat to use the following method in order to find proper data for intense rainfall:
Until recently Rijkswaterstaat always used the so called return levels of Braak as input for calculations including stormwater drainage systems. Recently a study has been executed by Buishand [5] with an update of these return levels for the current situation (see Figure 4.2). It is decided to use these return levels for the present situation which are presented in table 4 of the KNMI technical report 295 [5].

- The worst case scenario for intense rainfall is scenario W in which it is predicted that intense daily rainfall will increase with 27 percent in 2050 with a return period of once every ten years. This increase can be used to predict the Buishand return levels for 2050. Together with Rijkswaterstaat it is decided to use an increase of 30 percent in order not to appear over-precise. This increase from the W scenario can be used since:
  - It is not known how extremes, more extreme than the events that occur once every ten year, will change. Analyses of climate models are not clear over the trends. Therefore KNMI assumed that more extreme situations (lower probability than once every ten years) will change as much as the daily extremes that occur every ten years.
  - In the report of 2009 KNMI states that there is a margin in the daily rainfall extremes of the G and W scenario. This margin will probably count for a possible extra rise of intensity for shorter periods than one day (minutes). This possible extra rise has not been investigated yet in more detail by KNMI.
- It is allowed to use these extremes for intense rainfall for every location in the Netherlands since there is no proof for regional differences for short periods of rain (for intense daily rain there are differences for coastal and non coastal regions). This can be seen in the Thesis of Overeem [17].

If possible, the KNMI recommends to perform a sensitivity analysis taking into account the uncertainty of the change in rise of extreme rainfall. The uncertainty ranges between 17 and 41 percent, with the best estimate of being 27 percent (see above). Such a sensitivity analysis is recommended since it is very difficult to predict very extreme situations with a statistical basis from climate models. In the current study, calculations have been performed assuming the expected rise of 30%.

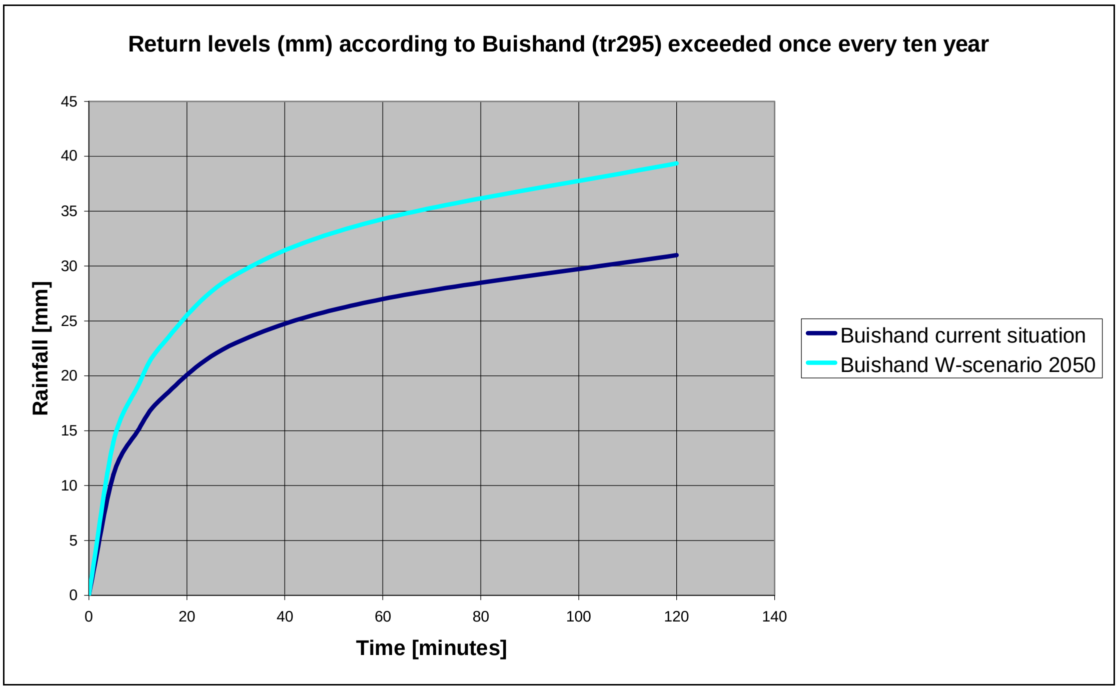


Figure 4.2 Return levels according to Buishand exceeded once every ten year

4.5 Conclusion

The most pessimistic scenario for each specific type of flooding is applied for the analyses. In appendix C the climate scenario's applied for the different analyses have been listed.

- Regarding flooding risks due to failure of defence structures maps are made with the present failure risks as a starting point.
- With respect to excess groundwater levels the climate effect on phreatic groundwater levels is simulated by doubling the effect calculated for GGE.
- For extreme rainfall calculations have been performed assuming an expected rise of 30%.

5 Flooding due to failure of flood defences

5.1 Failure of primary defence structures and high water levels for unprotected areas

5.1.1 Methodology

We discern flood risks to national highways situated inside dike ring areas[3], protected by primary defence (dikes and dunes) according to the current standards for flood protection and flood risks to highways in unprotected areas along the Dutch main water system (outside the dike ring areas). On map A2 in appendix D one can see the location of highways compared to the presence of dike ring area's.

The required basic data with regard to water depths within the dike ring areas has already been analyzed and collected in the projects FloRis, Flood Risks in the Netherlands, (in Dutch: Veiligheid Nederland in Kaart, VNK) and Flood protection for the 21st century (in Dutch: Waterveiligheid 21^{e} eeuw, WV21). These results will also be used in the making of the new Riskmap in 2013, according to the EU Flood Directive (in Dutch: Richtlijn OverstromingsRisico's, ROR). For unprotected areas the analysis that is made as part of the EU Flood Directive implementation plan for the Netherlands was used within current project.

The basic data (water depths) are derived from the results of hydrodynamic modelling of floods from coastal and/or fluvial origin. The flood is modelled assuming a failure of the primary flood defence due to excess load (i.e. water level) corresponding with the current standard for flood protection (in Dutch: toetspeilen). The calculated flooding scenarios for each dike ring area are combined into one water depth map by obtaining the maximum water depth of all the scenarios. This map is often referred to as the "Risk map".
The "Risk map" used in this analysis is a preliminary version of the Risk map that will become available according to the EU Flood Directive in 2013 and is based upon the flood scenarios that were delivered by the Provinces for the project Flood protection for the 21st century (WV21).

The direct usage of these water depths at the location of the highways is however not possible, due to inaccuracy of the elevation and positioning of the road in the flood models. Roads are often schematized as flow obstructing elements with heights corresponding with the lowest level of the road as a continuous barrier. Ramps and exits are thus neglected in these analyses, with as a consequence an incomplete picture of the vulnerability of the highway. Due to sampling to grid dimensions, also the exact position of the road can become inaccurate. In other words, the exact position of the road more or less disappears in the grid cells of 100 by 100 meters. A straight line becomes a blocked and wider line in the grid.

To overcome this problem the following methodology has been used:
The water level is calculated in a zone along the highway, based upon water depth in the zone not disturbed by the height of the highway and the level of terrain based upon AHN (the elevation model of the Netherlands) and, if available, AHN2 (an improved elevation model).
To calculate the water depth for the highway, this water level is combined with the elevation of the highway, based upon the lowest level of the road in each segment of 500 meter.

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[3] A dike ring area ('dijkringgebied' in Dutch) is an area that is protected from water (sea or rivers) by a primary water defence structure or by high grounds. These areas are appointed by law in The Netherlands.

The water depth on the highway is mapped with a legend according to the Risk map, which is expressed in terms of availability of the road for normal traffic or for military traffic.
Details of the methodology are explained in Appendix A.

5.1.2 Results

On map A1 in Appendix D the vulnerability can be seen:

1) Of the highways inside the dike ring areas for the flooding from failure of the primary defence structures.
2) Of the highways in the unprotected areas for high water levels in the main water system of the Netherlands.
3) In this map, the Risk map (water depth) is also shown.

In map A2 in Appendix D the same vulnerability as presented on map A1 is mapped in combination with probabilities. The probabilities in this map are accepted and maintained frequencies of exceedance of the design water levels of the defence structures and should not be interpreted as probabilities of flooding of the dike ring areas. The assessment of the actual probability of flooding of the dike ring areas is the main goal of the project FloRis (VNK2). At the moment, these results are only available for a limited number of dike ring areas and the results have been subject to large discussions of (water-) professionals.

It is clearly shown that, in case of flooding due to failure of the primary defence structures, almost every highway inside the dike ring areas can be affected. However, this is a compilation (worst case) of several flooding events, which in reality are highly unlikely to occur at the same time.
Therefore, the analysis has been refined by discerning the cause of flooding. On map A3, the flooding by large river discharges is presented and on map A4 coastal flooding scenarios, due to large storms are presented. It can also be seen that for certain areas (like Central Holland, dike-ring area 14) the water arrival time for these scenarios are very different. For instance, large flooding within 12 hours after the breach for coastal flooding scenarios occurs in contrast with flooding after 1 to 2 weeks after the breach with fluvial flooding scenarios.

Differentiation of the cause of flooding combined with the duration until flooded provides more insight in availability of roads and routes, both for evacuation in case of a threat of flooding as for rescue and restoration in case of actual flooding.

5.1.3 Limitations

- The risk map is a compilation (worst case) of several flooding events, which in reality are highly unlikely to occur at the same time. Differentiation by cause of flooding, by analysing the individual flood scenarios, has provided more insight. The actual probabilities of flooding of the dike ring areas are needed for a sensible risk assessment. This is the main goal of the project FloRis (VNK2) that will provide these not earlier than in the year 2015.
- The translation from water depths (being the only available outcome of the flood modelling) to water levels near the highway is based on several schematization steps, which all together introduce extra inaccuracies. Direct output of water levels from the flood calculations could overcome this problem. This output can be generated directly from the software but unfortunately this has not been done in the flooding calculations, provided by the projects FloRis and WV21.
- Road instability due to softening of soils, loss of bearing capacity, water (excess) pressures or seepage has not been considered, other than the presentation of the water depth according to the RiskMap at the side of the road. For instance, at the A12 west of

Woerden, one can see that no flooding of the road is calculated, but the area on both sides of the road can flood with a possible loss of stability of the road construction as a consequence. In a similar way other locations can be identified on map A1.
- For roads near the boundary of a dike ring area, the method with segment of 500 meter can introduce errors in the analysis, by taking water levels in account from another neighbouring dike ring area. For instance, this occurs in dike ring area 14 (Central Holland) in the neighbourhood of dike ring area 44 (Utrecht).
- Specific local provisions that prevent roads from flooding are not being taken into account. For instance flooding of tunnels and deep lying sections can be prevented by the use of dikes at the entries (kanteldijken in Dutch).
- The arrival time of the water that is presented is the arrival time of the water at the side of the road. If the elevation of the road is higher than the surface level of the surrounding, the actual water arrival time will be later than the presented arrival time. The presented arrival time is conservative.

5.1.4 Conclusion

For flooding caused by high water levels it is clearly shown that, in case of flooding due to failure of the primary defence structures, almost every highway inside the dike ring areas can be affected (map A1). However this is a (worst case) compilation of several flooding events, which in reality are highly unlikely to occur at the same time.
Therefore differentiation of the cause of flooding is provided in:

- Map A3: vulnerability of the highway for coastal flooding.
- Map A4: vulnerability of the highway for fluvial flooding.

In addition in these maps it is presented how much time it takes before highways are actually flooded after failure f the defence structures.

5.1.5 Recommendations for further research

For the dike ring areas that have already been analysed by the project FloRis (VNK2) a risk assessment for the highway can be conducted, by combining the probability of each flooding scenario with the effect of the flood i.e. the water depths on the road.
This analysis can be completed for the whole of the Netherlands by 2015, when the results of FloRis will be available for all dike ring areas.

5.2 Failure of regional defence structures

5.2.1 Methodology

A complete set of regional flooding scenarios, necessary for the analysis of the National Highways in the areas that are protected by regional defence systems, as well as the presence of highways in unprotected areas along regional water, is not available at this moment (first quarter of 2012). This will become available by the end of 2012 as part of the EU Flood Directive implementation plan (EU FD). This concerns the regional defence systems that are standardized by the provinces.

In this report, we used the preliminary results of the province of South Holland (which is leading in the delivery for the EU FD) as a pilot area with regard to the vulnerability of the highways for flooding from the regional water systems.

The method for the calculation of the water depths on the highway is similar with the method used for the primary defence structures. For the pilot area, the water levels in the polders due to flooding are directly available from the flood calculations. So the steps to derive water levels from water depths and surface levels were not necessary in this case.

The most important goal of the analysis in the pilot area is to investigate whether the data that will become available from the EU Flood Directive are sufficient to map the flood risks of the regional defence system for the highways.

In parallel to the above analysis, we have mapped those locations where the highway is near the regional defence structures (or the regional water system, if no defence structure was present) in combination with the elevation of the highway relative to the surrounding surface level. This is done by scoring the risk with a methodology according to Table 5.1.

	Levels of the highway compared to the surface level of the surrounding					
Distance to regional defence structure / regional water system	**< 0**	**0 - 0,25 m**	**0,25 - 0,75 m**	**0,75 - 1,5 m**	**1,5 - 3 m**	**> 3 m**
< 100 meter	5	5	5	4	2	1
100 - 500 meter	5	5	4	3	2	1
500 - 1 km	5	5	4	3	2	1
1 - 3 km	4	4	3	2	1	1
3 - 5 km	4	3	2	1	1	1
> 5 km	3	2	1	1	1	1

Table 5.1 Scoring table for risks of highways for failure of regional defence structures

	Risk:
5	very high
4	high
3	moderate
2	small
1	very small

Table 5.2 Qualitative risk classification

This method is used, because the (design) water levels of the regional water system, which would have been a better parameter for the method, are not yet available for all the regional systems.
By comparing the results for the pilot area, this quick scan method is evaluated to see if it is possible to get a more complete overview in a short time with the available data.

5.2.2 Results

In map A5 in Appendix D the result of the (normal) analysis for the pilot area is presented. The water depths on the highways in this map are based upon the water levels in the polders, These water levels are derived from the flood scenarios with boundary conditions which correspond with the frequency of exceedance of the polder. It should be noted that the map of the frequency of exceedance of the polders is preliminary and subject to change. It should only be used in the context of the development of the quick scan method.

In map A6.1 in Appendix D, the result of the quick scan analysis for the pilot area is presented. The conclusion, at least for the pilot area, is that the quick scan can identify most of the potential risks, with a tendency to overestimate the risk. However, a few spots were not identified by the quick scan method. The method should therefore be improved. Necessary information for an improvement are extra data about the maximum water levels. However, this is not available for all the regional water systems yet. Also, the difference in the amount of work compared with a complete assessment (like the one performed for the pilot area) will be small.

Map A6.2 in Appendix D presents the results of the quick scan analysis for the Netherlands.
For the dike ring areas it gives an overview of the vulnerability of the complete network for flooding from the regional system and it can be used to prioritize the detailed analysis like the one which now has been performed for the pilot area only (South Holland).
The road which was flooded in 1995 (near Den Bosch) is presented as a high risk. This must be considered as a potential risk (sensitive) and not an actual risk, because, after the flooding, local measures were taken to prevent this flooding. These measures have not and can not be taken into account in the quick scan analysis.

For the higher grounds (in dark grey on the map, for instance part of the provinces Gelderland/Overijssel and Brabant/Limburg) the proposed quick scan method does not give useful results. At this moment, there is no data available for the higher grounds to “calibrate” the quick scan analysis.

5.2.3 Limitations

- The quick scan method can be used to prioritize the detailed analysis for the dike ring areas. The method itself is, however, not robust (i.e. not all spots were identified).
- The map of the frequency of exceedance of the polders for the pilot area is preliminary and subject to change. Results of the pilot area should only be used as a verification of the quick scan method and as a demonstration that with the data provided by the regions, the analysis of vulnerability of the highways to flooding from regional water systems can be performed quickly.
- The quick scan method cannot be used for the flooding of the “higher grounds”.
- Specific local provisions that prevent roads from flooding are not being taken into account (such as the dikes that are constructed on both sides of the A2 near Den Bosch).

5.2.4 Conclusion

Currently it is only possible to assess the vulnerability of the highways to flooding due to failure of regional defence structures, based on regional flooding scenarios, for the province of Zuid Holland. For this province it was shown that the highways are affected. The probability of flooding of regional defence structures is generally higher than the probability of flooding of the primary defence structures.

Additionally a qualitative estimate of the risk level for all highways in the Netherlands is provided for flooding caused by failure of regional defence structures. Based on this quickscan, it can be concluded that highways in the whole of the Netherlands can be affected by flooding of regional defence structures. This quickscan identifies most of the potential blue spots, but is not robust (i.e. not all spots are identified).

5.2.5 Recommendations for further research

- Since, in general, the probability of failure for the regional defense structure is larger than the probability of the primary defense structure, we recommend analyzing the risk of failure of the regional defense structures for all the provinces, based upon the delivered flood scenarios by the end of 2012.

- Based upon the first delivery of data for the higher grounds it is possible to adapt the quick scan analysis for the higher grounds by the third quarter of 2012.

6 Flooding by intense rain and changing groundwater levels

6.1 Pluvial flooding

Possible effects of intense rainfall are pluvial flooding[4] and instability of the road foundation. The pluvial flooding risk discussed in this section is a potential risk that originates from the surroundings of the road, not from the road itself.

6.1.1 Background

Alterra [1] reported the "biophysical sensitivity" for extreme rainfall, under the W-scenario in the year 2050. Biophysical sensitivity is defined as the risk of pluvial flooding that originates from the surroundings of the road, not from the road itself. Their analysis was based on a study by Future Water [10] who considered the following factors and data:

- Surface elevation and slope, using AHN 100x100 m^2.
- Seepage and infiltration: STONE database (250x250m^2), interpolated to 100x100 m^2.
- Groundwater levels: soil map and observation well data, as published by Alterra [2].

Decision rules were then applied to derive a biophysical water sensitivity index (range 0-100%), which was subsequently combined with land use and climatic data to quantify the risk of pluvial flooding.

Alterra [1] already remarked that their analysis only provides insight in the potential risk of pluvial flooding, posed by the physical environment surrounding the highway. The map of inundation depth following pluvial flooding is presented in Map B-1.1 in appendix D.

The main reason to perform an update of the Alterra analysis [1] was that the exact elevation of roads was not considered. Instead, the inundation depth was calculated based on a regionally smoothened surface elevation dataset. Local topographic features such as buildings, dikes, road embankments and deepened stretches were to great extent removed by this smoothing procedure. Hence, the risk of pluvial flooding is overestimated for road stretches that lie, in reality, above the surroundings. The opposite is true for deepened road stretches, road cuts through accidented terrain and tunnel entries.

The calculation of inundation depths by Alterra [1] involved multiple, partly non-linear steps. These steps do not allow for a straightforward re-calculation of inundation depths with more accurate road elevations. Alternatively, the exact road elevation data can be used to verify the inundation depths calculated by Alterra [1] in a qualitative way.

6.1.2 Methodology

It was decided not to make a completely new calculation, because the risk of pluvial flooding from the surroundings was *a priori* estimated to be subordinate to the influence of the road construction. A new calculation would therefore be not very cost-effective. Our approach consisted of a comparison of the regionally smoothened surface elevation data used by Alterra [1] to the exact road elevation from the DTB. If the exact road elevation is higher than the smoothened elevation, the road lies above the surroundings, and the risk of pluvial flooding from the surroundings will be negligible. Conversely, if the exact road elevation is

[4] *Pluvial flooding is defined as flooding that results from rainfall-generated overland flow, before the runoff enters any watercourse or sewer*

lower than the smoothened elevation, the road lies below the surroundings. In that case there is a potential risk of pluvial flooding.

The difference between smoothened and exact surface elevation (ΔZ) was plotted on map, using the following legend:

- $\Delta Z < -1$ m : risk of pluvial flooding is much lower than reported by Alterra.
- -1 m $\Delta Z < -0.25$ m : risk of pluvial flooding is lower than reported by Alterra.
- -0.25 m $< \Delta Z <$ 0.25 m : risk of pluvial flooding is comparable to the risk reported by Alterra.
- 0.25 m $< \Delta Z <$ 1 m : risk of pluvial flooding is higher than reported by Alterra;
- $\Delta Z >$ 1 m : risk of pluvial flooding is much higher than reported by Alterra (in practice often mitigated by facilities in road construction).

6.1.3 Results

The resulting maps are shown in Appendix B-1.1. This map shows the difference in surface elevation according to the Alterra and DTB datasets (foreground), and the inundation depth according to Alterra [1] (background)[5].

The map clearly shows that many stretches showing an inundation depth in the Alterra map, in fact lie above the surroundings. This is notably the case in the lower parts of the Netherlands, where roads are often built on embankments flanked by ditches. Conversely, some stretches show a higher risk compared to the Alterra study. This mainly concerns (type 1) tunnel and aqueduct entries, deep-lying sections in urban areas such as the A10 West and A20, and (type 2) excavated sections of highways in slightly accidented terrain such as the A28 east of Utrecht and the Veluwe sections of the A12 and A50.

The type 1 spots are, as a rule, equipped with drainage and/or pumping facilities. The actual risk of pluvial flooding is negligible if design and maintenance of these facilities are adequate. Nevertheless the interviews with the road districts revealed some deep-lying stretches experiencing some sort of pluvial flooding problems. These are caused by stagnant rainwater, in turn stemming from poor design or maintenance of the rainwater discharge facilities. Examples are the Ringvaartaqueduct (A4) and the railway underpass in the A20 near Schiedam. Maintenance becomes more critical due to climate change. These kind of risks can be grouped somewhere between the functioning of the storm water drainage system (flooding type C) and pluvial flooding (flooding type B). We chose to range them in the category of pluvial flooding.

The type 2 spots are expected to experience little problems in general, because the unfavourable topographic setting is often compensated by favourable infiltration conditions (sand deposits) and low groundwater tables (hence large storage capacities). This is for example the case with the A28, as was confirmed in the interview with the road district managers.

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5. *Occasional differences between map B-1.2 and and the results as presented by Alterra [1] are a result of different upscaling procedures. The underlying data have a resolution of 50x50 m2, while the presented maps have a resolution of 250x250 m2.*

6.1.4 Limitations

As stated already, the limitation of the approach adopted is that it overestimates the pluvial flooding risk, because it depends on the quality of the construction and / or rainwater discharge facilities whether there is an actual problem relating to stagnant rainwater.

6.1.5 Conclusion

The over-all conclusion is that pluvial flooding appears to be a negligible risk for highways. With use of more detailed road information the output of the Alterra quickscan [1] of pluvial flooding is checked. It is concluded that the risk of pluvial flooding is generally low (lower than presented by Alterra). Tunnel and aqueduct entries and deep lying sections show a higher potential risk, however as a rule such stretches are equipped with drainage and/or pumping facilities. Also roads in excavation in slightly accidented terrain show a higher potential risk. Here the unfavourable topographic setting is often compensated by favourable infiltration conditions and low groundwater tables.

6.1.6 Recommendations for further research

As pluvial flooding poses a very limited risk for roads we do not recommend to perform a more detailed analysis.
However, some sort of pluvial flooding problems have been identified during the interviews at tunnel entries and deepened road stretches. The only way of gaining more detail is to look at these specific blue spots. It is advised to focus primarily on the blue spots that were identified from the interviews with the road districts. Secondarily, for a more detailed investigation on this kind of specific actual blue spots with an even geographical distribution across the Netherlands, more road districts need to be inquired. Aspects to be considered are:

- Design of rainwater discharge facilities, possible flaws and the extent to which resilience to climate change is incorporated. See also chapter 7.2.
- Maintenance.

6.2 Excess groundwater tables

Possible effects of excess groundwater levels are uplift and heave of roads in excavation, loss of bearing capacity, uplift of roads with an EPS foundation and leaching of pollution.

At first a national and rough analysis was carried out to identify potential blue spots. This is reported in chapter 6.2.2 through 6.2.3 Afterwards a more detailed analysis took place to identify likely blue spots. This is reported in chapter 6.2.4.

6.2.1 Methodology

6.2.1.1 Present situation

A nationwide analysis was carried out by intersecting the “mean highest phreatic groundwater depth”[6] map by NHI (version 2.1, raster 250x250 m) with the highway road network geometry, as contained in the DTB (using the lowest point on the road). NHI is the Dutch national hydrological instrument (www.nhi.nu). The NHI map is available for the Netherlands with the exception of South Limburg, and includes urban areas, unlike the groundwater map applied in the Alterra [1] study.

The NHI model runs that were used in this study were generated for simulating fresh water supply and regional groundwater patterns. Surface water levels maintained by e.g. water

[6] abbreviated “GHG”: a generally accepted measure of groundwater depth in The Netherlands, calculated as the 8-year average of 3 highest groundwater levels per year, determined on the 14th and 28th of every month

boards (Waterschappen) are used as model input to simulate groundwater conditions. The results can only be used for analyzing regional patterns and trends, e.g. “what is the expected change for the south-western part of the Netherlands. It is not justified to draw conclusions based on results for single pixels. An exception can be made for tunnel entries and low lying road stretches, where a very ‘’wet’’ pixel value obviously can be traced back to low surface elevations.

For South Limburg, time series of groundwater levels in observation wells within 200 m of the highway network were analyzed and summarized. The wells were selected on the basis of (1) their location within 300 m of a highway, (2) availability of measurements for at least ten years, of which at least one year after the year 2000, and (3) appropriate filter depths. The filter depths and lengths of these wells are highly variable, but are in general located in a permeable sand, gravel or fractured hard rock layer directly below the loam layer that covers the surface in large parts of South Limburg. Rainfall may induce stagnant water at the surface or perched groundwater tables in the loam layer. These are not recognized or measured as groundwater levels as discussed in this chapter, and are better classified under pluvial flooding (see chapter 6.1).

6.2.1.2 2050 situation

NHI provides scenario results for phreatic groundwater levels in two ‘’Delta scenarios’’:

- GGE: a combination of the meteorological “G”-scenario with the socio-economic scenario “Global Economy” [28].
- WPRC: a combination of the meteorological “W+”-scenario with the socio-economic scenario “Regional Communities” [28].

The W scenario unfortunately has not been calculated since this scenario seems to be the worst case. As motivated in chapter 4.3, an estimate of the changing groundwater levels in the W scenario for 2050 is made by doubling the effect of the GGE scenario.
Soil subsidence was also considered in the GGE scenario. In this scenario, it is assumed that soil subsidence is compensated for by lowering surface water levels to maintain the present freeboard[7]. In areas not near to surface water however, soil subsidence may still lead to a decrease in groundwater depth. On the other hand it is expected that highways are less subject to soil subsidence than their surroundings. See chapter 6.4 for further details on this aspect.

6.2.2 Results of national analysis

6.2.2.1 Present situation

Groundwater levels

The groundwater depth map for the present situation (B-2) shows the highway stretches with the mean highest phreatic groundwater depth less than 1 m highlighted. The mean highest phreatic groundwater depth was determined relative to the road surface as given in the DTB. The value of 1 m can be considered a minimum required groundwater depth in Dutch main road design. In the map therefore, a groundwater depth less than 1 m is considered an excess groundwater level.

Excess groundwater levels occur in different types of areas on the map:

(1) Peat areas in the western parts where high water surface levels are maintained. Examples that were confirmed in the interviews are the A15 near Sliedrecht and a

[7] *Freeboard is the difference between the surface water level and the surface elevation.*

part of the A205 Amsterdam - Haarlem. Recent road reconstructions often improve the actual situation.

(2) Areas prone to natural seepage occur at the foot of cover sand ridges and push ridges (low hills pushed up by glaciers in the Saalien glacial era) or brook valleys. It is not clear to what extent the risk in this type of areas is overestimated by the map. This type of risk is not experienced in practice in the relevant road districts (Utrecht, Amsterdam and Den Bosch). It may however be relevant in other risk areas that were not verified in the interviews.

(3) Deep-lying sections, tunnel and aqueduct entries. Many blue spots on the map can be attributed to this category. As a rule, drainage facilities, well fields or waterproof constructions are present along these low-lying stretches. These are often not represented by the model. Therefore the map overestimates the risk of excess groundwater levels. However, mitigating facilities are sometimes lacking or leaking. Examples mentioned in the interviews are the Schiphol tunnel (A4) and the Amelisweerd trench (A27).

In some areas, the risk of groundwater flooding remains to be verified. Notable examples are the areas around Enschede and south of Breda. Possibly they are of type 2 or a combination with type 3.

South Limburg
Time series of 21 groundwater observation wells, all accessible via DinoLoket, were visually inspected.

The groundwater depths, as derived from the time series, range from 2 to more than 20 m, with the exception of a stretch of A2 near Holtum and Roosteren where the highest measured groundwater levels are within 1 meter of the surface. However, this stretch lies relatively high above the surrounding area, as can be deduced from map B-10 showing differences between highway elevation and mean surface elevation according to NHI. The groundwater depth under the highway is therefore expected to be larger than represented in the time series.

Detailed results are shown in Appendix B.

6.2.2.2 2050 situation
Map B-3 shows the highway stretches with groundwater depths less than 1 m in 2050. Maps B-4 and B-5 show the differences in groundwater depth between 2050 and present, for special objects and tunnels, respectively. The effects on special objects and tunnels are discussed in detail in section 6.2.4.

There are three main types of response areas. (1) Groundwater levels rise most pronouncedly in elevated areas without surficial drainage. These are mainly the push ridges and cover sand ridges in East and South Netherlands where the increase in rainfall is transferred directly to groundwater recharge. (2) In the lower parts of the Netherlands, the increase in rainfall is transferred to an increase in drainage discharge. The groundwater depth remains more or less unchanged. (3) In areas with soil subsidence, the lowering of surface water levels, as a response to subsidence, is accounted for in this NHI scenario. Consequently, groundwater levels are also lowered. Because it is assumed that the road elevation remains at the present level, this results in an increase in groundwater depth on maps B-4 and B-5.

Because groundwater depths in type 1 areas are generally large, no substantial risk is expected as a result of future groundwater rises. In areas at the foot of these ridges however, risks may occur (or increase) due to an increase in seepage from the ridge. On the maps (B-2 vs. B-3), this seems apparent for the A28, east of Utrecht, and a number of isolated spots at the edge of the Veluwe and in Noord-Brabant. It is recalled here that in the interviewed districts of Utrecht and Amsterdam, no current problems with groundwater are experienced. It will remain unclear whether the present safety margins will suffice in 2050 unless some idea of groundwater levels in reality is gained by monitoring.

South Limburg
Phreatic groundwater levels and hydraulic heads in deeper aquifers are identical in South Limburg. While no climate scenario for phreatic groundwater is available from NHI for South Limburg, a GGE-scenario for a hydraulic head in a deep aquifer (NHI model layer 2) is available. Map B-6 shows this hydraulic head in the present situation.

Between the present and 2050, groundwater levels rise pronouncedly. This can be explained by the fact that South Limburg is an elevated area where, despite the presence of moderately permeable loam deposits at the surface, a considerable part of the increase in rainfall will be transferred to an increase in groundwater recharge. It must be realized that the water system behaves differently under different rainfall intensities. Under high intensities there is more surficial discharge, which is very visible in South Limburg and sometimes causes pluvial flooding. Under low intensities, there is more infiltration and groundwater recharge.

For most stretches, the simulated effects are less than 1 m. This will not cause excess groundwater levels when groundwater depths range between 2 and 20 m (see present situation). The exception is a stretch in the A79 near Meerssen, where a groundwater increase of more than 1 m is simulated with a present groundwater depth of 2 m.

6.2.2.3 Intermediate conclusion
Groundwater effects caused by climate change on the Dutch road network are limited. Stretches with substantial groundwater level increases in 2050 generally have no overlap with highway stretches currently at risk. It is recommended to assess the safety margins of groundwater depths at a limited number of road sections at the foot of push ridges and cover sand ridges. The assessment can be carried out by groundwater monitoring. The selection of locations can be based on the comparison of maps B-2, B-3 and B-4.

6.2.3 Limitations
The groundwater depths at low-lying road stretches may be underestimated (in reality be lower) because it is expected that the NHI model does not contain local drainage systems that were part of the road construction. Low lying model cells (250x250 m) are then dependent on higher ditch and drainage levels in neighbouring model cells.

As indicated before, the NHI model runs that were used in this study were generated for simulating the fresh water supply, and thus not dedicated to simulate flooding near highways. The results can therefore only be used for analyzing regional patterns and trends, e.g. what is the expected change for the south-western part of the Netherlands. Model results for individual road stretches with shallow groundwater levels may be used as an indication to look at these road stretches into more detail.

The limitations of the assessments of climate effects were already addressed in section 4.3. In addition, the results presented for 2050 are a possible realisation of what could happen under climate change; a scenario with less precipitation increase such as W+ must be regarded equally probable as the W scenario.

The limitations mentioned above do not influence the conclusions drawn in this section.

6.2.4 More detailed analysis

In a more detailed analysis specific attention has been given to special objects, tunnels and deep lying sections, since these are most vulnerable to a changing groundwater table.

6.2.4.1 Methodology

The inventory of special objects, tunnels and deep lying sections shows a variety of situations with different vulnerability. The analysis of the vulnerability proceeded as follows:

1. Eliminate locations that are not vulnerable because of geometrical characteristics, such as distance between the base level of the object and groundwater table. This step requires detailed design information that is currently available only for a number of locations.
2. Eliminate locations that are not vulnerable after analysis of the general design rules that were used at the time of construction, and the expected groundwater rise due to climate change. The general design rules will be obtained by interviewing experts of Deltares and RWS.
3. Prioritize locations according to importance. The importance decreases from applications in the main lanes, applications in entries and exit ramps, applications in secondary roads crossing highways to applications in auxiliary structures such as noise barriers.
4. Prioritize locations based on expected groundwater table rise due to climate change.
5. Eliminate locations that are not vulnerable after detailed analysis of the original design. This step is probably very time consuming, depending on the number of locations and accessibility of the data and was not executed within current project.

6.2.4.2 Results

The results of the steps 1 to 4 for the further analysis of the vulnerability of the special objects are described below.

In the first step, 12 locations were eliminated at which the bottom level of the EPS, foamed concrete or load transfer platform (at piled embankments) is above groundlevel.

The second step, identifying the resilience in the design methods used throughout the years, was based on information provided by Mr. Kees van den Akker and Mr. Hans Rijnen of RWS Centre for Infrastructure (tunnels), Mr. Arjan Venmans (EPS, foamed concrete), Mrs. Suzanne van Eekelen (piled embankments) and Mrs. Mieke Ketelaars (Waste Incinerator Slags), all Deltares. The fourth step produced the following results.

Tunnels, roads in excavation and underpasses

- The design of tunnels, roads in excavation and underpasses should have sufficient safety against uplift by groundwater pressure. Resistance is provided by heavy construction floors, tension plies or impervious layers.
  The design of tunnels, roads in excavation and underpasses after 2007 takes the effects of climate change into account. This means objects with year of completion 2010 or later are considered climate resilient. At present, this only applies to the Middelburg aqueduct in highway N57.

The constructions have been designed with a safety factor against uplift of 1.1. Assuming the weight of the road construction + floor to be at least 20 kPa, the margin for groundwater table rise is 0.2 m. This margin is used as criterion in prioritizing objects for further analysis.
Although general guidelines are available for objects completed before 2010, the RWS specialists recommend to assess all designs individually because of the complexity of the analysis.

EPS fills

- Period before 2000: the design groundwater level equals Average Highest Groundwater level (GHG) and the safety factor against uplift equals 1.1. For equilibrium EPS constructions this is equivalent to a resilience to groundwater level rise of 0.2 m. This margin is used as criterion in prioritizing objects for further analysis.
  For non-equilibrium EPS constructions, the resilience is larger, to be determined on a case-to-case basis.
- Period 2000 – 2012: The CROW guideline 150 was used. The design groundwater level equals ground level and the safety factor against uplift equals 1.3. For equilibrium EPS constructions this is equivalent to a resilience to groundwater level rise of 0.6 m. In case this approach does not lead to a feasible construction, an alternative probabilistic design is allowed. The probabilistic design uses the stochastic distribution of historical groundwater levels and material parameters, designing for a reliability index of 3.4. The resilience in the probabilistic design method is site specific, and will probably be equivalent to between 0.2 and 0.6 m groundwater level rise. For non-equilibrium EPS constructions, the resilience is larger, to be determined on a case-to-case basis.
- Period after 2012: an updated CROW guideline will be published explicitly dealing with resilience against climate change.

Foamed concrete fills

- Period before 2002: The design groundwater level equals Average Highest Groundwater level (GHG) and the safety factor against uplift equals 1.1. For equilibrium foamed concrete constructions this is equivalent to a resilience to groundwater level rise of 0.2 m. This margin is used as criterion in prioritizing objects for further analysis.
  For non-equilibrium foamed concrete constructions, the resilience is larger, to be determined on a case-to-case basis.
- Period 2002 – 2012: The CROW guideline 173 was used. The design groundwater level equals Average Highest Groundwater level (GHG) and the safety factor against uplift equals 1.1. For equilibrium foamed concrete constructions this is equivalent to a resilience to groundwater level rise of 0.2 m. For non-equilibrium foamed concrete constructions, the resilience is larger, to be determined on a case-to-case basis.
- Period after 2012: an updated CROW guideline will be published explicitly dealing with resilience against climate change.

Piled embankments

- Period 1999 (first application) – 2010: The CUR guideline 2002-7 was used. The load transfer platform is assumed to be above groundwater level; no specification for the clearance between the bottom of the load transfer platform and groundwater level is provided. Resilience also depends on the sensitivity of the construction to water. This is subject of research.
- Period 2010 – 2013: The CUR guideline 226 was used. The bottom of the load transfer platform should be above groundwater level. The resilience depends on variation in

groundwater level, and sensitivity of the construction to water. The latter is subject of research.
- Period after 2013: an updated CUR guideline will be published, probably dealing with resilience against climate change.

Waste incinerator slag fills
- Period 1989 (first application) – 1995: The IPO/VROM guideline ‘Environmental conditions for applications of unbound MSW slags on land in civil engineering works’ was used: The bottom level of the slags should be at least 0.50 m above either the Average Highest Groundwater level (GHG), or the average of the top 3 highest observed groundwater levels in the past 10 years, The resilience depends on the type of material used between the slags and the groundwater, and the actual settlement of the fill. All cases should be assessed individually.
- Period 1995 – 2008: The Building Materials Decree was used. The bottom level of the slags should be at least 0.50 m above Average Highest Groundwater level (GHG), very coarse sand to be applied in first 0.50 m below slags. Theoretically, the resilience is equal to a groundwater level rise of 0.45 m. There is zero margin for excessive settlements; in practice the resilience against groundwater level rise is frequently consumed by excessive settlements. All cases should be assessed individually.
- Period 2008 – 2011: The Soil Quality Decree was used. The bottom level of the slags should be at least 0.50 m above ground level. In areas with groundwater level below GL - 0.40 m, the bottom level of the slags should be at least 0.50 m above the 99% upper limit of the groundwater level. In polders, the bottom level of the slags should be at least 0.70 m above polder water level. The effects of climate change in the next 50 years must be included in the analysis. Sufficient margin against excessive settlements. Capillary rise of groundwater in the layer below the slags should be small enough to prevent water entering the slags. Theoretically, full resilience against climate change is achieved. It is not stated how the effects of climate change should be determined. In practice differences in interpretation may be found.
- Period after 2011: The Soil Quality Decree and additional RWS Component Specifications for Road Embankments v4.4 [18] were used:bottom level of the slags should be at least 0.50 m above Average Highest Groundwater level (GHG); the determination of Average Highest Groundwater level (GHG) should extend 30 years into the future. Sufficient margin against excessive settlements. Coarse sand to be applied in first 0.75 m below slags. Theoretically, full resilience against climate change is achieved. It is not stated how the effects of climate change should be determined. In practice differences in interpretation may be found.

Based on the NHI the groundwater table increase relative to NAP has been calculated. The NHI does not allow for interpretation at a cell (250m x 250m) scale. Therefore, a search radius for each object has been used of 750m (3 pixels). The maximum value of the groundwater table change within the search radius is compared to the resilience assumed in the design rules.

This leads to the following criteria for elimination:

- Tunnels, roads in excavation and underpasses: not vulnerable if designed in 2007 or later, i.e. with year of construction 2010 or later.
- Tunnels, roads in excavation and underpasses: not vulnerable if the maximum groundwater table rise is less than 0.10 m, irrespective of year of construction. This

knock out criterion has been set to half of the margin of 0.2 m against groundwater table rise, based on expert judgement.

- EPS and foamed concrete fills: not vulnerable if the maximum groundwater table rise is less than 0.10 m, irrespective of year of construction. This knock out criterion has been set to half of the margin of 0.2 m against groundwater table rise, based on expert judgement.
- Piled embankments: not vulnerable if the maximum groundwater table drops, irrespective of year of construction.
- Waste incinerator slag fills: not vulnerable if designed in 2008 or later, i.e. with year of construction 2009 or later.
- Waste incinerator slag fills: not vulnerable if the maximum groundwater table drops, irrespective of year of construction.

A number of MSW slag fills is vulnerable at present, predominantly because of excessive settlements after completion of the construction. These fills were not eliminated and are marked ** in appendix E.

The second steps leads to the elimination of:

- 8 tunnels.
- 8 roads in excavation.
- 4 aqueducts.
- 9 EPS fills.
- 4 foamed concrete fills.
- 4 waste incinerator slag fills.

In the third step, the special objects were prioritized according to importance, in the following classes of application:

1. Main highway lanes or connection roads (i.e. between two highways).
2. Roads with an important function in accessibility, i.e. highway entry and exit ramps, bus lanes.
3. Auxiliary roads, i.e. for services areas.
4. Secondary roads.
5. Fills not supporting pavements, i.e. noise wall.

In the third step no special objects are eliminated, but a level of relative priority for further investigation is assigned to the object. If, after a risk assessment further research is seemed necessary, a high priority object should be investigated before objects of medium and low priority. It does not indicate an absolute priority.

In the fourth step, the groundwater table increase relative to NAP has been calculated from on the NHI data. As already stated above, the NHI does not allow for interpretation at a cell (250m x 250m) scale. Therefore, a search radius for each object has been used of 750m (3 pixels).

For tunnels, roads in excavation, aqueducts, EPS and foamed concrete fills, the object has been assigned a medium priority for further investigation if within this search radius a cell is present where the groundwater table rises more than 0.1 m. If the average groundwater table rises more than 0.2 m within this search radius, the object has been assigned a high priority for further investigation.

For MSW slag fills and piled embankments, the object has been assigned a medium priority for further investigation if within this search radius a cell is present where the groundwater table rises. If the average groundwater table rises within the search radius, the object has been assigned a high priority for further investigation.

Table 6.1 shows a summary of priority for further investigation of all special objects.

Type	Priority 1	Priority 2	Priority 3
Aquaduct	8	11	9
Road in excavation	1	10	3
Tunnel	0	5	3
Waste Incinerator Slag	9	14	2
EPS	2	13	6
Piled embankments	2	6	0
Foamed concrete	0	2	1

Table 6.1 Summary of priority for further investigation of all special objects

The list of potential blue spots remaining after steps 1 to 4 is included as appendix E, in order of priority for further analysis on a case-to-case basis.

6.2.5 Conclusion

It is concluded that groundwater effects caused by climate change on the Dutch highways are limited. Stretches with groundwater level increases in 2050 generally have no overlap with highway stretches currently at risk, with the exception of stretches located at the foot of push ridges and cover sand ridges.
In a more detailed analysis specific attention has been given to special objects (EPS, foamed concrete and MSW slag fills), tunnels and deep lying sections, since locations on the highway with these constructions are most vulnerable to a changing groundwater table. Based on currently available general data it is difficult to perform such a detailed analysis. For most locations (107 out of 156) a specific analysis on a case-to-case basis (based on actual design information of the objects) is necessary. 22 locations are assessed to have a high priority in that research. For the other 49 locations with such objects it was possible to confirm that these locations are not vulnerable to a possible change of the groundwater table due to climate change.

6.2.6 Recommendations for further research

It is assumed and expected that the influence of excess groundwater levels imposed by the highway's surroundings is in many cases overruled by the condition of the road construction. It is advised to focus additional effort on (in order of priority):

1. Additional verification of map B-2 for some relatively extensive stretches with predicted excess groundwater levels (interview or inquiry by e-mail, telephone).
2. It is recommended to assess the safety margins of groundwater depths at a limited number of road sections at the foot of push ridges and cover sand ridges. The assessment can be carried out by groundwater monitoring. The selection of locations can be based on the comparison of maps B-2, B-3 and B-4.
3. Existing leakages in waterproof constructions are probably known by the road district managers. Solving these issues concerns tailor-made engineering, and is therefore beyond the scope of a potential follow-up of this study.

An assessment should be made of the vulnerability of underpasses for uplift by groundwater table rise.

Further elimination of potentially vulnerable special objects can be done as follows:

- Tunnels, roads in excavation and underpasses: all remaining cases to be assessed individually (step 5). The cases could be prioritized according to the magnitude of the potential rise of the groundwater table estimated from the NHI data, as indicated in appendix E1.
- EPS and foamed concrete fills: the remaining potential blue spots should be assessed on a case-to-case basis by experts (step 5). The cases could be prioritized according to the importance of the object and the magnitude of the potential rise of the groundwater table estimated from the NHI data, as indicated in appendix E2.
- Piled embankments: all remaining cases to be assessed individually. The cases could be prioritized according to the importance of the object and the magnitude of the potential rise of the groundwater table estimated from the NHI data, as indicated in appendix E2.
- Waste incinerator slag fills designed before 2008: all remaining cases to be assessed individually. The cases could be prioritized according to the importance of the object and the magnitude of the potential rise of the groundwater table estimated from the NHI data, as indicated in appendix E2.

6.3 Excess aquifer hydraulic heads

Possible effects of excess hydraulic heads, in the aquifer directly below the (mainly holocene) cover deposits, are uplift and heave of roads in excavation and in deep-lying polders.

6.3.1 Methodology

The methodology applied is essentially the same as the procedure followed for excess phreatic groundwater levels, except that the results for NHI model layer 2 are used in stead of layer 1.

Model layer 2 represents the ‘deep’ aquifer directly below the cover deposits in the major part of the Netherlands. In some areas, the boundary between phreatic and deep aquifer is less obvious. Locally, the aquifer hydraulic heads in the east and south of the Netherlands are (almost) identical to the phreatic surface.

6.3.2 Results

6.3.2.1 Present situation

Map B-6 for the present situation shows the difference between aquifer hydraulic head and the highway surface elevation. Focus of attention should be the stretches where the head is higher than the road surface; this represents a theoretical risk of uplift or heave. According to the map, this is almost entirely limited to tunnels, aquaducts, and roads in excavation.

During the interviews it emerged that these road stretches are, as a rule, designed to deal with these excess hydraulic heads. Also safety margins are generally assumed in these designs, to deal with fluctuations in the hydraulic head.

A few other stretches also show hydraulic heads that are more or less equal to the road surface. These stretches are subject to seepage because of their location in deep polders or next to a river. They lie relatively high above the surroundings and in any case above the ditch bottoms, which would be the first to suffer from increases in hydraulic heads. Therefore, these stretches are not expected to be at risk. Before the head increases above the road surface, measures will probably have been taken to safeguard other, lower-lying infrastructure objects.

6.3.2.2 2050 situation

The simulated climate change effects on the hydraulic head in 2050 are shown in map B-7 for "tunnels"[8] only. None of the stretches where the head is higher than or equal to the road surface show a head increase by 2050.

6.3.3 Limitations

See section on excess phreatic groundwater levels. It is assumed and expected that the influence of excess groundwater levels imposed by the highway's surroundings is in many cases overruled by the road construction.

6.3.4 Conclusion

The risk of a rise of aquifer hydraulic heads due to climate change on the Dutch highways is estimated as low. Road stretches are identified that currently show hydraulic heads in the first aquifer higher than the road surface. These stretches represent a theoretical risk of uplift or heave but are probably designed for this purpose as being tunnels and excavated road stretches. None of these stretches however show a head increase by 2050 due to climate change.

6.3.5 Recommendations for further research

Climate change effects are not expected to be relevant for tunnel constructions whose design and maintenance are adequate. More importantly, there is no overlap between effects on hydraulic heads and the location of tunnels, as shown on map B-7. No further actions are recommended.

6.4 Soil subsidence

Prior to presenting an analysis of soil subsidence[9], it is important to point out the effects that subsidence could have on highway flooding.

The type of soil subsidence addressed here is caused by regional surface water level adjustments in peat and clay areas. Local subsidence caused by e.g. a leaking sewer is not considered. Subsidence caused by mining of gas and salt is considered later in this section.

Highway embankments will hardly be affected by subsidence in peat and organic clay areas caused by lowering of the groundwater table. In these areas, subsidence has three components: oxidation of organic components, irreversible shrinkage of organic soil, and compaction of the soft layers. Because of settlement under the weight of the highway embankment, the base of the embankment is usually submerged, and the first and second component of subsidence are not present. There is a slight increase in effective stress after groundwater table lowering, that is small compared to the stress imposed by the embankment. Consequently, the third component of subsidence under a highway embankment is small. Worst case calculations have shown that a typical highway embankment will subside less than 1 mm in 10 years following a 0.10 m drop in groundwater table, compared to 0.05 to 0.10 m subsidence of the surrounding area.

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[8] See remark made for map B-6

[9] Lowering of the ground in larger areas, caused by oxidation of organic components, irreversible shrinkage of organic soil, compaction of the soft layers and mining.

In other words, the relative elevation of the highway will increase under soil subsidence, as already reflected in the increase in groundwater depth in subsidence areas shown on maps B-4 and B-5. It can also be argued that soil subsidence will not enhance the risk of pluvial flooding of highways, but on the contrary reduce the risk.

The interviews with the road districts however suggest that regional soil subsidence partly explains the actual consolidation problems in some districts.

Subsidence caused by mining from deep aquifers or rock layers is different in that it may cause an entire area to subside with uniform rate, highways as well as their surroundings. In these areas, pluvial and groundwater flooding risks remain unchanged.

A possible negative effect on road constructions can be a decrease in stability of the road embankment due to steepening of the embankment slopes when surrounding soil subsides. The decrease in stability will probably be limited and not critical in the majority of cases. Also can be stated that steepening of slopes is a long term effect, which is not expected to cause acute effects, and will be handled during major reconstructions.

6.4.1 Methodology

The analysis of soil subsidence by Alterra [1] was based on a national subsidence map by Haasnoot. Recently, more detailed national subsidence maps have become available [6]. The effects of water management to respond to soil subsidence are taken into account in these maps, as well as the effect climate change under the W+ scenario, and the influence of gas mining in the province of Groningen. Effects of salt mining in Friesland, Groningen and Twente are not taken into account, and would require further local research.

6.4.2 Results

Map B-8 shows the subsidence in 2050 when maintaining the freeboards, without the effect of climate change. The map shows subsidence of the surface next to the road and not the subsidence of the road itself. The resulting patterns are comparable to the Haasnoot map. In both maps the influence of gas mining in the province of Groningen is obvious. The recent maps show more detail, and this results in a number of subsidence hot spots not present in the Haasnoot map, most notable near Rotterdam (A20), Amsterdam (A2/A9), A28 Zwolle – Assen and N31 Drachten – Leeuwarden. Conversely for the stretch of A9 running north from Haarlem, less subsidence is simulated in the recent map as compared to the Haasnoot map.

Map B-9 shows the subsidence in 2050 including the effect of climate change under W+. The subsidence patterns are similar to the autonomous subsidence map for 2050, but the degree of subsidence is more severe. Most striking is the A20 / A12 between Rotterdam and Gouda which falls almost entirely into the class >40 cm now.

6.4.3 Limitations

Land subsidence has been determined for unfounded and non-preloaded surfaces. Subsidence of the road itself will therefore be far less for the reasons given at the beginning of this section.

6.4.4 Conclusion

Land subsidence is not expected to lead to an increase of the risks of pluvial flooding, rise of groundwater tables and rise of aquifer hydraulic heads on the Dutch highways. On the contrary, it can even be stated that land subsidence leads to a decrease of these risks.

6.4.5 Recommendations for further research

It is expected that autonomous soil subsidence will not influence the road heights (see the beginning of this section). This expectation can be verified with historic elevation measurements. If the measured subsidence is significant an extrapolation for future autonomous subsidence and subsidence under W+ can be estimated based on maps B-8 and B-9.

7 Flooding by incapacity of stormwater drainage and road surface

7.1 Development of a waterfilm on the road surface during heavy rainfall

7.1.1 Used information

7.1.1.1 Dutch guidelines

According to the Dutch New Guidelines for Design of Motorways NOA [23] it should be prevented that a water film layer with thickness larger than 2.5 mm develops during periods of heavy rain. Most road design manuals world wide require that the drainage gradient in all road sections must exceed 0.5 %, in order to avoid a thick water film during and after rainfall. A proper design measure to ensure runoff and prevent water burden on the road is to construct road pavements with a transverse slope of 2.5 % .

At present the following criteria for design of motorways are valid in the Netherlands:

- Intensity of rainfall 36 mm/hr (0.6 mm/min).
- Duration of rainfall 5 minutes.
- Thickness of water layer maximally 2 to 3 mm.
- Length of ponding about 10 m at maximum in one of the road tracks.

These criteria seem to descend from general rules of thumb. The mentioned rainfall event has a recurrence period of 4 times per year. The rain intensity criterion does not seem to yield very robust road design.

According to Wikipedia (citing literature [2] and consumer reports), cars aquaplane at speeds above 53 mph (72 km/h), where water ponds to a depth of at least 1/10 of an inch (2.5 mm) over a distance of 30 feet (9 meters) or more.
Nevertheless, near transitions between road sections with opposite slopes (so called slope warps[10]) problems might arise as in the centre of the transition the slope will be zero. These warps are usually present at bends to the left in direction of traffic on highway roads. Rohlfs [22] stated that car accidents due to runoff problems especially happen at these warps.

Near the section where rotation of the transverse slope occurs the length of the runoff streamline can get rather large, going from road width L up to $2*\sqrt{2}*L$ or even more, depending on the axis and centre of slope rotation (see figure 7.1).

The NOA document states that climatic characteristics should be identified in order to adapt the general guidelines to expected situations in future. The method of identification of climatic effects on the increase of rainfall runoff from highways is described in the next paragraph.

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[10] *A transition of the transverse slope of a roadway*

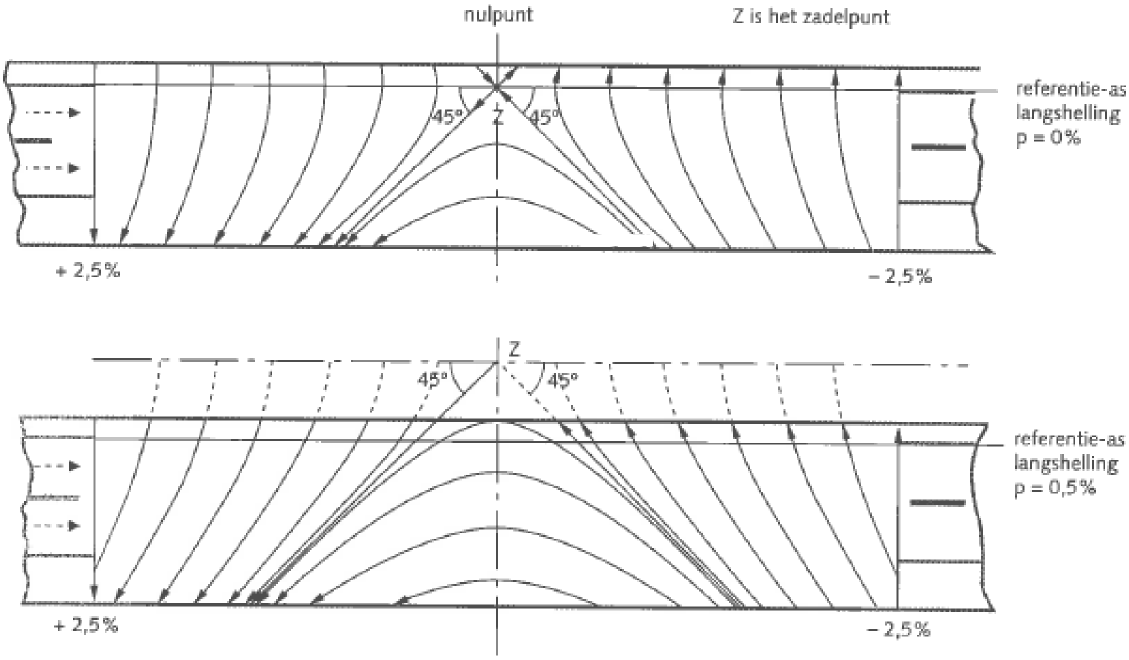

Figure 7.1 Patterns of runoff streamlines at changing slopes in motorways with differing grade in road direction [23]

7.1.1.2 *Climate data*

In our study, statistics of heavy rain showers are used according to Buishand [5] (see chapter 4.4). Buishand reports so called depths or return levels for different durations and recurrence times (frequencies or return periods). The concept of depths or return levels must be understood as cumulative amounts of precipitation (or rainfall depth). Table 7.1 presents an overview of rainfall depths for durations up to 120 minutes for events with frequencies of exceedance of once per year up to once per 1000 years.

Rainfall depth [mm]	**Depth-duration-frequency (DDF) acc. Buishand**					
D [minutes]	5	10	15	30	60	120
T [year]						
0.5 yr	4	5	6	8	10	13
1 yr	5	7	9	11	14	17
2 yr	7	10	11	14	18	21
5 yr	9	13	15	19	23	26
10 yr	11	15	18	23	27	31
20 yr	12	18	21	27	32	36
50 yr	15	21	26	32	38	42
100 yr	17	25	29	37	43	48
200 yr	-	28	33	42	49	54
250 yr	-	29	34	43	51	56
500 yr	-	32	39	49	57	62
1000 yr	-	36	43	54	64	69

Table 7.1 Cumulative precipitation for rainfall events (in mm) in the Netherlands for several recurrence frequencies for rainfall with durations of 5, 10, 15, 30, 60 and 120 minutes according to research by Buishand

By transforming to rainfall intensity rather than rainfall depth, leading to intensity-duration-frequency (IDF) curves, calculations of runoff are feasible. On the basis of the present Buishand curves the effect of the actual climate situation can be derived. For the design of stormwater drainage a frequency of exceedance of 10 years is acceptable.

To predict the situation of future climate change the W scenario for 2050 is used. According to Dutch climate research, a change of rainfall depth with a factor 1.3 is expected (see chapter 4.4). The resulting change in the Depth Duration Frequency (DDF) is presented in Table 7.2.

Rainfall	**DDF and IDF by Buishand, Recurrence frequency T=10 yrs**			
	Present climate		**Climate scenario W**	
Duration	Depth	Intensity	Depth	Intensity
[min]	[mm]	[mm/min]	[mm]	[mm/min]
5	11,0	2,20	14,3	2,86
10	15,0	1,50	19,5	1,95
15	18,0	1,20	23,4	1,56
30	23,0	0,77	29,9	1,00
60	27,0	0,45	35,1	0,59
90	29,0	0,32	37,7	0,42
120	31,0	0,26	40,3	0,34

Table 7.2 Rainfall depth and intensity in the Netherlands, for a recurrence frequency of 10 years for durations of 5, 10, 15, 30, 60 and 120 minutes; according to Buishand in actual climate and for W scenario 2050

It should be noted that the maximum rainfall intensity for shower duration of 5 minutes with a recurrence frequency of 10 years in the present climate situation is already about 3.7 times larger than the intensity of 0.6 mm/min according to NOA guidelines [23]. Due to climate change this intensity will rise to a value a factor 4.8 larger. The intensity of 0.6 mm/min according to NOA corresponds with a frequency of even more often than 2 times per year.

7.1.2 Methodology

7.1.2.1 Selection of normative phenomena related to rainfall, road runoff and water depth

The appearance of water on roads during heavy rainfall can lead to problems with availability of the road and safety for vehicles. The danger with regard to the safety aspect is constituted by the development of spray behind cars with resulting poor visibility and in the worst case by aquaplaning. The development of spray is only present if roads are used intensively. As a road must also be safe in times of low traffic, we eliminate the loss of water due to spray formation.

According to general information, aquaplaning of vehicles on wet roads occurs when a water film stays between the tires of a vehicle and the road. Tires have profile to disperse water from beneath the tire and to enhance high friction even in wet conditions. Aquaplaning occurs when a tire encounters more water than it can dissipate. Water pressure in front of the wheel forces a wedge of water under the leading edge of the tire, causing it to lift from the road. The reduction of friction can cause the tires to slip and loss of steering control follows. The risk of aquaplaning increases with the depth of standing water and the sensitivity of a vehicle to that water depth. In the international literature about car crashing [15, 3, 6] the aspect of sensitivity of vehicles to aquaplaning mostly is related to speed. We leave the aspect of aquaplaning and speed out of our consideration here.

The type of pavement influences the runoff and the severity of effects for the traffic. In the Netherlands most motorways are constructed using porous asphalt (PA, or in some publications called Open Graded Porous Asphalt OGPA and in Dutch “zeer open asphalt beton ZOAB”). In this study we will denote this type of pavement as PA. The advantage of PA is that this type of pavement reduces the formation of spray because water below the tires will be pushed away into the pavement by the pressure of the wheels of vehicles. Since this dissipation effect also reduces the formation of a waterfilm below tires, the hydraulic performance of PA is directly related to safety.

The background for a choice for the criterion with rainfall intensity of 36 mm/hour with a recurrence frequency of 4 exceedances per year is not clear. As traffic density has grown severely, we suggest to consider a more rigid criterion with a recurrence frequency of once per 10 years.

7.1.2.2 Types of pavement, hydraulic characteristics and stormwater runoff

In the research of possible blue spots we made a distinction between open or closed pavement layers. Except for open porous asphalt other closed types are also in use, like pavement types based on concrete and pavement repairs with a sealing effect on PA.

To optimize characteristics of PA the asphalt laboratories of large Dutch contractors on road construction have experimented with multiple layering and aggregate distributions. Single layer PA usually contains aggregate 11/16 mm. Top layers of twin layer PA contain aggregate mixtures 2/6 or 4/8 mm. Bottom layers of twin layer PA have mixtures like 4/8, 8/11, 11/16 or 16/22 mm. The granular distribution affects the porosity and permeability and runoff coefficients. Also, asphalt production procedures for PA affect the hydraulic characteristics. Because the application of PA on motorways mainly focuses on the improvement of noise reduction (hindrance for environment), information on hydraulic characteristics of pavements is rather scarce. To simplify the study in the present phase of investigations we made no distinction between the different types of PA pavements.

From literature [8,25], we found that permeability (in vertical direction K_v and in horizontal direction K_h) for porous asphalt PA in average circumstances after 5 years of traffic or just after construction varies as follows:

- K_v = 2.5 to 3.5 x10^{-3} m/s = 216 to 300 m/d.
- K_h = 0.8 to 1.4 x10^{-3} m/s = 86 to 120 m/d.

Corresponding values for in situ permeability testing with the Becker infiltration apparatus are 25 to 15 seconds.

The main characteristic for control of water depth is the storage capacity of PA. This characteristic depends on the porosity. In practice, the porosity of PA is between 15 en 25% with an average value of 22%.

7.1.2.3 Storage in PA during heavy rain showers on roads

Utilization of porous asphalt PA as road pavement material reduces aquaplaning problems due to stormwater on roads significantly. However, this is only partially the effect of water transport in this porous medium. A preliminary study (see appendix F and H) showed that the flow of water inside PA has hardly any effect on the surface runoff at events of heavy rainfall. The flow inside PA can only promote the drying of PA after rainfall events. An example explains the small contribution of water transport inside the porous material to the water management and safety on the road. According to Darcy's Law for flow of groundwater a PA layer with a thickness of 5 cm and an aggregate permeability of 1.10^{-3} m/s on a road with a slope of 2.5% transports at maximum a quantity of $75*10^{-6}$ m^3/min per m of road length. If an excessive rain shower with an intensity of 11 mm per 5 minutes happens to fall on a two-lane road with a width of 12 m, an amount of $26.4*10^{-3}$ m^3/min is occurring per m road length. Thus, the difference between the inflow and outflow is enormous.

Evaluating the example given above, we must conclude, that although the permeability of the layer is high, the ability of the porous material to transport water to the edge of the road is very small when compared to the amount of rain falling on the road during heavy showers.

The main properties of porous asphalt on roads are the storage and dissipation characteristic. The storage has a very positive aspect on the general water management of the road. With an average porosity of 22% for PA a 5 cm thick layer can contain as much as 11 mm per m^2. A short period with a heavy shower can be fully received by the porous pavement. However, if the porous layer has been saturated in previous wet circumstances it will take quite a long time to drain off the water that is stored in the PA.

However, if the PA contains a lot of rubber residue, the storage will be much smaller than 22%. Also, storage might play a negligible role if heavy rainfall occurs in generally wet periods and the pores in the PA are already completely filled with water before a heavy rainfall event.

Another important factor for hydroplaning also relates to the state of maintenance of the roads and concerns ruts in the pavement due to heavy traffic. We assumed that highway roads are well maintained and ruts do not occur. Therefore, the aspect of hydroplaning by ponding in ruts is not considered in this study.

The storage property of PA postpones hydroplaning problems on wet roads. In the following table is demonstrated that the storage in PA can delay the moment where a water film occurs on the road surface after start of a excess rainfall event.

DDF Buishand, present climate

Rainfall [mm]						
Dur. [min]	5	10	15	30	60	120
T = 0.5 jaar	4	5	6	8	10	13
1 jaar	5	7	9	11	14	17
2 jaar	7	10	11	14	18	21
5 jaar	9	13	15	19	23	26
10 jaar	11	15	18	23	27	31
20 jaar	12	18	21	27	32	36
50 jaar	15	21	26	32	38	42
100 jaar	17	25	29	37	43	48
200 jaar	-	28	33	42	49	54
250 jaar	-	29	34	43	51	56
500 jaar	-	32	39	49	57	62
1000 jaar	-	36	43	54	64	69

Rain intensity [mm/min]						
Dur. [min]	5	10	15	30	60	120
T = 0.5 jaar	0,80	0,50	0,40	0,27	0,17	0,11
1 jaar	1,00	0,70	0,60	0,37	0,23	0,14
2 jaar	1,40	1,00	0,73	0,47	0,30	0,18
5 jaar	1,80	1,30	1,00	0,63	0,38	0,22
10 jaar	2,20	1,50	1,20	0,77	0,45	0,26
20 jaar	2,40	1,80	1,40	0,90	0,53	0,30
50 jaar	3,00	2,10	1,73	1,07	0,63	0,35
100 jaar	3,40	2,50	1,93	1,23	0,72	0,40
200 jaar		2,80	2,20	1,40	0,82	0,45
250 jaar		2,90	2,27	1,43	0,85	0,47
500 jaar		3,20	2,60	1,63	0,95	0,52
1000 jaar		3,60	2,87	1,80	1,07	0,58

Time to fill 5cm OGPA [min]						
Dur. [min]	5	10	15	30	60	120
T = 0.5 jaar	13,86	22,27	27,93	42,22	68,53	107,65
1 jaar	11,07	15,85	18,52	30,51	48,42	81,17
2 jaar	7,89	11,07	15,13	23,89	37,43	65,15
5 jaar	6,13	8,50	11,07	17,54	29,16	52,25
10 jaar	5,01	7,36	9,21	14,46	24,78	43,62
20 jaar	4,60	6,13	7,89	12,31	20,87	37,43
50 jaar	3,67	5,25	6,37	10,37	17,54	31,99
100 jaar	3,24	4,41	5,71	8,96	15,48	27,93
200 jaar		3,94	5,01	7,89	13,57	24,78
250 jaar		3,80	4,87	7,71	13,04	23,89
500 jaar		3,44	4,24	6,76	11,65	21,55
1000 jaar		3,06	3,85	6,13	10,37	19,34

Period with water on surface [min]						
Dur. [min]	5	10	15	30	60	120
T = 0.5 jaar	<	<	<	<	<	**12,35**
1 jaar	<	<	<	<	**11,58**	**38,83**
2 jaar	<	<	<	6,11	**22,57**	**54,85**
5 jaar	<	1,50	3,93	**12,46**	**30,84**	**67,75**
10 jaar	<	2,64	5,79	**15,54**	**35,22**	**76,38**
20 jaar	0,40	3,87	7,11	**17,69**	**39,13**	**82,57**
50 jaar	1,33	4,75	8,63	**19,63**	**42,46**	**88,01**
100 jaar	1,76	5,59	9,29	**21,04**	**44,52**	**92,07**
200 jaar	5,00	6,06	9,99	**22,11**	**46,43**	**95,22**
250 jaar	5,00	6,20	**10,13**	**22,29**	**46,96**	**96,11**
500 jaar	5,00	6,56	**10,76**	**23,24**	**48,35**	**98,45**
1000 jaar	5,00	6,94	**11,15**	**23,87**	**49,63**	**100,66**

DDF Buishand, W climate scenario

Rainfall [mm]						
Dur. [min]	5	10	15	30	60	120
T = 0.5 jaar						
1 jaar						
2 jaar						
5 jaar						
10 jaar	14,3	19,5	23,4	29,9	35,1	40,3
20 jaar						
50 jaar						
100 jaar						
200 jaar						
250 jaar						
500 jaar						
1000 jaar						

Rain intensity [mm/min]						
Dur. [min]	5	10	15	30	60	120
T = 0.5 jaar						
1 jaar						
2 jaar						
5 jaar						
10 jaar	2,86	1,95	1,56	1,00	0,59	0,34
20 jaar						
50 jaar						
100 jaar						
200 jaar						
250 jaar						
500 jaar						
1000 jaar						

Time to fill 5cm OGPA [min]						
Dur. [min]	5	10	15	30	60	120
T = 0.5 jaar						
1 jaar						
2 jaar						
5 jaar						
10 jaar	3,85	5,66	7,08	11,11	19,00	33,37
20 jaar						
50 jaar						
100 jaar						
200 jaar						
250 jaar						
500 jaar						
1000 jaar						

Period with water on surface [min]						
Dur. [min]	5	10	15	30	60	120
T = 0.5 jaar						
1 jaar						
2 jaar						
5 jaar						
10 jaar	1,15	4,34	7,92	**18,89**	**41,00**	**86,63**
20 jaar						
50 jaar						
100 jaar						
200 jaar						
250 jaar						
500 jaar						
1000 jaar						

perc increase	>100%	65%	37%	22%	16%	13%

Table 7.3. Effect of storage of storm water in PA on duration of occurring water film on roads, comparing present and future climate situation

If we consider the rainfall data according to Buishand in Table 7.3 as single events we can deduce which intensity remains if the first part of the rainfall fills the available storage in the PA. In that case the surface runoff reduces due to the smaller (remaining) intensity. This is shown in the next tables.

Rainfall - storage [mm]						
Dur. [min]	5	10	15	30	60	120
T = 0.5 jaar	0	0	0	0	0	2
1 jaar	0	0	0	0	3	6
2 jaar	0	0	0	3	7	10
5 jaar	0	2	4	8	12	15
10 jaar	0	4	7	12	16	20
20 jaar	1	7	10	16	21	25
50 jaar	4	10	15	21	27	31
100 jaar	6	14	18	26	32	37
200 jaar	-	17	22	31	38	43
250 jaar	-	18	23	32	40	45
500 jaar	-	21	28	38	46	51
1000 jaar	-	25	32	43	53	58

Rainfall [mm]						
Dur. [min]	5	10	15	30	60	120
T = 0.5 jaar						
1 jaar						
2 jaar						
5 jaar						
10 jaar	3,3	8,5	12,4	18,9	24,1	29,3
20 jaar						
50 jaar						
100 jaar						
200 jaar						
250 jaar						
500 jaar						
1000 jaar						

Rain intensity [mm/min]						
Dur. [min]	5	10	15	30	60	120
T = 0.5 jaar	0,00	0,00	0,00	0,00	0,00	0,02
1 jaar	0,00	0,00	0,00	0,00	0,05	0,05
2 jaar	0,00	0,00	0,00	0,10	0,12	0,08
5 jaar	0,00	0,20	0,27	0,27	0,20	0,13
10 jaar	0,00	0,40	0,47	0,40	0,27	0,17
20 jaar	0,20	0,70	0,67	0,53	0,35	0,21
50 jaar	0,80	1,00	1,00	0,70	0,45	0,26
100 jaar	1,20	1,40	1,20	0,87	0,53	0,31
200 jaar	-	1,70	1,47	1,03	0,63	0,36
250 jaar	-	1,80	1,53	1,07	0,67	0,38
500 jaar	-	2,10	1,87	1,27	0,77	0,43
1000 jaar	-	2,50	2,13	1,43	0,88	0,48

Rain intensity [mm/min]						
Dur. [min]	5	10	15	30	60	120
T = 0.5 jaar						
1 jaar						
2 jaar						
5 jaar						
10 jaar	0,66	0,85	0,83	0,63	0,40	0,24
20 jaar						
50 jaar						
100 jaar						
200 jaar						
250 jaar						
500 jaar						
1000 jaar						
perc increase	>>	113%	77%	58%	51%	47%

Table 7.4. Effect of storage of storm water in PA on intensity and surface runoff on roads, comparing present and future climate situation

Comparing the calculated intensities in Table 7.3 and Table 7.4 it becomes obvious that storage in PA has a large impact on the amount of water that will turn into surface runoff. The representative value for intensity changes from 2.2 mm/min to 0.47 mm/min for present climate and for future W climate scenario it is not 2.86 mm/min but 0.85 mm/min.

However, given the uncertainty that heavy showers may occur after previous wet periods the safest option is to neglect the transport in the porous asphalt in our analysis and only focus on the runoff of stormwater from the surface of the road. For design of new roads we would advise to consider a worst case rainfall event with a duration of (just a little longer duration than) 5 minutes (same rain period as currently used in NOA [23].

The earlier mentioned hydraulic property of dissipation characteristic allows the porous pavement to drain the water film or excess water pressures below the wheel tires. Within the framework of this study, elaboration of dissipation phenomena was not demanded. However, it is obvious that this characteristic property of PA improves the handling of vehicles driving on wet roads.

7.1.2.4 Selection of a calculation method for stormwater runoff from roads

Dutch literature about runoff from roads [24, 21] introduced the calculation method according to Gallaway in the Netherlands (see Appendix H). This formula only accounts for runoff from the pavement surface and neglects Darcy flow inside porous asphalt. As we concluded previously, this simplification is no objection. Nevertheless, in Appendix H is shown that use of the Gallaway approach is troublesome because information about pavement roughness is lacking and the validity of the formula for Dutch roads combined with the climate situation in question is not clear.

We chose a simpler approach by adopting the empirical formula of Manning (see Appendix H for foundations of this choice).

The water depth on the pavement during heavy rainfall events can be calculated with

$$WD = \left(\frac{I \cdot L}{k_M \cdot \sqrt{S}} \right)^{3/5}$$

The parameters are:

- WD = Water Depth
- S = Pavement cross slope and grade
  The cross slope allows water to run down the pavement. Grade is the steepness of the road. The resultant of cross slope and grade is called drainage gradient or "resulting grade".
  $$S = \sqrt{i_L^2 + i_c^2}$$
- L = Drainage path length or Width of pavement [m]
  Wider roads require a higher cross slope to achieve the same degree of drainage.
- I = Rainfall intensity [m/s]
- k_M = Manning's coefficient for surface type, in [9] notated as 1/n or 1.486/n but depending on the unit system.

The Manning coefficient can be found by comparing tabulated values and estimations from field tests. From tables in hydraulic literature it was deduced that for flow over an asphalt layer the factor n is approximately 0.015. Thus, the Manning coefficient k_M for pavement has a value of around 67.

7.1.3 Results

The waterfilm depth could be calculated for varying road widths based on the Manning formula in a spreadsheet. The basic road data in the spreadsheet originated from Rijkswaterstaat database, presenting road identification, track length (divided in steps of 0.1 km), road width, pavement type and cross slope. From this database we derived that 75% of the pavement is porous asphalt and 25% is solid/closed.
In the description in the following paragraphs the effect of pavement type (closed asphalt concrete or open graded porous asphalt) was distinguished based on the amount of storage as indicated in paragraph 7.1.2.3.

7.1.3.1 Pavements without storage

For a first comparison we used as well NOA as Buishand and W climate rain intensity:

- In the NOA guidelines a rain intensity is mentioned of 36 mm/hr (matching with a 0.25 years recurrence period).
- The Buishand rain intensity is a rainfall event for the 10 years frequency with an intensity of 11 mm in 5 minutes.
- The W climate scenario 2050 has a 30% higher rainfall intensity.

The result of the comparison is shown in the following figures for 2 differing slopes.

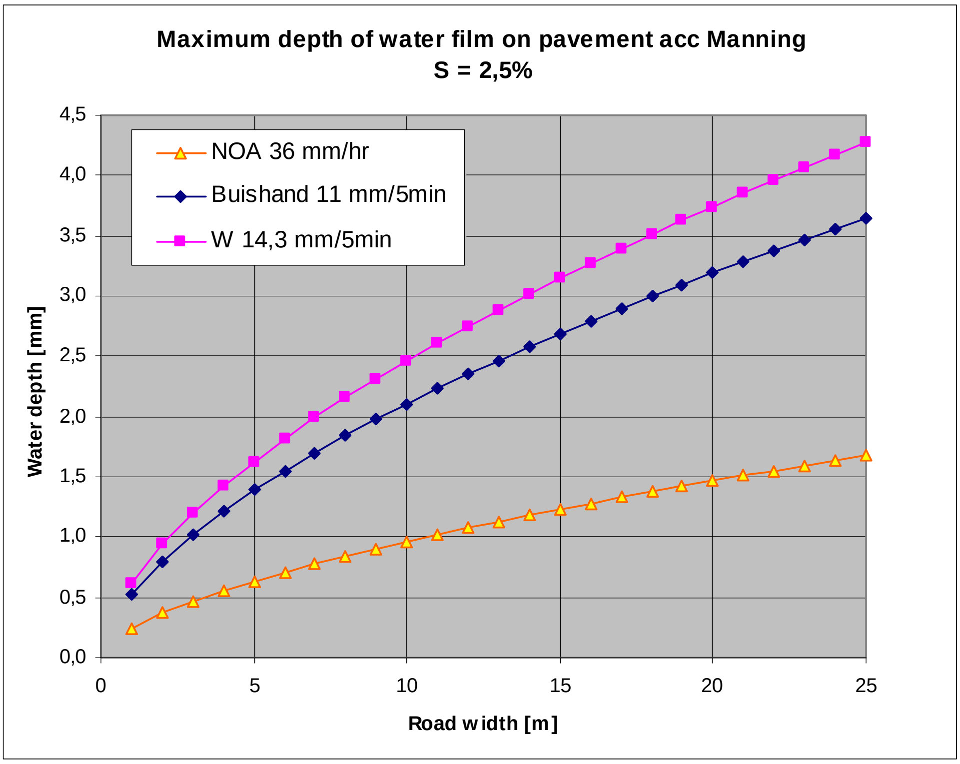

Figure 7.2 Comparison of NOA, present Buishand and future W scenario for 10 years recurrence period calculated with Manning formula for stormwater runoff on 2.5% sloped pavement without storage

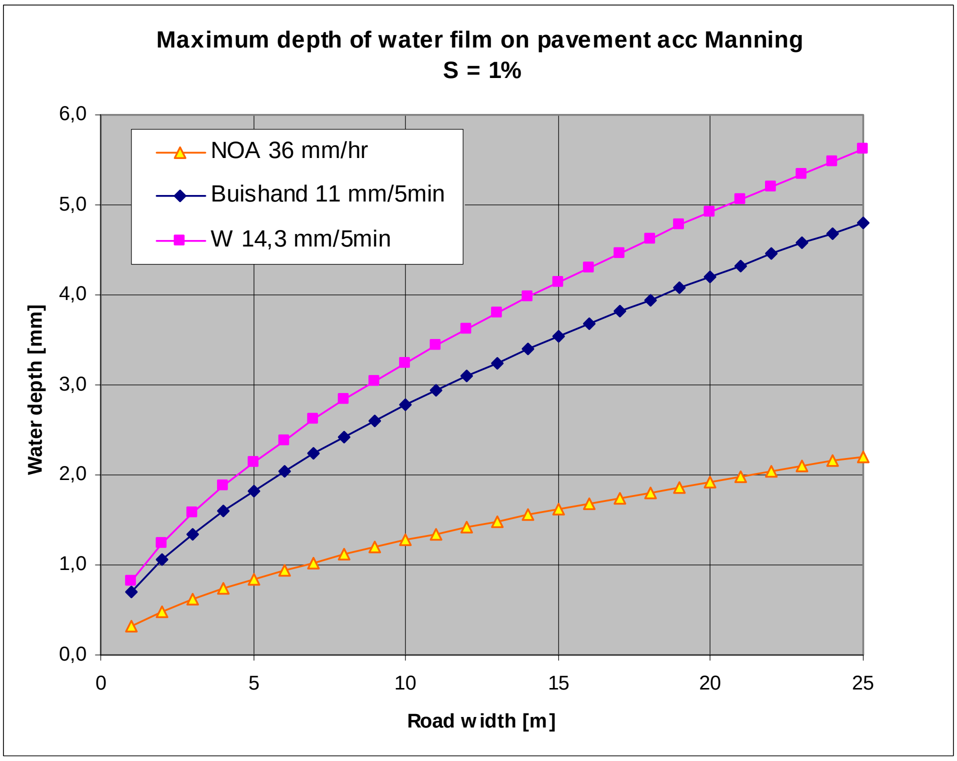

Figure 7.3 Comparison of NOA for 0.25 years recurrence period, present Buishand and future W scenario for 10 years recurrence period calculated with Manning formula for stormwater runoff on 1% sloped pavement without storage

With actual climate situation the water depth does not exceed the 3 mm film on pavement surface of motorway roads with two lanes per traffic direction (width is 12 m) and slope 2.5% as demanded by the Dutch NOA guidelines. For smaller slope inclinations the water depth goes up seriously, as can be seen in Figure 7.3. For motorway roads with two lanes per traffic direction (width is 12 m) considering the W climate scenario the water depth exceeds the NOA criterion of 3 mm. The rise of water depth is 25 to 35%, which is almost linear with increasing rainfall intensity.

At present climate the 10-years recurrence situation of rainfall is more severe than the prescribed intensity in the Dutch guidelines NOA. The selected exceptional precipitation amounts 11 mm per 5 minutes. The increase of water depth for high rainfall intensity at cross slopes for several road widths up to 23 m wide is not dramatic.
However at bends in the roads where the direction of slope changes (slope warps) the length of the flow path is much longer. If we consider a theoretical flow path with an angle of 45 degrees the length becomes $2\sqrt{2}$*B. The real flow length also depends on local data about change of cross slope and longitudinal slope of the road. Data about longitudinal slope are hard to extract from road information. For this study we took the mentioned theoretical flow path length as a rough approach.

The effects on water depth were calculated for a tire track at the lowest position on the right lane of 2-, 3-, 4- and 5-lane roads. Tire tracks can be positioned 4.15 m from the downside end of the service lane. The critical width of a 2-lane road pavement then becomes 12 – 4.15 = 7.85 m. The resulting water depths for several cross slopes and other road widths are gathered in Table 7.5 and Figure 7.4 for the present climate circumstances.

Present climate No storage cut off		**Flooding due to surface runoff from roads Water depth [mm]; Calculated with Manning's Formula for kM = 67**				
rainfall intensity i		Flow path	Cross slopes			
2,2 0,132	[mm/min] [m/hr]	from high end X [m]	S= 0,025	S= 0,02	S= 0,015	S= 0,01
road cross section						
2 lanes		7,9	1,8	1,9	2,1	2,4
3 lanes		11,5	2,3	2,4	2,7	3,0
4 lanes		15,2	2,7	2,9	3,2	3,6
5 lanes		18,8	3,1	3,3	3,6	4,0
with slope warp						
2 lanes		28,1	3,9	4,2	4,6	5,2
3 lanes		38,4	4,7	5,0	5,5	6,2
4 lanes		48,7	5,4	5,8	6,3	7,2
5 lanes		59,0	6,1	6,5	7,1	8,0

Table 7.5 Water depth of storm water runoff from pavements without storage for rainfall intensity of 11 mm/5 min, T=10 years (present climate situation) for several slopes and road widths, with flow path at straight cross direction and at slope warp

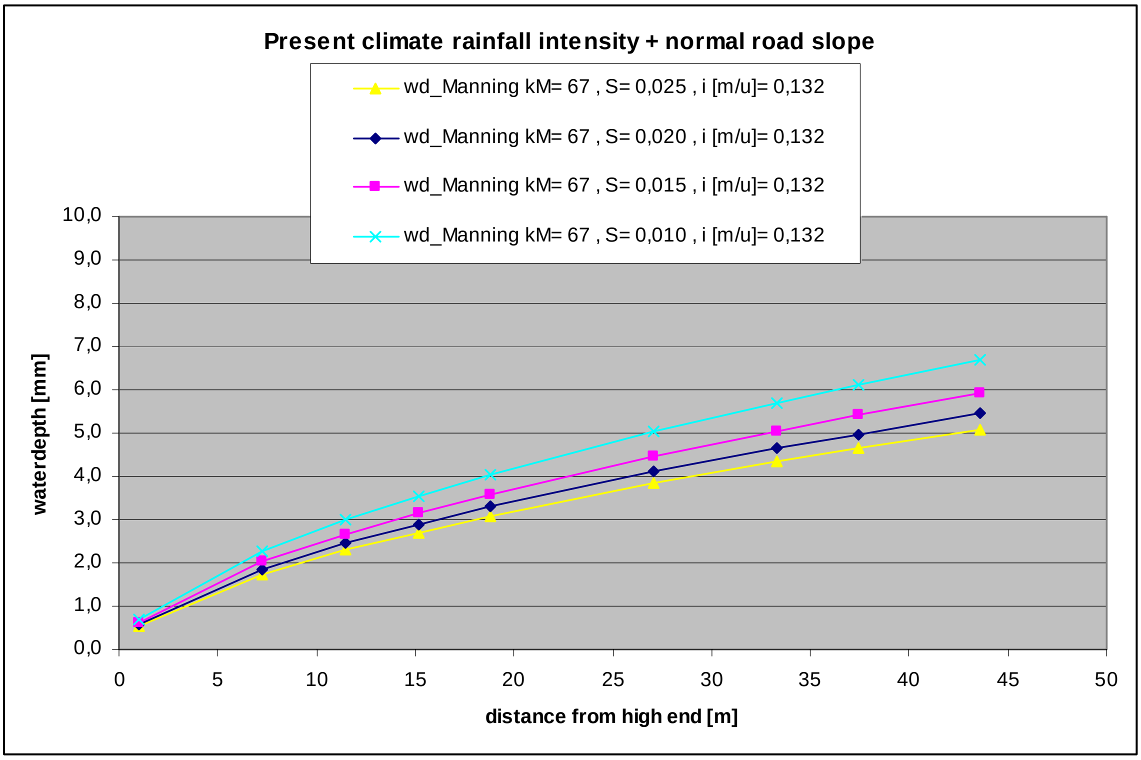

Figure 7.4 Water depth at 4 road widths (without and with slope warp) for pavements without storage at present Buishand scenario for 10 years recurrence period calculated with Manning's formula for stormwater runoff on varying sloped pavement

Considering a W climate situation the water depth on roads increases.
The increase of rainfall intensity is 30% and according to Manning's formula, where a power of 0.6 is set on this parameter, the water depth increases with 17%.

Future W climate No storage cut off		**Flooding due to surface runoff from roads Water depth [mm]; Calculated with Manning's Formula for kM = 67**				
rainfall intensity i		Flow path	Cross slopes			
2,86 0,1716	[mm/min] [m/hr]	from high end X [m]	S= 0,025	S= 0,02	S= 0,015	S= 0,01
road cross section						
2 lanes		7,9	2,1	2,3	2,5	2,8
3 lanes		11,5	2,7	2,9	3,1	3,5
4 lanes		15,2	3,2	3,4	3,7	4,2
5 lanes		18,8	3,6	3,9	4,2	4,7
with slope warp						
2 lanes		28,1	4,6	4,9	5,3	6,0
3 lanes		38,4	5,5	5,9	6,4	7,3
4 lanes		48,7	6,4	6,8	7,4	8,4
5 lanes		59,0	7,2	7,7	8,3	9,4

Table 7.6 Water depth of storm water runoff T=10 years from pavements without storage for W climate situation at several slopes and road widths, with flow path at straight cross direction and at slope warp

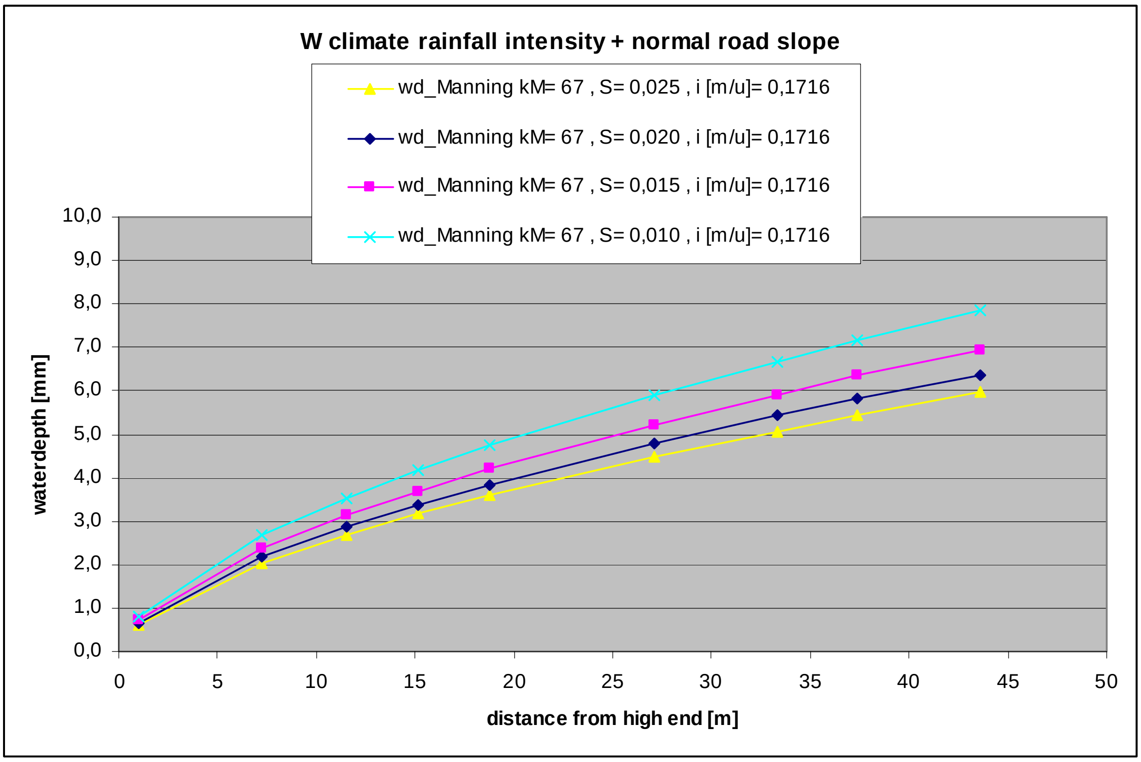

Figure 7.5 Water depth at 4 road widths (without and with slope warp) at future W climate scenario for 10 years recurrence period calculated with Manning's formula for stormwater runoff on varying sloped pavements without storage

For road pavements without storage Manning's formula is used to determine the extent the cross slope should be enlarged to keep the same safety (represented by water depth) in future as with current climate.

Water depth calculation Acc. Manning's Formula			**NOA scenario**	**Buishand scenario**	**W climate scenario**
Rain intensity i =	[m/hr]		0,036	0,132	0,1716
Manning kM =	[-]		67	67	67
Depth Wd_max =	[mm]		2	3	3
		distance from high end X [m]	desired slope [%]	desired slope [%]	desired slope [%]
road cross section					
2 lanes		7,9	0,1%	0,5%	0,8%
3 lanes		11,5	0,3%	1,0%	1,7%
4 lanes		15,2	0,5%	1,8%	3,0%
5 lanes		18,8	0,8%	2,7%	4,6%
with slope warp					
2 lanes		28,1	1,7%	6,1%	10,2%
3 lanes		38,4	3,3%	11,3%	19,2%
4 lanes		48,7	5,2%	18,3%	30,9%
5 lanes		59,0	7,7%	26,8%	45,3%

Table 7.7 Effect of criterion for storm water runoff, climate scenario and recurrence period on desired cross slope of roads with pavement without storage for several road widths; flow path length in cross direction and at slope warp

Because the formula is based on rainfall intensity divided by square root of slope, it is clear that if the rainfall intensity goes up with a factor 1.3, the slope must get steeper with a factor of 1.69. Thus, a present slope of 2.5% in future needs to be 4.2%. The calculated slopes for wide roads at locations of warps are not practical.

Visualisation with GIS of the calculated hydraulic situation on Dutch roads for actual and future climate situations was performed. A simplification neglecting storage in pavements for all Dutch highways resulted in a very high portion of the road tracks (3%) showing problems with exceedances of the allowable water depth (blue spots) at present climate, even when only straight cross slopes were considered. At almost all slope warps serious problems with waterfilm depth would occur.
Expecting more intense rainfall in 2050, this would lead to even more vulnerable spots on the map (10% of total highway track length). Since the result seems very pessimistic for the entire Dutch road system, additional calibration was sought by interviewing road administrators (see chapter 8). However, local predictions of larger water depths for closed asphalt concrete pavements are still valid.

7.1.3.2 *PA pavements with storage*

From interviews with road surveyors we received information that it would not be reasonable to present a large number of exceedances of the stormwater depth limit. The hydraulic situation with runoff due to stormwater on Dutch highways is much more favourable. This might be an effect of water losses such as by wind and spray of vehicles on wet roads but it is certainly an effect of storage in PA. For this moment all probable water losses were lumped in a storage factor. When we set this equal to thickness (0,05 m) times porosity (22%) in PA an amount of 11 mm can be taken as storage.
The effect of storage on water depth is shown in the next set of tables for both climate scenarios (present at the left and future at the right) and storage effect in PA (upper tables without and lower tables with storage) where exceedance of an occurring water depth of 3 mm is highlighted.

Present climate No storage cut off		**Flooding due to surface runoff from roads Water depth [mm]; Calculated with Manning's Formula for kM = 67**				
rainfall intensity i		Flow path	Cross slopes			
2,2 0,132	[mm/min] [m/hr]	from high end X [m]	S= 0,025	S= 0,02	S= 0,015	S= 0,01
road cross section						
2 lanes		7,9	1,8	1,9	2,1	2,4
3 lanes		11,5	2,3	2,4	2,7	3,0
4 lanes		15,2	2,7	2,9	3,2	3,6
5 lanes		18,8	3,1	3,3	3,6	4,0
with slope warp						
2 lanes		28,1	3,9	4,2	4,6	5,2
3 lanes		38,4	4,7	5,0	5,5	6,2
4 lanes		48,7	5,4	5,8	6,3	7,2
5 lanes		59,0	6,1	6,5	7,1	8,0

Future W climate No storage cut off		**Flooding due to surface runoff from roads Water depth [mm]; Calculated with Manning's Formula for kM = 67**				
rainfall intensity i		Flow path	Cross slopes			
2,86 0,1716	[mm/min] [m/hr]	from high end X [m]	S= 0,025	S= 0,02	S= 0,015	S= 0,01
road cross section						
2 lanes		7,9	2,1	2,3	2,5	2,8
3 lanes		11,5	2,7	2,9	3,1	3,5
4 lanes		15,2	3,2	3,4	3,7	4,2
5 lanes		18,8	3,6	3,9	4,2	4,7
with slope warp						
2 lanes		28,1	4,6	4,9	5,3	6,0
3 lanes		38,4	5,5	5,9	6,4	7,3
4 lanes		48,7	6,4	6,8	7,4	8,4
5 lanes		59,0	7,2	7,7	8,3	9,4

Present climate Storage cut off		**Flooding due to surface runoff from roads Water depth [mm]; Calculated with Manning's Formula for kM = 67**				
rainfall intensity i		Flow path	Cross slopes			
0,47 0,0282	[mm/min] [m/hr]	from high end X [m]	S= 0,025	S= 0,02	S= 0,015	S= 0,01
road cross section						
2 lanes		7,9	0,7	0,8	0,8	0,9
3 lanes		11,5	0,9	1,0	1,1	1,2
4 lanes		15,2	1,1	1,1	1,2	1,4
5 lanes		18,8	1,2	1,3	1,4	1,6
with slope warp						
2 lanes		28,1	1,5	1,7	1,8	2,0
3 lanes		38,4	1,9	2,0	2,2	2,5
4 lanes		48,7	2,2	2,3	2,5	2,8
5 lanes		59,0	2,4	2,6	2,8	3,2

Future W climate Storage cut off		**Flooding due to surface runoff from roads Water depth [mm]; Calculated with Manning's Formula for kM = 67**				
rainfall intensity i		Flow path	Cross slopes			
0,85 0,051	[mm/min] [m/hr]	from high end X [m]	S= 0,025	S= 0,02	S= 0,015	S= 0,01
road cross section						
2 lanes		7,9	1,0	1,1	1,2	1,4
3 lanes		11,5	1,3	1,4	1,5	1,7
4 lanes		15,2	1,5	1,6	1,8	2,0
5 lanes		18,8	1,7	1,9	2,0	2,3
with slope warp						
2 lanes		28,1	2,2	2,4	2,6	2,9
3 lanes		38,4	2,7	2,9	3,1	3,5
4 lanes		48,7	3,1	3,3	3,6	4,1
5 lanes		59,0	3,5	3,7	4,0	4,5

Table 7.8 Effect of storage of storm water in PA on water depth on roads, comparing present climate and future W climate scenario. Note: the two upper tables are the same as tables 7.5 and 7.6

Visualisation with GIS of the calculated hydraulic situation on Dutch roads for actual (green locations on map C3 in Appendix D) and future (red locations on Appendix D) climate situations was performed.

With the adjustment for storage in road tracks with PA the percentage of the highway system with blue spots for stormwater runoff is calculated at a much lower amount. For the present climate runoff blue spots are found at 1.4% of the total highway length and for future climate at 3.1% of total highway length.
The total length of all roadways, according to the Rijkswaterstaat database (IVON) is 5489,9 kilometre.
The part of the total length with solid/closed pavement is 25% and the length with open PA is 75%. In the part with closed pavement blue spots occur at 3.5% over the length of this pavement type at present climate, going up to 7% for W climate.
In the part with open pavement blue spots occur at 0.7% over the length of this pavement type at present climate, going up to 2.1% for W climate. A possibility to reduce the future number of blue spots is to anticipate on the increase of runoff problems by replacing closed asphalt by PA, enlarging the thickness of PA to create more water storage, enlarge the cross slope, change the road profile or consider drainage measures at coming road maintenance projects.

	Pavement type		
	Porous	**Closed**	**Total**
total highway length [km]	4118,7	1371,2	5489,9
blue spots present climate	0,72%	3,51%	1,42%
blue spots future W climate	2,09%	6,96%	3,31%

Table 7.9 Blue spots in Dutch highways due to stormwater runoff, taking assumed storage in porous pavements, comparing present climate and future W climate scenario

7.1.4 Limitations

- At short notice, it is too complicated to derive a formula with surface runoff and Darcy flow inside PA. In spite of this shortcoming, but given the project restrictions (budget and time) we proposed to use the Manning equation to calculate water depth during storm water runoff. However, that equation describes the static situation of runoff whereas a passing rain storm is a dynamic phenomena. For porous asphalt a better dynamic formulation is needed that distinguishes the first part of a stormwater event as storage to account for the net design rainfall intensity in flooding.
- Calibration of the proposed method to practical measurements was not possible due to lack of available data in literature.
- Given the amount of data for evaluation of surface runoff in the Dutch highway system, at present we only considered the effect of cross slope. With the present data is was not possible to evaluate the longitudinal slope of the roads and take this factor into consideration.
- We assumed that highway roads are well maintained and ruts do not occur. With poor maintenance and with presence of ruts, the waterfilm thickness will be larger than calculated.
- A cross-wind can enhance or lessen the development of a waterfilm. The effects of wind however have not been taken into account.

7.1.5 Conclusion

It appears that the number of blue spots more than doubles due to climate change, being present at 3,3% of the total road length. Blue spots are mostly present at locations with a change of transverse slope, many lanes and closed pavements.
This conclusion is based on the use of a simple calculation model with blue spots being defined as the development of a waterfilm with a thickness exceeding 3 centimetres.

7.1.6 Recommendations for further research

- To improve the hydraulic design method of Dutch roads it is advised to start research on a calculation method for dynamic behaviour of slope runoff.
- For future research we recommend performing field and lab tests on storm water runoff from Dutch roads. That would give a better insight into the effect of flow from PA and of the effect of texture of aggregate material in asphalt pavements. Probably the storage and flow in PA would give a reduction of the storm water runoff and water depth.
- A possibility to anticipate on the increase of runoff problems on Dutch highways at the changing climate is to replace solid pavement types by PA and to enlarge the existing thickness of PA at future pavement renewal projects to create larger water storage.

7.2 Road drainage system incapable for draining during periods of intense rainfall

7.2.1 Methodology

As a rule, runoff from the pavement will freely flow into the verge and infiltrate in the embankment. In most cases horizontal drains installed in the embankment will transport the water to the ditches alongside the highway. The ditches are connected to the surface water system of the surrounding area.

In a number of situations surface flow and infiltration into the verges is impossible or restricted and drainage is provided by a system of gutters and sewers. These situations are [18]:

- Tunnels.
- Roads in excavation without possibilities for gravity drainage.
- Bridges.
- Significant flow along the axis of the road, for instance when the longitudinal slope exceeds 0.5% and transitions to bridges and other constructions.
- Insufficient possibilities for infiltration, for instance when the transverse slope is directed to the center verge, or in the presence of obstacles such as curb stones, an elevated verge, sound barriers, buildings or other constructions.
- Danger of erosion of the verge, for instance near slopes or when the pavement width is large.

Information concerning the presence of gutters was derived from KernGIS. The layer containing ‘gutter’ (‘goten’ in Dutch) objects was used to produce the map with potential blue spots.

All locations with gutters could be considered potential blue spots. This approach however is rather rough and produces a relatively large amount of potential blue spots. Additional analysis therefore is required to assess the actual vulnerability.

The additional analysis was performed along the following tracks:

- For tunnels and roads in excavation pumping systems have been installed. A review of the design will reveal if the spare capacity is sufficient to cope with more intense rainfall. Because the number of locations is high the analysis on a case-to-case basis will require a substantial effort. An alternative approach is suggested, in which an analysis was made of the general design rules used at the time of construction, giving general conclusions for systems of a certain age. The general design rules were obtained by interviewing experts of RWS Centre for Infrastructure (DI).

- For bridge decks the design of the sewer system was reviewed, analyzing general design rules at the time of construction. This will give general conclusions concerning the spare capacity of systems of a certain age.
- For locations where the transverse slope is directed to the center verge, an analysis was made of the infiltration and storage capacity of the center verge during the actual events. This capacity is spare capacity that can be used to temporarily store the additional rainfall due to climate change, assuming the capacity of the drainage system is sufficient for the present day rainfall.
- For locations with danger of erosion of the embankment top layer, a more detailed analysis was made of the actual discharges that could occur for different pavement widths. These were compared to the discharges allowed for overtopping of waves over flood defences. Thus, the proper design rules for erosion resistance of the top layer were selected, and compared to typical conditions in road embankments.
- For all other locations, the design of the sewer system was reviewed. The first step is the analysis of general design rules at the time of construction, giving general conclusions for systems of a certain age.

7.2.2 Results

7.2.2.1 Drainage systems of tunnels and roads in excavation

Mr. Kees van den Akker and mr. Hans Rijnen, specialists at the department CT of RWS Centre for Infrastructure, have been interviewed to obtain design rules for the drainage systems of tunnels and roads in excavation.

With respect to the design rules, the following periods could be defined:

- Period before 1982: no explicit design rules present. The design was done by experts on the basis of construction codes, partial internal guidelines and experience.
- Period 1982 - 2006: guidelines were compiled; the intensity-duration curve of Braak (1933) was used. The guidelines served as starting point for the design. Often, the design was pragmatically adapted to local conditions by experts of RWS. In the period until 1990 different departments of RWS were responsible for the design, each department giving its own twist to the design rules.
- Period 2007 and later: the intensity-duration curve was adapted for the effects of climate change. The current intensity-duration curve is approximately equal to the Buishand W scenario. There is a time lag between the revision of the guidelines and the first tunnels constructed on the basis of the revised guidelines. Probably the revised guidelines were first applied in the design of the Middelburg aqueduct in the N57, that was finished in 2010.

Both specialists do not recommend the elimination of potential blue spots on the basis of guidelines valid for specific periods. Too many deviations from the general design rules (if present) make the suggested approach invalid. The main reasons for these deviations are:

- Designers at RWS used the guidelines only as a starting point, pragmatically adapting the design to local conditions and experience.
- Constructions may be as old as 1940; the design rules applying to constructions dating before 1982 are not easily identified.
- In the period before 1990 different departments of RWS were responsible for the design, each department giving its own twist to the design rules.

In addition, the actual capacity of the drainage systems is dependent also on the state of maintenance. Especially buried drainage systems, such as used in foil constructions, are more vulnerable than open collection systems used in concrete trenches.

In stead of the original approach suggested for the analysis, the specialists suggest an approach in which a team of experts makes a quick first assessment of the vulnerability on a case-by-case basis, using their knowledge of the local situation and the actual design.

From the analysis follows that only drainage systems of tunnels and roads in excavation designed in 2007 or later can be eliminated as blue spots. Given the time lag between design and completion, tunnels and roads in excavation with a construction date (KernGIS) 2010 or later can be eliminated. Probably the only construction that will be eliminated is the Middelburg aquaduct in the N57. The capacity of the drainage systems of all other tunnels and roads in excavation will need to be evaluated on a case-by-case basis. All tunnels and roads in excavation are presented in map B-11.

7.2.2.2 Pavement drainage systems on bridges

Mr. Kees van den Akker and mr. Hans Rijnen, specialists at the department CT of RWS Centre for Infrastructure, have been interviewed to obtain design rules of pavement drainage systems on bridges.

Generally, both specialists estimate the actual risk resulting from insufficient drainage capacity of the bridge decks as low. Even if the drainage capacity should be insufficient, the main effect will be a very temporary disruption of the traffic flow. Also, many bridge decks are relatively short (up to 50 m). Longer decks, such as found in fly-overs and bridges crossing large waters, will be more vulnerable.

With respect to the design rules, the following periods could be defined:

- Period before 1982: no explicit design rules present. The design was done by experts on the basis of construction codes, internal partial guidelines and experience.
- Period 1982 - 2006: guidelines were compiled; the intensity-duration curve of Braak (1933) was used. The guidelines served as starting point for the design. Often, the design was pragmatically adapted to local conditions by experts of RWS. In the period until 1990 three different departments of RWS were responsible for the design, each department giving its own twist to the design rules.
- Period 2007 and later: the intensity-duration curve was adapted for the effects of climate change. The current intensity-duration curve is approximately equal to the Buishand W scenario. There is a time lag between the revision of the guidelines and the first tunnels constructed on the basis of the revised guidelines.

As for the drainage systems of tunnels and roads in excavation, the specialists do not recommend the elimination of potential blue spots on the basis of guidelines valid for specific periods.

However, a distinction can be made between bridges with porous vs. non-porous pavements. Two possible mechanisms could provide spare capacity in the pavement drainage system. In the first place, storage in the porous asphalt wearing course is ignored. Second, part of the water on the road surface is converted to spray water by the traffic.

Assuming a porosity of 20% in a 50 mm porous asphalt wearing course, of which 2/3 is available for storage [24], the amount of water that effectively can be stored is 6.7 mm. Storage in nonporous wearing courses is nil. The amount of water converted to spray is approximately 1/3 for porous asphalt, and approximately 2/3 for nonporous pavements [12].

The following figures compare the pre-2007 design intensity-duration curves and the curves corrected for storage and spray.

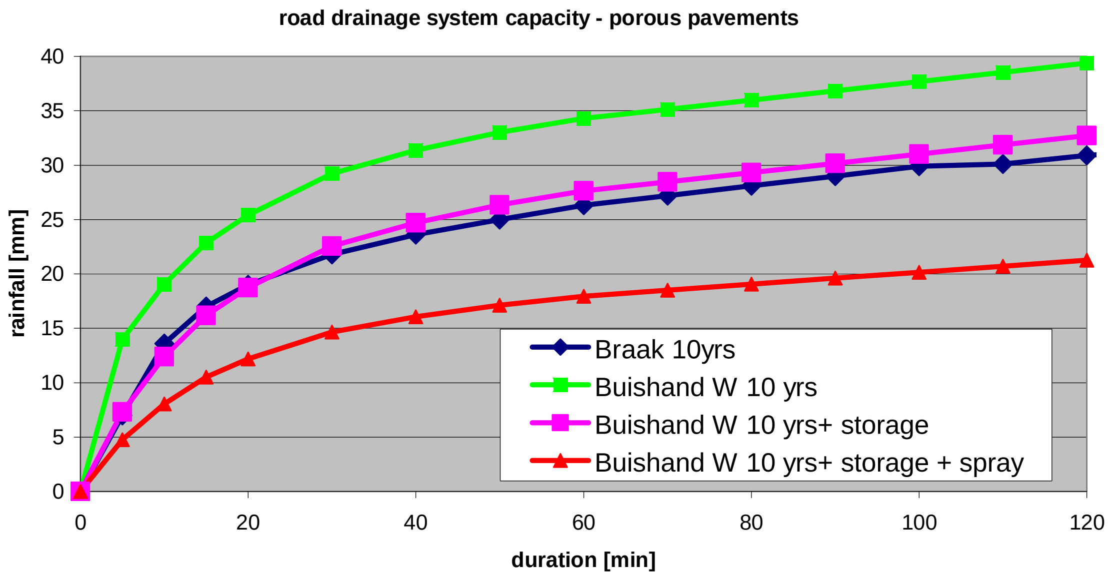

Figure 7.6 Intensity-duration curves pre-2007 and with potential effects of storage and spray, for porous asphalt

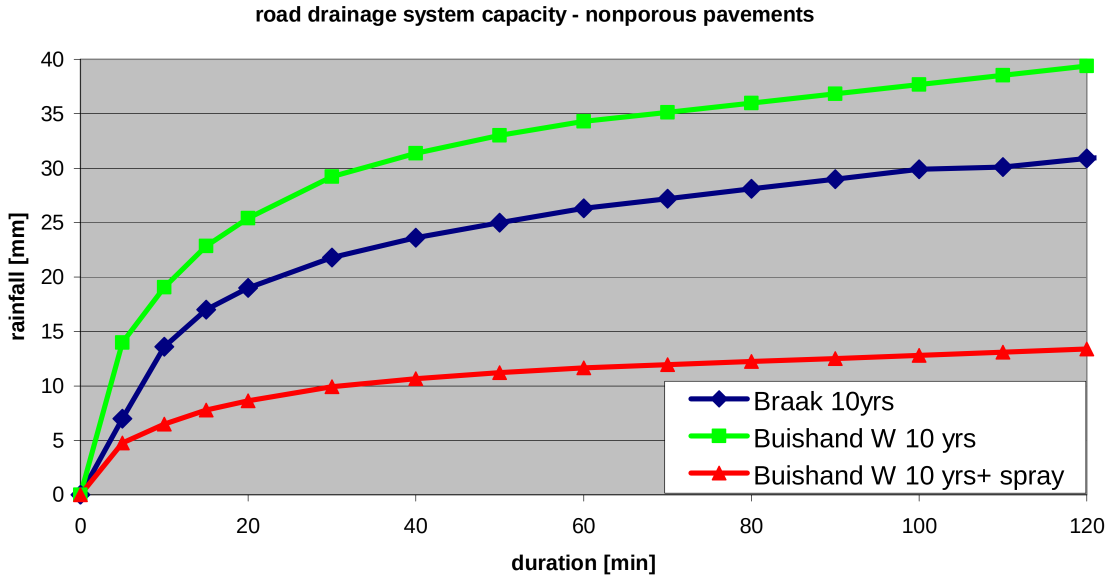

Figure 7.7 Intensity-duration curves pre-2007 and with potential effect of spray, for non-porous pavements

By comparing the pre-2007 curves and the potential effects of storage and spray, it is clear there may be a hidden spare capacity in the design of road drainage systems. For porous asphalt the effect of storage alone will largely offset the increase in rainfall due to climate change. For nonporous pavements, the spare capacity will depend on the amount of spray only.

However, the amount of spray quoted in [12] is valid for light to moderate rainfall. The amount of spray generated will most certainly drop significantly once the traffic speed decreases in heavy showers. For this reason, this study does not take reduction of the intensity by spray into account until further validation of the figures is found.

Also, any spare capacity may also be necessary to compensate for loss of capacity due to other factors, such as partial clogging of the sewers by poor maintenance. In order to allocate the spare capacity to counteracting the effects of climate change, maintenance will become more critical.

Connections of drainage systems on bridge decks to the sewers in the embankments are notorious weak spots in the drainage system. At present, it is estimated that at least one such connection fails every year, leading to internal erosion of the embankment and collapse of the pavement. These events will occur more frequently in the future as discharges increase. Again, inspection and maintenance of the drainage systems will become more critical.

The conclusions are:

- For porous pavements, the additional rainfall due to climate change will largely be stored in the porous pavement itself, which represents a potential spare capacity. Therefore, bridges with a porous pavement have been eliminated as blue spots.
  The spare capacity can be used if the gutter and sewer system is well maintained. Thus, maintenance will become more critical.
- For non-porous pavements the capacity of drainage systems designed before 2007 is not sufficient to cope with additional rainfall due to climate change. In KernGIS, the corresponding completion date of the gutters will be 2010. A considerable spare capacity may be found in the amount of spray reducing discharges to the drainage system [12]. Reliable figures are lacking for spray during heavy showers. Consequently, all bridges with non-porous pavements and completion dates before 2010 should still be considered blue spots.
  Specialists of the RWS Centre for Infrastructure suggest to focus further elimination of blue spots on the longer bridge decks, such as found in fly-overs and bridges crossing large waters.
- Inspection and maintenance of weak spots in the drainage systems will become more critical. Notorious weak spots are found at the connection of gutters on bridge decks to sewers in the embankments.

The locations of the remaining blue spots are depicted in map C1. Discrepancies may exist between the KernGIS data used for map C1 and the pavement type data obtained from DVS. For instance at the Moerdijkbrug, the KernGIS data suggest a porous pavement, whereas the DVS data indicates DAB, a non-porous pavement type. The actual pavement type may be closer to a epoxy bound surface treatment.

Also, the actual pavement type is missing or not specified in KernGIS for some highway sections that have recently been reconstructed, such as A2 Amsterdam-Utrecht, A12 Zoetermeer-Gouda and A73 Venlo-Maasbracht. Although the final pavement type will most likely be porous asphalt, bridges in these highway sections are currently classified as potentially vulnerable on map C1 .

7.2.2.3 *Storage capacity of the centre verge*

The storage capacity of the center verge was determined, with the following assumptions:

- A width of the centre verge of 3.4 m, the value given for highways in a limited space [20].
- A depth of the centre verge of 50 mm, the thickness of the wearing course.
- Conservatively, no infiltration into the soil is assumed.

The volume that can be contained in the verge itself is very limited. The emergency strip ('redresseerstrook') of 1.40 m to the left of leftmost lane can be added to the buffer volume, as depicted in Figure 7.8.

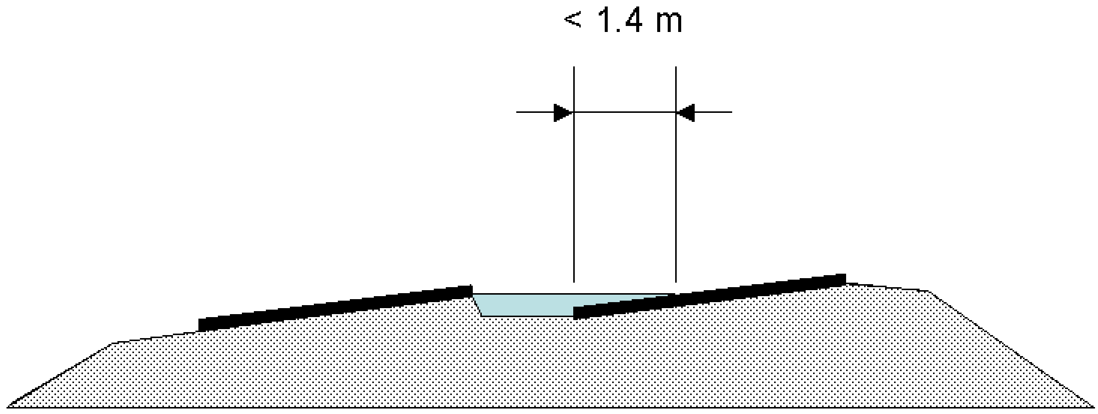

Figure 7.8 Storage in the centre verge

An analysis was made to determine if storage in the centre verge could offset the increase in rainfall due to climate change. The difference between the Braak design curve and the Buishand W design curve (10 years return period) represents a maximum additional rainfall of 8.4 mm. The storage capacity depends on the width of the pavement capturing the rainfall, and the transverse slope of the pavement. The higher the transverse slope, the larger the volume of water that can be stored in the 'pool' in Figure 7.8. Table 7.10 presents the minimal transverse slopes required to store the additional rainfall in the center verge.

Number of trafficked lanes	Pavement width [m]	Minimal transverse slope
1	8.30	-0.45%
2	12.00	0.47%
3	15.70	1.40%
4	19.40	2.32%
5	23.00	3.22%

Table 7.10 Minimal transverse slope required to store additional rainfall in the centre verge

The analysis was performed on the spreadsheet data obtained from RWS DVS.

The conclusions are:

- 16.5 % of the network length has a transverse slope directed towards the centre verge; 12.1 % has a porous pavement, 4.4 % has a nonporous pavement.
- For porous pavements, the additional rainfall due to climate change will be stored in the porous pavement itself (6.7 mm) and in the centre verge (1.7 mm). This is possible at all locations.
  Therefore, roads with a porous pavement have been eliminated as blue spots.
- For non-porous pavements all additional rainfall (8.4 mm) will be stored in the centre verge. Although partial flooding of the emergency strip will occur at many locations, the leftmost lane will be flooded along only 6.7 km (0.13 %) of the network length. The locations of the remaining blue spots are given in Appendix F.

This analysis assumes the existing drainage system in the centre verge is well maintained. With more intense rainfall, maintenance of the drainage system becomes more critical.

7.2.2.4 *Erosion of verge and slopes*

Pavements without gutters and 'normal' outward transverse slopes will discharge the runoff in the verges. If infiltration of the verge is limited by the formation of a sludge layer, most runoff will stream down the slopes.

An analysis was made of the potential erosion in case of heavy rainfall. It is assumed that:

- There is no reduction of runoff due to spray.
- There is no infiltration in verge or slope.
- The critical discharge is taken as the slope of the initial part of the Buishand W intensity-duration curve; the critical discharge occurs after saturation of the porous pavement.
- The slope is 1:2 or less.

Table 7.11 gives the discharges per meter road length.

Number of trafficked lanes	Pavement width [m]	Discharge down slope [l/s/m]
1	8.30	0.08
2	12.00	0.11
3	15.70	0.15
4	19.40	0.18
5	23.00	0.21

Table 7.11 Discharges down slope

The discharges were compared to the design rules for wave overtopping of grass covered river dikes [26]. According to the design rules the risk of erosion with the calculated discharges is very low in the following cases:

- If the quality of the vegetation cover is moderate to good, irrespective of the type of top soil.
- If the quality of the vegetation cover is poor, and the top soil has a clay percentage of 8% minimum.

However, erosion may be initiated at defects in the vegetation cover, such as rabbit holes. Because the velocity of the water flow and the duration of the event are limited, a single event is not believed to cause structural damage to the pavement and embankment. Repeated events without intermediate repairs may ultimately cause more serious damage. Therefore, inspection and maintenance will become more critical.

7.2.2.5 *Other pavement drainage systems*

The analysis of the other pavement drainage systems i.e. gutters and sewers along pavements on embankments, proceeds along the same lines as that for the drainage systems on bridges.

The following design rules for the road drainage systems were obtained from an interview with Mr. Henkjan Beukema, specialist geotechnical design at RWS Centre for Transport and Navigation, and an analysis of the design guidelines for road drainage systems [18]:

- Period before 2011: the intensity-duration curve of Braak (1933) was used, for a return period of 10 years. The design assumes that all precipitation on the pavement flows into the gutters and sewers. No explicit safety factors are assumed in the design.
- Period 2011 and later: the intensity-duration curve was adapted for the effects of climate change. The current intensity-duration curve is approximately equal to the Buishand W scenario, for a return period of 10 years. As before, the design assumes that all precipitation on the pavement flows into the gutters and sewers. No explicit safety factors are assumed in the design.

Given the time lag between a revision of the design rules and the completion of construction, it can be expected that at present no constructions yet exist that were designed according to the revised design rules.

A potential spare capacity may be found in storage in porous pavements and in spray water generated by the traffic. For reasons discussed before, the latter is not considered in this study.

The conclusions are:

- For porous pavements, the additional rainfall due to climate change will largely be stored in the porous pavement itself, which represents a potential spare capacity. This spare capacity can be used if the gutter and sewer system is well maintained. Thus, maintenance will become more critical.
  Therefore, roads with a porous pavement have been eliminated as blue spots.
- For non-porous pavements the capacity of pavement drainage systems may not be sufficient to cope with additional rainfall due to climate change. A considerable spare capacity may be found in the amount of spray reducing discharges tot the drainage system. Reliable figures are lacking for spray during heavy showers. Consequently, all gutters and sewers draining non-porous pavements should still be considered blue spots.

The locations of the remaining blue spots are depicted in map C2. It should be noted that in KernGIS the actual pavement type is missing or not specified for some highway sections that have recently been reconstructed, such as A2 Amsterdam-Utrecht, A12 Zoetermeer-Gouda and A73 Venlo-Maasbracht. Although the final pavement type will most likely be porous asphalt, gutters and sewers in these highway sections are currently classified as potentially vulnerable on map C2.

7.2.3 Limitations

in KernGIS the actual pavement type is missing or not specified for some highway sections that have recently been reconstructed. In reality, the future pavement type will most likely be porous asphalt. This will lead to a reduction of the number of potential blue spots.

The potential blue spots associated with non-porous pavements on bridge decks have been identified combining the corresponding layers in KernGIS. Because of small inconsistencies between the locations of the bridges and the pavements, the combination has been based on the absence of porous pavements near the bridge locations. This approach relies on the assumption that if the pavements on either side of the bridge are porous, the pavement on the bridge deck itself will also be porous. Some errors may have been introduced with this assumption.

The approach originally taken to eliminate potential blue spots relied on analysis of design rules used for constructions of different ages. This approach was suggested for drainage systems of tunnels and roads in excavation, and for pavement drainage on bridge decks.

Specialists of the RWS Centre for Infrastructure (DI) do not recommend this approach because of the frequent deviations of the design rules. In stead of the original approach suggested for the analysis, the specialists suggest an approach in which a team of experts makes a quick first assessment of the vulnerability on a case-by-case basis, using their knowledge of the local situation and the actual design. For pavement drainage on bridge decks the focus should be on the longer bridge decks, such as found in fly-overs and bridges crossing large waters.

For all parts of the road drainage system, maintenance will become more critical if the intensity of rainfall increases. Poor maintenance may reduce the capacity of drainage systems to such extent that even present day extreme events cannot be adequately processed. Thus, poor maintenance may be the cause of water related problems at locations not identified as blue spots.

7.2.4 Conclusion

For all mechanisms, inspection and maintenance of drainage systems, verges and embankment slopes will become more critical. Poor maintenance may reduce the capacity of drainage systems to such extent that even present day extreme events cannot be adequately processed. Thus, poor maintenance may be the cause of water related problems at locations not identified as blue spots.

The existence of blue spots due to incapacity of stormwater drainage systems has been identified for tunnels, bridges, locations where the transverse slope is directed to the center verge and other locations.

- All drainage systems of tunnels and roads in excavation designed before 2010 should be considered a potential blue spot and will need to be evaluated on a case-by-case basis.
- In the present representation all locations with gutters alongside non-porous pavements are considered potential blue spots since these stormwater drainage systems could not have enough capacity to deal with climate change. Storage in porous pavements compensates the additional rainfall due to climate change.
- For drainage systems alongside non-porous pavements discharging in the centre verge, sufficient capacity is present in the centre verge for temporary storage of the additional rainfall due to climate change, except for 6.7 km (0.13 %) of the network length. In this

approach, temporary flooding of the emergency lane left of the leftmost trafficked lane is allowed.

- Erosion of verges and slopes by runoff is likely to occur only at defects in the vegetation cover is damaged, for instance by rabbit holes. A single rainstorm event is not believed to cause structural damage to the pavement and embankment. Repeated events without intermediate repairs may ultimately cause more serious damage.

7.2.5 Recommendations for further research

Further elimination of potential blue spots is possible:

- By using the approach suggested by the specialists of RWS Centre for Infrastructure for drainage systems of tunnels and roads in excavation, and for pavement drainage on bridge decks.
- By obtaining more reliable numbers for the amount of runoff turned into spray during heavy showers for drainage systems alongside non-porous pavements.

8 Results of calibration by interviewing road administrators

Five road districts were interviewed: Den Bosch, Zuid-Hollandse Waarden, Rijnmond, Amsterdam and Utrecht.

The interviews were structured as follows. First, the interviewee was asked for known locations with water-related problems. This was done prior to showing the maps in order to prevent any prejudice with the interviewee. Second, the maps were shown and the blue spots indicated on the maps were evaluated. Conversely, it was checked whether all known locations mentioned by the interviewee are present on one the appropriate map.

Shown were draft versions of (1) maps B1 through B10, (2) the Alterra map (Map B-1.1, floodrisk due to heavy rainfall), and (3) the C-map showing the risk of waterfilms on the road after heavy rainfall (“ZOAB-map”)11. In Den Bosch also map A4 was discussed, because a stretch of highway A2 was inundated during the high river water levelsof January / February 1995.

8.1 Flooding due to failure of flood defences

The A-type maps were discussed only for the Den Bosch road district, because the other road districts have not experienced any flooding due to high water so far. The inundated stretch is indicated on map A5 as a blue spot. After the 1995 flooding, measures were taken here to prevent flooding in the future. This means that there is no actual risk anymore, but the stretch remains sensitive, as it depends on technical facilities to discharge water from the road. Map A4 should therefore not be interpreted in terms of actual risk but potential risk, c.q. dependence on technical facilities.

8.2 Flooding by intense rain and changing groundwater levels

In the road districts Amsterdam and Rijnmond, a fair amount of problematic locations were mentioned, in Den Bosch and Zuidhollandse Waarden almost none. Utrecht ranked in between. The reported problems relate mainly to (1) a stagnant discharge of rainwater, or (2) seepage of surface water or ground water through seams in the constructions of aqueducts, tunnels and deepened road stretches. No problems were reported that relate directly to shallow groundwater levels.

It turned out that the draft versions of maps B-2, B-6, B-8, and B-10 were most useful to evaluate. These maps represent the present situation and are most easily confronted with field experience within the road districts. The maps representing the future climate situation were shown, but not discussed in detail for this reason. The Alterra map turned out to be not distinctive enough to be used on the desired scale. Instead, map B-10 proved to be very useful, and many deep-lying stretches were recognized as such. Map B-1 had little added value to map B-10 in the context of the interviews[12].

The deep-lying stretches shown on map B-10 were all recognized. Broadly, they can be divided into two categories: (1) road cuts through push ridges and cover sand ridges, mainly

11. *This map has been updated after the interviews; the presented map in this report is not the same map as presented during the interviews (storage in asphalt has been taken into account in this report, based on the interview results).*

12 *This concerns the B-1 map version included in the interim report. In the definitive version, map B-1 was significantly improved.*

in the higher eastern and southern parts of The Netherlands, and (2) deepened road constructions, tunnels and aqueducts, mainly in the northern and western parts. There are three type 1 deep stretches present in the interviewed districts (all in Utrecht), and no problems are reported here with stagnant rainwater, because of the high permeability of the soil and deep groundwater levels (hence: sufficient soil water storage capacity). As for type 2 deep stretches, it depends on the quality of the construction and / or rainwater discharge facilities whether there is an actual problem relating to stagnant rainwater.

The major part of the blue spots on the groundwater-related map B-2, and all blue spots on map B-6, could be attributed to deep-lying stretches as shown on map B-10. It depends on the quality of the construction whether there is an actual problem relating to groundwater. In general this is not the case, however there are exceptions showing seepage of groundwater through seams, as described earlier in this section. A smaller part of the blue spots on map B-2 are located in areas with a small freeboard[13]. The majority of these stretches is equipped with drainage and / or pumping facilities to prevent groundwater problems.

The areas sensitive to soil subsidence shown on map B-8 turned out to partly coincide with road stretches that have to be re-leveled relatively frequently. In some subsidence-sensitive areas, no more frequent re-leveling is reportedly necessary, however. This may be due to recent reconstructions and associated road foundation improvements. Conversely, some road stretches need frequent re-leveling while they are not shown as ‘’sensitive’’ on map B-8. In the Rijnmond road district, it is suspected by the interviewee that truck overloading may be a cause for this. Especially the A29 experiences intense and heavy traffic to and from the port of Rotterdam.

The main point of attention for map B-10 is the fact that a number of deepened road sections are not shown, some of which not recently built. This was noted for one of the interviewed districts, and further supported after confrontation with known deepened stretches elsewhere in the Netherlands.

The stretches marked as “tunnel” on map B-10 are tunnels in the constructive sense, i.e., the road is overarched or roofed over a certain minimum distance. It may be built directly on the surface, and in that case it is not a tunnel in the hydrological sense. This explains the rather large amount of tunnels present on the map. It turned out that some non-recent piled road foundations (A27, A20) are not shown on map B-10. The A27 piled road actually is a low-lying bridge and not a piled embankment. As such it is not potentially vulnerable for the effects of more intense rainfall.

8.3 Flooding by incapacity of stormwater drainage and road surface

Some of the blue spots shown by the risk map of waterfilms on the road (“ZOAB-map’’) are well recognized, but the overall impression is that the map overestimates the number of blue spots in the present situation. The interviewed road administrators stated that replacement of pavement by porous asphalt improved the hydraulic performance significantly and eliminated earlier notorious blue spots. The storage of the first part of stormwater in the porous asphalt surely has a positive effect on the prevention of flooding and therefore has been accounted for in the analysis as presented in the current report. Better performance than calculated may also occur due to the presence of additional facilities for the discharge of rainwater, or recent reconstructions with (probably) improved rainwater discharge design. Stretches with actual water film problems but not shown on the map are very exceptional.

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13 *Freeboard = difference between surface level and surface water level*

8.4 Conclusions

Interviewing road districts has proven to be an important step in the validation of the results as presented in this report. The interviews carried out up until now are sufficient for drawing the general conclusions stated in this report. One or two interviews with districts in geographically different regions might provide added value. In that case the districts of Twente (push ridges and shallow clay layers) and South Limburg (sloping terrain, erosion) are suggested. If specific verification is desired for all the potential blue spots identified on the maps, all road districts have to be inquired.

The interviews learned:

- There is little experience with type A flooding in the interviewed road districts. Map A5 should not be interpreted in terms of actual risk but potential risk, c.q. dependence on technical facilities.
- Likewise, the maps showing type B flooding must be interpreted in terms of potential risk rather than actual risk. The actual risk is determined by the current quality of deepened road constructions and / or rainwater discharge facilities. In general, there is no actual risk, but there are a number of exceptions, notably in the districts of Amsterdam and Rijnmond.
- Map B-8 showing soil subsidence partly explains the actual consolidation problems in some districts, but some consolidation problems appear to determined by other (as yet unknown) factors. A possible suspect is heavy trafficking combined with truck overloading.
- The risk map of waterfilms on the road (map C3) that was shown during the interviews appears to overestimate the number of blue spots in the present situation. Based on the interviews, the analysis for run-off has been reassessed and calibrated on the interview results. The map that is currently presented in this report reflects the experiences of the 5 road districts.
- The interviews were useful in confirming the expectation that drainage facilities mitigate many of the potential risks. However, some exceptions were reported showing actual water-related problems. Stretches with reported actual water-related problems but not shown on one of the maps are very exceptional.

9 Conclusion

This report provides the results of a study by Deltares, commissioned by Rijkswaterstaat Centre for Transport and Navigation, on the identification of vulnerable spots due to flooding in the Dutch National Road Infrastructure Network with regard to climate change. Detailed results of the analyses are presented in chapters 5 to 7. In this conclusion chapter the over-all conclusions are repeated.

Risk assessment in order to gain insight in the acceptability of the risks of the blue spots was not part of the current research. Chapter 9.6 provides a short description of the activities that could be performed in such a risk assessment.

9.1 Objectives and basic assumptions

The objectives of the presented study are:

- To identify the vulnerable spots to flooding on the Dutch National Highway Network.
- To analyse the probability of flooding. Both now and in 2050, based on the worst case KNMI climate change scenario for each type of flooding.

Three different types of flooding are analyzed:

- Flooding due to failure of flood defences.
- Flooding by intense rain and changing groundwater levels.
- Flooding by incapacity of stormwater drainage and road surface.

In this report, a vulnerable spot to flooding is called a blue spot, with the following definition: A blue spot is a location on the Dutch National Highway Network that can be flooded in certain circumstances. A blue spot only refers to the probable cause of flooding and not to the consequences. Therefore the identification of a blue spot does not by definition mean that the risk of flooding on that location is unacceptable. Risk assessment will be the objective of further studies.

9.2 General conclusion

Table 9.1 provides an overview of the results of all analyses of the current investigation. At first, the current vulnerability is shown. Secondly, the effect of climate change, using the worst case climate change scenario for 2050 (see appendix C), is shown.

Table 9.2 gives insight in the impact of the different flooding types. It should be noted that the effects of the different flooding types are very different.

- For flooding type A it is shown that large part of the Dutch highways can be affected to such an extent that it is not possible to use the road anymore over large distances and over long periods of time.
- For flooding type B the roads will only be locally affected, but still for longer periods. Some special objects can become less available or even unavailable. Although these are only local points on road trajectories, a whole trajectory still can become unavailable.
- For flooding type C it is shown that the roads will be locally affected only for short periods of time. The effects of flooding type C are quite different from the effects of flooding type

B and C. Flooding type C will not lead to a complete unavailability of the road, except for tunnels and deep lying sections.

Type of flooding			Results		
			Current vulnerability	**Probability**	**Effect of climate change**
A	Failure of flood defences	Flooding from sea and large rivers	Almost every highway inside dike ring areas is affected	ranging from 1:10000 to 1:1250	no change of probability, consequences will be larger
		Flooding from small rivers/canals	Highways are locally affected	ranging from 1:1000 to 1:100	no change of probability, consequences will be larger
B	Water system in the area around the road is not capable for drainage / discharge of water	Pluvial flooding (overland flow after precipitation)	Only situation in 2050 is considered; the vulnerability is negligible	1:100	
		Increase of groundwater levels	“none”: depending on maintenance state	almost every year (use of mean highest water levels)	Limited effect, probably only for some special objects
		Increase of aquifer hydraulic heads	“none”: depending on maintenance state	almost every year (use of mean highest water levels)	no effect
C	Road surface not capable for enough drainage / discharge of water	Run-off on the road	1,4% of total road length affected	1:10	3,3% of total road length affected
		Flooding of the storm water drainage system	“none”: depending on maintenance state	ranging from 1:250 to 1:10	tunnels, roads in excavation and roads with non porous asphalt become potentially vulnerable

Table 9.1 Summary of results

It is concluded that climate change only leads to an increase of probabilities of the flooding types with a more local impact. Locally also the consequences in terms of the number of blue spots will increase due to climate change. It is important to take in mind the worst case climate change scenario for each type of flooding has been used.

Due to the effects of climate change almost all locations become more charged, when considering flooding types B and C because of existing overcapacity and safety in the design many locations will not become a blue spot. However poor maintenance can reduce the resistance against climate change. Therefore good maintenance and asset management will become more critical due to climate change.

<table>
<tr><th colspan="3" rowspan="2">Type of flooding</th><th colspan="2">Impact</th></tr>
<tr><th>Duration</th><th>Affected road length[14]</th></tr>
<tr><td rowspan="2">A</td><td rowspan="2">Failure of flood defences</td><td>Flooding from sea and large rivers</td><td>Weeks to months</td><td>Whole road network</td></tr>
<tr><td>Flooding from small rivers/canals</td><td>Days to weeks</td><td>Several kilometers</td></tr>
<tr><td rowspan="3">B</td><td rowspan="3">Water system in the area around the road is not capable for drainage / discharge of water</td><td>Pluvial flooding (overland flow after precipitation)</td><td>Hours to a day</td><td>Length of a tunnel entry of deepened road section</td></tr>
<tr><td>Increase of groundwater levels</td><td>Weeks to months</td><td>Length of a special object</td></tr>
<tr><td>Increase of aquifer hydraulic heads</td><td>Weeks to months</td><td>None</td></tr>
<tr><td rowspan="2">C</td><td rowspan="2">Road surface not capable for enough drainage / discharge of water</td><td>Run-off on the road</td><td rowspan="2">Minutes to hours</td><td rowspan="2">Several meters to kilometers</td></tr>
<tr><td>Flooding of the storm water drainage system</td></tr>
</table>

Table 9.2 Impact of an occurring event, for the different types of flooding in terms of duration and affected road length

9.3 Flooding due to failure of flood defences

For flooding due to failure of the primary defence structures it is shown that almost every highway inside the dike ring areas can be affected. However this is a (worst case) compilation of several flooding events, which in reality are highly unlikely to occur at the same time.
Therefore differentiation of the cause of flooding is done by discerning vulnerability of the highway for coastal flooding and vulnerability of the highway for fluvial flooding. In the provided maps it is presented how much time it takes before highways are actually flooded after failure f the defence structures.

Currently it is only possible to assess the vulnerability of the highways to flooding due to failure of regional defence structures, based on regional flooding scenarios, for the province of Zuid Holland. For this province it was shown that the highways are affected. The probability of flooding of regional defence structures is generally higher than the probability of flooding of the primary defence structures.
Additionally a qualitative estimate of the risk level for all highways in the Netherlands is provided for flooding caused by failure of regional defence structures. Based on this quickscan, it can be concluded that highways in the whole of the Netherlands can be affected by flooding of regional defence structures. This quickscan identifies most of the potential blue spots, but is not robust (i.e. not all spots are identified).

9.4 Flooding by intense rain and changing groundwater levels

Pluvial flooding appears to be a negligible risk for highways. It is concluded that the risk of pluvial flooding is generally low (lower than presented by Alterra in 2009). Tunnel and aqueduct entries and deep lying sections show a higher potential risk, however as a rule such stretches are equipped with drainage and/or pumping facilities. Also roads in excavation in slightly accidented terrain show a higher potential risk. Here the unfavourable topographic setting is often compensated by favourable infiltration conditions and low groundwater tables.

14. *It is noted that this is the impact of one occurring event and not the potential impact of all possible occurring events.*

It is concluded that groundwater effects caused by climate change on the Dutch highways are limited. Stretches with groundwater level increases in 2050 generally have no overlap with highway stretches currently at risk, with the exception of stretches located at the foot of push ridges and cover sand ridges.
In a more detailed analysis specific attention has been given to special objects (EPS, foamed concrete and MSW slag fills), tunnels and deep lying sections, since locations on the highway with these constructions are most vulnerable to a changing groundwater table. Based on currently available general data it is difficult to perform such a detailed analysis. For most locations (107 out of 156) a specific analysis on a case-to-case basis (based on actual design information of the objects) is necessary. 22 locations are assessed to have a high priority in that research. For the other 49 locations with such objects it was possible to confirm that these locations are not vulnerable to a possible change of the groundwater table due to climate change

The risk of a rise of aquifer hydraulic heads due to climate change on the Dutch highways is estimated as low. Road stretches are identified that currently show hydraulic heads in the first aquifer higher than the road surface. These stretches represent a theoretical risk of uplift or heave but are probably designed for this purpose as being tunnels and excavated road stretches. None of these stretches however show a head increase by 2050 due to climate change.

Land subsidence is not expected to lead to an increase of the risks of pluvial flooding, rise of groundwater tables and rise of aquifer hydraulic heads on the Dutch highways. On the contrary, it can even be stated that land subsidence leads to a decrease of these risks.

9.5 Flooding by incapacity of stormwater drainage and road surface

It appears that the number of blue spots more than doubles due to climate change, being present at 3,3% of the total road length. Blue spots are mostly present at locations with a change of transverse slope, many lanes and closed pavements.
This conclusion is based on the use of a simple calculation model with blue spots being defined as the development of a waterfilm with a thickness exceeding 3 centimetres.

For stormwater drainage, inspection and maintenance of drainage systems, verges and embankment slopes will become more critical. Poor maintenance may reduce the capacity of drainage systems to such extent that even present day extreme events cannot be adequately processed. Thus, poor maintenance may be the cause of water related problems at locations not identified as blue spots.

The existence of blue spots due to incapacity of stormwater drainage systems has been identified for tunnels, bridges, locations where the transverse slope is directed to the center verge and other locations.

- All drainage systems of tunnels and roads in excavation designed before 2010 should be considered a potential blue spot and will need to be evaluated on a case-by-case basis.
- In the present representation all locations with gutters alongside non-porous pavements are considered potential blue spots since these stormwater drainage systems could not have enough capacity to deal with climate change. Storage in porous pavements compensates the additional rainfall due to climate change.
- For drainage systems alongside non-porous pavements discharging in the centre verge, sufficient capacity is present in the centre verge for temporary storage of the additional rainfall due to climate change, except for 6.7 km (0.13 %) of the network length. In this

approach, temporary flooding of the emergency lane left of the leftmost trafficked lane is allowed.

- Erosion of verges and slopes by runoff is likely to occur only at defects in the vegetation cover is damaged, for instance by rabbit holes. A single rainstorm event is not believed to cause structural damage to the pavement and embankment. Repeated events without intermediate repairs may ultimately cause more serious damage.

9.6 Further use of results

Recommendations for further research have been listed in the chapters 5 through 8 of this report.

Many identified potential blue spots will probably prove not to be real blue spots due to limitations of the research and knowledge gaps. As stated, case to case studies need to be done to get more insight. It is not recommended to start with these studies, before the results of a risk assessment (see below) are present. The level of detail achieved in the current study should be sufficient to perform such a risk assessment. Consequently, if a risk is classified as being unacceptable, one of the measures can be to start more detailed research on these specific locations to be sure whether these locations actually will be blue spots.

In the current study locations of potential blue spots have been identified. However, these blue spots only refer to the probable cause of flooding. The consequences are not yet analyzed and therefore it is not yet possible to estimate the risk of flooding, the risk being a combination of cause and consequence.
A risk assessment is a logical following step. For instance the RIMAROCC framework [4] can provide a good basis for such a risk assessment. In general the following activities can be executed with use of this framework:

- The risks can be classified (according to probability and consequences of flooding).
- It can be checked whether the risks are acceptable.
- If the risks appear not to be acceptable measures can be identified.
- Together with a suitable adaptation strategy in which measures are prioritized and aligned.

References

1 Alterra (2009) Quickscan klimaat en landelijk snelwegennet. Alterra-rapport, ISSN 1566-7197, Wageningen.

2 Alterra (2009-2) Effecten van klimaatverandering op de watervraag in de Nederlandse groene ruimte. Alterra rapport 1791, Wageningen.

3 Badger, J.E., Investigating the hydroplane phenomenon, PNS A&I Co Ltd, Bloomington, Indiana, 1990.

4 Bles Thomas (Deltares), Ennesser Yves (Egis), Fadeuilhe Jean-Jacques (Egis), Falemo Stefan (SGI), Lind Bo (SGI), Mens Marjolein (Deltares), Ray Michel (Egis), Sandersen Frode (NGI), RIMAROCC (Risk management for roads in a changing climate), A Guidebook to the RIMAROCC Method, final version 30-09-2010.

5 Buishand, A. en Wijngaard, J., 2007: Technical report ; TR - 295 Statistiek van extreme neerslag voor korte neerslagduren, KNMI, De Bilt.

6 Chesterton, J. et al, The use of the Gallaway formula for aquaplaning evaluation, NZIHT & Transit NZ 8th annual conference, 2006.

7 De Lange, G., Oosthoek, J.T. and Gunnink, J.L.T., 2011. Maaivelddalingsprognose voor 2050, onderdeel van de scenarios van het Deltamodel, Deltares, Utrecht.

8 DWW, Verbetering aanvangsstroefheid Tweelaags ZOAB Proefvakken A15 te Leerdam, Eindrapport IPG 0500218, December 2005, DWW-2005-086.

9 Federal aviation administration, Department of Transportation, Airport drainage, July 1970.

10 Future Water 92008) Klimaatverandering en lokale wateroverlast ten gevolge van extreme neerslag in Nederland. Future water rapport no. 73, Wageningen.

11 Gallaway, B. M., et. al., Pavement and Geometric Design Criteria for Minimizing Hydroplaning, Federal Highway Administration, Report No. FHWARD-79-31, 1979.

12 Gallaway, B.M., R.E. Schiller Jr. and J.G. Rose, The effect of rainfall intensity, pavement cross slope, surface texture, and drainage length on pavement water depths, Texas Transportation Institute, Texas Highway Department, May 1971.

13 Grinsven, W.G.M.A. van; Muiswinkel, C. van, 2010. Motorway runoff in the Netherlands, runoff and spray pollution, legislation and prevention measures. Proc. ConSoil 2010, Salzburg, Austria.

14 Klein Tank, A.M.G. en G. Lenderink (red.), 2009: Klimaatverandering in Nederland; Aanvullingen op de KNMI'06 scenario's, KNMI, De Bilt.

15 NASA, W.B. Horne and R.C. Dreher, Langley research center, Phenomena of pneumatic tire hydroplaning, NASA technical note NASA TN D-2056, Washington DC, November 1963.

16 NEN-EN 12697-19; Bitumineuze mengsels Beproevingsmethoden voor warm bereid asfalt Deel 19: Doorlatendheid van proefstukken; Brussel 2004.

17 Overeem, A., 2009: Climatology of extreme rainfall from rain gauges and weather radar, thesis, Wageningen University, Wageningen.

18 Rijkswaterstaat Centre for Transport and Navigation, 2011. Component Specifications for Embankments. Version 4.4, 22 December 2011.

19 Rijkswaterstaat, Guidelines Road Construction – Design Drainage Systems (Dutch: Handleiding Wegenbouw – Ontwerp Hemelwaterafvoer); ISBN 903690071 9, Dienst Weg- en Waterbouwkunde, Delft, juni 1988.

20 Rijkswaterstaat, 2002. Kentallen Rijkswaterstaat. Deel IV, Reconstructie droge infrastructuur, hoofdelement wegvakken.

21 Rohlfs, R., Ontstaan van afwateringsproblemen, Afwatering in wentelende verkantingsovergangen 1, PT Civiele techniek 39 (1984), nr. 6.
22 Rohlfs, R., J.J.A. Gerardu, J. van Zwieten, Voorkomen en oplossen van afwateringsproblemen, Afwatering in wentelende verkantingsovergangen 2, PT Civiele techniek 39 (1984), nr. 7/8.
23 RWS, NOA Nieuwe Ontwerprichtlijn Autosnelwegen, Rijkswaterstaat Adviesdienst Verkeer en Vervoer, january 2007.
24 SCW, 1977. Water op de weg. Publication L, A.G. Welleman, Delft.
25 Steen, T. van der, EFFECTEN GRADERING OP DE FUNCTIONELE EIGENSCHAPPEN VAN TWEELAAGS ZOAB, TUDelft 2005.
26 TAW, 1994. Guideline Constructive Design, annexes 2F – erosion of the inner slope. Delft.
27 WL/Delft Hydraulics, afwatering landingsbaan Amsterdam Airport Schiphol, relatie regenintensiteit en waterlaagdikte, verslag fysisch onderzoek, Delft, september 1999
28 WLO, 2006. Centraal Planbureau, Milieu- en Natuur Planbureau en Ruimtelijk Planbureau. Welvaart en Leefomgeving - een scenariostudie voor Nederland in 2040.

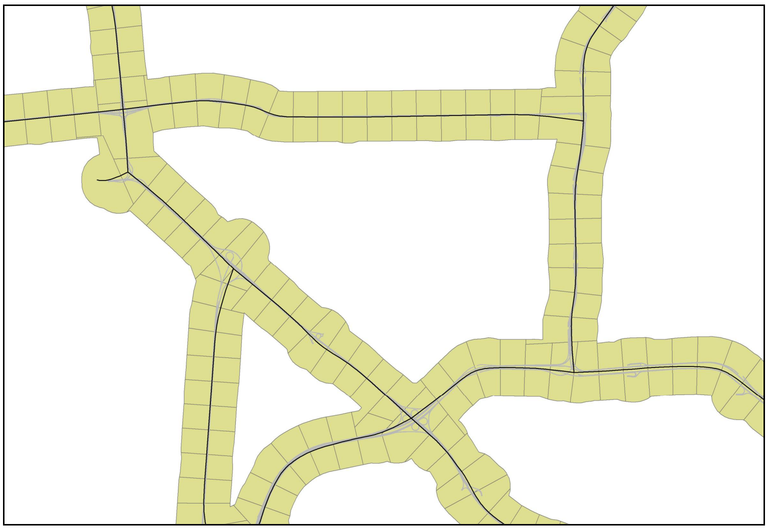
A GIS method for determining the vulnerability for flooding due to failure of a defence structure

A.1 Used data

- KernGis20110115.
- Digital Topografic Data (in Dutch 'DTB').
- Dike ring areas (Dijkringen bestand 3.2).
- BPS_banen.shp.
- Most recent "RiskMap" (grid 100 x 100 meter).
- Results of flood simulations for unprotected areas (in Dutch 'buitendijkse gebieden').
- Regional levees shapefile (if available).
- Filtered Dutch Elevation Model (in Dutch 'AHN') from TNO (100 x 100 meter).
- Shape file of polders (if available).

A.2 Calculation of the potential water depth on the road due to flooding of the primary defence system (map A1 and A2)

- From the DTB all the mark lines of the national roads are selected.
- These mark lines contain elevation data, This data is converted to a point shape.
- A buffer polygon is created form the "BPS_banen.shp" with a buffer distance of 500 meter. The BPS_banen shape is part of the dataset "KernGis20110115".
- Within the buffer polygon a central line is generated.
- On the central line, perpendicular lines were generated every 500 meter. A small number of manual corrections was performed. See example below:



- Based upon the constructed polygon shape, the lowest point of the road (ramps and exits included) is calculated for every area (of 500 by 500 meter).
- The value of the lowest point is assigned to the corresponding part of the central line.
- Within the buffer zone the data of the flooded area is converted to a point shape with water depths.
- The water depth points are linked with the nearest central line segment.
- The water level is calculated by combining the surface elevation (based on AHN) and the water depth for each point in the water depth point shape.
- For each central line segment the median of the water levels is calculated.

- The potential water depth on the road is determined by the median of the water level and the lowest level of the road.

A.3 Calculation of the potential water depth on the road due to flooding of the regional defence system (map A5)

- Create within the shape file of the polders a new attribute with water level based on the frequency of exceedance for each polder.
  I = 1/10 year
  II = 1/30 year
  III = 1/100 year
  IV = 1/300 year
  V = 1/1000 year
- Intersect the already available segments of the central line (of about 500 meters length with the minimal level of the road)) with the shape file of the polders created in step 1.
- Calculate for all segments the potential water depth on the lowest level of the road.

A.4 Quick scan method (map A6)

- Calculate the distance (with GIS near function) of the segments of the central line of the road to the levees of the regional system in the shapefile.
- For each segment of the central line of the road, the median of the elevation is calculated from the data of the national elevation model (AHN) within the buffer of 500 meter Calculate the median
- Create a new attribute (indication of risk) for each segment of the central line, based on the combination according to the table (see below).

	Levels of the highway compared to the surface level of the surrounding					
Distance to regional defence structure / regional water system	**< 0**	**0 - 0,25 m**	**0,25 - 0,75 m**	**0,75 - 1,5 m**	**1,5 - 3 m**	**> 3 m**
< 100 meter	5	5	5	4	2	1
100 - 500 meter	5	5	4	3	2	1
500 - 1 km	5	5	4	3	2	1
1 - 3 km	4	4	3	2	1	1
3 - 5 km	4	3	2	1	1	1
> 5 km	3	2	1	1	1	1

	Risk:
5	very high
4	high
3	moderate
2	small
1	Very small

B Groundwater levels in South Limburg

Highest groundwater levels derived from visual inspection of observation wells in South Limburg. All values are rounded to integer values to avoid a suggestion of too much detail.

Observation well ID	Location near highway	Surface elevation (m NAP)	Highest groundwater level (m NAP)	Monitoring period	Filter depth (m NAP)
B61H0084	A2 Eijsden	63	51	1980-2004	33-49
B61H0066	A2 Oost-Maarland	57	49	1987-2011	43-48
B61F1357	A2 Heer	50	47	1979-2009	41-42
B61F1362	A2 Maastricht	47	45	1979-2010	39-?
B61F0301	A2 Kruisdonk	50	47	1982-2006	38-40
B60C0860	A2 / A76 Kerensheide	65	47	1987-2011	46-49
B60C0029	A2 Graetheide	48	38	1952-2008	29-30
B60C0003	A2 Born	43	38	1952-2008	35-36
B60A1746	A2 Holtum	30	>29	1984-2011	24-26
B60A0352	A2 Holtum	30	>29	1982-2005	8-10
B60A0357	A2 Roosteren	28	>27	1984-2011	15-17
B60C0838	A76 Geleen	72	49	1980-2002	43-48
B60C1164	A76 Geleen-Oost	64	55	1997-2011	52-54
B60C0839	A76 Schinnen	76	74	1980-2011	61-66
B62B0837	A76 Hoensbroek	80	76	1980-2002	67-70
B62B0904	A76 Voerendaal	91	87	1992-2008	83-85
B62B0902	A79 Voerendaal	98	91	1992-2011	83-86
B62B0987	A79 Klimmen	110	108	1980-2011	100-101
B62A0440	A79 Hulsberg	134	114	1994-2011	113-116
B62A0449	A79 Valkenburg	73	62	1996-2011	52-57
B62A0391	A79 Meerssen	51	49	1987-2011	44-48

C Used information per map

In the table below is summarized what information is used to produce each map in the next appendices. Of course also road information is used. This information is described in chapter 2

Analysis	Map	Source of information	Time scale
Failure of primary defence structures	A1	"Risk Map"	Present situation
	A2	"Risk Map"	Present situation
	A3	"Risk Map" fluvial scenarios (river)	Present situation
	A4	"Risk Map" coastal scenarios (sea/lake)	Present situation
Failure of secondary defence structures	A5	ROR- regional scenarios Provence South Holland	Present situation
	A6.1	ROR- regional defence structures / regional water systems South Holland	Present situation
	A6.2	ROR- regional defence structures / regional water systems	Present situation
Pluvial flooding	B1	Alterra [1]	2050, Scenario W
Excess groundwater tables	B2	NHI (version 2.1), layer 1	Present situation
	B3	NHI (version 2.1), layer 1	2050, Scenario GGE translated to Scenario W
	B4	NHI (version 2.1), layer 1	Difference between present situation and interpreted scenario W (2050)
	B5	NHI (version 2.1), layer 1	Difference between present situation and interpreted scenario W (2050)
Excess hydraulic heads	B6	NHI (version 2.1), layer 2	Present situation
	B7	NHI (version 2.1), layer 2	Difference between present situation and interpreted scenario W (2050)
Soil Subsidence	B8	De Lange [6]	2050, no climate change
	B9	De Lange [6]	2050, scenario W+
Capacity of drainage systems of bridge decks	C1	KernGIS, interviews	2050, scenario W
Capacity of drainage systems of other pavements	C2	KernGIS, interviews	2050, scenario W
Water film on road surface at storm water run-off	C3	IVON, literature	current and 2050, scenario W

D Maps

E Remaining potentially vulnerable special objects

E.1 Tunnels, aquaducts and roads in excavation

* Legend for importance:
1. Main highway lanes or connection roads (i.e. between two highways).
2. Roads with an important function in accessibility, i.e. highway entry and exit ramps, bus lanes.
3. Auxiliary roads, i.e. for services areas.
4. Secondary roads.
5. Fills not supporting pavements, i.e. noise wall.

Highway	Km	Region	Location	Type	STEP 3: importance*	STEP 4: expected groundwater rise
A2	73.0	UT	Verdiepte weg in de Burg Jhr Hoeufftlaan	road in excavation	1	1
A2	139.6	NB	Verdiepte weg bij gemeente Best	road in excavation	1	1
A50	127.1	NB	Verdiepte weg in de Loo	road in excavation	1	1
A50	126.9	NB	Verdiepte weg in de Menzel	road in excavation	1	1
A50	104.0	NB	Verdiepte ligging in de Rijksweg onder de Nijnselseweg (incl. riolerings- en drainagesystemen)	road in excavation	1	1
A58	101.4	NB	Westelijk open deel	road in excavation	1	1
A58	90.6	NB	Westelijk verdiepteweg in de Gordelweg	road in excavation	1	1
A58	90.6	NB	Oostelijk verdiepte weg in de Gordelweg	road in excavation	1	1
A73	15.5	LB	Tunnel in de rijksweg	tunnel	1	1
A1	157.3	ON	Verdiepte weg onder Oosterbosweg, spoorlijn en Schildweg	road in excavation	1	2
A2	111.6	NB	Tunnelbak in de Brug. Godschalxstraat	road in excavation	1	2
A3	6.9	ZH	Tunnelbak in de weg	road in excavation	1	2
A4	7.1	NH	Oostelijke tunnels in de rijksweg onder de start-, landings- en rolbaan	tunnel	1	2

Highway	Km	Region	Location	Type	STEP 3: importance*	STEP 4: expected groundwater rise
A4	7.1	NH	Westelijke tunnels in de rijksweg onder de start-, landings- en rolbaan	tunnel	1	2
A4	7.1	NH	Tunnels in de rijksweg t.b.v. dienst verkeer, fietsers en openbaar vervoer	tunnel	1	2
A4	73.0	ZH	Westelijke tunnel in de rijksweg onder de Nieuwe Maas	tunnel	1	2
A4	73.0	ZH	Oostelijke tunnel in de rijksweg onder de Nieuwe Maas	tunnel	1	2
A4	21.4	NH	Westelijk aquaduct Ringvaart van de Haarlemmermeerpolder	aquaduct	1	2
A4	21.1	NH	Oostelijk aquaduct Ringvaart van de Haarlemmermeerpolder	aquaduct	1	2
A7	127.0	NN	Tunnel onder het Prinses Margrietkanaal	aquaduct	1	2
A9	51.5	NH	Wijkertunnel onder het Noordzeekanaal	tunnel	1	2
N11	7.2	ZH	Verdiepte weg voor voetgangers en fietsers	road in excavation	1	2
N11	12.5	ZH	Aquaduct Alphen a/d Rijn	aquaduct	1	2
N14	14.2	ZH	Tunnel tussen Prins Benhardlaan en Vlietweg	tunnel	1	2
A15	73.8	ZH	Tunnel onder de Noord	tunnel	1	2
A16	60.6	NB	Tunnelbak onder de rijksweg in de verlegde Leursebaan	road in excavation	1	2
A16	35.2	ZH	Oostelijke / westelijke tunnelbak in de Glazenstraat	road in excavation	1	2
A20	25.8	ZH	Verdiepte weg in de toerit rw20 onder de Spoorlijn Delft-Schiedam	road in excavation	1	2
A20	25.9	ZH	Verdiepte weg onder de Spoorlijn Delft-Schiedam	road in excavation	1	2
A20	41.2	ZH	Aquadukt Ringvaart Zuidplaspolder	aquaduct	1	2
A22	12.0	NH	Velserverkeerstunnel onder het Noordzeekanaal	tunnel	1	2

Highway	Km	Region	Location	Type	STEP 3: importance*	STEP 4: expected groundwater rise
A27	77.5	UT	Verdiepte ligging in de rijksweg	road in excavation	1	2
A32	10.0	ON	Oostelijke verdiepte weg onder spoorlijn Meppel Hoogeveen	road in excavation	1	2
A32	10.0	ON	Westelijke verdiepte weg onder spoorlijn Meppel Hoogeveen	road in excavation	1	2
A58	139.8	ZL	Tunnel onder het Kanaal door Zuid-Beveland	tunnel	1	2
A1	161.9	ON	Verdiepte weg in rijksweg	road in excavation	1	3
A15	46.6	ZH	Tunnel in de rijksweg onder de Oude Maas	tunnel	1	3
A16	59.3	NB	Verdiepte weg in de rijksweg t.h.v. hm. 59,3	road in excavation	1	3
A16	33.3	ZH	Tunnel onder de Oude Maas	tunnel	1	3
A22	15.2	NH	Verdiepte weg onder de rijksweg	road in excavation	1	3
A29	13.9	ZH	Oostelijke tunnel in het rijwielpad onder de Oude Maas	tunnel	1	3
A31	54.4	NN	Aquaduct Langdeel	aquaduct	1	3
A32	65.7	NN	Aquadukt in de Boorne	aquaduct	1	3
A32	61.6	NN	Aquaduct onder het Pr. Margrietkanaal	aquaduct	1	3
A32	56.0	NN	Verdiepte weg in de oude rijksweg onder rijksweg 32	road in excavation	1	3
A50	120.6	NB	Verdiepte weg in de zuidelijke randweg onder de rijksweg	road in excavation	1	3
A58	73.4	NB	Ongelijkvloerse kruising rijksweg	road in excavation	1	3
A73	38.5	LB	Verdiepte weg nabij Tegelen	road in excavation	1	3
A5	5.8	NH	Verdiepte weg in de hoofdweg west in beheer bij gemeente	road in excavation	4	3
A5	5.8	NH	Verdiepte weg in de hoofdweg oost in beheer bij provincie	road in excavation	4	3

Table E1 Remaining potentially vulnerable tunnels, aquaducts and roads in excavation

E.2 EPS and foamed concrete fills, piled embankments and MSW slag fills

* Legend for importance:

1 main highway lanes or connection roads (i.e. between two highways)
2 roads with an important function in accessibility, i.e. highway entry and exit ramps, bus lanes
3 auxiliary roads, i.e. for services areas
4 secondary roads
5 fills not supporting pavements, i.e. noise wall

** These MSW slag fills are vulnerable at present

Highway	Region	Location	Type	STEP 3: importance *	STEP 4: expected ground-water rise	Remarks
A2	UT	Buttress NE of KW9 A2 Beesd	piled embankment	1	1	
A2	UT	Buttress SE of KW9 A2 Beesd	piled embankment	1	1	
A4	NB	Bergen op Zoom, conmnection to ring road N	MSW slag fill	1	1	
A5	NH	Boesingheliede, connection roads in junction; 3 embankments	MSW slag fill	1	1	
A16	NB	Moerdijk, reconstruction for HSL; main highway lanes and connection from A16 to A58	MSW slag fill	1	1	
N33	NN	Appingedam; access ramps of bridge across Eemskanaal	MSW slag fill	1	1	
A50	NB	Paalgraven	MSW slag fill	1	1	
A50	NB	Paalgraven; ramps of 3 bridges in main lanes	MSW slag fill	1	1	
A50	NB	Veghel; ramps of 2 bridges in main lanes	MSW slag fill	1	1	
A73	LB	Bridge over duct Roermond	EPS fill	to be determined	1	exact location to be determined
A2	UT	Nieuwegein Spoedpakket F	EPS fill	to be determined	2	
A4	NH	Rijpwetering	EPS fill	to be determined	2	
A5	NH	Westrandweg embankment 1**	MSW slag fill	1	2	
A5	NH	Westrandweg embankment 3**	MSW slag fill	1	2	

Highway	Region	Location	Type	STEP 3: importance *	STEP 4: expected ground-water rise	Remarks
N11	ZH	Buttress W twin bridges	EPS fill	1	2	
N11	ZH	Middle part between twin bridges	EPS fill	1	2	
N11	ZH	Buttress E twin bridges	EPS fill	1	2	
A12	ZH	Nootdorp	EPS fill	to be determined	2	
A12	ZH	Zevenhuizen KW36	EPS fill	to be determined	2	
A12	ZH	Bleiswijk KW41	EPS fill	to be determined	2	
A12	ZH	Zoetermeer-Gouda KW42	EPS fill	to be determined	2	
A15	ZH	Sliedrecht, connection Sliedrecht-West (aka Wijngaarden)	piled embankment	1	2	
A15	ZH	Sliedrecht, connection Sliedrecht-West (aka Wijngaarden)	piled embankment	1	2	
A15	ZH	Sliedrecht, connection Sliedrecht	piled embankment	1	2	
A15	ON	Meteren	EPS fill	1	2	
A16	NB	Moerdijk, reconstruction for HSL; 4 locations in connections	MSW slag fill	1	2	
A27	UT	Stichtse Brug**	MSW slag fill	1	2	
A27	UT	Stichtse Brug**	MSW slag fill	1	2	
A44	ZH	Buttresses for widening tunnel in Menneweg Sassenheim	foamed concrete fill	1	2	
N50	ON	S access ramp of bridge over IJssel Kampen	MSW slag fill	1	2	
A2	NB	Eindhoven fly-over junction De Hogt	EPS fill	1?	3	
A2	NB	Eindhoven ring road KW176 en KW178	EPS fill	1?	3	
A4	ZH	Burgerveen-Leiden Willem vd Madeweg	EPS fill	to be determined	3	
A5	NH	Westrandweg embankment 2**	MSW slag fill	1	3	
A5	NH	Crossing of Hoofdvaart near Hoofddorp	foamed concrete fill	1	3	
A15	ZH	Rozenburg**	MSW slag fill	1	3	

Highway	Region	Location	Type	STEP 3: importance *	STEP 4: expected ground-water rise	Remarks
A4	NB	Halsteren; access ramps of bridge in connection between N259 and A4, entry and exits	MSW slag fill	2	1	
A2	UT	Entry/exit Lage Weide	foamed concrete fill	2	2	
A4	NH	Entry Hoogmade	piled embankment	2	2	
A4	ZH	Burgerveen; 2 filles for entries and exits between A4 and N207	MSW slag fill	2	2	
A9	NH	Heiloo - Alkmaar (Kooimeer); 2 buttresses of bridge in bypass between A9 and roundabout Kooimeer	MSW slag fill	2	2	
A12	UT	Connection A12 - N204 Woerden, exit	piled embankment	2	2	
A12	UT	Connection A12 - N204 Woerden, entry	piled embankment	2	2	
A15	ZH	Connection Sliedrecht-Oost (Zwijnskade)	EPS fill	2	2	
A15	ZH	Entry/exit Papendrecht to N3	EPS fill	2	3	
A13	ZH	Service area Ruijven W side	EPS fill	3	2	
A16	NB	Moerdijk, reconstruction for HSL; filles Hoofdstraat and embankments near railway station	MSW slag fill	4	1	
A12	NH	Duiven; ramps of bridge in junction Noordersingel N810 and A12	MSW slag fill	4	2	
A16	NB	Moerdijk, reconstruction for HSL; W parallel road	MSW slag fill	4	2	
A16	NB	Moerdijk, reconstruction for HSL; W parallel road	MSW slag fill	4	2	
A16	NB	Moerdijk, reconstruction for HSL; buttresses of bridge in crossing N258	MSW slag fill	4	2	
A16	NB	Moerdijk, reconstruction for HSL; buttresses of bridge in crossing Hoge Zeedijk	MSW slag fill	4	2	

Highway	Region	Location	Type	STEP 3: importance *	STEP 4: expected ground-water rise	Remarks
A16	NB	Moerdijk, reconstruction for HSL; near bridge in cross-ing Binnenmoerdijksebaan	MSW slag fill	4	2	
A4	NH	Buttresses in crossing road N446 Hoogmade	EPS fill	4	3	
N11	ZH	Junction N11 - Boskoop-seweg SW quadrant; widening Boskoopseweg N207	EPS fill	4	3	
A2	LB	Echt noise barrier above waterduct	EPS fill	5	1	exact location to be determined
A2	NB	Den Bosch noise barrier	EPS fill	5	2	exact location to be determined

Table E2 Remaining potentially vulnerable EPS and foamed concrete fills, piled embankments and MSW slag fills

F Locations with insufficient storage capacity in the centre verge to store additional rainfall

Highway	Lane	Km from	Km to	Average transverse slope	Wearing course type	Number of trafficked lanes
A4	0HRM	112.2	112.3	-0.28	DAB	1
A4	0HRM	112.9	113.0	-0.20	DAB	1
A4	0HRM	115.9	116.0	-0.44	CEO	1
A4	0HRM	117.1	117.2	-0.16	CEO	1
A4	0HRM	117.2	117.3	-0.26	CEO	1
A4	0HRM	118.4	118.5	-0.36	DAB	1
A9	0HRM	101.3	101.4	-0.08	SMA	1
A15	0HRM	4.5	4.6	-0.12	DAD	1
A15	0HRM	7.8	7.9	-0.14	SMA	1
A15	0HRM	14.2	14.3	-0.24	SMA	1
A15	0HRM	14.3	14.4	-0.14	SMA	1
A15	0HRM	14.9	15.0	-0.40	SMA	1
A15	1HRR	211.3	211.4	-0.30	DAB	1
A15	1HRR	215.1	215.2	-0.28	OAB-R	1
A15	1HRR	216.8	216.9	-0.06	DAB	1
A15	0HRM	226.6	226.7	-0.22	SMA	1
A15	1HRL	227.3	227.4	-0.40	SMA	1
A15	0HRM	234.3	234.4	-0.08	DAB	1
A31	0HRM	36.5	36.6	-0.42	DAB	1
A31	1HRL	40.1	40.2	-0.20	DAB	1
N33	0HRM	7.8	7.9	-0.42	EAB	1
N33	0HRM	11.8	11.9	-0.42	DAB	1
N33	0HRM	15.4	15.5	-0.44	DAB	1
N33	0HRM	23.2	23.3	-0.18	SMA	1
N33	0HRM	28.7	28.8	-0.12	OAB	1
N33	0HRM	58.3	58.4	0.00	SMA	1
N33	0HRM	63.4	63.5	-0.34	DAD	1
N33	0HRM	63.7	63.8	-0.36	DAD	1
N33	0HRM	64.9	65.0	-0.12	DAD	1
N33	0HRM	66.1	66.2	-0.42	DAD	1
A35	0HRM	74.4	74.5	-0.12	SMA	1
N57	1HRL	3.3	3.4	-0.16	DAB	1
N57	1HRL	3.5	3.6	-0.30	DAB	1
N57	0HRM	58.4	58.5	-0.44	DAB	1
N57	0HRM	70.3	70.4	-0.40	DAB	1
N57	0HRM	76.2	76.3	-0.38	DAB	1
N57	0HRM	77.3	77.4	-0.06	DAB	1
N57	0HRM	79.0	79.1	-0.32	DAB	1
A59	0HRM	0.0	0.1	-0.40	DAB	1
A59	0HRM	2.9	3.0	-0.02	SMA	1

Highway	Lane	Km from	Km to	Average transverse slope	Wearing course type	Number of trafficked lanes
A59	0HRM	3.4	3.5	-0.12	SMA	1
A59	0HRM	5.7	5.8	-0.20	DAB	1
A59	0HRM	16.3	16.4	-0.12	DAB	1
A59	0HRM	17.3	17.4	-0.08	DAB	1
A59	0HRM	18.8	18.9	-0.20	DAB	1
A59	0HRM	19.9	20.0	-0.18	DAB	1
A59	0HRM	27.6	27.7	-0.16	DAB	1
A59	0HRM	31.6	31.7	-0.14	DAB	1
A59	0HRM	36.8	36.9	-0.38	DAB	1
N61	0HRM	3.7	3.8	-0.40	DAB	1
N61	1HRL	5.8	5.9	-0.02	DAB	1
N61	0HRM	10.9	11.0	-0.36	SMA	1
N61	0HRM	14.7	14.8	-0.30	DAB	1
N61	1HRR	22.9	23.0	-0.10	SMA	1
N99	0HRM	2.3	2.4	-0.16	EAB	1
N768	0HRM	31.8	31.9	-0.10	DAB	1
N768	0HRM	32.2	32.3	-0.20	DAB	1
N772	0HRM	8.7	8.8	-0.08	DAB	1
N772	1HRR	40.3	40.4	-0.36	DAB	1
N772	0HRM	40.4	40.5	-0.22	EAB	1
N834	0HRM	19.2	19.3	-0.26	DAB	1
N834	0HRM	23.0	23.1	-0.36	SMA	1
N835	0HRM	12.0	12.1	-0.16	SMA	1
N835	0HRM	16.3	16.4	-0.44	DAB	1
N835	0HRM	25.2	25.3	-0.24	DAB	1
N835	0HRM	40.6	40.7	-0.36	DAD	1
N838	0HRM	21.9	22.0	-0.43	SMA	1

Table F1 Locations with insufficient storage capacity in the centre verge to store additional rainfall

G Characteristics for porous asphalt

G.1 Introduction

Literature about hydraulic characteristics of PA (or Dutch ZOAB) is very rare. In the few available reports information about permeability of the porous medium is defined in two different ways consistent with the determination of the characteristic.

In practice in the field the infiltration method by Becker is followed where during a falling head test the time lag for emptying a cylinder is measured. The Becker test gives a drainage capacity in seconds. The Becker test is standardized according to European standard [16].

In laboratory, the permeability in meters per second is measured with standard hydrogeological or geotechnical flow tests in drilled asphalt samples.

The standard permeability test performed in the laboratory is a constant head test according to prEN 12697-19.

In the next paragraph details about the Becker test are specified. Since correlations between both tests are lacking a general hydrogeological formula for infiltration testing was used to convert Becker drainage capacity to permeability. Only few results were found in literature to check the validity of this conversion method.

G.2 Becker infiltration test for permeability of PA (ZOAB)

The description of the test for drainage capacity was taken from the Dutch website of KOAC.

The Becker apparatus is a transparent tube on a metal support plate with a circular opening at the down side, placed on the PA pavement. By increasing the weight the rubber footing closes the pores in the pavement texture. After filling with water the time lag is measured for water level drop between upper and lower marker line on the tube. The larger time lag is measured, the lower permeability of PA is determined. The method is not very accurate due to flow losses around the tube opening and due to high permeability and water volumes needed for filling the pores.

As a result of research in the past temporary guide values for time lags in Becker tests are specified for single layer PA:

- New PA 15 to 25 seconds at maximum.
- Alert to start cleaning clogged PA if time lag is between 50 and 75 seconds.
- After cleaning operation time lag should be below 30 seconds.
- When time lags are larger than 75 to 100 seconds successful cleaning is hardly achievable.

In road maintenance contracts for double layered PA with a 4/8 aggregate top layer, demands are formulated to guarantee a Becker time lag of average 17 seconds and 20 seconds at maximum for each control measurement.

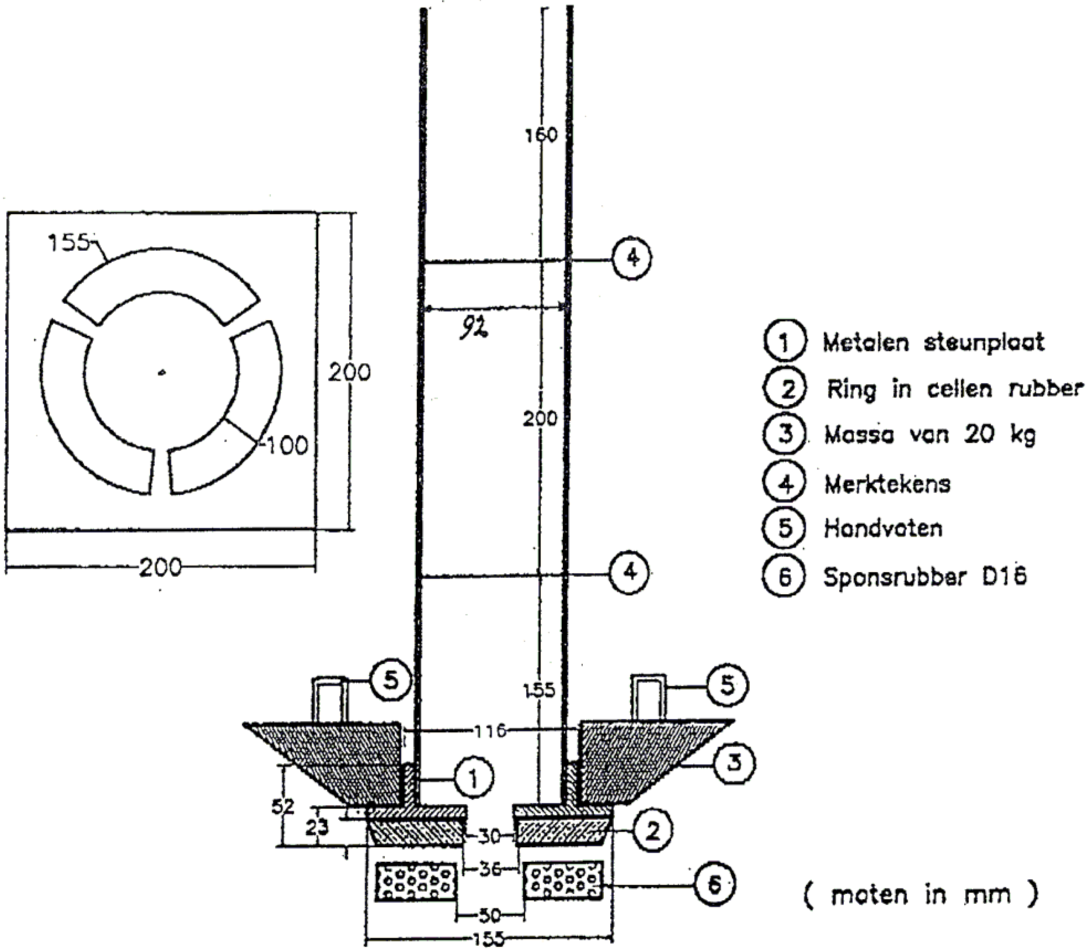


Figure G.1 Dimensions of the Becker apparatus for measuring drainage capacity of porous asphalt (http://www.koac-npc.com)

G.3 Relations between results from Becker tests and permeability testing

G.3.1 Results from Becker tests and laboratory tests of PA permeability

According to reported lab tests [8, 24] porosity of PA varies from 12 to 25 % but in average is 22%. Mentioned literature also reports values of (vertical) permeability of samples. Vertical permeability of samples prepared in the lab varies from 1.5 $\times 10^{-3}$ m/s to 10 $\times 10^{-3}$ m/s. Vertical permeability of samples drilled from roads varies from 1 $\times 10^{-3}$ m/s to 3 $\times 10^{-3}$ m/s. Horizontal permeability tends to be 1/3 of vertical permeability. The granular distribution of the top layer determines permeability the most.

In the document [8] lab tests on drilled samples of double layered PA are compared to Becker tests in situ. The results are gathered in the next table.

Granular distribution top layer	Permeability [m/s]		Porosity [%]		Becker test results [sec]
	horizontal	vertical	Top layer	Bottom layer	
4/8	1.0×10^{-3}	2.8×10^{-3}	24	19	18
2/6	0.6×10^{-3}	1.6×10^{-3}	20	19	12
2/8	0.7×10^{-3}	2.2×10^{-3}	20	20	20

Table G.1 Permeability and porosity of PA from lab and field tests

The lab results show that horizontal permeability is 1/3 of vertical permeability. This outcome conflicts with common sense of proportion for rolled asphalt pavement.

G.3.2 Theoretical evaluation of permeability from Becker infiltration tests

A Becker test is an infiltration test. For want of a better evaluation method, we propose to evaluate Becker infiltration tests according to the methods by Hvorslev for well tests. However, these methods describe flow of water from a groundwater monitoring well in a water bearing layer or aquifer. As simplification the PA porous layer is taken as aquifer. The horizontal permeability is calculated with:

$$k_h = \frac{d^2}{8L\left(t_2 - t_1\right)} \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right] \cdot \ln\frac{H_1}{H_2}$$

$$m = \sqrt{\frac{k_h}{k_v}}$$

With

- d = diameter of opening [m].
- L = layer thickness [m].
- D = diameter of tube [m].
- t_1 , t_2 = time steps in measurements (seconds).
- H_1 , H_2 = water level in tube at time steps [m +REF].
- m = factor of anisotropy.

If the Becker time lag $\Delta t = t_2 - t_1$, than H1 and H2 are 180 and 380 mm respectively, d = 30 mm and D = 92 mm according to [16], with $k_h/k_v = 1/3$ one can calculate from a Becker time lag Δt = 17 seconds for new PA witk a layer thickness of 50 mm that $k_h = 1.3 \times 10^{-3}$ m/s. This fits reasonably with lab tests.

H Calculation method for stormwater runoff from porous asphalt

H.1 Design calculations for flow inside porous asphalt pavements

Porous asphalt admits infiltration of rain water into the pavement.
The flow in the open graded asphalt layer can be described with formulas for groundwater flow according to Darcy's law for laminar groundwater flow:

$v = k \cdot h \cdot \frac{dh}{dx}$ gives the head in a non-sloping layer with permeability k = 5 x10^{-3} m/d.

It is shown that even with small rain intensity the layer will get fully saturated.

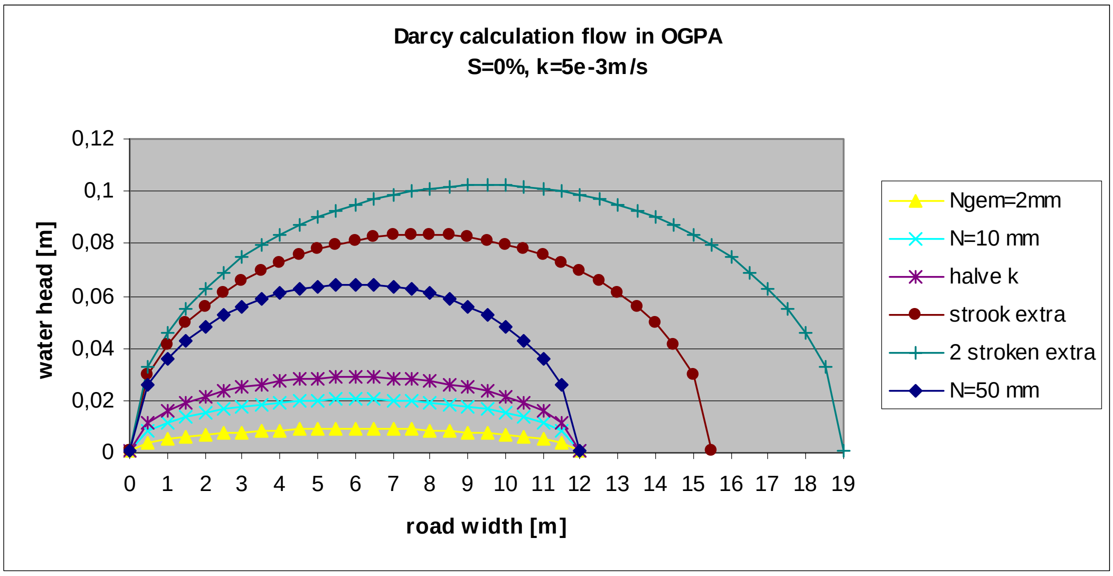


Figure H.1 "Groundwater head" in porous asphalt (non sloping) with varying road width and varying rainfall intensity (2, 10, 50 mm / day)

The storage in PA will delay the runoff form pavements.
The question is still what the maximum flow in PA would be in a sloping pavement. The gradient dh/dx is at least equal to the cross slope of the road and the head h is maximally equal to the layer thickness of the porous asphalt. As the gravel aggregate in the porous asphalt is very coarse, it is necessary to check what flow conditions are present (laminar or turbulent flow).

Considering normal slope of 2.5% and a Darcy permeability or conductivity of 10*10^{-3} m/s the Darcy flow velocity is 0.24*10^{-3} m/s. With a pore volume of 22% the real flow velocity becomes 1.2*10^{-3} m/s. The Reynolds number is calculated using a pore diameter of about 2 to 4 mm and a normal value for dynamic viscosity of 10^{-6} m^2/s. Thus, the Reynolds number is about 100. With a Reynolds number smaller than 2300 it is allowed to consider laminar flow in the open graded pavement and Darcy's law is valid.

If we consider an open graded porous asphalt layer with a permeability of $10*10^{-3}$ m/s or about 100 m/d, the maximum Darcy flow in a porous layer with 5 cm thickness and a cross slope of 2.5% is 0.125 $m^3/d/m^1$.

Note that this flow is very small compared to the amount of rain falling on a two-lane motorway of at least 12 m wide. An excessive rainfall event of 1 mm/min on such a road will give a flow rate of 17.28 $m^3/d/m^1$.

H.2 Calculations of combined flow in porous asphalt and surface runoff

Modeling experiments were performed to try out if general hydraulic models could help to estimate the flow and water depth due to rainfall on PA pavements.

Two types of models available at Deltares were tested:

1. Hydrogeological finite element model Plaxflow.
2. Hydraulic flow model Wanda.

H.2.1 Plaxflow

Plaxflow is a finite element model suited to calculate transient groundwater flow in saturated and unsaturated conditions. To use this model for flow in and over PA we had to make a scheme of a road section on top of the porous asphalt a fictitious layer to model the surface water runoff. However, this way of modeling only was applicable for small rainfall events. For heavy rain storms the modeling was hampered by numerical instability. It was not possible to solve this problem within this research project.

H.2.2 Wanda

The numerical Wanda model divides the road into two parts; the film on surface and the asphalt. For the open asphalt, Darcy's formula is used to calculate the resistance of the asphalt (which is considered as a porous media). The runoff part was modelled using the open free surface conduit element in Wanda which takes the film as free surface open channel where momentum and continuity equation are been applied (The Saint Venant Equations). The Wanda model solves these equations numerically.

The outcome showed that more than 80% of the flow moved as surface runoff. The calculated water depth was in the same magnitude as the surface roughness. The equations in Wanda are not suited to handle this situation. Therefore, the result is not very reliable and the Wanda model, designed for industrial pipe flows, may not be applicable for this kind of problems.

H.3 Empirical design rules for pavement runoff

A possibility to model surface runoff from pavements is to use the well known Manning formula [9]:

$$Q = k_M \cdot A \cdot R_h^{2/3} \cdot S^{1/2}$$

With Q as flow in m^3/s. Flow per unit of road length can be set equal to rainfall intensity times road width L. The hydraulic radius is in this case equal to the waterdepth on the pavement. After some mathematical elaboration the water depth can be calculated from

$$WD = \left(\frac{I \cdot L}{k_M \cdot \sqrt{S}} \right)^{3/5}$$

The parameters are:

- WD = Water Depth.
- S = Pavement cross slope and grade.
  The cross slope allows water to run down the pavement. Grade is the steepness of
  the road. The resultant of cross slope and grade is called drainage gradient or "resulting grade".
  $S = \sqrt{i_L^2 + i_c^2}$
- L = Drainage path length or Width of pavement
  Wider roads require a higher cross slope to achieve the same degree of drainage.
- I = Rainfall intensity
- k_M = Manning's coefficient for surface type, in [9] notated as 1/n or 1.486/n but depending on the unit system.

The Manning coefficient can be found by comparing with tabulated values or estimated in field tests. From tables in hydraulic literature can be deduced that for flow over an asphalt layer the factor n is approximately 0.015. Thus k_M has a value of around 67

Results of calculations when using Manning's equation for surface runoff will not be very accurate because the coefficient for PA is not known from practical tests. We have concluded that the water depth of the runoff is of the same order of magnitude as the roughness of the aggregate in the open asphalt. The calculations only give approximate results as Manning's formula is empirically derived.

Another option is to use experiences on this matter from foreign road design institutes. We found information from the US National Transportation Safety Board NTSB. According to the document TE-IA-85-1 about Highway Lorry Tyre Loading Investigative Alert a formula for calculating water depth by rainfall runoff is available:

$$WD = 0.00338 \cdot \left[\frac{TXD^{0.11} \cdot L^{0.43} \cdot I^{0.59}}{S^{0.42}} \right] - TXD$$

The formula is suggested by Gallaway et al [11] for the Texas Highway Department and the US Federal Highway Administration.
Most parameters are the same as in Manning's equation but here the roughness of the surface is expressed explicitly instead of Manning's coefficient with:

- WD = Water Depth (inch above aggregate surface).
- S = Pavement cross slope and grade.
  The cross slope allows water to run down the pavement. Grade is the steepness of the road. The resultant of cross slope and grade is called drainage gradient or "resulting grade".
  $S = \sqrt{i_L^2 + i_c^2}$
- L = Drainage path length or Width of pavement (feet)
  Wider roads require a higher cross slope to achieve the same degree of drainage.
- I = Rainfall intensity (inch/hour).

- TXD = Average Texture Depth (inch)
  The texture depth determines the roughness of the pavement surface.

This formula is mentioned in several international literature [6,3] but also in Dutch publications [21,24]. The Gallaway formula is empirically derived and stems from evaluation of large scale experiments.

Transformed to consistent metric units (meters, hours) the value of the coefficient changes and the formula reeds

$$WD = 0.00187 \cdot \left[\frac{TXD^{0.11} \cdot L^{0.43} \cdot I^{0.59}}{S^{0.42}} \right] - TXD$$

In literature [24] the coefficient is set to 0.01485 but then all dimensions are set in [mm] except for rain intensity in [mm/hour].

The Gallaway design rule is stated for normally paved asphalt roads but the source is not revealed. Which specific hydraulic conditions determine the validity of the formula is not clear. Most likely from a hydraulic point of view the formula is applicable for a limited range of rainfall intensities. In the Gallaway document it says that the formula is based on the following experiment parameters:

a) Drainage length up to 14.6 m.
b) Rainfall intensities up to 50.8 mm/h.
c) Slopes up to 8 %.

However the pavement or aggregate roughness is not mentioned.

The Gallaway and Manning formula differ only slightly. The road width was varied upto 25 m. As rainfall events the 10 years shower with an intensity of 11 mm in 5 minutes was introduced. At small widths the Gallaway formula gives erroneous results and certainly will not be valid if the width is less than 10 m. As Manning coefficient a value was chosen for rough concrete of 67. In the Gallaway formula a texture depth TXD was chosen of 2 mm.

The formulae give a stationary situation of water depth. In the calculations of the waterdepth and flow the delay due to the length of the flow path in the runoff was neglected.

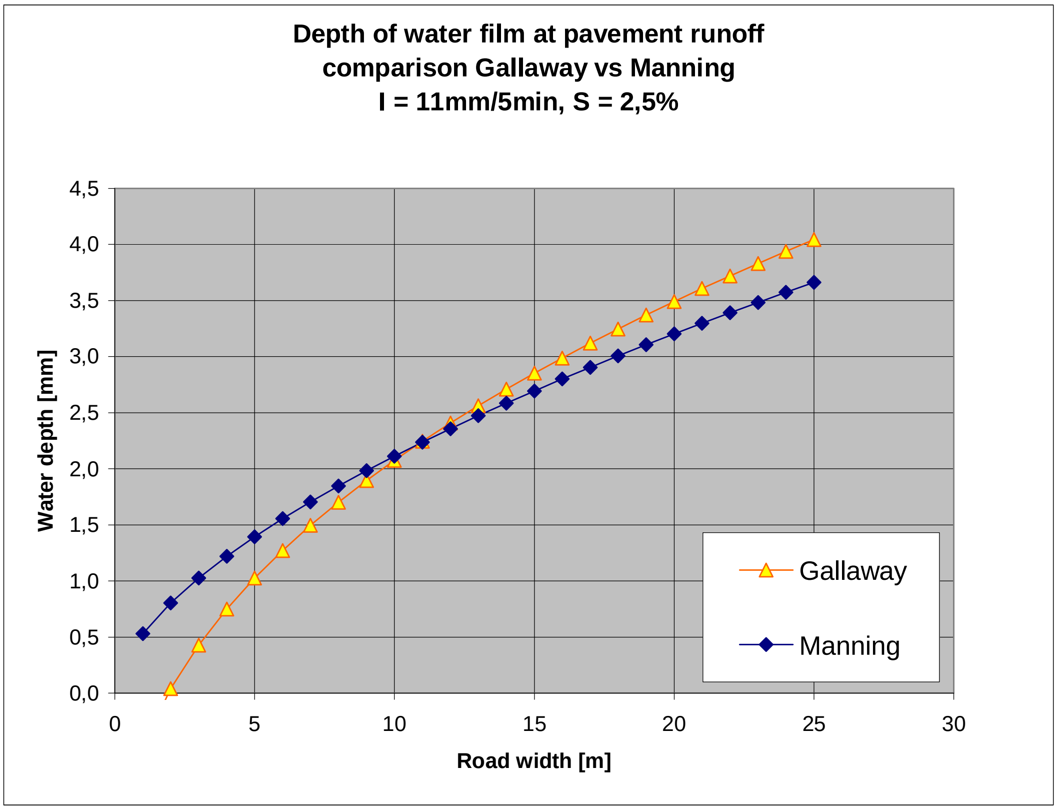


Figure H.2 Comparison of Gallaway and Manning formula for stormwater runoff

Varying of coefficients gave us the impression that the Gallaway formula is more sensitive to uncertainty in the texture or roughness parameter. Gallaway suggests to measure the roughness. This is possible with usual laser techniques for texture measurements. However, as far as we know the local texture depth of PA on Dutch Highways is not registered.

H.4 Validity of formulae and conformity to NOA requirements

Both Gallaway formula and Manning formula have coefficients (the Manning coefficient and the Average Texture Depth used by Gallaway) that might vary due to local circumstances related to type of pavement or other conditions. To determine whether to use either the Manning or the Gallaway equation we sought a possibility to calibrate the formulae with measurements from practical tests.

In 1999 a precipitation test was performed as described in [27] for an airport runway at Schiphol Amsterdam. The precipitation was set up in a ful scale lab test (22 m*0.5 m) with artificial rainfall intensities of 1 to 6 mm per 5 minutes. The asphalt layer was constructed according to work method in reality on board layers with a 4.5 mm rough aggregate (not OGPA but ordinary asphalt layer). The equivalent texture depth was measured to be 2.1 mm. The development of the water film depth during the test was monitored with high accuracy at two sections 5.5 and 17.5 m distance from upper end of the 1.5% sloping surface.

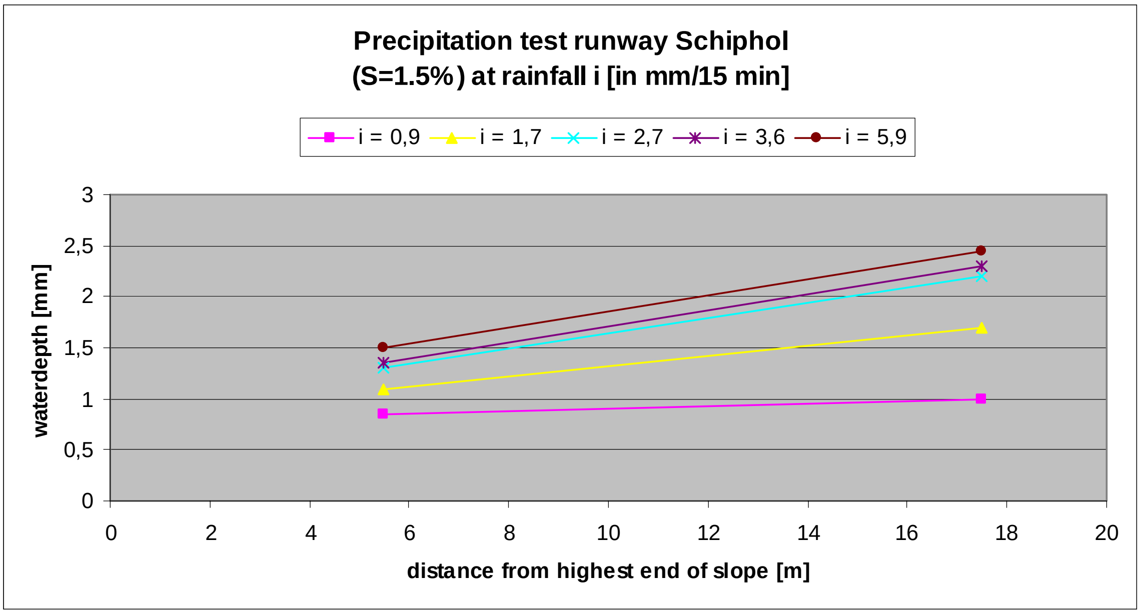


Figure H.3 Results of laboratory test for stormwater runoff of runway at Schiphol Airport

With these measurements of water film depth, it was possible to calibrate the empirical formulae of Gallaway and Manning.

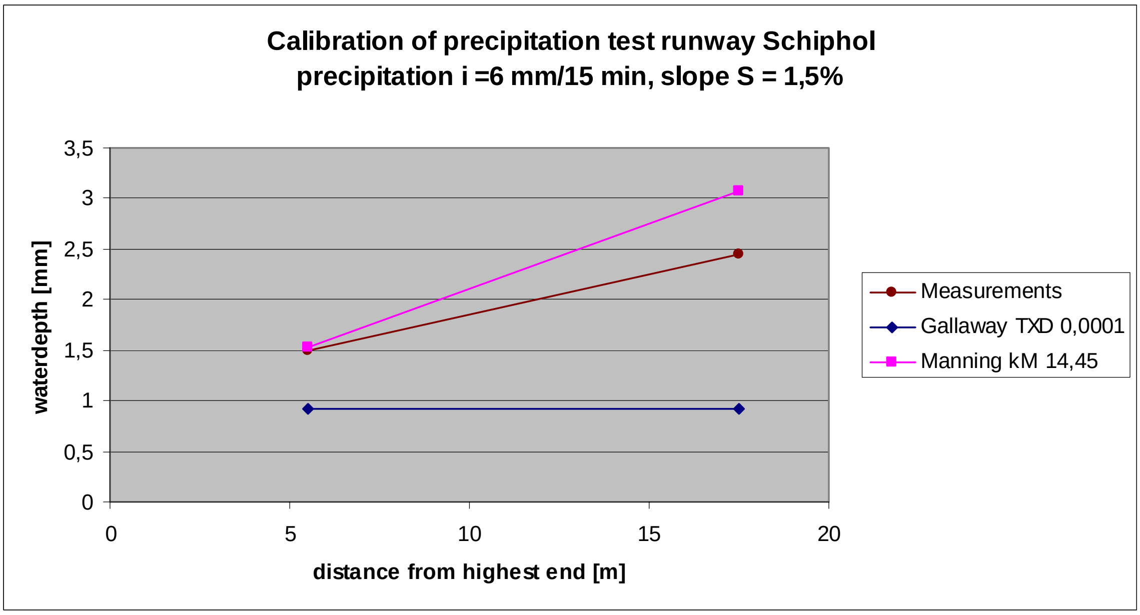


Figure H.4 Calibration of Gallaway and Manning Formula to laboratory test for stormwater runoff of runway at Schiphol Airport

It was impossible to find a proper fit with the Gallaway formula (high inaccuracy for expected texture depth of 0.002 m, smaller inaccuracy for unrealistic texture values). Probably the formula is not suitable for small slopes and small rainfall intensities. We did succeed in retrieving a very accurate solution with the Manning formula, giving a value for the Manning coefficient of kM=14.45. Nevertheless, this factor is relatively small, indicating a very rough surface with a large effect on the flow at small water depths (large turbulence).

Comparing the Gallaway and Manning formula on situations with slopes of 2.5% and higher rainfall intensities we found that a good comparison resulted for a Manning coefficient kM=67.

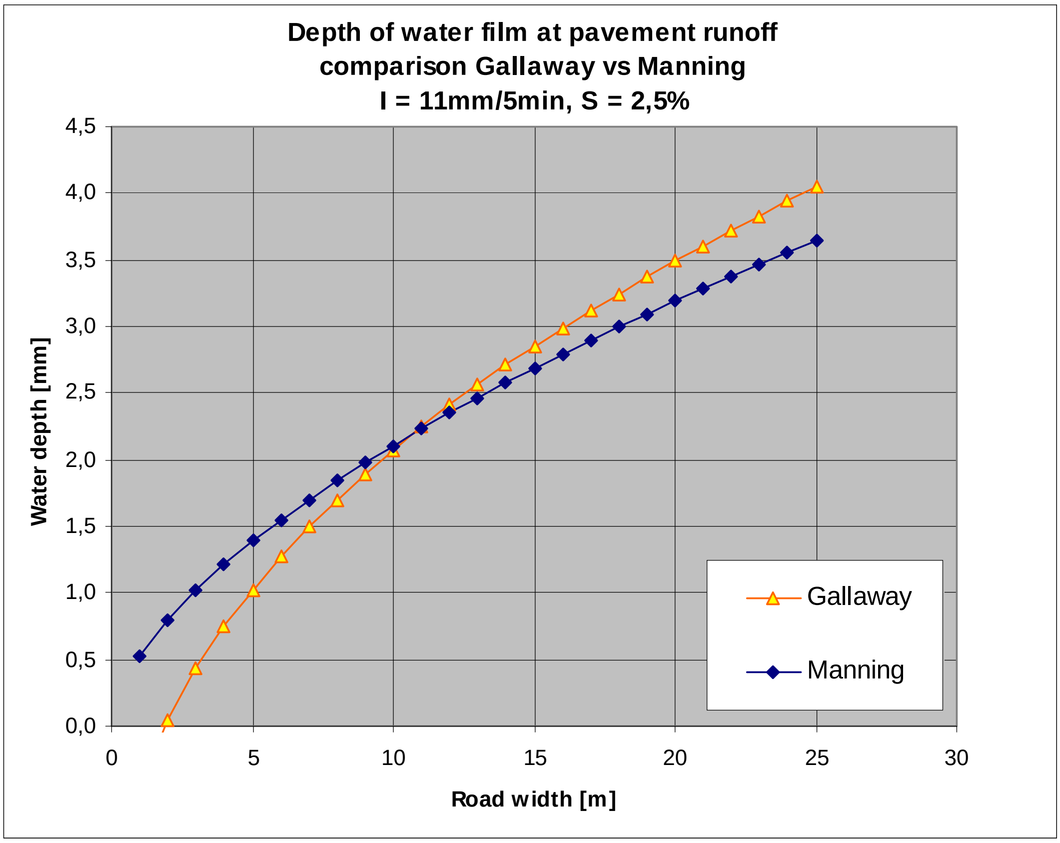


Figure H.5 Comparison of Gallaway and Manning Formula for stormwater runoff of highways

It is not satisfactory that we do not have laboratory test results to check how the formulae fit for rainfall events with a small frequency.
However, we concluded that the Manning formula is a proper method to study the development of water depth during rainfall for the Dutch climatic situation.

According to DVS, the current situation on the Dutch Highways is evaluated as an acceptable risk. We identified for varying slopes and road widths what the water depth is under climatic conditions as in NOA. Based on the Manning formula and Manning coefficient kM=67 a calculation was made for the geometrical and hydrological assumptions in the NOA document for road design.

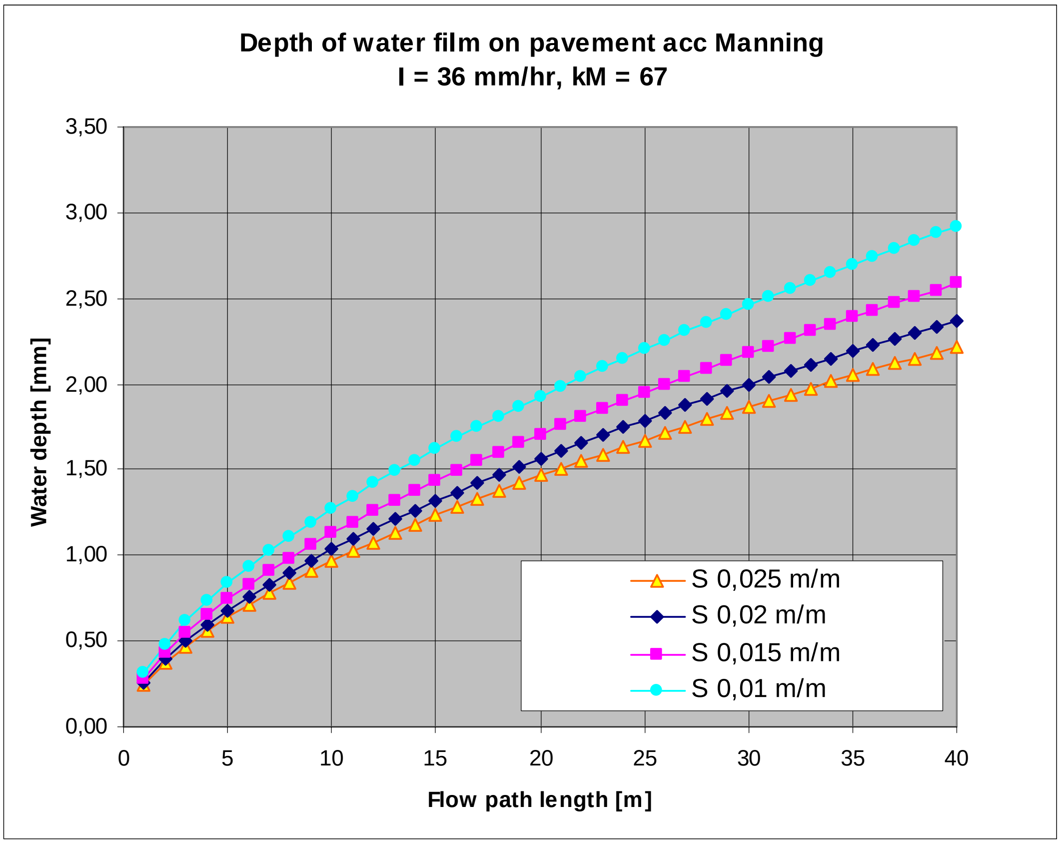
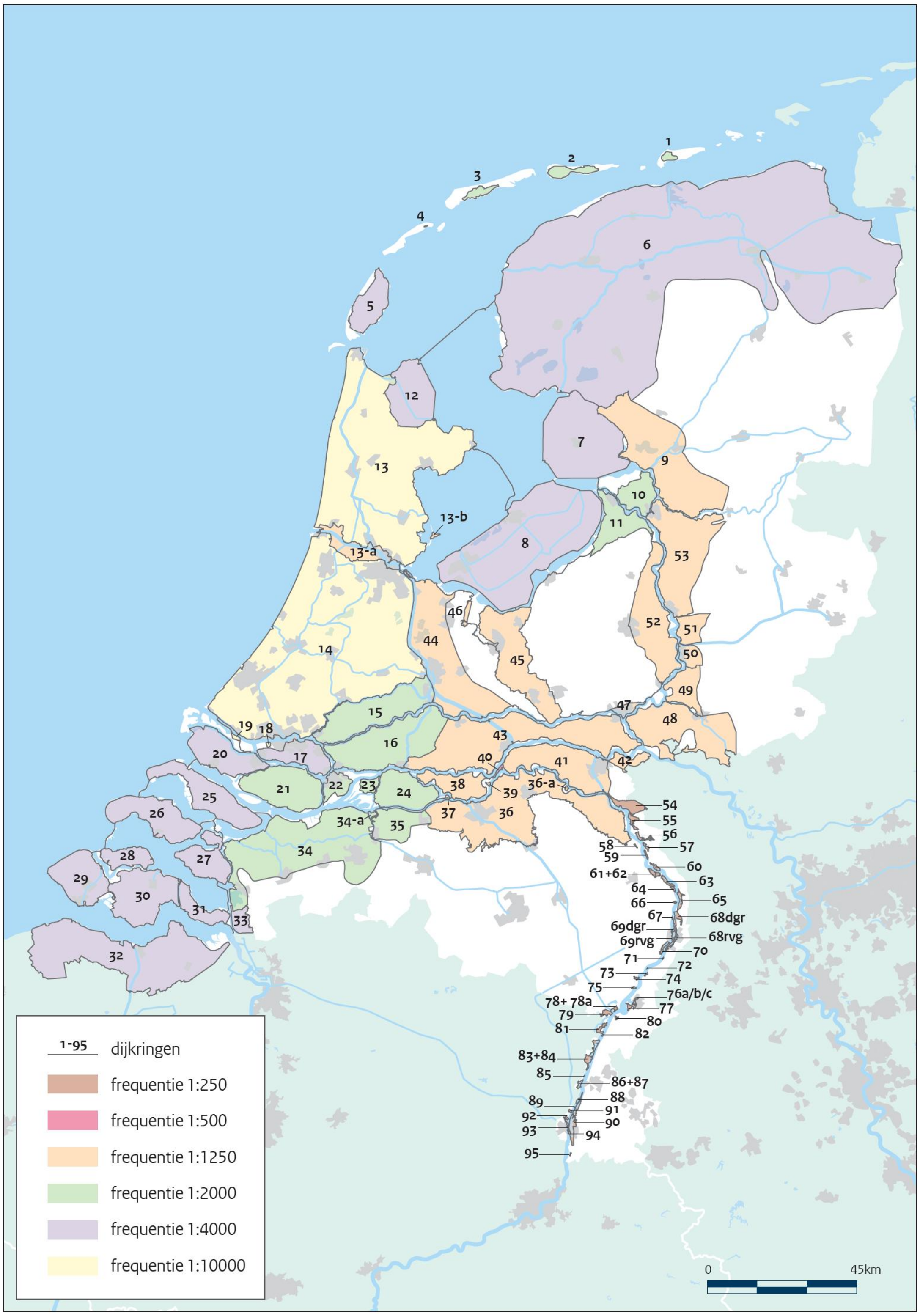


Figure H.6 Calculation with Manning Formula of stormwater runoff from highways for several cross slopes

The result for the rainfall intensity of 36 mm/hr as indicated in NOA shows that the water depth at several road widths and slopes stays within limit of 2 to 3 mm according to NOA.

I Dike ring areas



(source: www.helpdeskwater.nl)