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RADA WATER SUPPLY AND SANITATION PROJECT

RWSSP

ABOVE GROUND STORMWATER DRAINAGE IN AN URBAN AREA

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PREFACE.

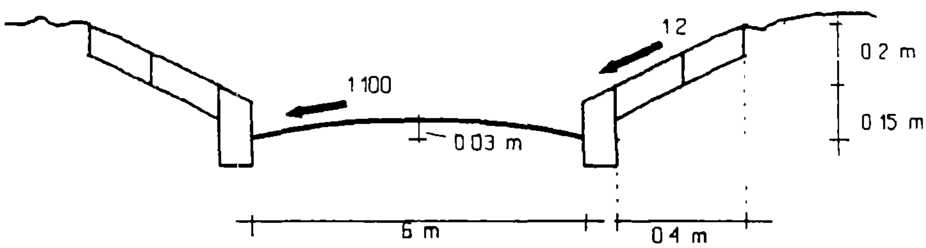
This report has been written for the Rada Water Supply and Sanitation Project (RWSSP); I assume that the reader is familiar with the situation, the objectives and works of the project. Consequently, in the main report only a short explanation of the situation is given, as an introduction to the study.
To complete the report, in the annexes, general background information (a description of Yemen, Rada, the project, etc.) and technical background information is given, as well as a more elaborate description of the study.

From October 1990 till January 1991 I was in Yemen for this project. Unfortunately I spent only 3 months (of the 6 months planned) in Yemen because of the Gulf-crisis. Because of this I missed the rainy season and due to this I had to change the subject of my thesis a bit. It has become more theoretical.
I would like to thank the team members and DHV, Consultants b.v. for helping me and making it possible to work at the project and finally Louise McIllwaine for helping me with the English.

SUMMARY.

Although the average rainfall in Rada is only 200 mm/year, rain intensities can be very heavy. There is no satisfactory drainage system. After heavy rainfall parts of the town are flooded for several days. The rainwater mixes with the solid waste and waste water on the streets. This mixture poses a danger to public health.
As a part of the RWSSP (Rada Water Supply and Sanitation Project) a stormwater drainage system will be designed and constructed. Other parts of this project are the construction of a drinking water system, a sewerage system, a solid waste collecting system and environmental health education.

For reasons of cost and maintenance the rain water will be discharged over the streets, which will be asphalted and shaped in a manner suitable for conveying stormwater. A typical cross section of a road is presented below:



The town has been divided into four parts, each part dividing the water to another spillway. The water is spilled either into the wadi (river bed) or onto the agricultural land east of the town.
The water levels on the streets have been determined with a computer model. The kerbs have such a height that all the water of a storm, which is equalled or exceeded once in 2 years (averagely) can be discharged. A design storm occurring with a return period of 2 years had been determined before, but seemed to be too small. Therefore a new design storm has been determined. Also a design of the drainage system had already been made, but the new design storm resulted in too high water levels in this present design of the drainage system.
The design of the drainage system has been changed where necessary. Only minor changes were needed to be able to discharge the new design storm.

With the new drainage system the water will be out of town within a few hours after the storm. Heavier storms than the design storm will still cause floodings, but the streets will be dry sooner than nowadays. The floodings will be much less frequent and the duration shorter.

TABLE OF CONTENTS.

MAIN REPORT.

ANNEXES.

LIST OF ABBREVIATIONS.

DGIS	Directorate General for International Co-operation, Dutch Ministry for Development Co-operation.
EHE	Environmental Health Education.
l/c/d	litres per capita per day.
MMH	Ministry of Municipalities and Housing.
MHUP	Ministry of Housing and Urban Planning.
NWSA	National Water and Sewerage Authority.
RIRDP	Rada Integrated Rural Development Project.
RSD	Road Surface Drainage.
RUA	Rada Urban Area.
RWSSP	Rada Water Supply and Sanitation Project.
WMO	World Meteorological Organisation.

1 OBJECTIVE.

The objective of this study is:

to check the present design of the drainage system for the Rada Urban Area and to give recommendations (to obtain an addequate design) if necessary.

Heavy storms can flood parts of Rada, after which depressions and some streets remain flooded for several days. Also solid waste and waste water end up on the streets. The mixture of solid waste, waste water and stormwater is a danger to public health. As a part of the Rada Water Supply and Sanitation Project (RWSSP) a stormwater drainage system will be constructed. The objective of the RWSSP is to improve the public health situation in the Rada Urban Area (RUA).

A design of the drainage system has already been made and is presented in the Final Design Report of 1989[1]. This is the present design of the drainage system and is based on 11 years of daily rainfall data only. However, at the end of 1990, more rainfall data were available. It appeared that the intensity and volume of the design storm used were too small.

A new design storm will have a higher intensity and a larger volume. It is the aim of this study to check whether the water levels in the present design of the drainage system, caused by this new design storm, are still acceptable; i.e to check whether the present design of the drainage system, is appropriate or not. If not, proper adjustments in the design have to be recommended.

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[1] Final Design Report RWSSP Volume 1, Main report, December 1989, [lit. 1]

2 INTRODUCTION.

2.1 Project area.

Rada is a town in the Al-Bayda province of Yemen (see figure 1, the map of what used to be North-Yemen). Yemen is located at the south-western point of the Arabic peninsula. The coast-line has a tropical climate. Further inland there are mountains up to 3760 m high. On the fertile terraces on the slopes of the mountains agriculture is possible. Behind the mountains lies a drier (but also fertile) plateau and next comes the desert (see figure 2).

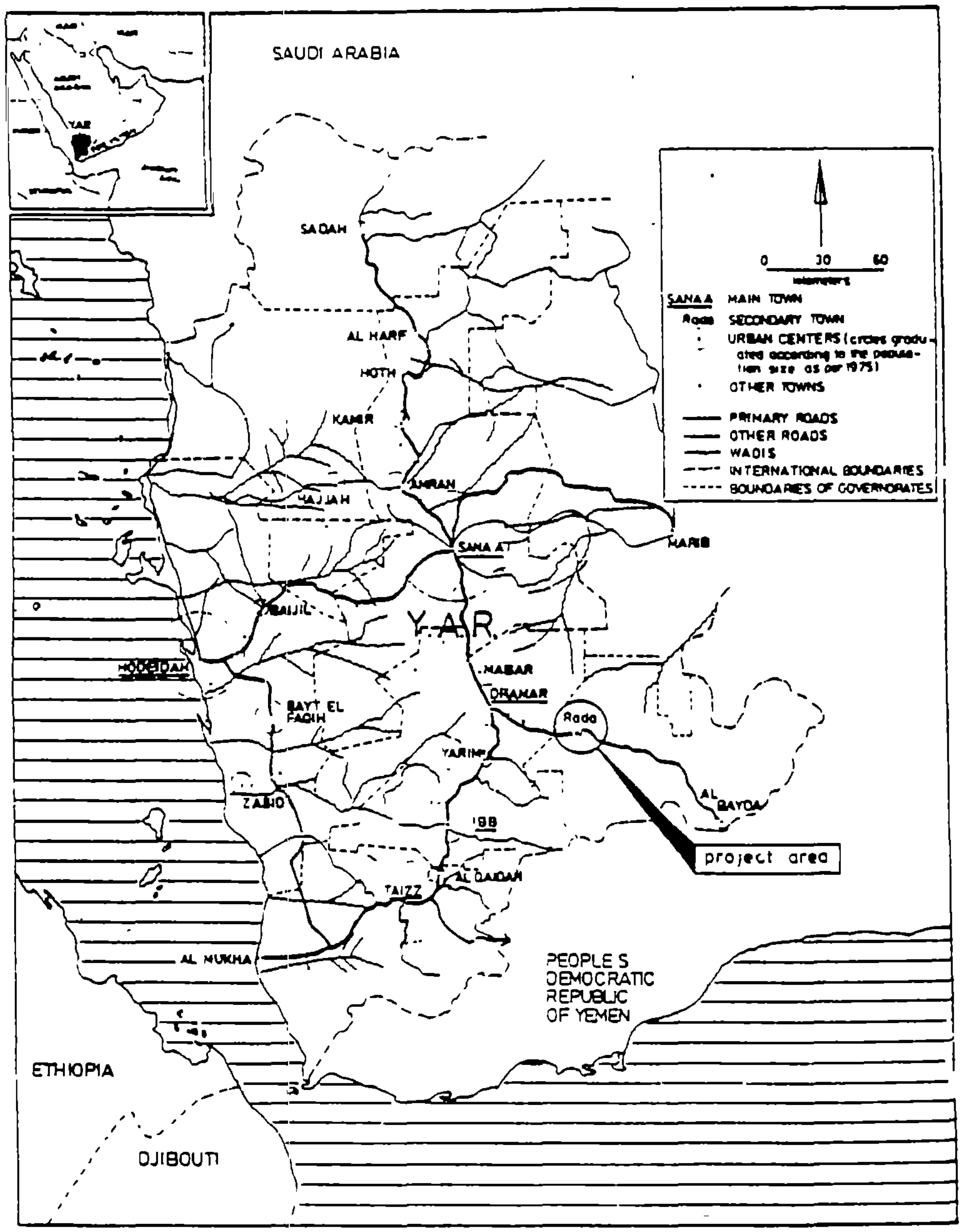

Figure 1: (former) North-Yemen.

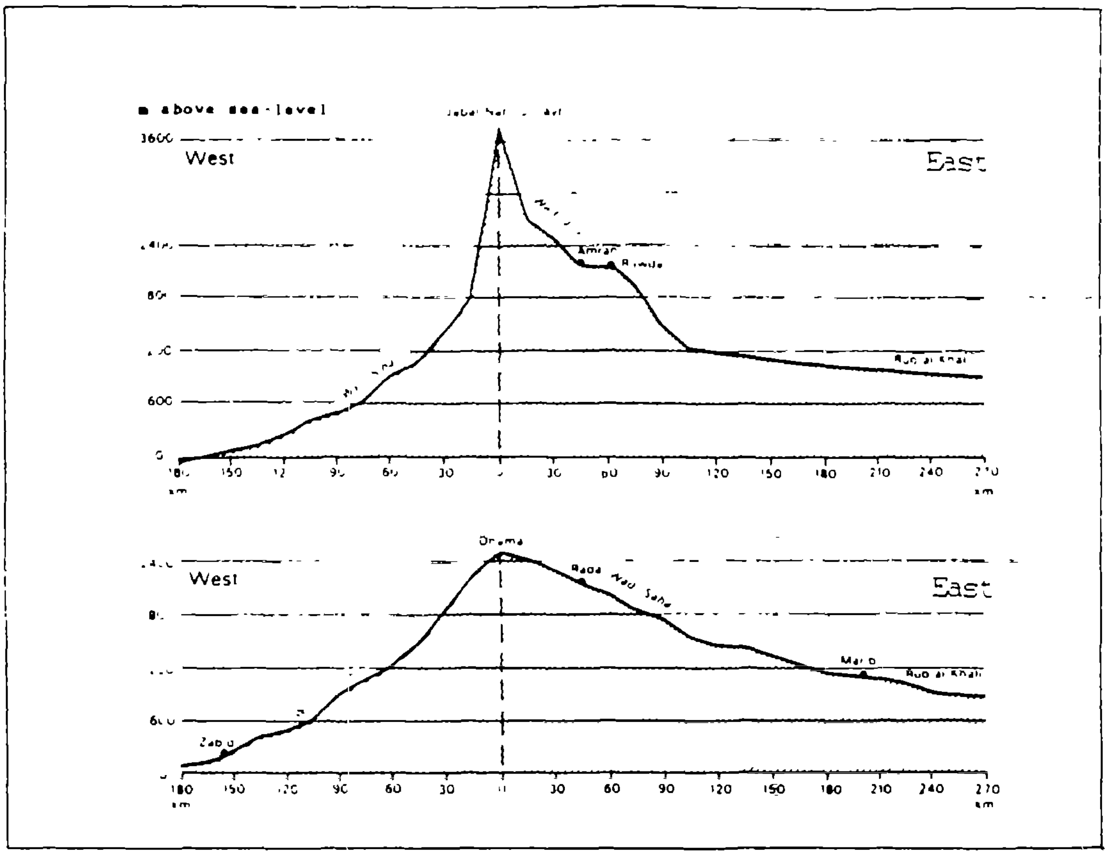

Figure 2: Cross-section Yemen.

Rada is located on the plateau at 2100 m above sea level, 155 km from the capital Sana'a. The Rada Urban Area (RUA) consists of two old settlements at the north of the (asphalt) road from Dhamar to Al-Bayda (see figure 3). In former days these old settlements (Rada and Musalla) were 1 km apart, but have now grown together.

In Rada a beautiful old fort is built on the rocks. The soil around the rocks is loess-like and the soil in the surroundings of Rada consists of volcanic outcrops. At the north of the town is the non-perenial riverbed, Wadi Al-Arsh.

The climate is semi desert, with temperatures ranging between $15^{\circ}C$ - $30^{\circ}C$ in the hottest months and $2^{\circ}C$ - 22° in the coldests. During the day, when the sun has warmed the surface, dust devils are formed by the difference in air temperature. These cause a lot of dust in the air.

In 1989 the population was estimated to be 35,000. For the two time goals of the project 1995 and 2010 the population is expected to be 50,000 and 75,000 inhabitants respectively.

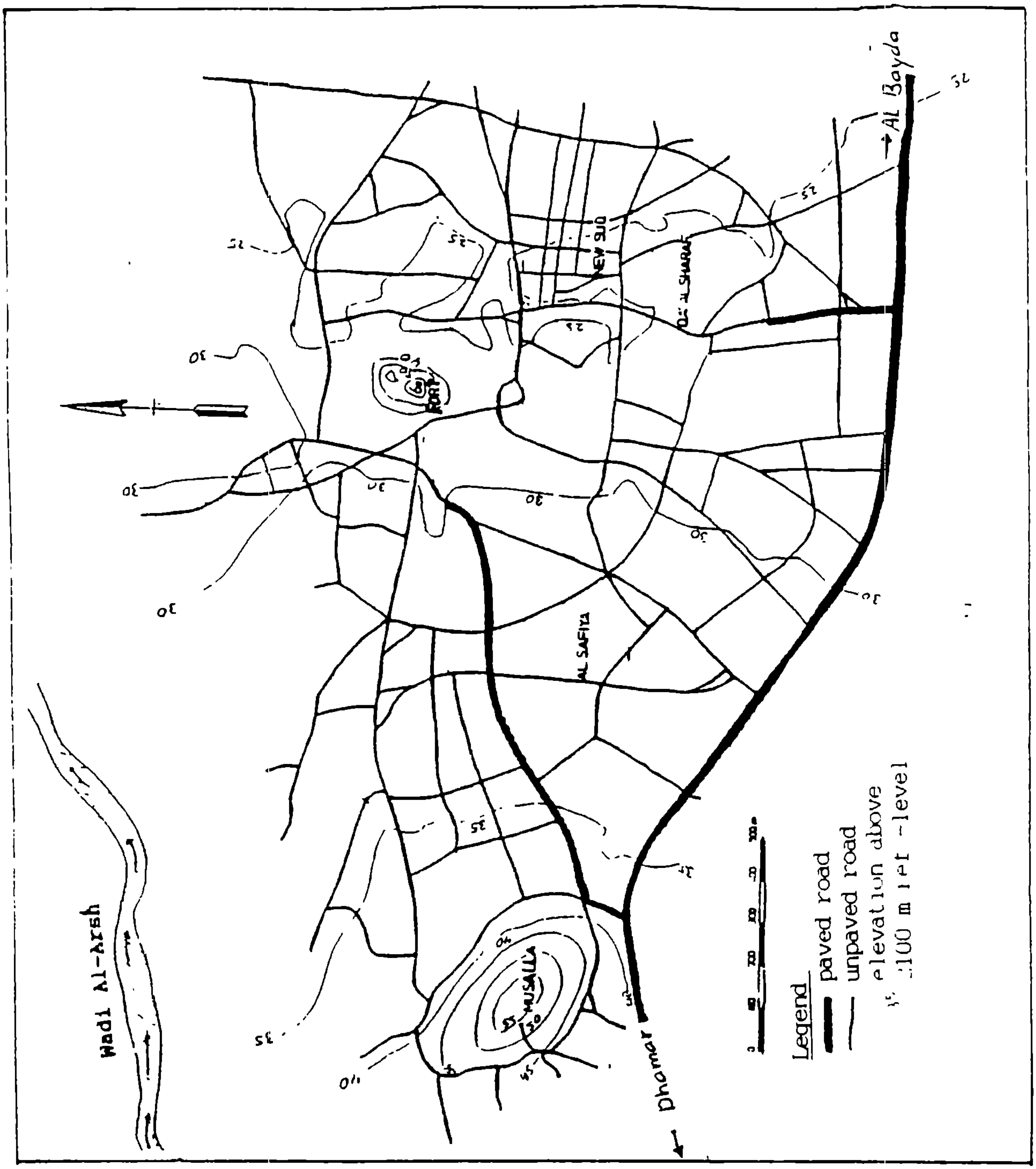

Figure 3: Rada Urban Area.

The public health in Rada is endangered because of the following problems:

The drinking water is unsafe.
The waste water ends in the streets.
The solid waste ends up in the streets.
Heavy rainfall causes floodings.

2.2 The RWSSP.

The Rada Water Supply and Sanitation Project (RWSSP) has got the objective to improve the public health situation through the implementation of a drinking water network, a sewerage system, a solid waste disposal system and a drainage system in the RUA, together with an Environmental Health Education (EHE) and an institutional strengthening programme.
The project is carried out by a joint venture of Euroconsult, DHV Consultants b.v. and Agrovision and the Yemenite counterparts, consisting of members of the Ministry of Municipalities and Housing (MMH) and the National Water and Sewerage Authority (NWSA). The project is partly financed by the Dutch Ministry for Development Cooperation, DGIS; Directorate General for International Cooperation.
In this report the drainage situation and solutions for the problems with the drainage of stormwater are discussed. In annex A the other subjects of the project are presented more elaborately.

3 STORMWATER DRAINAGE.

3.1 Problem survey.

Traditionally houses were built at the higher places or at locations with steep slopes, so the occasional heavy yearly rainstorms did not cause many problems. Rainwater was discharged over the streets out of town or to depressions, where it infiltrated and evaporated. Also some channels to discharge the water existed.

Nowadays the combination of alleys, streets and depressions is still the only means for the drainage of rainwater. This old drainage system is not functioning well any more. Due to urban growth, the less favourable places are being built-up too. This means a diminishing of the depression storage and the blocking of the previous courses for conveying the stormwater. The old drainage pattern is disturbed and after heavy rainfall parts of the town are flooded for at least a week. The solid waste on the streets contributes to this problem, by clogging the old drainage system.

Figure 4: Flooded streets in Rada.

Some traditional houses in Rada have got a foundation of loam. Long contact with water weakens these foundations. Recently some houses collapsed because of this.

In Rada almost all roads are unpaved[2]. These roads are nowadays more affected by the rainfall than before. The motorized traffic has increased. Because the drainage of the roads is poor, water remains in holes in the roads and makes the top soil softer. These holes are deepened by the heavy traffic, resulting in even larger holes with a depth of over half a metre.

The rainwater in the streets and depressions mixes with the garbage and waste water on the streets, causing dangerous situations for public health. It is in the scope of the project to reduce the health risks. In the future a sewer will be constructed to transport the waste water to a treatment plant out of town. At the moment the solid waste is already being collected and dumped at a dump site, but still a lot of garbage ends up in the streets.

Hence, for reasons of public health, protection of buildings, road maintenance and comfort, a proper stormwater drainage system will be needed.

Note: In the following, the terms 'present design report' and 'present design of the drainage system' are used, referring to the situation at the end of 1990; as presented in the Main Report and annexes of the Final Design Report, december 1989. It is emphasized that these are still only designs and at the moment no drainage system has been constructed yet.

3.2 Separate versus combined drainage system.

The stormwater can be transported either combined with the waste water, through a combined system, or separate from the waste water, through a separate system. With the latter system, no rainwater should enter the waste water system. In this case a seperate system is chosen, because a combined system has the following disadvantages:

A combined sewer system would require large dimensions of the pipes. On top of the pipes a cover is needed to protect them from being damaged. This would mean deep excavations. Because of the rocks in the ground, the excavation costs would be high.

The ratio between waste water and storm water is 1:100 to 1:300. So, in the long dry periods only little water flows through the large pipes of the combined sewer network, with small velocities and consequently deposition of solids, thus requiring extensive maintenance.

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[2] In figure 2 the asphalted roads are shown (as far as I can remember). Except for the road from Dhamar to Al-Bayda, the asphalted roads are in poor condition.

Finally, for a combined system, the waste water treatment plant would have to be designed for the maximum dry weather (waste water) flow plus part of the storm water flow. If not, for each heavy storm the plant would be overloaded, resulting in polluted untreated water to be discharged. The hydraulic capacity of a treatment plant designed for the storm water flow would have to be three times bigger than in case of a separate system, to be able to deal with the stormwater flow. Still, for a storm with higher intensities than the design storm, untreated (but diluted) water will have to be discharged.

3.3 Lay-out present design of drainage system.

There is a general slope of 1:150 to 1:100 from west to east over the town. The water can be discharged without pumping, if some low areas in town are filled up. The town has been divided into four separate sections: North-East, North-West, Middle and South, each discharging the water to another spillway. See figure 5 for a lay-out of the present design of the primary drainage system. North of Rada exists a non-perennial riverbed (Wadi Al-Arch). Water falling on the North Western section can be discharged to this wadi. The water from the other three sections is spilled on the agricultural land east of the town. The water falling south of the road from Dhamar to Al-Bayda does not enter the town and is discharged along the road to the east.

The drainage system consists of a primary, secondary and a tertiary system. The tertiary system is in the streets and alleys in the densely built-up areas and conveys the water to the primary and secondary systems. The secondary system is in the somewhat smaller roads. Water from the secondary system will be discharged into the primary system. The primary system follows the main roads. It collects the water of the secondary and tertiary systems and discharges the water out of town.

The design of the drainage system in this report only concerns the primary system.

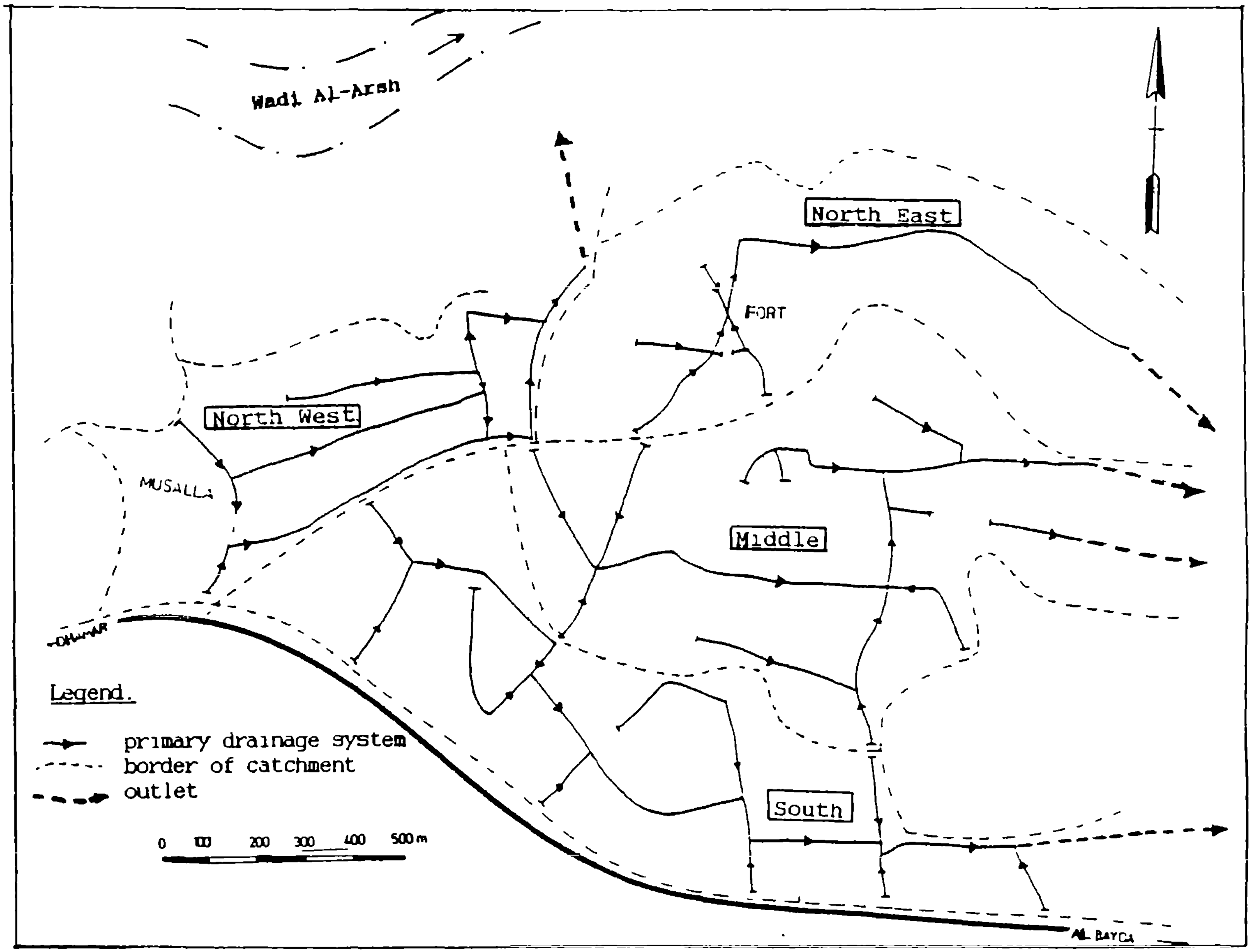
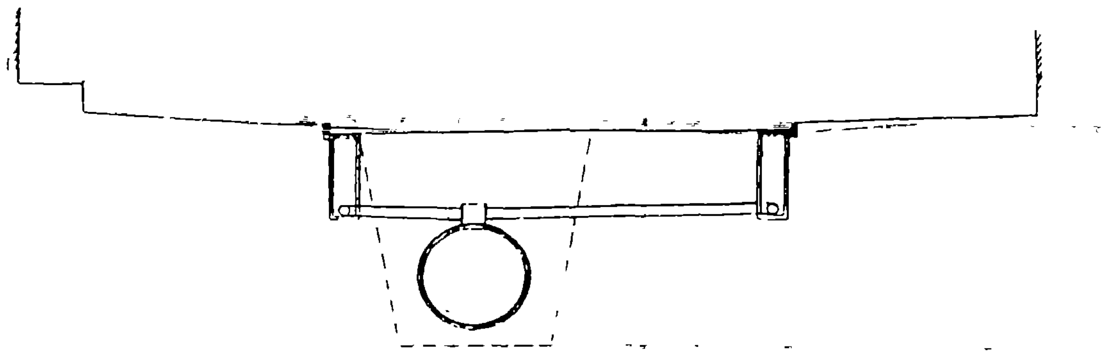


Figure 5: Lay-out present design of drainage system.

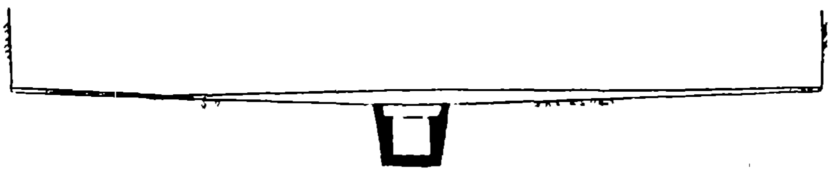
3.4 Type of drainage system.

The primary system can be of the following types:

1) **Closed drain structure.**
   - Collector drain under the road surface.

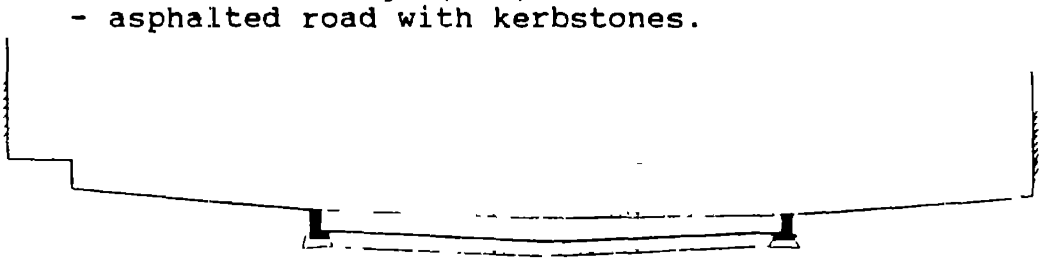


   - Closed culvert structure in the axis of the roads or at one or both sides of the roads.



2) **Open culverts.**
   - at one or both sides of the roads.

3) **Road Surface Drainage (RSD).**
   - asphalted road with kerbstones.



The RSD consists of paved roads with kerbstones so high, that the maximum water levels caused by the design rainfall, will be just below the top of the kerbstone.

An extensive comparison between these options has been made in "Considerations and recommendations for stormwater drainage, RWSSP" [lit. 3]. The conclusion is that for Rada the road surface drainage is the best solution.

The considerations are listed in the table 1:

	Closed	Open	RSD
Space required	+	-	+
Traffic	+	-	-
Safety		-	+
Maintenance	-	-	+
Costs	--	-	+

Table 1: Considerations type drainage system.

Space required.
For the closed drainage systems and for the RSD no separate space is required, whereas for the open culverts lack of space gives problems with fitting the system in the densely populated town.

Traffic.
A closed drainage system does not hinder the traffic. Open culverts have the disadvantage that cars might get stuck in them. At crossings the open culverts need special provisions so the cars can cross them.
For the RSD the traffic will be hindered at every heavy shower. This however will not last long. All the water will be discharged within a few hours. For the other types of drainage systems, this hindrance only occurs after rainfall intensities exceeding the design intensities.
For the RSD, the surface at the crossings should be shaped in such a way, that all the water is discharged and that no water will remain at the crossings after the rainfall.

Safety.
For safety of the public the open culverts should be covered in front of shops, houses etc. The open culverts make the effective width of the road smaller, reducing the space to walk. Because the RSD will consist of roads with high kerbs, the pedestrians will be reasonably save on the sidewalk.

Maintenance.
The closed and open drainage systems are easily clogged by solid waste, sand and stones.
To prevent sand and stones entering the closed system, the surface should be asphalted. This makes it even more difficult to carry out maintenance works. To keep the open culverts open, maintenance will be necessary. There is some experience with an open discharge channel through Rada: it was clogged very quickly, and hardly any maintenance was carried out. In the RSD sand and garbage will also collect, but this can be easily removed by sweepers.

Costs.
The costs of the construction of a RSD are less than half of that of the other possibilities. The maintenance costs are estimated to be half of those for the culvert system, and even 10 to 20 times smaller than for the closed drainage system.

Conclusion.
The RSD has been chosen for the drainage of stormwater in Rada. The low costs and easy maintenance of this alternative were the decisive arguments. This solution can be seen as an improved old situation. Originally the stormwater was drained over the streets too. The difference is, that now the roads will be asphalted and shaped and that kerbstones will be laid, so that the water will be conveyed through the streets, out of town.

A side advantage of the choice of the RSD is the following: It is the policy of DGIS not to finance projects constructing asphalt roads, but when the RSD is chosen, paved roads will be constructed and will be paid by DGIS. This will only be this way in the streets of the primary drainage system. To derive optimum profit of the paved roads, additional roads will have to be paved by the local authority. See annex B1, figure B1 for a map of the roads to be paved. The additional pavement will be financed by the local Authority.

For the safety of the public (pedestrians, children playing, etc), speed reducing devices (speed-humps) should be designed and provided. If these are omitted, the people will make them themselves, ruining the roads and the drainage system.

3.5 Cross section primary system.

To ensure that the whole street will not be covered with water after a small rainfall event, the roads should be sloping towards the sides or the middle. In consultation with the authorities concerned[3] a cross section such as in

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[3] MHUP: Ministry of Housing and Urban Planning

figure 6a has been chosen for the greater part of the primary system. See annex B2. The side slope will be small (i_s = 1:100) to limit the side depth.

a) main roads:

b) small roads:

Figure 6: Cross section primary drain. a) main roads,
b) smaller roads.

For the smaller roads and the roads with only small water levels, the road will be sloping to one side only; with a high and a low kerb (figure 6b).

4 PRESENT DESIGN STORM.

4.1 Introduction.

For the design of the drainage system, it is necessary to know how much rainfall can be expected and how much should be drained. If the design is made for a very heavy rainfall, which occurs very rarely, the system will be able to drain all the water in most occasions, but the costs will be high. On the other hand, a cheaper system, with a design based on a more frequent storm, will more often not be able to discharge all the water, and flooding will occur. Therefore a choice has to be made between costs and chances of failure.

The design storm used for the present design of the (primary) drainage system is presented in the Final Design Report of the RWSSP, December 1989. It was accepted that the drainage system had to be designed on a rainfall with a return period of 2 years. This is a commonly used return period for urban areas like Rada. This would mean that (according to the computations) the water level would be just below the top of the kerbstone once every 2 years. The present design storm was derived using 11 years of daily rainfall only. From these daily rainfall figures the rainfall intensities for durations of 15 minutes and longer were derived with the method as described in the paragraph below. This storm is referred to as the present design storm.

4.2 Rainfall intensities used for present design storm.

From 11 years of daily rainfall for each year the maximum rain falling in 1, 2, 3 or 4 consequent days is computed. This is the k-day rainfall for k = 1, 2, 3 or 4. Out of these maximum rainfall figures it is computed how much rain falls with a return period of T = 2, 5 or 10[4] years. This has been done with the method of Gumbell (see annex C.4.1). This way the maximum rain falling (with a return period of 2, 5 and 10 years), for periods of 1, 2, 3 and 4 days is computed. These 4 figures have to be converted for each return period into rainfall intensities for short periods. The following commonly used intensity-duration equation is used:

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[4] Since only 11 years of rainfall data were available, the rainfall with a return period of 10 years is uncertain

$$I_a = \frac{a}{(b+t_d)^n}$$

with I_a = average intensity in mm/hour
t_d = duration of rainfall in hours
a, b and n are constants.
This equation is for periods smaller than 24 hours.

In the computations of the present design storm the constant **b** for Rada was accepted to be 0.3. A line according to this equation is fit through the extreme intensities (giving the values for a and n). The rainfall intensities for short durations can now be found. Because rainfall intensities for durations of 15 minutes are derived from daily rainfall figures, the result is not too reliable and sensitive to little changes in the four k-day rainfall figures. The outcome is also uncertain, because the equation is meant for a period smaller than 24 hours, while the input consists of k-day rainfall figures.

According to the Final Design Report of the RWSSP, this method resulted in the following intensity-duration equation for a return period of T = 2 years:

$$I_a = \frac{27}{(0.3+t_d)^{0.93}}$$

However, this appeared not to be the best fit of the line through the k-day extreme rainfall figures. The best fit would give values of a = 18.7 and n = 0.77, which results in much smaller intensities for the short durations, than were computed.

With the computed intensity-duration equation a design storm for T = 2 years was created, using the so called USA Soil Conservation Procedure. This procedure has been explained in annex C.6.4 for another design storm. In figure 12 the result is shown as "present design storm."

4.3 Conclusion RWSSP design storm.

The rainfall intensities as given in the Final Design Report of the RWSSP, 1989 are not very reliable, because only daily rainfall figures were used. Furthermore, it appeared that the method as described in the Final Design Report had not been applied correctly. The resulting intensity-duration equation gives higher intensities than the theoretical correct solution (best fit). See figure 7.
In figure 7 also the largest rainfalls in 60 minutes, measured in Rada, are given. In the three years of which pluviographs are available, the measured rainfall exceeded the 30 mm in 60 minutes already 3 times. According to the results and data as presented in the Final Design Report, a rainfall of 30 mm in 60 minutes would correspond to a return

period of 5 years. It is very unlikely that a rainfall with a return period of 5 years, is exceeded 3 times, within a period of 3 years.

Consequently, it is concluded that the results as presented in the Final Design Report, December 1989 of the RWSSP, are probably too small.

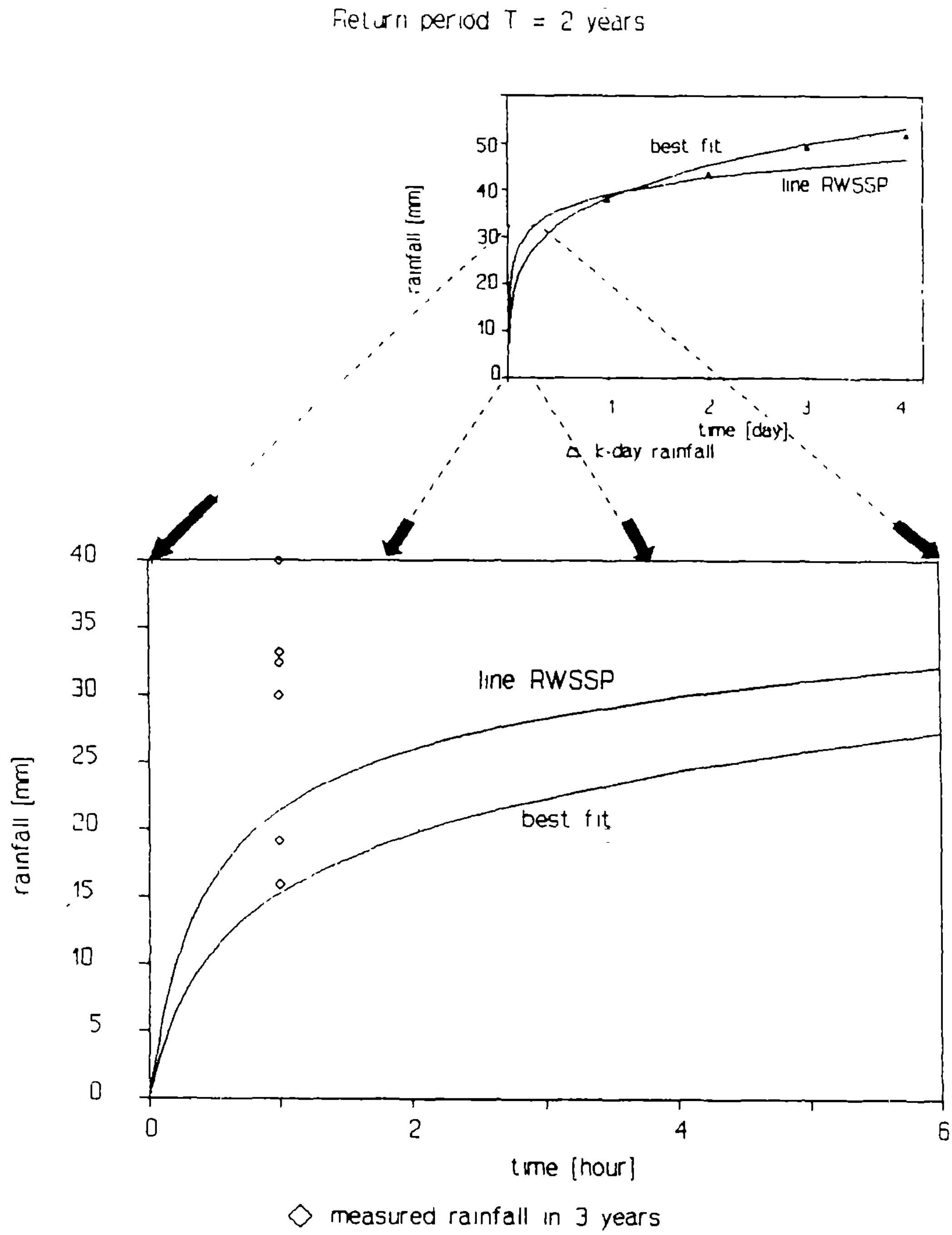

Figure 7: Rain-duration curves: RWSSP-curve and best fit.

5 NEW DESIGN STORM.

Due to the fact that more rainfall figures are available and because the present design storm seems to be too small, the computation of the design storm has been redone.
In annex C the way the design storm is derived is discussed more in detail.

5.1 General.

In Rada the average yearly rainfall is about 200 mm. There are two rainy seasons: The first, in which most of the rain falls, is from January until May. The second rainfall period is from July until September. See figure 8. In arid areas like Rada, rainfall is mostly caused by convective storms[5]. This means that the rain falls with a high intensity during a small period. The rainfall intensity is constant over the whole core of the shower, which has a diameter between 1 and 7 kilometre. Outside this core, the intensity decreases to 0 mm/hour within 10 kilometre.

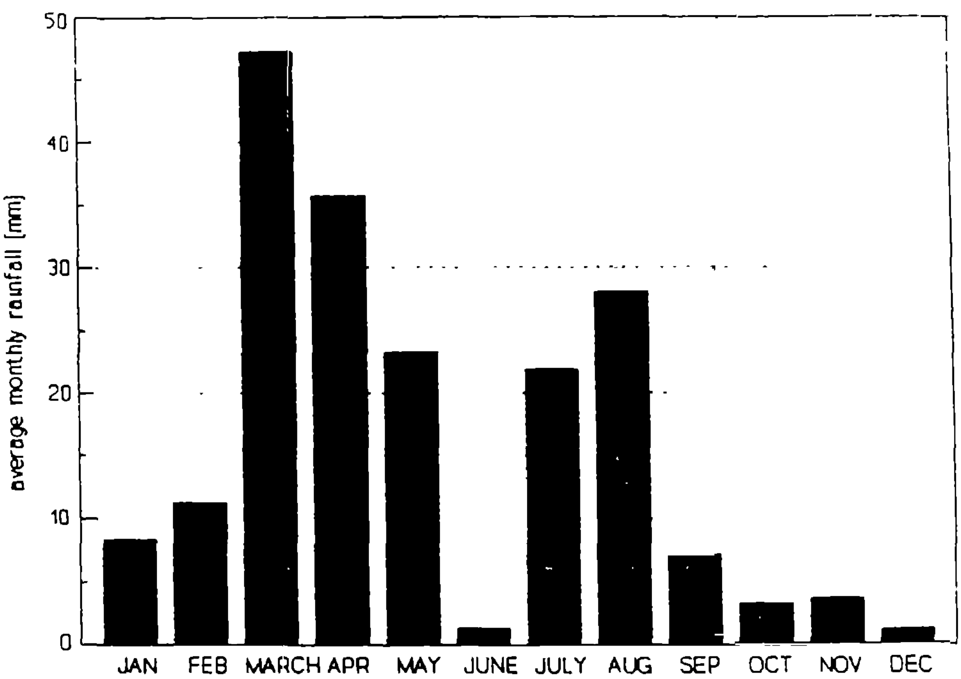

Figure 8: Average monthly rainfall Rada.

(5) Warm air at the surface rises to higher levels, where the water condensates and clouds are formed. These are transported by the wind and give local, short lasting showers, with high intensities. These showers mainly occur in the afternoon.

From the measured rainfall data of Rada it appears, that indeed the rain mainly falls in a few heavy showers. These showers can flood parts of the town.

To design the drainage system of Rada, a design storm has to be created. To do this, it is necessary to know the rainfall intensities for short periods like 15, 30, and 60 minutes. The design storm will be fed in a computer model of the drainage system, by which the water levels on the streets of the primary system and the discharges will be computed.

5.2 Methods of computing design intensities.

As pointed out in annex C.1, a long series of total daily rainfall and 3 years of pluviographs are available for Rada. Both series can be diverted into design intensities and a design storm (though with some difficulties). The methods are described below:

Total daily rainfall.
The first method is to convert the total daily rainfall figures into rainfall intensities for short durations. This is the method applied in the Final Design Report and described in sub-chapter 4.1.
As pointed out before, this method is not very reliable and sensitive to little changes in the four daily points.

Pluviographs.
For the method using pluviographs, the rainfall in short periods is directly read from the pluviographs. A drawback is that the amount of data is limited to 3 years, making it impossible to find the design storm with a reasonable return period out of the pluviographs only. Another point is that from the pluviographs rainfall in 30 minutes can be read, but the rain falling in 15 minutes is not clear (see figure C2 in annex C). Therefore an extrapolation to 15 minutes has to be carried out anyway.

Combination.
To predict a storm with a certain return period, both the long series of total daily rainfall and the pluviographs are used. A relation between rainfall in a short period and total daily rainfall is derived from the pluviographs. Next the daily rainfall can be converted into rain falling in shorter periods. With the Gumbell method (see annex C.2.2.1) the extreme values for different return periods can be computed. Finally an extrapolation using the same equation as in sub-chapter 4.1, gives the rainfall for periods of 15 minutes and longer.
This method is better than the former two, because of the following three reasons: the extrapolation is executed less extended and therefore more reliable. The equation is used

for periods smaller than 24 hours, for which it is meant. Finally, in this way all the information is used.

5.3 Design intensities.

In annex C.2. the way the design intensities are computed is given more comprehensively.
To be able to convert the daily maximum rainfall into rainfall in short periods, first a relation between daily rainfall and rainfall in short periods was derived from the pluviographs. Next the intensities occurring with an average of once every 2 or 5 were computed (the extreme rainfall intensities). Through these extreme rainfall intensities a line according to the intensity duration equation is fit.

For periods up to 24 hours the average rainfall intensities can be described with:

$$I_a = \frac{a}{(b+t_d)^n}$$

with I_a = average intensity in mm/hour
t_d = duration of rainfall in hours
a, b and c are constants.

For T = 2 years the intensity-duration curve is:

$$I_a(2) = \frac{48.8}{(0.5+t_d)^{1.03}}$$

The value of constant a for T = 5 years is 67.5. The constants b and n are equal for all return periods. With this equation the design intensities can be computed.
The results of the regression analysis is given in table 2 and figure 9.

duration	Return period T			
	2 years		5 years	
[hour]	[mm]	[mm/h]	[mm]	[mm/h]
0.25	16.4	65.6	22.7	90.8
0.50	24.4	48.8	33.7	67.5
0.75	29.1	38.8	40.2	53.6
1.0	32.1	32.1	44.5	44.5
2.0	38.0	19.0	52.5	26.3
24	43.4	1.8	60.1	2.5

Table 2: Resulting rainfall in [mm] and rainfall intensities in [mm/h] for return periods of 2 and 5 years.

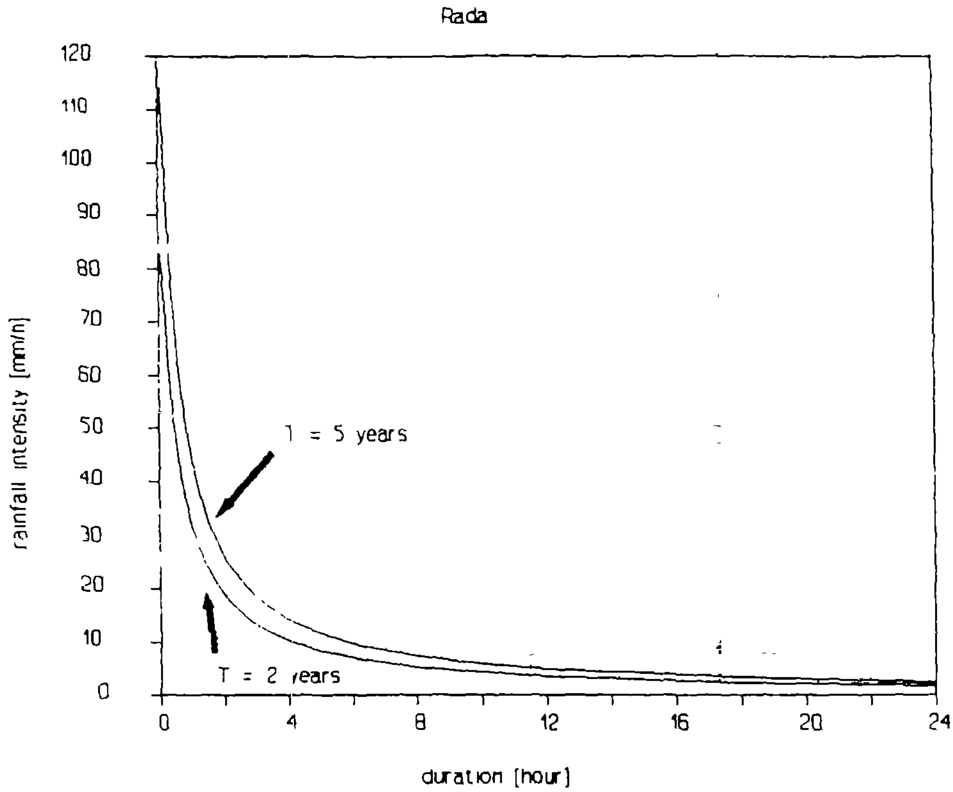

Figure 9: Intensity duration curve.

5.4 Design storm.

The return period to be chosen for the design storm should be related to the damage caused, if water leaves the drainage system, i.e. if the water levels rise above the top of the kerbstones.

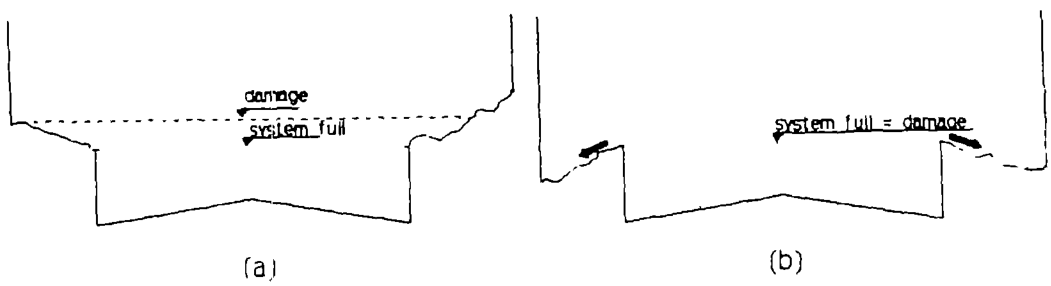

Figure 10: Cross-section of streets.

For a situation as in figure 10a, the houses will not be damaged if the water level rises above the kerb (until the houses are reached). In that case the return period to design the kerbstones can be smaller. If the situation is like in figure 10b, the houses will be flooded as soon as the water leaves the drainage system (water levels above the kerbstone). In that case the return period for the design should be higher.

It is not clear where the situation is like (1) and where like (2) (this has to be investigated on site). Therefore it has been accepted that the water levels should not rise above the kerbstone for the chosen return period. A return period of 2 years, which is commonly used for urban areas like Rada, has been chosen for the design of the drainage system. This means that, with an average of once every 2 years the water level is just below the top of the kerbstone[6].

In annex C it has been decided to design the drainage system on a design storm as given in figure 11.

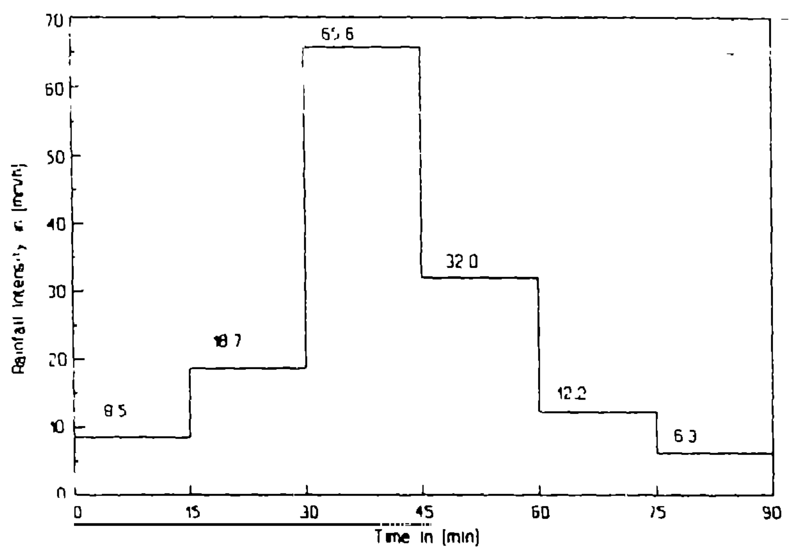

Figure 11: Created design storm (= new design storm) T = 2 years.

In figure 12 the 4 heaviest storms measured in Rada, as derived from the pluviographs, are presented, together with the old and the new design storm. It appears, that the peak of the present design storm is smaller. The maximum intensity of the present design storm for a period of 30 minutes, is 33 mm/h, while the average intensity of the measured storms, for a period of 30 minutes is 43 mm/h. Figure 12 shows, that the present design storm would probably be an underestimation of the storm with a return period of 2 years.

The average intensity for 30 min for the new design storm is 49 mm/h. This is higher than the average peak for 30 minutes of the measured storms (which is 43 mm/h). However, the resulting water levels caused by the measured storms and those caused by the new design storm, do not differ very much, as is shown in annex C7, table C9. Compared to the

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(6) Minus a certain percentage of the kerbstone hight (freeboard) for safety reasons. See sub-chapter 6.2.

measured storms, the new design storm might therefore be a small overestimation. It is emphasized that the measured storms are the storms of 3 years only. So from these data it is impossible to derive a design storm with a return period longer than 2 years.

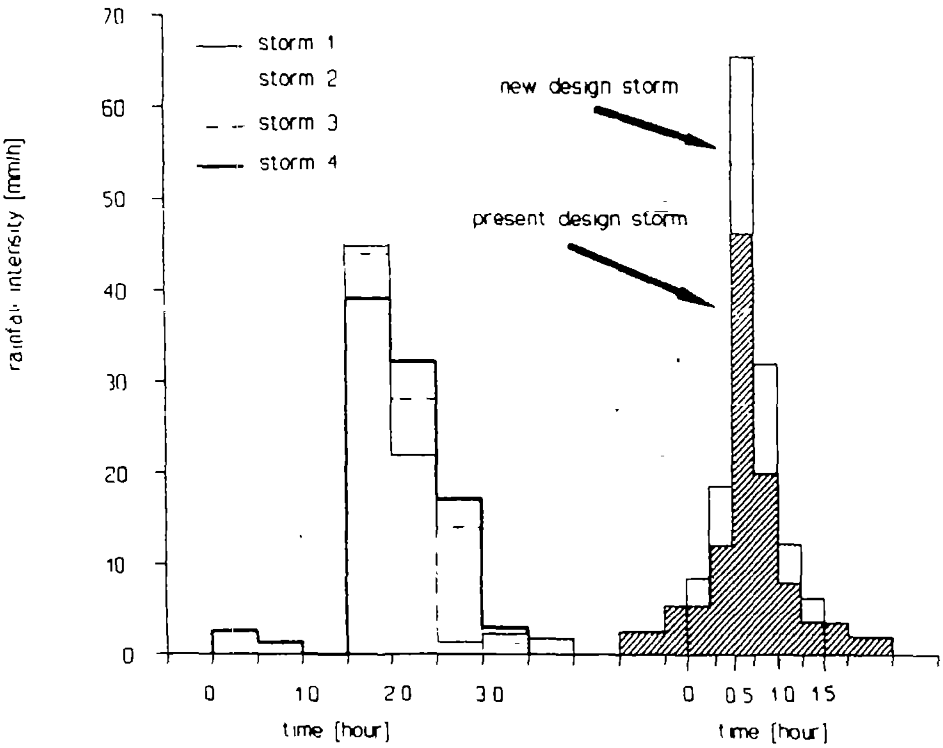

Figure 12: The 4 heaviest storms of Rada and present and new design storm.

5.5 Areal reduction.

The design rainfall will be accepted to be homogeneous for the whole catchment. The rainfall figures used, are maximum rainfall figures. If rainfall is the result of convective storms (as is the case in Rada), the difference in rainfall in two close points can be quite considerate. Consequently, the average rainfall figure for the whole catchment will be smaller than the maximum rainfall figures used. Generally a reduction of the rainfall is not used for area's smaller than 2 square kilometre. The town is divided into four sections, each not larger than 60 ha (or 0.6 km²). Therefore no areal reduction is taken into account.

6 PERFORMANCE PRESENT DESIGN OF DRAINAGE SYSTEM.

The present design of the drainage system is based on the design storm of chapter 4. The computer model CYCLONE [lit. 3] was used to compute the water levels in the system. Because the new design storm (chapter 5) is much larger and because some mistakes were made in the input of the model, it is accepted that the present design is no longer adequate.

6.1 Computer model.

The computer model CYCLONE is used to compute the expected water levels and discharges in the drainage system. The system is presented as a combination of nodes and conduits. The program transforms the precipitation on a catchment into an inflow hydrograph at a node. For this purpose the overland flow is computed, taking into account the slope, length and roughness of the catchment. In the system of nodes and conduits the varying water levels and discharges are computed at the nodes and in the conduits respectively. The input consists, amongst others of:

- catchment size, slope and typical length.
- roughness of conduits and catchments.
- surface and invert levels at nodes (see figure 13).
- length and shape of conduits.
- runoff coefficient.

See annex D for a more elaborate description of the model and the input.

Figure 13: Cross section road. The surface level is the ground level next to the road. The invert level is the lowest point of a cross section of a road.

6.2 Critical water levels.

The input of the new design storm in the present design of the drainage system[7], showed that the system is not adequate. Flooding occurs in the Middle and North-Western

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[7] As given in the annexes of the Final Design Report RWSSP, december 1989

section, and at the other sections relative water levels[8] are so high (>90%), that, regarding uncertainties in the input and the modelling, flooding might occur (see table 3).

section	relative water levels
North-East	95%
North-West	>100%
South	83%
Middle	>100%

Table 3: Maximum relative water levels caused by new design storm in present design drainage system.

A freeboard of 15% (i.e. relative water levels ≤ 85%) is accepted, because of the uncertainties in the input (design storm a.o.) and modelling of the drainage system (see annex E).

6.3 Difference in modelling.

As some errors were made in preparing the input to the model, the input was controlled and where necessary changed. For example:
Some node numbers were exchanged. To node 5331 of the middle section a catchment of only 0.1 ha was connected, while, this should be about 3 ha. Correcting this resulted in high relative water levels and even floodings at the downstream nodes. See figure 14 for the location of node 5331 (The area concerned is shaded).

The lengths of the conduits of the North-Western section were sometimes different from what could be read from the maps. The schematisation of the catchments discharging at a node is now accepted to be different from what was assumed in the Final Design Report. See annex D.4. This did not have a big impact on the flow through the system.
Also the average slope and the critical length of the catchments were measured again.

The new input is given in the annex D.

[8] The relative water levels is referred to as the waterlevels in a conduit divided through the kerb hight (surface level - invert level) in percentages.

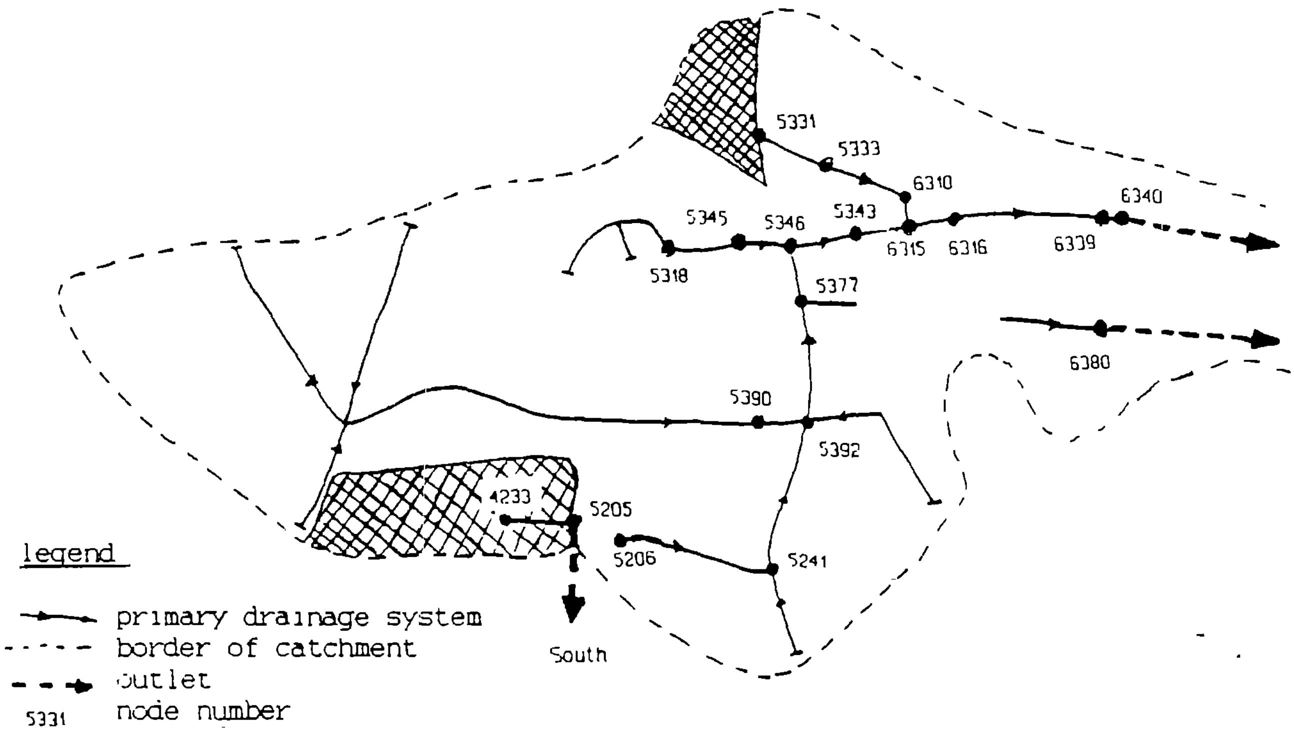

Figure 14: Middle section.

7 NEW DESIGN OF DRAINAGE SYSTEM.

The present design of the drainage system was altered in such a way, that the present design storm results in relative water levels smaller than or equal to 85%.

7.1 Recommendations.

The alterations in the design are not radical. They are of two kinds: The most important is to reduce the relative water levels to acceptable levels, while the other is to obtain limited sizes of kerbstones. The recommendations are given in table 4.

For the construction of the drainage system and the roads, it is easier to have only a limited number of sizes of kerbstones. The kerb stones will be 15, 25, 35, 40 and 50 cm high, with a kerbstone of 65 cm at one point. Because of this, some minor adjustments should be made to the present design of the drainage system.

Middle section.
The largest problems occur in the middle section. The water from nodes 5390 and 5392 is forced to the main street in the north, from where it flows out off town to the east. These problems can be reduced by disconnecting two nodes of this section. Water of nodes 4233 and 5205 can be discharged to the Southern section (without causing severe problems there), instead of to node 5206. See figure 14.

Further, the problems at branch 5331-5333-6310 can be solved by creating a steeper gradient and higher conduit depths. It is suggested to take the surface level of these nodes 5 to 15 cm higher. Now, to be able to discharge the water from the whole connected catchment, the low areas round node 5331 should be filled up a little bit (about 10 cm).
The invert level at 5331 should be higher, while at 5333 and 6310 it should be lower. This solution reduces the relative water levels to acceptable levels.

In the model, a node is added just before the spillway, to be able to connect the corresponding catchment to that point and to check the situation there. It appeared that the conditions at the spillways were satisfactory.

The relative water levels in the new design are up to 92%. So the accepted freeboard of 15% is not available. However, a relative water level of 92% only occurs at node 6315. This is in the main street. The higher the kerbstone will be the greater will be the hindrance to the many pedestrians in this busy street. A kerbstone of 50 cm has been designed. To reduce the hindrance, the kerbstone should be sloping as in figure 15. These high relative water levels are possible at this node because of the following two reasons:

Node	Surface level [cm] new	Surface level [cm] old	Invert level [cm] (cm above ref.level) new	Invert level [cm] old
North East				
4498	2962	2975		
4489	2945	2943		
5301	2980	2982		
5448	2506	2504		
6420	2503	2508		
6494	2356		2341 *	
6495	2355		2340 @	
North West				
3338	3428	3431		
4466	3285	3290		
4305	3315	3314		
4316	3341	3331		
4452	3222		3197 *	
4453	3121		3096 @	
4455	3122		3097 *	
Middle				
4233	to Southern section			
5205	to Southern section			
5236	left out of model, because very close to next node.			
5318	2646	2651		
5345	2562	2560		
5331	2470	2461	2455	2446
5333	2450	2444	2425	2429
6310	2435	2431	2410	2416
5346	2490	2491		
5377	2503	2500		
5390	2557	2547		
6316	2426	2428		
6339	2374		2349 *	
6379	2395		2370 *	
South				
4210	3162	3160		
4258	3071	3066		
4233	3013		2998	from middle section
5205	2880		2865	from middle section
5261	2835	2830		
5292	2740	2735		
5297	2727	2722		
6264			2525	2530
6286	2515	2520		

*new node before spillway.
@Spillway displaced.

Table 4: Recommendations surface and invert levels and nodes: differences between present and new design.

Firstly, because of the shops the kerbstones will be sloping as shown in figure 15. In annex D4 it is computed that for a water level of 0.5 m the schematisation of the cross section in the computer model, results in an overestimation of 10% of the water levels. Thus in reality the water levels will be smaller than computed.
Secondly, before the start of the RWSSP the ground level at that point was even 0.6 m higher than the surface level as planned in the annexes of the Final Design Report, RWSSP, December 1989. It is therefore likely that the adjacent shops and houses are above that high ground level. Consequently, flooding would not cause damage immediately.

At the other nodes the relative water levels will be ≤ 85%.

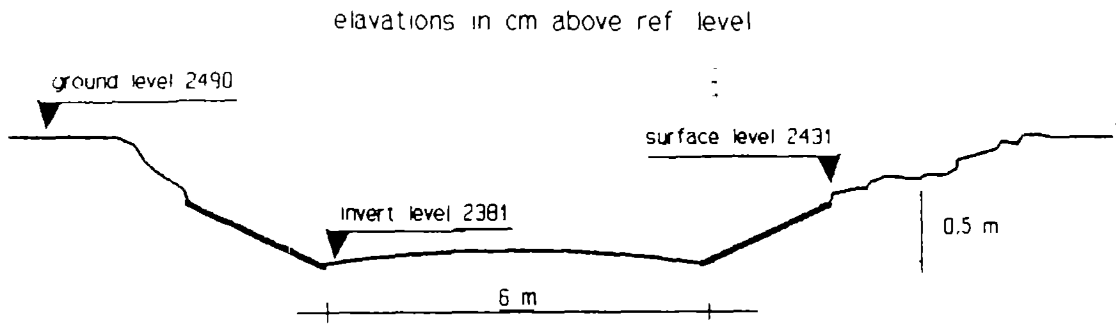

Figure 15: Cross section at node 6315.

North-Eastern section.
Only minor alterations in the design were necessary. The freeboard is ≥ 14%. At node 4479 (see figure 16) the kerb stone is even 65 cm high, because at that location the primary drain is cutting through a small hill.

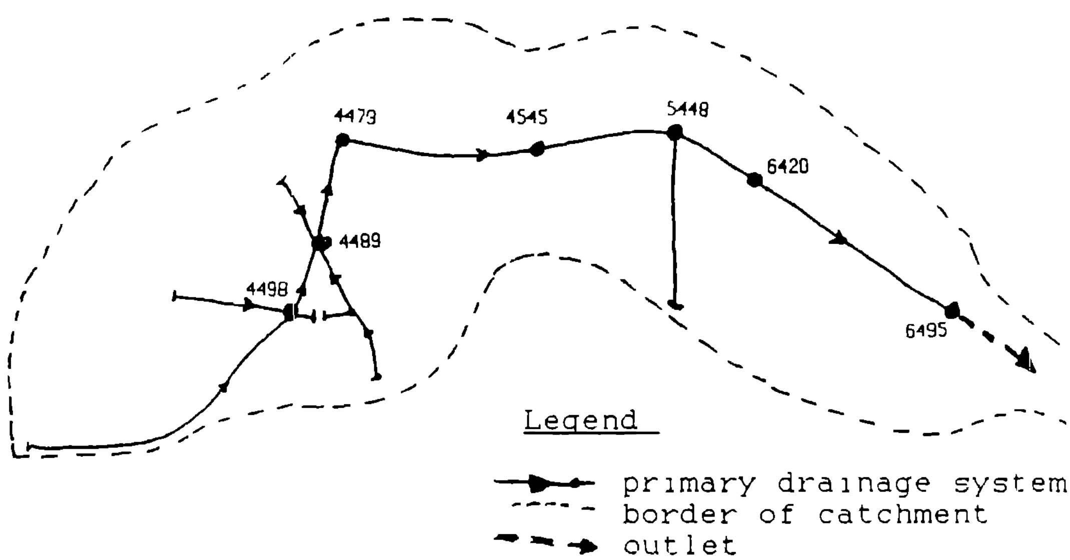

Figure 16: North-Eastern section.

Southern section.
Nodes 4233 and 5205 of the middle section were connected to node 5215. To prevent flooding or high relative water levels, the surface level at nodes 5261, 5292, and 5297 have been increased (increasing the conduit depth). See figure 17. At node 6264 the invert level is lowered. The results are freeboards of about 15%.

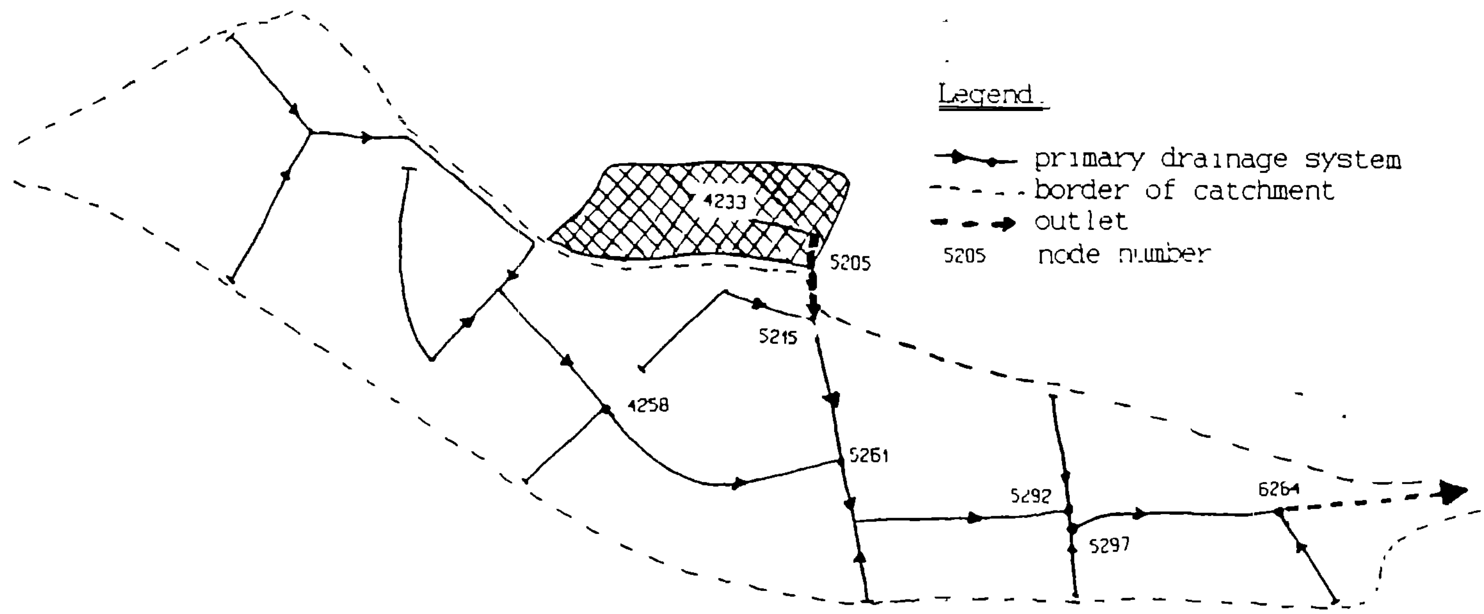

Figure 17: Southern section.

North-Western section.
The surface level at node 4316 can be 10 cm higher than the surface level in the Final Design Report, without causing problems for the over land flow. This results in a 10 cm higher kerbstone. The maximum relative water levels will then be 78% (at node 4305) (figure 18).
The spillway as presented in the annex of the Final Design Report of the RWSSP seems impossible. At 160 m from node 4466, an invert level of 32.06 m is too low to spill the water on the surrounding fields with a ground level of 32.60 m. A solution would be to have the spillway 200 m further to the north, with a conduit to it. This solution was derived from maps only. A proper solution has to be found by a visit of the location.
The water should not be spilled to the east, because then it might enter the North-Eastern section at node 4483.

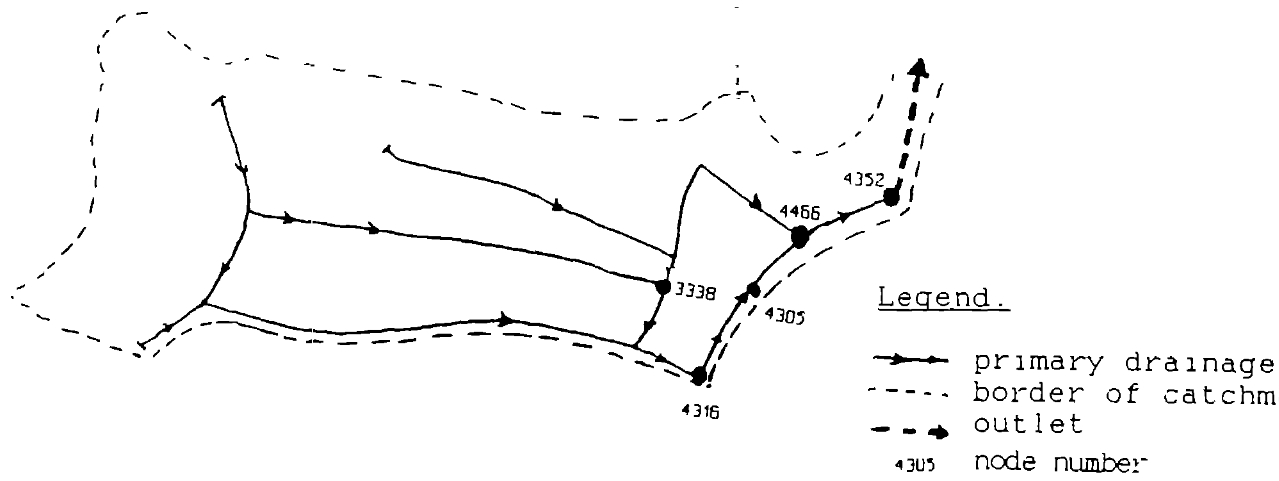

Figure 18: North-Western section.

7.2 Results.

With the recommendations presented in table 5, the relative water levels are (except node 6315) smaller than or equal to 86%:

	Relative water levels at design:	
section	**Present**	**New**
North-East	95%	86%
North-West	>100%	81%
South	83% *	86%
Middle	>100%	92% **

* Before adding 2 nodes of the middle section.
** At node 6315. The other relative water levels ≤ 85%

Table 5: Maximum relative water levels caused by new design storm.

8 ALTERNATIVE SOLUTIONS.

The water levels can be quite high in some parts of Rada. Especially between nodes 5346 and 6340 in the middle section, the kerbstones of 50 cm are a hindrance to the people, because the street concerned is one of the busiest in Rada. Alternate solutions for the discharge of the water to reduce the size of the kerbstones are: a retention reservoir upstream from the main street, an extra sewer along or under the main street, or wider roads.

Retention reservoir.
To reduce the water levels in the main street, a retention reservoir can be constructed at the (only) empty space north of node 5390 (see figure 14). It appeared, that with a reservoir of 750 m^3, the maximum water levels downstream of the reservoir would be about 5 cm smaller. For reasons of hygiene, the reservoir should be drained soon after the rainfall.
However, a reservoir of 750 m^3 is difficult to fit in at that location. Node 5377 seems to be the best node to discharge the water to, but there is only 70 cm of fall available between the empty space and node 5377. Consequently, if water is drained by gravity, the reservoir would have to be very shallow. For a depth of 0.5 m, the area of the reservoir would have to be 1500 m^2 . This is not available. Another option is to drain the reservoir by pumping, so the reservoir can be deeper. This solution is rejected, because then the drainage system would become less reliable. Furthermore, the danger that garbage will be dumped in such an empty space in town is considerable. In this way the hazards for public health would remain.

Extra drain.
A second alternative might be to have a kind of channel or sewer in the last part of the Middle section (from node 5346 or 6315 onwards). Nevertheless, an open channel will very soon be clogged with sand, rocks and garbage. A sewer requires a diameter of 1.5 m. This would mean excavations of 2.3 m to include a cover of 0.8 m. Because of the very mild slope at that part of the town, the sewer should go quite far out of town to be able to spill the water on the agricultural land. Finally a sand and garbage trap should be constructed at the beginning of the sewer, to prevent clogging. All this would make this alternative much more expensive than the Road Surface Drainage. The costs and the fact that the sewer might be clogged, is an important disadvantage of this solution.

Wider roads.
If the width of the roads is 8 m instead of 6 m, the water depth will be considerable smaller. Assume Chézy:

$$Q = bhC\sqrt{RI}$$

with	Q = discharge	$[m^3/s]$
	b = width of road	[m]
	h = waterdepth	[m]
	C = factor of Chézy	
	R = hydraulic radius	[m]
	i = longitudinal slope	[-]

The ratio between R and h appeared to be constant (about 0.92) for a waterdepth from 0.2 m to 0.5 m. The equation above can now be rewritten as:

$$Q = bh^{\frac{3}{2}}C\sqrt{I}\sqrt{\frac{R}{h}} = 0.92C\sqrt{I}*bh^{\frac{2}{3}}$$

Then if is accepted that the discharge is constant, a rough estimate gives that the waterdepth for a width of 8 m is almost a factor $(6/8)^{2/3}$ smaller than the water depth for a width of 6 m:

$$Q_1 = Q_2 = b_1h_1^{\frac{3}{2}} = b_2h_2^{\frac{3}{2}}$$

or

$$h_2 = (\frac{b_1}{b_2})^{\frac{2}{3}}h_1$$

In the present design of the drainage system some roads at critical locations are already 8 m wide. In figure B2 of annex B the width of the roads of the primary system is given. It is not possible to read from the maps whether other roads can be made wider too. It is recommended to investigate on site the possibilities of making some roads wider, to reduce some of high kerbs.

9 CONCLUSIONS.

As the present design storm[9] seemed to be too small, a new design storm was computed. This new design storm would cause floodings in the present design of the drainage system[9]. The new design of the drainage system consists of the present design with the recommendations as presented in table 4. In the new design of the drainage system the new design storm does not cause any floodings.

However, the water levels can be quite high and hence the kerbstones in some parts are high too. On the other hand, the duration of the high water levels, during which traffic will be impossible, is short. After this period the water levels will be small and the hindrance limited. Within 2 hours after the end of the storm all the water will be of the streets. Upstream in the system the roads will be dry even quicker.
Especially during the long dry periods with no rainfall at all, the high kerbstones might seem odd to the people.
A heavier rainfall than the design storm will still cause flooding[10]. It depends on the location, whether the flooding will cause problems. When the peak discharge has passed the water can enter the system again and will be discharged. Consequently, the hindrance will be less than at present, when the streets are flooded for at least a week after heavy rainfall.

The impression is often that the water levels and the freeboards in the system have been calculated accurately. However, the calculations are not that precise. Firstly, the modelling of the drainage system and secondly the design storm cannot be given exactly. It is estimated, that the return period T = 2 years should be interpreted as a return period between 1.5 and 3 years. See annex E.

The high kerbstones will be a nuisance to the people. Yet, alternative solutions to reduce the maximum water levels (such as a retention reservoir or a sewer in one part of the town) did not seem to be attractive alternatives. Wider roads is an option, which still has to be investigated.

Concluding:
With the new drainage system the water will be discharged out of town in a few hours, whereas nowadays parts of the town are flooded for several days after heavy rainfall.
Another advantage of the drainage system is, that the main roads will be asphalted.
These will be great improvements and will compensate for the high kerbstones and hopefully improve the living conditions of the inhabitants of Rada.

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(9) as presented in the Final Design Report, RWSSP, December 1989.

(10) A created storm with a return period of 5 years and a measured storm with the same return period indeed showed that in the Middle section flooding would occur.

Annex A:

BACKGROUND INFORMATION

CONTENTS.

A.1 SHORT HISTORY OF YEMEN.

In ancient times in Yemen a high civilization existed. About 600 B.C. at Marib (100 km north-east of Rada) a huge dam was in use, irrigating thousands of hectares. The old trade routes for spices and incense brought wealth to this area.

The Islam was introduced in Yemen in 628 A.D. In 1839 the English took power over Aden and parts of what later became South-Yemen. In North-Yemen the Imam had the absolute power, but in 1962 the military took over after a coup. In the civil war following, the republicans were backed by Egypt, while the Saudi Arabians were at the side of the royalists (Imam). Finally, in 1970 the moderate government of the republicans was recognized by the Saudi Arabians. From that moment on, the country accepted a more open course towards the West. Since the beginning of the Republic quite a few coups occurred and a war against South-Yemen broke out. Meanwhile, South Yemen had become Marxist after a revolution in 1967, dismissing the British and the traditional monarchs.

In June 1990 North and South Yemen were finally unified again in one land: The Republic of Yemen, forming a country with more inhabitants than the rest of the Arabic peninsula (12 million inhabitants) and about 12 times as big as the Netherlands. The soil is reasonably fertile, resulting in substantial rainfed agricultural activities where enough rain is available. Besides, the exploitation of oil (partly on the former border with South-Yemen) has been started. But until now the land has only a few revenues. Before the Gulf-crisis, many Yemenite were working in countries like Saudi Arabia, Kuwait, Iraq, etc, bringing or sending money home. Because of this money quite a lot of expensive goods like cars, diesel pumps and generators can be found in Yemen, although Yemen is said to be one of the poorest developing countries in the world.

In Yemen there are many beautiful traditional houses. Although Yemen is nowadays open for Western influence, the people preserve their old customs. Many men wear trousers during the morning, but in the afternoon, they change into their traditional clothing: a kind of skirt, a shirt, jacket and a beautiful embroidered belt with a big traditional knife (the Yambya) and a scarf round or on the head. Often they also have heavier weapons than the Yambya, like machine guns. It is custom to attend a qat-session in the afternoon. Qat is a stimulating (non-addictive) drug. The leaves are chewed and stored in the cheek during the qat-session, which can last until 8 o'clock in the evening. During qat-session problems are discussed and arrangements made. Qat is said to have a positive effect on the discussions. For qat a lot of money is spent daily; about 15 Dutch guilders.

Yemen is an Islamic state. Of course some differences in customs exist over the country, but specially between the former South-Yemen and North-Yemen. In the former South-Yemen the Islamic rules were (applied) less strictly. In Aden there even is a brewery. Now the two countries are one, it is the question whether the more fundamentalistic North-Yemen will demand that this brewery will be closed or not. Also the women are freer in South-yemen, but even in North-Yemen, differences in the position of the women exists. In Rada the women are virtually totally covered, when on the streets. Their whole face is hidden behind a veil. Inside the houses or at the premises, which are surrounded by high walls, the veils go off. The life of men and women is separated. Even at the wedding ceremonies there are separate parties for men and women.

A.2 PROJECT.

The Rada Water Supply and Sanitation Project (RWSSP) consists of the following subjects: the implementation of a drinking water network, a sewerage system, a solid waste disposal system and a drainage system in the Rada Urban Area (RUA), together with an Environmental Health Education (EHE) and an institutional strengthening programme. The drainage system is discussed in the main report. The other parts of the project are discussed here.

A.2.1 Drinking water.

In Rada there are a few small private drinking water networks and one big co-operation. The supply is intermittent and because there are no storage facilities in the system, the pressure falls during the peak hours. Therefore almost all the houses have a pump and a storage tank on the roof.
The quality of the water is poor. The water is saline and bacteriological unreliable because of the intermittent supply. Many pipes are above ground level and pass through pools with stagnant waste water, (sometimes) rainwater and solid waste. If the pressure is off, dirty water can enter the system. Bacteria in the contaminated water, can multiply if the water is in the storage tanks (on the roofs) in the sun for longer periods.

The water consumption is estimated to be 45 - 50 l/c/d[1]. This is expected to increase, as the number of taps and flush toilets increase. The water demand is estimated to be 100 l/c/d in the year 2010[2]. The new system will provide 24 hours supply with a minimum pressure of 20 m above street level. To meet this goal, first one and later two reservoirs will be needed. The first phase of the drinking water project will be for 95% of the 50,000 inhabitants in 1995. For the second phase to all (then 75,000) inhabitants drinking water should be supplied. On top of the maximum daily demand 20% has been reserved for unaccounted for water.

North of Rada a well field has been selected (see figure A1). The water only needs a safety chlorination.

(1) litres per capita per day.

(2) Water consumption for the Netherlands is 150 l/c/d.

A.2.2 **Waste water.**

Previously, all waste water was spread out over the land round the house, where it infiltrated or evaporated. In the traditional toilets the fluid and solid excrements were separated. The solid part was and is used as fuel for bath houses etc.
Because more houses are nowadays connected to the drinking water system, the water use has increased and more waste water is produced. The traditional toilets are being replaced by western type flush-toilets.
In an urban area like Rada so many people live close together, that there is too little space for the increasing amount of waste water to infiltrate in the ground. A part of it collects in pools with stagnant water, causing a considerable hazard to public health. Also the litter in the streets is part of this problem, because it clogs the topsoil and blocks the ways of discharge.

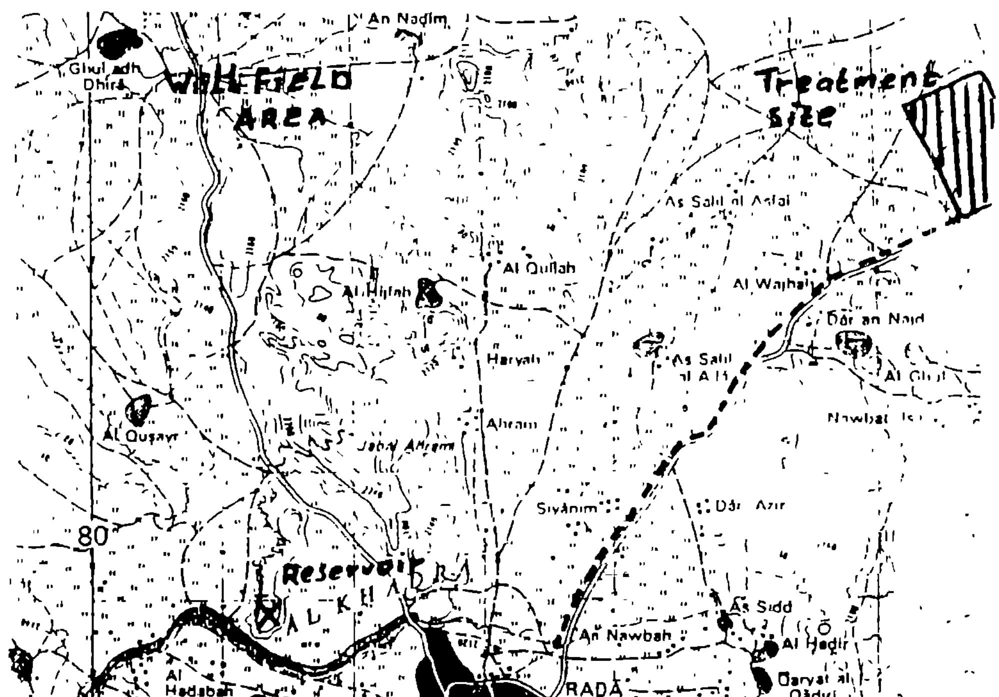


Figure A1: Surroundings of Rada Urban Area.

Nowadays some facilities to dispose the waste water are available; such as soak-away pits and a small sewer system. However, still about one third of the waste water of Rada is ending up in the streets.

As a part of the project, a sewerage system for the waste water will be constructed. The waste water is transported to a treatment plant 5 km north-east of Rada (see figure A1). The plant consists of screens, anaerobic ponds and a number of facultative ponds.

Soon after the start of the project, a number of immediate improvements were carried out. One was that an old channel through the town was restored, to discharge the waste water from a place with severe problems. This channel was, however, clogged very soon again (see figure A2).

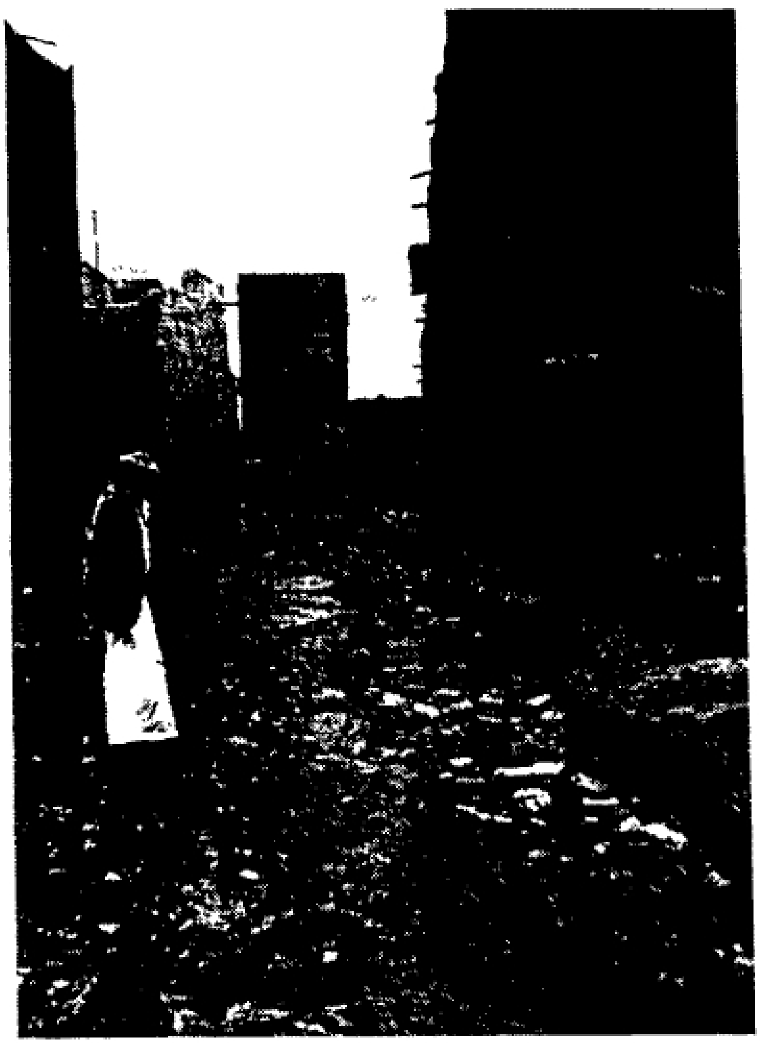


Figure A2: Channel clogged with solid waste.

A.2.3 Solid waste disposal.

It is common in Yemen to throw the solid waste over the wall round the house on the street. Previously, when almost all the garbage was organic, this didn't cause severe problems. But since plastic bottles and bags were introduced, they are everywhere on the streets. So problems as described above occur.
By the Municipality a solid waste disposal system had been introduced several years ago, but this was not working satisfactorily, probably due to the lack of information and lack of participation of the people.

As a part of the project the town has been cleaned up and containers have been spread out over the town. The garbage is collected by a compactor truck and dumped at a dump site out of town. To make solid waste collection successful, it is accompanied by a corresponding Environmental Health Education (EHE).

A.2.4 Environmental Health Education.

To convince the people of the purpose of the project and to make them confident with the works of project, an Environmental Health Education (EHE) was set up. The participation and the involvement of the population is important to prevent the failing of the project.
The EHE consists amongst others of information about the use of water and the containers. Videos are made and shown and a periodical is distributed. Schools, mosques and qat-sessions are used to give information. A considerable part of the EHE is addressed to the women.

Annex B:

DRAINAGE SYSTEM

CONTENTS.

.1 **ROADS TO BE PAVED.**

The roads to be paved are in figure B1. The additional pavement will be done (financed) by the local Authority.

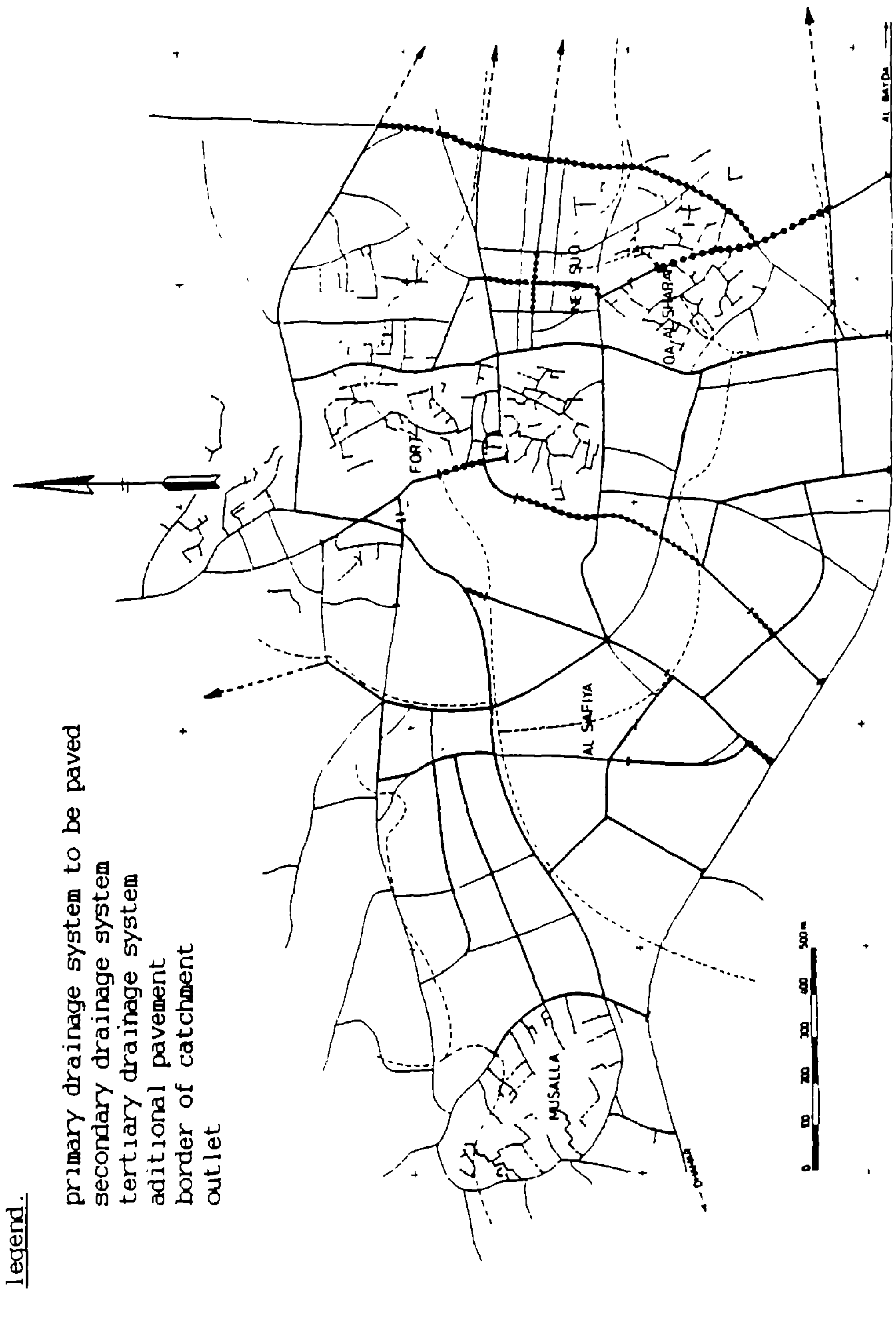


Figure B1: Roads to be paved.

B.2 CROSS SECTION PRIMARY SYSTEM.

To ensure the whole street will not be covered with water after a small rain event, the roads should be sloping to the sides or the middle. A typical cross section of the Road Surface Drainage can be one of the following:

Type 1: convex; V-shape.

Type 2: concave; roof-shape.

Type 3: sloping to one side.

A comparison of these 3 types is given below. In table B2 the results are listed.

Side depth.
Smaller kerbstones will be cheaper and give less hindrance to the users. If a maximum kerbstone level is accepted, then more water can be discharged without flooding.

The flow through a wet (cross sectional) area can be computed by means of the Chézy formula (assuming stationair flow conditions):

$$Q = AC\sqrt{RI}$$

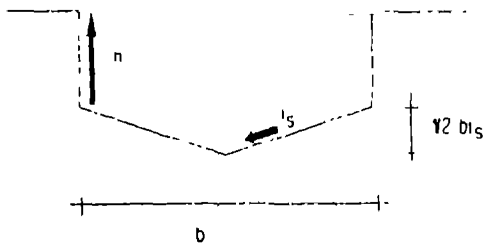
with A = wet (cross sectional) area [m^2]
R = hydraulic radius = A/O [m]
O = wet perimeter [m]
i = longitudinal slope [-]
C = Chézy factor

$$C = 12\log(\frac{18*R}{K_n})$$

with K_n = roughness of Nikuradse = 0.005 [m]

The wet perimeter O, the wet area A and the hydraulic radius for the 3 types (with a side slope i_s = 1/100) can be computed with:

Type 1:

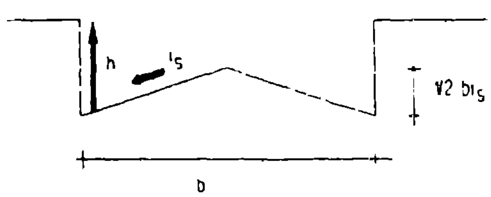


$$O = 2h+2\sqrt{(\frac{1}{2}bi_s)^2+(\frac{1}{2}b)^2} \approx 2h+b$$

$$A = hb+\frac{1}{4}b^2i_s$$

$$R = \frac{hb+\frac{1}{4}b^2i_s}{2h+b}$$

Type 2:

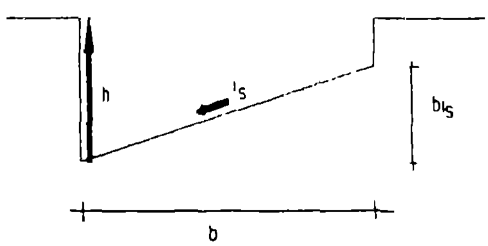


$$O \approx 2h+b$$

$$A = hb-\frac{1}{4}b^2i_s$$

$$R = \frac{hb-\frac{1}{4}b^2i_s}{2h+b}$$

Type 3:



$$O = 2h-bi_s+\sqrt{(bi_s)^2+b^2} \approx 2h-bi_s+b$$

$$A = hb-\frac{1}{2}b^2i_s$$

$$R = \frac{hb-\frac{1}{2}b^2i_s}{2h-bi_s+b}$$

For the critical rain events the waterlevels will be about 0.4 m. The most frequent road width is 6 m. For these values it appears that the hydraulic radius for type 1 is 8% (percentage of type 2) smaller than for type 2. This has a negligible impact on the factor of Chézy, which can therefore be assumed to be the same for all 3 types of cross sections. See table B1.

	R [m]	$(R_1-R_2)/R_2$ %	C	$R^{½}$ [$m^{½}$]	$((R_1-R_2)/R_2)^{½}$ %
type 1	0.37	+8%	37	0.60	+4%
type 2	0.34		37	0.58	
type 3	0.33	-4%	37	0.57	-2%

Table B1: Cross sections 1, 2 and 3.

In the formula of Chézy the discharge is proportional to the square root of the hydraulic radius R. The difference between the square root of R for type 1 and 2 is only 4%, which can be neglected. The equation of Chézy can thus be written as:

$$Q = constant*A \quad with\ constant = C\sqrt{Ri}$$

So to have the same discharge Q through the 3 types of cross sections, the wet area A should be the same (in fact, the cross section of type 1 would be slightly smaller because of the difference in friction).

The result is that for the same discharge, the side depth and kerbstone will be lowest for type 1 and highest for type 3.

For a width of 6 m and a side slope i_s = 1:100, the side depth for roads of type 1 will be only 3 cm lower than for type 2. For i_s = 1:50 the difference would be twice as big. The difference between type 2 and 3 is the same: For type 2 the side depth will be 3 cm smaller than for type 3 if i_s = 1:100. The conclusion is that for a side slope of 1:100 the difference is small.

Experience.
Roads of type 2 have been constructed all over the world. However, this is often in combination with a sewer or culverts at the sides of the roads. Also in Sana'a this kind of road (in combination with closed drains) is used for the drainage. Type 1 is very rarely used. Therefore the authorities are reluctant to choose this type of cross section.

Small discharges.
After a small rain event only a part of the cross section will be covered with water. For type 1 all the water will be in the middle, dividing the dry, accessable space into two small lanes. For type 2 the middle of the road will stay dry. Assume that to discharge the same amount of water the wet cross section A of type 1 and 2 should be equal. Then it appears that if for type 2 a lane of 2 m is free, for type 1 only 2 small lanes of 1 m would be dry. However, if the drainage system is working properly, the roads will be dry very quickly.

Maintenance.
In the middle of the roads garbage, sand and stones will provide more problems than at the sides, because it is more dangerous for sweepers to clean the roads in the middle than at the side of the roads. For Type 1 garbage, sand and stones on the roads will collect in the middle of the roads, whereas for the other two types thesed will be at the sides of the roads. After a rainfall small pools will be on the road.

	Type 1	Type 2	Type 3
Side depth	+	±	-
Experience	-	+	+
Small discharges	-	+	+
Maintenance	-	+	+

Table B2: Comparison 3 types of cross sections.

Choice.
It appears that for type 1 the kerbs (and thus the investment costs) will be lowest. On the other hand, because of the situation after small rain events and for reasons of maintenance and especially experience type 2 would be preferable. Together with the authorities concerned[1] a cross section of type 2 has been chosen for the greater part of the primary system. To minimize the kerbstones, a side slope of 1:100 has been chosen. Type 3 can be used for parts of the system where the waterlevels will be small.

[1] MHUP: Ministry of Housing and Urban Planning.

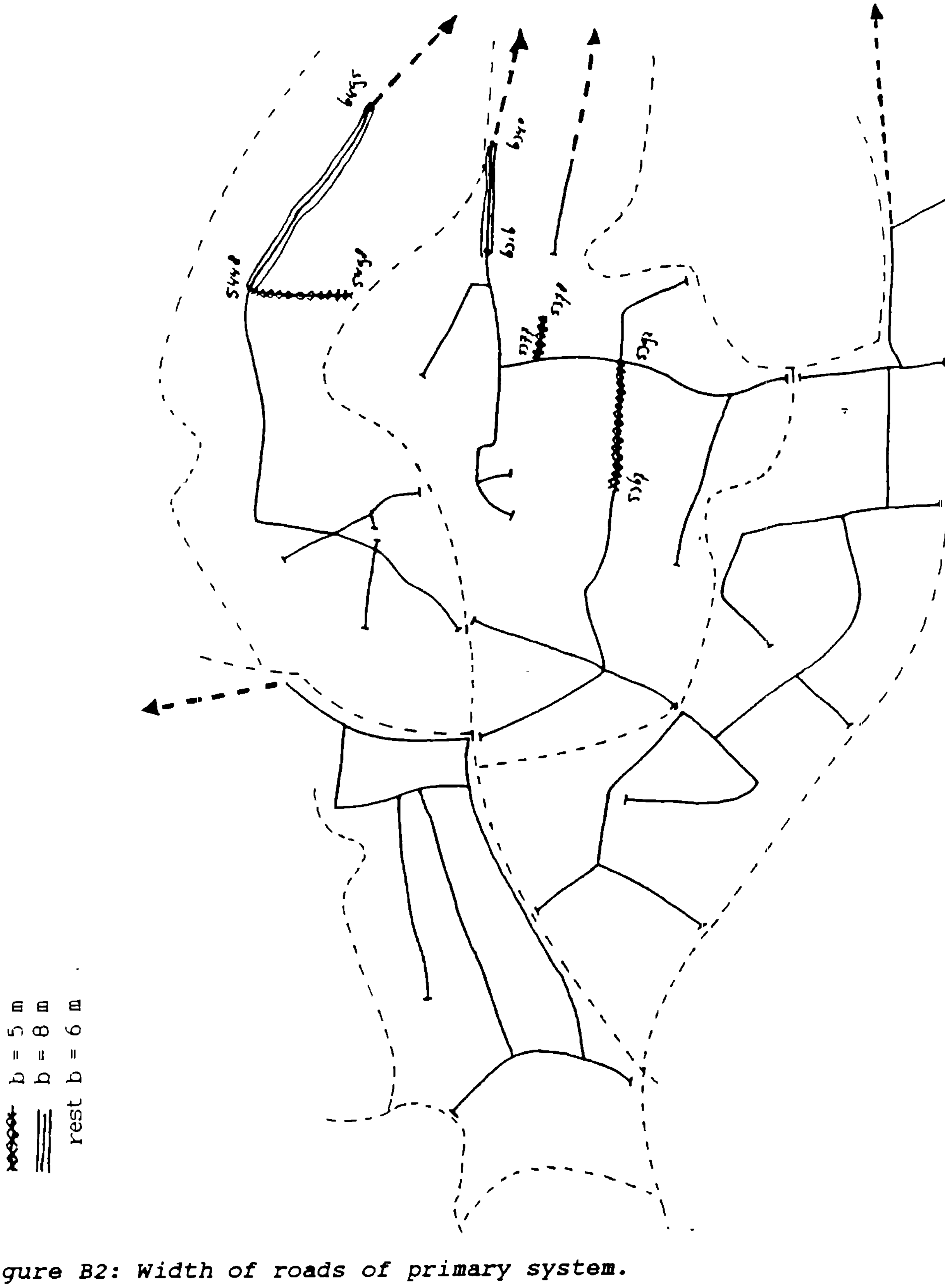


Figure B2: Width of roads of primary system.

Annex C:

RAINFALL

CONTENTS.

C.1 AVAILABLE INFORMATION.

In the surroundings of Rada the total daily rainfall has been measured, for 4 to 13 years, 10 stations, by the RIRDP (Rada Integrated Rural Development Project). See figure C1. These day totals are measured with mechanical recorders and standard rain gauges. However, it is not clear when which equipment was used. Also electronic recorders were used, but no information of these is available yet. See chapter C8, for a more elaborate description of the rain gauges.

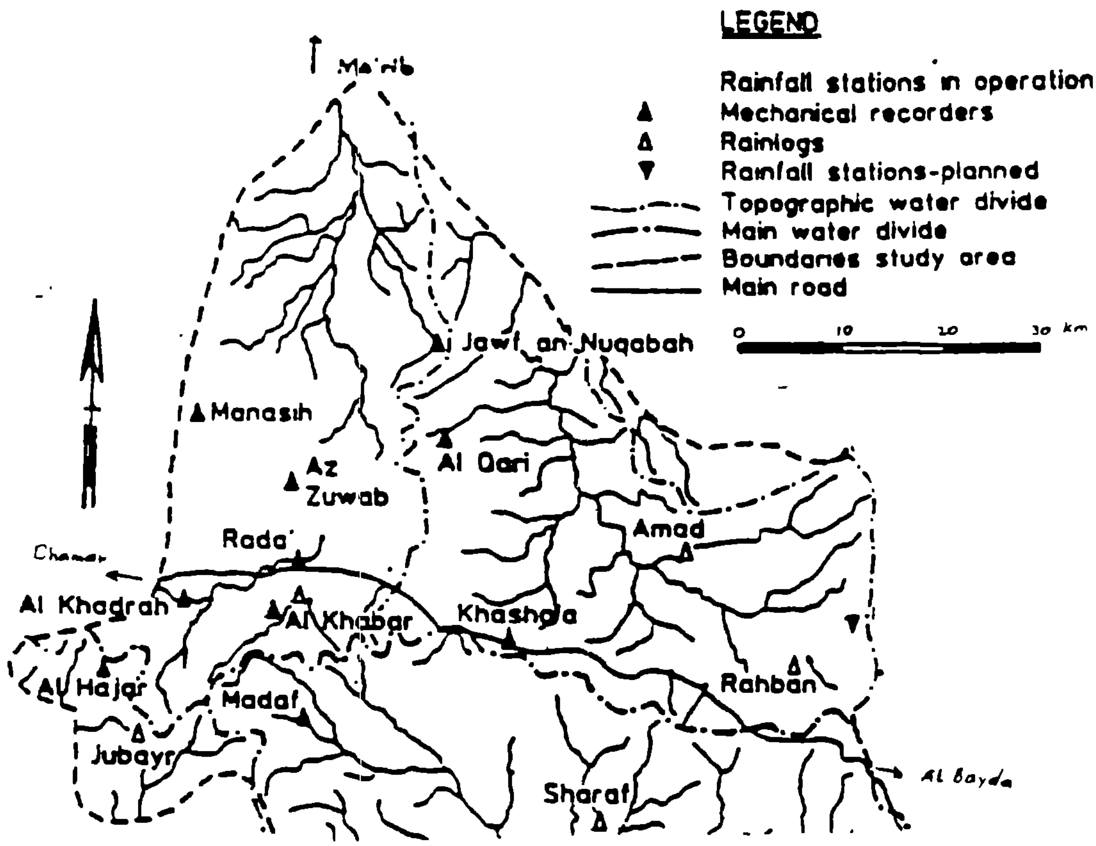


Figure C1: Location rainfall stations.

The average yearly rainfall totals are given in table C1. The stations Al-Khabar and Al-Khadra have an average about equal to that of Rada. These stations are located not far from Rada: Al-Khabar is only 4 km from Rada and Al-Khadra 11 km. The other stations have a considerately smaller year total. Therefore only the pluviographs from those 3 stations were collected. On pluviographs the mechanical recorders register the rainfall continuously. From these the rainfall can be read in 30 minutes. See figure C2 for an example of a pluviograph.

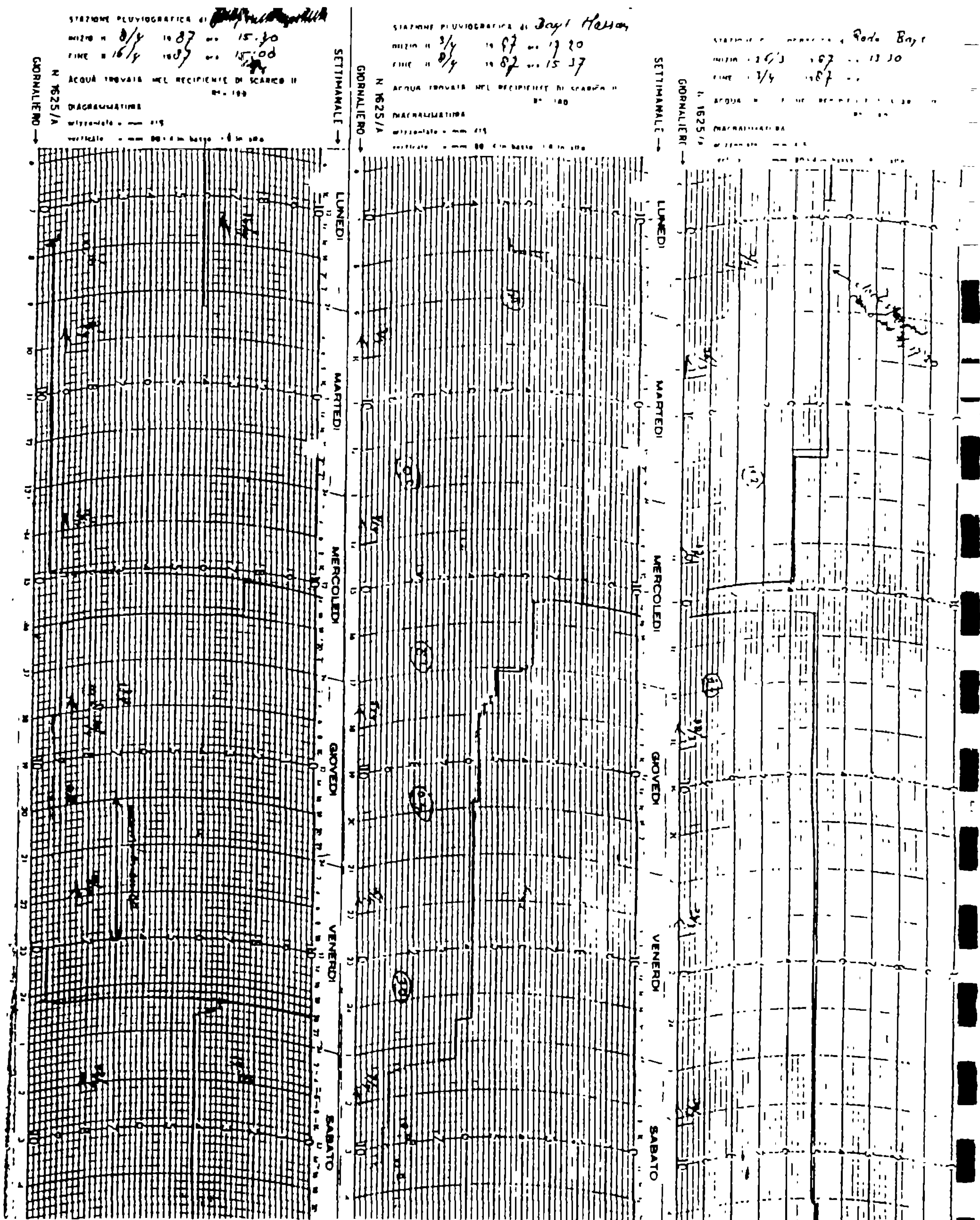


Figure C2: Example pluviographs.

Station	Average rainfall [mm/year]	Number of years
Rada	204	13
Al-Khabar	201	12
Al-Khadra	230	9
Al-Qarim	108	9
Al-Hajar	162	12
Khasha'a	113	9
Al-Madaf	128	9
Az-Zuwab	125	9
Jauf An-Nugabah	104	5
Manasih	118	4

Table C1: Average yearly rainfall.

Unfortunately only for a few years pluviographs are available:

Rada:	1986 from July	
	1987	
	1988	
	1989 until July	together 3 years.
Al-Khabar	1985 from July	
	1986 until May	
	1987	
	1988	
	1989	
	1990 until September	together 4.5 years.
Al-Khadra	1984 from September	
	1985	
	1986	
	1987	
	1988	
	1989	
	1990 until August	together 6 years.

Another drawback is that from the pluviographs of Al-Khadra some heavy storms have not been recorded properly.

C.2 METHOD OF COMPUTING DESIGN STORM.

As is explained in sub-chapter 5.2 of the main report, both the pluviographs and the daily rainfall figures are used to compute the design intensities and finally create the design storm.
The maximum total daily rainfall for each year has to be converted into rainfall in short periods. In the following chapter first a relation between daily rainfall and rainfall in short periods is derived from the pluviographs. With this relation the maximum total daily rainfall figures can be converted into rainfall in short periods.
In the subsequent chapter it is computed how much rain falls in those short periods with a certain return period: the Gumbell extreme values. Through the extreme values for a return period of 2 years a line according to the following (intensity-duration) equation (see sub-chapter C.4.3) is fit:

$$I_a = \frac{a}{(b+t_d)^n}$$

with I_a = average intensity in mm/hour
t_d = duration of rainfall in hours
a, b and c are constants.

Now the design intensities can be computed by filling in a duration in this equation.
In Chapter C.6 the design storm is created out of these design intensities.

C.3 CONVERSION OF DAILY MAXIMA.

The maximum daily rainfalls, ranging between 20 and 66 mm/day, have to be converted into rainfall in short periods. Hence, from the pluviographs only the heavy rainfalls are interesting for deriving the relation. For Rada only the storms with a daily total of more than 10 mm are used. This yields 15 showers or points. See figure C4. In table CC1 (at the end of annex C) rainfall read from the pluviographs is given. Of these figures the maxima for each rain event[1] are given in table CC2.

C.3.1 Pluviographs used.

The rainfall from the pluviographs of the station Al-Khabar and Al-Khadra are not used. Firstly, of Al-Khabar the rainfalls of the years, of which information of Rada is available are not used. This is because Rada and Al-Khabar are only 4 km apart, and there is a big chance that one storm is measured in both stations, so it would be used twice. In that case the data are no more statistically independent and not useful for the regression.
In 1990 the mechanical recorder of Al-Khabar worked, while no information is available of Rada. However, this only gives one storm of 21 mm, and because of possible differences between the stations (which is hard to detect with so little information), this storm is not used either.

Al-Khadra is 11 km from Rada. The rain in Al-Khadra appears to be less violent than in Rada and therefore does not give much extra information about the relation between rainfall in short periods and day totals for heavy daily rainfalls. Because, once again, it is not sure how to convert the data to rainfall in Rada, it has been decided not to use the pluviographs of Al-Khadra.

Consequently, only the pluviographs of Rada are used to find the relation.

The relation between rainfall in a short period and day totals, are the result of rain events of 10 to 45 mm/day. The maximum daily rainfalls are all in the range of 20 mm/day to 66 mm/day. If, however, only the rainfall bigger than 20 mm/day is used to find these relations, just 5 of the 15 points remain. There are 10 storms bigger than 14 mm/day registered in Rada. The results using only those 10 points are almost the same as the results computed with the 15 points.

(1) A rain event has ended when it is dry for at least 6 hours.

C.3.2 Maximum daily rainfall used.

The maximum daily rainfall figures of Al-Khabar of periods in which in Rada was measured too, are not used, because, as explained before, there is a chance that the data are not statistically independent. Only for 1990 data of Al-Khabar are available and not of Rada. In table C2 the measured rainfall is given.

	Rada [mm/day]	Al-Khabar [mm/day]	Al-Khadra [mm/day]
1977	20.5		
1978	52.5		
1979	39.4	38.0	
1980	34.4	24.4	
1981	24.5	24.3	38.8
1982	65.8	40.4	61.9
1983	40.7	35.2	18.0
1984	50.9	27.4	40.0
1985	40.8	60.8	23.8
1986	27.4	31.5	22.6
1987	45.2	36.0	27.2
1988	58.0	47.0	29.0
1989	50.6		15.4
1990		21.0	

Table C2: Maximum daily rainfall for 3 stations.

The maximum daily rainfall of Rada is on average 1.2 times bigger than those of Al-Khabar, so the maximum of Al-Khabar is multiplied by this factor to obtain a comparable rainfall for Rada.
Rada and Al-Khabar are located only 4 km apart, hence the difference in rainfall is remarkable. This might be caused by a systematical error in the measured data, or by the location of the stations with regard to the surrounding mountains.
For the same reason as explained in subchapter C.3.1 the data of Al-Khadra are not used.

According to the RIRDP (for which the rainfall was measured), the registered daily rainfall figures are 10 to 15% to small. This is due to wind errors. Some comment on this is given in chapter C.9. Therefore all day totals will be increased with 10%. In table C2 the rainfall figures have not yet been increased. In table C3 in the last column for Rada the daily rainfall increased with 10% is given.

C.3.3 Conversion maximum daily rainfall to rainfall in 24-hours.

In 24 hours more rain can fall than what is called the daily rainfall. The meteorological day for the measurement of rainfall is from 6 o'clock in the morning, until the next morning 6 o'clock. It is however possible that if the daily rainfall is measured during another 24 hours, more rain is falls in those 24 hours. For Yemen no information about the factor 24-hour rainfall divided through daily rainfall is available.
Because the rainfall is mainly caused by convective storms[2] and because according to figure C3 a factor 1.0 seems more likely than a factor 1.1, the 24-hours rainfall is accepted to be equal to the day total.

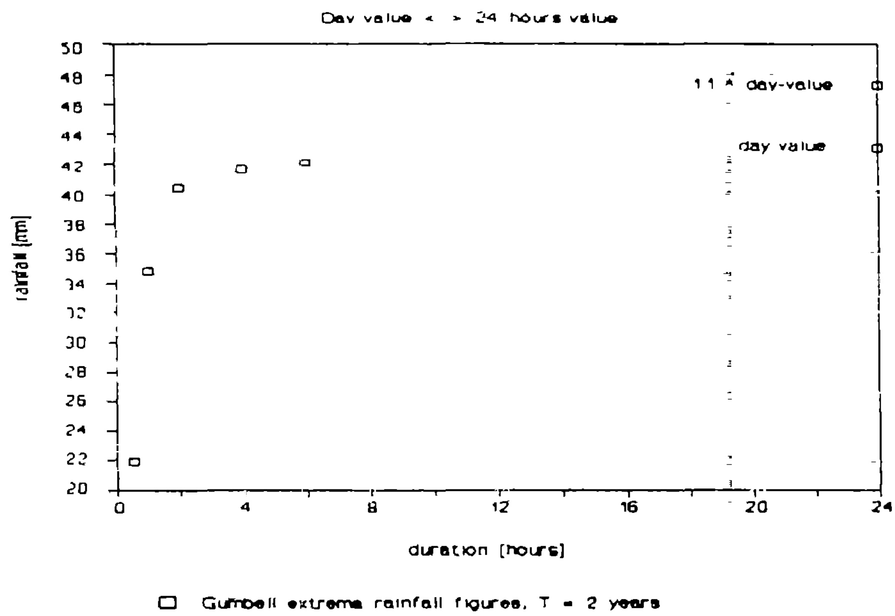


*Figure C3: Gumbell extrema 1.1*day-value and day value*

C.3.4 Regression.

If the rainfall in short periods is plotted against the daily total, it appears, that the relation between these figures can, with reasonable reliability, be described with the simplest relation: a straight line. Thus of the type y = **a*x + b**. However, (for reasons explained later) a line of type **y = a*x** is chosen. In these equations the daily rainfall is given by the variable **x** and the rainfall in a short period by **y**.

---

(2) Convective storms mostly occur in the afternoon, so within the meteorological day.

The straight line **y = a*x** is drawn through the points in such a way, that the sum of the squares of the distance (in y-direction) between the points and the line is smallest. The constant **a** is computed with:

$$a = \frac{\sum_{i=1}^{N} (x*y)}{\sum_{i=1}^{N} x^2}$$

Now for a given x-value, the expected y-value can be computed.
An indication of the correctness of the relation and estimation is given by the correlation coefficient. For linear regression this coefficient will be equal to 1 or -1 if all the points are on one straight line. Because a solution according to **y = a*x** is used, the correlation coefficient is computed with:

$$r = \sqrt{\frac{(\sum_{i=1}^{N} y*y') - N*\bar{y}^2}{(\sum_{i=1}^{N} y^2) - N*\bar{y}^2}}$$

In which y' is the expected y-value computed with **y = a*x** and $\bar{y}$ is the average y-value.

The results are:

$R_{1/2\ hour} = 0.51 * R_{day}$	corr. $r^2 = 0.84$
$R_{1\ hour} = 0.81 * R_{day}$	corr. $r^2 = 0.94$
$R_{2\ hours} = 0.94 * R_{day}$	corr. $r^2 = 0.96$
$R_{4\ hours} = 0.97 * R_{day}$	corr. $r^2 = 0.97$
$R_{6\ hours} = 0.98 * R_{day}$	corr. $r^2 = 0.98$

The correlation is reasonable for 1/2 hour. For the other periods the correlation is good. In figure C4 the rainfall in 1/2, 1 and 6 hours is plotted against the daily rainfall. It is clear that the majority of the rain falls in the first hours.
With these results the day maxima can be converted into rainfall in short periods. In table C3 the maximum daily rainfall of 14 years is converted into rainfall in short periods.

rainfall in [mm]						measured
[hours]	0.5	1	2	4	6	1 day
1977	11.5	18.3	21.2	21.9	22.1	22.6
1978	29.5	46.8	54.3	56.0	56.6	57.8
1979	22.1	35.1	40.7	42.0	42.5	43.3
1980	19.3	30.7	35.6	36.7	37.1	37.8
1981	13.7	21.8	25.3	26.1	26.4	27.0
1982	36.9	58.6	68.0	70.2	70.9	72.4
1983	22.8	36.3	42.1	43.4	43.9	44.8
1984	28.6	45.4	52.6	54.3	54.9	56.0
1985	22.9	36.4	42.2	43.5	44.0	44.9
1986	15.4	24.4	28.3	29.2	29.5	30.1
1987	25.4	40.3	46.7	48.2	48.7	49.7
1988	32.5	51.7	60.0	61.9	62.5	63.8
1989	28.4	45.1	52.3	54.0	54.5	55.7
1990	13.0	50.6	23.9	24.6	24.9	25.4

Table C3: Conversion of maximum daily rainfall of Rada (+10%) into rainfall in shorter periods.

The fact that a daily rainfall of 0 mm also means that the rainfall in a short period is 0 mm, might indicate that the relations are of the type **y = a*x**. However, because for the day totals a threshold of 10 mm is chosen, this does not necessarily have to be this way. On the other hand, if a relation of the type **y = a*x + b** is used, the results become physically impossible. The relation for 2 hours would be:

$$R_{2\ hours} = 1.09 * R_{day} - 4.5$$

For a daily rainfall more than 50 mm, the result would be that in 2 hours more rain falls than the daily total. It should be borne in mind that specially heavy rainfalls (up to 65 mm a day) are interesting. Therefore **b = 0** is chosen.

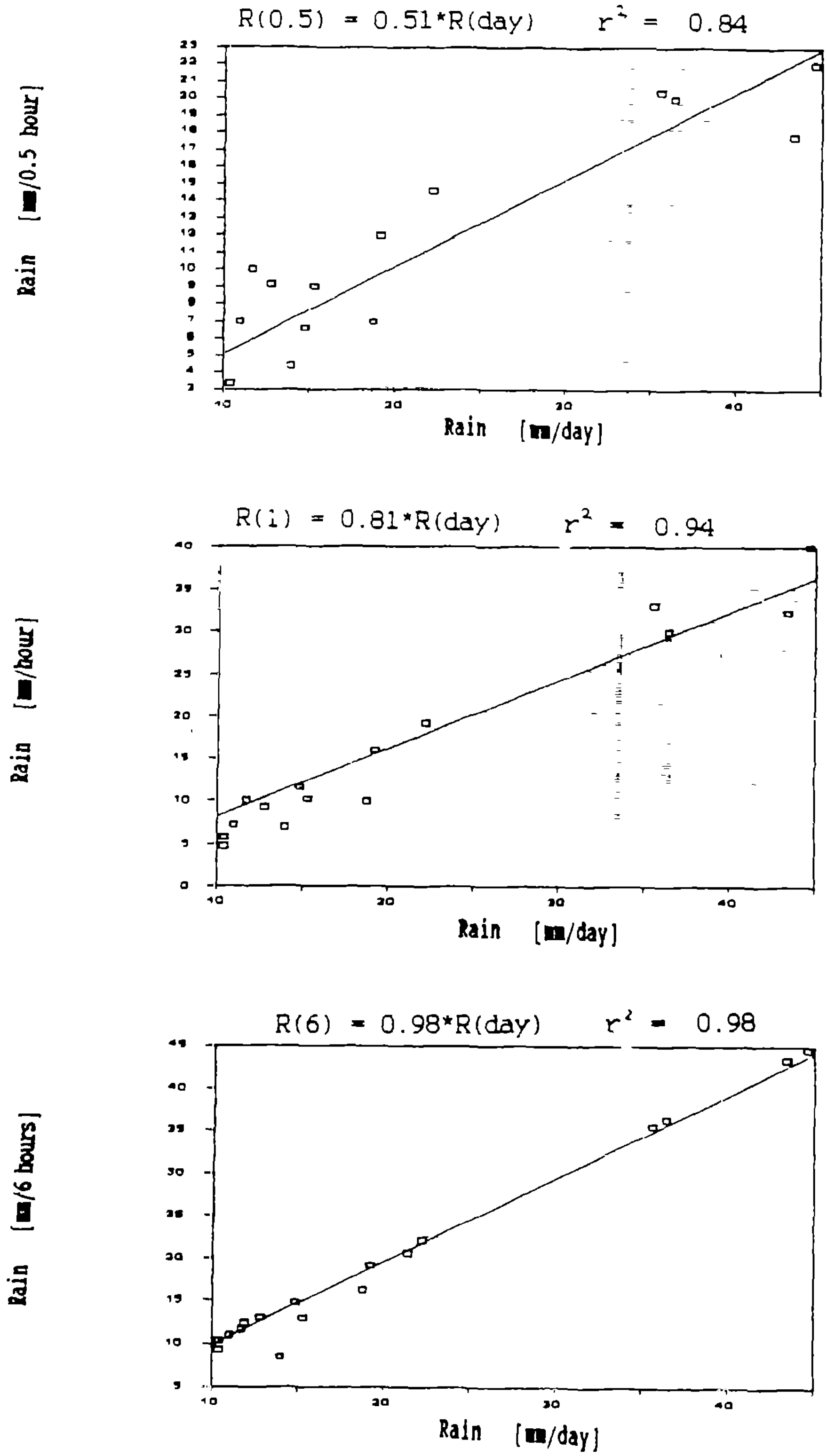


Figure C4: Rainfall in short periods against day total for Rada, for periods of 0.5, 1 and 6 hours.

C.4 DESIGN RAINFALL INTENSITIES.

First the extreme values are computed, from which the design intensities are derived.

The extreme values are the quantities of rainfall, that are equalled or exceeded with an average of once every T year, with T is the return period. A set of measured data usually has got a normal distribution, as the dotted line in figure C5: The average value occurs most and the chance is equal that a value is a certain amount smaller or larger than the average (a symmetrical distribution). Hydraulic extreme values, however, have an asymmetrical distribution. A frequently used method to compute the extreme values is Gumbells method.

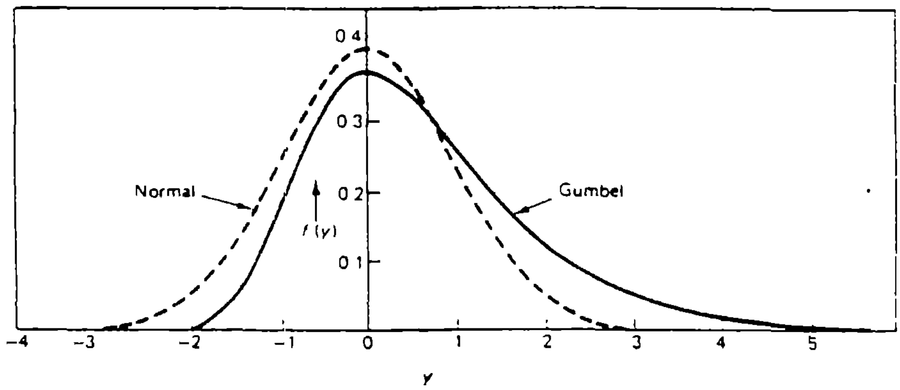


Figure C5: Normal and Gumbell density functions.

C.4.1 Gumbell distribution.

Gumbell defines a distribution of the extreme values **x** with the following formula:

$$F(y) = \exp(-e^{-y})$$

And thus a density function (see figure C5):

$$f(y) = \frac{dF(y)}{dy} = e^{-y}\exp(-e^{-y})$$

In these equations **y** is the reduced (and therefore dimensionless) variable of the extreme rainfall **x**, calculated as **y = α(x - u)**. The parameters **α** and **u** make that the density function of **x** coincides with that of **y**. See figure C6.

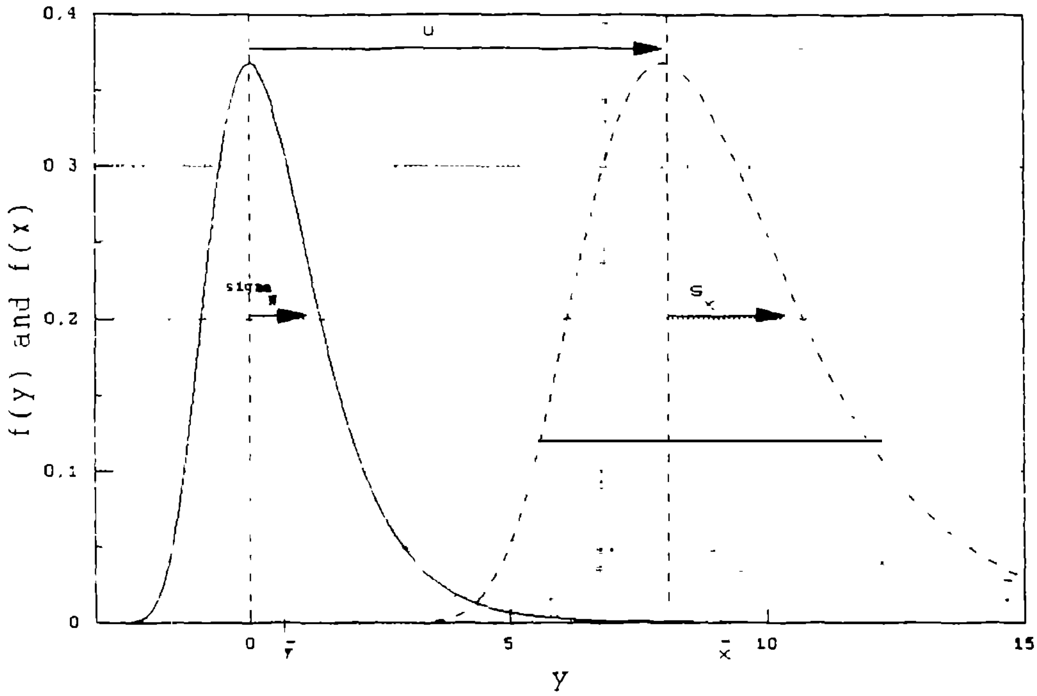


Figure C6: Density functions for y and arbitrary x.

If s_x and σ_N are the standard deviations of **x** and **y** respectively, then the parameter α is equal to s_x/σ_N. This parameter makes the sizes of the both curves equal. It can be seen that

$$y-\bar{y} = \frac{x-\bar{x}}{\alpha}$$

In this equation $\bar{x}$ and $\bar{y}$ are the average values of **x** and **y**. Consequently:

$$y = \alpha(x-\bar{x})+\bar{y} = \alpha x-\alpha(\bar{x}-\frac{\bar{y}}{\alpha}) = \alpha(x-u)$$

The parameter **u** is equal to $\bar{x} - \bar{y}$ ***s/σ_N** and represents the mode of the x-distribution (point of maximum density).

The total area below the density function equals 1:

$$\int_{-\infty}^{\infty} f(y) = F(y)\Big|_{-\infty}^{\infty} = \exp(-e^{-y})\Big|_{-\infty}^{\infty} = 1$$

The value of F for a certain y-value is the area left of the y-value and below the density function. See figure C7. What is left is the chance **P** that a certain value of **y** (extreme rainfall) is equalled or exceeded:

P = 1- F(y) = 1-exp(-exp(-y)).

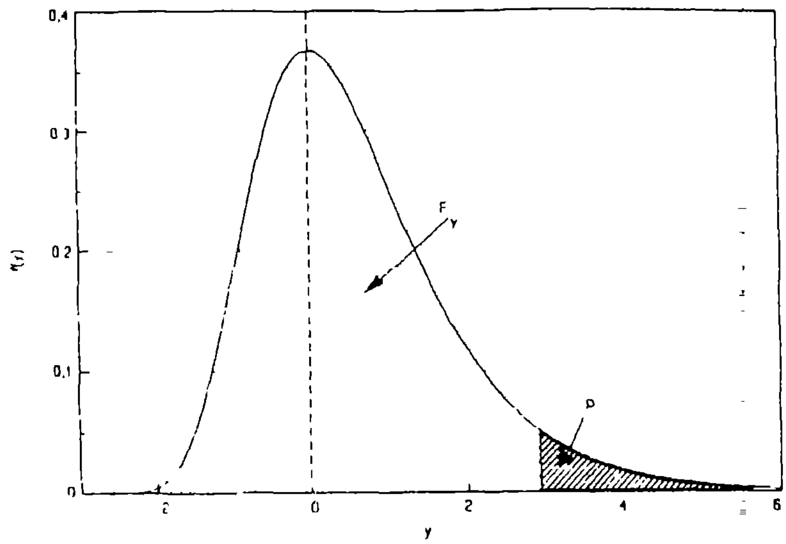


Figure C7: Gumbell Density function.

The return period is the reciproke of this chance (if the chance that a rainfall is equalled or exceeded is 0.1/year, then the return period is 10 years). This gives:

$$T = \frac{1}{1-\exp(-e^{-y})}$$

The reduced value for a particular return period T can be found with:

$$y = -\ln(\ln(T) - \ln(T-1))$$

For every year the maximum daily rainfall is taken. These maxima are ranked from big to small and numbered. According to Weibull, the return periods can be computed with:

$$T = \frac{N+1}{m}$$

In this formula N = number of values and
m = rank of value.
Thus the biggest maximum is on average equalled or exceeded once every N+1 years and the middle maximum once in 2 years.

The reduced variable y is given as function of T. The extreme rainfall can be computed from y with **x = y/α + u**. The values of the constants **α** and **u** can be obtained graphically or numerically.

Numerically:
With a computer it is easy to compute **α** and **u** numerically. In table C4 the maximum rainfall figures have been ranked. Also the return period (computed with the equation of

Weibull) and y are given. The values of α and u can be computed from the standard deviations and the average x- and y-values:

$$\alpha = \sigma_N/s$$
$$u = \bar{x} - \bar{y} * s/\sigma_N$$

After that the extreme values can be computed for return periods of 2 and 5 years.

Weibull return period			x = rainfall in [mm]					
n	T	y	0.5	1	2	4	6	1 day
1	15.0	2.67	36.9	58.6	68.0	70.2	70.9	72.4
2	7.5	1.94	32.5	51.7	60.0	61.9	62.5	63.8
3	5.0	1.50	29.5	46.8	54.3	56.0	56.6	57.8
4	3.8	1.17	28.6	45.4	52.6	54.3	54.9	56.0
5	3.0	0.90	28.4	45.1	52.3	54.0	54.5	55.7
6	2.5	0.67	25.4	40.3	46.7	48.2	48.7	49.7
7	2.1	0.46	22.9	36.4	42.2	43.5	44.0	44.9
8	1.9	0.27	22.8	36.3	42.1	43.4	43.9	44.8
9	1.7	0.09	22.1	35.1	40.7	42.0	42.5	43.3
10	1.5	-0.09	19.3	30.7	35.6	36.7	37.1	37.8
11	1.4	-0.28	15.4	24.4	28.3	29.2	29.5	30.1
12	1.3	-0.48	13.7	21.8	25.3	26.1	26.4	27.0
13	1.2	-0.70	13.0	50.6	23.9	24.6	24.9	25.4
14	1.1	-1.00	11.5	18.3	21.2	21.9	22.1	22.6
	avg	0.51	23.0	36.5	42.4	43.7	44.2	45.1
	std	1.01	7.5	11.9	13.8	14.2	14.4	14.7
	α		0.135	0.085	0.073	0.071	0.070	0.069
	u		19.2	30.5	35.4	36.5	36.9	37.7
return period T [year]			Gumbell extreme values expected rainfall in [mm]					
	2		21.9	34.8	40.4	41.7	42.1	43.0
	5		30.3	48.2	55.9	57.8	58.3	59.5

Table C4: Conversion of maximum daily rainfall and extreme values.

Graphically.
The maximum rainfall figures and the return periods (see table C4) are plotted on Gumbell paper. Gumbell paper has got a linear vertical axis on which the rainfalls are

plotted. On the horizontal axis the return periods are plotted on a double logarithmic scale (or the y-values are plotted on a linear scale, which is the same).
If the extreme rainfalls have a Gumbell distribution, all points should be on a straight line. The equation of this line is:

$$x(T) = u + \frac{y(T)}{\alpha} = u - \frac{\ln(\ln(T) - \ln(T-1))}{\alpha}$$

The parameter **u** can be found from the Gumbell plot as the x-value for y = 0 (T = e/(e-1) and α is given by the angle between the horizontal axis and the line.

The numerical method is faster and more accurate, but a Gumbell plot is still a good control measure because it shows whether the points are on a straight line.

In figure C8 and C9 the measured maxima and the Gumbell line are plotted for rain-durations of 0.5 and 24 hours.

C.4.2 Confidence interval extreme values.

The expected value of x ($\bar{x}$) is computed with:

$$x'(T) = \frac{y(T)}{\alpha} + u = \bar{x} - s * \frac{\bar{y}_N - y(T)}{\sigma_N}$$

From the diversion of the points round the line x'(T), an interval can be computed, in which 95% of the predicted points lie. A measure for the spread of the x-values round the computed x as a function of T is given by the variance of x.

$$Var(x'(T)) = \frac{s^2}{N}[(1+1.14(\frac{y_T - \bar{y}_N}{\sigma_N}) + (\frac{y_T - \bar{y}_N}{\sigma_N})^2(0.6 + \frac{0.5N}{N-1})]$$

Assume that the distribution of the points round the line is a t-distribution. A t-distribution is, like the normal distribution, a symmetrical distribution, but depends on the degrees of freedom. In this case the degrees of freedom is 12 (2 of the 14 are used to compute α and **u**). So for a interval of 95% (so 2.5% not covered on both each side of the line) the factor of the t-distribution is 2.179.

The validity of the t-distribution has not been checked and this distribution is probably not correct. However, the exact values of the confidence are not interesting. The confidence interval is computed to give an indication of the size of it.

The confidence interval can now be computed with:

$$x = x'(T) \pm t_{(\frac{1}{2}\alpha)} * \sqrt{Var(x'(T))}$$

In table CC3 for T = 2 and 5 years the borders of the confidence intervals are given. For a duration of 0.5 hour and 24 hours the confidence intervals are plotted in figure C8 and C9. The reliability for long return periods is not large. Even for T = 5 years the interval is already rather wide.

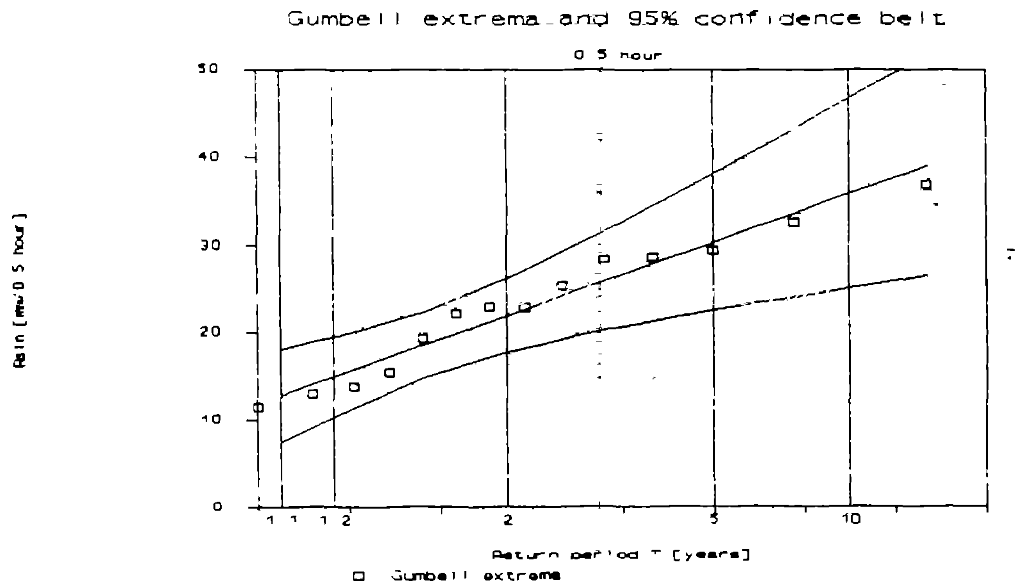


Figure C8: Gumbell plot wit confidence interval for 0.5 hour.

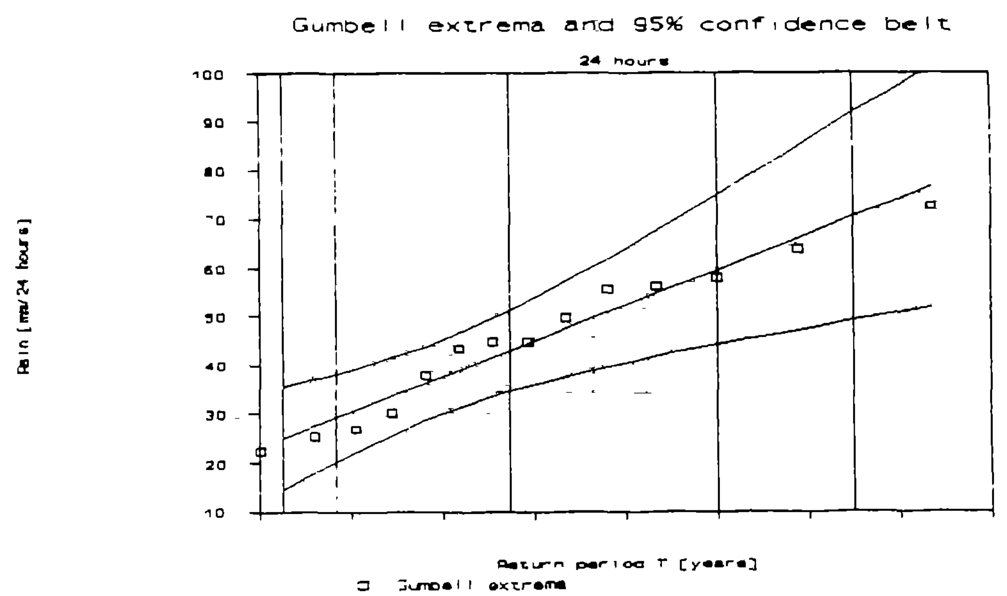


Figure C9: Gumbell plot wit confidence interval for 24 hours.

C.4.3 Intensity-duration equation.

In figure C10 an arbitrary rainfall which is expected to be equalled or exceeded with a certain return period (for example T = 2 years) is plotted against the duration of the rainfall.

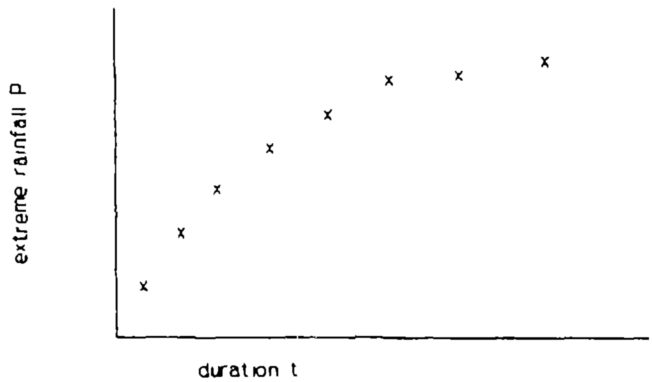


Figure C10: Extreme rainfall-duration plot.

For a duration t = 0, the rainfall P will be zero. The gradient of the line gets milder for longer durations, which is logical, since the extreme rainfall in 10 hours is only slightly larger than the rainfall in 9 hours.
The average intensity I_a for rainfall P_1 and duration t_1 is given by the tangents of the angle ß in figure C11. $I_a = \tan(\beta) = P_1/t_1$.

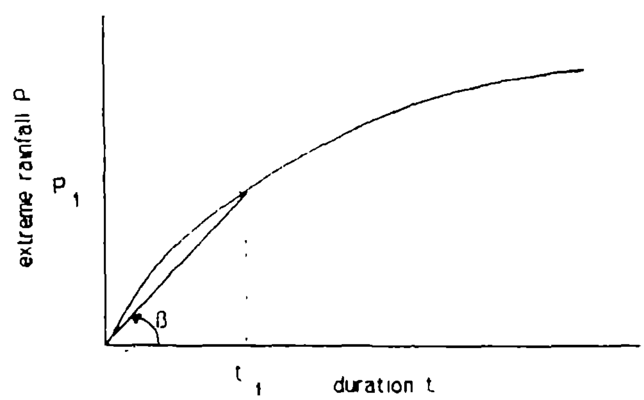


Figure C11: Average rainfall intensity.

For durations up to 24 hours, a commonly used equation for the average intensity is:

$$I_a = \frac{a}{(b+t_d)^n}$$

with I_a = average intensity in mm/hour
t_d = duration of rainfall in hours
a, b and c are constants.

The extreme rainfall P then is:

$$P = I_a t = \frac{at}{(b+t)^n}$$

In the following the values of n and b will be discussed.

The constant b.
The intensity for very small durations will be very high. The question is whether I_a is infinitive for t = 0, or that a certain start-value I_0 exists. The rainfall intensity for durations going to zero ($t \downarrow 0$) is given by:

$$I_{a\ for\ t\downarrow 0} = \frac{a}{b^n}$$

So if b = 0 then I_0 will be infinitive. Physically, the rainfall intensity can not become infinitive, because for very high intensities the rainfall is hindered by the air which has to move upwards. Therefore a value for I_0 exists and b should be greater then 0. Maybe it is possible to measure or compute this value.
In this case for b the value is taken, which gives the best fit of the intensity-duration equation through the extreme rainfall points.

The constant n.
Physically the expected rain P should increase for increasing durations. This means that dp/dt>0 for t>0.

$$\frac{dP}{dt} = \frac{a}{(b+t)^n}\left(1-\frac{nt}{b+t}\right)$$

dp/dt equals 0 if

$$1-\frac{nt}{b+t}=0 \qquad thus \quad t=\frac{b}{n-1}$$

So, if the value of n is between 0 and 1, then for all durations dp/dt will be positive. But, if n > 1, then for t = b/(n-1) there will be a maximum, and for t >1 dp/dt will be negative. This is physically impossible, because that would mean that the maximum rain becomes larger if the duration t gets longer.

So b should be positive and have a certain value (depending on the maximum intensity) and n should be between 0 and 1. Nevertheless for the calculation of the rainfall intensities of Rada, b and n will just be chosen so that the fit of the intensity-duration equation is best. As the smallest durations for which the intensity will be computed is 15 minutes, the intensities for very short durations are not important. Consequently, the value of b is physically not important. The value of n can become higher than 1 as long as the durations for which the equation is used is smaller than b/(n-1).

C.4.4 Rainfall intensities.

So the average rainfall intensities can be described with:

$$I_a = \frac{a}{(b+t_d)^n}$$

For each return period the constants a, b and n have to be computed. To make the equation linear it can be rewritten as:

$$\log(I) = \log(a) - n*\log(b + t_d),$$

If a value for b is chosen, the values of a and n can be computed with linear regression. The linear regression results in an equation **y = Ax +B**, with **A** and **B** are constants and **y** and **x** are the variables. In this case:

$$\begin{aligned} \log(I) &= y \\ \log(b + t_d) &= x \\ -n &= A \\ \log(a) &= B \end{aligned}$$

For each value of b there is a best combination of **A** and **B**, thus for **a** and **n**.

$$A = \frac{\sum_{i=1}^{N} (x_i*y_i) - N\overline{xy}}{\sum_{i=1}^{N} x_i^2 - N\overline{x}^2}$$

$$B = \overline{y} - a\overline{x}$$

The equation for A is not the same as in sub-chapter C.3.4, because there the second constant B was accepted to be 0. The correlation coefficient r gives is a measure for the correctness of the fit of the straight line through the point **x** and **y**. With no set value for B, the correlation coefficient is computed with:

$$r = \frac{\sum_{i=1}^{N} x_i * y_i - N*\overline{xy}}{\sqrt{(\sum_{i=1}^{N} x_i^2 - N*\overline{x}^2) * (\sum_{i=1}^{N} y_i^2 - N*\overline{y}^2)}}$$

For b = 0.8 the fit through the points of extreme rainfall was best. See figure C12 line 2. However, this line is physically impossible, giving more rainfall in 6 hours than in 24 hours (according to the intensity-duration equation the maximum rainfall would occur at t = b/(n-1) = 8 hours for b = 0.8). For b = 0.5 (line 1 in figure C12) the line is about horizontal for t ≥15 hours, (in fact there is a mximum for t = 17 hours). B has been chosen as the best possible fit for all return periods.

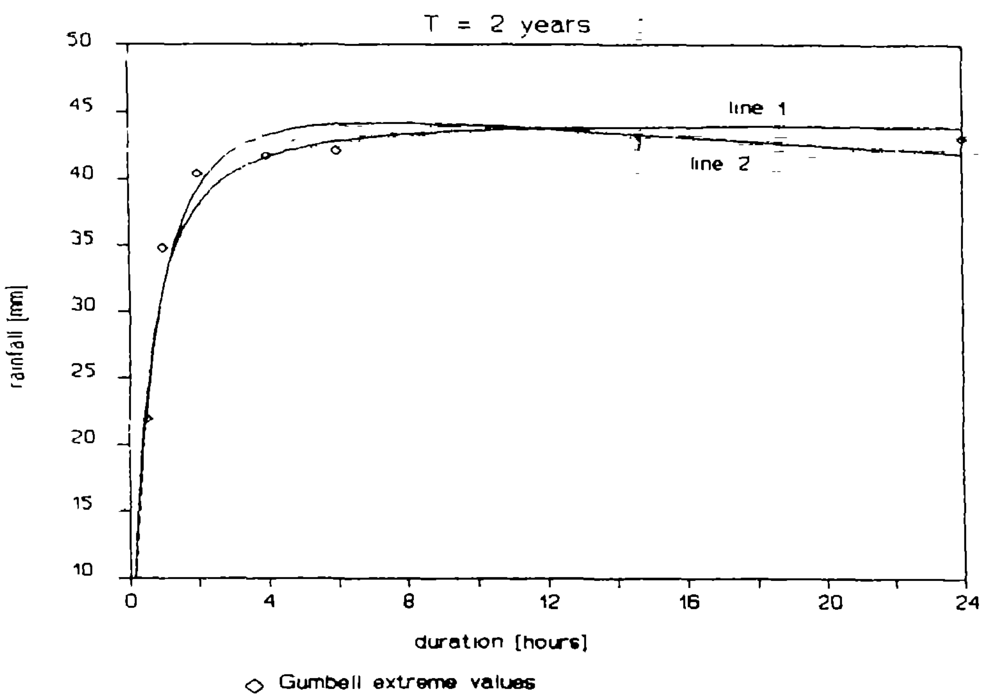


Figure C12: Best fit through extreme values.

b	r^2	a	n	b/(n-1) [hours]
0.5		48.8	1.03	17
0.8				8

Table C5: Value of constant b of intensity-duration curve for T = 2 years.

According to "manual urban drainage," DHV, lit. [7], it is advised for Saudi Arabia and Dubai that b = 0.2 and 0.3 respectively. But Saudi Arabia is a large country and it is not clear if the climatological situation is the same as in Yemen over there. The value for Dubai is found for periods up to 2 hours, while now the function is used for periods up to 24 hours. Consequently it seems right to neglect this advice and take the value giving the best fit.

Line 1 represents the best physical possible fit.
For T = 2 years the intensity duration curve is:

$$I_a(2) = \frac{48.8}{(0.5+t_d)^{1.03}}$$

The value of constant a for T = 5 and 10 years is 67.5 and 79.9 respectively. The constants b and n are equal for all return periods.
The results of the regression is given in figures C13 and C14 and table C6.

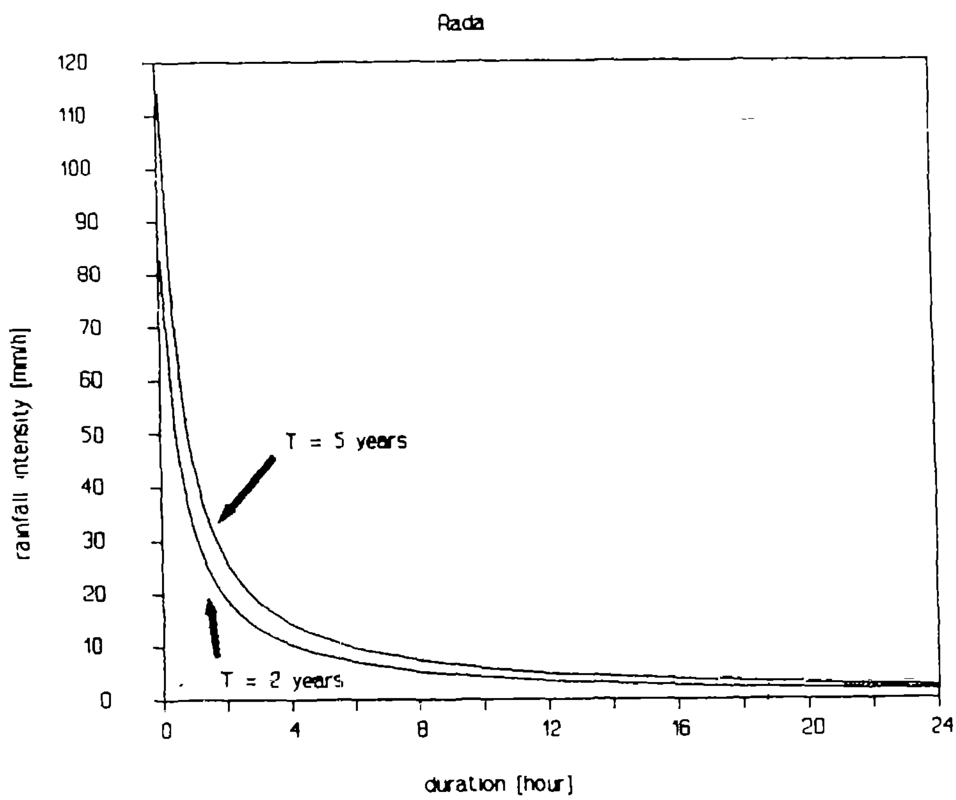


Figure C13: Intensity-duration curve.

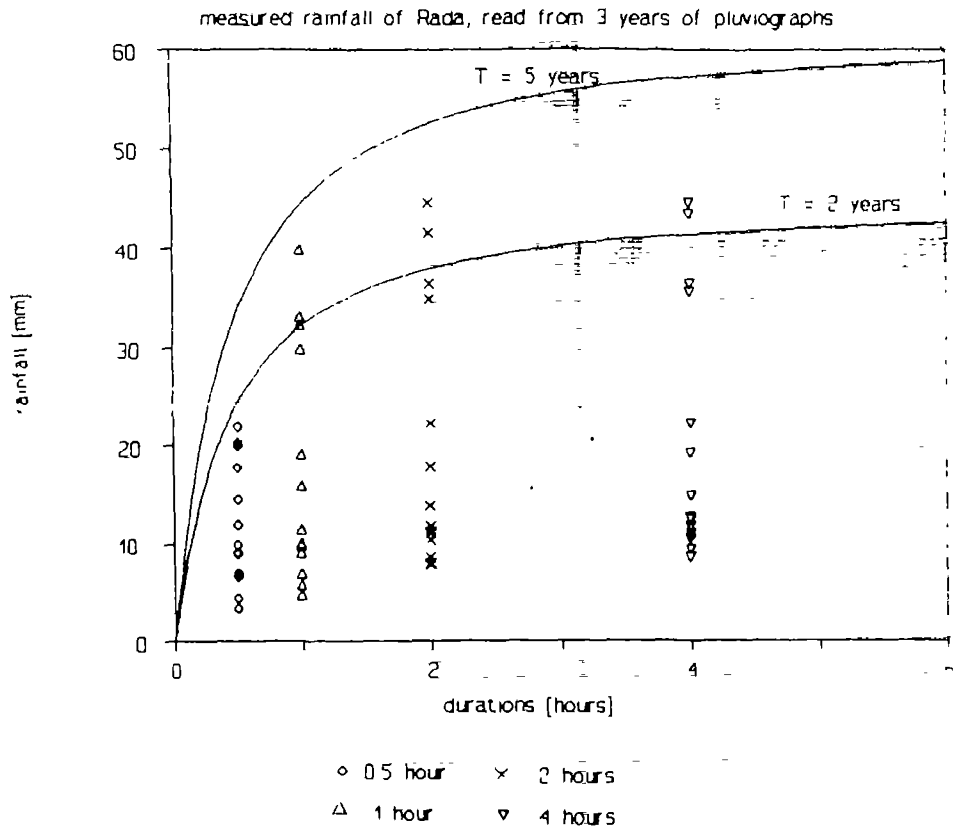


Figure C14: Rainfall-duration curve.

duration [hour]	Return period T			
	2 years		5 years	
	[mm]	[mm/h]	[mm]	[mm/h]
0.25	16.4	65.6	22.7	90.8
0.50	24.4	48.8	33.7	67.5
0.75	29.1	38.8	40.2	53.6
1.0	32.1	32.1	44.5	44.5
2.0	38.0	19.0	52.5	26.3
4.0	41.5	10.4	57.3	14.3
6.0	42.6	7.1	58.9	9.8
24	43.4	1.8	60.1	2.5

Table C6: Resulting rainfall in [mm] and rainfall intensities in [mm/h] for return periods of 2 and 5 years.

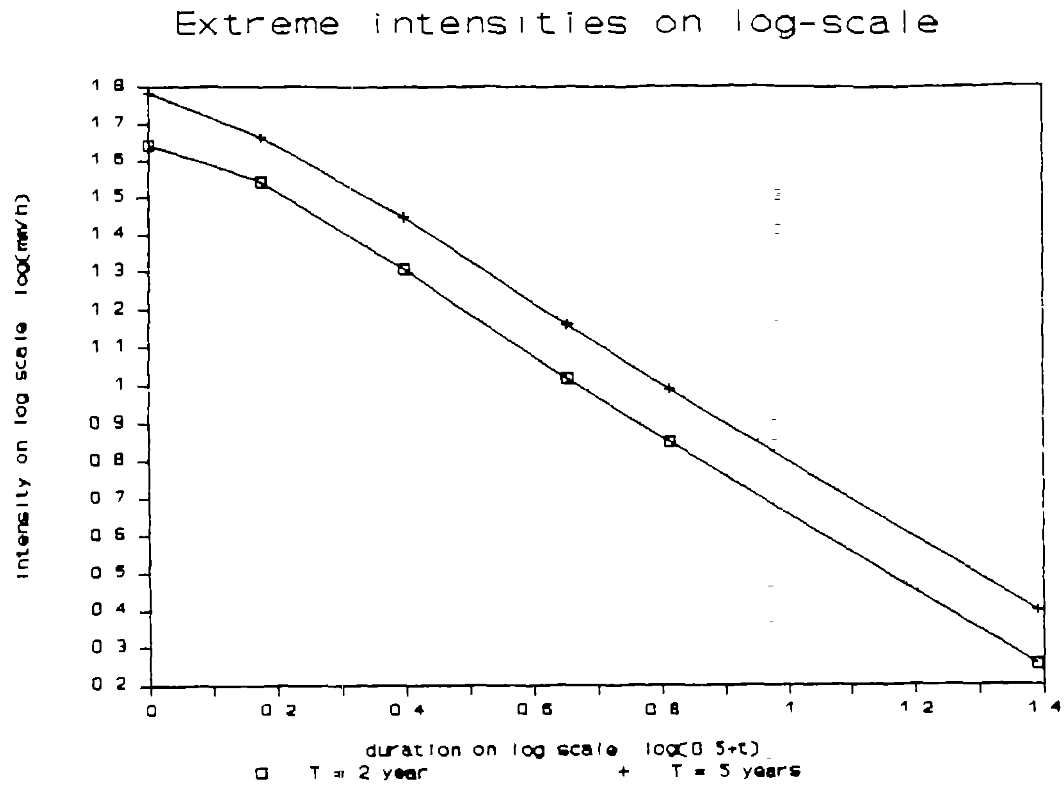


Figure C15: Logarithmic intensity duration curve.

Figure C15 shows that the points are not on one line, but that for t = 0.5 hour the values are smaller than expected. For this duration the correlation between rainfall in this period and the total daily rainfall was worse than for the other periods. Also it was sometimes difficult to read the rainfall in 30 minutes from the pluviographs. Therefore it might be decided to drop the information of rainfall in 30 minutes. However, this does not have a great effect on the resulting rainfall intensities (see table C7). It is decided to use the information of 30 minutes.

duration [hour]	Return period T			
	2 years		5 years	
	[mm]	[mm/h]	[mm]	[mm/h]
0.25	16.6	66.4	23.0	91.9
0.50	24.7	49.4	34.3	68.4
0.75	29.4	39.3	40.7	54.3
1.0	32.5	32.5	45.0	45.0
2.0	38.4	19.2	53.2	26.6
4.0	42.0	10.5	58.1	14.5
6.0	43.1	7.2	59.7	9.9
24	44.0	1.8	60.8	2.5

Table C7: Resulting rainfall and rainfall intensities for return periods of 2 and 5 years, derived without 0.5 hours.

A plot of the extreme rainfall intensities on logarithmic scale does not have to result in a straight line. This is due to the fact that for a small rainfall, the rainfall is the result of a different process in the air, then for a medium rainfall. And a very heavy rainfall is influenced by other factors again. This makes that there should be three parts in a rainfall duration curve. For this case only the middle part will be interesting, which can be accepted to be a straight line.

Out of so few points, a confidence interval for the design rainfall is hard to compute. However, the confidence interval of the extreme values gives an indication. For a return period of 2 years, it has been computed that with a duration of 0.5 hour there is 95% probability, that an extreme rainfall is between the expected value plus or minus 4 mm. It is accepted that the confidence interval will be as wide for the design rainfall. So there is a 95% probability that the design rainfall is between the expected value plus or minus 4 mm: between 20 and 28 mm.

C.4.5 Alternative way of computing design intensities.

Instead of the method using both the pluviographs and the daily rainfall figures, as described before, it might be possible to derive the design rainfall directly from the pluviographs. Since only three years of pluviographs are available, this can merely be a control of the results of the method using both sets of data.

For the extreme value distribution only the maximum rainfall data of each year or period are used. If all rainfall data is regarded, then hydrological data often has an exponentional distribution. The exponentional distribution is as follows:

$$n = b*N*e^{-a\frac{R}{\bar{R}}}$$

with: $\bar{R}$ = average rainfall
R = rainfall
N = number of rainfall data
n = times of occurrence or exceedance.
a and b are constants to fit the exponentional distribution through the real points.

The formula computes how often rainfall R is equalled of exceeded. From the number of times R is equalled or exceeded can be derived what the return period of that rainfall is. Or the other way round: For a certain return period can be computed what rainfall is expected.

It appeared that if all the half hours of rainfall from the pluviographs are regarded, the exponentional distribution doesn't fit very well through the measured points. This is because rainfall smaller than 2 mm behaves different than the heavy rainfall. This rainfall is the result of different meteorological conditions than those prevailing at heavy storms.
However, the rainfall smaller than 3 mm can be omitted because it is accepted that that rainfall does not contribute to the runoff. These small amounts of rainfall will be lost because of infiltration, retention and evaporation. So only rainfall ≥ 3 mm is used. The exponentional distribution now fits well through the measured points.

All rainfall figures (≥3mm) are sorted from low to high and numbered from high to low (to the smallest rainfall belongs a ranking number n = N and for the biggest rainfall n equals 1). If these rainfall figures have an exponentional distribution, then this ranking number is equal to n as computed with the formula above. The measured and computed rainfall figures are plotted in figure C16.

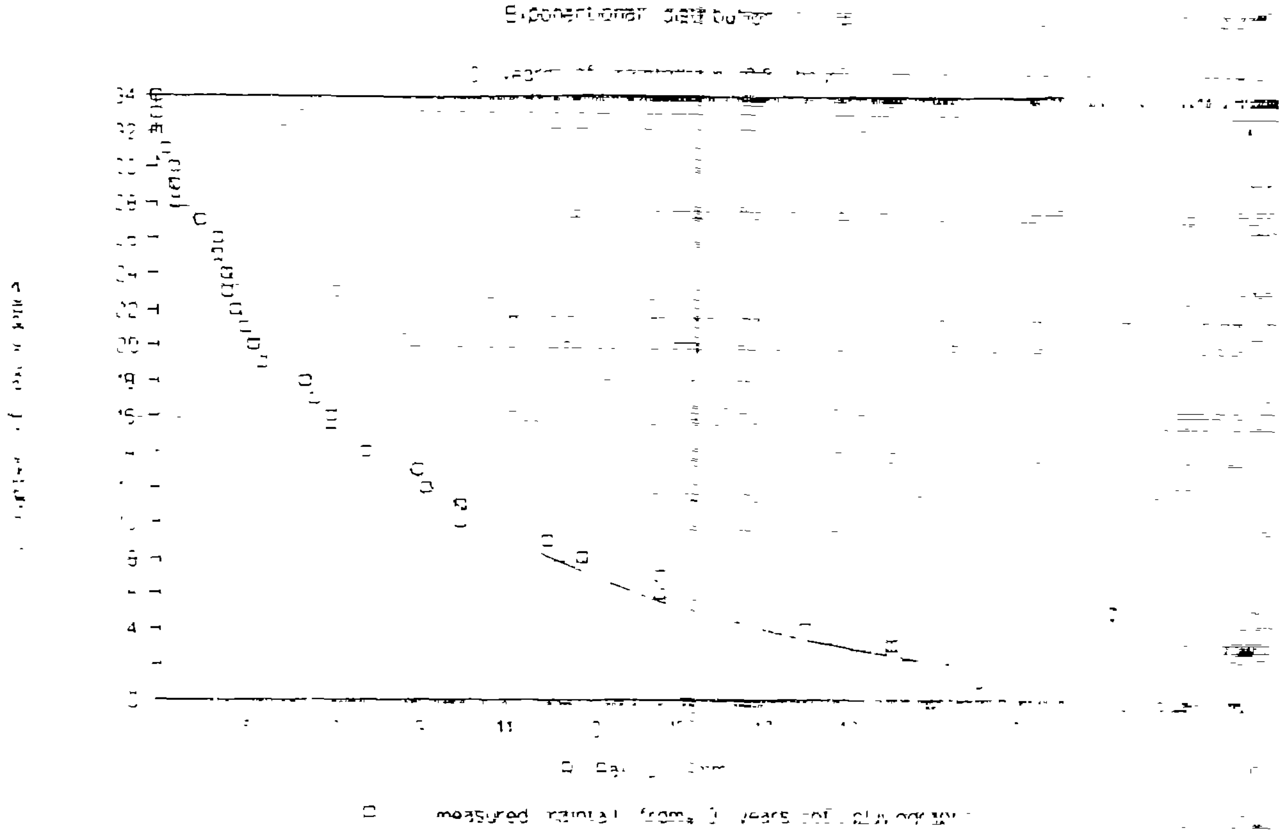


Figure C16: Exponentional distribution.

From the pluviographs of Rada 34 periods of 0.5 hour with rainfall ≥ 3 mm were read (N = 34). The average of these figures is 8.53 mm. The exponentional distribution does fit best for the following value of the constants **a** and **b**:

a = 1.26
b = 1.40

The rainfall equalled of exceeded n times is given by:

$$R = \frac{\bar{R}}{a}*\ln\frac{b*N}{n} = 6.77*\ln\frac{47.7}{n}$$

Let R_n be the rainfall that is equalled or exceeded n times in N years, then the chance P that R_n is equalled or exceeded is n/N. The return period T is the reciproke of P (in numbers of rainfalls). Consequently; to the rainfall R_n belongs a return period T = 1/p = N/n. There are N rainfalls in 3 years; N/3 rainfalls a year. So, a return period of T = N/m (rainfalls) means a return period of T_{year} = N*3/ (n*N) = 3/n year.

The measured rainfall is accepted to be 10% to small. Therefore the measured rainfall has been multiplied with 1.1. In table C8 the alternative expected rainfall has been compared with the extreme values as is computed in sub-chapter C.4.1, and the design rainfall as in sub-chapter C.4.4. It appears that the results are slightly different, but not very much. See also figure C17.

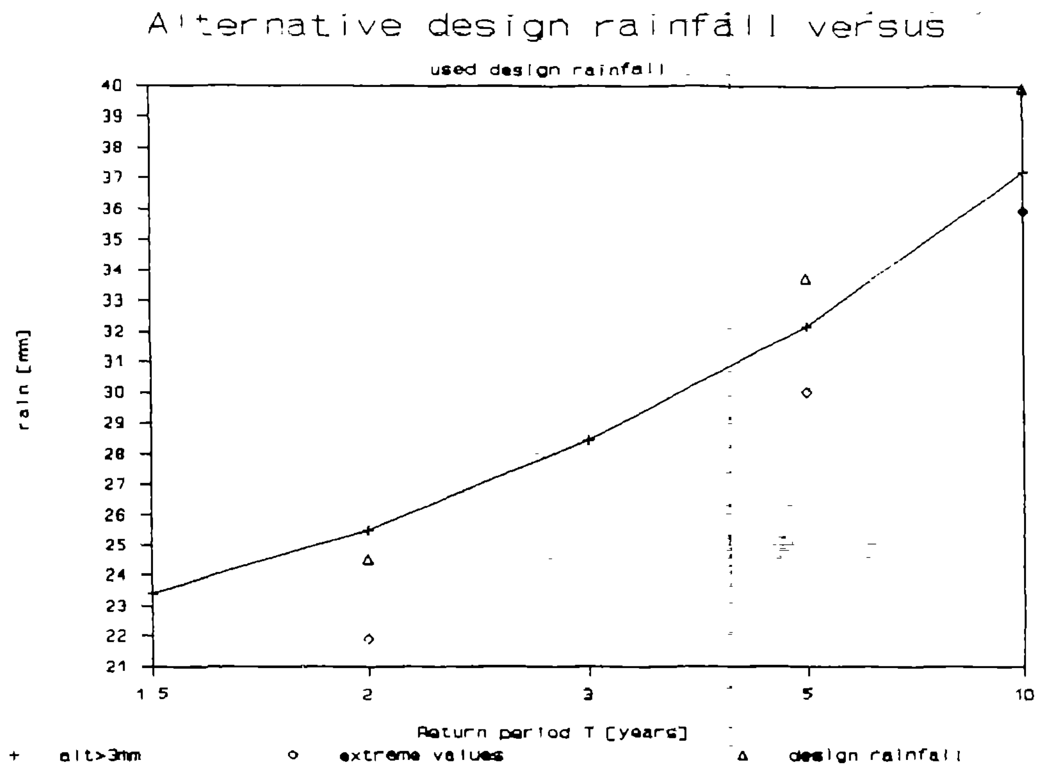


Figure C17: Alternative rainfall versus used design rainfall.

T [year]	Alternative expected rainfall			extreme value [mm]	design rainfall [mm]
	n	r [mm]	r+10% [mm]		
1.5	2	21.5	23.6		
2	1.5	23.4	25.8	21.9	24.4
3	1	26.2	28.8		
5	0.6	29.6	32.6	30.0	33.7
10	0.3	34.3	37.7	35.9	39.9

Table C8: Comparison alternative rainfall and design rainfall.

Conclusion.
The difference between the rainfall as computed in this alternative way and the design rainfall is very small. The difference is only about 5%.
The results of the alternative method are closer to the measured data than the design rainfall. Only 1 step is made to derive the (alternative) rainfall for a certain return period:

1 step: A direct fit of the exponentional distribution, with which the expected rainfall is computed.

For the other method 3 steps are needed:

First: Relation daily rainfall --> rainfall in short periods.
Second: Extreme values according to Gumbell.
Third: Fitting a intensity-duration-curve, through the extreme values.

On the other hand with method in 3 steps, more data is used.

Luckily the results of the alternative method support the results of the other method. There is no reason to change the design rainfall.

C.5 RESEARCH RAINFALL INTENSITIES IN YEMEN.

In Yemen for a reasonable period, rainfall has been measured. In this chapter the results of some researches concerning the rainfall intensities, are given.

C.5.1 Russian method.

In 1986 a Russian team published a research titled: Sana'a Basin Water Resources Scheme (literature [5]). This was carried out for the Ministry of Agriculture and Fishery of the (then) Yemen Arab Republic. In this research a relation between rainfall in a short period and 24-hour rainfall is given.

The results are given below:

$$R_{0.5} = 0.33 * R_{24}$$
$$R_{1} = 0.38 * R_{24}$$
$$R_{3} = 0.57 * R_{24}$$
$$R_{6} = 0.76 * R_{24}$$

These equations give considerable smaller intensities in small periods than what is found for Rada. The relations were derived from data of 5 stations in Yemen and 2 in the United Arab Emirates. Information about how these equations were derived is missing.
In the report a rainfall of 23.6 mm/0.5 hour for T = 10 years was found, while for Rada 39.9 mm/0.5 hour is computed!

C.5.2 Method RWSSP.

By the RWSSP the method given in chapter 4 of the main report was applied to obtain the design storm. The disadvantages of this method have already been discussed in sub-chapter 4.1 of the main report and the results are considered to be too small.

C.5.3 Method RIRDP.

The rainfall is measured for the RIRDP. This project is mainly interested in rainfall intensities for long durations (from 1 day up), but in one of its reports rainfall intensities for small periods are computed. This has been done with the same method as at the RWSSP, and also gives too small rainfall intensities for small durations.
The average rainfall intensities (according to the RWSSP), with a duration of 1 hour and for return period of 2 years and 10 years, were 15 mm/h and 24 mm/hour respectively. Undoubtedly, this is too small, because from the three years with pluviographs of Rada it is read that in that period the rainfall already exceeded 3 times the 30 mm/h for that period.

C.6 KINDS OF DESIGN STORMS.

A return period of 2 years is commonly used, therefore first a design storm of that return period is created. Later, if necessary, design storms for other return periods can be created too.

There are different methods to create a design storm. The simplest storm is one of constant intensity, during a critical period. This critical duration is dictated by the time a raindrop falling on the edge of a catchment, needs to reach the end of the drainage system (concentration time t_c). This time consists of a time for flow over land to the system and a time for flow in the system to that point.
However, such a storm is not likely to happen. In reality the intensity of the rainfall will vary during the storm. The rain will start with small intensities, then a peak and from that the intensity decreases to zero.

C.6.1 Method WMO

The WMO (World Meteorological Organisation) prescribes a distribution in percentages of the total rainfall. For catchment areas with concentration times smaller than 6 hours, a duration of the storm of 6 hours is advised. For areas with longer concentration times, this should also be the duration of the storm. The concentration time for Rada is just 1.5 hour, so a storm of 6 hours should be chosen. The distribution of the rainfall is as follows:

hour	**cum %**
0-1	8%
1-2	22%
2-3	70%
3-4	84%
4-5	92%
5-6	100%

C.6.2 Chicago design storm

Another distribution of the rainfall during the storm is given by the Chicago Design Storm. The peak appears at the moment that the ground is maximally moistened and the depressions filled up and therefore all the water will be discharged into the system.
If I_a is the average intensity for a duration t_d, then the precipitation p, falling in a period t_d is computed with:

$$p = t*I_a = \frac{a*t}{(b+t)^c} \quad [mm]$$

With: p = rainfall in mm
I_a = average rainfall in mm/hour
t_d = duration of rainfall in hours
a, b and c are constants.

The intensity at moment t then is:

$$I = \frac{dp}{dt} = \frac{a(b+t(1-c))}{(b+t)^{c+1}} \quad [mm/h]$$

In this case for a return period of 2 years, the constants are:

a = 51.9
b = 0.6
c = 1.03

For t > b/(c-1) = 20 minutes the intensity becomes smaller than zero, but such long durations won't be used.

A part of the total rainfall falls before the peak during a period t_b. The remaining part of the rain falls during t_a (afterwards). The total duration is $t = t_b + t_a$. The distribution of the total duration over t_b and t_a depends on the catchment area. This distribution is expressed in $r = t_b/t_a$. The course of the intensities before and after the peak is given in figure 18. For the surroundings of Chicago a value for r of 0.375 is advised. For Yemen this value has still to be determined.

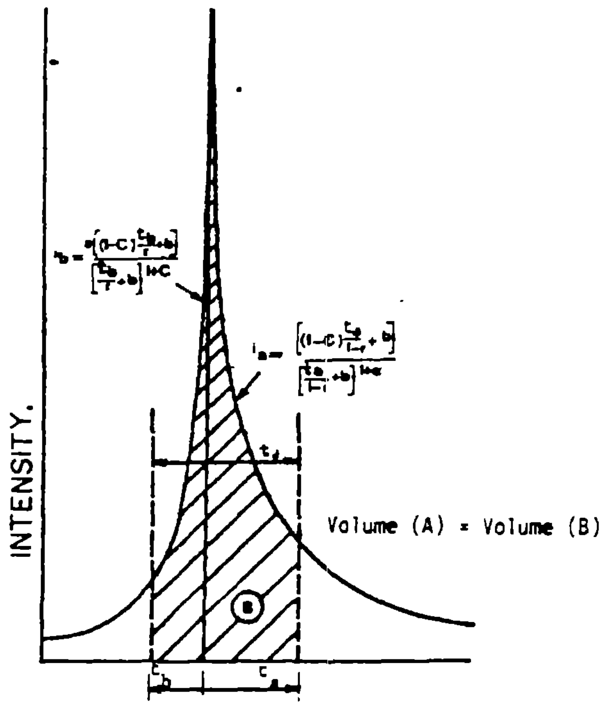


Figure 18: Chicago Design Storm.

C.6.3 Measured storms.

If a long series of heavy storm is available, then these can be used to design the drainage system, by entering them into the model and examining the result. However, for Rada only 6 heavy storms where measured, and also of Al-Khabar and Al-Khadra only a few heavy storms are available.
The 6 heaviest storms measured are in table C9. It appears that the peak is short and heavy, while the rainfall before and after this peak is mainly restricted to a period of 1.5 hour. Certainly some information is lost, because the pluviographs were read in periods of 30 minutes. It is assumed that a storm has ended, if for 6 hours no rain has fallen.
The duration of a shower appears to be 1.5 to 3.5 hours with an average of 2.25 hour.

heaviest measured storms [mm] +10%												
	Rada						Al-Khabar		Al-Khadra			
time	storm						stkhab		stkhad			
hours	1	2	3	4	5	6	1	2	1	2	3	4
0.5				1.3								
1.0				0.7			0.2					
1.5		19.8		0.0	3.3	0.9	4.8			2.0		
2.0	22.4	24.2	22.0	19.6	16.1	13.2	22.0	22.9	12.8	13.2	16.9	16.5
2.5	14.1	5.1	11.0	16.1	5.1	4.4	4.6	5.1	9.0	7.5	5.1	5.5
3.0	0.7		7.0	8.6		1.1		1.5	2.0		1.8	5.9
3.5	1.1			1.5		0.7		0.9	1.1			1.1
4.0	0.9					0.9			0.7			
4.5												
total	39.2	49.1	40.0	47.7	24.4	21.1	31.7	30.4	26.6	22.7	23.8	29.0

Table C9: Heaviest storms measured.

C.6.4 Composed design storm.

A third method is to build a design storm from short time steps with critical intensities. This method is called the USA Soil Conservation Method in the Final Design Report of the RWSSP. The duration of the peak should be so long that the discharge to the first points of the drainage system is maximum. In this case 15 minutes seems right.
The peak will have a period of 15 minutes. The maximum rainfall for 2 years falls in 15 minutes with constant intensity. Next is computed how much rain falls in the 15 minutes after the peak. This is the maximum rainfall in 30 minutes (T = 2 years) minus what fell in the peak of 15 minutes. What is left, falls with a constant intensity during the next 15 minutes.

Before this, again there is a period of 15 minutes of rainfall with constant intensity. The rainfall in those 15 minutes is the maximum rainfall in 45 minutes minus what fell in 30 minutes. This goes on until a storm of the chosen duration is reached (1.5 hours). See table C10 and figure C19.

Design storm T = 2 years				
time [hour]	duration [min]	total rain [mm]	rain [mm]	intensity [mm/h]
0.25	15	16.4	16.4	65.6
0.50	15	24.4	8.0	32.0
0.75	15	29.1	4.7	18.7
1.00	15	32.1	3.1	12.2
1.25	15	34.3	2.1	8.5
1.50	15	35.8	1.6	6.3
1.75	15	37.0	1.2	4.8
2.00	15	38.0	0.9	3.8

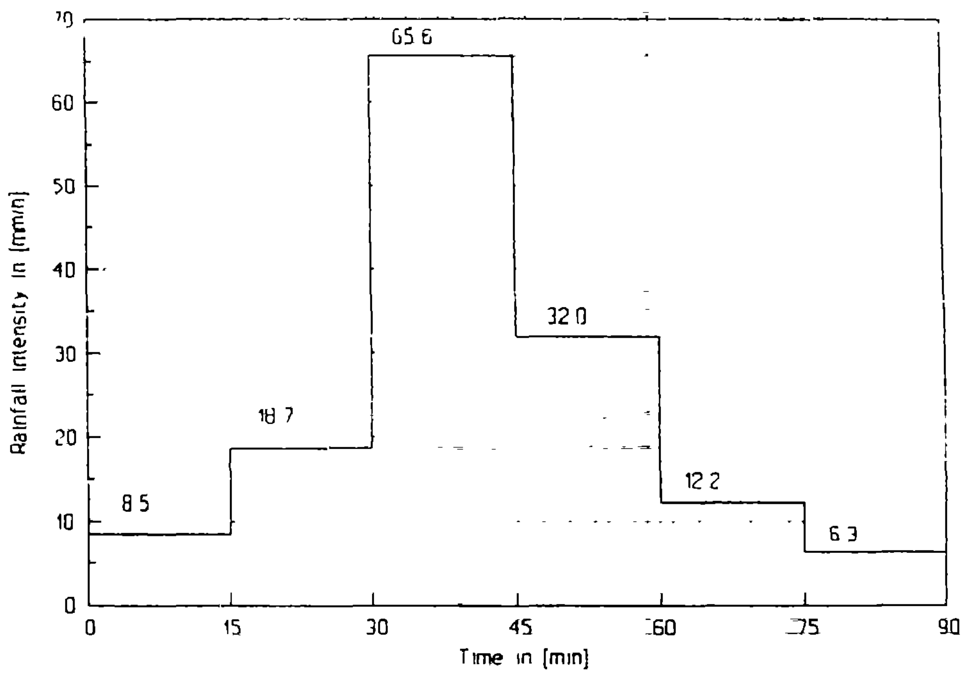


Table C10 and Figure C19: Created design storm. T = 2 years.

C.7 DESIGN STORM CHOSEN.

From the presented possibilities, a design storm has to be chosen. Because at least a little information about the size and shape of the real storms is known, the method of the WMO is rejected. Six hours is far too long. Also the Chicago Design Storm is rejected, because it will be very hard to define the factor r for the surroundings of Rada.

With the computer model and the fast computers it is however possible to get some insight in the effect of the different storms, by computing the water levels and the discharges. In the following storms of constant intensity, the measured storms and some created design storms are routed through the model, but first the rational method is tried.
For the North-Eastern section of the drainage system only the effect of these different design storms is investigated.

C.7.1 Rational method.

Instead of the advanced computer model, a simpler model can be used to have an indication of the maximum discharge. The maximum discharge at the end of the system, is computed with:

$$Q = c*i*A$$

with c = runoff coefficient
i = constant intensity in $m^3/s*ha$ for a duration t_c and a return period T.
A = catchment area in ha.

From computations with CYCLONE it appeared that a duration of 35 minutes is critical for a storm with constant intensity (see subchapter C.7.2). Consequently a duration of 35 minutes has been chosen for the rational method. For 35 minutes the design intensity for a return period of 2 years is i = 44.9 mm/h or 0.125 $m^3/s*ha$. The area[3] is 41.4 ha and an average runoff coefficient of 0.55 is used[4]. This results into a maximum discharge of 2.8 m^3/s.
The maximum discharge computed in this way is too high (provided that the runoff coefficient and the duration are correct), because of the following two reasons:
Firstly, in the rational method is assumed that all the catchment contributes to the runoff at the end of the 35 minutes. In reality, the water needs some time to reach the end of the system. Dependent on the distance, the slope and the configuration of the system, not yet from the whole catchment, the water will reach the end of the system at the

(3) North-Eastern section.

(4) The average of the runoff coefficients as given in the annexes of the Final Design Report, December 1989.

end of the 35 minutes.
The second reason is that the rational method does not take storage on the roads into account.
So the maximum discharge will be smaller than computed with the rational method.

In table C11 the result of the rational method is compared to the results of computations with a computer model of the drainage system (CYCLONE computations). For different storms the water levels and discharges have been computed with Cyclone. These storms are presented in the following sub-chapters.
To anticipate on the results of the CYCLONE computations, it can be concluded that the maximum discharge computed with the simpler rational method does not differ much form the other results. Compared to the measured storms, a design of the drainage system based on the rational method, would be on the safe (large) side, but not wrong.
An advantage of the CYCLONE computations is that they result in water levels throughout the drainage system. The effect of certain changes in the design can be studied, to come to an optimal design of the drainage system.

C.7.2 Constant intensity.

The computer model CYCLONE is used to compute the water levels and discharges in the drainage system.
To have an indication of the concentration time of the system, storms of constant intensities with a return period of 2 years, can be routed through the model. The intensities are computed with:

$$I_a = \frac{48.8}{(0.5+t_d)^{1.03}} \quad [mm/h]$$

In table C11 the results of these storms are given (for the North-Eastern section). For a duration longer than 90 minutes, the discharge becomes stationary. So then water of the whole catchment reaches the end of the system. The maximum water levels and discharges, do however occur at much smaller durations: between 30 and 35 minutes. Because of the time water needs to build up a hydraulic gradient, the peak will occur at the moment that not yet the whole catchment contributes to the discharge. For longer durations the design intensity will be smaller giving also a smaller discharge i.e. water level.
The most critical storm with constant intensity causes a water level of 35 cm.

A real shower doesn't have a constant intensity, but will have a peak intensity. The question is whether the peak will be at the beginning, middle or end of the storm. If the peak

Rational method					
kind of storm	total rainfall [mm]	duration [min]	return period [year]	max water depth [m]	Qmax [m^3/s]
const int	26	35	2		2.8

CYCLONE computations					
kind of storm	total rainfall [mm]	duration [min]	return period [year]	max water depth [m]	Qmax [m^3/s]
constant rainfall intensity	15	15	2	0.26	1.6
	24	30	2	0.35	2.3
	26	35	2	0.35	2.4
	28	40	2	0.34	2.3
	32	60	2	0.32	1.9
	36	90	2		1.5
Measured storms					
Rada					
storm 1	39#	150	2	0.34	2.2
storm 2	49#	90	5	>0.40 **	2.9
storm 3*	40#	90	3-4	0.34	2.1
storm 4	48#	90	3-4	0.33	2.0
Al-Khabar					
stKhab1	32#	120	<2	0.35	2.3
stKhab2 *	30#	120	<2	0.34	2.2
Design storms					
new	36	90	(>)2	0.37	2.58
RWSSP	9	180	<<2	0.30	1.7

* storm at 22-4-'88
** flooding
\# data from pluviographs + 10%.

Table C11: Maximum water depths and discharges.

comes first, then the storm with constant intensity gives an overestimation. For a storm with a peak in the middle or at the end, this will be an underestimation, because then, when the peak intensities occur, the soil is already wet and the depressions are filled up.

C.7.3 Measured storms.

The 6 heaviest rain events of Rada (see table C9), from the second half of 1986 until 1989, were entered in the model. Four of these showers (see figure 12 of the main report) yield a considerable discharge. Two of the 6 showers are not quite sure, due to unclear pluviographs (storm 3 and storm 6). Also the probable return periods have been computed from the duration of the biggest part of the shower and the total rainfall in that period.
The second storm of Rada, storm 2, has got a probable return period of about 5 years and (as the system is designed for a return period of 2 years), causes flooding. The other storms are smaller and cause no flooding in the North-Eastern section of the present design of the drainage system.
Storm 5 and storm 6 have probable return periods much smaller than 2 years, and therefore, cause no critical water levels. The return periods of storm 1, storm 3 and storm 4 are 2, 3-4 and 3-4 years respectively.
In Al-Khabar only 2 heavy storms have been registered (stkhab 1 and stkhab 2 in table C9). They do coincide with storm 6 and storm 3, which are unclear. The results of these storms are water levels of 35 and 34 cm, which is high, considering the probable return period is smaller than 2 years. The shapes of these 2 showers must have been more critical. A comparison of the course of those storms shows that a storm with a distinct peak is critical.

The maximum water levels caused by storm 1, storm 3, storm 4, stkhab 1 and stkhab 2 are about equal. For the 4 sections of the drainage system, there is not one storm critical (except storm 2, of course). The maximum percentages of relative water levels for a few storms are given in table C12 (first 5 columns).

section	storm 1	storm 2	storm 4	stkhab 1	created storm (cs)	Result cs - max storm %	[cm]
North-East	80%	>100%	-	83%	90%	7%	2
North-West	86%	>100%	84%	86%	95%	9%	2
South	90%	>100%	92%	86%	93%	3%	1
Middle	93%	>100%	92%	97%	>100%	3%	>1

Table C12: Fillings caused by measured ar created storms.

In table C9 the heaviest storms of the 3 stations are given. The measured showers have a peak of 1.5 hours, in which most of the rain falls. The average distribution of the rainfall over a storm is:

% rainfall	period
10%	0- 30 min
56%	30- 60 min
25%	60- 90 min
7%	90-120 min
2%	120-150 min

So the storms have a peak fairly soon after the beginning of the rain, and after that another period with reasonable high intensities.

These measured storms indicate that for a return period of 2 years, the maximum water levels should be about 35 cm.

C.7.4 Created storm.

The last method is building up a storm, as presented in the former chapter. This has been done as pointed out in chapter 6.4 and is worked out in figure 19 and table C10.
The results of this storm are water levels, a bit higher than the results of the heaviest measured storms and the storms with constant intensities (see table C11 "design storm; new").

The created storm presented, is critical, because it is built up out of critical rainfall intensities, for a chosen return period. In fact, in the created storm of 1.5 hours falls the maximum possible rainfall for T = 2 years. But within that period of 1.5 hours, there is a period of 1.25 hours in which falls the maximum amount of rainfall (for T = 2 years) for a period of 1.25 hours. Again in this period a

peak exists with the maximum rainfall for 1 hour, and so on... until a peak with a duration of 15 minutes is obtained.
The chance that this design storm will be equalled or exceeded is therefore smaller than the average of once every 2 years.

Another point is that, because the storm is derived from 3 years of pluviographs and 11 years of daily rainfall figures, the return period of 2 years is not too sure. More pluviographs might change the design intensities. To be on the safe side, a return period of 2 years should be interpreted as meaning that the return period is between 1.5 and 3 years.

C.7.5 Design storm.

The water levels resulting of the created storm are too high, thus on the safe side, because the return period is expected to be longer than 2 years. In table C12 (6^{th} column) also the fillings caused by the created storm are given. It appears that the difference between the results of the created storm and the measured storms is only small (table C12 last two columns): only 3% for the middle section, in which flooding is most likely. 3% of a conduit of 40 cm is about 1 cm. This is certainly negligible, considering the uncertainties in the design storm and the modelling.
The aim of this study is to check the present design of the drainage system, as presented in the Final Design Report, December 1989 and if necessary to give recommendations for adjustments.
Not just one of the measured storms is critical for all sections (except for storm 2), and if adjustments in the design are made, another storm might become decisive. The difference between the water levels caused by the different storms is only small, and often even negligible.
To check the present design of the drainage system, one storm is chosen: The created storm, which is on the safe side.

Thus the drainage system will be designed using the created storm as design storm.

C.8 RAIN GAUGES.

By the RIRDP a network to measure the rainfall was set up. See figure C2. The equipment used are mechanical recorders, standard rain gauges and rainlogs.

C.8.1 Mechanical recorders.

The mechanical recorders are of the tipping-bucket type. The funnel has got an opening area of 1000 cm^2 and is connected to one of the 2 buckets. If the bucket is full, then it tips over, and the water pours out in a collecting reservoir The other bucket is now under the funnel and the pen on the paper registers 0.2 mm of rainfall. The water in the collecting reservoir should be measured, to control the registered rainfall, but it seems as if this hasn't been done.
In the area of Rada the paper in the mechanical recorders had to be changed once a week. Often something went wrong with changing the papers, or the paper was only changed after it started raining. To safe guard the recorder against vandalism, they are installed on the roofs of the houses of the operators. The rim height is 65 cm above this roof level. So the top of the collector is far from the ideal height, which, because of the wind, is at ground level.

C.8.2 Standard rain gauges.

Except for the mechanical recorders, also standard rain gauges were used. It is however unclear when and where these have been used instead of the mechanical recorders. The standard rain gauge consists of a funnel with an opening area of 200 cm^2 on top of a reservoir. The water collected is poured into a measure cylinder and measured every morning at 6 o'clock.

C.8.3 Rainlogs.

Rainlogs are electronic recorders, which measure the pressure of the column of water caught and the outside air pressure. The pressure is intermittently registered at a chip. The time step between two measurements can vary. It is possible to have normally a big time step, but when it starts raining the time step becomes smaller (for example 5 minutes). When the rain has stopped the time steps become bigger again. After 14 months the rain log can be opened and linked to a computer. With the right software the results can be read.
Unfortunately no data of the rainlogs was available yet, at the beginning of 1991.

C.9 PROBLEMS WITH EXECUTION.

In creating the design storm the problems which occurred, had mainly to do with filling up missing data and the validity of the data.

C.9.1 Missing data.

The set of day totals is incomplete. For some months or parts of months nothing was measured and no data is available. Where possible, it was tried to fill up the missing data. In comparing the 3 stations (Rada, Al-Khabar and Al-Khadra), it appeared that a big rainfall in one of them did not necessarily mean that also a big rainfall occurred in the other stations. A comparison of the total weekly rainfall gives a better result, but still not even a reasonable relation between the stations.

For computing the intensities, the maximum daily rainfalls are used. If the rainfall data of a year was not complete, it was checked, whether it was likely that the maximum had fallen in the missing period, by comparing this period with the corresponding period in the other stations. If this was the case, this year was not used, otherwise the missing period was assumed to be dry. For Rada the data of the years 1976 and 1990 were not used. For Al-Khabar the years 1978 and 1989 and for Al-Khadra 1990 were omitted.

C.9.2 Difference pluviographs and day totals.

Another problem is the difference between the total daily rainfall read from the pluviographs and the given set of day totals. The storms with high intensities, read from the pluviographs, are for some years, but not all, smaller than the given set, while the small storms are the same. See figure C20, giving the difference between the daily rainfall of the 2 sets plotted against the daily rainfall of the pluviographs. From this figure no multiplying factor (for example dependent on the total daily rainfall) can be derived. It is odd that for all years the results are different:

Rada:	All the big rain events of the pluviographs are smaller than the given set day totals,
Al-Khabar:	1985 1986 1989 and 1990 exactly the same, but different in 1987 and 1988.
Al-Khadra:	Every year exactly the same!

Of course, it was only possible to compare the years of which pluviographs were available. The maximum daily rainfalls and the average yearly rainfalls of the other years are however not significantly smaller, so it is assumed that the data was always obtained in the same way.

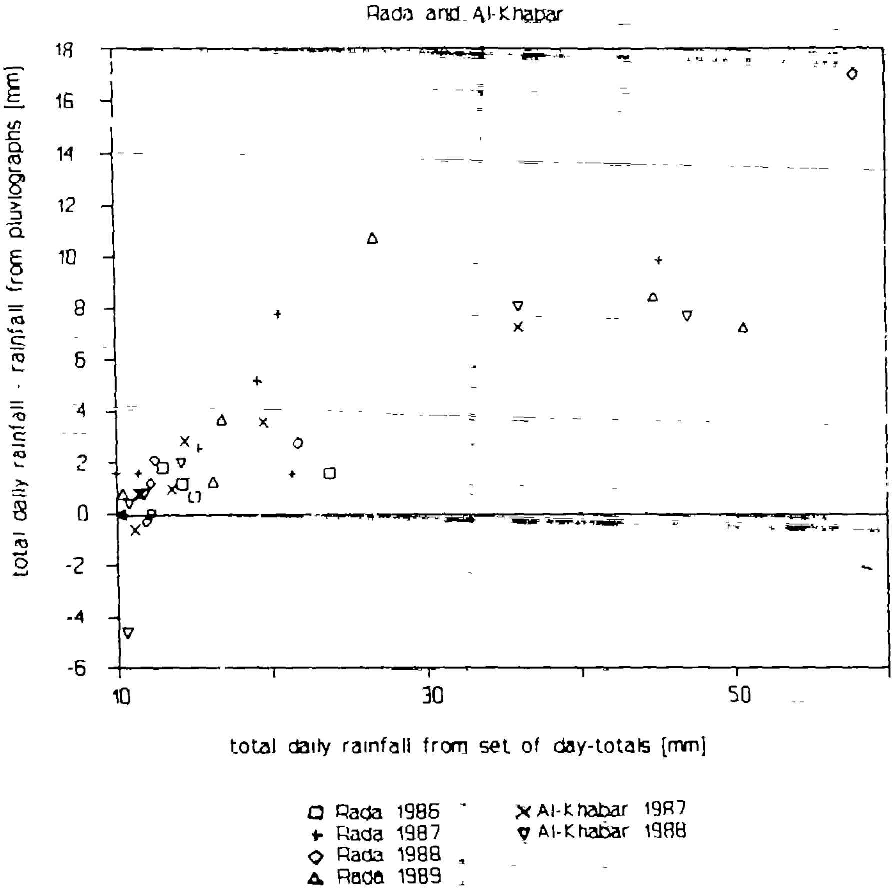


Figure C20: Difference pluviographs and given daily rainfall.

The difference might be explained with the use of different types of rain gauges. The different types have a different systematic error and different wind losses. However it is not possible to find out when what equipment was used.
From a research by the RIRDP it appeared that the rainfall figures registered are 10 to 15% percent too small. This loss was not yet accounted for in the day totals, so they still have to be increased. It seems likely that this loss is caused by the wind.
For the relation between rainfall in short period and day total, the loss of 10 to 15% is of some importance.
If it is assumed that the wind error is equally divided over the storm, than this loss doesn't have any impact on the relations between rainfall in short periods and daily rainfall. It is however likely that this loss is not divided this way, but that during high intensities disproportionately more water is spilled. In that case the relations found give a too low rainfall for the short periods. But the error, made by assuming the loss divided

proportional over the storm, is neglectible, as is shown below.
The 6 heaviest rain storms of Rada had an average distribution as given below.
If it is assumed that the loss is percentagewise divided over the 2 periods with the biggest intensities, then it appears, that instead of with 10% the maximum should be increased with 12.5%. A difference of 2.5% is of course neglectible.

```
10% --> 10%                      = 10%
56% --> 56% + 10%*56/(25+56) = 63%     7 is 12.5% of 56.
25% --> 25% + 10%*25/(56+25) = 28%
 7% -->  7%                      =  7%
 2% -->  2%                      =  2%
----                              ---- +
100%                              110%
```

C.9.3 Reading data from the pluviographs.

At reading of data from the pluviographs some difficulties did arise, because some of the pluviographs were not clear. For example the funnel was partly logged, so a storm was spread over a long period, or the line on the graph was drawn by hand. Sometimes the paper was stuck or the operator had forgotten to wind up the clock.
Because of this some of the (already few) heavy storm were lost.

rain read from pluviographs in periods of 30 minutes

duration (hours)	0.5 measured	1 → computed	2	4	5
1986					
2-8	0.6	0.6	0.6	0.6	0.6
	0	0.6	0.6	0.6	0.6
	0	0	0.6	0.6	0.6
	10	10	10.6	10.6	10.6
	0	10	10	10.6	10.6
	1.1	1.1	11.1	11.7	11.7
4-8	4.4	4.4	4.4	4.4	4.4
	2.6	7	7	7	7
	0.8	3.4	7.8	7.8	7.9
	0	0.8	7.8	7.8	7.8
	0	0	3.4	7.8	7.9
	0	0	0.8	7.8	7.8
	0.2	0.2	0.2	8	8
	0.6	0.8	0.8	8.6	[illegible]
	0	0.6	0.8	4.2	[illegible]
	0	0	0.8	1.6	8.6
	0	0	0.6	0.8	8.6
	0	0	0	0.8	8.6
	0.8	0.8	0.8	1.6	5
	0	0.8	0.8	1.6	2.4
	0	0	0.8	1.4	1.6
	0	0	0.8	0.8	1.6
	0	0	0	0.8	1.6
	0.8	0.8	0.8	1.6	2.4
	0	0.8	0.8	1.6	2.2
	0.4	0.4	1.2	2	2
	1.2	1.6	2.4	2.4	3.2
	0.4	1.6	2	2.8	3.6
	0.8	1.2	2.8	3.6	4.4
	0.4	1.2	2.8	4	4.8
	0.2	0.6	1.8	4.2	4.2
	0	0.2	1.4	3.4	4.2
	0.4	0.4	1	3.8	4.6
15-9	0	0	0	0	0
	14.6	17.6	17.6	17.6	17.6
	4.6	19.2	22.2	22.2	22.2
1987					
12-3	1	1	1	1	1
	0.2	1.2	1.2	1.2	1.2
	0	0.2	1.2	1.2	1.2
	0	0	1.2	1.2	1.2
	0	0	0.2	1.2	1.2
	0	0	0	1.2	1.2
	0	0	0	1.2	1.2
	1.4	1.4	1.4	2.6	2.6
	1.2	2.6	2.6	2.8	3.8
	9	10.2	11.6	11.6	12.8
	0.2	9.2	11.8	11.8	13

rain read from pluviographs in periods of 30 minutes

duration (hours)	0.5 measured	1 → computed	2	4	6
23-1	20.4	20.4	20.4	20.4	20.4
	12.3	33.2	33.2	33.2	33.2
	0.6	13.4	33.8	33.3	33.9
	[illegible]	1.6	34.3	34.3	34.8
	0.3	1.8	15.2	35.6	35.6
10-4	0.6	0.6	0.6	0.6	0.6
	1	1.6	1.6	1.6	1.6
	0.3	1.3	2.4	2.4	2.4
	[illegible]	3.3	5.4	5.4	5.4
	[illegible]	10	11.3	12.4	12.4
	0	[illegible]	10.3	12.4	12.4
	0	0	10	12.4	12.4
	0	0	[illegible]	12.4	12.4
	0	0	0	11.3	12.4
	0	0	0	10.8	12.4
	0	0	0	10	12.4
	0	0	0	7	12.4
	0	0	0	0	[illegible]
	[illegible]	1	1	1	11.3
	5.4	[illegible]	[illegible]	[illegible]	16.4
12-4	[illegible]	1.2	[illegible]	2.2	2.2
	5	[illegible]	7.2	7.2	7.2
	2.9	11.6	13.3	13.3	13.3
	0	5.6	12.3	13.3	13.3
	0.6	0.6	12.2	14.4	14.4
	0.4	1	7.6	14.8	14.8
[illegible]	9.2	9.2	9.2	9.2	9.2
	0	9.2	9.2	9.2	9.2
	0	0	9.2	9.2	9.2
	1.2	1.2	10.4	10.4	10.4
	0	1.2	1.2	10.4	10.4
	0.8	0.8	2	11.2	11.2
	0.8	1.6	2.8	12	12
	0.6	1.4	2.2	12.6	12.6
	0.4	1	2.6	3.3	13
1988					
17-4	[illegible]	1.2	1.2	1.2	1.2
	0	1.2	1.2	1.2	1.2
	0	0	1.2	1.2	1.2
	0.6	0.6	1.8	1.8	1.8
	3.4	4	4	5.2	5.2
	1.4	4.8	5.4	[illegible]	[illegible]
	0.2	4.6	[illegible]	[illegible]	[illegible]
	0.6	0.8	[illegible]	10.4	10.4
22-4	18	18	18	18	18
	22	10	40	40	40
	1.5	26.6	44.6	44.6	44.6

Table CC1: Rain read from the pluviographs of Rada.

rain read from pluviographs in periods of 30 minutes.

Duration: (hours)	0.5 measured	1 → computed	2	4	6
28-7	Heavy storm of totally 19 mm rain not sure				
	0.8	0.8	0.8	0.8	0.8
•	12	12.8	12.8	12.8	12.8
•	4	16	16.8	16.8	16.8
•	1	5	17.8	17.8	17.8
•	0.6	1.6	17.6	18.4	18.4
•	0.8	1.4	6.4	19.2	19.2
1-8	0.2	0.2	0.2	0.2	0.2
	0	0.2	0.2	0.2	0.2
	0	0	0.2	0.2	0.2
	0	0	0.2	0.2	0.2
	0	0	0	0.2	0.2
	0	0	0	0.2	0.2
	0.6	0.6	0.6	0.8	0.8
	0	0.6	0.6	0.8	0.8
	0.2	0.2	0.8	0.8	1
	0	0.2	0.8	0.8	1
	0	0	0.2	0.8	1
	0	0	0.2	0.8	1
	0	0	0	0.8	0.8
	0	0	0	0.8	0.8
	0	0	0	0.2	0.8
	0	0	0	0.2	0.8
	0.2	0.2	0.2	0.2	1
	0.4	0.6	0.6	0.6	1.4
	1	1.4	1.6	1.6	1.8
	3.4	4.4	5	5	5.2
	2.4	5.8	7.2	7.4	7.4
	1.8	4.2	8.6	9.2	9.2
	0.2	2	7.8	9.4	9.4
23-8	0.2	0.2	0.2	0.2	0.2
	7	7.2	7.2	7.2	7.2
	0.2	7.2	7.4	7.4	7.4
	0	0.2	7.4	7.4	7.4
	0.8	0.8	8	8.2	8.2
	1	1.8	2	9.2	9.2
	1	2	2.8	10.2	10.2
	0.8	1.8	3.6	11	11
1989					
6-2	Heavy storm of totally 36.4 mm unsure.				
•	20	20	20	20	20
•	10	30	30	30	30
•	6.4	16.4	36.4	36.4	36.4
29-2	1.2	1.2	1.2	1.2	1.2
	0.6	1.8	1.8	1.8	1.8
	0	0.6	1.8	1.8	1.8
	17.8	17.8	19.6	19.6	19.6
	14.6	32.4	33	34.2	34.2
	7.8	22.4	40.2	42	42
	1.4	9.2	41.6	43.4	43.4

Table CC1: Continuing.

Rainfall in [mm] for different durations (in [hours])					
0.5	1	2	4	6	1 day
10.0	10.0	11.1	11.7	11.7	11.7
4.4	7.0	7.8	8.6	8.6	14.0
14.6	19.2	22.2	22.2	22.2	22.2
9.0	10.2	11.8	11.8	13.0	15.3
20.4	33.2	34.8	35.6	35.6	35.6
7.0	10.0	11.8	12.4	16.4	18.8
6.6	11.6	13.8	14.8	14.8	14.8
9.2	9.2	10.4	12.6	13.0	12.8
3.4	4.8	8.6	10.4	10.4	10.4
22.0	40.0	44.6	44.6	44.6	44.6
12.0	16.0	17.8	19.2	19.2	19.2
3.4	5.8	8.6	9.4	9.4	10.4
7.0	7.2	8.0	11.0	11.0	11.0
20.0	30.0	36.4	36.4	36.4	36.4
17.8	32.4	41.6	43.4	43.4	43.4

Table CC2: Maximum rain read from pluviographs.

0.5 uur

T	YT	X'T	delta	Xo	Xb
2	0.37	21.9	4.2	17.7	26.1
5	1.50	30.3	7.8	22.5	38.1
10	2.25	35.9	10.8	25.1	46.7

4 uur

T	YT	X'T	delta	Xo	Xb
2	0.37	41.7	8.0	33.7	49.7
5	1.50	57.8	14.8	43.0	72.7
10	2.25	68.5	20.6	47.9	89.1

1 uur

T	YT	X'T	delta	Xo	Xb
2	0.37	34.8	6.7	28.1	41.5
5	1.50	48.2	12.4	35.8	60.6
10	2.25	57.0	17.2	39.8	74.2

6 uur

T	YT	X'T	delta	Xo	Xb
2	0.37	42.1	8.1	34.0	50.2
5	1.50	58.3	15.0	43.3	73.3
10	2.25	69.0	20.8	48.1	89.8

2 uur

T	YT	X'T	delta	Xo	Xb
2	0.37	40.4	7.8	32.6	48.2
5	1.50	55.9	14.4	41.5	70.3
10	2.25	66.1	20.0	46.2	86.1

dag

T	YT	X'T	delta	Xo	Xb
2	0.37	43.0	8.3	34.7	51.3
5	1.50	59.5	15.3	44.2	74.8
10	2.25	70.4	21.2	49.1	91.6

Table CC3: Results computation confidence intervals for extreme values.

Annex D:

COMPUTER MODEL

CONTENTS.

D.1 GENERAL.

To compute the water levels in the system the computer model CYCLONE has been used.

Cyclone is a computer model, computing discharges and water levels for unsteady flow conditions in sewerage systems and systems of open water courses. The hydraulic system is presented as a combination of nodes and conduits, with storage in the nodes (water levels) and flow in the conduits (discharges). The program deals with over land flow to transform rain falling on a catchment into an inflow hydrograph at a node. For this the kinematic wave approach is used.
If the water levels at a node rises above surface level, then flooding is simulated.

D.2 Computational method.

The flow conditions are described by the continuity equation in the nodes and the motion equation in the conduits. For long waves like in this case[1], the vertical component of the velocity can be ignored, resulting in an one-dimensional flow.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial s} = 0$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial_s}\left(\frac{Q^2}{A}\right) + gA\frac{\partial h}{\partial s} - gAi + \frac{gQ|Q|}{C^2RA} = 0$$

(1) (2) (3) (4) (5)

Q = flow rate	[m^3/s]
s = distance	[m]
A = wet cross sectional area	[m^2]
t = time	[t]
g = acceleration of gravity	[m/s^2]
h = water depth	[m]
i = bottom slope	[-]
C = Chézy coefficient	[$m^{1/2}/s$]
R = hydraulic radius	[m]

The first two terms of the motion equation (inertia and non-uniform velocity distribution) are neglected. So only (3) hydrostatic pressure, (4) gravity and (5) friction are taken into account. For the use of sewerage systems this is justified, because the friction is much more important than

---

(1) Long wave if wl (wave length) ≥ a (waterdepth). wl = c*t, t (duration) ≈ 1 hour. c = $(g*a)^{1/2}$. a ≈ 30 cm --> wl ≥ a.

the velocity distribution. This results in the diffuse wave equations.
The equations represent the flow conditions at every moment and place. To solve the equations: i.e. compute water levels and discharges, an approximation is made. The water levels and discharges are only computed on chosen times ($t = 0$, Δt, $2\Delta t$, etc.) and at certain locations (at nodes for water levels and between nodes for discharges). This way the finite difference method is obtained.

The equations can be solved implicitly or explicitly:
For the explicit way of solving the equation, a condition for stable calculations is:

$$\frac{\Delta t * c}{\Delta x} \leq 1$$

with,	Δt	= time step	[s]
	Δx	= x-distance	[m]
	$c = \sqrt{gh}$	= velocity of long wave	[m/s]

If Δx is small, in order to give a correct schematization of the drainage system, then, Δt has to be small too.

The **implicit** way has been chosen, because then the computation is always stable and Δt is independent of Δx. So a big time step can be chosen for small x-steps, which saves time. This method is less accurate. Therefore, to check the result also a run with a small time step should be made. This way of computing is more complicated.

The equations can be written as follows:

__Continuity:__

$$[\theta * F_{t+\Delta t} + (1-\theta) * F_t] * \frac{h(i)_{t+\Delta t} - h(i)_t}{\Delta t} =$$

$$\theta * \sum_j [Q_{ij} + Q_e - E + P]_{t+\Delta t} + (1-\theta) * \sum_j [Q_{ij} + Q_e - E + P]_t$$

with:

F	= horizontal wet surface (on which water is stored)	[m²]
h(i)	= water depth at node i	[m]
Q_{ij}	= Flow from node j towards node i	[m³/s]
Q_e	= external flow towards or from node i	[m³/s]
E	= evapotranspiration from the wet surface of the node	[m³/s]
P	= rainfall at node i	[m³/s]
θ	= weighing factor	[-]

Motion:

$$\theta\left[gA_s(h_i-h_j)+LA_sgi-Lg\frac{Q|Q|}{C^2RA_s}\right]_{t+\Delta t}+$$

$$(1-\theta)\left[gA_s(h_i-h_j)+LA_sgi-Lg\frac{Q|Q|}{C^2RA_s}\right]_{t}=0$$

with:
h_j = water level at node j [m]
L = length of conduit [m]

To solve the motion equation implicitly, it has to be linearized. For open channel flow the discharge at time t + Δt is calculated with:

$$Q_{t+\Delta t}=K_{ij}(h(i)_{t+\Delta t}-h(j)_{t+\Delta t})$$

In this equation K_{ij} is the flow coefficient in [m^2/s]:

$$K_{ij}=\frac{\sqrt{h(i)-h(j)}}{h(i)-h(j)}AC\sqrt{\frac{R}{L}}$$

with:
A = wet cross section, average between nodes i and j. [m^2]
C = Chézy coefficient, average between nodes i and j. [$m^{½}/s$]
R = hydraulic radius for conduit, average between nodes i and j. [m]

The flow coefficient is computed for each string. Each time step the hydraulic quantities A, C and R are computed. The program assumes hydraulic rough conditions. R is computed either with White-Colebrook:

$$C=18\log\frac{12R}{k}$$

or with Manning:

$$C=\frac{R^{\frac{1}{6}}}{n}$$

with:
k = Nikuradse wall roughness [m]
n = Manning's roughness coefficient [$s/m^{1/3}$]

Implicit means that all equations are solved simultaneously. The linearized motion equation is filled in the continuity equation. This results in one equation for each couple of nodes. The equations are solved in matrices:

$$\begin{vmatrix} -K_{ij} & +K_{ij} \\ +K_{ji} & -K_{ji} \end{vmatrix} * \begin{vmatrix} h_i \\ h_i \end{vmatrix} = \begin{vmatrix} -S \\ +S \end{vmatrix}$$

K_{ij} = flow coefficient for flow from node j to i [m^2/s]
h = unknown water levels [m]
S = known terms of out of the equation of continuity [m^3/s]

For each time step the water levels and discharges are determined in a predictor and corrector phase. First K is computed for time t, from which the water levels at time $t + \Delta t$ are derived. These are the predictor values. Next, with h for $t + \Delta t$, the parameters F and K at $t + \theta\Delta t$ are computed (the corrector values). With the corrector values the final values for h at $t + \Delta t$ are computed.

D.3 INPUT

For the use of the computer model, certain characteristics of the drainage system have to be entered in the model: The design storm, the lay-out of the system; surface and invert levels, lengths of conduits, etc. Also the roughness and shape of the conduits are input for the model. For the catchments the average slope and critical length, initial loss of rain, and roughness of the ground for overland flow have to be given. Finally the runoff coefficients and spillways are input for the model too. These data are to be put in a specially arranged input file. These input files are given in annex D.5.
Except for these input items, it is possible to introduce (amongst others) water level curves, evapotranspiration, reservoirs, pumps etc. These possibilities are described in the manual of CYCLONE.

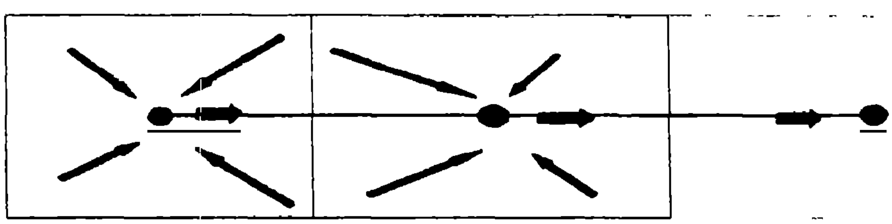
D.4 MODEL OF DRAINAGE SYSTEM RADA.

The modelling of the present design of the drainage system is given in Main Report and Annexes of the Final Design Report, 1989, of the RWSSP [lit 1 and 2]. As far as possible, for the new design the same input was used. Only for some points the input is different (and hopefully better).
It takes just a few minutes to compute the water levels in one section of the system, so by altering the input, it is easy to obtain some insight in the sensibility of the model for certain changes in the input.

Design storm.
For the design storm, see annex E.

Lay-out.
The lay-out of the present design of the drainage system is as in figure D1. The lay-out of the new design of the drainage system is just slightly differnet. See chapter 7 of the main report.

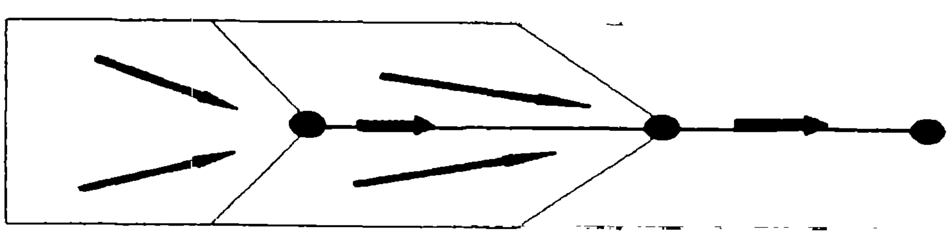
Catchments
In the Final Design Report of the RWSSP, the catchment area at a node was determined as the area above the node and the area round the node (see figure).



In reality, a part of this area does not discharge its water to that node. The water is discharged into the conduit after the node. The result is that for the average discharge (in the conduits), this method gives a good result, but for the water levels (at the nodes) an overestimation.

It appeared that not all catchment areas were entered correctly in the computer model.

Another way of defining the catchments is like this:



So only the water, discharged at a node, will reach that node. The rest of the water, will enter the drainage system at the next node (in reality it will enter in the conduit after the node). This method a gives more realistic water level at the nodes, but an underestimation of the average discharges in the conduits.

Because the water levels are important for the design of the system and not the discharges, the second method is chosen. Therefore all catchments were measured again from maps.

The roughness of the conduits is accepted to be 5mm. The conduits are made of asphalt, but it is likely that sand, small stones and litter will be on them. The rest of the town is mostly unpaved, so the over land flow goes over sand etc, consequently a bigger roughness coefficient has been chosen. For over land flow a value of 10 mm has been accepted.

The runoff coefficient is the ratio between the amount of runoff from a certain area and the amount of rain on that area. During a storm the rain will be retained, infiltrate and evaporate, so the coefficient will be smaller than 1. The coefficient is found after determing the catchments and the land use. For the rainfall conditions with a return period of 2 years, the coefficient is accepted to be:

Land use	runoff coefficient
Residential area:	
-low density, modern set-up	0.1 - 0.3
-medium density	0.3 - 0.5
-high density (traditional built up areas)	0.6 - 0.8
Commercial area (suq)	0.7 - 0.9
Industrial area	0.3 - 0.7
Paved areas (asphalt roads)	1

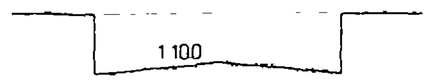
Table A1: Runof coefficients[2].

The walls surrounding the premises prevent the water to contribute to the runoff. The retention is therefore the most important factor for the runoff coefficient.

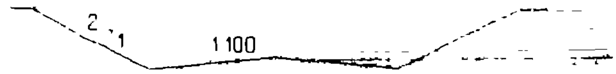
(2) Main Report, Final Design Report 1989, page 4-8.

Cross section.

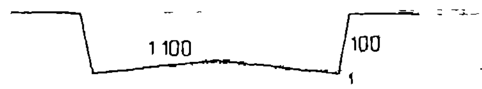
The cross sections of most of the roads will be like this: (I)



In front of shops and at parking places, the side of the road will have a different kerbstone: (II)



In the computer model cross sections are schematised as follows: (III)



This schematisation does have some impact on the calculations. If Chézy is accepted (see annexes B), then it can be seen that the important factors are the wet cross sectional area A and the hydraulic radius R, which has an impact on the coefficient of Chézy C. The hydraulic radius R = A/O with O is the wet perimeter.

For the width b = 6 m and side slope i_s = 1:100 the variablés A, O and R are computed for a side depth h = 0.3 m and h = 0.5 m.

		h = 0.3 m	h = 0.5 m
I:	$O \approx 2h + b$	6.60 m	7.0 m
		R = 0.259 m	R = 0.416 m
	$A = b*h - \frac{1}{2}bi_s\frac{1}{2}b$	1.71 m^2	2.91 m^2
II:	$O \approx 2h!5 + b$	7.34 m	8.23 m
		R = 0.257 m	R = 0.414 m
	$A = b*h - \frac{1}{2}bi_s\frac{1}{2}b + 2h^2$	1.89 m^2	3.41 m^2
II:	$O \approx 2h + b$	6.60 m	7.00 m
		R = 0.273 m	R = 0.429 m
	$A = b*h + h^2:100$	1.80 m^2	3.00 m^2

The hydrualic radius seems to be pretty constant for the different cross sections. On the coefficient of Chézy, which is a logaritmic funtion of R, the influence of the schematisation is therefore negligible.
The difference in wet (cross sectional) area between I (most common cross section) and the III (schematisation) is not very big. However, for II this schematisation introduces an underestimation of the wet area. For h = 0.5 m the wet area for cross section II is even 14% larger than for cross section III.
Assume that the discharge is equal (for II and III) if the wet area A is equal. To obtain a wet area of 3.0 m^2 for II the side depth would be 0.45 m while for III this would be 0.5 m.
If the outcome of the computer model gives a side depth of 0.5 m, the side depth for roads with cross section II will therefore be only 0.45 cm (10% smaller).

D.5. CYLCONE MODEL.

Figure D1: Node Numbers.

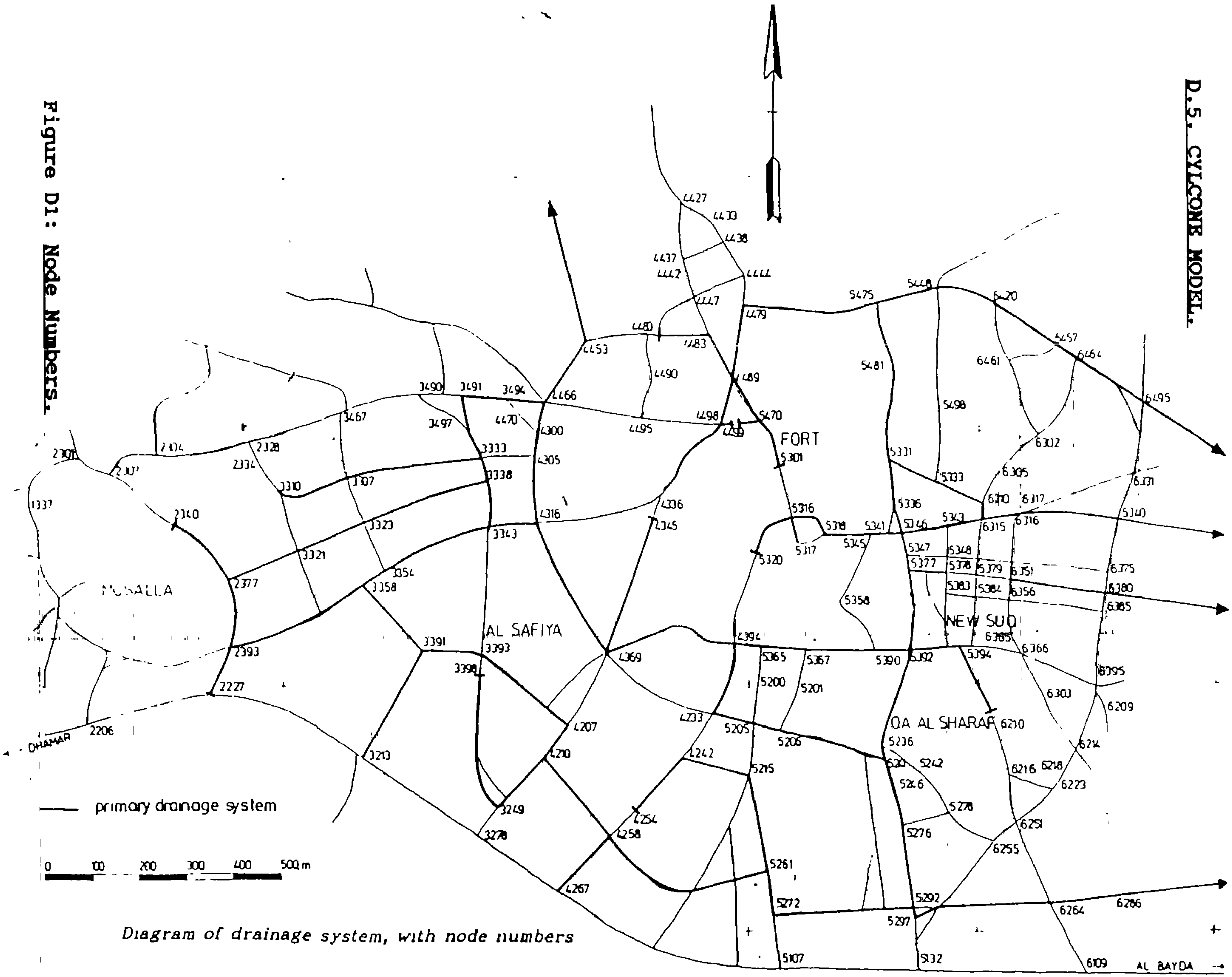


Diagram of drainage system, with node numbers

Middle section.

```
:YCLONE,  D H V  CONSULTANTS, AMERSFOORT, THE NETHERLANDS                                        DATE     6-24-1991          PAGE
***** L I S T   O F   T H E   I N P U T C A R D S *****

CARD COLUMN  10   15   20   25   30   35   40   45   50   55   60   65   70   75   80 * ERROR MESSAGES
NUMB          |    |    |    |    |    |    |    |    |    |    |    |    |    |    |
   1 *TEXT     RWSSP        RADA      YEMEN                                             *
   6 *GENERAL  0       50   -50   100  -100  50   30                                    *
   7 *CALC    25    1    2     1     5     0                                            *
   8 *NODE       0 4207 3180    0         0    0             1  30              1       *
   9 *NODE       0 4316 3331    0         0    0             1  40              1       *
  10 *NODE       0 4345 3160    0         0    0             4  40              1       *
  11 *NODE       0 4369 3130    0         0    0            72  40              1       *
  12 *NODE       0 4394 2961    0         0    0            48  50              1       *
  13 *NODE       0 5206 2740    0         0    0             7  40              1       *
  14 *NODE       0 5241 2680    0         0    0            33  40              1       *
  15 *NODE       0 5316 2697    0         0    0            35  60              1       *
  16 *NODE       0 5317 2785    0         0    0             6  60              1       *
  17 *NODE       0 5318 2646    0         0    0            13  60              1       *
  18 *NODE       0 5320 3000    0         0    0            14  60              1       *
  19 *NODE       0 5331 2470    0         0    0            33  60              1       *
  20 *NODE       0 5333 2450    0         0    0            30  60              1       *
  21 *NODE       0 5343 2450    0         0    0             8  50              1       *
  22 *NODE       0 5345 2562    0         0    0            21  60              1       *
  23 *NODE       0 5346 2490    0         0    0            20  60              1       *
  24 *NODE       0 5367 2675    0         0    0            25  50              1       *
  25 *NODE       0 5377 2503    0         0    0            22  60              1       *
  26 *NODE       0 5378 2650    0         0    0             5  60              1       *
  27 *NODE       0 5390 2557    0         0    0            45  60              1       *
  28 *NODE       0 5392 2530    0         0    0            30  60              1       *
  29 *NODE       0 5394 2568    0         0    0            17  60              1       *
  30 *NODE       0 6210 2796    0         0    0             4  60              1       *
  31 *NODE       0 6310 2438    0         0    0            12  60              1       *
  32 *NODE       0 6315 2431    0         0    0            13  60              1       *
  33 *NODE       0 6316 2426    0         0    0            10  40              1       *
  34 *NODE       0 6351 2522    0         0    0             7  60              1       *
  35 *NODE       0 6339 2374    0         0    0            25  40              1       *
  36 *NODE       0 6379 2395    0         0    0            15  60              1       *
  37 *ENODE        4207   20  100  100   30                                             *
  38 *ENODE        4316   20  100  100   30                                             *
  39 *ENODE        4345  120  150  100   30                                             *
  40 *ENODE        4369  320  120  100   30                                             *
  41 *ENODE        4394  280  140  100   30                                             *
  42 *ENODE        5206  100   50  100   30                                             *
  43 *ENODE        5241  250  140  100   30                                             *
  44 *ENODE        5316  300   60  100   30                                             *
  45 *ENODE        5317  120  100  100   30                                             *
  46 *ENODE        5318  180   10  100   30                                             *
  47 *ENODE        5320  240  120  100   30                                             *
  48 *ENODE        5331  220   90  100   30                                             *
  49 *ENODE        5333  180   90  100   30                                             *
  50 *ENODE        5343  100   80  100   30                                             *
  51 *ENODE        5345  200   40  100   30                                             *
  52 *ENODE        5346  200  150  100   30                                             *
  53 *ENODE        5367  200   60  100   30                                             *
  54 *ENODE        5377  150  100  100   30                                             *
  55 *ENODE        5378  100  200  100   30                                             *
  56 *ENODE        5390  260   80  100   30                                             *
  57 *ENODE        5392  300  140  100   30                                             *
  58 *ENODE        5394  180  100  100   30                                             *
  59 *ENODE        6210  100   40  100   30                                             *
  60 *ENODE        6310  240  100  100   30                                             *
  61 *ENODE        6315  120   60  100   30                                             *
  62 *ENODE        6316  150   50  100   30                                             *
  63 *ENODE        6351  100   90  100   30                                             *
  64 *ENODE        6339  300  230  100   30                                             *
  65 *ENODE        6379  300  230  100   30                                             *
  66 *SPILLWAY     6340 2373    0         0      0 2000  2349185                          *
  67 *SPILLWAY     6380 2394    0         0      0 2000  2370185                          *
  68 *TRAPPROF       21 4316 4369    0  600    1    1 3316 3115   50       280  100       *
  69 *TRAPPROF       40 4207 4369    0  600    1    1 3165 3115   50       150  100       *
  70 *TRAPPROF       53 4345 4369    0  600    1    1 3145 3115   50       270  100       *
  71 *TRAPPROF       55 4369 4394    0  600    1    1 3115 2946   50       315  100       *
  72 *TRAPPROF       57 4394 5367    0  600    1    1 2946 2660   50       155  100       *
  73 *TRAPPROF       80 5206 5241    0  600    1    1 2725 2665   50       220  100       *
  74 *TRAPPROF       85 5241 5392    0  600    1    1 2665 2505   50       230  100       *
  75 *TRAPPROF       98 5320 5316    0  600    1    1 2985 2682   50       105  100       *
  76 *TRAPPROF       99 5316 5317    0  600    1    1 2682 2770   50        80  100       *
  77 *TRAPPROF      100 5316 5318    0  600    1    1 2682 2631   50        80  100       *
  78 *TRAPPROF      101 5318 5345    0  600    1    1 2631 2547   50       120  100       *
  79 *TRAPPROF      102 5331 5333    0  600    1    1 2455 2425   50       125  100       *
  80 *TRAPPROF      106 5333 6310    0  600    1    1 2425 2410   50       100  100       *
  81 *TRAPPROF      108 5343 5346    0  600    1    1 2425 2450   50       110  100       *
  82 *TRAPPROF      110 5343 6315    0  600    1    1 2425 2381   50        50  100       *
  83 *TRAPPROF      111 5345 5346    0  600    1    1 2547 2450   50        80  100       *
  84 *TRAPPROF      112 5346 5377    0  600    1    1 2450 2463   50       100  100       *
  85 *TRAPPROF      115 5367 5390    0  500    1    1 2660 2532   50       155  100       *
  86 *TRAPPROF      116 5377 5378    0  500    1    1 2463 2635   50        90  100       *
  87 *TRAPPROF      117 5377 5392    0  600    1    1 2463 2505   50       160  100       *
  88 *TRAPPROF      119 5390 5392    0  600    1    1 2532 2505   50        90  100       *
  89 *TRAPPROF      120 5392 5394    0  600    1    1 2505 2553   50       110  100       *
  90 *TRAPPROF      121 5394 6210    0  600    1    1 2553 2781   50       290  100       *
  91 *TRAPPROF      145 6310 6315    0  600    1    1 2410 2381   50        40  100       *
  92 *TRAPPROF      146 6315 6316    0  600    1    1 2381 2376   50        75  100       *
  93 *TRAPPROF      147 6316 6339    0  800    1    1 2376 2349   50       220  100       *
  94 *TRAPPROF      152 6339 6379    0  600    1    1 2349 2370   50       130  100       *
  95 *TRAPPROF      154 6351 6379    0  600    1    1 2507 2370   50       205  100       *
  96 *TRAPPROF      155 6339 6340    0  600    1    1 2349 2348   50         1  100       *
  97 *TRAPPROF      156 6379 6380    0  600    1    1 2370 2369   50         1  100       *
  98 *OUCONDUIT      57  115  119   85  120  117  112  111  108  110  146                *
  99 *OUCONDUIT     147                                                                   *
 100 *RAIN      1    1    1    0   85   15   85   16  187   30  187   31  656   45  656 *
 101 *RAIN      1    1    2   46  320   60  320   61  122   75  122   76   63   90   63 *
 102 *RAIN      1    1    3   91    0                                                    *
 103 *END                                                                               *
```

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 ***** INPUT *****
 *****************

 G E N E R A L   D A T A
 ***********************

 DIFFUSIVE WAVE EQUATIONS
 IMPLICIT METHOD

 LENGTH OF A COORDINATE-UNIT     (M) :          1.000

 ROUGHNESS ACCORDING TO
 NIKURADSE (M)
 FOR OPEN CONDUITS                    :          .0050
 FOR CLOSED CONDUITS                  :          .0050
 EFFECTIVE CROSS-SECTIONAL AREA       :          1.000
 RUN-OFF COEFFICIENT (DEFAULT)        :           .250

 TOTAL CALCULATION TIME        (HOUR) :        2
 TIME STEP (CALCULATION)        (MIN) :        1
 TIME STEP (OUTPUT)             (MIN) :        5

 NUMBER OF TRAPEZIUM PROFILES         :       30
 NUMBER OF VARIABLE PROFILES          :        0
 NUMBER OF CLOSED CONDUITS            :        0
 NUMBER OF WEIRS                      :        0
 NUMBER OF NODES                      :       29
 NUMBER OF SPILLWAYS                  :        2
 NUMBER OF PUMPS                      :        0
 NUMBER OF INFLOW HYDROGRAPHS         :        0
 NUMBER OF RAIN CURVES                :        1
 NUMBER OF EVAPORATION CURVES         :        0
 NUMBER OF STORAGE CURVES             :        0
 NUMBER OF PROFILE CURVES             :        0
 NUMBER OF WATER LEVEL CURVES         :        0
 NUMBER OF MOVABLE WEIR CURVES        :        0
```

N O D E   D A T A

CATCHMENT AREA (CODE 1) 0- = HA   PUMP CAP. (CODE 2) 0- = M3/S
1- = M2   1- = L/S

NODE NUMBER	GROUND LEVEL (M)	WATER LEVEL (M)	X-COORD (M)	Y-COORD (M)	CATCHMENT AREA (CODE 1)	RUN OFF CFF	CURVE NUMBERS INFL	STORGE	EVAP	RAIN	WATLEV	CAP (CODE 2)	PUMP START (M)	STOP (M)	SPILL WIDTH (M)	LEV
4207	31.80	.00	0.	0.	1.0 0-	.30				1						
4316	33.31	.00	0.	0.	1.0 0-	.40				1						
4345	31.60	.00	0.	0.	.4 0-	.40				1						
4369	31.30	.00	0.	0.	7.2 0-	.40				1						
4394	29.61	.00	0.	0.	4.8 0-	.50				1						
5206	27.40	00	0.	0.	.7 0-	.40				1						
5241	26.80	00	0.	0.	3.3 0-	.40				1						
5316	26.97	.00	0.	0.	3.5 0-	.60				1						
5317	27.85	.00	0.	0.	.6 0-	.60				1						
5318	26.46	.00	0.	0.	1.3 0-	.60				1						
5320	30.00	.00	0.	0.	1.4 0-	.60				1						
5331	24 70	.00	0.	0.	3.3 0-	.60				1						
5333	24.50	.00	0.	0.	3.0 0-	.60				1						
5343	24.50	.00	0.	0.	.8 0-	.50				1						
5345	25.62	.00	0.	0.	2.1 0-	.60				1						
5346	24.90	.00	0.	0.	2.0 0-	.60				1						
5367	26.75	.00	0.	0.	2.5 0-	.50				1						
5377	25.03	.00	0.	0.	2.2 0-	.60				1						
5378	26.50	.00	0.	0.	.5 0-	.60				1						
5390	25.57	.00	0.	0.	4.5 0-	.60				1						
5392	25.30	.00	0	0.	3.0 0-	.60				1						
5394	25 68	.00	0.	0.	1.7 0-	.60				1						
6210	27.96	.00	0.	0.	.4 0-	.60				1						
6310	24.35	.00	0.	0.	1.2 0-	.60				1						
6315	24.31	.00	0.	0.	1.3 0-	.60				1						
6316	24.26	.00	0.	0.	1.0 0-	.40				1						
6351	25.22	.00	0.	0.	.7 0-	.60				1						
6339	23.74	.00	0.	0.	2.5 0-	.40				1						
6379	23.95	.00	0.	0.	1.5 0-	.60				1						
6340	23.73	.00	0.	0.											20.00	23
6380	23.94	.00	0.	0.											20.00	23.

C O N D U I T   D A T A

CONDT NUMBER	NODE NUMBER BEGIN	END	INVERT LEVEL BEGIN (M)	END (M)	LENGTH (M)	ROUGHNESS (*)	EFF. CROSS COEFF	BOTTOM WIDTH (M)	CATCHM. WIDTH (M)	SIDE SLOPES COTGA	COTGB	PROFILE CURVE NUMBER	CLOSED CONDUIT - SHAPE	HEIGHT (M)	WIDTH (M)	CHARACTE LOSSE BEGIN	E
21	4316	4369	33.16	31.15	280	.0050	1.00	6.00	0	.01	.01						
40	4207	4369	31.65	31.15	150	.0050	1.00	6.00	0	.01	.01						
53	4345	4369	31.45	31.15	270	.0050	1.00	6.00	0	.01	.01						
55	4369	4394	31.15	29.46	315	.0050	1.00	6 00	0	.01	.01						
57	4394	5367	29.46	26.60	155	.0050	1.00	6.00	0	.01	.01						
80	5206	5241	27.25	26.65	220	.0050	1.00	6.00	0	.01	.01						
85	5241	5392	26.65	25.05	230	.0050	1.00	6.00	0	.01	.01						
98	5320	5316	29.85	26.82	105	.0050	1.00	6.00	0	.01	.01						
99	5316	5317	26.82	27.70	80	.0050	1.00	6.00	0	.01	.01						
100	5316	5318	26.82	26.31	80	.0050	1.00	6.00	0	.01	.01						
101	5318	5345	26.31	25.47	120	.0050	1.00	6.00	0	.01	.01						
102	5331	5333	24.58	24.25	125	.0050	1.00	6.00	0	.01	.01						
106	5333	6310	24.25	24.10	100	0050	1.00	6.00	0	.01	.01						
108	5343	5346	24.25	24.50	110	.0050	1 00	6.00	0	.01	.01						
110	5343	6315	24.25	23.81	50	0050	1.00	6.00	0	.01	.01						
111	5345	5346	25.47	24.50	80	.0050	1 00	6 00	0	.01	.01						
112	5346	5377	24.50	24.63	100	.0050	1 00	6.00	0	.01	.01						
115	5367	5390	26.60	25.32	155	.0050	1.00	5.00	0	.01	.01						
116	5377	5378	24.63	26 35	90	.0050	1.00	5.00	0	.01	.01						
117	5377	5392	24.63	25.05	160	.0050	1.00	6.00	0	.01	.01						
119	5390	5392	25.32	25.05	90	.0050	1.00	6.00	0	.01	.01						
120	5392	5394	25.05	25.53	110	.0050	1 00	6.00	0	.01	.01						
121	5394	6210	25.53	27.81	290	.0050	1.00	6.00	0	.01	.01						
145	6310	6315	24.10	23.81	40	.0050	1.00	6.00	0	.01	.01						
146	6315	6316	23.81	23.76	75	.0050	1.00	6.00	0	.01	.01						
147	6316	6339	23.76	23.49	220	.0050	1.00	6 00	0	.01	.01						
152	6339	6379	23.49	23.70	130	.0050	1.00	6.00	0	.01	.01						
154	6351	6379	25.07	23.70	205	.0050	1.00	6.00	0	.01	.01						
155	6339	6340	23.49	23.48	1	.0050	1.00	6.00	0	.01	.01						
156	6379	6380	23.70	23.69	1	.0050	1.00	6.00	0	.01	.01						

* ROUGHNESS ACCORDING TO NIKURADSE (M)

R A I N   C U R V E S

RAIN CURVE												
	TIME (MIN)	0	15	16	30	31	45	46	60	61	75
1	INTENS. (MM/H)	8.5	8.5	18.7	18.7	65.6	65.6	32.0	32.0	12.2	12.2
	TIME (MIN)	91									
	INTENS. (MM/H)	.0									

M A X I M U M   W A T E R   L E V E L S   A N D   D I S C H A R G E S
****

CONDT NUMBR	NODE NMBR	SURFACE LEVEL (M)	WATER LEVEL (M)	INVERT LEVEL (M)	CONDT DEPTH (M)	FILLING (%) *\#	TIME (MIN)	TIME (MIN)	DISCHARGE (M3/S)	VELO-CITY (M/S)	GRADIENTS WATER (1:X)	GRADIENTS INVERT (1:X)	BOTTOM WIDTH (M)	REMARKS
OPEN CONDUITS WITH TRAPEZIUM PROFILES :														
21	4316	33.31	33.16	33.16	.15	1	46	46	.007	.04	144	139	6.00	
	4369	31.30	31.23	31.15	.15	51	63							
40	4207	31.80	31.65	31.65	.15	3	45	46	.005	.03	350	300	6.00	
	4369	31.30	31.23	31.15	.16	51	63							
53	4345	31.60	31.46	31.45	.15	10	50	51	.021	.09	1135	899	6.00	
	4369	31.30	31.23	31.15	.15	51	63							
55	4369	31.30	31.23	31.15	.15	51	63	62	.283	.73	183	186	6.00	
	4394	29.61	29.51	29.46	.15	35	61							
57	4394	29.61	29.51	29.46	.15	35	61	61	.518	1.27	54	54	6.00	
	5367	26.75	26.68	26.60	.15	55	61							
80	5206	27.40	27.28	27.25	.15	17	47	47	.046	.28	365	366	6.00	
	5241	26.80	26.68	26.65	.15	20	48							
85	5241	26.80	26.68	26.65	.15	20	48	48	.208	.31	161	143	6.00	
	5392	25.30	25.25	25.05	.25	81 *	52							
98	5320	30.00	29.86	29.85	.15	6	72	46	-.118	.53	35	34	6.00	
	5316	26.97	26.89	26.82	.15	46	47							
99	5316	26.97	26.89	26.82	.15	46	47	46	-.060	.25	97	90	6.00	
	5317	27.85	27.71	27.70	.15	8	51							
100	5316	26.97	26.89	26.82	.15	46	47	47	.461	.82	173	156	6.00	
	5318	26.46	26.43	26.31	.15	78	47							
101	5318	26.46	26.43	26.31	.15	78	47	47	576	1.09	133	142	6.00	
	5345	25.62	25.53	25.47	.15	39	47							
102	5331	24.70	24.61	24.55	.15	43	47	47	.283	.44	592	416	6.00	
	5333	24.50	24.40	24.25	.25	61	46							
106	5333	24.50	24.40	24.25	.25	61	46	48	.581	.67	790	666	6.00	
	6310	24.35	24.28	24.10	.25	70	54							
108	5343	24.50	24.45	24.25	.25	79	54	53	-2.450	1.51	275	440	6.00	
	5346	24.90	24.85	24.50	.40	86 *	53							
110	5343	24.50	24.45	24.25	.25	79	54	54	2.488	1.27	279	113	6.00	
	6315	24.31	24.27	23.81	.50	92 **	54							
111	5345	25.62	25.53	25.47	.15	39	47	47	.772	70	117	82	6.00	
	5346	24.90	24.85	24.50	.40	86 *	53							
112	5346	24.90	24.85	24.50	.40	86 *	53	54	-1.684	.86	1062	769	6.00	
	5377	25.03	24.94	24.63	.40	77	54							
115	5367	26.75	26.68	26.60	.15	55	61	58	.641	1.12	127	121	5.00	
	5390	25.57	25.47	25.32	.25	60	49							
116	5377	25.03	24.94	24.63	.40	77	54	46	-.050	08	63	52	5.00	
	5378	26.50	26.36	26.35	.15	5	39							
117	5377	25.03	24.94	24.63	.40	77	54	52	-1.515	.99	512	380	6.00	
	5392	25.30	25.25	25.05	.25	81 *	52							
119	5390	25.57	25.47	25.32	.25	60	49	50	969	.92	410	333	6.00	
	5392	25.30	25.25	25.05	.25	81 *	52							
120	5392	25.30	25.25	25.05	.25	81 *	52	47	-.186	.28	350	229	6.00	
	5394	25.68	25.57	25.53	.15	24	48							
121	5394	25.68	25.57	25.53	.15	24	48	46	-.042	.29	128	127	6.00	
	6210	27.96	27.82	27.81	.15	9	46							
145	6310	24.35	24.28	24.10	.25	70	54	48	.577	.35	4655	137	6.00	
	6315	24.31	24.27	23.81	.50	92 **	54							
146	6315	24.31	24.27	23.81	.50	92 **	54	54	3.103	1.19	755	1500	6.00	
	6316	24.26	24.17	23.76	.50	82 *	55							
147	6316	24.26	24.17	23.76	.50	82 *	55	55	3.139	1.26	472	814	8.00	
	6339	23.74	23.70	23.49	.25	85 *	56							
152	6339	23.74	23.70	23.49	.25	85 *	56	46	-.061	.12	4805	619	6.00	
	6379	23.95	23.73	23.70	.25	12	52							
154	6361	25.22	25.10	25.07	.15	18	46	47	.074	.45	149	149	6.00	
	6379	23.95	23.73	23.70	.25	12	52							
155	6339	23.74	23.70	23.49	.25	85 *	56	56	3.270	2.59	68	99	6.00	
	6340	23.73	23.69	23.48	.25	83 *	56							
156	6379	23.95	23.73	23.70	.25	12	52	52	.105	.58	98	99	6.00	
	6380	23.94	23.72	23.69	.25	12	52							

FREEBOARD CODE OPEN CONDUITS (% OF DEPTH): *** 0% - 5%; ** 5% - 10%; * 10% - 20%

FILLING OF CLOSED CONDUITS (% OF DEPTH\DIAM.): ### 95% - 100%; ## 90% - 95%; # 80% - 90%

POSITIVE DISCHARGE FROM LOW TO HIGH NODE NUMBER

TOTAL CATCHMENT AREA 57.6000 HA
THE BANDWIDTH OF THE SYMMETRICAL MATRIX IS 6 POSITIONS

S P I L L E D   V O L U M E S
****

NODE NUMBER	VOLUME (M3)	SPILLING TIME (MIN)
6340	8331	92
6380	247	92

W A T E R   B A L A N C E   I N   (M3)
****

INFLOW (HYDROGRAPHS)	:	0
INFLOW (FIXED WATER LEVELS)		0
RAINFALL	:	11035
TOTAL IN		11035
OUTFLOW (SPILLWAYS)		8578
OUTFLOW (PUMPS)		0
OUTFLOW (FIXED WATER LEVELS)		0
EVAPORATION		0
TOTAL OUT		8578
STORAGE CHANGE		2470
TOTAL OUT + STORAGE CHANGE		11049
BALANCE ERROR		.13 %

****** E N D   O F   C Y C L O N E ******

North-Eastern section.

```
 1 *TEXT      RWSSP        RADA        YEMEN                                                   *
 6 *GENERAL   0          50     50    100  100    50    30                                     *
 7 *CALC     25     1     2      1      5     0                                                *
 8 *NODE      0 4336   3151      0          0          0           30   60          1          *
 9 *NODE      0 4479   2925      0          0          0           25   70          1          *
10 *NODE      0 4483   3035      0          0          0           24   60          1          *
11 *NODE      0 4489   2945      0          0          0           27   60          1          *
12 *NODE      0 4495   3050      0          0          0           26   60          1          *
13 *NODE      0 4498   2962      0          0          0           45   70          1          *
14 *NODE      0 4499   2970      0          0          0            1   70          1          *
15 *NODE      0 5301   2980      0          0          0           10   60          1          *
16 *NODE      0 5445   2545      0          0          0           54   60          1          *
17 *NODE      0 5448   2506      0          0          0           73   30          1          *
18 *NODE      0 5470   2966      0          0          0            4   60          1          *
19 *NODE      0 5498   2616      0          0          0            4   60          1          *
20 *NODE      0 6420   2503      0          0          0           23   30          1          *
21 *NODE      0 6464   2419      0          0          0           42   40          1          *
22 *NODE      0 6494   2356      0          0          0           70   20          1          *
23 *ENODE       4336    280    150    100    30                                                *
24 *ENODE       4479    160    140    100    30                                                *
25 *ENODE       4483    240    110    100    30                                                *
26 *ENODE       4489    150    100    100    30                                                *
27 *ENODE       4495    200     80    100    30                                                *
28 *ENODE       4498    170     80    100    30                                                *
29 *ENODE       4499     10     80    100    30                                                *
30 *ENODE       5301    100    200    100    30                                                *
31 *ENODE       5445    300     60    100    30                                                *
32 *ENODE       5448    280     40    100    30                                                *
33 *ENODE       5470    100      5    100    30                                                *
34 *ENODE       5498     60    120    100    30                                                *
35 *ENODE       6420    140    100    100    30                                                *
36 *ENODE       6464    240    110    100    30                                                *
37 *ENODE       6494    300    130    100    30                                                *
38 *SPILLWAY    6495   2358      0            0         0 10000  2341185                         *
39 *TRAPPROF     10 4336 4498     0   600     1     1 3136 2937    50        205   100          *
40 *TRAPPROF     20 4479 4489     0   600     1     1 2860 2910    50        140   100          *
41 *TRAPPROF     30 4479 5445     0   600     1     1 2860 2510    50        280   100          *
42 *TRAPPROF     35 4483 4489     0   600     1     1 3020 2910    50        110   100          *
43 *TRAPPROF     40 4489 4498     0   600     1     1 2910 2937    50        150   100          *
44 *TRAPPROF     50 4489 5470     0   600     1     1 2910 2951    50         80   100          *
45 *TRAPPROF     65 4495 4498     0   600     1     1 3035 2937    50        170   100          *
46 *TRAPPROF     60 4498 4499     0   600     1     1 2937 2955    50         40   100          *
47 *TRAPPROF     70 4499 5470     0   600     1     1 2955 2951    50         40   100          *
48 *TRAPPROF     80 5301 5470     0   600     1     1 2965 2951    50         65   100          *
49 *TRAPPROF     90 5445 5448     0   600     1     1 2510 2471    50        120   100          *
50 *TRAPPROF    100 5448 5498     0   500     1     1 2471 2601    50        260   100          *
51 *TRAPPROF    110 5448 6420     0   800     1     1 2471 2463    50         90   100          *
52 *TRAPPROF    120 6420 6464     0   800     1     1 2463 2379    50        245   100          *
53 *TRAPPROF    130 6464 6494     0   800     1     1 2379 2341    50        160   100          *
54 *TRAPPROF    140 6494 6495     0   800     1     1 2341 2340    50          1   100          *
55 *OUNODE      4336   4479   4483   4489   4495   4498   4499   5301   5445   5448   5470       *
56 *OUNODE      5498   6420   6464   6495                                                     *
57 *OUCONDUIT     10     20     30     35     40     50     55     60     70     80     90       *
58 *OUCONDUIT    100    110    120    130                                                      *
59 *RAIN      1    1    1     0    85    15    85    16   187    30   187    31   656   45  656  *
60 *RAIN      1    1    2    46   320    60   320    61   122    75   122    76    63   90   63  *
61 *RAIN      1    1    3    91     0                                                          *
62 *END                                                                                         *
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DIFFUSIVE WAVE EQUATIONS
IMPLICIT METHOD

LENGTH OF A COORDINATE-UNIT      (M) :       1.000

ROUGHNESS ACCORDING TO
NIKURADSE (M)
FOR OPEN CONDUITS                    :        .0050
FOR CLOSED CONDUITS                  :        .0050
EFFECTIVE CROSS-SECTIONAL AREA       :       1.000
RUN-OFF COEFFICIENT (DEFAULT)        :         250

TOTAL CALCULATION TIME       (HOUR)  :       2
TIME STEP (CALCULATION)       (MIN)  :       1
TIME STEP (OUTPUT)            (MIN)  :       5

NUMBER OF TRAPEZIUM PROFILES         :      16
NUMBER OF VARIABLE PROFILES          :       0
NUMBER OF CLOSED CONDUITS            :       0
NUMBER OF WEIRS                      :       0
NUMBER OF NODES                      :      15
NUMBER OF SPILLWAYS                  :       1
NUMBER OF PUMPS                      :       0
NUMBER OF INFLOW HYDROGRAPHS         :       0
NUMBER OF RAIN CURVES                :       1
NUMBER OF EVAPORATION CURVES         :       0
NUMBER OF STORAGE CURVES             :       0
NUMBER OF PROFILE CURVES             :       0
NUMBER OF WATER LEVEL CURVES         :       0
NUMBER OF MOVABLE WEIR CURVES        :       0
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CYCLONE, D H V CONSULTANTS, AMERSFOORT, THE NETHERLANDS DATE · 6-20-1991 PAGE 4

***** INPUT *****

N O D E   D A T A

CATCHMENT AREA (CODE 1) 0- = HA 1- = M2   PUMP CAP. (CODE 2) 0- = M3/S 1- = L/S

NODE NUMBER	GROUND LEVEL (M)	WATER LEVEL (M)	X-COORD (M)	Y-COORD (M)	CATCHMENT AREA (CODE 1)	RUN OFF CFF	C U R V E N U M B E R S: INFL	STORGE	EVAP	RAIN	WATLEV	CAP (CODE 2)	P U M P: START (M)	STOP (M)	S P I L L W A Y: WIDTH (M)	LEVEL (M)	COEF
4336	31.51	00	0.	0	3.0 0-	.60				1							
4479	29.25	.00	0	0.	2 5 0-	.70				1							
4483	30.35	.00	0.	0.	2.4 0-	.60				1							
4489	29.45	.00	0.	0	2.7 0-	.60				1							
4495	30.50	.00	0.	0.	2.6 0-	.60				1							
4498	29 62	.00	0.	0.	4.5 0-	.70				1							
4499	29.70	.00	0.	0.	.1 0-	.70				1							
5301	29.80	.00	0.	0.	1.0 0-	.60				1							
5445	25.45	00	0.	0.	5.4 0-	.60				1							
5448	25 06	.00	0.	0	7.3 0-	30				1							
5470	29.66	.00	0	0.	.4 0-	.60				1							
5498	26.16	.00	0.	0.	.4 0-	.60				1							
6420	25.03	.00	0.	0.	2 3 0-	.30				1							
6464	24 19	.00	0.	0.	4.2 0-	.40				1							
6494	23 56	.00	0.	0.	7.0 0-	.20				1							
6495	23.55	.00	0.	0.											100.00	23.41	1.85

***** INPUT *****

C O N D U I T   D A T A

CONDT NUMBER	NODE NUMBER BEGIN	END	INVERT LEVEL BEGIN (M)	END (M)	LENGTH (M)	ROUGH NESS (*)	EFF CROSS COEFF	BOTTOM WIDTH (M)	CATCHM. WIDTH (M)	SIDE SLOPES COTGA	COTGB	PROFILE CURVE NUMBER	CLOSED CONDUIT - CHARACTERISTICS: SHAPE	HEIGHT (M)	WIDTH (M)	LOSSES BEGIN	END	SIL
10	4336	4498	31.36	29.37	205	.0050	1.00	6.00	0	.01	.01							
20	4479	4489	28.60	29.10	140	.0050	1.00	6.00	0	.01	.01							
30	4479	5445	28.60	25.10	280	.0050	1.00	6.00	0	.01	.01							
35	4483	4489	30.20	29.10	110	.0050	1.00	6.00	0	01	.01							
40	4489	4498	29.10	29.37	150	.0050	1 00	6.00	0	.01	01							
50	4489	5470	29.10	29.51	80	.0050	1.00	6.00	0	.01	01							
55	4495	4498	30.35	29.37	170	.0050	1.00	6.00	0	.01	.01							
60	4498	4499	29.37	29.55	40	0050	1.00	6.00	0	.01	.01							
70	4499	5470	29.55	29.51	40	.0050	1.00	6.00	0	.01	.01							
80	5301	5470	29.65	29.51	65	.0050	1.00	6.00	0	.01	.01							
90	5445	5448	25.10	24.71	120	.0050	1.00	6.00	0	.01	.01							
100	5448	5498	24.71	26.01	260	.0050	1.00	5.00	0	.01	.01							
110	5448	6420	24.71	24.63	90	.0050	1.00	8.00	0	.01	.01							
120	6420	6464	24.63	23.79	245	.0050	1.00	8.00	0	.01	.01							
130	6464	6494	23.79	23.41	160	.0050	1.00	8.00	0	.01	.01							
140	6494	6495	23.41	23.40	1	0050	1.00	8.00	0	.01	.01							

* ROUGHNESS ACCORDING TO NIKURADSE (M)

R A I N   C U R V E S

RAIN CURVE													
RAIN	TIME (MIN)	0	15	16	30	31	45	46	60	61	75	76	[illegible]
CURVE 1	INTENS. (MM/H)	8 5	8.5	18.7	18.7	65.6	65.6	32.0	32 0	12 2	12.2	6.3	6.
	TIME (MIN)	91											
	INTENS. (MM/H)	.0											

M A X I M U M   W A T E R   L E V E L S   A N D   D I S C H A R G E S
****

CONDT NUMBR	NODE NMBR	SURFACE LEVEL (M)	WATER LEVEL (M)	INVERT LEVEL (M)	CONDT DEPTH (M)	FILLING (%) *\#	TIME (MIN)	TIME (MIN)	DISCHARGE (M3/S)	VELO-CITY (M/S)	GRADIENTS WATER (1 X)	GRADIENTS INVERT (1 X)	BOTTOM WIDTH (M)	REMARKS
OPEN CONDUITS WITH TRAPEZIUM PROFILES :														
10	4336	31.51	31.39	31.36	.15	18	49	47	213	.38	110	103	6.00	
	4498	29.62	29.53	29.37	.25	66	49							
20	4479	29.25	28.70	28 60	65	16	51	51	-1.325	1.28	218	280	6.00	
	4489	29.45	29.34	29.10	.35	70	51							
30	4479	29.25	28.70	28.60	.65	16	51	51	1.543	1.57	82	80	6.00	
	5445	25.45	25.33	25.10	.35	64	52							
35	4483	30.35	30.22	30.20	.15	11	40	46	.203	.28	125	99	6.00	
	4489	29.45	29.34	29 10	.35	70	51							
40	4489	29.45	29.34	29.10	.35	70	51	49	-.854	.70	787	555	6.00	
	4498	29.62	29.53	29 37	.25	66	49							
50	4489	29.45	29.34	29.10	.35	70	51	46	-.137	.18	409	195	6.00	
	5470	29.66	29.54	29.51	15	20	47							
55	4495	30.50	30.39	30.35	.15	24	47	46	.237	.41	199	173	6.00	
	4498	29.62	29.83	29.37	.25	66	49							
60	4498	29.62	29.53	29.37	.25	66	49	37	-.009	.04	1768	222	6.00	
	4499	29.70	29.56	29.55	.15	5	46							
70	4499	29.70	29.56	29.55	.15	5	46	46	.004	.04	2254	1000	6.00	
	5470	29.66	29.54	29.51	.15	20	47							
80	5301	29.80	29.71	29.65	.15	37	46	46	.096	.38	391	464	6.00	
	5470	29.66	29.54	29.51	.15	20	47							
90	5445	25.45	25.33	25.10	.35	64	52	52	-1.915	1.22	383	307	6.00	
	5448	25.06	25.01	24.71	.35	86 *	54							
100	5448	25.06	25.01	24.71	35	86 *	54	46	-.042	.06	257	199	5.00	
	5498	26.16	26.02	26.01	15	9	46							
110	5448	25.06	25.01	24.71	.35	86 *	54	54	2.162	1.12	446	1125	8.00	
	6420	25.03	24.81	24.63	.40	45	51							
120	6420	25.03	24.81	24.63	.40	45	51	54	2.249	1.14	350	291	8.00	
	6464	24.19	24.11	23.79	.40	80 *	57							
130	6464	24.19	24.11	23.79	.40	80 *	57	57	2.387	1.34	277	421	8.00	
	6494	23.56	23.53	23.41	.15	82 *	57							
140	6494	23.56	23.53	23.41	.15	82 *	57	57	2.830	3.32	15	99	8.00	
	6495	23.56	23.47	23.40	.15	45	57							

FREEBOARD CODE *** 0% - 5%
OPEN CONDUITS ** 5% - 10%
(% OF DEPTH) * 10% - 20%

FILLING OF CLOSED ### 95% - 100%
CONDUITS ## 90% - 95%
(% OF DEPTH\DIAM.) # 80% - 90%

POSITIVE DISCHARGE FROM LOW TO HIGH NODE :

TOTAL CATCHMENT AREA 45.8000 HA

THE BANDWIDTH OF THE SYMMETRICAL MATRIX IS 6 POSITIONS

S P I L L E D   V O L U M E S
****

NODE NUMBER	VOLUME (M3)	SPILLING TIME (MIN)
6495	6134	95

W A T E R   B A L A N C E   I N   (M3)
****

INFLOW (HYDROGRAPHS)	:	0
INFLOW (FIXED WATER LEVELS)	:	0
RAINFALL	:	7794
TOTAL IN	:	7794
OUTFLOW (SPILLWAYS)	:	6135
OUTFLOW (PUMPS)	:	0
OUTFLOW (FIXED WATER LEVELS)	:	0
EVAPORATION	:	0
TOTAL OUT	:	6135
STORAGE CHANGE	:	1669
TOTAL OUT + STORAGE CHANGE	:	7804
BALANCE ERROR	:	.13 %

****** E N D   O F   C Y C L O N E ******

Southern section.

```
 1 *TEXT       RWSSP        RADA       YEMEN
 6 *GENERAL    0            50    50   100  100   50   30
 7 *CALC      25    1        2     1     5     0
 8 *NODE       0 3213  3517     0            0        0          23   30            1
 9 *NODE       0 3249  3231     0            0        0          56   30            1
10 *NODE       0 3391  3337     0            0        0          65   30            1
11 *NODE       0 3393  3287     0            0        0          40   30            1
12 *NODE       0 3398  3299     0            0        0           4   30            1
13 *NODE       0 4207  3180     0            0        0          36   30            1
14 *NODE       0 4210  3162     0            0        0          15   30            1
15 *NODE       0 4233  3013     0            0        0          40   40            1
16 *NODE       0 4242  3037     0            0        0          24   30            1
17 *NODE       0 4254  3071     0            0        0           7   30            1
18 *NODE       0 4258  3071     0            0        0          44   30            1
19 *NODE       0 4267  3201     0            0        0           5   30            1
20 *NODE       0 5107  2909     0            0        0           2   40            1
21 *NODE       0 5132  2800     0            0        0           1   30            1
22 *NODE       0 5205  2840     0            0        0          11   40            1
23 *NODE       0 5215  2860     0            0        0           9   40            1
24 *NODE       0 5261  2835     0            0        0          98   40            1
25 *NODE       0 5272  2800     0            0        0          45   40            1
26 *NODE       0 5276  2774     0            0        0          22   50            1
27 *NODE       0 5292  2740     0            0        0          42   50            1
28 *NODE       0 5297  2727     0            0        0          25   40            1
29 *NODE       0 6109  2570     0            0        0          18   30            1
30 *NODE       0 6264  2560     0            0        0          35   50            1
31 *ENODE        3213   340   120   100    30
32 *ENODE        3249   300   160   100    30
33 *ENODE        3391   420    90   100    30
34 *ENODE        3393   250   140   100    30
35 *ENODE        3398   120   100   100    30
36 *ENODE        4207   380   170   100    30
37 *ENODE        4210   140   100   100    30
38 *ENODE        4233   300   200   100    30
39 *ENODE        4242   260   190   100    30
40 *ENODE        4254   150   180   100    30
41 *ENODE        4258   260   110   100    30
42 *ENODE        4267   180   180   100    30
43 *ENODE        5107   120   120   100    30
44 *ENODE        5132    20   200   100    30
45 *ENODE        5205   120    90   100    30
46 *ENODE        5215   150   100   100    30
47 *ENODE        5261   460   130   100    30
48 *ENODE        5272   320   160   100    30
49 *ENODE        5276   300   370   100    30
50 *ENODE        5292   300   300   100    30
51 *ENODE        5297   220   270   100    30
52 *ENODE        6109   340   170   100    30
53 *ENODE        6264   280   130   100    30
54 *SPILLWAY     6286  2515     0           0      0 2000  2481185
55 *TRAPPROF      10 3213 3391   0  600   1   1 3502 3322  -50      295  100
56 *TRAPPROF      15 3249 4210   0  600   1   1 3216 3137   50      250  100
57 *TRAPPROF      20 3398 3249   0  600   1   1 3284 3216   50      100  100
58 *TRAPPROF      30 3391 3393   0  600   1   1 3322 3262   50      220  100
59 *TRAPPROF      40 3393 4207   0  600   1   1 3262 3165   50      230  100
60 *TRAPPROF      50 4207 4210   0  600   1   1 3165 3137   50       80  100
61 *TRAPPROF      60 4210 4258   0  600   1   1 3137 3046   50      230  100
62 *TRAPPROF      44 4233 5205   0  600   1   1 2998 2865   50       95  100
63 *TRAPPROF      65 4242 4254   0  600   1   1 3022 3056   50      175  100
64 *TRAPPROF      70 4242 5215   0  600   1   1 3022 2845   50      150  100
65 *TRAPPROF      80 4258 4267   0  600   1   1 3046 3186   50      130  100
66 *TRAPPROF      90 4258 5261   0  600   1   1 3046 2810   50      420  100
67 *TRAPPROF      95 5205 5215   0  600   1   1 2865 2845   50      100  100
68 *TRAPPROF     100 5107 5272   0  600   1   1 2894 2775   50      100  100
69 *TRAPPROF     110 5132 5297   0  600   1   1 2785 2702   50       90  100
70 *TRAPPROF     140 5215 5261   0  600   1   1 2845 2810   50      190  100
71 *TRAPPROF     150 5261 5272   0  600   1   1 2810 2775   50      100  100
72 *TRAPPROF     160 5272 5292   0  600   1   1 2775 2705   50      280  100
73 *TRAPPROF     165 5276 5292   0  600   1   1 2759 2705   50      150  100
74 *TRAPPROF     170 5292 5297   0  600   1   1 2705 2702   50       40  100
75 *TRAPPROF     180 6109 6264   0  600   1   1 2555 2525   50      240  100
76 *TRAPPROF     185 5297 6264   0  600   1   1 2702 2525   50      150  100
77 *TRAPPROF     190 6264 6286   0  600   1   1 2525 2480   50      125  100
78 *RAIN       1   1   1    0   85   15   85   16  187   30  187   31  656   45  656
79 *RAIN       1   1   2   46  320   60  320   61  122   75  122   76   63   90   63
80 *RAIN       1   1   3   91    0
81 *END
```

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DIFFUSIVE WAVE EQUATIONS
IMPLICIT METHOD

LENGTH OF A COORDINATE-UNIT       (M) :          1 000

ROUGHNESS ACCORDING TO
NIKURADSE (M)
FOR OPEN CONDUITS                                   0050
FOR CLOSED CONDUITS                  .              0050
EFFECTIVE CROSS-SECTIONAL AREA       :             1.000
RUN-OFF COEFFICIENT (DEFAULT)        .               250

TOTAL CALCULATION TIME         (HOUR) .              2
TIME STEP (CALCULATION)         (MIN) :              1
TIME STEP (OUTPUT)              (MIN) :              5

NUMBER OF TRAPEZIUM PROFILES         .              23
NUMBER OF VARIABLE PROFILES          :               0
NUMBER OF CLOSED CONDUITS                            0
NUMBER OF WEIRS                      .               0
NUMBER OF NODES                      .              23
NUMBER OF SPILLWAYS                  .               1
NUMBER OF PUMPS                      :               0
NUMBER OF INFLOW HYDROGRAPHS         .               0
NUMBER OF RAIN CURVES                :               1
NUMBER OF EVAPORATION CURVES         .               0
NUMBER OF STORAGE CURVES             .               0
NUMBER OF PROFILE CURVES             .               0
NUMBER OF WATER LEVEL CURVES         .               0
NUMBER OF MOVABLE WEIR CURVES        .               0
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CYCLONE, D H V CONSULTANTS, AMERSFOORT, THE NETHERLANDS DATE : 6-19-1991

***** INPUT *****
****

N O D E   D A T A
****

CATCHMENT AREA (CODE 1) 0- = HA PUMP CAP. (CODE 2) 0- = M3/S
1- = M2 1- = L/S

NODE NUMBER	GROUND LEVEL (M)	WATER LEVEL (M)	X-COORD (M)	Y-COORD (M)	CATCHMENT AREA (CODE 1)	RUN OFF CFF	CURVE NUMBERS INFL	STORGE	EVAP	RAIN	WATLEV	CAP (CODE 2)	PUMP START (M)	STOP (M)	SPILL WIDTH (M)	SPILL LEVE
3213	35.17	.00	0.	0.	2.3 0-	.30				1						
3249	32.31	.00	0.	0.	5.6 0-	.30				1						
3391	33.37	.00	0.	0.	6.5 0-	.30				1						
3393	32.87	.00	0	0.	4.0 0-	.30				1						
3398	32.99	.00	0.	0.	.4 0-	.30				1						
4207	31.80	.00	0.	0.	3.6 0-	.30				1						
4210	31.62	.00	0	0.	1.5 0-	.30				1						
4233	30.13	00	0.	0	4.0 0-	.40				1						
4242	30.37	.00	0.	0.	2.4 0-	.30				1						
4254	30.71	.00	0.	0	.7 0-	.30				1						
4258	30.71	.00	0.	0.	4.4 0-	.30				1						
4267	32.01	.00	0.	0.	.5 0-	.30				1						
5107	29.09	00	0.	0.	.2 0-	.40				1						
5132	28.00	.00	0.	0.	.1 0-	.30				1						
5205	28.80	00	0.	0.	1.1 0-	.40				1						
5215	28.60	.00	0.	0.	.9 0-	.40				1						
5261	28.35	.00	0.	0.	9.8 0-	.40				1						
5272	28.00	.00	0.	0.	4.5 0-	.40				1						
5276	27.74	.00	0.	0.	2.2 0-	.50				1						
5292	27.40	.00	0.	0.	4.2 0-	.50				1						
5297	27.27	.00	0.	0.	2.5 0-	.40				1						
6109	25.70	00	0.	0.	1.8 0-	.30				1						
6264	25.60	00	0.	0.	3.5 0-	.50				1						
6286	25.15	.00	0.	0.											20.00	24.

***** INPUT *****
****

C O N D U I T   D A T A
****

CONDT NUMBER	NODE NUMBER BEGIN	END	INVERT LEVEL BEGIN (M)	END (M)	LENGTH (M)	ROUGHNESS (*)	EFF. CROSS COEFF	BOTTOM WIDTH (M)	CATCHM. WIDTH (M)	SIDE SLOPES COTGA	COTGB	PROFILE CURVE NUMBER	CLOSED CONDUIT - CHARACT. SHAPE	HEIGHT (M)	WIDTH (M)	LOS BEGIN
10	3213	3391	35.02	33.22	295	.0050	1.00	6.00	0	.01	.01					
15	3249	4210	32.16	31.37	250	.0050	1.00	6.00	0	.01	.01					
20	3398	3249	32.84	32.16	100	.0050	1.00	6.00	0	.01	.01					
30	3391	3393	33.22	32.62	220	.0050	1.00	6.00	0	.01	.01					
40	3393	4207	32.62	31.65	230	.0050	1.00	6.00	0	.01	.01					
50	4207	4210	31.65	31.37	80	0050	1.00	6.00	0	.01	.01					
60	4210	4258	31.37	30.46	230	.0050	1.00	6.00	0	.01	.01					
44	4233	5205	29.98	28.65	95	.0050	1.00	6.00	0	.01	.01					
65	4242	4254	30.22	30.56	175	.0050	1.00	6.00	0	.01	.01					
70	4242	5215	30.22	28.45	150	.0050	1.00	6.00	0	.01	.01					
80	4258	4267	30.46	31.86	130	.0050	1.00	6.00	0	.01	.01					
90	4258	5261	30.46	28.10	420	.0050	1.00	6.00	0	.01	.01					
95	5205	5215	28.65	28.45	100	.0050	1.00	6.00	0	.01	.01					
100	5107	5272	28.94	27.75	100	.0050	1.00	6.00	0	01	.01					
110	5132	5297	27.85	27.02	90	.0050	1.00	6.00	0.	.01	.01					
140	5215	5261	28.45	28.10	190	0050	1.00	6.00	0	.01	.01					
150	5261	5272	28.10	27.75	100	.0050	1.00	6.00	0	01	.01					
160	5272	5292	27.75	27.05	280	.0050	1.00	6.00	0	.01	.01					
165	5276	5292	27.59	27.05	150	.0050	1.00	6.00	0	.01	.01					
170	5292	5297	27.05	27.02	40	.0050	1.00	6.00	0	.01	.01					
180	6109	6264	25.55	25.25	240	.0050	1.00	6.00	0	.01	.01					
185	5297	6264	27.02	25.25	150	0050	1.00	6.00	0	.01	.01					
190	6264	6286	25.25	24.80	125	.0050	1.00	6.00	0	.01	01					

* ROUGHNESS ACCORDING TO NIKURADSE (M)

R A I N   C U R V E S
****

RAIN CURVE 1	TIME (MIN)	0	15	16	30	31	45	46	60	61	75
	INTENS. (MM/H)	8.5	8.5	18.7	18.7	65.6	65.6	32.0	32.0	12.2	12.2
	TIME (MIN)	91									
	INTENS. (MM/H)	.0									

M A X I M U M   W A T E R   L E V E L S   A N D   D I S C H A R G E S

CONDT NUMBR	NODE NMBR	SURFACE LEVEL (M)	WATER LEVEL (M)	INVERT LEVEL (M)	CONDT DEPTH (M)	FILLING (%)	*\#	TIME (MIN)	TIME (MIN)	DISCHARGE (M3/S)	VELO-CITY (M/S)	GRADIENTS WATER (1:X)	GRADIENTS INVERT (1:X)	BOTTOM WIDTH (M)	REMARKS
OPEN CONDUITS WITH TRAPEZIUM PROFILES :															
10	3213	35.17	35.04	35.02	.15	12		48	49	.078	.29	169	163	6 00	
	3391	33.37	33.30	33.22	.15	51		54							
15	3249	32.31	32.22	32.16	.15	38		46	49	.246	50	361	316	6.00	
	4210	31.62	31.52	31.37	.25	62		60							
20	3398	32.99	32.85	32.84	15	5		46	46	-.020	.10	158	147	6 00	
	3249	32.31	32.22	32.16	.15	38		46							
30	3391	33.37	33.30	33.22	.15	51		54	56	.281	59	371	366	6.00	
	3393	32.87	32.70	32.62	.25	34		59							
40	3393	32.87	32.70	32.62	.25	34		59	60	.399	.78	238	237	6 00	
	4207	31.80	31.74	31.65	.15	59		62							
50	4207	31.80	31.74	31.65	.15	59		62	61	496	68	375	285	6.00	
	4210	31.62	31.52	31.37	.25	62		60							
60	4210	31.62	31.52	31.37	.25	62		60	62	.727	99	236	252	6 00	
	4258	30.71	30.55	30.46	.25	37		64							
44	4233	30.13	30.00	29.98	15	15		66	46	176	60	74	71	6 00	
	5205	28.80	28.73	28.65	.15	56		46							
65	4242	30.37	30.24	30 22	.15	12		48	52	-.024	.19	505	514	6 00	
	4254	30.71	30.58	30.56	.15	16		53							
70	4242	30.37	30.24	30.22	.15	12		48	48	107	.34	88	84	6 00	
	5215	28.60	28.54	28 45	.15	61		50							
80	4258	30.71	30.55	30.46	.25	37		64	47	- 021	.14	98	92	6.00	
	4267	32.01	31.87	31.86	.15	5		46							
90	4258	30.71	30.55	30.46	.25	37		64	64	.846	.97	186	177	6 00	
	5261	28.35	28.30	28.10	.25	80		63							
95	5205	28.80	28.73	28.65	.15	56		46	47	.256	.51	519	500	6.00	
	5215	28.60	28.54	28 45	15	61		50							
100	5107	29.09	28.94	28.94	15	3		46	46	013	05	102	84	6 00	
	5272	28.00	27.97	27.75	.25	86	*	64							
110	5132	28.00	27.85	27 85	.15	2		40	45	006	05	126	108	6.00	
	5297	27.27	27.14	27.02	.25	48		66							
140	5215	28.60	28.54	28.45	.15	61		50	52	.396	52	785	542	6.00	
	5261	28.35	28.30	28.10	.25	80		63							
150	5261	28.35	28.30	28.10	.25	80		63	63	1.518	1.22	299	285	6.00	
	5272	28.00	27.97	27 75	.25	86	*	64							
160	5272	28.00	27.97	27.75	.25	86	*	64	65	1.669	1.09	453	399	6.00	
	5292	27 40	27.35	27.05	.35	85	*	66							
168	5276	27.74	27.62	27.59	.15	17		61	49	100	.17	558	277	6.00	
	5292	27.40	27.35	27.05	.35	85	*	66							
170	5292	27.40	27 35	27.05	.35	85	*	66	66	1.916	1.53	193	1333	6.00	
	5297	27.27	27.14	27.02	.25	48		66							
180	6109	25.70	25.58	25.55	.15	23		67	50	.050	.09	7561	800	6.00	
	6264	25.60	25.55	25.25	.35	86	*	67							
185	5297	27.27	27.14	27.02	.25	48		66	66	1.996	1.58	94	84	6.00	
	6264	25.60	25.55	25.25	.35	86	*	67							
190	6264	25.60	25.55	25.25	.35	86	*	67	67	2.161	1.56	211	277	6.00	
	6286	25.15	24.96	24.80	.35	46		67							

FREEBOARD CODE OPEN CONDUITS (% OF DEPTH): *** 0% - 5%; ** 5% - 10%; * 10% - 20%

FILLING OF CLOSED CONDUITS (% OF DEPTH\DIAM.): ### 95% - 100%; ## 90% - 95%; # 80% - 90%

POSITIVE DISCHARGE FROM LOW TO HIGH NODE NUMBER

TOTAL CATCHMENT AREA 66.7000 HA

THE BANDWIDTH OF THE SYMMETRICAL MATRIX IS 5 POSITIONS

S P I L L E D   V O L U M E S

NODE NUMBER	VOLUME (M3)	SPILLING TIME (MIN)
6286	5997	82

W A T E R   B A L A N C E   I N   (M3)

INFLOW (HYDROGRAPHS)	:	0
INFLOW (FIXED WATER LEVELS)	:	0
RAINFALL	:	8736
TOTAL IN	:	8736
OUTFLOW (SPILLWAYS)	:	5997
OUTFLOW (PUMPS)	:	0
OUTFLOW (FIXED WATER LEVELS)	:	0
EVAPORATION	:	0
TOTAL OUT	:	5997
STORAGE CHANGE	:	2755
TOTAL OUT + STORAGE CHANGE	:	8752

BALANCE ERROR 18 %

****** E N D   O F   C Y C L O N E ******

North-Western section.

```
 1 *TEXT       RWSSP        RADA       YEMEN
 2 *TEXT
 3 *TEXT       DESIGN PRIMARY DRAINAGE SYSTEM
 4 *TEXT            NORTH-EAST AREA
 5 *TEXT             december 1990
 6 *GENERAL   0           50     50    100   100     50    30
 7 *CALC     25     1      2      1      5     0
 8 *NODE      0 2227    3936      0           0       0                  42   30         1
 9 *NODE      0 2340    3972      0           0       0                  32   40         1
10 *NODE      0 2377    3788      0           0       0                  30   30         1
11 *NODE      0 2393    3700      0           0       0                  17   60         1
12 *NODE      0 3307    3524      0           0       0                  33   30         1
13 *NODE      0 3310    3620      0           0       0                  24   30         1
14 *NODE      0 3321    3543      0           0       0                  20   30         1
15 *NODE      0 3323    3500      0           0       0                  22   30         1
16 *NODE      0 3338    3431      0           0       0                  16   30         1
17 *NODE      0 3333    3460      0           0       0                  24   30         1
18 *NODE      0 3343    3405      0           0       0                   1   40         1
19 *NODE      0 3354    3520      0           0       0                  17   30         1
20 *NODE      0 3492    3330      0           0       0                  57   30         1
21 *NODE      0 4305    3314      0           0       0                  11   50         1
22 *NODE      0 4316    3331      0           0       0                  17   50         1
23 *NODE      0 4466    3290      0           0       0                  25   60         1
24 *ENODE       2227     270     25    100    30
25 *ENODE       2340     150     10    100    30
26 *ENODE       2377     160     20    100    30
27 *ENODE       2393     200     10    100    30
28 *ENODE       3307     250    110    100    30
29 *ENODE       3310     280     80    100    30
30 *ENODE       3321     170     70    100    30
31 *ENODE       3323     180    300    100    30
32 *ENODE       3338     300    270    100    30
33 *ENODE       3333     370    140    100    30
34 *ENODE       3343     300    205    100    30
35 *ENODE       3354     300    170    100    30
36 *ENODE       3492     280    130    100    30
37 *ENODE       4305     120    100    100    30
38 *ENODE       4316     100    100    100    30
39 *ENODE       4466     150    150    100    30
40 *SPILLWAY    4453  3226      0          0        0  1000  3207185
41 *TRAPPROF     10 2227 2393     0   600     1     1 3921 3685    50       155   100
42 *TRAPPROF     20 2340 2377     0   600     1     1 3957 3773    50       200   100
43 *TRAPPROF     30 2377 2393     0   600     1     1 3773 3685    50       100   100
44 *TRAPPROF     40 2377 3321     0   600     1     1 3773 3528    50       185   100
45 *TRAPPROF     50 2393 3354     0   600     1     1 3685 3505    50       235   100
46 *TRAPPROF     60 3307 3310     0   600     1     1 3509 3605    50       130   100
47 *TRAPPROF     70 3323 3338     0   600     1     1 3475 3413    50       280   100
48 *TRAPPROF     80 3307 3333     0   600     1     1 3509 3445    50       295   100
49 *TRAPPROF     90 3321 3323     0   600     1     1 3528 3475    50       115   100
50 *TRAPPROF     95 3333 3338     0   600     1     1 3445 3413    50       100   100
51 *TRAPPROF    100 3333 3492     0   600     1     1 3445 3315    50       160   100
52 *TRAPPROF    110 3338 3343     0   600     1     1 3413 3380    50       105   100
53 *TRAPPROF    120 3343 3354     0   600     1     1 3380 3505    50       250   100
54 *TRAPPROF    130 3343 4316     0   600     1     1 3380 3306    50       105   100
55 *TRAPPROF    150 3492 4466     0   600     1     1 3315 3260    50       175   100
56 *TRAPPROF    160 4305 4316     0   600     1     1 3290 3306    50       125   100
57 *TRAPPROF    170 4305 4466     0   600     1     1 3290 3260    50       135   100
58 *TRAPPROF    180 4453 4466     0   600     1     1 3206 3260    50       168   100
59 *RAIN      1    1    1     0    85    15    85   16  187    30   187   31  656   45  656
60 *RAIN      1    1    2    46   320    60   320   61  122    75   122   76   63   90   63
61 *RAIN      1    1    3    91     0
62 *END

DIFFUSIVE WAVE EQUATIONS
IMPLICIT METHOD

LENGTH OF A COORDINATE-UNIT      (M) :          1 000

ROUGHNESS ACCORDING TO
NIKURADSE (M)
FOR OPEN CONDUITS                                 0050
FOR CLOSED CONDUITS                    .         .0050
EFFECTIVE CROSS-SECTIONAL AREA         :        1.000
RUN-OFF COEFFICIENT (DEFAULT)          :         .250

TOTAL CALCULATION TIME         (HOUR) :         2
TIME STEP (CALCULATION)         (MIN) :         1
TIME STEP (OUTPUT)              (MIN) :         5

NUMBER OF TRAPEZIUM PROFILES           :        18
NUMBER OF VARIABLE PROFILES            :         0
NUMBER OF CLOSED CONDUITS              .         0
NUMBER OF WEIRS                        .         0
NUMBER OF NODES                        .        16
NUMBER OF SPILLWAYS                    :         1
NUMBER OF PUMPS                        :         0
NUMBER OF INFLOW HYDROGRAPHS           :         0
NUMBER OF RAIN CURVES                  :         1
NUMBER OF EVAPORATION CURVES           :         0
NUMBER OF STORAGE CURVES                         0
NUMBER OF PROFILE CURVES               .         0
NUMBER OF WATER LEVEL CURVES           :         0
NUMBER OF MOVABLE WEIR CURVES          :         0
```

R A I N   C U R V E S
****

RAIN CURVE 1	TIME (MIN)	0	15	16	30	31	45	46	60	61	75	76	9
	INTENS. (MM/H)	8.5	8.5	18.7	18.7	65.6	65.6	32.0	32.0	12.2	12.2	6.3	6.
	TIME (MIN)	91											
	INTENS. (MM/H)	.0											

M A X I M U M   W A T E R   L E V E L S   A N D   D I S C H A R G E S
****

CONDT NUMBR	NODE NMBR	SURFACE LEVEL (M)	WATER LEVEL (M)	INVERT LEVEL (M)	CONDT DEPTH (M)	FILLING (%) *\#	TIME (MIN)	TIME (MIN)	DISCHARGE (M3/S)	VELO-CITY (M/S)	GRADIENTS WATER (1:X)	GRADIENTS INVERT (1:X)	BOTTOM WIDTH (M)	REMARKS
OPEN CONDUITS WITH TRAPEZIUM PROFILES .														
10	2227	39.36	39.24	39.21	.15	17	48	46	199	.67	67	65	6.00	
	2393	37.00	36.94	36.85	.15	60	42							
20	2340	39.72	39.64	39.57	.15	45	46	46	.232	.79	106	108	6.00	
	2377	37.88	37.76	37.73	.15	20	47							
30	2377	37.88	37.76	37.73	.15	20	47	46	.174	.56	121	113	6.00	
	2393	37.00	36.94	36.85	.15	60	42							
40	2377	37.88	37.76	37.73	.15	20	47	46	.196	.72	76	75	6.00	
	3321	35.43	35.35	35.28	.15	45	42							
50	2393	37.00	36.94	36.85	.15	60	42	44	666	1 11	134	130	6.00	
	3354	35.20	35.19	35.05	.15	94 **	47							
60	3307	35.24	35.19	35.09	.15	65	57	46	-.099	.34	147	135	6.00	
	3310	36.20	36.07	36.05	.15	13	46							
70	3323	35.00	34.87	34.75	.25	48	49	52	.397	65	444	451	6.00	
	3338	34.31	34.24	34.13	.18	61	59							
80	3307	35.24	35.19	35.09	.15	65	57	57	.181	.48	416	460	6.00	
	3333	34.60	34.48	34.45	.15	19	61							
90	3321	35.43	35.35	35.28	.15	45	42	45	.337	.70	240	216	6.00	
	3323	35.00	34.87	34.75	.25	48	49							
95	3333	34.60	34.48	34.45	.15	19	61	60	.106	.26	416	312	6.00	
	3338	34.31	34.24	34.13	.18	61	59							
100	3333	34.60	34.48	34.45	15	19	61	59	.142	.55	125	123	6.00	
	3492	33.30	33.21	33.15	.15	40	51							
110	3338	34.31	34.24	34.13	18	61	59	58	.468	.77	307	318	6.00	
	3343	34 05	33.90	33.80	.25	39	53							
120	3343	34 05	33.90	33.80	.25	39	53	49	-.712	1.06	193	200	6.00	
	3354	35.20	35.19	35.05	.15	94 **	47							
130	3343	34.05	33.90	33.80	.25	39	53	53	1.041	1.03	175	141	6.00	
	4316	33.31	33.30	33.06	.25	95 ***	53							
150	3492	33.30	33.21	33.15	.15	40	51	53	.330	.42	457	318	6.00	
	4466	32.90	32.83	32.60	.30	76	57							
160	4305	33.14	33.09	32.90	.24	80 *	55	54	-1.142	.89	609	781	6.00	
	4316	33.31	33.30	33.06	.25	95 ***	53							
170	4305	33.14	33.09	32.90	.24	80 *	55	56	1.185	.94	506	449	6.00	
	4466	32.90	32.83	32.60	.30	76	57							
180	4453	32.26	32.27	32.06	.20	100 ***	59	58	-1.657	1.27	296	305	6.00	
	4466	32.90	32.83	32.60	.30	76	57							

FREEBOARD CODE OPEN CONDUITS (% OF DEPTH)	***	0% - 5%	FILLING OF CLOSED CONDUITS (% OF DEPTH\DIAM.)	###	95% - 100%	POSITIVE DISCHARGE FROM LOW TO HIGH NODE NUMBER
	**	5% - 10%		##	90% - 95%	
	*	10% - 20%		#	80% - 90%	

TOTAL CATCHMENT AREA 38.8000 HA

THE BANDWIDTH OF THE SYMMETRICAL MATRIX IS 4 POSITIONS

S P I L L E D   V O L U M E S
****

NODE NUMBER	VOLUME (M3)	SPILLING TIME (MIN)
4453	3849	83

W A T E R   B A L A N C E   I N   [M3]
****

INFLOW (HYDROGRAPHS)		0
INFLOW (FIXED WATER LEVELS)	:	0
RAINFALL	:	4960
TOTAL IN		4960
OUTFLOW (SPILLWAYS)	:	3850
OUTFLOW (PUMPS)		0
OUTFLOW (FIXED WATER LEVELS)	:	0
EVAPORATION	:	0
TOTAL OUT		3850
STORAGE CHANGE	:	1117
TOTAL OUT + STORAGE CHANGE	:	4966
BALANCE ERROR	:	.13 %

****** E N D   O F   C Y C L O N E ******

Annex E:

ERROR ANALYSIS

CONTENTS.

E.1 ERROR ANALYSIS.

The results of the computations are not as accurate and certain as they may seem because of uncertainties in the input and the modelling. Therefore a freeboard of 15% is taken into account.

A simplified equation to compute the discharge is:

$Q = C*A*i$	with	Q = discharge in	[m^3/s]
		C = runoff coefficient	[-]
		A = Catchment area	[m^2]
		i = rain intensity	[m/s]

In this equation only the most important factors for the discharge are present. These factors have the biggest impact on the water levels. They will be discussed further on. The sophisticated computer model of the drainage system also takes less important factors into account; such as slope and roughness. These have a smaller impact on the water levels.

E.1.1 Rainfall.

Because of errors in the measurements of the rainfall the rainfall figures have already been increased with 10%. According to a study of the RIRDP[1] the measured rain figures are 10% to 15% too low.
The relation between daily rainfall and rainfall in short periods was derived from 3 years of pluviographs only. This makes the outcome uncertain.
From the measured rainfall figures a 95% confidence interval was derived. This interval indicates that 95% of the extreme values of the rain intensities for a duration of 0.5 hour are between 18 and 26 mm/h and therefore that 97.5% of these extreme are below 26 mm/h. So there is a 2.5% chance that a 2 year rain intensity for this duration is below this value (the expected value is 22 mm/h for a duration of 30 minutes). A more reasonable value would be the value which is exceeded only with a chance of 10%. This results in i = 22 mm/h + 10% ≈ 24 mm/h.

A 10 % bigger extreme rain intensity results also in a about 10% bigger design storm. This has a big impact on the relative water levels in the system.

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[1] Rada Integrated Rural Development Project, by Euroconsult.

E.1.2 Runoff coefficient.

The runoff coefficient depends on the soil type and use. Values for C are given in annexes D.4. The soil type and use have been read from maps. Development of the town might increase (more paved areas) or decrease (more walls built round premisses, hindering the water flow) this coefficient.

E.1.3 Catchment area.

The catchment areas are read from maps. The catchment size at a node can change if houses, walls or roads are constructed, because they can block or change the current water flow.

The runoff coefficient multiplied with the catchment area (C*A) gives the area of which all the water will be dischar-ged. It is assumed that in this case C*A is estimated with an accuracy of 20% (taking futur changes into account).
For the Middle section a 20% smaller and a 20% bigger factor C*A were entered in the model.
The 20% smaller factor C*A resulted in floodings at node 6315 and high relative water levels (95%) at node 5346 and 6339. By filling in a design storm for T = 1.5 years, comparable water levels occurred to the normal design situation (T = 2 years).
For a 20% smaller factor C*A, the return period could be about 3 years without flooding (maximum filling of 93% at node 6315).

E.1.4 System Characteristics.

The other input data in the model is read from the maps or obtained from books in combination with experience and site visits. In the model the inital rain loss is accepted to be 3 mm. If this would be only 1 mm, then the water levels would be about 2% higher.
Decreasing the roughness of the catchments, from 1 to 0.5 mm results in 1% higher water levels.
A roughness of the conduits of 0.1 instead of 0.5 mm makes the water flow faster. This doesn't make the water levels higher higher. A bigger roughness of 1 mm results in 4% higher water levels, because the water is longer in the drainage system.

The conclusion is that changing these variables does not have a great impact on the (relative) water levels. The model is quite insensitive to errors in these variables.

The length of the conduits, length and slope of a catchment are obtained from the maps. It is assumed that these variables are quite accurate (upto ± 20%). The small errors in these will therefore have a smaller impact on the relative water levels.

REFERENCES.

lit [1] Final Design Report, RWSSP, Volume I - main report, December 1989

lit [2] Final Design Report, RWSSP, Volume I - Annexes, December 1989

lit [3] Cyclone manual; DHV, Consultants bv.

lit [4] Technical note no. 42, monitor report, February 1991; RIRDP.

lit [5] Introduction to Hydrometeorology; J.P. Bruce & R.H. Clarck.

lit [6] Manual: Design of Urban Drainage Studies; DHV, Consultants b.v.

lit [7] Hydrometeorology; C.J. Wiesner.

lit [8] Handbook on the Principles of Hydrology; D.M. Gray.

lit [9] Sana'a Basin Water Resources Scheme; Ministry of Agricultur and Fischering YAR:
v/d SELKHOZPROMEXPORT.

lit [10] Yemen a travel survival kit.

lit [11] Hydrology; Wister & Brater.

lit [12] Urban Stormwater Drainage Management; Scheafer, Wright et.al.

lit [13] Stochastic Water Resources Technology; N.T. Kottegoda.

lit [14] Development in Water Science 14; Stormwater Hydrology & Drainage; D.Stephenson.

lit [15] Applied Hydrology; Ven Te Chow, David R. Maidment and Larry W. Mays.

lit [16] Lange golven (b73) deel 1; C.Verspuy, TUD, 1987.

lit [17] Numerieke berekeningen in waterbouwkunde en hydrology (b84N); C.B. Vreugdenhil, TUD, 1985.

lit [18] Een streep in het zand; AMOK, 1991.

lit [19] Considerations and recommendations for stormwater drainage, RWSSP, Draft March 1989.

Lit [20] Handbook of Applied Hydrology; Ven te Chow.

Lit [21] Noord-Jemen, Landendocumentatie 1981 nr.3; KIT.