Technical note on drainage systems

design of pipes and detention facilities for rainwater

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Content

NOTES

Technical note on drainage systems – design of pipes and detention facilities for rainwater

This technical note will present simple but widely used methods for the design of drainage systems. The note will primarily deal with surface water (rainwater) which on a satisfactorily way should be transport into the drainage system. Traditional two types of sewer systems exist: A combined system, where rainwater and sewage is transported in the same pipe, and a separate system where the two types of water are transported in individual pipe. This note will only focus on the separate rain/stormwater system, however, if domestic sewage should be included in the dimensioning procedure, it's not major different than described below - just remember to include this contribution for combined systems where the surface water (rain) and sewage are carried in the same pipes in the system and change some of the parameters for failure allowance (this will be elaborated further later on).

The technical note is divided into four main topics: First, a short review of the precipitation in Denmark as well as how historical (actual) rainfall data can be used advantageously to those simple design methods, then how pipelines and reservoirs can be dimensioned and finally how safety in the design can implemented.
The document is only to be used in education, since newer (but smaller) changes to some of the statistical parameters are not implemented in this.

Precipitation

In Denmark, precipitation mainly falls as rain and snow. The average rainfall is not uniform over Denmark despite the small size; from 500 mm/year in the east to 900 mm/year in the west (Figure 1). The total amount of rainfall per year is in the design context uninteresting - on the other hand, it is the extreme intensities under the single rain events that govern, for example the needed pipe diameter for a stormwater system. The Water Pollution Committee (SVK) in collaboration with DMI, see Figure 2 has a large number of stations where the varying precipitation intensity of all rain events are recorded.

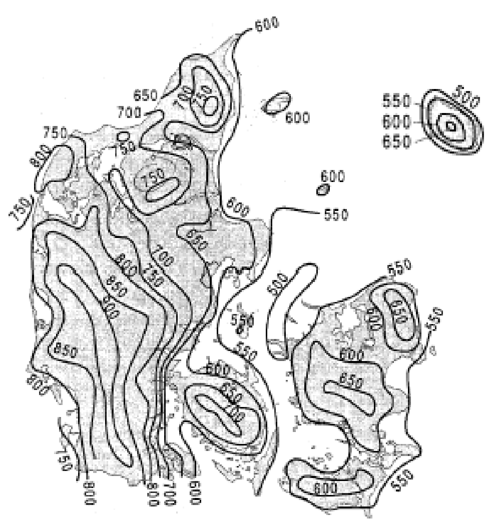

Figure 1 Mean yearly precipitation in Denmark [Linde et. el, 2002]

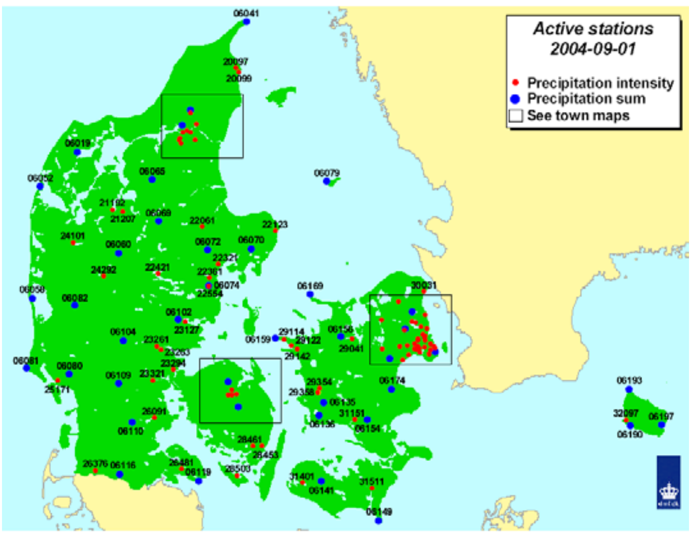

Figure 2 Rainfall measuring stations [DMI, 2005]

The intensity measured every minute during all rain events over a longer period can be used for dimensioning. However, this requires a lot of work if this is to be done by hand calculations. However, there are several programs on the market that works with real measured rain e.g. MIKE URBAN from DHI. To facilitate the dimensioning procedure - numerous statistical studies based on the measured precipitation in Denmark has been carried out. These statistics are gathered in the so-called "landsregnrække" - a table that sums up the statistical properties of Danish rainfall. This method is also used worldwide in devolved countries.

The intensity of rainfall over a rain event could, for example, appear as shown in Figure 3 with different intensities every minute during the event. Instead of having to apply the varying rain intensity of the event, moving averages over different time intervals (eg. 5 min, 10 min, 15 min…) (the black curve in Figure 3) has been calculated and for each averaging period stated the maximum mean intensity over the averaging period ("rain duration").
The mean value of the specified duration is subsequently assumed to be descriptive of precipitation intensity over this period (shaded area in Figure 3, hence "kasseregn" in Danish)

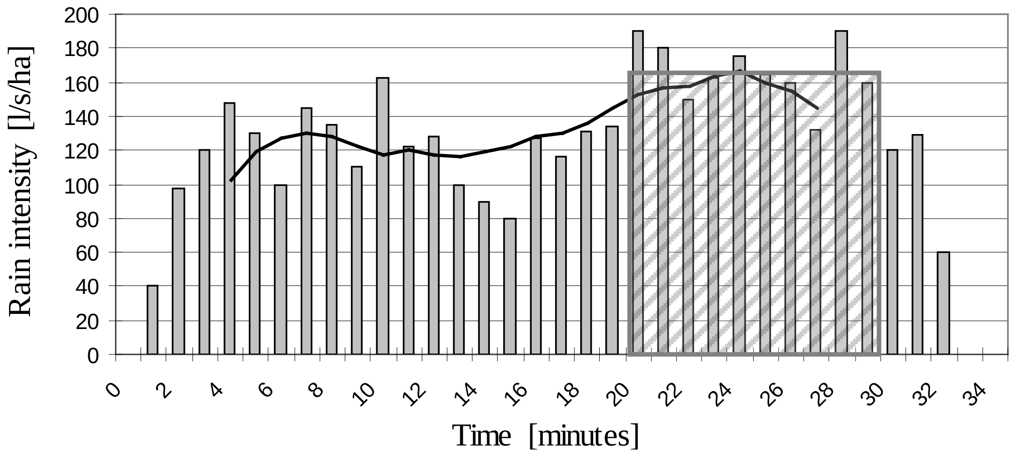

Figure 3 Bars: Measured rain intensity in 1 minute intervals over one single rain event. Black curve – moving average over 10 minutes. Shaded area – maximums mean intensity over 10 minutes.

The procedure of calculating mean rainfall intensities over different time intervals (5 min, 10 min, 15 min, etc.) have been carried out for all measured Danish rainfall over many years, after this, all the maximal intensities for a given duration have been ranked in size. This gives a statistical view of:

"How often you can <u>expect</u> a rain event with a given intensity and corresponding duration"

All this information is gathered in a table (and equation) where data from the whole country is collected as shown in table 1. Due to the spatial variation of precipitation in Denmark, it is possible to use/construct "local rain tables" – however it's not the scope of this note to do so and the effect of doing it might even be negligible or drown in other uncertainties.
In the table, it can be seen that e.g. one time every 10 years (a return period of T=10 yr) it can be expected that over a 15 minutes period will occur a rain event that has a mean intensity of 190 l/s/ha (litres per second per hectare)

The unit for rain intensity is somewhat a matter of religion. For dimensioning the unit *l/s/ha* is straight forward, however many people also uses µm/s (micrometer per sec) or mm/hr (millimeter per hour), hence remember the unit to avoid major errors in the dimensioning process. This note will only use *l/s/ha* as unit for intensity.

Later on in the note it will be elaborated which rain durations and intensities that should be used in the dimensioning procedure for pipes and retention pond systems.

Table 1 National rain statistic determined from measurements from 1933-62. Intensities are in l/(s·ha).

Return period T (years)	Duration, t_r (minutes)								
	5	10	15	20	25	30	40	60	120
20	350	280	240	205	172	149	119	86	64
10	310	230	190	170	142	123	98	72	43
5	260	190	160	128	108	94	76	56	33
2	200	140	114	92	78	68	56	43	26
1	150	110	88	72	61	54	44	33	21
0.5	110	83	64	53	46	41	34	26	17
0.2	80	52	40	34	29	26	22	17	11

When using return periods of less than one year, it may be appropriate to remember that the rain does not fall evenly throughout the year. The most intense rain (thunderstorms) included in the statistics is most likely to fall in the summer months. The return period T = 0.2 year (5 times a year) does not mean that, for example to expect a 5 minute event with an intensity of 80 l/s/ha every 2 ½ months. These rain events occur with high probability all during the summer months and perhaps all within a week.

For practical application all the information in table 1, has been further analysed and IDF-curves (Intensity-Duration-Frequency) have been created. These mathematical expression for these curves (not shown) are typically gathered in one equation of the type (1)

$$i = c \cdot t_r^{-\alpha} \qquad (1)$$

where i is the rain intensity [l/s/ha], t_r is the rain duration [s]. The parameters c and α depend on the chosen return period for the given problem that needs to be solved (see table 2). Note that there are limits for the validity of equation 1. It's possible to modify equation 1, so it's capable of dealing with rain durations below the given lower time limits in table 2. A simple way (and not very imprecise) is just to use the closest value from table 1 (or linear interpolated values from the table)

Table 2. National parameters for the rain intensity equation (1)

Return period T (year)	c	α	Limitation for t_r
20	55600	0,79	20 min – 4 days
10	45960	0,79	20 min – 4 days
5	28070	0,76	15 min – 4 days
2	16290	0,73	15 min – 4 days
1	10980	0,71	15 min – 4 days
0,5	5300	0,65	10 min – 3 hours
0,5	11200	0,73	3 hours – 4 days
0,2	2730	0,62	5 min – 4 hours
0,2	9490	0,75	4 hours – 4 days

Dimensioning of drainage systems (pipes or open channels)

There is no need for an in-depth explanation of the drainage systems / sewage systems purpose and function, as it must be clear to all. To be precise - these systems carry water (preferable by gravity), contaminated or not away from one location and to another. For dimensioning of stormwater systems it is quite different. For this purpose a variety of options exists depending on the complexity of the system. In this note only method will be provide – the simplest approach- the "rational method". This method is probably the most used method in practice for the design of storm water drainage systems.

It is clear that the necessary pipe diameter or channel dimension of a given section may depend on the amount of waste water or rainwater that it needs to carry. *Small water flow → small pipes, large water flow → large pipes*. Through fundamental hydraulics, this postulate, however, needs to be modified somewhat when the required diameter of eg. a given pipe needs to be found. It's beyond the scope of this note to derive the fundamental hydraulics of pipe and channel flows, hence it must be emphasized to read further on this subject elsewhere. However in general the capability of a pipe/channel to carry a certain amount of water away per time is as previous mentioned a matter of cross sectional size (diameter), slope of the pipe/channel and pipe roughness. Several equations exists, that combines these features, the most commonly used for pipe and channel dimensioning is the Manning equation (2)

$$V = M \cdot R^{2/3} \cdot \sqrt{I} \qquad (2)$$

where V is the full running velocity [m/s], M is the Manning number [$m^{1/3}$/s], R is the hydraulic radius [m] and I is the slope on the energy line.

Typical Manning numbers can be found in literature – note there are differences on Mannning M and Manning n in the literature $n = 1/M$. For concrete pipes a typical value of M will be 75 $m^{1/3}$/s and around 90 $m^{1/3}$/s for a PVC pipe, however after many years of use the roughness tends to be the same for different materials.

For practical application in the dimensioning procedure some assumptions are made and some of the terms in 2 have been substituted as shown in eqn 3

$$\begin{aligned} &Q = A \cdot M \cdot R^{2/3} \cdot \sqrt{I_0} \\ &\text{since } Q = V \cdot A \end{aligned} \qquad (3)$$

where Q is the discharge/flow [m^3/s], A – the cross sectional area of the pipe/channel, I_0 is the slope of the pipe/channel.

The hydraulic radius R is to be understood as the ratio between cross sectional area of the pipe/channel and the wetted perimeter of the pipe/channel. E.g. for at full-flowing pipe the hydraulic radius is (4):

$$R_{full,pipe} = \frac{\pi \cdot r^2}{2 \cdot \pi \cdot r} = \frac{r}{2} = \frac{D}{4} \qquad (4)$$

Where r is the radius of the pipe and D is the diameter of the pipe
So in order to relate the Manning equation (2) to the full-flowing capacity of a pipe of given size to carry a certain amount of water away per time unit equation 2 could be written:

$$Q_{full} = \pi \cdot \left(\frac{D}{2}\right)^2 \cdot M \cdot \left(\frac{D}{4}\right)^{2/3} \cdot \sqrt{I_0} \qquad (5)$$

Rational method for designing pipe and channel rainwater systems

Basically, the dimensioning ensures that eg. pipes can carry the highest possible water flow from the upstream catchment area, within established criteria for how often the capacity of the pipes to be exceeded. How often we allow a pipe to lose its capacity is depending on the consequences of flooding. Normally we allow the pipes (for only rainwater) to exactly lose its capacity ones a year – keeping in mind that the pipes are places below ground – it means in practice that approximately only every 5th year water will stand on the ground. No law is stated on this field, so it's a matter of the service level you are willing to pay for, that decides the return period for failure.

The rational method relies on the assumption, that the maximal flow in a pipe occurs when it rains exactly as long time as it takes for the furthest away landing droplet of rainwater to enter the pipe that is to be dimensioned (an example will be provided later on). Remember that the rain with exactly this duration has a corresponding rain intensity (eqn 1) for a given chosen return period.

The time it takes the water to travel to the pipe that should be sized is called the concentration time defined as t_f in this note, is the sum of the time it takes the water to run on the surface (t_s) (eg. on the road) and the time it takes to run in previous connected pipes (t_p).

The surface runoff time, depends in principle of many factors, size of the catchment, slope, pavement type, wind etc. This time will often be very difficult to state, hence a surface runoff time is often set to be 5-10 minutes.

In respect to the surface – it must also be remembered that it might not be all of the water that lands on the catchment that ends up in the sewer system, e.g. in a park almost all the rain will infiltrate to the groundwater – not reaching the pipe system. To include these different pavement types in the design phase a new area is defined -> *the reduced area* F_r.

Reduced area

To characterise how big a percentage of a given catchment that contributes with water to the sewer, a runoff coefficient φ is implemented, hence the runoff coefficient φ is a number between 0 and 1, 1 if all the water that lands on the surface runs to the sewer. Table 3 shows typical values for runoff coefficients.

Table 3. Typical values for runoff coefficients

Area	**Degree of imperviousness**
City center	0.7 -1.0
Industrial areas	0.5 – 0.9
Institutions, public spaces	0.3 – 0.6
Mixed residential & business	0.4 – 0.6
Block of flats	0.4 -0.6
Detached houses	0.2 – 0.35
Green areas / parks	0.05 – 0.15

In theory the runoff coefficient is the multiple of three other catchment parameters:

Degree of imperviousness:

The impervious area F_{imp} of a catchment area is that part of the total area F_{tot}, where the surface limits the natural infiltration. Here we are talking about both fully impervious and semi-impervious surfaces e.g. asphalt, flagstone pavements, macadams, roofs
Hence the degree of imperviousness β can be defined as:

$$\beta = \frac{F_{imp}}{F_{total}} \qquad (6)$$

Degree of connection (δ)

When the impervious area has been determined, you should be aware of that there might be areas of these that is not connected to the stormwater system. In principle you should determine whether the runoff from some of the impervious areas is handled in another way (it could be infiltration trenches, direct discharge to receiving waters or pervious areas etc.)

$$\delta = \frac{F_{connected}}{F_{imp}} \qquad (7)$$

For normal design practice set δ =1. (Safe side)

Hydrological reduction factor (θ)

This is the parameter that describes how big a percentage of the semi-impervious and impervious connected area that contributes to runoff. The reduction factor reflects that not all the connected impervious area contributes 100 % to the runoff due to infiltration and evaporation.

Runoff coefficient (φ)

By combining all above mentioned parameters you will get the overall runoff coefficient, that again states how big a percentage of a given catchment that contributes with water to the sewer or channel.

$$\varphi = \beta \cdot \delta \cdot \theta \qquad (8)$$

Note that there are quite large confusion about this term and different definitions worldwide (and national), hence you will find people talking about runoff coefficients but meaning e.g. hydrological reduction factor and vice versa.

Design flow

The design flow Q_d, in respect to the rational method, can be calculated as given in equation 9:

$$Q_d = \varphi \cdot F_{tot} \cdot i(t_f) = F_r \cdot i(t_f) \qquad (9)$$

Remember the units used in this equation: The calculation of the rain intensity (eqn. 1) should use *[seconds]* for time unit, giving the rain intensity in *[litres/second/hectare]*, the total catchment area or the reduced catchment area should be in *[hectares]* given the design flow in *[litres/second]*. Subsequently an example of the use of the rational method is given.

Example 1

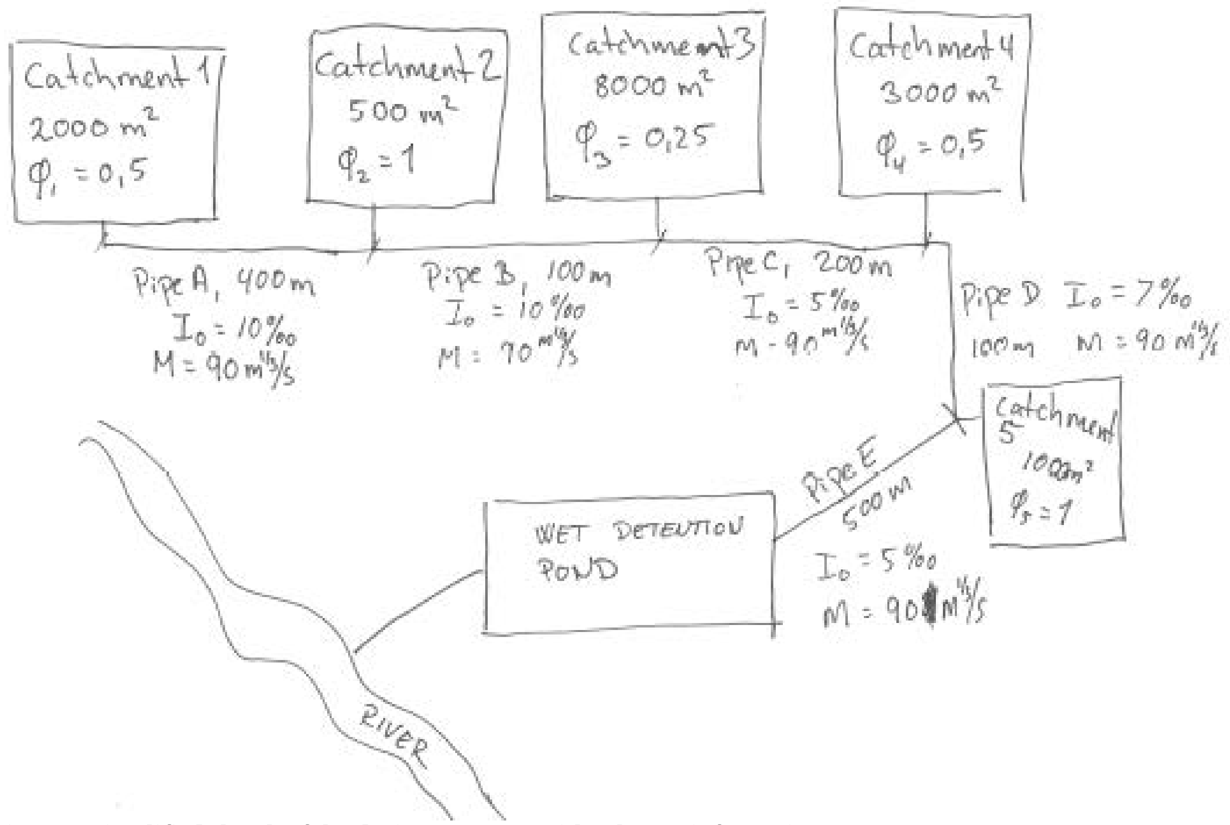

Figure 4 Simplified sketch of the drainage system with relevant information.

For some reasons the city been decided that the service level of the system is increased compared to traditional levels, hence it has been decided to use a return period of T= 5 years for the design of the pipe system. That means in other words that the pipes may run just full ones every 5 years. The design formulation could then be written:

$$\frac{Q_{d,pipe}}{Q_{full,pipe}} \leq 1 \qquad (10)$$

All surface runoff times (t_{fo}) from the catchment to the connected pipes are assumed to be 9 minutes

Pipe A:

First the design flow is calculated and according to the rational method, the intensity of the rain that should be used, corresponds to the rain with a duration of exactly the time where the whole catchment connected to pipe A is contribution with water. That means for pipe A, that after 9 minutes of rain, the most far placed corner of the catchment is contributing with water in pipe A, hence it should be the intensity of the rain with a duration of 9 minutes for a return period of 5 years that is the design flow for pipe A.

Since the duration is 9 minutes for the design rain, the rain intensity equation (1) is not valid (see table 2, the rain intensity for a duration of 10 minutes from table 1 (T=5 years) is used instead:

$$Q_{d,pipe,A} = \varphi \cdot F_{tot} \cdot i(t_f) = 0,5 \cdot 0.2ha \cdot 190 \, {}^{l}\!/_{s \cdot ha} = 19l/s = 0.019m^3/s$$

The capacity of the pipe with the properties given in figure 4, can be calculated with the Manning equation:

$$Q_{full,pipeA} = A \cdot M \cdot R^{2/3} \cdot \sqrt{I_0} = \pi \cdot \left(\frac{D_A}{2}\right)^2 \cdot 90\frac{m^{1/3}}{s} \cdot \left(\frac{D_A}{4}\right)^{2/3} \cdot \sqrt{0.01}$$

In respect to the design criteria following could be stated for pipe A, giving one equation with one unknown – the diameter of pipe A.

$$\frac{Q_{d,pipeA}}{Q_{full,pipeA}} \leq 1 = \frac{0.019\frac{m^3}{s}}{\pi \cdot \left(\frac{D_A}{2}\right)^2 \cdot 90\frac{m^{1/3}}{s} \cdot \left(\frac{D_A}{4}\right)^{2/3} \cdot \sqrt{0.01}} \leq 1$$

The diameter can be found buy guessing, or a simple iteration procedure, or different solver programs, leading to a necessary diameter of 0.154m for pipe A. Please remember, that this is the internal diameter of the pipe and not all dimensions are traded. Plastic pipes are sold as external diameter, hence the next pipe in line (always upwards) that fulfill the design criteria for pipe A, is a 200mm pvc pipe with an internal diameter of 191mm.

So far so good, now it comes to pipe B, however the principle is the same. The design flow should be calculated, however caution should be taking know in using the right rain durations and catchment areas:

So first question is, when do all the catchments connected to pipe B contribute with water in pipe B? Answer: When the water landing on catchment 1 contributes with water in pipe B, since the surface runoff time (9 min) is the same for catchment 1 and 2. Remember that it's a totally new situation we are calculating on now, and the design procedure for pipe A should be forgotten now – also remember that since this is a rather small area we are dealing with – it rains on all catchments (1-5) on the same time.

So the design flow for pipe B:

Concentration time to the beginning of pipe B = design rain duration, hence the rain intensity that should be used for designing pipe B, should correspond to a rain-duration of:

$t_{f,2}$: 9 minutes ($t_{s,1}$) + the time it takes the water to travel in pipe A. (t_{pA})

A good estimate of the transportation time (t_{p1}) in pipe 1 could be calculated since the water velocity easily can be calculated from the definition of flow:

$$V_{full,pipeA} = \frac{Q_{full,pipeA}}{A_{pipeA}} = \frac{0.019\,\frac{m^3}{s}}{\pi \cdot \left(\frac{0,154m}{2}\right)^2} = 1.03m/s$$

This velocity is of cause not totally true, since in reality a larger pipe has been placed and the design intensity (duration) for pipe B is not the same as it was for pipe A. This is however a minor issue. The traveling time for pipe A, can simply be calculated as follows:

$$t_{pA} = \frac{\text{Lenght of pipe A}}{\text{Velocity in pipe A}} = \frac{400m}{1.03m/s} = 390s$$

The total concentration time to point B is then

$$t_{f2} = 9\min + 390s = 930s\,(15\tfrac{1}{2}\min)$$

Since the design rain duration for pipe B, should be 15½ min, the rain intensity equation (1) can be used to calculate the design rain intensity.

If the surface runoff time for catchment 2 for example was 18 minutes. The design rain duration should have been 18 minutes! By using 18 minutes in this case, it would have ensured that both catchment A and B was contributing with water in pipe A at the same time.

Since the surface runoff time for catchment 2 is 9 minutes, then by using 15½ min for the design rain duration, it ensures that exactly on the minute 15½ both catchment A and B is contributing with water in pipe B.

The design rain intensity is calculated from eqn. 1 with constants from table 2, for T=5 years.

$$i = c \cdot t_{f2}^{-\alpha} = i = 28070 \cdot 930s^{-0.76} = 156\ {}^{l}\!/_{s \cdot ha}$$

Note that the design intensity is falling, due to the longer duration of the design rain. The design flow can then be calculated – just keep in mind that not only the intensity has changed also the connected area to pipe B has increased!

$$Q_{d,pipe,B} = \sum(\varphi \cdot F_{tot}) \cdot i(t_f) = (0,5 \cdot 0.2ha + 1 \cdot 0.05ha) \cdot 156\ {}^{l}\!/_{s \cdot ha} = 23l/s = 0.023m^3/s$$

And the necessary pipe diameter for pipe B

$$\frac{Q_{d,pipeB}}{Q_{full,pipeB}} \le 1 = \frac{0.023\,\frac{m^3}{s}}{\pi \cdot \left(\frac{D_B}{2}\right)^2 \cdot 90\,\frac{m^{1/3}}{s} \cdot \left(\frac{D_B}{4}\right)^{2/3} \cdot \sqrt{0.01}} \le 1 \Rightarrow D_B = 0.166m$$

$$V_{full,pipeB} = \frac{Q_{full,pipeB}}{A_{pipeB}} = \frac{0.023\,\frac{m^3}{s}}{\pi \cdot \left(\frac{0{,}166m}{2}\right)^2} = 1.08m/s$$

$$t_{pB} = \frac{100m}{1.08m/s} = 93s$$

Same procedure goes on for pipe C

$$i = c \cdot t_{f3}^{-\alpha} = i = 28070 \cdot (930s + 93s)^{-0.76} = 145\ {}^{l}/_{s \cdot ha}$$

$$Q_{d,pipe,c} = \sum(\varphi \cdot F_{tot}) \cdot i(t_f) = (0{,}5 \cdot 0.2ha + 1 \cdot 0.05ha + 0.25 \cdot 0.8ha) \cdot 145\ {}^{l}/_{s \cdot ha} = 51l/s$$

$$\frac{Q_{d,pipeC}}{Q_{full,pipeC}} \le 1 = \frac{0.051\,\frac{m^3}{s}}{\pi \cdot \left(\frac{D_C}{2}\right)^2 \cdot 90\,\frac{m^{1/3}}{s} \cdot \left(\frac{D_C}{4}\right)^{2/3} \cdot \sqrt{0.005}} \le 1 \Rightarrow D_C = 0.253m$$

$$V_{full,pipeC} = \frac{Q_{full,pipeC}}{A_{pipeC}} = \frac{0.051\,\frac{m^3}{s}}{\pi \cdot \left(\frac{0{,}253m}{2}\right)^2} = 1.01m/s$$

$$t_{pC} = \frac{200m}{1.01m/s} = 198s$$

Same procedure goes on for pipe D

$$i = c \cdot t_{f4}^{-\alpha} = i = 28070 \cdot (930s + 93s + 198s)^{-0.76} = 127\ {}^{l}\!/_{s \cdot ha}$$

$$Q_{d,pipe,D} = \sum(\varphi \cdot F_{tot}) \cdot i(t_f) = (0{,}5 \cdot 0.2ha + 1 \cdot 0.05ha + 0.25 \cdot 0.8ha + 0.5 \cdot 0.3ha) \cdot 127\ {}^{l}\!/_{s \cdot ha}$$
$$= 63l/s$$

$$\frac{Q_{d,pipeD}}{Q_{full,pipeD}} \leq 1 = \frac{0.063 \frac{m^3}{s}}{\pi \cdot \left(\frac{D_D}{2}\right)^2 \cdot 90 \frac{m^{1/3}}{s} \cdot \left(\frac{D_D}{4}\right)^{2/3} \cdot \sqrt{0.007}} \leq 1 \Rightarrow D_D = 0.258m$$

$$V_{full,pipeD} = \frac{Q_{full,pipeD}}{A_{pipeD}} = \frac{0.063 \frac{m^3}{s}}{\pi \cdot \left(\frac{0{,}258m}{2}\right)^2} = 1.21m/s$$

$$t_{pD} = \frac{100m}{1.21m/s} = 83s$$

And finally for pipe E

$$i = c \cdot t_{f5}^{-\alpha} = i = 28070 \cdot (930s + 93s + 198s + 83s)^{-0.76} = 120\ {}^{l}\!/_{s \cdot ha}$$

$$Q_{d,pipe,E} = \sum(\varphi \cdot F_{tot}) \cdot i(t_f) = (0{,}5 \cdot 0.2ha + 1 \cdot 0.05ha + 0.25 \cdot 0.8ha + 0.5 \cdot 0.3ha + 1 \cdot 0.1ha)$$
$$\cdot 120\ {}^{l}\!/_{s \cdot ha} = 72l/s$$

$$\frac{Q_{d,pipeE}}{Q_{full,pipeE}} \leq 1 = \frac{0.072 \frac{m^3}{s}}{\pi \cdot \left(\frac{D_E}{2}\right)^2 \cdot 90 \frac{m^{1/3}}{s} \cdot \left(\frac{D_E}{4}\right)^{2/3} \cdot \sqrt{0.005}} \leq 1 \Rightarrow D_E = 0.289m$$

To sum up – the necessary dimension for pipe A-E is shown in table 4.

Table 4 Necessary pipe diameters and trade dimensions

Pipe	Necessary internal diameter	Trade dimension
A	154 mm	200 mm
B	166 mm	200 mm
C	253 mm	315 mm
D	258 mm	315 mm
E	289 mm	315 mm

Important remarks to the rational method

Note that the rational method is not applicable on larger complex systems with very long transportation times in pipes and with large variation in sub-catchment sizes. If applied without thought the system will be under-dimensioned in worst case. A good sign of the shortage of the method is – if you get falling design flow in the flow direction. In this case simply use the highest flow calculated in the previous pipe as design flow for the given pipe. A worse situation can occur if e.g. catchment 5 were very large (say 5 ha) and the 4 previous catchments where very small (say a few square meters). If the rational method should be used in this case without any thought, you should, following the method, use the rain intensity corresponding to the longest travel time when designing pipe E. Keeping in mind that longer rain duration gives lower rain intensities, this will truly under dimension pipe E, since catchment A,B,C & D has such small catchment areas, that in praxis they doesn't really contribute with a significant flow in pipe E. In this case pipe E should probably only be designed for a rain intensity corresponding to the surface runoff time for catchment 5.

Design of detention ponds

Detention ponds associated with drainage systems are built with different purposes and functions. The primary reason for the construction of ponds is to reduce the hydraulic load (water flow) on the drainage system e.g. the wastewater treatment plant or to the recipient (recipient = the receiving water for the drainage system like rivers, estuaries, seas or groundwater). By reducing the load on the following drainage system, you will be able to reduce pipe sizes, reducing the slope on connected sewer lines or even prevent flooding further down in the system, during heavy rain. The primary reason for establishment of ponds in the end of a system, are to reduce the flow of rain water, maybe from cubic meters per second down to a few litres per second discharged to e.g. a small stream to prevent flooding and erosion of these ecosystems. In praxis these ponds also work (especially those which have a permanent water pool) as small low-tech treatment plants separating settleable solids, with different pollutants like heavy metals and xenobiotics attached. A lot of focus has been put into this in the later years, however since the scope of this note is pure hydraulics of the systems, the reader should seek more information elsewhere e.g. in *Bentzen (2008).* The following three figures show typical locations of ponds in a drainage system, type 1 and 2 are in primarily used in combined systems, however type 2 is also seen in separate systems, where the topography (very little slope), else would have demanded very large pipes. Type 3 is only used for separate systems for stormwater.

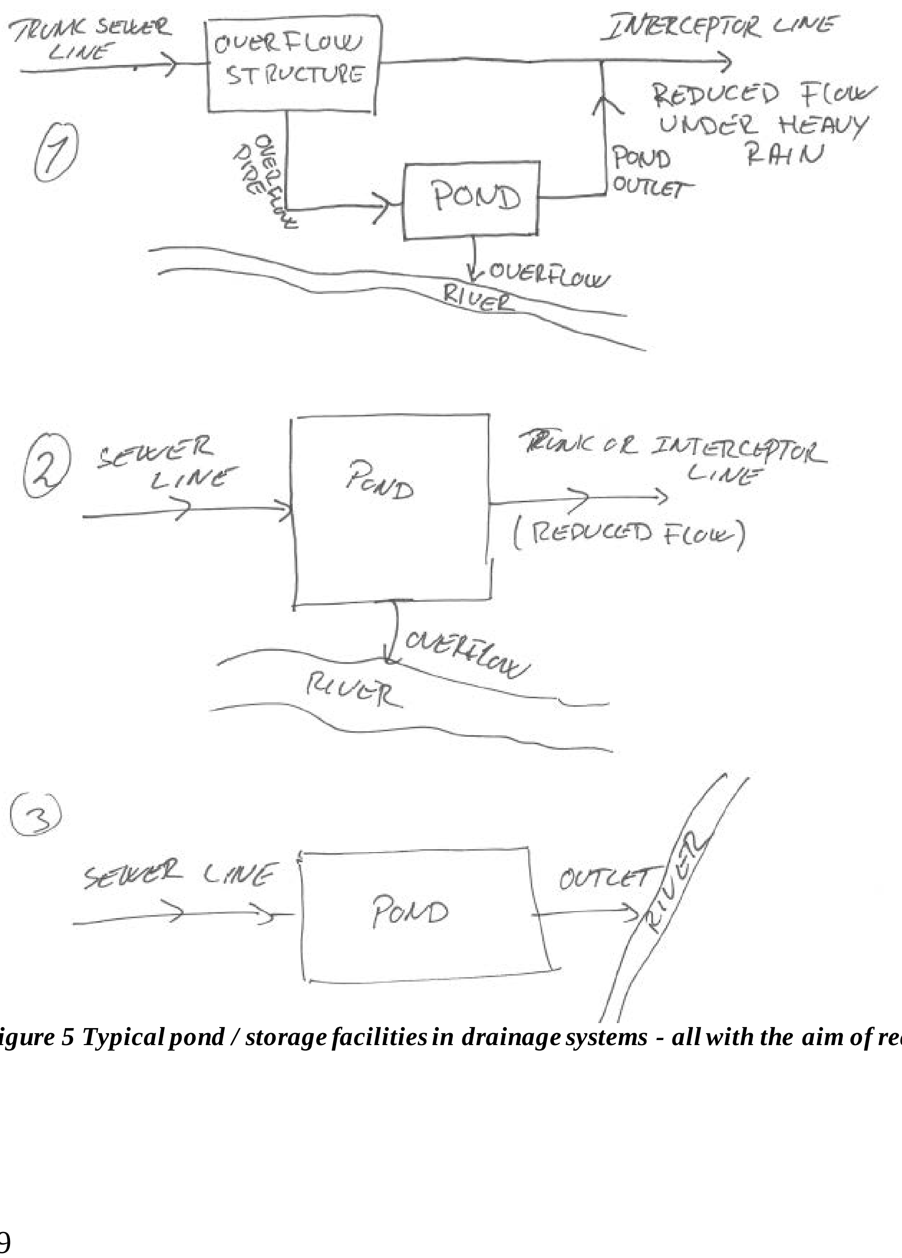

Figure 5 Typical pond / storage facilities in drainage systems - all with the aim of reducing the discharge

In principle two types of ponds exits - wet detention ponds and a dry detention ponds. In respect to dry detention ponds two types exits – one with a reduced outlet (pipe) to recipient or an infiltration system, where the soil properties allows the rain water to infiltrate to the groundwater (see figure 6).

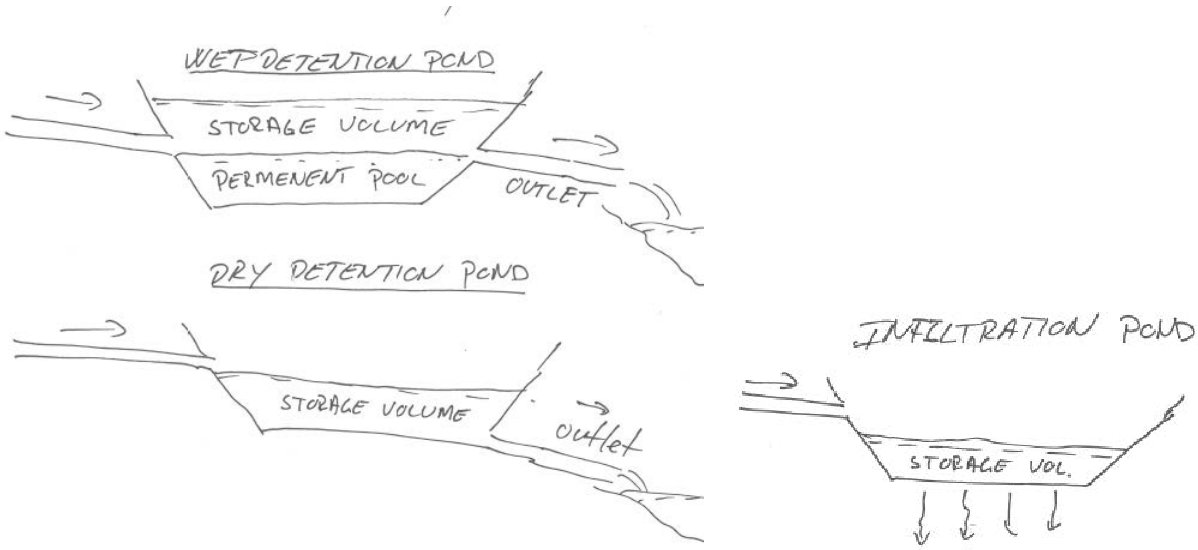

Figure 6. Principle sketch of detention/storing facilities

Pros and cons exist for these different types of ponds. A wet detention pond is more expensive to construct, since it should have more volume than the dry pond, due to the presence of the permanent water pool, it should also be sealed e.g. with clay, membranes or concrete to prevent infiltration and additional this type demands more maintenance than dry ponds. On the other hand wet ponds facilitate a more effective treatment of polluted stormwater and are more aesthetical than dry ponds. One the other hand dry ponds are cheaper since they don't need such a big volume nor the sealing of the bottom. Infiltration ponds becomes very large if the soil properties (in terms of infiltration capacity) are bad, however infiltration ponds also facilitates a very high degree of treatment and they might be the only solution if the system is placed far from e.g. rivers or seas.

Subsequently a simple method for sizing of the detention/storage volume (figure 5) of dry and wet pond is provided after this a calculation example is provided for the dimensioning of the wet detention pond from previous example 1.

Design procedure by use of "uniform rain"

The easiest way of designing (and the most common way) a detention pond, is by using the statics of the rain as present previously in this note. The procedure is even more simplified than the rational method used for pipe dimensioning – since the concentration time (surface and pipe runoff times) of the system is neglected. This is a fairly okay assumption since the concentration times of the drainage system might be in the timescale minutes to a few hours – and the residence time in the ponds might be several hours to days. In praxis this means that we are just looking at all the sub-catchments to the pond as one single catchment with no surface runoff time nor transportation time in the pipe system. This will be on the safe side in the design procedure. The simplification is shown on figure 7. From basic continuity consideration, it can be seen that the necessary volume of a pond is the net-flow to the pond – in other words the difference between the inflow Q_{in} and the outflow Q_{out} multiplied with the duration of the rain (t_r).

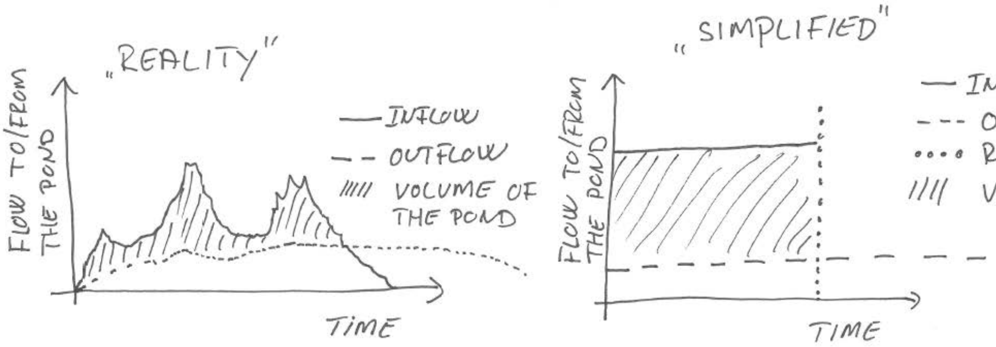

Figure 7 Real and simplified dynamics of a pond.

The necessary volume of a pond an arbitrary "rain" from table 1 can be expressed as:

$$Vol = (Q_{in} - Q_{out}) \cdot t_r \qquad (11)$$

The discharge out of the pond Q_{out} is in this case assumed constant. This is more or less possible by introducing a water brake in the outlet structure. If the outlet only consist of a pipe with a reduced dimension compared to the inlet pipe, this will not be totally true, since the outlet discharge then will depend on the pressure – in other words the water level in the pond.
The outlet discharge is a matter of choice if the pond is of type 1 or 2 on figure 5. If the system is of type 3, the outlet discharge is often set by the responsible authorities for the receiving water and is a matter of the sensitivity of the receiving water. Typically this outlet permit is given on the form *litres/seconds/hectare of catchment connected to the pond.* A typical value of this for a small stream will be in the order of 1 l/s/ha – meaning that Q_{out} for a pond with 10 ha connected is 10 l/s. Mind whether the permit is given based on reduced catchment or total catchment area (there is a lot of confusion about this in praxis).

Since the inflow (Q_{in}) depends on the rain intensity (i) and the reduced catchment area (F_r) equation 11 can be rewritten to:

$$Vol = (i \cdot F_r - Q_{out}) \cdot t_r \qquad (12)$$

However it is not obvious which rain intensity to use for a chosen return period, like it was for the pipe design. It must here be pointed out – that the design inflow for the pond is not the same as the design flow for dimensioning of pipe E in example 1 (figure 4). Design rains for pipes are often short duration rains with high intensity – however these rains don't contribute to a lot of volume due to their short duration. For pond design – it's typically longer lasting rain with a little lower intensity that defines the necessary volume. So following mathematical operation has the purpose of finding exactly that rain that gives the largest volume of the pond for the given return period:

The rain intensity (i) in equation 12 can be described with equation 1, substitute this into equation 12 gives:

$$Vol = (c \cdot t_r^{-\alpha} \cdot F_r - Q_{out}) \cdot t_r \qquad (13)$$

Since both c, α, F_r and Q_{out} are constants for a given pond and return period, the volume of the pond depends only of the rain duration (t_r). This duration can, as normally for continuous function, be found by differentiating the function and setting the derivative to zero.

$$\frac{dVol}{dt_r} = -(Q_{out} \cdot t_r^{\ \alpha} + c \cdot (\alpha - 1) \cdot F_r) \cdot t^{-\alpha} = 0 \tag{14}$$

which gives:

$$t_{r,\max} = \left(\frac{-c \cdot (\alpha - 1) \cdot F_r}{Q_{out}} \right)^{1/\alpha} \tag{15}$$

The maximal necessary pond volume can then easily be calculated by substitution t_r in eqn. 13 with $t_{r,max}$ from eqn. Remember to use correct dimensions in the equations. If c and α are used to calculate the rain intensity in *l/s/ha* – the reduced area F_r should be in hectares and the outflow Q_{out} should be in *l/s* – giving the pond volume in litres.

The volume calculated in this way – is the necessary detention volume for a wet pond or the total volume of a dry pond. If this is a wet pond the storage volume should be added to the permanent volume, which then gives the total necessary volume. No official rules are provide for the size of the permanent water volume, however to ensure that living organisms can survive during winter a minimum permanent depth should be 0.8 m. Furthermore, typical values for the permanent wet volume are 200-250 m^3 per reduced hectare.

For infiltration ponds the procedure is the same – here it is the hydraulic properties of the underlying soil that gives the outflow Q_{out} for the design. Infiltration capacities for soil is typical ranging from 10^{-4} *m/s* for very coarse sand to more or less impermeable clay 10^{-10} *m/s* – so values for designing an infiltration pond should be taken with great caution, since the nature of soil is complex, hence the real values on the site might vary several decades. Core samples on the site will help estimating the texture of the soil. Furthermore, clogging of the bottom of the pond will occur during years of operation, leading to a significant fall in infiltration capacity.
In the design of an infiltration pond – simply substitute Q_{out} with the infiltration capacity multiplied with the bottom area of the pond and convert to the right dimensional unit.

Example 2

The wet detention pond in example 1 (figure 4) should be designed. It has been stated that the pond may only exceed its capacity every ten year (T=10 yr). The municipality has given an outlet permission of Q_{out} = 5 l/s.

For a return period of T=10 years, the constants c and α for the rain intensity equation can be found in table 2 giving: $c=45960$ and $\alpha=0.79$. The reduced catchment area F_r is 6000 m^2 (0,6 ha). Hence from equation 6 we get:

$$Vol = (45960 \cdot t_r^{\ -0,79} \cdot 0.6\,ha - 5\,l/s) \cdot t_r \tag{9}$$

Instead of differentiating this (eqn. 9) and setting the derivative to zero in order to find the rain duration that gives the largest volume, equation 9 has been plotted in figure 8 for various rain durations (t_r).

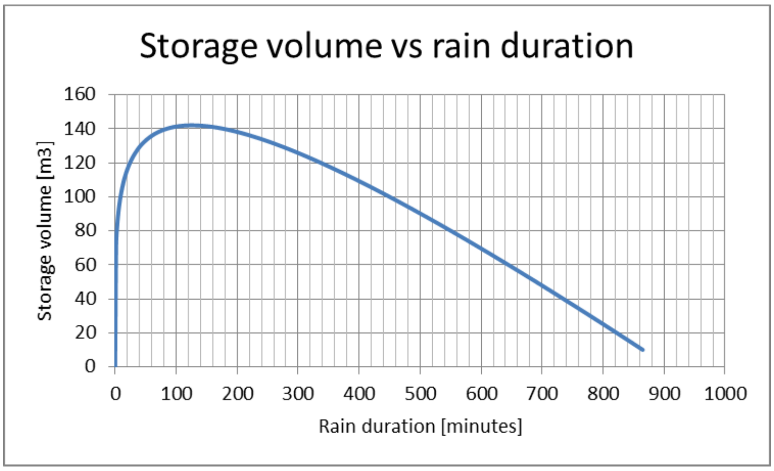

Figure 8 Storage volume as function of rain duration (t_r).

From figure 8, it can easily be seen that there is an optimum on the function. From the figure it can be seen that a rain duration around 125 minutes gives the largest storage volume of around 140 m^3. For longer rain durations – it's easily seen that the needed volume is decreasing again, since the intensity is falling – meaning that the ratio between outlet and inlet discharges is increasing.

The result deduced from figure 8, can also directly be calculated by use of equation 8:

$$t_{r,\max} = \left(\frac{-45960 \cdot (0.79 - 1) \cdot 0.6\,ha}{5\,l\,/\,s} \right)^{1/0,79} = 7554\,s \approx 126 min$$

Inserted in equation 6:

$$Vol = (45960 \cdot 7554s^{-0,79} \cdot 0.6\,ha - 5\,l\,/\,s) \cdot 7554s = 142096\,l = 142m^3$$

The storage volume for the wet detention pond in the system shown on figure 4 should be at least 142 m^3 (the volume placed above the outlet pipe from the pond)

How the geometrical shape of the pond should be, is for dry detention ponds irrelevant. The storage function works equally well for rectangular, circular or irregular shaped ponds. For wet detention ponds, attention should be paid to the geometrical design, so that the pond shape facilitates an improved settling possibility for the pollutants carried with the stormwater.

The easy approach – is stating that the pond is a rectangular pond made out of concrete. The ground plane of the pond could e.g. be 20 m long and 10 m wide – giving a necessary height above the outlet pipe of 0.71 m. In addition to the storage volume the permanent water level (the water depth below the outlet pipe) should be 0.8 m. In total this will sum op in a wet detention pond of 300 m^3.

Climate changes and uncertainties

The above mentioned methods for designing pipes and detention facilities hold to a certain degree. If more security is to be placed in the design phase; to compensate for the rather simple methods used, eventually more catchment connected to the system in the future (change of area usage) and to the more or less well documented climate changes that is ongoing at the present time and the future.

To incorporate these uncertainties and future rain statistics several safety factors have been proposed in the later years. Especially the climate factor is not a static parameter, since new rain data is ticking in all the time. However based on The Water Pollution Committees (SVK) paper no. 27 and 29, following safety factors are proposed (but not a legal requirement) [SVK, 2005 & SVK, 2008]

Table 5. Proposed safety factors. *) Note that already planned areas should be incorporated in the design phase, this factor is only covering the unforeseen changes of the connected reduced area. **) By use of more complicated models in the design phase as verification of the system, the low value should be used.

Increased catchment area	γ_a	1.1*
Statistical/model uncertainty	γ_s	1.2-1.3**
Climate change (DK)	γ_c	1.2 (2 years), 1.3(10 years), 1.4 (100 years)

For a system that has a lifetime of 100 years – this sums up to following safety factor:

$$\gamma = \gamma_a \cdot \gamma_s \cdot \gamma_c = 2$$

The use of safety factors (all or only one or two of them) is of cause of discussion. It must also be remembered that e.g. the pipe dimension is always rounded up – this gives in some cases a significant contribution of safety as well. In principal the choice of safety factors must rely on the consequences for failure of the system and the service level that should be provided.

For visualising where the safety factors should be incorporated in the design phase, parts of example 1 and 2 is given below:

For the pipe dimensioning, the safety factor should multiplied the design flow

$$Q_d = \gamma \cdot \varphi \cdot F \cdot i(t_f) = \gamma \cdot F_r \cdot i(t_f) \qquad (16)$$

Else the procedure is the same. If a safety factor of 2 has been used on example 1 following pipe dimension would have been the result:

Table 6. Necessary pipe diameters and trade dimensions from example 1, including safety factors

Pipe	Necessary internal diameter	Trade dimension
A	199 mm	250 mm
B	220 mm	250 mm
C	335 mm	400 mm
D	344 mm	400 mm
E	385 mm	500 mm

As can be seen from the table – all pipes have increased one or two dimensions. However the difference in material cost for these types of pipes are not large compared to the total construction costs of a system.

For the detention pond example, you should be a little more careful where to incorporate the safety factor. The right placement of the safety factor is given below.

$$\begin{aligned} Vol &= (\gamma \cdot Q_{in} - Q_{out}) \cdot t_r = (\gamma \cdot i \cdot F_r - Q_{out}) \cdot t_r \\ &= (\gamma \cdot c \cdot t_r^{-\alpha} \cdot F_r - Q_{out}) \cdot t_r \end{aligned} \tag{17}$$

Leading to the rain duration which gives the largest volume

$$t_{r,\max} = \left(\frac{-\gamma \cdot c \cdot (\alpha - 1) \cdot F_r}{Q_{out}} \right)^{1/\alpha} \tag{18}$$

It must be remembered that $t_{r,max}$ calculated from equation 18 should be used in combination with equation 17 and NOT equation 13, else it would provide a storage volume less than without safety factors! By use of a safety factor of two – the necessary storage volume has been increased to 342 m^3.

Note that the implementation of a very high climate factors for pond design, might be an overestimation, especially for very long design durations (rain durations), since the ongoing research on climate changes don't directly show that the intensity of long lasting rains should increase in the future.

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