

Hydraulic Engineering – Summary

Hydraulic engineering contains the knowledge on the design, construction and maintenance of structures and systems that are designed in answer to community needs for infrastructure that has to deal with water in all kind of ways.

After this course students are able:

- To explain the types, principles and objectives of flood embankments and hydraulic structures and their functioning, construction aspects, failure mechanisms and safety assessment in their societal context.
- To explain the failure mechanisms: retaining height, wave overtopping, piping, macro-stability, structural stability and probability of closing.
- To apply appropriate calculation tools for these failure mechanisms.
- To design measures to prevent these failure mechanisms.
- To perform a safety assessment and make a conceptual design of flood defence systems.

Chapter 1: Safety policy

1.2 Safety policy

Four approaches that have been considered in the Netherlands are (TAW, 1998a):

- Overload approach per dike ring/section (probability overload larger dan max overload).
- Flood (inundation) probability/risk approach (consequences of a flood (victims, damage)).

A dike trajectory (Dutch: dijktraject) is (a part of) a primary food defence with a certain safety requirement (Figure 1.1). A dike trajectory is a part of a flood defence that when flooded results in similar flood consequences after a breach somewhere in the trajectory. For example, the Afsluitdijk is one trajectory. A dike section (Dutch: dijkvak) is a part of a flood defence which has a more or less constant geometry and strength and which experiences similar hydraulic loads. The Afsluitdijk consists of a multitude of dike sections. Probability of failure for dike section smaller than for dike trajectory.

The safety standard (norm) of a flood defence is the allowable probability of flooding and can be expressed in the alert safety value (3 times smaller then lower threshold value) and the lower threshold safety limit (may not be exceeded).

1.3 Failure mechanisms and the failure probability budget

The failure probability budget (Dutch: faalkansbegroting) is the distribution of allowable failure probabilities over the different possible failure mechanisms so that the combination of these allowed failure probabilities does not exceed the safety requirement of the flood defence.

1.4 Safety assessment procedure

Uncertainty is included in a safety factor in the safety assessment since the used failure mechanism models are a schematization of reality.

Limit state design: The Limit State Design (LSD) design philosophy is widely used for the risk-based design and assessment of structures (Vrouwenvelder, 2002), as it allows to define the general limit state in a mathematical form. This general equation describes the failure state of a structure due to a certain deterioration process.

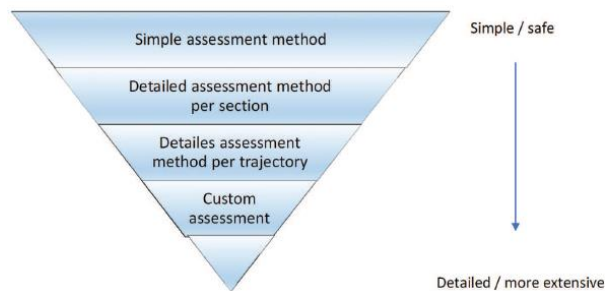
Fully probabilistic methods: Diermanse (2012) describes the fully-probabilistic approach as the 'royal route' to perform the assessment compared to the semi-probabilistic approach. In a fully probabilistic approach, the inputs are probability distributions, and the result is a calculated probability of failure

that can be directly compared to the legal norm. However, the computational load is for the fully probabilistic determination is usually significant, depending on the failure mechanism. Additionally, the fully probabilistic method can only be executed by experts, and the methods are relatively complicated.

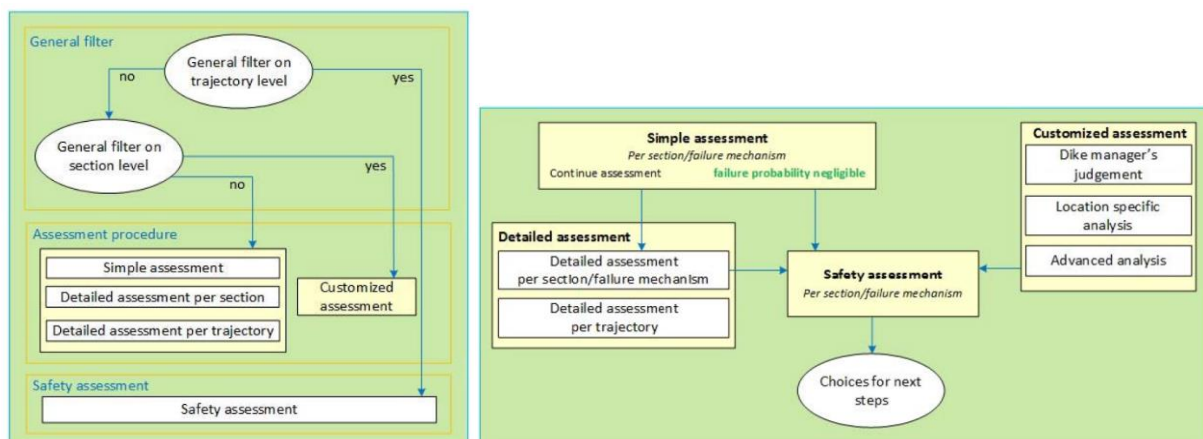
Semi-probabilistic methods: The semi-probabilistic approach gives a less accurate reflection of the reliability of the flood defence but is faster to perform. In a semi-probabilistic approach, the inputs are characteristic values for the parameters, derived from their probability distributions. The outcome of a semi-probabilistic assessment is a determination whether or not it can be assumed with enough confidence that the probability of failure is smaller than the target probability. The semi-probabilistic approach also involves safety factors that are to be very broadly applicable and therefore sometimes unnecessarily strict (conservative) (Diermanse, 2012).

Deterministic methods: Finally, there are the classic, deterministic methods, which at first glance appear similar to the semi-probabilistic methods. These methods also involve inputting deterministic values for load and strength properties into a limit state function. An essential difference between deterministic and semi-probabilistic methods is however that in the former there is no explicit relationship with required failure probabilities (ENW, 2017).

Safety assessment tracks: The idea behind the different levels of assessing a dike is to follow the path from coarse to more detailed. If a failure mechanism can be excluded in a simple generic way, the dike can be marked safe without an expensive analysis. If not, the dike must be examined in more detail using more sophisticated, but costly methods.



Safety assessment procedure: The safety assessment of a dike has to be done by following a certain flow chart, where the assessment goes from a coarse level (easy, conservative) to a very fine level (costly, more accurate, sharper). The assessment starts by applying two general filters: a dike trajectory filter and a dike section filter. If it is already known that the failure probability of the dike trajectory is much higher or much lower than the lower safety limit, an immediate conclusion may be given. If this dike trajectory filter is not applicable, a general filter needs to be applied to the dike sections. If application of the generic WBI tests may lead to unreliable conclusions, an advanced or customized test can be executed, which consists of a more advanced analysis.



1.5 Hydraulic boundary conditions

The design water level can be determined using numerical modelling or extreme value theory. In this section the determination of the water levels using extreme value theory is described and consists of three steps:

1. Selection of historical annual peak water levels or peaks above a certain threshold level with connecting the probability of exceedance (independent high water peaks → hydraulic years).
2. Fit the peak water levels to an extreme values distribution of probability (first rank peak values then use Weibull or Gringorten formula to determine probability of exceedance of different water levels).
3. Extrapolation of the fitted extreme values distribution of probability to the design return period (use Gumbel distribution to find the design water level for the accepted probability of exceedance).

Bretschneider equation is used to determine the wave height and wave period, based on wind, water depth, and water surface characteristics. The Bretschneider equations assumes that the wind blows over a water surface with a constant speed and that the wind blows until the maximum wave height is reached. The time that a wave needs to develop completely over the maximum available fetch (Dutch: strijklengte), which is approximately 5 km, depends on the wind speed and is 45 to 60 minutes.

Soil mechanics:

1. Total vertical stress: $\sigma_{zz} = d * \gamma_{soil}$
2. Effective vertical stress: $\sigma'_{zz} = \sigma_{zz} - p$
3. Effective horizontal stress: $\sigma'_{xx} = \sigma'_{zz} * K$
4. Total horizontal stress: $\sigma_{xx} = \sigma'_{xx} + p$

Exceedance probability closed structure = 10*exceedance probability open structure = 100*collapse probability

Resilience is the ability of the system to withstand a loading above its design standard. Resilience strategies are more important for systems with low return periods than for systems with high return periods.

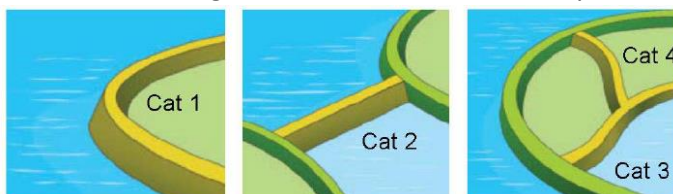
Chapter 2: Flood embankments (e.g. dikes/levees) → Soft structures

Types, principles, failure mechanisms and safety assessment tools

2.1 Dike categories

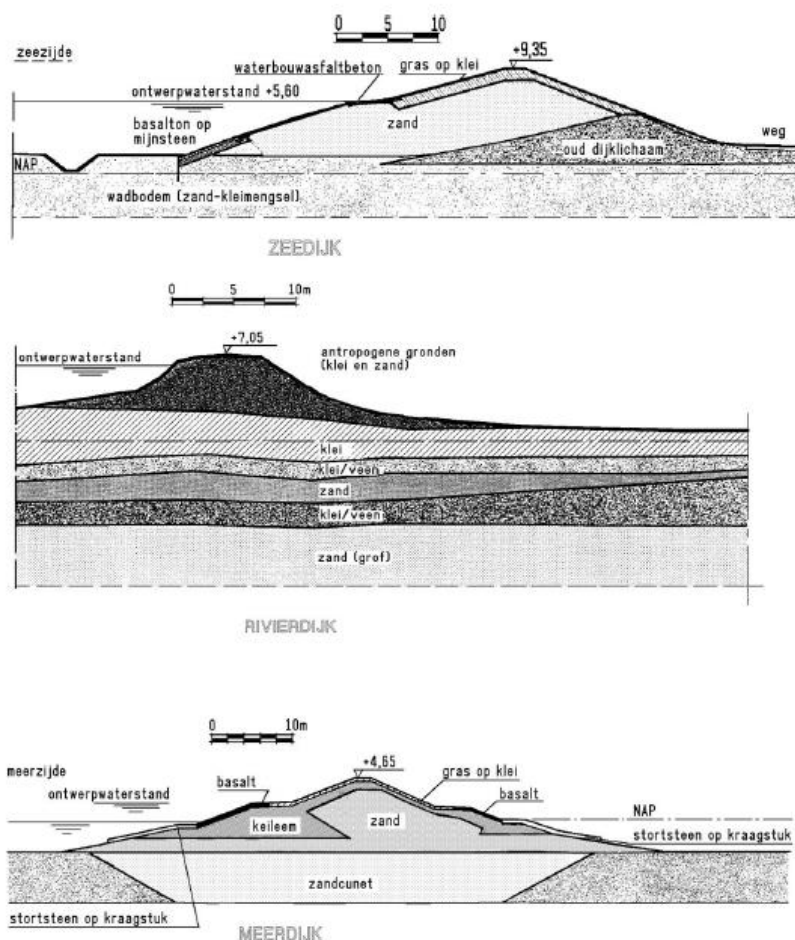
Categories based on its function:

- Category 1: Direct protection (sea/river) → Separate outside water and land
- Category 2: Connection dams → Direct retention of outside water between two other structures. No land between the two structures, only water.
- Category 3 and 4: Indirect protection → Prevent the occurrence of too high levels of water behind the dike and limit the loads on the flood defences separating the water behind them from the land in front of them. Flood defences in category 3 are situated in front of a dike ring area and retain outside water. Flood defences in category 4, as in category 3 are situated in front of a dike ring area but do not retain directly outside water (backup system).



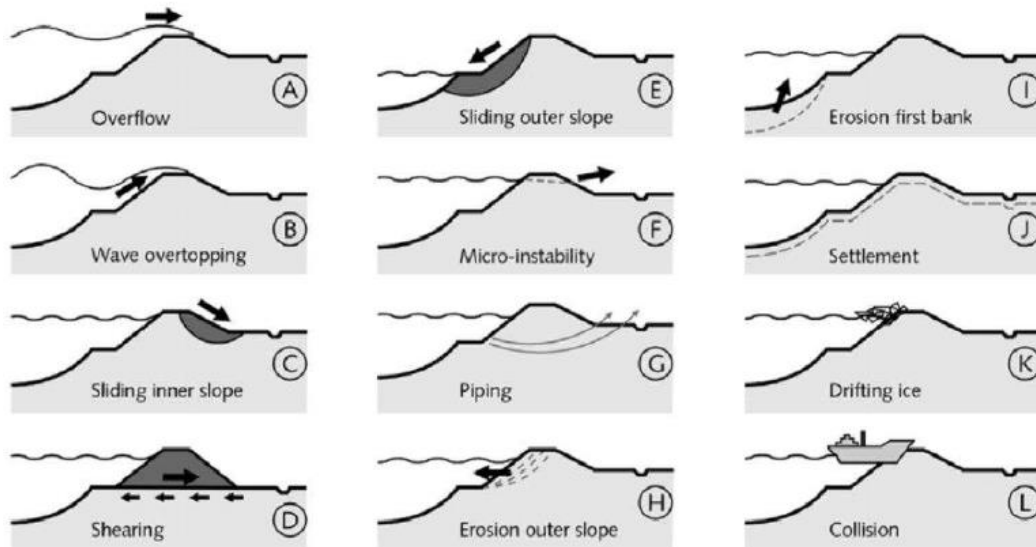
Categories based on its location:

- Sea dikes: The duration of the loads on sea dikes is short but the loading by waves is very strong. Protecting sea dikes against forces like wave overtopping and scour at the toe is important. This can be counteracted by creating an outer slope with a greater roughness, steeper slope or by constructing an outer berm, which should break the waves to ensure that the wave run-up is less. A modern sea dike is made out of sand and clay. Sand because it is cheap, available and it will drain the water. Clay because it is more or less water-tight and has a reasonable resistance against erosion.
- River dikes: On river dikes the duration of the high water is longer than on sea dikes (order days instead of hours). Therefore, river dikes suffer more from saturation of the dike than of direct damage due to wave action. River dikes began to be built by the inhabitants in early centuries. In the Eastern part of this country their foundation is stiff sandy material but very permeable. Making the dikes and the subsurface watertight was often a major problem. Due to the sandy subsoil often the failure mechanism piping was the main source for dike breaches. In the western part of this country settlement and stability problems often created problems as well, caused by the soft subsoil.
- Lake dikes: Lake dikes sometimes have to retain water all the time, so seepage water will always be present. Controlling the groundwater (phreatic water) inside the dike needs special attention in the construction phase. Failure of lake dikes is often related to groundwater problems, e.g. a drainage system that is not functioning. Excavating the foreshore and giving way to an excess of water penetrating the subsoil are known causes for failure. Other times the water resistant layer at the outer slope is leaking. For lake dikes, the wave action is larger than for river dikes, but not so large as for sea dikes.



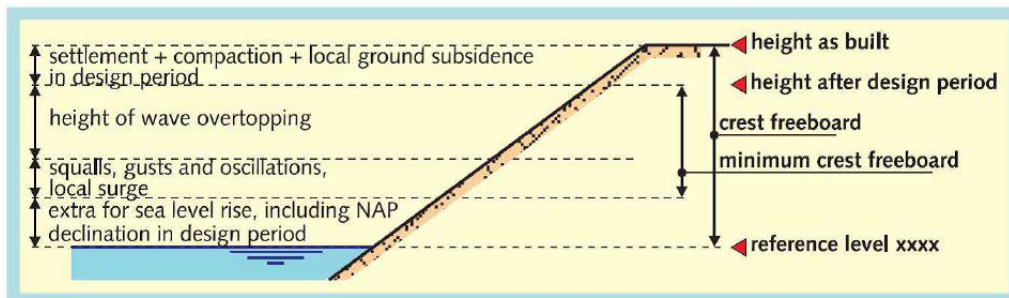
Common Dutch practice for a dike is to apply a 1:3 slope on the landward (inner) slope and between 1:3 and 1:5 on the waterside (outer) slope. The minimum crest width is 2m to be able to drive over the dike for inspections.

2.2 Flood embankment failure mechanisms



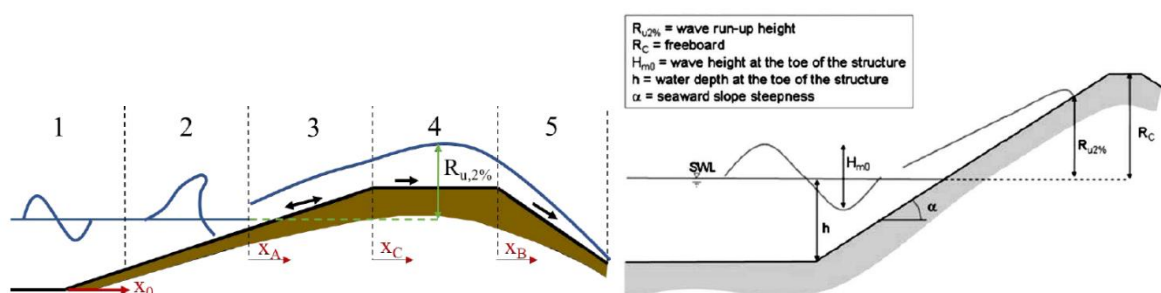
2.3 Dike retaining height

The most important hydraulic boundary conditions for a dike safety assessment are the design water level and the design waves that depend amongst others on wind speed and wind direction.



2.4 Wave run-up and overtopping

The effect of wave overtopping can be separated in the water load, defined as the amount of water that overtops the dike, and the erosional effect on the landward slope (the strength of the grass cover). Either one of them is the limiting factor (usually the quality of the grass cover). The water load is described by either the wave run-up, the average discharge q [m^3/s] or the overtopping volume V [m^3]. The wave run-up determines the condition for wave overtopping. When the run-up height $R_{u,2\%}$ (measured vertically from still water line) is larger than the free crest height R_c , the waves overtop.



Often a ditch behind a dike to get rid of the water is overflow occurs.

Influence factors:

- Oblique waves: When a wave comes perpendicular onto a dike this will cause the greatest impact on the dike. When the waves approach the dike at an angle the force of the wave on the dike will be reduced.
- Waterside berm: A berm is often introduced to reduce wave run-up and wave overtopping if the wave height is large, which is often the case in sea dikes or lake dikes. The most effective berm is when the berm level is located at the still water line (=limited to $\gamma_b = 0.6$).
- Roughness of waterside slope: The the effect of roughness decreased with the increasing of the mean overtopping discharge. The wave steepness at the toe of the structure also has an influence on the roughness factor, but not influenced by the slope gradient.
- Vertical wall on top of flood embankment: A relatively small vertical wall on a slope reduces the wave overtopping compared to a 1:1 slope (or gentler), which is present at the top of a dike profile.

Measures to prevent wave overtopping:

- Add berm at water (outer) side of the dike to break the waves earlier.
- Less steep slope at water side of the dike

2.5 Uplift and heave

→ Do not directly lead to dike breach but indirectly do affect flood embankment stability, mainly by increasing piping and macro stability risks.

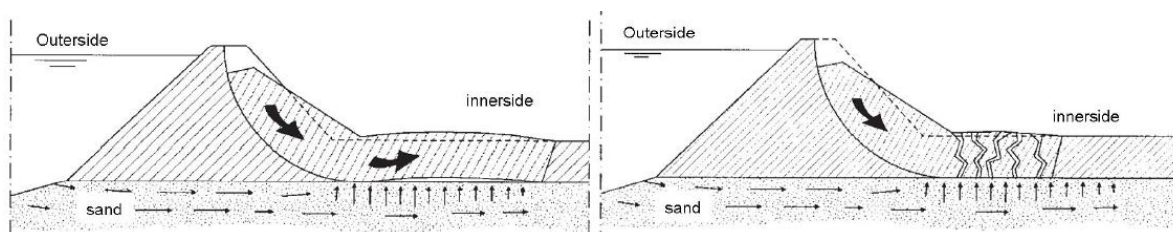
Uplift occurs if the upward water pressure below an impermeable set of cohesive cover layers is larger than the downward pressure of the layers themselves, due to their combined weight.

- Loss of shear between the impermeable, cohesive layer (or aquitard) and the sand layer (or aquifer) → More vulnerable to macro-instability.
- Lifting may cause cracks in the cohesive layer → Enables enhanced seepage and piping.

Heave may occur if cracks are present in the cohesive layer. Heave is the process of liquefaction of sand due to upward seepage flow.

- Floating sand particles → Reduction internal shear and → Sand loses its strength.
- Heave is required for piping to occur (seepage).

Usually, heave occurs before uplift occurs and uplift is therefore usually normative (uplift is a requirement for heave to occur). Only in case of thick peat cover layers (in the West of the Netherlands), the heave criterion is normative.

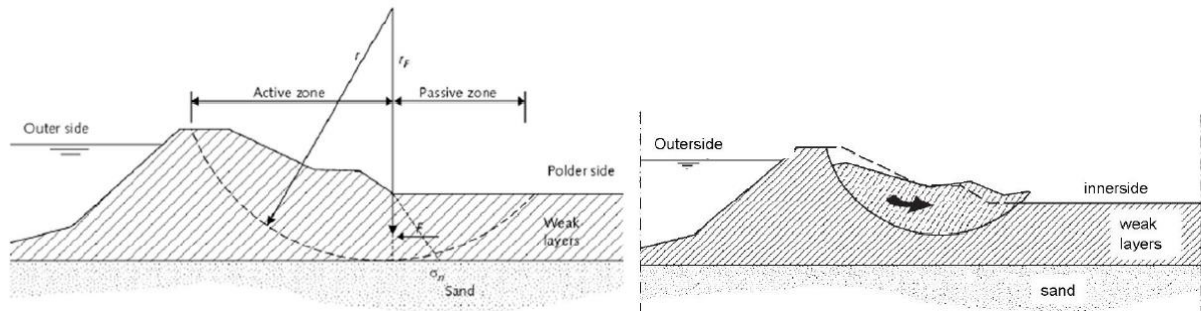


Measures to prevent uplift:

- Increase thickness/weight cohesive layer → gentler landside slope or berm.
- Increase the phreatic head in the covering layer (i.e. increase the groundwater level) to reduce hydraulic head difference.

2.6 Macro-stability (inner and outer slope)

Macro-stability is the resistance against shearing of large sections of the dike slope, along straight or curved slip planes. Instability is a result of a lost equilibrium between the active or driving forces and the passive or resisting forces. The driving forces consist of the weight of the soil of the dike in the active zone. The resisting forces consist of the soil and/or water next to the dike and the shear resistance along the slip plane. Macro instability can occur on the inside (due to high water level) or outside (due to a quick fall of the water level) of the dike.



Saturation of a dike (due to increase in water level outside of the dike) leads to smaller effective stresses and therefore less internal shear stress, since the water pressure in the soil increases. Moreover, the soil weight will increase which increases the driving forces.

Bishop's method is used to determine the slip circle with the lowest safety factor by taking different slip circles with different radii and centre point divided in different slices and determine the forces on the different slices.

Uplift-Van Limit Equilibrium Model does not take into account horizontal parts of slip plane since shear along the bottom of the impermeable layer is negligible if uplift is present.

Spencer-van der Meij Limit Equilibrium Model can account for any shape of slip plane.

In the end, the model which results in the slip plane with the lowest safety factor is the normative slip plane.

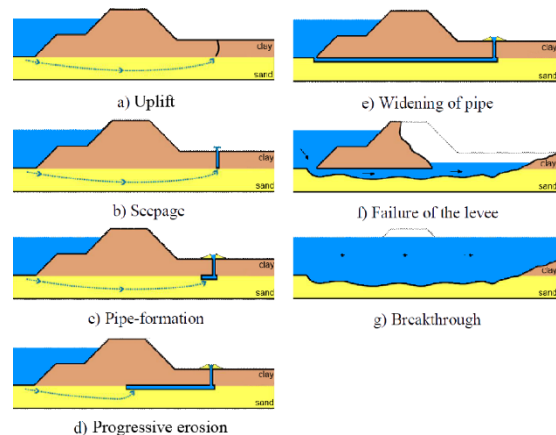
Measures to prevent macro-instability:

- Increase resisting forces by adding a berm at landward side of the dike.
- Add sheet piles so that the location of the slip-circle changes

2.7 Piping

→ Backwards erosion

- Increased water pressures can cause crack in blanket layer → Heave/uplift
- Water flows through the cracks → Seepage
- Sand transport → Sand boil behind the dike
- The pipe grows → Process slows down as a result of decreasing force of the flow as the pipe grows = limit state
- The pipe has reached the other side → Contact between polder and river
- Pipe erodes quickly
- Breach



Bligh's rule only takes into account the horizontal seepage length with correction for vertical piping channel at the landward side of the dike which results in an additional resistance of $0.3 \times \text{thickness of the impermeable layer}$.

Sellmeijer is a more detailed method → Takes into account soil type, porosity and grain size variations.

Lane equation is used when seepage length is not only horizontal but also vertical due to an obstruction in the ground below the dike. When structure is on piles → Zero horizontal seepage length.

Measures to prevent piping:

- Increase seepage length → Add impermeable berm at waterside of the dike.
- Prevent uplift (increase thickness covering layer inner slope)

2.8 Other failure mechanism

Micro-stability concerns the stability of soil layers of very limited thickness on the surface of the inner slope. Micro-stability is caused by a high phreatic line in the core of the dike, resulting in groundwater flowing out of the dike at the inner slope. Under influence of the flow of out flowing groundwater, soil particles are washed away from the slope.

Erosion of the waterside slope of the dike construction is mainly caused by strong currents in the water that flow close to the dike toe. When the stone or grass revetments start to get loose and move from its original position, this is called instability of revetments. After this the erosion of the under layer can take place. → Erosion on the outer slope of the dike can also be caused by wave attacks.

Settlement through consolidation (Dutch: zetting) is a process by which soils decrease in volume. It occurs when a load is applied to a soil, that causes the soil particles to pack together more tightly, and therefore reducing its bulk volume. When this occurs in a soil layer underneath a dike which is saturated with water, the water will be squeezed out of the soil, with settlement of the dike as result. Settlement through subsidence (Dutch: klink), is the compaction of the subsoil due to the weight of the dike or structure itself.

A pipeline within a dike reduces the dike safety in two ways: 1) by leaking of the medium from the inside due to a calamity with the pipeline or 2) by leakage of water along the pipeline surface.

Buildings influence loading, geometry, soil behaviour, water pressure development and erosion. It is necessary to check whether the buildings influence stability negatively with respect to micro-stability (wind loading, own weight and vibrations), settlement (due to the vibrations of turning windmills), piping (if basements of buildings intersect covering layers), erosion (depending on the location and the overtopping capacity) and micro stability (if the buildings are on the inner slope, they may intersect a covering clay layer and give rise to leaks along the wall).

Problems with ice mainly occur due to ice that gets blown onto the dikes and breaks up the dike.

Collision of shipping on a dike.

Non water retaining elements are all kind of elements that can occur on dikes, but have no water retaining function (e.g. buildings and trees). These objects often increase the risk of dike damage.

Rock revetment stability

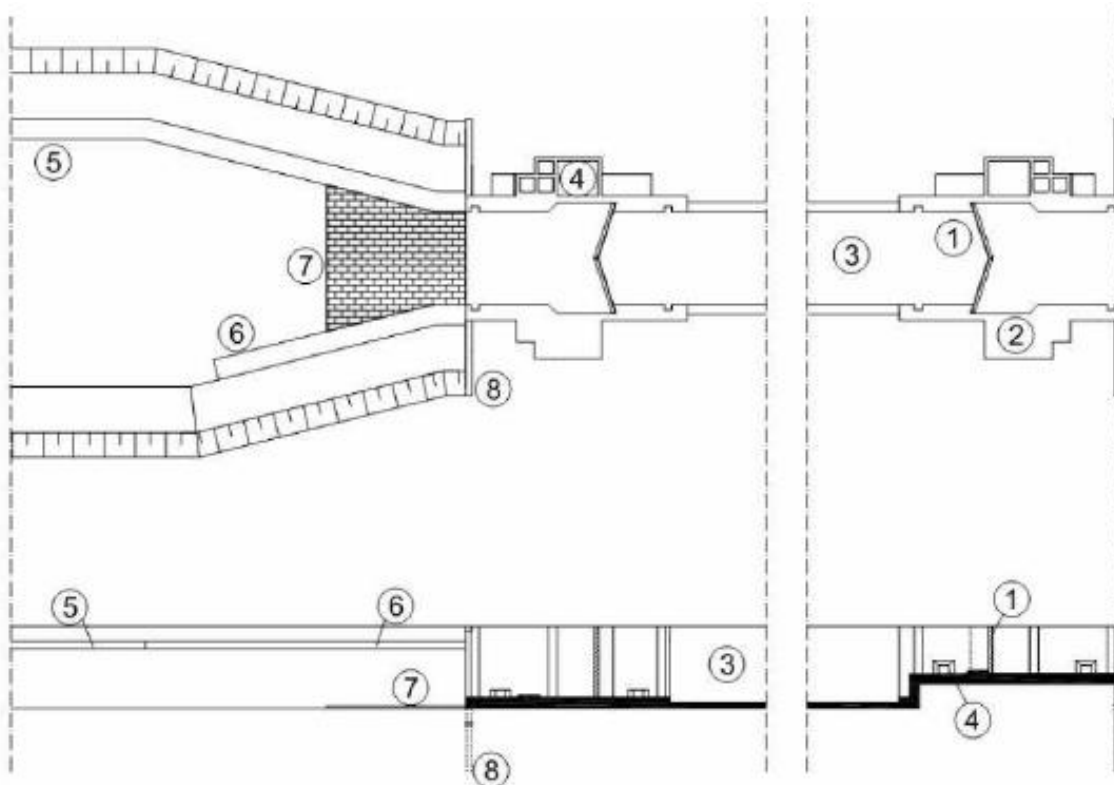
Blocks ensure subsoil of the structure to stay in place. Stability of the blocks determined by hydraulic load (force of the waves, breaking and run up and down of waves) and the weight of the blocks itself. Structure of the blocks determines how the force of waves gets to the subsoil. Most important failure of rock revetment is that the subsoil under the blocks will be 'flushed' away → Block will 'inzakken'.

Chapter 3: Hydraulic structures (e.g. barriers, locks) → Hard structures

Types, principles, failure mechanisms and safety assessment tools

3.2 Lock construction

1. Lock gates
2. Lock heads
3. Lock chamber
4. Filling and emptying system
5. Waiting berths
6. Guard wall or lead-in jetty
7. Bottom protection → Change in water level by lock + propeller wash caused by navigating ships causes local currents which can cause a scour hole in front of the lock head
8. Cut-off walls and screens (sheet piles) → Prevent piping underneath/along the lock

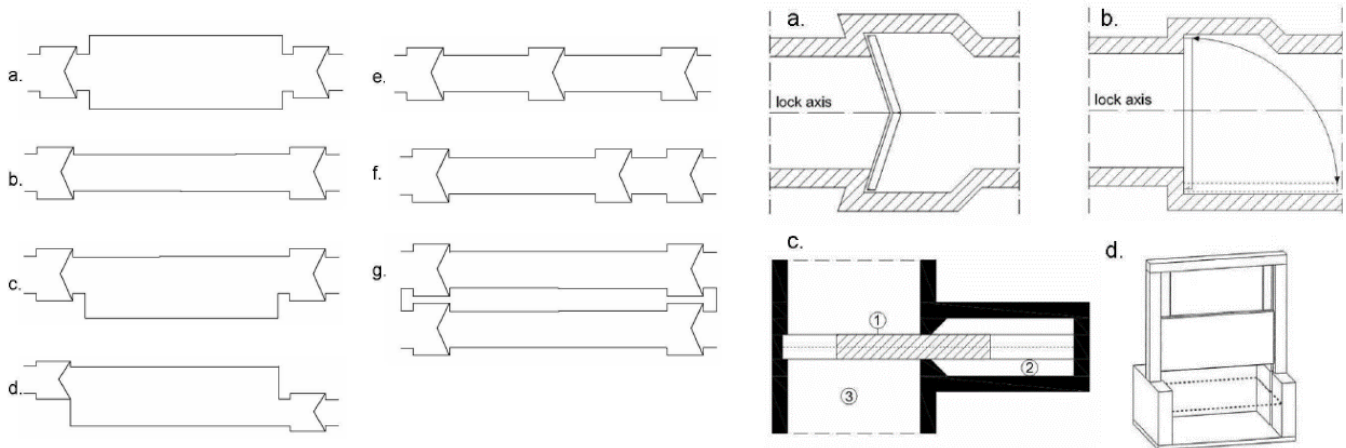


Types of locks:

- a. Lock with inner fronts → More ships, more change of collision, more space needed
- b. Smooth lock → Most common, ships behind each other so longer gate, much water loss in pounding process
- c. Chamber lock → More ships, more space, LiFo
- d. Bayonet lock → More ships, more space, FiFo
- e. Staircase lock → coupled smooth lock to overcome large water level differences
- f. Lock with intermediate lock head → staircase lock with different chamber sizes to adapt to ship size so less water loss
- g. Twinned lock → Busy channels, less waiting time, exchange water between chambers, less water loss

Types of gates:

- a. Mitre gates → Unsuitable as weir or flood gate since they cannot deal with significant currents or waves when partially open, cannot be used when water levels vary between outside water and lock chamber
- b. Single leaf gate → Only for locks with a small width, can be closed in flow conditions, cuts through water with small surface, relatively large amount of material needed
- c. Rolling lock gate and → Used in locations with varying water levels between the outside and lock chamber, no limiting vessel height, lot of space needed for lock head
- d. Lift gate → High guide towers needed, little space needed for lock heads, limited vessel height



Filling and emptying system:

- Shutters at bottom of the lock → Barrier behind shutter in chamber to break inflowing water
- Water flow through the walls or floor of the chamber → Bypass channel alongside the lock which allows for controlled water inlet into the chamber

Foundation:

- Shallow foundation (funderen op staal) → When soil can bear the load of the lock directly or improve the soil so it still can bear the load (remove non-bearing soil underneath structure)
- Pile foundation

Lock walls:

- Sheet pile wood or steel → Use anchors to resist the forces of the soil
- Concrete → carry forces of the soil onto the foundation

Lock dimensions:

- 1.1 times the length of the design vessel
- 1.25 times the width of the design vessel

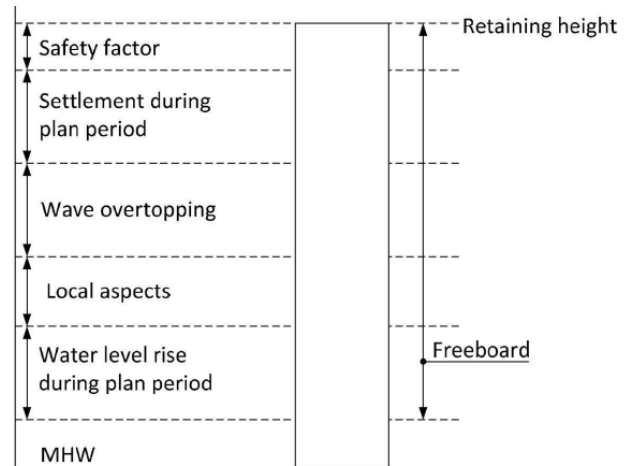
3.3 Failure mechanisms for hydraulic structures

Failure of a hydraulic structure can occur due to two reasons: there is too much water in flow (condition 1, Dutch: waterbezwaar) OR the structure collapses (conditions 2, assessment track: strength and stability of the structure).

3.4 Retaining height (HTKW)

The difference with dikes is that generally for structures overflow or overtopping can be allowed, because structures are less prone to erosion and storage might be available behind structures.

To determine if there has to be added an extra height for overflow (due to wave overtopping or overflow) there first has to be determined if overflow is allowed → Depends on maximum capacity behind structure (q_{store}), discharge due to wave overtopping (for DWL condition, q_{wave}) or maximum overflow due to damage on the structure (q_{flow}).



Settlement is the decrease of bed level below a structure, caused by the load of that structure. Often the case if the structure is built on weaker soils like peat. Two phases:

- Primary settlement: Expulsion of water from the voids in the soil layer
- Secondary settlement: Change in volume of a fine-grained soil caused by the adjustment of the internal structure.

3.5 Reliability of closure mechanisms (BSKW)

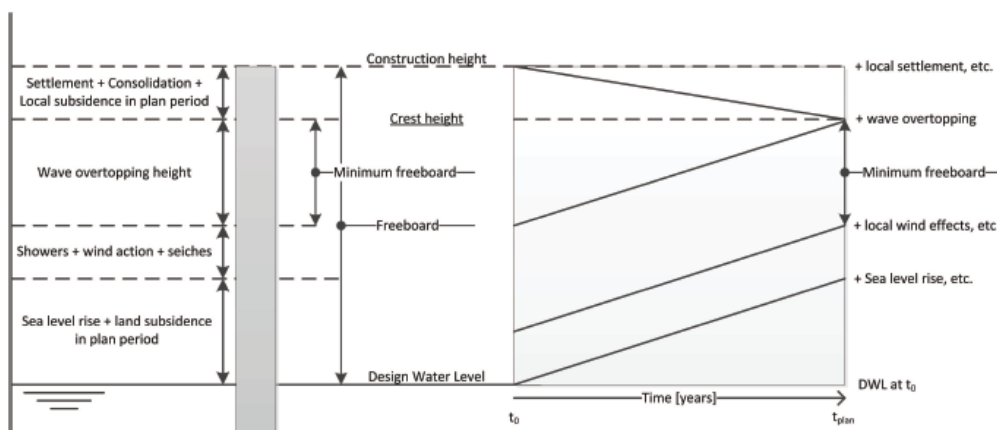
Simple method requirements:

1. High water warning system
2. mobilization procedure
3. Closure procedure
4. Reliability of closure components

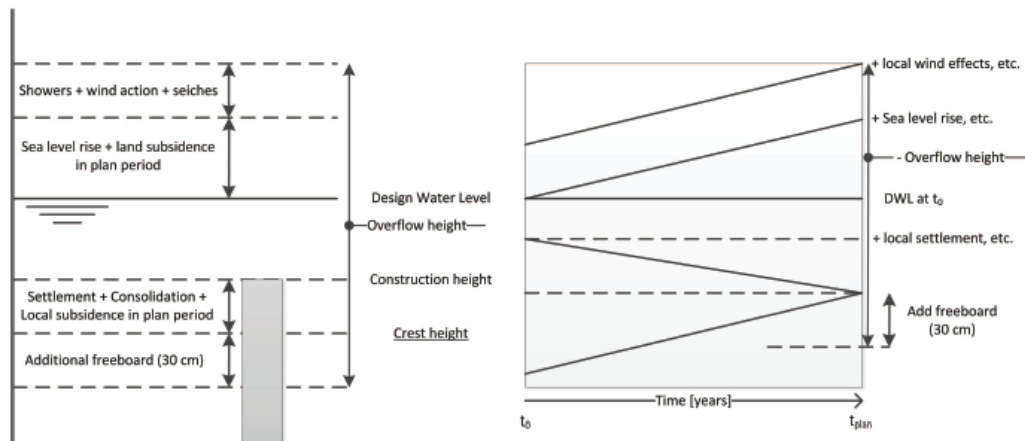
Detailed method steps:

1. Investigation of functions and possible states of the lock
2. Determining the retaining height of the open construction for every state
3. Determining the exceedance probability of the retaining height of the open construction for every state (n_j)
4. Probability of non-closure per state (P_{nc})
5. Comparison with the norm ($P_{\text{mechanisms, section}}$, the maximum allowed exceedance probability of due to reliability of closing for the dike section)

a) Crest height > DWL



b) Crest height < DWL



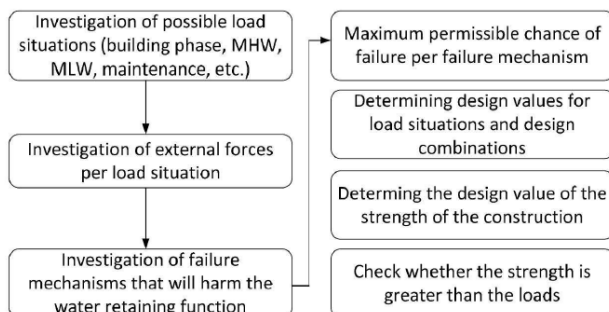
3.6 Piping (PKW)

Piping can result in sliding of the structure (less horizontal resistance), breaking of the structure (less vertical strength) or turning of the structure (less vertical strength at one place causing a moment over the structure).

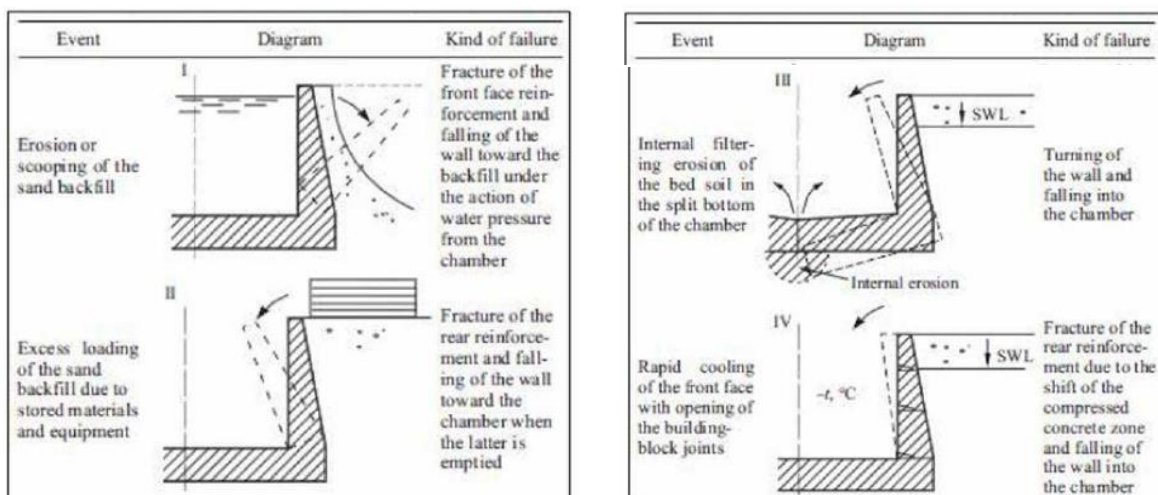
For piping along the structure (only horizontal seepage lengths) the method of Sellmeijer is used. When vertical screens or sheet piles are present, the method of Lane is used → in case of a pile foundation, the horizontal piping length in the Lane equation is set to 0.

3.7 Strength and stability (STKW)

P_{cf} is maximum allowed probability of constructive failure.

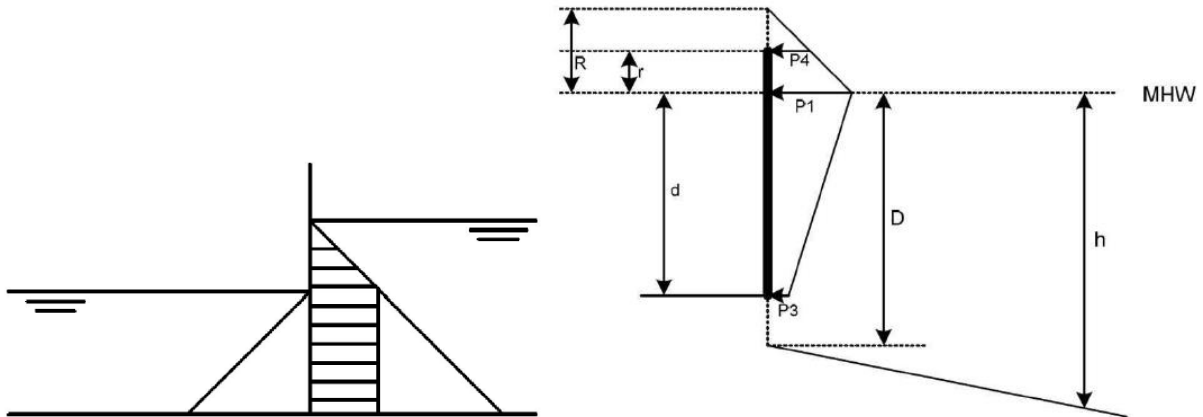


Failure of lock chamber walls:



Hydrostatic pressure: Pressure from the water level at both sides of the gates $\rightarrow P = \rho g z$

Hydrodynamic pressure: Pressure due to waves attacking the gates $\rightarrow$ Determined with Method of Goda.



3.8 Stability of bottom protection

Water is owing in and out the lock when the gates are opened. This can lead locally to high velocities around the lock heads. These velocities can cause sedimentation, which will decrease the stability of the lock. Therefore, this should be prevented by using sediment that is difficult to move.

3.9 Other failure mechanisms

Earth removal can take place due to tidal difference and water level difference. This will undermine the strength and stability of the lock. Can be prevented by placing sheets before the construction or apply bottom protection which is less likely to be eroded.

