6. TECHNICAL DESIGN

To convert Bergenmeersen from a flood control area (FCA) to a flood control area with controlled reduced tide (FCA-CRT), the existing dykes were modified and a new inlet and outlet construction was built. This chapter outlines the hydraulic and geotechnical design. This encompasses raising the existing ring dyke around the area, the new stability calculations and the modified dyke revetment along the water and land side. The inlet and outlet structure is also described. The hydraulic boundary conditions are extremely important to the design.

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6.1 Hydraulic boundary conditions

These hydraulic boundary conditions are important for the design: water levels, water velocities, wind waves and ship induced waves.

6.1.1 Water levels

For the updated Sigma Plan, the dyke (crest) heights are fixed. They were determined for the entire Sigma area on the basis of a study of social costs and benefits. It follows from the assumptions of the update study for the Sigma Plan that dykes along the Sea Scheldt and tributaries must be water-retaining to water levels equal to 0.5 m below the crest of the dyke. Non-water-retaining infrastructure such as overflow dykes and inlet and outlet constructions may not fail structurally at these water levels. In concrete terms, this means that the constructions are designed for a high water level of +7.5 m TAW for Bergenmeersen.

No unique return period was assigned to the water level to be retained (or the design water level). Because the updated Sigma Plan is being implemented on a phased basis, safety will gradually increase in the period 2010-2030 (or the probability of the design water level will decrease). The interplay of climate change, economic and ecological developments and additional measures will then maintain the risks of flooding at socially acceptable levels.

In addition to the maximum water level of +7.5 m TAW, the variation of the level is also
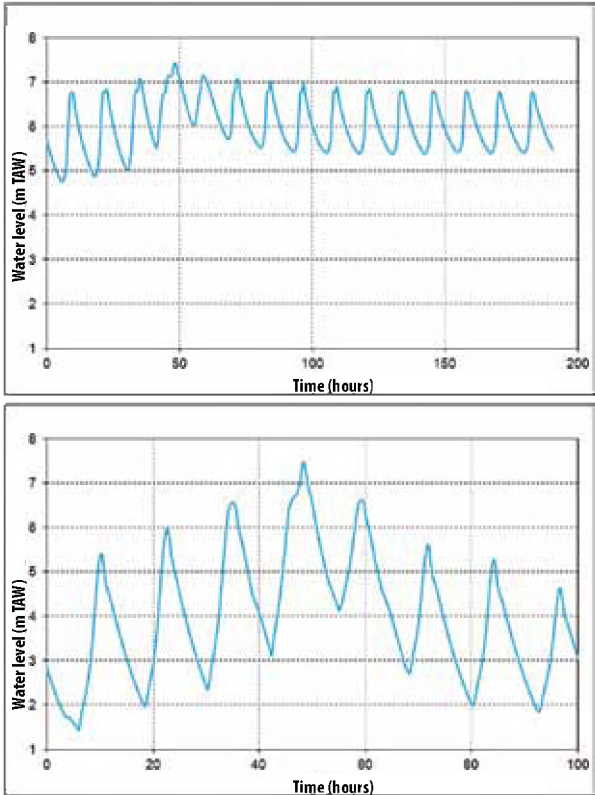
Figure 6.1. Two examples of level variations used for the design: small level variation (above) and large level variation (below)
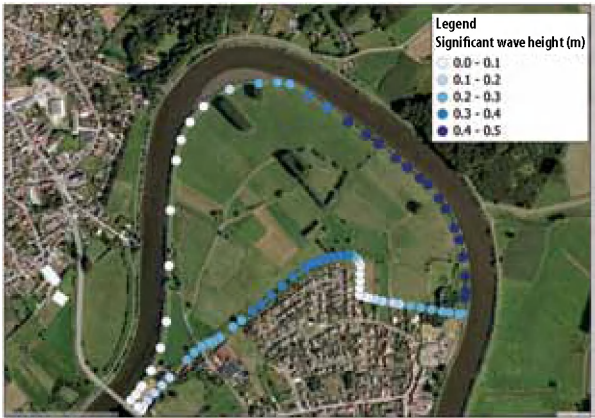

Figure 6.2. Wind wave load on the dykes of Bergenmeersen

extremely important. A rapid fall or rise in the water level, for example, leads to high flow speeds through the outlet structure. If the water level remains more or less constant for a long time, the load on a grass covering on the dyke increases. Allowance was therefore made for different variations in level. In total, 15 different hydrographs were taken into account in each case. These hydrographs were determined on the basis of a thorough statistical analysis of water levels in the Scheldt. Figure 6.1 shows two extremes. Both are characterised by the same maximum water level of +7.5 m TAW, but the variation of the levels differs significantly.

6.1.2 Water velocities and waves

In addition to the design water level, the load on the hydraulic structures is also caused by water velocities, ship induced waves and wind waves. The water velocities were determined on the basis of one-dimensional hydrodynamic simulations of the Scheldt; the ship waves were calculated using the DIPRO method. As with the water levels, no return period was assigned to the design ship waves and water velocities.

Wind waves, on the other hand, were estimated for various return periods. They were calculated using Wilson's empirical formula, given the wind direction, wind speed and fetch length, and then combined with the design water levels.

Figure 6.2 shows an example of the wind wave load on the flood area side of the overflow dyke and the ring dyke. The eastern dyke segments clearly experience a greater load than the western segments.

6.2 Dykes: calculation method and safety

6.2.1 General

Bergenmeersen is surrounded by dykes with a total length of 3.2 km, comprising 2 km of overflow dyke and 1.2 km of ring dyke. The crest of the overflow dyke is at +6.4 m TAW, the crest of the ring dyke at +8 m TAW. The slopes are relatively gentle, with a gradient of 12/4 on the Scheldt side and 16/4 on the land side.

The dykes are inspected for macrostability (sliding of the land or river slope), uplifting of the water-retaining layer (microstability) and seep erosion (piping and heave) beneath and along the dyke.
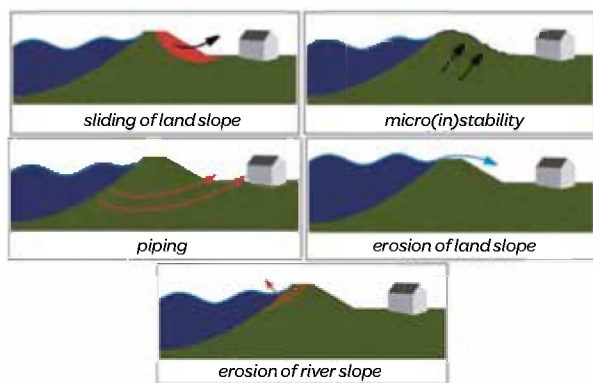

sliding of land slope	micro(in)stability
piping	erosion of land slope
	erosion of river slope

Figure 6.3. Failure mechanisms investigated

The land side of the overflow dyke is loaded by the high flow speeds of the overtopping water when the flood area is filling. The land side of the ring dykes can be loaded by overtopping water as a result of wind waves. A suitable revetment must prevent erosion of the land slope as a consequence of this

phenomenon. Every effort was made to find an ecological solution.

The water side of the ring dyke is loaded by the impact of wind waves. Once again, a revetment that is suitable but preferably as natural as possible protects the dyke against possible erosion of the river slope.

Strictly speaking, settlement is not a failure mechanism. However, settlement calculations were carried out to be able to determine the initial settlement allowance of the dykes.

6.2.2 General stability

To check general stability, both stationary (with constant values for the water levels) and non-stationary calculations (with variable water levels) were carried out. The calculations were carried out with the Plaxis finite element package and in Geostudio.

An undrained calculation was also carried out in Plaxis. This allows the implementation period of the various construction phases to be taken into account. Stability in Plaxis was determined using the φ-c-reduction. Within this calculation, the tan φ and cohesion c of the ground are gradually reduced until (part of) the structure fails.

For the interim phases (such as the construction of the sand core), a minimum safety of 1.1 was required; for the final phase (construction of final dyke profile, possibly fully consolidated), a minimum safety of 1.3. This value was achieved for all dyke segments. Figures 6.4 and 6.5 show two typical results of this modelling.

6.2.3 Uplifting

To avoid uplifting, it was checked whether the upward water pressure below the impermeable layer on the land side is greater than the dead weight of the earth package. If that is the case, earth will be pushed up and a leak will appear on the land side. If that phenomenon occurred, further checks were performed to see whether the resulting flow through the dyke remains limited. No problems with uplifting were found for the dykes of Bergenmeersen.

6.2.4 Seep erosion

Two types of seep erosion play a role: piping and heave. Piping develops along the underside of the structure, more or less in a horizontal direction. This forms an erosion channel. Heave is vertical, and is associated with the disappearance of the effective grain pressures. The grains of sand are picked up and rinsed away by the water, as it were. The critical (outflowing) hydraulic gradient is limited to 0.5.

Piping is irrelevant for dykes on permeable and cohesionless subsoil. No piping can occur if material rinses out at the inner toe of a dyke. This is because the hole immediately caves in, since sand is cohesionless. For dykes on a subsoil that is cohesive and poorly permeable (clay and peat), piping will only occur if the earth can be lifted up outwards from the dyke.

If the dimensions satisfy one of the following criteria, there is no danger of piping:

- $\Delta g / \Delta w > 1$
- $L / 18 > \Delta H - 0.3 d$

No seep erosion problems were found for Bergenmeersen.
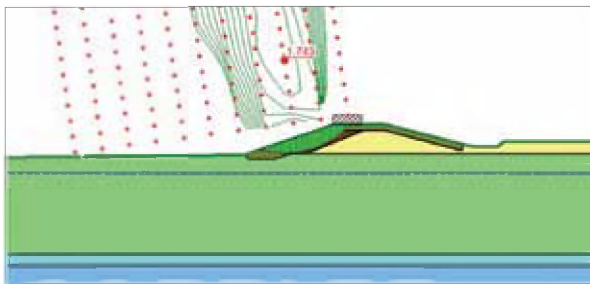

Figure 6.4. Example of sliding at the GEO-08/228-S7 test bore (outwards LW linear, Geostudio)
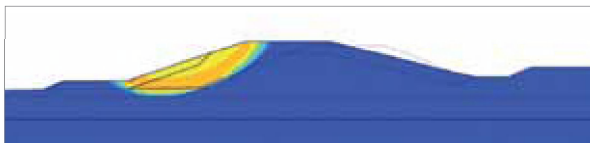

Figure 6.5. Example of sliding at the GEO-08/228-S7 test bore (consolidated LW Plaxis)
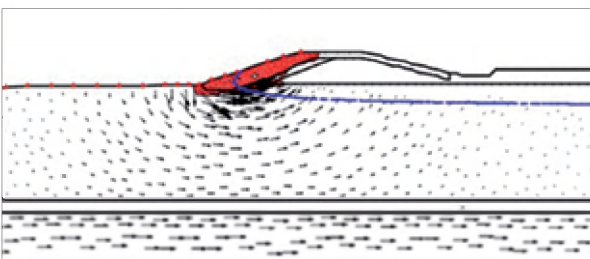

Figure 6.6. Flow at the GEO-08/228-S7 CPT. There is flow below the dyke, but the hydraulic gradient at the outflow does not exceed the critical value.
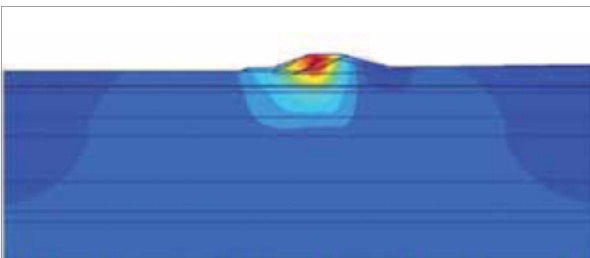

Figure 6.7. Vertical movements at CPT S5, consolidated condition (Plaxis, maximum 13 cm)

6.2.5 Settlement

Even after the dykes have been constructed, further settlement occurs. Settlement was checked using Plaxis and verified by manual calculations (according to Terzaghi). Based on the soil characteristics and layer structure, a choice was made beforehand of where the minimum and maximum settlement is expected (based on the CPT numbers).

The settlement was checked in drained and undrained conditions. For the undrained condition, a realistic estimate was made of the minimum implementation periods for the different phases. In this way, the expected consolidation of the materials was taken into account.

The settlement of the ring dyke is 3 to 13 cm. It was proposed that this dyke be constructed with a settlement allowance of 15 cm at the crest and that the settlement allowance be decreased to 0 cm towards the toe of the dyke.

6.2.6 Erosion of the land slope

The water that flows over the overflow dyke is moving at high speed and can cause erosion of the slope and the toe of the dyke on the side of the flood area. To avoid dyke instabilities, slope protection and soil protection at the toe are provided.

The supercritical flow over the slope changes to a subcritical flow downstream from the toe of the dyke. This occurs with a hydraulic jump. To position the hydraulic jump above the soil protection, an energy dissipater (stilling bassin) is installed at the foot of the overflow

dyke. This takes the form of a lined channel that not only drains water, but also acts as a dissipater.

To dimension the energy dissipater channel, a software program was developed that calculates the water levels and water velocities from the crest of the overflow dyke, over the slope and into the channel, as far as the flood area. The calculations took account of the filling of the area behind. The result: a channel with a depth of 0.5 m, a bottom width of 2 m and 8/4 slope gradients is sufficient to fix the hydraulic jump.

The maximum water speed over the overflow dyke is reached between the start of the overflow and the maximum level in the flood area. The maximum water velocity for Bergenmeersen is 6.3 m/s. In that case, the most suitable slope protection on the dyke is open stone asphalt (OSA). The open structure prevents water overpressures from the dyke acting on the revetment. To prevent erosion, the channel also has to be lined with OSA.
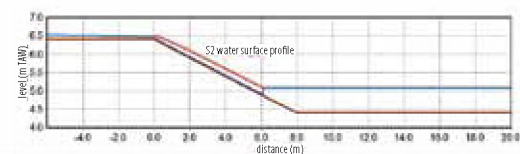
Figure 6.8. Calculation of the hydraulic gradient line over the overflow dyke
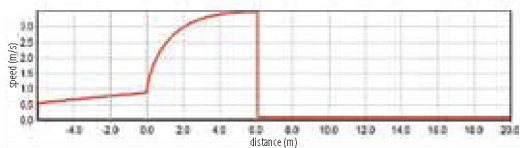

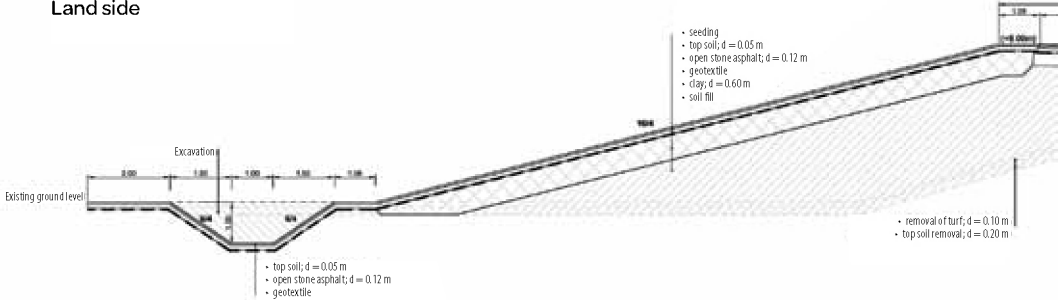Figure 6.9. Speeds of the flowing water over the overflow dyke

Land side

- seeding
- top soil; d = 0.05 m
- open stone asphalt; d = 0.12 m
- geotextile
- clay; d = 0.60 m
- soil fill

Excavation

Existing ground level

- removal of turf; d = 0.10 m
- top soil removal; d = 0.20 m

- top soil; d = 0.05 m
- open stone asphalt; d = 0.12 m
- geotextile

Figure 6.10. General structure of the covering of the overflow dyke

This lining is continued in the direction of the polder over a distance of 2 m.

The land side of the ring dyke can also be loaded by the flow of the water. The ring dyke is higher than the overflow dyke and therefore cannot flood directly. However, wind waves can overtop the dyke in highly exceptional situations. The wave overtopping flow rate was calculated. A good grass covering appeared to suffice to prevent erosion through overtopping.

6.2.7 Erosion of the water side of the ring dyke

In principle, this is the failure mechanism of erosion of the river slope. For the slope of the ring dyke on the side of the flood area, the most important load is caused by wind waves. These wind waves were calculated with Wilson's empirical formula. The final wave height can rise to 0.32 m for a return period of 100 years and 0.39 m for a return period of 4,000 years.

It was first investigated whether grass with a good layer of clay beneath it is sufficient as a revetment for the dyke. For grass, the duration of the load is very important, as well as the

wave height. The lower section of the dyke in particular experiences long-term loading from wind waves. Figure 6.11 illustrates this for one of the calculated hydrographs.

A grass revetment by itself did not appear sufficient for the lower sections of the dyke. A combination of rip rap and grass was therefore sought as revetment. Calculations show that adequate strength is obtained if the bottom 1.5 m of the dyke is covered with rip rap (layer thickness 45 cm with standard sorting from 5 to 40 kg) and the top 3 m with good erosion-resistant grass.
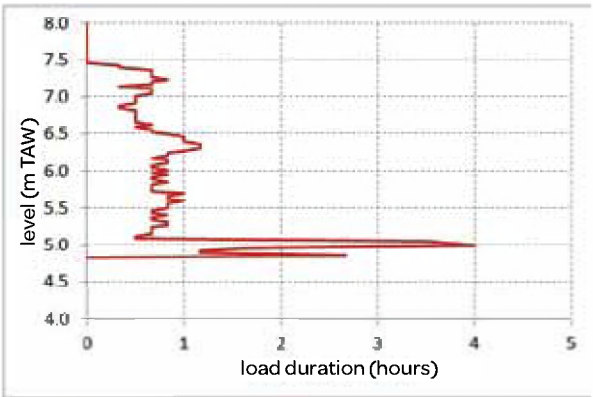

Figure 6.11. The total load duration of wind waves at different levels on the slope of the ring dyke

6.3 Inlet and outlet sluice

6.3.1 General

A structure was chosen in which the inlet openings are situated above the outlet openings. The advantages of this configuration have already been discussed above. Six inlet openings were installed, each with a width of 3 m and a height of 2.2 m. The desired sill level for each opening can be further adjusted with stop-logs. Three openings are required for outlet flow. These are also 3 m wide, but only 1.1 m tall.

Figure 6.12. The Scheldt side of the inlet and outlet structure. The 3 drainage shafts are visible at the bottom (each shaft consists of 2 sections), and the 6 inlet shafts at the top (also each consisting of 2 sections).
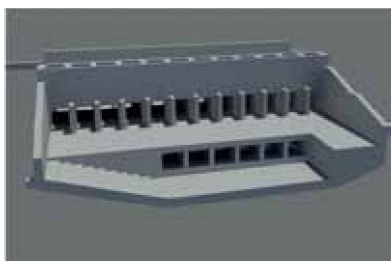

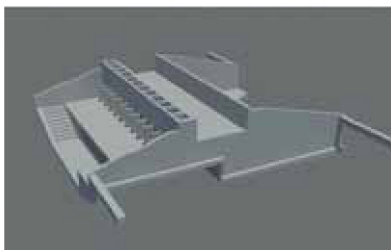Figure 6.13. Side view of the inlet and outlet structure

Figure 6.14. The polder side of the inlet and outlet structure. A number of these shafts are inlet shafts only. The central shafts are used as both inlet and outlet shafts.

6.3.2 Dimensioning

The dimensioning of the drainage structure is based on European standards and directives. The following loads act on the construction:

- Dead weight of the structure and its components.
- Earth pressures.
- Water level of the Scheldt: the water pressure on the construction is calculated according to the design level (7.5 m TAW). To be certain that the situation with the worst impact is taken into account, various scenarios were studied. There are four types of load: water pressure on the even openings, on the odd openings, water pressure on the Scheldt side and on the side of the flood area. All load combinations were combined with each other.
- Water pressure during heave: the worst scenario for heave was studied. This occurs just before the flood area is filled. The Scheldt is then as high as the overflow dyke, the flood area is dry and the non-return valves are closed. The water pressures were investigated according to the design level.
- Traffic loads on the crest of the dyke: the traffic load on the dyke road was determined according to standard ENV 1991-3 (2002). Although just one theoretical lane is required on the Bergenmeersen roadway, it was decided to study two theoretical lanes. That scenario can occur if appurtenances are installed or maintenance work is carried out with two vehicles standing alongside each other on top of the construction. Two scenarios were considered that are the mirror image of each other. In each of the scenarios, the lanes are loaded with a UDL

(uniformly distributed load) and a convoy. The convoy was positioned at different places in the lane to obtain the most negative loads. UDL 1 was also applied to the side walls of the construction to be able to withstand any loads alongside the construction.

- Overloads: the wire-floors and accessible zones were calculated with an overload of 300 kg/m².

The various load conditions were merged into combinations. Three major situations were examined. The first situation encompasses the earth pressure and the overload on the roadway. These combinations took account of the different positions of the convoys. The second situation encompasses earth and water pressures. These combinations took account of the various water level scenarios. The third situation encompasses earth and water pressures combined with the uniformly distributed overload on the dyke road.

The foundation characteristics are based on the study of the dyke bodies. The construction was calculated on an elastic bed. Characteristics for earth that is reloaded were assumed.

The environmental class and ambient class were set at XC4 and EE3 respectively.

The construction (concrete structure and steel girders) was calculated with the SCIA Engineer finite element package. The results of the computational model were used to calculate the reinforcement. This calculation was carried out on the basis of EN 1992-1-1 (2005) and separately for each element in the construction. The maximum capac-ity for the elements was determined with a standard net. Where overruns were found, additional reinforcement or a modified net was determined.

When checking heave, settlement and contact stresses, settlement of between 1 and 10 mm was still found in the worst scenario for heave. The relative settlement was finally limited to 5 mm, which is within the acceptable limits.

6.4 Modifications to the design

6.4.1 Feedback from public consultation

The original design was submitted to various services within the Flemish Government. The public was also given the opportunity to view the plans. The purpose: to create support for the plans and adapt them where necessary.

As indicated by the service that will be responsible for management and maintenance, further modifications to improve the safety and accessibility of the structure were applied. Thus an extra entrance was added, as well as a staircase. It was also ensured that the trash screens can easily be removed. An anti-theft system prevents the screens from being removed.

The residential area in question and the action committee set up there were concerned about the effect of litter. This could flow into the area through the sluice when this is operating daily. The trash screen was therefore narrowed further (from a gap of 10 cm to 5 cm), but without making the construction impassable to fish.

6.4.2 Optimization for fish migration

Due to Benelux and European directives, any sluice has to be maximally adapted for fish migration. This was achieved in Bergenmeersen by the optimization of the stilling basin.

At low tide fish can actively swim upstream into the CRT or passively out of the CRT, as long as the outlet gates are open (cf. Lippenbroek). As the length of the culvert is considerable, there is the risk of stranding by the end of the low-tide. By extending the stilling basin into the CRT area, the water body inside de sluice is made continuous with the CRT creek and provides a permanent and safe passage way. Juveniles and larvae, on the other hand, are expected to enter the CFA passively trough the inlet sluices, possibly dropping onto the floor slab. The addition of a stilling basin breaks their fall and again provides a connecting water body to the CRT creek.

Preliminary data in Chapter 8 already demonstrate that fish migration for larvae and adults is indeed possible through the Bergenmeersen CRT sluices, suggesting migration through both inlet and outlet culverts.

6.4.3 Sedimentation pond

Special interest goes to the amount of sediment entering the system each tide and partially remaining entrapped within a CRT-area. Intensive monitoring at the Lippenbroek shows that the suspended solid concentrations (SSC) entering the Lippenbroek are related to the amount of sediment in the Scheldt, but clearly smaller in absolute quantities. Following two different kinds of monitoring techniques, yearly sedimentation heights ranging between 2 and 3 cm are found.

Although excessive sedimentation will have adverse effects, it should be stressed that in order to obtain a well-functioning freshwater intertidal habitat, settlement of sediment is a necessity. A high inlet sill together with the implantation of the sluices somewhat more landward (Figure 7.4) create a self-maintaining sediment pond on the river side, limiting the sediment load entering Bergenmeersen. In addition, it is believed that yearly sedimentation heights will diminish after years of functioning (e.g. due to consolidation and subsidence). Moreover, when the amount of sedimentation becomes too high in terms of safety, appropriate action will be taken.

6.5 References

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- **Stilling Basin Optimization for a Combined Inlet-Outlet Sluice in the Framework of the Sigmaplan.** *J. Vercruysse et al. (2013) in IWLHS 2013: international workshop on hydraulic design of low-head-structures ISBN 978-3-939230-04-5 / www.vliz.be/imisdocs/publications/88/243588.pdf*
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