THE CANALIZATION OF
THE LOWER RHINE

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RIJKSWATERSTAAT COMMUNICATIONS

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Figure 1. Catchment area of the Rhine.
I. Why canalization with locks, sluices etc. was needed

1. Introduction

A few kilometres after crossing the Dutch frontier the Waal splits off from the river Rhine and flows to the left carrying 70% of the Rhine's water. A few kilometres further down, the IJssel splits off and flows to the right carrying 12% of the water coming down the Rhine. The river Rhine turns to the left carrying the remaining 18% of its original quantity. The river is then called Neder-Rijn (Lower Rhine) and after some 50 km westwards its name is changed into Lek.

This must be regarded as Nature's distribution; the layout of the canals and cities, and the dimensions of dikes, sluices and bridges were all conditioned by it.

On the other hand, the water coming down the river had to be put to the best possible use, so the flow had to be redistributed or at all events had to be made redistributable. The exigencies of both water management and inland shipping made a certain degree of regularisation desirable, particularly when the Rhine discharge was low.

To meet the principal requirements, a scheme was worked out by Rijkswaterstaat (the Water Control and Public Works Department) which:

a. when the discharge was either average or high would maintain the original distribution over the three branches;
b. when the discharge was low would enable to control distribution of the Rhine's water between the Waal, IJssel and the Lower Rhine.

When the discharge is low, the Lower Rhine is reduced to a temporary canal the discharge of which is controlled and the navigation depths of which are optimized by three sets of weirs and locks.

The scheme is known as the Lower Rhine canalization project. The Lower Rhine will be acting as a canal for six to nine months every year, depending on the natural regime of the river Rhine and on the exigencies of water management and transport.

During the remaining period of higher discharges, when the arguments against artificial distribution prevail, the weirs will be raised, thus reinstating Nature's distribution of the discharge over the three branches.
2. Basic information and considerations

The catchment area of the river Rhine covers the greater part of Switzerland and vast areas of France, Germany, Luxembourg, Belgium and the Netherlands; it totals to about 160,000 square kilometers (Figure 1).

The river is mainly fed by rain water but in spring and early summer melting snow helps to swell the river quite considerably. Low water periods, sometimes protracted, generally occur in autumn (Figure 2).

2.1. Water management

The Rhine runoff is discharged through the three branches known as the Waal, Lower Rhine/Lek and IJssel and the distribution is fairly stable, as the following table shows.

<table>
<thead>
<tr>
<th>Discharges in cub.m.per sec.</th>
<th>Lowest recorded</th>
<th>Average</th>
<th>Highest recorded</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rhine</td>
<td>630</td>
<td>2,140</td>
<td>13,500</td>
</tr>
<tr>
<td>Waal</td>
<td>490</td>
<td>1,490</td>
<td>8,250</td>
</tr>
<tr>
<td>Lower Rhine</td>
<td>90</td>
<td>390</td>
<td>2,700</td>
</tr>
<tr>
<td>IJssel</td>
<td>50</td>
<td>260</td>
<td>2,300</td>
</tr>
</tbody>
</table>

The reader might get the impression that this would be amply sufficient to satisfy all the low lying country’s possible fresh water needs.

But it is not. The minima give rise to three problems viz.

a. The intrusion of salt water through river mouths and locks;
b. The welling up of saline groundwater;
c. The lack of water for flushing polluted water.

Large quantities of river water in excess of the country’s normal consumptive requirements are needed to combat these evils. The situation is aggravated by the fact that the Rhine water itself is highly contaminated.

The reader will understand that it is becoming increasingly essential for the Netherlands to formulate and pursue a sound water-conservation policy in view of the irregularity of the discharges and the growing demands for water.

One way of doing so is to provide fresh water stocks in the south-west and centre of the country (Figure 3).

The former Zuyderzee (now called Lake IJssel) is a natural reservoir, provided of course that sufficient water can be made available. To ensure adequate supply, it was considered desirable that the runoff of the IJssel effluent be improved. This involved
both enlargement of the lower discharges and reconditioning of the river-bed. These points are dealt with in paragraph 3.2.

2.2. Inland navigation

It will be clear from the low discharge figures in the above table that in dry periods navigation is greatly hampered by lack of depth.

In both the Lower Rhine and the IJssel the water in the navigation channel is sometimes less than one metre deep.

During OLR conditions (i.e. the conditions obtaining when the runoff is below the ordinate low river runoff, which occurs for 20 days in an average year) incidental depths of not more than 2.40 m in the Waal, 1.50 m in the Lower Rhine and 1.40 m in the IJssel are recorded.

These figures are obviously below the standards for inland navigation (i.e. 2.50 m-2.70 m) and for international Rhine traffic (3.00-3.30 m).
Figure 3. The Rhine and its effluents in the Netherlands.
2.3. Remedies

These considerations show that in dry periods there is simply not enough water for three rivers but the runoff could do for two (although even then the requirements are not always met).

In order to solve the problems it was decided to stem the discharge of the middle effluent in dry periods by means of a weir at Driel near Arnhem. The water held back in this manner will then swell the natural discharges of the IJssel and the Waal.

Below this master weir two additional weirs have been built to control the water level in the lower course of the river and ensure that it remains navigable without causing serious flooding.

A similar condition must of course be set for the IJssel: flooding might result here if the Lower Rhine discharge were to be stemmed during higher discharges. These conditions are dealt with in paragraph 3.1.

2.4. River bed characteristics

In view of what is said below about the lay-out of the weirs, the operation of the system and flooding it is essential that the ultimate configuration of the longitudinal section and cross-section of the rivers be studied.

In cross-section (Figure 4) the river bed consists of two parts: firstly, the minor bed, the width of which is fixed by means of groynes and embankments; it increases as one goes downstream and is of such dimensions as will ensure that the minor bed will carry normal discharges while providing a reasonable navigation channel.

This principle has been adopted for 150 years; it is based upon practical and theoretical considerations, including sand movement calculations and model tests.

The second part consists of the flood plain which is generally found on both banks between the main dikes. The main dikes protect the low country behind them against flooding. The flood plain is often protected against small summer floods by means of "summer dikes".

Both the minor bed and the flood plain are kept as clear of obstacles as possible so as to provide sufficient capacity for floods and ice.

Bibliography:
a. "The Ingenieur" 1956, No. 37 (The Rhine Canalization, summary by E. M. H. Schaank) No. 39 (The construction of Hagestein weir, by H. C. Wentink) and No. 41 (Bed Load computations, by K. van Til);
A longitudinal section of the Lower Rhine is shown in Figure 5. Gradients decrease from $15 \times 10^{-5}$ in the upper reaches to $10 \times 10^{-5}$ in the middle reaches and decrease still more towards the sea.

It should be noted that the bed material of the upper reaches of the Rhine effluents consists of sand. Medium grain sizes range from 1.5 to 0.5 mm.

2.5. Hydro-electric generating plant

The design of Hagestein weir included a 2,500 KVA hydro-electric generating plant. The 10 KV AC electricity generated is fed directly into the grid.

The semi-automatic plant runs without attention. A full description is given in a separate chapter.

The idea of equipping the two other weirs with generating plants was abandoned, since the power obtainable would have been less than that produced at Hagestein weir and even that was not particularly great.
3. **The project**

The set-up of the project is described in paragraph 3.2. The object of the undertaking must be considered before the scheme itself can be understood. Figure 3 should be consulted for the topography.

3.1. **Objectives**

It has been pointed out that certain limitations must be set to the measure of runoff control. They may be formulated as follows:

- **a.** IJssel discharges should be kept above 250 cub.m. per sec. if possible to facilitate navigation;
- **b.** IJssel discharges should not be forced above 350 cub.m. per sec. to prevent the unnecessary inundation of flood plains;
- **c.** Lower Rhine discharges should be kept above 50 cub.m. per sec. to satisfy the needs of water-users along its course and to maintain a minimum flushing current;
- **d.** levels above the three weirs should be kept below +9.20 m, +6.00 m and +3.00 m (zero = N.A.P.*) to prevent the unnecessary inundation of flood plains;
- **e.** a lock should be built in a by-pass adjacent to each weir in the main river;
- **f.** runoff or bed-load transport conditions should not be altered by the implementation of the project.

It was found that the adoption of three steps between Arnhem level and sea level would give satisfactory results.

These boundary-conditions determined the project almost completely. There were further conditions because sites had to be found for the weirs and locks.

The locks were called Driel, Amerongen and Hagestein, after neighbouring villages. Amerongen is 30 km below Driel and Hagestein is 23 km below Amerongen.

3.2. **Operation**

As already stated, the IJssel discharges should be kept between 250 and 350 cub.m. per sec. for the longest possible period. What discharge will eventually be chosen will depend on water management requirements, especially in the northern part of the country. This will determine the level to which the water will have to be backed up by the master weir at Driel and for how long.

Within the stated limits of 250 and 350 cub.m. per sec. the weirs will function from between 160 and 270 days in an average year and the level above the weir will fluctuate accordingly between +8.20 m and +9.20 m.

Figure 6 gives the figures for an arbitrary runoff year and a programme based upon a desirable IJssel discharge of 300 cub.m. per sec. and a minimum Lower Rhine

* N.A.P. = Amsterdam Ordnance Datum (A.O.D.)
discharge of 50 cub.m. per sec. The quantities discharged by each of the three branches are shown in the figure.

There is no need to interfere with the natural distribution during the winter months nor would it be possible to do so. But in April, when the IJssel discharges fall below the 300 cub.m. per sec. minimum, the level is boosted by partly closing the weirs. This also increases the Waal discharge. The Lower Rhine discharge is correspondingly reduced. The May discharges do not require boosting and the weirs are opened (i.e. raised right out of the water).

But in the period from June to mid-August they must be lowered again.

From early July onwards the 300 cub.m. per sec. minimum cannot be adhered to because of the 50 cub.m. per sec. that must be reserved for the Lower Rhine. So in this case the IJssel discharges will drop to 200 cub.m. per sec. and the Waal discharges to 1,050 cub.m. per sec. A short rise in August will allow human intervention to cease for a while but then the autumn drought will bring another operational period.

On the worst day the Rhine runoff will be slightly above OLR but owing to the weir the IJssel discharge will be 175 cub.m. per sec. instead of 100 cub.m. per sec. and the Waal discharge will be 825 cub.m. per sec. instead of 750 cub.m. per sec. This will give a navigable depth of 2.00 m instead of 1.40 m in the IJssel and 2.50 m instead of 2.40 m in the Waal.

The figures are given by way of illustration and are not claimed to be exact.

Figure 6. Effect of canalization on the Rhine effluents in a fictitious year.
3.3. Adapting the river bed

The ratio in which the backed-up Lower Rhine discharges are distributed over the IJssel and Waal depends mainly on their discharge capacities at their junctions with the Lower Rhine, i.e. by the slopes, widths and bottom levels of their upper reaches.

As it was considered essential that the IJssel effluent should receive the major share, the upper reaches of this effluent had to be modified accordingly. The width was one factor that could not be changed because of its effect on the sand transport characteristics. The slope and bottom level could be changed, however, without fear of complications. This was confirmed by model tests.

An increase in slope was effected by straightening out two major bends, one 9 km, the other 25 km, below the bifurcation. This shortened the river by $3\frac{1}{2}$ and $4\frac{1}{2}$ kilometres respectively. The 30 km stretch was cut to 22 km so the average slope (the average being $13 \times 10^{-5}$) was increased by nearly 25%.

It is not yet fully effective, because it will be a few years before the bottom level has adjusted itself. Nevertheless, bottom erosion is expected to proceed upstream, eventually reaching the bifurcation.

Then the third factor will automatically come into effect. If this automatic process is too slow, dredging may be considered. In any case, there are means by which the IJssel may be helped to swallow its calculated and programmed share.

If they are not employed forthwith, it is because gentle adjustment by nature is preferred to an enforced regime, the consequences of which are hard to predict.

3.4. Consequences

The effects of the scheme on bed load transport and bottom levels, especially near the two bifurcations, have been very carefully weighed indeed. Studies and model tests preceded the operation, which is being watched continuously to ensure there is the closest possible conformity between the plan and its realization.

After all, this low-lying country’s defences against flooding, either by the sea or by the rivers, may not be exposed to unpredictable risks.

There might be unpredictable risks if the distribution of flood water over the Rhine effluents got out of hand, or if the presence of the weirs and ancillary structures in the river bed raised the maximum flood levels.

The effect of building the weirs, etc. out of commission was investigated. Recommendations were made and the river bed was reshaped accordingly.

Although as a rule the design water levels will not harm the riparian land, some of the lower parts of the flood plains will be affected by the unusual ground-water levels. Complaints were investigated by an arbitration commission, who recommended either that indemnification be paid or that preventive measures be taken. In most cases the second expedient is preferred. Raising low ground, draining low ground towards the river below a weir and pumping are some of the remedies.
4. How the project was carried out

4.1. Building operations

The implementation of the scheme commenced in 1954-1955 when two bends were straightened out: one in the Lower Rhine a few kilometres above Arnhem, the other in the IJssel some 25 kilometres below Arnhem.

Then, in the years 1954-1961, the first weir and lock near Hagestein were built. The downstream complex was built first because the work at this point would at once improve the navigability of part of the river above the weir, whereas there would always be sufficient depth in the tidal area below the weir.

It should be borne in mind that neither of the downstream weirs (i.e. those at Hagestein and Amerongen) can affect the volume of runoff. Work on the upstream weir at Driel, on the other hand, would have affected the Lower Rhine discharge immediately and made the whole river virtually impossible to navigate. It was reasonable, therefore, that this "main tap" should be the last to be constructed and not to put it into commission until both downstream weirs were completed.

Figure 7 shows the Hagestein site on the inner bend of the river where there was room in the flood plain to excavate a pit for the foundations and to build a dike around it to prevent flooding.

When the weir and the lock were finished the river was made to flow through the newly dug bed and the old bend was dammed off.

The middle weir near Amerongen was built by similar methods in the period between 1958 and 1967 but here it was impossible to realign the river by straightening out one bend so the new bed traverses two consecutive bends (Figure 8). Apart from this, the procedure was the same as that adopted for the Hagestein complex.

The "main tap" or master weir near Driel (Figure 9) was constructed in the years 1962-1967, and the adjacent lock in the years 1967-1970. There was not enough room between the hills in the north and the main dike in the south to excavate a pit big enough for the foundations of both weir and lock and place a dike around it that would remove the risk of flooding.

Therefore it was decided to build the weir close to the southern main dike with a temporary dike first, then shift the river into its new bed through the weir, and finally build the lock on foundations surrounded by another temporary dike in the old river bed.

The work was interrupted by high discharges, but that is usual when carrying out river projects. There were no exceptional floods, however, and the work went according to plan.

The last major work in the project involved the straightening out of a large bend in the IJssel some nine kilometres below Arnhem; this was done in 1969. There were several minor works and adaptations but they are not described in this article.
Figure 7. Location of Hagestein weir showing dam across former river.

Figure 8. Location of Amerongen weir.

Figure 9. Location of weir at Driel.
4.2. Management

Each weir with its lock will be operated by eight men. There is housing-accommodation for the staff on the site. The operation period will be from six to nine months in an average year. The rest of the time (i.e. when the discharges are high) is spent on maintenance work. The mechanical and electrical systems are serviced by specially trained mechanics.

The Lower Rhine is part of the Rhine system and comes under the International Rhine Commission regulations, which stipulate that vessels must be free to use the river without let or hindrance and without having to pay any toll.

Accordingly, the locks are manned round the clock and there is no charge for their use.

Broad directives are issued by the Central Water Conservation Board; detailed instructions are sent to the operating staff by the Arnhem headquarters. In accordance with the scheme described in the foregoing pages, the instructions will usually be limited to the following points:

a. for Driel weir: maintaining the water level at the IJssel bifurcation at a given gauge level, depending on the runoff and requirements, so as to maintain the IJssel discharge planned;

b. for Amerongen and Hagestein weirs: to keep their respective head-waters at given levels (normally +6 m and +3 m A.O.D. respectively) and to adjust the water levels gradually when a rise or a fall is expected.

4.3. Cost

Expenditure on the project may be broken down into the following items:

<table>
<thead>
<tr>
<th>Item</th>
<th>Amount</th>
</tr>
</thead>
<tbody>
<tr>
<td>acquisition of sites</td>
<td>Dfl. 5.3 million</td>
</tr>
<tr>
<td>earth and river works</td>
<td>Dfl. 51.0 million</td>
</tr>
<tr>
<td>weirs and locks</td>
<td>Dfl. 78.0 million</td>
</tr>
<tr>
<td>housing</td>
<td>Dfl. 1.2 million</td>
</tr>
<tr>
<td>miscellaneous</td>
<td>Dfl. 1.5 million</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>Dfl. 137.0 million</strong></td>
</tr>
</tbody>
</table>

The other river reconstruction and adaption works referred to which were directly connected with the canalization scheme involved expenditure totalling Dfl. 25 million.

All the designs were prepared by the Water Control and Public Works Department, who also supervised the actual work.
Photograph 1. Aerial view of Hagestein weir looking downstream with lifted visor gates.

Photograph 2. Hagestein weir looking upstream.
Figure 10. View of weir looking upstream showing headroom
II. The construction of the weir complexes

1. General

The preceding section explains why it is necessary to canalize the Lower Rhine and Lek.

The present section deals at some length with the hydraulic structures in concrete that will be needed, chief among them being the three weir and lock complexes (see Photograph 1, Hagestein).

First of all, there are a few general comments to make on the work.

The designs will have to meet requirements laid down by the rivers department. One of the principal requirements is that preferably there should be one opening only in the weir and that the opening should be wide enough to accommodate both upstream and downstream traffic; alternatively there should be two passages, one for upstream and one for downstream traffic, separated by a central pier, each passage having an interval width of 45-50 m and a vertical clearance below the raised gate of at least 9.10 m above the highest recorded water level (a stipulation which was obviously made by the Central Commission for Rhine Traffic at Strasbourg).

A study prepared by directie Sluizen en Stuwren (the Locks and Weirs Directorate), which will be responsible for designing and building these works, shows that a single passage would be very costly and would incur a substantial risk of vibration and operational unreliability.

Consequently, the only solutions considered are those featuring two ship passages.

Since the designer has proposed the visor type of gate (about which more later) for closing the ship passage, and since this would not meet the requirements on vertical clearance at the sides, it has been decided to set the net span at 48 m, so that there will be 38 m in the centre of the passage within which the stipulated headroom will be assured.

A further important requirement is a fine control opening with adequate capacity (100 cub.m. per second) for dealing with an average head.

It must be possible to adjust the weirs with sufficient accuracy to match the water drainage needs; very large gates were not thought suitable for this (though their level can be very precisely controlled).

For reasons of symmetry and to prevent erosion of the banks the designer has sited the fine control not at the sides but in the middle of the pier separating the two ship passages.

In dry periods the visor gates can then be lowered and left on the sill to facilitate the carrying out of maintenance work on the cables and winches.
Figure 11. Cross-sections of pier and micro-adjustment valve (cylindrical gate valve).
This gave an indication of the required pier width, and this in turn governed the ultimate width of the river bed at the weir.

For the Hagestein weir this was \(2 \times 48 + 14 = 110\) metres. For the weirs at Amerongen and Driel the width is slightly less, as the pier can be somewhat narrower in these weirs because there is no water turbine.

In all other respects the three weirs are practically identical. There may be slight differences in the superstructure due to differences in flood discharge.

All three locks have the same \(18\) m horizontal clearance, and are equipped with mitre gates in order to give unlimited headroom.

The locks have an intermediate bay to save water in periods of drought.

During periods of flood discharge the locks may be completely under water. To prevent scouring, baffles are fitted in the upstream bays (they can take the form of simple sliding doors), while a dividing wall between the weir and lock channels prevents cross currents.
Figure 13. Upstream lock bay and current deflector.
Figure 14. Plan and longitudinal section of floor of weir.
2. Brief description of the three weir complexes

2.1. Hagestein complex

At Hagestein the weir and lock were built in a single excavation in the southern flood plain.

When the excavation had been filled in and the access channels dug, a low-topped dam was built in the old river. When the water level is very high the river water spills over this dam, so the width of the winter bed is adequate.

As there is a deep pocket of sand on the site, the seepage path was lengthened mainly by incorporating a thick horizontal bottom covering. Since there might be a covering layer of clay peat over much of the surface, infiltration or drainage strips were provided upstream and downstream to prevent any disruption of the bed through sudden opening or closing.

The lock chamber is 225 m long; this seems relatively short, but since the weir is just above the mouth of the locks at Vreeswijk, vessels plying between Amsterdam and Rotterdam do not have to pass through the Hagestein lock.

The waiting berths are of steel sheet piling supported by a stiffener beam on piles. This gives great stability and has made it possible to build the second lock immediately alongside the first.

Further details are given in paragraph 3.

Figure 15. Closed floor of weir showing drainage system.
Figure 16. Cross-section of approach.

Figure 17. Cross-section of central pier at cylindrical gate valve.
2.2. Amerongen complex

At Amerongen the works are also on steel foundations and are located in the flood plain on the northern shore of the river. Since there is no water turbine generator, the pier here is only 12.5 metres wide.

In the fine control opening in the middle of the pier there is a cylindrical valve with an air tube behind the seating through which oxygen can be added to the water (as is also the case at Driel).

As at Hagestein and Driel, the waiting berths for ships on the upstream side are considerably shorter than those downstream; this is because more small vessels are expected to pass upstream (these are, without extra fuel, raised by about 9 m) than downstream, since the latter traffic goes along the Waal where it is helped by the current. At Amerongen, as at Driel, the lock chambers are 260 m long.

The waiting berths are of the same design as at Hagestein, except that the piling is of Vibro piles with a prestressed Diwidag bar as the tie rod.

Figure 18. Cross-section of approach.
2.3. Driel complex

At Driel the subsoil consists of layers of clay and sand washed up along the Veluwe. The clay is a preglacial, hard type; it was dredged away on the site of the works and replaced by sand.

The layers of sand underneath the abutments and piers were heavily compacted by deep vibration. Underneath the sills they were slightly vibrated; the sills were also weighted with lead slag concrete.

This very largely prevented uneven settling due to differing foundation pressures.

Because of the small space between the confining embankments work on the weir proper started on the southern side.

The river was diverted; then the lock was built on the old river bed.

Since there may be some uneven settling in the lock, the joints have been made watertight by means of PVC-rubber sealing strips. In this it differs from the other locks. The entrance guides are also designed to suit local circumstances. They are sprung braking constructions made from steel tubes with linking sleepers, which can be moved without great expense. They will absorb 4 tm energy at each point of impact at normal stresses, i.e. they are suitable for ships of 2,000 tons.

Since there are no longer any glass-eels in this stretch of the river, there are no eel passages in the fish-ways.

Figure 19. Longitudinal section of excavation for foundations showing improved soil structure beneath foundations.

Figure 20. Cross-section of weir sill showing lead-clay concrete.
Figure 21. Joints between lock sections. Left: Hagestein and Amerongen locks, right: Lock at Driel.
Figure 22. View and cross-section of approach to lock at Driel.
3. Description of Hagestein weir complex

This weir differs in a number of respects from the types found elsewhere in the Netherlands. The gates are semicircular and have a free span of about 54 metres, the fish-ways have been changed into fish locks and there is a water turbine generator in the central pier.

3.1. Design of gates

The special shape of the gates sprang from a number of theories subscribed to by the designer, namely, that

a. if the gate took the shape of part of a cylinder with the water pressure on the "hollow" side, it would only have to withstand tensile loads, so it could be a light construction;

b. if the bottom edge was rounded the water would flow out radially when the gate was only slightly raised and the hydraulically sound flow distribution would consequently make a light bed protection adequate even if only one gate was being used for drainage;

c. the resulting non-rigid structure of the gate and the curved drain aperture would preclude serious vibration since much of the gate would always be steadied by the mass of water behind it.

Figure 23. Diagram showing vibration of flat vertical-lift gate and of visor-type gate.

Apart from small disturbances due to a slight tendency of the gate to lift and the effect of the girders along the edges, the stresses set up in the plating are mainly dependent on the water pressure and the radius. With a span of 54 metres or a radius of 27 m and a gauge water pressure of 4.5 tons per sq.m., the stress set up in the plating is \( 0.45 \times 2,700 = 1,215 \) kg per sq.cm.

Figure 24. Stresses in curved plate.
Figure 25. Stress diagram of visor gate.

Figure 26. Direction of current downstream when visor gate is partially raised.
The plating is 8 mm thick, so the tensile stress is about $1,215 : 0.8 = 1,520$ kg per sq.cm., which can be taken as very low indeed for an S.52 steel. Since the extent to which vibration is a hazard is not known, a safety margin of 600 kg per sq.cm. is included in the calculation.

There is a piano-hinge with rubber seal in the centre of the gate; this facilitates dismantling and helps to reduce temperature stresses.

Because of its shape and the method of lifting, this type of gate is called a "visor gate", since it is reminiscent of the visor on a mediaeval knight's helmet.

When the gate is down, it is subjected to tensile stresses only. During the raising operation, stresses are set up in the edge girders; they become less as the gate rises. In the raised position, the edge girders are subjected to compression stresses.

The shape is hydraulically so effective that the coarse-grained sandy bottom does not need protecting. There is no edge turbulence.

The level of the upper stretch of water must be adjusted by means of the visor gates whenever the discharge of the river exceeds 100 cub.m. per sec. (up to this value all the water passes through the fine-control system). Above 550 cub.m. per sec. the gates are fully open. The maximum head at this rate of discharge is 3.50 metres. The minimum vertical flow clearance is 15 cm.

In view of the flow characteristics of the Rhine, the gates will be in their partly-raised position for about four months of the year; it is therefore important that there should be no very strong vibration.

### 3.1.1. Hydraulic study

The Hydraulics Laboratory was responsible for the whole of the hydraulic study.

To reproduce the flow phenomena in the prototype in a model it is necessary that for a constant flow pattern the various internal and external forces acting on a particle of water should continue to act in the same proportions.

For this reason the Laboratory prepared the following appraisal, which was prepared by Ir. P. A. Kolkman.

The following terms are used in the equation for the dynamic balance of a water particle:

Acceleration

$$\rho \frac{\delta v_x}{\delta t} + \rho \left( v_x \frac{\delta v_x}{\delta x} + v_y \frac{\delta v_x}{\delta y} + v_z \frac{\delta v_x}{\delta z} \right),$$

in which

- $\rho$ = density of fluid
- $v$ = velocity
- $x$, $y$, and $z$ = coordinates
- $t$ = time
Pressure
\[ \frac{\delta P}{\delta x} \text{ etc. in which } P = p + \rho \cdot g \cdot h \]

in which
- \( p \) = local pressure
- \( g \) = acceleration due to gravity
- \( h \) = height of the point relative to a datum level

Viscous shear stress
\[ k_x = \frac{\delta \tau_x}{\delta y} + \frac{\delta \tau_x}{\delta z} \text{ etc. which corresponds to } \]
\[ \rho v \left( \frac{\delta^2 v_x}{\delta x^2} + \frac{\delta^2 v_x}{\delta y^2} + \frac{\delta^2 v_x}{\delta z^2} \right) \text{ etc. } \]

Free fluid surface.
At the surface \( p = p \text{ atm. (constant)} \)
\[ \frac{\delta p}{\delta x} = \rho \cdot g \cdot \frac{\delta h}{\delta x} \]

Surface tension
\[ \left( p_{\text{gas}} - p_{\text{fluid}} \right) = \pm \sigma \left( \frac{1}{R_1} + \frac{1}{R_2} \right) \]
in which \( \sigma \) = surface tension
- \( R_1, R_2 \) = radii of curvature of surface

If all lengths are to be reduced by \( n_L \), we can determine the scale factor by introducing the invariance of the relationship between the terms.

a. From the acceleration formula it follows that:
\[ n_t = n_{t_{/n_v}} \]

b. From a combination of the pressure and acceleration formulae it follows that:
\[ n_p = n_{\rho \cdot n_v^2} \]

c. From the viscous shear stress formula and the acceleration formulae it follows that:
\[ n_v \cdot n_L/n_{v \cdot L} = 1 \]

The Reynolds number (\( Re = vL/\nu \)) is constant.

d. From a combination of the free fluid surface and the acceleration formulae it follows that:
\[ n_{v^2}/n_{\rho n_L} = 1 \]
The Froude number (\( Fr = v^2/gL \)) is constant.
e. From the surface tension formula it follows that:

\[ n_p = n_o/n_L, \] and so combined with the term \((n_p = n_o n_z^2)\) given in b. above:

\[ n_p \cdot n_L \cdot n_o^2/n_o = 1. \]

The Weber number \(\left(\text{We} = \frac{\rho L v^2}{\sigma}\right)\) is constant.

The constancy of the Reynolds, Froude and Weber numbers provides the conditions for the velocity scale, while the pressure and time scales follow through the relationships a. and b. from the velocity scale chosen.

*If the same gas or fluid is used in model and prototype alike, the conditions for velocity are always contradictory.*

In fact,

\[ n_p = n_o = n_o = n_o = 1 \]

This means that the flow in a \(1:20\) model should be 20 times faster according to the Reynolds scale, \(\sqrt{20}\) times slower according to the Froude scale, and \(\sqrt{20}\) times faster according to the Weber scale.

*The choice will depend upon the purpose of the study.*

Since the water in the sea or in rivers has surface waves or is turbulent and since hydraulic engineering structures are seldom perfectly streamlined, the particles of water therein must always be in a state of violent acceleration or deceleration. It is found that viscosity and surface tension play a minor role in this compared with the forces of acceleration and gravity acting on a particle (i.e. Re. and We. are very large). Even when there is a steady flow in large pipes or concrete irrigation channels the turbulence caused by the roughness of the walls is so great that the effect of viscosity is very slight.

This fact allows us some freedom not to incorporate the correct value of Re. in the model, provided the value we do incorporate in the model is sufficiently high.

There is ample information on this point in the literature on the subject. It need hardly be said that certain inaccuracies in the model determined in the laboratory are accepted for practical reasons.

It follows from the foregoing that so long as we do not go below the critical lower limit of Re. there can be a free choice of velocity scale in a model without a free fluid surface. If there is a free surface, the constancy of Fr. will determine the velocity scale.

As a means of compensating in a model for too small a value of We. or Re., we might mention magnification of the vertical scale with free surface flow and the turbulence mesh to make the boundary layer turbulent more quickly. Obviously, both these methods prejudice the conformity of the flow pattern.

This completes our examination of the hydraulics side of the question.
The following observations concern the elastic properties of models.

The final vibration amplitude of a single mass spring system with periodic excitation is shown in the following diagram:

The shape of the curve is dependent solely on the damping.

The scale of $\omega_r$ must also be equal to the scale of the excitation frequency (e.g. the wave or turbulence frequency)

$$n_\omega/n_{\omega_r} = 1$$

(2)

The relative damping $j.$ (damping/critical damping) must be the same in both prototype and model:

$$n_j = 1$$

(3)

Conditions (1) and (2) determine the mass scale:

$$n_M = n_c/n_{\omega_r}^2 = n_K/n_L \cdot n_\omega^2$$

(4)
Since during the vibration of a structure in fluid an apparent increase in mass occurs, the scale of the model mass and that of the water mass must satisfy these conditions.

For elastic models of compound structures condition (1) can be replaced by:

\[ n_{\varepsilon} = n_\sigma / n_E = n_K / n_F n_E = 1 \]  

(5)

\( \varepsilon \) = relative distortion, \( \sigma \) = material stress  
\( E \) = modulus of elasticity

For shearing, similarly,

\[ n_K / n_F n_G = 1 \]  

(5a)

On combining (5) and (5a) we get \( n_G = n_E \); this is in most cases automatically satisfied.

For the combination of fluid with model, where there is flow without a free surface:

for the flow we found \( n_p = n_\rho n_\phi \) consequently

\[ n_K = n_\rho n_\phi \]  

(6)

The spring stiffness of an elastic model follows from (1)

\[ n_{cnL} / n_K = n_{cnL} / n_\rho n_\phi n_\phi = n_C / n_\rho n_L n_\phi = 1 \]  

(7)

The Cauchy number \( \left( Ca = \frac{C}{\rho L V^2} \right) \) is constant.

For the frequency of excitation by flow or waves

\[ n_t = n_L / n_v \] or \( n_\omega = n_v / n_L \)

In combination with (2) we get for the resonance frequency of the model:

\[ n_L n_\omega / n_v = 1 \]  

(8)

The Strouhal number \( \left( \frac{\omega r L}{v} \right) \) is therefore constant.

For the mass scale we find, from (4), (6) and (8):

\[ n_M / n_\rho n_\phi = 1 \]  

(9)

The mass number \( \left( \frac{M}{\rho L^3} \right) \) is therefore constant.

The total mass, i.e. \( \frac{M_{\text{struct.}} + M_{\text{water}}}{\rho L^3} \)

must in accordance with (9) be geometrically reduced as it were but must keep the same density with respect to the surrounding fluid.

This is already the case with the water mass vibrating with it, so that it is sufficient to have \( M_{\text{struct.}} \) satisfy these conditions.
For mass spring systems it is therefore possible to obtain a correct model reproduction of the vibration phenomenon of each velocity by adjusting the spring stiffness, the mass of the model being independent of the velocity.

For compound structures it is preferable, when using the same fluid, to make the model from the same materials as the prototype structure. The mass will then be correct for all forms of vibration.

It follows from (5) and (6) that, as $n_E = n_p = 1$, then:

\begin{equation}
\begin{aligned}
    n_e &= n_K/n_p^2 n_E = n_p n_p^2 n_p^2/n_p^2 n_E = n_p^2 = 1
\end{aligned}
\end{equation}

This means that the prototype velocity must be used for the flow in the model.

For the combination of fluid with model with a flow with a free surface:

As an extra condition we now introduce the invariance of the Froude number. This gives:

\begin{equation}
\begin{aligned}
    n_V &= (n_p n_L)^{1/3}
\end{aligned}
\end{equation}

The stiffness of a model of mass spring systems follows from the invariance of the Cauchy number.

For compound structures we must, according to (5) and (6), write:

\begin{equation}
\begin{aligned}
    n_e &= n_p n_p^2 n_p^2/n_p^2 n_E
\end{aligned}
\end{equation}

This gives us

\begin{equation}
\begin{aligned}
    n_L/n_E = 1
\end{aligned}
\end{equation}

We also have to satisfy

\begin{equation}
\begin{aligned}
    n_p \text{struct.} = n_p = 1
\end{aligned}
\end{equation}

There is no such material for the conventional scales. In compound structures, however, the stiffness increases linearly with the plate thickness (with the exception of local bending stiffness of plates and massive girders), so the stiffness can be corrected by sacrificing the geometry. There is a practical preference for adjusting the model for too low an $E$ and consequently for too large a plate thickness. Up to now plastic models have been employed, with $n_E \approx 60$ compared with a steel prototype.

According to (12), this ought to amount to $n_L = 60$, but this would make the models so small that the Reynolds number would preclude their use.

In our studies of the visor gates at Hagestein we used $n_L = 20$, so that $E$ was too small by a factor of 3. This was compensated for by multiplying the plate thickness by 3. In view of the fact that the plastic material (Trovidur) used was $5\frac{1}{2}$ times lighter than steel, the 3-fold greater thickness of the plate still gave too light a model.

The correction for this was effected in such a way that the stiffness was unchanged. To do this, small lead weights were applied at certain points, distributed as nearly as possible in proportion to the deficiency in mass.
The damping requirement that \( n_j = 1 \) naturally obtains at all times. The \( j \) may be the same in geometrical models made of the same material as the prototype, since \( j \) is to a large extent a material constant.

The damping of parts subject to friction, hinges and rubber seals is not automatically to scale. The damping caused by the fluid may also be out of scale. The damping will certainly be to scale if it is due to the drag of beams and the like in turbulent currents but not if viscosity is a criterion (e.g. a plate vibrating in its own plane). The damping of hinges and the like often needs to be considered separately and the question as to whether the fluid damping is to scale must also be gone into.

The damping is a criterion when determining the equilibrium amplitude when there is resonance. As a rule, however, whenever resonance occurs the design needs to be changed to prevent it.

\[ As \ long \ as \ the \ damping \ in \ the \ model \ is \ low \ enough, \ preferably \ lower \ than \ in \ the \ prototype, \ the \ model \ will \ show \ whether \ or \ not \ resonance \ will \ occur. \]

The damping factor is important even if there is no resonance, since the response to non-periodic excitation resulting from the turbulent flow field also contains components of the resonance frequency, although their effect is less than those concomitant with resonance vibration. Damping is of minor importance in respect of the response to impact phenomena, since the maximum amplitude occurs shortly after impact and there can be little dissipation of energy.

The damping inherent in the material is too high when plastic is used for the models. On the other hand, when making check measurements at Hagestein it was found that the friction of other parts predominated, so the model as a whole was often even less damped than the prototype.

The distortion in elastically similar models can be measured directly by means of expansion strips. Since \( n_t = 1 \), the correct strain value can be read off direct. If the material is excessively thin, the adhesive and waterproof covering may not affect its elasticity; for this reason, strain gauges are only used to a limited extent on models.

### 3.1.2. Model studies on gates

\[ On \ the \ basis \ of \ the \ foregoing, \ model \ studies \ were \ carried \ out \ on \ the \ Hagestein \ visor \ gates, \ and \ extensive \ check \ measurements \ were \ carried \ out \ on \ the \ prototype. \]

Briefly, the model studies comprised the following stages:

a. Vibration patterns and resonance frequencies were determined, using the elastically similar model.

As the model lacked rigidity in the horizontal plane, the minimum resonance frequency was very low indeed, viz. 1.3 cycles per second dry and 0.6 cycles per second when immersed.

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b. Pressure fluctuations were measured on a rigid model of a gate section. The pressures generated all the frequencies between 0 to 10 cycles per second, so some vibration can always be expected. Variations in the shape of the lower edge had little effect on vibration.

c. Lower edge tests: using a model of a gate section (scale 1 : 6) suspended from springs, tests were carried out to find out whether resonance vibrations occur when there is horizontal or vertical movement at various resonance frequencies. There were none in the profile selected.

d. Tests on the elastically similar model. Perforation of the bottom stiffener was found to reduce the vertical vibrations substantially.

Systematic readings with the gate at various heights and with various water levels showed no appreciable vibration amplitudes; the maximum was 7 mm for horizontal movement and 1.7 mm for vertical movement.

e. Scale tests on the model as described in c and a 20 : 1 scale model of this; the vibrations set up by currents and the damping effects of the water were compared.

Since the results of the tests described in d showed that there was a substantial margin of safety and since the results of the tests described in e did not suggest there would be any very great scale effects, the design is considered to be sufficiently safe.

There is some resonance in parts of the plating under certain circumstances; this can be overcome by raising or lowering the gate very slightly. Scarcely any measurable stresses are set up. This was not observed in the elastic model, since the components are much too stiff as a consequence of the excessive plate thickness.

Exhaustive experiments were carried out on the prototype to test the model technique, the model being set up afresh to ensure that all the conditions obtaining when readings were taken were faithfully reproduced. The stiffness of the rubber side seals was tested separately under dynamic loading and the stiffness was reproduced in the model. Since the cables had not yet been fitted when the dry excitation tests were carried out, the gate was temporarily suspended by strips. This was a help when taking the check readings, because otherwise the elasticity of the cables would have had too great an effect and the conclusions regarding the model would have been less reliable.

The horizontal tangential and the horizontal radial periodic excitation were compared.

The agreement in frequency in both the dry and the wet state is satisfactory. As a rule, the oscillation was greater in the model than in the prototype.

The effect of the water on the resonance frequencies was adequately reproduced in the model.

We also compared the figures for vertical excitation.

The excessive torsional stiffness of the model (due to extreme simplification of the shape) gives too high a resonance frequency. This was only to be expected. A re-
markable thing is the powerful damping effect of the water in the prototype; during the tests there were high-frequency vibrations in the plating which presumably absorbed a great deal of energy. The vibrations were also recorded during normal operating, and there was satisfactory agreement between the figures obtained from both model and prototype.

The study shows that the model technique used is reasonably reliable, as long as prior allowance is made for the difficulties arising from excessive material damping in the model and as long as a great deal of attention is paid to details such as the elasticity and damping effect of the rubber seals, friction in and damping effect of hinges, elasticity in the cables.

The tests on the prototype show that the design of gate adopted is the best from the point of view of vibration.

Because of the extra stress allowance of about 600 kg per sq.cm., the structure was too heavy. The use of high-strength bolts, also for the work done in the factory, has also produced excessively heavy structures. In view of present-day welding techniques, the use of these bolts is justifiable only for work done on site.

3.2. The fishways

3.2.1. Remarks

Even before the Second World War a number of weirs incorporating fishways were built in the River Maas in Holland.

As our knowledge of the biology of fishes was incomplete, the fishways were of the Denil or basin-ladder type; the argument was that "a fish is a fish".

Since then we have discovered that there is a basic difference between fish that travel very far upstream to spawn (such as trout and salmon) and the white fish found in tidal rivers. Salmon and the like are strong fish easily able to overcome differences in water level by leaping upwards to higher reaches.

Fish in tidal rivers are far less strongly developed, since they live in slow-flowing waters. These "lazy" species are unable to negotiate a basin-ladder or Denil fishway.

Consequently, the Fishery Inspectorate and the Water Control and Public Works Department have together developed a fishlock at Lith on the River Maas which raises fish enticed into the lock by a "lure current". This was so effective that the system has also been adopted for the Rhine canalization scheme.

The system operates in the following manner:

The lock has an upstream and a downstream gate. In the lock-chamber behind the upstream gate (which operates as an overspill) there is a chute with an overspill profile. Beneath the floor of the lock are two "lure current" conduits with diameters of 50 cm and 90 cm. Each conduit has a stopcock at the upstream end.

Fish pass through the lock in three stages, as follows:

The downstream gate is raised, the upstream overspill provides a moderate inflow
of water and the stopcocks of the large and small conduits (A and B respectively) are opened. The strong lure current from the large conduit entices the fish from the river away from the weir gates to the fishlock, while the lure current from the small conduit and that of the water from the transfer basin entices the fish into the lock.

After a time the gate is suddenly closed (by letting it drop freely against a brake). The large conduit stopcock A remains open. As the small conduit stopcock B is closed the overspill gate drops, allowing the lock to fill.

Stopcock B then opens, producing a countercurrent in the lock chamber. Since the large conduit stopcock A remains open, there is also a lure current passing through the lock passage from the upper pound. These two currents together attract the fish into the upper ground. The idea of leaving the large conduit stopcock A open (also during the filling of the lock, i.e. with the gate closed) is to keep the lower lure current flowing.

The overspill gate is closed completely when the fish have left the lock chamber. Stopcock B remains open and the lock chamber is emptied by slowly raising the gate until it is wide open. The entire process is then repeated. The gates and stopcocks open and close automatically at set intervals.
3.2.2. Eel passages

Separate arrangements have to be made in the lower effluents to allow the glass-eel (which is brought to the coasts of Europe from the Sargasso Sea by the Gulf Stream when it is a few years old, still transparent (hence the name) and about 10 cm long) to pass weirs on its journey upstream. These eels move close to the bank and can only cope with weak adverse currents. They are unable to get through the fishlocks, because they are not yet strong enough to do so but they will be when they reach the uppermost weir complex at Driel; they will have grown by then and will be able to negotiate the fishlock there.

For this reason each abutment has been provided with an eel passage having its entrance and exit immediately next to the bank. It is basically a "climbing-channel" with a gradient of 1-in-12 filled with year-old willow-twigs through which a slight current flows. The elvers wriggle upwards through the twigs, arriving halfway up the passage and at the upper end in resting basins where they can gather fresh energy for the remainder of the climb upstream. They still have to pass through the upstream gate into the upper pound; there a straw basket is provided against the top gate for the purpose. Water is let into the basket, which contains straw, from the upstream pound via an overspill in the gate, and the young eels have to crawl vertically upwards through the straw until they are finally enticed into the upper pound by the lure current. The volume and velocity of the water flowing through the basket and channel are set by raising or lowering the gate to suit the water level at the weir. This is done by a hand-operated winding mechanism.

Provision has to be made for renewing the straw and twigs in the eel passage. Wire netting on frames is placed over the upstream and downstream ends of the passage to keep predatory birds and crabs out. The upper end is shielded by the upper floor of the abutment, so the fish too are well protected against attacks from without.

There are perspex observation windows in the walls and underwater lamps attached to the opposite walls through which the operation of the fishway can be observed from a shaft alongside them.

Figure 29 and Figure 30. Cross-sections of fish lock and eel pass.
3.3. Hydro-electric generating plant

When work on the plans for the weir complexes began in 1950, the question was studied as to whether it would pay to incorporate hydro-electric generating plant in them. The conclusion was that at Hagestein, where advantage could be taken of the ebb and flow below the weir, it would pay to install a turbine with an intake capacity of 66 cub. m. per second and an annual output of $6 \times 10^6$ kWh. It was found that, for Amerongen and Driel, whose annual output would be $4 \times 10^6$ kWh, the capital investment would not be justified.

In view of the very complicated conditions under which the turbine had to operate, only one supplier was negotiated with on a cost price basis and when the price had been agreed the turbine was ordered and designed.

A turbine of the Kaplan type was chosen for the hydro-electric power station in the central pier at Hagestein weir, because ebb and flow produce a constantly varying head at this point.

In this type of turbine both the stator and the rotor blades are adjustable during operation. A cross-section of the turbine is shown in Figures 31a and b.

The turbine governor adjusts the stator and rotor blades to suit the constantly changing head.

To avoid having to construct deep foundations for the turbine discharge flume, the installation was designed on the siphoning principle, a vacuum pump being fitted to initiate and maintain the siphoning action.

The turbine, with a maximum output of 2,590 hp, rotates at 62.5 r.p.m.; it has a maximum intake capacity of 66 cub.m. per sec. and a head of 3.8 metres. Gears are used to step up the speed so that the generator operates at 750 r.p.m.

The generator (maximum output approx. 2,500 kVA) is connected directly to the 10 kV network of the "Provinciale Utrechtse Elektriciteits Mij." (PUEM) and is controlled for constant cos $\phi$ to allow for any variations in the mains voltage. An automatic voltage control adjusts the voltage during the running-up period, and after parallel switching functions together with the cos $\phi$ controller.

When the generator has been automatically coupled in parallel with the grid the entire installation comes under the control of a Rittmeyer controller. Water-level measuring points built close to the weir complex, both upstream and downstream, have floats with transducer equipment which measure the head.

In addition, there is Rittmeyer volume metering equipment in the pier to measure the amount of water flowing through the turbine. A receiver installation working in conjunction with the turbine governor ensures that the quantity of water flowing through is kept constant irrespective of the head at any particular time.

The turbine can only be put into operation if a number of essential conditions are met: the turbine governor oil circuit must be pressurized, there must be a vacuum in the siphon, and the temperature of the control-gear oil and lubricating oil must be
high enough. There must also be sufficient oil between the surfaces of the Mitchell block, which acts as a bearing carrying the vertical load.

The plant comes into operation in the following manner. A servo motor sets the rotor blades in the starting position. A second servo motor linked to the control ring of the guide apparatus then opens the 24 stators, allowing water to flow through the turbine from the volute and the turbine and generator gather speed.

Mounted on the generator shaft is an oscillating-type generator with permanent magnets coupled electrically to the main turbine governor motor.

The governor has two major components, one of which reacts to the turbine's speed of rotation and the other to the gear ratio. There is also an oil accumulator containing control-circuit oil under air pressure provided by an electrically driven compressor. The oil in the accumulator and the servo motors referred to above can close both the stators and the turbine blades under all conditions. It takes about six seconds to close the stators.

To prevent water hammer, the control ring actuates two venting valves in the volute housing. Should the stators fail to close during a shut-down, two further vent valves, each fitted in a recess in the pier above the volute, will open to break the siphon vacuum.

Mains water is pumped round a closed cooling circuit for the generator, gearbox and governor, and is itself cooled by river water in a heat exchanger. The river water inlet is fitted with a changeover filter which can be cleaned during operation. Two electric pumps are fitted in both the river and mains cooling water supply lines, each acting as a standby to the other. Should one of the pumps fail due to an electrical or mechanical fault, the other will automatically take over. A magnetic filter is incorporated in the lubricating oil system. An electrically driven grease pump automatically lubricates the plain bearing of the turbine shaft.

The temperature of the control-circuit oil and lubricating oil must be right before the turbine is started; should it be too low, electric heater elements will raise it.

The temperature of all the bearings in the installation and of the generator cooling air is shown on thermometer dials in the control room. The thermometers are fitted with switches which automatically shut down the turbine if the bearings or cooling air reach inadmissibly high temperatures, and a relay drops an annunciator flap to indicate the location of the fault. In addition to these visual warnings, there is also an audible alarm which can be heard virtually throughout the weir complex. When a mechanical breakdown occurs, the load on the plant is reduced and the generator cut off from the grid; this minimizes the rise in turbine speed.

If an electrical breakdown occurs, the generator will cut out immediately, irrespective of the level of the load, and in most cases even the generator rotor will be demagnetized.

All the relays needed to protect the generator are fitted. It was decided to have the plant shut down automatically in the event of a breakdown, because the power plant would be minded by lock-keepers, who are not trained electrical engineers.
Figure 31a. Cross-sections of pier showing "Kaplan" turbine.
Figure 31b. Detail of "Kaplan" turbine.
The electricity for this and other purposes must come from a source wholly independent of the three-phase mains supply. Accordingly, it is obtained from an automatically charged set of accumulators giving 110 V.

The electricity supply for the operation of the weir and locks, which must be maintained at all times, is in the event of a breakdown in the 10 kV supply taken over by a diesel standby generator unit, which will start up automatically within 7 seconds of any power failure. The standby unit is capable of supplying 70 kVA, or 56 kW at $\cos \phi 0.8$. The mains voltage is then 220/380 V.

The standby unit switching is arranged so that the signal lights for shipping, the lighting at the weir complex and a group of units regarded as vital, such as coolant pumps, oil pumps, the air compressor and greaser pump, will be switched over automatically in the event of a breakdown.

The non-vital group is not connected to any emergency source of current; this group includes such units as the heater elements and the vacuum pump, which do not need to function when the turbine unit is being shut down.

The power-consuming groups used in weir and lock operation, such as those operating the weir and lock gates, cannot draw electricity from the standby generator unit until the turbine has stopped.

To reduce the turbine shut-down time the generator is fitted with a hydraulic brake which comes into operation when the shaft speed has dropped to about 30% of its nominal figure. The frequency-responsive braking relay is linked to the oscillating generator for the purpose.

The hydro-electric plant is remote controlled from a switch console in the central control building. The system has been automated as far as possible, so that after the start-up signal has been received by the turbine's own operating equipment (which then comes into action entirely automatically in a predetermined order) the turbine will run up of its own accord and the generator will be automatically switched into the grid.

The Rittmeyer equipment referred to then controls the load on the generator automatically, and the correct operation of the switches is indicated on the console by a panel of lights.

Both deliberate and emergency stoppage (to deal with breakdowns) is likewise automated. The installation can also be brought into operation "by hand" by a fully-qualified engineer. There is a control panel behind the switch console on which the following items of control equipment are clearly laid out:

a. The Rittmeyer apparatus, which carries out the functions described above and also records the upstream and downstream water levels on chart-paper rolls; the water throughput, both that preset and that actually passed by the turbine, can also be read off.

b. The automatic voltage control, which during operation acts in conjunction with c. The automatic $\cos \phi$ control, to keep the $\cos \phi$ at the 0.7 level required by the PUEM, and the reverse-feed cutout.
d. The automatic Oerlikon parallel-switching unit.

The electric cut-outs referred to are fitted with individual annunciator flaps and are housed in a separate relay panel.

The switch console also has a mimic circuit diagram of the 10 kV switching and distribution point. It is at this point that the generator and the PUEM grid are linked. A transformer providing the current to operate the weir and locks is also shown.

This 200 kVA transformer converts the 10 kV voltage into a 220/380 V supply for the weir and locks.

Besides the conventional voltmeters, ammeters, wattmeters, etc. the turbine console also carries instruments to show the position of the aperture limiter, the stators and the head control roller.

The console also carries a revolution counter and the synchronizer instrument which indicates any discrepancy in voltage, frequency and "phase position" between the PUEM grid and the generator during the parallel-switching operation.

Lastly, the console has panel lights which indicate mechanical and electrical faults and show immediately in which group of annunciator relays the trouble is to be found.

Photograph 3. Hagestein weir looking downstream.
4. The visor gates

The preceding photograph shows the design of the visor gates; they consist basically of 8 mm steel plates stiffened along the top and bottom edges which can be raised by means of lattice-work lifting arms.

The drawings show details of the seal along the lower edge, the central hinge and the pivots. As the gates must be capable of replacement, the upper side of the deck below the pivot is three metres lower.

In addition to the recesses and steel seatings for the side seals and the housings for the pivot shaft bearings for the gate, the abutments and piers are also provided with flushing conduits which flush river sediment from the visor gate recesses. The general arrangement of the flushing conduits and some details of the conduit sluices are shown in the accompanying figure.

4.1. Lifting gear

The machinery for lifting the gates is housed in the top of the machine building on the arch, as shown in earlier illustrations. Each of the trunnions of the gate pivot shafts is carried in a self-adjusting barrel bearing with a conical adjusting sleeve. The maximum static load on the most heavily-loaded bearing is 700 tons. The bearing housing is completely filled with oil; a standpipe ensures adequate static pressure.

Sleeve packing is used for sealing the trunnion.

The pivot points are situated on the low-water side, so they are above water for the better part of the year.

Each set of lifting mechanism is powered by two electric motors giving 3 hp at 1,000 r.p.m.; the weir can be opened fully in approximately $2\frac{1}{2}$ hours.

The hoist rate at the drum is only 2.25 mm per second to prevent excessive wave attack along the river, so quite large reduction gearing is required. The gearbox provides a ratio of $1,340:1$ and the two other sets of reduction gearing bring the ratio up to the required $50,300:1$.

The two large cast-steel gearwheels on the cable drum have a pitch circle diameter of 3,530 mm (module 36.98 teeth). Four galvanized 66 mm diam. hoist cables are wound on the 2,250 mm diam. drum; they are attached to the suspension arms of the gate by a set of equalizer sheaves and a spring device.

The flexible coupling between the motors and the gearbox takes the form of a brake disc. The brake weight is lifted by a brake-lifting magnet; there are two brakes to each winch unit. All the shafts run in barrel bearings. A spindle of one of the gearboxes drives the spindle-end switches. The apparatus for ensuring synchronism and indicating position is actuated by one of the spindles of the other gearbox.

A hydraulic closing device is fitted to the gate at Hagestein to prevent wastage when the river is low and ensure that all the water flows through the turbine. This is not the
Figure 32b. Details of side and bottom seals and central hinge.
case at Amerongen and Driel, because there are no turbines there, so a certain amount of leakage is of no consequence.

There are watertight machine rooms in the two abutments and in the pier. Each hydraulic installation comprises a double-acting hydraulic cylinder, a gear pump unit and a spindle-end switch. The cylindrical gate valve seal is steel-to-steel on the seating and a rubber slab against the fixed cover.
Figure 34. General view and details of flushing culvert gate valve.

Figure 35. General view of lifting gear showing visor gate pivot.
Figure 36. Detail of lifting gear.
5. The cylindrical gate valve

The cylindrical gate valve is of the lidded type; details are shown in Figures 37a and b. In the latter illustration the gate is shown raised.

The rising floor of the inlet with its fixed guide vanes distributes the water evenly over the gate valve after its oblique inflow. At Amerongen and Driel the water enters symmetrically from two sides, since there are no turbines there.

Maintenance and inspection ways are provided above weir level.

Since the high velocities and consequent low pressure involve the risk of cavitation of the concrete at the overflow lip, the latter is encased in cast steel plating.

The maximum discharge capacity is 100 cub.m. per second. The gate valve is raised 60 cm higher than the theoretically necessary level for full throughput, thus safeguarding it against vibration.

The gate valve runs along a fixed steel guide system, four pairs of guide rollers fitted with rubber bushes affording enough elasticity for horizontal movements of the gate. There is also a device to prevent the gate from turning on its vertical axis.

The intakes for both the turbine and the cylindrical gate valve are protected by duckweed gratings, and the openings can be closed off with steel shutters when there is ice in the river.

5.1. Gate valve lifting gear

The lifting gear for the cylindrical gate valve is mounted on a foundation inside the shaft. The lifting speed is 1.25 cm per second, and lifting can be either continuous or in stages of 5 cub.m. per second throughput. The entire mechanism is powered by a 6 hp motor.

Facilities for emergency manual operation are provided.

The motor drives an endless Gall chain (breaking strength 150 t) through 1,300 : 1 reduction gearing.

A balance is hinged to the chain, and a vertical guide runs from the balance to the steel framework. A spring-loaded tensioning pulley further ensures that stretch and wear in the chain do not interfere with the movement.

Two spindle-end switches act as end stops for the gate and a flexible coupling with an electro-magnetically-operated brake is fitted between the electric motor and the reduction gearing.

The steel mitre gates are opened and closed by a Panama wheel system and the lock chamber is filled through sluices set very low in the gates.

Maximum hawser stresses are \[ P = \frac{W}{C} \], in which \( W \) is the water displacement and \( C \) is 1,000 if the force units are tons. The gates take 100 seconds to open or close.
Figure 37a. Cross-section of pier showing cylindrical gate valve.
Figure 37b. Detail of cylindrical gate valve.
Figure 38. Panama wheel in lock gate operating gear.
Figure 39. General view of steel sluice-gate.
6. Concrete weir and lock structures

The maximum foundation pressures are set up during construction and may reach 2 kg per sq.cm. This presents no problem on the good, hard sand underlying the site, as the foundation rafts reach at least 2 metres below the floor, and cofferdams prevent the sand from being forced out at the edges.

As already stated, the sills between the abutments and the pier have been designed to present an unbroken surface to the water to limit seepage.

Water-tightness is ensured by a layer of clay and this is compressed by the concrete slabs which also protect the clay against erosion.

A conduit had to be laid beneath the floor of the river to carry the cables. This has been made fairly roomy so that it can also be used as a service tunnel (since the gate cannot be crossed when it is raised). Lifts are provided in both the abutments and the pier, since 14 metres separate the cable and service tunnel and the deck.

In places where fast-flowing currents are deflected by sills, the concrete is clad with a protective coating of steel-grain concrete, and the bottom stop of the weir rests on a prefabricated and accurately set sill of steel-grain concrete.

The rear and bottom stops of the lock are also made of steel-grain concrete; they are prefabricated, adjusted in place on support fixing frameworks and then concreted-in. The lock is equipped with replaceable level gauges and stop strips. The lock chamber has a rectangular profile, and is divided into sections 22.5 m long by joints filled with tarred coconut rope to prevent the underlying sand from being washed out through the gap.

At Hagestein the concrete was made with 325 kg blast-furnace slag cement per cubic metre, while for the sand and gravel mixture efforts were made to keep as close as possible to the Fuller best mix curve. The maximum sieve size was to pass a 32 mm grain.

At Amerongen we used 275 kg blast-furnace slag cement and 50 kg trass per cubic metre of concrete, while the sand and gravel mixture was the same as for Hagestein. Moreover the water/cement ratio was about 0.52-0.55, the trass counting as cement.

Lastly, at Driel, a long series of tests resulted in the use of 300 kg blast-furnace slag cement per cubic metre of concrete, with a plasticizer and air-bubble former and a compact Fuller mix with a water/cement ratio of 0.48.

For the bulk concrete we used only 275 kg cement. Where there was a great deal of shuttering per cubic metre, the quantity was increased to 325 kg.

This weir has the best concrete work.

Experience has shown, however, that the standards set by the contractor and his team count for a very great deal when it comes to producing high-quality work.
Figure 40. Cross-section of sill, weir floor and cable conduit below weir floor.

Link between turbine and cylindrical gate valve outlet culvert and stilling basin.
Figure 41. Prefabricated sill set on supports prior to concreting.
Figure 42. Prefabricated bottom and rear seating set on supports prior to concreting.

Figure 43. Cross-section of lock chamber.
Figure 44. Joints showing tarred coco hawser.
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