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Datum: 12-2-1960
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International Course in Hydraulic Engineering

Navigation Canals

by Mr. A. Zanen

INTERNATIONAL COURSE IN HYDRAULIC ENGINEERING

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Contents of the lectures:

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1. Scope of the Subject.

Under navigation canals are to be understood artificially constructed waterways whose water level and possible stream velocities are more or less under control.

Thus in this lecture we shall not be discussing canals which function exclusively is for conveyance of water, such as irrigation canals, feed-channels to hydro-electric stations etc.

Canals will be dealt with which are chiefly intended to serve for transport by boat, although these, as it will appear later, will often have a secondary function in helping the water-conservancy of the district crossed.

2. Development.

The oldest means of transport is doubtless that by boat. Even in primitive forms of civilization, transport, although often deficient, took place by boat through streams and rivers.

During the period in which Egyptian and Chinese civilizations flourished, there appears already to have been mention of artificial waterways, and so canals.

But actual canal construction only became significant after the lock-gate had been invented in the second half of the 17th Century, whereby it became possible to bring ships from one waterlevel to another in a simple manner. In this way the possibility arose of crossing the watershed between two rivers by ships, and thereby increase their radius of action.

So long as trading activity is still limited, chiefly the natural waterways will be used. However, as soon as a district reaches such a stage of development that it goes over from the sum-total of isolated components, to a society in which exploitation, industrial and commercial centres can be distinguished, it becomes necessary to create the means for transporting persons and goods from one place to another.

This transport can take place over land (road and railway), in the air, by water (natural and constructed waterway).

Now it is not the case that an arbitrary choice is made from these possibilities.

The nature of what is to be carried will often determine the most economical means of transport.

It will also often happen that the transport from producer to user takes place by various ways eg. by boat or rail and by lorry, whereby the product must be transferred from the one means of transport to the other under way.

A typifying difference between the various means of transport lies in the quantity which is possible per journey, and the speed with which the journey can take place.

Transport by water, for example, takes place with a limited speed, but it is possible for great quantities simultaneously. It lends itself also especially well for the transportation of large units.

Transport by aeroplane is quick, but is with only small quantities simultaneously.

Transport by truck or rail lies between the two extremes mentioned.

Clearly, perishable products (eg. fruit, flowers, etc.) are transported by rapid means. Valuable products too (eg. diamonds) will be carried quickly for preference, because loss of interest and greater chance of theft begin to play with prolonged transit.

Piece goods will, in general, be more valuable than bulk goods, and a higher transport cost can be afforded if on the other hand transit is quicker. In a similar way it will often be more advantageous to use a boat for transport of large amounts of low value bulk goods (eg. coke) over big distances.

It is clear that a certain means of transport is only possible if

certain conditions have been satisfied, such as the building of works (roads, railway, airfields, canals, river improvements) and the investment of capital in material (trucks, goods-trains, aircraft, ships).

Before one goes over to creating a certain possibility of transport, the clearly indicable advantages and disadvantages will be weighed up one against the other, and the result is used as a chief guide.

Typical of an economically well ordered society, is good connection between the various means of transport, their completion of one another, and the conveyance of products by the most suitable means.

Hereby the transport costs will be as low as is possible, which raises the economic exchange-value of the products by a very large measure. According to some economists, the exchange-value of products is actually proportional to $(\text{lowering in freight-charges})^2$.

In Holland, with its many natural waterways, there arose only in the beginning of the 19th Century, in connection with the increase in trading activity, more and more need for improvement and mutual connection of the waterways. From 1820 onwards, a great number of canals was dug. Some of these are intended for connection of harbours with the sea (eg. North-Holland Canal, Canal through Voorne, Ems-Canal, North Sea Canal), but most serve more especially for internial shipping.

Abroad there is a development to be seen. In France for example, a canal system was built for connection of Seine, Marne and Rhine, and of Rhine and Rhone, However, there is abroad markedly decreased activity in the field of canal-building to be seen when about the middle of the 19th Century the railway appeared and this was seen as an unconquerable competitor in goods transport.

In Holland, both means of transport were considered as coexistent and useful, and building of canals went on. In the 19th Century there were opened:

North-Holland Canal, Zuid-Willemsvaart, Sas van Gent-Terneuzen. Apeldoorn Canal, Canal through Voorne, Overijsel Canals, peat Canals, Noord-Willems Canal, Ems Canal,South Beveland Canal, Walcheren Canal, North Sea Canal, Merwede Canal,

Neither has the development of road-transport, which has been set forward since World War I, had any slowing-down influence here on the canal-building.

Since 1919 have been laid consecutively the Wilhelmina Canal, the Meuse-Waal Canal, the Canal Wessem Nederweert, the Juliana Canal, the Twenthe Canals, the waterway Groningen-Lemmer and the Amsterdam-Rhine Canal, Also in the future, important canals will still be built (Antwerp-Moerdijk), Emmen-Delfzijl, etc.).

In order to give an impression of the relationship between the various means of transport in Holland, we give some figures:

Weight carried (tons):

Boat 37%	Rail 14%	Road 49%

Amount carried (ton-kilometers):

Boat 48%	Rail 26%	Road 26%

From this it can be seen that the average distance of conveyance differs rather widely for the various means of transport, by boat c. 100 km, by rail c.150 km and by road c.25 km.

The above-mentioned ratios have stabilised in Holland and each of the means has thereby proved its right of existence.

In the other West European countries the ratios are quite different.

Of the ratios of the lengths of the communication networks are compared for rail, boat and road, the following pattern is obtained:

	Rail		Boat		Road		km. waterway per km^2 area
Holland	1	:	2,4	:	9	:	0,209
Belgium	1	:	0,3	:	9	:	0,058
France	1	:	0,2	:	6	:	0,024
Germany	1	:	0,1	:	4	:	0,017
England	1	:	0,2	:	10	;	--

Holland is thus quite justly described as "Waterland". The reason for this must be sought in our many natural waterwaysand also in the flat nature of our country, for one will not so quickly proceed to build a canal in hilly country as in flat.

That elsewhere in the world water is considered as a very important line of communication is nicely illustrated by the fact that in Canada it is apparently considered justified to invest very big capital in increasing the navigability of the St. Laurence, a river which is unnavigable anyway for 5 to 6 months of the year because of ice.

3. <u>The composition of the inland shipping fleet.</u>

In order to be able to set up the requirements which a canal to be built must satisfy, it should be found out what types of boat will use the canal, and with what frequency.

In this way, the dimensions of the biggest ship which it is thought to admit to the canal will play a role in fixing the cross-section and course of the canal, according to the frequency with which it is expected on the canal.

Also the hydraulic phenomena which will occur in the transit of a towed boat, differ from those excited by a self-propelled boat. This can exert influence on the desired cross-section or on the construction of the bank revetments of the canal.

Although a certain type of ship can more often occur in a certain district than one would presume from reviewing the composition of the inland fleet, yet can this composition form a useful starting point for the prediction which will have to be made concerning the traffic expected.

The Dutch inland fleet, divided according to size is composed of the following approximate percentages:

	number	carrying-capacity
21 - 100 ton	39%	8,3%
100 - 200 "	26%	12,6%
200 - 400 "	15%	15,7%
400 - 600 "	9%	16,8%
600 - 1000 "	6%	16,7%
1000 - 1500 "	3,5%	17,3%
1500 "	1,5%	12,6%

Division according to method of propulsion is approximately as follows:

	number	carrying-capacity
Towed vessels	31%	58%
Motor vessels	52%	35%
Vessels with side-screw	5%	3%
Vessels with push-boat	10%	3,5%
Sail vessels and steamvessels	2%	0,5%

Seeing that a boat has quite a long a life-tune (35 to 40 years) any possible variations come about slowly. Even so, there is a clear tendency towards strong motorisation to be observed. The very big towboats are gradually being driven out by smaller boats with self-propulsion There is increasing activity expected from boats of 200 - 600 tons and decreasing from the boats under (sure) 200 tons and above (possibly) 1500 tons.

The Dutch inland fleet, as appears from these figures, marks a great differentiation in dimensions. This is not so everywhere, at least not in this measure. For example in the North-French canals ships are met with of near enough similar dimensions.

Considering that in the future the waterways of the various countries will be connected more and more with one another, it is logical that there is always more need for some standardization in ship dimensions.

Thus five boat-types have been arrived at for the mainland of Western Europe. In building locks and bridges, the dimensions can be then so chosen that the types of boat which will use the canal, will be able to pass. This division is as follows:

Class	Tonnage	Type of Vessel	Maximum Dimensions in metres
			l x b x d x h
I	1350-2000 t.	Rhine-Herne	80 x 9,50 x 2,50 x 4,40
II	600-1000	Dortmund-Ems Canal	67 x 8,20 x 2,50 x 3,95
III	300- 500	Kempenaar	50 x 6,60 x 2,35/2,50 x 4,20
IV	100- 300	Spits	38,50 x 5 x 1,90/2,20 x 3,55
V	100	-----	h = max. height when unloaded.

For the dimensions of 2000 ton vessels can be reckoned 100 x 12 x 2,80 x 6,70 m. In Western Europe no reckoning is yet made in the building of locks and canals for the introduction of push-tow navigation which outside Europe finds quite a lot of application.

In designing a canal one will have to take into account any possible coasters, which more and more try to push their way inland in order to save transver-loading costs. The dimensions of the coasters are to be found in the handbooks. They are characterised by a greater draught. Attention must also be paid to the big motor capacity of such ships, which make cruising at a high speed possible.

4. Considerations which can lead to the digging of a canal.

In general it can be said that construction of a canal is justified if the costs incurred in the building and exploitation plus the costs of travelling through that canal in the broadest sense, are such that goods transport will be able to take place with advantage through the new navigation channel.

It can, however, be put forward more broadly and account also be taken that the construction of a waterway can diminish the isolation of certain districts, and that the presence of a canal often opens possibilities of improved water conservancy in the district traversed, which increases the production of the soil and thereby, indirectly, trading activity.

A consideration which sometimes counts abroad is the possibility of generation energy in an economical manner at the sluices in a lateral canal, since the discharge through the canal can be regulated independently of the river-discharge, whereby a constant energy output can be guaranteed.

Political arguments can also come into play, as was the case for example in the construction of the Juliana Canal and is now the case with the connection of Antwerp with Dordrecht, which is being considered.

Finally, in periods of unemployment, canal-construction can serve as a means of providing work.

Thus a complicated set of factors are concerned and the decision to proceed to the construction of a canal will only be able to be taken after much weighing the pros and cons all the more so because it is difficult to predict how the future development will be.

In Holland, a commission specially set up at that time carried out an investigation into the economics of the projected Twente canals. One of the expectations of the Commission was that the industries, after the construction of the canal, would bring up their coal by boat instead of by train. For various reasons, this expectation was not correct. In the first place because the railways promptly lowered their tariffs, and secondly because industry often buys 20 ton coal many times rather than 600 tons in one lot.

The estimated increase in value of the land along the canal had little effect because buyers did not come forward.

The decisive argument of better water conservancy in the intersected area was, however good.

It thus certainly appears that in the setting up of future expectations, there yet always exist the possibility of favourable or unfavourable surprises. This arises because the composition of such a commercial reckoning rests on subjective judgements. There can exist very important differences between the reckoning of the advocates and opponents of the canal construction which does not make the affair any easier.

It is to be hoped that with help of modern sciences such as sociography, market analysis etc., the subjective element will be able to be more and more pushed back.

Sometimes the decision is not difficult. This was the case for example for the Meuse-Waal Canal, which formed the continuous link in the transport of coal from the Limburg mines to central Holland. In such a case there need be no mention at all of remunerativeness.

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5. Course, Length-profile and Water conservancy.

A canal can:

a) form the connection between two rivers or two seas.
b) connect two points on the same river.
c) on one side connect into a waterway.

In the first case the canal forms a link between two natural waterways (eg. Meuse-Waal Canal).

In the second case the canal replaces a portion of the river which does not satisfy the requirements, either by too great detours or by bad navigability (eg. Juliana Canal).

In the third case the canal serves to open up a district (eg. Twente canals).

In <u>all</u> cases, the course is broadly fixed by the function which the canal will have to fulfill.

In general the terrain which will be intersected by the canal will not be everywhere at the same height and even less will the water-table be everywhere at the same level.

In case a) this is seen clearly demonstrated because the canal must hereby pass the watershed between the two rivers. Moreover, in cases a) and b), the waterlevels into which the canal at both ends connects, will be different in height.

In connection with this a canal is divided into reaches with different levels so that the canal more or less follows the terrain.

With this, there is a means of limiting soil movement for the construction, and the possibility of working with a soil balance.

By means of a lock-gate ships can come from the one canalreach into the other. Since a lock is an expensive structure and moreover causes delay for shipping, one will aim at limiting the number of locks and thus also the number of canal reaches.

This means a greater fall to be retained at each lock, but also deeped excavations and higher embankments.

Aan important point is the question how to take care that the reaches contain enough water, and further, in what measure the water-table will be influenced by the canal in the intersected area.

It can occur that it is of importance that certain geological formations are not broken into, and this can be of influence in fixing the depth and level or the course of the canal.

Sometimes existing natural or dug streams can be made use of in determining the course.

The suitability of the soil as a foundation soil can influence the siting of structures such as locks and bridges. It can be useful to lay the course such that industrial establishment in population centres is stimulated. One can also consider making such centres accessible to navigation by means of a branch-canal.

The way in which crossings of land and water can be most favourably adapted to the existing systems will have to be gone into.

Along with all these considerations, it must be tried to give the canal an aspect harmonises with the landscape.

The height of the canal embankments, siting and manner of storing the soil removed, routing with gentle curves are points, along with others, which crop up here.

All in all, the routing of the course, taking into account the factors mentioned, is only possible after a sound study of the advantages and disadvantages of various possible courses.

6. <u>Cross-Section and Bank-Revetments I.</u>

If a prediction has been made concerning the shipping which is to be expected, this results in a certain insight being acquired into the dimensions of the boats, and also into frequency with which boats of various size will appear.

Now the difficulty always comes up that we grope in uncertainty about any possible development of shipping traffic in the future.

The quantity of goods to be transported is tied up with the extent of industrialisation - especially in the matter of this last factor, predictions are speculative.

Eeven if we were in a position to predict the extent of shipping in the distant future, it would still not be economical to fix the dimensions of the canal from this.

Certainly there would then for a long time be a canal which would be bigger than necessary, whereby unnecessary strips of land would be taken up for that time.

A sensible line of action seems to be to dimension the canal generously on the shipping which is expected in the near future, and then take into account that later broadening may possibly have to be carried out. For example, a strip of ground may be purchased broad enough for the widened profile, or leaving room for a future second set of locks next to the initially built locks.

The ground which is not directly necessary can be exploited (eg. by leasing) as long as there is no need to proceed to the widening of the canal.

In this way, the advantage is realized that later, for the widening, no land has to be bought up which has just risen in value through the presence of the canal.

In designing bridges over the canal, is is worth considering dimensioning these directly on the widened canal profile, since a later rebuilding is possibly more expensive than the loss of rent if a bigger bridge is built directly.

Possible future widening of the cross-profile can have influence on the construction of the bank revetments too.

If the canal is so made that widening will take place on one side, there a bank revetment will be designed which will have a more temporary character, and thus often differ importantly from the more permanent bank revetment on the opposite side.
If it is thought that it can be predicted that the shipping on a canal will never increase importantly, then naturally no room need be reserved for widening and permanent bank revetments will be laid on both banks.

From the foregoing, it follows that there is in view a certain extent and frequency distribution of the size of the shipping when one proceeds to design the cross-section.

Now the biggest ship which must be able to navigate the canal will occur much less often than the ships which will mostly be met with on the canal. This is most accentuated on the canals in the great navigation-routes.

So, for exemple, the Meuse-Waal Canal must be navigable for ships of 2000 ton, whilst the "average" ship remains below the 400 tons. On economic grounds, a speed-limit can be laid on the few large ships in such cases, and, in designing the cross-section, more reckoning be held with the smaller ships which appear many times and on which it is undesirable to lay restrictions.

In order to arrive at the dimensions of the cross-section consideration can be given on the one side to the standpoint of the skipper who will have certain minimum desires for good navigation and on the other side it can be asked how the maintenance of the canal can be limited by keeping the hydraulic effects aroused by the transit of the ships through the canal below certain limits.

Naturally there will be a connection between both these preliminary considerations. It will appear later how the dimensions of the cross-sections can be arrived at taking into account both aspects.

For this it is necessary to go into what hydraulic phenomena can occur and at the same time into how far these are limiting for the dimensions of the cross-section or for the construction of the bank revetments.

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7. Hydraulic Phenomena.

The hydraulic phenomena which can occur in a navigation canal can depend on the traffic through that canal, directly or indirectly, but can also be due to other causes.

Among the first mentioned phenomena can be counted the condition of flow and the wave-pattern which accompanies the moving vessel and also the consequences of filling or emptying a lock-chamber, in which a quantity of water is given to or taken from a canal-reach in a relatively short time.

Other causes can be: wind, drainage and ground-water flow.

In succession, we shall look at:

a. Primary phenomena which occur when a vessel moves through a canal (surface-drop and return flow)

b. Secondary phenomena which occur when a vessel moves through a canal (ship-waves)

c. Screw action

d. Flow and level variations due to filling or emptying a lock chamber

e. Ditto - due to drainage

f. Ditto - due to surging

g. Waves due to wind

h. Ground-water flow

i. Combinations of the effects described in a - h.

a. Surface-drop and return-flow as a result of a ship's transit through a canal.

That both these effects do occur can be observed by standing on the bank and watching a vessel pass.

The occurrence of these effects is to be understood if we consider that the travelling vessel pushes water forward and at the rear leaves a void behind. It is obvious that the quantity of water pushed up in front, flows backwards alongside and under the vessel to fill up the void. Through this flow, a velocity-head arises and thus there is a lowering of the water-level in the region of the vessel.

By Krey equations were established in 1913 with the help of which the surface-drop and velocity of the return-flow can be calculated if we can begin with given values for:

wetted perimeter of the canalsection, F

area of the biggest cross-section of the vessel, f

width of the canal at the water-line B

and the velocity of the vessel, v.
Herewith, for simplifying the problem, the following approximate assumptions are made:

a. the vessel drops, for its whole length, as much as the water-level.

b. the velocity distribution of the return-flow round the vessel is uniform.

c. the friction of the flowing water along the side-slopes and the bottom of the canal and along the hull of the vessel are neglected.

d. the turbulence-losses round the prow of the vessel are neglected.

The equations established by Krey are the continuity equation and the equation of Bernoulli, both with respect to a co-ordinate axes system moving with the vessel.
They appear as:

$$v.F = (v+u)(F-f-Bz) \text{ and}$$

$$\frac{v^2}{2g} = \frac{(v+u)^2}{2g} - z$$

in which u represents the velocity of return-flow and z the surface-drop.
A moving system of axis has been chosen because in the derivation of the formula of Bernoulli a steady motion is assumed.
The flowpattern for a ship in motion is only steady if the system of axes is made to move the same speed as the ship.
In the two basic-equations are unknowns u and z (properly speaking, v too).
In principle, solved as 3[rd] degree equation.
Solution time consuming, for F assume a value, for v assume a value.
It is not seen how u and z are influenced.

Attention was devised to this by ir. Schijf and Professor Jansen at the International Navigation Congresses at Lisbon and Rome.
To find z: eliminating u from both equations gives:

$$1 - \frac{f}{F} - \frac{z}{h} - \left(1 + 2\frac{z}{v^2} \cdot \frac{gh}{h}\right)^{-\frac{1}{2}} = 0 \qquad \text{average depth } h = \frac{F}{B}$$

To find u: eliminate z

$$1 - \frac{f}{F} - \frac{1}{2} \cdot \frac{v^2}{gh} \left\{ \left(1 + \frac{\frac{u}{\sqrt{gh}}}{\frac{v}{\sqrt{gh}}}\right)^2 - 1 \right\} - \left(1 - \frac{\frac{u}{\sqrt{gh}}}{\frac{v}{\sqrt{gh}}}\right)^{-1} = 0$$

Two dimensionless diagrams, which show:

$\frac{z}{h}$ and $\frac{u}{\sqrt{gh}}$ as function of $\frac{f}{F}$ and $\frac{v}{\sqrt{gh}}$

With the help of these diagrams, for each value of $\frac{f}{F}$ for every value of

$\frac{v}{\sqrt{gh}}$, the operating $\frac{z}{h}$ and $\frac{u}{\sqrt{gh}}$ can be read off, and from this the value of the surface-drop and return-flow corresponding to the chosen $\frac{f}{F}$ and navigating speed v, can be derived.

In this way we have succeeded in giving a more manageable form to the basic equations of Krey.

If we consider the two graphs once more, we see that the curves for all values of $\frac{z}{h}$ resp. $\frac{u}{\sqrt{gh}}$ lie below the upper lines, which form the envelopes.

Apparently with a value of $\frac{f}{F} = 0,3$ for example, $\frac{v}{\sqrt{gh}}$ cannot reach a value higher than 0,37.

For higher values than this, no value is read off for $\frac{z}{h}$ resp. $\frac{u}{\sqrt{gh}}$

On purely mathematical considerations, the conclusion is reached that a so called natural limiting speed exists, which cannot be exeeded. When this was discovered, the physical explanation of the natural speed-limit was sought. It was then seen that, seen hydraulically, it made a lot of difference where the propelling force of the vessel came from.

If it is drawn from a force of reaction on the water, such as for example with a ship's screw or a paddle wheel, the following reasoning can be followed.

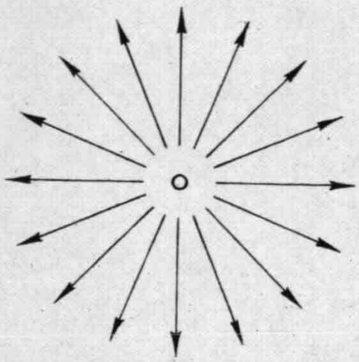
If the vessel moves with steady speed then the resulting force in the direction of motion is zero (there is anyway no acceleration or retardation).

If the resultant force of the water on the vessel is zero, also the resultant force from the vessel exerted on the water is zero.

If the section of the canal is considered which rests in equilibrium, then it cannot be otherwise than that the water-level at two points (one before and one behind the ship) must be at the same height. This is an important conclusion which forms a key to the physical explanation sought. The faster the vessel travels, the more water must flow along and around the vessel from front to back in the same time. This can be compared with a pier in flowing water.

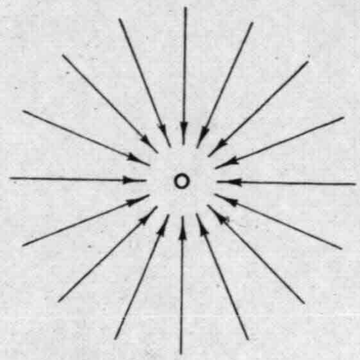
Through every section there flows the same quantity of water. The stream velocity by the pier must therefore be larger, which means that the velocity-head is greater there. From hydraulics, we know that the discharge has a maximum if the velocity-head $\frac{v^2}{2g} = \frac{\text{Total Energy Head}}{3}$. With increasing velocity (and thus velocity-head) the discharge decreases again.

If the discharge of the river is greater than the maximum which can be a



Bleeder well

b



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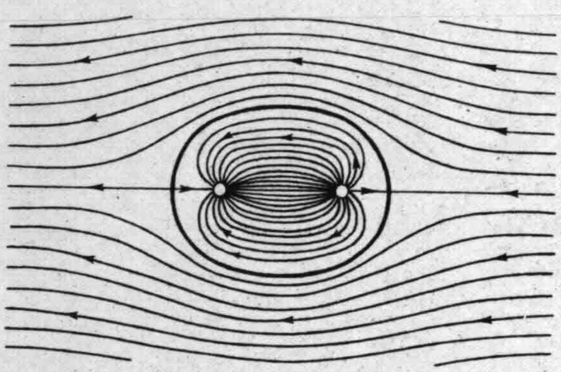
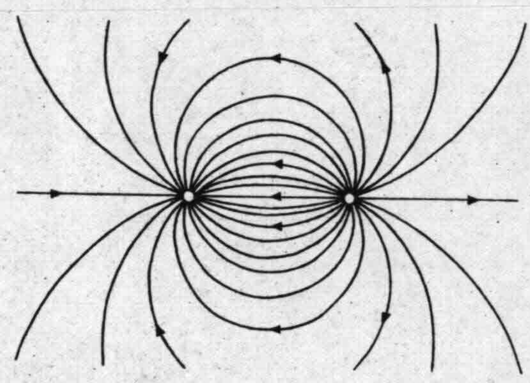
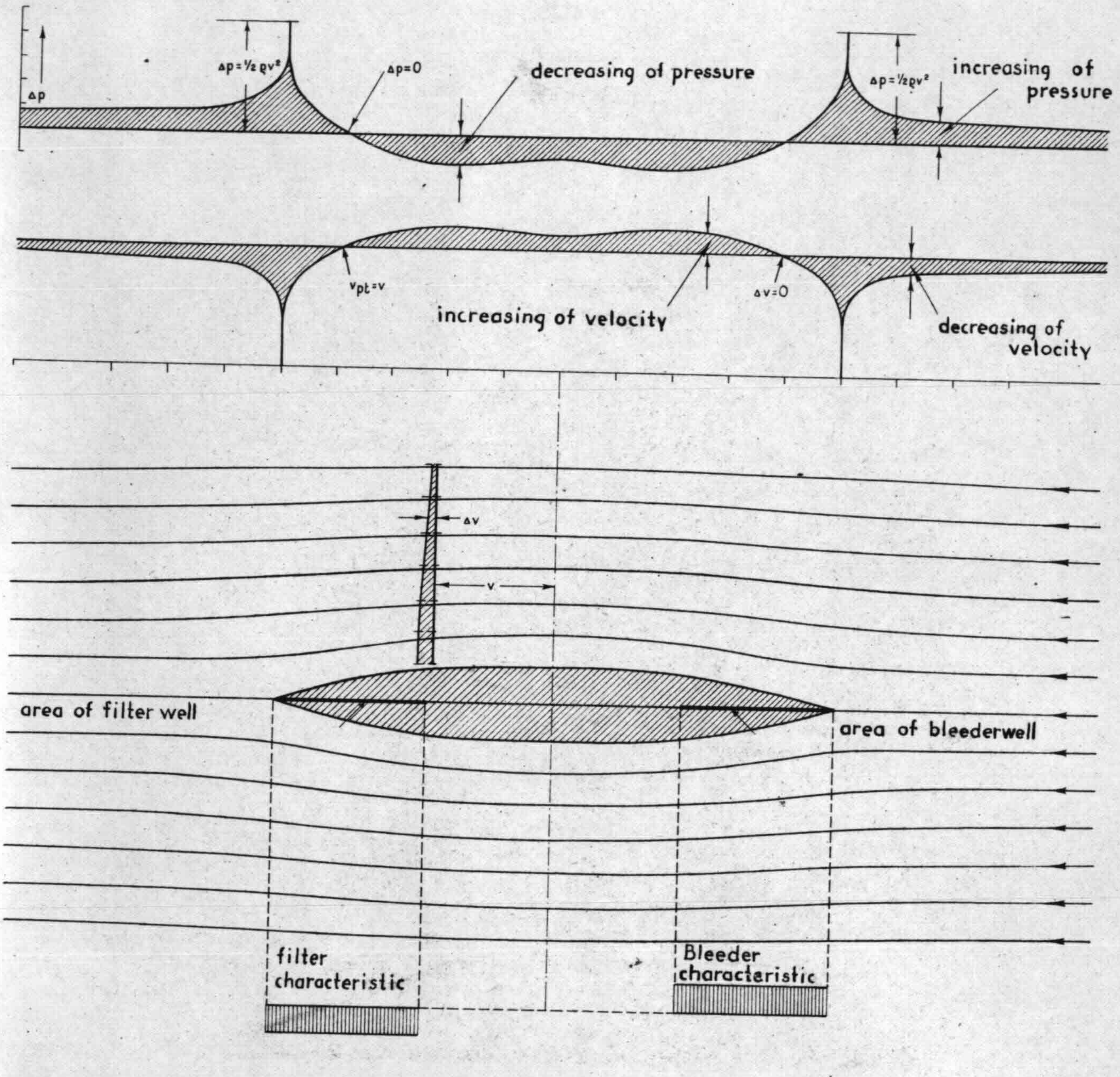


Combination a + b and transportflow

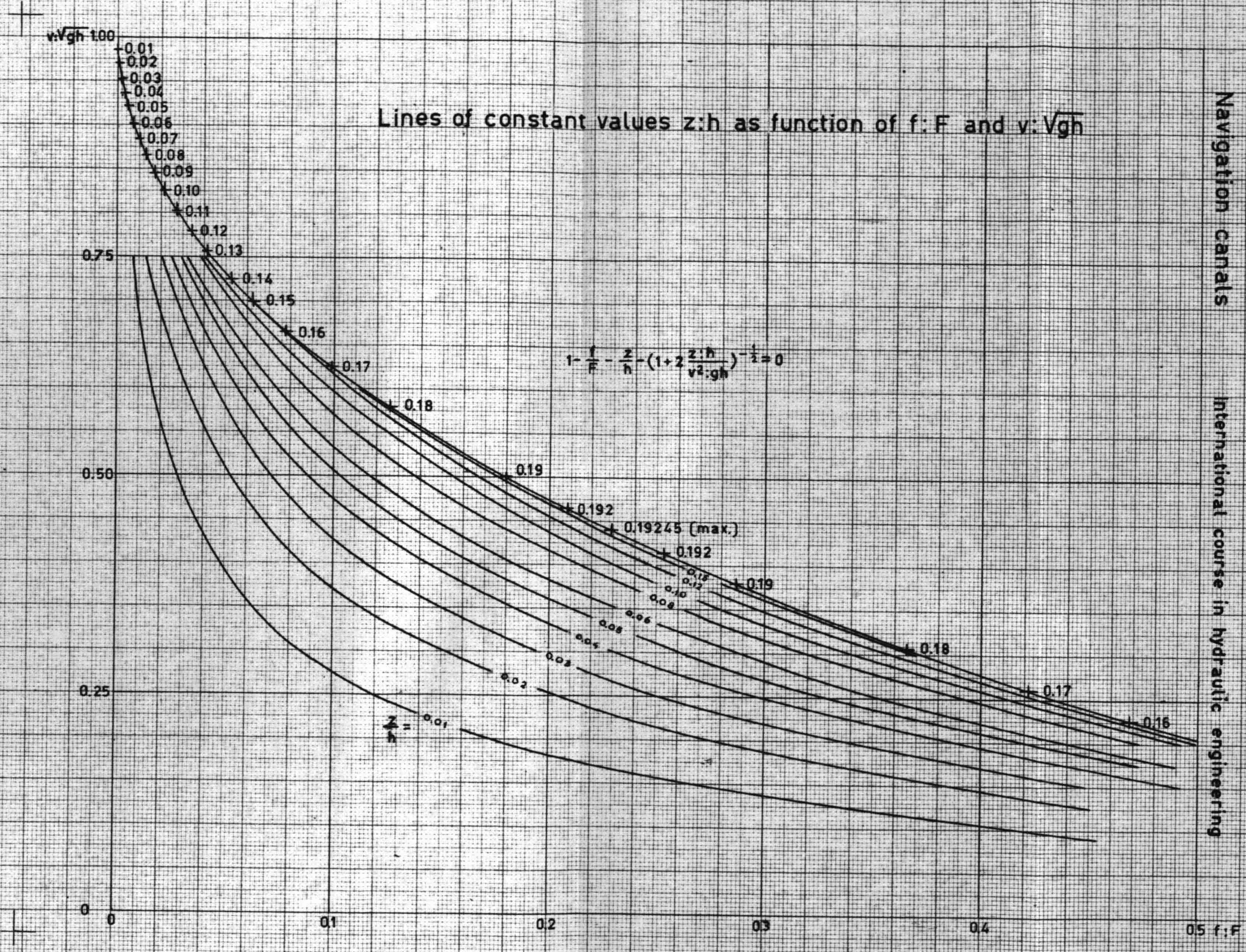
c



Combination a + b



Streamlines, velocities and pressures in a two-dimensial flow around the form of a ship (according to Horn)


Lines of constant values z:h as function of f: F and v: √gh
$1-\frac{f}{F}-\frac{z}{h}-\left(1+2\frac{z:h}{v^2:gh}\right)^{-\frac{1}{2}}=0$
v:√gh 1.00
0.75
0.50
0.25
0
0.1
0.2
0.3
0.4
0.5 f:F
0.01
0.02
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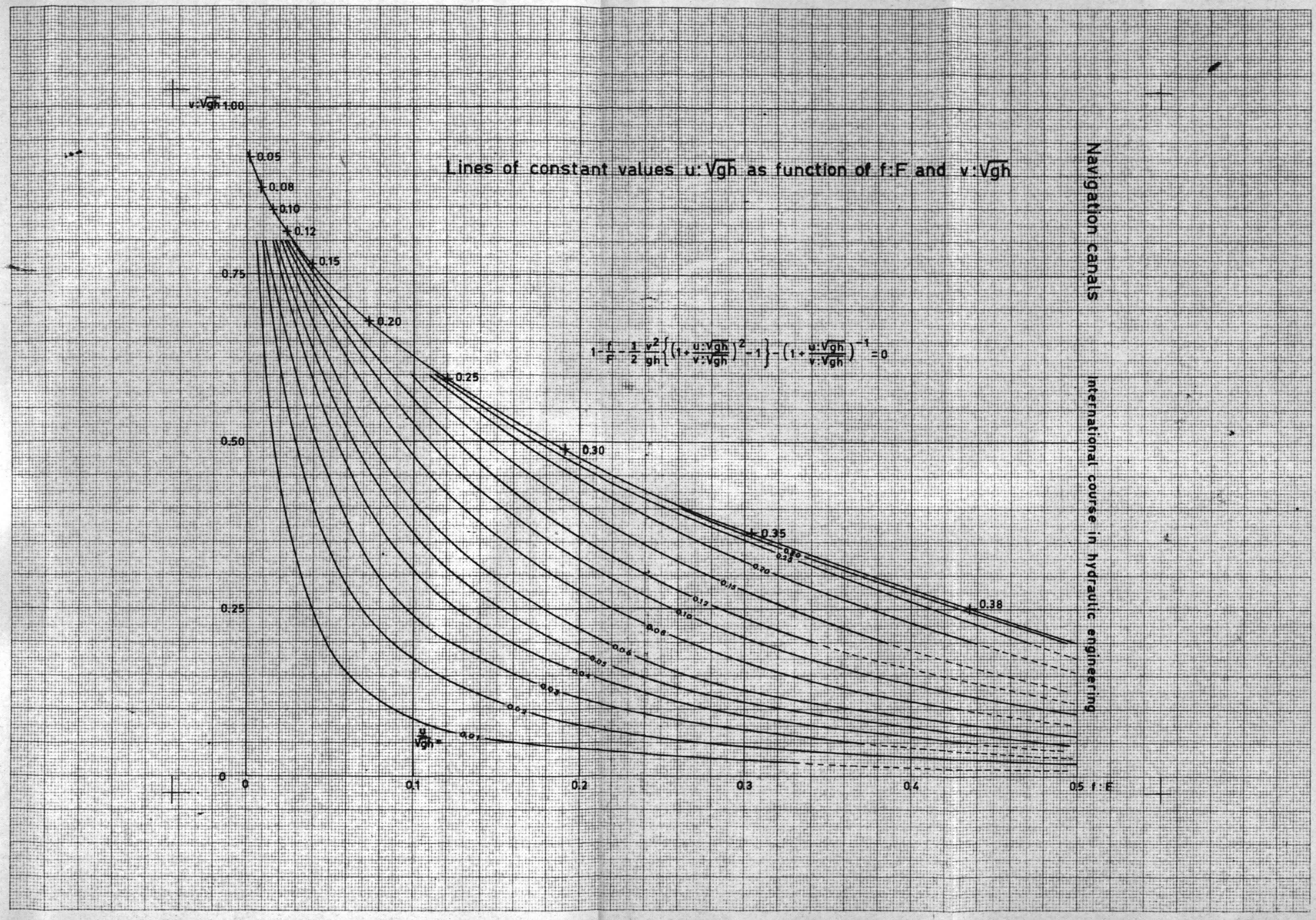
Lines of constant values u: √gh as function of f:F and v:√gh
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$1-\frac{f}{F}-\frac{1}{2}\frac{v^2}{gh}\left\{\left(1+\frac{u:\sqrt{gh}}{v:\sqrt{gh}}\right)^2-1\right\}-\left(1+\frac{u:\sqrt{gh}}{v:\sqrt{gh}}\right)^{-1}=0$
v:√gh
1.00
0.75
0.50
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f : F
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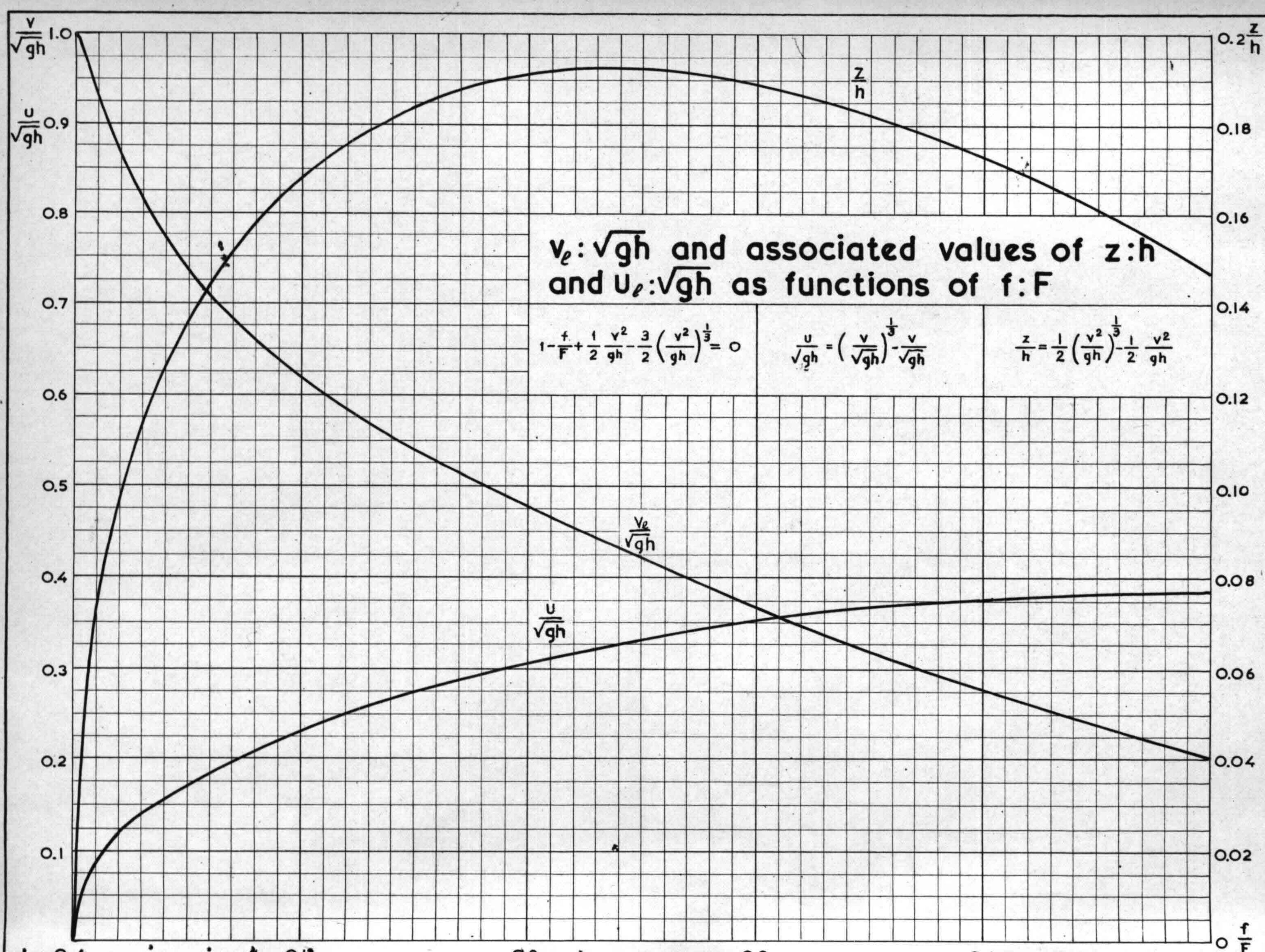
0.25
0.30


$v_\ell : \sqrt{gh}$ and associated values of $z:h$
and $u_\ell : \sqrt{gh}$ as functions of $f:F$
$1 - \frac{f}{F} + \frac{1}{2}\frac{v^2}{gh} - \frac{3}{2}\left(\frac{v^2}{gh}\right)^{\frac{1}{3}} = 0$
$\frac{u}{\sqrt{gh}} = \left(\frac{v}{\sqrt{gh}}\right)^{\frac{1}{3}} - \frac{v}{\sqrt{gh}}$
$\frac{z}{h} = \frac{1}{2}\left(\frac{v^2}{gh}\right)^{\frac{1}{3}} - \frac{1}{2}\frac{v^2}{gh}$
$\frac{v}{\sqrt{gh}}$
$\frac{u}{\sqrt{gh}}$
$\frac{z}{h}$
$\frac{v_\ell}{\sqrt{gh}}$
$\frac{f}{F}$
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
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0.18
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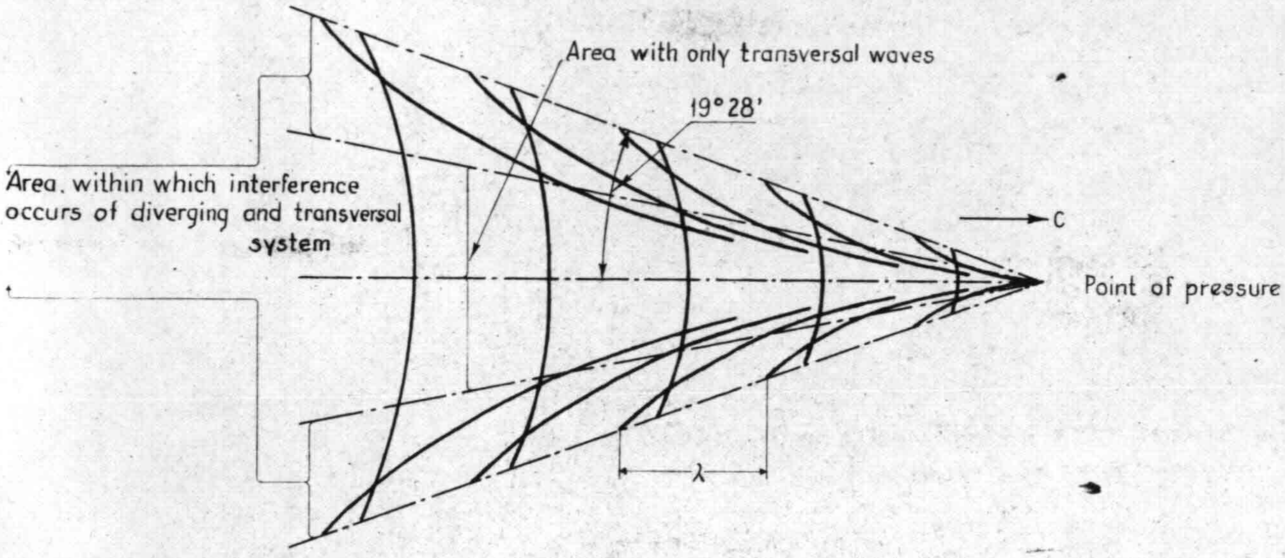
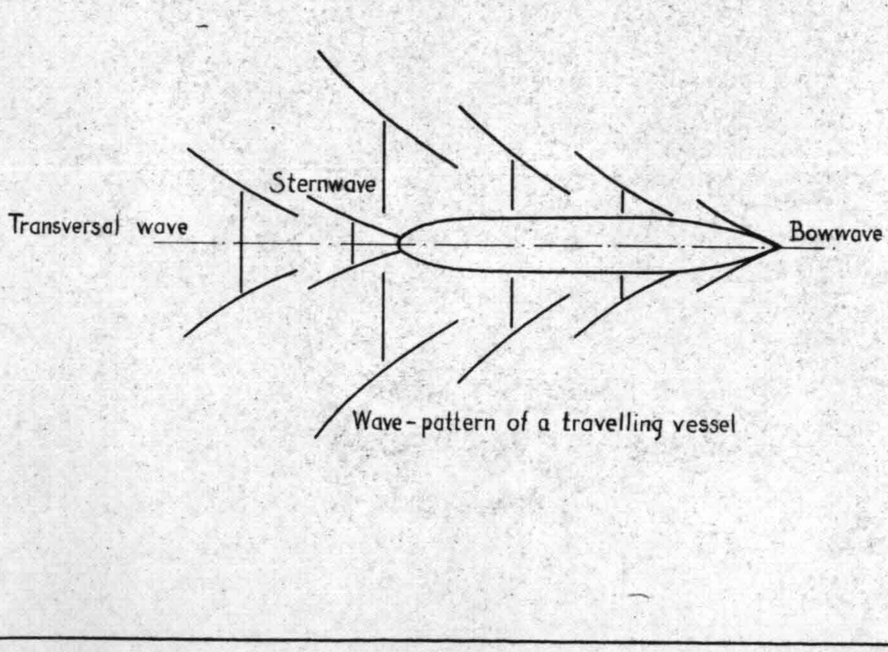
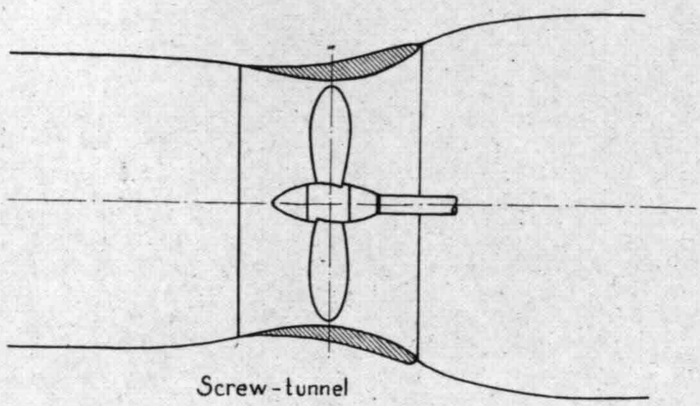
0

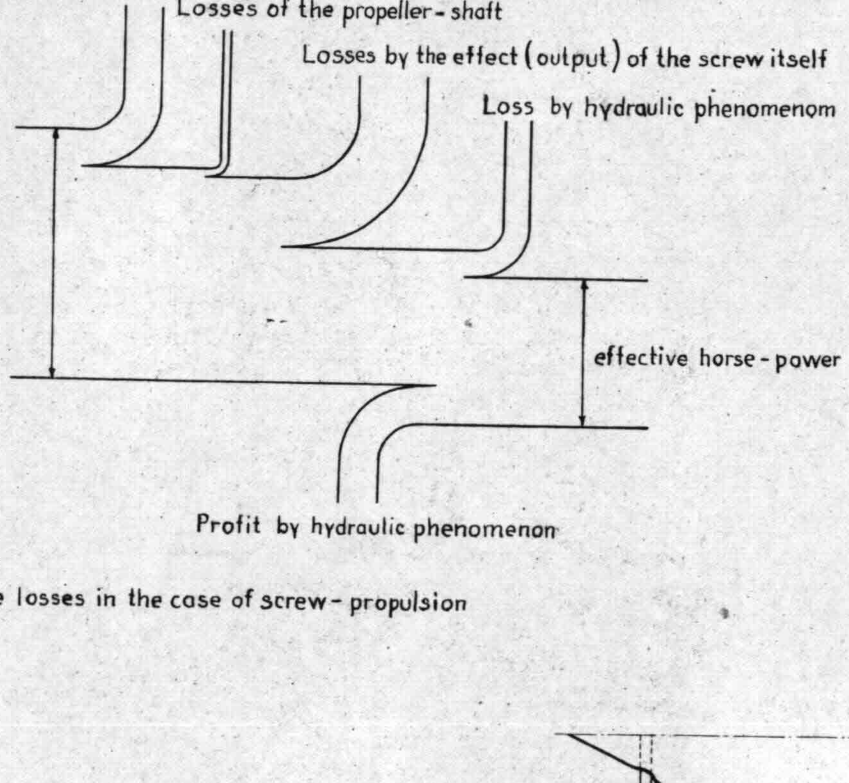




Wave-pattern of a travelling vessel

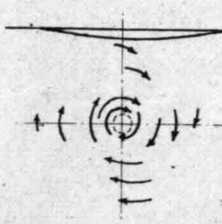


Screw - tunnel

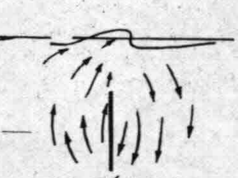


Survey of the losses in the case of screw - propulsion

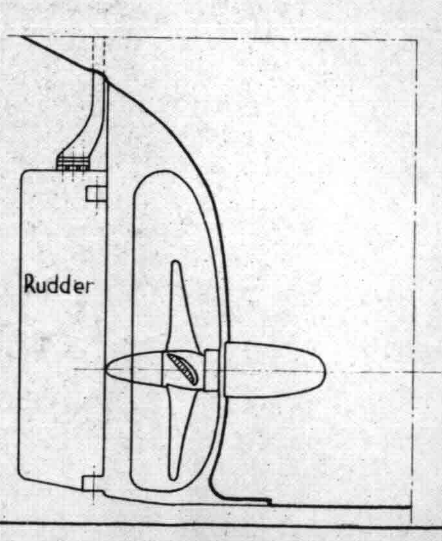
Longitudinal section of the screw-jet



Cross-section of the screw-jet



Cross-section of the screw-jet with rudder



hereby reached, all the water cannot pass. The waterlevel, and with it the total head upstream of the pier rises and does so until the discharge can pass.

For the self-propelled vessel, we have shown that the waterlevel before and behind the vessel is the same. This means that no damming up takes place and that there is thus a maximum discharge, thus a maximum velocity-head and thus a maximum navigation speed. If the vessel is being drawn from the bank, then, if the navigation speed is steady, there is a force exerted on the vessel which is equal in magnitude but opposed in direction to the force K with which the vessel is being drawn.

If we now consider a canal-section, then for equilibrium, the water-level in a point before the ship must be higher than that in a point behind the ship. (The difference in water pressure force K). If we pull harder K increases, then the difference in level between the two points, mentioned before, increases, whereby the desired quantity of water can pass.

In this case, there is no natural limiting velocity. It is possible, even if at the cost of much energy, to reach any desired velocity.

We limit ourselves here to the vessels which propel themselves through the water. From this, we know what the greatest speed is which can be reached. From the diagrams, the corresponding values of $\frac{z}{h}$ and $\frac{u}{\sqrt{gh}}$ can be read off.

The fact that the h occurs in the denominator in both of these terms and in the term $\frac{v}{\sqrt{gh}}$ signifies that the greatest average velocity can be reached with a big average depth, and also that for a certain velocity, the velocity of the return-flow and the surface-drop are smaller if the average depth is bigger.

<u>To know this can be of importance in designing the cross-section of the canal</u>. Apparently in contradiction to the above-mentioned theory, is the large velocity which little boats of a certain form can reach (hydro-planes). Because when in motion the boat is partly lifted out of the water by the form of the prow, the value of f/F decreases further and further. In the diagrams we see that then a greater value of $\frac{v}{\sqrt{gh}}$ becomes possible. The boat planes as it were along the curve towards the value $\frac{v}{\sqrt{gh}} = 1$ which cannot, however, be reached since the value $\frac{f}{F}$ cannot become zero.

In the report of the International Congress on Navigation at Rome, a diagram is given, from which the connection between the value of the various quantities can be directly read off. We can see that the greatest surface-drop, which amounts to c.0,2 h, occurs with a ratio $\frac{f}{F} = 0,23$.

If the surface-drop is a limiting factor, then this is the most dangerous vessel. If, on the other hand, the size of the return current is the limiting factor, then the biggest vessels are the most dangerous.

We will return to the design of the cross-section.

In practice it appears that the resistance which a ship must overcome in travelling, increases inproportionately as the natural limiting velocity is approached. This means an inproportionate fuel consumption, which implies economic brake on the skipper.

<u>Navigation will seldom be faster than 90% of the natural limiting velocity.</u>

In connection with the simplified assumption which were made establishing the equations, it was necessary to test the results obtained by calculation, on measurements in the laboratory and in practice.

Thereby a) the existence of a natural limiting speed was established b) the lowering of the vessel was observed to be on the average about equal to the surface-drop. Through action of the screw flow and the form of the stern, however, the lowering of the stern can be quite a lot higher. The surface-drop is not at the same level over the whole canal width.

There appeared to be agreement between calculation and practice if in the second basic equation, a coefficient $\pm$ 1,1 was applied to the term

$$\frac{(v + u)^2}{2g}$$

Naturally, if the vessel departs from the axis of the canal, both the wave movement and the return velocity (and thus also the surface-drop) will make themselves most felt between the vessel and the nearest bank.

INTERNATIONAL COURSE IN HYDRAULIC ENGINEERING

Navigation Canals

by Mr. A. Zanen

On purely mathematical and on physical considerations the existence of a natural limiting speed for a ship travelling through a canal is proved. Also it was stated that navigation will seldom be faster than 90% of this natural limit.

With aid of the theory developed by Mr. Schijf we are able now to make a computation of the surface-drop and of the velocity of the returnflow in the case that one ship is moving through a canal.

In addition to one, also two ships can be simultaneously in a canal section.

If two ships meet each other, the two flow systems partly cancel each other out. Moreover, the ships are alongside each other for a fairly short time.

Except that the attention of the skippers is required, there are no great difficulties for the ships to overcome. This becomes different when two vessels overtake each other. The problems which crop up here are again to be approached with the help of the two basic equations, established for various stages of the overtaking-manoeuvre. It would be going too far to work this out.

We will be satisfied with giving some typifying situations during the process of overtaking.

The overtaking vessel has two obstacles to overcome. The first occurs as soon as the overtaking vessel comes by the overtaken vessel.

It comes then in a section which has been narrowed near to the section of the vessel to be overtaken and in which return-flow from that vessel prevails. The resistance thus becomes greater and the limiting-speed of the overtaking vessel comes to stand lower (smaller F and flow contrary). Since it is customary in the world of shipping to reduce speed if another vessel wants to overtake, this first obstacle will be able to be easily overcome. The second obstacle often offers more difficulties. This occurs when the bows of the overtaking vessel have come past those of the vessel to be overtaken.

The most forward part of the overtaking vessel lies in its own surface-drop and the rear part in the surface-drop of both vessels together.

The vessel must thus, as it were, travel up the slope, which naturally implies decrease of speed. Added to this, the vessel to be overtaken travels down the slope. That is, its bows lie in the combined surface-drop. This vessel experiences increase of speed.

In this way the danger exists that both ships will go down the canal moving as one system.

In that case, only setting off the engine of the overtaken vessel offers a way out. In order to avoid this undesirable situation the condition must be fulfilled that, at the moment that the danger of travelling as one system exists, the overtaking vessel can develop enough propulsive force to work its way up the slope and thereby complete the overtaking manoeuvre. we see here a balance between the size of the canal profile (that is with a broader profile, the slope to be overcome becomes smaller) and the capacity of the ship's engine and screw.

As illustration of the navigating speeds and the hydraulic phenomena during overtaking, it has been calculated what happens.

<u>a</u>. when a 1000 ton ship travels through the Maas-Waal Canal

<u>b</u>. when two 1000 ton ships overtake with various speeds of the overtaken ship

<u>c</u>. when a 1000 ton ship is overtaken bij a 200 ton ship

<u>d</u>. when two 200 ton ships move trough the canal as one system (thus an unsuccessful overtaking manoeuvre).

In the table the values found for the speed of return flow and the surface-drop are given.

ase	Speed of Travel Ship to be overtaken/Overtaking ship	Speed of Return Flow	Surface Drop	Remarks
aip in he axis	Limiting speed = 2,85 m/sec. 0,9 x " = 2,56 m/sec	1,30 m/sec. 0,70 "	0,55 m 0,27 m	
ship long the ank	0,9 x " = 2,56 m/sec	Max.1,05 m/sec	Max. 0,40	Estimated from flow pattern
vertaking anoeuvre self- ropelled essels of 000 ton	V_1=V_2= 2,23 m/sec = V limiting group V_1= 2,0 m/sec V_2= 2,3 m/sec V_1= 1,8 " V_2= 2,33 " V_1= 1,5 " V_2= 2,42 " V_1= 1,0 " V_2= 2,45 "	1,85 " 1,80 " 1,75 " 1,59 " 1,56 "	0,565 m 0,56 " 0,53 " 0,505 " 0,495 "	
vertaking anoeuvre, self- ropelled essel of 0 ton and itto of 000 ton	V_1= 2,4 " V_2= 2,86 "	1,38 "	0,49 "	
self ropelled essels 200 t	V_1=V_2=3,18 m/sec = V limiting group	1,33 "	0,495 "	

We see thus that the strongest speed of return-flow occurs with an unsuccessful manoeuvre, in which two self-propelled vessels of the biggest type move as one system through the canal. The surface-drop corresponding to this is of the same order of size as for the case of one ship with the natural speed for the canal navigation.

From what has been dealt with above, we can for every case predict fairly accurately what surface-drop and speed of return-flow will occur.

b. <u>Secondary Phenomena in the passage of a vessel through a canal.</u>

It is a known fact that a certain wave-formation occurs as a result of a region of pressure moving relative to the fluid. If this relative velocity lies below 0,233 m/sec., then <u>no</u> waves occur as a result of surface tension.

For velocities of importance in engineering, a system of gravity waves developes.
In 1891 a mathematical derivation was given of this by Lord Kelvin which agrees with observations, but not useful in practice as the formulae are very complicated.

The system consists of slightly curved diverging and transversal waves. Every point of pressure arouses its own wave-system with similar speed of propagation c = V. The resulting wave-system of a ship is chiefly governed by the great sudden pressure-change at bow and stern and influenced by the regions of strong pressure decrease following the bows and preceding the stern (the so-called for' and after shoulder).

The research which has been carried out in this field, had as aim to go into how far the form of the ship is related to the wave-system excited and in how far it would be possible to come to an optimum form. In the wave-system which runs with the ship, there is a certain amount of energy stored, which is supplied by the means of propulsion of the ship. Because the transverse-wave and the diverging waves both occur in a certain region, interference will arise. The result of this is what is observed, i.c. a wave-system running up obliquely against the bank and a region where the interfering transversal bow- and stern-waves occur.

From the latter, the energy is gradually expended by internal friction. By this no damage on the canal can be inflicted.

This is otherwise with the wave-system running obliquely up the bank, which can cause erosion of the talus.

In respect of the height of the waves, the theory gives no quantitative solution and we are thus referred to the result of observation.

The wave phenomena in the model and in actuality have been measured by Krey. It appeared that with the navigation speeds with which one is concerned in practice, the height of the waves near the bank can be near enough equated to the surface-drop Z, which can be calculated.

In normal cases, the place where the waves reach the banks lies behind the rear end of the ship, where the surface-drop has already considerably decreased again. However, the subtended angle of the diverging wave-system which in "deep" water amounts to $19^{\circ}28'$ becomes greater with very high navigating speed. This arises because, with V, λ increases and the water changes from "deep" to "shallow" with respect to the wave. The component $\perp$ the bank then becomes smaller.

For the natural limiting speed, the angle subtended is actually 90°. This all follows from the formulas.

If it is desired to observe the worst wave, in respect of erosion of the bottom and low parts of the bank, it has been shown that these should be allowed to occur with a still-water level lowered by Z.

On the other hand, the normal water level will be taken if we are concerned with examining erosion on higher parts of the bank. Apparently the interests of the ship designer go along with those of the canal constructor. The ship designer will try to design the ship so that it will experience the last resistance in travel. This means less energy-loss, thus fewer waves and turbulences in the water. This is desirable for the canal designer, too. However, with the ship-builder it is a question of higher returns. A decreased resistance by a good shape of the ship opens for him the possibility of reaching a higher velocity expenditure of the same amount of energy, because as it has appeared, the necessary energy per ton-km. is approximately inversely proportional to the navigation speed.

To a higher navigation speed, belong again higher waves and moreover, the primary phenomena of surface-drop and return-flow will increase.

It is possible that in the future, the means of propulsion of the ship will be perfected.

We will have to reckon thus that the navigation speed in the future will be increased and that pressure will be brought to bear on the canal designer to take care that, by a broader profile, the natural limiting speed will come to lie higher.

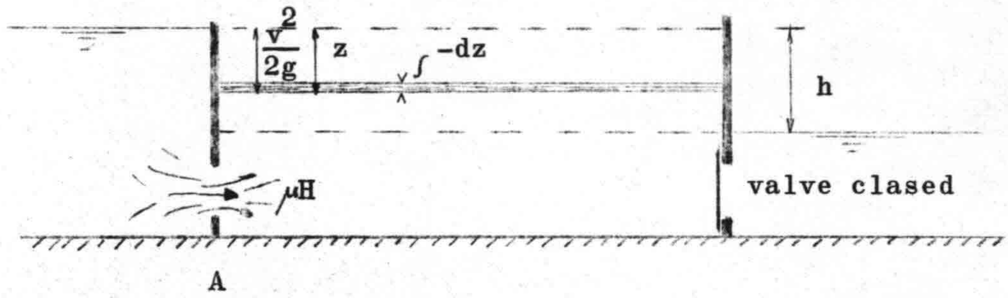
Just as the road-builder has had to keep adapting himself to the increased demands of road traffic, so the canal-builder too, on economic grounds, will have to meet the desires of shipping. In many cases it will not be justified to make a very broad profile directly, yet it seems in buying the land to reserve a strip ready for a future widening of the profile. This is done in many cases.

INTERNATIONAL COURSE IN HYDRAULIC ENGINEERING

Navigation Canals

by Mr. A. Zanen.

Filling of a lock-chamber.



During the filling of the lock-chamber z is a function of the time. To calculate the time necessary to fill the chamber we make use of the equations of motion and of continuity.

In the area of acceleration around the valve A for the equation of motion Bernoulli's law may be applied.

In the situation as showed in figure 1 the velocity-head near the valve A $\frac{V^2}{2g}$ will be equal to z. So: $V = \sqrt{2gz}$.

On that moment the discharge through the valve A will be:

$\underline{Q} = \mu\ Av = \mu\ A\sqrt{2gz}$ (μ = contraction coëfficient).

Now we make the assumption that the waterlevel rises gradually over the whole lock-chamber (without translation waves). During the time interval dt the rise of the waterlevel in the lock-chamber is $-\ dz$.

Because of continuity we can state:

I : $\boxed{\mu A \sqrt{2gz}\ .\ dt = -\ dz\ .\ 0}$ (0 is surface of waterarea in lock-chamber)

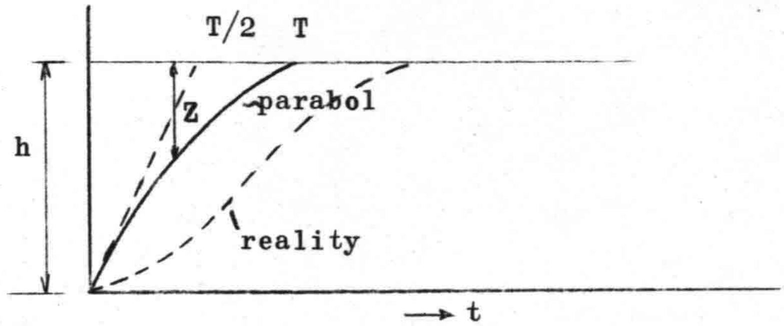
Integration of equation I will give t as a function of z:

$$\int dt = -\frac{0}{\mu A\sqrt{2g}}\int\frac{dz}{\sqrt{z}} \longrightarrow t = -\frac{0}{\mu A\sqrt{2g}}\ .\ 2\sqrt{z} + C.$$

The condition that for $t = 0$ $z = h$ gives:

$C = \frac{0}{\mu A\sqrt{2g}}\ .\ 2\sqrt{h}$, and so:

II: $\boxed{t = \frac{20}{\mu A\sqrt{2g}}\left(\sqrt{h} - \sqrt{z}\right)}$ This is a parabol. (see fig. 2).



The lock-chamber is filled if $z = 0$

So the time of filling $T = \dfrac{2\,O\sqrt{h}}{\mu A\sqrt{2g}} = \dfrac{2\,Oh}{\mu A\sqrt{2gh}}$.

Now $\mu A\sqrt{2gh}$ is the amount of water which enters the lock-chamber per sec. when $t = 0$.

$O.h$ is the total amount of water to be brought into the lock-chamber.

(If the velocity $v = \sqrt{2gh}$ (when $t = 0$) could be maintained during the whole process of filling the lock-chamber should be filled in $\frac{T}{2}$ sec.; so the value of T is easy to remember).

We want to know Q as a function of t.

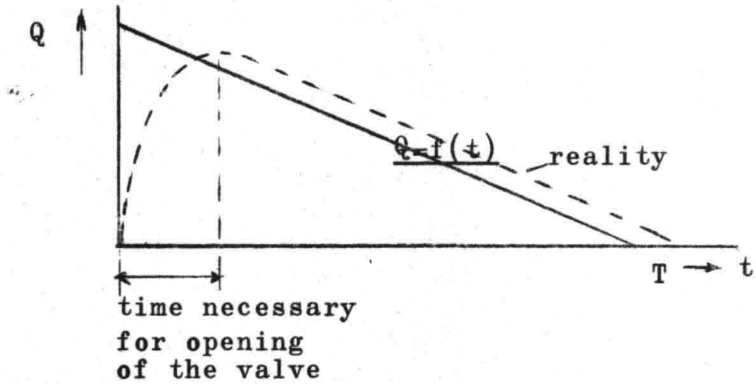
We have: $Q = \mu A\sqrt{2gz}$

and $t = \dfrac{2\,O}{\mu A\sqrt{2g}}(\sqrt{h} - \sqrt{z})$

Eliminating the factor z gives:

III: $\boxed{Q = \mu A\sqrt{2gh} - \dfrac{\mu^2 A^2\, 2g}{2\,O}\, .\, t.}$ (linear function).

see fig. 3.



So far the time necessary for opening the valve is neglected. Therefore the function $Q = f(t)$ in fig. 3 cannot be correct in reality.

Now we assume that during the time of opening the valve the cross-section A will increase in a linear way.

$\mu A = Ft$ (F is chosen such that for the value of t to open the valve: $F = \frac{\mu A}{t}$).

Substitution of $\mu A = Ft$ in equation I gives:

$Ft \sqrt{2gz}\, dt = -\, dz \,.\, 0$ or

$Ft\, dt = -\frac{0}{\sqrt{2g}} \cdot \frac{dz}{\sqrt{z}}$, and after intergration:

IV: $\boxed{\frac{1}{2} Ft^2 = \frac{2\,0}{\sqrt{2g}} (\sqrt{h} - \sqrt{z})}$

For illustration an example is given now:

Example: Given: $\mu A = 4{,}2$ m^2

h = 2,- m

length of lock-chamber = 140 m.

width " " = 14 m.

The valves can be opened in 60 sec.

and during this time the cross-section A increases in a linear way.

Calculation of the total time necessary for the filling of the lock-chamber:

If t = 60 sec. $F = \frac{4{,}2}{60} = 0{,}07$.

The first step is the calculation of the waterlevel after 60 sec. (valve opened totally).

Equation IV gives:

$$0{,}07 \cdot \frac{3600}{2} = \frac{2.140.14}{\sqrt{2g}} (\sqrt{2} - \sqrt{z}), \text{ and } z = 1{,}62 \text{ m.}$$

Now equation $T = \frac{2\,0\sqrt{h}}{A\ \ 2g}$ gives the additional time necessary for the filling of the chamber. Here h is 1,62 m.

So $$T = \frac{2.140.14 \sqrt{1{,}62}}{4{,}2 \sqrt{2g}} = 268 \text{ sec.}$$

Total time of filling is 60 + 268 = $\underline{\underline{328}}$ sec $\approx$ $\underline{\underline{5\frac{1}{2}}}$ minute.

Remarks: 1) Equation III gives the relation between Q and t for the period that the valve is opened totally. (In the example for h to take 1,62 m.).

During the opening of the valve this relation does not hold. The right relation can be found, for we know during that period $Q = \mu A \sqrt{2gz}$, and

$\frac{1}{2} Ft^2 = \frac{2\,0}{\sqrt{2g}} (\sqrt{h} - \sqrt{z})$. In example to take h = 2m.).

Elimination of z will give the relation between Q and t. during the opening of the valve.

2) If the cross-section A will not increase in a linear way during the opening of the valve, the calculation will be more complicate. In that case $\mu A = Ft^n$ has to be taken, in which n is chosen such that the formula represents the alterations in the cross-section of the valve during the opening manoeuvre.

INTERNATIONAL COURSE IN HYDRAULIC ENGINEERING

Navigation Canals

by Mr. A. Zanen.

7-c. Screw Action.

The action of the shipsscrew rests on the fact that pressure on the water is exerted by moving a surface of special shape and set, through the water.

The reaction-pressure of the water on that surface thendelivers the force propulsion required.

By the sucking action on the front side of the screw, there arises a pressure-lowering whereby the rear part of the ship undergoes an extra amount of in-sinking.

The principle of the screw action has found its practical application in a hub with 2, 3 or 4 blades.

The speed of the screw is so chosen that the surface of the turning screw-blade has a velocity directed backwards with respect to the water, so that an extra velocity is given to the oncoming water (the screw-jet).

This has as consequence:

1. the already mentioned axial component, directed forwards, delivers the propelling force of the ship;
2. a certain amount of energy in the water propelled backwards, which is lost in eddies and finally in heat in the water.

An important part of this energy is in the rotation which is given to the particles of water in the screw-stream by the screw blade, in turning through the water, giving the screw-stream a tangential velocity.

In spite of the many improvements which have been made to the screw in the course of time, the efficiency is still rather low (30 to 40%). This means that about two-thirds of the screw energy is lost and largely finishes up in the water.

In normal cases, the screw-stream is axially directed and will only be able to reach a small section of the walls of the canal because, before this, the internal friction and diffusion will have done their useful work.

As a result of the rotation of the water particles in the screw stream, however, bottom erosion can occur of the rudder hinders a normal rotation. Naturally this injurious effect occurs in much less measure if either a double rudder or two screws are applied.

With a travelling ship, bottom and wall of the canal are only for a short time exposed to the screw-stream. In places where the possibility exists that the ships have no or a small velocity (bridges, locks) the attack is of a more sustained nature and thus more dangerous. In such places, extra precautions could be considered. It is immediately clear that test-vuns may only be allowed in a place where the screw-jet can cause no erosion of consequence. The complaint, often heard formerly, of bottom erosion through screw action of deeply loaden ships is almost not heard any more since the margin between the under-side of the ship and the canal bottom has, for other reasons, been chosen bigger.

A water-mattress of 1 m. is apparently sufficient to prevent erosion.

The danger of damaging the side slopes by screw action from a ship travelling close along the bank remains, however, in existence. For example, this can be happen in meeting or overtaking of each other.

7-d. Stream and level-alterations in filling or emptying a lock chamber.

If the lock chamber of a lock is brought from the level of the one reach to the level of the other, in the locking of ships from one reach to the other, then in a relatively short time, a big lot of water is taken from or added to a canal reach. From hydraulics, we learn that this causes a so-called "long wave" that is both positive and negative.

The height of this long wave is directly proportional to the quantity of water which per unit time is taken or added, and inversely proportional to the width of the canal and to the speed of propagation of the wave. In a formula: $Z = \frac{Q}{b.c}$

Considering that the value of Q varies with time, Z will vary in the same way, whereby, neglecting alterations in the form, the shape of the wave is determined.

In this way for example the slope of the wave front must not be so steep as to cause trouble to ships situated in that canal reach.

The slope of the front is

$$\frac{\partial Z}{\partial s} = \frac{1}{b.c} \frac{\partial Q}{\partial s} = \frac{1}{b \, . \, c^2} \frac{\partial Q}{\partial t}$$

The value of $\frac{\partial Q}{\partial t}$ is dependent on the rate of lifting of the valve and on the shape of the valve. The slope $\frac{\partial Z}{\partial s}$ is thus under control.

The speed of propagation C of a long wave is as a first approximation $C = \sqrt{gh}$, and as second approximation $C = \sqrt{gh} \, . \, (1 + \frac{3Z}{2h})$ if the size of Z can no longer be neglected in comparison with h.

From the formula it follows that the top of the wave has a greater speed of propagation than the lower parts, whereby alteration in shape

arises. The front will become continually steeper and the back continually more gradual.

At the same time, the wave will be damped by friction etc. whereby front as well as back of the wave will become more gradual.

The decrease from the original wave height Z_o due to friction, is well approximated in the formula:

$$\Delta \left(\frac{1}{Z_o}\right) = \frac{S}{\psi . \frac{C^2}{g} R . h}$$

Where ψ is a coefficient which varies from 1 to $1\frac{1}{2}$

S is the distance covered

R is the hydraulic mean radius

h is the average depth of the canal.

The use of this formula is demonstrated in an example.

Given: canal h_{av} = 3.- m.

R = 2.80 m.

C = 55 $m^{\frac{1}{2}}$/sec.

$\psi = \frac{4}{3}$

Z_o for s = 0 for instance 0,40 m.

Computation: $\left(\frac{1}{Z_o}\right) = \frac{s}{\frac{4}{3} . \frac{3000}{10} . 2,8 . 3,0} = 0,30$ m./km.

Table

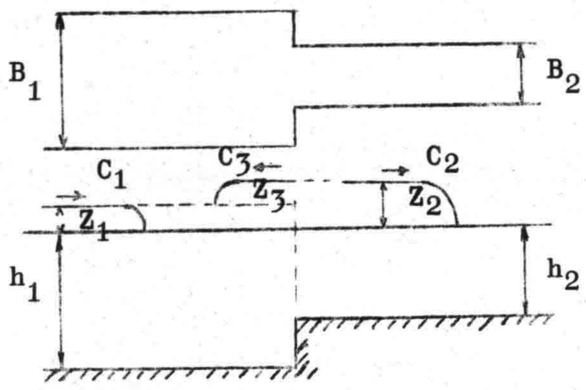
S	Z_o	$\frac{1}{Z_o}$	$\left(\frac{1}{Z_o}\right)$
0 km.	0,40	2,5	-
1 "	0,36	2,8	0,30
2 "	0,32	3,1	0,60
5 "	0,25	3,99	1,49
10 "	0,18	5,48	2,98

Tried out in practice.

Since the damping generally is the governing factor, the result of both factors is that the slope becomes more gradual, the front slowly, the back quickly.

This is also a reason to take care that $\frac{\partial Q}{\partial t}$, and so $\frac{\partial Z}{\partial s}$ do not get too big, because then the alteration in shape due to the difference in speed of propagation might well be able to get the upper hand, whereby finally such a steep front arises, that the wave propagates itself through the canal as a sort of bore, with all the disadvantageous consequences of this. The velocity of flow which is aroused by a long wave is in general not of much significance. A fairly uniform velocity distribution arises in the vertical. $V = \frac{Z}{h_o + Z} \cdot c$. For a positive translation-wave the stream velocity is in the direction of the wave, for a negative translation-wave, the water flows in a negative sense.

With junctions or alternations in profile, the translation-wave behaves according to the formulae



$$Z_2 = Z_1 \frac{2}{1 + \frac{B_2\,C_2}{B_1\,C_1}}$$

$$Z_3 = Z_1 \frac{1 - \frac{B_2\,C_2}{B_1\,C_1}}{1 + \frac{B_2\,C_2}{B_1\,C_1}}$$

Z_3 indicates thus a portial reflection of the primary wave with height Z_1.

It can be of importance to investigate this in, for example designing the head-room of structures.

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Navigation Canals

by Mr. A. Zanen.

e. Flow and change in level due to drainage.

If the canal has a function in drainage, this means that transport of water takes place and that flow occurs in the direction of the axis of the canal. Considering that a difference of head is necessary to the occurence of flow, changes in level will occur simultaneously.

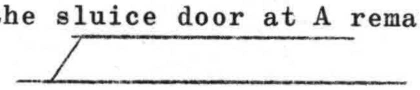
It happens quite frequently, that in a canal which at the same time has a draining function, sluices are situated next to the locks which separate the various canal-reaches from each other.

This case will be discussed more fully here.

Other cases, such as that somewhere in a canal reach water is added to or taken from the canal can then be easily reduced to the case discussed.

Consider a canal reach AB, into which water is let at A from the higher reach, and from which at B, the same quantity of water is let into a lower reach. If the sluice at A comes into action by a sliding door being turned open, an effect is obtained that is analogous to that in emptying the lock chamber A into reach AB.

The only difference is that in the last case a translation-wave is obtained of which the Q runs up from 0 to a certain value and afterwards decreases again to 0,  while a Q is now obtained which runs up from 0 to a certain maximum and thereafter stays near enough constant as long as the sluice door at A remains open.



The height of the translation wave, its speed of propagation and the slope of the front can be calculated with the formulae mentioned under d.

If just as much water is let out at B as is let in at A, a permanent condition of flow will establish itself. This condition is not uniform because bottom and water-surface do not run parallel now.

To find the course of the surface corresponding to this permanent condition, we can go to work as follows.

The general equation of motion is:

$$\frac{\partial v}{\partial t} + v\,\frac{\partial v}{\partial s} = -\,g\,.\,\frac{\partial h}{\partial s} - g\,.\,I - g\,\frac{v/v/}{C^2\,R}$$

In this $\frac{\partial v}{\partial t} = 0$ since movement is steady

$I = 0$ since bottom is horizontal

$R \quad h$ = average canal depth.

We now obtain:

$$v \frac{\partial v}{\partial s} = - g \frac{\partial h}{\partial s} - g \frac{v^2}{C^2 h}$$

or $$\frac{d}{ds} \left(\frac{v^2}{2g}\right) g - \frac{\partial h}{\partial s} - \frac{v^2}{C^2 h} \quad (\bar{v} = \frac{g}{h})$$

$$\frac{d}{ds} \left(\frac{q^2}{2gh^2}\right) = - \frac{\partial h}{\partial s} - \frac{q^2}{C^2 h^3}$$

$$\frac{q^2}{2g} \cdot - 2 h^{-3} \cdot \frac{dh}{ds} = - \frac{dh}{ds} - \frac{q^2}{C^2 h^3}$$

$$\frac{q^2}{gh^3} \cdot \frac{dh}{ds} = \frac{dh}{ds} + \frac{q^2}{C^2 h^3}$$

$$\frac{dh}{ds} = \frac{\frac{q^2}{C^2 h^3}}{\frac{q^2}{gh^3} - 1} = \frac{\frac{q^2}{C^2}}{\frac{q^2}{g} - h^3}$$

Now the order of magnitude of $\frac{q^2}{g} \approx \frac{0,8^2}{9,8} \approx 0,07$

and that of $h^3 \approx 45$, so that we may write:

$$\frac{dh}{ds} = - \frac{\frac{q^2}{C^2}}{h^3}$$

or $$h^3 \cdot \frac{dh}{ds} = - \frac{q^2}{C^2} .$$

$$\frac{d}{ds} \left(\tfrac{1}{4} h^4\right) = - \frac{q^2}{C^2}$$

$$d \left(\tfrac{1}{4} h^4\right) = - \frac{q^2}{C^2} ds$$

Integrated this gives:

$$\tfrac{1}{4} h^4 = - \frac{q^2}{C^2} \cdot S + \text{constant.}$$

The constant must be taken from a boundary condition. We find this at sluice B. [illegible].

If we assume that the lock-keeper there manipulates his volve that a waterlevel h_o is maintained there, then for $s = L$, $h = h_o$ (L = length of reach AB).

This gives

$$\tfrac{1}{4}\, h_o = -\frac{q^2}{C^2} \cdot L + C$$

or $C = \tfrac{1}{4}\, h_o^{\;4} + \dfrac{q^2}{C^2} \cdot L$

Thus: $\tfrac{1}{4}\, h^4 = \dfrac{q^2}{C^2} \cdot s + \tfrac{1}{4}\, h_o^{\;4} + \dfrac{q^2}{C^2} \cdot L$

$$\boxed{\tfrac{1}{4}\,(h^4 - h_o^{\;4}) = \frac{q^2}{C^2}\,(L - s)}$$

h appears, as expected, to be a function of s and can be represented by a curve of the 4th degree.

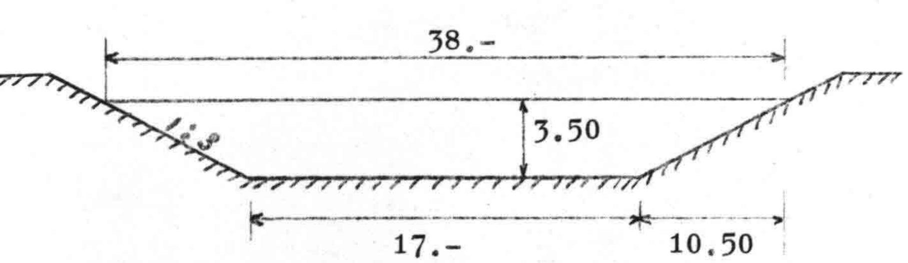
If it is required to know the highest water level in the reach at A, 0 is filled in in place of S:

$$\tfrac{1}{4}\,(h_A^4 - h_o^{\;4}) = \frac{q^2}{C^2} \cdot L$$

or $h_A^{\;4} - h_o^{\;4} = \dfrac{4\,q^2}{C^2} \cdot L$ $\qquad q = \dfrac{Q}{B}$ = average discharge per unit width.

For a canal, the coefficient C lies always in the region of 45.

As example a certain concrete case should be investigated for what water level differences we get. As cross-section, we assume that for a canal for vessels with a maximum load-capacity of 550 ton.



For this $F = 96{,}2\ m^2$.

$R = \dfrac{96{,}2}{39{,}16} = 2{,}54$ m.

If the discharge $Q = 30\ m^3$./sec. then $q = \dfrac{30}{38} = \pm\ 0{,}8\ m^2$./sec.

For C we take 45 $m^{\frac{1}{2}}$./sec. and for h_o 3,60 m.

Suppose that reach AB has a length L of 10 km. then we find h_A = 3,67 m.
Thus difference in height between A and B = 0,07 m.
The maximum velocity of flow at B, where the water surface is lowest,
i.e. $v = \frac{0,8}{3,6} \approx 0,22$ m./sec.

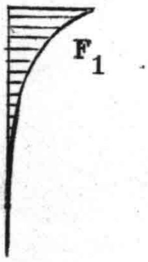
For a canal reach with a length L = 30 km., a level difference is found between A and B of 0,19 m. and for the reach with L = 60 km. a level difference between A and B of 0,36 m.

For long canal reaches, these differences in level become significant.

f. <u>Flow and changes in level due to storm surges.</u>

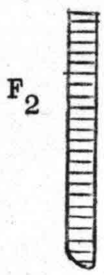
If wind blows over a free water surface, this will take along the upper water-particles with it by friction (which is proportional to the square of the wind speed). By internal friction between the water-particles, those under the surface will be taken along too.

There arises as a result of the wind, a velocity distribution in the vertical which appears as follows.



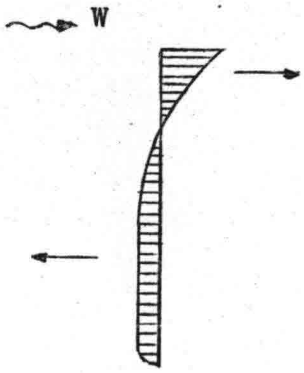
Thus, water flows in the direction of the wind. However, if there is a barrier in this direction (eg. lock at the end of a canal reach) then the water wells up and a gradient arises which makes the water flow back.

For this, the velocity distribution appears as follows:



an equilibrium condition will arise if just as much water flows to the left as to the right.
In other words, if the areas F_1 and F_2 are equal.

The result of both superposed upon each other is:



From various considerations, it follows that the slope of the surge is proportional to W^2 and inversely proportional to the depth.

The formula for the level difference z that as a result of surge between two points at distance 1 can arise appears also then:

$$\boxed{z = \alpha \cdot \frac{W^2}{d} \cdot 1 \cdot \cos \varphi}$$

Here φ is the angle the wind direction makes with the canal axis and α = factor of proportionality.

For broad canals α lies at a value of approximately $0{,}2 \cdot 10^{-6}$. For the wind-velocity W, the greatest value need not to be taken, since the calculated gradient can only set in after the course of several hours, and the average value during that time may be taken.

With this formula we arrive at a surface gradient of approximately $2 \cdot 10^{-5}$ (2 cm per km), if we take a windspeed W = 20 m/sec. and the direction of the wind precisely in the (straight) canal axis.

According to communications at the International Navigation Congress at Brussels, a maximum gradient due to storm surging of $1 \cdot 10^{-5}$ has been measured. This is considerably less than can be found according to the formula. It should, however, be remembered here that in calculation both the max. wind speed and the most unfavourable wind direction have been considered, while in practice these two conditions will only coincide very exceptionally, and will be measured even more exceptionally.

It may be assumed that with an average speed of 20 m/sec., a surface-flow directed with the wind of max. 0,20 m/sec. and an underflow directed against the wind of max. 0,10 m/sec. will occur.

If it should happen that, when the water is standing at its maximum gradient, the wind suddenly completely drop or changes, the water is going to flow back again and a damped oscillation about the equilibrium condition will occur. Here too, no velocities of flow are to be expected greater than about 0,20 m/sec. and then still only very temporary.

As a rule the alterations in level and the flow due to storm surges, do not play a big rôle in canals, since the effect remains small as soon as the direction of the canal changes, which will almost always be the case. There are only a few cases known in which definite measures have had to be taken in connection with surging.

These measures then consist of the closing of a lock which normally stands open. Hereby the length of a long canal reach is devided into smaller parts.

g. Waves due to wind.

In respect to these waves, the same really holds as for surging.

Only seldom will wind-waves in a canal be able to grow to a great height owing to the limited fetch of the wind. Waves with a height of 0,30 m. are already exceptionally high. Moreover, these will run in the direction of the canal axis and can only on the side slopes do any damage. These slopes must, however, be protected in connection with the higher ship waves, so that in designing a canal the influence of wind-waves almost never need to be taken into account.

h. <u>Ground-water flow.</u>

Ground-water flow from or to the canal will occur if a head-difference exists between the water level in the canal and the piezometric level of the ground-water in the adjoining soils at some distance from the canal.

This difference in heifht can be a normal condition.

The undisturbed water level is at the same height in the whole canal reach, which cannot be said of the ground-water level over the whole course, especially where the canal runs through more or less undulating land. The difference in height can also arise through the water level not being constant in connection with its water-conservancy, but showing oscillation: and at the same time if the ground-water level is subject to changes which are connected with weather-conditions or season.

Finally, the water-level changes due to the waves, which is a very swiftly-passing more or less periodic process.

Groundwater currents present a danger for the walls of the canal as soon as they have become so strong that they carry along soil particles with them, which is naturally only possible if the flow is directed toward the canal.

If the outflow of the ground-water in the canal is obstructed or hindered by constructing the walls of the canal more or less water-tight, it must be reckoned that behind the construction an excess water pressure arises which can raise the construction or bring in danger the stability of the whole construction.

If the soil has a homogeneous composition a judgement can be made about the course of ground-water flow by drawing an orthogonal net-work.

For this, there should be considered:

<u>a</u>. If the ground above the phreatic surface (ground-water-table) is filled with capillary water or not and thus whether or not it assists the seepage.

<u>b</u>. If the place of exit drawn of the water above the canal waterlevel corresponds with the ground-water-table, which can be derived from the net drawn.

<u>c</u>. If equipotential lines or stream lines lie through the boundary conditions. An edge-stream line is for example the boundary of a water-tight layer or the profile-line of a dike filled with capillary water. An edge-equipotential-line can be a slope situated beneath the canal water level and the bottom.

From the orthogonal network which satisfies the given boundary conditions, the direction of flow is known at every point, and the speed of this can be calculated if the permeability (k-value) of the soil is known. (This should be determined by the laboratory, preferably by testing on the site).

If the soil consists of homogeneous layers with various k-values, or if the k-value is greater in the horizontal direction than in the vertical direction, an orthogonal network can still, even if with much more difficulty, be drawn.

If the soil is arbitrarily non-homogeneous, it is difficult to make a prediction of the course of the flow, and safety must be introduced by assuming an unfavourable course of the flow.

In general, the drawing of an orthogonal network should be given preference over calculation of the flow, since hereby homogeneous soil must be able to be assumed at the same time, and moreover a fairly strong schematisation must be brought in (slopes vertical). This schematisation must be brought in because the boundary lines usually are not lines which can be cast in a simple manner in a mathematical way. The flow which is in force at some distance from the boundary condition, is calculated accurately, yet not with the flow which must be considered at the exit from a talus, whilst this is just so important for our aim. It is also possible with the help of an orthogonal net to see well how the speed of the outward flow can be influenced by altering the boundary conditions (e.g. giving a different slope to a canal talus or making the covering of them water-tight over a portion).

Finally, in the choice of the construction of the free-board one must take into account also what consequences the ground-water flow can have on the stability of the slopes if the canal reach could run empty by an accident.

From the foregoing does follow how important it is for the canal designer to acquaint himself accurately with the geological and hydrological conditions of the soil through which he wants to cut.

i. Combinations of the phenomena described under a to h.

It is of importance to the canal designer to know in how far the various phenomena discussed can re-inforce one another.

We will discuss this as far as velocity of flow, changes in water level and waves are concerned.

The velocity of the flow resulting from translatory waves and surging is only slight. That resulting from the transit of vessels is far-away the most important.

A vessel which moves against the current for example as a result of a translatory wave, will locally reinforce the current already present with its own return-flow.

However, for this case, the natural limiting speed (and thus also the cruising-speed) lies just as much below that in stagnant water as the velocity of flow which opposes the vessel.

If. for instance, the own current in the canal is 0,20 m/sec., and the maximum admissable current is 0,80 m/sec. then the ship may not cause a return flow of more than 0,60 m/sec.

As we know the value of $\frac{f}{F}$, and also the value of the term $\frac{u}{\sqrt{gh}}$, the ordinate $\frac{V}{\sqrt{gh}}$ (and V) can be found from the diagram.

If the value of V is lower than 90% of $V_{lim.}$, a restriction in the form of a speed limit must be ordered when an "own" current in the canal occurs. The coincidence of all other flows not directly dependent on the movement of the vessel is never decisive in design. For the calculation of maximum possible velocity of flow, that velocity can be taken which occurs when a vessel travels at the natural limiting velocity in a canal which itself has no flow.

Concerning the <u>variations in level</u> the case is somewhat different. The reason why the variations in level interest us is two-fold.

In the <u>first place</u> one must, on the one hand for the fixing of the bottom depth and on the other hand for determining the headroom of bridges, take into account the lowest and highest level of water respectively in which the vessels travel.

In the <u>second place</u>, the wave attack, which will be discussed later, takes place in a zone which is strictly dependent on the waterlevel.

<u>It is clear that in fixing the bottom depth:</u>

<u>1st</u> the vessel with the maximum speed is allowed to cruise so that the maximum sinking-in is reached, (vessel loaded),

<u>2nd</u> the negative translatory wave is taken into account which results from locking at the end of a canal reach, which gives a lowering of the water surface which is also dependent on the distance from the lock,

<u>3rd</u> the possible surface lowering by surging is brought into calculation.

<u>4th</u> any possible lowering due to drainage is involved in the considerations.

The coincidence of 1st and 2nd occurs very often. The frequence is dependent on the intensity of navigation. 3rd can, however, occur with a much lower frequency. That 4th should simultaneously occur with 1st, 2nd and 3rd is still possible. The frequency of the most unfavourable case thinkable is so small that then the play between underside of vessel and canal bottom can be taken very small, thereby accepting some scour as a result of screw-action.

If it is a question of fixing the height of the underside of the bridges, then

1. the vessel will be allowed to travel very slowly so that almost no surface-drop occurs (vessel unloaded)
2. a positive translatory wave resulting from locking at the end of the canal reach will be taken into consideration
3. care will be taken with the rise in water-level due to surging and
4. ditto due to drainage.

If all the factors are reckoned as unfavourable as possible in this way, the play between underside of bridge and upper side of vessel can be kept quite small.

Between the lowest and highest possible water-level calculated in this way, the water-levels occur among which the average level has the greatest frequency. The frequency of the water-level decreases according as it deviates more from the average level.

A qualitative insight into wave formation due to shipping may be obtained just as was dealt with before. A positive pronouncement is possible as far as velocity of the propagation and the period of these waves are concerned. We know that the wave height is dependent on the form of the vessel and the speed of navigation, while a measure for the wave height was taken from empirical data.

An impression has also been gained of the direction of propagation. This appeared also to depend on the speed of navigation.

These ship waves can interfere with wind waves, because they can only develop to some extent if the wind direction coincides with the direction of the canal and then the waves send no energy in the direction of the banks. It appears to be wise, considering the uncertainty in ascertaining the most unfavourable wave, to bring in a sufficiently great safety-margin.

For the ship-waves we did this already by reckoning, for the average level, the water-level lowered by the whole surface-drop. This was when we were concerned with the wave most unfavourable for erosion of the bottom and low parts of the bank.

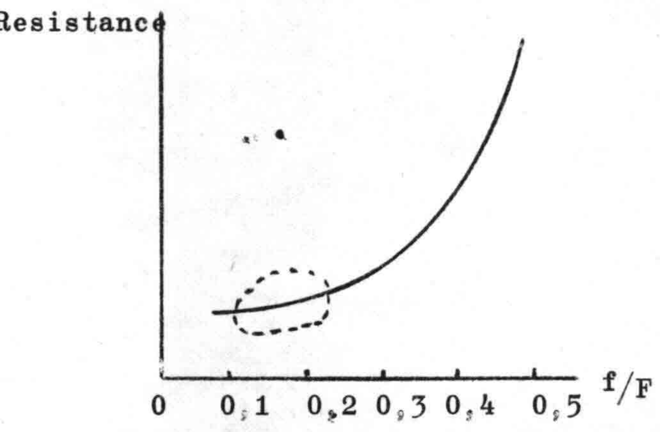
8. Resistance of Vessels. Dependence on the canal profile.

In order to arrive at an economical means of transport, the resistance which the vessels experience in sailing through a canal may not improportionately large.

The assumption that the resistance which the ship experiences in transit is dependent on the size of the canal profile is obvious.

Wit the help of model tests and by measurements in practice, this dependence has been shown. Both the width and the depth of a canal are decisive for the resistance which the vessel experiences in transit.

The relation which exists between this resistance and the ratio $\frac{f}{F}$ is approximately as follows:



The practical value lies in the region of $\frac{f}{F}$ = 0,2 - 0,15. Enlarging the canal profile gives then hardly any advantage, while the resistance rises very quickly for values of $\frac{f}{F}$ becoming greater. This result, which is gained if the problem is approached from the side of the skipper, is in nice agreement with the result from the considerations of the canal designer and administrator.

Now there is, especially with large canals, a very great difference between the <u>biggest</u> vessel which the canal can carry and the <u>most frequently occuring</u> vessel. With what f to reckon, is more closely discussed under point <u>12</u>. (Cross-section and bank protection II).

INTERNATIONAL COURSE IN HYDRAULIC ENGINEERING.

Navigation Canals

by Mr. A. Zanen.

The first step for the designer of the cross-section, and especially of the revetment of the banks, of a canal is to compute the value of currents and waves, and of the alternations of the waterlevel under different circumstances.

The second step is to know the nature of flow, waves and groundwater-flow

The third step is to design the construction such that no damage, or damage to a certain limit, will occur by the force of the phenomena mentioned.

Second step.

9. The nature of the flow, waves and ground-water-flow.

A. Flow.

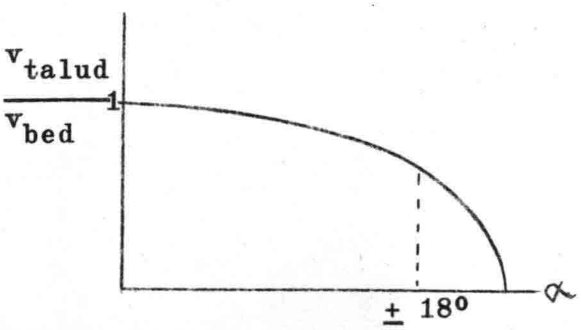
As a result of flow, erosion can occur if the strength of the current is big enough to displace soil particles.

In general, the current which can occur in a canal, will be too small for this. An exception to this is formed by the return flow which occurs around a moving vessel. It is possible for this to reach the order of 0,40-1,00 m/sec. This current will work now in one direction, then in the opposite direction. Moreover, it occurs locally during only a very short time.

Erosion of the bottom of the canal need not be feared, although there will probably be some motion of the particles.

This is different with the slope, because there gravity acts on the soil particles in addition. A derivation of the ratio between $v_{talud\ cr.}$ and $v_{bed\ cr.}$ is given in the lectures "Revetments".

For various values of α, the ratio $\frac{v_{talud}}{v_{bed}}$ is plotted (v = critical velocity), we get: (see figure).



It seems desirable so to choose that the conservation of the tali is not too decisive in the allowable velocity of flow. From the graph it appears that making more gentle than 18° gives little advantage, but that for greater values of α the critical velocity of flow for the tali descends sharply.

Naturally we have been thinking here of a granual material. In many cases there will be clay- or alluvial component, and as a result, cohesion, for which the above consideration no longer holds. Yet this points in a certain direction, and since 18^{0} almost corresponds to a slope of 1 in 3, (a slope which has also been arrived at from practical considerations) the result is not unsatisfactory.

B. Waves.

As we saw, translatory (or long) waves manifest themselves in a relatively weak current and in differences in level. They can of themselves bring about little damage.

This is different with the so-called "short waves", that is the waves caused by shipping and wind. The mechanism of the short wave is dealt with by Ir. Mostertman in the lecture "Short Waves" and also in short in the lectures "Revetments".

We saw the bed disturbs the original orbital motion, and in greater measure according as the water "shallower" is.

For a wave which comes in gradually shallower water, the velocity of propagation and the form are more and more influenced by the bed. This disturbing action of the bed culminates in the breaking (turning over) of the wave-head. This happens if at a given moment the velocity of the particles in the uppermost orbital circles has become greater than the continually decreasing velocity of propagation of the wave. The particles then fly as it were, out of the wave is characterised by a heavy turbulence by means of which the wave gives up a great portion of its energy. It is this liberated energy which threatens the wall of a canal.

In general, a wave breaks where the depth reaches less than 1,5 x wave height.

It is necessary to protect against erosion the talus of the region which is situated under the breaking wave up to the highest point which waves can reach.

In designing the bank defences, a decision must be taken on principle A construction can be made with a vertical wall up to such a depth that the wave does not break and to such a height that the wave is reflected, or a construction can be made whereby the talus runs down to the water line, whereby the waves break and a talus defence has to be brought in which must prevent occurence of erosion as a result of any turbulence caused.

In the first case, the water after the vessel will remain restless for a long time, while in the second case the wave motion is quickly damped out.

Now in a canal, little disadvantage or inconvenience is in general found from wave movement, so that the choise is mainly determined by <u>cost-price considerations.</u>

<u>Remark:</u> At first sight, making the canal broader should imply lower waves, since then the energy which is pumped into the water by the screw is spread over a greater area. However, by broadening, the possibility of faster navigation is created, so that the favourable effect of the broadening on the wave height is lost again.

C. <u>Ground-water Flow.</u>

Due to the ground water which flows from the surroundings to the canal, the stability of the sand grains which lie at the surface, eg. in the side-slopes, can be brought into danger.

In considering the equilibrium of a soil particle in flowing ground water, it is convenient to bring in the concept of "flow-pressure". This is not, as the name might perhaps suggest, a dynamic pressure, but a static.

This flow-pressure acts in the direction of the flow and amounts to $\gamma_w . i$ on an element of unit volume in which i is numerically equal to the pressure difference between the front and rear side of this. This pressure difference can be derived from the orthogonal net. Of course the pressure gradient is the greatest where the velocity of flow is the greatest, thus where the orthogonal elements (squares) are the smallest.

Now the direction of flow for the case when the water runs freely out of a side-slope is very different from the case when the water runs out of a side-slope under a free water surface, in the latter case the slope being an equipotential surface.

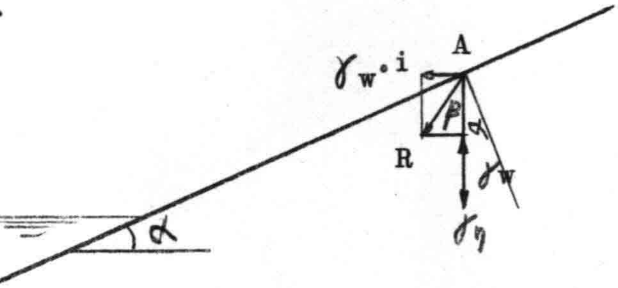
In the first case, the direction of flow for a relatively gentle slope may be taken as horizontal as approximation, in the second case the outflow takes place perpendicularly to the slope. We now go into what are the equilibrium conditions for both cases.

<u>1st Case.</u>

About the same force will act on every unit of soil at the surface of the slope.

If each of the units is in equilibrium, than no disturbance of the equilibrium has to be feared.

So it is sufficient to take into consideration the equilibrium of one unit, and leaving out of account the pressure of the surrounding units on the considered one.



On an element of unit volume in point A act:

γ_n = density of the wet soil, vertically downwards

γ_w = upward pressure of the displaced water, vertically upwards

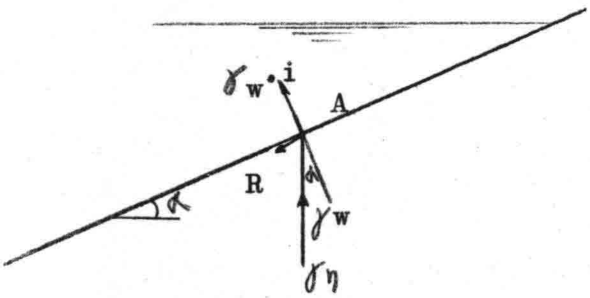
$\gamma_w \cdot i$ = horizontally-directed flow-pressure.

The resultant of these forces is R if $\gamma_n = 2$ and $\gamma_w = 1$,

then it appears from the figure that $\tan \alpha = \tan \beta$ and thus $\alpha = \beta$

R thus makes an angle $\alpha + \beta = 2\alpha$ with the normal to the slope A condition of equilibrium is that this angle must be smaller than the angle of internal friction φ of the material, thus must $2\alpha < \varphi$ or $\alpha < \frac{1}{2}\varphi$. If therefore the water leaves the slope freely horizontally, and if the angle of internal friction is for example 30°, then a slope with a slope of 15° is still just in equilibrium. This slope is very gentle ic. 1:3,7.

<u>2nd Case.</u>



In the figure are given again the forces which act on the element of unit volume in A. The resultant R may make, in connection with the equilibrium, an angle of φ at the highest with the normal to the slope. The components of R perpendicular to and parallel to the slope respectively amount to:

$$(\gamma_n - \gamma_w)\cos\alpha - \gamma_w \cdot i$$

and $(\gamma_n - \gamma_w)\sin\alpha$

The equilibrium condition is thus

$$\tan\varphi \geq \frac{(\gamma_n - \gamma_w)\sin\alpha}{(\gamma_n - \gamma_w)\cos\alpha - \gamma_w \cdot i}$$ or is $\gamma_n = 2$ and $\gamma_w = 1$

$$\tan\varphi \geq \frac{\sin\alpha}{\cos\alpha - i}$$

If the i has been calculated from the orthogonal net, then angle α can be expressed in angle φ.

In the numerical calculation for a certain flow pattern, it appears that in this case the angle α, at which the grains are still just in equilibrium, differs less from φ than in the 1st case.

It is thus obvious that one must always attemt to prevent free escape of water for example by bringing in suitable draining, with which the phreatic surface (water table) can be beneficially influenced.

The most important conclusions from the foregoing are:

1. Try to prevent the water leaving a slope freely.
2. The angle under which a slope is stable does have some connections with the angle of internal friction ϕ , yet may not as soon as there is flowing or stationary ground water present, be equated with this angle.
3. The course of a ground water current can be influenced by placing certain boundary conditions.

If. due to flow in the canal, there is at the same time a force exerted on the grains in the longitudinal direction of the canal, this has to be taken into consideration (lectures Revetments).

Navigation Canals

by Mr. A. Zanen.

10. Other factors which can be the cause of damage to the boundaries of the canal profile.

Direct damage by frost can arise if water is situated somewhere in the construction, which has no opportunity for expansion. The damage then consists of breakage by freezing or working apart of the construction. By bringing in dilatation joints or other forms of construction, this damage can be prevented.

Indirect damage du to frost can be caused by ice that, as a result of flow or shipping along the canal walls and exerts on these an impact and grazing action, which will be the heavier, the more irregularities and discontinuities the wall contains.

Also the ice can bind itself to the wall which can lead to damage if for any reason the water-level undergoes a change. If icing-up is of much concern, the shape of both the construction and the situation can be influenced by this.

Rain or run-off water can cause erosion of slopes above water.

The danger of this is greater according as the slopes become steeper and higher, and according as the bond between the soil particles less good is. In so far as these slopes are not defended against washing by waves, it is desirable to meet the danger of erosion with a sturdy grass cover. The condition for this is a maximum slope of 1 : 2 and a sufficiently thick layer of growing-soil.

The natural activities of animals such as moles, rats etc. which are to be seen in holes and burrows, give local damage, which, however, often can lead to more extensive damage.

This holds too for local damage, which by malevolence or lack of consideration, can be brought about by persons (anglers).

Through the detailing of the construction, the infliction of damage can be hindered. For example an adhesive overcover for a stone revetment will hinder the loosening of stones.

Further, the concern is that the authorities, by careful oversight will take care that damage from persons will be exceptional and that damage from animals is limited in extent.

1. Defence of the bed and walls of the canal.

In designing a canal, one has the choice between a profile which is so roomy that defence of bed and walls against erosion due to flow is not necessary, or a narrow profile with bed and walls protected against erosion.

If the canal is only intended for navigation, then the choice between these two systems will be fixed on economic grounds.

Where the ground has a high value and the material for covering the bed and walls is to be had for a reasonable price, the narrow defended profile will doubtless be chosen. On the other hand in a little-developed district, where the value of the ground is low, the choice of the broad undefended profile would quickly be made. However, it does not usually lie so clearly and comparative estimates of costs will have to be set up in order to reach an informed choice.

For navigation canals a certain minimum profile is necessary for a safe navigation, and also to limit the resistance of travel.

If it appears from a calculation, that in this profile flow velocities can still occur which will take bed material along in significant measure, then a relatively small extension of that minimum profile can reduce the maximum velocities below the highest permissible. If a canal serves for drainage besides navigation, the matter lies once more less clear.

For a small drainage discharge, the profile will, in general, be designed somewhat broader.

However, if the drainage discharge is large (which naturally is disadvantageous for shipping), one can consider discharge this through a separate canal. It are again economic considerations which determine the choice.

After the foregoing, it will be clear that the profile of a navigation canal will in general be designed so broad that protection of the walls against erosion due to flow is not necessary.

Another question is that the banks need a local protection to avoid erosion from the waves (short).

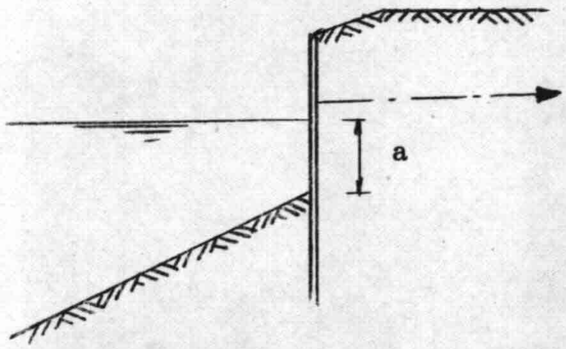
Under 9 B, it has already been said that one is here confronted with a decision on principle, ic. if the waves are allowed to break or allowed to reflect.

From this it follows that the canal banks in any case must be provided locally with a protective construction. Also it is clear that the region to be protected becomes greater, the more the water-level varies.

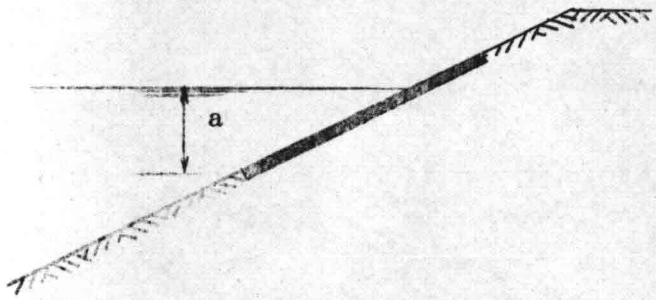
Because of this too, the alterations in level dealt with earlier are of great importance.

We make distinction between three basic types of bank defence.

1. The <u>first type</u> is that in which turbulence is prevented by seeing that a sufficient depth lies by the bank. The depth <u>a</u> must therefore, at the lowest water-level, reacht 1,5 H. In order to prevent such a wall from being pushed over, this is either braced or anchored throughout. At the same time, the material of the wall above water must be weather resistant. Likewise must the wall be soil-proof and remain so, to prevent hollowing out behind the wall and making the canal shallow.



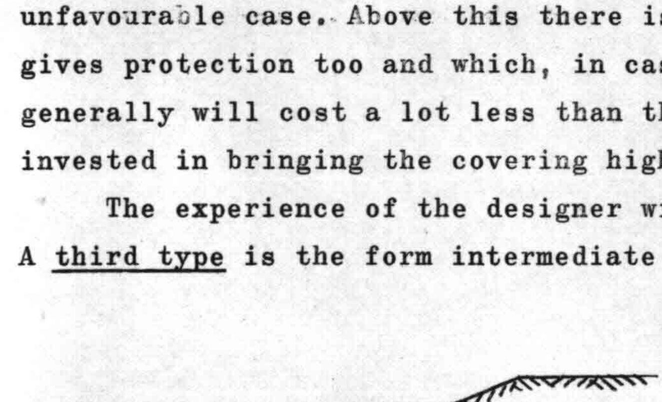
2. The <u>second type</u> is that in which the turbulence caused by breaking waves is simply accepted and the slope so covered that the soil particles under this are fixed. The lower edge of the covering must reach down to <u>at least</u> the forementioned depth <u>a</u> below the lowest possible water-level. Further underneath, the water particles have still a velocity as a result of the orbital motion. These are calculable with the formulae (lecture Short Waves), from which it can be concluded whether the covering must be brought still further down.



How far up the covering must reach is naturally bound up with the highest possible water-level and with the wave height. It is not necessary to bring the covering up so high that no water can come above this in the most unfavourable case. Above this there is anyway a grass cover present, which gives protection too and which, in case of damage, requires maintenance that generally will cost a lot less than the interest would reach on the capital invested in bringing the covering higher.

The experience of the designer will have something to say here.

3. A <u>third type</u> is the form intermediate between the two types mentioned. Hereby is it possible to apply to a material for the vertical wall that only <u>under water</u> need be weather proof. From the point of view of stability, the soil tightness and the depth in front of the wall, the same conditions hold as for the first type. The waves will in this type be partially reflected and partially give their energy in turbulence. This therefore again poses requirements on the

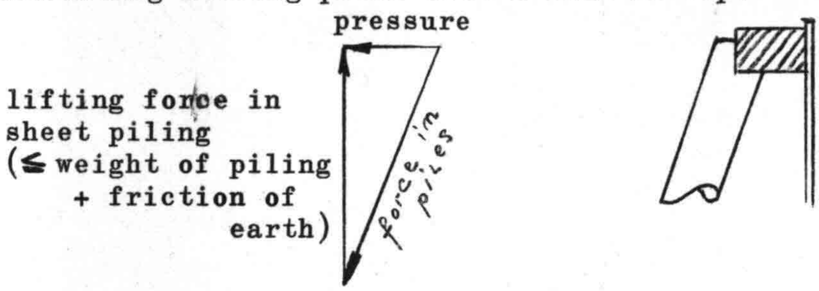


protection of the bank above the vertical wall.

We saw there are 3 types of construction of the bank protection of a canal. Now we will consider the materials to be used for the various constructions, and also with which special care must be taken in design.

As mentioned already the vertical wall of type 1 must hold against a fairly big soil pressure, and will in general be constructed of steel or reinforced concrete sheet-piles. A wooden construction does not come into consideration, in connection with its smaller weather-resistance.

The measures which are necessary to bring about the stability of the wall form a disadvantage to this simple and generally handsome construction. Formerly in general the stability of this kind of constructions was obtained by constructing shoring piles with a beam on top.



But formerly usually there was no depth before the wall. Now, in relation with the depth a according to the new insights, vessels can reach the wall of the canal and a construction of shoring piles will irrevocably be broken by collision with vessels. Moreover these shoring piles form extremely valuerable components of the construction in periods of icing-up and they also cause turbulence in the water. One is thus thrown back onto an anchoring-construction. This consists usually out of a waling in front of or behind the sheet pile wall, steel anchor rods with (handscrews?) and anchor-plates or a continuous anchor wall. This construction is naturally fairly dear in material and requires, especially with high banks, a considerable soil replacement.

It is true that the anchors are sometimes driven in by use of a horizontal setting (in clay and peat) or sprayed in (in sand), yet before the fixing of the anchors in the anchorings construction, an excavation must in every case take place.

As mentioned before stability can also be reached by designing the wall so long that a clamping action in the ground is obtained. This requires also thicker piles, naturally, since the moments occuring in this case become considerably bigger than in an anchored wall.

The length of the clamped-in wall can be somewhat limited by not driving these in vertically, but leaning outwards a little. If this wall is provided moreover with a weighted concrete cap, then the component of the weight of the cap exerts, perpendicular to the wall, a force which acts in the opposite smaller and the construction somewhat lighter. On the other hand, it will always have to be said, however, that driving a pile wall at an angle is more troublesome (and thus more expensive) and also that the weighted concrete cap costs something. It is thus again a question of cost-estimation in coming to a choice between the two systems.

Finally there is the requirement of <u>soil-tightness</u> of the wall, already mentioned before.

A steel sheet wall satisfies the condition of soil-tightness. A wall of ordinary reinforced concrete sheet piles certainly does not satisfy this. So with this, extra measures should be taken. These can consist of the application of special systems of concrete wall which are soil-proof, or the transport of fine soil particles can be hindered by means of a drain with rubble or gravel behind the wall. Such a drain is, however, difficult to construct with sufficient exactitude in the wet.

For the vertical wall of type III which is as a rule constructed of wood, the conditions stated for stability and soil-tightness likewise hold. The shored construction applied fairly often formerly can also here, in connection with the great depth before the wall, not be applied. The support-length of this wall is, however, smaller and if an anchoring becomes expensive owing to local circumstances, a more obvious choice is clamping-in at the bottom, with or without some slant on the wall outwards.

The requirement of soil-tightness demands special provisions here also.

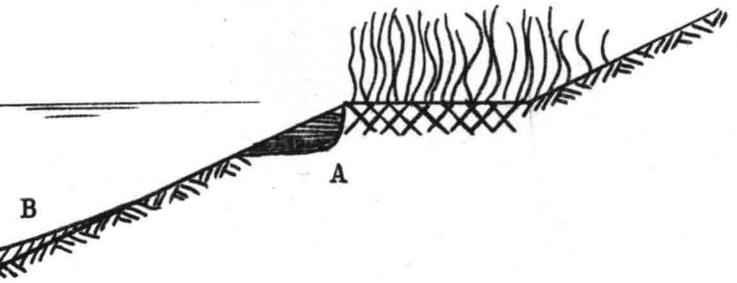
It is true that a new wooden sheet pile wall of which the planks engage in a tongue and groove, together with swelling of the wood, has a fairly high measure of soil-tightness, but it has appeared that, after a time, due to wave action and to permanent or periodic flow through of ground water, the wood is subject to mechanical wear from which, over the upper portion of the planks, fairly broad cracks arise. Fixing battens from behind against the seams of the wall piles has appeared to be a worthless measure.

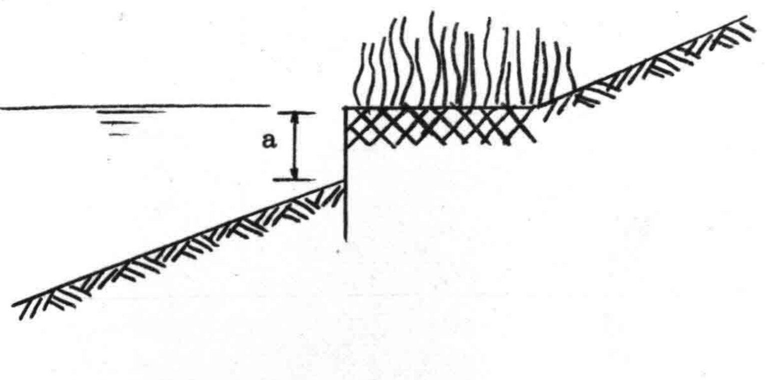
In the Twenthe-canals, favourable experience has been had with nailing eternite plates against the wall from behind in order to make the wall watertight over the dangerous height.

It is true that, also from this, the pattern of ground water flow is somewhat altered, although <u>under</u> the eternite plates the wall is not watertight so that the water pressure behind a wall so fitted cannot reach as high as against the special systems of concrete wall mentioned which are watertight over the whole height.

A smaller loading comes on the vertical wall with type III if behind this a <u>rush-berm</u> is made. Here the rushes themselves have the function of so smothering the waves that the slope rising behind needs no defends. Further, the roots of the rushes must hold the soil together. On the canal-side the rush-berm is bounded by a protecting construction, which, together with the root system must form the vertical wall. In a canal with not too big a wave-attack on the banks, the depth in front of the wall need not be deeper than the depth to which the system of roots of the rushes develops. In that case the protecting construction has only a supporting function and can, for example, consist of one or two heavy continuous rows of faggotting. However, it should here be kept in mind that the system of roots is not there directly, but must have opportunity to develop. During this time there is thus, besides the supporting function also a soil-retaining function. If the depth has to be greater than that to which the roots can hold the soil fast, in connection with the expected wave attack, then the vertical wall has also a remaining function and a sheet wall construction will soon be chosen, either made water-tight or not. The active earth pressure on this wall from behind will be extremely small due to the action of the rush roots holding the soil fast, so that this wall can be constructed without anchoring.

Before we proceed to discuss slope defence, two remarks should be made here. The first concerns especially the rush-berm. As minimum breadth a measure of 1 m. was given in the report for the Navigation Congress at Brussels. According to examinations in the Twenthe-canals a measure of 3 m. is chosen. The breadth which is chosen will depend on the question how the growingbed is, thus on what expectations are set upon the quality of the rushes. Now a rush berm can be severely damaged if cattle can come onto it either to eat the rushes or drink water. Thus in bank defence with the help of the rush berm, there ought to be, if the adjacent land is grazed, a fence which makes the rush berm out of reach to the cattle.





The second remark is of a more general nature. Especially formerly it happened fairly often that the slope was allowed to run straight from below to the water surface. At this point, the front edge of the rush berm was laid. When the shipping grew in extent there appeared a berm to have formed under water as a result of the erosion from the turbulence caused by the breaking waves.

The soil which was washed out from A comes to rest in B in the corners of the canal, from where it had to be removed again by dredging. It is obvious to dig out the soil at A already, directly when building so that the maintenance dredging work mentioned is prevented. The following step is the bringing forward of the whole freeboard construction. For the bit of vertical wall, it is true, higher requirements should be set in connection with decreased passive earth pressure, on the other hand, on the land side of the canal landcosts are saved of a fairly considerable order.

The value of the land formed by the choice between the two forms plays a chief rôle.

If with type III no rush berm is applied, the rising slopes must be protected over some distance.

Light defences such as piled sods, turf heaps, rubble dumping etc. will no longer be applied now that effort is being directed to limiting the speed of travel as little as possible, and so strong hydraulic effects must be taken into consideration. The revetment of natural stone, sometimes applied formerly, will not be so readily applied any more because of high costs and investments. It is obvious to apply the material concrete or brick in place of this natural stone.

A covering of the slope with concrete slabs contains the danger of hollowing out behind the covering, since the slabs do not sink as well if the soil particles are sucked away through the toe construction or the seams between the concrete slabs. The consequence is a strengthened water movement under the cover and finally very expensive restoration. In the outer reach of the Wilhelmina Canal it has indeed appeared that these objections are not theoretical. A covering of the slope with an asphalt mixture seems to be destroyed after a time by plants which grow through it. Construction in tight asphalt concrete or in tarmacadamized stone can probably prevent this. It seems still too early to pass a sufficiently founded judgement on these constructions.

A handsome and fairly commonly applied construction is that with the hexagonal concrete blocks, of height 15 or 20 cm. In straight or gradually curved portions of a canal, they can be laid with good closure against one another by less skilled labour than is necessary for a revetment of natural stone.

In order yet further to hinder the (fairly small) danger of soil particles being sucked away between the concrete blocks, the blocks are layed on a mattress pegged to the ground or layer of rushes or on a thin layer of fine sea-shell gravel or cinders.

A provision whereby good results can likewise be obtained is the following which, among other places, has been applied along the banks of the

Merwede Canal in South Holland (Vianen Gorinchem).

This consists of a double layer of bricks laid loose as herringbone work with sometimes a pegged mattress under these layers. By tamping, a flat revetment can easily be obtained if the foundation is of clay or peat. In sand this is not so easy and preference will probably be given to the concrete block covering.

It is clear that the toe joint of every revetment on no account may fall over, since the whole revetment then becomes loose and must be reset afresh. Also on the upper edge a (lighter) closure is desirable (eg. a course of capping or a concrete strip either placed in the work or not), in order to prevent damage as mentioned before.

How the slope defence of type II will look, is very much dependent on the manner in which the canal is dug. If this takes place in the dry, then the whole slope defence can be made as described above for the slope defence above water for type III.

Digging out a canal in the dry is, however, not the rule, because removal of soil in the wet is cheaper in general. Besides this, in sandy soil, excavation can only be done in the dry if the watertable is artificially lowered during construction, which is very costly. Yet sometimes circumstances do occur which lead to excavation in the dry, which appears indeed from the fact both the Maas-Waal Canal and the Twenthe canals have been excavated in the dry.

In clayey soil, the possibility exists, although excavating in the wet, to first dig trenches in which the free-board embankments are made and then later to dredge the canal out further.

However, that may be, in my opinion, a cover of blocks or bricks beneath the canal level is <u>not</u> so fine because, in eventuality that damage arises below water for any reason , repair is very difficult.

If this objection is accepted, a construction is arrived at which, above the canal level, consists of a slope covering as has been described for type III and beneath this of a construction which is to be placed in the wet and thus also to be repaired. There is no great choice here as the application of a hanging mattress is almost compulsory. This is willow mattress composed in the same way as a fascine mattress with 2 layers of filling which can be, as it were, continuously made on the job so that no seams arise. The upper edge of the hanging mattress is as a rule laid on a little berm that lies at such a height that the upper edge of the sloping mattress will always remain wet to prevent rotting of the faggotting. Below, it reaches as far as is necessary to prevent erosion of the slopes by turbulence from the breaking waves or by the normal orbital motion of the water. For such a construction, it is often difficult to make a tight connection between the hanging or slope mattress and the slope cover which lies above these. Because the berm must be kept c. 40 cm. beneath the canal level, the lowest part of the stone revetment must be set in water, which does not promote careful construction and whereby the condition of soil-tightness comes into question. If, during construction, the canal level can be temporarily lowered, this objection naturally disappears.

A logical desire follows from the above i.c. for a covering frome one piece (thus in which the weak point of the connection between two different constructions disappears) which can be placed both in the wet and in the dry and in the objection does not exist which was indicated for a stone-revetment in one piece.

One's thoughts here turn onto a product which is becoming more and more applied in hydraulic constructions, i.e. bituminous product.

Covering the slopes to be defended with an asphalt coat in one piece would fulfill the expressed wish, if the following conditions could be satisfied:

1. Continuous production, whereby seams are avoided, or a good solution for the mutual connection of the pieces.
2. Water and weather resistance. Here we think primarily of the effect of so-called "stripping" of asphalt products (i.e. the loosening of the asphalt skin from the mineral).
3. An acceptable price for such a provision.

A more aesthetic colour has not been here mentioned as a condition because the black colour is inherent in bituminous products and will be therefore difficult to avoid.

As a matter of fact, Shell have developed a method which makes a slope cover possible of which the opinion is that it satisfies the above conditions.

In the year 1954 the first test length was constructed ans also afterwards several kilometers of canal bank with such protection were constructed.

INTERNATIONAL COURSE IN HYDRAULIC ENGINEERING

Navigation Canals

by Mr. A. Zanen.

12. Profile of Cross-Section and Bank Protection II.

With the previously dealt with theory and the considerations mentioned in fixing the profile of cross-section, we must be in a position to design the canal.

At the same time, we can calculate for an already existing canal what hydraulic effects can be expected under certain given circumstances.

Generally speaking the biggest vessels expected must be able to pass. Because these occur but seldom, a speed limit can be imposed on these vessels in order to save on the profile.

On the vessels which normally occur on the canal, one will, for preference, not impose any limits. We consider these "design" vessels as decisive for the cross profile. After having chosen a cross section as a first design, the surfacedrop and returnflow are calculated for the design vessels. The result of this shows whether the cross section tried is satisfactory.

If necessary, the calculation is repeated with a revised cross section, until a satisfactory result is obtained. For the mean speed of the returnflow, will in general no value higher than 0,7 - 1 m/sec. be allowed. The calculated surface-drop is especially important for the design of the bank protection.

It should be mentioned here that in practice higher values for surface-drop are sometimes measured than would follow from the calculation. It seems safe to reckon on a max. surface-drop which is c. 1,3x the calculated.

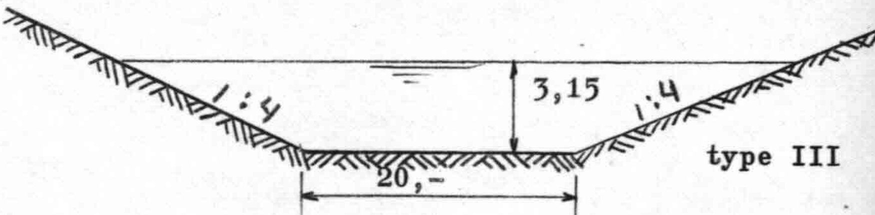
The depth $\underline{a}$ thus becomes $1{,}5 \times 1{,}3\ z + z \approx 3\ z$ beneath the normal water level.

z = surface-drop = wave height; taken with the waterlevel lowered by the surface-drop. To be on the save side sometimes 3,5 z is taken.

In order to illustrate how this material is handled, there follows here a description of the way in which a student, (now a degreed engineer) for the graduation design, set about fixing the speed limits which would possibly have to be imposed on shipping in the given circumstances.

For certain reasons, not here to be further discussed, the canal profile was fixed at the following dimensions:

Width of bottom 20 m.
Depth 3.15 m.
Slope of submerged banks 1:4



type III

Also the construction might not differ from that accepted in that region.

From this, a depth of the bank of 0,65 m (a) was available. This must thus be reckoned with.

For the allowable mean return flow velocity 0,75 m/sec was assumed and for the allowable surface-drop $\frac{0,65}{3,5} = \pm\ 0,19$ m. By the designer the following diagram was then established: see diagram. In the left part are given lines which on the basis of the formulae treated give the relation between the velocity of travel v and the return velocity u, for various increasing values of f/F.

In the right-hand part are given lines which give the relation between velocity of travel v and the depth in front of the bank (a) for various values of f/F. From the drawing it does appear that for intermediate values of f/F, interpolation can be used with sufficient accuracy. In the left hand part of the figure lie the allowable speeds of travel, thus on or on the left of the vertical through u = 0,75 m.

The points of intersection of this line with the series of curves for f/F are brought over to the right hand part of the figure where a straight line is obtained. Here too the allowable speeds of travel lie on or on the left of this line.

The allowable speeds of travel are, however, also determined by the maximum allowable Z in connection with any possible scour of the bank protection. The speeds of travel must thus lie on or on the left of the protection. The speeds of travel must thus lie on or on the left of the vertical a = 0,65 m.

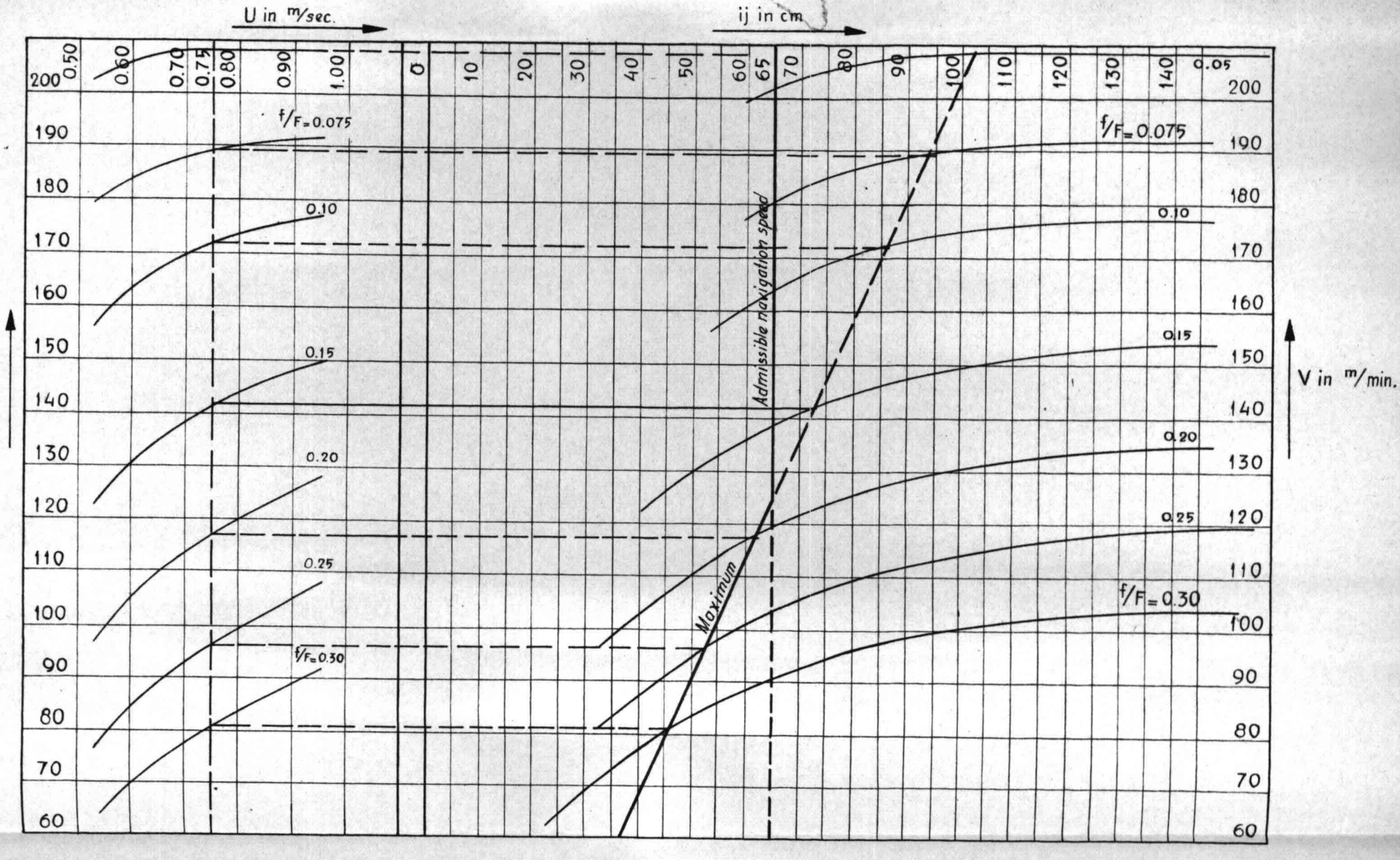
We thus see that sometimes the size of the surfacedrop and then again the size of the return-flow is decisive for the allowable speed of travel.

By means of the diagram, the maximum allowable speed can now be determined for the various vessel types. The result of this is then compared with the calculated possible limiting speed (see table).

In general it appeared that the ration between allowable and highest possible velocity lay between 80 and 95%.

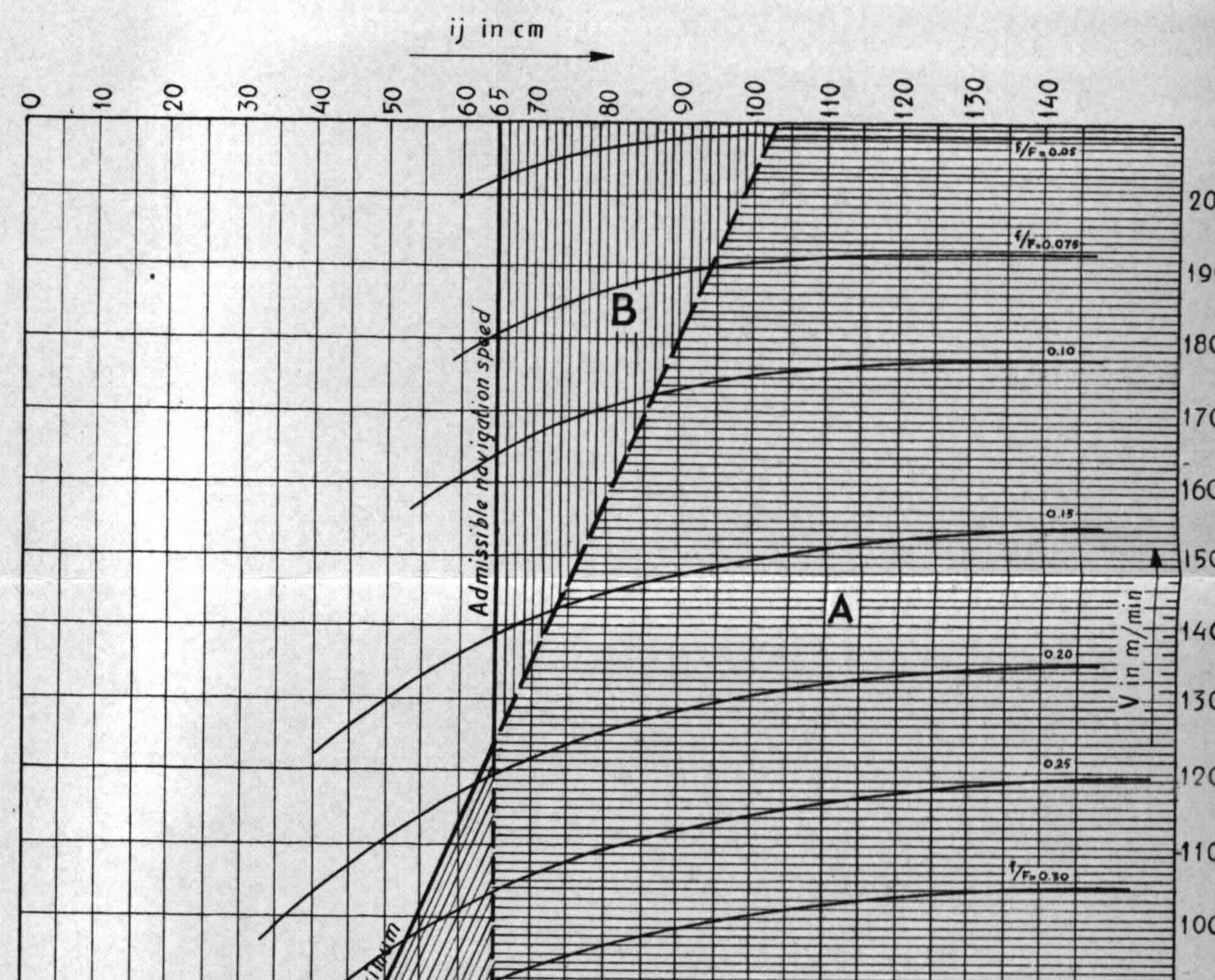
Considering that especially the smaller vessels seldom travel with 90% of the limiting speed, as a result of their motor-capacity a speed-limit for these is superfluous.

For the bigger vessels and the coasters the ratio decreases to 80%. These vessels travel often near to the limiting speed for a well-paying exploitation.



Type of ship		l in m	b in m	d in m	f in m²	f/F	Navigation speed: Natural limiting speed V_ℓ in m/min	Navigation speed: Admissible speed V_a in m/min	V_a/V_ℓ in %
Inland navigation ships									
2000	ton	95	11.50	2.70	30.0	0.295	108	85	79
1350	,,	80	9.50	2.50	23.0	0.225	129^5	110^5	85
900-1000	,,	67	8.20	2.50	20.0	0.195	139^5	122	87^5
400-600	,,	50	6.60	2.50	16.0	0.156	153	136	89
500-600	,,	50	6.60	2.40	15.3	0.149	156	139	89
250	,,	31^5	6.20	2.10	12.7	0.124	166	152	91^5
170	,,	31^5	5.40	2.00	10.5	0.102	178	162	91^5
150	,,	29	5.20	1.80	9.0	0.088	185	171	92^5
80-120	,,	24	5.00	1.60	7.7	0.075	192	181	94
30-65	,,	21	4.20	1.40	5.6	0.054	206	198	96
Coasters									
500	B.R.T.	51	8.30	3.40	26.9	0.263	116	93	80
400	,,	47	7.90	3.10	23.6	0.230	127	106	84
300	,,	41	7.20	2.80	19.4	0.189	140	123	88
200-250	,,	39	6.70	2.60	16.6	0.162	150	133	88
180	,,	35	6.30	2.20	13.3	0.120	170	153	90
140	,,	31	5.50	2.00	10.6	0.103	176	162	92

$h_{av} = 2.35$ m



Inadmissible because of :

A : to high value of ij and u
B : ,, ,, ,, ,, ij
C : ,, ,, ,, ,, u

Since they, however, occur relatively little, for the time being the imposition of a speed limit has been rejected for these vessels too.

With the knowledge thus obtained, it is known fairly accurately what can be expected if a newly built canal is made available for shipping.

You will understand that there are objections to setting up general rules for the choice of the type and construction of the free board. The data are different every time, and these must be taken up with insight and knowledge of affairs.

We shall now examine some constructions executed in the Netherlands:

Type I The figure show forms as they have been applied respectively along the Gouwe-Aar Canal, the canal Ghent-Terneuzen and the Amsterdam-Rhine Canal. These constructions are in general fairly costly and come into consideration if the available space is limited or if the adjacent land is costly. Now that according to modern knowledge a certain minimum depth is designed for the earth retaining wall to prevent the breaking of the waves, this type of free board lies once again more open as a possible choice. But then certainly all soil above a height of about 1,50 m. must be retained.

Presumably on the ground of these considerations, the construction over a great length of the Amsterdam-Rhine Canal has been built.

A concrete wall has the objection of not being sufficiently soiltight so that at present, means are being sought to promote tightness especially where the soil to be retained consists of sand.

For the discussion of the concrete walls of type III this will be gone into more deeply.

Type II The figures represent respectively constructions which have been applied along the North Sea Canal, the Twenthe canals, the canal through South Beveland, the Betuwe Reach of the Amsterdam-Rhine Canal, the Winschoterdeep and the harbour Oudenbosch.

The constructions with a sloping mattress have been executed in the wet. As concerns the Twenthe canals, it was the parts which had been allowed to stand as dam for excavation in the dry (provision of work) and which later had to be brought under profile by means of dredging. As opposed to the free board provision made there in the dry, (which consisted of twisted faggotting with faggot filling), it appears that this construction is good.

The construction as applied in the canal through South Beveland could naturally only be made in the dry. Any possible damage beneath the waterlevel will be repaired with difficulty. An improvement of this construction has been applied in the Betuwe Reach of the Amsterdam-Rhine Canal. The stone cover has there been impregnated with an asphalt-sand mixture.

In the Winschoter Deep some trial lengths have been made with a covering layer of asphalt-sand and asphalt-concrete (tarmac and macadamized-stone). The experiences with this tarmac construction were not entirely favourable. It appeared that the plants grow through the construction, whereby the construction is destroyed after passage of some time. In spite of counter-measures with poisons and hormone-killers, there has not yet been success in getting rid of this nuisance.

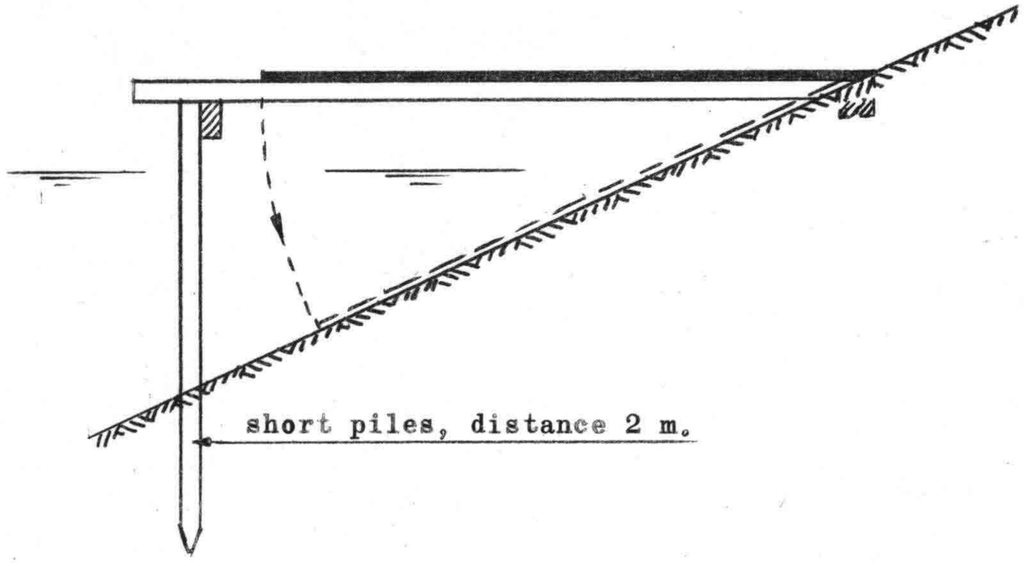
These constructions can naturally only be made in the dry.

By Shell, a method has been developed which makes it possible to lay a bituminous cover without joints also in the wet. In the year 1954 a short test length 100 m. long was made in the harbour of Oudenbosch.

The results were such that there, in 1955, 1½ km. of canal bank has been provided with such a covering.

It seems that with this construction the conditions, which have been stated, are more or less satisfied.

This is an introduction to give here a short description of the composition and way of placing this construction.



Since the pouring of the asphalt mixture should take place on a horizontal surface, the construction can <u>not</u> be made directly on the spot.

The following method of working has therefore been thought out: the asphalt plate is made on an auxiliary construction. The floor consists of a platform of planking which is supported on one side by the bank and on the canal-side by a beam fastened to auxiliary piles. When the asphalt plate which is continuously being made has cooled off, the platform can be pulled away plank by plank perpendicular to the line of the bank whereby the asphalt plate finally comes to lie on the slope. In this operation, the plate locally takes up a screw form from which all sorts of stresses will occur. In order to be able to take up these stresses, a reinforcement is placed in the plate of 1" galvanised steel gauze which is sufficiently strong for a plate of 4 m. breadth and 5 cm. thickness.

It is not avoidable that the plate remains hanging from its reinforcement overnight and at week-ends. To prevent the curring through the asphalt, the asphalt plate is provided with a skeleton of crushed gravel 25-40 mm. which has been previously enclosed in a skin of bitumen.

The schedule of work is thus as follows:

a. The slope is made even.
b. The auxiliary construction is put up.
c. The platform is covered with waterproof paper to prevent the asphalt sticking to the platform.
d. 1" iron gauze is laid on top of this whose courses are bound with soft iron wire.
e. Over this c. 40 kg/cm^2 crushed gravel is spread, coated with c. $1\frac{1}{2}$% by wight Mexphalt R85/40. To prevent the stone sticking together after coating, it is dropped into water directly it comes out of the asphalt plant. The stones, which have been "shocked" into entirely separate elements will not even stick together during or storage.
f. c.20 kg/m^2 Mexphalt R85/40 is poured onto the stones.
g. After about $\frac{1}{2}$ hour the removal of the platform can be begun, whereby the asphalt plate drops onto the slope.

Coverings of impregnate stone, tarmac etc. are water-tight. They have to be so, otherwise, in the course of time, the bitumen skin loosens from the material whereby the bonding is lost (stripping). Water-tight coverings imply the possibility of unbalanced water pressure arising inside. On the other hand, we have the relatively small weight of the bituminous covering.

In an article about this question is declared that such an unbalanced water pressure from inside will lift the asphalt plate somewhat (probably locally) from the slope. Thereby arise between asphalt plate and slope, narrow channels through which the water can flow downwards, the unbalanced pressure disappears and the equilibrium of the asphalt plate is again restored. It should only be taken care that the drainage water at the underside of the construction can flow away. It has been calculated that even with several meters excess water pressure behind a tight covering layer on a slope of coarse sand a system of channels of some millimetres height is sufficient. Naturally, a slope with water draining off will have to have a slope more gentle than the angle of internal friction, in connection with the stability, as has already been shown. The slope can be made steeper if care is taken, that, by means of an artificial drainage, the water does <u>not</u> leave the slope. In that case, one must, however, be certain that such a drain cannot become blocked.

The article mentioned was published by an engineer who works for a company that <u>sells</u> asphalt constructions. So we cannot be sure that he was objective when writing this article.

Even this is a very important question for futural construction. So there is established a working-group that is studying this problems. This group did not yet finish its study, and so I cannot tell you about the results now.

A disadvantage of thin bituminous coverings under water could be that, if there was damage from the keel of vessels with shallow draught, the sand can escape and a fairly large area can come into a bad condition, while repair will not be simple.

<u>Type III</u>. This type, which consists of a vertical earth retaining wall up to approximately canal level and a protection for the slope lying above, is applied in very great measure and in a great number of variations.

The rush berm + provision on the canal side, is also classed under this type and will be examined first.

The figures show such constructions along the <u>Twenthe canals</u>, the outer reach of the <u>Wilhelmina Canal</u> and in the north of the country the frequently applied Frisian Wattle-matting.

Concerning the constructions applied in the <u>Twenthe canals</u>, the following remarks may be be made: The quality of the reed and of the root system is dependent in great measure on the condition of the soil in which it must grow. In places where the rushes grew less strongly the toe-construction appeared to be not strong enough and eating away of the slope arose. In such a case, and also in a case where shipping is present before the rushes have had a change to strike root, a strong side strengthening should be made. Such constructions have been given. They were applied over a considerable length in the continuation of the branch-canal to Almelo. The cheapest is the construction with rush mats. These are largely made on the site by means of a press from rushes taken from the rush-berms of the canal. The bond in the mats is obtained by weawing galvanised iron wire in the mats.

The length of life of these mats is closely dependent on the speed with which the iron wire rusts through, and this is again closely dependent on the purity of the water.

A somewhat more expensive but also more durable construction is that with clamped-up wooden boarding in place of rush mats.

In these constructions, both the stakes and the rush mats or wodden fences were installed on the spot by means of spraying.

Concerning the constructions applied in the outer reach of the Wilhelmina Canal, the following remarks may be made: The construction as made in the laying of the canal led to a great disappointment.

This consisted of a wooden sheet wall driven along a waling with concrete slabs joining up with the wall at its upper edge. Because this construction appeared not to be sand tight, there arose hollowing out under the concrete slabs of very great extent. This construction was abandoned. The pile wall with waling now functions still as side-strengthening. By excavating the canal embankments which are in general very broad, a 3 m. broad berm has been formed on which rushes are set. To prevent scouring of the rising slope during the period that the rushes must develop, a light protection of concrete blocks and turf blocks has been applied. In places where the canal had to be widened, a side reinforcement of stake fencing with clamped up wooden boarding or of wooden sheet wall was brought in. In general, the introduction of the rush berm is considered here as a great improvement. Here it should be said that the local water level is influenced by the tidal movement and varies daily between NAP-0,35 m. and +1,20 m. It has been observed that the foremost rows of rushes which are placed on approximately N.A.P., lay behind the higher placed rows in growth. This demands some maintenance (extra planting). Yet it is remarkable that under these unfavourable circumstances, the application of a rush berm still appears possible.

The Frisian wattle matting can be applied if no greater depth is present along the bank than 0,50 to 0,70 m.

Hereby the tempory function is given to the willow mattress of protecting the berm during the time that the rushes must develop and the roots still do not form an interwoven and bound-together mass. At diverse places along the Ems canal, it has been possible to observe that the parcel of roots reached down to 1 m. and more below canal level.

In a report of the Provincial Public Works Department of Groningen, it has been calculated that with the given condition of 0,60 m. depth along the bank, the speed of the vessels in the canal must be regularly limited by about 10% in relation to the speed allowable with a depth of 1,25, before the bank (in combination with a sheathing). The application of this construction is thus again dependent on economic considerations. However, a regular limitation of the speed of travel remains always a measure which can be easily transgressed. Checking of this is a difficult matter.

It thus appears that the Frisian wattle berm can offer a good and cheap solution in canals with navigation of little frequency.

Construction in the dry does seem desirable. The construction is also executed in the wet.

Let us now consider several constructions which are the most often applied of type III ie. with a sheet-wall construction up to canal level and a slope protection above this, not consisting of rushes. The figures show the constructions as these occur in the Maas-Waal canal, the Rhine-Amsterdam Canal, the Twenthe canals, the Merwede Canal in South Holland and the West Frisian Canals.

For the construction in the Maas-Waal-Canal the defence has to be brought a great height above canal level, considering that in connection with the water conservancy of the canal, the canal level can vary between N.A.P. +7,50 m. and +8,50 m.

As for the wooden sheet-wall, it has appeared also here that the upper parts of the wall became increasingly less tight due to mechanical wear. Since the soil behind the wall consists of sand, this had as consequence that the revetment came into bad condition in many places.
Since 1945 it has been attempted to counter this objection by replacing the sand adjoining the wall by a mixture of sand with asphaltemulsion, which is more compact. Presumably just as good success could have been expected from replacing this sand by gravel (which in this district is present in the bed and thus can be obtained in a cheap way). Also the upper part of the seams could have been closed with eternite plates, as was done in the Twenthe canals.

The construction which occurs in the side branch to Vreeswijk of the Amsterdam-Rhine Canal owes its form to the soft subsoil. Considerable subsidence of the sheet was feared and therefore a wall was made with sheet piles which must carry over to firmer soil layers the weight of the construction and the vertical component of the active soil pressure.

For the continuation of the Twenthe canals concrete sheet wall constructions were applied over a considerable length. Since sandy soil had to be retained, a sand tight construction was sought out. In both edges of the 8 cm thick and 50 cm broad concrete sheet piles a hollow was made of 2 cm radius.

The so-called wooden "swelling pole" which, with its circular cross-section, fits in the space between the concrete piles, was repeatedly, together with a concrete pile, brought into the job by spraying. This gave in general few difficulties. However, it appeared that pieces easily broke from the edges of the concrete piles which did not do the tightness of the wall any good.

An improvement was found with a variant joint construction. In this, the edges of the piles are stronger and the swelling pole, as a result of its rectangular shape, is cheaper.

In the Merwede Canal in South-Holland it was also tried to make the seams of the concrete sheet wall tight. In general, the work was not in sandy soil here for which the cleaning out of a hollow space by means of the spray-lance is not possible, as is also the simultaneous spraying in of the pile. After many unsuccessful attempts, it was yet possible to find a construction capable of good execution. Here too, the swelling pole construction has been applied.

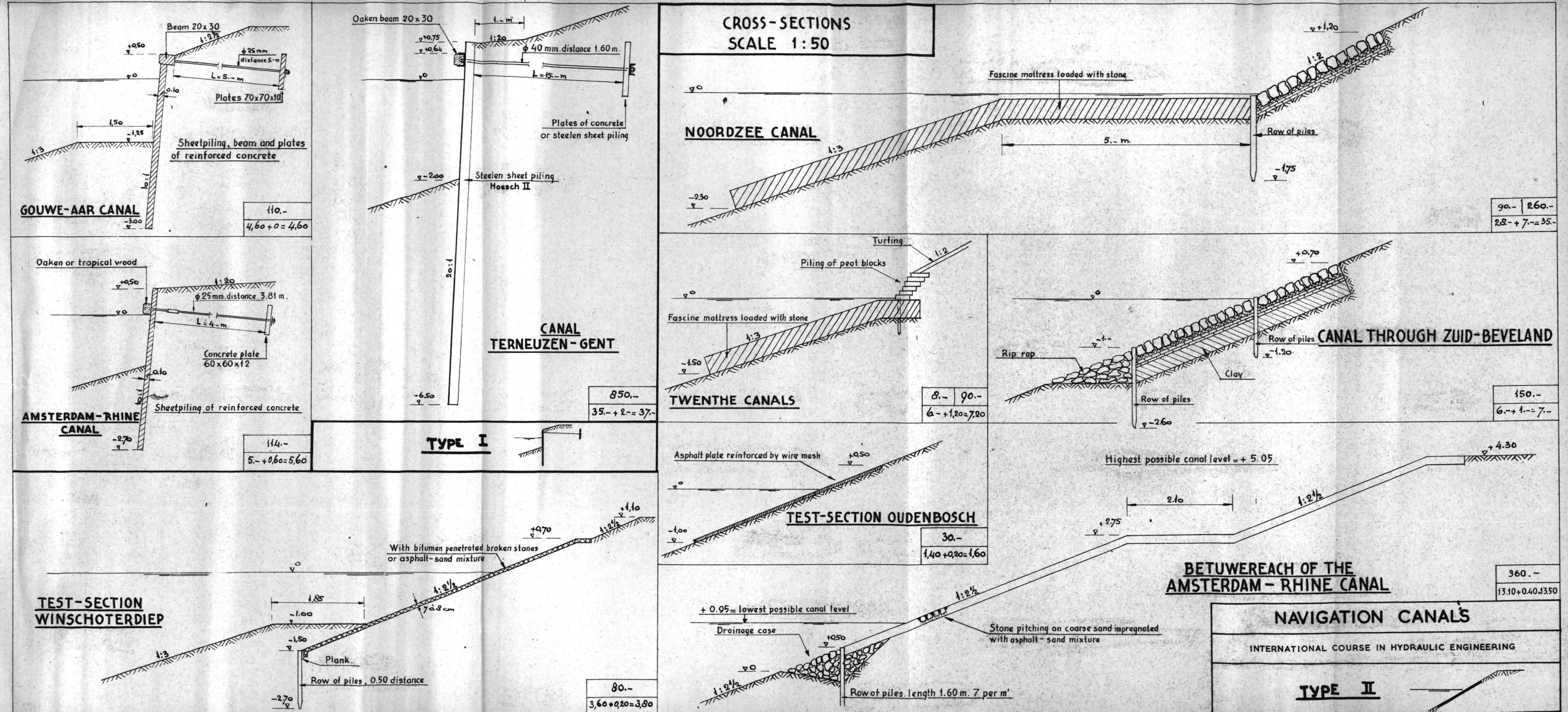
The question with this is always: 1st How are the piles to be held in the correct position during driving 2nd How is the material to be removed from the grooves between the piles 3rd How is the wooden pole to be set in. Initially it was specified that during driving, the piles must be led along iron bars or tubes into the grooves of the piles, which stuck 1 m. out above the piles so as to be able to withdraw them. By this method great practical difficulties were run into. In the first place, because of the rods sticking out above the pile, provisons had to be made in the driving-cap. Further, it appeared that, through not driving the piles exactly in the same plane, the rods could be withdrawn with great difficulty and were curved from the grooves. Moreover the set of the piles did not become any better from the pulling and tugging on the rods. Also the setting of the wooden pole often did not succeed because this broke. It was also attempted to use the wooden pole as guide and to drive these in simultaneously with the piles. In soft ground this did succeed, but in sand or in firmer soil, the pole received the blow in preference to the pile, with the result that this broke and the groove was not filled with the pole over the whole length.

Finally a good solution was found. Over a length of 0,75 m. on one side of the piles, in place of a groove, a tongue was made. Hereby, the piles again got guidance from another. Moreover the tongue makes the groove in the previous pile completely clean, so that directly after driving, the wooden pole can be set in without difficulty.

In practice it appears quite possible to place the pole in the groove of the pile as soon as the pile has been driven in 0,75 m. The pole then goes down along with the pile. The fact that the wall is not soil-tight over the lowest 0,75 m. is of no importance since here no soil tightness is required.

The construction applied in the West Frisian Canals shows a wooden wall with a concrete cap cast on top, which reaches to 0,15 m. + canal level, so that still only a narrow slope protection is necessary. The wooden walings serve as floor-shuttering for the concrete beam. This construction should be made in the dry or temporarily lowered canal level.

Now we have reviewed all these constructions, the question arises which construction should be chosen in a given case.


CROSS-SECTIONS
SCALE 1:50
GOUWE-AAR CANAL
Beam 20 x 30
1:2½
+0,50
0
ϕ25mm distance 5.-m
L = 5.- m
0.10
Plates 70x70x10
1,50
-1,25
1:3
Sheetpiling, beam and plates of reinforced concrete
10:1
-3.00
110.-
4,60 + 0 = 4,60
Oaken beam 20 x 30
1.- m
+0,75
+0,64
1:20
ϕ 40 mm. distance 1.60 m.
L = 15.- m
0
Plates of concrete or steelen sheet piling
-2.00
Steelen sheet piling Hoesch II
20:1
CANAL TERNEUZEN-GENT
-6.50
850.-
35.- + 2.- = 37.-
Oaken or tropical wood
+0,50
1:20
ϕ 25mm. distance 3.81 m.
0
L = 4.- m
Concrete plate 60 x 60 x 12
0.10
10:1
Sheetpiling of reinforced concrete
AMSTERDAM-RHINE CANAL
-2.70
114.-
5.- + 0,60 = 5,60
TYPE I
NOORDZEE CANAL
0
Fascine mattress loaded with stone
+1,20
1:2
1:3
Row of piles
5.- m.
-1,75
-2,30
90.- | 260.-
28.- + 7.- = 35.-
Turfing
1:2
Piling of peat blocks
0
Fascine mattress loaded with stone
1:3
-1.50
TWENTHE CANALS
8.- | 90.-
6.- + 1,20 = 7,20
+0,70
0
-1.-
Row of piles
CANAL THROUGH ZUID-BEVELAND
-1.20
Rip rap
Clay
Row of piles
-2.60
150.-
6.- + 1.- = 7.-
Asphalt plate reinforced by wire mesh
+0,50
0
-1,00
TEST-SECTION OUDENBOSCH
30.-
1,40 + 0,20 = 1,60
+1,10
1:2½
+0,70
With bitumen penetrated broken stones or asphalt-sand mixture
0
1:2½
7 à 8 cm
1.85
-1.00
TEST-SECTION WINSCHOTERDIEP
1:3
-1.50
Plank.
Row of piles, 0.50 distance
-2.70
80.-
3,60 + 0,20 = 3,80
Highest possible canal level = + 5.05
+4.30
2.10
1:2½
+2.75
BETUWEREACH OF THE AMSTERDAM-RHINE CANAL
360.-
13.10 + 0.40 = 13.50
1:2½
+ 0.95 = lowest possible canal level
Drainage case
+0,50
Stone pitching on coarse sand impregnated with asphalt-sand mixture
0
1:2½
Row of piles. length 1.60 m. 7 per m'
NAVIGATION CANALS
INTERNATIONAL COURSE IN HYDRAULIC ENGINEERING
TYPE II

CROSS-SECTIONS SCALE 1 : 50

NAVIGATION CANALS

INTERNATIONAL COURSE IN HYDRAULIC ENGINEERING

Besides the considerations brought forward the question what price will have to be paid for the various constructions will play a very important rôle in the choice.

In order to be able to answer this question, we should know the building costs, the length of life of the construction and also the normal yearly maintenance costs. Of these, the last two are difficult to estimate.

For each of the constructions discussed, various amounts have been mentioned in the following manner:

p + q
r + s = t

Here p + q represent the building costs, if possible divided according to the cost of slope defence (p) and of the foot construction (q). r represents the amount which must be put out yearly at compound interest of 4% in order to be able to renew the construction completely after the expiry of its life-span. For this, the varying life-span of the components of the construction is taken into account as much as possible. s represents the amount that according to estimates must be spent as a yearly average on maintenance of the construction. Finally, t represents the total yearly amount which must be considered as the "price" of the construction. This amount may thus be used as comparison between the various forms of construction. For this, as the probable length of life, the figures are assumed which are mentioned in the report to the XVIth International Navigation Congress at Brussels.

The prices are based on costs of workers'wages and materials in 1st Jan. 1955. They are composed partly from information forwarded by canal managers, partly from estimates made of costs. Naturally, the prices do not lie at a constant value for various places in the country and in various situations. The prices given must then be considered as indicating the order of size. They are often therefore rounded off. In fixing the prices, the difference in land occupation and of terrain width which is the consequence of greater or less width demanded by the freeboard construction has been left out of consideration since these two factors vary too much with local circumstances.

For the price of the foot construction (q) the costs of a possible concrete beam on the top of the sheet wall are included.

If the results obtained are examined, it appears that by application of a rush berm with edge-closure (thus in a case of not too busy traffic), a "price" of c.fl.1,50/m. freeboard must be reckoned with. From this, it follows likewise that the Frisian wattle matting mainly applied in Friesland, which certainly is not stronger than the given constructions with rush berm and edge-closure, really do not come into consideration because of the high price of fl. 2,50.

For the constructions which come into consideration in busy traffic, for a foot-construction + stone revetment, a "price" must be reckoned with of about fl. 5.00 to fl.7,00/m. For a complete slope protection (type II) the prices differ widely.

The <u>favourable impression</u> which the price of asphalt constructions makes is striking. It seems advisable, by a broader application (possibly in the form of test lengths) to amplify experience with this sort of construction.

Likewise, it appears that, what for the rest was to be expected, the price of the freeboard is highest in canals with a variable canal level and above all in canals which can be navigated by seagoing vessels.

13. <u>Junction or Crossing between Canal and River.</u>

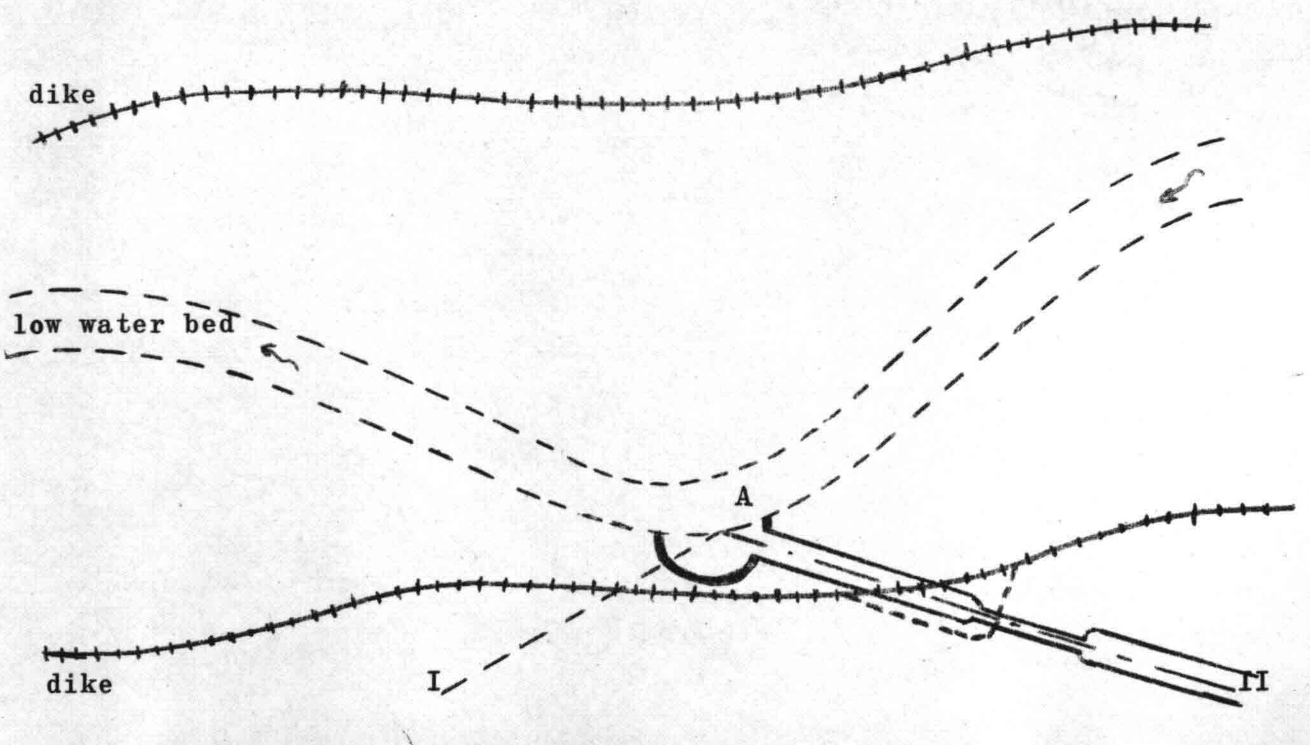
Since a certain shipping-route in general runs via rivers and canals, we shall be concerned with a junction of the two, and sometimes even with a crossing.

A. <u>Junction.</u>

In general the water levels in a river will vary strongly because of irregular discharge. For a canal, such a strongly varying waterlevel has the objection that it must be provided with high dikes (to contain the high water), a deep-lying bed (in connection with low water levels) and that a very broad strip of the banks must be protected in connection with the varying situation of the wave attack.

<u>Mostly</u> therefore a <u>lock</u> will be built at the junction whereby the water levels of <u>river and canal</u> are <u>separated.</u>

The lock fulfills at the same time also the function of holding back the water at the point of juncture. (The river dike present is broken there).



As will be known from the lecture "Rivers", river meanders, if nature is allowed to have its way, move.

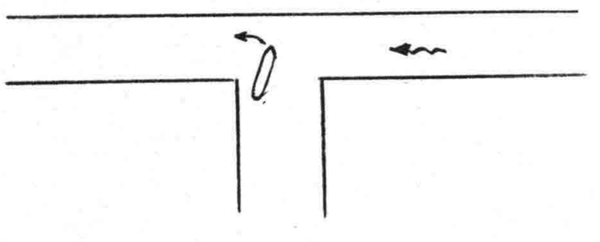
Naturally this gives great difficulties for canal joining. If the junction was made at a deep convexity, in the course of years at the junction there could well be very little depth in the river which is naturally not allowable.

It is thus a <u>first</u> requirement that at the place of juncture the banks are fixed whereby a movement of the meander is prevented. Also to prevent switching of the channel, the riverbed has to be fixed some bends upstream of the juction. It does not need to be said that in a regulated or normalized river, no more banks need be stabilised because that has already partly or entirely taken place. About the way in which river banks are stabilised, one may be referred to the lecture "Rivers" from Prof. v. Bendegom.

Seeing that in the convexities of the river the greatest navigation depth is met with, such a bend will be chosen for the junction eg. junction in the neighbourhood of point A.

Now the entry of a canal from a river by boat is a manoeuvre which is not so simple and requires craftmanship. Besides the craftmanship of the skipper it is however also necessary that the situation is designed as favourable as possible for sailing in and out.

If the junction is in a direction perpendicular to that of the river, this gives great difficulties for the skipper, because at a given moment, if the vessel is traveling with a small speed owing to the sharp bend, the foremost part of the ship has arrived in the stationary canalwater, whilst the rear part is pushed round by the current.



A perpendicular connection does therefore not look very preferable. It is better to choose the junction so that the vessel needs to change course as little as possible during entry. The junction ought thus to be made along I or II.

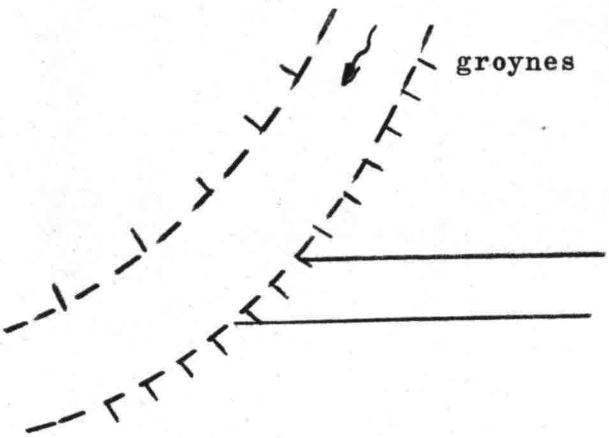
Now there are all sorts of arguments why course II gains preference. In the first place, it is an advantage that the vessels have a countercurrent when leaving the river, Their speed is then relatively small whilst the vessel still responds well to its rudder.

In the second place, the bed-load carried along the bottom cannot enter the mouth so easily as if the junction were made "scooping" along I. This holds likewise for ice.

If the junction is laid along II, the vessels which come from upstream must first sail past the canal mouth, turn round in the river and then move into the canal mouth against the current.

As we shall see later in an example, there is sometimes, for very definite reasons, divergence from a course along II, whereby the disadvantages attached to course I are accepted.

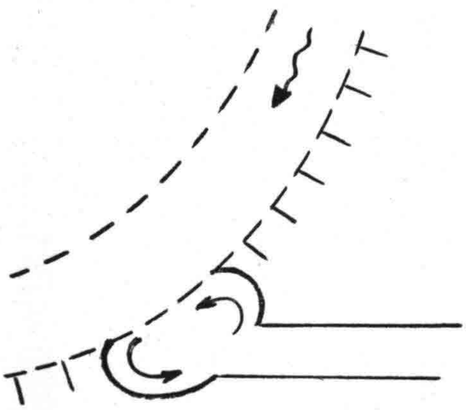
If one is concerned with a regulated river, then one or more groynes must be removed to give entry to the canal. This cannot happen without any disadvantage.



At the place where the groynes are missing, the river gets the chance to choose a broader profile of flow. The consequence of this is that at that point a current paralysis occurs, whereby, just before the entrance to the canal, shallows can form.

In order to decrease the dredging work necessary because of this, and the inconvenience to shipping, it has been attempted, by means of a model investigation, to design the form of the canal junction such that a big eddy can develop, which then so guides the discharging riverwater that it remains inside the normal-line and thus no current paralysis arises.

The dams which separate the canal from the water meadows may on no account hinder the high river discharge. They may thus not lie higher than groynes present or other obstacles present in the water meadows, such as wharves, cart-tracks etc.



The length of canal between the junction and the lock must be sufficiently long to give the vessels opportunity to reduce their speed to zero in good time and must at the same time offer room for waiting for the lock.

If the water meadows at that point must assist with high river discharge, there would arise for the waiting vessels a very troublesome cross-current. In that case, the lock must be built so far inside the dike that current-free spaces for waiting could be formed. The connection of the dikes to the lock then demands much earth work.

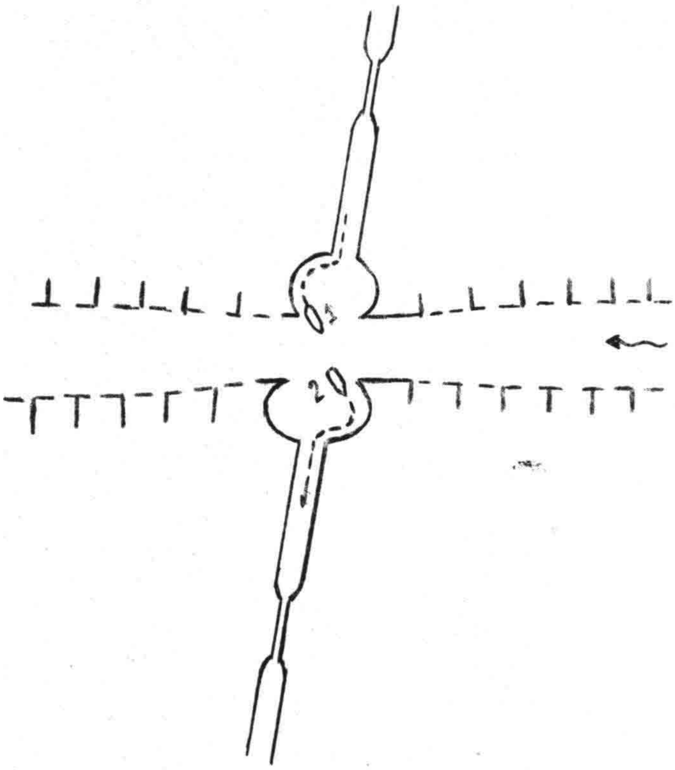
If the water meadows are not submerged with high river discharges, then the lock can usually be built nearer the existing dikes. The lock is then often built quite close behind the existing dike, the dike are connected onto this sideways, and the piece in front of the lock is dredged out.

Mostly, a double set of doors in a lock will be required in connection with the possibility of a defect or breakage by collision of one set.

B. <u>Crossing.</u>

Again, the problems are different if a canal must cross a river, especially if the river normally offers insufficient draught to the vessels which must navigate the canal. The crossing would then be the bottleneck for the vessels. Certain provisions should then be met with on the river.

Such a crossing can, for example, appear as follows:



If the shipping on the two shipping lanes (canal and river) carries straight on, there is no reason to make a smouth connection.

The shape of the junction will be so designed that big eddies arise which guide the discharging river-water.

If the canal offers a draught of 4 m. for example and the river in times of low discharge only 3 m. for example, a local deepening of the river bed can be obtained by making a local narrowing of the low-waterbed. b.h. = constant. The extent of the narrowing is thus very much dependent on the desired deepening.

A crossing-manoeuvre runs now as follows: the vessel travels into and against the eddy and reaches the following river-water in position (1). The boat travels on in this direction and drifts at the same time with the current so that it reaches the far side in position (2) and again against the eddy in the canaljunction enters the waiting basin for the lock.

The advantage of a stable eddy is that the unavoidable shoaling is always obtained in the middle of the eddy where the velocity of flow is the smallest and where the dredger hinders shipping the least. Designing such a situation naturally requires a thorough model investigation.

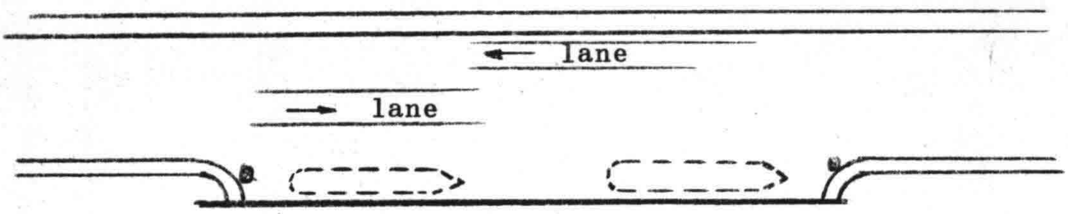
14. Equipment of a Canal.

In the equipment of a canal one must count lay-bys, loading and emptying, wharves, canal harbours, turning basins, waiting basins, mooring piers, fendering etc.

Further we meet structures such as locks, bridges over the canal, culverts under the canal, outlet works, ferries etc.

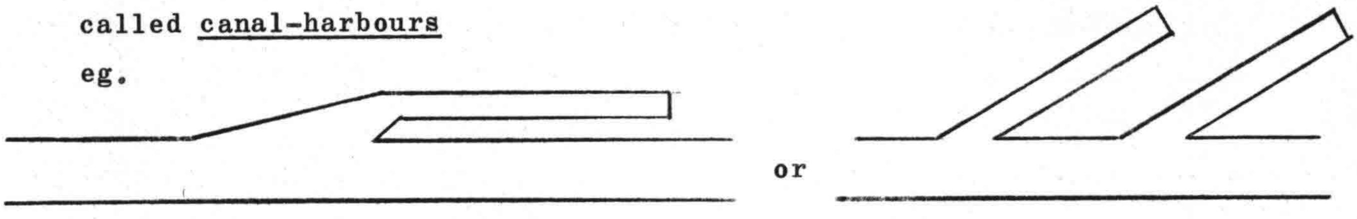
All these must be made such that the shipping can be best served and the least hindrance is caused.

So, if lay-bys or loading and unloading wharves are projected, care will be taken that, by bringing in a local widening, the normal space remains available for through shipping.



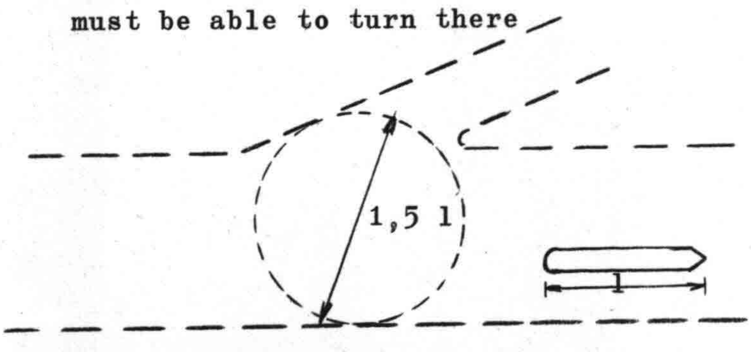
As soon as the extent of loading and unloading increases it is better to design the loading and unloading wharves on side branches of the canal, so-called canal-harbours

eg.



or still more extensive, according to the need. At such loading and unloading wharves (which are often the end point of a journey) there often exists the need for the possibility of turning a vessel.

There is often a need for this in the region of locks too. For this, turning-basis should be made. The centre line of such a turning basis must be designed on at least 1,5 x the lenth of the biggest vessel that must be able to turn there



Waiting basins are always necessary on both side of locks. These must be so projected that

1. the ships waiting give no hindrance for the ships sailing out of the lock
2. the vessels can easily and quickly leave the waiting basin to move into the lock chamber. If there is a prevailing wind direction, the waiting basins must lie on that side of the canal such that the vessel automatically drifts away from the mooring place to which it was moored, due to the wind.
3. in connection with the fact that at these waiting basins the velocity of vessels is zero, there exists here the danger from the vessels' screws which cause turbulent currents for a long time at the same place. Extra depth must thus be provided here to hold the bed free from erosion and an extra strong bank protection to prevent scour of the banks.

The mooring-piers and fendering are met with in the canal where sailing cannot take place normally, thus at locks, bridges, harbours, loading and unloading wharves etc.

These can, according to need, consist of one or a combination of single piles, or of more complicated constructions, possibly provide with gangways to give the skipper opportunity to leave his vessel in order to do his buying-in etc. These structures are not gone into here and d one is referred to the lecture "Locks" from ir. Josephus Jitta. But it must be mentioned that the fairly rigid constructions of the past are being replaced by more flexible constructions.
These offer the advantage of being better able to give, under impact from a vessel, whereby the energy of the impact is spent more gradually whereby damage to both sprung construction and vessel will arise less easily.

As concerns bridges, which will necessarily cross the canal to give land traffic opportunity to follow its course, the following may be remarked. By the shipping, requirements will be set on the headroom. If these cannot be satisfied or if it is too dear, the bridge can be made movable (unlimited headroom).

A movable bridge is troublesome for land traffic because it forces the traffic to wait. Therefore it is being more and more decided, if the situation allows it to lay the movable bridges so high that a great proportion of the shipping can pass without the bridge needing to be opened for this.

The shipping will also set requirements concerning the navigable width. Moreover it will prefer to see, especially if the traffic is busy, two navigable openings, that is one for each direction. These two openings were formerly offered by the then very customary swing-bridge with centre pier. Now the roads become continually busier, the bridge will also have to be broader. In these cases, a great part of the canal width is occupied by the open bridge.

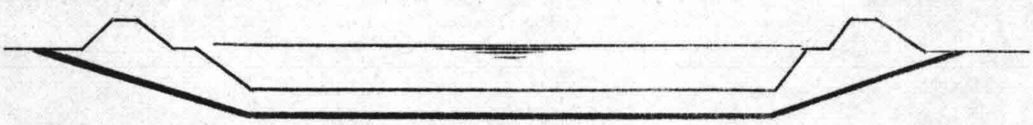
Swing-bridges are therefore only suitable if the roads are of minor importance and therefore fairly narrow.

No designer will succeed in being able to build a fixed bridge, however, high over the Vliet. The possibility of letting floating cranes and hoists which exist at the moment pass, will not be bought at any price.

The Hoorn Bridge in the Rijksweg is an example of a movable high level bridge which need not, by a long way, be opened for all shipping, whereby the delay for the road traffic is limited.

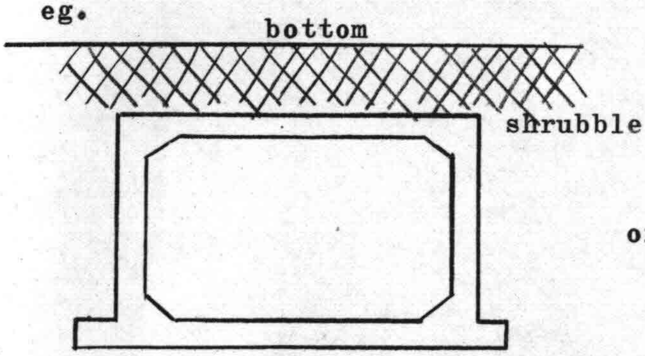
As a rule, the existing drainage system of the intersected land will be disturbed by the laying of a canal. It can, for example, happen that a part of a polder is isolated from the rest. Such a cut off polder could be made independent and be provided with its own pumping.

It can also be arranged that the existing drainage system remains intact by maintaining the water-connection between the two parts of the polder. This can be achieved by building culverts. These are pipes or sewers which are carried under the bed of the canal. In fixing the upper edge of the culvert, one must keep in mind any possible deepening of the canal in the future.



Also efforts must be made to prevent damage to the culvert from ships anchors.

eg.



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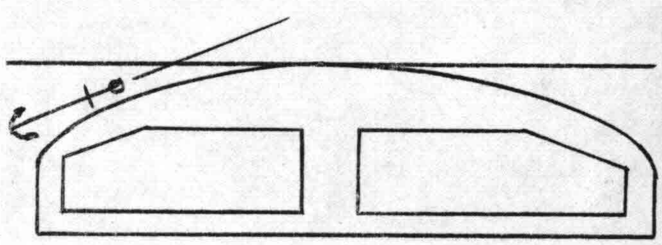

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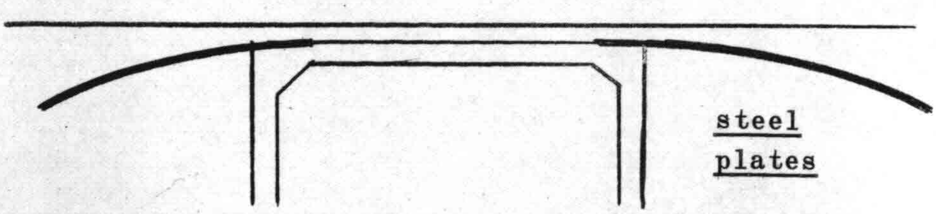


Since culverts are naturally deeply founded, the building costs will be high. Especially in sandy soil, local drainage by pumping, which is costly, will be necessary to get and keep the construction trench dry.

It will therefore be of importance to limit the depth of the underside of the culvert as much as possible.

This can be achieved by making the culvert long and broad. In place of a broad culvert, it offers more advantages to make two culverts or two mains next to each other so that, when necessary, there is opportunity to repair the culvert opening by opening if damage has arisen.

The cross section necessary for the culvert should naturally be determined by a calculation of the quantity of water which must be able to pass per unit time.

If water is let into the canal from the side somewhere, the <u>outlet structure</u> should be so installed, 1st that the cross-current remains so small that the shipping experiences no hindrance from the flow and 2nd that no sand can be brought into the canal by the water flowing into it.

As a link in very unimportant roads, one does see ferries used. In a single special case, so-called ferry-cars are made. These are platforms on high legs which ride on wheels on rails on the bed.

Crossing by a ferry or ferry-car may only take place if the shipping allows this. The shipping is thus always <u>primary.</u>

15. Some Particulars of the Maas-Waal Canal.

To finish off the lecture "Navigation Canals" there now follows an oversight of one of the big Dutch canals, the Maas-Waal Canal.

Successively there will be discussed:

A. Reason for existence
B. Capacity
C. Course
D. Canal level
E. Locks
F. Water conservancy
G. Free board
H. Movement of shipping
J. Diverse

A. The Maas-Waal Canal has been laid as a component of the works which must serve to bring about a good shipping communication between South Limburg and the remainder of the Netherlands.

Through this canal (which was opened for shipping in 1927) the Maas, which had been in the meantime canalised, was connected with the great shipping lane, the Waal.

B. The canal has been designed for vessels of 2.000 ton (assumed dimensions: length 100 m. breadth 12 m. draught 2,80 m. and greatest height above the water surface 7 m.). On the portion of the canal over a length of 800 m. to the south of the lock at Weurt, are even vessels allowed with a length of 120 m. breadth of 14 m. draught 3.10 m. and greatest height above the water-level of 9 m. (see too C4 and E). The cross-section is so roomy, that two vessels of 2.000 tons can pass each other at a point where a similar vessel lies along the bank (see drawing).

C. In fixing the course, the following considerations held:

1. Very roughly, the line Mook-Nijmegen was obvious

2. To limit the usuage of land, advantage was taken of the low land between the Rijksweg Nijmegen-Maastricht and the Overasseltse Hills. This Low-lying country reaches from the Maas to the Rijksweg Nijmegen-'s-Hertogenbosch.

3. It was considered desirable not to separate the villages Hatert and Neerbosch from Nijmegen by the canal.

4. Nijmegen wanted the mouth of the canal as near as possible to the town. There had, however, space to remain over for the building of a harbour for the Council of Nijmegen, which would be accessible via the Maas-Waal Canal.

5. The junction in the Maas by Heumen was made upstream for the following reasons:

a. The shipping from and to Limburg strongly prevails in respect of the shipping from and to Grave.

b. The canalised Maas is not so wide that the tows or big vessels can turn in the last easily. By laying the mouthing upstream, sailing in and out were thus made easier, although with this the disadvantage of sanding up and ice-jams had to be accepted. Moeover, the harbour before the lock had to be fairly long to give the vessels opportunity to reduce speed.

The minimum radius of bends in the canal and in the basins were fixed respectively at 2.000 m. and 1.500 m.

On the grounds of these considerations the course was chosen from <u>Heumen</u> (0,7 km. below the rail-bridge by Mook) to <u>Weurt</u>.(2,5 km below the rail bridge by Nijmegen). The canal was given a length of <u>13,4 km</u>. with a basin on the Maas 1,6 km. long and a basin 1,1 km. long on the Waal.

D. In principle, the canal stands in open communication with the Maas and is shut off from the Waal at Weurt by means of a lock. The canal level is thus the same as the weir level of the Maas above Grave = NAP + 7,50 m.

This level may rise to a maximum N.A.P. + 8,50 m. If the level of the Maas becomes higher, then the canal is shut off from the Maas at Heumen by means of a lock.

1. Locks by Weurt. Since the water level on the Waal at this point varies between N.A.P. + 4,91 and N.A.P. + 13.00 m., the locks at Weurt must be able to retain water on both sides. The lock-gates have been constructed as rolling-doors.

Through the lock must be able to pass the biggest Rhine boats (3.000 to 3.500 tons) which are to be expected in the harbour of Nijmegen.

The lock was given a navigable width of 16.00 m. and a chamber length of 260 m. (possible to be divided into two shorter chambers by an intermediate abutment). In fixing the height of the sill, a possible bed lowering of the Waal of 1 m. was taken into account. Already it has appeared that this assumption was too favourable and a greater bed lowering must be expected.

2. Locks by Heumen. This lock only needs to function of the level of the Maas rises above N.A.P. + 8,50 m. Higher water levels on the canal than N.A.P. + 8.50 m. were considered undesirable because then much water would seep through the highly permeable soil to the polders and estates, and also because then the fixed bridges would have to be laid higher and the banks would have to protected over a greater height against wave attack.

If the Maas-level remains below N.A.P. + 8,50 m. (about 3/4 of the year), then the lock stands open. Normally in the closed position, the flood doors are in function. The lock is, however, at the same time provided with one set of reserve ebb-doors, in order to prevent the canal running empty if the level of the Maas should sink below N.A.P. + 7,50 m. Locking between canal and Maas is then not possible, which is not important because this case can only occur by an accident to the weir at Grave or if this weir is lifted in connection with loose ice floating on the Maas. In these circumstances there is no change of any shipping.

Because when the lock stand open, it is sailed through at normal speed, the navigable width has been kept copious, i.e. 16.00 m. for vessels with a maximum width of 12.0 m.

The sill lies at N.A.P. + 3,70 m., so that with normal canal level, (NAP + 7,50 m.) 3,80 m. water stands above it. The shape of the lock chamber is about the same as in that at Weurt.

Adjoining both locks, space has been reserved for the building of a possible second lock.

F. In connection with the fact that canal level = weir level of the Maas above Grave, the canal-level will only be able to sink below N.A.P. + 7,50 m. in the cases stated under E 2. Because there is then no shipping anyway there can be no mention of a water shortage. An excess of water occurs if the level of the Maas rises above N.A.P. + 8,50 m. (the lock at Heumen is then closed) and the level of the Waal is also higher than N.A.P. + 8,50 m. In order to prevent further rise of the canal level, a pumping-station has been installed at Heumen which pumps the superfluous water into the Maas.

G. Because the ratio between the wet cross-section of the canal and the beam of the biggest vessel is favourable (with the lowest water level 5,5 : 1) and because the slopes of the banks are gentle (1:3), it was the opinion that by applying a berm of c. 2,0 m. at N.A.P. + 7,30 m., a relatively light free board provision could be accepted.

This consists of a revetment of hexagonal concrete blocks from N.A.P.+7,50 tot N.A.P. + 9.00 m. which are laid directly on the soil foundation consisting of sand and gravel. This revetment rests against a wooden waling which is supported every 1,50 m. by 2.10 m. wooden pile, whilst a 1,30 m. long and 0,06 m. thick sheet wall must prevent scouring of the covering of blocks. In the basins, a free board of the same type has been applied (with longer sheet wall and shoring-piles). The experiences with this free-board construction were not completely favourable. The 2.0 m. wide berm was lowered from 0,20 to 0,60 m. below the canal level through waves and flow resulting from the shipping. However, this gave almost nowhere any instigation to slumping or bursting of the wood construction. In spite of this, scour of the covering of blocks appeared to arise in many places. Presumably this is caused by the fact that, after the berm had been lowered to 0,60 m., below canal level, for every wave washing up, there occured a pressure of sand and water from the land side against the sheet wall. Through the cracks in the wall the water pushes its way out, and later, when the cracks have been sufficiently eroded, water + sand.

The lowered berms are gradually brought up to height again by means of dumping on top coarse gravel or concrete rubble, while the sunken block-covering is set afresh on the foundation filled up with gravel.

H If the data concerning the shipping movement through the Maas-Waal Canal collected by the Central Bureau for Statistics, are subjected to a closer examination, then the following appears:

The flow of goods which passes through the Canal is chiefly in a northerly direction. The number of tons carried north amounts to about 6x the number of tons carried south. The number of vessels which travels in the northerly direction is about 2x as big as what moves in a southerly direction. From this it can be derived that of these latter vessels, about 1/3 part is loaded and 2/3 part is empty and also that the half of the total number of vessels travels along another route to South Limburg. In 1948 the total number of vessels amounted to 31.000 with a total loading capacity of c. 12.000.000 ton (average c. 400 ton), In percentages of the total tonnage was carried, among other

36% sand, gravel etc.	in N.	direction
30% coal, coke etc.	" N.	"
9% " " "	" S.	"
7% stone	" N.	"
4% chalk	" N.	"
3% artificial manure	" N.	"
1% salt	" S.	"

J. Various water-courses ware cut off by the canal, which carry the water from east to west. These were taken up into an aquaduct which was dug along the eastern boundary of the land bought and which restored the water-connection with the region to the west of the canal by means of a culvert at three places. The upper side of the culverts lies in the centre 0,60 m. below the canal bed. The culverts have a useful cross-section of respectively 1,50, 1,15 and 2 m^2, and a length of 120 m. They consist respectively of three, two and three round concrete pipes adjacent in a concrete sheath.

Over the canal have been built 2 draw-bridges, 3 bowstring arch bridges and 3 truss-bridges.

The draw-bridge over the exterior lock abutment at Heumen has a head room of 4,80 m. (in respect of canal level) in a closed position, and of 10,35 m. in ipen position, which is 0,20 m. more than the head-room of the rail-bridge by Mook, which was raised after canalisation of the Maas.

The draw-bridge over the lock chamber of the locks at Weurt has a head-room of 8.85 m. in the closed position, and of 12,90 m. in respect of canal level in the opened position.

If locking takes place with the highest permissible water level (N.A.P. +12,80 m.), this head-room still amounts to 7,60 m.

The three bow-string bridges which span the canal at Malden, Hatert and Neerbosch respectively, have in the centre over a width of 40.0 m. a head-room of at least 8,45 m. in respect of canal level.

Because some settlement was to be expected in the abutments (bow-string bridges with high level deck give high transversal on the abutments), the arches were constructed as a three hinged span.

The three girder-bridges lie next to one another, i.e. two for the rail-way and one for the motorway from Nijmegen to 's-Hertogenbosch. In respect of canal level they have a head room of 8,45 m.

On the eastern bank of the canal are two loading and unloading wharves i.e. by Hatert an by Neerbosch with a length of 200 m. and 125 m.

The spoil from excavating the canal was used for raising the lock-plateaus, laying the railway (which was relayed), for making the approaches to the bridges and for making dikes along the canal.

These structures had to be finished before the canal could be reached with dredgers.

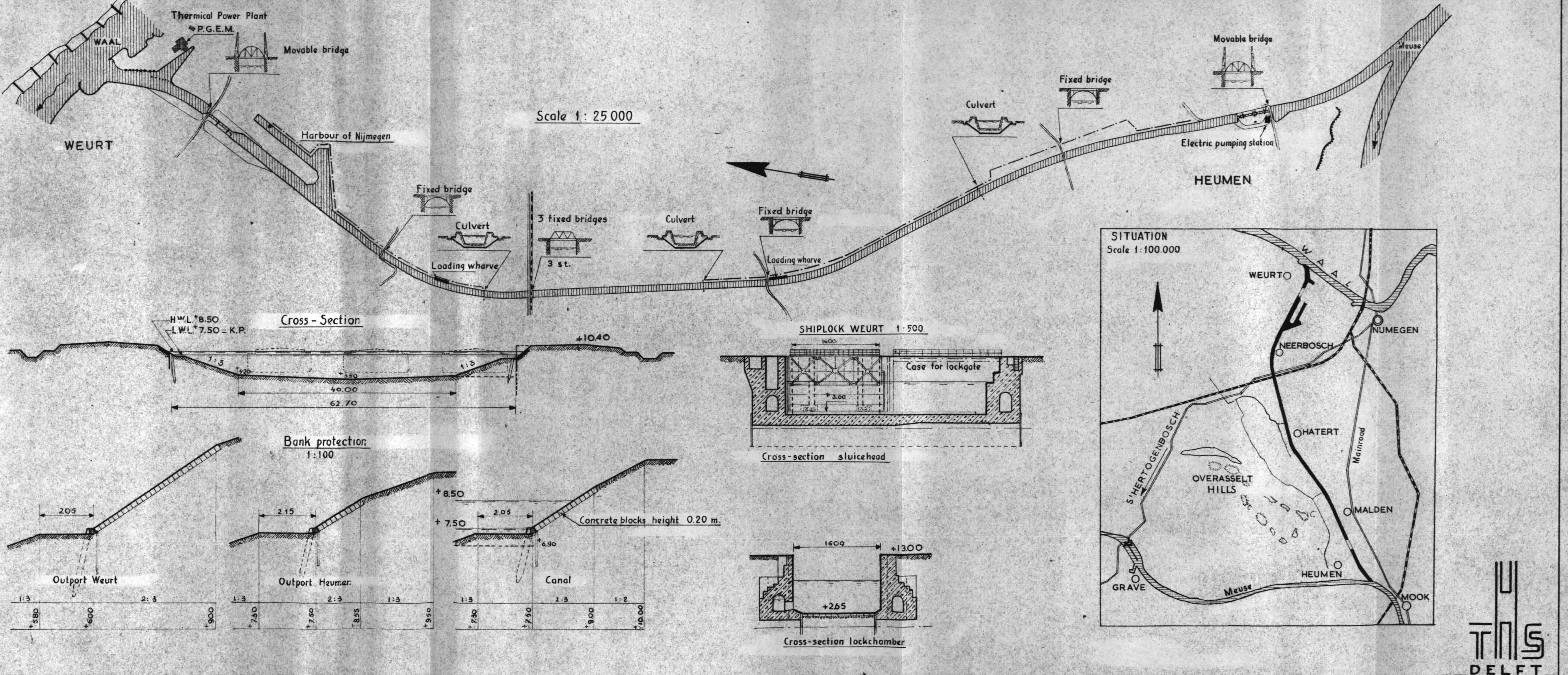
Therefore the excavation took place largely in the dry with the help of excavators whereby, by means of a pumping plant, 200 - 700 l/sec.had to be pumped away.

This took place in stretches of 500 - 1000 m. between which dams of about 60 m. were saved. The basins were dredged, while finally the dams mentioned above were removed by dredging. The sub-soil consists chiefly of coarse sand and gravel. The canal is under direction and maintenance of the state.

In 1944, the three concrete arch bridges were destroyed by acts of war, in addition to the draw-bridge over the lock at Weurt. For clearing up the broken pieces of the concrete bridges, dams were made in the canal on both sides of the bridge, with the help of scrapers and bulldozers, from the sand from the approaches to the bridge. The space between these was pumped dry and the pieces of concrete pulled out of the canal by bulldozers and later demolished. The resulting rubble was used for dumping on the canal berm in front of the sheet wall construction.

Shipping through the canal came into action again in August 1945. In the reconstruction of the bridges, the shipping was not significantly hampered, because the parts of the three-hinged span were prefabricated, and by means of hoists, were placed on a wooden auxiliary construction (which did not hinder the passage of shipping), after which the bridges were finished off.

N.B. In addition to the harbour designed for the Nijmegen Council to the south of the locks at Weurt being constructed, to the north of these locks a harbour was dug on behalf of the Electricity Power-Station of the Provincial Electricity Company of Gelderland. In the mouth of the Maas-Waal Canal troublesome shoaling initially occured. In connection with this a special form was given to the mouth, which was fixed after tests in the Hydraulics Laboratory at Delft. Hereby the extent of the Maintenance dredging work has decreased markedly.



MEUSE - WAAL CANAL

NAVIGATION CANALS

INTERNATIONAL COURSE IN HYDRAULIC ENGINEERING

THS DELFT

W en W