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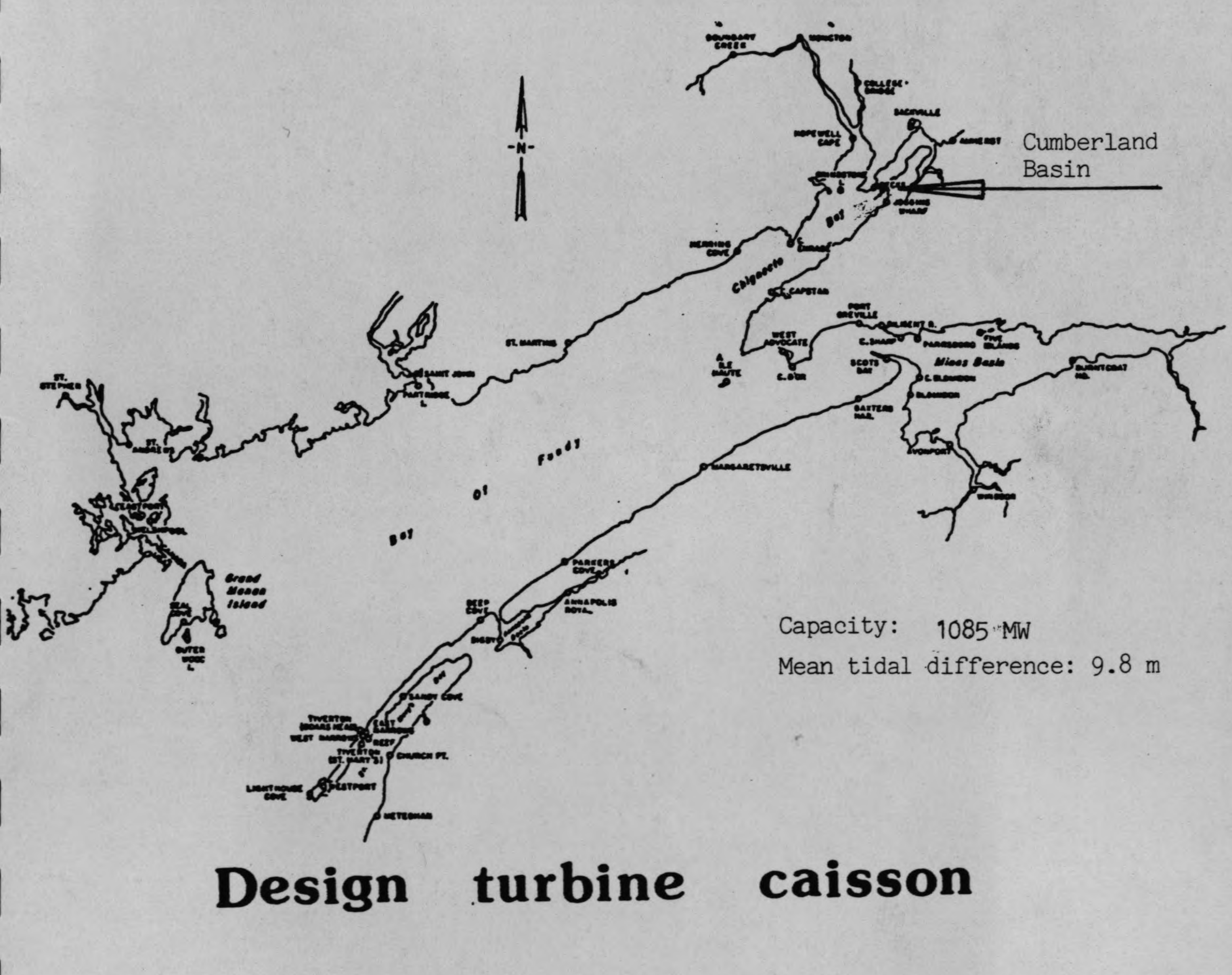
BAY of FUNDY

TIDAL POWER PROJECT

Location: Cumberland Basin



Capacity: 1085 MW
Mean tidal difference: 9.8 m

Design turbine caisson

This study was guided by prof. ir. J.F. Agema, prof. ir. Ch.J. Vos, prof. ir. A. Glerum, ir. J. Stuip and ir. J.A.G. Küppers and is made up with use of their experience. The ice part is written with help of dr. D.B. Muggeridge, Memorial University St. John's, Nfld. and dr. A.B. Cammaert and mr. S.Th. Lavendor, Acres Engineering, Toronto, Canada. Lots of thanks is given to all these people, but especially to prof. ir. J.F. Agema, whom most of the back ground ideas come from.

September 1985
Marielle Tavenier
Delft University
graduate student

Introduction.

This thesis is written as a part of my finishing study at the Delft University, Holland.

In the Bay of Fundy, the highest tidal water level differences devellop; three meters at sea side of the bay, increasing to 15 - 17 meters at the back at spring tide. Since the beginning of this century, people have been thinking how to make use of the tidal energy. In 1965, the Tidal Power Review Board, Halifax, presented the first designs. Two for plants in Chignecto Bay, with a capacity of 1085 MW and 1550 MW, and one for a plant in Minas Basin, with a capacity of 3800 MW. At that time, tidal power attracted a lot of interest.Even a small plant (20 MW) to test the straight flow turbine, which was especially develloped for this type of energyproduction, was built. As export market of the energy from the Bay of Fundy, the Maritime Provinces (Nova Scotia, New Brunswick and Prins Edward Island) New York and New England,were taken into consideration. The delivery distance to New England,Ludlow (Western Massachusets) is 860 km, to New York, Pleasant Valley, is 980 km. At the moment, politics have changed and interest in tidal power has dropped.

In an oriëntating study, attention is paid to the energy production and the lay out of the dam for a plant in Cumberland Basin, 1085 MW (§ 1-3). The main subject of this study was the design of the turbine caisson. Based on the design principle of the turning height of the dam and criteria concerning the type of turbine, an estimate of the caisson dimensions is made (§ 4). The loading -hydraulic, wave and ice loads- is discussed in § 5-7. Some attention is paid to the foundation as well (§ 8). Eventually, the stability of the caisson during the different construction fases and its final dimensions are determined (§ 9).

For mistakes made in the english writing, I would like to apologize.

Summery.

In the first fase of this study, attention is paid to the cross section of the dam for a plant in Cumberland Basin, capacity 1085 MW.
Some details of this project are:

length of the dam (HW):	2780 m
mean tidal water level difference spring tide:	14.50 m
basin surface:	73 km²

Chosen is for generation over the ebb with use of bulb turbines. The main subject of this study was the design of the turbine caisson.

Design principle.

For energy production, a hydrostatic water level drop over the turbine and a flow through the turbine is needed. Waves don't influence the head. Wave overtopping from sea side fills the basin and thus could be a gain of energy, wave overtopping from basin side is a loss of basin water and thus a loss of energy. This means, in order to accomplish the main function of the dam, energy production, just turning height from basin side is needed. A turning height of mean high water basin side, increased with the year design wave height is taken as design value (+ 9.50 m GSCD). Efficiëncy loss as a result of wave overtopping over or leakage underneath the dam proves to be negligible.Secondly, the dam would preserve a good facility for a road connection between New Brunswick and Nova Scotia. In the design , as proposed here, the road would have to be put on piles on top.

Dimensions.

The dimensions of the intake and outlet works are predicted by the turbine manifacturers. This determines the length of the caisson. To obtain sufficiënt stabillity during transport, three turbine sections per caisson are needed. The foundation depth depends on considerations concerning cavitation and wave running into the

outlets. Compared to a design, in which no wave overtopping is allowed, the caissonheight is reduced with approximately 15 meters.

Loading.

The caisson is loaded by hydrostatic waterpressures, wave- and ice loads. For the overall stability of the caisson, the wave- and hydrostatic loads determine the design value. For the local loads on the caisson, the ice loads give the determining values.

Foundation.

The stability of the sub soil proved sufficiënt. Attention would have to be paid to sealage of the sub soil, influences of wave loads and fixation of the bottom during construction. Schematically, some foundation methods are proposed.

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PART ONE

ORIËNTATION

All levels refer to Geometric Standard Canadian Datum, GSCD.

All calculations are based on a probability of exceedence of 10^{-4} during the life time of the constructions. A life time of 100 year is assumed.

1. Tidal Energy.

Tides are generated by the interacting forces of the sun, earth and moon. Their movements causes an enormous wave, which we perceive as ebb and flood,or high and low water. In the ocean, this wave has an amplitude of approximately one meter. In shallow coastal area's, this wave can increase enormously . Examples are the river La Rance, in France, with tidal differences up to 13 meters, the Bay of Fundy in Canada, with a maximum tidal difference of approximately 17 meter, the Severn Estuary in Wales, with maximum 13 meters and Secure Bay in Australia, with a maximum up to 11 meters.
For generation of tidal energy, the same principal as for a conventional water power station can be used, where water from a higher level is guided through turbines to a lower level. The mechanical energy of the turbines can be transmitted to electrical energy by a generator. The main difference between the conventional and the tidal power station is the variation in head. A conventional station usually has a fairly constant headdifferential over the turbine; at tidal stations, the head varies a lot.

1.1 Problems related to tidal energy.

Generation of tidal energy has disadvantages:

The energyproduction varies in time with ebb and flood, the daily variations of the tides and the spring and neap tides. The energyproduction in such a system is related to the fases of the moon. Our living habits, and thus the demand for energy, are based on the suncycli. This means tidal energy can only be used as a supply to the local system of energyproduction.
In tidal stations, the difference in head is very small. To produce a certain amount of power, large flows are needed and thus a large basin and many turbines and sluices. The station has to be built in a tidal area with high tidal differences and usually high currents. The resulting high buildingcosts are the main reason that until now

only three stations are built, one in La Rance in France, 240 MW, one in Kislaya Guba in Rusia, 0.4 MW,and one in the Bay of Fundy, Canada, 20 MW.

1.2 Generation.

The design of a tidal plant can be based on the following design purposes:

-constant energyproduction
-a constant capacity
-energyproduction with a constant head over the turbine
-maximum pumpstorage capacity
-minimum investment

Based on one of these designpurposes, the generationscheme is made up. The following schemes can be considered:

1. single basin scheme with
   a) ebbgeneration
   b) floodgeneration
   c) ebb- and floodgeneration

2. dubble (or more) basin scheme with a, b or c

1. Single basin scheme.

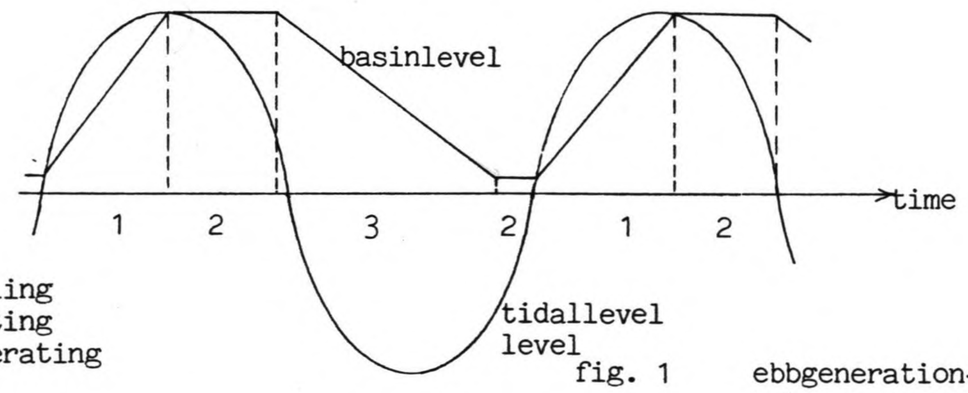
In a single basin scheme, the energy is produced with help of one basin in one or more flowdirections.

2. Double basin scheme.

In a double basin scheme, two or more basins are used. With such a scheme, a more constant level of energyproduction can be obtained. High investmentcosts (long dams, many turbines and sluices) have to be paid for this.

a) ebbgeneration.

With use of ebbgeneration, the basin is filled during the flood. As soon as the maximum waterlevel in- and outside is reached, the sluices are closed. For the energyproduction has to be waited, until sufficient headdifference over the turbines is obtained. From that moment onwards, the basinwater is guided through the turbines and energy can be produced. During the ebb, the head over the turbines increases; during the next flood, it decreases again. As soon as the head is too small, the turbines are closed. When the waterlevels in- and outside are equal, the turbines and sluices are opened to refill the basin during the last part of the flood. When the basin is full, the whole cyclus restarts.

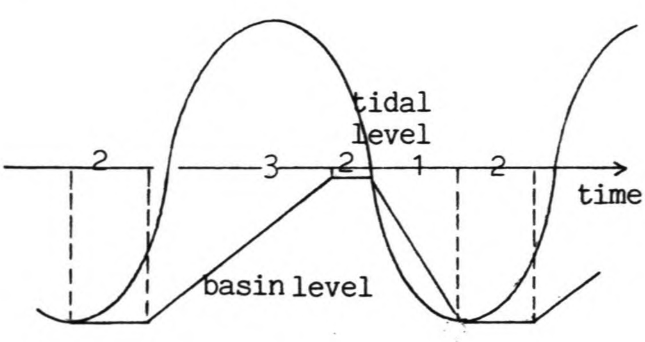


1. filling
2. waiting
3. generating

fig. 1 ebbgeneration-scheme

b) floodgeneration.

With a floodgenerationscheme,the energy is produced during the flood; the sluices are used to empty the basin. With use of this scheme, the waterlevels in the basin are much lower. This reduces the basin-surface and thus the energyproduction per generationcyclus. The lower waterlevels are considered to be a disadvantage for the navigation and the environment.



1. filling
2. waiting
3. generating

fig. 2 flood generation-scheme.

c) ebb- and floodgeneration.

Ebb- and floodgeneration means that the water is guided through the turbines during the ebb and during the flood,with the result that during both periods energy can be produced. This scheme offers more workingflexibility and more possibilities to adjust the energyproduction to the demand. Disadvantages of this scheme are:

1. The equipment needed is more expensive. Because the workingperiod is smaller, more turbines are needed. The turbine must be operational in both flowdirections.
2. The turbineblades can never be used optimally in both directions.
3. The headdifference available for generation is smaller. Pumping or more sluices can increase the headdifference a little.

1. generation
2. waiting

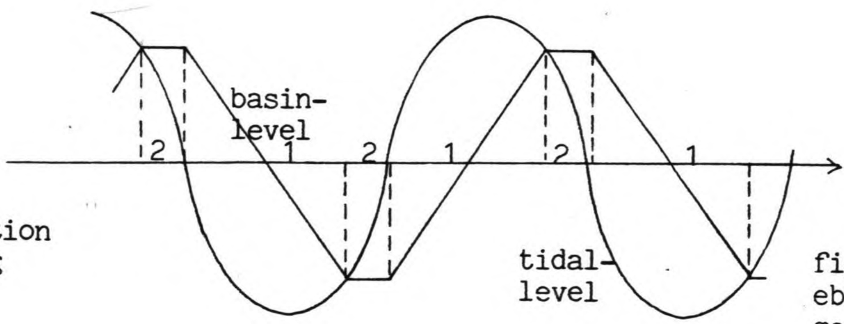


fig. 3
ebb- and flood-
generationscheme

By using the turbines as pumps, the head and working period available for generation can be increased. In this way, it even is possible to deliver energy at each moment of the generationcyclus. However, this reduces the total output a lot.

1.3 Choise of the generationscheme.

Tidal power delivers a fluctuating energy output. The ebb- and floodgenerationprinciple, pumpingcapacity and double basin systems are efforts to reduce these variations and to offer the possibility to be able to deliver energy at each moment of the day. They all imply:

- an increase in investmentcosts.

- more complicated hydraulic equipment
- higher maintenancecosts.

From studies during the last couple of years has been concluded that, when a modern electrical infrastructure in the surrounding area's is available, the design of a tidal plant would always be based on direct production of the energy against the lowest possible costs; a one-way generationscheme with use of a single basin would be prefered. For environmental considerations, the ebbgenerationscheme is chosen.

2. Tidal energy in the Bay of Fundy.

2.1 Introduction.

Tidal energy delivers a basis for energyproduction varying in amount and time. Due to the enormous increase in energy producing facilities and the connection and fusion of individual systems, more flexibilty in energyproduction became possible and the question of tidal power raise. At the moment lots of energy from the provinces Ontario, Quebec and New Brunswick is exported. As exportmarket for tidal power from the Bay of Fundy, the Maritime Provinces (Nova Scotia, New Brunswick and Prins Edward Island), New England and New York were taken into consideration. The delivery distance to New England, Ludlow (Western massachusetts) is 860 km and to New York, Pleasant Valley, is 980 km. However, I get the impression politics have changed and at the moment interest in tidal power has dropped.

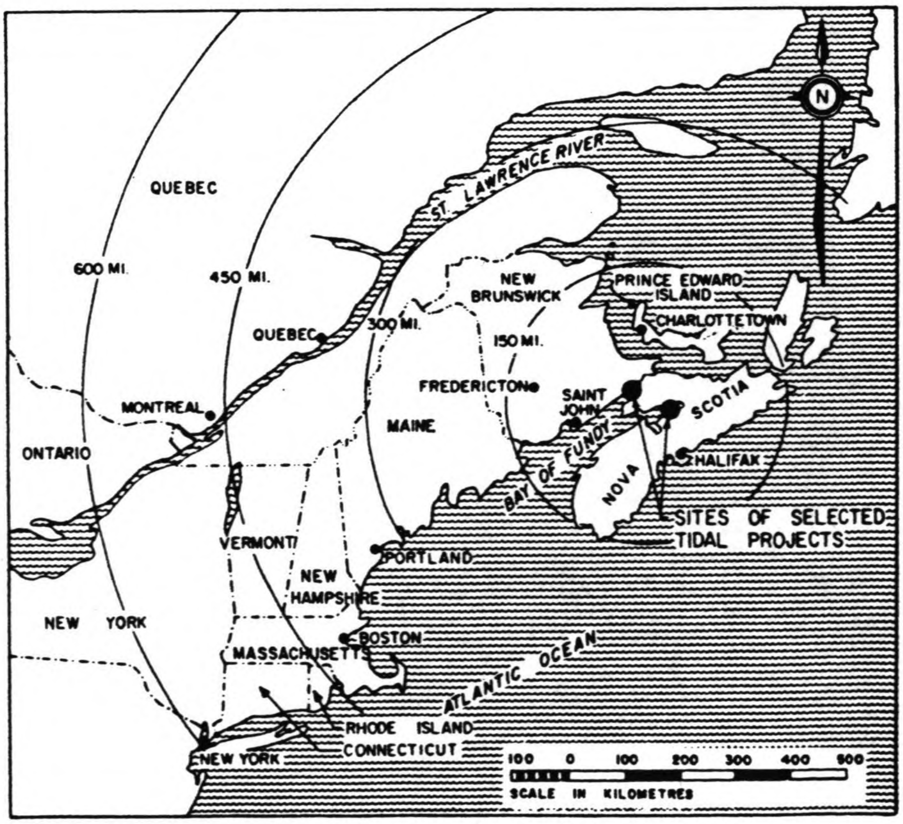


fig. 4

2.3 Possible locations .

The main factors concerning the locationchoi~~s~~ce of a plant in the Bay of Fundy are:

1. The amount of energy wanted.
2. The costs of the produced energy.
3. The environmental consequences.

1. The amount of energyproduction wanted.

The energyproducing facilities of the Maritime Provinces, called Maritime Integrated Systems (M.I.S.), can absorb the energy delivered by a station with a capacity up to 1500 MW. Stations with higher capacities will depend on exportmarkets. Locations with a capacity less than 1000 MW are less attrective for M.I.S..

2. The costs of the delivered energy.

The kWh-price of tidal energy mainly depends on the costs of the investments, and the capacity and the servicelife of the station. The energyproduction is equivalent with the flow through and the head over the turbines. The biggest tidalwaterleveldifferences are found, when the station is located in the upper regions of the bay, bigger flows at locations more seawards. The most important contributing factors in the costs of the station are the length of the dam, the depth of the basin and the amount of turbines and sluices needed. Several alternatives for the location are studied by the Review Board (1977) (fig. 7). From these, location A8, B9 and A6 proved to to be interesting for further study (fig. 7). Based on this information, the Board has studied the economical and financial feasibility of the alternatives A6, A8 and B9. Some information as follows from the rapport of the Tidal Power Review Board is reviewed here:

B9:	capacity:	3800 MW
	market:	M.I.S. and NEPOOL (New England Power Pool); not all the produced energy can be absorbed within M.I.S., a big part has to be exported. This means the economical feasibility also depends on the contractbasis with NEPOOL.
A8:	capacity:	1085 MW
	market:	M.I.S.; This project does not depend on a secondary market; all the energy can be absorbed within M.I.S.. The best scenario would involve 250 MW storage device added in M.I.S. with NEPOOL as secondary market. The optimal transmissioncapacity between M.I.S. and NEPOOL would be 500 MW.
	influences:	The nuclear power installation schedule would remain unaffected. A netreduction of 1000 MW is expected. Two oil-fired in installations would have to be closed, and 600 MW gasturbines installed.
A6:	capacity:	1550 MW
	market:	M.I.S. and 500 MW export to NEPOOL.
	influences:	The nuclear power schedule would remain unaffected. Oil-fired stations with a total capacity of 900 MW would have to be closed; 300 MW gasturbines would have to be installed.

The results from a benefit-cost analysis are given in figure 5 :

location	B/C	breakeven period
A6	0.9	-
A8	1.2	30-35 year
B9	1.2	30-35 year

fig. 5

Based on the changes in marketing strategy, higher oilprices and new construtionmethods, in 1982, an update of the results of the Review Board, called Update '82, was made up for the projects A8 and and B9. The change in marketing strategy was a more export promoting policy. An energydistribution over the markets of concern, as given in figure 6 , seamed realistic.

	Maritimes	New York	New England
B9	10%	45%	45%
A8	40%	-	60%

fig. 6

As a result of the long constructionperiod, needed for this enormous project, interest is one of the higher contributors in the total costs. With new constructionmethods, a reduction of the constructionperiod could be realised. This had the following consequences for the design:

-the closuregap was proposed to be closed with caissons as long as possible. The caissons would be placed first. The final closure would be completed with dykes in the inter-tidal zones.
- to reduce the waterleveldifferences between the sea- and basinside before closure, different sorts of caissons, varying in depth and function,would have to be designed.
-the delivery of the turbines is slower than the fabrication of the caissons. The installation of the turbines after the caisson has been sunk to its final place,was considered.

3. The environmental consequences.

Because of the enormous impact of tidal energy on the ecological system of the Bay of Fundy, for the choise of tidal power, these consequences should be taken into account. The most important matters of concern are:

-Agrical productivity:

In the basin, the mean waterlevel increases, the tidal difference reduces. This means that the surrounding areas will have a shorter period to void their redundant water, while the groundwaterlevel is expected to raise.

-Birds.

The intertidal flats and marshlands in Cumberland Basin are considered to be important stop, cover and staging ares for migratory birds and to be significant for waterfowl production.

-Fish.

From observations of fishpopulations in the Bay of Fundy, only grown-up fish is known. Probably, this area has no breeding function. This means that the closure of a small part of the bay would not have enormous consequences for the fishpopulations. The passagepossibilities in the dam determine the fish stock in the closed off part. The running speed of the turbines is so small that fish can easily pass without getting hurt. Other passagepossibilties give the sluices. For fish swimming at the surface, a passage has to be created.

-Floods and drainage.

Within the tidal basin, three changes are expected:

1. Because of the lower high tide levels, both around the basin and within the estuaries of tributary streams, reduced flooding is expected.
2. As a result of the decrease in tidal exchange, the mixing energy in the headpond would be reduced and in some areas a gradual decline in salinity would probably be found so that iceformation would increase. Lower high waterlevels could lead to increased accumalations of raft ice on the intertidal flats at the mouths of tributary rivers, although the overall reduction in tidal area, the lower production of ice due to the reduced tides and the lack of sea ice, would all tend to counteract this effect. At the moment, in this area, ice-accumalations often cause flooding.

3. Sediments transported by the strong tidal currents tend to settle at the mouths of incoming rivers, thereby reducing the hydraulic capacity and contributing to ice-jamming. This is thought to be the principal reason for periodic flooding at the mouth of some rivers.

There are indications that, while the sediments in the bay originate from both erosion and drainage of inland areas, the large build-up of sediments at the mouths of several rivers is derived from seaward sources. This implies that a barrage would lead to reduced floodplain inundation.

2.3 Choise of the location.

An optimal locationchoise for a plant in the Bay of Fundy depends on financial,economical and social considerations and is not studied here. As subject for my study, I have chosen for smallest location, A8, in Cumberland Basin. That's big enough for me!

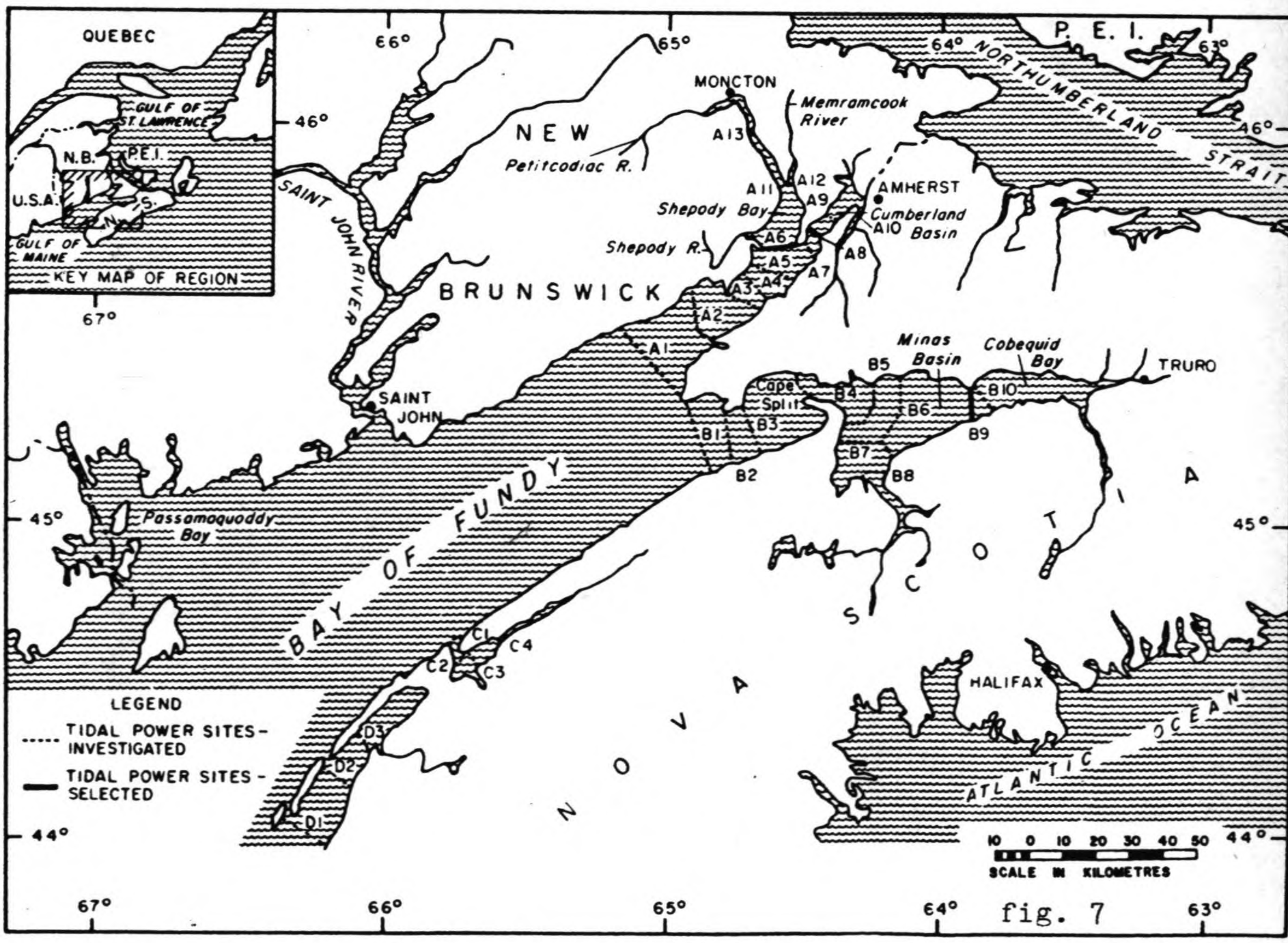
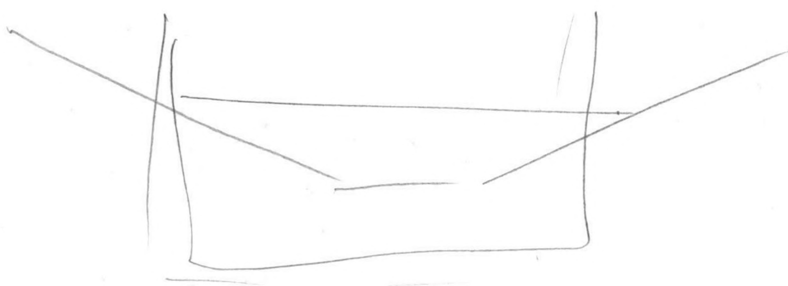


fig. 7



3. Cumberland Basin project A8.

3.1 Energy production.

The capacity of a tidal plant depends on the flow through and the head over the turbines and can be calculated with:

$$N = \eta \rho g Q H$$

with: N = capacity. (W)
Q = flow. (m/s)
H = head. (m)
η = efficiency factor.

The energyproduction can be determined by:

$$E = \int_0^t \eta \rho g H Q \, dt$$

with: E = energyproduction. (J)
t = generationperiod. (s)

This means:

$$E = \int 10 \eta Q H \, dt$$

with: ρ = 1025 kg/m³
g = 9.81 m/s²

The flow and head vary during the generationperiod. In order to be able to determine the energyproduction, the following assumptions are made:

1. A constant flow during generation.
2. A constant basinsurface for varying waterlevels.

Now the resulting energyproduction per cyclus is:

$$E_c = 10\,\eta\,Q_m\,H_m\,t \quad (kWh)$$

with: H_m = mean head. (m)
Q_m = mean flow. (m^3/s)

The mean flow can be determined by:

$$Q_m = \frac{A\,H_o\,10^6}{3600\,t} \quad (m/s)$$

with: H_o = waterleveldrop in the basin per cyclus. (m)
A = basinsurface. (km^2)

The avarage generationtime per cyclus is approximately 12 hours and 45 minutes. With 705 cycli per year, this gives a yearly energyproduction of:

$$E = 7050\,\eta\,H_o\,H_m\,A\,10^6 \quad kWh$$

H_o and H_m can be expressed in the total tidal difference Z as:

$$H_o = \alpha_1 Z \qquad \text{and} \qquad \alpha_1\,\alpha_2 \approx 0.4 \qquad \text{(ref. 3)}$$
$$H_m = \alpha_2 Z$$

The resulting energyproduction is (η = 0.8) per year is:

$$E = 2256\ Z^2\ A * 10^6\ kWh$$

mean tidal difference: 9.8 m
mean basin surface: 73 km^2

Energyproduction per year: 1230 GWh

3.2 Choise of the turbine.

3.2.1 Introduction.

Characteristic for tidal power is a large flow and a very small and varying head. Types of turbines with a high efficiency when working under a small head are the bulb, kaplan, propellor and straight flow turbine (app. 2). When working with very large flows, an axial turbinetype, this is a turbine with a horizontal shaft, is prefered: a bulb or straight flow turbine. In low head turbines, the curving of the flow is a big contributor in the total energyloss.

With the bulbturbine, the runner and generator are fitted up the horizontal shaft. The generator is placed in the bulb, usually upstream of the runner. Because the generator is located very close to the shaft, to obtain enough inertia, a big generator and consequently a big bulb is needed. This turbinetype can be fabricated with variable blades. For every position of the blades, the flow is electronically directed with the stator. In this way, the position of runner and stator can be optimized for variations in head.

In a straight flow turbine, the generator is placed at the outside, around the runner. Here the runnerblades serve as the spokes of the generatorwheel. A high inertia of the generator, but fixed runnerblades are the consequences of this design. This generatortype is cheaper than the bulbgenerator and less sensitive for netdisturbances. Because the part of the turbine, placed in the flow is smaller, less flowdisturbance takes place and smaller in- and outletworks are needed. Adjustment of the turbine for variations in head can just be realised by varying the position of the stator, the runner is fixed. This efficiencyloss could be reduced by operating the turbine for every head at its optimal velocity. Then, the connection with the net must be adjusted with rectifiers and convertors. This way of operating is proposed for the Severn Barrage; no experience is obtained with it yet.

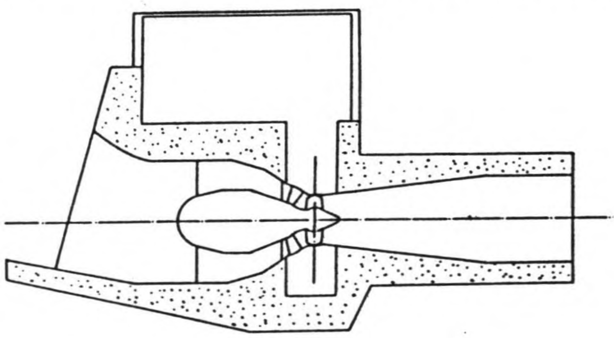
3.2.2. Turbine choice.

The variations in headdifferential of concern here are up to 40-50 % of the mean head. Thus optimizationcapacity concerning head variations is a very important aspect. For this reason, the bulbturbine would be prefered. For the costs of the civil engineering works, the intakes and outlets, the disturbance of the flow is the most important factor. That would apply for the straight flow turbine. Aswell, this turbine-type is less sensitive for net disturbances. When I first started this study, the Dutch said the bulbturbine would be best, the Canadians the straigt flow turbine. I am Dutch, so I chose for the bulbturbine.

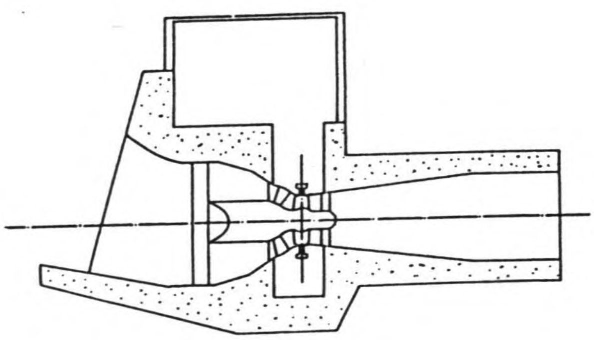
Details of the turbinetype chosen are[1] :

Rated output:	31 MW
Rated head:	6.5 m
Runner diameter:	7.5 m
Speed:	67.7 r.p.m.
Specific speed:	1148
Unit power:	33.26 MW

Of this turbinetype 39 turbines are needed.



bulb turbine



straight flow turbine

fig. 8

3.3 Generationscheme.

Chosen is for one-way generation over the ebb with use of the bulb-turbine (§ 1.3,3.2). The most important factors concerning the optimization of the generationscheme are the choise of the turbine and the expected waterlevels and headdifferentials. From studies of the Review Board is concluded that, with use of this type of turbine, optimal efficiency can be reached when the generation is started with a head of 4 meters and ended when the head is reduced to 3 meters. An estimate of the scheme, based on the waterlevels at spring tide, is made here. A constant flow through the turbines is assumed. The extreme waterlevels are corrected for the closure:

extreme waterlevels sea side: HWS: +7.50 m
LWS: -6.95 m

The waterlevelfluctuations in the Bay of Fundy devellop very regularly, like a sinus. They are schematized as:

$$h(t) = \underset{(1)}{0.3} + \underset{(2)}{7.2} \sin \omega t$$

with:
$h(t)$ = waterlevel sea side. (m GSCD)
(1) = mean waterlevel slack water. (m GSCD)
(2) = tidal waterlevel fluctuation. (m)
$\omega = \frac{2\pi}{T}$ and T = tidal period (44700 sec.)

A review of the generationscheme is given in figure 9 and appendix
The maximum head over the turbines is found when the waterleveldrop in- and outside is equal:

$$\left.\begin{aligned} &\frac{d\,h(t)}{dt}\text{ (inside)} = \frac{d\,h(t)}{dt}\text{ (outside)} \\ \text{and}\quad &\frac{d\,h(t)}{dt}\text{ (inside)} = -2.28*10^{-4} \\ &\frac{d\,h(t)}{dt}\text{ (inside)} = 7.2\,\omega\cos\omega t \end{aligned}\right\} \Longrightarrow \text{t = 17460 sec. after the generationstart}$$

resulting head: 10.2 m

The maximum head over the turbines at spring tide is 10.2 meters and takes place approximately 4 hours after the generationstart.

fase	time (s)		basin level begin	end	sea level (m GSCD) begin	end
filling	10583	2 h. 55 m.	+1.00	+7.50	+1.00	+7.50
waiting	3276	55 m.	+7.50	+7.50	+7.50	+3.50
generation	28454	7 h. 55 m.	+7.50	+1.00	+3.50	-2.00
waiting	2387	40 m.	+1.00	+1.00	-2.00	+1.00

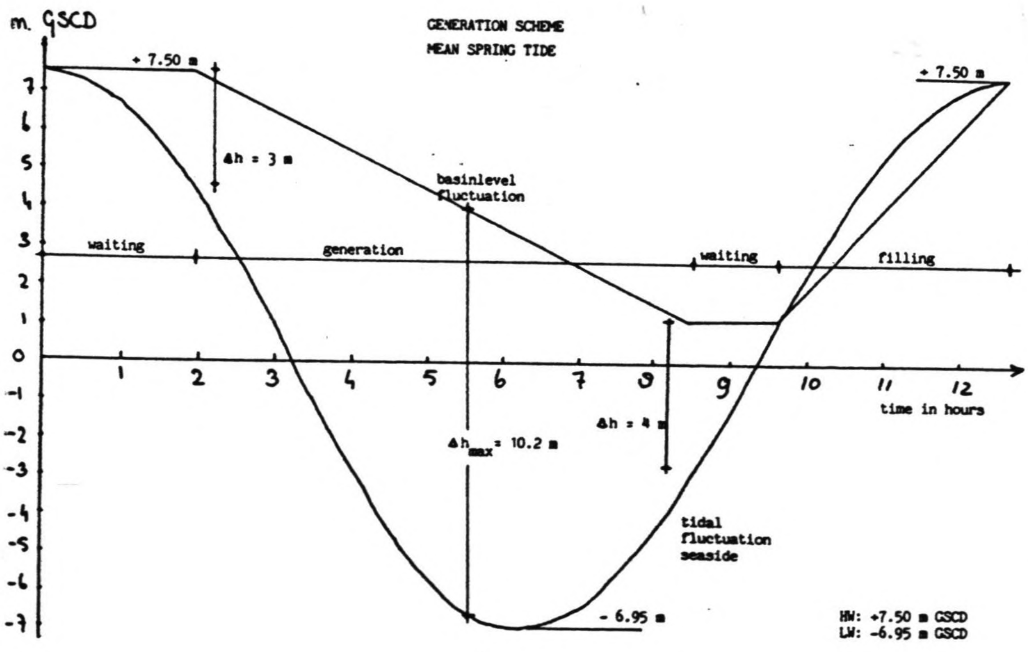


fig. 9

3.4 Lay out of the dam.

The main functions of the dam for this project are:

1. energyproduction.
2. roadconnection.

In order to be able to produce tidal energy, the following functions have to be accomplished:

- damming up of the water
- generation of the energy
- filling of the basin

The generation is proposed to take place by letting the water outoff the basin through the turbines. To fill the basin again, the turbine-inlets can be used. However, these inlets are too small to let the water in quick enough to reach the scheme as proposed. Additional inlets will have to be constructed (§ 3.6). The most usual solution would be the construction of sluices in the dam. Within this study, no alternatives are considered. To dam up the water, all inlets should be able to be closed off. The following constructions would have to be built in the dam:

1. turbines
2. sluices
3. other constructions closing off the closuregap.

In this part of the Bay of Fundy, no navigation is found.

The flow outoff the turbinesections during generation and outoff the sluices during the filling fase is expected to spread under an angle of approximately 1:20. When the individual turbinesections and the individual sluices are placed close to each other, the individual flows will stabilize each other. If not, the flow is expected to start start twistling, with the result that a lot more foundationprotection-material would be needed. Thus installation of the individual turbine-

sections and the individual sluices, aswell as the complete sluice-section and turbinesection, as close to each other as possible, would be prefered . For cavitationconsiderations, the turbinecaissons would be placed in the deepest parts of the cross section.
In these enormous projects, interest is a big contributor in the overall costs . For that reason, in order to be able to start generating within the quickest time, the rest of the closure would preferably be done with caissons as long as possible (prefabrication assumed § 3.5). This possibility depends on the length of the slack water period, which decreases as the gap becomes narrower.
The inter tidal zone would probably be closed from the shore inwards.
Schematically, the following cross section is proposed:

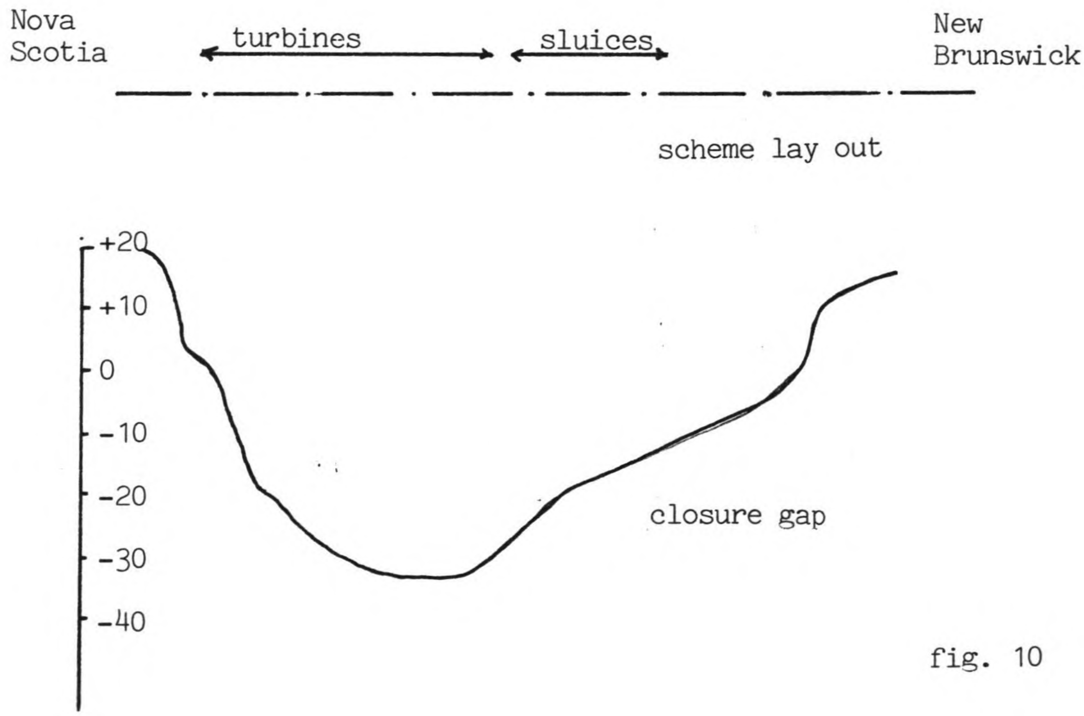
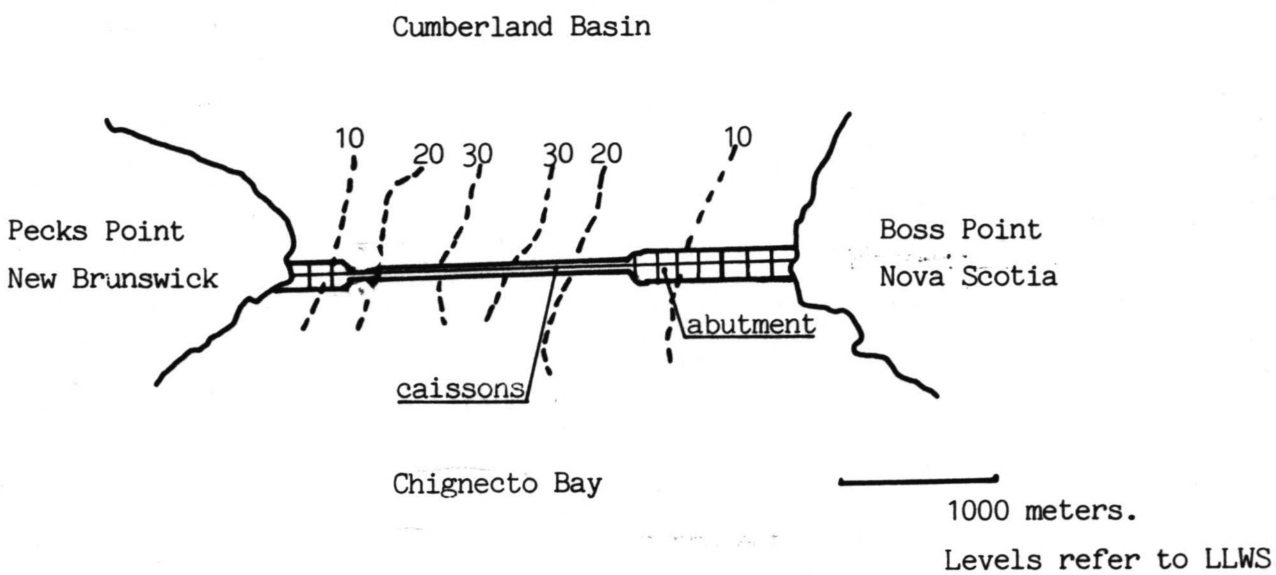


fig. 10

The construction of the dam preserves a good facility for a road-connection between New Brunswick and Nova Scotia. If the dam would be built, a road would have to be put on top.



Scheme of the lay out of the Cumberland Basin project.

3.5 Construction method.

First, the choise has to be made between prefabrication and construction at the location itself.

Contruction at the location itself.

When constructing at the location itself , the work could be completed in two fases. For the closuregap of concern here (fig.10), first a buildingpit would be constructed in the deepest part near the Nova Scotia shore. During that constructionfase, the flow would have to be transferred to the New Brunswick shore. This implies an enormous impact on the environmental system. With the generation could only be started when the last part of the buildingpit is removed. The construction of the buildingpit actually implies an extra closure to to be made. At the La Rance tidal power plant, the costs of the buildingpit were 30% of the overall costs. However, construction of the plant, bottomprotection and other surrounding works is a lot easier than with use of prefabrication.

Prefabrication.

With use of prefabrication, the caissons would be built in one or more buildingpits, transported and sunk to their final place. Eventually, the finishing works could be done. With the generation could be started as soon as the closure is finished. With use of this method the buildingpit(s) can be made in an area where the currents are much lower. However, transport and placement of the caisson, construction of the bottomprotectionworks and the foundation, could be very difficult now.

Both methods have a lot of advantages and disadvantages. Within this study, prefabrication is assumed.

3.6 Impression overall flowpattern during generation.

In order to get an impression of the overall flowpattern when the plant is working, an estimate of the open scross section of sluices and turbines, needed to reach the schedule as proposed, is made.

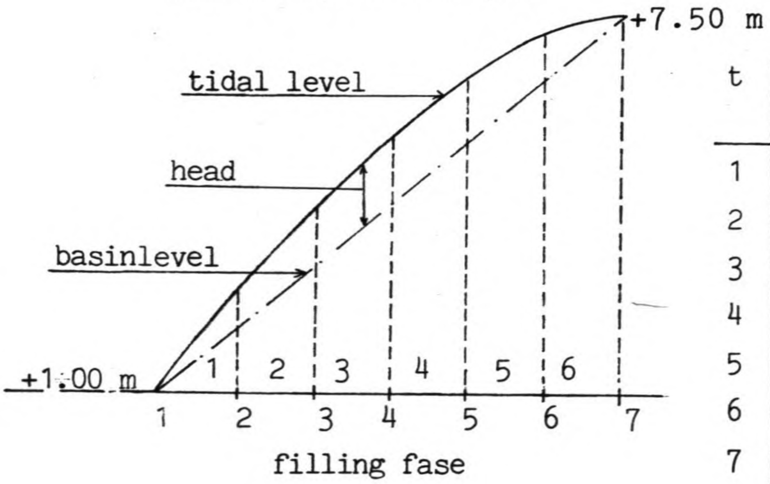
Calculation.

During the filling fase, the basin is filled through the inlets of the turbines and sluices.The cross section needed can be estimated with the following formula:

$$V = (\mu_T A_T + \mu_s A_s)\sqrt{2g}\int_0^t \sqrt{\Delta h}\, dt$$

with: V = volume of water. (m^3)
A = cross section. (m^2)
μ = flow coëfficient.
T = turbine.
s = sluice.
Δh = head. (m)

A flowcoëfficient of 1.6 for the turbines and 1.2 for the sluices is assumed. The total inflow during the filling fase is calculated numerically in 6 steps:



t	h_{inside}	$h_{outside}$ (m)	Δh
1	1.00	1.00	0.00
2	2.08	2.72	0.64
3	3.17	4.30	1.13
4	4.25	5.63	1.38
5	5.33	6.65	1.32
6	6.24	7.30	0.88
7	7.50	7.50	0.00

The mean head per section is:

section	1	2	3	4	5	6
head (m)	0.32	0.89	1.26	1.35	1.10	0.44

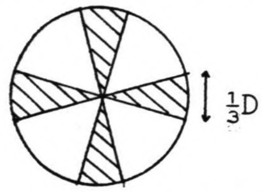
$$\left.\begin{array}{l}\Delta h_{total} = 5.36 \text{ m} \\ dt = 1747 \text{ s.}\end{array}\right\} \Rightarrow \sum_{1}^{6} \sqrt{\Delta h}\, dt = 4042$$

The total volume of water needed is:

$$\left.\begin{array}{ll}\text{total head:} & 6.5 \text{ m} \\ \text{basin surface:} & 73 \text{ km}^2\end{array}\right\} \Longrightarrow V = 47.5*10^6 \text{ m}^3$$

During the filling fase the turbines are placed in their vane position. The net cross section per turbine is :

$$A_T = \tfrac{1}{4}\pi D^2 - \tfrac{1}{4}\pi(\tfrac{1}{3}D)^2 = 39.3 \text{ m}^2$$



The total cross section of the turbines is: $A_T = 1490 \text{ m}^2$

The resulting cross section of the sluices is: $A_s = 7830 \text{ m}^2$

During the filling fase, approximately 16 % of the flow is guided through the turbines, 84 % through the sluices. The resulting mean velocities in the turbine- and sluicesections are:

during generation:	1.1 m/s
during filling:	0.5 m/s

PART TWO

DESIGN TURBINE CAISSON

(All levels refer to GSCD.)

4.1 Designprinciple .

When using the ebbschedule, HW in the basin usually is assumed to be at the HW sea level; LW a little above the mean water level. In studies of the Review board, the following levels were proposed:

HW basin: +7.50 m
LW basin: +1.00 m

To generate energy, a head over and a flow through the turbines is needed. With use of ebbgeneration, the head is obtained by damming up the water from basin side and is determined by the hydrostatic waterpressuredrop over the turbine; waves don't influence the head. Thus, for generationpurpose, waveovertopping can be allowed. Waveovertopping from sea side fills the basin and could be a gain of energy. Waveovertopping from basin side is a loss of basin water and thus a loss of energy. To determine the turning height of the dam, the following criterion is used:

Waveovertopping from basinside allowed once per year.

The resulting turning height from basin side is:

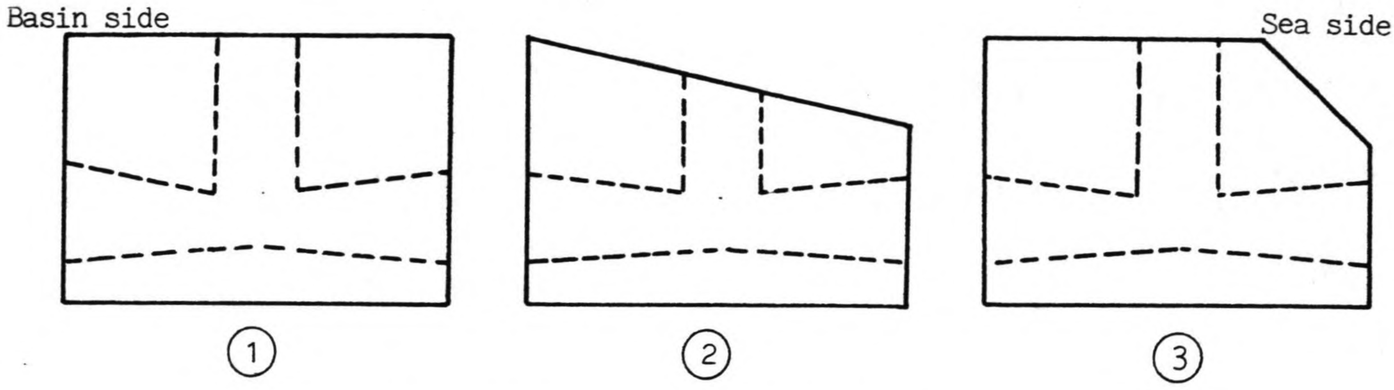
HW basin:	+7.50 m
one year design wave basin side:	2.00 m
turning height:	+9.50 m

In most designs, usually no waveovertopping is allowed. In that case, standing waves devellop against the vertical face of the caisson. Now at sea side, the complete standing wave, $H_{1\%}$ = 12.1 m, has to be turned. An extra caissonheight of approximately 15 meters would be needed. A turning height from basin side of +9.50 m GSCD is taken as designcriterion.

The roadconnection would have to be constructed on piles on top.

4.2 Alternatives.

The following alternatives are considered:



The choise is based on the following considerations:

Construction: During construction, placement of the turbines and other works on top of the caisson would be done. A horizontal deck is prefered.

Maintenance: For maintenancepurposes, admittance to the turbine at every moment should be possible. Perhaps an entrance in the upper deck or the bridge pile could be used. When the turbine has to be lifted out, the hatch of the shaft has to be removed. In order to be able to carry the wavepressures of breaking waves on top, a heavy hatch is needed and probably the same crane as used to lift out the turbines, would be used. This usually is a mobile gantry. Turbine and hatch are heavy. Preference would be given to a horizontal upper deck.

Ice accumalation: With alternative 2, ice accumalation is expected on top of the caisson from sea side. In that situation, admittance to the shaft and turbine might be blocked. Aswell ice could give enormous loading on top of the caisson. Both should be prevented.

Wave reflection: With alternative 2 and 3, the waves from sea side are only partly reflected. This means a reduction in waveloads of approximately 20 % is expected. Waves from sea side, coming from the Bay of Fundy and the Gulf of Maine,have a long wavelength. At LW sea side, the head over the turbine is maximum and the maximum amount of energy is produced. Then, these waves will introduce pressuredisturbances in the turbine outtake, which will influence the efficiëncy of the turbine a lot. In front of alternative 2 and 3, the waves are only partly reflected and a slight decrease in pressuredisturbance might be expected. Perhaps a better solution for this problem would be the construction of a hybrid wall, as proposed by the Tidal Power Consultants, Montreal, in their design for the Cumberland Basin plant, 1985.

Chosen is for alternative 1, with a horizontal upper deck.

4.3 Dimensions needed for the in- and outtake.

The minimum dimensions of the in- and outtake depend on the type of turbine. For a bulbturbine with a runnerdiameter of 7.5 meter, the following dimensions are needed:

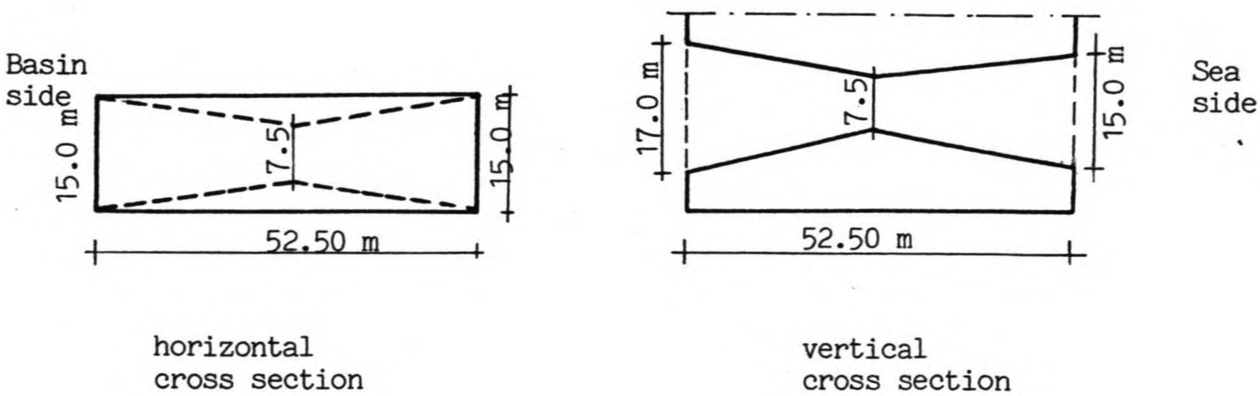


fig. 11 Dimensions in- and outtake

4.4 Foundationdepth.

For the foundationdepth of the turbines, the following has to be considered:

1. Cavitation.
2. Waves running into the in- and outlets of the turbines.

4.4.1 Cavitation

Cavitation is a result of underpressure behind the blades of the turbine. Usually, the waterlevels at the intake are considered to determine the cavitationconditions. However, when big watervelocitydifferences in the intake are concerned, the waterpressuredrop here,has to be taken into account.

The depth needed to prevent cavitation can be calculated with the following formula:

$$H_s \geqslant B - \sigma * H$$

with:

- B = barometric pressure at sealevel
- H_s = leveldifference between the highest point of the turbineblades and the lowest waterlevel.
- H = waterpressure at the turbinelocation
- σ = cavitation coefficiënt

The σ- value can be found by extrapolation of the added Kaplangraph (app. 3).

Calculation.

In the intake of the turbine the watervelocity increases. This means that, when no turbulence at the edges of the intake occurs, the loss of energy in the intake can be neglected and the Bernouilli equation can be used:

$$H = z + h + \frac{u^2}{2g} = \text{constant}$$

with:
- H = energy level. (m)
- z = reference level (GSCD)
- u = watervelocity. (m/s)
- h = waterpressure. (m)
- g = gravitational acceleration. (m/s^2)

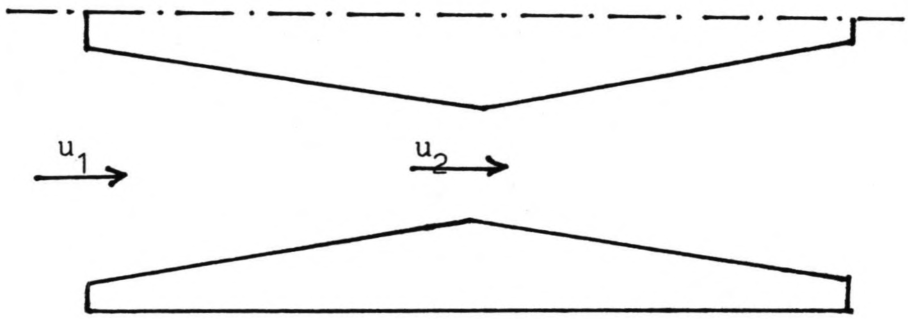


fig. 12

Data: $Q = 428\ m^3/s \Longrightarrow u_1 = 1.7$ m/s

$u_2 = 10.9$ m/s

LW Basinside: +1.00 m

From the Bernouilli equation follows a waterpressure at the place of the turbine of - 4.90 m GSCD.

specific speed turbine (n_s): 1148 $\Longrightarrow \sigma = 1.8$ (app. 3)

barometric pressure: 10 N/m^2

$$\Longrightarrow \quad H_s \leqslant 10 - 1.8 * 10.2 = -8.4 \text{ m.}$$

The foundationdepth needed to prevent cavitation is:

$$-4.90 - 8.40 - 0.5(7.5 + 17.0) = -30.65 \text{ m GSCD.}$$

4.4.2 Waves in in- and outlets.

When waves are running into the in- and outlets, air can be closed in. This can cause very high pressurewaves and should be avoided by placing the in-and outlets underneath the wavevalley. The design waterlevels and wavehights and the needed levels of the in- and outlet are:

Basinside:

winddirection	waterlevel	waveheight $H_{1\%}$	upperside inlet
NE	+1.60	4.4	-1.40
SW	+0.15	0	0.15

Seaside:

winddirection	waterlevel	waveheight $H_{1\%}$	upperside outlet
NE	-8.30	0	-8.30
SW	-4.95	15.6	-10.75

fig. 13

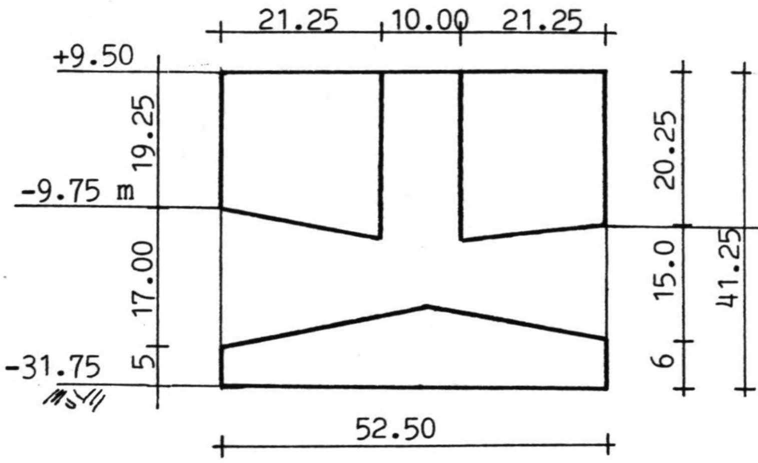
To avoid pressurewaves, an outletlevel of -10.75 m GSCD is needed. The foundationdepth needed is -31.75 m GSCD.

4.4.3 Conclusion

The incoming waves from seaside determine the foundationdepth. A level of -31.75 m GSCD is chosen.

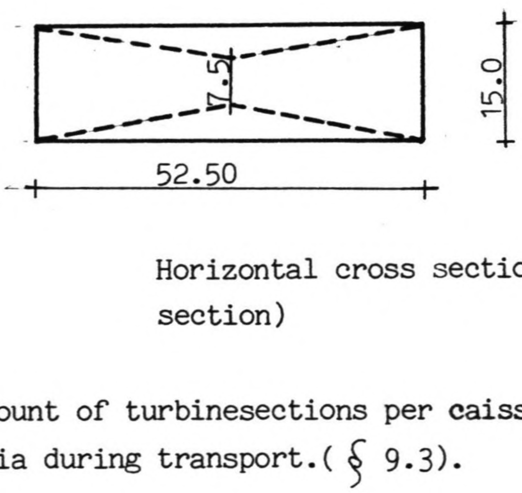
4.5 Estimate caisson dimensions.

The resulting minimum caissondimensions are:



(all dimensions in meters)

fig. 14 Vertical cross section turbine caisson



Horizontal cross section turbine caisson (one turbine section)

The amount of turbinesections per caisson depends on the stability-criteria during transport.(§ 9.3).

4.6 Some efficiëncy considerations.

Loss of basin water takes place as a result of leakage underneath the dam and waveovertopping.

4.6.1 Leakage underneath the dam.

Leakage underneath the dam can be estimated with the following formula:

$$q = k \frac{dh}{dx}$$

with: q = flow velocity. (m/s)
k = permeability coëfficiënt. (m/s)
dh = head. (m)
dx = length of the flow. (m)

From tests of the Tidal Power Review Board, the permeability of the bed proves to be $1*10^{-7}$ - $7.5*10^{-7}$ m/s.

For this estimate, no sealing of the sub soil is assumed.

Calculation.

Mean head during generation:	6.2 m
Length of the flow:	52.50 m
Depth of the flow:	20 m
Length of the dam:	2500 m
Duration of the generationcyclus:	28500 sec.
Permeability of the ground:	$7.5*10^{-7}$ m/s

Loss of basin water per cyclus: 126 m^3

A loss of 126 m^3 water will not be noticed on a basinarea of 73 km^2.

4.6.2 Waveovertopping.

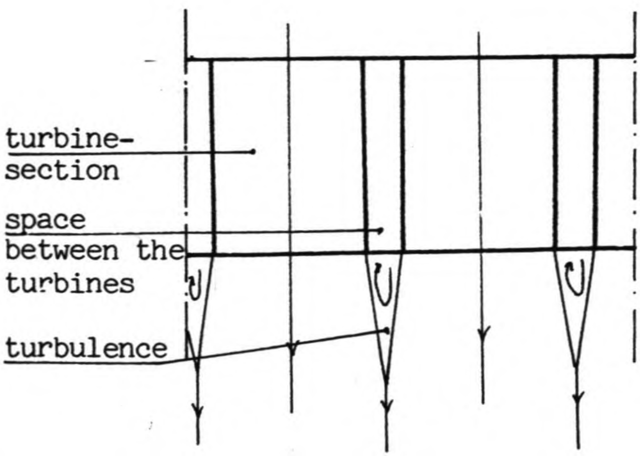
During the wintermonths, a southwesterly wind prevails. In storms, waves from sea side will overtop the dam and fill the basin. This would have an efficiëncy increasing effect. In summertime, northwesterly winds prevail. With a damheight of +9.50 m GSCD, with a frequency of once per year, during storms, waves will overtop. On the yearly output, this would be neglectible.

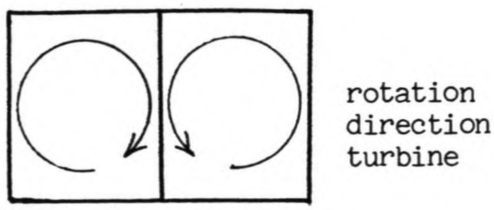
4.6.3 Disturbances in the turbine outtake as a result of wave-influences.

Incoming waves from sea side are generated in the Bay of Fundy or even in the Gulf of Maine. Consequently, they will have a long wavelength, which will cause movements in the water on high depths. This means, around LW, these waves will cause pressuredisturbances in the turbine outtake, which could influence the efficiëncy of the turbine a lot. At LW, the head is at its maximum and the maximum amount of energy is produced. Some solutions for this problem are mentioned in §4.2

4.6.4 Position of the turbine sections and rotationdirection of the turbines.

In order to decrease the turbulence in the outflow as much as possible, the turbinesections would be placed as close to each other as possible. Perhaps letting the next-door turbines rotating in each other opposite direction would have a turbulence decreasing effect aswell.





horizontal cross section | fig. 15 | vertical cross section

Loads.

The caisson is loaded by the hydrostatic loads resulting from the waterleveldifferences over the caisson, by wave and ice loads. The hydrostatic loads and the waveloads are determined in § 5, the ice loads are studied in § 6.

5. Hydraulic loads.

5.1 Introduction.

The hydraulic loads on the dam are caused by waterleveldifferences and waves. The calculations are based on the waterfluctuations as given in the generationscheme (app. 4). The extreme mean waterlevels are:

basinside		seaside	
LW	+7.50	HHWS	+7.50
HW	+1.00	LLWS	-6.95

fig. 16

Metereological variations of these waterlevels are calculated with the use of windmeasurements of stations in the surrounding of the Bay of Fundy. A review of the windspectrum resulting from these measurements is given in figure 17 . The designcalculations are based on a failurefrequency of 10^{-4} in a period of 100 year.

Period (year)	Winddirection (km/h) NE	SW overland	SW oversea
50	106	111	133
100	113	116	139
200	119	121	145
500	129	129	155
1000	135	135	162

fig. 17 Windspectrum Bay of Fundy

5.2 Drive up due to windinfluence.

The friction of wind along the watersurface causes a variation of the watersurface. This can be calculated as explained in figure 18.

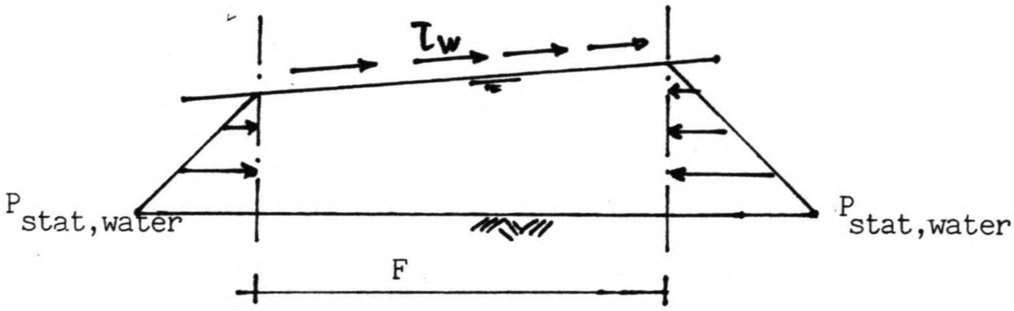


fig. 18 Balance between the resulting hydrostatic water-pressure and the windfriction.

In this equilibrium, bottomfriction can be neglected. Balance between the hydrostatic pressure and the windfriction gives the following equation:

$$P_w = \rho_1 v_w^2 \, c_1 F = 0.5 \rho_w \, gd^2 - 0.5 \rho g(d + S)^2$$

(1) (2)

with:

P_w	=	windpressure	(kNm^{-2})
v_w	=	windvelocity	(ms^{-2})
F	=	length of windfriction	(km)
ρ_w	=	density of water	(kNm^{-3})
ρ_1	=	density of air	(kNm^{-3})
(1)	=	windfriction	
(2)	=	static pressure	

This equation results in:

$$S = C * \frac{v_w^2}{gd} * F \quad (m) \qquad \text{with } C = 4*10^{-6}$$

As can be concluded from the map in app. 1 wind from northeasterly and southwesterly directions will cause the highest drive up. The designwindvelocities from these directions are:

NE	v_w = 31.4 m/s		
SW	v_w = 38.6 m/s	freq.: 10^{-2}/year	

In the bay, the mean depth is 75 meter. Here the wind can drive up the water over a distance of more than 250 kilometers. In the basin the mean depth is 12 meter and windfriction can devellop over only 12 kilometers. The results of the calculations of the drive up of water due to windfriction are given in figure 19 .

wind		drive up (m)	
direction	velocity (m/s)	basin side	sea side
NE	31.4	+0.60	-1.35
SW	38.6	-0.85	+2.00

fig. 19 Waterlevelvariations due to windfriction

The designloads are based on the extreme waterlevels. The resulting designwaterlevels are:

Basinside:	maximum	+8.10 m GSCD
	minimum	+0.15 m GSCD
Seaside:	maximum	+9.50 m GSCD
	minimum	-8.30 m GSCD

5.3 Waves.

5.3.1 Introduction.

Besides drive up of the waterlevel, friction of the wind over the watersurface has another influence, waves. On deep water, with a winddirection perpendicular to the coast, the significant waveheight, H_s, and period T_s, only depend on the windvelocity, the length of the windfriction and the gravitational accelleration. Formula's for these relations are:

$$\frac{gH_s}{u^2} = 0.282 \tanh \left(0.0125^* \left(\frac{gF}{u^2}\right)^{0.42}\right)$$

$$\frac{gT_s}{u^2} = 7.54 \tanh \left(0.077^* \left(\frac{gF}{u^2}\right)^{0.25}\right)$$

Grafically, these formula's given in app. 5 . These formula's can only be used for deep watercircumstances. By the approach of shallow water, two other fenomina occur:

1. shoaling
2. refraction

In the following paragraphs, these fenomina are discussed.

5.3.2 Shoaling.

The wavevelocity depends on the waterdepth:

$$c = \frac{L}{T} = \frac{gL}{2}\sqrt{\tanh\frac{2\pi d}{L}}$$

with:	c =	wavevelocity	(m/s)
	L =	wavelength	(m)
	T =	waveperiod	(s)
	d =	waterdepth	(m)

On deep water, the formula can be simplified to:

$$c = \sqrt{\frac{gL}{2\pi}} \qquad \text{deep watercriterium:} \quad d \geqslant 0.5L$$

In shallow water, the following equation has to be used:

$$c = \sqrt{\frac{gL}{2} * \frac{2\pi d}{L}} = gd \qquad \text{shallow watercriterium:} \quad d \leqslant 0.25L$$

From the equations, mentioned above, can be concluded that the waveheight will not be influenced as long as the waterdepth is big in relation to the wavelength ($d > 0.25L$). In shallow water however, the wavevelocity decreases and the short wave from the deep water will start behaving like a long wave. By approaching the coast, the ammount of energy of a group of waves is:

$$U = E^* c_g = E^* c^* n \qquad \text{per m' wavecrest}$$

The power of a group of waves approaching the coast remains constant, just like its waveperiod. The wavevelocity decreases and the waveheight increases. The balance between the wave-energy in deep and shallow water gives:

$$\underbrace{\frac{1}{8}\, g\, H_o^2\, c_o\, n_o}_{\text{energy deep waterwave}} = \underbrace{\frac{1}{8}\, g\, H_1^2\, c_1\, n_1}_{\text{energy shallow waterwave}}$$

As follows from this equation, the relation between the waveheight in deep and shallow water can be determined with the shoalingcoëfficiënt:

$$K_{sh} = \frac{H_1}{H_o} = \frac{c_1}{c_o} * \frac{1}{2n}$$

For shallow water, the coefficient can be simplified to:

$$K_{sh} = 0.2821\sqrt{\frac{L}{d}}$$

In reality, the waves, which are approaching the coast, will break before they reach an infinite height. The breakingcriteria for waves are:

1. $\frac{H}{L} = 0.142$ (maximum healing)
2. $\frac{H}{d} = 0.7$ (waterdepthcriterium)

This is considered in the calculations in § 5.3.4.

5.3.3 <u>Refraction.</u>

Generally, waves don't approach perpendicularly to the coast. When a wavespectrum approaches a coast, with depthlines parallel to the coast, with an angle smaller than 90^o to the coast, the wavevelocity nearer to the coast will decrease and the wavecrests will turn in the direction of the coast (fig. 20). By assuming that the wave-energy of a wavecrest between two orthogonals remains constant, the effect of refraction on the waveheight can be calculated.

$$U_1 b_1 = U_2 b_2 \Longrightarrow E_o n_o c_o b_o = E_1 n_1 c_1 b_1 \quad \text{and}$$

$$E = \frac{1}{8} \rho g H^2$$

The resulting refractioncoëfficiënt is:

$$K_r * K_{sh} = \frac{H_1}{H_o} = \sqrt{\frac{1}{2n} * \frac{c_o}{c_1} * \frac{b_o}{b_1}}$$

The angle with the coast can be calculated with Snell's law:

$$\frac{\sin \varphi_o}{\sin \varphi_1} = \frac{c_o}{c_1}$$

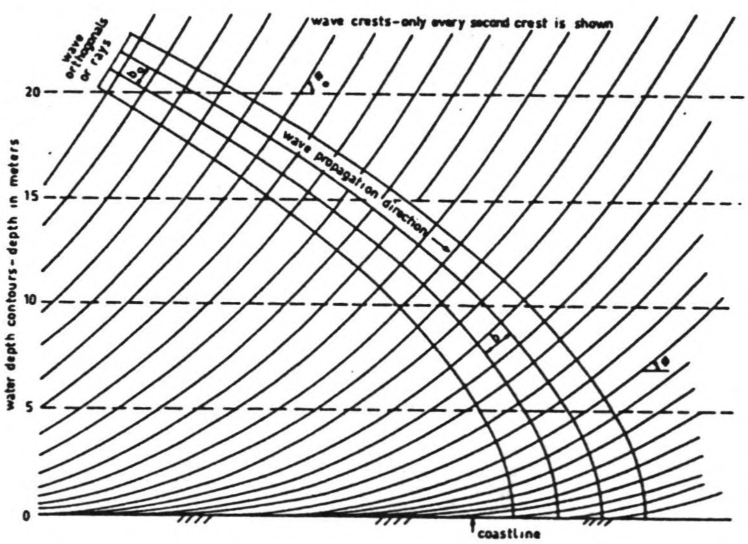


fig. 20

App. 6 shows the wavepropagation in the Bay of Fundy from winds from southwesterly directions. Hardly any refraction will take place. Refraction is neglected in the wavecalculations.

5.3.4 Calculation of the design waveheights.

Waves on the basinside are caused by winds from northeasterly directions. The fetchlength of the wind is 17 kilometers. The basindepth increases regularly, with a mean depth of 12 meter. For the wavecalculations, a mean depth of 12 meter over the whole length of the basin has been assumed. Waves from seaside are generated in Fundy Bay and Chignecto Bay by winds from southwesterly directions over a fetchlength of more than 250 kilometers. The mean depth in Fundy Bay is 75 meter. In Chignecto Bay, the depth decreases quickly up to a mean depth of 25 meter at the location of the dam. The waves from seaside are corrected for shoaling. The waveheight is determined with the Pierson Moskowitz spectrum (app.6)

Design waveheight basin side:

$$\left.\begin{array}{l} F = 17 \text{ km} \\ d = 12 \text{ m.} \\ v_w = 31.4 \text{ m/s} \end{array}\right\} \Longrightarrow H_s = 3.8 \text{ m} \Longrightarrow H_{1\%} = 4.4 \text{ m}$$

Design waveheight sea side:

$$\left.\begin{array}{l} F = 250 \text{ km} \\ d = 75 \text{ m} \\ v_w = 38.6 \text{ m/s} \end{array}\right\} \Longrightarrow H_s = 11.0 \text{ m}$$

correction for shoaling:

$L_{shallow} = \sqrt{gd} * T = 469$ m ($\frac{d}{L} = 0.16$ -shallow water assumption correct)

HW:	$d = 42.0$ m	$K_{sh} = 0.96$
LW:	$d = 24.2$ m	$K_{sh} = 1.24$

HW:	$H_s = 10.5$ m	$H_{1\%} = 12.1$ m
LW:	$H_s = 13.6$ m	$H_{1\%} = 15.7$ m

5.4 Loads from waterleveldifferences and waves.

A scheme of the loads, resulting from hydrostatic pressuredifferences on both sides of the dam is given in figure 21.

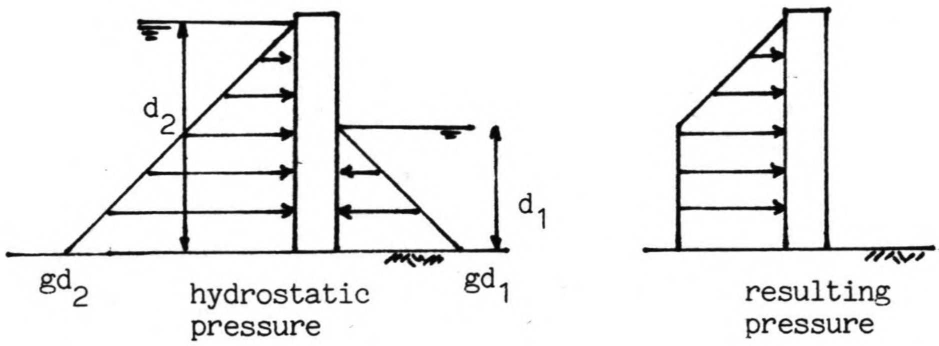
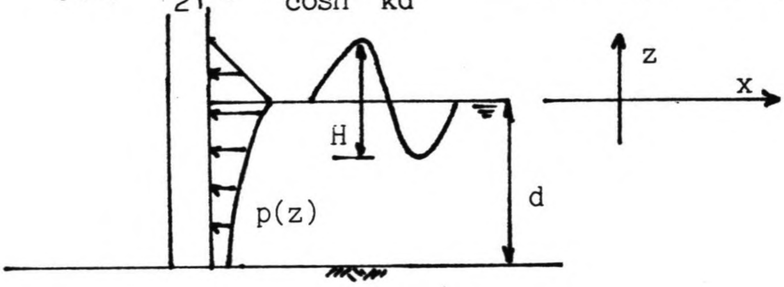


fig. 21

Waves give dynamic forces on the dam. The pressures, resulting from a wave, reflected by a vertical wal, can be calculated with the following formula:

$$p(z) = \frac{1}{2}\rho g\, H \frac{\cosh k(\, z + d\,)}{\cosh\ kd} \cos\,(\omega t - kx\,)$$



pressure under a wave fig. 22

When a wave is reflected by a vertical wall, the waveamplitude is doubled ; a standing wave devellops.

To determine the designloads, the following situations are considered:

1. wind NE basin HW: 7.5 + 0.6 =+8.1 m GSCD
   sea LW: -6.95-1.35=-8.3 m GSCD
   wave basinside: $H_{1\%}$ = 4.4 m

V

2. wind SW basin HW: 7.50 - 0.85 = + 6.65 m GSCD
sea LW:-6.95 + 2.00 = - 4.95 m GSCD
wave seaside: $H_{1\%}$ = 15.7 m

3. wind SW basin LW: 1.00 - 0.85 = +0.15 m GSCD
sea HW: 7.50 + 2.00 = +9.50 m GSCD
wave seaside: $H_{1\%}$ = 12.1 m

Waveconditions for these situations are:

situation	d	T	$H_{1\%}$	L	kd	d/L	cosh kd
1	39.8	7.0	4.4	76	3.3	0.50'	3.5
2	26.8	17.3	15.7	284	0.6	0.12"	1.29
3	41.2	17.3	12.1	350	0.8	0.10'	1.19

" shallow water: $L = \sqrt{gd} * T$
' deep water: $L = gT^2/2\pi$

The resulting hydrostatic loads are:

situation	horizontal load (kN/m')	arm (m)	moment (kNm/m')
1	5236	16.3	84470
2	3798	16.4	61514
3	3434	18.4	63203

The waves are only partly reflected when they overtop the crest. A reflectioncoëfficiënt of 0.8 is assumed. For total reflection, the coëfficiënt is 1.0. The resulting waveloads are:

situation	horizontal load (kN/m')	arm (m)	moment (kNm/m')	reflection coëfficiënt
1	770	26.1	20107	0.8
2	2309	10.2	23550	1.0
3	1555	17.1	26594	0.8

The total wave- and hydrostatic loads are:

situation	horizontal load (kN/m')	excentricity (m)	moment (kNm/m')	wave reflection
1	6006	17.4	104577	0.8
2	6684	13.7	92147	1.0
3	4989	18.9	89797	0.8

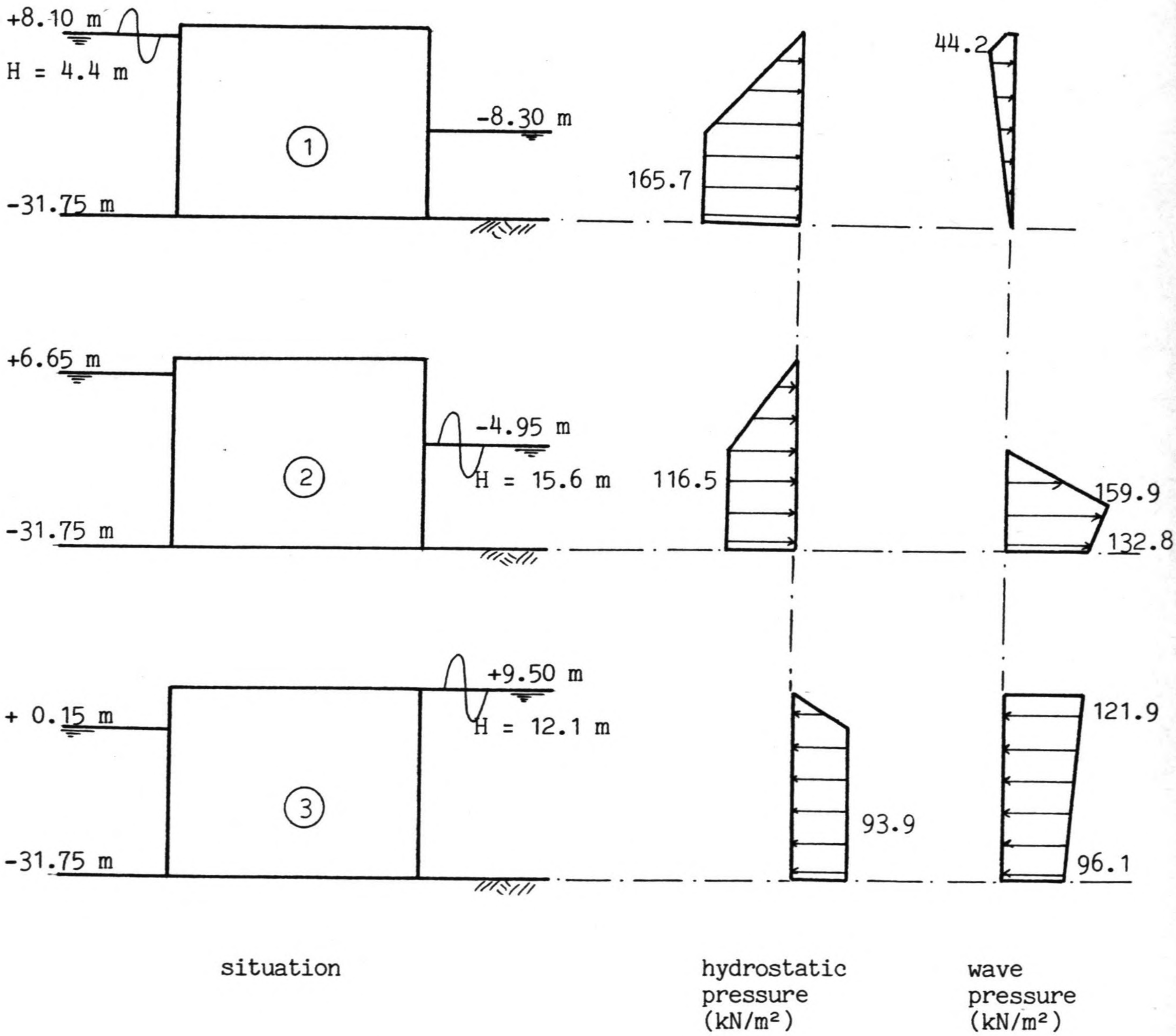
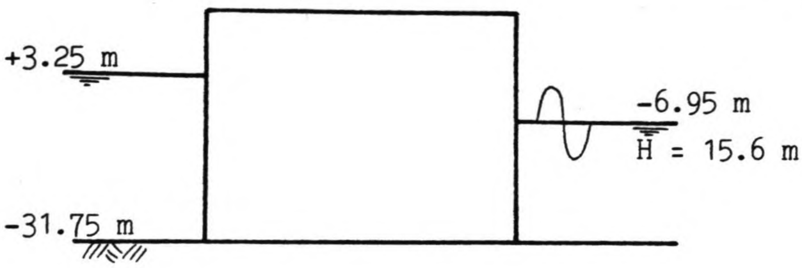


fig. 23

5.5 Some considerations concerning the approach of the design-criteria.

The designcriteria used here actually never take place. When the plant is working, the maximum waterleveldrop over the caisson is 10.2 meters. (app. 4). The hydrostatic pressuredifference, designed for here, is 16.4 meters and can only devellop when the plant is outoff order and the basin water is at its highest level. When the plant is not working, to prevent the extreme waterleveldifference of 16.4 meters, some basin water would have to be discharged. That could be forgotten. However, the chance that, the plant is outoff order, discharging of basin water is forgotten and the design wave height from sea side devellops, all take place at the same moment, is much smaller than the designcriterion, 10^{-4}. Perhaps the following designcriteria would be better:

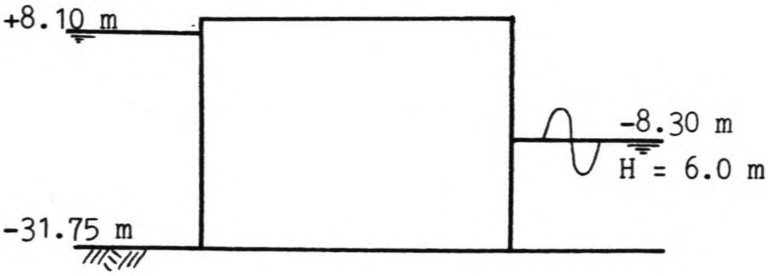
1. designconditions when the plant is working:



maximum waterleveldifference during generation: 10.2 m
100-year design wave sea side: 15.6 m (LW)

	hydrostatic loads	wave loads	resulting design loads.
F (kN/m')	3000	2309	5309
M (kNm/m')	51494	23550	75044
z (m)	17.1	10.2	14.1

2. designconditions when the plant is outoff order:



maximum waterleveldifference: 16.4 m
1-year design wave sea side: 6 m

	hydrostatic loads	wave loads	resulting design loads
F (kN/m')	5236	980	6216
M (kNm/m')	84470	12250	96720
z (m)	16.3	12.5	15.5

Now, the designconditions would be determined by the maximum water-leveldrop, when the plant is outoff order combined with the one-year design wave height. The resulting designloads are:

$$F_x = 6216 \text{ kN/m'}$$
$$M = 96720 \text{ kNm/m'}$$
$$z = 15.5 \text{ m}$$

This gives a reduction of 8 % in the designloads. Ofcourse this approach depends on environmental considerations aswell.

To determine the caissondimensions, the designcriteria as mentioned in § 5.4 are used.

6. Iceconditions and ice loads.

6.1 Description of the iceconditions in Cumberland Basin and Chignecto Bay.

From Januari to early April, much of Cumberland Basin is covered with drift ice and its shoreline with an icefoot. Each year, the conditions vary a lot. In these months, basically three modes of ice occurrence are observed:

1. drift ice.
2. an icefoot along the shore.
3. a frozen icecrust.

The iceconditions are made up from icereports of Icecentral Halifax (1969) and observations of local inhabitants.

Drift ice.

The drift ice cover increases through early Januari and mid-Februari, reaching a maximum in late Februari. Hardly any time, the field area becomes frozen over with a continuous unbroken ice cover. The tidal currents break up the large icefloes and keep the blocks in continual motion. Winds add a variable cross-bay component to the drift ice motions, which can produce enormous differences in foreshore zone appearence from day to day. Four distinct types of drift ice can be distinguished:

- Slush ice.

Slush ice is composed of an unconsolidated mass of ice crystals and water with no definite shape or structure. The individual ice crystals are derived from the initial freezing stages of seawater, snowfall and erosion of iceblocks by collisions during transport.

- Pan ice.

Pan ice is approximately 10 to 15 cm in thickness and may reach up to 10 m across. It has a smooth upper surface and forms from the freezing of surface waters in the sheltered embayment and estuaries. Individual pieces of pan ice are broken off by the rise and fall of the tide and are redistributed by the currents and winds.

- Cake ice.

Cake ice forms as independent, near- equidimensional ice blocks, commonly less than 1 m across, from the direct accretion of seawater onto smaller pieces of drift ice. This ice usually has an elliptical base and rimmed top edges, which result from collisions with other blocks of drift ice and from grounding out at low tide.

- Composite ice.

Composite ice is built up from all types of ice present and forms from freezing together of small pieces of ice. Subsequent enlargement occurs by rafting of other blocks onto its surface or the direct accretion of sea water and snowfall. The size and shape of the composite ice is variable, but it generally occurs as semi-equant blocks, which range up to more than 5 m in thickness. The keel of these blocks usually is somewhat elliptical as a result of collisions or grounding.

Any type of drift ice can become stranded in the intertidal zone as the tide drops. Most blocks float off as the tide rises again, but some become frozen to the bottom during the low tide period and cannot be lifted by the rising water. Ice that has been anchored to the bottom in this manner is called rooted ice. Blocks of rooted ice grow by accretion of seawater and may achieve sufficient buoyance to break free eventually, often taking some of the substrate with them. The formation of rooted ice is most common in the higher parts of the intertidal zone, where blocks are stranded for a longer period and on the supratidal marshes, where they are left behind during the spring tides. This drift ice contains a wide variety of sediment sizes, ranging from clay up to cobbles and larger.

Icefoot along the shore.

The icefoot begins to form along the high tide level in late December and remains throughout the winter. It grows during periods of subfreezing air temperatures through the combined accretion of wave-spray and wave wash, snowfall and overtopping by the high spring tides. Also drift ice floats onto the icefoot and becomes stranded during the high spring tides and/or onshore storms producing an irregular and hummocky surface. The ice is relatively dense due to the repeated cycles of freezing and thawing. Throughout the whole winter the icefoot remains attached to the bottom and can therefore be considered as shorefast. The height and width of the icefoot is related to the foreshore slope and the general shoreline morphology. On the intertidal shores, with relatively shallow gradients, an icefoot of 1.5 m to 2 m height and 10 to 30 m across forms. On the truncated edges of saltmarsh deposits,the icefoot reaches 4 m in height. The greatest thickness of shorefast ice forms on the bedrock headlands, where rocky edges are exposed at low tide and the intertidal foreshore is relatively steep. In these areas, the icefoot can reach heights up to 9 m with a width generally less than 5 m. Lots of sediment is incorperated in the icefoot. During warm days, the icefoot melts. Subsequent refreezing of meltwater tends to smooth out its upper surface. Progressive melting concentrates the sediment near the upper surface.

Frozen icecrust.

During the wintermonths, a frozen crust of ice covers and immobilizes most of the intertidal sediments. This crust is usually continuous over several hundred meters and varies in thickness, from about 10 mm, when newly formed, to a maximum of 45 to 50 cm. Internally, the crust is composed of alternating ice and sediment laminae, up to 10 mm in thickness, which appear to be laterally continuous and parallel to its surface over several meters.

6.2 Ice formation.

Before ice can be formed, the water must be cooled to freezing temperature. As seawater (salinity > 2.74 ‰) becomes havier by further cooling, cooled water will move to the bottom and warmer water will appear at the surface. Thus, before ice can be formed, the water has to be cooled over its whole depth.
The just formed ice is comprised of small disc-shaped crystals with a maximum diameter of 5 mm and a thickness ranging between 0.025 mm and 0.1 mm. During the period of nucleation and active production of frazil, the icecrystals agglomerate to form frazil flocks of very low icecontent. Once the water comes back to 0 °C, the flocks of inactive frazil oscillate in the turbulent flow and those reaching the surface serve as a frame work for regular growth of ice from heat exchange with the atmosfere. As soon as the flocks are formed, they have a tendency to concentrate at the surface of the flow to form the usual frazil slush. Because of the continuous heat exchange with the atmosfere, the slushballs become denser and in case of a low flow velocity, they stay at the surface long enough so that a continuous layer of ice can be formed in their upper parts to produce ice pancakes (fig. 24). Because of collisions with their neighbours, they have a peculiar saucerlike appearance . At low water slack, when the watervelocity

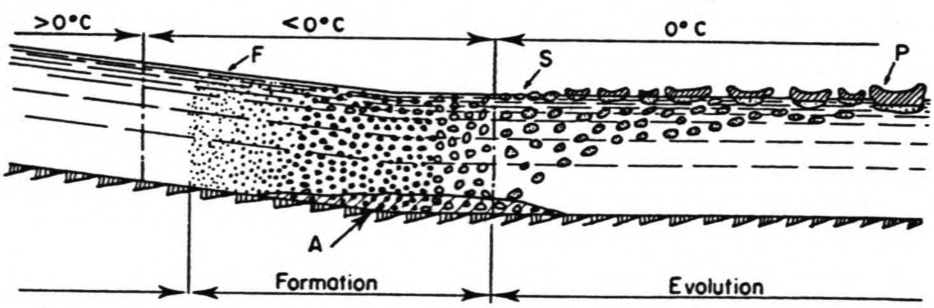


Fig. 24 Development of frazil in a river, from frazil discoids (F), to frazil slush (S) and pancakes (P). Anchor ice (A) is also shown. From Michel (1965).

is very small and the cooled water is spread thinly over the shoals, the conditions for freezing are most favourable. Ice will first form in the shallowest sections and gradually grow in more or less rounded cakes, usually following the contours of the shoals. The diameter of the cakes increases from the upper to the lower sections of the estuary by sintering of the individual pancake floes together. When the ice drifts over a long distance in and out of the bay with the tidal currents, a sizeable area of the watersurface may become covered with floating slush, icepans and floes. Once the icecover reaches 100 % of the surface, the pancakes press together to form a continuous icedeck. From that moment, the icecover progresses very quickly by simple juxtaposition of incoming frazil slush and icepans (fig. 25).

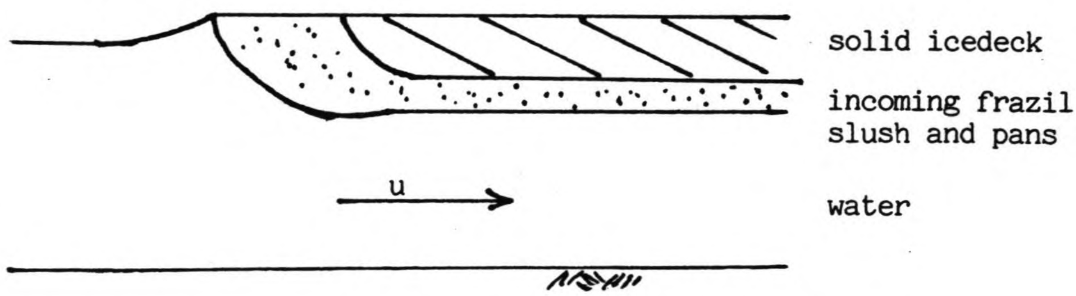


fig. 25 Icedeck progressing by incoming frazil slush and pans

Most of the solid ice blocks are formed during the periods of low tides. Some of the ice cakes move upstream with the tide, touch bottom at the moment of high water and strand when the tide starts dropping. They are pressed against the bottom by the still upstream flowing water (inertia). By the time the flow reverses, the tide has dropped so much that a lot of the icecakes can not be removed by the outgoing water. The contact between the icecakes and the ground is strengthened by frozen water trapped near the points of contact. When the subsequent high tide arrives, this bond is so strong that the buoyance of the ice cakes is not sufficient to refloat them. If this tide is higher than the previous one, other cakes may strand on top. In this way, in a week time, during the increasing tides from neap to spring, iceblocks of 4 to 5 m high can grow.

6.3 Design conditions.

The ice cover is built up from ice floes with a wide variation in diameter. Occasionally, a big ice block from the mudbanks is trapped in . Except under very severe weatherconditions, the ice deck will be broken up due to the high watercurrents. The percentage of coverage varies a lot each year. Under influence of wind and currents, the floes are moved around in the bay. Due to the high waterlevel differences, attachment to the shore will never take place. For the design conditions, the following ice thicknesses are taken into account:

return period (year)	max. thickness 10/10 cover (m)
20-30	0.5
100	1.0

fig. 26 Ice thickness in Cumberland Basin and Chignecto Bay.

Southwestwards of Cape Chignecto, no ice is expected, if any it would be slob ice.

Ridging ussually takes place when the ice is pushed into a narrowing basin. In Cumberland Basin and Chignecto Bay, tides with high currents up to 4-5 knots drive the water and ice in and out. When a complete cover is formed, in Chignecto Bay, on the incoming tide, a first ridge will form from Cape Enragé eastwards, a second one from Cape Maringuoin to Joggins Pt. (app. 7). These ridges can not build up very high. The ice is saline and formed in water of a relatively high temperature and with a lot of turbulence. Consequently it is much softer than fresh water ice with the result that high shear shearstresses won't devellop. On the ebb, the ridges will solve again. Cumberland Basin is shaped nearly rectangularly; no ridges will devellop.

For the design of the structures, two sorts of ice loads has to be designed for:

1. 'static' pressure of the rubble in front of the structures.
2. impact loads from ice floes or blocks hitting the structures at a certain speed.

Because of the enormous tidal waterlevel differences, adfreezing of ice to the structures is not expected.

6.4 Pressure of the rubble.

6.4.1 Behaviour of the ice sheet against the dam.

The ice sheet is driven against the dam by the environmental friction forces of wind and current. Because of its ruggged surface failure of the ice sheet does not take place simultaneously over its entire length, but only at a few places along its edge. When the sheet is just formed, its strength will be very small with the result that in the intake areas rubbleformation can be expected. Thus from the moment an ice deck has formed a rubblefield will be found on both sides of the structures. The sheet and rubble are moved towards and from the structures by the current and wind forces. As the sheet moves towards the structures, it will accumalate the rubble in front of it, until it has reached a thickness equal to the thickness of the sheet. When the sheet continues to move forward, the rubble located in front of it is thrust either over or under the sheet. The forces exerted on the sheet by the displaced rubble bend the sheet and thus bending stresses, which may at some time exceed the strength of the sheet are created (fig. 27).

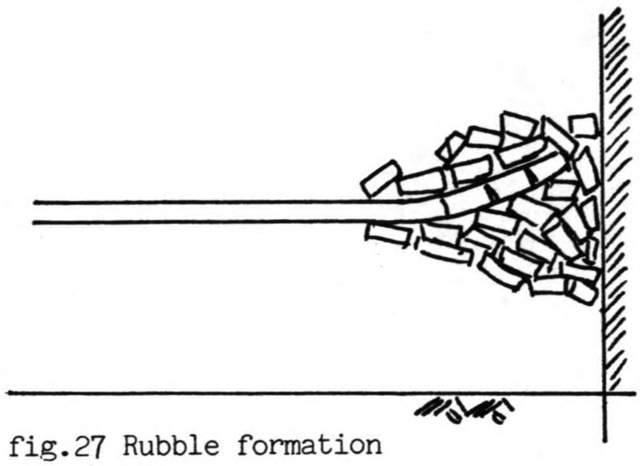


fig.27 Rubble formation

The ice cover, as described in § 6.2 , progresses by juxtaposition of incoming frazil slush and ice pans. When wind and current friction push the sheet against the dam, in a progressing cover, the stresses increase until the internal resistance of the cover is exceeded.Then the cover shoves or thickens until a new equilibrium is is reached (fig. 28).

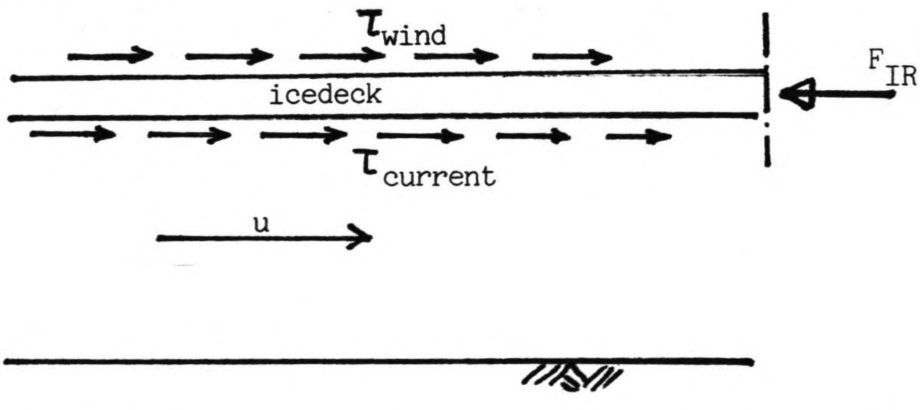


fig. 28 Equilibrium of stresses in the cover due to the driving forces and its internal resistance.

During generation and filling phase, the currentdirection turns and the process of incoming slush and ice pans reverses. Now stresses in the cover build up in the other direction.

6.4.2 Method to predict the pile-up height.

When being pushed against the dam by the tidal current- and wind-forces, the ice will load the dam. As the cover progresses upstream, stresses in the cover increase. The forces, which increase the ice-stress include the hydrodynamic shearing force of the flow under the cover, the shearing stress of the wind on the cover, the weight of ice along the slope of the ice/water interface and the hydrodynamic thrust on the leading edge of the cover. These forces must be opposed by the internal resistance of the cover and the resistance of the banks, otherwize the cover will be unstable and will shove or thicken until the resistance increases sufficiently.

From Michel (1971), the hydrodynamic force is defined as:

$$F_T = \frac{\gamma}{2g} D \left(1 - \frac{d}{D} \right)^2 v_u^2 W = 5000 \left(1 - \frac{d}{D} \right)^2 v_u^2 W$$

with: F_T = hydrodynamic thrust of the flow. (N)
D = depth of water upstream of leading edge. (m)
d = depth of flow under the leading edge. (m)
v_u = velocity under the leading edge. (m/s)
W = width of ice cover. (m)
γ = weight density of water. (N/m³)

The force on the ice from frictional drag is:

$$F_D = C_c \rho_w A v_c^2$$

with: F_D = friction drag force. (N)
C_c = watershear coefficient.
ρ_w = density of water. (kg/m³)
A = under-surface area of ice exposed to flow. (m²)
v_c = watercurrent velocity. (m/s)

The weight of the cover is based on simple boyance criteria and is independent of porosity, n. It is calculated as:

$$F_W = 9025\ S\ V$$

with: F_W = gravitational force acting along the channel. (N)
V = volume of ice cover. (m^3)
γ = 9800 (kN/m^3)

The force exerted on the cover due to the wind is calculated as follows:

$$F_{WD} = C_{10}\ \rho_a\ A\ v_w^2$$

with: F_{WD} = wind drag force. (N)
C_{10} = windshearcoefficient for windspeeds measured at a height of 10 m above surface.
ρ_a = airdensity. (kg/m^3)
A = ice surface area. (m^2)
v_w = windspeed at a height of 10 m above sea level. (m/s)

Bank reaction is comprised of an ice-over-ice frictional term related to the internal stress transferred to the banks and a cohesive (freezing) term. The cohesive term expression is given as follows:

$$F_C = 2\ C\ t\ L$$

with: F_C = force of cohesion of ice to two riverbanks. (N)
C = cohesion per unit of ice/bank interface. (Pa)
t = average thickness of ice cover. (m)
L = length. (m)

The hydrodynamic forces exerted on the icecover in streamwize direction create stresses in the ice, which are spread laterally towards the shore or banks. The lateral stress results in a reaction of static friction at the riverbank, which acts as a stabilizing influence on the cover. From Pariset and Hausser (1966):

$$F_F = 2\ f\ t\ L\ K_1\ \tan\varphi$$

with: F_F = friction force of ice along the riverbank. (N)
f = stress in the icecover in the direction of the flow. (N/m²)
K_1 = a coefficient equal to the ratio of lateral stress to longitudinal stress in the icecover. (a ratio less than or equal to 1.0)
φ = angle of friction of ice.

The $K_1\tan\varphi$-value is 0.150.

The capacity of ice to spread the environmental loads towards the shore is limited by its shearstrength. When a very wide icefield is pushed onto the shore and starts piling up along the shore, the icecover will fail in shear parallel to the winddirection. The equilibrium width of failure is:

$$B_e = \frac{2\tau\, t_i}{f}$$

with: B_e = effective width. (m)
τ = shearstrength. (N/m²)
t_i = icethickness. (m)
f = stress in the direction of loading. (N/m²)

Once this failure has occurred, the reaction of the cover is free to move downwind. In that case, the bankreaction, F_F, can be neglected and the other forces are determined by the failurewidth, B_e.

The internal resistance of the icecover (after Pariset et al. 1961, 1966) is given as:

$$F_{IR} = \rho' \left(1 - \frac{\rho'}{\rho}\right) \frac{g\ t^2}{2}\ K_2\ W$$

which can be reduced to:

$$F_{IR} = 361\ K_2\ t^2\ W$$

with: F_{IR} = internal resistance of the fragmented ice-cover. (N)

t = icethickness. (m)

K_2 = a coefficient analogous to Rankine's passive coefficient in soils. (= 8.7)

W = riverwidth. (m)

Values of K_1, K_2, and $\tan\varphi$ have been based on actual observations (after Pariset et al 1966), which show that:

$$K_1\ K_2\ \tan\varphi = \mu = 1.3 - 1.6$$

Although the individual values of K_1, K_2 and $\tan\varphi$ have not been determined by prototype observations, comparative simulation with the mathematical model have indicated that the predictions of shoves and hence icethicknesses are relatively insensitive to the choise of their individual values, provided that $K_1\ K_2\ \tan\varphi = 1.3 - 1.6$.

The total loading force is:

$$F = F_D + F_T + F_W + F_{WD} - F_c - F_F$$

If this force, F, exceeds the internal resistance of the cover, F_{IR}, then a shove is assumed to occur to permit the ice to thicken to the appropriate value necessary for stability. When a shove occurs, the required stable thickness of the cover can be determined by:

$$t_{sh} = \left(\frac{F}{361\ K_2\ W} \right)^{0.5}$$

6.4.3 Calculations.

For the designcalculations, only the loading situation on the dam is observed. The amount of ice needed to reach the calculated pile-up is assumed to be available. For the loading area, the surface area from ridge to ridge is taken into account. The pile-up is calculated for the low water situation and the moment of maximum current (app. 2).

designconditions Cumberland Basin side.

In Cumberland Basin, no ridging takes place. The loading area is:	$A = 16 * 10^6$ m²	(app. 7)
Because of the geometry of the bay, the hydrodynamic thrust is neglected:	$F_T = 0$	
mean waterdepth:	$h = 12$ m	
watershear coefficient:	$C_c = 2.5 * 10^{-3}$	
windshear coefficient:	$C_w = 1,2 * 10^{-3}$	
ice shearstrength:	$\tau = 600$ kN/m²	

maximum current situation.

maximum mean flow velocity:	$v_c = 1.2$ m/s	
slope of the watersurface:	$S = 5.9 * 10^{-5}$	(app. 7)

ice thickness: 1.0 m

v_w (NE)	B_e	F_{WD}	F_C	F_W	F_{TOT}	F_{IR}	t_{sh}
(m/s)	(m)	(kN/m')					
7	20	1.2	51.7	7.5	60.4	3.1	4.4
15	19	5.2	51.7	7.5	64.4	3.1	4.6
25	16	14.7	51.7	7.5	73.9	3.1	4.9

icethickness: 0.5 m

v_w (NE)	B_e	F_{WD}	F_C	F_W	F_{TOT}	F_{IR}	t_{sh}
(m/s)	(m)	(kN/m')					(m)
7	5	1.2	51.7	3.7	56.6	0.8	3.3
15	5	5.2	51.7	3.7	60.6	0.8	3.4
25	4	14.7	51.7	3.7	70.1	0.8	3.6

($B_e < W \Longrightarrow F_F = 0$)

slack water.

icethickness: 1.0 m

v_w	B_e	F_{WD}	F_{IR}	t_{sh}
(m/s)	(m)	(kN/m')		(m)
7	1050	1.2	3.1	1.0
15	228	5.3	3.1	1.0
25	82	14.7	3.1	1.7

($F < F_{IR}$)

icethickness: 0.5 m

v_w	B_e	F_{WD}	F_{IR}	t_{sh}
(m/s)	(m)	(kN/m')		(m)
7	262	1.2	0.8	0.5
15	57	5.3	0.8	1.0
25	21	14.7	0.8	1.7

designconditions Chignecto Bay side.

In Chignecto Bay, a ridge devellops, from Cape Maringuoin to Loggins Pt.;	
the loading area is:	$A = 7 * 10^6$ m² (app. 7)
hydrodynamic thrust (loading area from ridge to ridge):	$F_T = 0$
mean waterdepth:	$h = 22$ m
watershear coefficient:	$C_w = 2.5 * 10^{-3}$
windshear coefficient:	$C_c = 1.2 * 10^{-3}$
ice shearstrength:	$\tau = 600$ kN/m'

maximum current situation.

maximum mean flow velocity:	$v_c = 1.5$ m/s
slope of the watersurface:	$S = 4.3 * 10^{-5}$ (app.8)

ice thickness: 1.0 m

v_w (SW)	B_e	F_{WD}	F_C	F_W	F_{TOT}	F_{IR}	t_{sh}
(m/s)	(m)	(kN/m')					(m)
7	12	0.3	23.1	1.6	25.0	3.1	2.8
15	12	1.5	23.1	1.6	26.2	3.1	2.9
25	11	4.2	23.1	1.6	28.9	3.1	3.0

ice thickness: 0.5 m

v_w	B_e	F_{WD}	F_C	F_W	F_{TOT}	F_{IR}	t_{sh}
(m/s)	(m)	(kN/m')					(m)
7	48	0.3	23.1	0.8	24.2	0.8	2.8
15	47	1.5	23.1	0.8	25.4	0.8	2.8
25	43	4.2	23.1	0.8	28.1	0.8	3.0

($B_e < W \Longrightarrow F_F = 0$)

slack water.

ice thickness: 1.0 m

v_w(NE)	F_{WD}	F_{IR}	t_{sh}	
7	0.3	3.1	1.0	($F < F_{IR}$)
15	1.5	3.1	1.0	($F < F_{IR}$)
25	4.2	3.1	1.2	

ice thickness: 0.5 m

F_{IR}	t_{sh}	
0.8	0.5	($F < F_{IR}$)
0.8	0.7	
0.8	1.2	

6.4.4 Interpretation of the results.

The maximum pile-up calculated is:

basin side:	3.5 - 4.0 m	at max. flow
	1.5 - 2.0 m	at slack water
sea side:	3.0 m	at max. flow
	1.0 - 1.5 m	at slack water

The maximum driving force from basin side is approximately 70 kN/m', from sea side 30 kN/m' (§6.4.3). These loads are much smaller than the design wave loads (§ 7.0).

The most important driving force is the current drag. Due to the high tidal currentfriction, the icedeck will be broken up (B_e very small). Pile up can only devellop when the pressure can not be released sideways. When the plant is working, during the filling phase, the flow will concentrate towards the sluices, during generation towards the turbinesections. As during these periods the flow, and thus the driving force concentrates, pile up is expected to devellop on the basin side of the turbine caissons and on the sea side of the sluices. In designconditions, in front of the turbine sections, the ice is expected to accumalate up to a thickness of 3.0 m during generation and 2.0 m at HW. With this amount of pile up,no overtopping of the caisson is expected from the basin side. On the other side, when generating,the ice will be broken up and blown away by the outcoming water. During the next phase, the filling period, the flow will concentrate towards the sluices. Now, at the sea side of the turbine caissons, the ice will have the possibility to spread its loads sideways with the result that hardly any pile-up will devellop. No significant accumalation of ice is expected on top of the caisson (fig. 29 , app. 9)

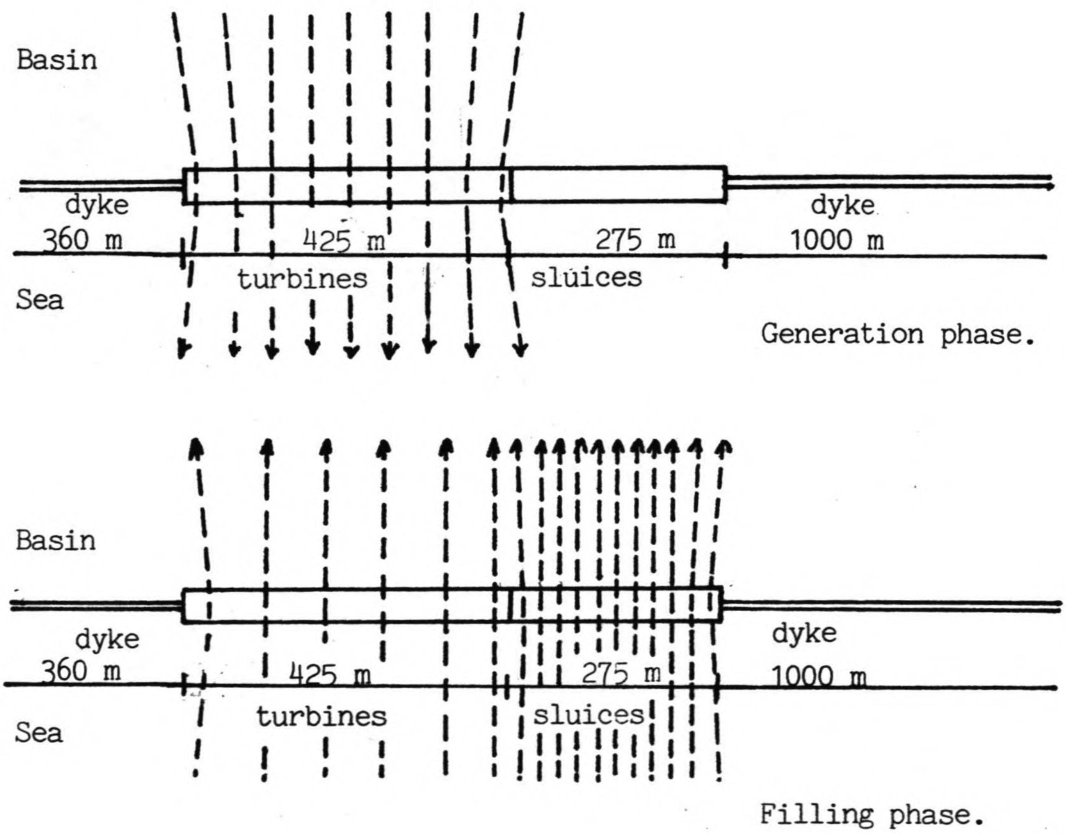


fig.29 Flow scheme when the plant is working.

6.5 Impact loads.

Impact loads can be expected from:

1. icefloes, which, moving under influence of water- and windfriction, hit the caisson.
2. icepieces, smashed against the structures by the waves.

6.5.1 Method to predict impact loads.

When an icefloe collides with a massive structure, the contact zone will fail by crushing and will increase in size as the resisting force is steadily increased. The kinetic energy is then decreased until a new equilibruim point is reached. To calculate the impact of the floes or ice pieces, the following designassumptions have been used:

- When an icefeature collides with a massive structure the leading edge of the icefeature will be progressively crushed and the force acting on the structure during impact will be that of a steady increase from zero to a certain maximum value, as a result of kinetic energy-dissipation during the icecrushing.

- the shape of the ice floe has been assumed to be round.

- the deformation of the structure has been neglected, since it normally is an insignificant portion of the overall displacement.

- the load is introduced very quickly, a crushing failure is assumed.

When an ice floe, which moves at a velocity v, hits the caisson, it creates an impact load in addition to the hydrostatic load.

The collision force can be expected to reach its maximum, when the kinetic energy of the floe is completely dissipated. During the impact the contact area is assumed to increase steadily by crushing of the contact area up to a maximum A_c. Hence the impact-load is a function of the penetrationdepth and can be expressed as:

$$F_x = A_x \sigma_{cr}$$

The contactarea for a penetrationdepth x is defined by:

$$A_x = 2\ t\ (\ 2\ R\ x - x^2\ )^{0.5}$$

with: t = ice thickness (m)
R = floe radius (m)

From these equations follows:

$$F_x = 2\ t\ (\ 2\ R\ x - x^2\ )^{0.5}\ \sigma_{cr}$$

The virtual displacement of the floe can be considered as:

$$W = k\ W_i$$

with: W_i = weight of the ice (kg)

For these relatively thin floes a k-value of 1.1 is taken into account.

The maximum value of penetration can be determined by equating the kinetic energy of the floe to the energy absorbed by crushing of the ice:

kinetic energy: $E_k = \dfrac{W\ v^2}{2g} = 1.1\ \dfrac{W_i\ v^2}{2g}$

crushing energy: $E_c = \int^{x_m} F_x\ dx = 2\ t\ \sigma_{cr} \int^{x_m} (\ 2\ R\ x - x^2\ )^{0.5}\ dx$

An approximate expression for x_m can be found by dropping the x^2-term in the last expression. This gives:

$$E_c = 1.89\ t\ \sigma_{cr}\ R^{0.5}\ x_m^{1.5}$$

The resulting penetration and impactload are:

$$x_m = \left(\frac{0.291\ W_i\ v^2}{t\ \sigma_{cr}\ R^{0.5}\ g} \right)^{2/3}$$

$$F_m = 2\ t\ (\ 2\ R\ x_m - x_m^2\ )^{0.5}\ \sigma_{cr}$$

The floevelocity is determined as follows:

Relative to the watercurrents, a floe is set into motion by the windfriction and increases its speed until balanced by the water-shear.

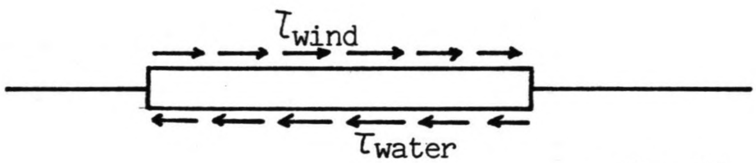


fig. 30

The watershear is estimated as follows:

$$\tau_c = C_c\ \rho_w\ v^2$$

with: τ = watershear (N/M²)
C_c = watershear coefficient (= 2.5 * 10^{-3})
ρ_w = density of water (kg/m³)
v = floevelocity relative to water (m/s)

Windshear can be expressed by:

$$\tau_a = C_{10}\ \rho_a\ v_w^2$$

with: τ_a = windshear (N/m^2)
v_w = windspeed relative to ice (m/s)
C_{10} = windshear coefficient (= $1.2 * 10^{-3}$)

Balance between wind and watershear gives:

$$v = \frac{1}{46} v_w \qquad (\tau_a = \tau_c)$$

The resulting floevelocity is:

$$v_{floe} = v_{current} + 0.02\, v_{wind}$$

6.5.2 Impact from icefloes.

Due to the enormous tidal waterleveldifferences, the icedeck isn't attached to the shore and is free to move up and down river under influence of current and wind. Because of the high currents, the floes are not expected to grow up to a larger diameter than 100 m. While moving, these floes may hit the structures. The impact of floes of several diameters is calculated, a review is given in app. 10 . The design impactload from icefloes is:

	basin side	sea side
crushing pressure:	1 MPa	1 MPa
windvelocity:	NE: 25 m/s	SW: 25 m/s
maximum floevelocity:	1.7 m/s	2.0 m/s
impact load:	8800 kN	9800 kN
crushing surface:	8.8 m²	9.8 m²

In case the floe hits just one caisson, the floe impact from both sides is much smaller than the design waveloads and thus isn't a designvalue for the overall stability of the caisson. For the local loads on the concrete, the impactloads will have to be taken into account.

6.5.3 Waveattenuation in icefields.

Longer waves with periods of 10 seconds or more penetrate over very long periods into icefields. Such waves are not influenced by the size of the floe. Waves of periods less than 10 seconds attenuate quickly. In that case attenuation depends on the floe size rather than its thickness. Fieldobservations have given the following information:

In waves with periods shorter than 10 seconds, floes of approximately 1.5 m thick and a diameter smaller than 40 meter behave as ridgid floating bodies. With such waves, no detectable penetration into the icefield took place, if the floes were half a wavelength or more in diameter. If the floes are less than 1/6 of the wavelength or more in diameter, little energyloss takes place. The main energyloss takes place when the floes are 1/3 of the wavelength. In these situations, the floethickness is not important. However, when the waveperiods are 11-12 seconds, the penetration of long ocean swells into icefields consisting of floes larger than half of the wavelength takes place by bending of the floes. In these cases, the thickness of the icefield influences the wave-energy. In a broken icefield covering the whole surface, the following attenuation is measured:

icethickness (m)	period (sec)	reduction (%)
3	5	50
3	6	33
1.2	7	15-20

Waves with periods smaller than 5-6 seconds are completely attenuated within the first 50 meters of the cover.

6.5.4 Impactload of icepieces in waves.

Under severe weatherconditions, larger icefloes are expected to accumalate on both sides of the dam. In that case, most of Cumberland Basin will be covered with ice with the result that only smaller waves can devellop. These will be attenuated in the cover very quickly. From the sea side, in the case of a southwesterly storm, waves from the Bay of Fundy and the Gulf of Maine will penetrate through the icedeck and smash the ice against the structures. To calculate these impactloads a 10-year designwave is taken into account:

10-year designwave: $H_s = 8.0$ m $H_{1\%} = 9.2$ m

Under these circumstances, the cover will be broken up; the percentage of coverage is uncertain. For designpurpose, no reduction of waveheight in the cover is assumed. Devellopment of this wavefield takes about 10 hours. In an increasing wavefield, after a couple of hours, the larger icefloes in front of the structures will be broken up. A maximum floesize of 2.0 m in diameter is expected.

For the overall stability of the caisson, the loading of a 10-year wave containing ice has to be compared with the 100-year design waveloads. As soon as the floesize is reduced to a maximum of 2.0 m in diameter, most of the ice will be crushed and the distribution of icepans will be random. This means that the impact of the icepieces will not take place at the same time over the whole length of the caisson. The crushed ice has no strength and will hardly devellop any impact. The 100-year design waveload is taken into account for the overall stability. For the local stabilitycriteria, the impact of the single icepieces is calculated.

Calculation.

Icepans, drifting on waves, move with the orbital partical velocity of the wave:

horizontal: $$u = \frac{\omega H}{2} \frac{\cosh kd}{\sinh kd} \cos (kx - \omega t)$$

vertical: $$v = \frac{\omega H}{2} \frac{\sinh kd}{\sinh kd} \sin (kx - \omega t)$$

The designwave characteristics are:

waveheight: $H_{1\%} = 9.2$ m

period: $T = 10.5$ sec. $\omega = 0.6\ \text{sec.}^{-1}$

mean depth in Chignecto Bay: $d = 41$ m

wavelength: $\lambda = 160\ \text{m} \Longrightarrow k = 0.04$

$$\left. \begin{array}{l} d = 41\ \text{m} \\ k = 0.04 \end{array} \right\} \Longrightarrow kd = 0.6$$

(Pierson-Moskowitz spectrum)

The resulting orbital velocities are:

horizontal: $u_{max} = 3.0$ m/s

vertical: $v_{max} = 2.8$ m/s

The maximum impact loads are develloped when the floe hits horizontally to the vertical face of the caisson and perpendicularly to the upper face of the caisson.

The maximum floesize is: diameter: 2.0 m

thickness: 0.5 m

The weight of the floe: 1400 kg

The crushing strength: 1 MPa

The resulting penetration and impactloads are:

	vertical face	upper face
penetration (mm)	8.2	7.5
impactload (kN)	130	120

6.6. Inflow of ice into the turbines.

As far as ice blocking the turbine intakes is concerned, two different problems appear:

1. inflow of frazil ice.
2. inflow of iceblocks.

Frazil ice.

High turbulent rivers usually carry a lot of frazil ice. These crystals, formed at the formation stage of the ice, tend to stick to the trash rags and intakes, eventually blocking off the complete intake. The formation of frazil ice stops as soon as a complete icedeck has formed.
At hydroplants in rivers with severe iceproblems, the growth of a complete icedeck usually is forced at the beginning of the winter by stopping the plant for a couple of days, until a strong enough icedeck is formed. To prevent the frazil iceproduction, they try to keep the icedeck the whole winter through.
In Cumberland Basin, this method is not applicable as a complete icedeck can not be kept.

Iceblocks.

Inflow of a considerable amount of iceblocks will not take place (app. 9). Trash rags, if considered needed, would usually be placed on an inclined slope.

7.0 Resulting designloads.

Overall stability: Wave and hydrostatic pressures.

situation	horizontal load (kN/m')	arm (m)	moment (kNm/m')
1	6006	17.4	104577
2	6684	13.7	92147
3	4989	18.9	89797

Local loads: Impact loads of icefloes.

		crushing surface (m^2)	impact load (kN)
vertical face	basin side	8.8	8800
	sea side	9.8	9800
upper deck	basin side	-	-
	sea side	0.12	120

8. Foundation.

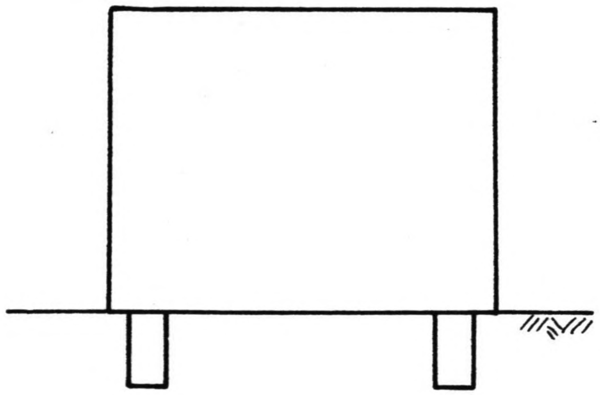
8.1.1 Introduction.

The foundation has the following functions:

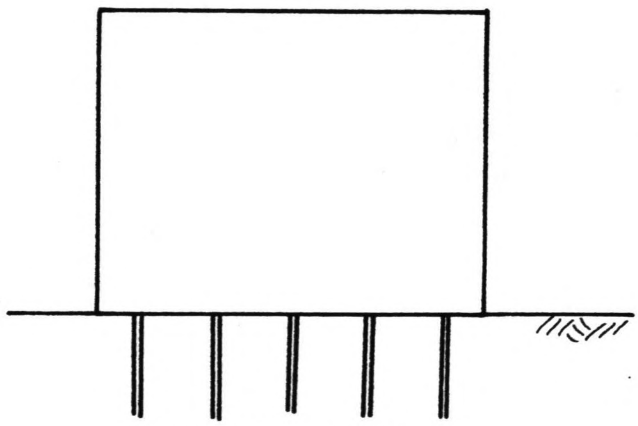
1. A foundating function,to bring the loads to the underground.
2. A sealing function, to prevent seapage underneath.

Principally, three methods or combinations of these methods, are used:

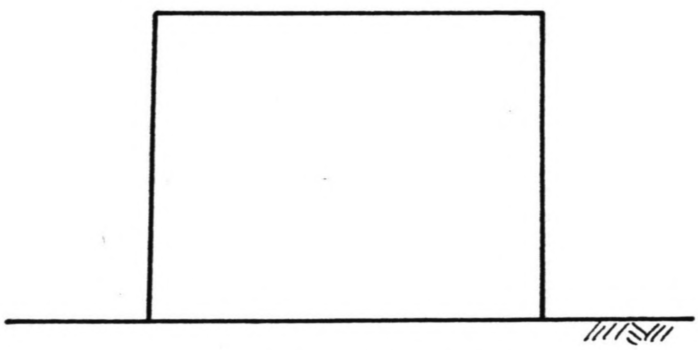
1. A foundation on piles.
2. A foundation on pits.
3. A foundation directly on the ground.



foundation on pits



foundation on piles

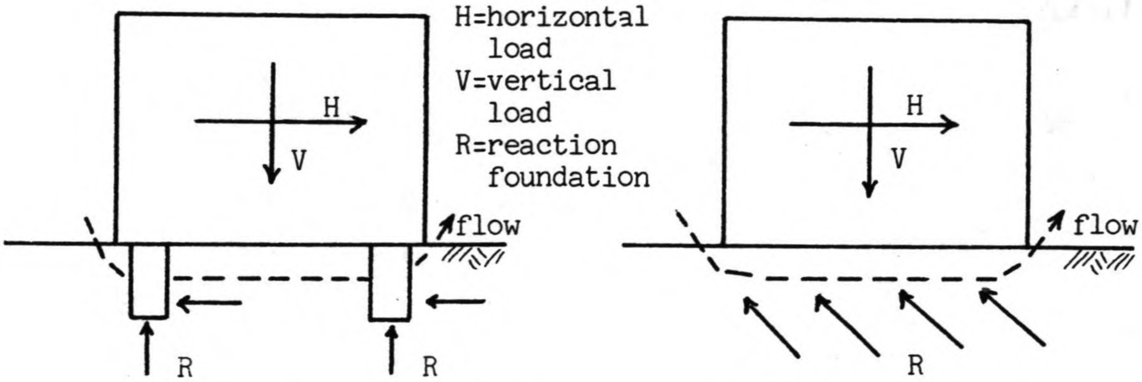


foundation directly
on the ground

fig. 31

8.1.2 A foundation on pits or piles.

With these types of foundation, sealing and foundation are seperated in function. The threshold has a sealing function; the pits or piles a foundating one. At the places, where the pits (or piles) are constructed through the threshold, the sealage is interrupted and erosion might be a problem. Because the functions of the seal and the foundation are seperated here, the consequences are less than with a foundation directly on the ground.



Foundation on pits fig. 32 Foundation directly on the ground.

One of the bigger problems, when foundating on pits (or piles), is the space, usually found between the caissonbottom and the underground. This results from a difference in setting of the ground, immediately underneath the caisson,and the ground on which the pits or piles are foundated. Thus an easy way for the flow is created, which could result in high flowvelocities and erosion. Some solutions for this problem are:

- a skirt
- a sheet pile
- a 'groutsausage'.

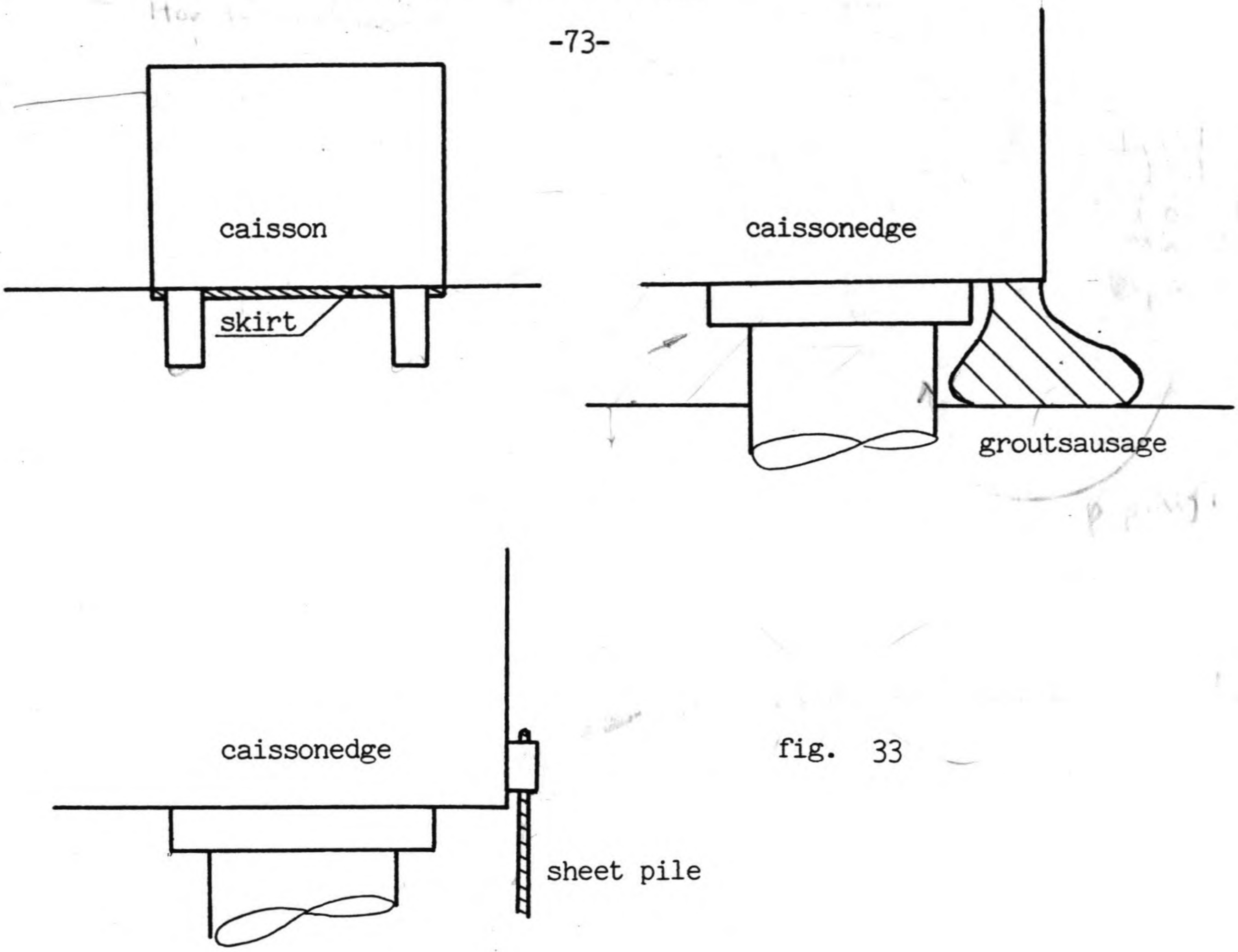


fig. 33

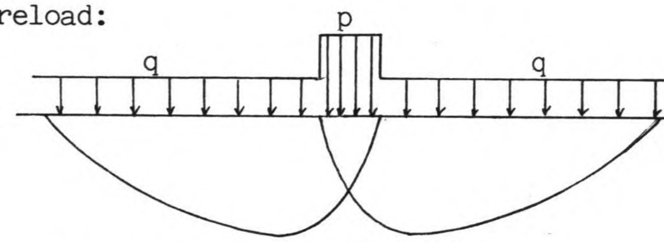
8.1.3 A foundation directly on the ground.

A foundation directly on the ground is only possible when the underground is stable enough to carry the vertical loading and has enough shearstrength to carry the horizontal loads. With this type of foundation, the underground has a double function: a sealing and a foundating one (fig. 32). This means erosion underneath the caisson would have desastrous consequences for the foundation aswell. In order to prevent seapage underneath the caisson, a good connection between the caissonbottom and the foundationbottom has to be realised. For this reason, after ballasting, the caisson sometimes is undergrouted. To keep the grout in, the same solutions as proposed for the sealage of a foundation on pits or piles (fig. 33) could be used.

In order to get an impression of the problems, the stability and sealage of the underground is estimated first.

8.2 Stability of the foundation.

The stability of the underground is calculated with the method of Brinch Hanssen. This method, based on the theory of Prandtl, gives the following relation for the failureload:



Prandtl

$$p = c\,N_c + q\,N_q + \tfrac{1}{2}\,\gamma\,B\,N_\gamma$$

(1) (2) (3) (4)

with: (1) = failureload. (kN/m²)
(2) = cohesionterm. (kN/m²)
(3) = loading pressure of the caisson. (kN/m²)
(4) = loading of the ground. (kN/m²)

For horizontal loads, the coëfficients in this formula have to be corrected as follows:

B is reduced to: $B_{eff.} = B - 2\,\frac{H}{V}\,exc.$

with: H = horizontal load. (kN)
V = vertical load. (kN)
exc. = excentricity. (m)
B = width of the caisson. (m)

As well, for horizontal loads, the coëfficiënts N_c, N_q, N_γ have to be reduced with the following factors:

$$i_c = \left[1 - \frac{t}{c + p\tan\varphi}\right]$$

$$i_q = i_c^2$$

$$i_\gamma = i_\gamma^3$$

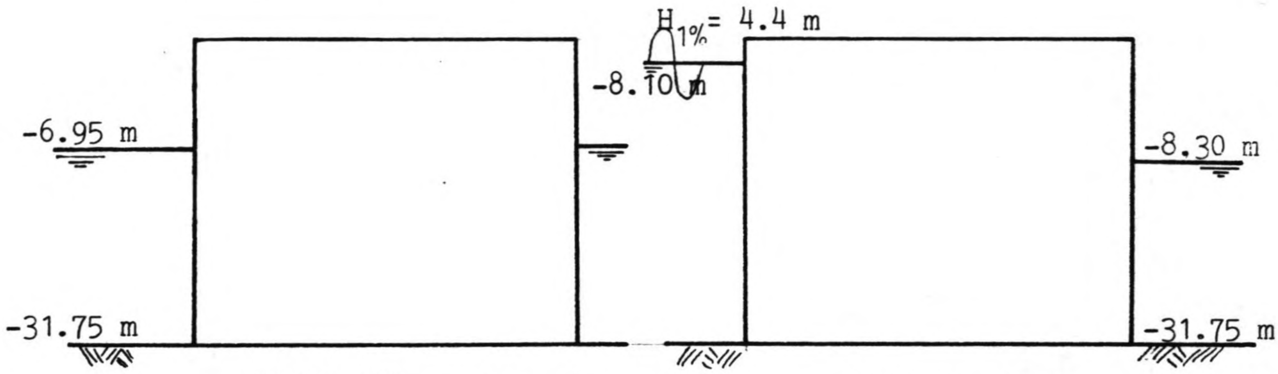
with: p = vertical pressure. (kN/m²)
t = horizontal pressure. (kN/m²)
φ = internal friction of the ground.
c = cohesion of the ground. (kN/m²)

These N-values depend on the internal friction.

Calculation.

The stability of the underground is checked for the following situations:

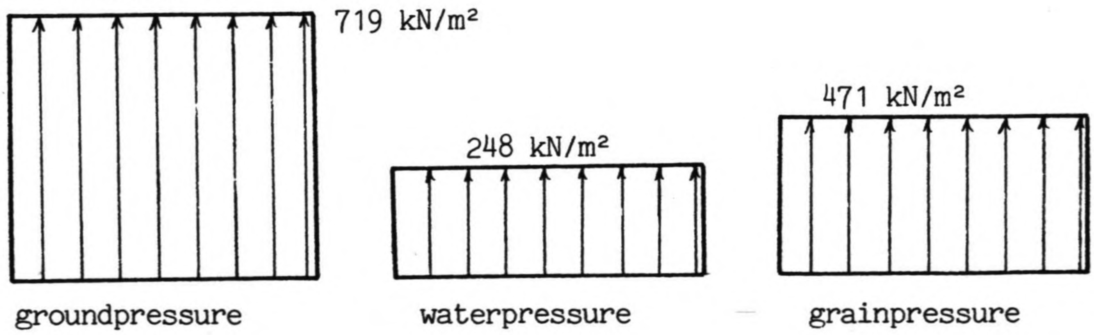
1. Static situation LW (fig.34).
2. Designcondition situation 1 (fig.35).



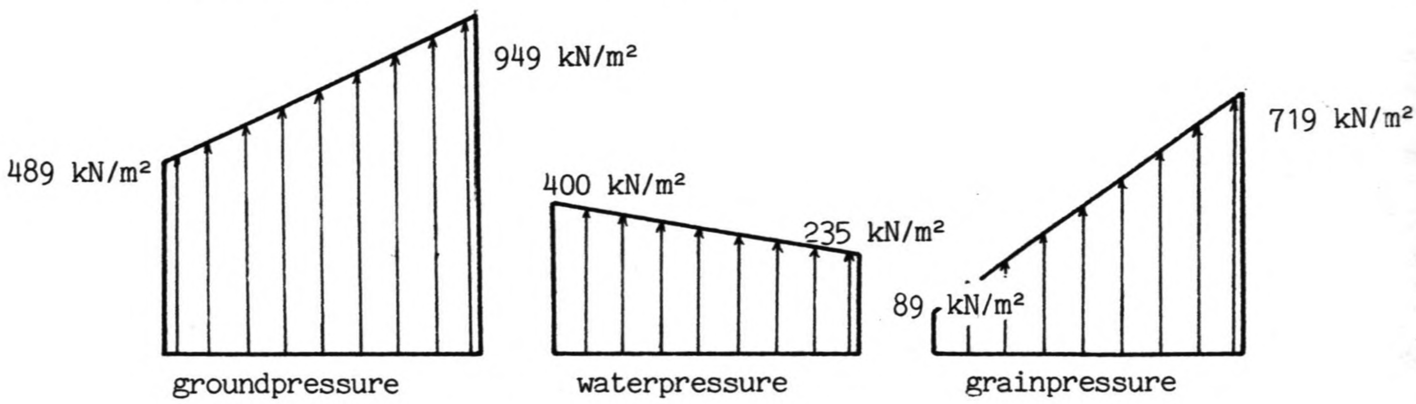
Static LW fig. 34 Designcondition maximum moment fig. 35

The resulting ground-, grain- and waterpressurres underneath the caisson are:

LW situation:



Designcondition 1 (maximum moment):



As follows from consolidationtests, the c-value varies from 5.2 to 34.5 kN/m². A c-value of 5 kN/m² is assumed.The internal friction of the ground approximately is 30°. The resulting N-values are:

N_c = 30.14
N_q = 18.40
N_γ = 15.07 (ref. 7)

<u>Static situation:</u> p = 471 kN/m²
q = 21 kN/m² (4 meter ground, ρ = 17 kN/m³)

The resulting failurepressure is : 3312 kN/m²

<u>Designcondition 1:</u> p_{mean} = 402 kN/m²
t = 114 kN/m²
q = 21 kN/m²
exc. = 17.4 m

i_c = 0.54
i_q = 0.30
i_γ = 0.16

The resulting coëfficientvalues are:

i_c = 0.54	N_c = 16.4	B_{eff} = 42.6 m
i_q = 0.30	N_q = 5.4	
i_γ = 0.16	N_γ = 2.4	

The resulting failurepressure is : 533 kN/m²

From these calculations, the stability proves sufficient.

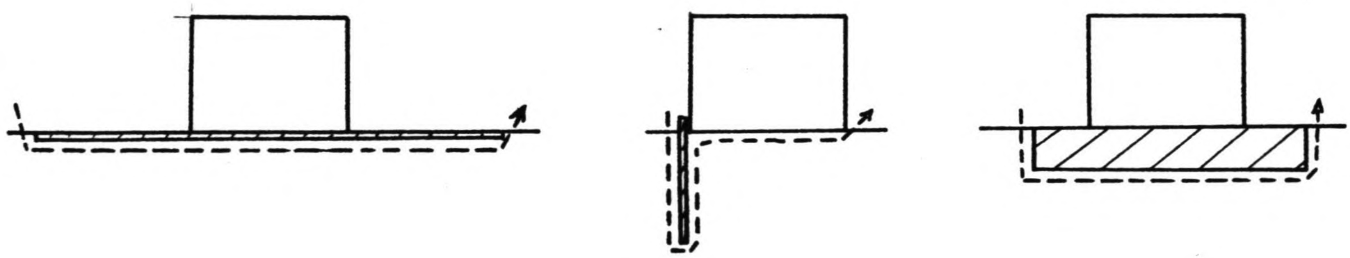
8.3 Sealage of the foundation.

As a result of the high waterleveldifferences over the caisson, seapage through the foundation might be caused. When the watervelocities through the ground are too high, material can be carried away. Erosion, from the bottom inwards, called piping, might devellop. The flow through the ground can be estimated with the following formula:

$$q = k \frac{dh}{dx}$$

with: q = discharge velocity. (m/s)
k = permeabilitycoëfficient of the ground. (m/s)
dh = waterleveldifference. (m)
dx = length of the flow. (m)

The length of the flow, needed to secure a stable foundation, principally can be obtained as follows:



horizontally vertically combination

fig. 36

The length needed is estimated with the method of Lane:

$$C_L = \frac{\sum L_{vert.} + \frac{1}{3} \sum L_{horz.}}{h}$$

with: $L_{vert.}$ = vertical length of the flow. (m)
$L_{horz.}$ = horizontal length of the flow (m)
h = waterleveldifference over the caisson. (m)

This method is based on a good connection between the caissonbottom and the foundation.

Calculation.

The extreme waterlevels are:

$$\left.\begin{array}{ll}\text{maximum basin side:} & +8.1\text{ m}\\ \text{minimum sea side:} & -8.3\text{ m}\end{array}\right\} \Longrightarrow \text{resulting head:}\quad 16.4\text{ m}$$

$$\left.\begin{array}{ll}\text{maximum sea side:} & +9.5\text{ m}\\ \text{minimum basin side:} & +0.1\text{ m}\end{array}\right\} \Longrightarrow \text{resulting head:}\quad 9.4\text{ m}$$

The designwaterleveldifference over the caisson is: 16.4 m (freq. 10^{-4})

Foundationground: coarse sand and gravel $\Longrightarrow C_L \approx 4.5$

The length needed is:

horizontally: 221 m
vertically: 28 m
or a combination .

8.4 Influences of waveloads on the stability of the foundation.

Waveloads are variable loads of a short period. In the foundation, when the overpressed water can not be released quick enough, the short period loads will be taken up by the waterpressures. Then the grainpressure decreases and the stability of the underground could be endangered. In order to get an impression of these influences, the reduction in grainpressure is estimated. A one-year designwave is taken into account.

1-year designwave: $H_{1\%} = 6.0$ m
$T = 9\text{-}10$ s.

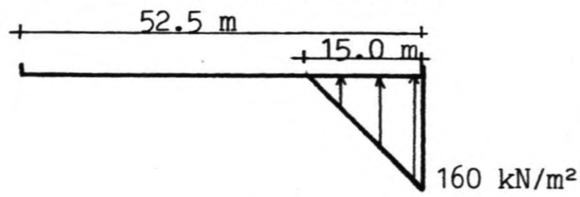
The mean ground-,grain- and waterpressures at foundationlevel at HW are:

groundpressure: $\sigma_g = 618$ kN/m²
waterpressure: $\sigma_w = 391$ kN/m²
grainpressure: $\sigma_k = 227$ kN/m²

Two sorts of variable waveloads take place:

- horizontal loads on the vertical face of the caisson
- breaking waves on top of the caisson.

8.4.1 Influence of the horizontal wavepressures on the vertical face of the caisson.



1-year wave load: $F = 1320$ kN/m'
$M = 16518$ kNm/m'
$z = 21.5$ m

Resulting pressure.

The maximum pressure at the caissonedge, 160 kN/m², is 40 % of the mean grainpressure.

8.4.2 Influence of breaking waves on top of the caisson.

The load resulting from a breaking wave with a height of 6 meter is:

F = 367 kN/m' (calculationmethod § 9.5.3)

The maximum pressure is expected to devellop within the first 20 meters from the caissonedge. For a loadingdistance of 5 and 10 meters from the edge, the resulting pressures in the foundation are estimated:

breaking distance from the caissonedge (m)	resulting max. pressure in foundation (kN/m²)	perc. of mean grainpressure
5	40	18
10	15	7

8.4.3 Conclusions

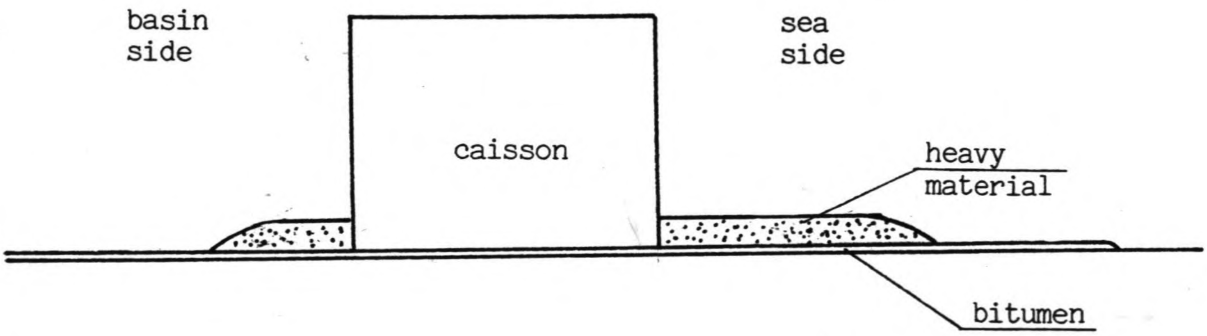
The duration of maximum pressure for breaking waves on top of the caisson is approximately 0.1 second. During such short periods, hardly any release of waterpressure can be expected. This means the total loading will have to be taken up by overpressure of the water. An eguivalent decrease in grainpressure devellops. In order to secure the stability of the caissonedges, extra protection of the foundationmaterial probably is needed.
Assuming no release of wateroverpressure due to the one-year design wave loads on the vertical face of the caisson, a reduction of grainpressure up to 40 % would take place. However, during the variationperiods of concern here, 9-10 seconds, a release in waterpressure is expected to devellop. If the underground is not permeable enough to secure a stable foundation, a filter underneath the caisson will have to be constructed.

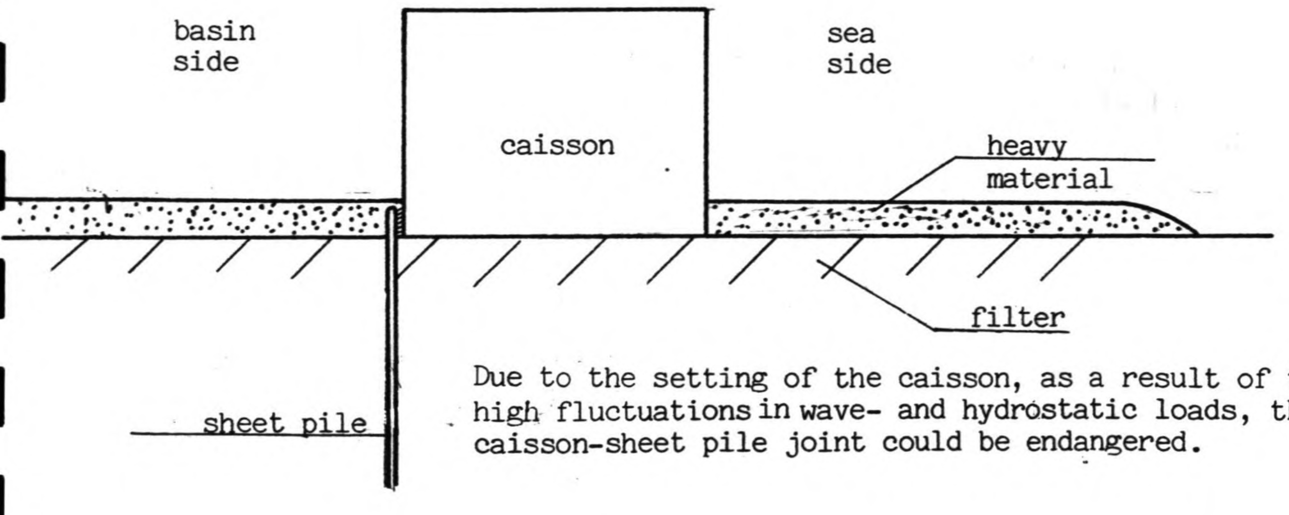
8.5 Stability of the foundation during construction.

As the closure proceeds, the flowvelocities will increase enormously. To fix the bottom, heavy material or a layer of bitumen could be used. When using heavy material, a filter will probably be needed in order to prevent mixing of the underlying ground with the upper layers. With use of a bitumenprotection, at the places where wateroverpressures are expected at the underside of the layer, additional ground will have to be put on top. Bitumen has a sealing function aswell.

8.6 Interpretation of the results.

As far as can be concluded from these rough calculations, the stability of the underground seems sufficient. Attention would have to be paid to the sealage of the ground, the influence of waveloads and the fixation of the bottom during construction. Schematically reviewed, perhaps the following solutions could be possible.





Due to the setting of the caisson, as a result of the high fluctuations in wave- and hydrostatic loads, the caisson-sheet pile joint could be endangered.

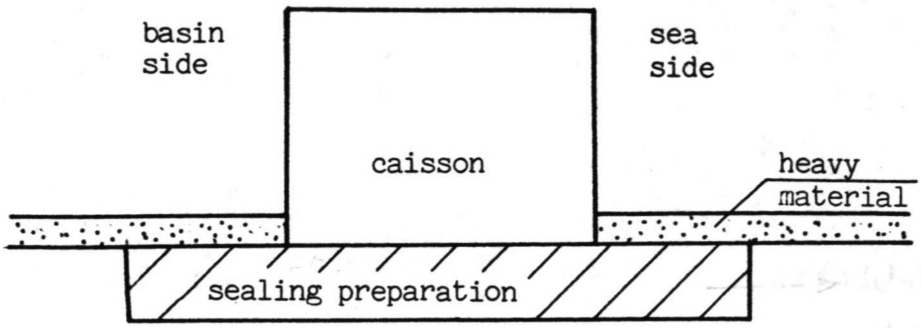


fig. 37

From an efficiency point of view, leakage underneath the caisson is neglegible (§ 4.6.1).

Within the reach of this study, no more attention is paid to the the foundationaspects.

9. DIMENSIONS AND STABILITY

9.1 Introduction.

In order to save in costs of the building pits, construction of the caisson in two parts is considered here. Both parts would be built in dry pits, towed out and *placed on top of each other afterwards. Eventually towage to a waiting place and to its definitive location would take place. Because after some time, the fabrication of the turbines is expected to be slower than the caisson construction, installation of the turbine before and after lowering of the caisson would probably be done both.
The design of the caisson is based on the minimum dimensions needed for the turbine in- and outtake. An estimate of caissondraft and -weight, based on these dimensions is calculated in appendix 10. Dimensions and stability of the caisson in the different constructionfases are considered in this paragraph; rough calculations are made.

9.2 Stabilitycriteria during the lifetime of the caisson.

To stabilize the caisson sufficiently, turning over and sliding has to be prevented. The stability of the foundation itself is calculated with the method of Brinch Hanssen (§ 8.2).

9.2.1 Sliding.

The sliding resistance of the caisson depends on the internal friction of the foundationground and the pressure of the caisson on its foundation. The sliding resistance can be checked with the following formula:

$$\gamma H \leqslant V \tan \varphi$$

with: φ = internal frictionangle. (°)
H = horizontal load. (kN)
V = vertical load. (kN)
γ = safetycoefficient.

The caisson is packed in by foundationmaterial over a height of 4 meters (fig. 38). The active and passive pressures of the ground have been calculated with the following formula's:

$$\sigma_{k,\text{horz.}} = \sigma_{k,\text{ vert.}} * \lambda$$

$$\lambda_a = \frac{\cos \delta}{\cos^2(\varphi + \delta)} * \left[\cos \delta + \sin\varphi \sin(\varphi + \delta) - 2\sqrt{\sin\varphi \sin(\varphi + \delta) \cos \delta}\right]$$

$$\lambda_p = \frac{\cos \delta}{\cos^2(\delta - \varphi)} * \left[\cos \delta + \sin\varphi \sin(\varphi - \delta) + 2\sqrt{\sin\varphi \sin(\varphi - \delta) \cos \delta}\right]$$

with: φ = internal friction angle. (°)
δ = frictionslope between concrete and ground. (°)
σ_k = groundpressure - waterpressure. (kN/m²)

Calculation of the horizontal groundpressures.

foundation characteristics:

$\varphi \approx 30°$ $\lambda_a = 0.3$
$\delta \approx 2/3\,\varphi = 20°$ $\lambda_p = 5.7$
$\rho = 17$ kN/m³ (density foundationmaterial)

Resulting horizontal loads:

$F_a = 15.6$ kN/m'
$F_p = 321.4$ kN/m' $F_{total} = 305.8$ kN/m' (fig.38)

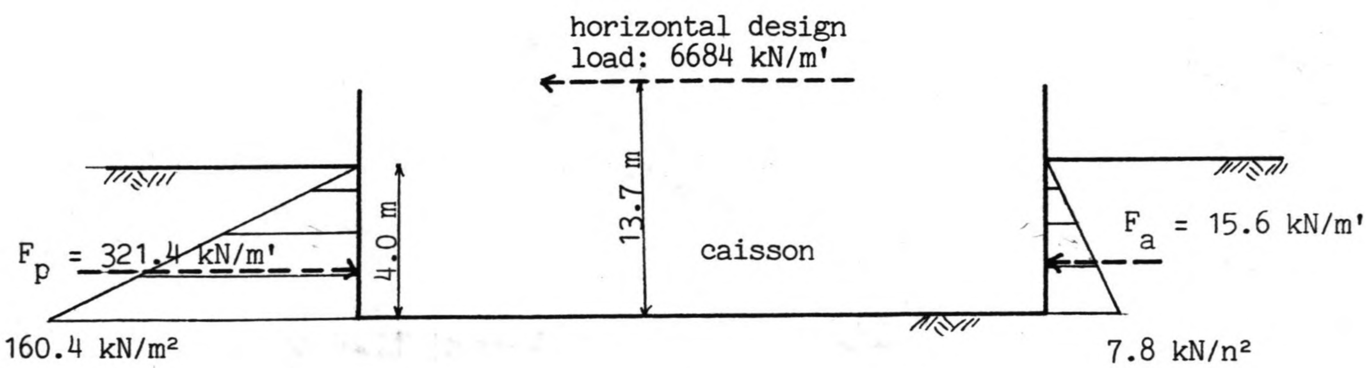


fig. 38

To obtain sufficient resistance against sliding, the force resulting from the horizontal design load and horizontal ground pressures has to be smaller than the sliding resistance of the caissonbottom.

Horizontal load on the caissonbottom:

	load (kN/m')	height above foundationlevel (m)
horz. groundpressure	306	1.3
horz. design load (app.)	6684	13.7
resulting load	6378	14.3

Safety against sliding:

criterium: $\gamma H \ll V \tan\varphi$

with: $H = 5378$ kN/m' (§ 7.0)

$V = 17178$ kNm/m' (§ 7.0)

$\varphi = 30°$

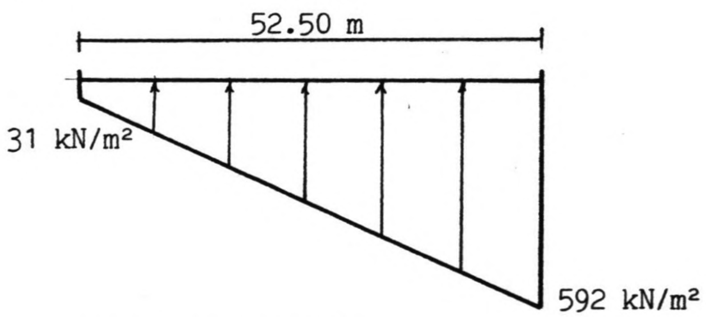
resulting safety: $\gamma = 1.4$

9.2.2. Turning over.

To check the stability against turning over, the resulting ground-pressure for the 100-year design load (§ 7.0) is calculated. A safety factor of 1.3 is taken into account.

100-year design load: $M = 104577$ kNm/m', $z = 13.2$ m

$\gamma = 1.3 \Rightarrow M_u = 135950$ kNm/m'



resulting pressure.

9.3 Stability during transport.

For safe transport with tugboats, sufficient selfturning capacity is needed. During transport, due to wind- and waveïnfluences, the caisson will start rolling. An increase of these movements will take place when the frequency of the waves and the natural resonance frequency of the caisson are in the same range. Both, the stability, when drifting, and the stability in waves, are considered.

9.3.1 Stability when drifting.

The stability of a drifting caisson depends on the height of the centre of boyance B, the centre of gravity G and the metacentre-height, h_m. Ballance is illustrated in figure .

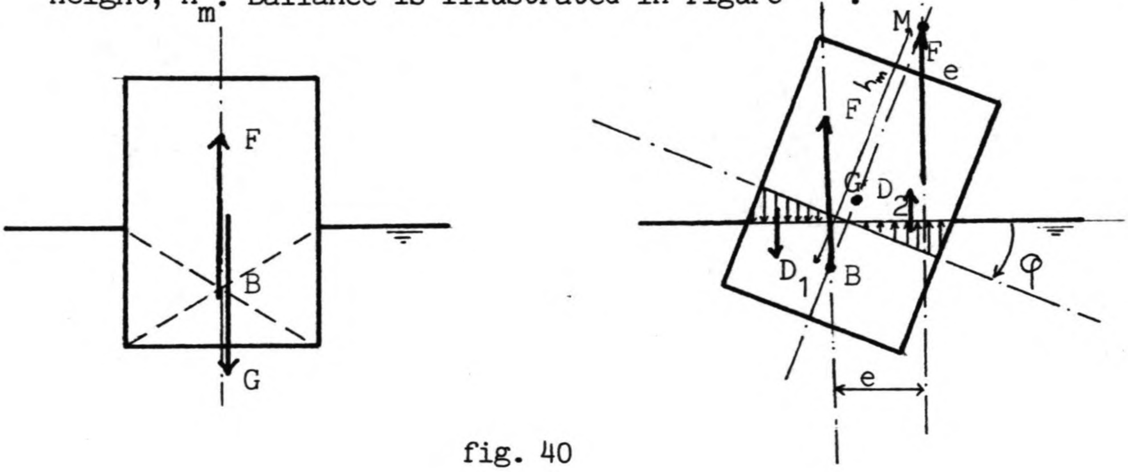
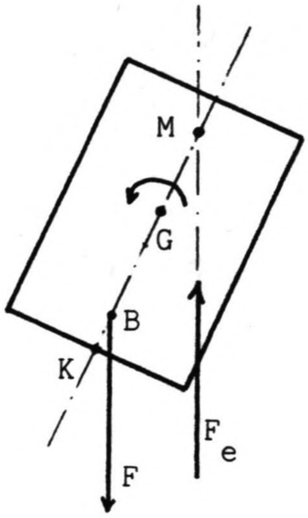
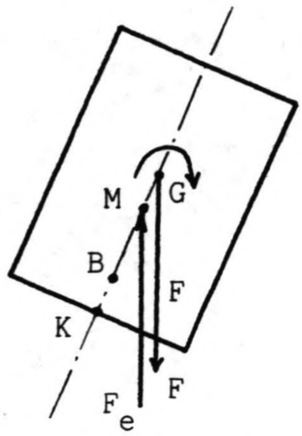


fig. 40

When rotated over an angle φ , the caisson will be loaded by extra waterpressures (fig. 40). The forces D_1 and D_2 can be combined to a force F_e of the same size as F, but at a distance e of the original centre of boyance B. In extension of F_e and the axis of symmetry of the caisson, the metacentre M is found. The distance MG is called the metacentreheight h_m. The caisson has selfturning capacity, when the metacentreheight is positive (fig. 41). A usual criterion for sufficient stability is a metacentreheight bigger than one meter.



positive metacentreheight



negative metacentreheight

fig. 41

Calculation of the metacentreheight.

The forces D_1 and D_2 bring to bear a momentum:

$$D_x = \rho_w\, g\, x \tan\varphi \quad dx \qquad M_x = \rho_w\, g\, x^2 \tan\varphi\, dx$$

and $$M = \int_{-\frac{1}{2}b}^{\frac{1}{2}b} M_x\, dx = I\, \rho_w\, g \tan\varphi$$

Combination of M aan F gives a force F_e at a distance e from the centre of boyance B:

$$F_e * e = \rho_w\, I \tan\varphi$$

F_e is equivalent to the boyance force F:

$$F_e = F = \rho_w\, g\, V$$

From these equations follows:

$$e = \frac{I}{V} \tan\varphi$$

From figure 40 can be concluded:

$$\sin\varphi = \frac{e}{h_m + BG}$$

The resulting metacentreheight is:

$$h_m = \frac{I}{V \cos\varphi} - BG \qquad \frac{I}{V} - BG \qquad (\varphi = \text{small})$$

$$GM = h_m = BM + KB - KG$$

with: I = inertia of the caissonpart under water. (m^4)
V = volume of the replaced water. (m^3)

Calculation.

draft of the caisson: 18.8 m   centre of boyance height: KB = 9.4 m
centre of gravity height: KG = 18.1 m

Resulting stability parallel to the turbine in- and outtakedirection:

$BM_{//} = 12.2$ m $h_{m\,//} = 3.5$ m

The stability perpendicular to the turbine in- and outtakedirection depends on the amount of turbinesections per caisson (x). A review of the stability for several caissondimensions is given in figure .

x	length of the caisson (m)	width of the caisson (m)	$BM_{\perp}$ (m)	$h_{m\perp}$ (m)
1	52.5	17	1.1	-7.6
2	52.5	33	4.8	-3.9
3	52.5	49	10.7	2.1

For all calculations, installation of the turbines after lowering of the caisson is assumed. The turbines are placed low in the caisson and thus have a stabilising effect during transport. A caisson of three turbinesections proves to have sufficient stability in both directions. Smaller caissons have to be stabilised, bigger ones are less manoeuvreble. A caisson of three turbinesections is chosen. The stability of upper and lower part of the caisson, before placement on top of each other,is checked aswell. Both prove to be sufficient (fig.42).

upper part:

draft: 8.1 m $\Longrightarrow$ KB = 4.1 m $\qquad BM_{\perp} = 24.7 \qquad h_{m\perp} = 20.8$ m
centre of gravity: KG = 8.0 m $\qquad BM_{//} = 28.4 \qquad h_{m//} = 24.5$ m

lower part:

draft: 8.7 m $\Longrightarrow$ KB = 4.4 m $\qquad BM_{\perp} = 23.0 \qquad h_{m\perp} = 16.6$ m
centre of gravity : KG = 10.8 m $\qquad BM_{//} = 26.4 \qquad h_{m//} = 20.0$ m

fig. 42

9.3.2 Stability in waves.

Amplification of the movements of the caisson takes place when the natural resonance frequency of the caisson and the frequency of the waves are in the same range (fig.43).

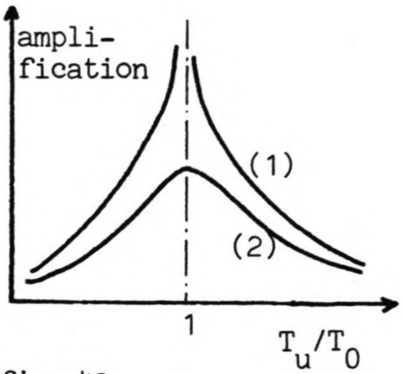


fig. 43

(1) without demping
(2) with demping

T_u = waveperiod
T_0 = natural resonance frequency of the waves.

Calculationmethod to determine the natural resonance frequency of the caisson.

The movements of the caisson can be described with the following formula's:

$$\left.\begin{array}{l} I_p \ddot{\varphi} + h_m\, m\, g = 0 \\ I_p = j^2\, m \end{array}\right\} \Longrightarrow j^2\, m \ddot{\varphi} + h_m\, m\, g = 0$$

with: I_p = polar momemtum of inertia. (m^4)
m = mass. (kg)
j = polar radius of giration round the horizontal axis through the centre of gravity. (m)
$\left(j = \sqrt{\frac{I_p}{A}}\right)$

Movements within the range of the natural resonance frequency of the caisson will be amplified. These can be described as:

$$\varphi(t) = \hat{\varphi} \sin \omega t$$

with: ω = natural resonance frequency. (rad/s)
$\varphi(t)$= rolling movement. (°)
$\hat{\varphi}$ = maximum angle. (°)

This results in:

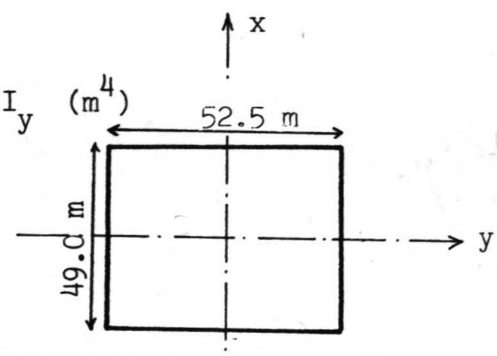
$$-\hat{\varphi}\,\omega^2\, j^2 \sin \omega t = h_m\, g \sin \omega t$$

The natural resonance period is:

$$\omega = \frac{1}{j}\sqrt{h_m\, g} \qquad T_o = \frac{2\pi j}{\sqrt{h_m\, g}} \quad \frac{2j}{\sqrt{h_m}} \qquad (\pi \approx \sqrt{g})$$

<u>Calculation.</u>

polar momentum of inertia: $I_p = I_x + I_y$ (m^4)



$$\begin{aligned} I_x &= 514714 \; m^4 \\ I_y &= 590871 \; m^4 \quad + \\ \hline I_p &= 1105585 \; m^4 \\ A &= 2573 \; m^2 \end{aligned} \Bigg\} \Longrightarrow j = 20.7 \text{ m}$$

The resulting natural resonance frequency is:

-in the direction of the turbine in- and outtakes: T_o = 28.6 s.
-perpendicular to the turbine in- and outtakedirection: T_o = 22.1 s.

Transport and ballasting will never take place at waveheights higher than 2-3 meters. These waves have periods up to 4-5 seconds. No amplification of the movements of the caisson in the waves is expected.

9.4 Stability during ballasting

In a tidal estuary, sinking of the caisson has to be completed within the slack water period. As closure proceeds, the closuregap will become narrower and the slack water period will decrease. This means the caissons will have to be placed within a very short period. For this reason, ballasting of the caisson will preferably be done with water. Water as ballasting material gives better controlancepossibilities over the sinkingmanoeuvre than firm materials like gravel. When using water to ballast the caisson, a free watersurface in the caissoncompartments is formed. This reduces the stability.

Calculationmethod of the stability during ballasting.

The ballastwater in the caissoncompartments reduces the selfturning capacity (fig. 44).

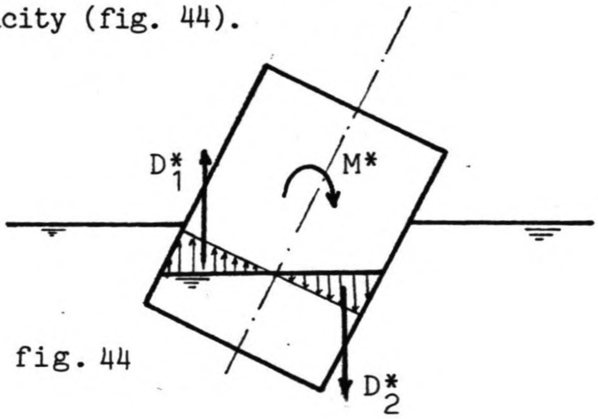


fig. 44

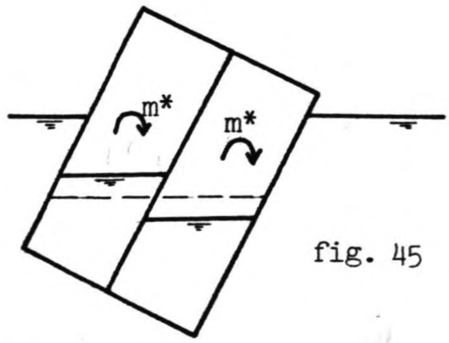


fig. 45

The resulting selfturning capacity is: M - M*

with: M = selfturning capacity
M* = momentum of the ballastwater.

Using the same equations as in , the resulting metacentre-height is:

$$h_m = BM_{res.} + KB - KG \quad (m)$$

and $BM_{res.} = BM_{caisson} - BM_{ballastwater}$

This means: $BM_{res.} = \dfrac{I - \sum i}{V}$ fig. 45

with: i = momentum of inertia of the ballastwater in a compartment. (m^4)

Calculation.

Compartilising of in-and outtake is needed. Placement of an extra division plate halfway is assumed (fig.46).

draft (m)	KB (m)	$BM_{\perp}$ ballast (m)	$BM_{//}$ water	$h_{m\perp}$ resulting metacentre (m)	$h_{m//}$
20	10	0.8	1.1	1.8	3.0
25	12.5	0.8	4.5	4.4	2.1
30	15	0.6	3.8	7.0	5.3
35	17.5	0.5	1.6	9.6	10.0
40	20	0.5	1.9	12.1	12.2

unballasted caisson: centre of gravity: KG = 18.0 m

$BM_{\perp}$ = 10.7 m

$BM_{//}$ = 12.2 m

The stability during ballasting proves to be sufficient in both directions.

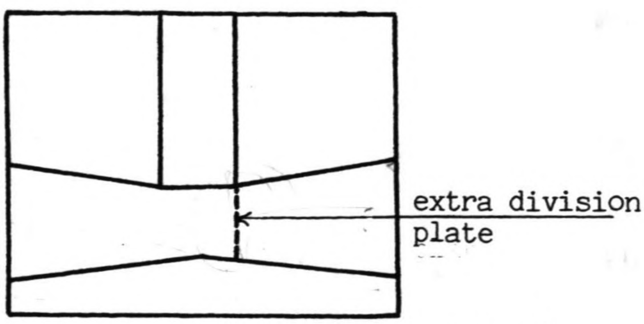


fig. 46

9.5 Dimensioning of the caisson.

9.5.1 Introduction.

For the dimensions of the caissonwalls, the following loadingsituations are taken into account:

1. extreme local loads over a period of 100-year.
2. loading during transport.
3. loads when a turbine is under repair.

1. Extreme local 100-year loads.

The caissondesign is based on the extreme local pressures due to the 100-year wave- and hydrostatic loads. Figure 47 gives a review. The outerwalls are loaded by iceloads aswell (§ 7.0). The upperdeck has to be able to carry the loads of breaking waves and the loads of a crane used to lift out the turbines if needed. These loads are calculated in § 9.5.3 and § 9.5.4.

2. Loading during transport.

During transport, due to leakage, one of the compartments might be filled with water. In that situation, when the divisionplates break, the stability of the caisson might be endangered. Thus these plates are calculated for the full hydrostatic waterpressure.

3. Loading when a turbine is under repair.

When a turbine is under repair, the water will be pumped outoff the shaft and the in- and outtake sections. The strength of the walls has to be checked for this loadingsituation.

9.5.2 Extreme local pressures of the 100-year wave- and hydrostatic loads.

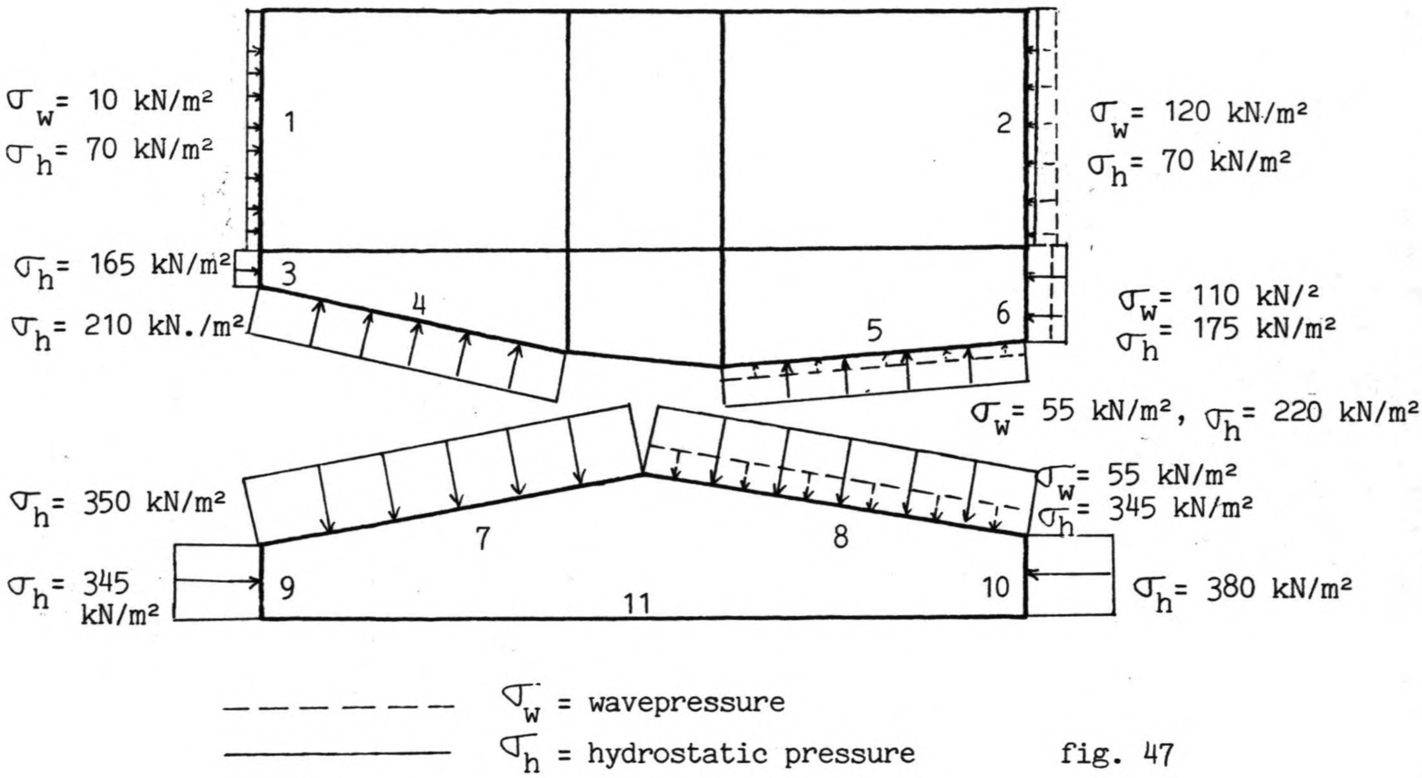


fig. 47

9.5.3 Loads of breaking waves on top of the caisson.

At HW, waves from seaside will break on top of the caisson. For the design of the deck, a 100-year wave is taken into account:

$$H_{1\%} = 12.1 \text{ m}$$

Loads of breaking waves can be estimated with the following formula:

$$p = \rho_w \, g \, q \, H \qquad (N/m^2)$$

with: P = maximum pressure. (N/m²)
H = waveheight of the undisturbed wave. (m)
q = coefficient dependent on α .
α = slope angle

V

The waves are interrupted abruptly; a high q-value, 2.5, is assumed. The width, over which the maximum pressure devellops, approximately is 0.4 H.

Calculation:

$$\left.\begin{array}{l} q = 2.5 \\ b = 0.4\ H = 4.8\ m \\ H_{1\%} = 12.1\ m \end{array}\right\} \Longrightarrow \begin{array}{l} p = 302\ kN/m^2 \\ F = 1452\ kN/m' \end{array}$$

9.5.4 Crane load.

To lift out the turbines, a mobile gantry usually is used. The capacity needed is 15.000 kN. The weight of the crane on the wall of the shaft is estimated as follows:

cranecapacity:	15.000 kN
craneweight (1.5*capacity):	22.500 kN +
total:	37.500 kN
safety factor γ = 1.5:	56.250 kN
craneload on the wall of the shaft:	28.125 kN

9.5.5 Calculations.

Based on these loads, an estimate of the caissondimensions is made up. For these first estimates, the local loads resulting from the impact of icefloes (§ 6.5) and the loading of breaking waves on top is not taken into account. A review of the calculations is given in appendix 12.

9.6 Connection of the upper and lower part of the caisson.

The joint between the upper and lower part of the caisson has to be able to carry the loading on the upper part. The extreme loadings-situations and loads (freq. 10^{-4}) are:

hydrostatic loads:	F (kN/m')	M (kNm/m')	wave loads:	F (kN/m')	M (kNm/m')
HW sea side(+9.50 m) LW basin side(+0.15 m)	1069	5707	$H_{1\%}$ = 12.1 m sea side	1200	6720
LW seaside(-6.95 m) HW basin side(+7.50 m)	1235	6923	$H_{1\%}$ = 4.4 m basin side	400	3200

The resulting design load is: F = 2270 kN/m'

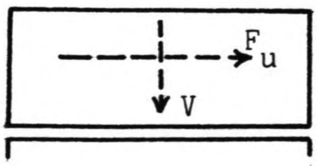
M = 12430 kNm/m', from sea side.

M_u = 18645 kNm/m', F_u = 3405 kN/m' (γ = 1.5)

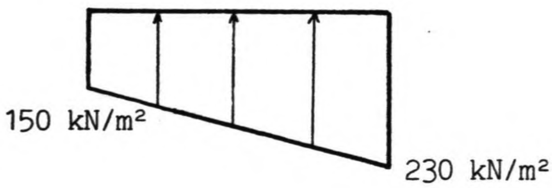
Under designconditions, the mean waterlevel over the caisson is +4.60 m GSCD; the weight of the caisson in the water is:

dry weight of the upper part:	12920 kN/m'
boyance force:	2888 kN/m' -
effective weight:	10032 kN/m'

The connection is loaded as follows:



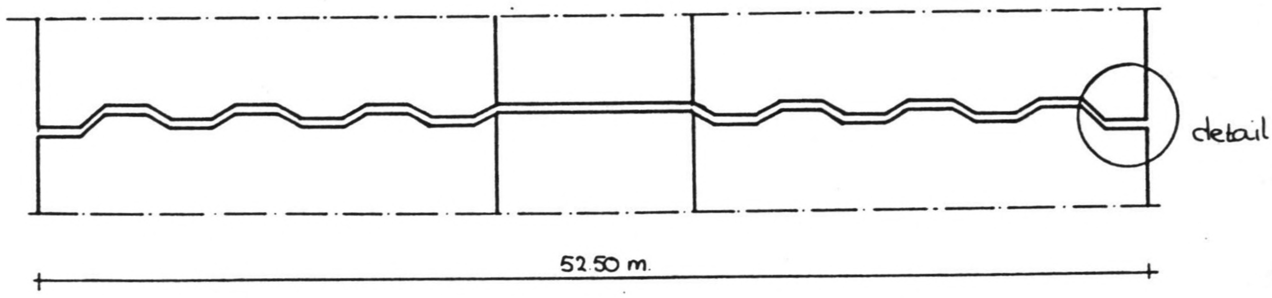
Designloads

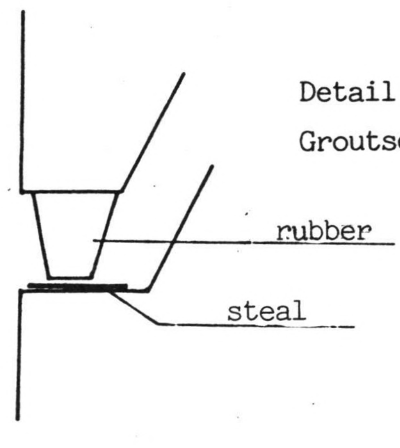


Resulting pressure

The joint will always be loaded by shear- and pressureforces, never by tension. The following construction is suggested:

Joint upper and lower part of the caisson.



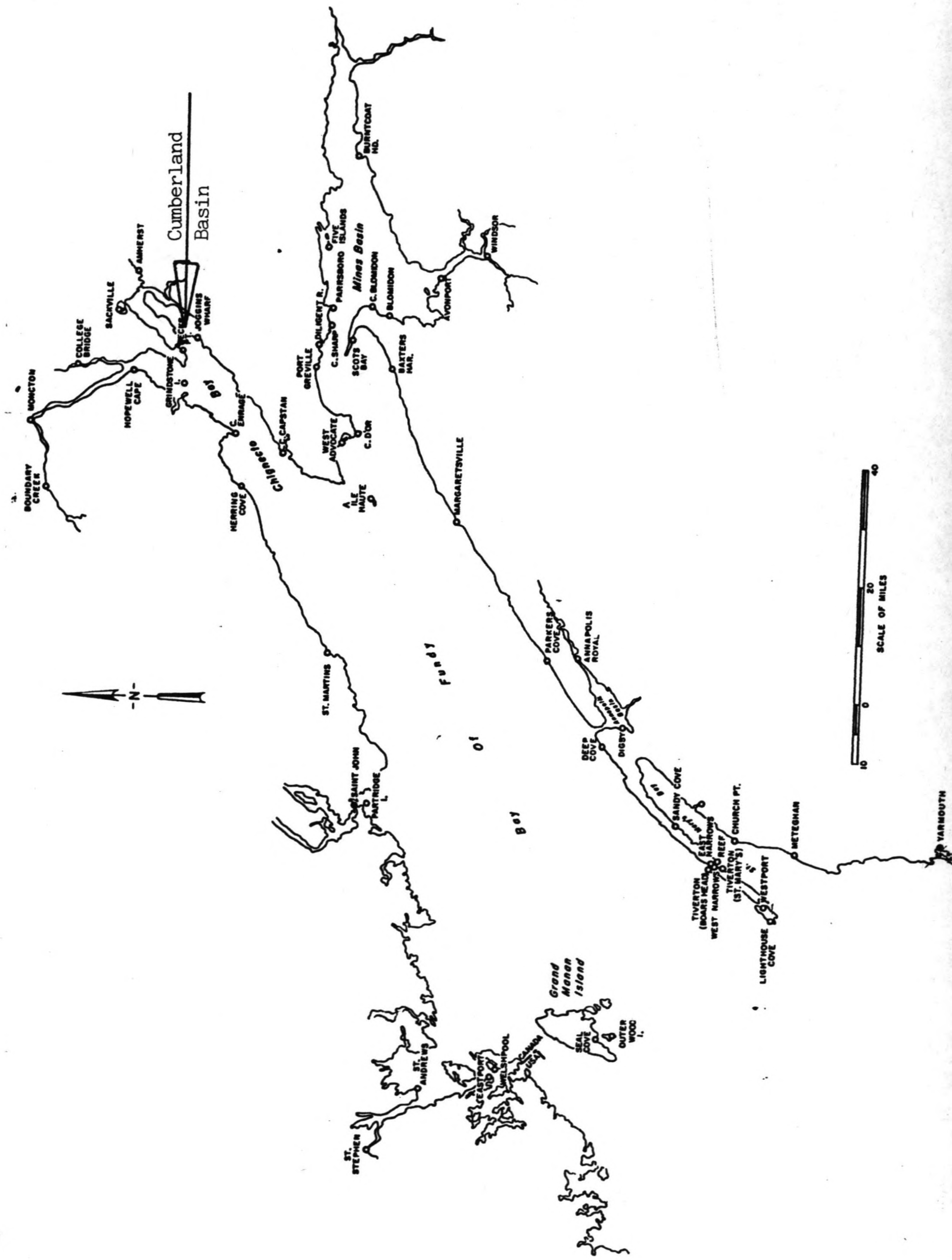


Detail A.
Groutseal.

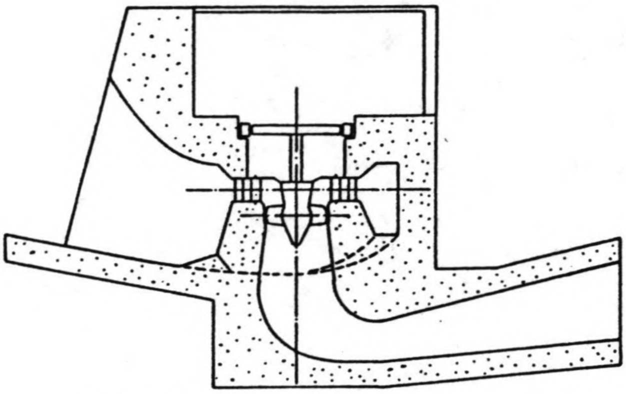
APPENDICES

(All levels refer to GSCD.)

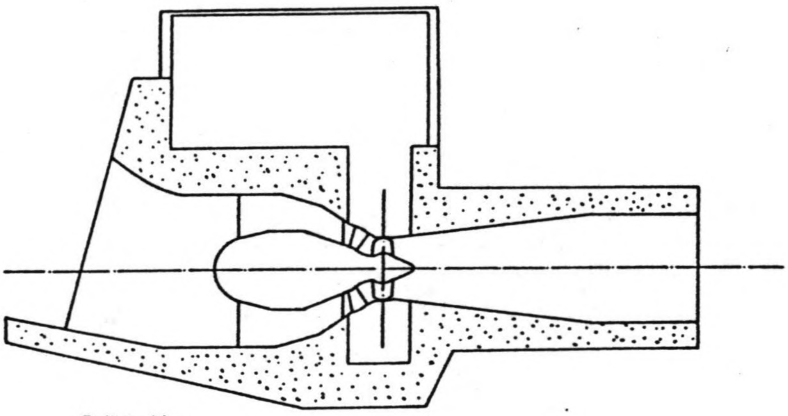
Appendix 1


Cumberland Basin
-N-
AMHERST
SACKVILLE
COLLEGE BRIDGE
MONCTON
BOUNDARY CREEK
HOPEWELL CAPE
GRINDSTONE I.
JOGGINS WHARF
C. ENRAGE
C. CAPSTAN
Chignecto
Bay
HERRING COVE
WEST ADVOCATE
C. D'OR
ILE HAUTE
PORT GREVILLE
C. SHARP
PARRSBORO
FIVE ISLANDS
BURNTCOAT HD.
Mines Basin
C. BLOMIDON
BLOMIDON
SCOTS BAY
BAXTERS HAR.
AVONPORT
WINDSOR
MARGARETSVILLE
ST. MARTINS
Bay of Fundy
SAINT JOHN
PARTRIDGE I.
PARKERS COVE
ANNAPOLIS ROYAL
DEEP COVE
DIGBY
SANDY COVE
EAST NARROWS
WEST NARROWS
CHURCH PT.
METEGHAN
YARMOUTH
WESTPORT
LIGHTHOUSE COVE
Grand Manan Island
SEAL COVE
OUTER WOOD I.
ST. ANDREWS
ST. STEPHEN
WELSHPOOL
EASTPORT
SCALE OF MILES
10
0
20
40

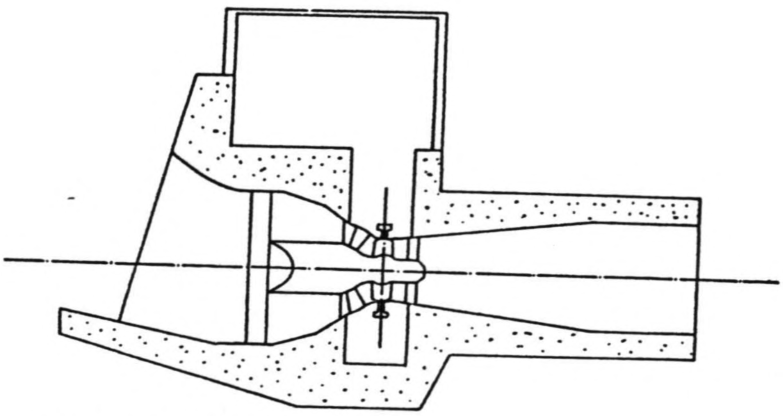
Appendix 2



Kaplan turbine

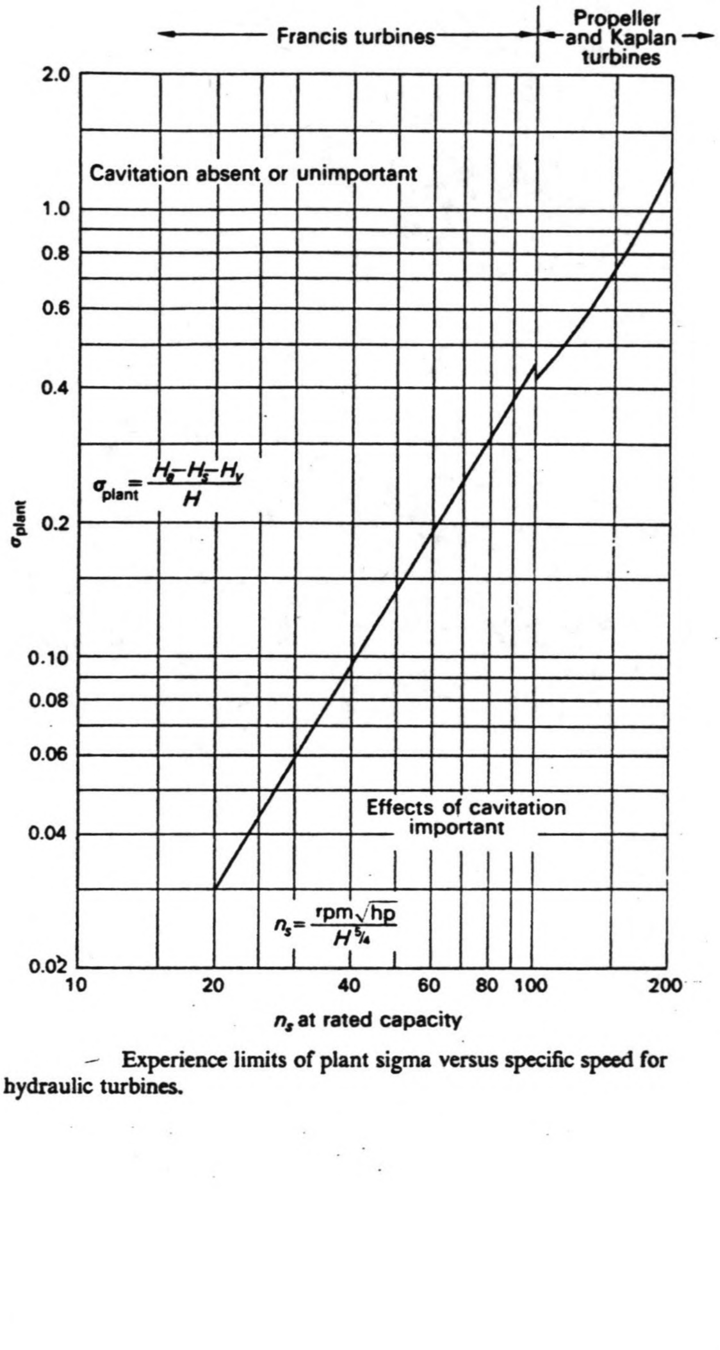


Bulb turbine



Straight flow turbine (Rim generator)

Appendix 3

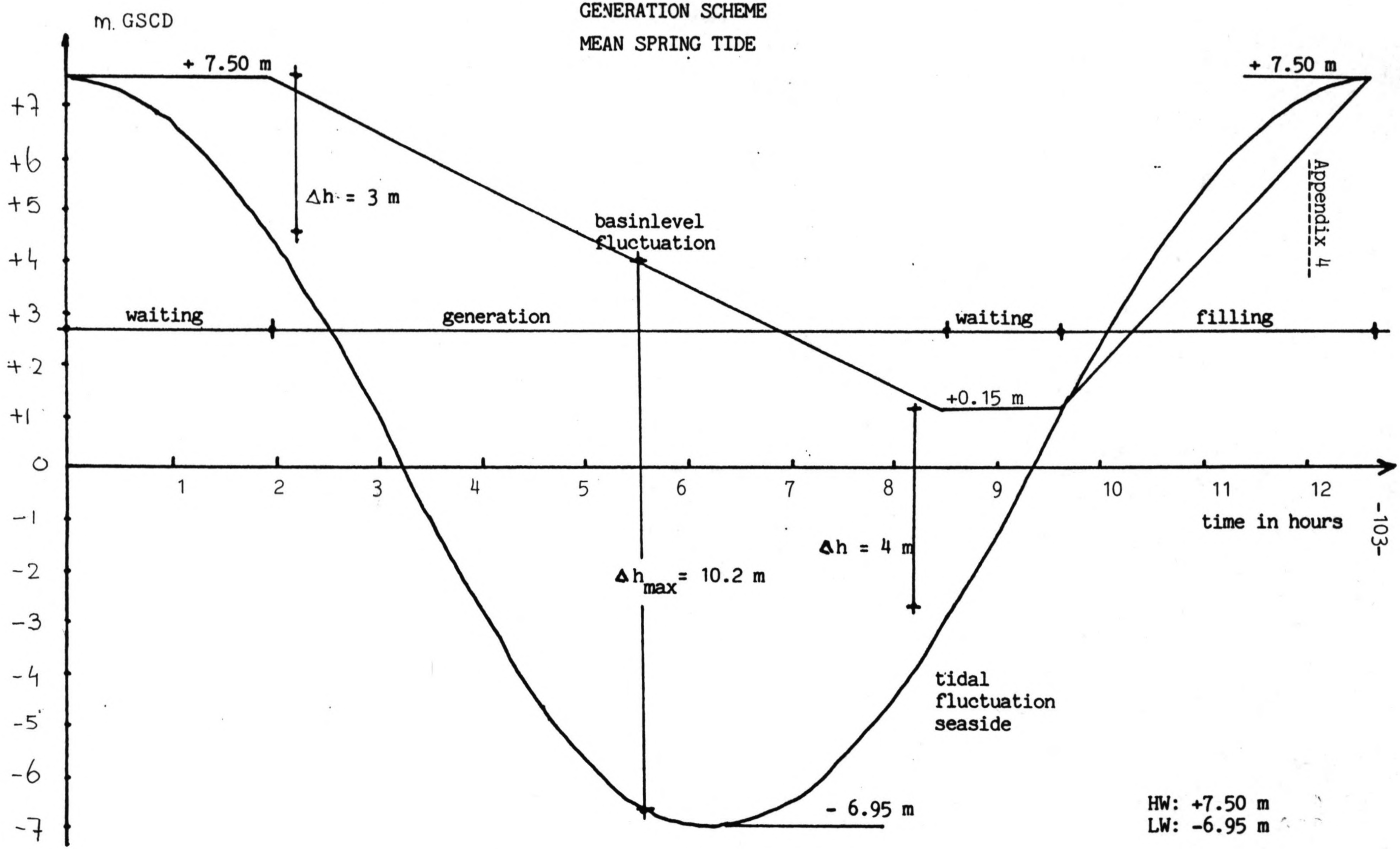


— Experience limits of plant sigma versus specific speed for hydraulic turbines.

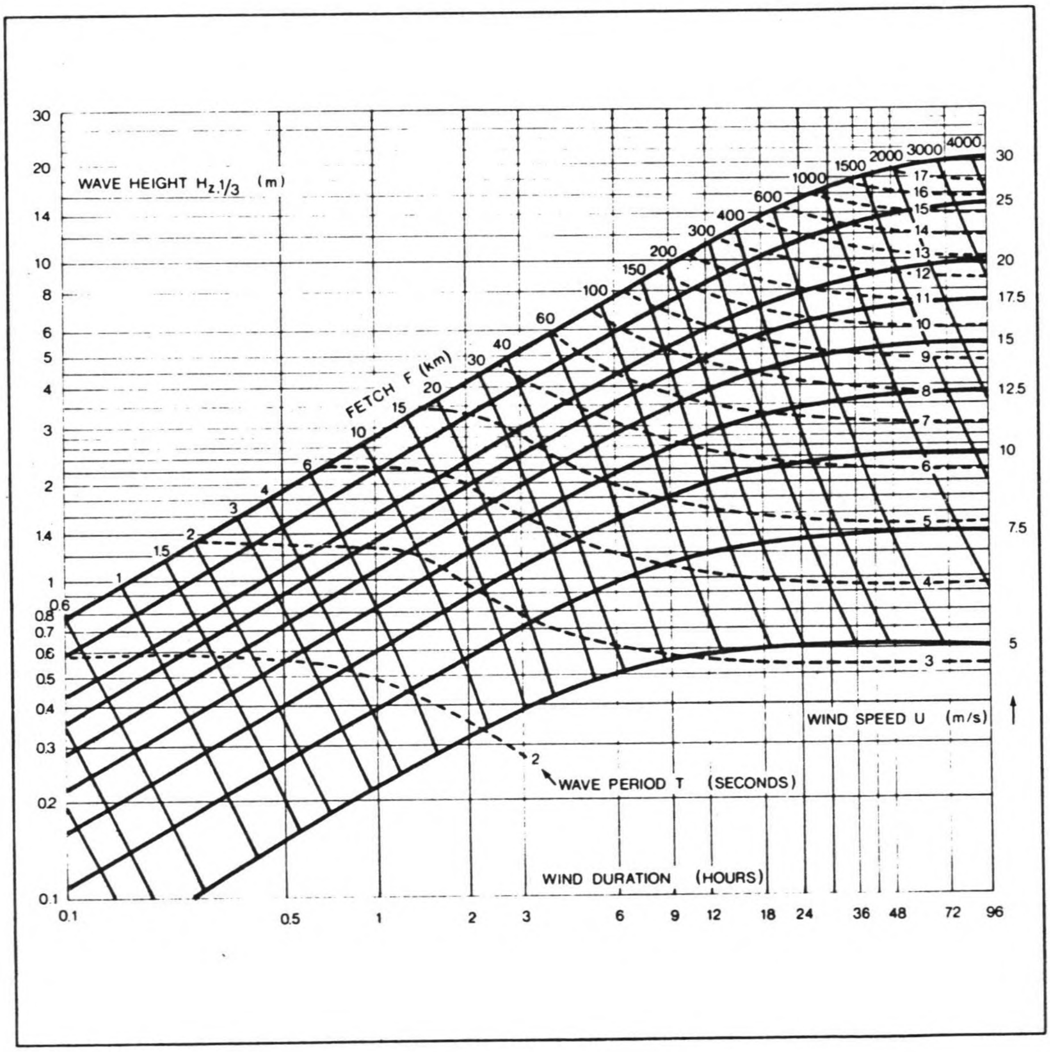
Appendix 4

GENERATION SCHEME

MEAN SPRING TIDE

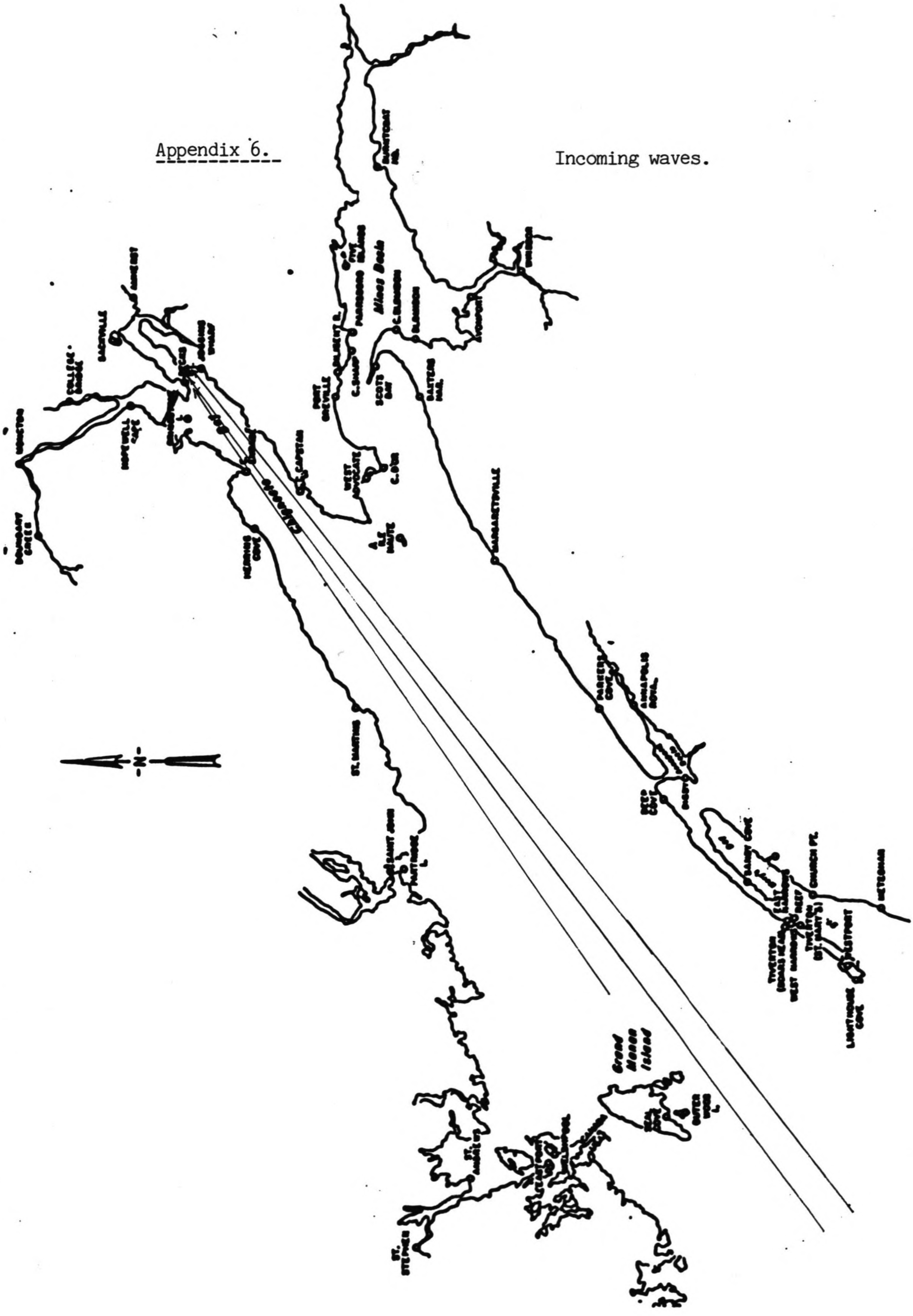


Appendix 5. Wave forecasting graph

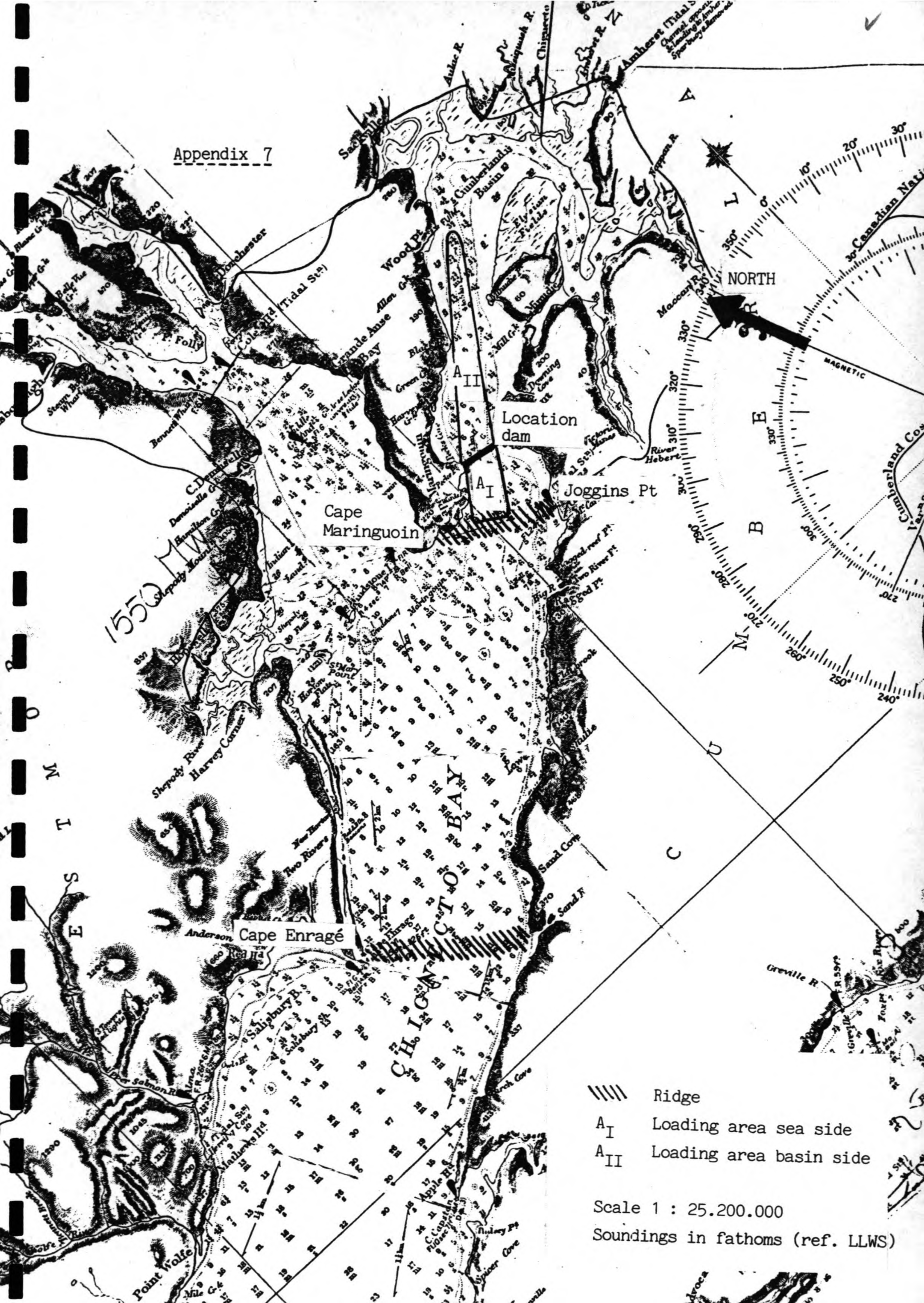


Appendix 6.

Incoming waves.



Appendix 7


NORTH
A_{II}
Location dam
A_I
Joggins Pt
Cape Maringuoin
Cape Enragé
CHIGNECTO BAY
Ridge
A_I Loading area sea side
A_{II} Loading area basin side
Scale 1 : 25.200.000
Soundings in fathoms (ref. LLWS)

Appendix 8

The slope of the watersurface is estimated as follows:

$$u = C\sqrt{R\,S} \Longrightarrow S = \frac{u^2}{C^2\,R}$$

with: u = mean velocity under the ice deck. (m/s)
C = Chezy-coefficient ($/s^2$)
R = hydraulic radius (m)
S = surface slope

In very wide rivers with an ice deck: $R \approx 0.5\ d$ (d=waterdepth)

The Chezy-coefficient is determined with:

$$C = 18 \log \frac{12\ R}{k}$$

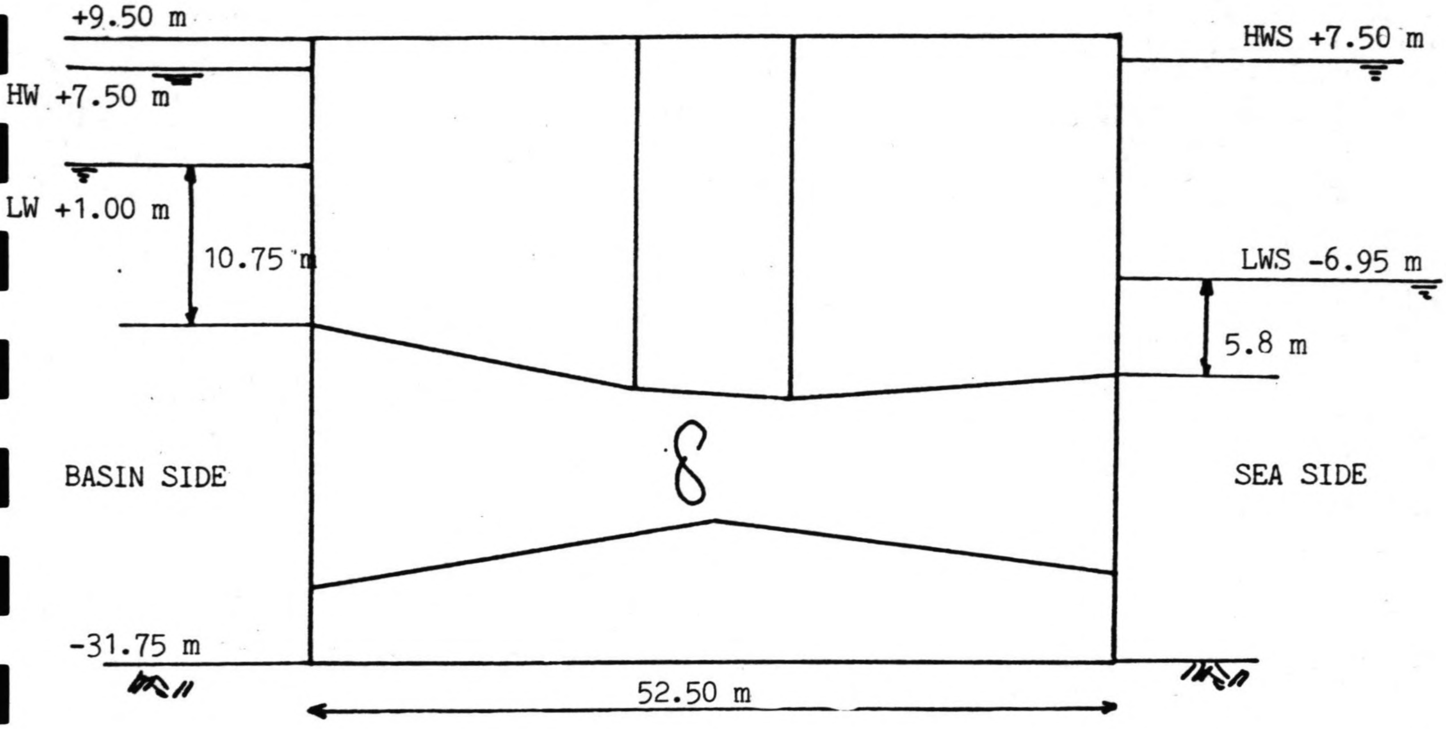
with: k = roughnesscoefficient of Nikauradse

$$\left.\begin{array}{l} k_{ice} = 10^{-2} \\ k_{bottom} = 3 * 10^{-2} \end{array}\right\} \Longrightarrow k_{total} = 2 * 10^{-2}$$

Sea side: waterdepth: $d = 22$ m; flowvelocity: $v = 1.5$ m/s $\Longrightarrow S = 4.3 * 10^{-5}$

Basin side waterdepth: $d = 12$ m; flowvelocity: $v = 1.2$ m/s $\Longrightarrow S = 5.9 * 10^{-5}$

Appendix 9 Pile up.



max. pile-up (m)	plant not working	working	period
LW sea side	1.2	-	generation
LW basin side	1.7	4.9	end generation
HW sea side	1.2	3.0	end filling
HW basin side	1.7	2.0*	begin generation

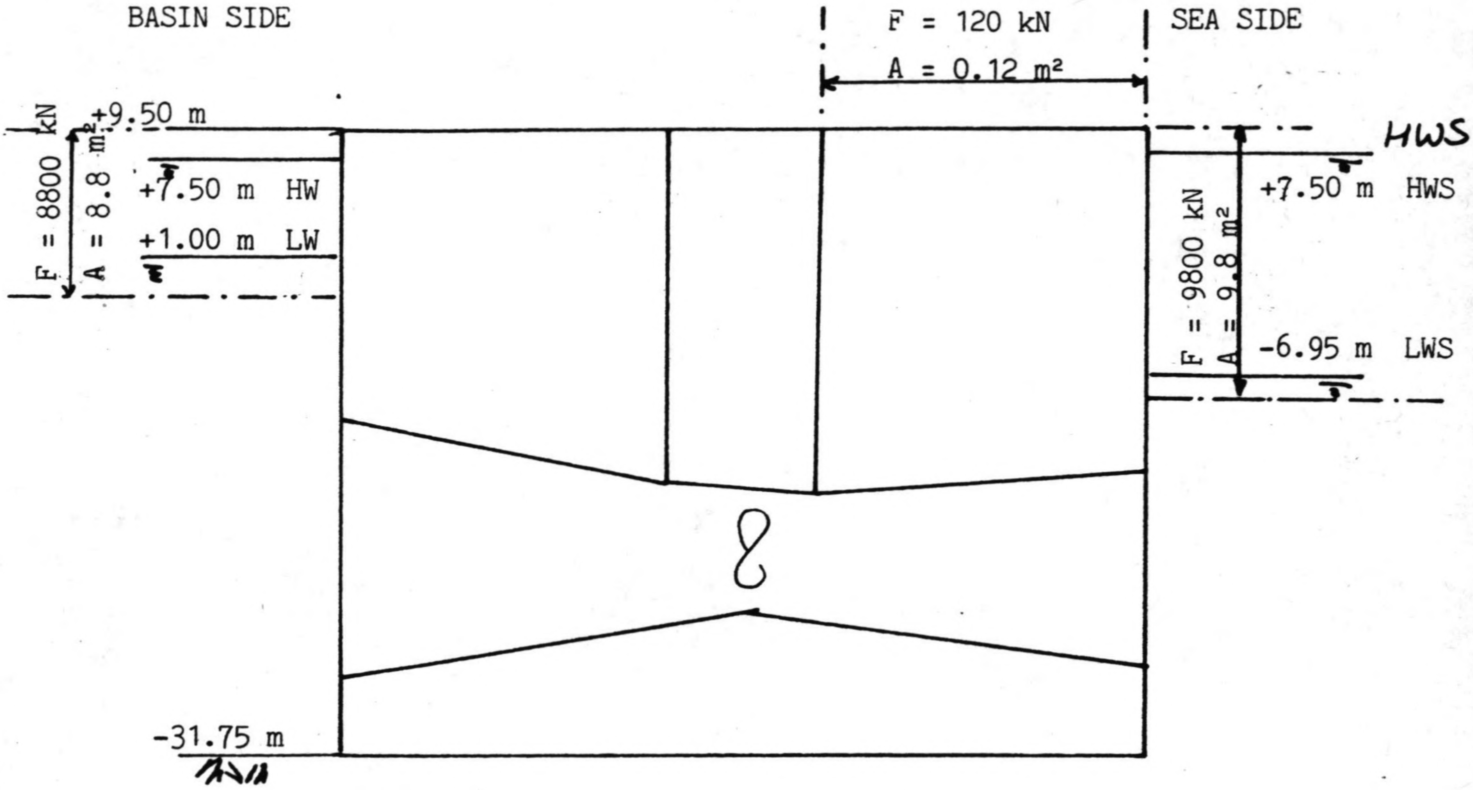
*(max. during generation is 5.0 m)

Sailheight h_s - keelheight h_k relation is: $h_s : h_k = 1 : 3$

No overtopping nor ice-inflow into the turbinesections is expected.

Appendix 10

Impact loads.



F = impactload. (kN)

A = contactarea. (m^2)

Appendix 10

Impact loads of various floe sizes.

Basin side: flowvelocity: 1.2 m/s
windvelocity: 25 m/s (NE) } ⇒ v_{floe} = 1.7 m/s
crushingstrength: 1 MPa

floe radius (m)	weight $*10^6$(kg)	penetration (m)	load (kN)
50	7.0	0.19	8800
40	4.5	0.16	7000
30	2.5	0.12	5300
20	1.3	0.09	3700

Sea side flowvelocity: 1.5 m/s
windvelocity: 25 m/s (SW) } ⇒ v_{floe} = 2.0 m/s
crushingstrength: 1 MPa

floe radius (m)	weight $*10^6$(kg)	penetration (m)	load (kN)
50	7.0	0.19	9800
40	4.5	0.16	7800
30	2.5	0.12	5900
20	1.3	0.09	4100

Appendix 11 Estimate caisson weight and draft.

Ballasted caisson:

concrete and ballast:	31019
water:	5775
turbines and valves:	938 +
Total dry weight:	37732
Boyance force HW:	20554 +
Effective weight (HW):	17178

Transport:

weight:	9844 kN/m'
draft with turbine:	20.6 m
draft without turbine:	18.8 m
centre of gravity	18.1 m

Lower part:

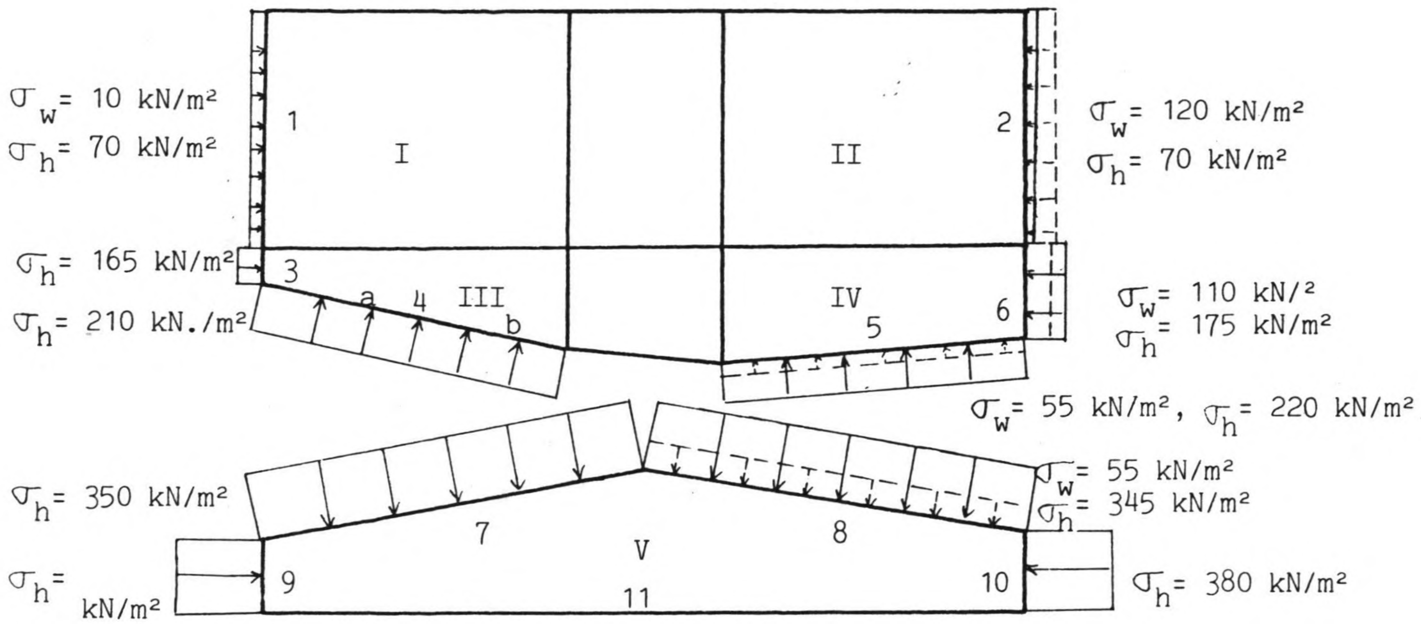
weight:	4219 kN/m'
draft with turbine:	9.9 m
draft without turbine:	8.7 m
centre of gravity:	10.8 m

Upper part:

weight:	4250 kN/m'
draft:	8.1 m
centre of gravity:	8.0 m

Appendix 12. Review calculations caisson dimensions

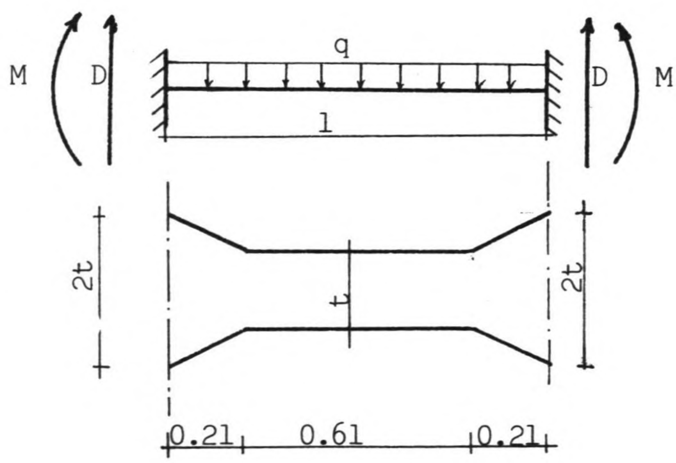
Extreme local pressures of the 100-year wave- and hydrostatic loads.



σ_w = wavepressure

σ_h = hydrostatic pressure

Assumptions:



$M = 0.1\ ql^2$

$D = \frac{1}{2}\ ql$

Platethickness

Shearstrength concrete: 550 kN/m²

Concrete: B 17.5

Outerwalls sea- and basin side:

Section 1: 7.5-15.0 m

Loads:

Constructionfase:

waves:	10 kN/m²
hydrostatic:	70 kN/m² +
total:	80 kN/m²

During use:

waves:	35 kN/m²
hydrostatic:	70 kN/m²
ballast:	45 kN/m² +
total:	60 kN/m²

$q = 80$ kN/m² $\quad q_u = 136$ kN/m²

$D_u = 510$ kN $\quad 2t = 1.00$ m, $t = 0.50$ m

$M_u = 765$ kNm $\quad \frac{M_u}{bh^2} = 765$

Section 2: 7.5-15.0 m

Loads:

waves:	120 kN/m²
hydrostatic	80 kN/m²
ballast:	45 kN/m² +
total:	155 kN/m²

$q = 155$ kN/m² $\quad q_u = 264$ kN/m²

$D_u = 988$ kN/m² $\quad 2t = 1.80$ m, $t = 0.90$ m

$M_u = 1482$ kNm $\quad \frac{M_u}{bh^2} = 457$

Section 3: 3.0-15.0 m

Load: 165 kN/m²

$q = 165$ kN/m² $\quad q_u = 280$ kN/m²

$D_u = 761$ kN/m² $\quad 2t = 1.40$ m, $t = 0.70$ m

$M_u = 430$ kNm $\quad \frac{M_u}{bh^2} = 219$

<u>Section 4:</u> 10.0-15.0 m

Loads:	part a	part b
hydrostatic:	210 kN/m²	210 kN/m²
waves:	70 kN/m² +	85 kN/m² +
total:	140kN/m²	125 kN/m²

part a:	$q = 140$ kN/m²	$q_u = 238$ kN/m²
	$D_u = 1190$ kN	$2t = 2.2$ m, $t = 1.1$ m
	$M_u = 2380$ kNm	$\frac{M_u}{bh^2} = 492$
part b:	$q = 125$ kN/m²	$q_u = 213$ kN/m²
	$D_u = 1062$ kN/m²	$2t = 1.95$ m, $t = 1.00$ m
	$M_u = 2130$ kNm	$\frac{M_u}{bh^2} = 560$

<u>Section 5:</u> 6 .0-15.0 m

Loads:	hydrostatic:	275 kN/m²
	ballast:	85 kN/m² +
	total :	190 kN/m²

$q = 190$ kN/m²	$q_u = 323$ kN/m²
$D_u = 969$ kN/m²	$2t = 1.80$ m, $t = 0.90$ m
$M_u = 1163$ kNm	$\frac{M_u}{bh^2} = 359$

<u>Section 6:</u> 5.0-15.0 m

Loads:	hydrostatic:	285 kN/m²
	ballast:	14 kN/m² +
	total:	271 kN/m²

$q = 271$ kN/m²	$q_u = 461$ kN/m²
$D_u = 1152$ kNm	$2t = 2.10$ m, $t = 1.05$ m
$M_u = 1153$ kNm	$\frac{M_u}{bh^2} = 261$

Section 7: $5.0-15.0$ m

Load: hydrostatic: 350 kN/m²

$q = 350$ kN/m² $q_u = 595$ kN/m²

$D_u = 1488$ kN $2t = 2.70$ m, $t = 1.35$ m

$M_u = 5950$ kNm $\frac{M_u}{bh^2} = 816$

Section 8: $5.0-15.0$ m

Load: hydrostatic: 400 kN/m²

$q = 350$ kN/m² $q_u = 595$ kN/m²

$D_u = 1190$ kN $2t = 2.2$ m, $t = 1.1$ m

$M_u = 952$ kNm $\frac{M_u}{bh^2} = 197$

Section 10: $5.0-15.0$ m

Loads

hydrostatic:	430 kN/m²
ballast:	14 kN/m² +
total:	416 kN/m²

$q = 416$ kN/m² $q_u = 707$ kN/m²

$D_u = 1768$ kN $2t = 3.2$ m, $t = 1.6$ m

$M_u = 1768$ kNm $\frac{M_u}{bh^2} = 173$

Section 11: $5.0-15.0$ m

Loads (LW):

dry weight caisson with ballast:	37732 kN/m'
boyance force:	15750 kN/m' +
effective weight:	16710 kN/m'

This is: 318 kN/m²

Loads:

groundpressure:	318 kN/m²
ballast:	68 kN/m² +
total:	250 kN/M²

$q = 250\ kN/m^2$ $q_u = 425\ kN/m^2$

$D_u = 1063\ kN$ $2t = 1.95\ m,\ t = 1.00\ m$

$M_u = 1063\ kNm$ $\frac{M_u}{bh^2} = 279$

Outerwalls sides:

Designcondition: just the hydrostatic pressure during transport.

Wall I 15.0-15.0 m

Load: 70 kN/m^2

$q = 70\ kN/m^2$ $q_u = 119\ kN/m^2$

$D_u = 893\ kN$ $2t = 1.6\ m,\ t = 0.8\ m$

$M_u = 2678\ kNm$ $\frac{M_u}{bh^2} = 1046$

Wall III:

Load: 190 kN/m^2

$q = 190\ kN/m^2$ $q_u = 323\ kN/m^2$

part a: $D_u = 485\ kN$ $2t = 0.90\ m,\ t = 0.45\ m$

3.0-10.0 m $M_u = 291\ kNm$ $\frac{M_u}{bh^2} = 395$

part b: $D_u = 808\ kN$ $2t = 1.5\ m,\ t = 0.75\ m$

5.0-10.0 m $M_u = 808\ kNm$ $\frac{M_u}{bh^2} = 359$

Wall IV: 6.0-14.0 m

Load 190 kN/m^2 190 kN/m^2

$q = 190\ kN/m^2$ $q_u = 323\ kN/m^2$

$D_u = 969\ kN$ $2t = 1.80\ m,\ t = 0.90\ m$

$M_u = 1163\ kNm$ $\frac{M_u}{bh^2} = 359$

Wall V: 5.0-8.0 m

Load: 380 kN/m²

$q = 380$ kN/m² $q_u = 610$ kN/m²
$D_u = 1525$ kN $2t = 2.8$ m, $t = 1.4$ m
$M_u = 1525$ kNm $\frac{M_u}{bh^2} = 195$

Shaftwalls: 7.5-15.0 m

Ballastload: 128 kN/m²

$q = 128$ kN/m² $q_u = 218$ kN/m²
$D_u = 818$ kN $2t = 1.5$ m, $t = 0.75$ m
$M_u = 2180$ kNm $\frac{M_u}{bh^2} = 969$

Craneload: 28125 kN Contactarea of 2 m² needed.

Plates:

Loadingconditions: 1. transportcondition: full hydrostatic pressure.
2. definitive situation

Plate I & II: 15.0-15.0 m

Load (transport): 70 kN/m²

$q = 70$ kN/m² $q_u = 119$ kN/m²
$D_u = 893$ kN $2t = 1.60$ m, $t = 0.8$ m
$M_u = 2678$ kNm $\frac{M_u}{bh^2} = 1046$

Loads (ballasting): plate loaded by tension.
Ballastload: 64 kN/m²

$q = 64$ kN/m² $q_u = 109$ kN/m²
$2T = 818$ kN/m' $f_b = 0.9$ N/mm²

Plate III: 4.0-15.0 m

Load (transport): 20 kN/m²

$q = 20$ kN/m² $q_u = 34$ kN/m²
$D_u = 68$ kN/m² $2t = 0.12$ m

Load (design)
Hydrostatic pressure: 210 kN/m²

$q = 210$ kN/m² $q_u = 357$ kN/m²
$2P = 1785$ kN $h_{min} = 0.13$ m

A minimum platethickness of 0.40 m is taken.

Plate IV: 5.0-15.0 m

Load (transport): 25 kN/m²

$q = 25$ kN/m² $q_u = 42.5$ kN/m²
$D_u = 106$ kN $2t = 0.20$ m.

Load (designcondition):
Hydrostatic pressure: 275 kN/m²

$q = 275$ kN/m² $q_u = 468$ kN/m²
$D_u = 4680$ kN $h_{min} = 0.35$ m

A minimum platethickness of 0.40 m is taken.

Plate V: 6.0-15.0 m (max.)

Load (transport): 30 kN/m²

$q = 153$ kN/m² $q_u = 51$ kN/m²
$D_u = 153$ kN $2t = 0.30$ m

Load (designcondition): 250 kN/m²

$q = 250$ kN/m² $q_u = 425$ kN/m²
$D_u = 2125$ kN $h_{min} = 0.15$ m A minimum platethickness of 0.4 m is taken.

Pressurefluctuations in in- and outtake:

Extreme conditions: 1. Turbine under repair.
2. Designconditions

The shape of the in- and oittakes varies from round to rectangular. For these estimates. a round shape with a diameter of 10 meters is as assumed; the wallthickness assumed is 1.00 m.

Load repairsituation: 280 kN/m²

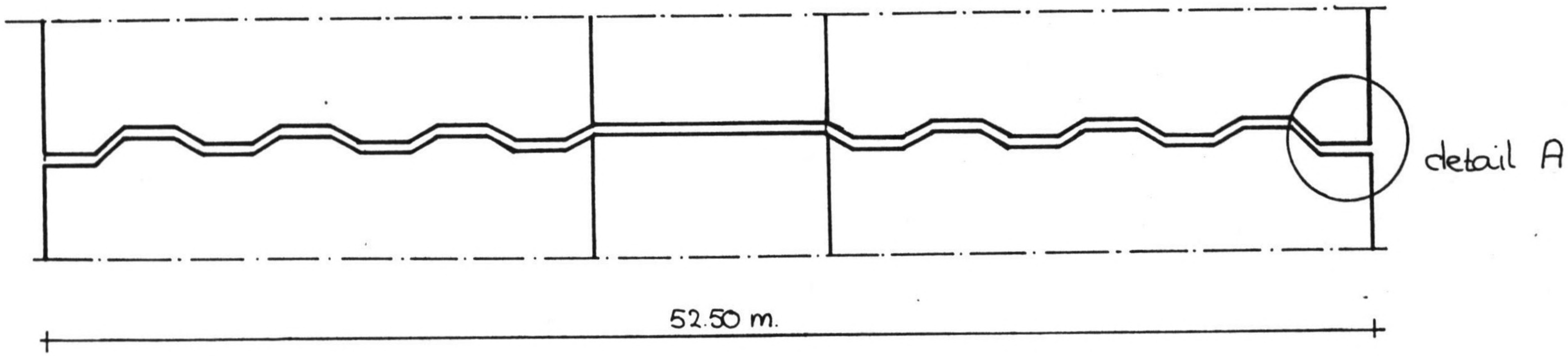
$q = 280$ kN/m² $q_u = 480$ kN/m² $p = 1.5$ N/mm² (pressure)

Load designcondition: 300 kN/m²

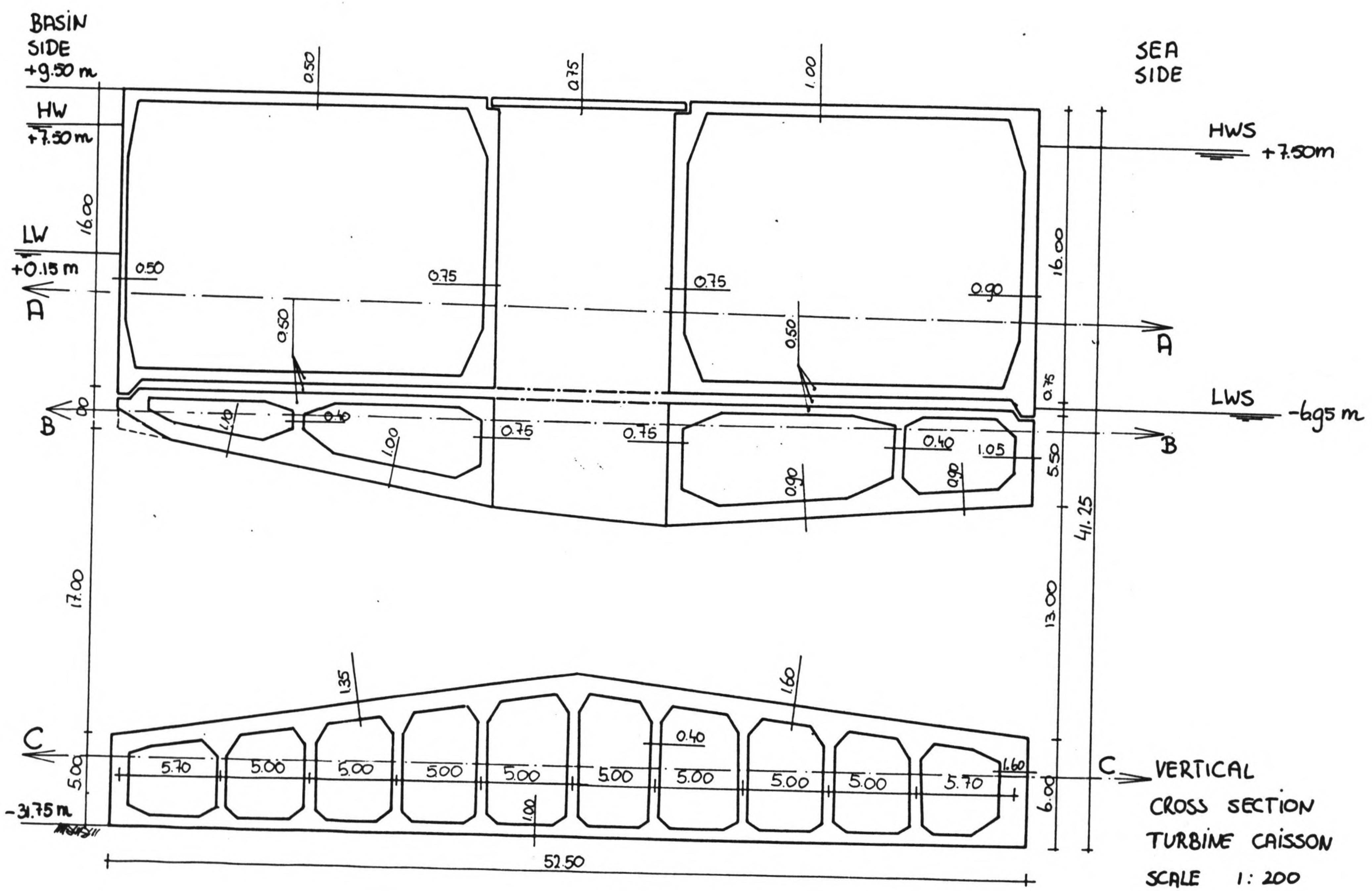
$q = 300$ kN/m² $q_u = 510$ kN/m² $t = 2.6$ N/mm² (tension)

Weight of the caisson.

After these calculations, the weight of the caisson proves to be 614064 kN (dry weight without ballast). The corresponding draft during transport is 23.4 m.



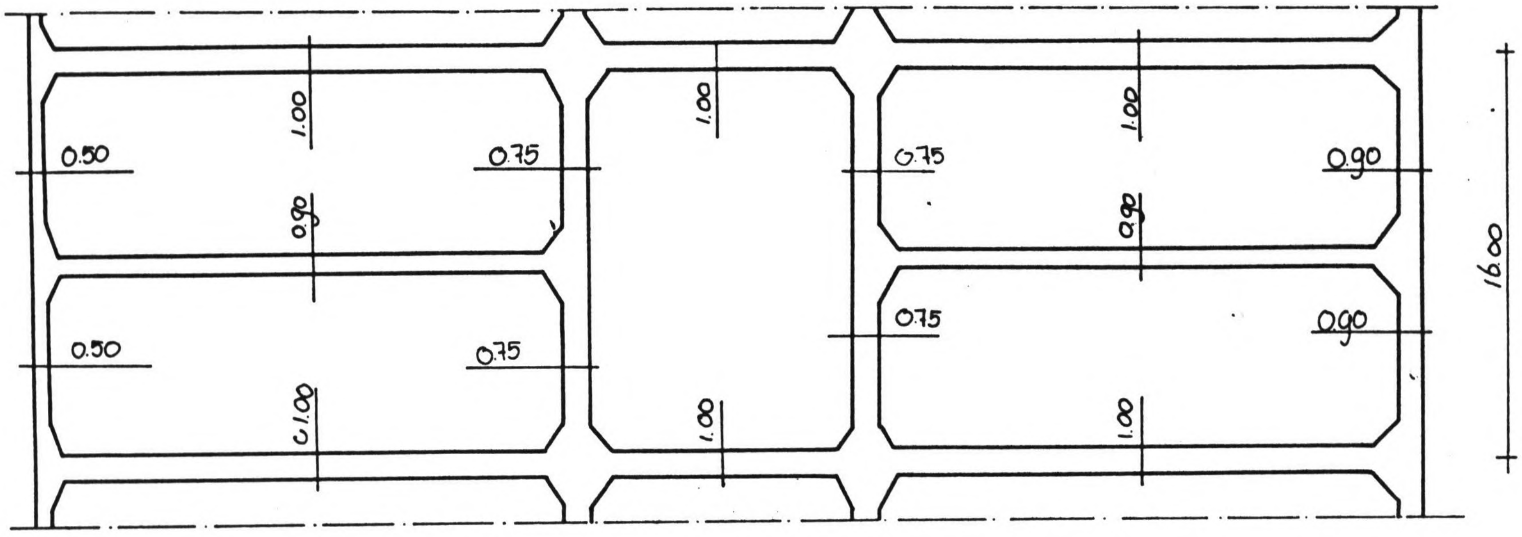
detail A
52.50 m.



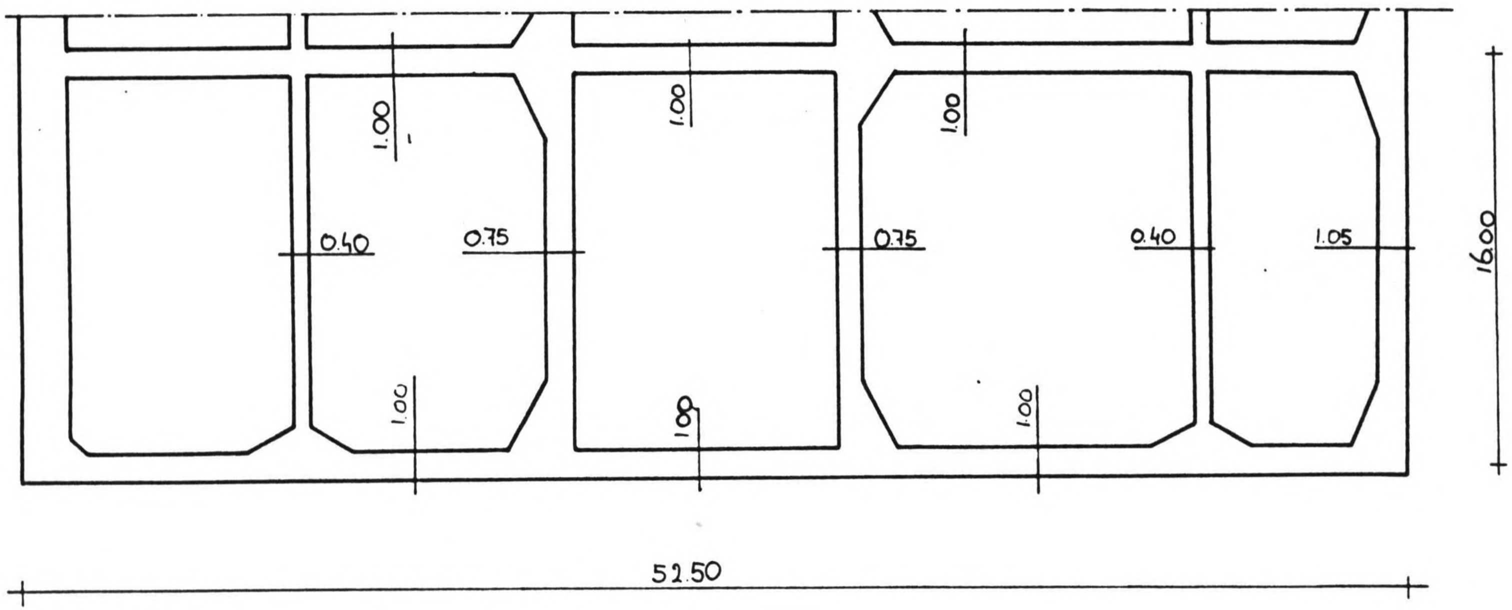
VERTICAL CROSS SECTION TURBINE CAISSON

SCALE 1: 200

LEVELS REFER TO GSCD

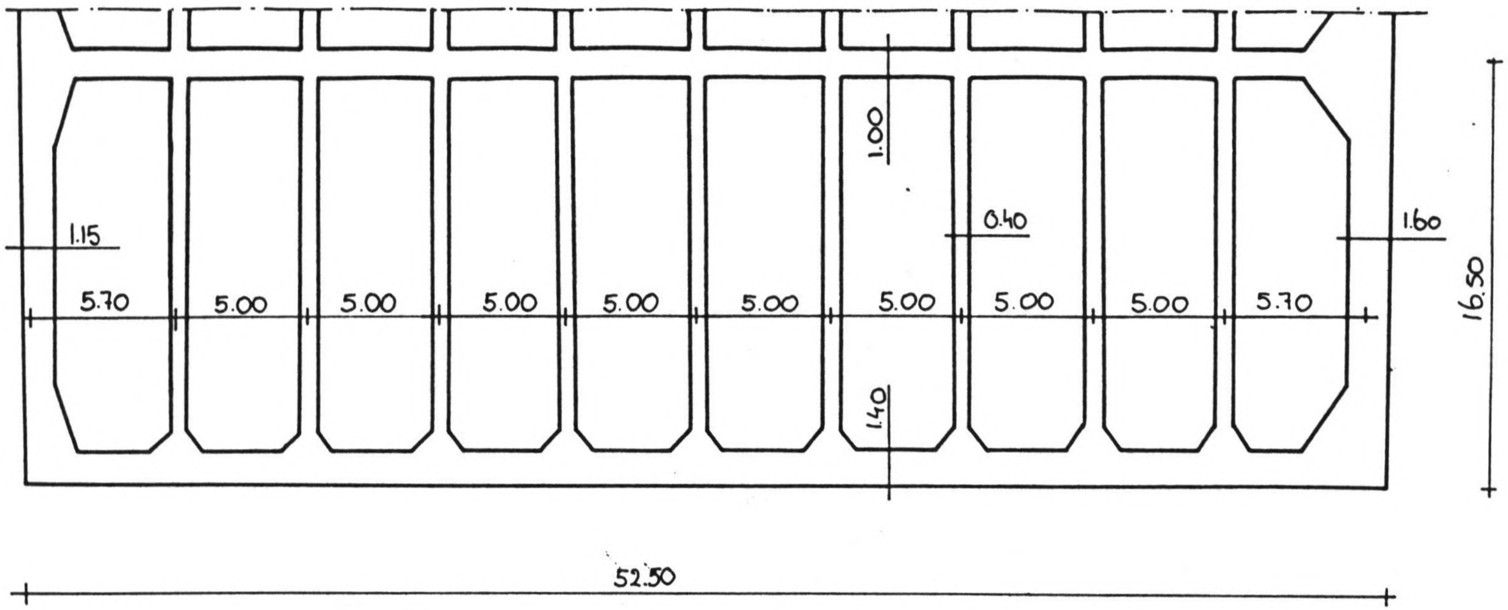


CROSS SECTION
A-A

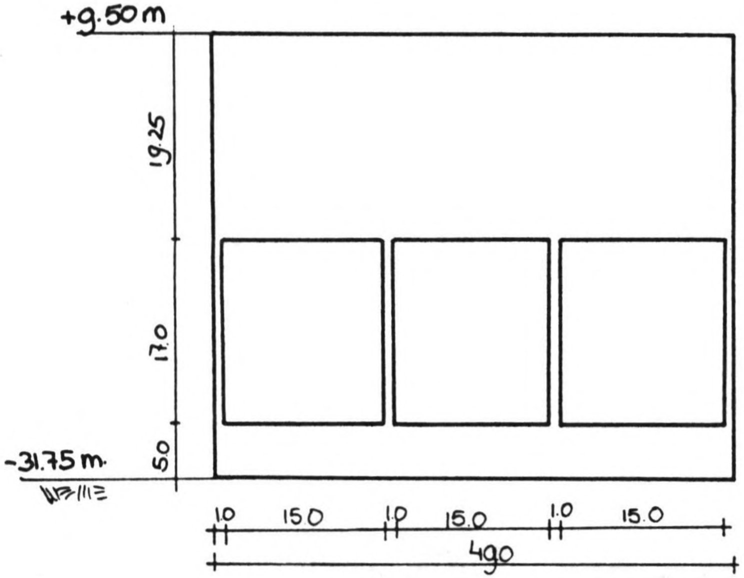


CROSS SECTION
B-B

SCALE 1:200



CROSS SECTION
C-C
scale 1:200



FRONT FACE
BASIN SIDE
scale 1:500

LEVELS REFER
TO GSCD

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Errata.

p.12 $E = \int 10\ Q\ H\ dt$ (kJ)

p.13 $\alpha_1 \alpha_2$ varies for different conditions. This calculation is ment as an indication of the energy production.

p.16 In the generation scheme, the inertia of the flow is neglected. This means in reality, the maximum water-level in the basin would be a little lower.

p.19 The length needed for the turbine and sluice sections is obtained from the Reports of the Tidal Power Review Board.

p.23 one year design wave: this is the significant wave height. This means wave overtopping would take place a couple of times per year.

p.73 A groutsausage is designed to keep the grout in; not to prevent piping.

p.81 The sliding resistance of the caisson, foundated on a bitumen foundation protection is doubted.

p.85 For the calculation of the sliding resistance, the passive ground should not be taken into account. Passive pressure develops when the caisson sets. Sufficiënt weight is needed to prevent caison movements.