Storm surge barrier Eastern Scheldt

Evaluation of wave load and dynamic response studies
for SVKO (Prefeasibility study)

Literature study

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delft hydraulics

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EVALUATION OF WAVE LOAD AND DYNAMIC RESPONSE STUDIES FOR SVKO
Prefeasibility study.

1. Introduction

1.1 Scope of the study

In the framework of the design and construction of the Storm Surge Barrier in the Eastern Scheldt a large number of studies have been performed. Different types of study can be discerned, viz.: desk studies, studies with mathematical models, physical model studies or combinations of different types. The study set-up and analysis were aggravated to the specific structure and demand for the Storm Surge Barrier (SVKO).
Now that the construction of the Barrier is completed the need is felt to evaluate the studies in a more broader perspective in order to formulate more generally usable design tools.

The aim of the presently reported study is to investigate the possibility of a useful evaluation of SVKO studies on the following subjects:
1. Wave impact forces and resulting excitation and response of structures; relation to probabilistic design techniques.
2. Wave forces and the combination of wave and flow forces; especially the problem of superposition.

Later on a study proposal has to be formulated on the subjects of which enough data for a useful evaluation is found in the SVKO studies.

1.2 Terms of reference

In his letter dated June 5, 1989, the "Hoofd-Ingenieur-Directeur van de Dienst Weg- en Waterbouwkunde van de Rijkswaterstaat" commissioned DELFT HYDRAULICS to perform the prefeasibility study "Evaluation of wave load studies for SVKO".

The study was performed by Mr. J. Wouters of the Harbours, Coasts and Offshore Technology Division of DELFT HYDRAULICS, who also drew up this report.

2. Procedure of prefeasibility study

Different phases can be determined in the procedure of the presently reported prefeasibility study, viz.:

1. Making an inventory of the study reports related to subjects of interest.
2. Clustering of the studies into related subjects.
3. Summarizing and/or describing of the final result of the studies on the particular subjects.
4. Comparing the final result to the state of the art on the particular subject.
5. Formulating options for the eventual evaluation.

Items 1 and 2 are described in Section 3, while in Sections 4 and 5 item 3 will be dealt with. In Section 6 the result of the studies on the subjects of interest are evaluated and the direction for further evaluation is presented.

3. Survey of relevant studies

A survey of all relevant reports on SVKO studies is presented in Table 1. For the composition of the survey following sources have been used:

1. The library data base "Deltalit".
2. Summary of all SVKO studies (internal library data base of SVKO studies).
3. List of references of the reports themselves.

In Table 1 the studies are arranged by project number; publications on subjects of interest are also included in this table.

From the first global inventory it appeared that the studies could be divided into three major clusters, viz.:

1. Studies on wave impacts against stirn bodies.
2. Studies on the dynamic behaviour of the structure caused by wave loads, including wave impacts.
3. Quasi-static wave loads with and without flow.

The reports related to the above mentioned clusters are rearranged in the Tables 2, 3 and 4, respectively. As far as reports of DELFT HYDRAULICS are concerned the references mentioned in the reports are presented too in these tables. In this way a chronological overview of the studies can be made.

Separately from the present report a summary of the reports was made. The summary consists of copies of the following sections of the reports, viz.:

- scope of the study,
- summary and conclusions,
- list of references.

4. Wave impact forces, dynamic response and vibrations caused by flow

4.1 General

The problems related to the wave impact forces and resulting behaviour of different parts of the SVKO have been studied interactively. Following phenomena can be discerned:
- wave impact forces on rigid structures,
- resulting vibrations of structural parts and the influence of the dynamic behaviour on the wave impact.

The interactive study set-up as used for the SVKO is illustrated in Figure 1.

The two subjects (impact forces and vibrations) were handled by two different disciplines within DELFT HYDRAULICS, viz.:
- The Maritime Structure Branch handled the impact forces, while
- The Locks, Weirs and Sluices Branch handled the dynamic behaviour of gates and beam structures.

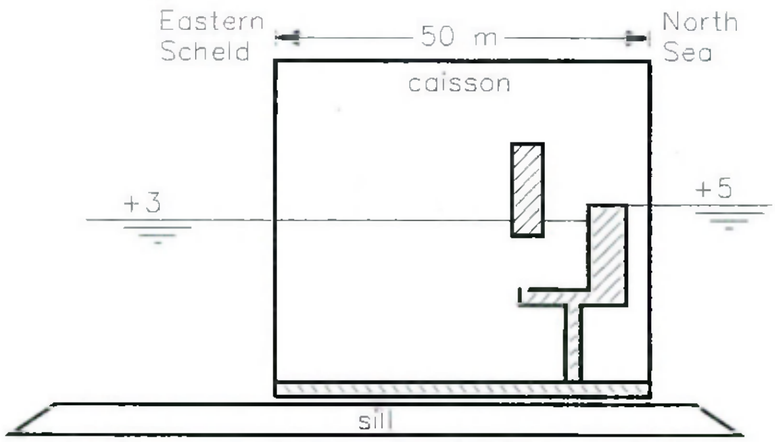
4.2 Wave impact loads

The nature of wave impacts on rigid structures and the resulting loads are very much related to the shape of the structure. It is almost impossible to predict theoretically the wave impact loads on structures that differ from a flat surface.
Physical model tests often appeared to be the most suitable tool for research on this subject.

In the framework of the SVKO scale model studies on wave impact loads three main problem areas were discerned, viz.:
1. How to analyse the complex wave impact phenomenon.
2. How to interprete model data to design loads.
3. How to adapt the design to reduce the external loads.

The analysis of the complex wave impact phenomenon was divided into two major subjects, viz.:
- what is the most critical structure configuration with respect to wave impact,
- what is the correlation of wave impacts and wave characteristics.

In M1320 IV, M1335 I, M1381, model tests on wave impact loads on different caisson solutions for the storm surge barrier are described. The caisson was placed in relatively deep water, therefore the waves did not break in front of the caisson. As a result of protruding parts of the caisson, however, the water motion was stopped abruptly, causing impact loads. In M1320 IV the influence of the distance between the two vertical parts of the caisson (see figure below) on the impact load are described.



In M1335 I the influence of wind on the impact forces was taken into account, while the studies M1381 I and II were aggravated to the loads on either steel or concrete grating gates.

Study of phenomena occurring in the case of a gate with alternative girder structures at the sea side was the objective of the studies M1504, M1664, M1723/M1687 and M1835. In these studies special attention was paid to the reproduction of test results. For regular waves it appeared that it was almost impossible to get a good reproduction of impact loads, for random waves the exceedance curve of the load was comparable for identical test-series.

A theoretical and numerical description of a piston type model was made in M1335 II. In M1835 the comparison between theoretical values for wave impact loads and model results was not very successful. In publication [51] of Kolkman four different types of impacts are described, viz.:
- Ventilated shock (comparable to slamming), water can disappear sideways.
- Hammer shock, which is analogue to a shock wave in water.
- Hammer shock for a water-air mixture.
- Compression shock, a limited column of water compresses an air cushion.

Different kind of wave impact asks for a different relation between scale value and nature value.

The investigations in the early stages of the Barrier design (caisson type barrier) resulted in wave impact pressures occurring at protruding elements in front of entrapped air that could be described with a compression model, see M1335 II and III. The phenomena occurring in the case of the gate with girders do not fit in the afore mentioned model.
The following conclusion can be drawn for the measured wave impacts.
The non-oscillating and the oscillating part of the wave impact have to be considered as behaving according to different scale laws;

- the non-oscillating part according to the "slamming model" which implies $n_p = n_L$,
- the oscillating part according to the non-linear air compression model, since the oscillations can be considered as being caused by air enclosures, scaling factor is $n_p \approx \sqrt{n_L}$.

Although it was one of the major objectives of the studies a general correlation of wave impacts and wave characteristics could not be found. The fact that this correlation was not found was the reason that a probabilistic design approach could not be used in this case.

With respect to the wave impact the following can be said:

- The test results are only of interest for structures comparable to the tested ones. Qualitative information on the influence of the structure shape on the existence and/or quality of the impact can be gained from these tests, however.
- The studies on the different phenomena involved and the resulting scale laws are of general interest. These parts are very well described in different publications [51, 54, 56].
- Verification of model test result by nature measurements would be of major interest.
- To find the correlation of wave impacts and wave characteristics would be very important. In the framework of the SVKO studies a lot of data was gained, so if there is any correlation it should have been possible to find it in this data. This was not the case, however.

4.3 Dynamic response of gates and beams/vibrations caused by flow

In Chapter 4.2 the studies on wave impact forces are described. For the described studies it was assumed that the structural parts of interest are rigid. The gates and beams of the barrier, however, cannot be considered as rigid structures. For this reason not only the wave- and current loads themselves are of importance but also the resulting dynamic behaviour of the different parts of the barrier.
A design objective was to minimize the weight of the steel gates. Consequently the significance of material stresses and fatigue aspects were growing more important for the design of the gates. For the dynamic behaviour of the different sections of the barrier it is of importance that the concrete beams (upper- and sill beam, see Figure 2) are supported by elastic bearings.

Predictions with regard to the dominant vibration modes cannot be made with certainty for such a complex structure as the SVKO is. Moreover the water will cause coupling phenomena. For these reasons scale model studies on the dynamic behaviour of gate and beam structures were necessary. The following sequence can be noticed in these studies:

1. Studies on the phenomena involved and their physical interpretation.
2. Studies on the possible consequences of these phenomena for the structure designs. Mathematical models did support the different studies.
3. Examination on the subject whether these phenomena could occur under realistic circumstances.

A list of all reports produced by DELFT HYDRAULICS on these subjects are presented in Table 3.

The studies M1322, M1424 I, M1494 and M1582 can be considered as studies on the phenomena (item 1). Results of theoretical analyses and model tests on virtual mass and unstable vibrations are compared in report M1322. Hereto the results for several L-shaped gates and for gates with different elliptical under edges were used. The study is more or less a sequel to the doctors thesis of Dr. P. Kolkman: "Flow induced gate vibrations". A very practical approach was followed in the studies M1424 I and M1494. In a scale model of a section of the barrier it was studied whether or not and under which circumstances vibrations occurred. And as far as vibrations were dealt with it

appeared that relatively simple adjustments of the design could eliminate these vibrations. In report M1582 a study on the virtual mass is described, the study was performed with the use of an electric analogon.
The dynamic behaviour of gates with plate girders and concrete beams was studied in the projects M1561, M1594 and M1648. Hereto an elastic scale model was used. The studies can be considered as studies to check the dynamic behaviour in flow and wave conditions and to obtain the actual loads on different parts of the structure (item 2). The influence of flow was the main subject in M1561. Under flow condition vibrations occurred; it appeared, however, that it was sufficient to modify the construction shape in such a way that the flow pattern, which caused the vibration was radically changed. It appeared that waves reduced or prevented vibrations due to the flow. For the determination of quasi static wave loads the measurement system of the elastic gate model was extended with strain gauges on the flanges of the main girders so that the distribution of the horizontal load along the height of gate could be determined. According to the wish to be able to compute a force spectrum at every wave spectrum, transfer functions for horizontal and vertical loads on the gates and upper beams were established in M1594. An important aspect was, that a representative combination of flow and wave conditions had to be established. Some difficulties arose because only the disturbed prototype wave spectrum on stagnant water was known. Starting from this spectrum a model spectrum on flowing water ($v < 3$ m/s) was determined. The applied method has been outlined in Appendix III of report M1594. The quotient of the undisturbed wave spectrum and the measured response spectrum finally gave the desired transfer function. Prototype loads can be determined with the aid of these transfer functions starting from a wave spectrum in which also the influence of flow refraction has been taken into account (see Figure 3). An important comparison has been made between the test results of M1594 (elastic model) and M1469 (rigid model). From this comparison it appeared that for quasi statical loads an elastic model gives the same result as a rigid model. The study M1648 can be considered as a sequel of M1594. This time, however, wave impact forces were included in the study. The circumstances in which maximum wave shock pressures can be expected and the magnitude of these pressures were known from scale model tests on rigid structures (see Chapter 4.2). The elastic scale model has been used to measure responses of gates and beams on pressure shocks and to measure the "total wave shock impulse". This could not be derived from the rigid models because of the limited number of pressure

cells. The measured response functions were used for mathematical models. The Froude law has been used for the transformation of model values to prototype values. This was considered to be a conservative approach.

Although the knowledge about, and the possibility to predict the dynamic behaviour of structure, has grown significantly the need to compare "model" results with prototype results is still felt.

The study R2279 can be considered to belong to item 3 (what is the chance of occurrence of wave impacts). In this study it is tried to incorporate the study results on wave impact on the upper beam in a probabilistic design philosophy.
Because of the fact that the wave impacts could not be related directly to wave parameters a good statistical approach could not be realized (see also Chapter 4.2).

5. Quasi-static wave loads

In the years between 1976 and 1980 several scale model studies on wave loads (quasi-static wave loads) have been performed. Two groups of studies can be discerned, viz.:

1. Loads on piers and gates (together) in the operational situation. This means a situation in which all gates are closed or only one, not working gate, is open.
2. Loads on the piers in construction phase of the barrier.

Operational situation

In the course of the years the design of the SVKO has been changed significantly. Three major solutions can be discerned, viz.:

- caisson (M1320, M1355),
- piers on a concrete cell (M1396, M1453, M1469),
- monolith pier (M1507, M1516, M1543, M1593).

All these solutions have been tested.

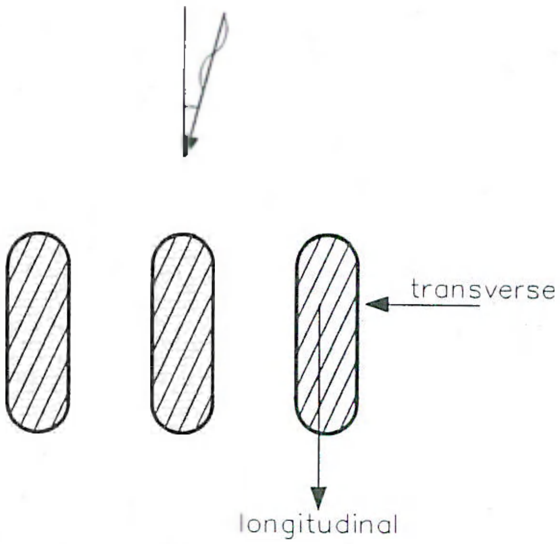
The caisson solution has been tested intensively in the studies M1320 and M1355. In the scope of M1320 a large number of caisson shapes were tested under perpendicular wave attack. The total wave force on the caisson and the reflection coefficient were measured. Based on the results it was possible to make a computer programme with which it was possible to calculate the total force and resulting moments and the "total" reflection coefficient. The phase shift due to different location of the several parts of the caisson front side could be taken into account, too. (The source of this programme is not available any more).
In M1355 obliquely approaching waves were used together with the situation in which one gate of the barrier does not close, so all gates except one are closed.

The same strategy has also been followed for the two other solutions. First tests with perpendicular wave attack and afterwards tests with oblique waves and one open gate have been performed.
The model tests M1469 were used to calibrate a mathematical wave load programme developed by R.W.S. The reflection coefficient is an important input parameter for this programme. It showed that if the right reflection coefficient was used the measurements and calculations were well comparable.

The wave loads appeared to be linear to the wave height.
Further no attempts have been made to find theoretical approaches for the wave loads. For the situation that an open gate is present in a closed barrier (with relatively large head difference between both sides of the barrier) theoretical approaches to calculate the wave loads were not made at all.

Construction phase

The wave loads on the piers of the barrier in the construction phase turned out to be of major importance. As a result of the interaction between more than one structure the transverse wave load on the piers turned out to be very large. The influence of this interaction was first noticed from mathematical calculations on this subject (Berkhoff diffraction model; publ. 206 [53]).



These calculations were done on three adjacent steel cofferdams for the solution piers on concrete cells. The model tests M1483 I were meant to prove that the calculations were right and the model tests M1483 II to prove that the transfer function between loads and wave height found with regular waves is also valid for random waves. Both above mentioned aspects turned out to be right. On basis of these results the solution piers on concrete cells was abandonned.

Similar tests, as performed for the solution piers on concrete cells, were performed on the monolith piers. Not only the interaction between the piers is of importance in the construction phase but also the interaction between waves and current. Tests on this subject have not only be performed by DELFT HYDRAULICS but also by N.S.M.B. (Netherlands Ship Model Basin).

The testresults of M1506 I showed that combined wave and current longitudinal loads were a little bit higher than expected from theoretical calculation; the transverse loads, however, showed to be non-computable.

First of all it was tried to prove whether linear superposition of wave- and current forces is possible. Therefore the magnitude of non-linearities in the hydrodynamic forces were calculated. The drag component in the Morison formula showed to be less than 10% of the inertia component for the piers. The maximum drag force is also out of phase with the maximum inertia forces, so the influence of the drag force on the maximum wave force could be considered negligible (see calculation below [59]).

Morison formula:

$$F_{wave} = M \cdot \frac{du}{dt} + D\,|u| . u.$$

M = inertia coefficient

D = drag coefficient

u = water velocity

In case of a combination of waves and current, the total force can be written as

$$F_{tot} = M \frac{d(\hat{u} \sin \omega t + V_c)}{dt} + D|\hat{u} \sin \omega t + V_c| \;.\; (\hat{u} \sin \omega t + V_c)$$

$\hat{u}$ is about 0,6 m/s for the largest waves, whereas V_c is about 2 m/s, so one can also write

$$F_{tot} = M\,\omega\,\hat{u} \cos \omega t + D\,(\hat{u} \sin \omega t + V_c)^2$$

$$F_{tot} = \hat{u}\,\sqrt{M^2\,\omega^2 + 4D^2V_c^2}\;\sin(\omega t + \varepsilon) + D(V_c^2 + \tfrac{1}{2}\hat{u}^2) - D\,\tfrac{1}{2}\hat{u}^2 \cos 2\omega t.$$

Form preliminary tests, M1506 I, it appeared that the transverse wave force was at least twice the current force, so

$M \,.\, \omega \,.\, \hat{u} > 2 \,.\, D \,.\, V_c$, and consequently $M > 20$ D.

Neglecting further the drag component of the wave force, the ratio of the wave force amplitude with and without current is

$$\frac{F_{tot} - F_{current}}{F_{current}} = \sqrt{1 + \frac{4\ D^2\ V_c^2}{M^2\ \omega^2}} = 1{,}04$$

The error in the average force is

$$\frac{F_{tot} - F_{current}}{F_{current}} \simeq \frac{\frac{1}{2}a^2}{V_c^2} = 0{,}045.$$

Although it was realized that other interactions might exist, these possible errors were that small, that sufficient confidence was obtained to proceed with the tests, under the assumption that linear superposition would be possible. Tests to verify this assumption were also started. The major results of the tests (M1532) were:

- the fluctuating part of the transverse force is strongly influenced by the presence of a steady current,
- the total force (waves plus current) is larger than the sum of the average current force and wave force.

In the report M1532 and in publ. 206 [53] the interaction effect are discussed in more detail. Attempts to describe these interactions quantitatively failed.
Especially the transverse force gave large problems because of the influence of the current on the diffraction pattern.

Although theoretically the problem of superposition of wave- and current loads was not solved, it appeared from the tests that the wave force was predominant in the determination of the design load.

The main problem for the calculation of the total force of waves and current together was the influence of the current on the diffraction pattern. Now, that programmes in which the diffraction in a current can be calculated this problem can be solved. At least for not too complicated structures.

A special place in this chapter can be given to the Delta flume tests of wave forces on (and pressure in the foundation underneath) the caisson of the SVKO (M1620). Still a lot of data of these tests are not really evaluated.

6. Evaluation

In the present chapter subjects that lend themselves to further evaluation will be dealt with. Suggestions will be given on the direction for further research. The Paragraphs 6.1 and 6.2 are written by Mr. G. v. Vledder and Mr. T. Jongeling, respectively.

6.1 Analysis of impact forces

Waves breaking on a coastal structure may generate high pressures during relatively short time intervals (in the order of 50 ms) and concentrated in small regions (in the order of 1m^2). Such pressures are called impact or shock pressures and knowledge of these pressures is of interest for the calculation of the strength of the structure. Whether or not shock pressures are expected can be determined on the basis of the checklist of Appendix 1.

In the present design procedure for large coastal structures the effect of wave impacts is normally taken into account by means of model experiments and experience gained in previous experiments. In these experiments the dynamic behaviour of the structure as a whole is investigated. An analysis of the effect of individual wave impacts on the structure is not yet feasible. Since the effect of wave impacts on the structure as a whole strongly depends on the geometry of the structure no general rules exists to relate incident wave conditions to wave impact loads. By considering the effect of individual wave impacts on a structure improved design rules may be developed.

Field and laboratory experiments have shown that the magnitude of wave impact pressures and impact pressures varies considerably per event. This is not only true for random waves but also for regular waves. These variations are mainly due to the varying air content in the waves breaking against the structures. For that reason, in the past, attempts have been made to describe these pressures or forces in a statistical way (e.g. Witte, 1988). One of the aims of those investigations was to relate parameters of the incident wave field to parameters of the impact pressures and impact forces. Unfortunately, these attempt have not been successful, since the scatter in the results was too large to draw firm conclusions.

One of the main problems with the analysis of these impact forces is that the magnitude of the impacts depends on many variables. The most important of these are the amount of air in the water, the geometry of the structure, the water level in relation to the structure, and the directional spread of the incident wave field. Due to this dependencies, large variations occur in the impact pressures. Physical explanations for these variations have been identified, but not been used to classify the measurements into distinct classes, with less scattered results.

Research on impact forces on coastal structures and dike slopes (e.g. Führböter, 1966; Witte, 1988) shows that the statistical distribution of maximum impact pressures follows a log-normal distribution. This holds for small scale experiments with regular waves, as well as for (full scale) field experiments with random waves.

Impact pressures and impact forces on storm surge barriers have been investigated by e.g. Witte (1988) and DELFT HYDRAULICS (1990). Witte published a statistical description of impact forces at the Eiderdamm in the North of Germany. In the work of Witte no satisfactory relation exists between the magnitude of mean impact forces and parameters of the incident wave field. DELFT HYDRAULICS (1990) has published some preliminary results of wave impact pressure measurements, however, these data should be further analysed to relate incident wave conditions with the total wave loads.

Recent research (e.g. Grüne, 1988) provided detailed information on the time and spatial characteristics of impact pressures on dike slopes. This information could only be obtained by using measuring equipment capable of measuring quick pressure variations, typical of wave impacts. Similar studies for coastal structures may be useful for detailed simulations of wave loads to study the dynamic behaviour of the structure.

A shortcoming of these investigations is the lack of reliable information about the width of the impact area. Such information is also of major interest for the computation of wave loads.

New design methods should be based on detailed time and space simulations of the effect of individual wave impacts. In addition, the random character wave impacts should be incorporated in the design procedure.

Considering the above remarks it is suggested that further research should be aimed to get a relation between parameters of the wave field and characteristics of the impact forces. Such a study should contain the following elements:

1) A detailed analysis of the time and spatial behaviour of individual wave impacts (including an analysis of the width of the impacts) on the storm surge barrier.
2) Statistical analysis of these results per class of incident wave, current and water level conditions.
3) Determination of a general relation between characteristics of the incident wave field and characteristics of wave impact loads.

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Conditiebewaking stormvloedkering Oosterschelde, meting tijdens hoogwatersluiting, 14 februari 1989, eigenfrequentiemeting schuif S13, 23 october 1989.
Concept rapport Q605.

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Ph. D. thesis, Technischen Universität Carolo-Wilhelmina zu Braunschweig.

6.2 Dynamic response analysis

The dynamic response of gates and beams to hydrodynamic forces is distinguished in:

a. response to flow forces,
b. response to wave forces.

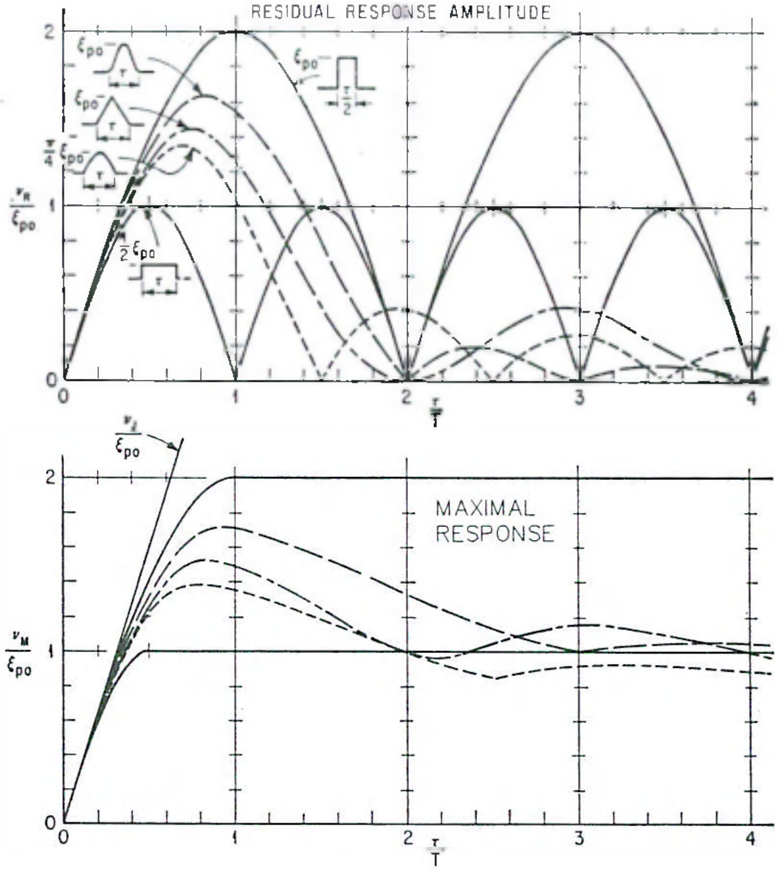
The response to flow forces is extensively studied in scale models (elastically supported rigid section models, elastically similar models) and is understood well. Vibrations which appeared during the investigations could be prevented by shape alterations or appeared to occur at hydraulic conditions with a probability lower than the design probability.
Important scale model effects are not expected, but a verification at real prototype conditions is useful, also because the prototype gate design is different compared to the investigated gate models (space truss instead of plate girders). A check of local vibrations is useful, because these vibrations were not investigated in scale models, but were evaluated in a desk study (truss members only).
In the framework of the "CONDITS" project a measurement and data acquisition system has been installed on the storm surge barrier, with which also the dynamic behaviour of two gates and an upper beam can be recorded during closing conditions. It is expected that these measurements will yield sufficient information to check the occurrence of vibrations and to compare the response with scale model measurements.

The response to wave impact forces is both dependent on the impact characteristics and the structure characteristics.

The wave impact depends on parameters such as:
- quantity of air in water (dissolved or airbubbles),
- pressure of air between water surface and the attacked structure,
- the position of the mean water level in relation to the attacked structure,
- the density of the water,
- the shape of the structure,
- the extend of the attacked area and phase differences between processes at different locations,
- wave and flow conditions, geometry of the approach area.

These parameters determine the time dependent wave impact characteristics; The duration and shape of the wave impact, the maximum amplitude and the "ascending" and "descending" time. In fact also the rigidity of the structure may have some influence.
The response of the structure to the wave impact force is mainly a function of rigidity, the mass and the damping. The rigidity and mass (including hydrodynamic mass) determine the natural periods of the structure. The

response is strongly dependent on the ratio of impact duration τ and natural period T, as is shown in the following figure, which is valid for an undamped single oscillator system. This system is loaded with impact forces with various time history, but with equal pulse. It appears that the relative response is small when $\tau/T \ll 1$, so in case of a short shock duration or in case of a non-rigid system.



The wave shock problem related to gates and upper beams of the storm surge barrier was tackled in a very practical manner: the maximum shock pulse was determined with elastically similar models also using a mathematical response model and results of pressure measurements in rigid models. This approach was possible thanks to the availability of various types of scale models and mathematical models. In general, however, this is not the case and designers are in need of a procedure to estimate the maximum response to wave shocks.

In the estimation the influence of the afore mentioned parameters (related to both the wave impact and the structure) has to be taken into account, with special emphasis to natural periods of the structure, damping, shape and attacked area.

The results of the storm surge barrier investigations form a good starting point for the development of a more generally applicable design procedure. Mathematical models to sustain this development are available, varying from simple multiple-degree-of-freedom systems to the finite element package DIANA.

A special problem is the influence of enclosed or entrained air on the wave impact load and on the structure response. Air may reduce the amplitude of the wave impact load but not necessarily the pulse and consequently not the response of the structure. The influence (qualitatively) can be estimated using mathematical models.
The air problem is also of importance for scale model investigations: the airpressure is usually not scaled and the air has thus a too high rigidity. When the phenomena are understood, correction of these scale effects is possible.
In the framework of the CONDITS project wave shock pressures are measured on one gate and one upperbeam as well as the response thereto. These measurements may serve as a helpful additional input in the study.

6.3 Quasi-static wave forces

As the studies M1320 IV and M1335 I provide a lot of qualitative information on the influence of protuding elements on the quality of impact forces, the research work M1320 I...III provides both qualitative and quantitative information about quasi-statical wave loads on non-vertical surfaces. This data-set can be considered as a unique one.
It would be worth while to make this information better accessible. The development of a mathematical model to calculate the quasi-statical wave loads on non-vertical structures is possible (see Chapter 5). Combining this work with the unique data-set collected in the Delta flume (M1612). The last mentioned data-set makes it possible to compare or even to calibrate existing formulas on wave loads to these unique measurements.

The combination waves, flow and diffraction could not be handled mathematically during the studies for the SVKO. Nowadays sophisticated models on these subjects are available (e.g. Pharos model). Especially for the solution piers on concrete cells during construction phase a comparison of measured transverse loads to calculated ones would be of major interest for the development of models on this subject.

7. Epiloque

In the present study an overview of the studies on wave loads performed for the SVKO is presented.
Subjects on which a reasonable number of data is available are described and those on which further evaluation is possible are pointed out. For this last group it is pointed out in which direction further evaluation would be desirable.

In Appendix I the general design procedure for a structure in a wave field is presented in the form of a flow scheme.
By means of SVKO it is indicated whether some information about the subject can be found in the study reports for the SVKO.

A more detailed outlining of plans for further evaluation can only be made, after discussion about the value of such an evaluation in general.

1.	M 1507	Schaaleffekten bij golfklappen op een talud	1979
2.	M 1294	Doorlaatcaisson O.S. Onderzoek naar belastingen door onregelmatige golven	1975
3.	M 1320 I	Golfbelasting caisson Oosterschelde. Oriënterend onderzoek naar de invloed van de geometrie van de caisson op de door de caisson op de drempel overgedragen en de op de schuif uitgeoefende totaalkrachten	1976
3A.	M 1320 II	Golfbelasting caisson Oosterschelde. Onderzoek naar de invloed van de geometrie van de caisson en de golfkondities op de door de caisson op drempel overgedragen totaalkrachten zowel voor bouw- als eindfase	1979
3B.	M 1320 II	Onderzoek naar totaalkrachten op de pijler oplossing met een dubbele kering	1976
3C.	M 1320 IV	Onderzoek naar de golfklapgevoeligheid van de pijleroplossing met een dubbele kering	1976
4.	M 1322	Toegevoegde watermassa en instabiele trillingen van schuiven met een vertikale bewegings-mogelijkheid	1977
5.	M 1327	Krachten en afvoercoefficienten bij rooster-schuiven. Onderzoek in stijve modellen	1978
6.	M 1335 I	Golfklappen op schuif in de Oosterscheldecaisson	1977
7.	M 1335 II	Golfklappen: een zuigermodel met samendrukbaar water	1979

Table 1 References

8.	M 1335 III	Golfklappen: een literatuuroverzicht en schaal-effekten in modelonderzoek	1979
9.	M 1335	Golfbelasting caisson Oosterschelde op pijlers uitgeoefende dwarskracht t.g.v. scheve golf-aanval, al of niet gekombineerd met verval	1979
10.	M 1381		
10A.	M1381	Golfklappen op stalen roosterschuiven	1976
10A.	M 1381 II	Golfklappen tegen betonnen roosterschuiven	1978
11.	M 1381 I		
12.	M 1381 II		
13.	M 1396 I	Belasting in gesloten toestand; oriënterend onderzoek loodrechte golfaanval	1977
14.	M 1396 II	Belasting in gelosten toestand en bij falende kering; oriënterend onderzoek scheve golf-aanval	19..
15.	M 1396 III	Belasting in gesloten toestand, oriënterend onderzoek golfklappen, scheve golfaanval	1982
16.	M 1422 I	Belasting in gesloten toestand; diepe sectie Roompot loodrechte golfaanval	1977
17.	M 1422 II	Belasting in gesloten toestand; ondiepe sectie noordelijk deel Hammen loodrechte golfaanval	1977
18.	M 1424 I	Krachten en trillingen bij hefschuiven in de pijlerdam; vooronderzoek met een sectiemodel 1:40 in een kleine stroomgoot	1978

Table 1 References (continued)

19.	M 1453	Vergelijkend onderzoek naar de invloed van de pijlergeometrie op de golfbelasting op pijlers in de gebruiksfase	1980
20.	M 1469	Diepe en ondiepe sektie systematisch onderzoek bij loodrechte aanval van regelmatige en onregelmatige golven	1978
21.	M 1483 I	Golfbelasting op kuipen in de uitvoeringsfase (regelmatige golven)	1978
22.	M 1483 II	Golfbelasting op kuipen in de uitvoeringsfase (onregelmatige golven)	1978
23.	M 1494	Vooronderzoek m.b.v. een stijf sectiemodel naar stroom- en golfbelasting op dorpelbalken, bovenbalken en plaatliggerschuiven. Vooronderzoek naar het trillingsgedrag van de plaatliggerschuiven	1981
24.	M 1504	Oriënterend onderzoek naar golfklappen op de plaatliggerschuiven sektie R15 loodrechte golfaanval en aanstroming	1982
25.	M 1506 I	Gecombineerde stroom- en golfbelasting op pijlers in de uitvoeringsfase	1979
26.	M 1506 II	Golfbelasting op pijlers in de uitvoeringsfase	1979
27.	M 1507 I ... III	Belasting bij gesloten en bij weigerende schuif Sektie R15, hart op hart afstand pijlers 40 m scheve golfaanval	1980
28.	M 1509	Kwasi statische golfbelasting op de plaatliggerschuif sektie R15, loodrechte golfaanval en aanstroming	1982

Table 1 References (continued)

29.	M 1516	Belasting bij gesloten, weigerende en sluitende schuif; sektie R15, hart op hart afstand pijlers 40 m, loodrechte golfaanval	1980
30.	M 1532	Bouwfase-onderzoek; superpositie-onderzoek stroom- en golfbelasting	1978
31.	M 1543	Belasting bij gesloten en weigerende schuif. Sektie Roompot 15, hart of hart afstand pijlers 45 m, scheve golfaanval	1981
32.	M 1561	Onderzoek naar trillingsgedrag van plaatligger-schuiven en balken met betrekking tot een elastisch gelijkvormig model	1981
33.	M 1582	Onderzoek naar toegevoegde watermassa's plaat-liggerschuiven m.b.v. een elektrisch analogon	1981
34.	M 1593 I	Stroom- en golfbelasting op de dorpelbalk bij diverse schuifstanden; rechthoekige dorpelbalk sektie R15; loodrechte golfaanval en aanstroming	1980
35.	M 1593 II	Idem voor rechthoekige en trapeziumvormige dorpel	1980
36.	M 1593 III	Stroombeelden bij diverse schuifstanden, recht-hoekige en trapeziumvormige dorpel	1980
37.	M 1594	Golfbelastingonderzoek plaatliggerschuiven en bovenbalk van de pijlerdam m.b.v. een elastisch gelijkvormig model	1982
38.	M 1612	Golfbelasting caisson Oosterschelde Deltagoot proeven	1982
39.	M 1614	Golfbelasting op hefcylinder	1982

Table 1 References (continued)

40.	M 1648 I	Onderzoek m.b.v. een elastisch gelijkvormig model naar het responsiegedrag van de bovenbalken bij golfklapbelasting. Berekening van golfklapdrukken m.b.v. een wiskundig massaveer systeem model	1981
41.	M 1648 II	Onderzoek m.b.v. een elastisch gelijkvormig model naar het responsiegedrag van de plaatliggershuiven bij golfklapbelastingen. Berekening van optredende krachten in de bewegingswerken van de schuiven	1981
42.	M 1648 III	Berekening van optredende krachten in de aanslagen van de vakwerkliggerschuiven	1984
43.	M 1664	Golfklappen op de hefschuiven; optimalisering plaatliggerschuif	1980
44.	M 1723/M 1687	Vertikale golfbelastingen op de vakwerkschuiven. Loodrechte golfaanval	1982
45.	M 1835	Golfklappen op de eindkokers	1982
46.	R 1155	Golf- en stroomkrachten op slanke cylinders (niet in het kader v.d. SVKO)	1977
47.	R 1280	Bewegingsgedrag schuiven onder invloed van het beweegsysteem en de wrijving bij de glij-opleggingen; wiskundig model massa veersysteem met Coulombse wrijving	1980
48.	R 1280 I	Invloed richtingsgevoeligheid wrijvingskracht op het bewegingsgedrag van de schuiven; aanvullende berekeningen met wiskundig model massa-veersysteem met Coulombse wrijving	1980
49.	R 1280 II	Dynamische verschijnselen bij vertikaal bewegen en belasten van de vakwerkliggerschuiven. Berekeningen m.b.v. een geschmatiseerd massa-veersysteem model met Coulombse wrijving	1982

Table 1 References (continued)

50.	R 2279	Nadere beschouwing golfklapbelasting en responsie bovenbalk bij golfklappen	1985
51.	AV 236	Kolkman, P.A. Invloed lucht op golfklappen; maximale golfdrukken volgens het stromingsmodel, het schokgolfmodel en het waterpiston model	
52.	publ. 49 IV	Kolkman, P.A. Elastisch gelijkvormige modellen van waterbouwkundige konstrukties	1967
53.	publ. 206	Berkhoff, J.C.W. en Weide v.d. J. Wave forces on a row of cylindrical piles of large diameter	1978
54.	publ. 207	Ramkema, C. A model law for wave impacts on coastal structures	
55.	publ. 305	Linderberg, J. et al Wave induced pressures underneath a caisson a comparison between theory and large scale tests	1983
		Hydraulic Aspects of Coastal Structures	1980
56.		Wave impact forces, consequences for gate design	
57.		Vibration of gates and beams	
58.		Sill-beam loads due to flow and waves	
59.		Wave and current loads on the piers in the construction stage	

Table 1 References (continued)

Number	Year	Author	References
M 1320 IV	76	v. Hijum	
M 1335 I	77	Ramkema	M 1381 I, M1320 I
M 1335 II	79	Ramkema/Flokstra	M 1335 I
M 1335 III	79	Ramkema	M 1335 II
M 1057	79	v. Doorn	publ. 207, M 1335 I, II, III
M 1381	76	Ramkema	M 1335 I
M 1381 II	78	Ramkema	M 1335 I, M 1320 I, M 1381 I
M 1664	80	Stans	M 1335 I
M 1504	82	Korthof	M 1057, publ. 207, M 1335 I, M 1648, M 1664, M 1509
M 1543	81	Stans	
M 1723/M 1687	82	Korthof	R 1155 M, 1504, M 1664
M 1835	82	Korthof	M 1723/1687, publ. 207, M 1057, M 1614
AV 236	81	Kolkman	
publ. 207	78	Ramkema	
HACS	80		Wave induced forces, consequences for gate design

Table 2 Wave impact forces

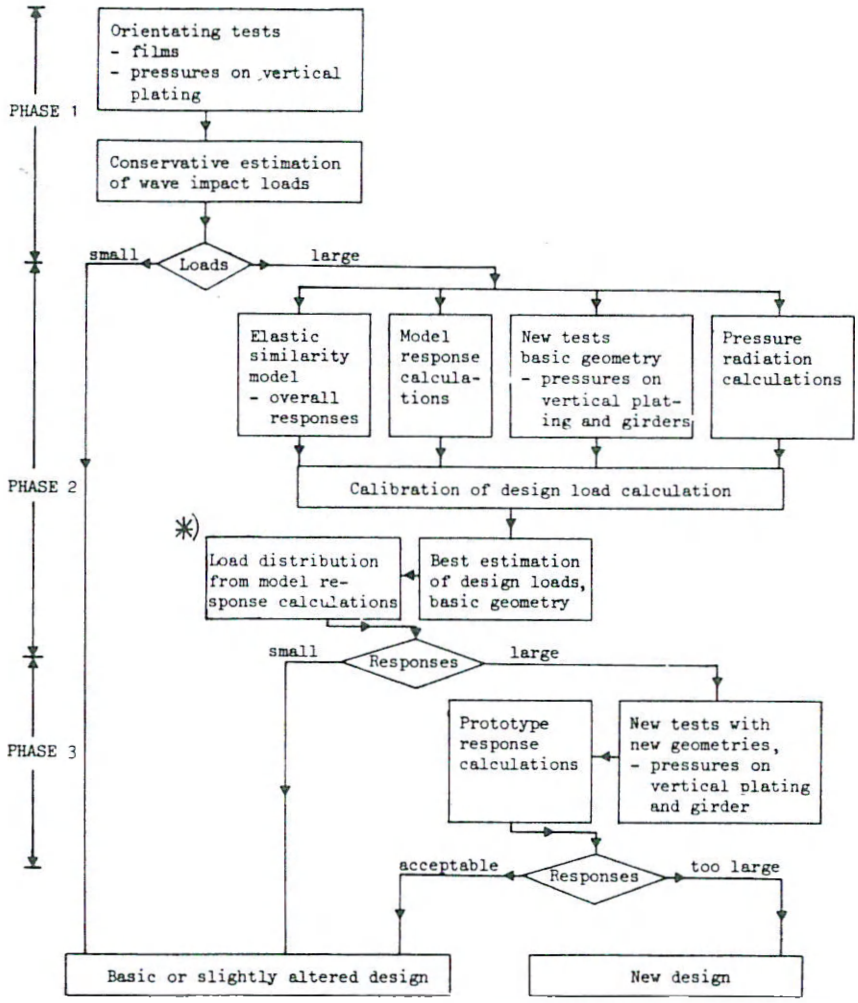
Number	Year	Author	References
M 1322	77	A. Vrijer	Flow induced gate vibrations doctor thesis of P. Kolkman
M 1424 I	78	R. de Jong	
M 1494	81	Jongeling	M 1424, M 1504/M 1509, M 1507, M 1648 I, M 1561, M 1594, M 1543, M 1593 I, M 1447 I, M 1451, M 1324, M 1516, M 1419, M 1487, M 1664
M 1561	81	v/d Wal Jongeling Perdijk	M 1582, M 1424, M 1494 M 1509, M 1594, R 1280, M 1648 HACS
M 1582	82	Deelen	publ. 164, M 1322, M 1561, M 1648 II
M 1594	82	Jongeling Perdijk	
M 1648	81	Jongeling	M 1335 I, II, III, M 1561, M 1594, M 1504 and M 1509, M 1664, M 1494, R 1280, M 1582, M 1057, HACS
R 1280	82	Deelen	
R 2279	85	Jongeling	

Table 3 Responses

Number	Year	Author	References
M 1320 I	76	v. Hijum	M 1335
M 1320 II	79	v. Hijum	
M 1320 III	76	v. Hijum	
M 1355	79	v. Híjum	
M 1396 I	77	Stans	
M 1396 II	79	Stans	M 1335, M 1320
M 1422/I + II	77	Stans	M 905, M 1320
M 1453	80	Wouters	
M 1469	78	Korthof	M 1320, M 1422
M 1483 I	78	Wouters	
M 1483 II	78	Voogt	
M 1506 I	79	Wouters	
M 1506 II	79	Wouters	
M 1507	80	Stans	M 1516
M 1516	80	Stans	M 1507, M 1320 I
M 1532	78	Vis	
M 1543	81	Stans	M 1507, M 1516, M 1593 I, M 1664 M 1648 I
M 1593	80	Stans/Korthof	
publ. 206	78	Berkhoff	
publ. 305	83	Lindenberg	
HACS	80		Wave and current loads on piers in the construction stage

Table 4 Quasi-statical loads

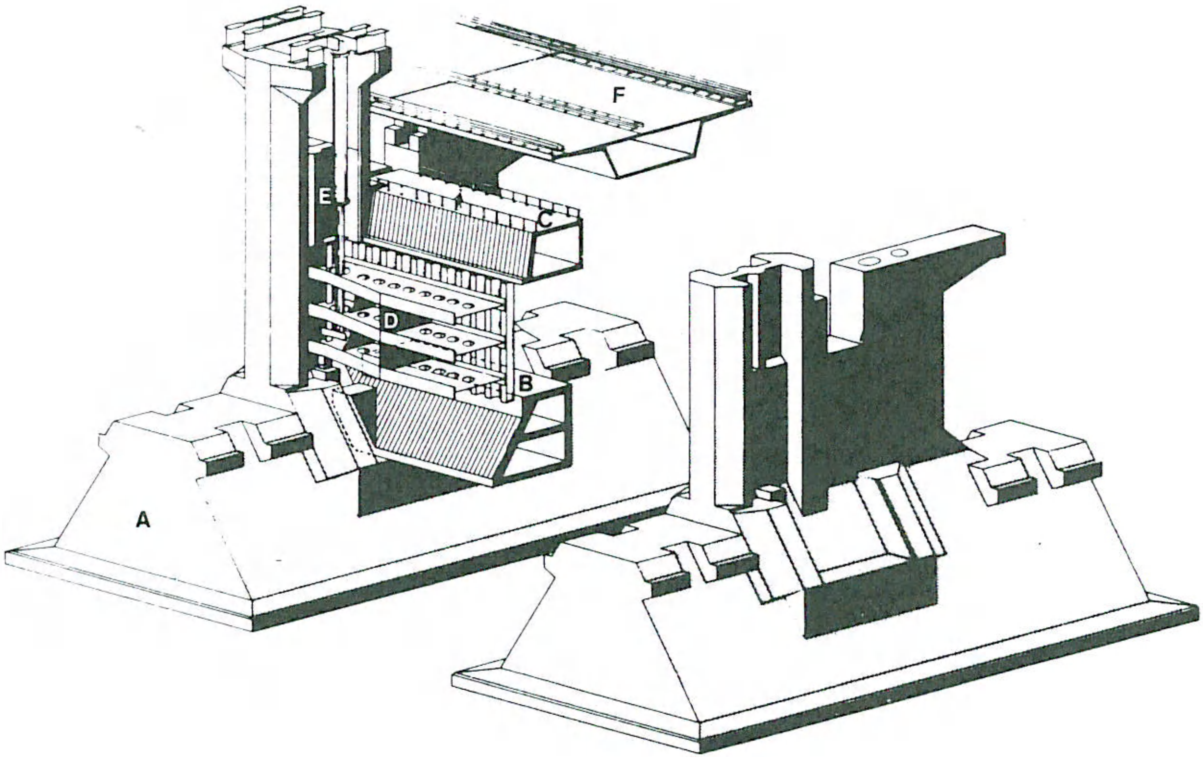
Research strategy



*) This is only possible if the response has been measured, so that calculations can be compared to measurements

STUDY SET-UP

DELFT HYDRAULICS | H 905 | FIG. 1



A. pier
B. sill beam
C. upper beam

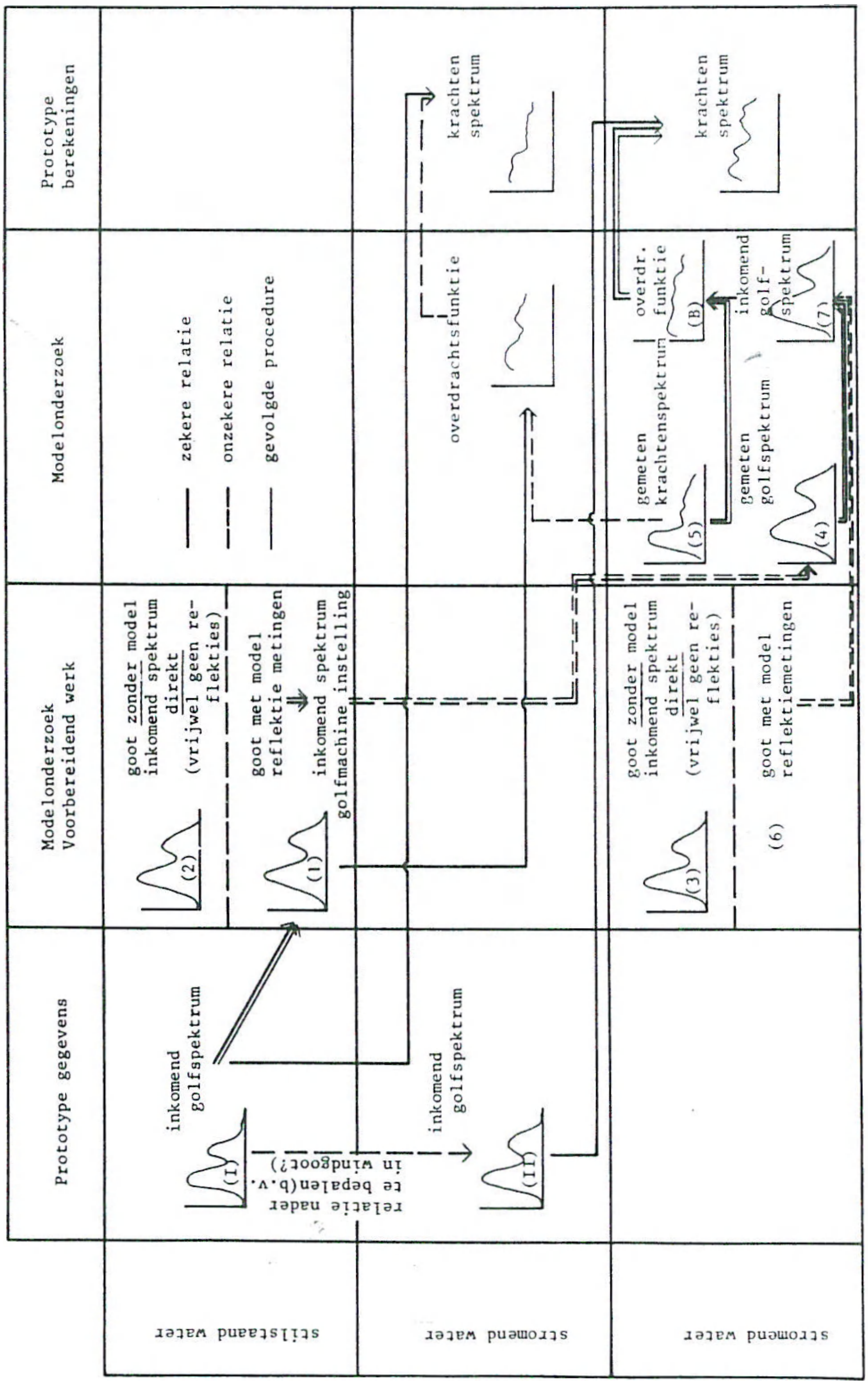
D. gate
E. recess
F. box girder bridge

DESIGN SVKO

DELFT HYDRAULICS

H 905

FIG. 2



STUDY SET-UP COMBINATION
WAVES AND FLOW LOADS [33]

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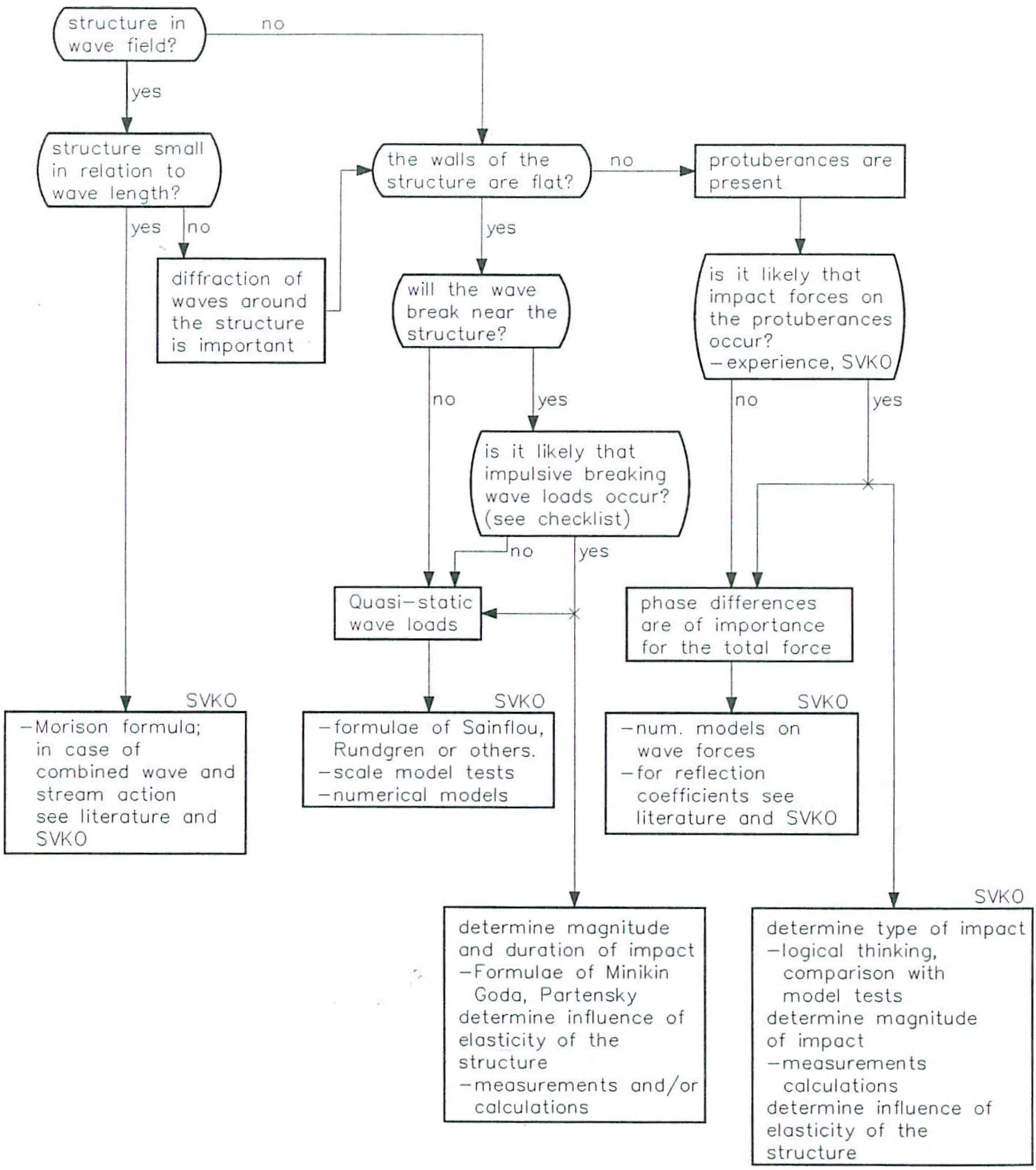
H 905

FIG. 3

APPENDIX I

Design procedure for a structure in waves



Appendix I : Design procedure for a structure in waves

Checklist for judging the danger of impulsive breaking wave loads.

1.	Is the angle between the wave direction and the structure less than 20°?	no →	little danger
	↓ yes		
2.	Is the sea bottom slope steeper than 1/50?	no →	little danger
	↓ yes		
3.	Is the steepness of the equivalent deep water wave less than about 0.03?	no →	little danger
	↓ yes		
4.	Is the breaking point of a progressive wave (in absence of the structure) located only slightly in front of the structure	no →	little danger
	↓ yes		
5.	Is the crest elevation so high as not to allow much overtopping	no →	little danger
	↓ yes		

Danger for Impulsive Loads Exists.

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