THE USE OF MODEL TESTS FOR THE DESIGN OF MARITIME STRUCTURES WITH REGARD TO WAVE ACTION

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1. General considerations

The hydraulic model is playing an increasing part in the process of hydraulic designing and it seems even to have taken over the dominant position that experience and tradition had held for such a long time. But, however important the model tests have become, they are not the only means available to the designer, nor does he have to rely solely upon them. Experience still holds a prominent place, while investigations in situ procure for the designer basic information he seldom can dispense with. A close interaction between investigation in model and prototype is a condition for obtaining reliable results.

Beside these relations in the hydraulic field, a design is influenced by other considerations, such as in the fields of technical construction and economy.

The model (hydraulic, mathematical or other) has its own place between several other components of the design.

As already stated, the influence of the hydraulic model in this interaction is increasing and is now tending to become the central point in the process of designing. This is to a large extent due to progress in modeltechniques: simulation, even of very intricate hydraulic phenomena, has been greatly improved by experience as well as through basic research, while also the instrumentation of the laboratory is steadily improving.

The other fields concerned with hydraulic design should keep pace with this progress. This applies especially to hydraulic investigation and research in the prototype. These field investigations should produce many of the basic data required for the determination of boundary conditions and for testing the model. As the technique of simulation improves, the need for better and more exact field data grows accordingly. But we seldom find the same facilities for field investigations in the prototype as the laboratory can offer. Measurements in nature are often restricted by practical circumstances and need a large amount of organization.

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Furthermore, the repeatability is less, due to varying conditions, while the occasion to investigate under exceptional situations very seldom occurs. For these and other reasons, the investigations in the prototype lag behind these carried out in the laboratory, with the result that a gap of increasing width threatens to arise between these two fields of research.

The progress of the technique of running model tests challenges the designer with regard to the aspects of construction. He is bound to exploit to full extent the increasing possibilities of hydraulic research and of the ready information obtainable from this field. This requires at least some fundamental understanding of model tests, of the range of their validity, and of their limitations. The designer should be able to give the testresults their proper place and weight alongside his other hydraulic and non-hydraulic considerations. But as the technique of the model test has become more and more specialized, its scope has become less accesible to "outsiders", among whom often also the designer himself should be ranked. This may be
** the cause of a growing "mental distance" between the designer and the laboratory staff. This is a danger against which both sides should be on their guard, because such an alienation may lead either to an insufficient contribution of the model test to the design or, probably or more often, to a growing dictate of the model.

The designer should follow the progress of the test and should have a critical insight into what is done in the model. However, with the growing complexity of many models, it is often not easy for him to find his way, especially when - which too often happens - he has not had any special experience or training in this field. He should be on his guard against what may be called the suggestive power of hydraulic models; having water as medium they seem to be a true reproduction of the prototype, tempting the spectator to jump to rash conclusions by visual observation. This risk is not run with mathematical models and electric analogons. Nevertheless, in some cases the illustrative character of the hydraulic model has also its benefits.

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The improving thechnique of measuring hydraulic phenomena accurately in the model should be exploited as fully as possible. As the exact numerical evaluation of the information derived from the model is not always feasible, the designer often cannot make full use of the data which the model can supply him. On the one hand, this should restrain him from asking data from the model which he cannot take properly into account for his design, but on the other hand this lag should urge him to bring his criteria up to the level of the model.

These general considerations regarding the relations between model test and design hold good for all hydraulic designs but especially for those where the hydraulic phenomena involved are intricate and not easily accesible to observation and simulation. To this last category belong many of the phenomena of wave motion with which the designer of maritime structures has to deal. Though much progress has already been made in wavestudy and wavesimulation in models, there are still many problems to be solved in this field by both the laboratory and the designer.

2. Simulation of wavephenomena

In developing the technique of wavesimulation, two main practical problems are encountered: measurement of complex wavemotions in the prototype and the true reproduction of these data in the waveflume. Various attempts have been made to measure the effects of waveattack on structures in nature, but as already stated it is very difficult and wearisome to get reliable results of a sufficient scope from measurements in the prototype, especially of such a complex phenomenon as the breaking of waves on structures. And as the registration of the process of waveaction in the prototype is already a difficult matter under normal conditions, it is especially so during storms. This is one of the reasons why up till now it is still largely necessary to rely on a mathematical approach to the problem. But it is not possible to go far on this theoretical path without checking the assumptions by the facts observed and measured in the prototype. The same applies to the simulation of waves in a model. This can be based to some extent on scale laws, but if a close resemblance to nature is required measurements in the prototype are indispensable.

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For a long time we have had to be content with the approximation of the real wavemotion by more or less regular waves, not only because they could not be measured exactly in the protortype but also because it was technically impossible to reproduce them in the flume. Some declared that this simplified simulation of the wave spectra would serve the purpose, trying perhaps to make a virtue of this forced limitation. But the point has now been reached where it is possible to reproduce in the wave flume almost exactly the real wave spectrum. And the test results obtained so far seem to contradict to some extent the opinion of those, who trusted that tests with more or less regular waves would always give enough information on which to base a design.

As it has now become possible to simulate the wave spectrum truly in the wave flume, even though only unidirectionally, the need is felt to improve the investigation in the prototype accordingly, expecially with regard to the very complex phenomenon of the deformation of waves in the neighbourhood of structures.

We should aim here at a close cooperation between the research in the prototype and in the laboratory, especially with regard to the measuring of the forces exerted by wave impact against maritime structures.

With the simulation of a unidirectional spectrum in the wave flume, there still remains a restriction as compared with the prototype, because the aspects of the wave pattern are not yet brought into account. In the flume this pattern is almost regular that is, without the transversal differences that occur in the prototype, especially when there is question of a system of crossing wave trains. Though the neglect of this complication will often be permissible there may be cases where a closer investigation in this respect is desirable. This question arose, for instance, during the investigation of the wave impact against the gates of the Haringvliet sluices, where it was important to have information about the transversal extension of the wave impact in order to determine the total load on a gate. This problem deserves special consideration now that the simulation of wavemotions has been so much improved.

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3. Evaluation of flume tests

As has already been observed, the designing engineer frequently has to cope with the problem that the criteria of failure of the structure are not well defined. Therefore it is often difficult to make full use of the impacts measured in the model when determining the strength of the structure to resist the wave attack. In these cases, they can only give him qualitative indications.

In addition, the designer has to give the proper weight to the hydraulic data in comparison with other non-hydraulic considerations, when dealing with questions of construction and economy. He may be fortunate enough to have all his considerations, hydraulic and nonhydraulic, point to the same direction. But very often there will be some contradiction between these different considerations. In those cases he has to decide which of them should prevail: whether the hydraulic evidence has to be decisive or some constructive or economic aspects should dominate. This all depends, of course, on the type and the character of the design.

Special attention will be given here to three problems:

- wave impact and wave run-up on the slope of a sea wall
- wave attack on sluice gates and
- wave attack on a rubble dam.

Some practical questions connected with the evaluation of model tests regarding these phenomena will now be discussed briefly.

a. Wave attack on the slope of a sea wall

This is a complex phenomenon consisting of the direct impact of the breaking wave on the revetment and, in addition, the run-up of a mass of water which eventually may overtop the crest of the wall. From observations in the prototype it is obvious that the process of a breaking wave can be affected by the backwash caused by the preceding wave. The reflux from a wave may smother the impact of the following wave and check its run-up. This may explain the fact that in model tests, carried out with irregular waves, a direct relationship between the run-up and the height of the idividual waves could not be detected. This was probably because the interaction of two succesive waves was not taken into account. The same difficulty should arise when trying to find the relation between the individual wave height and its impact on the slope.

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Further research into this problem is recommended, especially now that tests with a truly simulated irregular wave spectrum have become possible. However, for the time being the data available both on wave impact and wave run-up seem to be sufficient for the designing engineer, considering the vagueness of the criteria he applies.

The process of the wave impact on slopes has been measured in the model as well as in the prototype; reference may be made here as an example to the investigations carried out in this respect on the smooth concrete slope of the Westkappelle sea wall. (figure 1). These have given some insight into the process and the magitude of the impact. Furthermore, the results obtained from the measurements in the prototype seem to correspond to a fair extent with those obtained from similar model tests. But it is still very difficult to apply these data to the design of the revetment. The engineer can only take them into account very globally and has to content himself mainly with the knowledge that the impact on a slope decreases considerably with its gradient.

As to the wave run-up, the designer will especially be interested in the amount of overtopping that may be expected. This quantity of overtopping water, in relation to a certain wave spectrum and water-level, can be determined fairly correctly in a model for any sea wall design; data can be given regarding the overflow per unit length of seawall, as shown in figure 2. In order to take these data into account in a proper way the designer should know how much overflow his structure can stand and also how much overflow can be accepted on the hinterland behind the sea wall. But all too often he has to decide on these aspects without adequate information.

Investigations after the disastrous flood 1953, which broke so many dikes in Holland, showed that most damage was caused by overflow eroding the inner slopes of the seawalls. These slopes ordinarily are not protected by a stone revetment but simply by a grass cover which is only to some extent resistant against overflowing water. The exact cause of the dammage was found to be the sliding away of the protecting top layer of soil in which the grass was rooted, due to saturation by the overflowing water.

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It was found also that the resistance of the top layer can be considerably improved by drainage, making sure that this layer is in any case less permeable than the soil beneath, and further by decreasing the gradient of the inner slope. These rules were applied when rebuilding and reinforcing the dikes. But as this problem is not yet accessible to exact calculations, it is still impossible to assess the critical amount of overflow that can be accepted. So the engineer is still all a loss when he has to decide how much overtopping he may permit and for what duration.

This problem also exists when the crown and inner slope are protected by a more resistant material, for instance by an asphalt revetment. Though more overflow might be accepted here, no reliable criterion is available in this case either.

This uncertainty prevents an exact economical evaluation being made of the relation between the wave run-up obtained by model tests and the cross-section of a sea wall. And as there should be no taking of unknown risks, even very small overflow is only allowed in very exceptional cases. Normally the socalled 2% rule is applied in Holland, which means that 2% of the waves during design storm will reach the crown of the dike. This means that a point is chosen far on the left of the steep part of the curve.in figure 2. Obviously, looking at the shape of this curve, much could be gained economically, if more overtopping dare be allowed.

The importance of this problem depends on the situation. For instance, in dealing with a polder dike protecting low land, as shown in the upper picture of figure 3, a variation of the crestlevel will have more consequences than in the case of a dike protecting an industrial harbour site situated on a comparatively high level, like those being constructed in the sea near the Hook of Holland (Maasvlakte). When comparing these two examples it must also be pointed out, that in the case of the polder dike, the consequences involved, are much greater because the land to be protected here lies far below sea level. In assesing the freeboard of dikes, this aspect should also be taken into account.

Further investigations into the problem of the resistance of different types of revetments should be stimulated.

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b. Wave impact on the gates in the Haringvliet

The Haringvliet sluice, with its circularshaped segment gates is an interesting example of a design in which an important hydraulic desideratum had to yield to technical and economic considerations. With regard to the severe wave attack to which these gates will be exposed, their seaward inclination is very unfavourable. Wave impacts increase considerably with the angle of inclination, as is clearly showed in figure 4. However, the enormous weight of these gates (width 56 m, height 10 m), asked for a design that would keep the force required for the lifting of a gate within reasonable limits. Therefore the circular shape was chosen so that the resultant hydrostatic forces would act axially; the larger wave impacts due to this shape had to be accepted here as the lesser evil.

The process of the wave impact on this type of structure (figure 5) is of another character than on a flat slope as was shown in figure 1, and seems to be vary considerably with the shape of the wave. An eagerness to check the forces measured in the model by tests in the prototype resulted in several pressure recorders being built in one of the gates (figure 6). It will be interesting to follow the results of these investigations in the prototype and to compare them with the data obtained from the model.

4. Wave attack on the new breakwater at Hook of Holland.

a. Assessing type and weight of armour

The cross section of these breakwaters is shown in figure 7, They are composed of a core of quarry stones protected by an armour of concrete cubes. As the model tests that were carried out to check the stability of these blocks are dealt with in the paper of Norwegian colleagues, discussion here will be confined to some considerations in the fields of construction and economy that played an important part in assessing type and dimension of the blocks.

From the beginning it was without question that the strength of the wave attack that would have to be expected on the dams excluded the use of quarry stone for the armour and that consequently concrete blocks would be needed here.

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Starting from this assumption, technological investigations were carried out in order to discover the maximum specific weight that could be obtained at reasonable cost. This turned out to be approximately 2.65. In addition, an approximate estimate had to be made of the maximum weight that could be handled by the tools that would be transported and put into place by ship.

** Next, to ascertain a large output, it had to be considered that the blocks would have to be fabricated, handled and transported in an easy way. This consideration asked for a simple shape, and therefore further tests were started with the cube, as this seemed to be the most advantageous shape in this respect.

Based on these facts and desiderata, stability tests were started in the model, where the relations between wave height, rate of damage, specific weight and special shapes of the blocks were investigated.

** One of the interesting results of these primary tests was that the stability of special more or less interlocking shapes (such as Akmons) was relatively less than had been expected. This was largely due to the fact that the rubble dam has its crest situated some 2 m below storm surge level. Consequently severe wave attack has also to be expected on the innerslope, where the interlocking systems are less adequate. Therefore -- although by using a special type of block, such as the Akmon, a certain reduction in block weight and total armour volume could be obtained -- the difference with cubes was too small in this case to be decisive.

Taking into account its lesser cost per unit weight and its easier handling, the final choice was made in favour of the cube.

The test that had to be performed to assess the dimension of the blocks were mainly based upon the rather arbitrary assumption that slight damage might be allowed only about once in 100 years, corresponding with a design wave of 8,5 m significant.

This criterion was checked on its econmical merits, trying to assess the optimum combination of initial and capitalized maintenance cost, and a fairly good agreement was obtained. The value of this check was limited, however, because it was not possible to assess accurately the relation between construction and maintenance cost for this type of structure in Holland. But as the construction cost was not appreciably influenced by the dimension of the blocks, the chosen weight of 43 tons that was regarded as approximately the maximum to be handled by the equipment without difficulty, appeared to be also about the most economical.

b. Measurements against erosion in front of the breakwater

The adjacent part of the coast where the breakwaters are built will be liable to considerable changes, not only due to the construction of the breakwaters but also to the Delta Works, e.g., the closure of the Haringvliet. As a consequence of these works, accretion may be expected along the first part of the southern breakwater which willconsist of a sanddam, and erosion along the most protruding part which runs almost parallel to the coast, and will be constructed as a rubble dam. The parts where accretion or erosion is expected are roughly indicated in figure 8, in which is also shown the approach channel that has to be dredged to give access to tankers up to 225.000 dwt.

It was necessary to anticipate these changes in the design, and especially the threat of erosion in front of the toe of the rubble dam. However, no reliable data could be obtained, either from the prototype or from the model, as to the ultimate extension of the eriosion and of the time it will take to develop. What could be done was to determine to what extent the erosion would be acceptable without endangering the stability of the breakwater, and then to plan what measures would have to be taken successively to keep the erosion within these bounds. The programme set up for that purpose is shown in figure 7.

In the first instance, a blanket of limited breadth consisting of gravel, will be placed before and under the dam, the sea bottom being locally excavated to the required depth for this foundation. Then, if erosion starts at the toe of this blanket, it will be extended horizontally for 40 à 50 m. If after this supplementary protection has been provided erosion still continues at the end of this berm, the blanket will be extended once more, this time sloping.

The ultimate profile as shown in figure 7 (No. 5) has been assessed in the model. The tests that were carried out with irregular waves indicated that although the boundary conditions of the wave motion were not changed, the impact on the rubble dam increased considerably when a certain depth in front of the breakwater was exceeded. The tést showed, furthermore, that this phenomenon did not occur if a berm was kept in front of the dam as indicated in figure 7 (No. 3). Further increase of depth outside that berm did not worsen conditions.

This example shows that, although model tests cannot (yet) give reliable evidence as to the extension and pace of an erosion, it can nevertheless procure essential information with regard to the limits within which it should ultimately be kept for safety of the structure.

5. The use of model tests

In the preceding pages it has been pointed out that the results of model tests may have in many cases only a very limited value with regard to their quantitative interpretation. Sometimes this is due to certain limitations of the model, but often also to the circumstance that the criteria handled by the designer cannot be put into exact figures.

Therefore, though a certain quantitative evidence may sometimes be obtained from the model, its chief value lies in its contribution to the qualitative interpretation and appreciation of various solutions for the design.

The success of a model test is to a large extent dependent on the right choice of parameters. Their number should be restricted and limited to those whose influence is really important and can be evaluated by the engineer. For the choice of parameters, the engineer should not only be guided by their importance for the hydraulic effects on the design, but also by their influence on constructive and economic consequences.

The decision as to what boundary conditions should be applied is also very important. Special attention has to be paid here to the question whether in the prototype these conditions may be subject to changes. Such changes may arise from alterations of the topography of the foreshore, effected either by the structure itself or by natural hydrographic changes. This may be expecially occur along sandy coasts - like that of the Netherlands - liable to scour or siltation.
In these cases a structure should be tested in the model under different boundary conditions, corresponding with the changes that may be expected to occur in the prototype.

The boundary conditions may also be varied as parameter, in order to determine the design wave that goes with the most economic combination of construction cost and capitalized maintenance cost.

Finally, it can be asked to what extent the hydraulic model might be replaced by a mathematical one. This may be possible for those phenomena that can be simulated with sufficient precision by formulae based on theoretical considerations. But many wave phenomena are still too complex to be captured in a mathematical model. Some of these may be superficially represented by formulae derived empirically from hydraulic model tests, but as these formulae do not give a basic insight, it is dangerous to apply them on a design without considering if they hold good for that very case.

With the growing accuracy of simulation, the importance of the hydraulic model increases, especially for those designs which have an exeptional character.

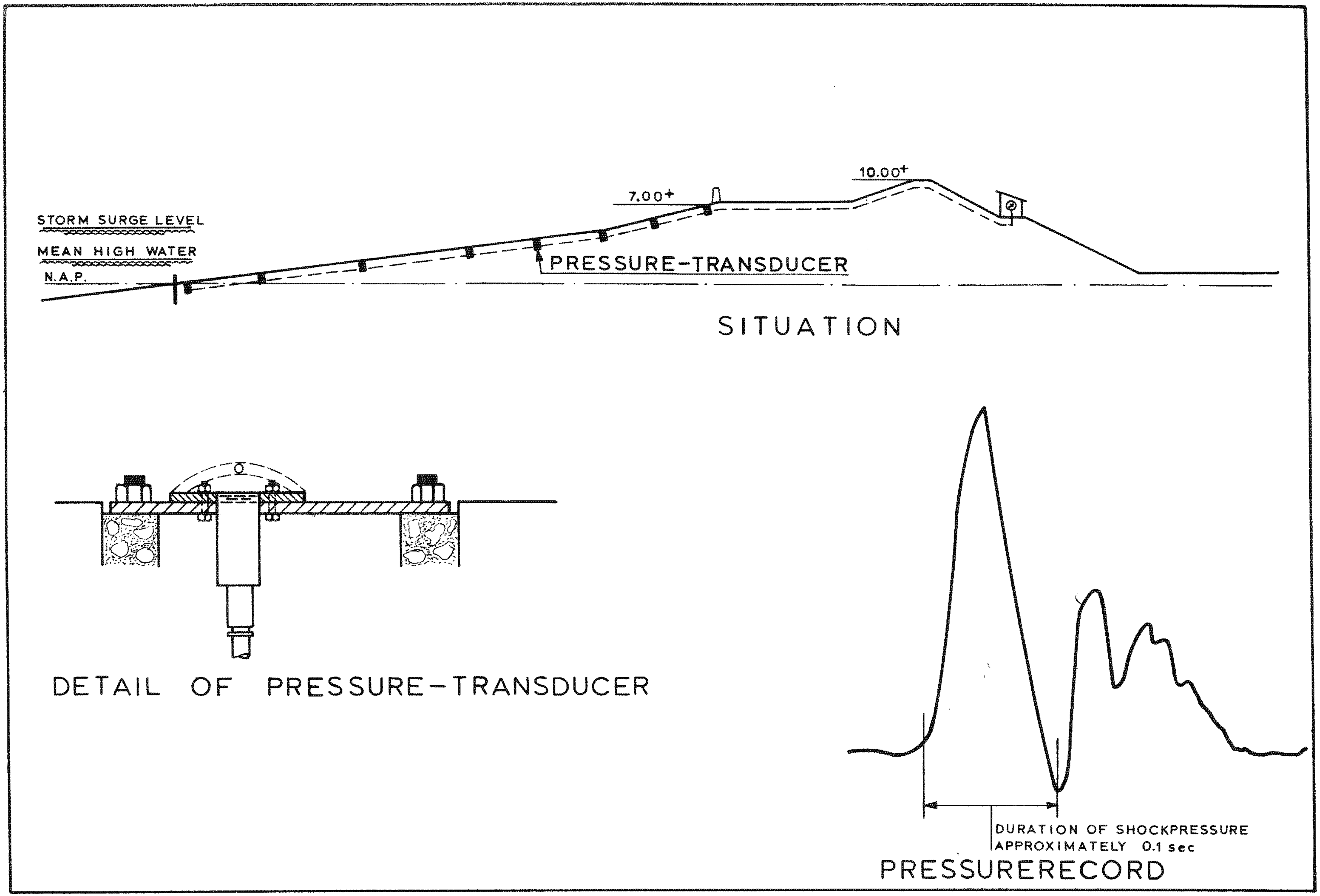

FIG.1. MEASUREMENTS OF WAVEPRESSURE ON THE WESTKAPELLE SEAWALL IN PROTOTYPE

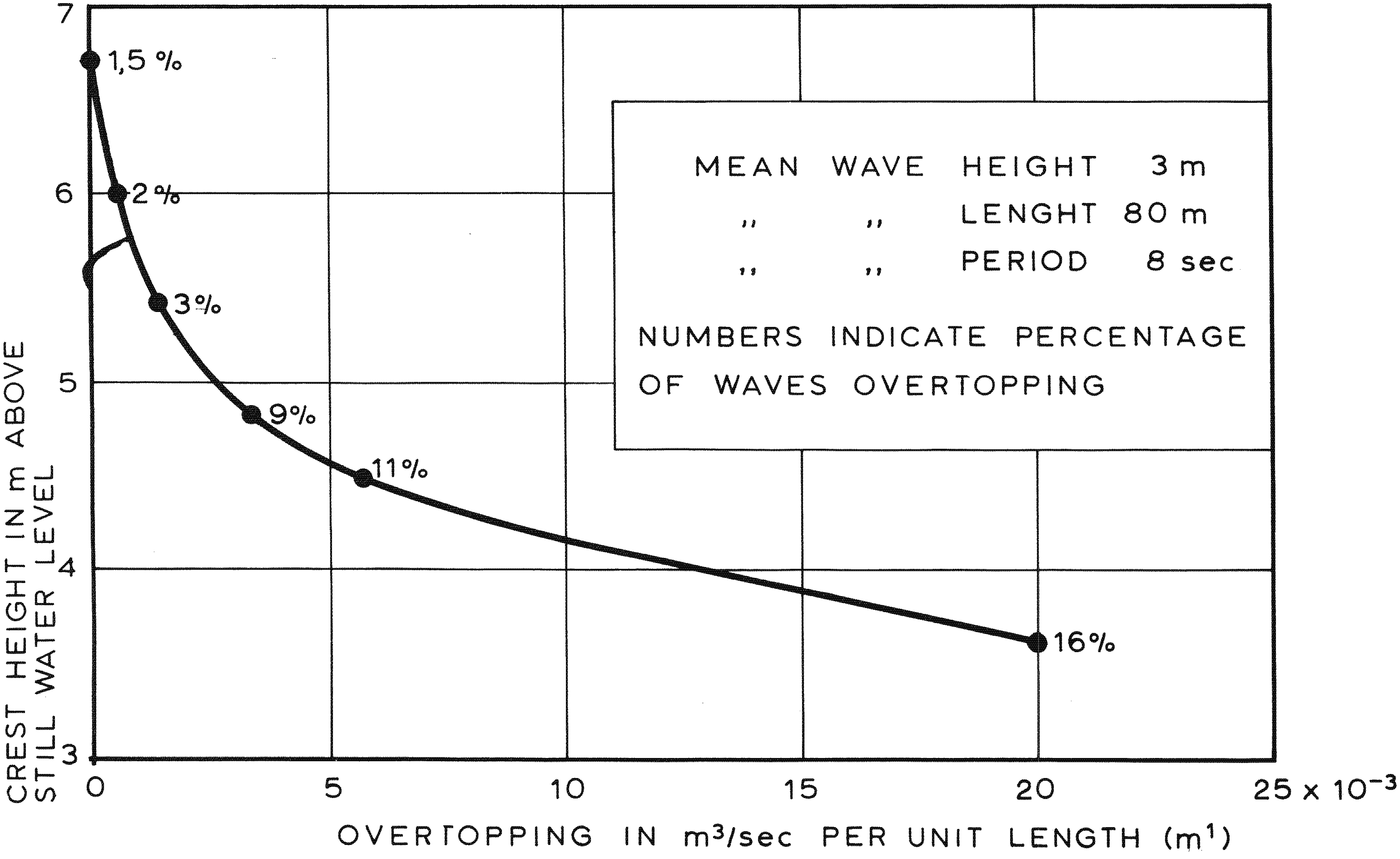

FIG.2. RESULTS OF MODELTESTS IN WAVE OVERTOPPING OF A SEAWALL

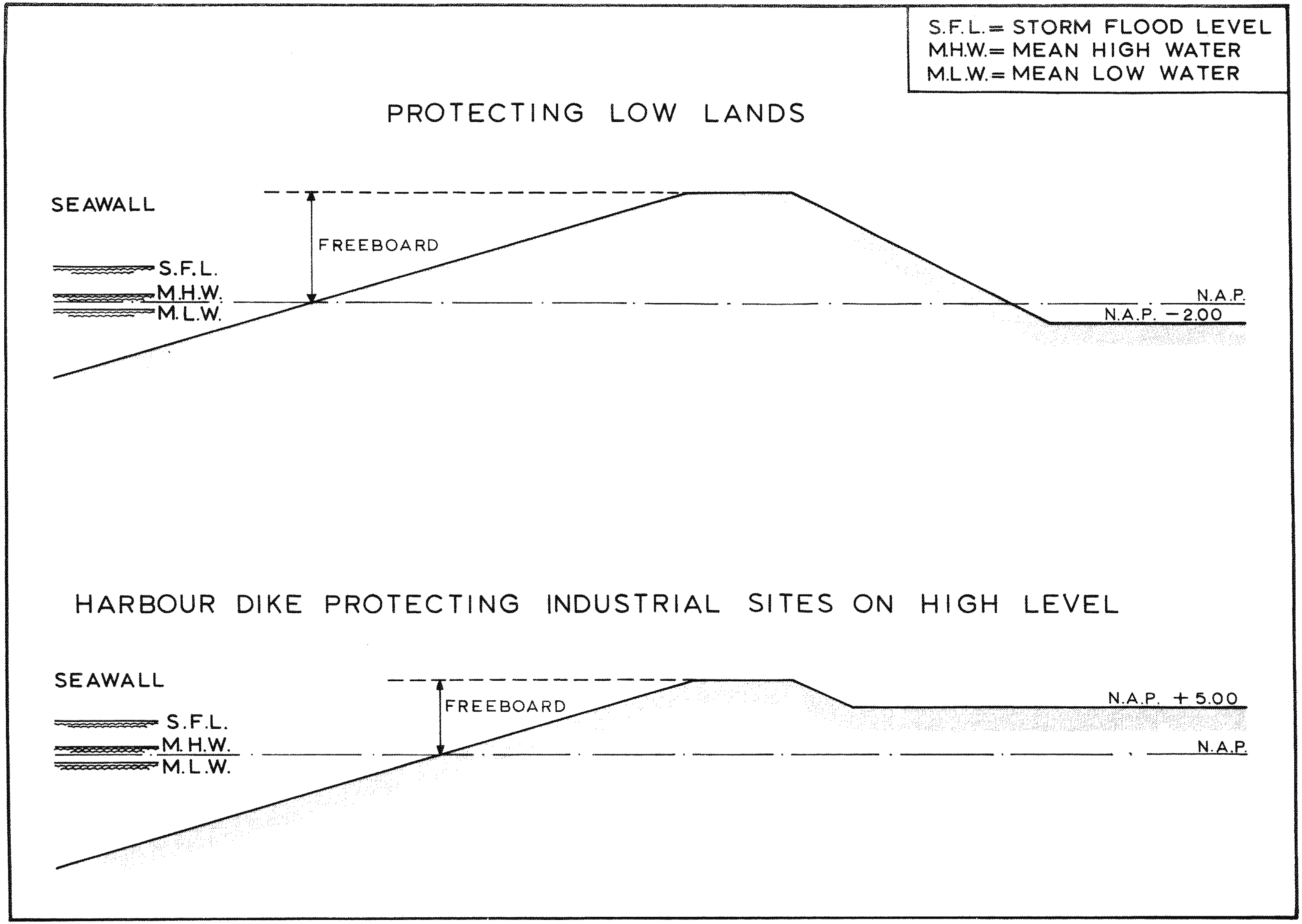

FIG. 3. FREEBOARD OF SEAWALLS IN RELATION TO LEVEL OF HINTERLAND.

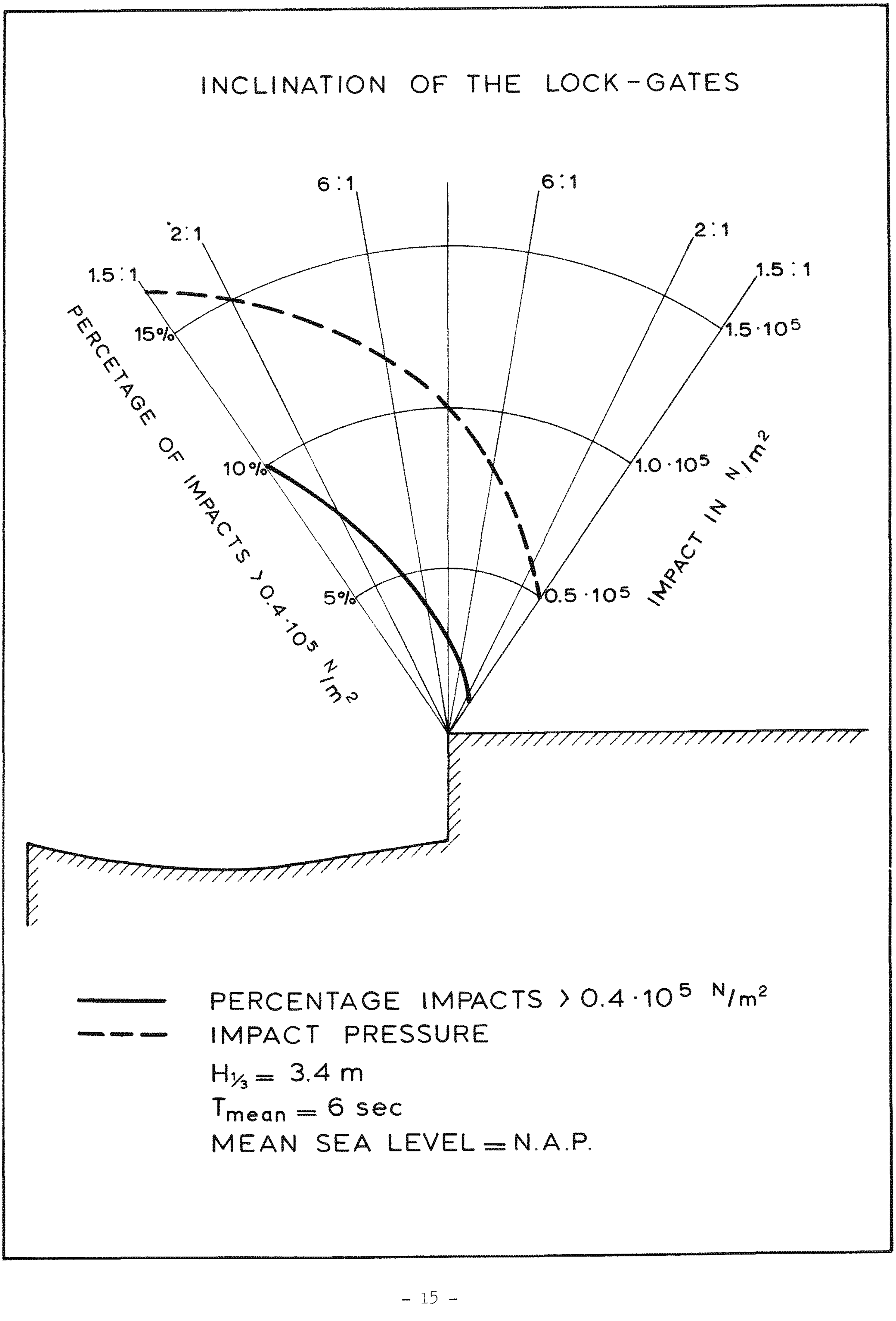

FIG. 4 IMPACT OF THE WAVES AS FUNCTION OF THE INCLINATION OF THE LOCK-GATE SEA-SIDE (HARINGVLIET SLUICES)

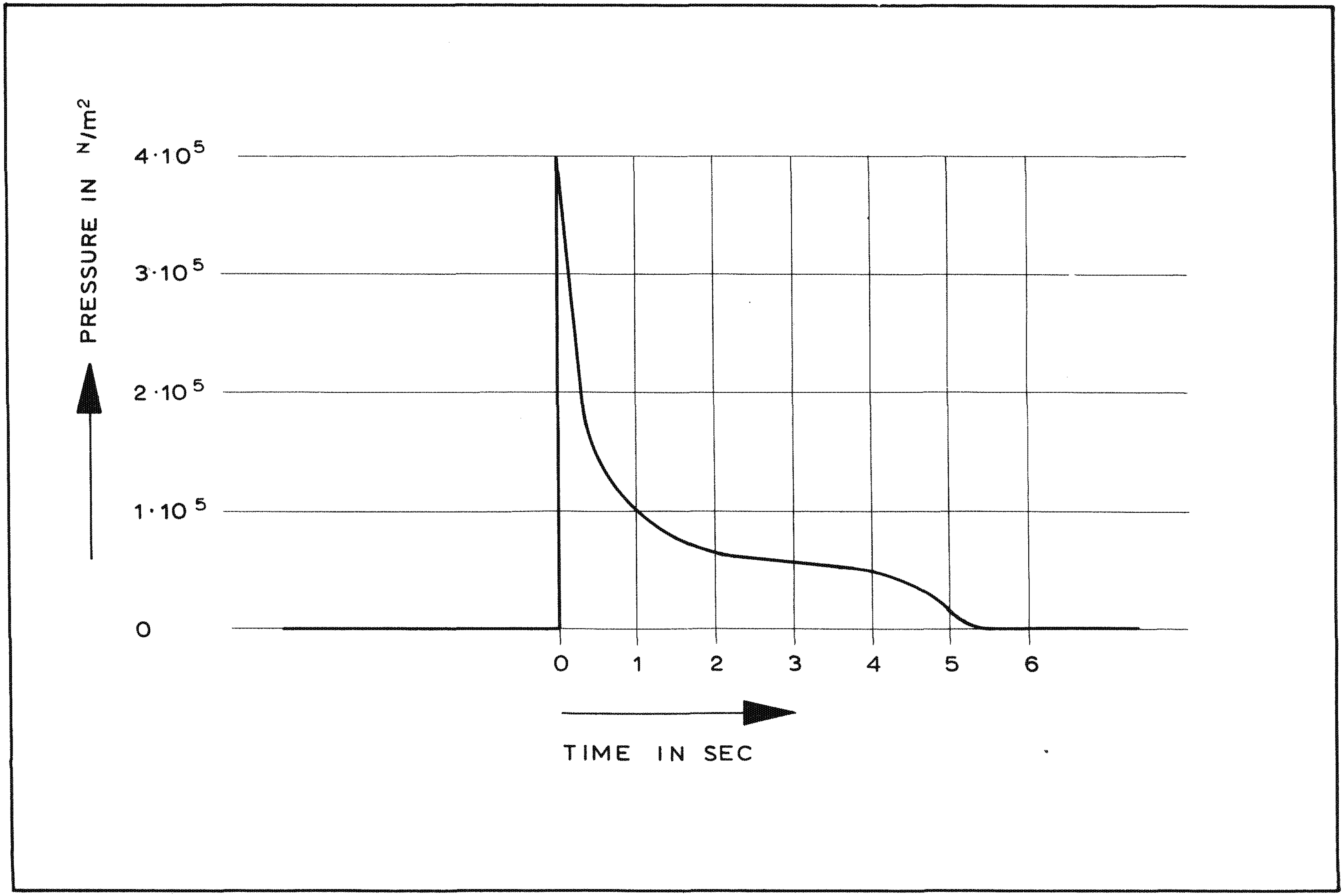

FIG. 5. RECORD TYPICAL OF THE PRESSURE ON THE LOCK-GATE SEA-SIDE (HARINGVLIET SLUICES)

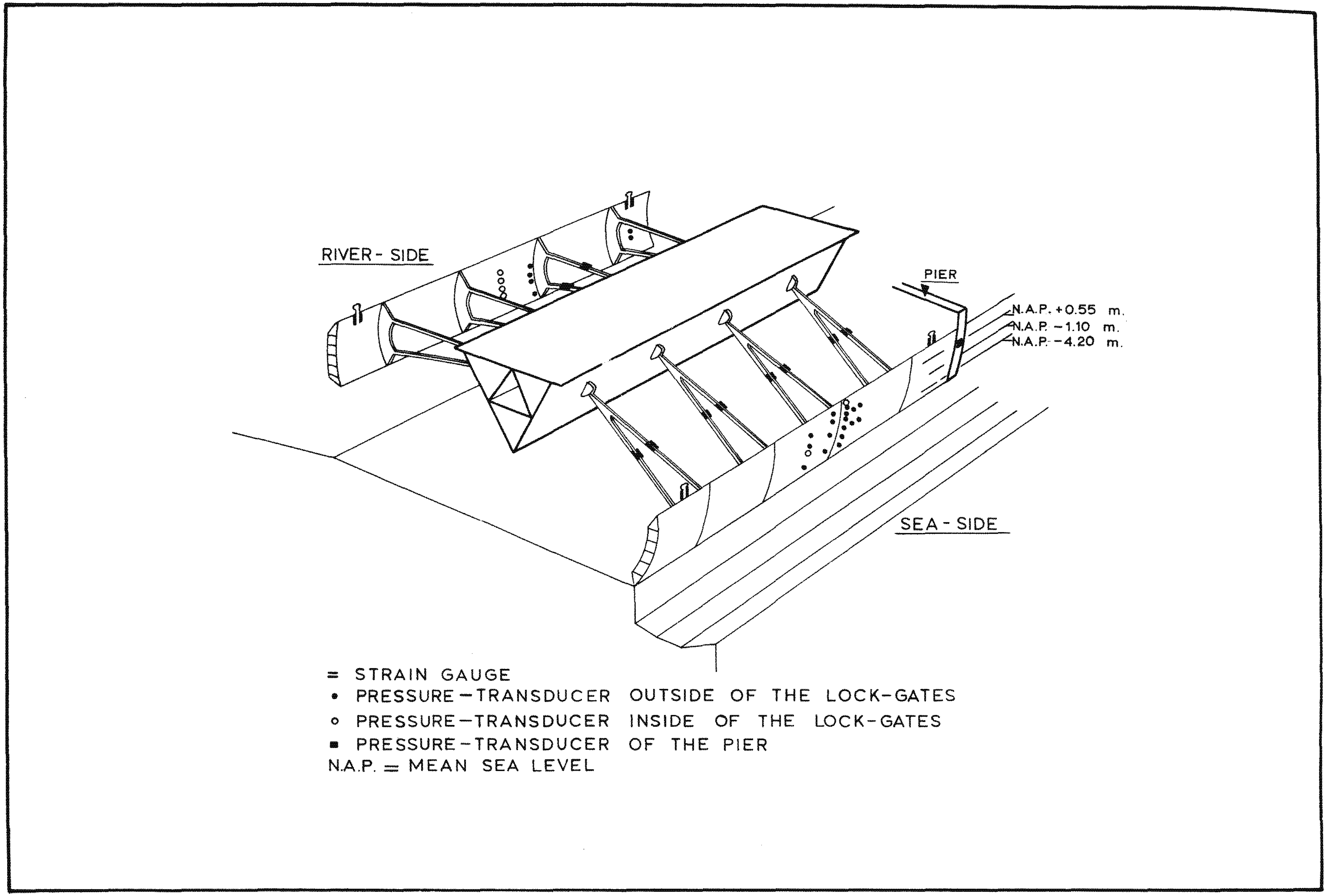

FIG. 6. REVIEW OF MEASURING-SECTION WITH LOCALINDICATION OF STRAIN GAUGE AND PRESSURE-TRANSDUCERS (HARINGVLIET SLUICES)

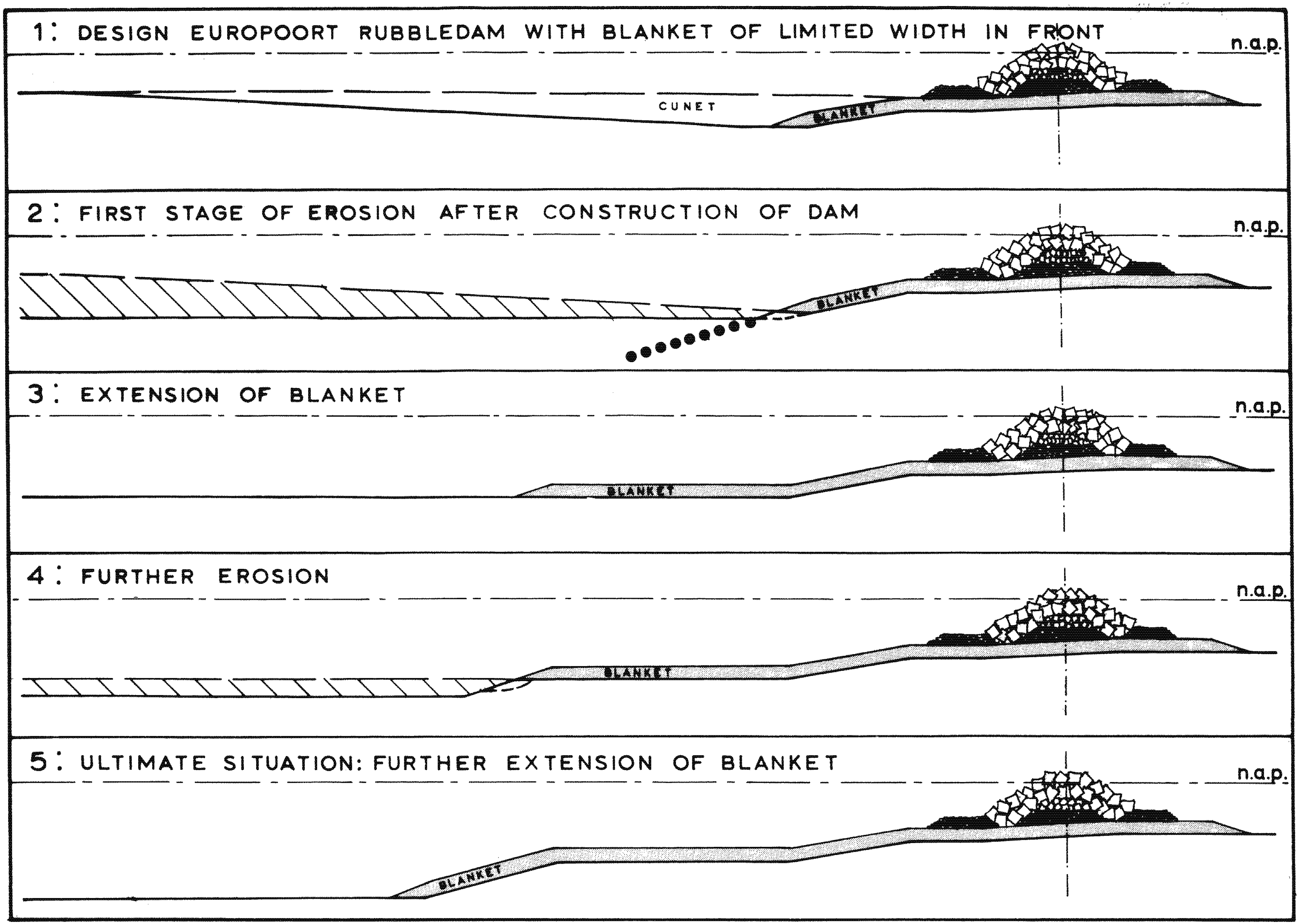

FIG. 7. PROGRAM FOR BOTTOMPROTECTION IN FRONT OF SOUTHERN EUROPOORT BREAKWATER

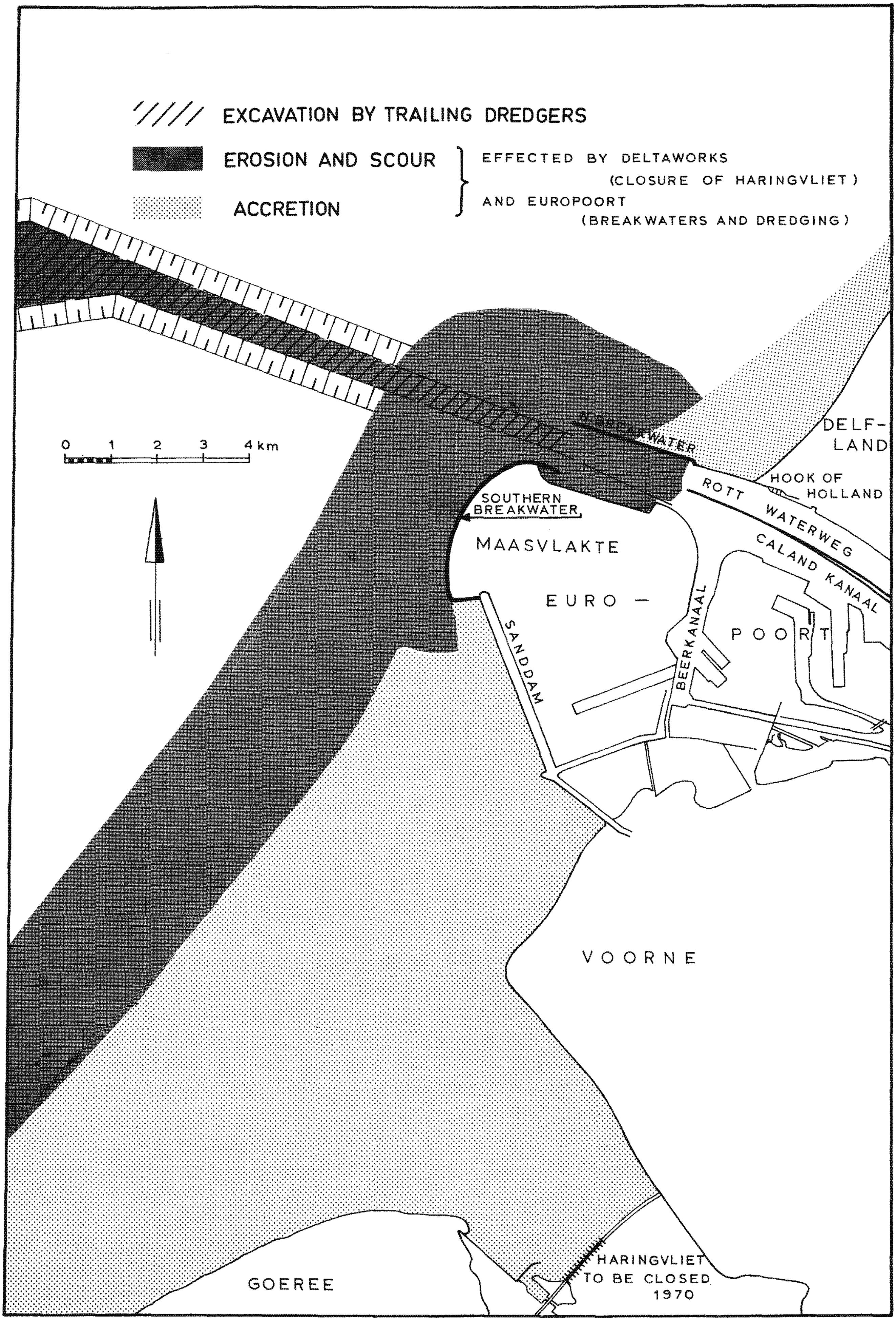

FIG. 8. EXPECTED COASTAL EROSION AND ACCRETION NEAR HOOK OF HOLLAND