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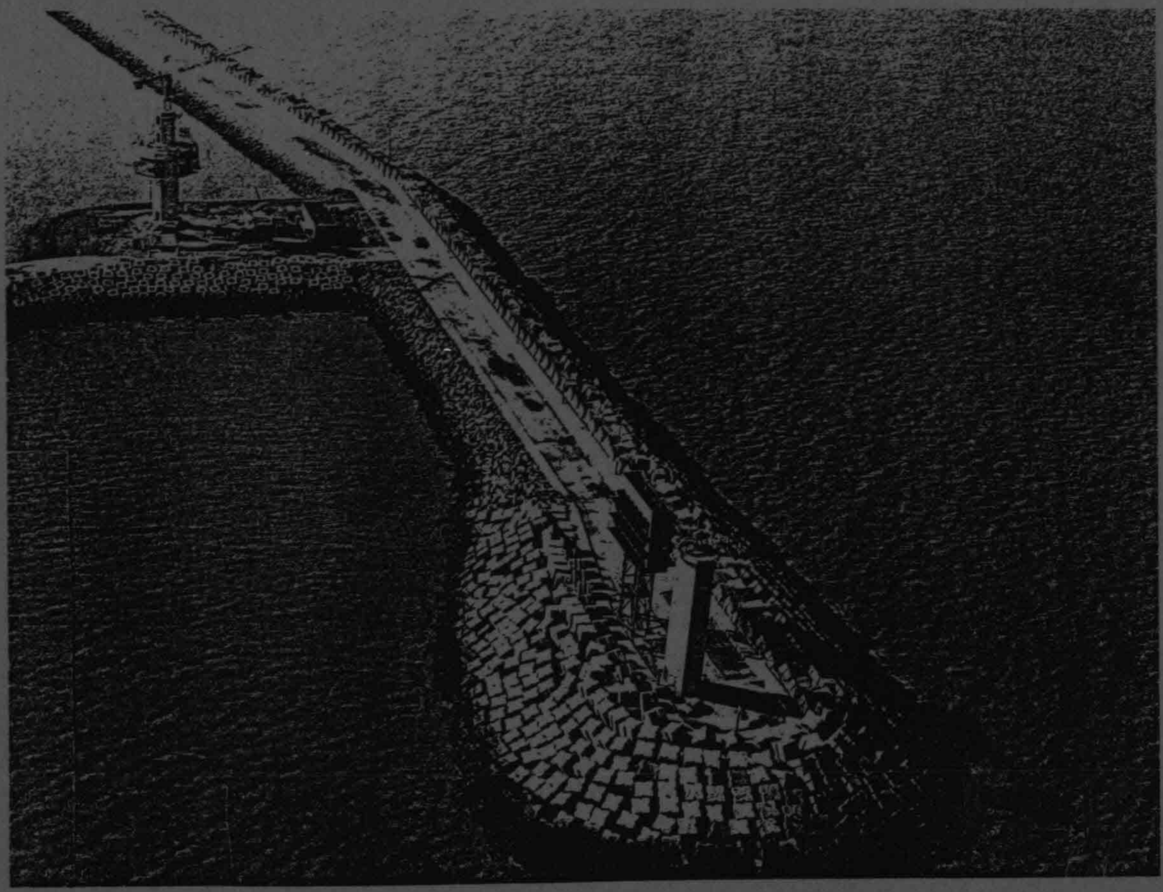

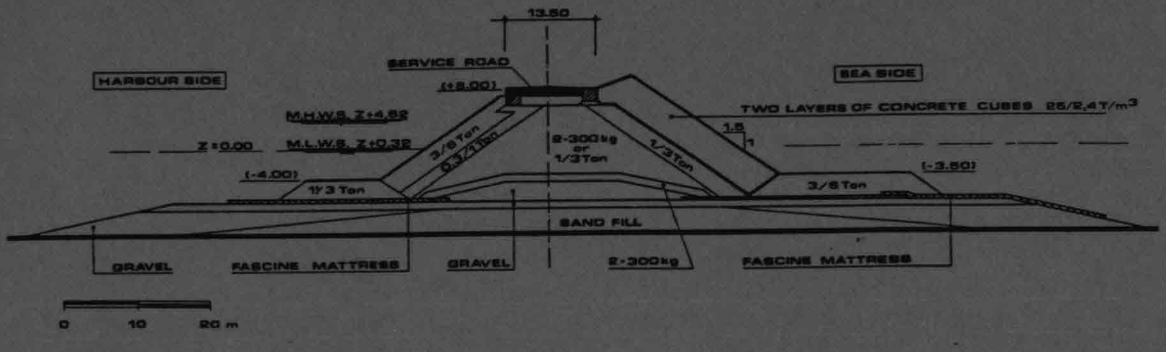

The Reliability of Coastal Structures in Vietnam in Relation to the Reliability of Wave and Water Level Data

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THE RELIABILITY OF COASTAL STRUCTURES IN VIETNAM IN RELATION TO THE RELIABILITY OF WAVE AND WATER LEVEL DATA

Master of Science Thesis

By

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Abstract

In designing marine and coastal structures there always exists a considerable uncertainty, especially regarding the reliability of the data to be used. This is primarily due to the stochastic nature of sea waves - one of the most complex, volatile, pertinacious and uncomprehensible of nature's forces. Indeed, even if using all the available sources and methods presently available, the confidence level in the determination of the design load still falls far short of what is expected in other branches of civil engineering.

In comparison with countries like Holland, U.S.A, Japan, etc. the problem with the uncertainty in designing and constructing marine structures is far more serious in Vietnam, and perhaps in many other countries of developing world. This is a consequence of usually uncertain hydraulic design conditions, the lack of knowledge and experience in design and construction, the pressure in time, constraints in money, and so on.

The purpose of this study is to provide solutions to the problems discussed above. This entails working out an appropriate design philosophy and alternatives to cope with uncertainties in designing and constructing coastal structures (in Vietnam), and thereby enhance their reliability.

Chapter-1 gives a brief description of coastal engineering in Vietnam: the typical hydrometeorlogical features of the South China Sea, necessity and potential for constructing coastal engineering works, present design practice and shortcomings.

Chapter-2 and chapter-3 deal with hydraulic environmental conditions and hydraulic design parameters (HDP). In chapter-2, emphasis is given to methods of determination of the main HDP and to identification of sources of uncertainty related to them. In chapter-3, some possibilities and measures for improving the reliability of the main HDP are investigated.

Chapter-4 is devoted to the determination of general principles/ philosophies to aid in design of coastal structures in Vietnam, at present and in the coming years.

In chapter-5 and chapter-6, sensitivity analysis for sea dikes and breakwaters is performed, aiming at obtaining relevant types of structures and armour units for the situation of coastal engineering in Vietnam.

Chapter-7 deals with possibilities to control the hydraulic loading and structure's strength, while in chapter-8, probabilistic design approach and possibility of applying it in Vietnam is considered.

Conclusions and recommendations emerging out from this study are presented in chapter-1.

Chapter 1

INTRODUCTION

1.1 Coastal engineering in Vietnam

*** General**

Lying on the west bank of the South China Sea, Vietnam has a long coastline which spreads almost from the altitude of 8° north till 22° north. Along the coast, from north to south, there are a number of estuarine and sea ports (mainly small ones) and several large sea port development projects at the study stage. Also along the coast, there are many lowland areas needing to be protected by sea dikes, and existing sea dikes need to be upgraded or rehabilitated.

The most remarkable hydrometeorlogical feature of the South China Sea is the presence of typhoons with very high frequency of occurrence. Every year, in average 8 to 10 typhoons strike the coast of vietnam, mainly concentrating on the central and north part of the country. Strong winds, large waves and high waters usually combined with intense rainfalls cause a lot of damages to coastal structures and losses to coastal economy.

*** Present design practice and shortcomings**

In Vietnam, the method presently used for designing coastal structures is the traditional deterministic method, mainly based on the Russian system of design codes and criteria.

An important shortcoming in the design practice is that due attention is not given to the collection of hydraulic environmental data in the first place, and then to the data processing and analyzing for deriving the design values of hydraulic parameters.

Regarding existing data, these are generally scarce and usually of unsatisfactory quality. Further, data collected during storm conditions which are actually used in design are not available, for most cases.

All these facts combined lead to the situation that hydraulic design conditions are usually uncertain; large uncertainties exist in the estimates of hydraulic design parameters.

Concerning the construction aspect, the present state of the art in construction is still low in Vietnam. This situation is caused by the lack of experience, and particularly the lack of various devices for control and inspection of the work. The inevitable consequence is the usually large deviations between designs and as-built-structures.

1.2 Aim and scope of the study

* Working out an appropriate design philosophy, aiming at minimization of the uncertainties in design and construction of coastal structures in Vietnam;

* Investigating other alternatives for improving the reliability of coastal structures in general;

* Performing sensitivity analysis for sea dikes and breakwaters, aiming at deriving relevant types of structures and armour units for the situation in Vietnam.

1.3 Conclusions and recommendations

<u>CONCLUSIONS</u>

Though the reliability of marine structures in general and coastal structures in particular primarily depends on the reliability of the hydrometeorlogical data used, this still significantly depends on many other aspects. They are:

* the general knowledge regarding the physical processes in wave-structure (or structural element) interaction, usually presented in the so-called design formulas which describe various failure mechanisms,
* the degree of difficulty in the design specifications, construction conditions and the

percentage of underwater work,

* the experience and skill of constructors, the availability of suitable construction equipment, construction materials as well as devices for control and inspection,
* the management and maintenance aspects, etc.

It should be stressed that there exist almost always a chance of damage or failure of marine structures. This is mainly due to the lack of reliable long-term statistical data on waves and water levels, necessary for deriving the real estimates of the hydraulic design loads. Additionally, this is also a consequence of:

* the lack of reliable design formulas/criteria, experience in design, etc. resulting in "not-as-required" designs,
* the difficulties in construction, usually caused by unfavourable construction conditions (winds, waves, etc.) and underwater work leading to "not-as-designed" structures.

In coastal engineering practice in Vietnam, the problem discussed above is of course more serious. This is because hydraulic design conditions are usually uncertain and many shortcomings and limitations still exist in design, construction and maintenance aspects.

In this study, various aspects involved and possible alternatives/solutions to the uncertainty problem have been considered. These are all combined in an effort to minimize the uncertainties in designing and constructing coastal structures in Vietnam, and thereby increase their reliability. From the study, the following conclusions can be drawn:

■ Since the reliability of environmental data to be used has a dominant effect on the reliability of coastal structures, it is wise to pay due attention to this aspect in the design process.

Water levels and wave characteristics, typically used in design of coastal structures are the extreme events with low probability of occurrence which can only be determined, in most cases, by extrapolation. It is therefore crucial to obtain water level and wave data over a period of time sufficiently long to extrapolate with confidence.

In Vietnam, since a long series of measurements hardly exists, all possible approaches: short-term measurements, long-term hindcast, old documents, etc. should be combined with procedures of quality control and rational use of the available data in order to obtain reliable design criteria. In brief, the reliability of hydraulic design parameters can be improved by:

- maximizing the data base,

- controlling/improving the data quality and
- extracting the maximum amount of information from the available data.

■ Realizing the existence of many shortcomings and limitations related to the design and construction boundary conditions in Vietnam, at present and in the near future, some general principles have been worked out to aid in design process. They can be summarised as follows:

* Structure should be hydraulically as less sensitive as possible. This principle aims at minimizing the risk of failure due to large uncertainties in hydraulic design parameters.
* Structure should be as simple as possible. The word "simple" implies the simplicity in functional and structural design. This principle is essential to prevent large damage.
* Structure should be as flexible as possible. Since there is always a chance of deformation, damage or even failure to structures, the flexibility, i.e the capability of structures to accommodate/neutralize small deformations or initial damage or to avoid catastrophic failure, is of great advantage.
* Structure should be easy to construct and maintain. This implies there is no high accuracy, no special construction method or equipment and no problem with inspection of damage and repair. This principle is important for minimizing the deviations between as-built-structures and designs (or between as-built-strength and designed strength).

In addition to these design principles, designs with the maximum use of material locally available and labour intensive construction can be important and thus should be given with due attention.

■ The fact that the designs of structures and their locations affect the hydraulic loads offers possibilities, within certain limit, to choose these loads. By manipulating the location or layout of a structure, more favourable hydraulic conditions can be obtained. Also, by a proper selection of the crest height, slope steepness, construction material, geometry and configuration, it is possible to choose the size, the sort and the place of attack of the hydraulic loads.

A better insight in the wave-structure interaction and various failure mechanisms gives possibilities to control the structure's strength. By increasing the weight or density of elements, the degree of "cooperation" between individual elements (or layers) or permeability factor (for rubble structures), it is possible to improve the structure's stability.

■ Concerning design approach, probabilistic approach is evidently the best method to guarantee safety and economy. However, the application of this approach in the actual design is still mainly confined with semi-probabilistic calculations due to many reasons. In principle, level-2 probabilistic methods are possible to apply in Vietnam; however level-1 calculations might be of more practical use (in design of rubble mound breakwaters).

■ In case of sea dikes, from the performed sensitivity analysis using the wave overtopping criterion, the gentle slope dikes without a berm appear to be less sensitive to variations in wave height and period than those with a steep slope and a berm. However, quite gentle slope dikes can also be risky when overflowing failure mechanism is considered. This is particularly the case when uncertainties in the design water levels is large, while the design wave heights are low.

■ In case of breakwaters, it appears that the dynamically stable concept is generally the most relevant alternative for the situation in Vietnam. The fact that small stones can be used and large construction tolerances can be accepted allowed the use of common construction equipment available in Vietnam, and particularly allows construction with limited-skilled labour. However, the decisive point that makes the dynamically stable concept relevant in Vietnam is the high flexibility of this concept. Under circumstances of the large uncertainties in hydraulic design parameters, low risk and economically-efficient solutions demand robust and flexible designs with a wide margin between start of damage and total failure. To face large uncertainties, economic optimization leads to the very conservative and therefore very expensive designs for rigid structures, and to less expensive and safer designs for flexible ones.

Nevertheless, it should be stressed that the correct type of structure, whatever it be, is very site-specific. For certain conditions, dynamically stable concept may be not relevant. Then more conventional rubble mound structures with concrete armour units are likely to be appropriate, also due to their relatively high flexibility compared with the vertical wall concept. In this case, the so-called D-armour breakwater (Figure 6.8) appears to be a good solution. In the D-armour design, the armour thickness around still water level is significantly increased, compared with the conventional designs. This has two advantageous effects:

- stability improvement of the most vunarable part of armour layer due to the increase of permeability in this part,
- increase of the margin between start of damage and failure due to some reservation

in armour thickness for the safe erosion when the design loads are exceeded.

■ Regarding armour units, rock and simple shaped concrete units like cube, antifer cube, etc. are generally more relevant than tetrapod and other complicated shaped units. However, accropode deserves attention, whenever concrete units are involved in consideration, due to having high stability, strong design and relatively simple shape. By applying an adequate safety factor to face uncertainties, both economy and safety might still be achieved with this unit.

Finally, it is necessary to mention that the best economy and safety can be achieved, if designers are also flexible, i.e are not too rigid in the approach to the problem of coastal structure design and prepared to adapt to natural local conditions. The best way to guarantee economy and safety is to work with the sea rather than against it, and use the materials which are locally and economically available rather than insist on fancy quality of the work at greatly increased expense.

RECOMMENDATIONS

To face large uncertainties in designing and constructing coastal structures (in Vietnam), the following recommendations are considered to be appropriate.

a) For coastal structures in general:
- * Structure should be hydraulically less sensitive, simple in functional and structural design, flexible and easy for construction and maintenance.
- * Dynamically stable concept.

b) For sea dikes:
Relatively gentle slope dikes with no berm.

c) For breakwaters:
Berm breakwaters,
D-armour breakwaters

d) For armour units:
Rock,
Simple shaped concrete units like cube, antifer cube, etc.

Chapter 2

HYDRAULIC ENVIRONMENTAL PARAMETERS

2.1 Hydraulic boundary conditions

For the design of coastal structures, a good understanding of the coastal environment at the site under consideration is essential. This encompasses coastal morphology, hydraulic boundary conditions, geological and geotechnical boundary conditions and sedimentation and lithological processes. However, within the scope of this paper, only hydraulic boundary conditions are to be considered. Two aspects of primary concern are:

* water level

* wave climate

2.1.1 Water level

Water level is an important aspect in the design of marine structures. Besides water level determines the level of the hydraulic loads or the magnitude of the hydrostatic pressure, it may affect the principle hydraulic load - the waves. In shallow water, wave parameters are dependent on water depth, i.e on water level.

In coastal waters, water level is a summation of the four principle components: tides, storm-surges/wind set-up, wave set-up and secular changes in mean sea level (sea level rise). Among these four components, tides and surges are the major contributors to the water level fluctuations whereas sea level rise and wave set-up have minor effect.

- *Sea level rise*

For many years it has been known that the sea level is rising, though very slowly. This can only be detected and quantified, for a given area on the world, if there are long-term (century) data on both water level and land level, as land level also changes, especially in delta areas where land subsidence is the common phenomenon.

For engineering purposes, not the absolute value of the sea level rise, but the rise relative to the land level is of interest, and this simplifies the matter since only long time series of water level observations is required. Nevertheless, it is still a problem because these long time data series is not available, in most of cases.

Fortunately, the rate of sea level rise in the last century was quite small, possibly no more than 20 cm/century, though it is expected to increase in the next century. This can be judged through the well-known relative rise of approximately 20 cm/century in the Netherlands where land subsidence is obvious. As a result, sea level rise should be taken into account in the design of coastal structures only in so far as it significantly affects the design wave heights and beach response within the life time of structures.

- *Wave set-up*

When the wave train approaches the shore, the forward movement of the water particles under the waves is not entirely compensated by the backward movement due to bottom friction. As a consequence, there is a general movement of water in the shore-ward direction. This movement causes a rise in water level within the surf zone which is known as wave set-up.

In addition to the effect of enhancing the water level, wave set-up may produce rip currents and longshore currents due to the possible different set-up levels caused by the variations of wave heights along the coast.

- *Tides*

The water level changes in seas, bays and estuaries are largely determined by tides. There is no need to give a detailed account of tidal phenomenon in this paper, as the matter is fully dealt with in many books. However, it might be necessary to stress the influence of tides on coastal wave climate and coastal processes which, in turn, have an impact on the designs of coastal structures.

As already mentioned above, in shallow coastal areas, (maximum) wave height may strongly depend upon water depth while the latter can be significantly increased or decreased according to the tidal motion. What is more, both vertical and horizontal tide affect the longshore current pattern and therefore affect the sediment transport along the coast.

Regarding tidal levels, these can be directly obtained by measuring, but these can also be predicted for a given location by using tidal constants derived from a limited period of local water level observations (during usual weather) in harmonic analysis, or by scaling values to the nearest place where tidal levels are known. Concerning prediction based on the tidal constants, one should, however, be aware of the fact that tide along the coast or in estuaries may change due to morphological changes or large civil works. Consequently, long term tidal predictions should be treated with care to avoid gross errors caused by the effect of those changes on tide, i.e on tidal constants.

- ***Storm surges/ wind set-up***

Along the coast, sea levels can be significantly raised high above normal tidal levels by storm surge/ wind set-up (actually by reduced atmospheric pressure and the drag effect of strong winds over the water surface). This can happen during stormy weather, but this can also be caused by a very long lasting wind system like a monsoon or a trade wind.

Effects that can play a part in determining storm surge are wind set-up, reduced atmospheric pressure, rotation of the earth and storm motion. Of these the largest effect is generally produced by wind set-up which is strongly dependent on wind speed, wind fetch, bottom topography and elevation and coastline geometry. Significant wind set-up can be expected in shallow seas or bays with convergent coastlines like South China Sea, the northern gulf of Bengal in North Sea, etc. For sea of limited extent, storm motion effects may be capable of exciting the resonance of the sea basin and accordingly increasing storm surge levels. For large bays, it is possible for the natural modes of oscillation of the bay to excited as well, and this can further amplify the surge level.

Storm surges can be estimated using various formulas or computational models, but the results are not very reliable due to the complexity of the matter concerned. Therefore, it is common practice to obtain storm surges by subtracting the predicted tidal levels from observed ones. This method is also applied to give storm surges caused by typhoons along the coast of Vietnam.

It should be noticed that, though tides and surges are quite different in their origin of generation, in shallow water, tide-surge interactions may occur, that is tides and surges are not independent due to the fact that surges depend upon water depth.

2.1.2 Wave climate

In the design of marine structures, knowing the wave climate at the site of interest may mean already solving the most important and difficult part of the problem. Unfortunately, for most places, this is not the case due to the very complex nature of the matter concerned and the scarcity of long-term statistical data necessary for establishing wave conditions.

Waves commonly observed in oceans and seas are generated by the wind action. In areas where waves are being generated, the sea surface appears very confused because of interaction between water and winds and between waves. Moving out of the areas of growth, waves adopt more orderly pattern, but generally they are still random in height and direction.

In deep water, wave climate can often be effectively determined by the application of parametric wave hindcasting techniques using long-time series of recorded wind data that are usually available at some on-shore anemometer station at or near the site of interest.

The problem arises when it comes to determine the wave climate in shallow waters near shore. Approaching shallow areas, deep water waves experience the effects of refraction, diffraction, shoaling and breaking, and as a consequence, wave properties (wave height, direction, steepness, etc.) may be significantly changed. Further, in shallow waters, wave properties can also be affected by the presence of tidal variations or strong currents. All these combined make the wave climate in coastal waters very site-specific and very difficult to be established.

For site with (relatively) simple sea-bed topography, wave characteristics can be derived from forecast deep water waves taking into account shallow water effects, manually or with the help of numerical models. Nevertheless, for site with complex sea bed topography, the transformation of deep water waves becomes unreliable. In this situation, if there is no suitable existing wave data, site measurements are necessary.

It should be stressed that wave and wind are not always correlated. This may occur when some decayed swell from other place comes and interferes with locally generated seas.

2.2 Hydraulic Design Parameters (HDP)

2.2.1 General

Sea conditions can be identified as normal and extreme (storm) conditions. It is known that marine structures are not sensitive to small waves during normal weather; but they are most prone to response to largest waves generated during rare storm events. Thus severe storms become superimposed on the normal weather as an added variable.

In the design of coastal structures, the determination of the hydraulic design conditions is the result of quantification of the local conditions in combination with a certain level of safety. In this way, the design condition, some selected extreme event, is usually defined and presented in the form of water level, wave parameters/characteristics and an estimate for the duration of the design condition. Of these, the most important hydraulic design parameters are :

- water level
- wave height and
- wave period.

2.2.2 Determination of HDP

In the engineering practice, water level and wave information for design studies usually cover a period of observations or prediction that is much shorter than the return period required in the design of most of coastal structures. Consequently, the determination of the extreme values of the design water level and wave parameters has to be based on extrapolations of the available data to a longer period. The general procedure is to represent available statistical data by a certain probability law which governs the data, and then obtain necessary information for design from the probability function.

The major problem is that each of the probabilistic distributions attributed to normal and rare events can not be simply added or combined. Also, if the design studies rely on usual weather data only, the extreme event wave properties will probably be vastly underestimated;

if based on extreme values in attempt to synthesize the properties of extreme events, these could be widely in error due to lack of data.

- ***Design water level***

For determining an extreme value of the design water level, the method of Jenkinson known as "General Extreme Value Method" may be used. This method relies on statistical analysis of annual maxima and on the assumption that a series of extreme events is random or contains a linear trend. Although this method is relatively simple, it requires decades of water level observations (15 years as minimum, Lenon & Suthons, 1963) to allow extreme value analysis of the annual maxima to be carried out reasonable confidence. Also, it is difficult to obtain consistent, stationary results due to variable trends in the maxima (Graff, 1981).

Quite often , only a few years of sea level data are available, and as a matter of course, the method of extreme value analysis is not applicable. In this case, it is necessary to resort to detailed and laboursome joint probability analysis of tides and surges. This entails separating tides and surges in tidal record (by harmonic filtering techniques) so that probability distributions can be derived for each. These are then combined to produce the probabilities of extreme high water level. The combination of frequencies of predicted tides and residual surges provides a synthesized record of extreme water levels extending to events with return period of far much longer than the record time, and therefore enables extrapolations to be made usually within the range of observed residual surges and predicted tides.

Nevertheless, when analyzing tides and surges as separate components, it is important to recognise the seasonality of events, particularly if data sets are not complete. Also, there is implicit in this approach the assumption that the probability distribution of surges is the same for all tidal levels. However,in shallow water, a tide-surge interaction can occur, i.e tides and surges are not independent, as already mentioned. If this is likely to happen, determining separate surge probabilities related to different part of the tidal range is necessary.

- ***Design wave height***

Depending on the type and amount of data available at the site of interest, wave heights necessary for design of coastal structures can be estimated by the following methods:

* Estimates of the design wave heights may be directly obtained by extrapolating a series of wave records made at the site. This is possible if there exists at least one year of wave data and provided that representative storms did occur in the relevant fetch areas during the recording period.

* Another method is to apply wave forecasting technique to generate offshore wave data from wind data which are usually available at or near any site along the coast. Next step is to transform predicted offshore waves to the structure's location by taking into account typical shallow water effects of the inshore sea bed topography. Then extrapolation of predicted wave heights at the considered inshore site may be carried out to obtain the estimates of the design wave heights.

A very important point to be considered in determination of the design wave heights by extrapolation is that there may exist some physical process which interrupts the probability distribution found at a higher frequency. The wave breaking mechanism and other dissipation mechanisms controlled by water depth automatically limit wave height in shallow water. Further, fetch restrictions may also limit the wave height.

Of course, before any extrapolation can be made, wave height data, either predicted or measured, should be fitted to some probability distribution law. There are a number of probability distributions that have been found and used in engineering practice; they are : Exponential,Log-normal, Weibull, Gumbel, Fisher-Tippet, Frechet and Gompertz. Depending on a given case, one of these distributions may best represent the data set.

However, it may sometimes be found that one distribution will fit the lower wave heights well while another one will fit the higher wave heights better. In this situation, the distribution with the best fit to the larger waves should be used for extrapolation, as the interest is of large waves, but small ones. In addition, since wave height may be limited by breaking criteria, the Fisher-Tippet distribution/ in which the wave heights are restricted by an upper limit, may give the best fit.

2.3 Uncertainty in the HDP

2.3.1 Sources of uncertainty

Identification of the possible sources of uncertainty contributing to the total uncertainties in the design values of hydraulic parameters is necessary for :

- Estimation of uncertainties in the hydraulic design parameters which enable the evaluation of the reliability of designs to be made

- Improvement of the reliability of the hydraulic design parameters.

The following elements are the main contributors to the total uncertainty :

* Errors/inaccuracies existing in data sets. These may be :

    - Errors introduced from the measurements which are related to instrument response or observer's experience. Also, the position of measurement might cause errors due to local effects. Characteristics of shallow water waves can vary considerably in areas with complex sea bed topography. Surge levels and accordingly sea levels may be affected by local geography.
    - Variability and errors caused by different and imperfect hindcasting methods or models
    - Statistical sampling errors due to short-term randomness of the variables (variability within stochastic process, e.g two 20-min records from a stationary storm will give two different H_s)

* Variability due to different data processing and analysis techniques, that is different algorithms, smoothing and filter limits

* Statistical uncertainties related to extrapolation from short data samples to design events of low probability of occurrence, i.e extrapolation to a range remote from the exist data. First, small sample itself is likely to introduce deviations from the distribution of a given population, for there is hardly any chance that the small sample is a representative of the general (wave) climatology. Simply, fundamental laws of statistics are based on the assumptions of large population numbers and not on a small sample of observations which may be taken at "wrong" time. Second,there is no guarantee/certainty that the correct distribution function can be derived from sample analysis, and also the true long-term distribution of population is unknown, e.g for significant wave height H_s. Many distribution forms, e.g Weibull and Gumbel, often fit the wave data sample well but may give quite different estimates of extreme values.

In addition to the above listed sources of uncertainty, the possible long-term changes in the climatic pattern may also contribute to the uncertainty in the hydraulic design parameters, as extrapolation techniques are based on the assumption that the wave or surge-generating mechanism does not change with time. In shallow water, the long-term morphological changes may affect the wave climate, the tides and surges, and consequently may cause uncertainty in the hydraulic design parameters as well.

2.3.2 Estimation of uncertainty

Based on the personal experience and judgment, a tentative estimation of overall uncertainties inhering in the main hydraulic design parameters typically used in the design of the coastal structures in Vietnam has been made. The estimates of uncertainties are given in terms standard deviation, σ, or normalised standard deviations, σ', equal to standard deviation, σ, divided by mean value, μ (Table 2.1).

Hydraulic design parameters	Method of obtaining data	Mean value μ	Uncertainty σ or σ'
Water level	- Measurements - Tide prediction and wind set-up calculations		σ = 0.5-1.5
Significant wave height offshore	- Hindcast, SMB - Visual observations	3-5m	σ = 0.2-0.3
Significant wave height nearshore	- Manual calculations from offshore waves - Measurements - Visual observations	2-3m	σ = 0.2-0.5
Significant wave period	- Manual calculations - Visual observations	7-10s	σ'= 0.2-0.3
Wave direction	- Visual observations - Based on wind data		σ = 15°-30°

Table 2.1 *Estimates of uncertainties in the main HDP in Vietnam*

Chapter 3

IMPROVING THE RELIABILITY OF HDP

3.1 Reliability of HDP

Reliability of HDP has a large, if not decisive, influence on the reliability or the risk of failure of maritime structures, and subsequently, on the cost of construction and maintenance. Indeed, small variations in the selection of design wave heights, for breakwaters for instance, will significantly influence the required block weight of armour units, as this is known to be proportional to the third power of wave height. Other dimensions of structures such as the thickness of armour and under layers, crest height and width, etc. are also sensitive to variations in wave height.

Reliability of HDP depends upon quality and quantity of the available data on winds, waves and water levels used for deriving these parameters. For reliability of HDP, quality of data is important, but the extension of the time base is particularly crucial because the estimates of HDP are derived by extrapolations, as already mentioned. Estimates of extreme design conditions (obtained by extrapolation) rely on years for which observations were used being *typical*. If during the observation period storms were particularly severe or mild, then the extrapolations will give overestimates or underestimates of extreme values, respectively. However, the extrapolations should become more reliable if the data sample covers a greater number of years. For example, it is evident that the estimates of waves having an average return period of, say, 50 years would be quite reliable if there existed 50-100 years of wave observations. But instead, there may exist only 1, 5 or 10 years of observations, requiring questionable extrapolation to the 50-year probability level.

In shallow coastal waters, water levels and wave climate are quite site-specific, as analyzed in chapter-2. Accordingly, reliability of HDP to a great extent depends on whether there is data directly applicable to the study site or not. This is because the data obtained by hindcasting techniques or transformation of data from other places are subject to various errors. However in practice, far too often there is little or no data directly applicable to the project site; almost inevitably there will be no data based on actual measurements of waves and water levels. Consequently, a site measurement program is necessary, but this is unlikely to result in much more than one year's worth of data, perhaps due to restrictions in both time and money. It is therefore necessary to have recourse to an examination and analysis of other sources of data such as visual observations of ships in passing, wind data from shore stations, etc..

3.2 Improving the reliability of HDP

Reliability of HDP can be improved by the following ways :

- maximizing the data base
- controlling/ improving the data quality and
- extracting the maximum amount of information from the available data.

▪ *Maximizing the data base*

The ideal way to ensure the reliability of HDP is to have instrumental wave and water level data collected at or near the required site during a long time span, not in the order of years but decades. For water levels, this is unquestionably possible, provided there is no restrictions in time. For waves, this is presently unattainable and will remain so for most locations, at least in the foreseeable future.

Though each of the data sets actually available suffers from one or more shortcomings, if compared with those obtained by the ideal way of collection sketched above, they nevertheless contain at least some useful information. It is wise therefore to reject none of them, but to maximize the data base. Getting data related to the site and problem concerned may require a thorough and laborsome search in various files, from various institutions; and perhaps not less effort in making use of all found data, as these may significantly differ in quality and observation length caused by different purposes, and therefore different ways of observing. This is however well worth doing for major projects, considering large consequences or enormous costs of under-design or over-design caused by unsatisfactory reliability of the HDP.

The maximization of the data base should be both in quantity and quality . The quantity here refers to coverage in space (i.e not only the site under consideration, but also the adjacent areas are of interest) and time (observation period) and to variables involved. The quality, in general, implies procedures of quality control, intercomparisions, interpolations, and so forth.

- Since the extension of the data base is of particular importance, the inclusion of long-term wind data, and perhaps visual wave data is almost always necessary. This in turn requires necessary procedures presented below :
  - Hindcasting waves or wind set-up from wind data
  - Correlation of wave data with wind data
  - Validation of visual data with instrumental data.

- The data base can also be expanded by carrying out extra measurements. In principle, it is desirable to have site observations extending for as long as possible. However, for most locations, long-term measurement programs are unlikely to be feasible due to different reasons, for example restrictions in time or money. In some cases, the gain obtained in extending the measurements is so small that it can not be economically justified. In other cases, they are not necessary at all.

For water levels, site observations are usually required to enable the correlation with data at other locations or with hindcasting data to be established. However, if the site is remote from any long-term data recording locations or is likely to experience increased or decreased levels due to local effects, a long time program (at least 4-5 years) of water level observations is necessary to enable the collecting data being used alone. Alternatively, the long-term measurement program can be replaced by a short-term period of observation combined with the use of an appropriate storm surge numerical model.

For waves, due to the very site-specific character, data collected at the project site are of great value. Therefore, in the absence of adequate existing data, site measurements should be taken. The obtained data are primarily used to verify hindcasting or visual wave data, or to enable correlations of waves at the study site with wave data in deep water or at other locations to be found. Nevertheless, site observations may also be used alone if the extension of measurement period is sufficiently long to include typical storm conditions of the area considered. Site observations are particularly necessary where:

- complex sea bed topography at the site would render transformation of hindcast offshore waves unreliable

- wind data for wave hindcasting at the site is unsuitable or of doubtful quality
- the presence of currents or tidal variations in the study area is likely to affect the wave characteristics significantly
- the presence of swell or long wave action is likely to be significant, but is often not included in the standard wave observations.

■ ***Controlling/Improving the data quality***

Reliable design criteria can only be based on reliable environmental data. The reliability of data, in turn, depends on procedures of quality control/improvement, first during data collecting stage and later during data processing, analyzing and determining the design conditions.

- Regarding recording instruments, it is important that they should be robust and suitable for the marine environment, as well as being accurate and properly designed. However, malfunction of instruments is a common problem, both due to inherent faults and interference from outside. Therefore, to avoid loss of data, it is desirable to duplicate the facility or to have back-up system and, in any case, to check them regularly. Although many of the newly developed instruments appear to offer advantages over the older systems, owing to their higher accuracies and large storage capacities, a simpler and well tried robust instrument is generally more reliable and therefore likely to yield a higher data return. In other words, the simpler the measuring principle, the more reliable the system will be. It should also be mentioned that for simpler systems, their functioning can be controlled easily and repairs can be done locally; and this is essential for a country like Vietnam.

- Another aspect that can affect the reliability of data is the installing and siting of recording-instruments. Generally, instruments should be installed properly and sited away from places which are potentially affected by local effects such as reflection, diffraction or whatever. Some considerations for siting wave recorders are: To determine the general offshore wave climate, the wave recording instruments should be sited in deep water, relative to the expected wave lengths, adjacent to study areas, if possible in a position from which wave orthogonals can be easily transferred to the points of interest nearshore. Also, they should be sited away from local reflective and diffractive features and from uncharacteristically strong currents which may affect the wave climate.

- Reliability of HDP can also be improved by paying due attention to the quality control in data processing and analysis and in extrapolations. The following are some considerations on this aspect:

Quite often data sets contain gaps. The fact that gaps do exist in data series, of course, reduces the data quality; however if gaps are small, relative to the record length, and randomly distributed, they cause no significant negative effects on the reliability of data, and even can be filled by interpolations. Conversely, if gaps are systematic and occur at particular times of the year, e.g recorders are often damaged during storm season, the reliability of data become questionable because in this case gaps may produce a bias in statistics.

Extrapolation techniques used for deriving extreme design values are based on a large number of events of a population and their independence. Accordingly, the original data set should be statistically independent and care should be taken that this requirement is not overlooked in an attempt to increase a limited data base by including non-independent observations. For example, one year of wave observations with a small sampling interval of 3 hours will provide a data base of more than two thousands values. This large number of values could be extrapolated. However, the results of the extrapolations are doubtful, since the original values sampled at every 3 hour interval cannot be considered as statistically independent.

To avoid bias which may be introduced by the effect of seasonality, it is important that only a complete set of data, i.e a complete year or years are used for extrapolations to obtain extreme design values. Of course, this refers to the case when all observed values, taken at every 6-8 hours for waves for instance, are being used to extrapolate. In case when a extreme value or some extreme values (which occur during storm season) in every year are being used for extrapolations, only data observed in storm season every year are required. From technical and economical point of view, the last case is quite attractive, but more years of observations may be required, and care should be taken, as extreme events may occur outside the usual storm season. Also, since extrapolation is sensitive to the inclusion of large waves, which are generated during storms, it is crucial to ensure that typical storm conditions have occurred during the observation period.

Broadly, visual wave height data are not quite reliable. However, the reliability of these data can be improved if they are treated jointly with the wind data, as proposed by Hogben and investigated by Batties (ref.1). It was found that the improved visual wave height statistics are in faire agreement with instrumental data, provided the latter are also of sufficiently long duration. Additionally, many visual data sets are even better than instrumental ones with the gaps caused by the failure of instruments during storms.

- ***Extracting the maximum amount of information from the available data.***

As shortage of data is a common problem in engineering practice,
maximizing the data base and controlling/ improving the data quality are the two approaches for improving the reliability of HDP. Another method is extracting the maximum amount of necessary information from data for the purpose at hand. Two typical examples of this method are as follow :

a) Long-term wave statistics are extremely valuable in providing information for the design of marine structures. Nevertheless, a problem always exists in the use of this information for design. That is, data in severe seas, which are indeed necessary for design, are unreliable because such data are sparse.

   One method for solving this problem was proposed by Michel K.Ochi in the paper titled "On long-term Statistics for Ocean and Coastal Waves" (ref.11). The method is based on the statistical inference concept to establish the confidence domains from the data, taking into account the correlation between significant wave height and period. The general procedure is to find the joint probability density function of significant wave height and wave period. Then, based on this function, confidence domains are derived through some intermediate transformations. The significant benefit of drawing the confidence domains, as pointed out by the author, is that information in severe seas, where the data are always sparse, can be reasonably estimated from the overall data.

b) Extreme high sea levels can be estimated by using one of the traditional methods of extreme value analysis of annual maxima. These methods are quite handy and reliable if there is available a long series of observed annual maximum sea levels. However, these methods have serious limitations, concerning conditions for applying and rational use of the available data. The requirement of long time series of data (many decades) which rarely exist for most locations, and the large waste of data (only a maximum value of the whole year water level observations is used).

   In contrast, methods based on joint probability analysis of tides and surges, as already presented in Chapter 2, can rationally make use of the available data. The method was first used by Ackers and Ruxion (1974), and then further developed by Pugh and Vassie. It is given in the paper titled " Extreme sea level from tide and surge probability " (ref.15). This method can extract the maximum amount of necessary information from the available data, and as a result it produces realistic estimates for extreme sea levels by comparison with traditional methods, but from much shorter duration of data .

Summary:

Water levels and wave characteristics, typically used for design of coastal structures are the extreme events with (very) low probability of occurrence.These can only be determined, in most cases, by extrapolation. Therefore, it is crucial to obtain water levels and wave data over a period of time sufficiently long to cover the life time of the structures, i.e to guarantee the prescribed reliability of HDP.

And since a long period of measurements rarely exists, all possible approaches: short term measurements, long-term hindcast, old documents, papers, witnesses, records of the past destruction, etc, have to be combined with procedures of data quality control and maximum use of information, containing in every piece of data in an effort to obtain reliable hydraulic design criteria.

Chapter 4

DESIGN PHILOSOPHY

4.1 Problem Identification

It is not intention of this study to work out the design philosophy for coastal structures in general, but the one that is mainly specified for the situation of coastal engineering practice in Vietnam at present and in the coming years.

It should be emphasised that there always exist a chance of damage or failure to marine structures. First, this is because there are almost always a significant uncertainty related to the estimates of the design wave height and not small uncertainties in determination of other hydraulic design parameters and geotechnical conditions, as already mentioned in the previous chapter. Second, the lack of knowledge concerning the physical processes in wave structure interactions, the behaviour of different types of armour units or structure's elements under random wave attacks, etc. cause considerable uncertainties in various design formulas/criteria, and consequently, lead to the uncertainties in the structural design. Last but not least there are uncertainties related to the construction and maintenance aspect. The difficulties in construction or maintenance caused by usually unfavourable construction conditions (wind, waves, varying water levels, etc.) and under-water works are unavoidable for many types of coastal structures, especially for breakwaters. For under-water works, the degree of control and inspection available under water is very limited and the visibility frequently restricted to a matter of metres. As a consequence, large inaccuracies and uneven distribution of materials on underwater sections are common, a "not-as-designed" structure is inevitable.

Now back to the situation of coastal engineering in Vietnam, the problem with uncertainties inherent in designing marine structures in general and coastal structures in particular is, of course, much more serious, compared with that in countries like Holland, United States of America, Japan, etc.. This is due to :

* The scarcity in instrumental data, especially wave data and the lack of hindcasting models for generating wave data

* The difficulty in accessing modern achievements in coastal engineering

* The lack of experience in design and construction of coastal structures.

* The lack of soft-ware to support in the design process

* The fact that projects are often carried out under large political or commercial pressures and also the constraints in money leads to the situation that there is usually not enough time and money for complete and detail design programs as required to produce reliable designs.

4.2 Design Philosophy

The nature of the problems addressed and the situation in design, construction and maintenance practice of coastal structures in Vietnam, at present and in the coming years, suggest some general design principles to cope with uncertainties in designing coastal structures, and thereby increase their reliability.

■ ***Structure should be hydraulically as less sensitive as possible***

As long as there are large uncertainties in the hydraulic design parameters, this principle is essential. Hydraulic sensitivity here refers to the rate (degree) of response of a structure to the possible variations in the hydraulic design conditions. The whole idea of this design principle is to minimise the influence of uncertainties in the hydraulic design parameters on the structure's reliability. The hydraulically less sensitive structure means the less risk that the as-built strength of the structure will be exceeded by loads.

Hydraulic sensitivity can be analyzed and evaluated by :

* The rate of increase of the required crest height of a structure or required block

weight of armour units and

* The rate of damage (and mode of failure).

Obviously, the lower these rates, the less sensitive structures are.

For structures like sea dikes, the rate of increase of the required crest height is a very important parameter since wave overtopping is one of the critical design criteria. For rubble mound structures or rubble slope protections, where the stability of armour is essential, the rate of increase of the required block weight is of primary concern.

The rate of damage (and the mode of failure) is another important parameter to be considered. The damage evaluations, of course, depend very much on types of coastal structures. At least three types of structures can be identified due to significant differences in sensitivity to exceedance loads (Figure 4.1). The most ductile failure is that of rubble mound berm type structures, while the most brittle failure is associated with vertical wall type structures.

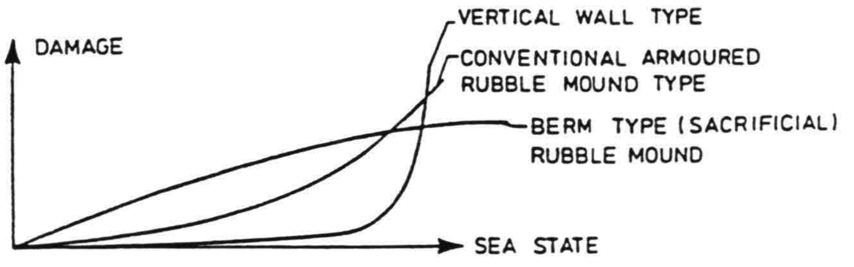


Figure 4.1 *Damage sensitivity of various types of structures, Rietveld & Burcharth (1987)*

For all structures, it is desirable to have a low rate of damage and ductile mode of failure; however this is not always possible. In any case, it is essential to design a structure so that the rate of damage and the mode of failure correspond to the quantity and quality of information constituting the boundary conditions and to the accepted risk.

■ ***Structure should be as simple as possible***

As long as reliable data on wave climate are not available and as long as the physical processes in wave structure interactions are not well understood, simplicity in design is essential. This means :

- ***simplicity in functional design,***

- ***simplicity in (overall) structural design and***

- ***simplicity in unit structural design.***

- Simplicity in functional design implies ***the separation of functions***, keeping the design function of a structure as close to its primary (basic) function as possible. This principle is crucial to restrict the risk of large damage.

  The primary function of sea defences, for instance, is flood prevention of low hinterland, of breakwaters is prevention of wave propagation, of groynes is dissipation of wave energy and currents, etc. However, from economical point of view, it often seems attractive to assign additional functions to the basic function of a structure. For example, a very large expenditure associated with the construction of a rubble mound breakwater can be justified by its use as a berthing facility or storage area, perhaps for special cargoes of a hazardous nature which have to be kept clear of general port operations. As a consequence, crown wall, access road, galleries for pipe-lines or conveyors, etc. may be necessary. And these appendages remove it from the initially simple concept of a rubble mound which, in principle, is easy to construct and not sensitive to the catastrophic damage if the design loads are exceeded.

- Simplicity in structural design is necessary to cope with the limitations in knowledge regarding hydraulic loading and structural response. This principle means ***there are no special structural element or feature*** which may complicate the physical processes in wave-structure interactions and the behaviour of which is still poorly understood. For instance, under wave attack the stability of rock armour on a straight slope is well described by Van der Meer formulas, while on a broken slope this is still a problem. Another example is the crown walls on rubble mound breakwaters. Besides crown walls are very harmful obstacles because they reflect the wave run-up, and therefore, increase the down-rush which endangers the stability of the armour units, they present their own uncertainties. The present state of knowledge in determining loading on crown walls is still vague. There is no general method of computing the wave forces on a crown wall for all configurations. There is also wide divergence between measured and calculated data. Generally, there are no reliable guidelines for design of crown walls.

- Simplicity in unit structural design means the ***simplification of the geometrical shape of the structural units or elements***. This aims at minimising the uncertainties related to hydraulic stability and mechanical strength of units due to the lack of good understanding in their hydrodynamic and structural behaviour under wave attack.

  Geometrically-complicated shaped armour units like dolos, stabit, tribar, etc. are very attractive due to their good hydraulic stability relative to their masses. However, the fact that their hydraulic stability to a large extent depends on the interlocking effect causes uncertainty in this high hydraulic stability due to the potential for structural breakage of units. For more massive and simple shaped units such as cube, antifer cube, rectangular blocks, and so on breakage of units is not a common phenomenon, while for geometrically-complicated shaped units this is a real problem. In randomly placed pattern of units, it is impossible to prevent rocking impact which may cause breakage of units if they are not designed strong enough or, it is highly possible, if they suffer impacts during placement or contain invisible cracks as a result of stress induced during casting and curing.

Thus, as long as the wave climate is not well predicted and as long as a proper design method, which takes into account also the mechanical properties of the construction units, simplicity in unit structural design is essential.

■ ***Structure should be as flexible as possible***

Since there are always uncertainties in the design boundary conditions, damage or deformations to structures are unavoidable. Under this circumstance, the flexibility of structures, i.e their ***capability to accommodate/neutralize small deformations and initial damage*** is of great advantage.

Rock structures like rubble mound breakwaters or rubble slope protections are good examples of flexibility. A certain displacement or settlement of rock units can be accepted with hardly any significant influence on the structure's stability due to self-healing effect of loose and randomly placed materials. On the other hand, structures of vertical wall type or armour slopes with blocks placed in pattern appear to be more sensitive to deformations and initial damage. For structures of vertical wall type, small sub-soil settlement or toe scour can serve as a beginning of a serious damage or even failure. For slopes of placed blocks, the damage or removing of one block can easily lead to a progressive damage.

■ ***Structure should be easy to construct and maintain***

The easiness for construction/maintenance of structures is necessary not only for reducing the construction cost, but also, and may be more importantly, ***for minimising the deviations of as-built structures from designs***, and therefore, increasing the reliability of structures. However, it should be mentioned again that the easiness here is mainly specified for the case of construction and maintenance practice of coastal structures in Vietnam, considering the low level of mechanisation, the lack of devices for controlling and inspecting the construction works, especially underwater works and the lack of experience. Under this situation, the following requirements for the designs of coastal structures are important:

- ***No high accuracy***

- ***No special construction method or equipment***

- ***No problem with maintenance.***

• Some types of structures are very sensitive to the construction tolerances; their strength greatly depends on how accurate they will be built or maintained. This holds for structures like placed block revetments, breakwaters with armour blocks placed in pattern (hollowed cubes, sea bees, etc.) or structures of vertical wall type. These structures are only appropriate for conditions where construction at low tide can be done in the dry, i.e accuracy and quality of the construction works can be easily controlled, as they demand an even under layer, accurate placement and good toe support.

Rubble mound structures with randomly placed armour units, in general, are less sensitive to the construction tolerances compared with the above mentioned ones. For these structures, lower accuracies can be accepted, and therefore, the deviations of as-built structure's strength from designed one should be smaller, of course provided under the same construction conditions and level.

Further, among rubble mound structures with randomly placed armour units, conventional type with relatively gentle slope, in turn, is generally less sensitive to the construction inaccuracies than wave wall type with steep slope and high interlocking armour units, whereas dynamically stable structures like berm breakwaters are an extreme example of this aspect. For dynamically stable structures, the initial profile, i.e as-built profile has little or no effect on the developed (final) profile, and as a result, large construction tolerances can be allowed without the fear of negative effect on the structure's stability.

For rubble mound structures or rubble slopes, it is advantageous to design more layers, as in this case, smooth transition between layers will be achieved. However, the designs with many layers cause difficulties in construction and increase the risk of damage due to storms to partially finished work.

Some structural elements, e.g concrete armour units with high interlocking degree, may be attractive in terms of rational use of concrete and having high hydraulic resistance, as already mentioned; however they may involve such construction difficulties that under the state of the art of the construction practice in Vietnam, it is either too difficult to guarantee their quality or economically not feasible.

For certain conditions, caisson type structures may appear to be more relevant than the other types, possibly because good quality rock is not available near the site while subsoil is good for foundation. Also, the fact that the ability to construct a major portion of the structure on dry land which enables a much greater degree of quality control and inspection to be achieved than is possible in underwater situation makes it even more relevant. However, the high accuracy and quality required, the necessary equipment and techniques and the lack of experience, in turn, make it less relevant for the situation in Vietnam at present and in some years to come.

- Maintenance or even rehabilitation of coastal structures is necessary, for there is always the chance of damage or failure due to the limitations of knowledge regarding wave climate and structural response, and due to the nature of the design procedures that, for most of coastal structures, are based on the optimization of construction and maintenance costs. How easily a structure can be maintained or rehabilitated and to what extent its original strength can be restored depend both on the type of the structure and on its structural design.

The influence of the type of structures on the maintenance aspect can be demonstrated by considering again rubble mound type and vertical wall type. Damage to rubble mound structures can be easily repaired, while if failure occurs it is seldom total failure and the remains of the structure can normally be built up again to perform its full function without undue difficulty. The reason for this is that under-designed structure of this type can be improved by flattening the slope on the seaward side, and in the course of failure, the wave action usually perform this function, so when the top part is built up again, the rehabilitated structure is even more stable than formerly. For structures of vertical wall type, in comparison, it is extremely difficult to repair, for instance, the inclination of the massive wall that may be caused by settlement of sub-soil, scour or whatever reason.

Thus, the chosen type of structure is desirably to be easy both for construction and for maintenance. Concerning the structural design, this should incorporate features to facilitate maintenance which include repair of local damage and renewal of degraded materials. Elements requiring periodic maintenance must be easily accessible for inspection and renewal. Furthermore, the design should makes allowance for changes to the structure due to its maintained service life.

▪ Besides the above design principles, two other aspects/requirements, though not quite related to the problem considered, should also be taken into account in the design process. They are :

- ***The maximum use of materials locally available and***
- ***Labour intensive constructions.***

In order to reduce the cost of construction, it is generally required that the structure's design should rely on locally available materials as much as possible. Also, considering the cheap workmanship in Vietnam at present and in the near future, designs with labour intensive construction methods should be given due attention.

Summary :

The above presented and elaborated design philosophies are mainly specified for design of coastal structures in Vietnam and proposed to cope with many shortcomings and limitations related to the design boundary conditions : natural boundary conditions, knowledge boundary conditions and construction and experience boundary conditions. In addition to those design philosophies, it should be mentioned, however, that the correct type of structure, whether it be rubble mound or caisson or blockwork or whatever, is very site-specific.

Chapter 5

SENSITIVITY ANALYSIS FOR SEA DIKES

5.1 General

As mentioned in chapter-4, the primary function of sea defences in general, and sea dikes in particular is the flood prevention of the (low) hinterland. Under storm conditions, these structures should withstand the combined action of storm surges, waves and strong winds. On the other hand, they should fulfil the assigned functional requirements, i.e protection of hinterland from adverse effects of high water and waves.

For sea dikes, since the high water protection is required, the structure's height in relation to the design storm surge level or to the maximum level of wave run-up during design storms is one of the most important structural parameters to be determined. This directly depends on the character of hinterland to be protected. In general, some amount of wave overtopping may be allowed under design conditions.

Regarding the structural design, since sea dikes are generally subjected to considerable wave attack and varying water levels, they have a relatively gentle slope on sea-side, usually with a berm at the storm surge level, and a heavy revetment/ slope protection. There are numerous types of revetments. They can be:

- a rubble slope protection, i.e randomly placed stones or concrete units,
- regular placed stones or concrete blocks and
- other types of revetments like gabions, asphalt, grass on a clay layer, geotextiles, etc.

The choice of a revetment depends on the local conditions: the load and geotechnical conditions, the availability of construction materials and construction equipment, the experience in design and construction, the environmental impact degree and local preference. Nevertheless, a revetment, whatever type it is, should be stable, flexible, durable as well as easy for construction and inspection of damage and repair. Under the circumstances when there are large uncertainties in the natural boundary conditions, especially in hydraulic design parameters as in Vietnam, structure's flexibility and ease for construction and maintenance are very advantageous and may be the determining factor in revetment selection.

5.2 Objective

This chapter aims at :

* investigating the sensitivity of the some typical used dike cross-sections to the uncertainties in the main hydraulic design parameters (significant wave height H_s and wave period T_p);

* investigating the relative influence of H_s and T_p on the structure's design;

* considering the application aspects in relation to the design boundary conditions in Vietnam.

5.3 Investigated alternatives

* Sea dikes with no outer berm
  Outer slope: m = 3
  m = 4
  m = 5

* Sea dikes with an outer berm
  Berm width: B = 4m
  B = 6m

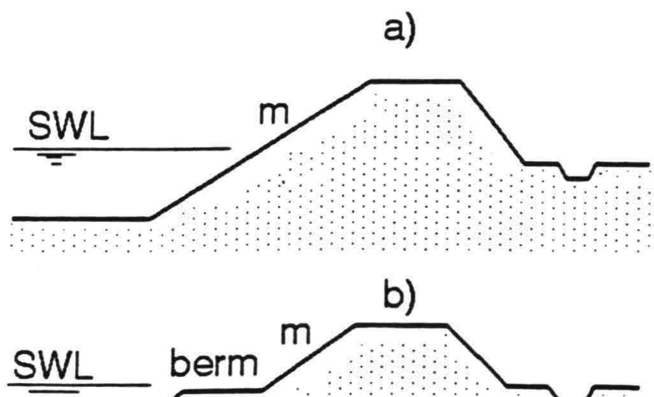


Figure 5.1 *a) Dike with no outer berm*
b) Dike with an outer berm

5.4 Sensitivity analysis

5.4.1 Method and formulas

METHOD

For analysis, the wave overtopping criterion is used. That is during the design storms, the discharge over the structure's crest should be less than some specified quantity, q litres/second per running meter of a dike. The allowable value of q primarily depends on the quality of the inner slope. Two typical cases may be identified:

a) q_{max} = 1 l/s/m for good inner slopes

b) q_{max} = 10 l/s/m for inner slopes with hard protections (stones, concrete blocks, asphalt, etc.)

In this case study, the criterion q_{max} = 1 l/s/m is used.

FORMULAS

The wave overtopping formulas derived by Van der Meer (1993) are adopted for investigation.

* ***For plunging waves***

$$Q = 0.06 \exp(-4.7\frac{R}{\gamma}) \tag{5.1}$$

With

$$Q = \frac{q}{\sqrt{gH_s^{\,3}}}\sqrt{\frac{H_s/L_o}{\tan\alpha}} \tag{5.2}$$

and

$$\mathbf{R} = \mathbf{R_c/(H_S\xi_p)} \tag{5.3}$$

** For surging (non-breaking) waves*

$$Q = 0.2 \exp\left(-2.3\frac{R}{\gamma}\right) \tag{5.4}$$

With
$$Q = q/(gH_s^3)^{-0.5}; \quad R = h_k/H_s \tag{5.5}$$

For transition between plunging and surging waves, $\xi = 2$ is used.
Where:

q = overtopping rate (m^3/s per metre width)
R_c = crest freeboard (m)
γ = $\gamma_b \gamma_f \gamma_\beta \gamma_h$, total reduction coefficient for the influence of berm, roughness, oblique wave attack and depth limited wave attack.
L_o = deep water wave length
ξ_p = $\tan\alpha/(S_p)^{-0.5}$, breaker parameter
α = slope angle
S_p = wave steepness for the peak period T_p

It has been found that run-up (or overtopping quantity) can be better described using the peak period, $T_{p,}$ instead of using the mean period, T_m; further the peak period is nearly equal to $T_{1/3}$.

It should also be noticed that in the above formulas, the coefficients 4.7 and 2.3 are 90% exceedance values used in deterministic methods. The real averages are 5.2 and 2.6, and the standard deviations are $\sigma=0.55$ and $\sigma=0.35$, respectively.

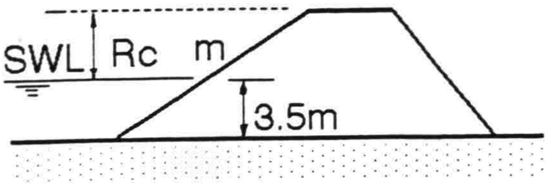
5.4.2 Computational results

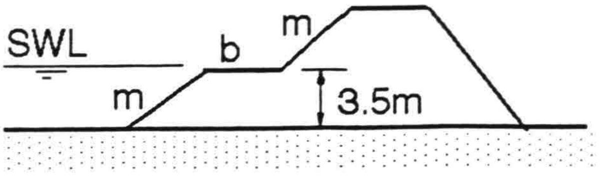
*** Hydraulic design conditions**

Assuming:
- Significant wave height: H_s = 1.5m
- Peak wave period: T_p = 5 sec
- Incident angle: β = 0°
- Depth of bottom at the structure's toe = 3.5m
- Depth of bottom in front, at a distance of half wave length from the toe: 6.0m

*** Structural parameters**

Figure 5.2





The computational results are presented in Table 5.1-5.6 and Figure 5.2-5.5.

Berm width b=	m=	Crest freeboard, R_c (T_m = constant)					
		H_s	$1.1H_s$	$1.2H_s$	$1.3H_s$	$1.4H_s$	$1.5H_s$
0m	3	3.76	3.98	4.17	4.33	4.48	4.61
	4	2.73	2.92	3.06	3.19	3.30	3.39
	5	2.17	2.30	2.41	2.51	2.59	2.67
4m	3	2.73	2.96	3.17	3.36	3.53	3.69
	4	2.18	2.36	2.51	2.64	2.77	2.88
	5	1.81	1.95	2.07	2.18	2.27	2.36

Table 5.1

Berm width b=	m=	Crest freeboard, R_c (H_s = constant)					
		T_p	$1.1T_p$	$1.2T_p$	1.3_p	$1.4T_p$	$1.5T_p$
0m	3	3.76	4.19	4.61	4.61	4.61	4.61
	4	2.76	3.08	3.40	3.73	4.06	4.39
	5	2.17	2.42	2.68	2.94	3.20	3.45
4m	3	2.73	3.04	3.34	3.34	3.34	3.34
	4	2.18	2.43	2.69	2.94	3.20	3.47
	5	1.81	2.02	2.23	2.45	2.66	2.88

Table 5.2

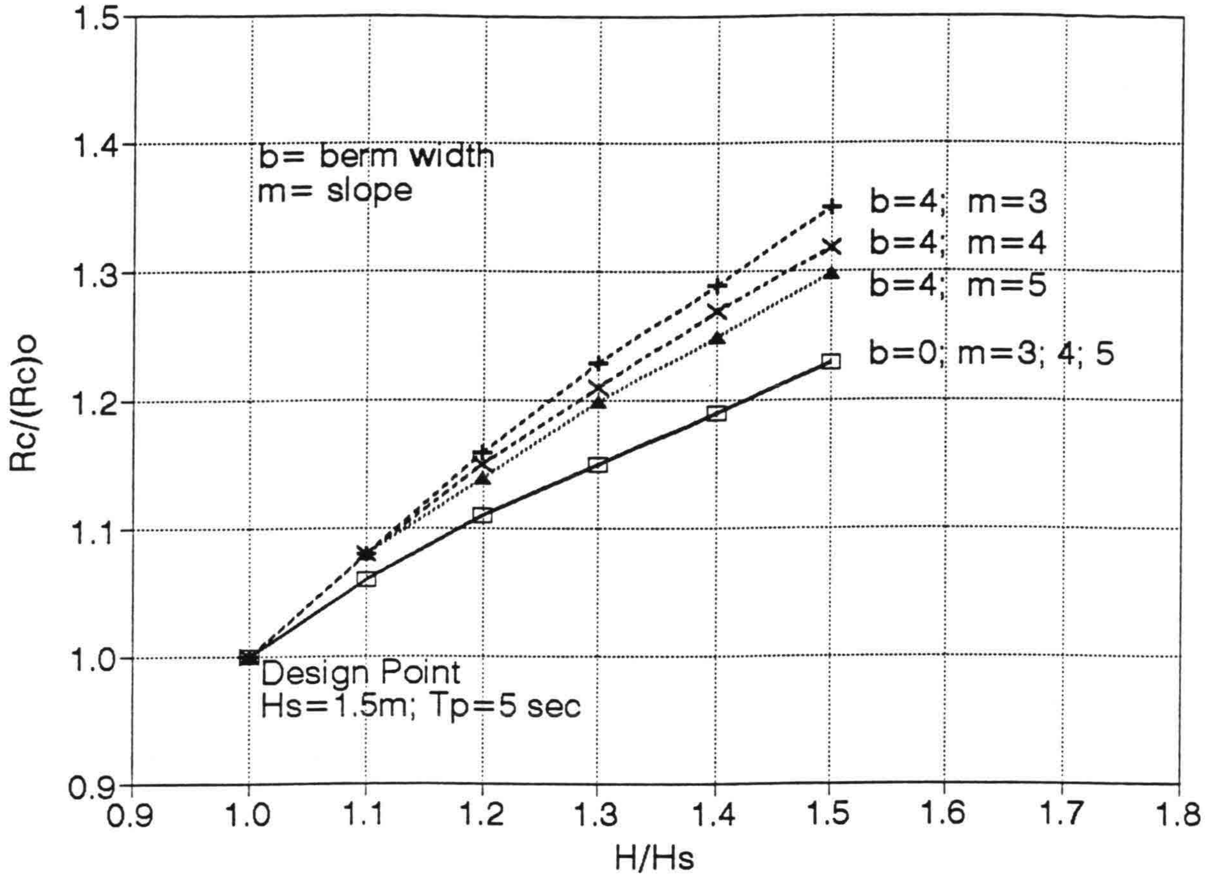


Figure 5.3 *Relative crest height, $R_c/(R_c)_o$, versus relative wave height, H/H_s.*

H/H_s	Relative crest height, $R_c/(R_c)_o$					
	b = 0m			b = 4m		
	m = 3	m = 4	m = 5	m = 3	m = 4	m = 5
1.0	1.0	1.0	1.0	1.0	1.0	1.0
1.1	1.06	1.06	1.06	1.08	1.08	1.08
1.2	1.11	1.11	1.11	1.16	1.15	1.14
1.3	1.15	1.15	1.15	1.23	1.21	1.20
1.4	1.19	1.19	1.09	1.29	1.27	1.25
1.5	1.23	1.23	1.23	1.35	1.32	1.30

Table 5.3

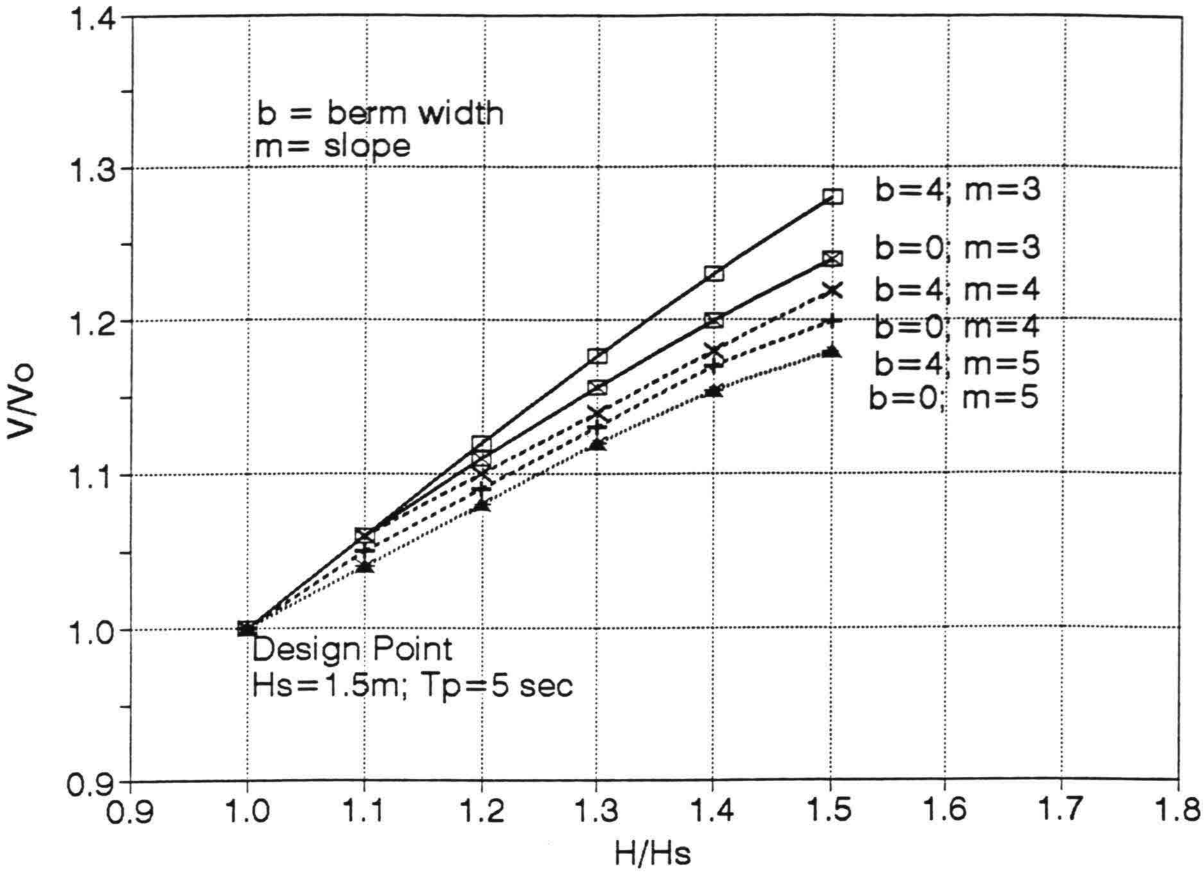


Figure 5.4 *Relative volume of embankment, V/V_o, versus relative wave height, H/H_s*

H/H_s	Relative volume of embankment, V/V_o					
	b = 0m			b = 4m		
	m = 3	m = 4	m = 5	m = 3	m = 4	m = 5
1	1	1	1	1	1	1
1.1	1.06	1.05	1.04	1.06	1.06	1.04
1.2	1.11	1.09	1.08	1.12	1.10	1.08
1.3	1.15	1.13	1.12	1.18	1.14	1.12
1.4	1.20	1.17	1.15	1.23	1.18	1.15
1.5	1.24	1.20	1.18	1.28	1.22	1.18

Table 5.4

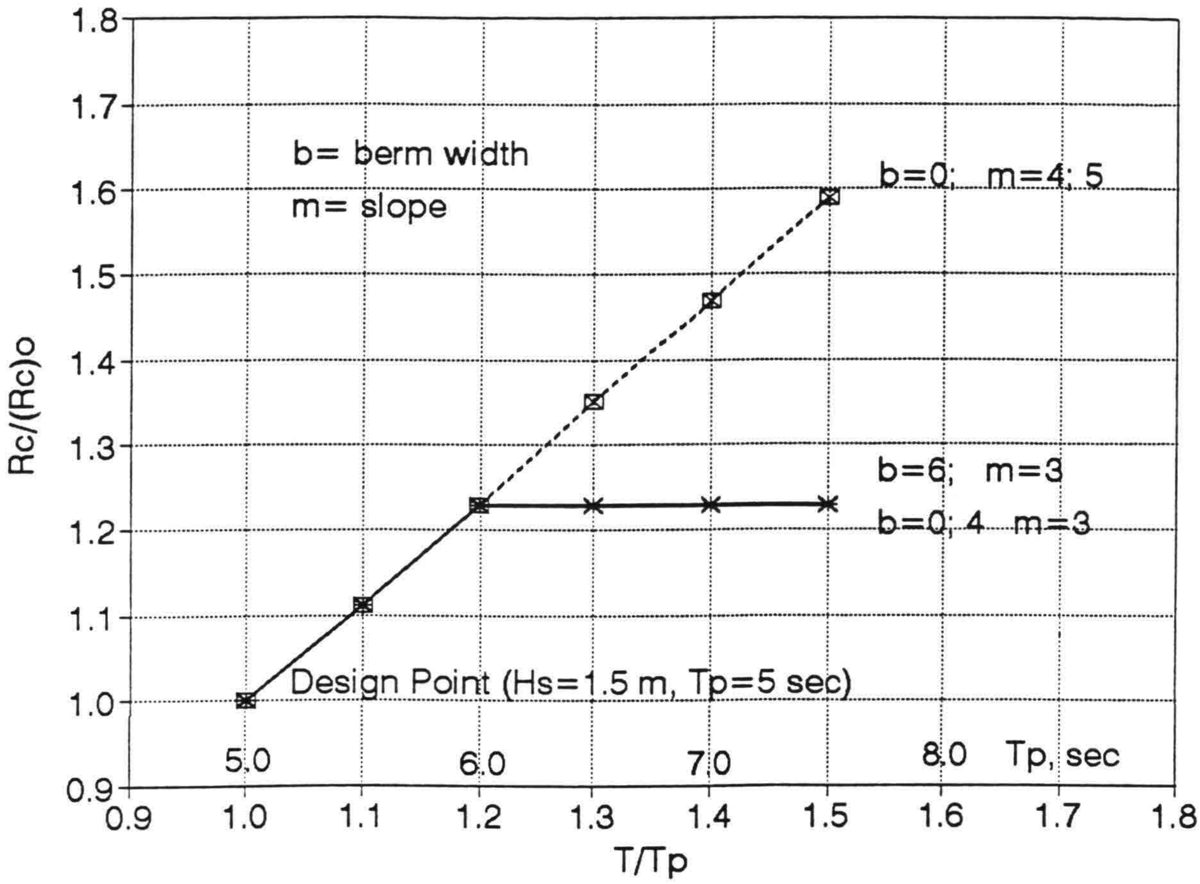


Figure 5.5 *Relative crest height, $R_c/(R_c)_o$, versus relative wave period, T/T_p*

T/T_p	Relative crest freeboard, $R_c/(R_c)_o$					
	b = 0m			b = 4m		
	m = 3	m = 4	m = 5	m = 3	m = 4	m = 5
1	1	1	1	1	1	1
1.1	1.10	1.11	1.11	1.11	1.11	1.11
1.2	1.23	1.23	1.23	1.23	1.23	1.23
1.3	1.23	1.35	1.35	1.23	1.35	1.35
1.4	1.23	1.47	1.47	1.23	1.47	1.47
1.5	1.23	1.59	1.59	1.23	1.59	1.59

Table 5.5

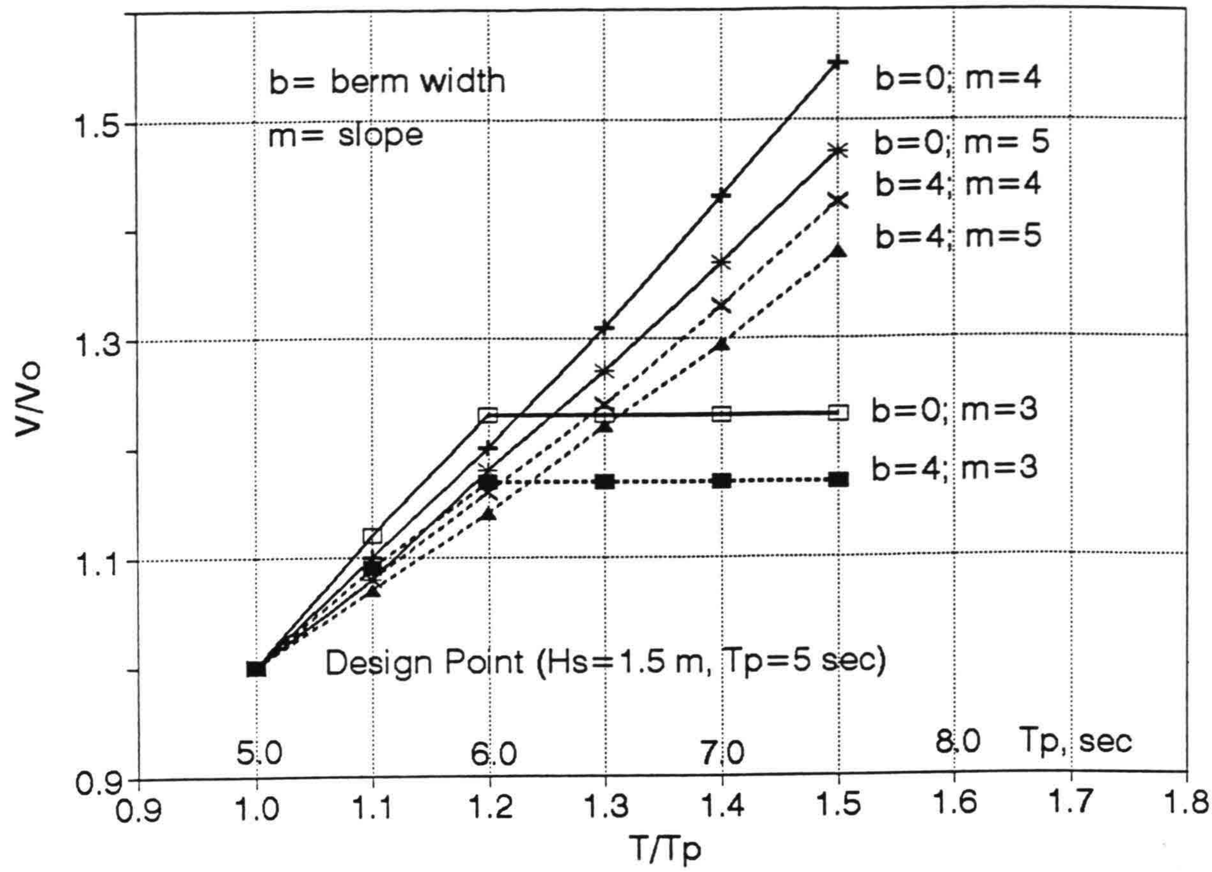


Figure 5.4 *Relative volume of embankment, V/V_o, versus relative wave period, T/T_p*

T/T_p	Relative volume of embankment, V/V_o					
	b = 0m			b = 4m		
	m = 3	m = 4	m = 5	m = 3	m = 4	m = 5
1	1	1	1	1	1	1
1.1	1.12	1.10	1.08	1.09	1.08	1.07
1.2	1.23	1.20	1.18	1.17	1.16	1.14
1.3	1.23	1.31	1.27	1.17	1.24	1.22
1.4	1.23	1.43	1.37	1.17	1.33	1.29
1.5	1.23	1.55	1.47	1.17	1.42	1.38

Table 5.6

5.4.3 Sensitivity

- **Sensitivity to variations in wave height ,H_s**

* As can be seen from Figure 5.1, the dikes without an outer berm are equally sensitive, in terms of the relative required crest height $R_c/(R_c)_o$, but less sensitive than the dikes with a berm . Further, among the latter (dikes with a berm) the steeper the slope, the more sensitive they are.

* In term of the relative required volume of embankment, V/V_o, the flatter the slope, the less sensitive the dikes are. This holds both for the dikes with a berm and for the dikes without a berm, but the dikes with a berm are more sensitive than those without a berm.

- **Sensitivity to variations in wave period, T_p**

From Figures 5.3 and 5.4 and the computational results, the following points can be drawn:

* For the considered case (H_s=1.5m, T_p=5 s) and within the specified range of the wave period (T_p=5-7.5 s), the dikes with slope m=4 and m=5 are around 15% more sensitive to wave period than to wave height. Additionally, in terms of $R_c/(R_c)_o$, they are equally sensitive to wave period, while in terms of V/V_o the dike with flatter slope (m=5) is less sensitive than the dike with a steeper slope (m=4).

* The dike with the slope m=3 behaves quite different, however. Within the range T_m=5-6 sec, its behaviour is in accordance with the dikes with the slope m=4; 5. But from the point T_m=6 sec and further, it becomes insensitive. This can be explained by the wave run-up formulas (Van der Meer, 1993):

$$R_{2\%} / H_s = 1.6\xi_p \qquad \text{for } \xi_p \leq 2$$
$$R_{2\%} / H_s = 3.2 \qquad \text{for } 2 \leq \xi_p < 4$$

Where :

$R_{2\%}$ = Run-up on smooth plane slopes, defined as the vertical height above still water level which is exceeded by 2% of the waves in a wave field.

At the point T_p=6 sec (H_s=1.5, cotα=3), the breaker parameter has the critical value

$\xi_p=2$ separating plunging ($\xi_p \leq 2$) and surging ($\xi_p > 2$) waves. After this point and within the range $2 < \xi_p < 4$, run-up is not dependent on wave period any more, as described by the expression:

$$R_{2\%} / H_s = 3.2.$$

For slopes m=4 and m=5, this situation will occur when the peak period T_p is more than 8 sec and 10 sec, respectively.

5.5 Considerations

From the computational results and analysis the following considerations can be made :

a. It appears that under the situation when there are large uncertainties in the design wave height and wave period, the gentle slope dikes without a berm are more relevant than those with a steep slope and a berm. This is because the gentle slope dikes with no berm appears to be less sensitive to variations in wave height and wave period than the dikes with a berm and a steep slope.

b. A flatter slope reduces the wave force on the revetment and therefore leads to the less wave run-up. Subsequently, the crest height can be lowered resulting in the less volume of the embankment. However, this does not necessarily imply that lower earth volume coincides with lower cost. An expensive part of the embankment, which comprises the revetment of the outer slope and slope surface, increases as the slope angle decreases.

   On the other hand, it should be noticed, however, that a quite gentle slope can also be risky. The reason is that so far we have considered only the sensitivity of dikes to the uncertainties in the design wave height and wave period. The uncertainty in the design water level hasn't been considered yet. There are large uncertainties related to the estimates of the design water levels as well (chapter 3). Though the dikes with a gentle slope is less sensitive to variations in wave height and wave period compared to those with less gentle slopes, they may be more risky than the latter in respect of overflowing failure mechanism. Indeed, the dikes with a gentle slope have lower crest heights, and therefore, have more chance to be overflowed by high water than those with a less gentle slope (i,e higher crest elevations). This is particularly the case when the uncertainties in the design water levels are large, while the design wave heights are small.

Additionally, the dikes with a less gentle slope still have another positive point in respect of sensitivity to variations in wave period . That is the steeper the slope, the narrower the range of wave periods (higher than the design value) that structures are still sensitive to. For the case study, these range are:

* T_p = 5 - 6 sec for the dike with slope m = 3
* T_p = 5 - 8 sec for the dike with slope m = 4
* T_p = 5 - 10 sec for the dike with slope m = 5

And this point is important since the wave run-up or overtopping quantity is more sensitive to wave period than wave height

c. The main function of a sea side berm is to reduce the wave run-up and thereby reduce the structure's elevation. The lower structure's elevation means the lower pressure force on the foundation, and this is essential for areas with weak/soft subsoil. But a berm is also important for maintaining the revetment (when the slope length is in order of 20m or more). In addition, by providing a berm, it could be possible to reduce the volume of expensive revetment by growing a good grass cover on a high and gentle berm.

Nevertheless, the application of a berm for obtaining the effect of the wave run-up reduction in Vietnam is not advised. The arguments for this is that:

* the sea dikes with a berm are more sensitive to the uncertainties in the design wave height and wave period, as stated in point (a).

* research has indicated that in order to obtain the maximum run-up reduction effect, the berm position should be approximately at the still water level (of the design storm surge). Also, the reduction effect will nearly disappear when the berm position is more than $\sqrt{2}H_s$ above or below the still water level (Van der Meer, 1993). This makes the application of a berm for the run-up reduction risky, as there is usually large uncertainty in determining the design water level as well.

d. The used design criterion q_{max}= 1 l/s/m is generally applied for the inner slopes with a good grass cover on clay. The value of 1 l/m/s is actually the average over the considered dike section. In reality, under the design storm conditions, the overtopping discharges along a dike section are different from place to place (and from time to time) and the maximum values can be significantly more than 1 l/m/s. Nevertheless, if the grass cover is of good quality, this situation causes no problem. It is therefore essential to apply appropriate measures for maintaining a good grass cover on the inner slope, particularly during the storm seasons.

Summary:

* The application of the dikes with a berm for wave run-up reduction is generally not preferable because this type of dikes is more sensitive to the uncertainties in the design wave height, wave period and water level than those without a berm.

* Generally, dikes with a gentle sea-side slope appear to be somewhat more relevant than those with a steep (less gentle) slope. In respect of the overtopping failure mechanism, the dikes with a gentle slope are preferable, while regarding the overflowing failure mechanism, the dikes with a steep slope are advantageous. The choice of either a gentle or a steep slope should therefore be made on the basis of given conditions. If the uncertainty in the design water level is large while, the design waves are small, a steep slope may be more relevant. In the opposite case, a gentle slope may be a good choice.

Chapter 6

SENSITIVITY ANALYSIS FOR BREAKWATERS

6.1 Objective

The primary objective of carrying out the sensitivity analysis for breakwaters is to determine types of structures and armour units that are most suitable for applying in Vietnam, where hydraulic design conditions are often uncertain. In addition, sensitivity of different design parameters is also being investigated, where is necessary, in order to figure out the relative influence of these parameters on the stability of structures.

For achieving this objective, sensitivity of different types of rubble mound breakwaters, i.e the degree or rate of response of a structure to the possible variations in the main hydraulic design conditions (H_s, T_m and storm duration, N) is being investigated.

The sensitivity analysis is only confined with rubble mound breakwaters because, besides the fact that availability of various design formulas and computational models enables the analysis to be done, they are likely suitable to be applied in Vietnam due to many features/ advantages, as already considered in Chapter-4. Concerning vertical wall type breakwaters, they are generally not relevant for the situation in Vietnam. This is due to many reasons, but the most important one is the fact that they have very brittle mode of failure, and therefore require accurate estimates of design waves.

6.2 The investigated alternatives

* Rubble mound breakwater (with and without a crown wall)
  - rock structure
  - armoured with cubes
  - armoured with tetrapods
  - armoured with accropode
* Berm breakwater

Among these structures, rubble mound breakwaters belong to the group of statically stable structures. They should be designed in such a way that no or minor damage is allowed under design conditions. Damage here is generally defined as displacement of armour units . In general, design of a breakwater is based on an optimum solution between design conditions, allowable damage and costs for construction and maintenance during the life-time of the structure. The distinct features of these types of breakwaters are the presence of large concrete structures (crown walls) or heavy armour units.

Berm breakwaters can be regarded as unconventional rubble mound structures which more or less belong to the group of dynamically stable structures. For this type of breakwaters, the initial profile characterised by a very steep seaward slope and a horizontal berm) is allowed to reshape under wave action; during the first storms armour stones are displaced by waves forces to form an S-shaped profile and afterward the structure will be more or less statically stable.

Compared with conventional rouble mound structures, berm breakwaters can be built with considerably smaller materials (stones). They are characterised by a very gentle part of sea-side slope around still water level, whereas above and below it, the slope is much steeper. This part of gentle slope reduces appreciably the wave forces on the armour stones.

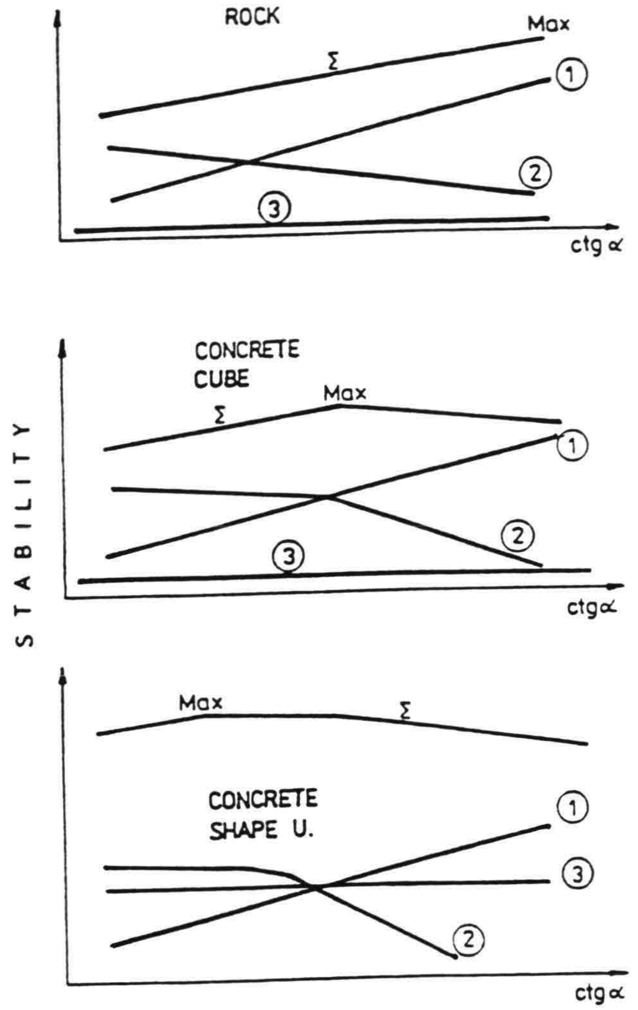
6.3 Rubble mound breakwater

6.3.1 Stability and stability formulas

STABILITY:

Armour layers of statically stable structures in general and rubble mound breakwaters in particular consist of loose materials, either large rock or concrete units which are subjected to large hydraulic forces during design conditions. Under severe storm conditions, armour units can stay in place or not, this primarily depends on their weight (gravity forces), skin friction and geometrical shape. Weight or gravity forces directly or indirectly (via "squeezing forces" from neighbouring units caused by gravity and friction forces) help to stabilize units. Also the shape of armour units can significantly increase the stability of units by interlocking effect. Gravity-related "squeezing forces" are highly slope-dependent, while interlocking is a geometrical property of armour units that does not depend upon slope.

The contributions of weight, friction and interlocking of units to hydraulic stability of rock, cube and shaped concrete units as function of slope cotα are shown in Figure 6.1 given by Kozakiewiecz (Hydraulic Stability of Rubble Mound, 1977). Also, the so-called optimum stability slope for different types of armour units is given by Losada (see Table 6.1).



Notes:

1 = effect of weight
2 = effect of friction between units
3 = effect of interlocking

Figure 6.1 *Hydraulic stability of armour units, Kozakiewiecz (1987)*

Type of armour units	Cot$\alpha_{(optimum)}$
- Quarry stones	5
- Parallelopipedic blocks	4 - 6
- Stabits	3 - 6
- Tetrapod	2 - 2.5
- Dolos	1.75 -2

Table 6.1 *Optimum stability slope*

STABILITY FORMULAS

Under wave attack, stability of a rubble mound breakwater as a whole depends on the stability of all its components but primarily depend on the stability of armour units which is described by so-called stability formulas. These formulas should give stable mass of armour units.

There exist sofar many stability formulas for armour units, but most formulas in contemporary use have been based on the semi-empirical approach of Irribaren. The weight of a stable armour unit reads :

$$W = \frac{C_D^3 \rho_a H^3}{8\Delta^3(\mu\cos\alpha - \sin\alpha)^3} \tag{6.1}$$

in which :

H - Wave height
C_D - Drag coefficient
μ - Friction factor, block versus mounds
Δ - Relative mass density. $\Delta = (\rho_a/\rho_w - 1)$ where ρ_a and ρ_w are the mass densities of armour units and water respectively.
α - Slope angle of armour layers.

Subsequent developments include primarily the well-known Hudson formula :

$$M = \frac{\rho_a H^3}{k_D \Delta^3 \cot\alpha} \tag{6.2}$$

or rearranged form in terms of stability number N_s and nominal diameter of armour units D_n

$$N_s = \frac{H}{\Delta D_n} = (k_D \cot\alpha)^{\frac{1}{3}} \tag{6.3}$$

The distinction between the two formulas, Iribaren's and Hudson's, consists of safety factor k_D in the Hudson formula. However in both approaches the incident wave is regular, at right angle, non-breaking and non-overtopping.

Although there exist many shortcomings in the Hudson formula, as found by different researchers, it has been widely used for more than three decades to aid in the design of rubble mound breakwaters, mainly due to its simplicity.

More recent modification improvements accounting for wave steepness proposed by Walton & Weggel (1981), Carver & Davidson (1982), accounting for wave period done by Pilarczyk (1985) and accounting for wave length done by Chen, Kao and Tang (1986).

Van der Meer (1988), based on extensive model tests, has essentially improved the stability formulas by considering and including the effect of all possible factors on the stability of armour units. Aside from taking into account the influence of stability governing parameters such as wave height, wave period, storm duration structures permeability, etc.

Van der Meer made a distinction between plunging and surging waves. And this is very important because the mechanism causing instability is different between plunging and surging waves. In the plunging wave region, the fast run-up after breaking of waves is decisive for stability, while in the surging waves region, wave run-down is decisive factor.

Therefore, in this case study, Van der Meer formulas are adopted for sensitivity analysis.

Van der Meer formulas for rock

* Plunging waves:

$$\frac{H_s}{\Delta D_n} = 6.2P^{0.18}\xi_m^{-0.5}\left(\frac{S}{\sqrt{N}}\right)^{0.2} \tag{6.4}$$

* Surging waves:

$$\frac{H_s}{\Delta D_n} = 1.0P^{0.13}\xi_m^{P}\sqrt{\cot\alpha}\left(\frac{S}{\sqrt{N}}\right)^{0.2} \tag{6.5}$$

The transition from plunging to surging waves is defined by critical value of ξ_{mc}:

$$\xi_{mc} = [6.2P^{0.31}\sqrt{\tan\alpha}]^{1/(p+0.5)} \tag{6.6}$$

For $\cot\alpha \geq 0.4$, formula 6.4 is used.

It should be noticed, however, that there are still some limitations in these formulas in terms of the application scope. They are valid only for structures with straight slopes and high crests. It is not yet established that they can safely be applied to real breakwater cross-sections with broken slopes. For low crested breakwaters, a factor f_i was derived by Van der Meer to apply to $H_s/\Delta D_n$ calculated using above formulas to account for the reduction in relative stone size needed as the crest level reduces.

$$f_i = \frac{1}{[1.25 - 4.8\frac{R_c}{H_s}(\frac{S_m}{2\pi})^{0.5}]} \tag{6.7}$$

This is valid for $0 < R_c/H_s < 1$ where R_c is crest free board.

Van der Meer formulas for concrete units

* For cubes :

$$\frac{H_s}{\Delta D_n} = [6.7\frac{N_{od}^{0.4}}{N^{0.3}} + 1]S_m^{-0.1} \tag{6.8}$$

* For tetrapods :

$$\frac{H_s}{\Delta D_n} = [3.75\frac{N_{od}^{0.5}}{N^{0.25}} + 0.85]S_m^{-0.2} \tag{6.9}$$

* For accropode :

For no-damage criterion $N_{od}=0$

$$\frac{H_s}{\Delta D_n} = 3.7 \tag{6.10a}$$

For failure criterion $N_{od}=1.5$

$$\frac{H_s}{\Delta D_n} = 4.1 \tag{6.10b}$$

The formulas for concrete armour units are only valid for structures with slope $\cot\alpha=1.5$ for cubes and tetrapods, $\cot\alpha=1.33$ for accropode and permeability factor $P=0.4$ for all considered units.

Notes :

H_s = Significant wave height

K_D = Safety coefficient taking into account the influence of all unknown factors (e.g the way wave height is measured, wave period, block shape, etc.)

N = Storm duration or number of waves

ξ_m = Surf similarity parameter in terms of mean period T_m

S = Damage number for rock

N_{od} = Damage number for concrete units

D_n = Nominal diameter of armour units

6.3.2 Damage and damage number

Under severe storm wave conditions, individual stones or concrete units in armour layers can start moving, can be displaced or even can be broken due to hard impact/rocking. The amount of displaced or moving units is called damage.

In the design process of a statistically stable rubble mound breakwater, the required mass of armour units is determined, based on an economically optimum solution, where construction costs are compared with maintenance costs. Consequently, it is essential to know the expected damage within the structure's life time.

The damage level of a rock armour layer can be described by the dimensionless damage number, S :

$$S = A/(D_{n50})^2$$

A = erosion area around still water level

Damage number, S, indicates the number of cubic stones with the side of D_{n50}. The actual number of stones eroded within this D_{n50} wide stripe of a structure can be more or less than damage number, S.

For armour layer with concrete units, damage can be defined as the relative damage, N_o, which is the actual number of displaced, rocking or moving units within a width of one nominal diameter D_n :

N_{od} = Number of displaced units
N_{or} = Number of rocking units
N_{omov} = Number of moving units, $N_{omov} = N_{od} + N_{or}$

The lower and upper damage levels for rock armour layers and for armour layers of concrete units are given in Table 6.2 and Table 6.3.

Slope cotα	Start of damage	Failure (filter layer visible)
1.5	S = 2	S = 8
2.0	2	8
3.0	2	12
4.0	3	17
5.0	3	17

Table 6.2 *Damage criteria for rock slopes, Van der Meer (1988)*

Concrete units	Slope cotα	Start of damage	Failure
Cubes	1.5	$N_{od} = 0$	$N_{od} = 2$
Tetrapods	1.5	0	= 1.5
Accropode	1.33	0	> 0.5

Table 6.3 *Damage criteria for slopes of concrete units, Van der Meer (1994)*

6.3.3 Stability and damage comparison

Stability of rock, cubes, tetrapods and accropode can be compared by plotting curves $H_s/\Delta D_n$ (dimensionless stability number) versus wave steepness S_m in one combined graph for all units. These curves have been computed using equations (6.4 - 7.0) for specified slopes and storm duration and for both upper and lower damage levels, i.e start of damage and failure (Table 6.4 and Fig.6.2).

Wave steepness	Stability number $N_s = H_s/\Delta D_n$							
	Rock		Cubes		Tetrapods		Accropode	
S_m	S= 2	S= 8	N_{od}=0	N_{od}=2	N_{od}=0	N_o =1.5	N_{od} = 0	N_{od}> 0.5
0.01	1.51	2.01	1.58	2.85	2.14	3.69	3.7	4.1
0.02	1.32	1.74	1.48	2.66	1.86	3.21	3.7	4.1
0.03	1.38	1.82	1.42	2.56	1.71	2.96	3.7	4.1
0.04	1.48	1.96	1.38	2.48	1.62	2.80	3.7	4.1
0.05	1.57	2.07	1.35	2.43	1.55	2.68	3.7	4.1
0.06	1.64	2.17	1.32	2.38	1.49	2.58	3.7	4.1

Table 6.4 *Stability numbers vs wave steepness (N=3000, cotα=1.5 for rock, cubes and tetrapods, cotα=1.33 for accropode)*

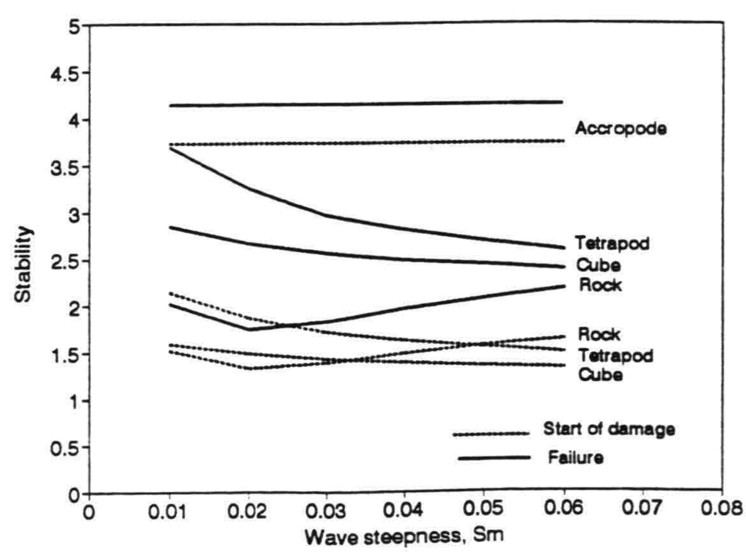


Figure 6.2 *Comparison of stability of rock, cubes, tetrapods and accropode.*

From Figure 6.2, the following comparisons can be made :

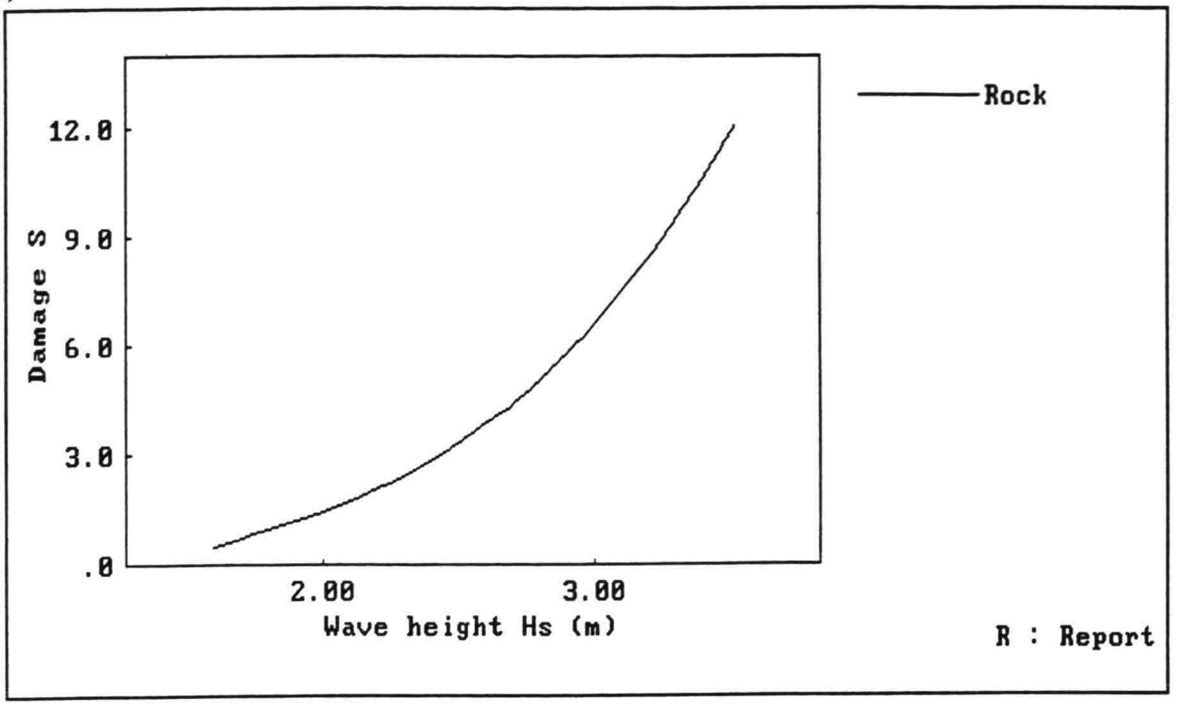
* The initial stability of rock, cubes and tetrapods are close to each other; however, the initial stability of tetrapods is slightly higher than that of rock and cubes
* The initial stability of accropode is much higher than that of rock, cubes, and tetrapods.
* The start of damage for rock, cubes and tetrapods are close although the initial damage for tetrapods is at slightly higher stability number $H_s/\Delta D_n$.
* Failure (severe damage) is reached first for rock, then cubes, tetrapods and finally accropde.
* Wave steepness or wave period shows no influence on the stability of accoropode, slight influence on cubes and fairly strong influence on rock and tetrapods.
* The margin between initial damage and failure for cubes and tetrapods are almost the same, and fairly large; for rock this is quite small, while for accropode it is even smaller than the margin for rock.

Based on the above comparisons, it can be expected that slope armoured with cubes or tetrapods will damage with more or less the same speed; slope with rock will damage considerably faster than slope with cubes or tetrapods, while slope with accropode will fail much faster. The start of damage and failure of accropode are very close and at high $H_s/\Delta D_n$ number (highest) means that up to a certain high wave height, accropode are completely stable, but after the initiation of damage at this high wave height the slope will fail dramatically.

Similar results can be obtained by analyzing and comparing "wave height-damage" curves computed and drawn for all considered armour units (see figure 6.3). The input data are as follows:

-	Mass of unit:	$M = 3000$ Kg
-	Mass density of unit:	$\rho_r = 2600$ Kg/m^3
		$\rho_c = 2400$ Kg/m^3
-	Mass density of water:	$\rho_w = 1030$ Kg/M^3
-	Wave steepness:	$S_m = 0.04$
-	Number of waves:	$N = 3000$
-	Permeability factor:	$P = 0.4$

a)



b)

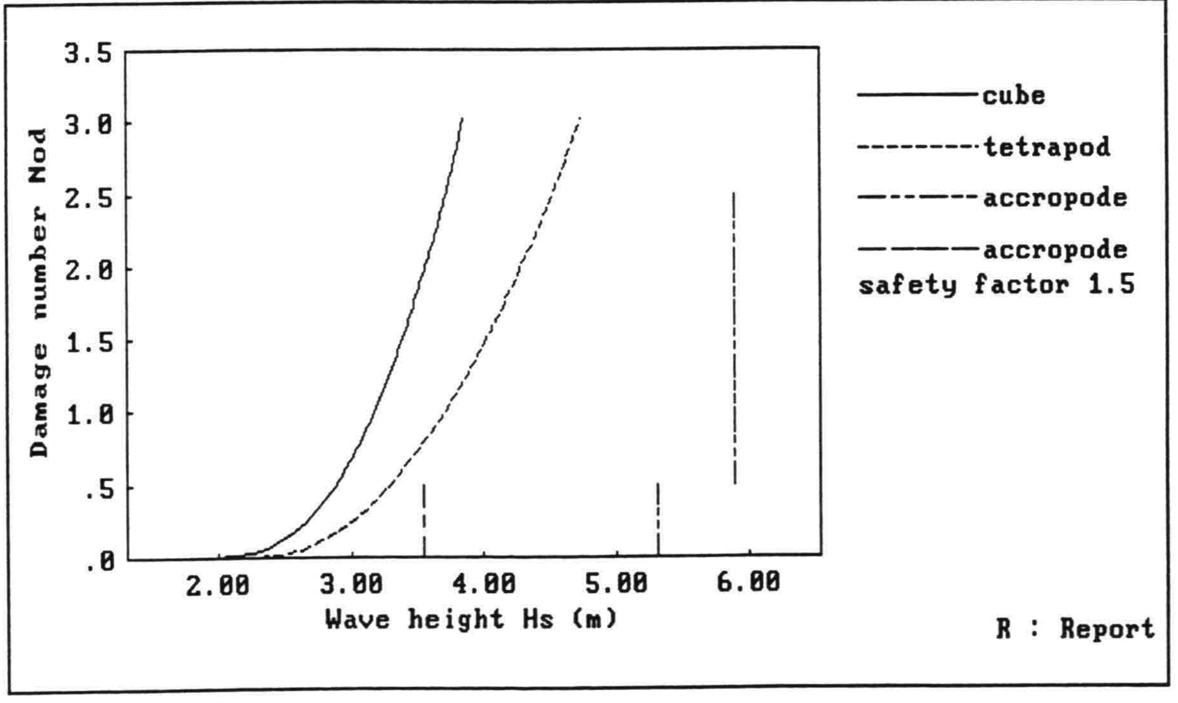


Figure 6.3a,b : *Wave height - damage curves for rock, cubes, tetrapods and accropode. W=3000 Kg, N=3000, P=0.4*

For concrete units, since damage to their slope is defined by the same damage number N_{od}, comparison can be made directly. For rock, in order to be able to compare with concrete units, the damage, S, to a rock slope should be first converted into N_{od} by using approximation $S \approx 2N_{od}$.

The rate of damage dS/dH_S or dN_{od}/dH_s for the slope of each type of unit can be compared visually by the steepness of the curves (the steeper the curve, the higher the damage rate) or by the average rate of damage given in the last column of Table 6.5.

Armour unit	Wave height, H_s (m)			Damage, N_{od}		Average rate of damage $\Delta N_{od}/\Delta H_s$
	Start of damage	Fail-ure	ΔH_s	Start of damage	Failure	
Rock	2.15	3.15	1.0	1 (S=2)	4 (S=8)	3/1 $\Delta S/\Delta H_s$=6/1
Cubes	2.0	3.5	1.5	0	2	2/1.5
Tetrapod	2.3	4.0	1.7	0	1.5	1.5/1.7
Accropode	5.3	5.9	0.6	0	>0.5	>0.5/0.6

Table 6.5 *Wave height versus damage criteria*

From Figure 6.3 and Table 6.5, the following conclusions can be drawn :

* Significant wave heights, H_s, at the start of damage to rock, cube and tetrapod slopes are low and close to each other (2.15m, 2.0m and 2.3m respectively), while to accropode it is much higher (5.3m).

* The differences between H_s at the initial damage and failure to cube and tetrapod slopes are almost the same (1.5m and 1.7m respectively); for rock slope the difference is considerable smaller (1m) and for accropode slope this is much smaller (0.6m) than those for cube and tetrapod slopes.

  Compared with the damage rate of tetrapod slope, the damage rate of cube slope is somewhat higher; the damage rate of rock slope is considerably higher, while that of accropode slope is much higher.

For rock slope, the damage rate is judged by comparing its ΔH_s-value and the average converted rate of damage, $\Delta N_{od}/\Delta H_s$, with appropriate factors for cubes and tetrapod. For accropode slope, since the criterion for comparison $\Delta N_{od}/\Delta H_s > 0.5/0.6$ is unclear, the rate of damage is judged on the basis of ΔH_s-value and the steepness of the "wave height- damage" curve. This curve is vertical, i.e. slope of accoropode fails abruptly, though at very high wave height.

It should be noticed that the above comparisons and conclusions are made for considered armour units on the basis of specified storm duration and slopes ($N=3000$, $\cot\alpha=1.5$ for rock, cube, tetrapod and $\cot\alpha=1.33$ for accropode). In reality both storm duration and building slope vary in large ranges (say $N=1000$ - 10000, $\cot\alpha=1$ - 5) and this can affect the above comparisons and conclusions.

For storm duration, hopefully, it does not or hardly influence the hydraulic stability of armour units, but the damage level (defined as S or N_{od}) of slopes. This influence of storm duration on damage to slopes is taken into account as $S^{0.2}/N^{0.1}$, $N_{od}^{0.4}/N^{0.3}$, and $N_{od}^{0.5}/N^{0.25}$ in equations (6.4, 6.5, 6.8, 6.9) for rock, cube and tetrapod slopes respectively. In other words, storm duration does not influence the hydraulic damage rate, but the degree of damage and therefore does not affect, fortunately, the validity of the stability and damage rate comparisons made above for rock, cube, tetrapod and accropode slopes.

Slope angle, however, has a distinct influence on the stability of armour units, as analyzed in section 6.3.1 (Stability and stability formulas) and shown in figure 6.1. This influence is described by $(\cot\alpha)^{1/3}$ in the re-arranged Hudson's formula and $(\cot\alpha)^{0.5}$ and $(\cot\alpha)^{(0.5-P)}$ in the Van der Meer formulas for plunging and surging waves, respectively. From Figure 6.1 and Table 6.1 it can be seen qualitatively that maximum stability is reached for rock on a very gentle slope ($\cot\alpha=$ 4.5 - 5.5) and for cubes on less gentle slope ($\cot\alpha=$ 3 - 4), while for tetrapod and accropode on steep slope ($\cot\alpha=$ 1.5 - 2.5). That is for the present case where $\cot\alpha=$ 1.5 & 1.33, tetrapod and accropode are nearly on their best position (slope), while rock and cubes are not.

Now consider the situation when rock, cubes, tetrapod and accropode are built on the optimum slope for cubes, say $\cot\alpha=3$. Then it is interesting to know the effect of this gentle slope on the stability and the rate of damage for slope of the considered armour units and the relationship between them, compared with the old situation. As roughly shown in Figure 6.1, stability of rock and cubes increases (cubes are at maximum stability level) considerably;

stability of tetrapod reduces, but only slightly and therefor still at considerably higher level than that of rock and cubes. This is due to the large contribution of interlocking to the total stability.

Consequently, the rate of damage of rock slope decreases. As can be seen from Figure 6.4, "wave height-damage" curve for slope cotα=3 becomes gentler than that for slope cotα=1.5. It can be expected that the same holds for a cube slope, while for tetrapod and accoropode slopes, it is still unclear. Logically, their rate of damage should increase, however this needs to be confirmed by model tests.

Thus, the change in slope angle, α, leads to the change in the hydraulic stability of armour units as well as damage rate of their slopes. Different armour units, however, have strongly varying characteristics (mass density, geometrical shape, skin friction) upon which their hydraulic stability and rate of damage primarily depend. As a result, in spite of some influence of slope angle, α, the above comparisons and conclusions made for rock, cube, tetrapod and accropode slopes are still valid, to a certain extent.

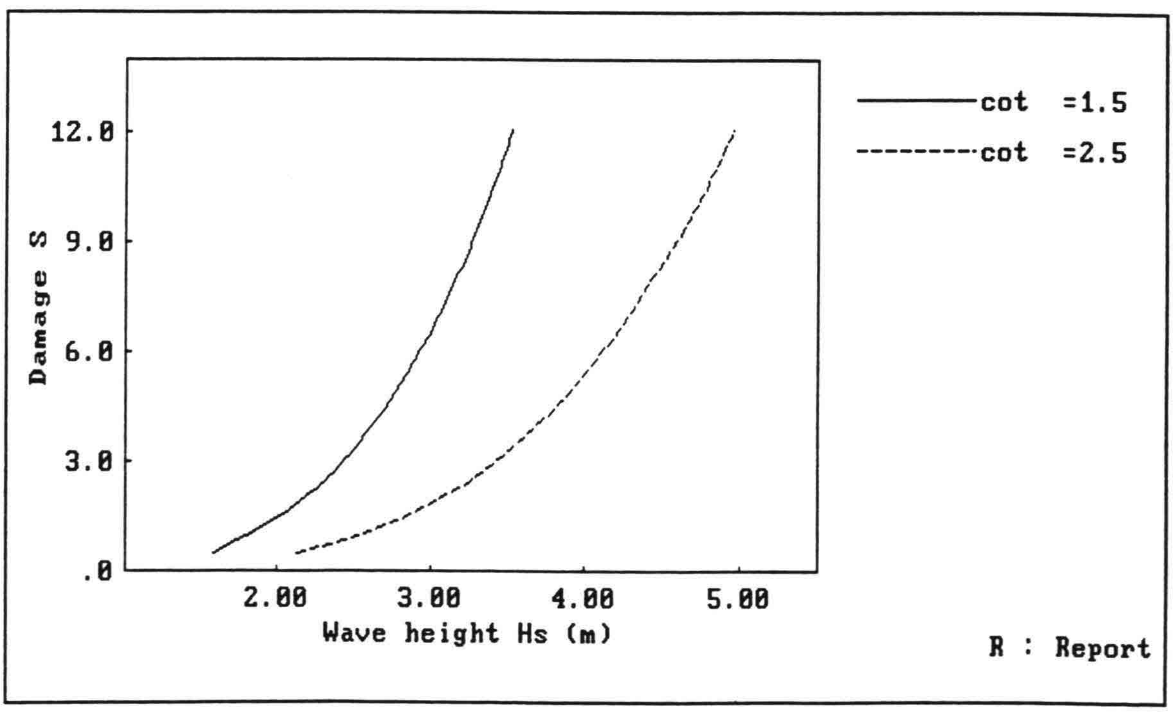


Figure 6.4 *The influence of the slope angle on the rate of damage for rock.*

6.3.4 Sensitivity to uncertainties

For each type of armour unit, besides the characteristic hydraulic stability and the rate of damage, its sensitivity or response, in terms of the change in required block mass to possible variations/ errors in the design parameters is an important property. In the same way as for dikes, the sensitivity for armour units can also be analyzed by investigating the influence of the variations in significant wave height, H_s, and mean wave period, T_m, on the required block mass.

Consider the case with the boundary conditions :

- Significant wave height $H_s = 3m$
- Mean wave period $T_m = 7sec$
- Storm duration $N = 3000$
- No damage criterion
  - for rock $S = 2$
  - for concrete units $N_{od} = 0$

Computations are given in Table 6.6 - 6.12 and plotted in Figure 6.5-6.6.

The sensitivity of armour units can be judged and compared through the rates of increase of the relative block mass $d(W/W_o)/d(H/H_s)$ and $d(W/W_o)/d(T/T_m)$ or relative block mass W/W_{rock}.

Where:

W - Required block mass related to a certain percentage of increase of H_s or T_m.

W_{rock}- The same as W, but specified for rock only.

W_o - Required block mass related to the design values of H_s and T_m.

<u>ROCK</u>

H/H_s	1	1.1	1.2	1.3	1.4	1.5
W_{50}	6140	7608	9254	11080	1309	15288
$W_{50}/(W_{50})_o$	1	1.24	1.51	1.80	2.13	2.49

Table 6.6

T/T_m	1	1.1	1.2	1.3	1.4	1.5
W_{50}	6140	7084	8071	8996	8557	8302
$W_{50}/(W_{50})_o$	1	1.15	1.31	1.46	1.39	1.35

Table 6.7

T/T_m	$W_{50}/(W_{50})$	
	$\cot\alpha = 1.5$	$\cot\alpha = 2.5$, 3 and 4
1	1.0	1.0
1.1	1.15	1.15
1.2	1.31	1.31
1.3	1.46	1.48
1.4	1.39	1.66
1.5	1.35	1.72

Table 6.8 *(The influence of slope cotα)*

<u>CUBE</u>

H/H_s	1.0	1.1	1.2	1.3	1.4	1.5
W	10426	14279	19029	24781	31648	39740
W/W_o	1	1.37	1.83	2.37	3.04	3.80

Table 6.9

T/T_m	1.0	1.1	1.2	1.3	1.4	1.5
W	10426	9847	9360	8912	8520	8397
W/W_o	1	0.94	0.90	0.85	0.82	0.80

Table 6.10

TETRAPOD

H/H_m	1.0	1.1	1.2	1.3	1.4	1.5
W	6425	9056	12387	16524	21577	27600
W/W_o	1	1.41	1.93	2.57	3.36	4.30

Table 6.11

T/T_m	1.0	1.1	1.2	1.3	1.4	1.5
W	6425	5733	5163	4692	4291	4165
W/W_o	1	0.89	0.80	0.73	0.67	0.65

Table 6.12

ACCROPODE

H/H_s	1.0	1.1	1.2	1.3	1.4	1.5
W	1763	2346	3046	3872	4837	5949
W/W_o	1	1.33	1.73	2.20	2.74	3.37

Table 6.13

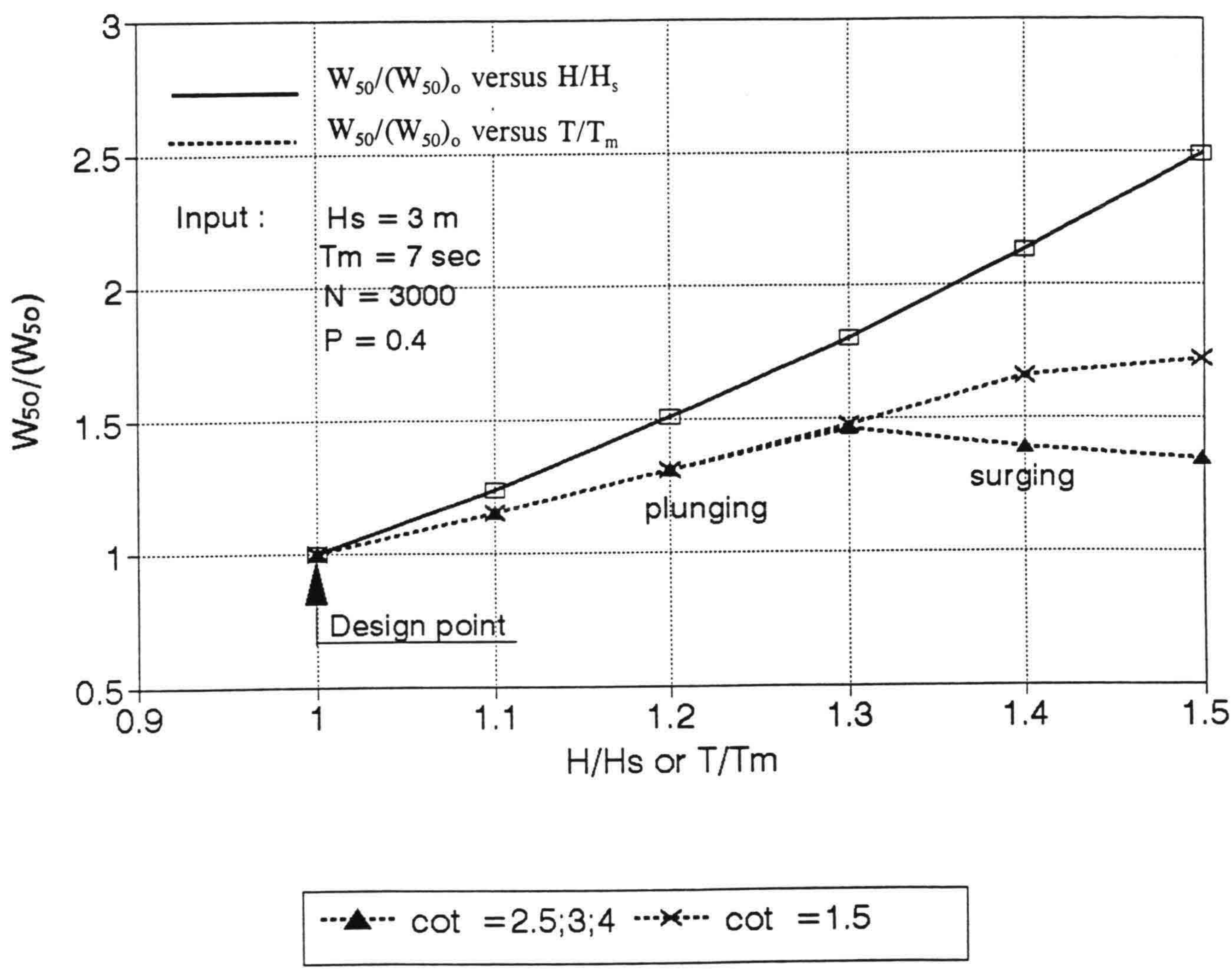


Figure 6.5 *Relative block mass of stones $W_{50}/(W_{50})_o$ versus relative wave height H/H_s or wave period T/T_m. $(W_{50})_o$ - the block mass related to the design values of H_s and T_m.*

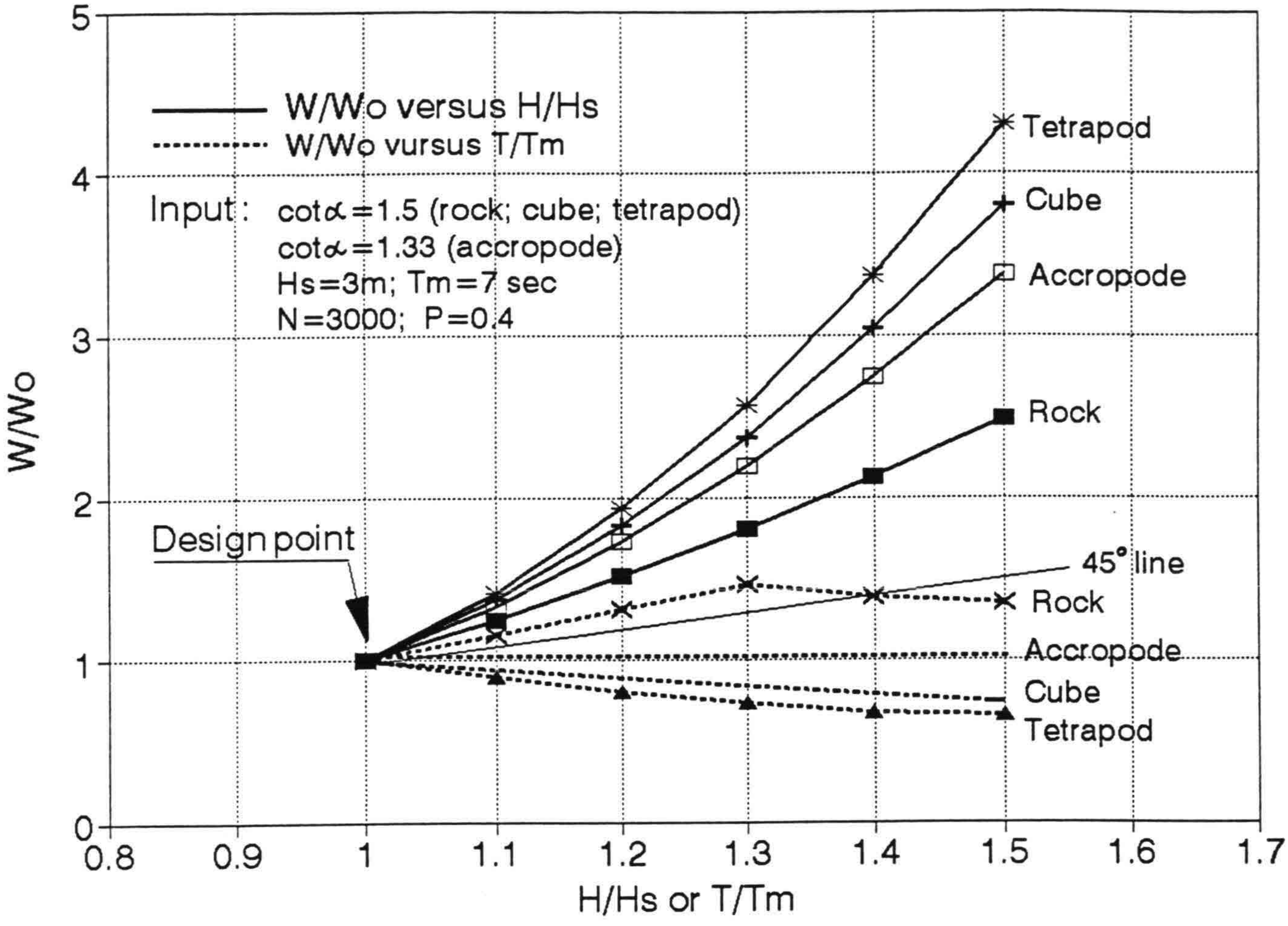


Figure 6.6 *Relative block mass, W/W_o, versus relative wave height, H/H_s, and wave period, T/T_m, for the considered armour units. W_o - the block mass related to the design values of H_s and T_m.*

Amour unit	H_s, T_m		$1.2H_s$, T_m		
	W_o	$W_o/(W_r)_o$	W	W/W_o	W/W_r
Rock (r)	6140	1	9254	0.51	1
Cube	10460	1.70	19029	0.83	2.05
Tetrapod	6425	1.05	12387	0.93	1.34
Accropode	544	0.09	940	0.73	0.1
	1763*	0.29*	3046*	0.73*	0.33*

Table 6.14 *Comparison of the required block mass for the considered armour units. The note (*) indicates the values with a safety factor of 1.5 on nominal diameter. The(*) indicates the values with a safety factor of 1.5 on nominal diameter.*

In general, all considered armour units are much more sensitive to the variations in wave height than in wave period. That is :

* For variations in wave height, ***tetrapod appears to be the most sensitive one, next are cube, accropode and rock*** (Figure 6.6). The sensitivity of cubes and accoropode, however, is closer to the sensitivity of tetrapod than that of rock, e.g 20% increase in H_s leads to the increase of the block mass of 93%, 83%, 73% and 51% for tetrapod, cubes, accropode and rock, respectively (Table 6.14).

* For variations in wave period, rock, in contrast, is the most sensitive one among the considered units. Cubes and tetrapod show less sensitive than rock, while accropode does not appear to be sensitive. Neither wave period nor storm duration shows influence on the stability of accropode. Also, it is interesting to notice that, for this specified slope $\cot\alpha=1.5$, the increase in wave period leads to the decrease of the required block mass for cubes and tetrapod.

The rate of increase of the block mass according to wave height, $d(W/W_o)/d(H/H_s)$, does not depend upon slope, $\cot\alpha$, while the rate of increase of the block mass according to wave period, $d(W/W_o)/d(T/T_m)$, is slope-dependent. The influence of slope on $d(W/W_o)/d(T/T_m)$ is identified for rock (Table 6.8 and Figure 6.5). For concrete units, this still can not be done, as empirical formulas derived for them are based on the analysis of the tests for only one slope ($\cot\alpha=1.5$ for cubes and tetrapod, $\cot\alpha=1.33$ for accropode).

At the design point, compared with rock, cubes should be around 70% heavier, tetrapod - 5% heavier, while for accropode the required mass is only about 10% of the mass of rock or tetrapod. If applying a safety factor of $1.5^3=3.4$ on the mass of accropode (1.5 on the nominal diameter, D_n), then the required mass of acrropode reaches about 30% of the mass of rock or tetrapod, i.e still much lighter than rock and tetrapod. When wave height increase by 20%, the mass of block will increase by 51%, 73%, 83% and 93% for rock, accropode, cubes and tetrapod, respectively.

Considerations:

To date the behaviour of concrete armour units are not well- know. Van der Meer's stability formulas for cube, tetrapod and accropode, as mentioned above, are developed on the results of the model research that is limited to only one cross-section (i.e one slope angle and permeability factor) for each type of armour units. Consequently, these formulas can not soundly and completely describe the behaviour of those concrete armour units, unless they are improved by more extensive model tests and research.

Nevertheless, based on the analysis and comparisons in the previous sections, some conclusions can be made:

* Compared with rock, the considered concrete armour units appear to be more sensitive (in terms of required block mass) to the variations in wave height, and among them, tetrapod are the most sensitive ones.

* With the increase of wave height, damage to a slope of cubes or tetrapod develops less progressively than slope of rock, while slope of accropode shows no damage up to a certain high wave height and just after the initiation of damage, it collapses.

* For rock, cube and tetrapod, the initial stability levels are more or less in the same order of magnitude, while accropode distinguishes itself by very high initial stability.

Finally, from results of the analysis and considering the situation that there are large uncertainties in the environmental design parameters, particularly in the hydraulic design parameters and uncertainties related to design and construction aspects in Vietnam, it follows that:

* Rock and simple shaped concrete armour units like cube, antifer cube etc. are more relevant than tetrapod and other complicated shaped units.

* Though accropode slope has very brittle mode of failure, this new type of concrete armour unit deserves due attention whenever concrete units are involved in the design. This is due to the fact that accropode has very high stability number, perhaps as high as that of dolos, while it is mechanically stronger than dolos and simpler in shape compared with tetrapod. if no safety factor is applied, the required mass of accropode is only around 10% of the mass of rock or tetrapod. Applying a safety factor of 1.5 on the stability number $H_s/\Delta D_n$, as recommended by Van der Meer (1988c) taking into account the brittle mode of failure, the required mass then reaches about 30% the mass of rock or tetrapod. In the same manner, uncertainties in the HDP can also be accounted for by applying another safety factor and after this, accropode might still be a good option.

6.3.5 Rubble mound breakwaters with a crown wall

Quite often, rubble mound breakwaters are provided with aconcrete crown wall. Usually, the main function of a crown wall is to improve the overtopping performance of the breakwaters. However, a crown wall can also be used for access, for carrying pipe-lines or conveyors, to provide a working platform for maintenance or even for cargo handling (breakwater combined with quay walls), and so on.

The application of a crown wall, for any purpose, raises question about its stability and the influence of its presence on the stability of the crest and front armour layer, and consequently of the structure as a whole. As already discussed in chapter 4, crown walls, on the one hand, reflect the wave run-up and thereby increase the wave run-down which may cause instability of armour units. On the other hand, designing a crown wall is still a difficult problem for the present state of knowledge. Indeed, there are neither reliable methods for determining wave loads exerted on crown walls nor guidelines for designing them.

In case a crown wall is used for carrying pipe-lines, conveyors or whatever, the stability and right functioning of the crown wall is of great importance. Damage or failure to the crown wall, in this case, is not only damage or failure to the structure itself, but also (and maybe more serious) damage to the related activities which directly depend upon stability and right functioning of the crown wall.

In general, the uncertainty in securing the stability of a crown wall, its potential harmful effects on the structure's stability and the increase of the risk (the product of the probability of damage or failure and consequences), all together make the rubble mound breakwaters with crown walls less relevant than those without crown walls, particularly for circumstances when there are large uncertainties in hydraulic design parameters and uncertainties related to design and construction aspects.

6.4 Berm breakwaters

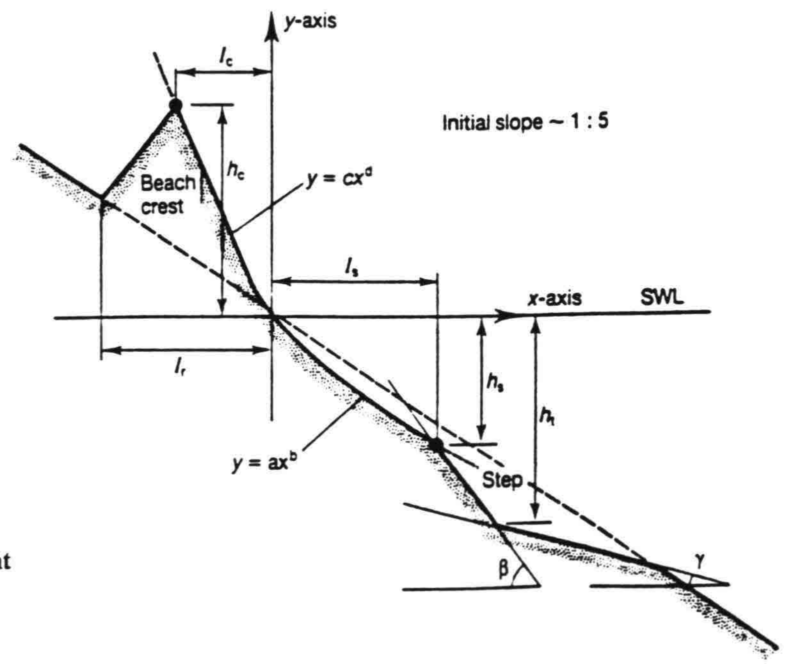
6.4.1 General

Conventional rubble mound breakwaters require heavy artificial armour blocks or natural rock, placed in a uniform slope, and that only little damage to the slope is allowed under severe design conditions. The wave-structure interaction results in hydrodynamic loadings. These loadings intensively focus on a certain part of an armour layer and hence endanger the stability of this particular part, while other parts may still be stable or even fare more than stable. Therefore, it is ideal, from both economic and technical point of view, to construct a breakwater, every part of which can work together well, not confronting but in harmony with waves/flow field, and has more or less the same stability level. To achieve this (at least to some extent), a breakwater should be constructed with a geometry and stone weight such that the natural profile readjustment, similar to the profile development of beaches and dunes under storm conditions, can take place. In this case, nature forms a profile for which the hydrodynamic loads are not concentrated on a certain part, and therefore are minimised. Allowing displacement of stones offers the opportunity to construct with smaller material.

A breakwater which is designed to reshape to a dynamically stable profile is termed as a berm breakwater. Under wave-structure interaction, material (stones) is redistributed into a profile which acts to minimize the applied forces by altering the flow field kinematics. The amount of stones that is available to contribute in the reshaping process has to be sufficient to form a dynamically stable profile in such a manner that the core is not exposed to the wave attack. This can be achieved if after reshaping the core is still covered by primary armour stones in a layer of at least two stone diameter thick.

6.4.2 Dynamic stability and computational model BREAKWAT

Dynamic stability is characterised by the formation of a profile which can be substantially different from the initial profile. Unlike static stability, which is described by the development of damage to the profile, dynamic stability is described by a developed profile. The developed profile is characterised by a number of length and height parameters and angles (Figure 6.7) which are related to the wave boundary conditions and structural parameters.



Notes:

l_r - **Run-up length**
h_c - **Crest height**
l_c - **Crest length**
h_s - **Step length**
l_s - **Step length**
h_t - **Transition height**
β **and** γ - **Angles**

Figure 6.7 *Schematised profile on 1:5 initial slope, Van der Meer (1988)*

Extensive model tests show the influence of several parameters on the development of the dynamically stable profile. The profile is clearly influenced by wave height and period, angle of wave attack and storm duration, diameters and grading of stones, water depth and variations of water level. Parameters, which have no or minor influence on the developed profile, are spectral shape, shape of stones, initial slope and crest height.

Based on the test results, the relationships between the characteristic profile parameters and the hydraulic and structural parameters were established (Van der Meer, 1988). These relationships were used to develop the computational model BREAKWAT. The boundary conditions for this model are :

- $H_s/\Delta D_{n50}$ = 3-500
- Crest above still water level
- Arbitrary initial slope.

The application scope of the model :

- Design of berm and S-shaped breakwaters ($H_s/\Delta D_{n50}$ = 3-6)
- Design of rock and gravel beaches ($H_s/\Delta D_{n50}$ = 6-300)
- Prediction of behaviour of core and filter layers for breakwaters under construction during yearly storm conditions
- Sensitivity analysis on a designed profile.

6.4.3 Sensitivity analysis for berm breakwater

DESIGNING A BERM PROFILE

The basis criterion in the design of a berm breakwater is to determine the cross-section with the minimum required amount of armour stones which, under design wave conditions, are being displaced to form an expected dynamically stable profile in such a way that the core is kept protected. To state more simply, the design process should result in the optimum dimensions of the structure (upper slope, lower slope, berm length).

Consider again the case with the hydraulic boundary conditions :

- Significant wave height H_s = 3m
- Mean wave period T_m = 7s
- Storm duration N = 3000

- Water depth h = 6.5m
- Incident wave angle β = 0°

The available construction material :

- Quarry run
- Wide-grading stone class : M = 100-1000Kg, D_{85}/D_{15} = 2, M_{n50} = 500Kg, relative mass density Δ = 1.52.

Apply BREAKWAT model for investigating possible profile-alternatives with different upper and lower slopes, it results in an optimum berm profile shown in Figure 6.8. This profile is characterised by :

- Crest height R_c = 4.5m (above still water level)
- Berm width b = 6.0m
- Upper and lower slopes n = m = 1.5

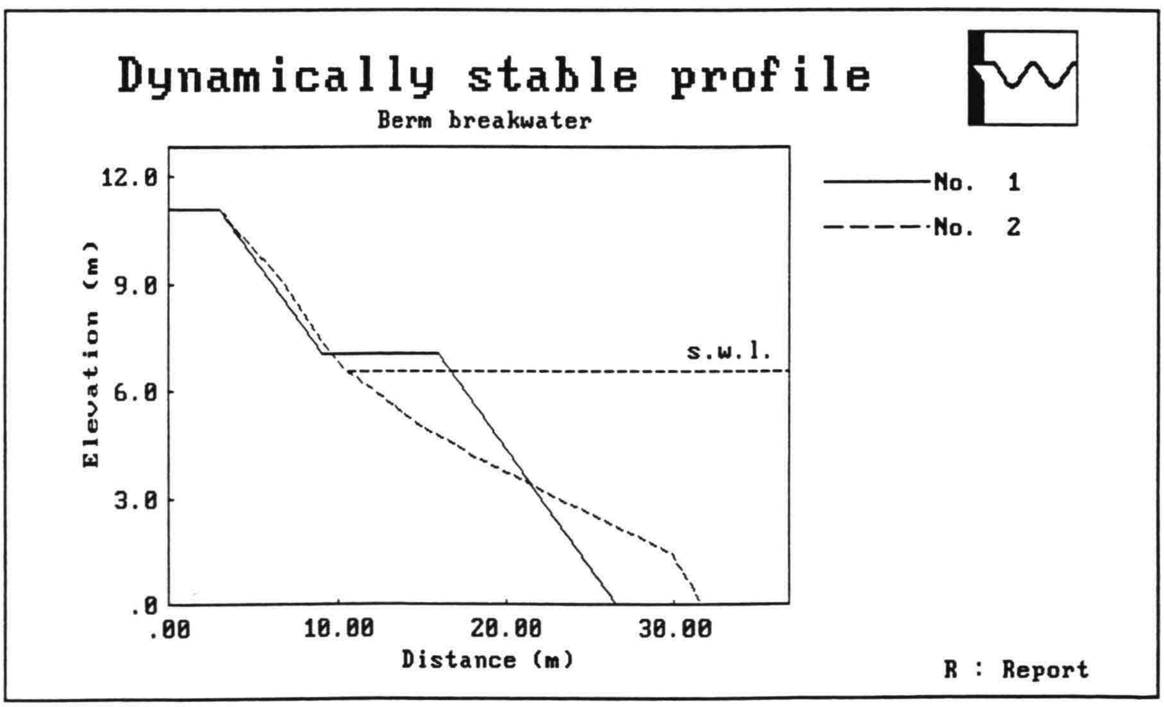


Figure 6.8 *N_o1 = initial berm profile*
N_o2 = development profile

The criteria for determining the above parameters are :

- The crest height of the initial profile, R_c, should meet the condition $R_c S_{op}^{1/3}/H_s \geq 0.25$ (in which S_{op} is the wave steepness for peak period) to avoid the damage to the rear of breakwater caused by overtopping waves.

- The berm width, b, should be determined in such a way that the upper point of the beach crest is not a part of the erosion profile. In other words, the upper point of the beach crest should lay on the initial slope in order to prevent erosion of the crest of the initial profile.

In designing berm breakwaters, there are two typical problems : the so-called longshore transport and the rock degradation. Transport of stones along the breakwater from cross-sections to cross-sections caused by oblique wave attack takes place when armour stones are not sufficiently large. Transport of stones causes problem at the round head and the end of the breakwater as well. Another problem involving in designing berm breakwaters is the rock degradation. At a berm breakwater, repeatedly displacement of armour stones during storms causes much more degradation of rock than in case of conventional breakwaters. This degradation causes stones to become smaller, and subsequently decreases their resistance to wave forces. This then results in changing of the equilibrium profile, and finally may lead to serious damage or failure due to the consequent more intensive transport leading to the core exposer.

SENSITIVITY ANALYSIS

The sensitivity or the degree of response of dynamically stable profile, under design wave conditions, to the possible more severe wave conditions is being investigated. This can be done by comparing the profiles related to the new wave conditions with the profile developed under design wave conditions. The parameters under consideration are wave height, H_s, wave period, T_m and storm duration, N which have major influence on the profile formation.

Different wave scenarios, the appropriate profiles and the combinations of profiles have been computed and given in Table 6.15 and Figures a1-a20 (see appendix).

Wave scenarios	H_s (m)	T_m (sec.)	N	Profile
- *Initial profile*	-	-	-	1
- Design wave condition	3.0	7.0	3000	2
- H_s increased by 20%	3.6	7.0	3000	3
30%	3.9	7.0	3000	4
- T_m increased by 20%	3.0	8.4	3000	5
30%	3.0	9.1	3000	6
- N increased up to 5000	3.0	7.0	5000	7
8000	3.0	7.0	8000	8
- H_s and T_m increased by 20%, N increased to 5000	3.6	8.4	5000	9
- H_s and T_m increased by 20%	3.9	9.1	3000	10

Table 6.15

From analysis of profile and comparison between them it follows:

* Wave height, H_s, and wave period, T_m, have dominant and similar effect on the profile formation, while the number of waves, N, shows less influence in this respect compared with H_s or T_m.

* The response of the developed profile to the increase of the design wave height or wave period by 20%-30% is not significant.

* After reshaping under design wave conditions, the developed profile is more or less statically stable.

6.5 Considerations

Berm breakwaters have been used more extensively in the past decade and it has been found that this type of structures is significantly less expensive than conventional breakwaters. The essential point is that armour stones used for this type of structures can be much smaller, at least five times lighter in weight than those required by conventional breakwaters. This allows for the design to be based on the actual quarry run rather than some pre-conceived specifications for stones for which a quarry must be found. With this design concept cost savings, compared to that for a conventional design, in the order of 40% have been achieved. Further, at many locations, dynamically stable breakwaters may be the only realistic type of structures. This is possibly because large stones are not available or the wave conditions are so severe that conventional breakwaters would required unrealistic large stones.

Considering the different types of rubble mound breakwaters in relation to the boundary conditions for design and construction of coastal structures in Vietnam, it appears that dynamically stable concept, in general, is the most relevant alternative. The fact that small stones can be used and large construction tolerances can be accepted allows the use of common construction equipment available in Vietnam, and particularly allows construction with limited-skilled labour. As a result, construction cost of this concept can be expected to be significantly lower than that of conventional structures. However, the decisive point that makes the dynamically stable concept relevant in Vietnam is the high flexibility of this concept. Under circumstances when there are large uncertainties related to the estimates of the hydraulic environmental design parameters, low risk and economically-efficient solutions demand robust and flexible designs with a wide margin between start of damage and total failure. To face large uncertainties, economic optimization leads to very conservative and therefore very expensive designs for rigid structures, and to less expensive and safer designs for flexible ones.

Nevertheless, it should be mentioned again that the right type of structures is very site-specific. In many cases, dynamically stable concept may not be relevant, perhaps due to the lack of rock with the suitable quality for armour layer near the project site or whatever reason. Then the more conventional rubble mound structures with concrete armour units are likely to be appropriate, also mainly due to their relatively high flexibility compared with vertical wall concept. In this case, it is necessary to select both the type of structures (cross-sections) and type of armour units. For the type of armour units, simple shaped concrete units like cubes, antifer cubes, etc. are likely to be relevant, as already mentioned in section 6.3.

Regarding the type of structures, this may be conventional, D-armour, S-shaped or Tandem breakwaters (Figure 6.8).

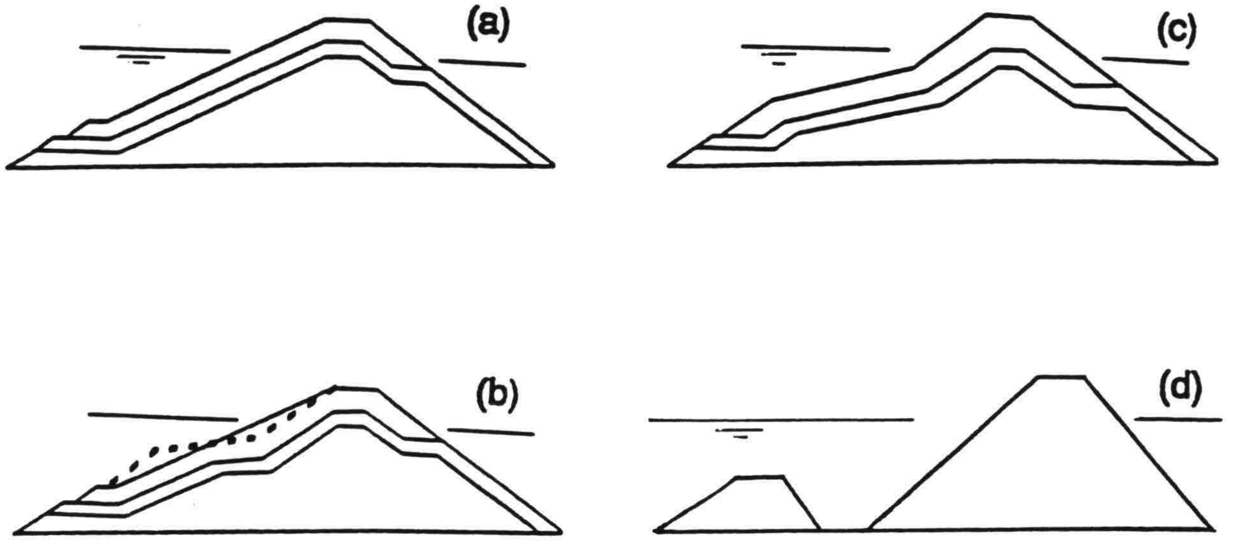


Figure 6.8 *Rubble mound breakwater cross-sections: a) Conventional; b) D-armour; c) S-shaped; d) Tandem.*

- Conventional cross-section has been used worldwide for many decades and thus much experience and understanding has been gained from this simple cross-section. The simplicity in shape is advantageous for understanding the structural behaviour as well as for realising the design. However, this simple cross-section has a serious disadvantage. It is prone to confront the wave loadings more than work in harmony with them. This leads to the situation that hydrodynamic loadings focus on a part of slope around still water level and endanger its stability.

- The so called D-armour cross-section is similar to a conventional design, however with a significant increase of the armour layer thickness around still water level. The increase in the armour thickness has two positive effects :

  - Improvement of the stability of the most vulnerable part due to the fact that permeability of this part is increased.

- Increase of the margin between start of damage and failure due to some reservation in armour thickness for safe erosion when the design loads are exceeded. When armour erosion increases, the D-armour cross-section can also transform to an S-shaped profile.

As a result, the structural behaviour of the D-armour cross-section is similar to the conventional one at low level of armour erosion, but better, while the allowable progressive reshaping to an S-shaped cross-section significantly increases the resistance comparable to an S-shaped design. It should be realised that though D-armour design is expected to reshape in life time, it is totally different from berm breakwaters.

- S-shaped breakwater is considered one of the most economic design. The special shape of the cross-section allows the whole profile to work together in harmony with the flow field and thereby minimizes the hydraulic loadings exerted on armour units. Waves are damped by the structure gradually, first from the toe, then along the profile before finally dying on the structure. As a result, the resistance to the hydrodynamic loadings is significantly increased and smaller armour units may be possible.

- A system composed of a conventional breakwater and a submerged-reef breakwater working in tandem is called tandem breakwater. This design is actually originated from the developed profile of a berm breakwater. A berm breakwater stabilizes by forming an S-shaped profile with a bench. It was recognised that the bench was really excess filler material and could be removed with little impact on the structure performance, forming a wave stilling behind the toe. The result was a tandem breakwater system with an outer submerged reef and an inner main breakwater. In many aspects, tandem breakwater is similar to an S-shaped design, but less construction material is required due to the removal of the bench in an S-shaped profile. Nevertheless, this is an innovative breakwater concept which requires further research before its behaviour can be well understood and guidelines for design can be established.

Relating the above considered alternatives to the design and construction boundary conditions in Vietnam, it appears that D-armour design might be more relevant than the others. This is because :

- it is more resistant to total destruction than a conventional breakwater,
- it is more flexible than an S-shaped breakwater and
- it is relatively simple in structural design.

Summarising this chapter, it might be necessary to mentioned that the best economy and safety can be achieved if designers are also flexible, i.e are not too rigid in the approach to the problem of breakwater design and are prepared to adapt to natural local conditions. It should also be realised that the best way to secure economy and safety is to work with the sea rather than against it, and use the best material which are locally and economically available than insist upon achieving high theoretical quality of the work at greatly increased expense.

Chapter 7

CONTROLLING THE HYDRAULIC LOADING AND STRUCTURE'S STRENGTH

7.1 Possibilities to Control the Hydraulic Loading

For all structures it is true that the design and the location affect the loads which are exerted on them. Hydraulic structures like sea dikes and breakwaters are extreme examples of this since small variations in water depth, slope steepness, structure's permeability, etc. cause large variations in the loads.

Even though hydraulic loads are stochastic in nature and exhibit extreme variations and even though there always exist uncertainties in hydraulic design parameters (water level, wave height, etc.), it is however possible, within certain limits, to control the hydraulic loading by making use of the fact that loads exerted on a structure are influenced by its own design. By manipulating the location or layout of the structure, more favourable hydraulic conditions can be obtained. For a given design condition it is still possible to choose the size, the sort and the place of attack of the hydraulic loads by a proper selection of slope steepness, construction materials, geometry and configuration. The following are some considerations on of different ways to control the loads for the case of sea dikes and breakwaters :

- By proper allocation of the structure it can be possible to get relatively less severe hydraulic conditions and consequently the required structure can be less heavy or more reliable while its function and effectiveness remain the same.

• For breakwater the structure location can not be freely manipulated in order to get more favourable hydraulic conditions. This is because of the fact that a breakwater is never an individual structure designed for its own sake, it is always a part of a harbour or coastal protection scheme. The function can, therefore, be set by an integrated design of breakwaters, access channels, moorings, loading systems, calling frequency, harbour operation, signal system, etc.. Nevertheless, for many cases it is still possible, to some extent, to choose or to decide which hydraulic condition the structure is subjected to. Much attention should be paid to the water depth , since small variations in water depth (in front of the structure) can produce large variations in the loads.

It is well known that in shallow water, waves will break if the relative wave height reaches a certain value (say $H/h=0.7$ in which H and h are wave height and water depth respectively). Consequently, by limiting the water depth in front of structure one can more or less control the wave height.

• For sea dikes, however,compared with breakwaters, the structure's location can be more freely manipulated to avoid unfavourable hydraulic conditions. This is due to the fact that the primary function of coastal defence measures is to protect the hinterland from attack by high water and waves, and as a result, although an integrated design is always necessary, sea dikes are more independent structures than breakwaters. As in the case of breakwaters, maximum design wave height can be controlled by limiting the water depth in front of the structure.

Since sea dikes are located more or less parallel to the shore line, there are two typical positions to be considered (see Figure 7.1) :

- Position A is at about the edge of land and water
- Position B is far in land, rarely submerged

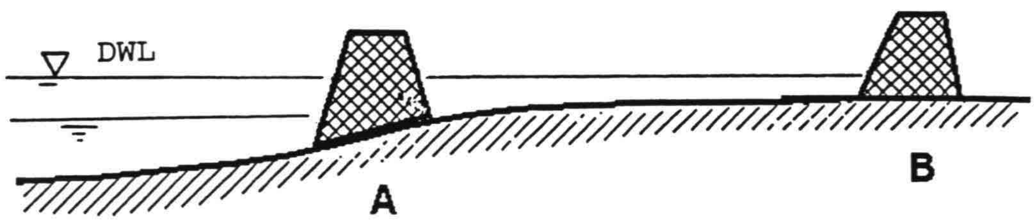


Figure 7.1 *Sea dike allocation*

During severe design conditions, dike at position A is likely to be subjected to large wave loads as water depth is increased considerably due to storm surge. Consequently, the dike at this position, whatever type it is, should be strong enough to withstand those possible heavy wave loads ; and this is a disadvantage because on one hand the structure is expensive and on the other hand it is very difficult to ensure a prescribed reliability level of the dike due to the unavoidable large uncertainty inhering in design wave height. However position A has an obvious advantage due to the fact that the stripe of land between position A and position B is protected and therefore can be used more effectively than under natural conditions. In comparison, even during severe storm conditions the wave loads exerted on the dike at position B should be much smaller than those at position A due to the depth restrictions. As a result, the dike at position B is less expensive and risky than the dike at position A.

It is impossible to say which of these two positions is more relevant for this is very site-specific matter. The choice could only be made by an integrated design taking into account all possible influences : the possible constraints in space, the importance of the area to be protected, the acceptable risk, etc. However, under circumstances when there is little or no data related or applicable to the site of interest, and thus there are inevitably large uncertainties in the hydraulic design parameters, position B is much more desirable than position A. This is not only due to the favourable hydraulic conditions there, but also due to the fact that there is less uncertainty related to the estimates of the design wave height, the dominant design parameter, as already mentioned above. At position B one can be sure that during design conditions wave heights are small because of the small water depth available. Here depth limit is a decisive factor.

- By changing the slope steepness, the breaker type, i.e the form of wave breaking on a structure or beach, can be influenced (Figure 7.2).

Wave action on a slope and some of its effects can be described by the breaker parameter or the surf similarity parameter ξ :

$$\xi = \tan\alpha/(s_o)^{0.5} = \tan\alpha/(H/L_o)^{0.5} \qquad (7.1)$$

in which:

α = slope angle
L_o = deep water wave length

The value of the breaker parameter is not only decisive for the type of breaker but also for the levels of run-up and run-down. For a given value of the (fictitious) wave steepness H/L_o, the value of ξ increases with increasing slope steepness. The type of the breaker in itself determines the way a breaking wave exerts loads on a slope. This can be either with a huge wave impact on the slope or with large masses of water running up and down the slope. Since the levels of wave run-up and run-down are also influenced by the value of the breaker parameter ξ , the slope steepness determines the required crest elevation, the level where maximum wave impact takes place and the level where other damage mechanisms endanger the stability of the structure. Thus it is possible to influence the breaker type and consequently to choose the critical damage mechanism by manipulating the slope steepness.

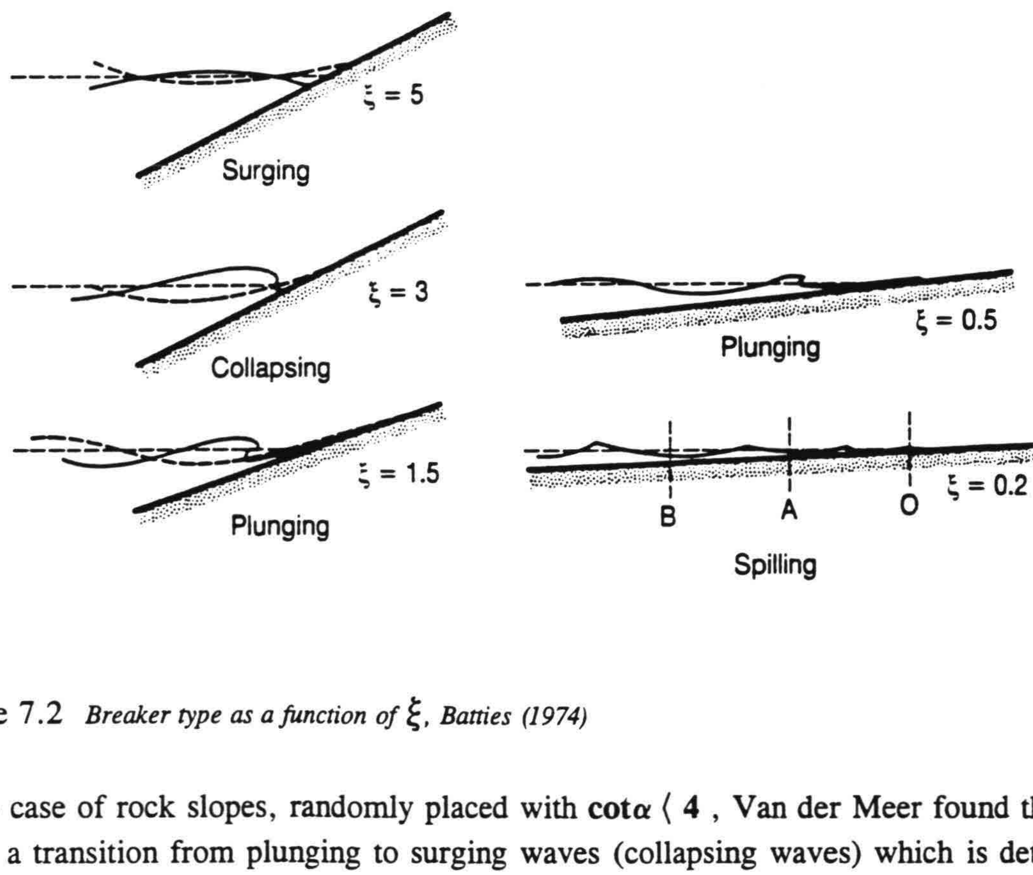


Figure 7.2 *Breaker type as a function of ξ, Batties (1974)*

In the case of rock slopes, randomly placed with **cotα** ⟨ **4** , Van der Meer found that there exists a transition from plunging to surging waves (collapsing waves) which is determined by the critical value of the breaker parameter ξ_{mc} :

$$\xi_{mc} = [6.2P^{0.31}\sqrt{\tan\alpha}]^{\frac{1}{0.5+P}} \qquad (7.2)$$

in which P is the permeability factor.

It should be noticed that the critical value of the breaker parameter ξ_{mc} does not depend upon wave steepness. Actually ξ_{mc} appear to be a property of randomly placed rock slopes. The situation when $\xi = \xi_{mc}$ is critical and undesirable for structure since the stability of armour stones, under this circumstance, is at the lowest level, as can be seen from Figure 7.3.

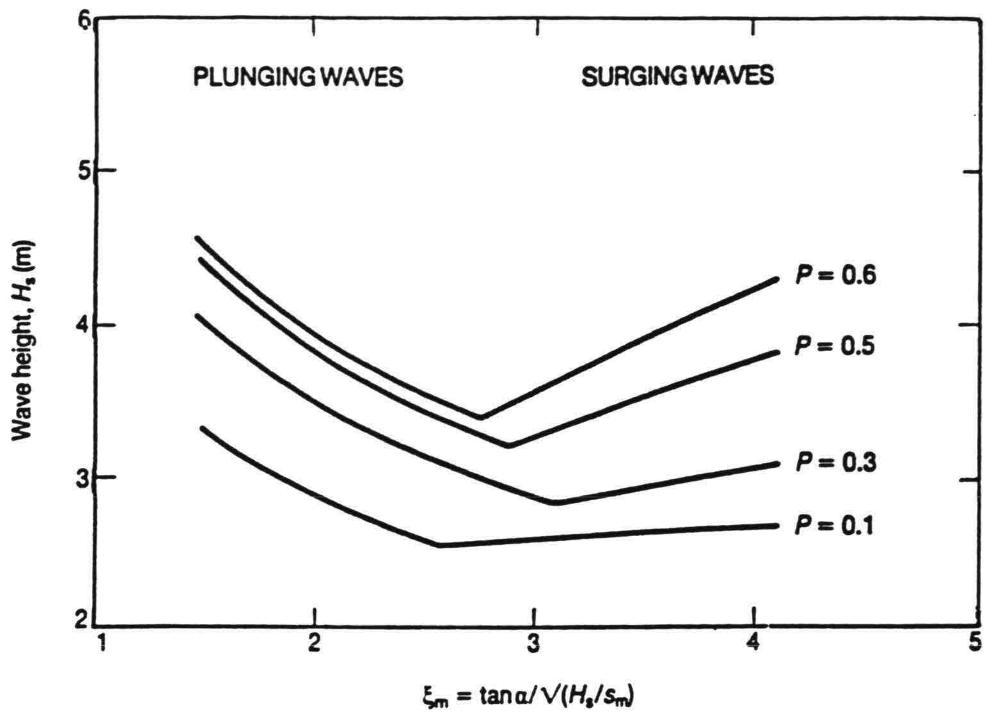


Figure 7.3 *Wave height versus breaker parameter; influence of permeability, Van der Meer (1993).*

For a given wave steepness $S_{om} = H/L_o$ and a structure's permeability P, the worst situation when $\xi = \xi_{mc}$ could be avoided by selecting an appropriate slope steepness.

- By variation of toplayer permeability it is possible to focus the hydraulic loads on certain part of the structure and to relieve other parts. So the design can be optimised for the locally available construction materials, equipment and techniques.

A less permeable toplayer leads to fairly limited pressure variations in the sublayers, also during large pressure variations outside. As a result, the internal stability can be guaranteed, but the stability of the top layer is endangered.

With a very permeable toplayer, on the contrary, the hydraulic loads on the toplayer will always be relatively small, also during severe wave attack, but in this situation, the loads on the sublayers are large because not so much damping occurs through the toplayer. Consequently, the stability of the toplayer can be guaranteed while the stability of the sublayers is endangered by the possible erosion (sucking out) of fine materials through large openings in the toplayer. Thus hydraulic loading can be partly controlled by manipulating the permeability of different parts of a structure.

- By changing the crest height, hydraulic loading can be significantly influenced.

It is obvious that when the crest level of a structure is low, part of energy in the uprushing waves can pass over the structure. In this case the wave loads on the front side of the structure are reduced since there is less energy left (lower run-down wave forces). As a result, the armour blocks on the front side can be smaller than those on a less or non-overtopped structure; however the crest and rear need to be armoured.

For sea dikes, this way of controlling hydraulic loadings is of limited application, as their primary function is to protect hinter land from flooding. However, for other coastal structures, particularly breakwaters, this is a very helpful way of controlling hydraulic loading and deserves due attention.

For placed block revetments, hydraulic loading can also be controlled in the following ways:

* Application of wide graded or fine filter materials reduces the filter permeability and therefore increases the stability of the placed blocks. Nevertheless, for the wide graded filters, there is a danger of internal instability (suffosion) when the fine fraction is eroded from in between the coarse fraction, as a consequence increasing the permeability of the filter. In this situation there is also a real chance of undermining the toplayer by the erosion of the fine part of the filter material.

* Application of a very thin granular filter layer underneath the toplayer reduces the upward gradients over the toplayer and consequently leads to a reduction of the required weight of the toplayer. However, the loads on the base will increase and this may lead to instability at the interface between base and filter, where the fine base materials (sand or clay) may be eroded.

7.2 Possibilities to Control the Structure's Strength

It seems somewhat illogical, but the possibilities for manipulating the strength of a structure are more limited than those for manipulating the loads. On the other hand, it should be mentioned that often the change of a construction detail influences both strength and loads. Also, the fact that the strength of a structure depends not only on the unity stability (the ability of individual units to stay in place) and the structural stability of units, but also on geotechnical stability and the stability of the structure as a whole makes it more complicated to control the structure's strength. The improvement of the strength of only some parts of a structure without taking the rest of the structure into consideration will usually solve only part of the problem. Following are some ways to increase the strength with emphasis on the stability of the top or armour layer, bearing in mind that these are not complete solutions.

* By increasing the block weight, the toplayer can be strengthened. This can be done either by increasing the weight of individual armour blocks or by grouting blocks together. Grouting blocks together allows the use of smaller blocks or stones for the armour layer; however due care is necessary to avoid decreasing the permeability of the armour layer too much leading to the danger of uplift pressure.

* By increasing the density of the armour blocks, the stability of the armour layer can be very effectively improved, for a 10% increase in density reduces the required weight by approximately 50% .

* By improving the degree of cooperation between individual blocks or elements, the stability of the structure can be increased. A good cooperation between blocks can be achieved by increasing the skin friction, contact area and interlocking degree; this mainly refers to concrete blocks. It should be mentioned that the stability of the armour layer can be essentially increased by improving the degree of interlocking between armour blocks. The contribution of the interlocking effect of shaped units like tetrapod, dolos,etc. to the total stability could reach approximately as high as 30% (Figure 6.1).

* By improving the degree of cooperation between layers or parts, the stability of the structure can be increased. The overall stability of a structure depends to a large extent on how different parts of the structure work together of different parts of the structure. For rubble mound structures or rubble slopes which are composed of loose materials and built in layers, each layer consists of materials of a specified type and

size, to ensure a good cooperation,i.e a smooth transition between layers is very essential. The cooperation between layers can be improved either by using more layers so that the difference in size of materials in two successive layers is not large, or by using wide graded materials for sublayers. However, division into many layers will increase construction cost, due to sorting of construction materials and difficulty in execution while using wide graded materials can lead to the danger of small materials migrating through toplayer.

* By increasing the structure's permeability, its stability can also be improved. For rubble mound structures or rubble slopes, permeability has a strong influence on the stability of the armour layer and consequently the stability of the structure as a whole. Higher permeability has two effects. First, hydraulic forces caused by wave action do not focus only on the armour layer but can penetrate deeper into other parts of the structure; thus more parts of the structure can be mobilised to work together with the armour layer "sharing" the same hydraulic forces, and as a result, this relieves forces acting on the armour layer. Second, the water level in the structure can better follow the variations in water level outside, and therefore, the up-lift pressure caused by hydraulic gradients over the armour layer becomes smaller. Consequently, the stability of the structure increases with increasing permeability, as can be seen from figure .

Summary

The fact that the designs of structures and their locations affect the hydraulic loads offers possibilities, within certain limit, to control these loads, i.e more or less to choose the intensity of loads exerted on the structural elements. By manipulating the location or layout of a structure, more favourable hydraulic conditions can be obtained. By a proper selection of the crest height, slope steepness, construction materials, geometry and figuration, it is possible to choose the size, the sort and the place of attack of the hydraulic loads.

A better insight in the wave-structure interaction and various failure mechanisms provides possibilities to control the structure's strength, i.e to guarantee its stability. To some extent, this can be achieved by increasing the weight or the density of units. This can also be obtained by increasing the degree of "cooperation" between different structural elements and parts or layers; and by increasing the structure's permeability (rubble mound structures or rubble slope protections).

Nevertheless, during the process of designing a coastal structure, numerous more or less subjective choices should be made. These are influenced by the considerations about the functional requirements, cost of construction, locally applicable techniques, technical restrictions, etc. and above all, the reliability of the structure in relation to the reliability of hydraulic design parameters.

Chapter 8

DESIGN APPROACH, DETERMINISTIC VERSUS PROBABILISTIC

8.1 Design process and objective

The motivation of any design process is the expected improvement, in terms of construction measures, of the risk-benefit balance compared to the existing situations. And the primary objective is that a maximum level of functioning of the future structure is required against a minimum cost during the life time of the structure.

Any design process is composed of two stages: geometrical/layout design and structural design. In the stage of geometrical design, general layout and concept is developed, aiming at an optimum geometry. This optimisation process is carried out from different view points. The most important of which are: the functional requirements, expected loads, environmental impact, available materials, transportation, future use and management.

In the structural design, the dimensions of the various components of the structure are determined. In the most general sense, the objective of the structural design is to provide detailed technical specifications that will enable the construction being realised with a certain strength-load ratio, the safety margin. This ratio is the ultimate criterion that follows from the functional requirements of maintaining a prescribed state of the structure under the expected design conditions.

At present, traditional deterministic and relatively newly developed probabilistic methods are the two options to secure safety margins for structures.

8.2 Design approach

In marine engineering, most loads, strength properties and design formulas are rarely known with certainty and should therefore be treated as random variable or stochastic processes.

Almost all available design formulas used in coastal engineering practice are semi-empirical, based on central fitting to model test results. The usual considerable scatter in test results cause uncertainties in these design formulas, which normally express only the mean values. Various design formulas are also stochastic in nature and this character should be accounted for in the design process, and preferably in a rational manner.

Strength, R, and loading, S, are two stochastic quantities, characterised by their respective statistical distributions:
The interrelationship of these determines the structure's reliability, the expected damage and the consequent maintenance costs. In general, loading and strength are both functions of a number basic variables/parameters regarding to environmental, material or structural characteristics (e.g wave height, block weight, material density, etc.). Often these basic variables/parameters are also stochastic in nature.

In the structural design, an appropriate distribution of strength R, relative to a given distribution of loading must be chosen to obtain the desired safety level. The presently available probabilistic design approach differs from the traditional deterministic method just in the way in which safety is provided for.

DETERMINISTIC APPROACH

Conventional design practice for coastal structures is deterministic in nature and is based on the concept of the design load, S_D, which should not exceed the structure's resistance, R_C, a certain characteristic value defined at a chosen limit state condition, ULS. The design load, S_D, is usually defined on a probabilistic basis as a characteristic value of the load, however often without consideration of the involved uncertainties. The safety factor then is defined as

$$F_S = R_C/S_D$$

By choosing a characteristic strength value, R_C, that exceeds the design load, S_D, sufficiently, the resulting probability of failure, $P(S>R)$, is kept low. So through the choice of F_S, a certain safety margin is maintained.

Limitations:

In this design approach, the uncertainties caused by stochastic character of the strength and loading and the uncertainties inherent in various design formulas are accounted for very roughly by imposing an overall safety factor F_S. Also, due to the lack of general standards for selecting the safety factor F, the choice of F_S has to be largely based on experience gained from the existing structures. This generally occurs in a rather subjective and conservative manner.

Another important limitation of this design approach is that no account is taken of loadings below or exceeding the design value S_D, and their contributions to the development of damage are neglected. This is a serious shortcoming when future damage must be estimated for maintenance assessment. As a consequence, using deterministic approach it is neither possible to optimize, nor to avoid over-design.

PROBABILISTIC APPROACH

In order to overcome limitations in the traditional deterministic design methods, to avoid costly conservatism, the probabilistic design approach was developed. To a certain degree, this approach is the logical extension of the traditional methods, but the problem of the choice of the appropriate safety factor is solved systematically by using statistical tools to describe the stochastic properties of both strength and loading variables.

In this approach, the design is based on an analysis of failure probabilities, which in turn entails the intensive and complicated analysis of joint probability density function of strength and loading variables. However, the efforts of this approach will be paid back in cost savings due the rational way of determining a safety margin. In fact the safety margin is obtained implicitly, by taking into account all the uncertainties of the loading and strength variables used in the design formulas, and of these formulas as well. By accepting a certain probability of failure, the safety margin can be adjusted in a rational process. Additionally, an improved appreciation of the stochastic elements in both loadings and strength also provides a means to evaluate the expected damage during the structure's lifetime.

In summary, the major advantages of the probabilistic design approach compared with the traditional deterministic one are:

- Better appreciation of strength and loading statistics

- Prevention of unnecessary conservatism leading to cost savings

- Provision of means for maintenance assessment and data for the structure's management.

In the scope of this paper, it is considered not relevant to go into details of the basis theory of the probabilistic method. This is confined to the general considerations above, and the classification of different methods given below. According to international convention, probabilistic methods can be distinguished on 3 levels :

Level-3 In these methods, the complete probability density functions of the strength and loading variables are taken into account.

Level-2 This level comprises a number of approximate methods which use the schematised distribution functions for the strength and loading variables.

Level-1 On this level, calculations are based on characteristic values and partial coefficients of safety. In fact, these are semi-probabilistic methods.

Deterministic methods that use one safety factor F_S, between the characteristic strength and loading are often indicated as level-0 probabilistic methods.

Strictly speaking, calculations at level-1 do not involve failure probabilities, but provide a method of checking whether a defined level of safety is satisfied. By using partial coefficients of safety derived from higher level calculations for various probabilities, a practical method of introducing an approximate representation of failure probabilities is possible.

8.3 Possibility of applying probabilistic method in design of coastal structures in Vietnam

The probabilistic design approach is increasingly being applied, directly or indirectly, in actual practice due to its obvious advantages over the traditional methods. In the hydraulic engineering field, this method has been used since the mid-1970s. However, the degree of application and areas where probabilistic methods can be applied in design practice are still limited. The reason behind this situation is that the approach is still in the developmental stage; and therefore many grey areas and blank spaces still exist about the knowledge that is needed.

In the coastal engineering sphere, two areas of primary concern are sea dikes and breakwaters. For sea dikes, the probability of some known failure mechanisms such as overflowing, piping, macro-instability of inner slope, etc. can be estimated using the probabilistic method. However, determining the overall probability of failure of a dike through the use of probabilistic calculations is still a very complicated problem. This is because of the multi-failure modes of a dike and possible correlations between these. In fact, for the collapse or failure of a dike, the failure function is usually very complicated due to the complicated interaction between water, soil, revetment, etc. and also due to the effect of the structure's length.

For breakwaters, especially conventional rubble mound breakwaters, probabilistic calculations are possible at the present. Nevertheless, the practical application of this approach is mainly confined with Level-1 calculations. These calculations use existing design formulas and incorporate partial coefficients of safety to provide any desired probability of failure within the service life time of structures. The approach, though it still needs further development and verification, enables designers to make a more rational and objective assessment of the reliability of conventional rubble mound breakwaters against a particular failure mode.

Considering the situation of coastal engineering in Vietnam at present and in the foreseeable future, the following considerations on the applicability of the probabilistic approach in design practice in Vietnam can be made :

* Level-2 probabilistic methods are possible, provided there are available necessary computational models. The application of these methods should aim at:

  - determining the failure probability of structures (primarily sea dikes and breakwaters) due to some typical failure mechanisms and

  - determining the relative contribution of various parameters/variables used in the design formulas to the probability of failure. Knowing this, rational data collection programs can be established avoiding the waste of time and money for less important design parameters.

* Level-1 calculations, i.e semi-probabilistic methods, which use partial coefficients of safety to the characteristic values of the design load and structure's strength, might be possible in design of conventional rubble mound breakwaters. This is primarily due to the fact that the method is quite handy and no computer software packages are required.

Summary:

Probabilistic approach is a logical step-forward in an effort to deal with the uncertainties caused by stochastic character of the loading, strength variables/characteristics and the uncertainties related to the design formulas used. This approach provides an excellent method to assess the reliability or failure probability of structures by a better appreciation of stochastic variables. A better insight into how the failure probability is built up from the various uncertainties results in cost saving due to:

* a more balanced design of components of the overall system. This can be achieved by avoiding the coexistence of over-designed components blowing up costs, and of under-designed elements causing risk.

* a soundly-based and rational safety margin can be produced.This avoids subjective and unnecessary conservatism.

In spite of the evident advantages of the probabilistic approach over the traditional design methods, its application in the actual design is still mainly confined with semi-probabilistic calculations. This is primarily because the probabilistic method is still in the development stage.

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APPENDIX

Figure 1

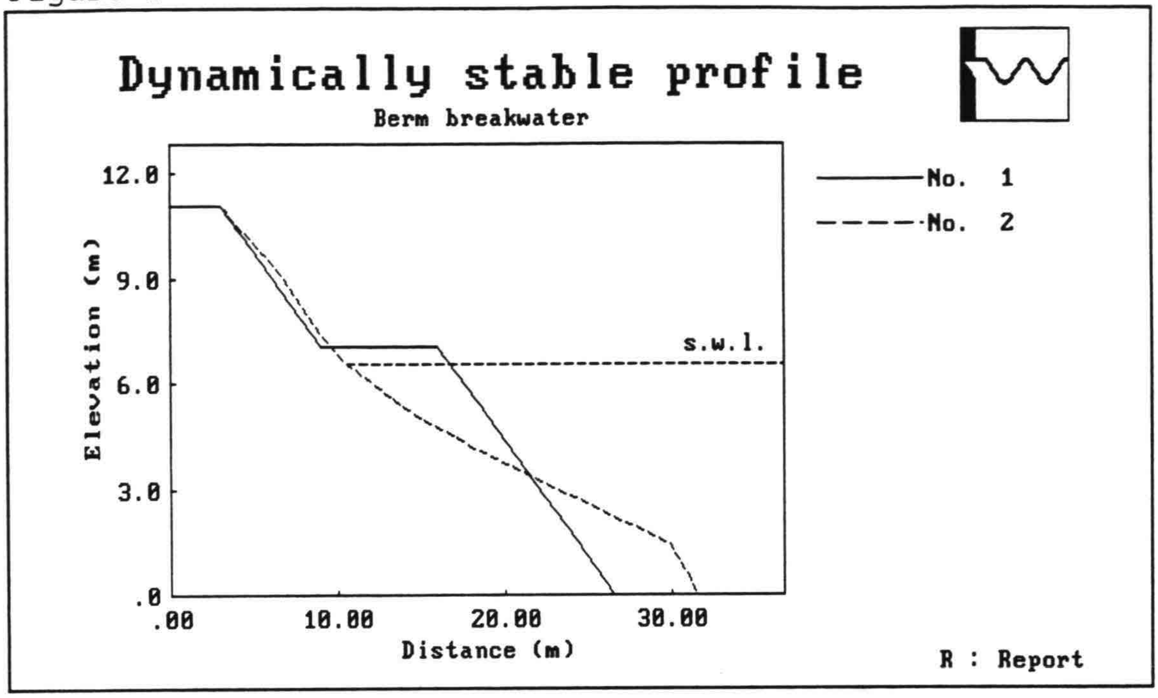

Figure 2

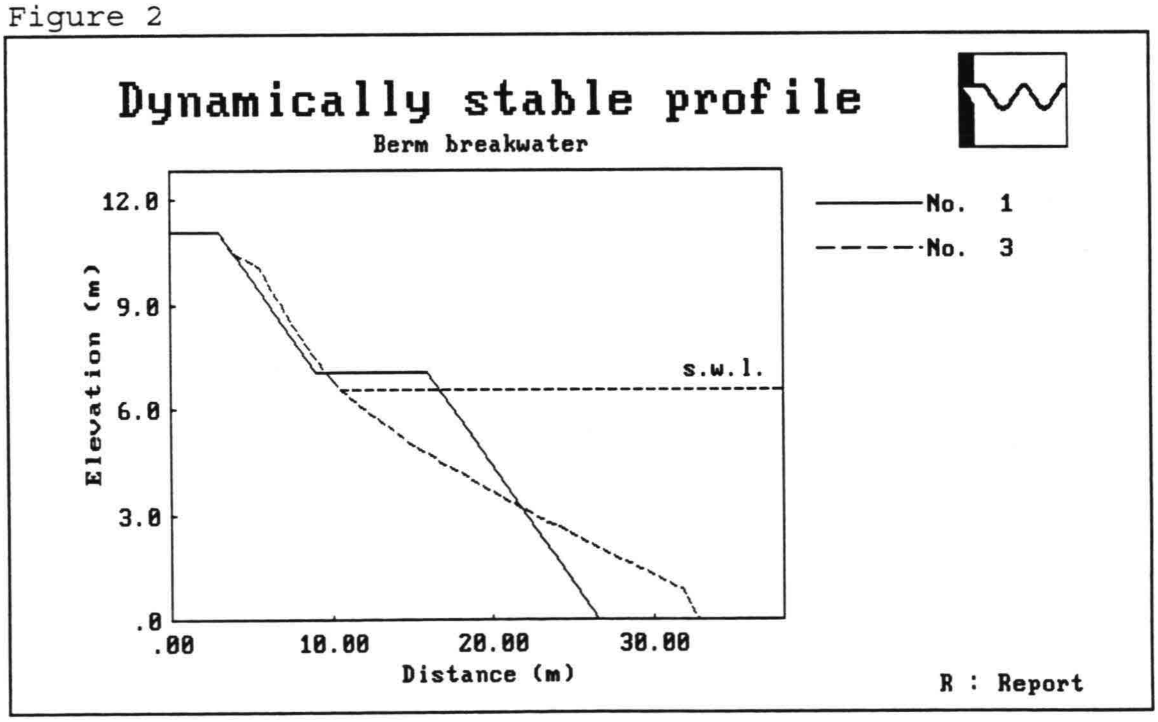

Figure 3

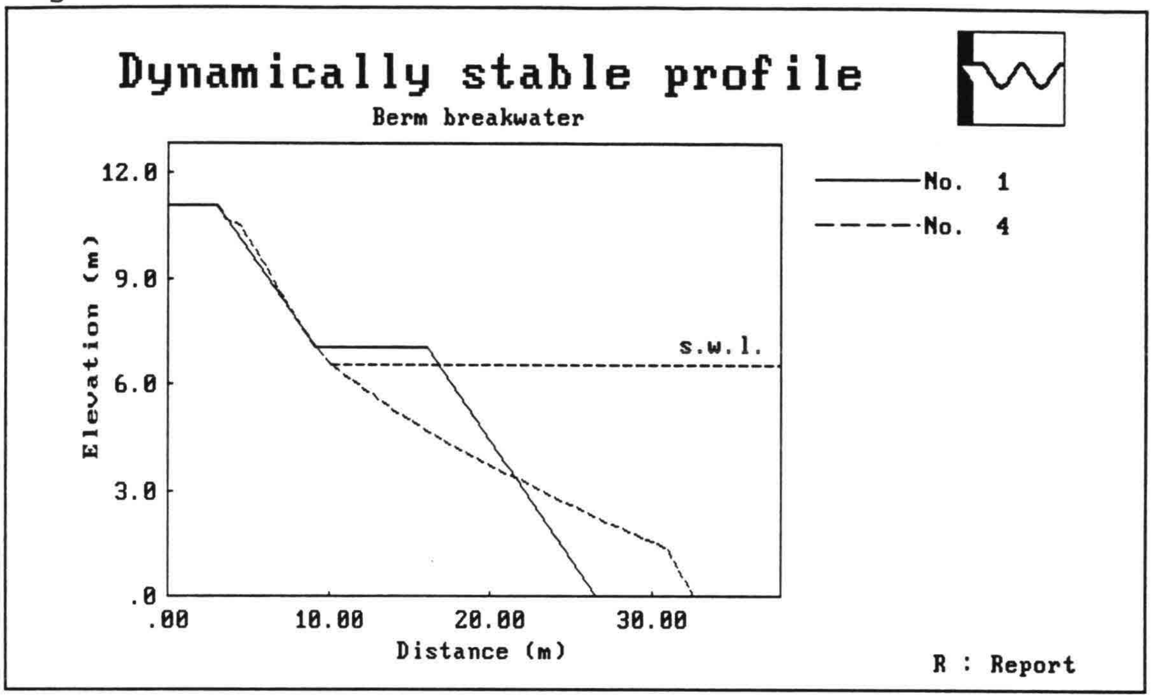

Figure 4

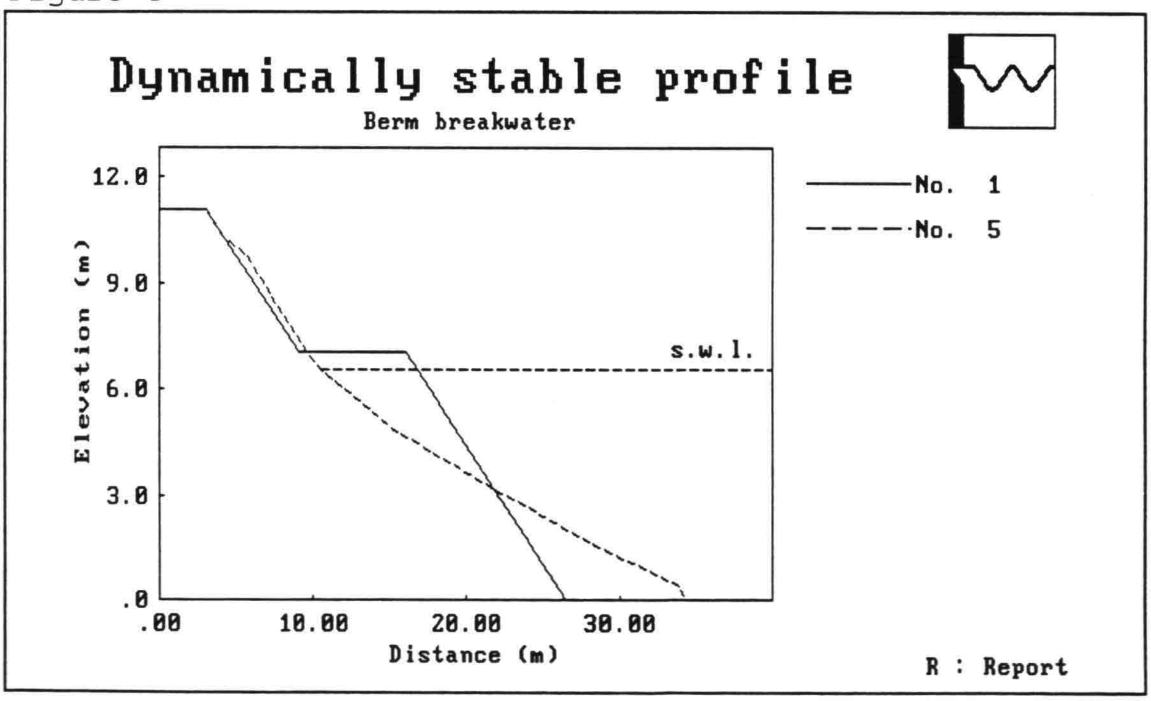

Figure 5

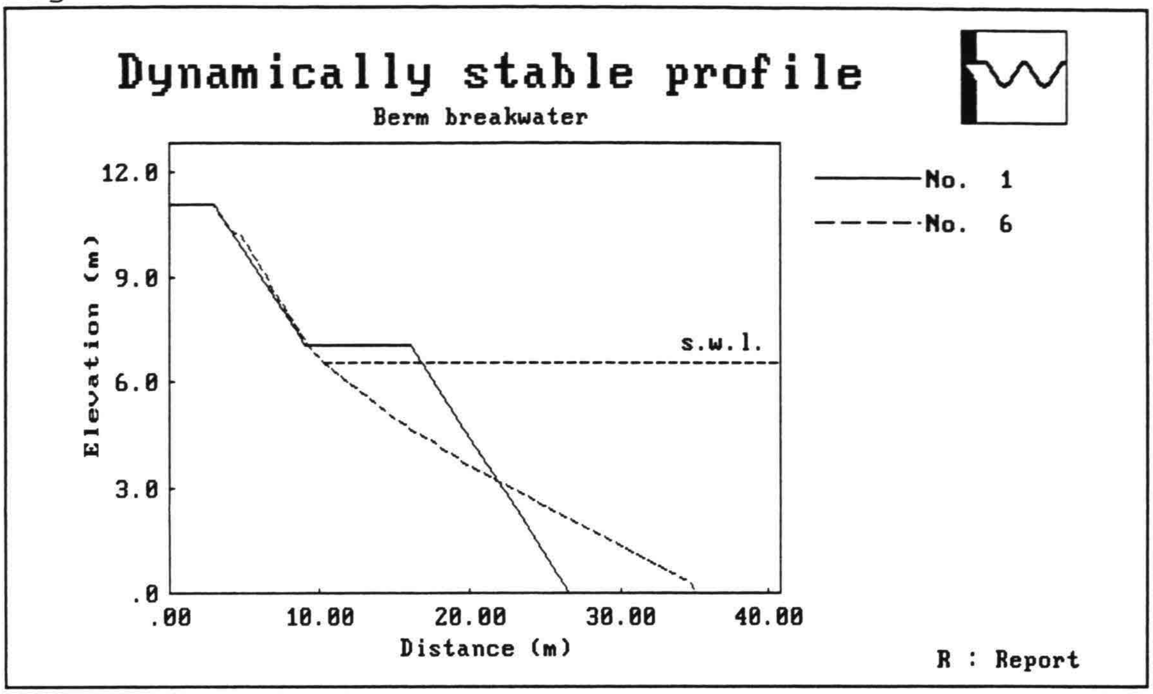

Figure 6

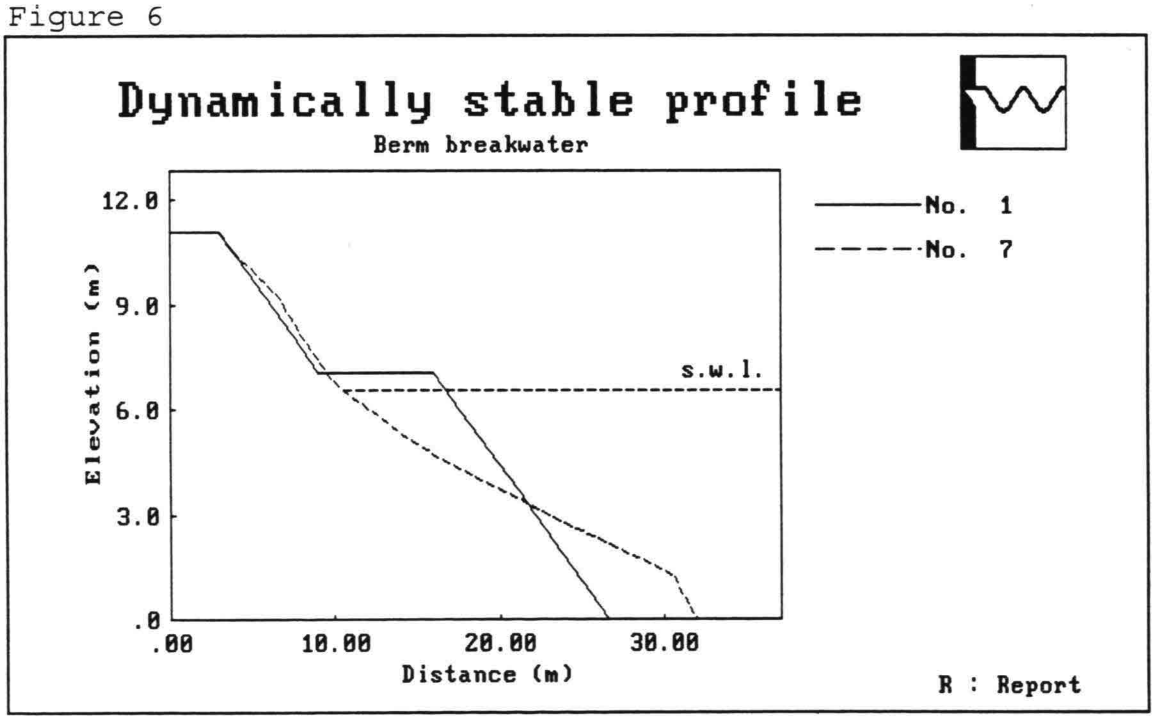

Figure 7

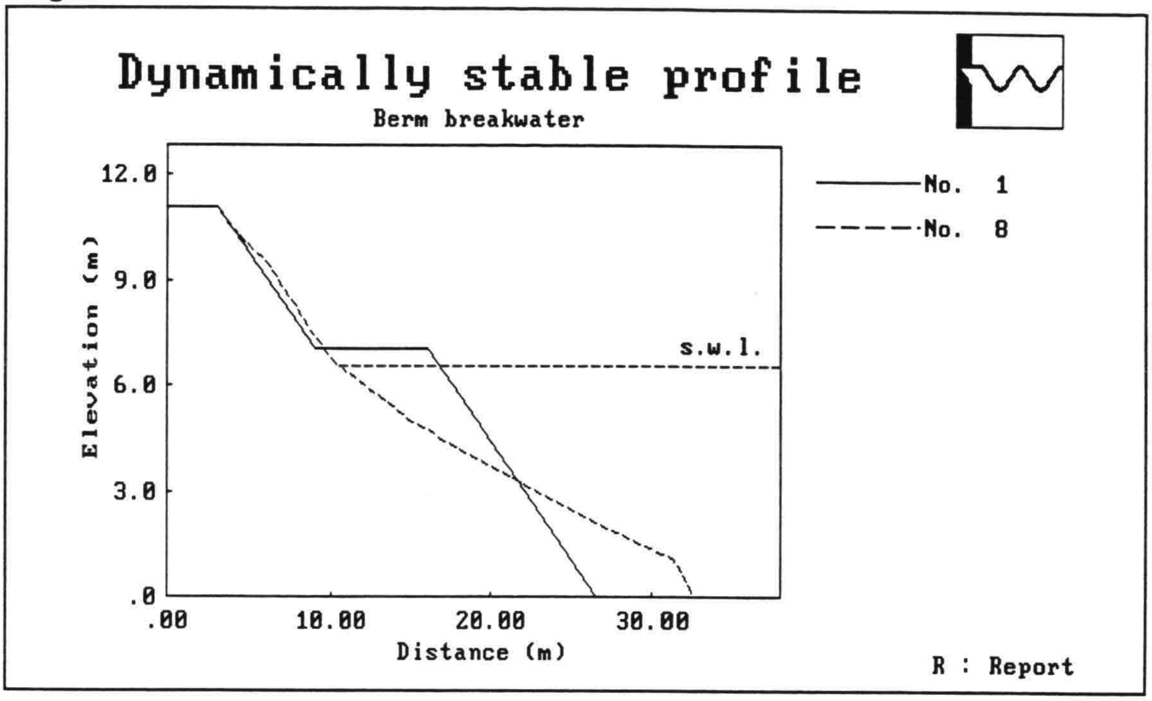

Figure 8

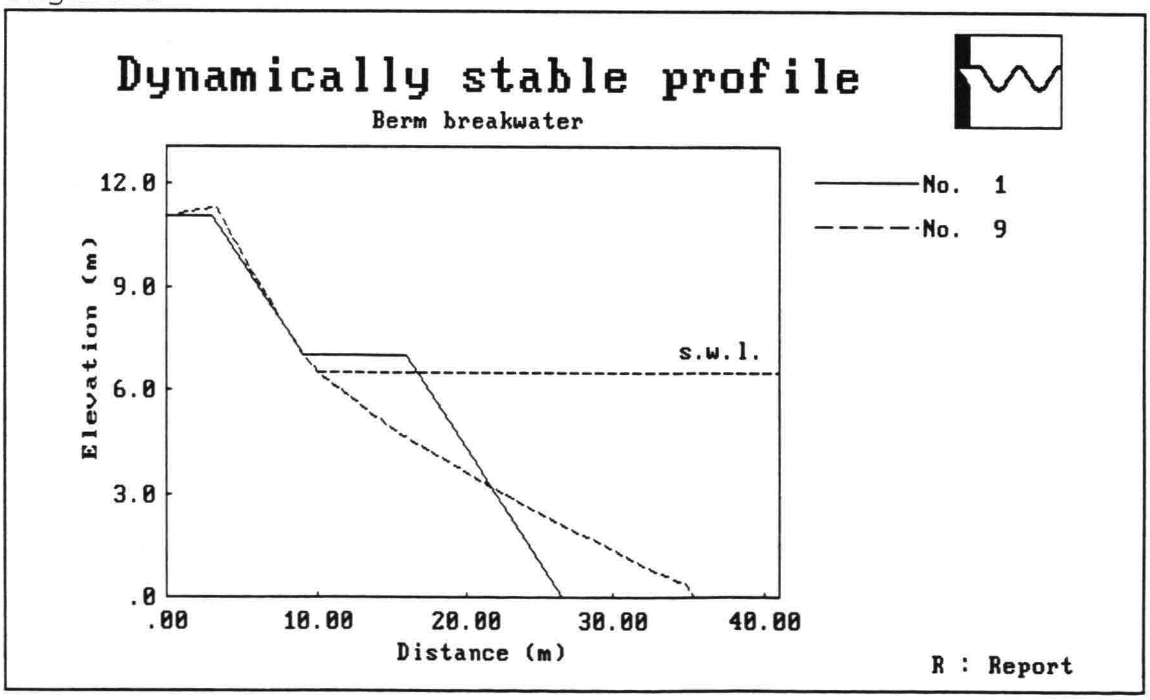

Figure 9

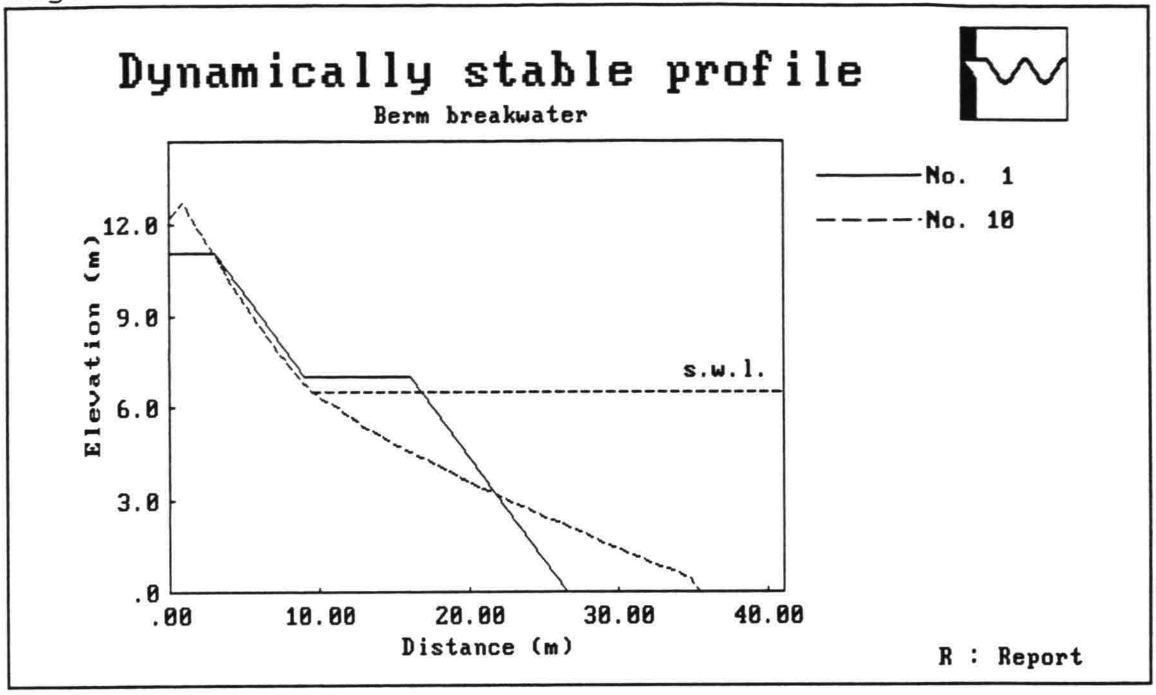

Figure 10

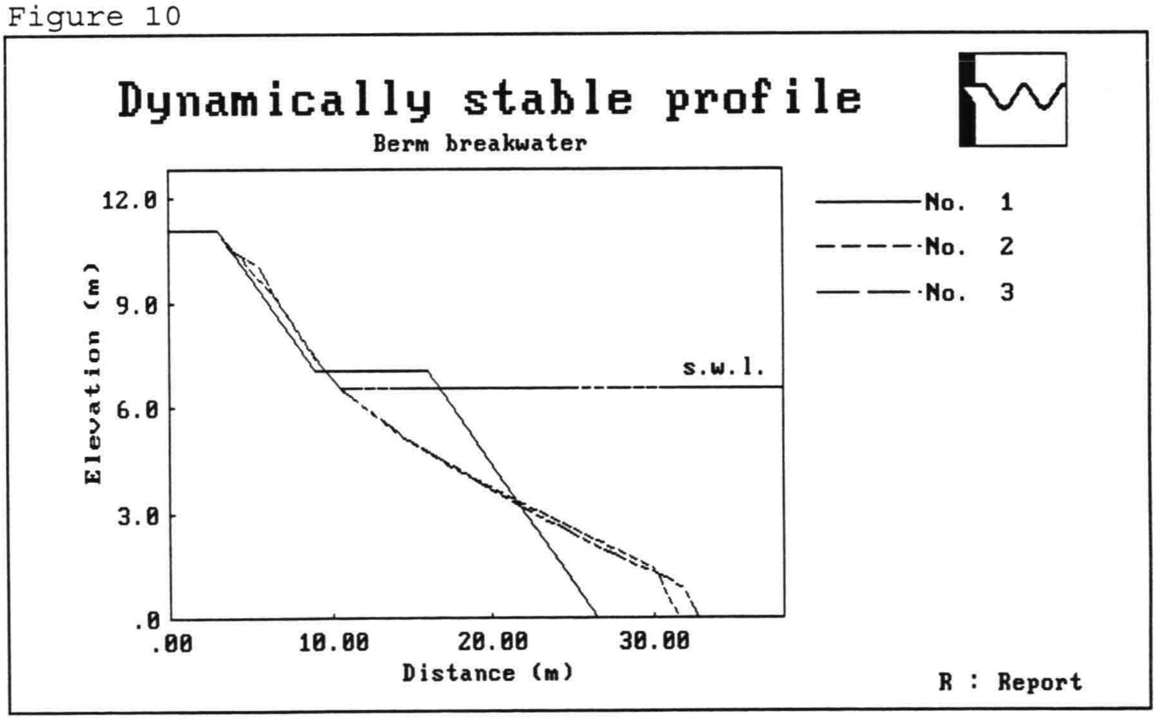

Figure 11

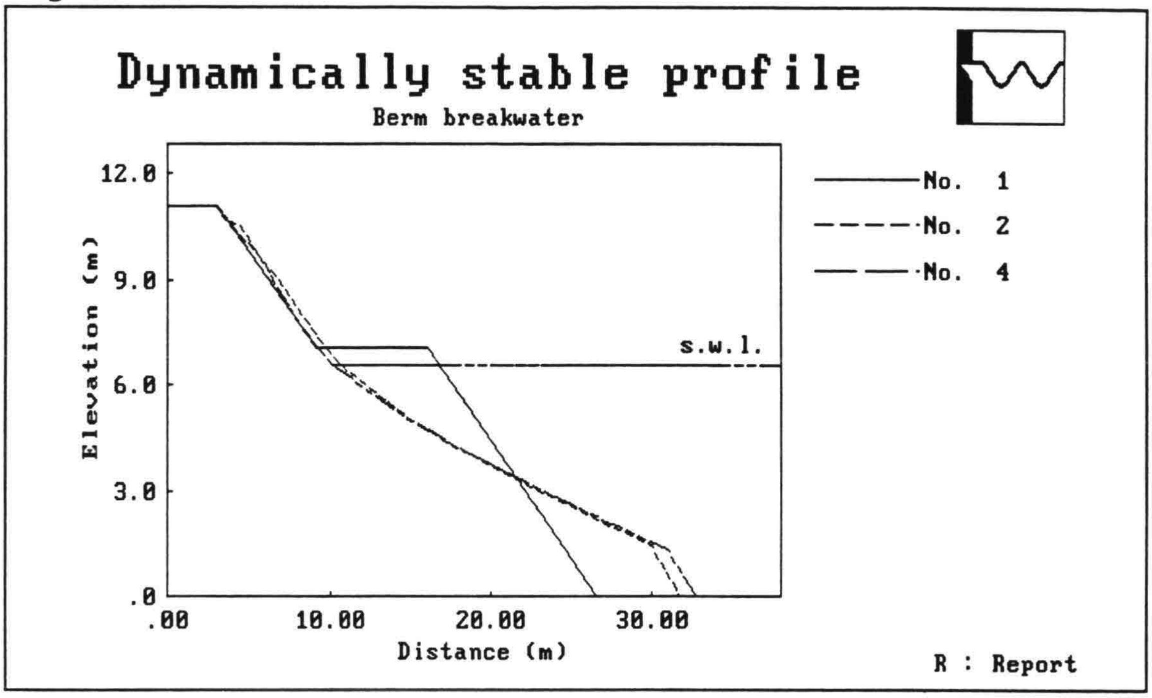

Figure 12

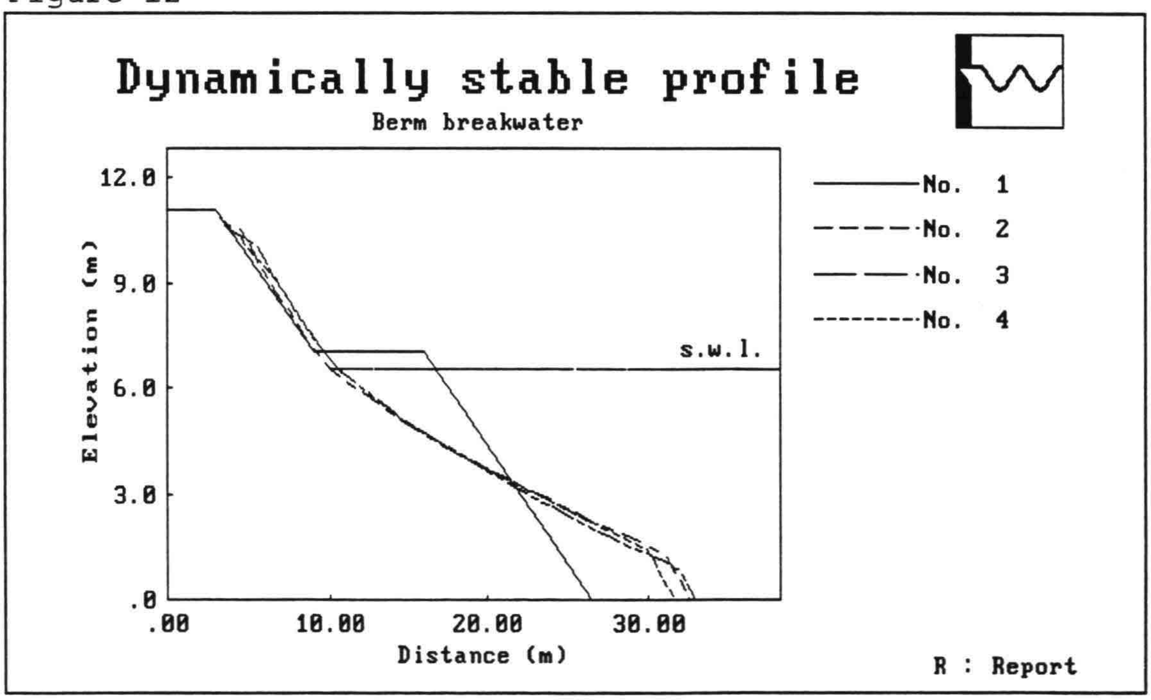

Figure 13

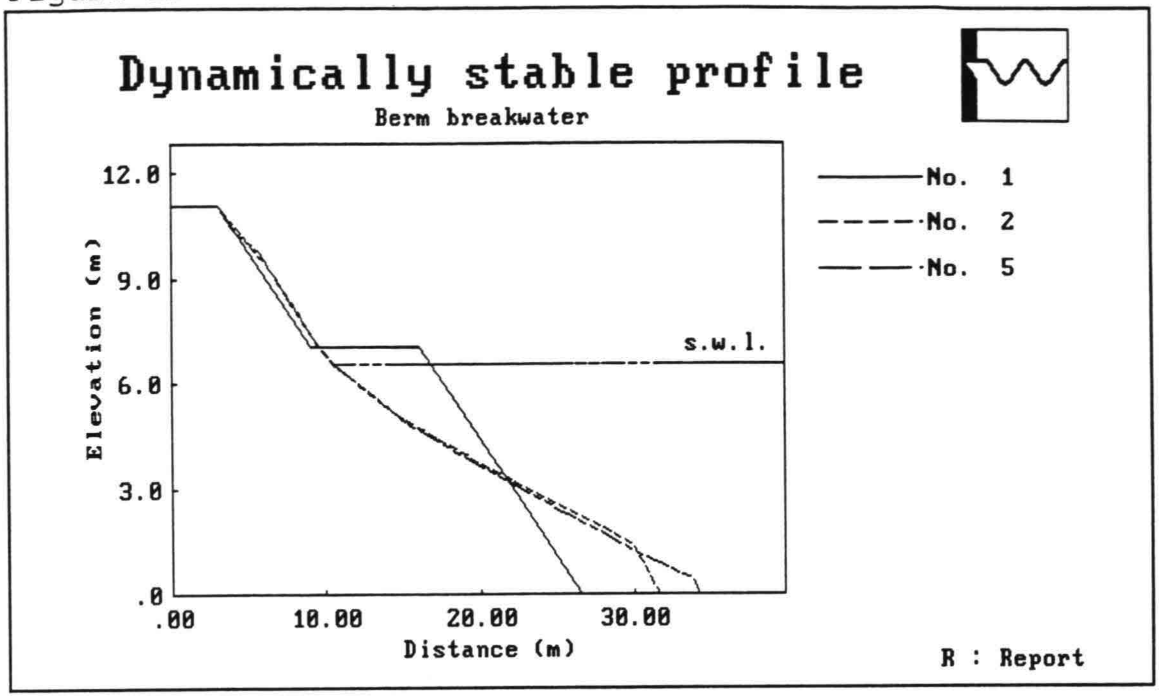

Figure 14

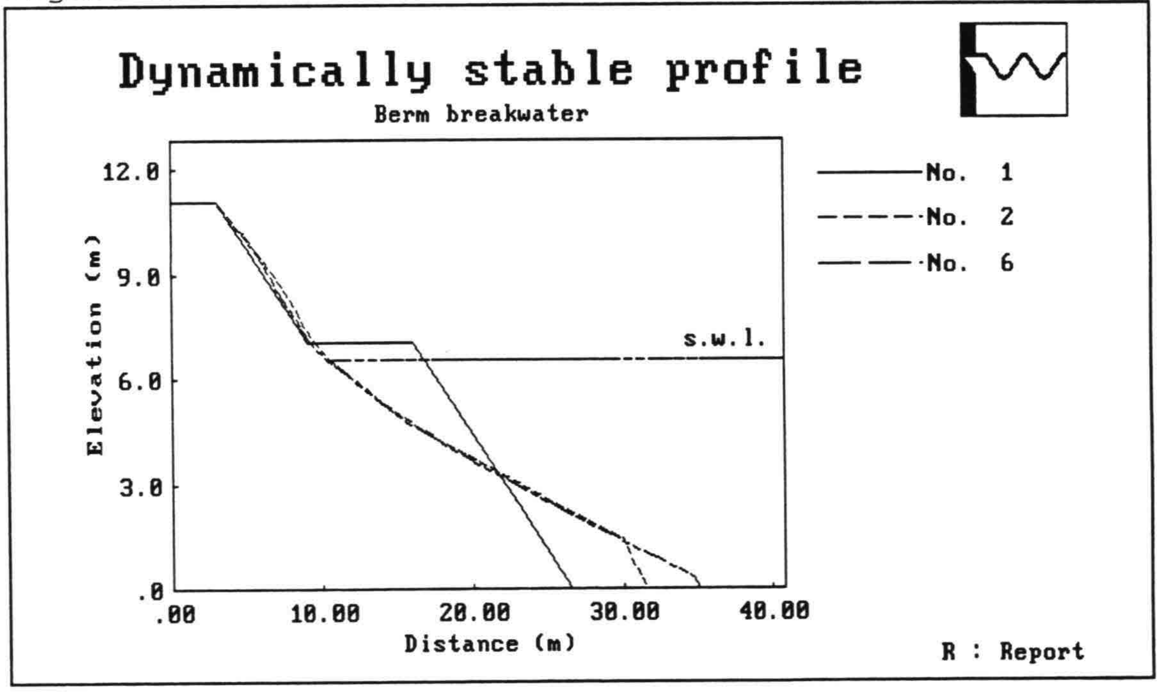

Figure 15

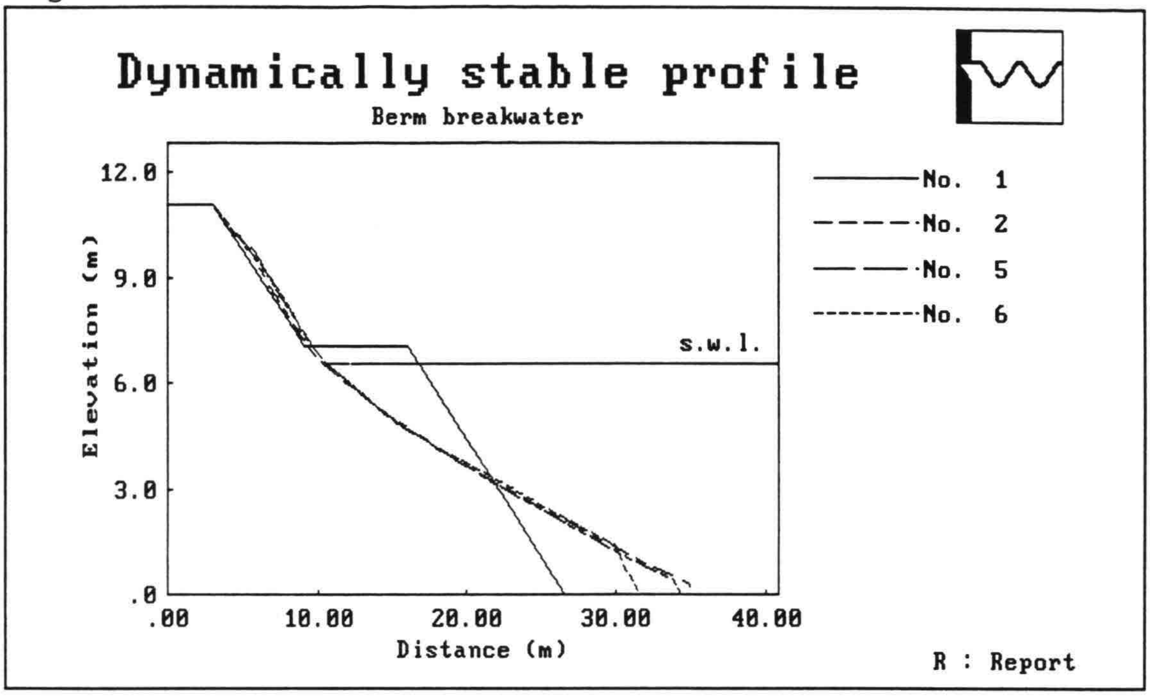

Figure 16

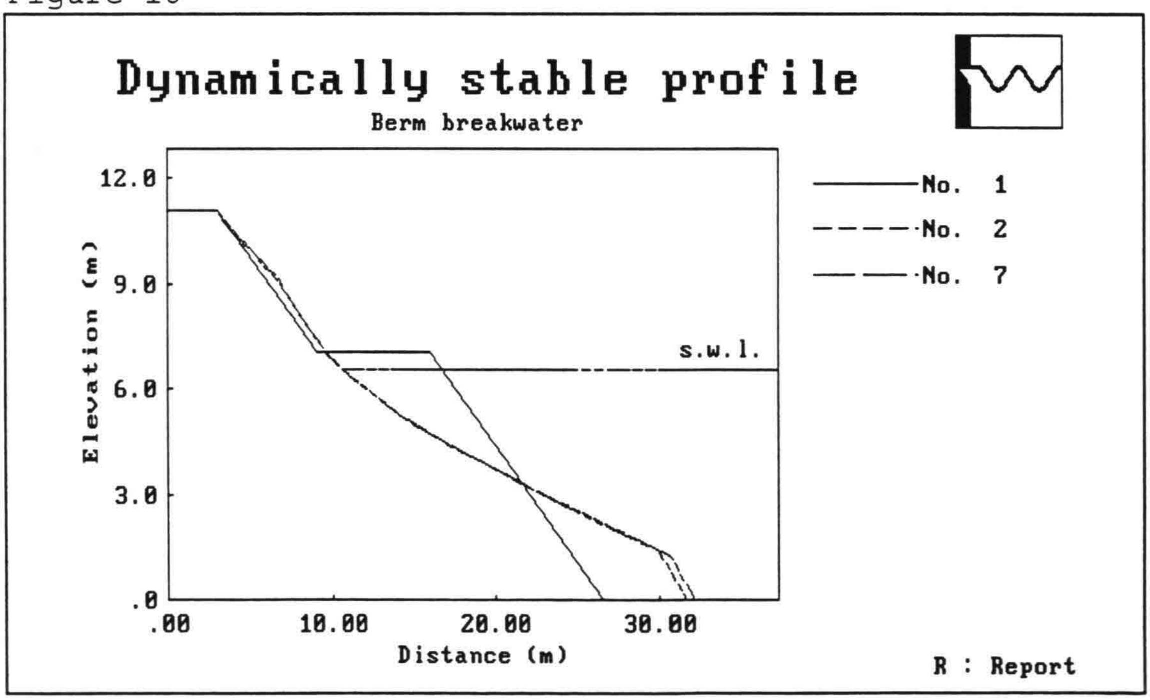

Figure 17

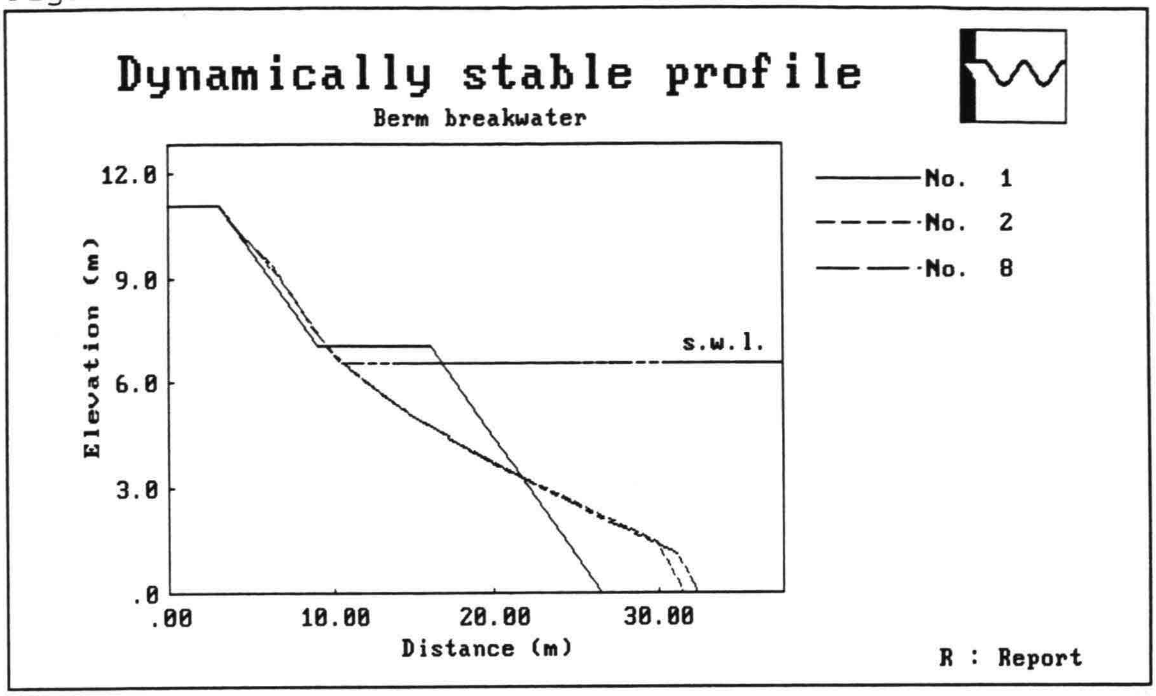

Figure 18

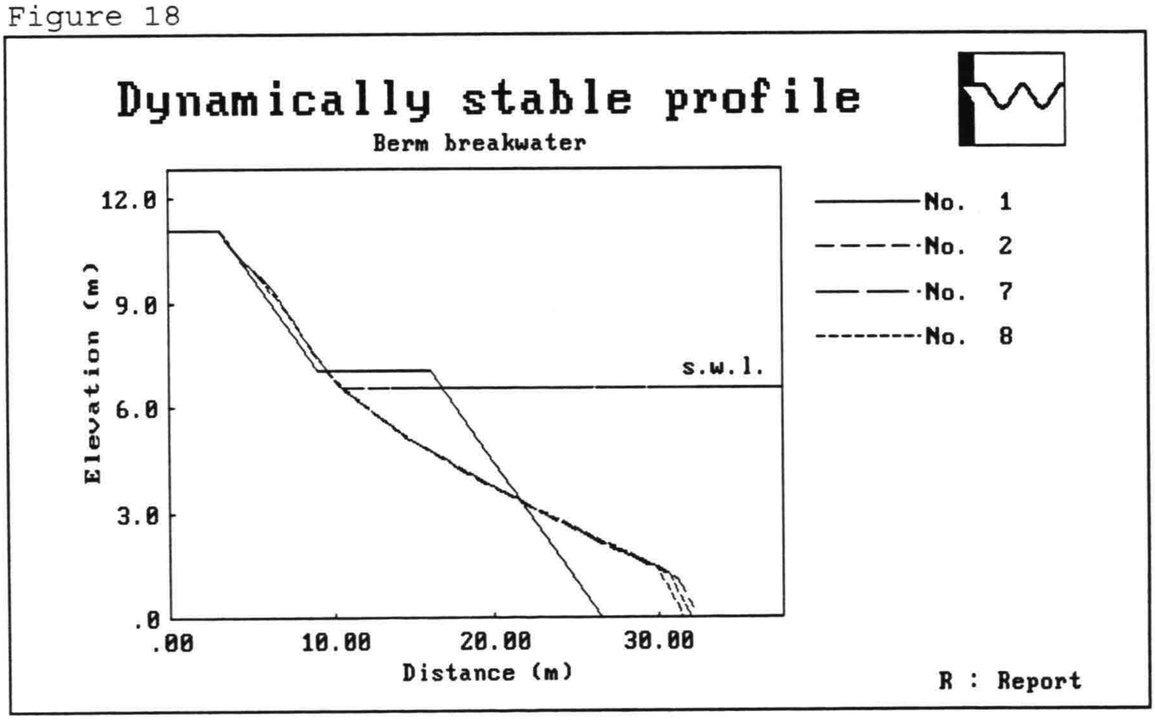

Figure 19

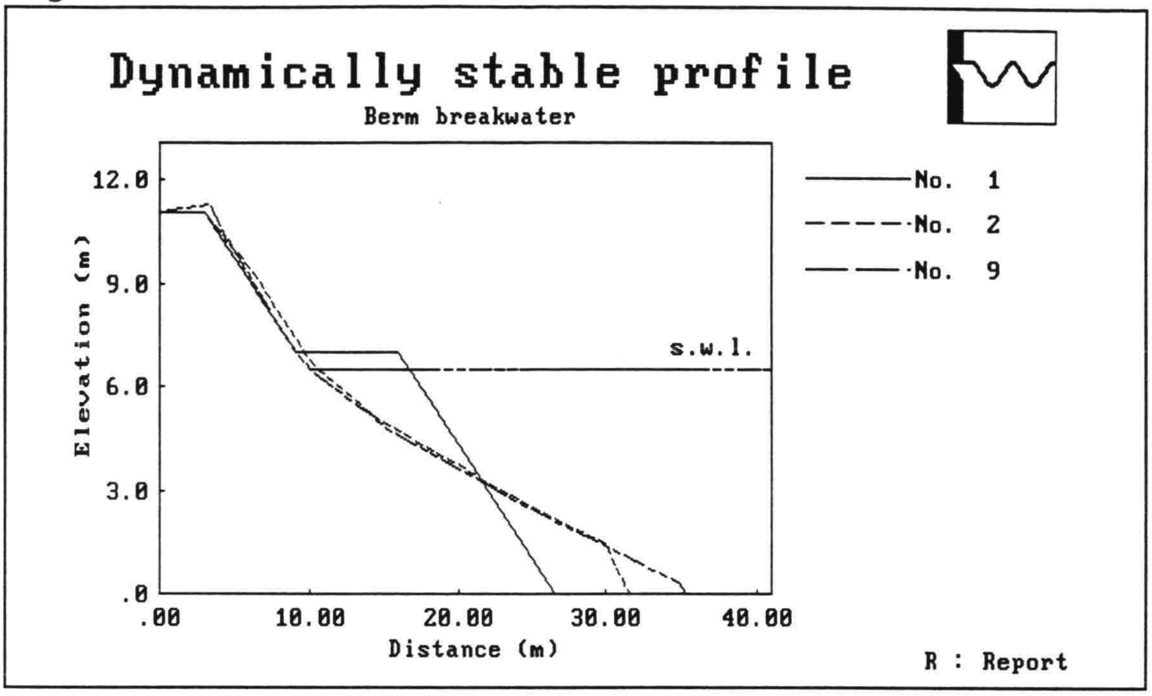

Figure 20