DESIGN OF SEAWALLS ALLOWING FOR WAVE OVERTOPPING

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ABSTRACT Based mainly on an extensive series of model tests the report presents design procedures to evaluate the overtopping discharge with irregular waves for existing seawalls, and to determine the seawall profile required for proposed new seawalls to restrict the overtopping discharge to a tolerable amount. The procedures are derived for simple and bermed seawalls of a generalised profile which can broadly be described as embankments; the methods are not applicable to complicated seawall geometries, such as those equipped with wave return walls for which specific model tests will still be required.

The report details the various parameters which are required to evaluate an existing seawall or design a new one, and outlines some of the methods available for determining such variables as design still water level, significant wave height, mean wave period, seawall roughness and allowable overtopping discharge. Worked examples are included at each stage to illustrate the techniques involved.

This design report was commissioned jointly by the Central Electricity Generating Board, the Severn-Trent Water Authority, the Wessex Water Authority, and the Hydraulics Research Station.

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INTRODUCTION

During 1978 and 1979 an extensive series of model tests was carried out at the Hydraulics Research Station to determine the overtopping discharges for a range of seawall designs subjected to different wave climates. The seawalls were all of the generalised profile shown in Figure 1, consisting essentially of a flat-topped embankment fronted in some cases by a flat berm. The seawalls were subjected to random waves, and the tests were designed to determine the effects on overtopping discharge of angle of wave attack, seawall slope, berm and crest elevations, berm width, and wave steepness. A detailed description of the test procedures, together with tabulations of measured data, and discussion of results obtained is given in a separate report[1]. The purpose of this present report is to publish the results of those and other studies in such a way that the design engineer can use the data either to calculate the expected overtopping discharge over an existing seawall, or to design a new seawall of the same generalised profile.

The studies were financed jointly by the Central Electricity Generating Board, the Severn-Trent Water Authority, the Wessex Water Authority, and the Hydraulics Research Station. The range of parameters for model testing was therefore selected to cover most of the situations likely to be encountered by these authorities. Great care should therefore be taken in extrapolation of the results beyond the ranges tested, unless some specific guidance is included in the text on the extrapolation of particular parameters.

SELECTION OF DESIGN PARAMETERS

Seawalls are generally assessed on their performance under a given design storm, i.e. for given tidal and wave conditions. The parameters which are required to evaluate the overtopping discharge of a given design of seawall are:-

The design still water level, SWL

The significant wave height, H_s

The mean zero-crossing period of the wave train, $\bar{\bar{T}}$

The predominant wave direction

It is not the purpose of this report to describe in detail how each of these should be evaluated, since this can be a very complex subject which could alone form the basis of a complete report. However some guidance on the methods available is worthwhile.

Design still water level

There are now a large number of tide recording stations around the coastline of the United Kingdom producing records which are sufficiently reliable for detailed analysis. From this data it is usually possible to calculate, for example, the 100 year High Water Level, i.e. the High Water Level which is expected to occur on average for once in every 100 years. There are at present two established methods of carrying out this calculation — the methods of Annual Maxima and of Surge Residuals.

(a) Annual Maxima. This is the more widely used method, and involves the selection of the annual maximum level for each year, i.e. the highest water level recorded during that year. From the frequency of occurrence of each annual maximum water level a probability distribution is fitted to the data, and extrapolated to the required return period. The work by Suthons[2] and more recently Blackman and Graff[3] are examples of this method. The calculations can be carried out fairly quickly, but several decades of data are necessary to obtain a confident forecast of the 100 year water level for example, since each year's tide recording yields only one data value.

(b) Surge Residuals. This method is mostly used when there is only a relatively short record of tidal heights in the locality, although it can be used with any record length. For every tide during the record period a comparison is made between the recorded and predicted high water levels. The difference is termed the High Water Surge Residual, and probability distributions are fitted to these surge residuals, and also to the predicted tidal heights. The two individual probability distributions are then re-combined to give the joint probability distribution for the total water level, i.e. predicted tide plus surge level. To achieve this an assumption has to be made about the correlation between tidal height and surge residual, which is the main disadvantage of the method. However the assumption made can be checked fairly easily against the recorded data, and it is generally found that the occurrence of given surge residuals is independent of tidal height. Apart from its requirement for less data than the method of Annual Maxima (since 700 data points are generated by each year's tidal records) the second method has the further advantage that it clearly separates out the regular astronomical effects (tides) from random meteorological effects (surges). As we shall see later, it is relatively easy to include further effects, such as for example the occurrence of waves. The paper by Ackers and Ruxton(4) gives an example of the application of the Surge Residuals method.

Significant wave height

The best method of obtaining the significant wave height is obviously to set up a wave recording system, either a Waverider buoy or a pressure transducer, at the site of the proposed seawall, and to monitor wave heights for as long a period as possible. The prediction of the design significant wave height is then fairly simply obtained by noting the frequency of occurrence of given wave heights, fitting a probability distribution to the data obtained, and then extrapolating to the return period required. A recent HRS Report(5) on wave heights recorded in the Severn Estuary gives a good example of this approach. Unfortunately however there are very few localities where such detailed wave measurements are available. In these cases therefore it is necessary to calculate the wave heights.

Waves can be broadly divided into two categories — locally generated wind waves, and distantly generated swell waves. The height of locally generated waves depends on the wind speed, the duration for which it has been blowing, the effective fetch length over which it has been blowing, and the average depth of water over that fetch. The height of swell waves depends on all these parameters measured at the distant location at which the waves are generated and also at all intermediate points on their route to the study site. The prediction of swell waves is therefore a mammoth undertaking requiring a large numerical model of the sea or ocean, such as the NORSWAM(6) model, or that developed by the UK Meteorological Office(7). The prediction of swell waves will therefore not be discussed further in this report.

Locally generated waves are of two types — fetch limited or duration limited. Fetch limited waves are governed by the wind speed and the fetch length, no matter how long the wind continues to blow. Duration limited waves are governed by wind speed and wind direction, no matter how long the fetch length is. In order to determine which of these applies at a particular location it is first of all necessary to calculate the effective fetch length for each wind direction. The method for this calculation is based on that given in the Shore Protection Manual(8), and an example is given in Fig 2. For a given wind direction radials are drawn from the point of interest at angular increments of 7.5° within ±45° of the wind direction. These lines are extended until they first intersect the shoreline, and the length of each line is measured in the direction parallel to the wind: this distance is then multiplied by the cosine of the angle between the ray and the wind direction. The resulting values are then summed over all radials and divided by the sum of the cosines to give the effective fetch length.

Using standard forecasting curves, such as those given in the Shore Protection Manual for example, the wave heights are then calculated for a given windspeed and either (a) the given fetch length or (b) the wind duration. If the wave height for the given fetch is lower than for the given duration, then the waves are fetch-limited. If the reverse is true, then the waves are duration limited. Alternatively Fig 3 can be used. For the given windspeed, the values of the dimensionless fetch length gF/U^2 and dimensionless duration gt/U are calculated, where F is the fetch length, t is the wind duration, and U is the windspeed measured at 10m above water level.

The values of gF/U^2 and gt/U are located on the horizontal axes of Fig 3. If the value of gt/U is to the right of gF/U^2 then waves are limited in size by the length of fetch F. If on the other hand gt/U lies to the left of gF/U^2 the waves will be duration limited.

In order to obtain the wave height we also need to know the water depth over the fetch area. This is obtained by estimation from hydrographic charts, averaging the water depth within the quadrant covered by the radials drawn for the fetch length calculation. The water depth will of course depend on the tidal stage. However since interest is mainly in the wave conditions at or near High Tide, a water level equal to Mean High Water Springs is generally used.

Armed now with the wind speed, direction, water depth and either fetch-length or wind duration, depending on whether the waves are fetch or duration limited, the wave heights can be read off standard shallow-water wave forecasting curves, such as the series given in the Shore Protection Manual(8). Alternatively use can be made of the universal dimensionless wave forecasting curves reproduced in Fig 3. The dimensionless fetch gF/U^2 and dimensionless duration gt/U have already been calculated: the dimensionless water depth gd/U^2 is now found for the given water depth d and windspeed U. For fetch-limited waves a vertical line is drawn through the calculated value of gF/U^2 until it intersects the line of calculated gd/U^2. The dimensionless wave height gH_s/U^2 is then read off the vertical axis, and converted back to give the significant wave height H_s. For duration limited waves a vertical line is drawn through the value of gt/U until it intersects the value of gd/U^2, and as before the corresponding value of gH_s/U^2 is read off the vertical axis.

The foregoing method enables wave heights to be calculated when the wind speed, direction and duration are known. However in order to extend the wave predictions to long return periods some knowledge of the frequency of occurrence of these given wind conditions must be available. Fortunately there are many locations around the United Kingdom where accurate wind measurements have been obtained for many years. Tables giving the recorded frequency of occurrence of given wind speeds, directions and durations can usually be obtained from the Meteorological Office. For each class interval used in these tables the resulting wave height can be calculated as above, and the percentage occurrence of a given wave height can be obtained by summing the percentage occurrences of all possible wind speeds/directions/durations which can give rise to this wave height. A probability distribution of the Weibull(5) or Fisher Tippett(5) type is then fitted to the wave height occurrences, and then extrapolated to more extreme return periods. For situations where the waves are mostly fetch-limited these calculations can be accomplished fairly easily, as in the case of the upper parts of the Severn Estuary for example(9). However for duration limited waves the calculations can become extremely complex, and other simplified methods are usually chosen. Perhaps the simplest of these is to examine all past wind records during major storms, and to estimate the worst possible wind speed and duration which could occur with a given return period for each onshore wind direction. The corresponding wave heights are then calculated for each direction, and all values are retained for future reference. (It could be that at a particular location a wave height of say 4m strking the seawall at an angle of 15° gives greater overtopping than a wave height of say 5m striking the seawall head on. The largest wave does not necessarily therefore give the worst case).

Mean zero-crossing wave period

The determination of the mean zero crossing period is closely associated with the determination of wave heights. Ideally both should be obtained from lengthy wave recording at the particular site. There are two methods by which the wave period for extreme waves can be estimated from these records. The first of these is identical to the determination of wave heights — the frequency of occurrence of given wave periods is noted, and a probability distribution fitted to the data. This probability distribution is then extrapolated to obtain the wave period for a given storm return period. However even if the significant wave height and the mean zero-crossing wave period are each determined for the same return period it does not necessarily imply that they occur simultaneously. The second method therefore examines the joint distribution of wave height and period. The recorded wave data is plotted as a scatter diagram as shown in Fig 4. Within each category of wave heights and periods the figure indicates the number of records having those wave heights and periods. For example, Fig 4 shows that at this particular site there were 74 occasions when the recorded waves had a significant height between 1.0 and 1.25m coupled with a mean zero-crossing period of between 4 and 5 seconds. From this scatter diagram it can be seen that for the higher waves in particular the wave records are grouped together forming a fairly well defined band of data. Onto these scatter diagrams are plotted lines of constant wave steepness, S, where S is defined as the ratio of significant wave height to mean deepwater wavelength, $H_s/\bar{L}_o$. This is done by plotting the equation $H_s = S\bar{L}_o = S \cdot g\bar{T}^2/2\pi$ for different assumed values of S. In Fig 4 this equation has been plotted for S = 0.035, 0.045, 0.055 and 0.065. After plotting these lines the steepness values which define the band of data are noted: in this example the data, for the higher wave heights at least, is banded by the lines S = 0.065 and S = 0.035. When the significant wave height has been determined for a given return period from the probability distribution, the mean wavelength corresponding to these two steepnesses is then calculated from $S = H_s/\bar{L}_o$. The corresponding mean zero-crossing wave periods are then calculated from $\bar{L}_o = g\bar{T}^2/2\pi$. For the particular data shown in Fig 4 for example the probability distribution yields a 5 year significant wave height of 3.04m. With steepness values between 0.035 and 0.065 the 5 year wavelength therefore varies between 87 and 47m, giving corresponding wave periods between 7.5 and 5.5 seconds. A probability distribution fitted directly to the recorded wave periods yielded 7.2s, which is within the range produced by the steepness method. Even with recorded wave data therefore it is impossible to determine a unique wave period for a given significant wave height — instead a range of periods is obtained. Since overtopping discharge is a function of wave period, overtopping calculations should be carried out for the two wave periods at the extremes of the possible range: generally the longer wave period will give the greater overtopping.

When no wave measurements are available, then wave periods have to be calculated in much the same way as wave heights. The effective fetch length and the average water depth for the site are calculated, and the wind speed and duration are noted. After determining as before whether the waves are fetch-limited or duration limited, the wave period is read off standard forecasting curves for the required windspeed, duration or fetch, and water depth. Alternatively the value of $g\bar{T}/(2\pi U)$ can be found from Fig 3 for the calculated values of gF/U^2 or gt/U, and gd/U^2. Most wave forecasting methods will give a value of wave period such that the wave steepness S is uniquely defined, and most probably lies within the range 0.05 to 0.06.

To obtain the wave period for more extreme events the tables of frequencies of occurrence of given windspeed, direction and duration are again used. For each class interval in these tables the resulting wave period is calculated and the percentage occurrence of a given wave period can be obtained by summing the percentage occurrences of all possible windspeeds/directions/durations which can give rise to the wave period. A probability distribution is then fitted to this data, and extrapolated to the required return period. As before, these calculations can be accomplished fairly easily when the waves are mostly fetch limited, since in this case the wave height and period are very closely related. However, for situations where the waves are mostly duration-limited the calculations are very much more complicated. In these cases simplified methods are usually taken, as described in the previous paragraphs for determining wave heights in this situation. A range of wave periods will then be obtained, and each of these retained for future calculation of overtopping discharges.

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Example Problem 1 Examination of the charts shows that at Frampton-on-Severn the longest fetch occurs with winds from direction 240°N, and calculations give an effective fetch length of 3415m (see Figure 2). At Mean High Water Springs the average depth of water over the fetch area is about 8 metres. Calculate the significant wave height and mean period for a Force 9 gale, direction 240°N, blowing for 2 hours.

Solution: A force 9 gale on the Beaufort scale covers windspeeds in the range 21 to 24m/s, so take a mid-range windspeed of 22.5m/s.

The dimensionless fetch gF/U^2 is therefore

$g \times 3415/22.5^2$

or

$gF/U^2 = 66.18$

The dimensionless duration $gt/U = g \times 2.3600/22.5 = 3139.2$

The dimensionless water depth $gd/U^2 = g \times 8/22.5^2 = 0.1550$

Turning to Fig 3 and plotting $gF/U^2 = 66.18$ and $gt/U = 3139.2$, we find that gt/U lies well to the right of gF/U^2 and the waves are therefore fetch-limited. To obtain the significant wave height we therefore enter Fig 3 with $gF/U^2 = 66.18$, and $gd/U^2 = 0.1550$, and read off $gH_s/U^2 = 1.83 \times 10^{-2}$. We therefore have

$$H_s = 1.83 \times 10^{-2} \times \frac{U^2}{g} = 0.946\text{m}$$

Similarly, with $gF/U^2 = 66.18$ and $gd/U^2 = 0.155$ we read off $g\overline{T}/(2\pi U) = 0.236$.

$$\overline{T} = 0.236 \times \frac{2\pi U}{g} = 3.40\text{s}$$

* *

Predominant wave direction With winds blowing from a given direction waves will be generated within a sector of about ±45° of that direction. However in deep water the predominant direction of locally generated waves will closely follow the wind direction. At most coastal sites it is usually fairly obvious from which offshore direction the worst waves are likely to occur, either because the fetch length is significantly greatest in that direction, or because the strong winds most frequently blow from that direction. If this offshore wave direction makes an angle of more than about 10° with the normal to the seabed contours then the wave direction at the shoreline is likely to be significantly affected by refraction. The calculation of the inshore direction is described in a later chapter.

EVALUATION OF SITE CHARACTERISTICS

The overtopping discharge of either an existing or projected seawall will depend on the characteristics of the site; in particular the depth of water at the toe of the seawall, the foreshore gradient, and the dimensions of any naturally occurring berm all affect the height of the wave as it strikes the seawall.

Depth of water at toe

The foreshore level at the toe of the existing or projected seawall can be easily obtained by carrying out a survey. The depth of water at the toe is then obtained by subtraction of the foreshore level from the design Still Water Level. If the seawall includes a berm, then the toe of the seawall is defined by the toe of the berm.

Foreshore gradient

The seabed profile should be surveyed from the seawall either down to the point where the water depth is equal to about twice the expected significant wave height, or for a distance offshore which is equal to the expected deepwater wave length, whichever gives the least distance to be surveyed (see next chapter for calculation of deepwater wave length). The mean gradient of the foreshore will be calculated at a later stage when the exact design wave period and water depth are known.

Berm dimensions

At many sites the seawall will be fronted by saltings or mudflats which in effect form a natural berm. The two important dimensions of a berm are its width, and the depth of water over such a berm at the design water level. Generally the surface of natural saltings is very nearly horizontal: the water depth is therefore simply obtained by subtracting the general level of the saltings from the design water level. The width of the saltings is measured from the seawall to the outer edge of the saltings, in a direction perpendicular to the seawall. Many saltings terminate at a near-vertical face, and the seaward edge is therefore easily defined. At other locations however the saltings may gradually merge with a beach or foreshore: in these cases the seaward edge of the berm is defined as the approximate point at which the surface ceases to be horizontal.

PRELIMINARY CALCULATIONS

The section on selection of design parameters dealt with the evaluation of such items as the design still water level, the design significant wave height, the design mean zero-crossing wave period and the predominant wave direction. Each of these parameters are essentially offshore values, applicable over a fairly wide area. The inshore wave heights and directions however are likely to be modified significantly depending on the seabed contours, the design wave period, and on the water depth at the toe of the wall. Ideally the inshore wave conditions should be determined by the application of a numerical wave refraction model[10,11]. In simple situations, however, particularly where the seabed contours are relatively straight and parallel, the approximate inshore wave conditions can be determined using theoretical methods derived for regular waves. Firstly, however, certain additional wave characteristics need to be determined.

Deepwater wave length

The mean wave length in deep water, i.e. the average distance between successive wave crests, is obtained from the design mean zero-crossing wave period by the equation

$$\overline{L}_o = g\overline{T}^2/2\pi$$

Deepwater steepness

The deepwater steepness is defined as the ratio of the design significant wave height to the calculated wavelength, i.e. $S_o = H_s/\overline{L}_o$. Generally S_o will lie between about 0.03 and 0.07. A value higher than this probably indicates a miscalculation at some stage. A lower value indicates that the waves have been generated over a considerable distance, and are probably distantly generated rather than locally-generated waves.

Wavelength at seawall toe The wavelength in a depth of water equal to the design water depth at the seawall toe is given by the equation

$$\overline{L}_s = \overline{L}_o \tanh \left(\frac{2\pi d_s}{\overline{L}_s}\right)$$

However this is not a particularly convenient equation, and the value of wavelength may instead be determined from Figure 5 for a given water depth and wave period.

Wavelength on the berm If there is a berm, then the wavelength on the berm is given by $\overline{L}_B = \overline{L}_o \tanh (2\pi d_B/\overline{L}_B)$ or obtained from Figure 5.

Wave refraction and shoaling As mentioned earlier, the following simplified method of calculating wave refraction and shoaling should only be used as a very approximate guide to the inshore wave conditions, and should not be used at all if the seabed topography is complex.

Let d_g be the water depth in the wave generating area, H_{sg} the significant wave height in the generating area, $\overline{T}$ the mean wave period, and α_g the angle in the generating area between the normal to the seabed contours and the direction of wave travel. The first step is to check whether the generating area can be considered as deep water. If the ratio of the water depth to the deepwater wavelength $d_g/\overline{L}_o > 0.5$ then the generating area is in deepwater: the deepwater wave angle is then given by $\alpha_o = \alpha_g$, and deepwater wave height $H_{so} = H_{sg}$. If $d_g/\overline{L}_o < 0.5$ then the generating area constitutes transitional or shallow water: in this case Fig 6 is used to estimate the equivalent deepwater conditions. Using d_g and $\overline{T}$ one can calculate the value of $d/g\overline{T}^2 = d_g/g\overline{T}^2$. Using this and $\alpha = \alpha_g$ the corresponding point is found on Fig 6. The value of α_o is then read off the vertical axis, and the wave height coefficient K_RK_S is interpolated between the lines of constant K_RK_S. The equivalent deepwater wave height is then given by

$$H_{so} = H_{sg}/(K_RK_S)$$

Having thus established the actual or equivalent deepwater wave angle α_o and significant wave height H_{so} the next step is to calculate the angle of wave attack and the wave height at the seawall. Using the values of $d_s/g\overline{T}^2$ and of α_o the inshore wave angle α_i and the wave height coefficient K_RK_S are read from Fig 6. The inshore wave height is then obtained from $H_{si} = K_RK_SH_{so}$. The inshore wave angle is the angle between the direction of wave travel and the normal to the foreshore contours. Knowing the angle between the seawall and the foreshore contours the angle at which the waves hit the seawall, β, can thus be determined.

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Example Problem 2 Waves with a significant height 2.5m and mean period 7.0s are generated over an area where the average water depth is 8.0m. If the waves make an angle of 30° with the normal to the contours in the generating area, what will be their height and direction in a water depth of 2.0m?

Solution: The deepwater wavelength is given by

$$\overline{L}_o = \frac{g\overline{T}^2}{2\pi} = \frac{g \times 7^2}{2\pi} = 76.5\text{m}$$

The ratio $d_g/\overline{L}_o$ is therefore $8.0/76.5 = 0.105$, which is significantly less than 0.5. The generating area cannot therefore be considered as deepwater. The value of $d_g/g\overline{T}^2$ is equal to $8.0/(9.81 \times 7.0^2) = 0.017$. Entering Fig 6, mark this off and draw a vertical line to intersect the curve $\alpha = 30°$. At this intersection read off the value of the equivalent deepwater wave angle from the vertical scale, giving $\alpha_o = 43.5°$. Also at the intersection read off the value of K_RK_S, in this case interpolating between the contours $K_RK_S = 0.80$ and $K_RK_S = 0.85$, giving a value just under 0.85. The equivalent offshore wave height is therefore given by

$$H_{so} = H_{sg}/(K_R K_S)$$
$$= 2.5/0.85$$
$$= 2.94\text{m}$$

At this inshore site, $d_s/g\bar{T}^2 = 2.0/(9.81 \times 7.0^2) = 0.004$

Re-entering Fig 6 with this value and $\alpha_o = 43.5°$ we read off by interpolation between the lines α = constant that $\alpha_i = 16°$, and similarly $K_R K_S = 1.00$. The inshore wave height is therefore given by $H_{si} = K_R K_S H_{so} = 2.94 \times 1.00 = 2.94$m, and the waves make an angle of 16° with the normal to the foreshore contours.

* *

Breaking wave height

The design wave height is determined on the assumption that no wave breaking occurs. However it may not be possible for that design wave height to reach the seawall before breaking: in that case the overtopping at the seawall will be less than expected. In the following paragraphs a method is described whereby the design wave height is replaced by an equivalent post-breaking wave height H_{sb} which can then be used in all subsequent overtopping discharge calculations. It should be noted that H_{sb} is not necessarily the wave height which would be obtained by direct measurement. It is an *equivalent* wave height designed to give the correct overtopping discharge as confirmed from those tests where significant wave breaking took place.

To check for breaking the first step is to evaluate the ratio of the design wave height to the water depth, H_{si}/d_s. Next, for the given value of the foreshore gradient m_f and the previously calculated ratio $d_s/g\bar{T}^2$ the value of the breaking wave ratio γ is determined from Figure 7. In this context γ is defined as the ratio of the equivalent post-breaking wave height to the water depth at the toe of the seawall, i.e. H_{sb}/d_s.

If the calculated design ratio H_{si}/d_s is less than the breaking ratio γ then the design wave height can indeed strike the seawall. However if the design ratio is greater than the breaking ratio then the wave height is limited by breaking. The new design wave height at the seawall is therefore given by

$$H_{sb} = \gamma\, d_s$$

where γ is the breaking ratio as determined from Figure 7. It is assumed that the mean zero-crossing wave period is unaffected by breaking.

In carrying out this calculation of wave breaking it is necessary to examine closely the definition of the foreshore gradient, m_f. Since waves generally take about 1 wavelength to complete the breaking process, then m_f should ideally be defined as the mean gradient of the foreshore over a distance of 1.0 $\bar{L}_s$ immediately approaching the seawall, where $\bar{L}_s$ is the mean wavelength at the seawall toe. In situations where the foreshore is markedly convex upwards this will probably give a slight overestimate of the wave height after breaking. Where the foreshore is markedly concave there will probably be an underestimate of the broken wave height, and in this case it may be worth re-calculating the foreshore slope over a distance of 0.5 $\bar{L}_s$, and re-working the broken wave height to determine its sensitivity to the assumed foreshore slope. In situations showing marked sensitivity it may well be necessary to carry out a short series of laboratory studies to determine exactly the broken wave height.

* *

Example Problem 3 At a specific site for a new seawall a topographic survey has shown the following water depths over the foreshore:-

Distance from seawall toe	Water depth at design SWL
m	m
0	2.0
10	2.4
20	3.1
30	3.5
40	3.8

Under design storm conditions, waves with a significant height of 2.5m and mean period 7.0s are generated in a water depth of 8.0m, making an angle of 30° with the normal to the contours. What is the design wave height and direction at the seawall.

Solution: Example Problem 2 showed that after refraction and shoaling the inshore wave height in a water depth of 2.0m was 2.94m, and the waves make an angle of 16° with the normal to the foreshore contours. However the wave height is based on the assumption that no breaking occurs, and this must therefore be checked.

For a wave height of 2.94m and a water depth of 2.0m the ratio $H_{si}/d_s = 2.94/2.0 = 1.47$. This ratio must be compared with the breaking ratio γ obtained from Fig 7. First we need $d_s/g\overline{T}^2 = 2.0/(9.81 \times 7.0^2) = 0.0042$.

Secondly we need the foreshore gradient m_f. From Fig 5 we see that for a 7.0s wave the wavelength in 2.0m of water is 30.0m: we therefore evaluate the mean gradient of the foreshore between 0 and 30m off the toe of the seawall. This gradient is (3.5 — 2.0)/30, or 1:20. From Fig 7, with $d_s/g\overline{T}^2 = 0.0042$ and $m_f = 1{:}20$ we have $\gamma = 0.8$. This is significantly smaller than the ratio of incident wave height to water depth, $H_{si}/d_s = 1.47$, so the waves will certainly break. For the design of the seawall therefore we take the broken wave height

$$
\begin{aligned}
H_{sb} &= \gamma\, d_s \\
&= 0.8 \times 2.0 \\
&= 1.6\text{m}
\end{aligned}
$$

Since the foreshore is neither markedly convex nor concave, this is probably a reasonable estimate of the design wave height.

* *

Waves breaking on the berm

The calculation of the wave height which can occur on the berm is very similar to the foreshore calculations. Knowing the water depth over the berm at the design water level, d_B, the ratio $d_B/g\overline{T}^2$ can be calculated. With a berm gradient of 0 (horizontal) Figure 7 is entered to obtain the breaking wave height ratio γ. If the design ratio H_s/d_B is less than the breaking ratio then the design wave height can traverse the berm without breaking. If the design ratio is greater than the breaking ratio, then the design wave height will break on the berm. The wave height at the seawall will therefore be limited to the value $\gamma\, d_B$. However model tests have shown that the wave-breaking process on the berm occupies a distance of about one wavelength $\overline{L}_B$. In other words, if the berm width W_B is less than the wavelength $\overline{L}_B$ then the breaking process is not complete, and the wave height striking the seawall is greater than the broken wave.

OVERTOPPING OF EXISTING SEAWALLS

With few exceptions, the design of a new seawall is intended to alleviate flooding which occurs or is expected to occur with an existing system of sea defences. In order to make a valid judgement on the benefits to be obtained with the new seawall it is therefore first necessary to calculate the overtopping discharges under design storm conditions for the existing seawall. However it is worth emphasising that there is no such thing as an absolute discharge: because the wave heights and periods exhibit a random distribution about a given mean, the discharge will also vary randomly. All that can be calculated therefore is the expected mean discharge for a wave sequence having a given significant wave height and period, together with the expected standard deviation about that mean. From standard statistical tables it is then possible to find the range of values within which the discharge is expected to lie for a given percentage of the time. In the next few pages the main attention is devoted to the evaluation of the expected mean discharge: calculation of the statistical range of discharge values is dealt with later.

The first step of course is to carry out surveys and measurements of the existing wall, particularly with regard to the crest elevation and the shape of the seaward profile. The seawall will almost certainly fall into one of the four main types shown in Fig 8, namely vertical, simple, composite or bermed seawalls. Each of these types can also have either an essentially flat-topped crest, or they could be surmounted by a wave return wall. The present report is based entirely on tests carried out for simple and bermed seawalls with flat topped crests, i.e. seawall types 2 and 4 on Fig 8. However some guidance will be given on the assessment of overtopping discharges for composite seawalls based on published data of wave run up on composite slopes. Seawalls which are surmounted by wave return walls are so variable in geometry from site to site that it would be impossible to generalise their overtopping characteristics: for these seawalls the only practicable method of determining the overtopping of a specific seawall is by model testing.

Dimensionless freeboard and discharge

Throughout the remainder of this report the height of the seawall and the discharge which overtops that seawall will be expressed in terms of the dimensionless freeboard and the dimensionless discharge respectively. The dimensionless freeboard, R_*, is defined as

$$R_* = \frac{R_c}{\bar{T}\sqrt{gH_s}}$$

where

R_c is the crest elevation above still water level

$\bar{T}$ is the mean zero-crossing wave period

and

H_s is the significant wave height

The numerical value of R_* is small when a low seawall is attacked by large waves or by long period waves. The physical significance of R_* is perhaps best appreciated if it is re-written in the following form:

$$R_* = \frac{R_c}{H_s}\sqrt{\frac{S}{2\pi}}$$

where S is the wave steepness.

Substitution of $S = H_s/\bar{L}_o$ and $\bar{L}_o = g\bar{T}^2/(2\pi)$ in this last expression will confirm that these two definitions of R_* are identical. However the second definition indicates that for waves of constant steepness R_* is simply related to the ratio (seawall height/wave height).

The dimensionless discharge Q_* is defined as

$$Q_* = \frac{\bar{Q}}{\bar{T}gH_s}$$

where

$\bar{Q}$ is the mean overtopping discharge in terms of volume/unit time/unit length of seawall, eg $m^3/s/m$.

Again the physical significance of Q_* is perhaps best illustrated by re-writing it as

$$Q_* = \frac{\bar{Q}}{\sqrt{gH_s^3}} \cdot \sqrt{\frac{S}{2\pi}}$$

making the same substitutions for s and $\bar{L}_o$.

For waves propagating onto a beach or up a sloping seawall the peak discharge at the point of wavebreaking is given approximately by

$$Q_b = \frac{1}{2\sqrt{\gamma}} \cdot \sqrt{gH_s^3}$$

where

Q_b is the peak wave discharge at breaking, and

γ is the ratio of the wave height to the water depth at breaking.

It can be seen therefore that Q_* can be re-written as

$$Q_* = \frac{\bar{Q}}{Q_b} \cdot \frac{1}{2\sqrt{\gamma}} \cdot \sqrt{\frac{S}{2\pi}}$$

or in other words, for constant wave steepness S and breaking ratio γ the dimensionless discharge Q_* is simply related to the ratio (overtopping discharge/breaking wave discharge).

Simple seawalls

Simple seawalls are precisely defined by the values of

d_s the design water depth at the toe of the seawall

1:m the gradient (vertical:horizontal) of the seaward slope

R_c the crest elevation above design still water level.

In order to evaluate the overtopping discharge the design significant wave height H_s, mean zero crossing period $\bar{T}$, and angle of the wave crests to the seawall β are required.

From these values the dimensionless freeboard $R_c/(\bar{T}/\sqrt{gH_s})$ is calculated, and Figure 9 is entered with this and the measured seawall slope. The dimensionless discharge $Q/(\bar{T}gH_s)$ is then read off the vertical axis, and multiplied through by $\bar{T}gH_s$ to obtain the dimensional discharge $\bar{Q}$ in units of volume/unit time/unit length of seawall (in SI units $\bar{Q}$ is in $m^3/s/m$). Figure 9 gives the discharge for waves attacking the seawall at normal incidence: if the design waves make an angle with the seawall then the discharge has to be calculated using the methods described later.

In evaluating the overtopping discharge it is possible that the calculated dimensionless freeboard $R_c/(\bar{T}\sqrt{gH_s})$ will be such that the expected dimensionless discharge $\bar{Q}/(\bar{T}gH_s)$ will be less than 1.0×10^{-6}. If this is the case then extrapolation of the curves is possible: the dimensionless discharge is related to the dimensionless freeboard by an equation of the form

$$Q_* = Ae^{-BR_*}$$

where

$Q_* = \bar{Q}/(\bar{T}gH_s)$

$R_* = R_c/(\bar{T}\sqrt{gH_s})$

and A and B are constants depending on the seawall geometry. Table 1 gives the values of A and B for simple seawalls of various seaward slopes. For seawall slopes of 1:1, 1:2 and 1:4 the values of A and B were determined experimentally for the following range of parameters:-

$0.05 < R_* < 0.30$

$10^{-6} < Q_* < 10^{-2}$

$1.5 < d_s/H_s < 5.5$

$0.035 < H_s/\bar{L}_o < 0.055$

For the other seawall slopes the tabulated values of A and B are interpolations based on published data on the run up of waves on simple slopes of various gradients[8]. In using the constants tabulated in Table 1 it should therefore be remembered that the results are known to be accurate only for the range of model tests carried out: outside this range the equation should only be used to give a broad estimate of the expected overtopping discharge. In order to determine the constants at intermediate seawall slopes the values of A and B from Table 1 are plotted in Fig 10, and smooth curves drawn through the plotted points.

* *

Example Problem 4 An existing seawall has the following dimensions

Seawall slope	1:2
Crest elevation	+5.5m ODN

What is the overtopping discharge when

Tide level	+3.0m ODN
Significant wave height	1.75m
Mean wave period	5.0s

Solution: At this tide level, the crest elevation relative to still water level is 5.5 − 3.0 = 2.5m. The dimensionless freeboard is therefore

$$R_* = \frac{R_c}{\bar{T}\sqrt{gH_s}} = \frac{2.5}{5.0\sqrt{g.1.75}} = 0.12$$

Entering Fig 9, draw a vertical line through the seawall slope of 1:2 to intersect the line $R_* = 0.12$. At this intersection, read off the corresponding value of Q_* from the vertical axis. In this case $Q_* = 9.0 \times 10^{-4}$.

Alternatively we could enter Fig 10 with a seawall slope of 1:2 to obtain the values of the coefficients A and B in the equation $Q_* = Ae^{-BR_*}$. In this case $A = 1.26 \times 10^{-2}$, and $B = 22.1$. For the given value of dimensionless freeboard we therefore have

$$Q_* = A\,e^{-BR_*}$$
$$= (1.26 \times 10^{-2})\,e^{-22.1 \times 0.12}$$
$$= 8.9 \times 10^{-4}$$

giving close agreement with Fig 10.

To obtain the actual discharge we have

$$\bar{Q} = Q_*\,\bar{T}\,g\,H_s$$
$$= (9 \times 10^{-4}) \times 5.0 \times g \times 1.75$$
$$\bar{Q} = 7.7 \times 10^{-2}\ m^3/s/m. \text{ run of seawall.}$$

* *

Bermed seawalls In order to define a bermed seawall it is necessary to determine the values of:-

- d_s the design water depth at the toe of the seawall
- m_2 the slope of the seaward edge of the berm
- w_B the width of the berm (normal to the seawall)
- d_B the design water depth over the berm
- m_1 the slope of the seaward face of the seawall
- R_c the crest elevation above design still water level

All the tests on which this report is based were carried out with the slope of the seaward edge of the berm, m_2, equal to the seawall slope m_1. However it is believed that in most cases the exact value of m_2 is relatively unimportant in terms of the degree of overtopping, unless the berm elevation is above Still Water Level.

The model tests were carried out for three seawall slopes (1:1, 1:2 and 1:4) for the berm elevation and berm width combinations shown below

		Berm width: metres				
		5	*10*	*20*	*40*	*80*
	−4		x			
Berm	−2	x	x	x	x	x
elevation:	−1	x	x	x	x	x
m. SWL	0		x			

Figures 11 to 22 give the variation of overtopping discharge with seawall slope and crest elevation for each of the berm elevation/width combinations tabulated above. If by good fortune the dimensions of the seawall for which the overtopping discharge is required happen to coincide with one of the conditions tested, then the relevant figure from 11 to 22 is used. For example, if the seawall dimensions are a seawall slope of 1:2, a berm elevation of 1.0m below SWL, and a berm width of 20m, then Figure 18 would be used. From the wave height H_s, the wave period, $\bar{T}$, and the crest elevation R_c, the dimensionless freeboard $R_c/(\bar{T}\sqrt{gH_s})$ is calculated. Entering the Figure with the given seawall slope and the calculated dimensionless freeboard, the dimensionless discharge $\bar{Q}/(\bar{T}gH_s)$ can then be obtained from the vertical scale. The actual discharge is then calculated by multiplying through by $\bar{T}gH_s$.

However in most cases the actual seawall will probably have dimensions which lie between those values tested: in these cases some method of interpolation is required. As mentioned earlier, the test results showed that the dimensionless discharge for a given seawall is related to the dimensionless freeboard by an equation of the type

$$Q_* = A\,e^{-BR_*}$$

where A and B are constants depending on the seawall geometry. Figures 23 to 26 have been produced from the model results to show the variations of these constants with seawall slope: each Figure gives the values of A and B for a fixed berm elevation, and on each figure curves are given for different berm widths, including a berm width of zero, ie a simple seawall. For example, Figures 24(a) and (b) respectively give the values of A and B for a berm elevation of 1m below Still Water Level, with berm widths of 0, 5, 10, 20, 40 and 80m. In these Figures the plotted points are derived from the model tests for the seawall slopes, berm elevations and berm widths tabulated above, and under the following range of test parameters:

10m berm,

$0.05 < R_* < 0.30$

$10^{-6} < Q_* < 10^{-2}$

$1.5 < d_s/H_s < 5.5$

$0.035 < H_s/\bar{L}_o < 0.055$

5, 20, 40 and 80m berms,

$0.10 < R_* < 0.20$

$10^{-6} < Q_* < 10^{-2}$

$1.75 < d_s/H_s < 3.5$

$0.035 < H_s/\bar{L}_o < 0.055$

The plotted points are connected by solid lines: dashed lines indicate values of A and B derived by interpolation from the results of the model tests. Table 2 lists the values of A and B derived from the model tests, for bermed seawalls.

From Figures 23 to 26 interpolation between standard berm widths for a standard berm elevation is simply achieved. For example Figures 24(a) and (b) show that for a seawall slope of 1:2 with a 10m wide berm at 1m below Still Water Level the values of A and B are 3.4 x 10^{-2} and 53.2 respectively. If the berm width was instead 8m, then interpolation between the lines for 5 and 10m berms gives 2.7 x 10^{-2} and 47.0 respectively. For a seawall slope of 1:2, a berm elevation of 1m below Still Water Level and a berm width of 8m the dimensionless discharge is thus given by the equation

$$Q_* = (2.7 \times 10^{-2})\, e^{-47.0R_*}$$

When the berm elevation differs from a standard value tested, interpolation is necessary between two pairs of Figures where berm elevations span the required elevation. For example if a berm elevation of 1.5m is required, it is necessary to interpolate for A between Figures 24(a) and 25 (a), and for B between Figurs 24(b) and 25(b). For B the interpolation is straightforward and linear. For A, the interpolation is linear, but has to be based on lnA.

To illustrate the way in which Figures 23 to 26 are interpolated consider the following example.

* *

Example Problem 5

Given:	Seawall slope	1:2.2
	Crest elevation	+5.5m ODN
	Berm elevation	+1.7m ODN
	Berm width	8.0m
Find:	Overtopping discharge when:	
	tide level =	+3.0m ODN
	significant wave height H_s =	1.75m
	mean wave period $\overline{T}$ =	5.0s
Solution:	Berm elevation relative to still water level =	+3.0m ODN − 1.7m ODN
	=	1.3m below SWL

Interpolation is therefore necessary between Figures 24 (for −1m) and 25 (for −2m).

Entering Figs 24(a) and 25(a) with a seawall slope of 1:2.2, and interpolating linearly between the curves for berm widths of 5 and 10m, we obtain by interpolation the following values of A at a berm width of 8m:-

Berm elevation −1.0m, $A = 2.85 \times 10^{-2}$, $\ln A = -3.56$

−2.0m, $A = 8.2 \times 10^{-3}$, $\ln A = -4.80$

By linear interpolation on ln A, we therefore have

$$\ln A_{1.3} = \ln A_{1.0} + \frac{(1.3 - 1.0)}{(2.0 - 1.0)} (\ln A_{2.0} - \ln A_{1.0})$$

$$= \ln (2.85 \times 10^{-2}) + 0.3 (\ln 8.2 \times 10^{-3} - \ln 2.85 \times 10^{-2})$$

giving $\ln A_{1.3} = -3.93$

$A_{1.3} = 1.96 \times 10^{-2}$

To obtain the value of B, enter Figs 24(b) and 25(b) with a seawall slope of 1:2.2, and interpolating between berm widths of 5 and 10m, we have for a berm width of 8m for the berm elevations

−1.0m, B = 49.3

−2.0m, B = 26.0

In this case, straightforward linear interpolation gives

$$B_{1.3} = B_{1.0} + \frac{(1.3 - 1.0)}{(2.0 - 1.0)} (B_{2.0} - B_{1.0})$$

giving $B_{1.3} = 42.3$

For the given seawall we therefore have the equation

$$Q_* = (1.96 \times 10^{-2})\, e^{-42.3\, R_*}$$

For the given tide level, the seawall crest elevation R_c relative to Still Water Level is 5.5m ODN − 3.0m ODN = 2.5m. The dimensionless freeboard

$$R_* = R_c/(\bar{T}\sqrt{gH_s})$$
$$= 2.5/(5.0\sqrt{g \times 1.75}) = 0.12$$

Using equation $Q_* = A\, e^{-BR_*}$

$$Q_* = (1.96 \times 10^{-2})\, e^{-42.3 \times 0.12}$$

giving $Q_* = 1.22 \times 10^{-4}$

Since $Q_* = \bar{Q}/(\bar{T}gH_s)$, we have

$$\bar{Q} = Q_* \bar{T} g H_s$$
$$= (1.22 \times 10^{-4}) \times 5.0 \times g \times 1.75$$

or $\bar{Q} = 1.05 \times 10^{-2}\ m^3/s$ per metre run of seawall.

* *

As mentioned earlier, the widest berm which was tested in the model studies was 80m: if the natural berm is wider than this the the best solution is to take the 80m values of A and B. This method will give a conservative answer, ie an overestimate of overtopping discharge, since the model studies showed that discharge was still decreasing as berm width increased, even at 80m.

In past methods of designing seawalls with a wide berm it has sometimes been the practice to calculate the breaking wave height on the berm, and apply this height to the simple seawall backing the berm. The tests on which this report is largely based covered a wide range of berm width/wavelength ratios, and from the results it was apparent that the wave breaking process occupied a distance of just under one wavelength on the berm, $\bar{L}_B$. For berm widths less than one wavelength ($W_B < \bar{L}_B$) the effective wave height at the backing seawall was somewhere between the incident wave height and the broken wave height. Former practice would therefore yield an underestimate of overtopping discharge, and should thus not be used. For berm widths greater than one wavelength ($W_B > \bar{L}_B$) the effective wave height started at a value approximately equal to the breaking wave height, but gradually reduced as berm width increased, due to frictional losses and wave-wave interaction on the berm. For these berm widths therefore the assumption of breaking wave height yields an overestimate of overtopping discharge, with the error increasing at greater berm widths. Although this gives a conservative design of seawall, it is preferable to use the values of overtopping discharge derived from the relevant parts of Figures 11 to 26.

Composite seawalls

In order to define a composite seawall it is necessary to evaluate:-

- d_s the design water depth at the toe of the seawall
- m_1 the slope of the upper portion of the seawall
- d_r the water depth at which the seawall slope changes (d_r can be negative if the change in slope occurs above the design still water level)
- m_2 the slope of the lower portion of the seawall
- R_c the crest elevation above design still water level

Unfortunately there is very little experimental data on the run-up and overtopping of composite seawalls. As far as is known, the only systematic tests were a series carried out by Saville [12] in the mid 1950s where the run-up and overtopping discharges were measured for a range of wave heights, wave periods and crest elevations. Unfortunately however all the experiments were carried out with regular or mono-chromatic waves, with a lower slope of 1:3 and an upper slope of 1:6, and with the slope change located at design still water level (ie $d_r = 0$). Plotting Saville's results against seawall crest elevation shows that the run-up and the overtopping discharge on the composite 1:3/1:6 seawall were intermediate between the results for the 1:3 seawall and 1:6 seawall respectively. In other words, the results showed that the composite seawall was in effect behaving like a simple seawall of slope somewhere between 1:3 and 1:6. Based on these results, and on similar tests with bermed seawalls, Saville[13] proposed the method which is now well known for evaluating the run-up of regular waves on composite and bermed seawalls. Later, the Hydraulics Research Station extended this method to obtain the theoretical run-up of irregular waves on composite and bermed seawalls[14]. Unfortunately however no similar methods have evolved for the overtopping discharge on such seawalls, even for regular waves. In addition, it is not possible to calculate the overtopping discharge from the run-up predictions because there is no means of determining the volume of water carried over the seawall crest by an overtopping wave. Expressing this mathematically, it can be shown that for seawalls the dimensionless overtopping discharge can be expressed either as

$$Q_* = Ae^{-BR_*}$$

or as

$$Q_* = f(N),$$

where N is the number of waves which overtop the seawall, as a proportion of the number of waves incident upon the wall. However, the exact form of the function f(N) changes with seawall geometry. For composite seawalls therefore, even if the value of N can be determined from run-up calculations, there is no way of knowing whether the function f(N) should relate to the lower slope, the upper slope, or some intermediate value. Until comprehensive series of model tests are carried out with irregular waves for a wide variety of composite seawalls it is therefore not possible to establish with any accuracy the likely overtopping discharge. In the meantime all that can be stated is that the overtopping discharge will lie somewhere between the values obtained for simple seawalls with slopes equal to the lower and upper slopes respectively of the composite seawalls. Depending on the relative heights of the two portions of the seawall, the designer will then have to make a reasoned guess to determine which end of this discharge range is more likely.

Effect of wave angle

All the preceding sections on evaluating overtopping discharge have been based on the assumption that the waves strike the seawall orthogonally. However in many cases this will not be so, and some of the tests on which this report is based were therefore designed to examine the effect of the angle of wave attack on the quantity of water overtopping the seawalls. The tests were carried out for 1:1 and 1:4 simple seawalls, and also for the same seawall slopes with berms of width 10 and 80m, and a berm elevation of 2m below still water level.

In the absence of any better information, many seawall designers in the past have made the assumption that a wave attacking a seawall of slope 1:m at an angle of β behaves in the same way as a wave attacking normally a seawall with slope 1:m/cos β. In other words, a wave hitting a 1:2 seawall at 60° gives the same run-up and overtopping as a wave hitting a 1:4 seawall normally. However the test results showed that this was a gross over-simplification, and could lead to a serious under-estimate of the overtopping discharge when the angle is less than about 35°. Although there was some variation from test to test, the results showed that both for simple seawalls and for seawalls with narrow berms the overtopping discharge was greatest at about 15°, and at 30° was very similar to the discharge at 0°. Only for angles greater than about 40° was there any significant reduction in discharge. For the seawalls with an 80m wide berm the overtopping discharges at 0°, 15° and 30° were all very similar with significant reduction only occurring at larger angles.

Based on the results of these tests the following procedure is recommended when the waves attack the seawall at an angle. Firstly for the given seawall geometry, ie, seawall slope, berm width, berm elevation etc, the values of the constants A and B in the discharge equation $Q_* = A^{-BR_*}$ are determined for normal attack as described in the previous sections. Then the values of A and B are corrected for the wave angle. Figure 27 is entered with the required angle, and the correction factors are read off for A and B. The values of A and B at 0° are multiplied by the relevant correction factor at angle β to obtain the new values of A and B. The calculation then proceeds as before.

* *

Example Problem 6 Given: Seawall slope 1:2.2

Crest elevation +5.5m ODN
Berm elevation +1.7m ODN
Berm width 8.0m

Find: Overtopping discharge when: Tide level = +3.0m ODN
Significant wave height H_s = 1.75m
Mean wave period $\bar{T}$ = 5.0s
Mean wave angle β = 25°

Solution: From Example 5 we found that for normal wave attack the values of A and B for this seawall at this tide level were 1.96×10^{-2} and 42.3 respectively.

Entering Fig 27 with an angle β of 25°, we find from the curve for A a correction factor of 1.4. The value of A for this seawall at an angle of 25° is therefore:

$A_{25°} = 1.4 \times 1.96 \times 10^{-2} = 2.74 \times 10^{-2}$

Entering the same figure 27 with an angle of 25°, we find from the curve for B a correction factor of 1.02. The value of B for this seawall is therefore:

$B_{25°} = 1.02 \times 42.3 = 43.1$

For the given seawall at an angle of 25° we therefore have the equation

$Q_* = (2.74 \times 10^{-2})\, e^{-43.1R_*}$

For the given tide level, crest elevation, wave height and wave period we had from the previous example $R_* = 0.12$.

For this seawall at these wave conditions striking at an angle 25° we therefore have

$Q_* = (2.74 \times 10^{-2})\, e^{-43.1 \times 0.12}$

giving

$Q_* = 1.55 \times 10^{-4}$

since

$Q_* = Q/(\bar{T}gH_s)$, we therefore calculate

$\bar{Q} = Q_* \bar{T}gH_s$

$= (1.55 \times 10^{-4}) \times 5.0 \times g \times 1.75$

or

$\bar{Q} = 1.33 \times 10^{-2}$ m³/s per metre run of seawall

Note that for this particular seawall and wave climate waves striking the seawall at an angle of 25° give approximately 27% greater overtopping discharge than if the waves approached the seawall normally.

* *

As stated earlier, the model tests at different wave angles showed some variability. However the procedure recommended above will give at worst a conservative result, ie, the actual overtopping may well be less than calculated for a particular seawall.

Effect of seawall roughness

All the model tests carried out for this report were for smooth faced seawalls, since the models were constructed of plywood painted with polyurethane varnish. However full sizes seawalls are constructed of in-situ concrete, concrete blocks, set stones, or even grassed embankments, and each of these materials has a definite texture which will influence the overtopping of the seawalls.

Examination of all the available literature on seawall design shows that there is virtually no information on the effect of surface roughness on overtopping discharges for seawalls, either for regular or random wave conditions. However there is a considerable body of data on wave run-up on simply sloping roughened seawalls(15) and this can be used to provide some guidance on the overtopping discharge to be expected in such cases.

For a given seawall, the dimensionless overtopping discharge can be related to the number of waves which overtop the seawall crest by an expression of the type $Q_* = f(N)$

where

$Q_* \; = \bar{Q}/(\bar{T}gH_s)$

and

N is the number of waves which overtop the seawall as a proportion of the number of waves incident upon the wall.

The exact form of the function f(N) changes from seawall to seawall. However the degree of change is relatively small for minor variations in seawall geometry. In the absence of any definite information to the contrary, we will therefore make the assumption that the function f(N) is unchanged by surface roughness. The effect of surface roughness on overtopping discharge is therefore assumed to be limited only to its effect on N, the number of waves overtopping the seawall.

Now for seawalls subjected to irregular waves having a Rayleigh distribution of wave heights(8), the proportional number of waves N which overtop the seawall is given by the equation

$N = \exp\left(-2\,(R_c/R_s)^2\right)$

where R_c is the crest elevation above still water level and R_s is the height to which the significant wave would run up the seawall if the crest elevation was sufficiently high to prevent overtopping. R_s will of course change for different seawall geometries and also for different wave steepness values. However for a given seawall the effect of surface roughness on the wave run-up has been expressed in the literature as a roughness value, defined as

$r = R_r/R_{sm}$

where

r is the effective roughness value

R_{sm} is the height to which a given wave will run up a smooth seawall

R_r is the height to which the same wave will run up a rough seawall

By substituting the values $(R_s)_r = r(R_s)_{sm}$ in the equation for N, we therefore obtain as the proportional number of waves overtopping a rough seawall

$$N = \exp\left[-2\left(\frac{R_c}{r\,(R_s)_{SM}}\right)^2\right]$$

This equation can be rewritten in a slightly different way

$$N = \exp\left[-2\left(\frac{R_c/r}{(R_s)_{SM}}\right)^2\right]$$

which shows that the number of waves overtopping a rough seawall with crest elevation R_c and roughness value r is identical to the number overtopping a smooth seawall with an effective crest elevation of R_c/r. From the expression $Q_* = f(N)$ with the assumption that the function f is unchanged by surface roughness it therefore follows that the discharge Q_* over a rough seawall with crest elevation R_c and roughness r is also identical to the discharge over a smooth seawall with effective crest elevation R_c/r.

Returning to the more familiar expression for Q_*,

$Q_* = A\,e^{BR_*}$

we have for smooth seawalls $R_* = R_c/\bar{T}\sqrt{gH_s}$

and for rough seawalls therefore $R_* = (R_c/r)/(\bar{T}\sqrt{gH_s})$

or

$(R_*)_r = (R_*)_s/r$

giving

$(Q_*)_r = A\,e^{-B(R_*)_s/r}$

or

$(Q_*)_r = A\,e^{-(B/r)(R_*)_s}$

For a rough seawall therefore the overtopping discharge is the same as for a seawall of effective freeboard R_*/r, or in other words the coefficient B is corrected by a factor 1/r. Table 3 gives a summary of the published values for the effective roughness r for a range of common seawall constructions. The Table does not include values for 'artificial' roughness such as stepped or ribbed seawalls, since these vary considerably in their geometries. Reference 12 gives more information on this subject. The minimum value of r for these artificially roughened seawalls is unlikely to be less than 0.5.

* *

Example Problem 7

Given:
Seawall slope 1:2.2
Crest elevation +5.5m ODN
Berm elevation + 1.7m ODN
Berm width 8.0m
Seawall construction: pitched stone

Find:
Overtopping discharge when Tide level = +3.0m ODN
Significant wave height H_s = 1.75m
Mean wave period $\bar{T}$ = 5.0s
Normal wave attack

Solution:
From Example 5 we found that for normal wave attack on a smooth seawall the values of A and B for this seawall at the given tide level were 1.96×10^{-2} and 42.3 respectively. From Table 3, we see that for seawalls constructed in pitched stone the quoted roughness value varied from about 0.85 to 0.9. Assume a value of 0.9 which would give the higher overtopping discharge.

The value of A is unchanged by surface roughness, and therefore remains at 1.96×10^{-2}.

The value of B for this seawall is obtained from

$$\begin{aligned} B_r &= B_{sm}/r \\ &= 42.3/0.9 \\ &= 47.0 \end{aligned}$$

For the given tide level, crest elevation, wave height and wave period we had from the previous example $R_* = 0.12$. For this rough seawall at these wave conditions striking the seawall normally we therefore have

$$Q_* = (1.96 \times 10^{-2})\, e^{-47.0 \times 0.12}$$

giving

$$Q_* = 6.96 \times 10^{-5}$$

and

$$\bar{Q} = Q_* \bar{T} g H_s$$
$$= 5.97 \times 10^{-3} \text{ m}^3\text{/s per metre run of seawall}$$

For this particular seawall at these wave and tidal conditions the effect of replacing a smooth finish by a pitched stone finish has thus been to reduce the overtopping discharge by 43%.

* *

Note that since this method of correcting for surface roughness is based on the assumption that the function f(N) in the expression $Q_* = f(N)$ is unchanged by surface roughness, then the results can only give an approximation of the true overtopping discharge. However, since f(N), if it changes at all, is likely to reduce in value, then this approximate method will give a conservative answer, ie the actual overtopping discharge will probably be less than predicted.

Effect of onshore winds

Almost all data on wave overtopping has been obtained in laboratory studies without taking wind effects into account. However in nature large waves will frequently be associated with onshore winds. These winds may influence the overtopping discharge in several ways, including:-

1. Raising the still water level (wind set-up)
2. Increasing wave run-up on the seawall
3. Blowing spray over the seawall.

The relative importance of each of these factors will depend largely on the type of seawall being considered. For example a vertical seawall can result in a considerable volume of water being thrown into the air when the waves break against it: with a moderate onshore wind a significant proportion of this will be blown over the seawall. For the simple and bermed seawalls described in this report the quantity of water thrown into the air by waves breaking on the wall is relatively small: here the wind effect would probably be a combination of all three factors.

Only a very limited number of tests have ever been carried out to determine the magnitude of the combined effects of the wind on overtopping discharge and the results have been contradictory. The main difficulty is involved in the modelling of wind effects, particularly in representing air-borne water, since the droplet size in the model is almost identical to the droplet size in nature. The other difficulty lies in reproducing identical wave conditions both with and without wind, since the addition of wind will usually also affect the laboratory wave conditions. With these provisos on the usefulness of model tests for the study of wind effects, Reference 15 quotes various results of laboratory investigations showing that wind effect reduces for steep seawalls, although the slopes tested varied only from about 1:7 to 1:3. On the other hand the Shore Protection Manual(8) quotes a formula for wind effect which implies that the total wind effect becomes larger for steeply sloping seawalls. Although this formula is said to be unverified, and also no derivation is given, it is stated that the formula is "believed to give a reasonable estimate of the effects of onshore winds of significant magnitude". The formula can be applied for seawalls with slopes up to vertical.

In the tradition of the Shore Protection Manual, the formula as given is applicable only to mono-chromatic waves, and its usefulness in random or irregular waves is therefore severely restricted. However it can be modified to give an approximate estimate of the maximum and minimum effects of the wind for a given seawall. The calculated overtopping rates are multiplied by a wind correction factor K_w, where K_w lies inside the range

$$1.0 + 1.1\ W_f \sin\theta > K_w > 1.0 + 0.1\ W_f \sin\theta$$

where

θ is the angular slope of the seawall

ie $\theta = \tan^{-1} 1/m$

or $\sin\theta$ $1/\sqrt{1+m^2}$

and

W_f is a wind factor, whose value depends on the onshore component of wind speed.

For no wind, W_f has a value of 0: for a windspeed of about 13m/s W_f is 0.5, and for a speed of about 26m/s W_f is 2.0. Values of W_f for intermediate speeds can be derived by interpolation.

From the above formula it can be seen that for a 1:1 seawall with a 26m/s wind (Force 10 Beaufort scale) the overtopping discharge must be corrected by the factor K_w, where $2.66 > K_w > 1.14$. For a 1:4 seawall in the same conditions $1.53 > K_w > 1.05$. Within the range of values, the correction factor K_w will be lowest for seawalls where the dimensionless freeboard R_* is very low, and will be greatest when R_* is very high. In other words, when a substantial amount of water overtops the seawall then the effect of the wind is relatively insignificant. When only a small quantity of water is overtopping, then the volume of wind-blown spray becomes comparable with the volume carried directly by the overtopping waves. In almost all newly designed seawalls the value of K_w will probably approach the upper end of the range of possible values, since the seawall will presumably be designed for little overtopping.

Evaluation of total overtopping volume during a tidal cycle

All the calculations discussed so far have related to the overtopping discharge at a fixed water level — by implication this is normally at High Water Level. However in most cases it is necessary to know the total volume of water which overtops the seawall during the complete tide, since this determines the degree of flooding behind the sea defences, or the area which has to be designated for flood storage during severe storms. Several different methods can be derived for calculating this cumulative volume of overtopping from the data presented in this report; the design engineer may thus wish to use alternative methods in particular circumstances, rather than the one described below. However for all methods it will be necessary to know the peak tidal level for the design storm, a typical tide shape curve (preferably for tides of similar range and height to the design tide), and the design wave height, wave period and wave direction.

In the method now described, the first objective is the derivation of a curve of overtopping discharge against tide level for the design wave conditions, by calculating the overtopping discharge at a few standard water levels. These water levels should be selected to cover the range from High Water Level down to the tide level at which the discharge is about 1 per cent of the overtopping discharge at High Water. At each water level, the calculations fall into three parts:-

(1) Check to confirm that the design wave height can approach the seawall without breaking. If not, replace design wave height by the equivalent post-breaking wave height. This check is carried out using Fig 7, and knowing the foreshore gradient m_f, the depth of water at the toe of the seawall, d_s, and the mean wave period $\bar{T}$.

(2) Evaluation of the coefficients A and B from Figures 23 to 26 knowing the seawall slope m, and if necessary the berm elevation relative to the Still Water Level (tide level) d_B, and the berm width W_B.

(3) Calculation of the overtopping discharge $\bar{Q}$ from the equations

$$Q_* = Ae^{-BR_*}$$
$$R_* = R_c/(\bar{T}\sqrt{gH_s})$$
$$Q_* = \bar{Q}/(\bar{T}gH_s)$$

knowing the seawall crest elevation R_c relative to the Still (tide) Water Level. These values should be modified if necessary, to take the angle of wave attack, sea wall roughness and wind effects into account.

A graph is then prepared of overtopping discharge $\bar{Q}$ against still water level SWL, plotting log $\bar{Q}$ against linear SWL. Having derived this graph, the next stage is to examine the typical tide curve, firstly to determine over what period the water level is high enough to give significant overtopping discharge, ie not less than 1 per cent of the peak overtopping discharge at High Water. After selecting a suitable timestep, the tide levels are then read off at fixed time intervals before and after High Water, continuing to the time when the water level falls to such an extent that overtopping is insignificant. Using the previously prepared graph, the overtopping discharge is then estimated for each water level, and the overtopping volume is then calculated from $\bar{Q}\triangle t$ where $\triangle t$ is the selected timestep. The total volume of water which overtops during the tidal cycle is thus $\Sigma\bar{Q}\triangle t$, or for a constant timestep this becomes $\triangle t\Sigma\bar{Q}$.

* *

Example Problem 8 Given:

Given:	Seawall slope	1:2.2
	Crest elevation	+5.5m ODN
	Berm elevation	+1.7m ODN
	Berm width	8.0m
	Toe elevation	0.0m ODN
	Foreshore slope	1:10
	Seawall construction: smooth in-situ concrete	
Find:	total overtopping volume during the tide when:	
	Peak tide level	+3.0m ODN
	Significant wave height	1.75m
	Mean wave period	5.0s
	Normal wave attack	
	Typical tide curve as shown in Fig 28	

Solution: Set up a calculation sheet as shown in Table 4. Beginning with a still water level equal to the peak tidal level, calculate the depth of water at the seawall toe,

d_s = 3.0m ODN − 0.0m ODN = 3.0m

Calculate $d_s/g\bar{T}^2 = 3.0/(9.81 \times 5^2) = 0.0122$

From Fig 7, with $d_s/g\bar{T}^2 = 0.0122$, foreshore gradient m_f = 1:10, we have the breaking ratio $\gamma_b = 0.76$. However the ratio of design wave height to the depth H_s/d_s is 0.58. Since this is less than the breaking ratio, the design wave can approach the seawall without breaking and we can therefore use H_s = 1.75m for all subsequent calculations.

Next, evaluate the coefficients A and B. At a tide level of +3.0m ODN, the berm elevation relative to still water level is

d_B = 1.7m ODN − 3.0m ODN = −1.3m

By interpolation between Figs 24a and 25a, we have A = 1.96×10^{-2} for a seawall slope of 1:2.2 and a berm width of 8.0m (see Example 5 for details of the interpolation). Similarly by interpolation between Figs 24b and 25b we have B = 42.3. In order to calculate the overtopping discharge, the crest elevation relative to still water level is given by

R_c = 5.5m ODN − 3.0m ODN = 2.5m, and

hence

$$R_* = R_c/(\bar{T}\sqrt{gH_s}) = 2.5/(5.0\sqrt{g.1.75}) = 0.1207$$

$$Q_* = Ae^{-BR_*} = (1.96 \times 10^{-2})\, e^{-42.3 \times 0.1207}$$

$$= 1.19 \times 10^{-4}$$

$$\bar{Q} = Q_*\bar{T}gH_s = (1.19 \times 10^{-4})\, 5.0 \times g \times 1.75$$

$$= 1.02 \times 10^{-2}\ m^3/s/m$$

The peak overtopping discharge is thus $1.02 \times 10^{-2} m^3/s/m$ run of seawall, so that the calculations have to be continued until the discharge falls to about $1 \times 10^{-4} m^3/s/m$. Choosing a suitable level increment (in this case 0.25m), the calculations are repeated for different water levels, until for a tide level of 1.75m ODN we find that the overtopping discharge falls to 1.00×10^{-4}. (Note that at this water level the ratio H_s/d_s is greater than the breaking ratio γ: the design wave height therefore breaks before reaching the seawall, and the breaker wave height $H_{s_b} = \gamma d_s = 0.93 \times 1.75 = 1.63$m must therefore be used for discharge calculations). The calculated discharges are then plotted against water level, as shown in Fig 29.

From these discharge calculations we see that the overtopping discharge becomes insignificant when the water level is +1.75m or less, ie 1.25m or more below the peak tide level. Figure 28 shows the typical tide curve, in this case having a high water level of +2.8m ODN (It is unlikely that the design tide has ever been recorded). From this curve we see that water levels 1.25m or less below High Water Level, (ie above +1.55m ODN on the typical curve) occur from about 1 hour before High Water to about ¾ hour after High Water. In other words overtopping is significant for about 1¾ hours. A timestep of 15 minutes would therefore seem adequate for the overtopping volume calculations. Starting at 1 hour before High Water (ie when the water level is close to 1.25m below High Water Level), the water levels relative to the peak tide level are therefore read from the typical tide curve at 15 minute intervals, until the level falls again to about 1.25m below High Water Level. Knowing the design High Water Level, the absolute water levels are then calculated, and the relevant overtopping discharges read off Fig 29 and tabulated, Table 5. With a 15 minute timestep, the volume of water overtopping during the timestep is therefore $\bar{Q} \times 15 \times 60$, or $1.02 \times 10^{-2} \times 15 \times 60 = 9.18 m^3/m$ for the 15 minute timestep centred around High Water. The total overtopping volume during the tide is then simply the sum of all the incremental volumes, giving $24.09 m^3/m$ run of seawall. Multiplication by the total length of seawall therefore gives the bulk volume for the site.

* *

Standard deviation of expected overtopping discharge

Because wave heights and periods exhibit a random distribution about a given mean there is no such thing as an absolute value of the overtopping discharge: the discharge will also vary randomly. Suppose that the discharge averaged over a duration of $100\bar{T}$ is denoted by Q, then over several successive sequences of $100\bar{T}$ the discharge Q will be expected to vary randomly about some mean value $\bar{Q}$. All the expressions presented so far in this report relate this mean discharge $\bar{Q}$ to the significant wave height H_s and the mean period $\bar{T}$ for a range of seawall designs. However it is now worth examining the possibility or chance that the actual discharge Q for a particular sequence of $100\bar{T}$ exceeds the expected mean value $\bar{Q}$.

The model tests on which this report is mainly based included measurements of the overtopping discharge for at least 5 sequences of 100 wave periods for each wave condition and seawall configuration. From these individual measurements it was possible to calculate not only a series of expressions relating mean discharge to wave conditions, but also the standard deviation of the measured discharges about the fitted expressions. Because these expressions took the form

$$Q_* = Ae^{-BR_*}$$

or

$$\ln Q_* = \ln A - BR_*$$

the standard deviations were in fact calculated on the basis of $\ln Q_*$. On the basis the standard deviation $\sigma_{\ln Q_*}$ was typically about 0.7: in other words the dimensionless discharge at one standard deviation above the mean was typically $Q_* e^{+0.7}$ or $2Q_*$ and at one standard deviation below the mean was $Q_* e^{-0.7}$ or $Q_*/2$. Alternatively the discharges at one standard deviation above and below the mean are obtained by multiplication and division respectively by a factor of $\exp(\sigma_{\ln Q_*}) = e^{0.7} = 2$. At two standard deviations the factor is $\exp(\sigma_{\ln Q_*}) = e^{1.4} = 4$. Assuming that the logarithms of the dimensionless overtopping discharges exhibit a normal probability distribution about a mean value, then from statistical tables it is possible to estimate the chance or probability that the actual discharge for a particular wave sequence will fall outside certain limits. On this basis the following table has been prepared.

Percentage probability of exceedance	*Overtopping discharge factor*
50	1.0
20	1.8
10	2.4
5	3.2
2	4.2
1	5.1
0.5	6.1
0.2	7.5
0.1	8.7

This table shows for example that there is a 1% chance that the overtopping discharge in a particular sequence of 100 waves will exceed $5.1Q_*$, where Q_* is the expected mean discharge. Equally, there is a 1% chance that the discharge will be less than $Q_*/5.1$

Example Problem 9

Given:

Seawall slope	1:2.2
Crest elevation	+5.5m ODN
Berm elevation	+1.7m ODN
Berm width	8.0m
Design storm: Tide level	+3.0m ODN
Significant wave height H_s	1.75m
Mean wave period $\overline{T}$	5.0s

Find: Overtopping discharge to be expected with 95% confidence

Solution: From Example Problem 5 we found that for this seawall configuration the overtopping discharge formula was

$$Q_* = (1.96 \times 10^{-2})e^{-42.3R_*}$$

and the expected mean overtopping discharge Q for this design storm was $1.05 \times 10^{-2} m^3/s$ per metre run of seawall.

We now require the overtopping discharge which is unlikely to be exceeded for 95% probability, in other words the discharge which could be exceeded with only a 5% probability. From the table, we see that for 5% probability the discharge factor is 3.2. The overtopping discharge which is unlikely to be exceeded for 95% of the time is therefore $3.2 \times 1.05 \times 10^{-2}$, or $3.36 \times 10^{-2} m^3/s$ per metre run of seawall.

* *

Return period for overtopping discharge

The calculations so far have assumed that the overtopping discharge is required for a storm of a given return period, say 100 years. Ideally however a more useful figure would be the 100 year return period overtopping discharge, which need not necessarily be generated by the 100 year storm.

As we have seen, the overtopping discharge for a particular seawall depends on the tide level and on the wave height and period. If the probability of occurrence of given wave conditions is totally dependent on the probability of occurrence of given tide levels, ie the 100 year wave height always occurs simultaneously with the 100 year tide level, then this condition will always generate the 100 year overtopping discharge. However this is not the case if the probability of occurrence of given wave conditions is totally independent of the probability of occurrence of given tide levels. Suppose for example that we consider a 100 year storm: this could be composed of a 5 year tide level with a 20 year wave height, a 10 year tide level with 10 year wave height, a 20 year tide level with 5 year wave height, or any other combination leading to a combined return period of 100 years. Without calculation it is usually not possible to predict which of these combinations is likely to give the greatest overtopping discharge, or to predict the value of overtopping discharge with a return period of 100 years. For this assumption of total independence of tide levels and wave heights these calculations are relatively straightforward although rather tedious, and for total dependence of waves on tidal heights the calculations are trivial. However at most localities the wave heights and tide heights are partially correlated, and research is still continuing into the best methods of determining overtopping discharges for a given return period in these circumstances.

In principle, the calculations for both independence and partial dependence of wave heights on tidal heights are identical. Firstly a range of possible tide levels and a range of possible wave heights are selected. The overtopping discharge for each possible combination of wave height and tide level is then calculated. From analysis of available tide and wave records the probability of occurrence of each possible tide level/wave height combination is calculated, and this probability of occurrence is then attached to the calculated overtopping discharge generated by this particular tide level/wave height combination. The probability distribution for overtopping discharge is then obtained by summation, and the discharge for given return periods read off the resulting plotted curve.

The difficult step is the evaluation of the probability of occurrence of each possible wave height/tide level combination. The difficulties arise because:-

1. Wave records are very rarely available, and if they are they generally cover a very short timespan — at most about 5 years. Wave heights therefore have to be calculated from available records of wind speed and direction.
2. Simultaneous wind and tide records are rarely available over a sufficiently long timespan to enable the probability of occurrence of extreme events to be accurately assessed. To obtain the overtopping discharge even for a 100 year return period requires several decades of simultaneous data.
3. Where extreme events have not occurred during the record period there is no known way of extrapolating the data to more extreme occurrences (unless waves and tide levels are either totally dependent or independent).

To overcome these difficulties, the present method is to calculate the return period for overtopping discharge on both of the assumptions that tides and waves are totally dependent or are totally independent. As mentioned earlier, for the total dependence case the calculations are trivial, since the 100 year discharge is caused by the 100 year tide level occurring simultaneously with the 100 year wave height. For the total independence case, the available tide records are used to determine the probability of occurrence of given tide levels, and the available wind or wave records are used to determine the probability of occurrence of given wave conditions. Each of these probabilities can be extrapolated if necessary to more extreme events. The joint probability of occurrence of a given combination of wave height/period and tide level is then simply the product of the individual probabilities of the occurrence of the given wave height/period and of the given tide level respectively. This method has been used in two recent studies at HRS[16, 17].

Armed with these results the design engineer then has to make a judgement as to whether at his particular site the waves and tide levels are likely to be relatively dependent or almost independent of each other. Relatively dependent situations might arise for example where the wave height reaching the seawall is always limited by breaking: higher tide levels therefore allow greater wave heights to reach the seawall. Similarly in situations where the meteorological system generating surge tides causes strong winds from a direction having appreciable fetch length then here again the wave height and tide levels would be fairly strongly dependent. In these situations therefore the assumption of total dependence would be adopted, giving a possible slight overdesign of seawall. Alternatively the surge generating system might be associated with offshore winds at a particular site. In this case the assumption of total independence would be used, again giving a slightly conservative design of seawall in the sense that the return period for a given overtopping discharge is likely to be slightly longer than calculated.

As mentioned earlier, research is still continuing in this topic, particularly with a view to exploring the possibility of linking the probabilities of occurrence of predicted tide levels, surge residuals, and wave heights.

DESIGN OF NEW SEA WALLS

Many of the steps and calculations involved in the design of a new seawall are of course very similar to those required for the evaluation of the performance of existing sea defences. In fact in many cases the new seawall configuration will be selected on the basis of intuition and experience, and the calculations used to confirm or otherwise the suitability of the design, in the same way as the adequacy of an existing seawall is examined. Most of the detailed steps in the calculation procedure will not therefore be repeated in this chapter: rather, attention will be devoted to some general guidelines in the design of new seawalls.

Design discharge The design parameters which have to be selected are the design storm (still water level, wave height, wave period, and wave direction) and the allowable overtopping discharge. Selection of the design storm parameters has already been discussed in some detail, so attention here will be devoted to the design value of the allowable overtopping discharge.

Depending on the nature of the particular scheme, the selection of the allowable overtopping discharge has to satisfy various requirements, including:-

1. The stability of the crown and backface of the seawall
2. The discharge capacity of drainage channels behind the seawall
3. The total volume available for storage of flood waters behind the seawall until the tide level falls sufficiently for tidal outfalls to come into operation
4. The possibility of damage or injury to buildings, vehicles or members of the public located behind the seawall.

1. The stability of the crown and backface of the seawall will be most severely tested at the height of the storm when the overtopping discharge is at its peak. The stability will depend on the method of seawall construction: for example a concrete or asphalt seawall would be expected to withstand a considerably higher overtopping discharge before failure than would a grassed embankment. The only published information relating backface stability and overtopping is a paper by Goda(18) describing work carried out in Japan following a very severe typhoon in 1959 which killed 5000 people. For each seawall which had been damaged during the typhoon the peak overtopping discharge was estimated from the results of idealised model tests. By comparing the degree of damage with the estimated overtopping discharge the following values were recommended for the design of seawalls for no damage:-

Type of seawall/construction	*Threshold overtopping discharge $m^3/s/m$ run*
Seawall with back slope (embankment):	
Crown and back slope unprotected (eg clay, compacted soil, grassed)	less than 5×10^{-3}
Crown protected, back slope unprotected	2×10^{-2}
Crown and back slope protected (eg concrete layer)	5×10^{-2}
Seawall without back slope (revetment):	
Apron unpaved	5×10^{-2}
Apron paved	2×10^{-1}

For example, an overtopping discharge of $1 \times 10^{-2} m^3/s/m$ run would be expected to cause damage to an unprotected embankment, but no damage should occur if the crown or crest of the seawall is protected.

2. The discharge capacity of drainage channels immediately behind the seawall will also depend mostly on the peak overtopping discharge during the storm. In other words if the discharge of the drainage channel is known or can be calculated then this defines immediately the allowable overtopping discharge at the peak of the storm.

3 . The volume available for flood storage behind the seawall depends on the total volume of water which overtops during the complete tide. In other words, if it is known that the flood waters will pond in a certain area and that only a finite depth of flooding can be tolerated, then the calculated flood storage volume defines precisely the allowable volume of overtopping during the tide. However it is not possible to design the seawall directly from this information: to do so it is necessary to estimate the peak overtopping discharge at the height of the storm. The seawall dimensions are then chosen to give this peak overtopping discharge, and calculations of total volume overtopping are performed for the resulting seawall. The calculated overtopping volume is then compared with the allowable overtopping volume for flood storage, and the seawall design modified as necessary. In order to estimate this peak overtopping discharge from the total allowable overtopping volume it is necessary first of all to assume a likely duration of overtopping at the new seawall. Dividing the overtopping volume by the duration then gives the mean overtopping discharge during the overtopping period. The peak overtopping discharge at the height of the storm is likely to be a factor between about 2 and 5 times greater than the mean overtopping discharge.

4 . The possibility of damage or injury to structures or personnel behind the seawall will be greatest at the height of the storm. If it is possible to obtain an allowable discharge for no damage or injury, then this precisely defines the peak overtopping discharge at the seawall at the height of the storm. The difficulty lies in establishing the no-damage discharge values. The only published information relating to this concerns some full-scale measurements carried out in Japan on seawall overtopping(19). Overtopping discharges were measured for several storms, and the overtopping waves filmed simultaneously with the measurements. The films were then shown to eight port and harbour research engineers for their individual assessments of the likely damage or injury to a walking person, an automobile, and a house in each case situated either immediately behind or about 10m behind the seawall. Based on the engineer's assessments and on the associated measured overtopping rates, the following limiting values of overtopping discharges were quoted,

1. For a person to walk immediately behind the seawall with a little discomfort,

   $Q < 4 \times 10^{-6}\ m^3/s/m$

2. For a person to walk immediately behind the seawall with little danger,

   $Q < 3 \times 10^{-5}\ m^3/s/m$

3. For an automobile to pass immediately behind the seawall at high speed

   $Q < 1 \times 10^{-6}\ m^3/s/m$

4. For an automobile to pass immediately behind the seawall at low speed

   $Q < 2 \times 10^{-5}\ m^3/s/m$

5. For a house located immediately behind the seawall to suffer no damage,

   $Q < 1 \times 10^{-6} m^3/s/m$

6. For a house located immediately behind the seawall to suffer no substantial flooding or damage although experiencing partial damage to windows and glazed doors,

   $Q < 3 \times 10^{-5} m^3/s/m$

When the danger was assessed at a distance of 10 metres behind the crest of the seawall then it was found that each of these limiting discharges could be increased by a factor of about 10. For example, a person could walk without danger at a distance of 10m from the seawall with overtopping discharges up to about 3 x $10^{-4}m^3/s/m$. Each of the limiting discharges quoted was said to have a 90% safety standard: in other words 7 out of the 8 engineers questioned thought this to be a safe limit.

The above paragraphs have discussed various possible ways in which the allowable overtopping discharge can be determined. In the Netherlands the maximum overtopping discharge which seawalls are designed for is said to be 2 x 10^{-3} $m^3/s/m$ (ie 2 litres/s/m run of seawall), although no published information relating to this discharge value has been found.

The published Dutch practice(20) is to design seawalls not on the basis of discharge, but on the principle of adopting a crest elevation such that only 2 per cent of the waves overtop the seawall during the design storm. However this gives widely differing values of overtopping discharge depending on the seawall geometry, and it is believed that where this '2 per cent' criterion leads to an excessive overtopping discharge then Dutch engineers stipulate an upper limit of 2 x $10^{-3}m^3/s/m$. No information is available which describes the reason for the choice of this value. However in view of the fact that the majority of sea dikes in the Netherlands are of unprotected construction it is interesting to note that the allowable discharge is very similar to that quoted by Goda for unprotected embankments (5 x $10^{-3}m^3/s/m$).

Finally it is worth examining the concept of designing the seawall for 'nil' overtopping discharge. Before the randomness of waves was fully understood it was thought possible to build a seawall sufficiently high for nil overtopping to occur. However the probability distribution for wave height, and an examination of the relationship $Q_* = Ae^{-BR_*}$ shows that however high the seawall there will always be a finite albeit very small overtopping discharge. In the design of seawalls against attack by random waves it is therefore always necessary to define the allowable overtopping discharge, either by using one of the methods described previously, or possibly by comparing the overtopping discharge with the quantity of water delivered by an intense rainstorm.

Design constraints

As well as these design parameters, at most sites there also exist design constraints, such as a limitation on the crest elevation which can be adopted because of environmental or constructional difficulties. Also, where the foreshore is composed of erodible sands or silts it will be necessary to consider the effect of the seawall on foreshore bed levels. These constraints should be identified as early as possible for the particular site.

Initial seawall design

For the initial design of seawall, the best approach is probably to assume that the new seawall will closely resemble the form of the existing seawall or of a nearby seawall. In other words, if there is no existing berm at the site, assume that the initial design is a simply sloping seawall. If there is an existing berm, assume for the initial design that the berm dimensions are unchanged. The selection of a suitable design then follows in a series of steps, some of which could be omitted in some cases.

Simple seawalls

(a) For the existing seawall slope, evaluate the coefficients A and B by the methods described earlier, making allowances if necessary for the wave direction and the roughness of the seawall.

(b) Check that the design wave height can approach the seawall without breaking when the tide is at its design still water level.

(c) Using the design tide, wave and discharge parameters evaluate the required dimensionless free board from the equation

$$R_* = \frac{1}{B} \ln \left(\frac{A}{Q_*}\right)$$

and hence calculate the required seawall crest elevation.

(d) If the resulting crest elevation is impractical, repeat the calculations for two or three alternative seawall slopes. Generally a flattening of the seawall slope will lead to a lower requirement for the crest elevation for a given overtopping discharge. Under some conditions however, particularly where a high dimensionless overtopping discharge is allowable, or for seawalls having slopes between about 1:1 and 1:2, flatter slopes may sometimes require a higher crest elevation.

(e) As a guide to possible seawall slopes, Fig 9 may be used when the waves strike the seawall normally. Knowing the value of crest elevation which is practical, the value of R_* is calculated for the design storm. This value is then used with the calculated value of Q_* to read off the required seawall slope.

(f) Assuming that one or more feasible designs arise out of these calculations, make a rough assessment of the environmental and financial cost of the various possible designs.

Bermed seawalls

If the existing seawall is bermed, or the above calculations fail to yield a suitable design of simply sloping seawall, then similar calculations have to be carried out for a bermed seawall.

(a) For the existing seawall slope, berm width and berm elevation evaluate the coefficients A and B, making allowances if necessary for seawall roughness and wave direction.

(b) Check that the design wave height can approach the seawall without breaking.

(c) Calculate the dimensionless freeboard from the equation

$$R_* = \frac{1}{B} \ln \left(\frac{A}{Q_*}\right)$$

and hence determine the required crest elevation.

(d) If the resulting crest elevation is unsatisfactory, then three dimensions can be varied — seawall slope, berm width, and berm elevation. In a similar way to simply sloping seawalls, a flattening of the slope of a bermed seawall will generally lead to a lower crest elevation for a given overtopping discharge. For berm dimensions, the required crest elevation will usually be lower when the berm elevation is close to still water level, and also for wider berms. With berm elevation, the major reduction in crest elevation occurs when the berm elevation is raised from 2 metres to 1 metre below still water level, with only slight improvement by a further rise. For berm widths, the reduction in required crest level elevation occurs for berms about 10m or wider, and then continues indefinitely as berm width increases. For berm widths less than 10m the required crest elevation for a given overtopping discharge may actually be increased compared with a simple seawall, the increase depending on the seawall slope and berm elevation.

(c) If no berm already exists at the site, then from the above paragraph it seems likely that a berm elevation of 1m below still water level and a berm width of 10m should prove a useful starting point for the design of a suitable berm. If the seawall is reasonably smooth, and the waves strike the seawall near-normally, then Fig 15 can be used to assist in the design. If a berm already exists, then the improvements will have to be based on the existing dimensions, using the above guidelines to achieve a suitable design. It will probably prove possible to use one of the standard berms as the first estimate of an improved design, in which case the relevant Figures 11 to 22 can be used for assistance. Knowing the allowable dimensionless overtopping discharge Q_*, either the required crest elevation R_* can be estimated for the preferred seawall slope, or the required slope can be estimated for the largest practicable value of R_*.

(f) Assuming that one or more feasible design arises out of these calculations, a rough assessment is made of the cost of the various possible schemes.

Final seawall design

After various possible designs of seawall have emerged from the initial design studies, the most promising schemes are selected for more detailed analysis. This analysis should include, but not be limited to, an assessment of:-

(a) the total volume of water overtopping the seawall during the design tide. This is calculated by the method described earlier.

(b) The overtopping discharge for the designed seawall under other, generally less severe, storm conditions. For example, if the seawall has been designed for a specific storm, then its behaviour under different wave and tide combinations should also be assessed, to confirm that the design is suitable for lesser storms. This conclusion is by no means automatic: for example it is possible that a 1000 year storm having water level +5.2m ODN, wave height 1.0m, and wave direction 0° will give less overtopping discharge at a particular seawall than a 100 year storm having water level +4.5m ODN, wave height 1.75m, wave direction 15°.

(c) The effect on foreshore levels. If the foreshore is composed of erodible sands or silt, then wave reflection at the seawall would cause significant erosion and hence lowering of the foreshore. Falling foreshore levels will allow larger waves to reach the seawall, which could in turn lead to higher overtopping discharges. Reduction of wave reflection can be achieved by roughening the face of the seawall (with the additional benefit of reducing overtopping) or by adopting a design with a relatively flat seawall slope (preferably about 1:3 or flatter).

(d) Other environmental effects, including for example the visual impact of high crest elevations.

(e) Financial implications, including capital cost of construction, ease and cost of any maintenance etc.

On the basis of these assessments the final design is selected according to the priorities attached to each factor by the designing authority.

* *

Example Problem 10

Given: Existing seawall dimensions:-

Slope	1:1.5
Crest elevation	+5.25m ODN
Toe elevation	0.0m ODN
Foreshore slope	1:20
No berm	
In-situ concrete construction	

Find: (a) The overtopping discharge at the existing seawall when:-

tide level =	+3.0m ODN
Significant wave height H_s =	1.25m
mean wave period $\bar{T}$ =	4.0s
normal wave attack	

(b) A suitable design of seawall to achieve an overtopping discharge of 2 x $10^{-3}m^3/s/m$ run for the above design storm.

Assume that for various reasons the seawall crest elevation cannot be raised by more than 1.0 metre.

Solution:

(1) preliminary calculations

Deepwater wavelength $L_o = \frac{g\bar{T}^2}{2\pi} = 25.0m$

Deepwater steepness $H_s/\bar{L}_o = 0.05$

Breaking wave height: Water depth at toe $= 3.0 - 0.0 = 3.0m$

The ratio $d_s/g\bar{T}^2 = 0.019$

$\therefore$ from Fig 7 for $m_f = 1{:}20$

$\gamma = 0.52$

$\therefore H_{sb} = 0.52\ d_s = 1.56m$

Design wave height is less than breaker height $\therefore$ design wave can reach seawall without breaking.

(2) overtopping at existing seawall

Dimensionless freeboard: Crest elevation $R_c = 5.25 - 3.0$

$= 2.25m$

$$R_* = \frac{R_c}{\bar{T}\sqrt{gH_s}} = 0.161$$

Discharge coefficients: From Fig 10 for simple seawalls, a slope of 1:1.5 gives

$A = 1.02 \times 10^{-2}$, $B = 20.1$

No allowance is necessary for seawall roughness or for wave angle.

Dimensionless discharge $Q_* = Ae^{-BR_*}$

$= (1.02 \times 10^{-2})\ e^{-20.1 \times 0.161}$

$= 4.01 \times 10^{-4}$

Absolute discharge $\bar{Q} = \bar{Q}_*\bar{T}gH_s$

$= 4.01 \times 10^{-4} \times 4.0 \times g \times 1.25$

$= 1.97 \times 10^{-2}\ m^3/s/m$ run of seawall

This discharge is well above the required discharge of $2.0 \times 10^{-3} m^3/s/m$ so a new seawall is required.

(3) Alternative seawall designs

Design dimensionless discharge: allowable discharge

$\bar{Q}_d = 2 \times 10^{-3} m^3/s/m$

$$Q_{*d} = \frac{\bar{Q}_d}{\bar{T}gH_s} = \frac{2 \times 10^{-3}}{4.0 \times g \times 1.25}$$

$= 4.08 \times 10^{-5}$

(i) Assume that seawall slope is unchanged, then required crest elevation is obtained from

$$R_* = \frac{1}{B} \ln\left(\frac{A}{Q_*}\right)$$

$$= \frac{1}{20.0} \ln\left(\frac{1.12 \times 10^{-2}}{4.08 \times 10^{-5}}\right)$$

$= 0.281$

$R_c = R_*\bar{T}\sqrt{gH_s} = 3.94m$

The absolute crest elevation required is therefore $3.0 + 3.94 = 6.94m$ ODN. Since the existing crest elevation is 5.25m ODN, the seawall would have to be raised by 1.69m: this is more than the stated practical limit of 1.0m for this example.

(ii) Since a feasible design with the existing seawall slope is not possible, try alternative designs using different slopes, either maintaining the existing crest elevation, or adopting the highest practicable crest elevation (+6.25m ODN). With the existing crest elevation we know that $Q_{*d} = 4.08 \times 10^{-5}$ and $R_* = 0.161$. From Fig 9 these conditions are satisfied by a slope of about 1:3.4. To check this, from Fig 10 we have for a slope of 1:3.4, $A = 1.75 \times 10^{-2}$, $B = 37.4$. For the required discharge we have therefore

$$R_* = \frac{1}{B} \ln \left(\frac{A}{Q_{*_d}}\right) = 0.162$$

which is very slightly above the existing seawall (in fact 0.01m higher). This is therefore a possible design — No. 1.

Alternatively with a crest elevation of +6.25m ODN the dimensionless freeboard becomes

$$R_* = \frac{R_c}{T\sqrt{gH_s}} = \frac{3.25}{4.0\sqrt{g.1.25}} = 0.232$$

With this freeboard, and the design discharge $Q_{*_d} = 4.08 \times 10^{-5}$, Fig 9 shows that a seawall slope of about 1:2.4 is possible. Again as a check, from Fig 10 for a slope of 1:2.4 we have $A = 1.41 \times 10^{-2}$, $B = 25.2$. We therefore have

$$R_* = \frac{1}{B} \ln \frac{A}{Q_*} = \frac{1}{25.2} \ln\left(\frac{1.41 \times 10^{-2}}{4.08 \times 10^{-5}}\right)$$

giving $R_* = 0.232$, which is equal to the value for a crest elevation of 6.25m ODN. This therefore becomes another feasible design — No. 2.

(iii) The two feasible designs obtained so far are therefore
No. 1 — seawall slope 1:3.4, crest elevation + 5.25m ODN
No. 2 — seawall slope 1:2.4, crest elevation + 6.25m ODN

There are of course an infinite number of intermediate designs, of which these two are the extremes. We therefore would now like to make a rough cost comparison between these two. Suppose for this example that we can assume that the cost of constructing the seawall is directly proportional to the volume of fill material required. Suppose also that the complete profile of the existing seawall consists of the 1:1.5 sloping seaward face, founded on the foreshore at a level of 0.0m ODN, a 5m wide crest at an elevation of 5.25m ODN, and founded on dry land at +3.25m ODN, and a backslope of 1:1.5 also founded on land at 3.25m ODN. (Fig 30). The volume of the existing seawall is therefore

$$\left[\frac{1}{2} \times 1.5 \times (5.25)^2\right] + \left[5 \times 2\right] + \left[\frac{1}{2} \times 1.5 \times 2^2\right] = 33.7\text{m}^3/\text{m run}$$

For design No. 1, assuming that the backslope is unchanged, the volume is 59.9m^3/m run, and for design No. 2 is 68.6m^3/m run. Design No. 1 therefore requires 26.2m^3/m of imported fill, and design No. 2 requires 34.9m^3/m, making design No. 2 approximately 33% more costly than No. 1 on the crude yardstick adopted.

(iv) In addition to these designs using simply sloping seawalls, there are also an infinite series of possible solutions using bermed seawalls. Suppose for example that we examine the possibility of retaining the existing crest elevation and seawall slope, and reduce the overtopping by placing a berm in front of the seawall. With the existing crest elevation we have $R_* = 0.161$, and also we know that $Q_{*_d} = 4.08 \times 10^{-5}$. We now scan through Figs 11 to 22 looking for possible solutions. Starting with a berm elevation of −2.0m SWL we see from Figs 11, 14 and 17 that berm widths of 5, 10 and 20m respectively give too high a dimensionless discharge at the existing dimensionless freeboard $R_* = 0.161$ and existing seawall slope 1:1.5, whereas Fig 19 shows a 40m berm giving too low a discharge. The required berm width for a berm elevation of −2.0m SWL (or +1.0m ODN) would therefore be somewhere between 20 and 40m — probably around 25m. This is a possible berm design — No. 1 — which can be examined in more detail when other possible berm solutions are identified. Repeating the scanning of Figs 11 to 22 for a −1.0m SWL berm elevation we see from Figs 12 and 15 that a berm width between 5 and 10m would be required — probably about 8.0m. Similarly for a berm at SWL Figs 9 and 11 show a berm width between 0 and 10m is required — probably about 5m.

The three possible berm designs with the existing seawall slope of 1:1.5 and existing crest elevation +5.25m ODN are therefore

No. 1 Berm elevation −2m SWL/+1.0m ODN, Berm width about 25m
No. 2 Berm elevation −1m SWL/+2.0m ODN, Berm width about 8m
No. 3 Berm elevation 0m SWL/+3.0m ODN, Berm width about 5m

If these berms are founded on the foreshore at an elevation of 0.0m ODN then the volume of fill required for No. 1 is about $25m^3$/m run, for No. 2 about $16m^3$/m, and for No. 3 about $15m^3$/m. Berm design No. 1 is therefore significantly more costly than either No. 2 or No. 3 so concentrate on detailing these last two.

The first stage is to establish the exact berm width required, which necessitates a trial and error process. Taking design No. 2 as an example, and using the estimated width of 8m, from Fig 24 we have for a seawall slope of 1:1.5 and a berm elevation of −1.0m SWL, $A = 1.96 \times 10^{-2}$, $B = 41.1$.

For the existing dimensionless crest elevation the dimensionless overtopping discharge would therefore be

$$Q_* = Ae^{-BR_*}$$
$$= (1.96 \times 10^{-2})\, e^{-41.1 \times 0.161}$$
$$= 2.62 \times 10^{-5}$$

This is lower than the design dimensionless discharge (4.08×10^{-5}) so the berm is unnecessarily wide. Therefore we try a width of 7m. From Fig 24, $A = 1.86 \times 10^{-2}$, $B = 38.7$, giving $Q_* = 3.66 \times 10^{-5}$. This is still too low, so we try 6.7m, giving $A = 1.83 \times 10^{-2}$, $B = 37.9$ and $Q_* = 4.10 \times 10^{-5}$. This is acceptably close to the design discharge of 4.08×10^{-5}, so the required berm width is 6.7m.

By a similar trial and error process we find that for berm design No. 3 the required berm width is 5.0m. The detailed berm designs are therefore:

No. 2 Crest elevation +5.5m ODN, slope 1:1.5, berm elevation 2.0m ODN, berm width 6.7m
No. 3 Crest elevation +5.5m ODN, slope 1:1.5, berm elevation 3.0m ODN, berm width 5.0m
and the volumes of fill required are $13.4m^3$/m and $15.0m^3$/m respectively, making berm design No. 2 marginally the more economical of the two designs.

(v) These berm designs all assume an unchanged seawall slope and crest elevation. Obviously we could repeat the process to find alternative designs using differing seawall slopes or crest elevations. For example, if the maintenance of the 1:1.5 slope of the existing seawall was proving troublesome, a decision might be taken to adopt a slope of say 1:2 for the new seawall. Repeating the calculations for a slope of 1:2 with the existing crest elevation would then give the following possible berm dimensions:-

Berm elevation −2.0m SWL/+1.0m ODN, Berm width 19.4m
Berm elevation −1.0m SWL/+2.0m ODN, Berm width 5.3m
Berm elevation 0.0m SWL/+3.0m ODN, Berm width 4.0m

Again a berm elevation of −1.0m SWL gives marginally the most economical design in terms of the quantity of fill required, needing $17.5m^3$/m run for flattening the seawall slope and constructing the berm.

(vi) After carrying out the necessary calculations for several feasible designs for the new seawall it is usually possible to pick out what promises to be the most economical design. In this example it looks as though the least costly design would be:-

Seawall slope 1:1.5 (existing), crest elevation +5.25m ODN (existing), Berm elevation +2.0m ODN, Berm width 6.7m. The existing seawall and the proposed improvement are shown in Fig 30. This design would then be subjected to a much more detailed analysis as outlined earlier.

* *

In the above example, a design discharge of 2 x 10^{-3}m³/s/m (2 litres/s/m run) was used. This is the figure which is sometimes used in the design of seawalls in Holland, as an alternative to the design method where the seawall is required to limit the number of waves overtopping to 2 per cent (See section on design discharge).

ADDITIONAL COMMENTS

Although the preceding chapters give all the information necessary for evaluating existing seawalls or designing new ones, a few additional comments may be helpful.

1. In situations where it is expected that even the most severe waves will approach the seawall without breaking it will sometimes be more convenient to use the alternative forms for the dimensionless discharge Q_* and the dimensionless freeboard R_*, as

$$Q_* = \frac{\bar{Q}}{\sqrt{gH_s^3}}\sqrt{\frac{S}{2\pi}}$$

and

$$R_* = \frac{R_c}{H_s}\sqrt{\frac{S}{2\pi}}$$

This is because the value of the wave steepness S may be changed from storm to storm, and it may therefore be unnecessary to calculate the wave period directly.

2. Where the seawall is subject to breaking waves in all major storms it is unnecessary to carry out detailed calculations to determine the offshore wave height: all that is required is a check that the waves are sufficiently large that they will break before hitting the seawall. However in these circumstances the derivation of wave period retains a great importance, since the overtopping discharge depends strongly on wave period.

3. Since wave period is such an important parameter in deriving the overtopping discharge, and since it can vary substantially for a given wave height, it is probably worth further comment. All commonly used forecasting methods for predicting wave period are for the situation where the waves are controlled directly by the wind — the so-called wind sea. For all these wind seas the wave steepness S is reasonably constant, so that the significant wave height H_s and the mean zero-crossing wave period $\bar{T}$ are uniquely related. Most forecasting methods predict wave steepness S values of about 0.05. In practical terms this means that the forecasting methods apply throughout the growth period of a storm, over the peak, and possibly for the initial decay of the storm. However during later stages of the storm the waves are no longer directly related to the contemporary windspeed, the so-called swell sea. For swell seas there is a wide range of wave steepness values, so that wave heights and periods are no longer directly related, and normal forecasting techniques cannot be applied.

If instead of using forecasting methods the waves are measured directly, then scatter diagrams such as Fig 4 also show a wide range of wave steepnesses. This of course is to be expected since the recorded waves include those occurring during the decay of a storm as well as those occurring during the growth and at the peak of the storm. Strong tidal currents can also affect the wave steepness. When such direct wave measurements are available then the overtopping discharge should preferably be calculated for the lowest likely wave steepness, ie the longest likely wave period. In Fig 4 this steepness would probably be taken as 0.035.

4. In many cases it may be worthwhile taking a few rudimentary measurements at an existing seawall, and carrying out some preliminary calculations before embarking on a full-scale survey of the seawalls and foreshore. Such preliminary calculations might for example identify whether or not wave breaking was likely to occur, and hence to determine the need for a detailed survey of the foreshore. The exact slope of the foreshore will be more important where wave breaking occurs.

5. Finally it should be emphasised that no laboratory or full scale measurements exist to confirm the method proposed for adjusting overtopping discharges when the seawalls are rough. This is particularly so for bermed seawalls, where very few measurements are available to determine the effect of roughness even on wave run-up, let alone on overtopping discharge. The roughness adjustment should therefore be used with caution, particularly for bermed seawalls.

CONCLUSION

Throughout this report the emphasis has been placed on designing seawalls on the basis of overtopping discharge, rather than by wave run-up on the wall or by a certain percentage of waves overtopping the wall. This approach has been adopted because the design engineer is far more concerned with the volume of water overtopping the seawall rather than the number of waves coming over, since he has to determine the capacity of the drainage system, or the possible depth of flooding. This approach is a relatively recent development, and further studies will undoubtedly extend knowledge of overtopping discharges at composite and rough seawalls, of overtopping due to wind-blown spray, and of the use of the data to determine return periods of given overtopping discharges.

In the meantime, the design procedures outlined in this report enable the evaluation of the overtopping discharge with irregular waves for simple and bermed seawalls, and allow estimates of the effects of angle of wave attack and of seawall roughness. For more complicated seawall geometries, such as those equipped with wave return walls, it will probably be necessary for specific hydraulic model tests to be carried out to determine the overtopping discharge, or the suitability of a particular design.

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NOTATION

A,B Coefficients in the dimensionless equation $Q_* = Ae^{-BR_*}$. The coefficients are functions of seawall geometry.

d Water depth

d_g Water depth in the wave generation zone

d_s Water depth at the toe of the seawall

d_B Water depth on the seawall berm

d_t The water depth at which the slope changes in a composite seawall

F Effective fetch length

g Acceleration due to gravity

H_s Significant wave height. In a sequence of irregular waves, a zero-crossing occurs whenever the water surface crosses the mean water level. The wave height is then defined as the vertical distance from the lowest trough to the highest crest between successive zero-down crossings (see Fig 31). The significant wave height is the mean height of the highest ⅓ of those waves

H_{sb} The effective value of significant wave height after breaking occurs

H_{sg} Significant wave height in the wave generation zone

H_{so} Significant wave height offshore in deep water

H_{si} Significant wave height inshore

K_R Refraction coefficient: the factor by which offshore wave heights are modified by wave refraction as they approach inshore

K_S Shoaling coefficient — the factor by which overtopping discharges are increased by onshore winds

K_W Wind correction factor — the factor by which overtopping discharges are increased by onshore winds

$\bar{L}$ The mean wavelength of a sequence of irregular waves

$\bar{L}_o$ Mean wavelength in deep water offshore, $L_o = gT^2/(2\pi)$

$\bar{L}_s$ Mean wavelength at the water depth at the seawall toe

$\bar{L}_B$ Mean wavelength at the water depth on the seawall berm

1:m The slope of the seaward face of the seawall (vertical:horizontal)

1:m_1 The slope of the seaward face of the berm, or of the lower portion of a composite seawall

1:m_2 The slope of the seaward face of the seawall for a bermed seawall, or of the upper portion of a composite seawall

1:m_f The slope or gradient of the foreshore

N The number of waves overtopping a seawall as a proportion of the number of waves incident upon that seawall

Q Overtopping discharge. In this report all discharges are the average discharge over a sequence of 100 waves

$\bar{Q}$ Mean overtopping discharge: the mean value of several measurements of overtopping discharge, each obtained over a sequence of 100 waves

Q_* Dimensionless overtopping discharge $Q_* = Q/(\bar{T}gH_s)$

R_c Seawall crest elevation relative to still water level

R_s The height to which the significant wave would run up the seawall if that seawall were sufficiently high to prevent overtopping

R_{sm} The height of wave run-up on a smooth seawall

R_r The height of wave run-up on a rough seawall

R_* The dimensionless freeboard, $R_* = R_c/(\bar{T}\sqrt{gH_s})$

r The roughness coefficient for a rough seawall

S The mean steepness of a sequence of irregular waves, $S = H_s/\bar{L}_o$

$\bar{T}$ The mean wave period. The wave period is the time between two successive zero down-crossings (Fig 31). The mean wave period is the average value for a sequence of waves. In this report all wave periods are for deep water

t The wind duration, or the time for which the wind has been blowing with a given speed and from a given direction

$\triangle t$ Time step

U The windspeed at an elevation of 10m above mean water level

W_B The width of the seawall berm, measured perpendicular to the seawall

W_f The wind factor, with a value depending on the onshore component of wind speed

α The angle between the direction of wave travel and the normal to the seabed contours

α_o The value of α in deep water offshore

α_g The value of α in the wave generation zone

α_i The value of α inshore (after wave refraction)

β The angle between the direction of wave travel and the normal to the seawall

γ The ratio of equivalent post-breaking wave height to water depth at the seawall toe

$\sigma_{\ln Q_*}$ The standard deviation of measured values of $\ln Q_*$ about a mean value in the expression $Q_* = Ae^{-BR_*}$, or $\ln Q_* = \ln A - BR_*$

θ The angular slope of the seawall, or $\tan\theta = 1/m$

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TABLES

TABLE 1 VALUES OF THE COEFFICIENTS A AND B FOR SIMPLE SEAWALLS

Seawall slope	**A**	**B**
1:1	7.94×10^{-3}	20.12
1:1½	*1.02×10^{-2}*	*20.12*
1:2	1.25×10^{-2}	22.06
1:2½	*1.45×10^{-2}*	*26.1*
1:3	*1.63×10^{-2}*	*31.9*
1:3½	*1.78×10^{-2}*	*38.9*
1:4	1.92×10^{-2}	46.96
1:4½	*2.15×10^{-2}*	*55.7*
1:5	*2.5×10^{-2}*	*65.2*

Bold type — values determined from model tests[1]

Italic type — values derived by interpolation based on published run-up data[8]

TABLE 2 VALUES OF THE COEFFICIENTS A AND B FOR BERMED SEAWALLS

Seawall slope	Berm elevation m SWL	Berm width m	A	B
1:1	- 4.0	10	6.40×10^{-3}	19.50
1:2			9.11×10^{-3}	21.50
1:4			1.45×10^{-2}	41.10
1:1	- 2.0	5	3.40×10^{-3}	16.52
1:2			9.80×10^{-3}	23.98
1:4			1.59×10^{-2}	46.63
1:1	- 2.0	10	4.79×10^{-3}	18.92
1:2			6.78×10^{-3}	24.20
1:4			8.57×10^{-3}	45.80
1:1	- 2.0	20	8.80×10^{-4}	14.76
1:2			2.00×10^{-3}	24.81
1:4			8.50×10^{-3}	50.40
1:1	- 2.0	40	3.80×10^{-4}	22.65
1:2			5.00×10^{-4}	25.93
1:4			4.70×10^{-3}	51.23
1:1	- 2.0	80	2.40×10^{-4}	25.90
1:2			3.80×10^{-4}	25.76
1:4			8.80×10^{-4}	58.24
1:1	- 1.0	5	1.55×10^{-2}	32.68
1:2			1.90×10^{-2}	37.27
1:4			5.00×10^{-2}	70.32
1:1	- 1.0	10	9.25×10^{-3}	38.90
1:2			3.39×10^{-2}	53.30
1:4			3.03×10^{-2}	79.60
1:1	- 1.0	20	7.50×10^{-3}	45.61
1:2			3.40×10^{-3}	49.97
1:4			3.90×10^{-3}	61.57
1:1	- 1.0	40	1.20×10^{-3}	49.30
1:2			2.35×10^{-3}	56.18
1:4			1.45×10^{-4}	63.43
1:1	- 1.0	80	4.10×10^{-5}	51.41
1:2			6.60×10^{-5}	66.54
1:4			5.40×10^{-5}	71.59
1:1	0.0	10	9.67×10^{-3}	41.90
1:2			2.90×10^{-2}	56.70
1:4			3.03×10^{-2}	79.60

All values determined from model tests(1)

TABLE 3 ROUGHNESS VALUES FOR VARIOUS TYPES OF SEAWALL CONSTRUCTION

Seawall construction	Roughness value r
Smooth, Impermeable	1.0
Stone blocks, pitched or mortared	0.95
Concrete blocks	0.9
Stone blocks, granite sets	0.85 to 0.9
Turf	0.85 to 0.9
Rough concrete	0.85
One layer of stone rubble on impermeable base	0.8
Stones set in cement, ragstone etc	0.75 to 0.8
Dumped round stones	0.6 to 0.65
Two or more layers or rubble	0.5 to 0.6

TABLE 4 CALCULATION OF DISCHARGE CURVE FOR EXAMPLE 8

Still Water Level m ODN	3.0	2.75	2.5	2.25	2.0	1.75
Berm elevation relative to SWL d_B:m	−1.3	−1.05	−0.80	−0.55	−0.3	−0.05
A	1.96×10^{-2}	2.67×10^{-2}	2.77×10^{-2}	2.68×10^{-2}	2.60×10^{-2}	$2.51 \times 10^{-[illegible]}$
B	42.3	48.1	50.0	51.0	51.9	52.8
Water depth at toe, d_s:m	3.0	2.75	2.5	2.25	2.0	1.75
H_s/d_s	0.58	0.64	0.70	0.78	0.88	1.0
$d_s/\bar{L}_o$	0.077	0.070	0.064	0.058	0.051	0.045
γ	0.76	0.79	0.82	0.86	0.89	0.93
Effective H_s	1.75	1.75	1.75	1.75	1.75	1.63
Crest elevation relative to SWL R_c:m	2.5	2.75	3.0	3.25	3.5	3.75
$R_* = R_c/(\bar{T}\sqrt{gH_s})$	0.1207	0.1327	0.1448	0.1569	0.1689	0.1876
$Q_* = Ae^{-BR_*}$	1.19×10^{-4}	4.51×10^{-5}	1.99×10^{-5}	8.97×10^{-6}	4.06×10^{-6}	$1.25 \times 10^{-[illegible]}$
$\bar{Q} = Q_* \bar{T} g H_s$	1.02×10^{-2}	3.87×10^{-3}	1.71×10^{-3}	7.70×10^{-4}	3.48×10^{-4}	$1.00 \times 10^{-[illegible]}$

Seawall slope	1:2.2
Crest elevation	+5.5m ODN
Berm elevation	+1.7m ODN
Berm width	8.0m
Toe elevation	0.0m ODN

Significant wave height H_s 1.75m

Mean wave period $\bar{T}$ 5.0s

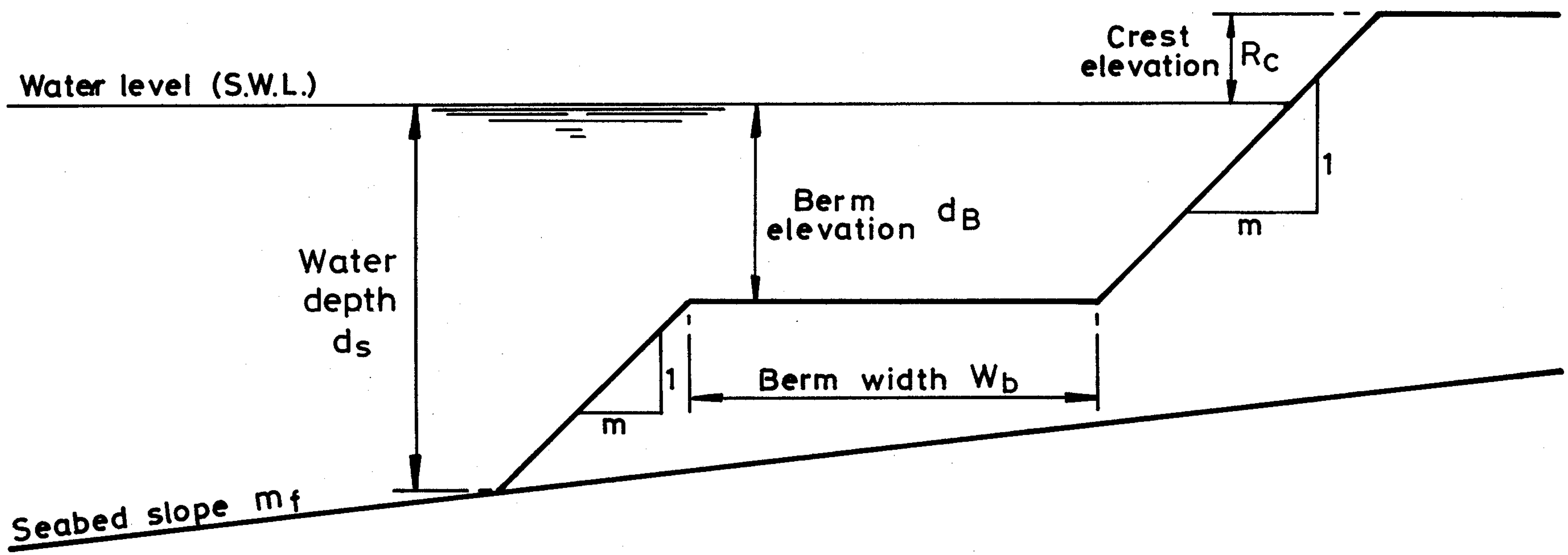
Deepwater wavelength $\bar{L}_o = \dfrac{g\bar{T}^2}{2\pi}$

TABLE 5 TOTAL OVERTOPPING VOLUME FOR EXAMPLE 8

Time relative to High Water	Still Water Level relative to High Water level:m	Still Water Level m ODN	Overtopping Discharge Q $m^3/s/m$	Overtopping Volume m^3/m
− 1hr 0m	− 1.3	+ 1.70	7.3×10^{-5}	0.07
− 0hr 45m	− 0.8	+ 2.20	6.9×10^{-4}	0.62
− 0hr 30m	− 0.4	+ 2.60	2.3×10^{-3}	2.07
− 0hr 15m	− 0.12	+ 2.88	6.3×10^{-3}	5.67
HW	0	+ 3.00	1.02×10^{-2}	9.18
+ 0hr 15m	− 0.15	+ 2.85	5.6×10^{-3}	5.04
+ 0hr 30m	− 0.56	+ 2.44	1.4×10^{-3}	1.26
+ 0hr 45m	− 1.12	+ 1.88	2.0×10^{-4}	0.18

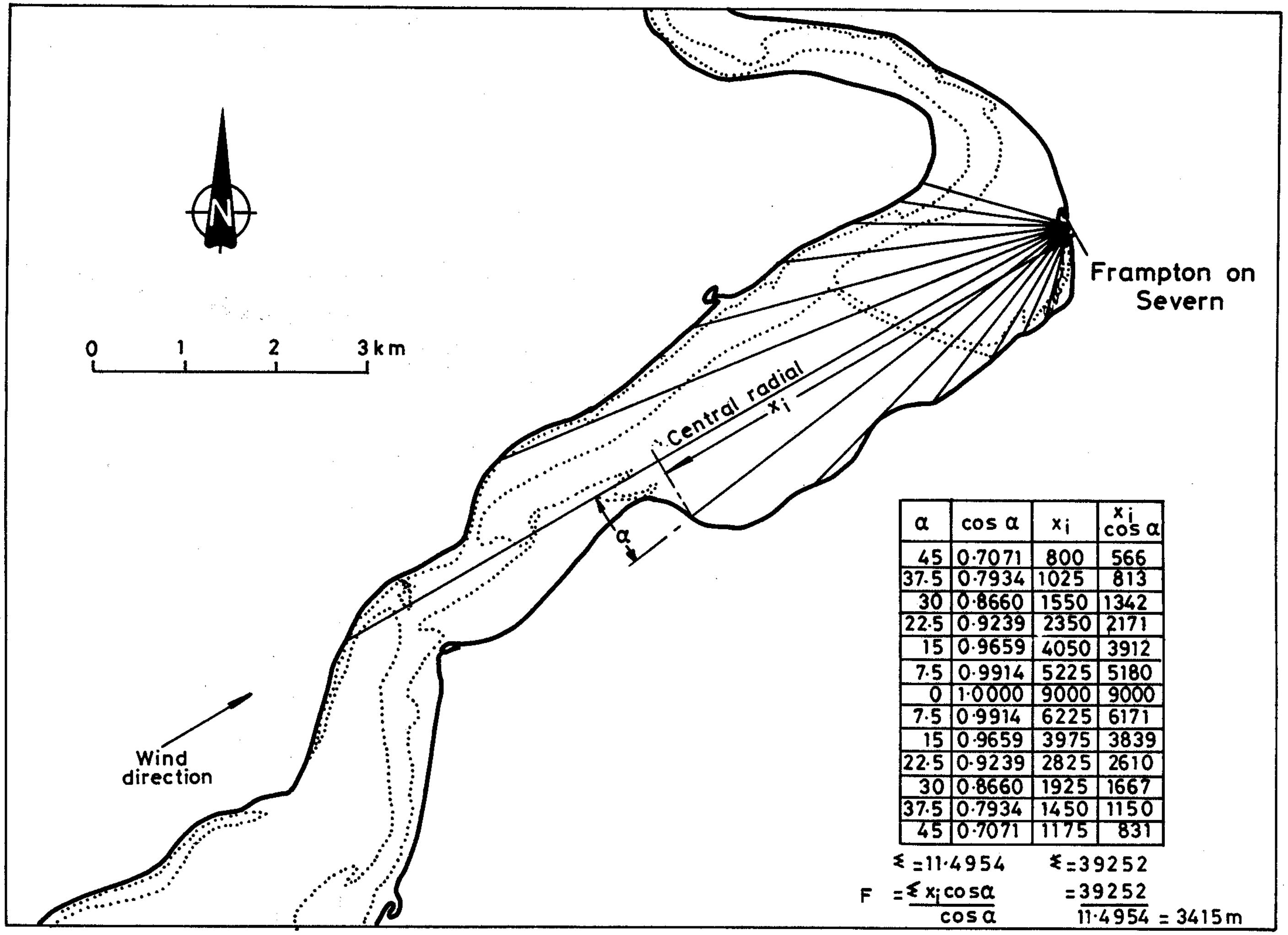
Total overtopping
Volume per tide 24.09m^3/m run of seawall

FIGURES



Generalised seawall profile

Fig.1



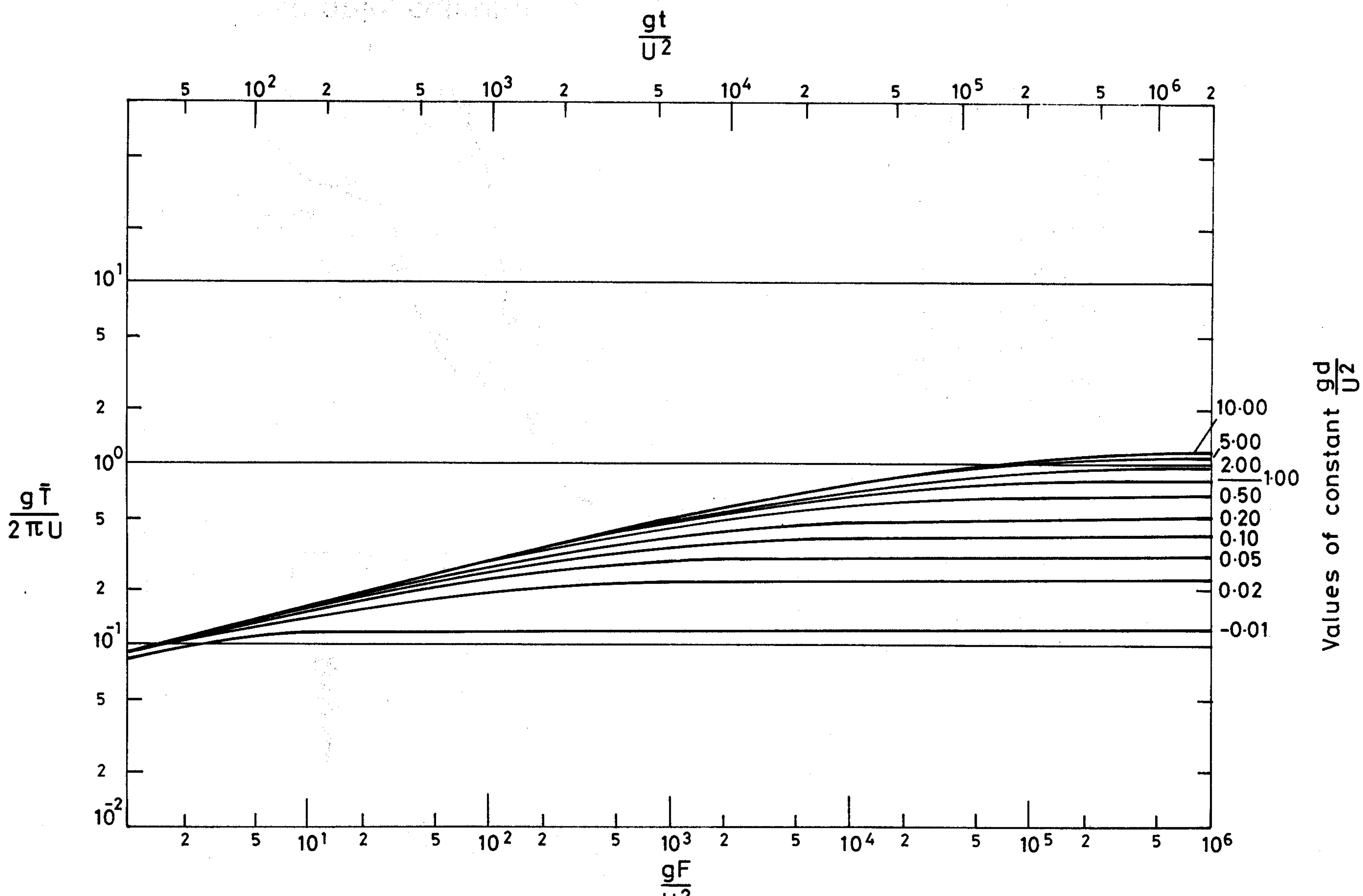
α	cos α	x_i	$x_i \cos\alpha$
45	0·7071	800	566
37·5	0·7934	1025	813
30	0·8660	1550	1342
22·5	0·9239	2350	2171
15	0·9659	4050	3912
7·5	0·9914	5225	5180
0	1·0000	9000	9000
7·5	0·9914	6225	6171
15	0·9659	3975	3839
22·5	0·9239	2825	2610
30	0·8660	1925	1667
37·5	0·7934	1450	1150
45	0·7071	1175	831

$\Sigma = 11{\cdot}4954 \qquad \Sigma = 39252$

$$F = \frac{\Sigma x_i \cos\alpha}{\cos\alpha} = \frac{39252}{11{\cdot}4954} = 3415\,m$$

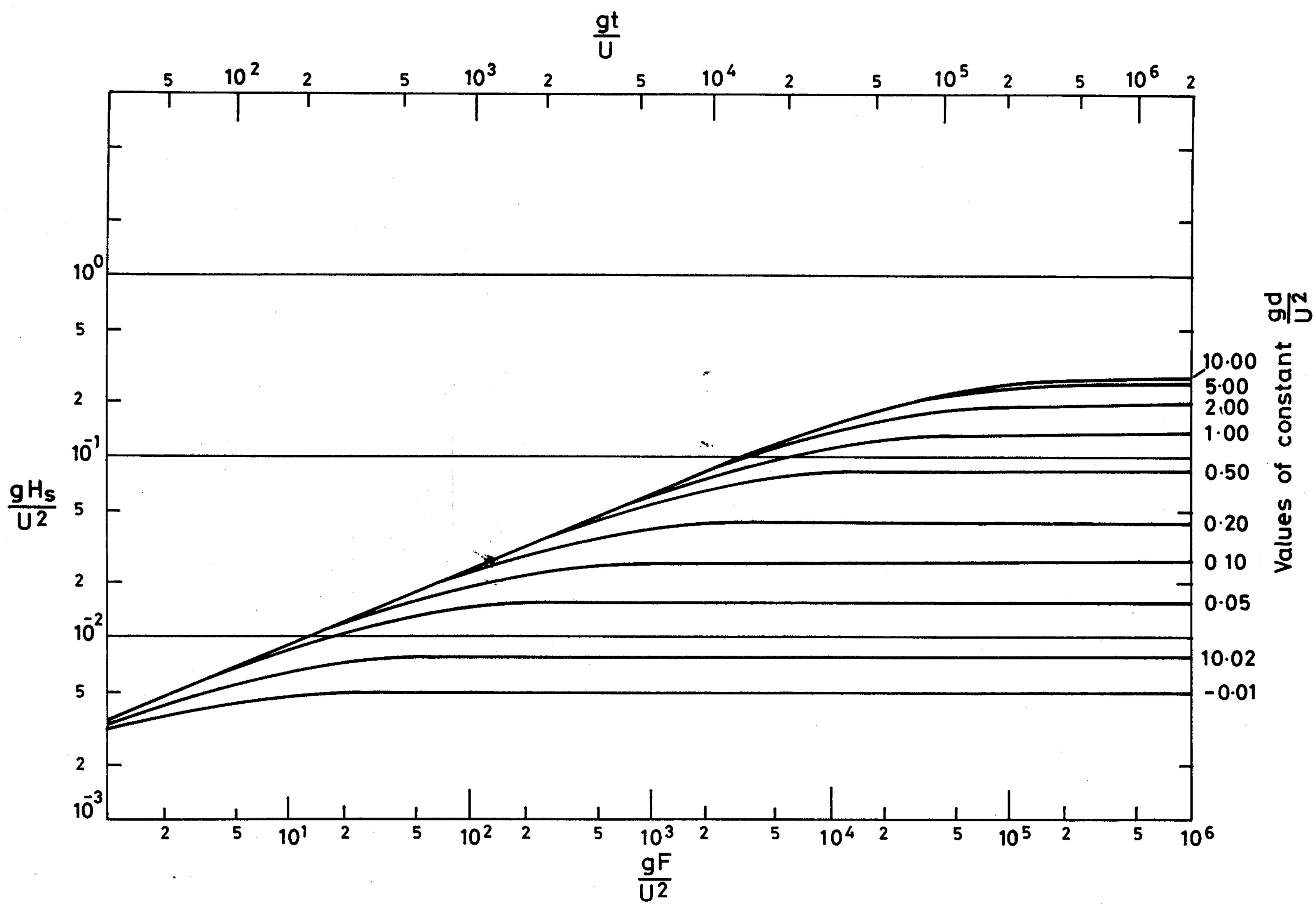
Example calculation of effective fetch length

Fig.2



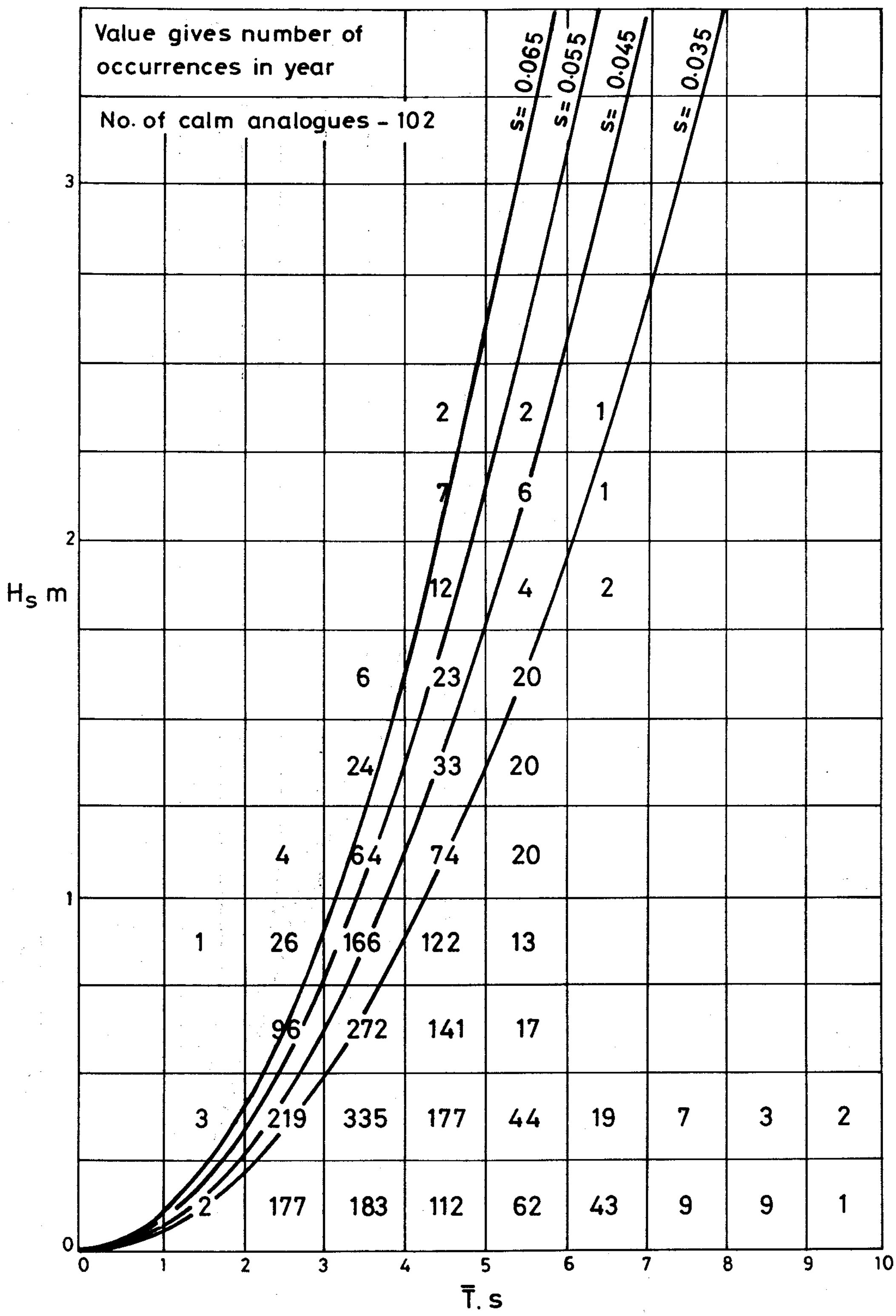
Dimensionless wave forecasting curves

Fig.3(a)



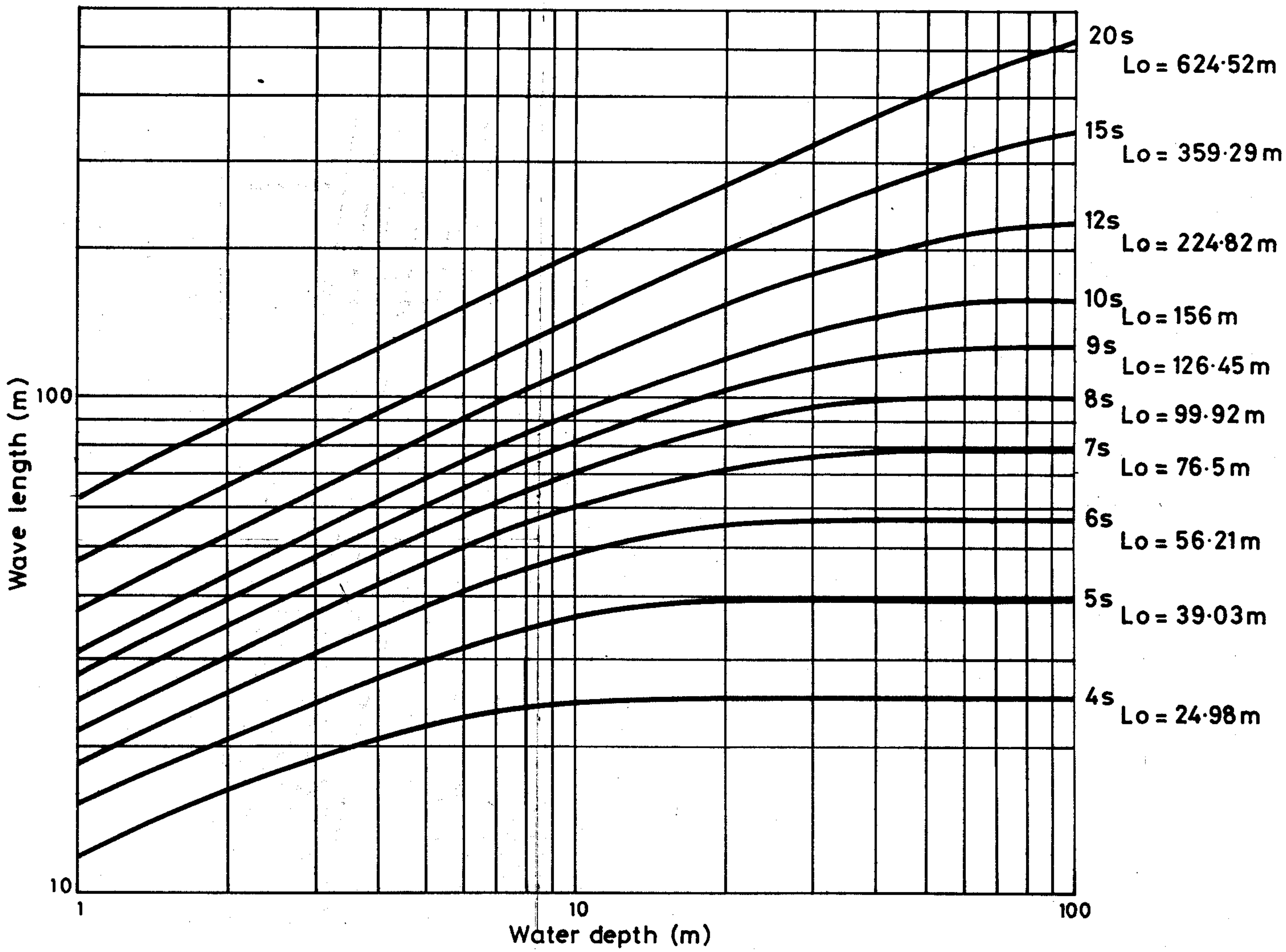
Dimensionless wave forecasting curves

Fig.3(b)



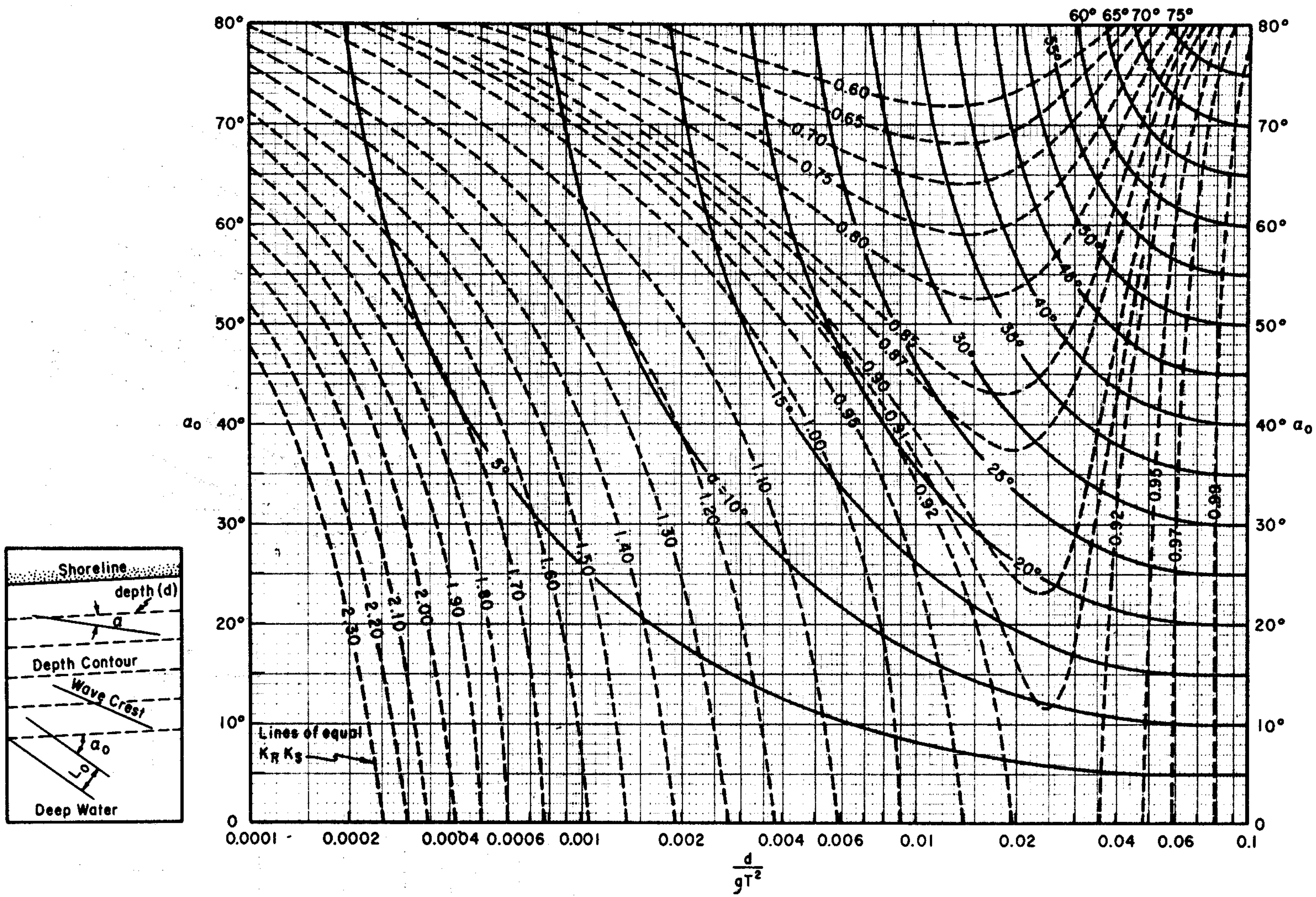
Scatter diagram of wave heights and periods

Fig.4



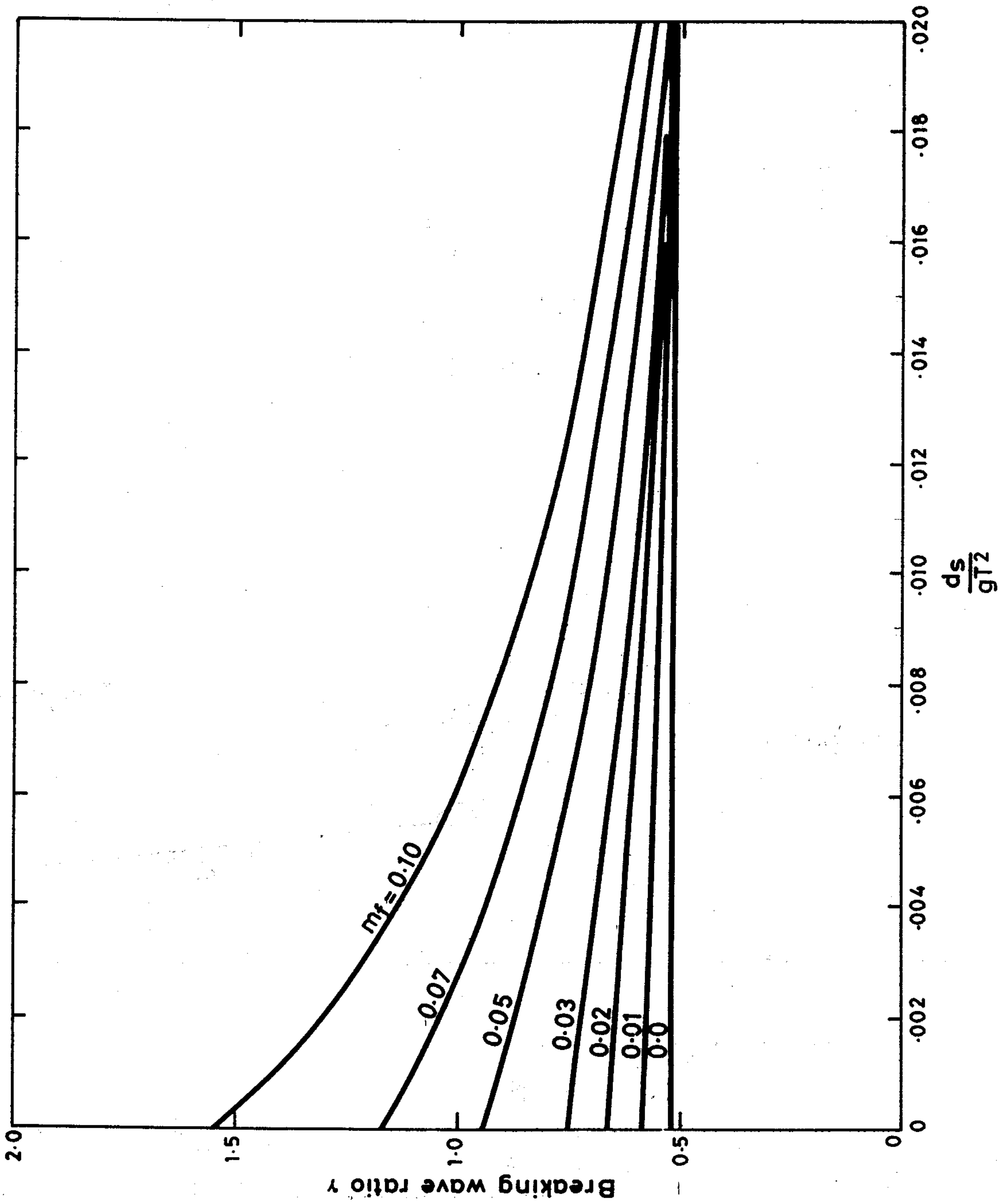
Wave length in varying water depths

Fig.5



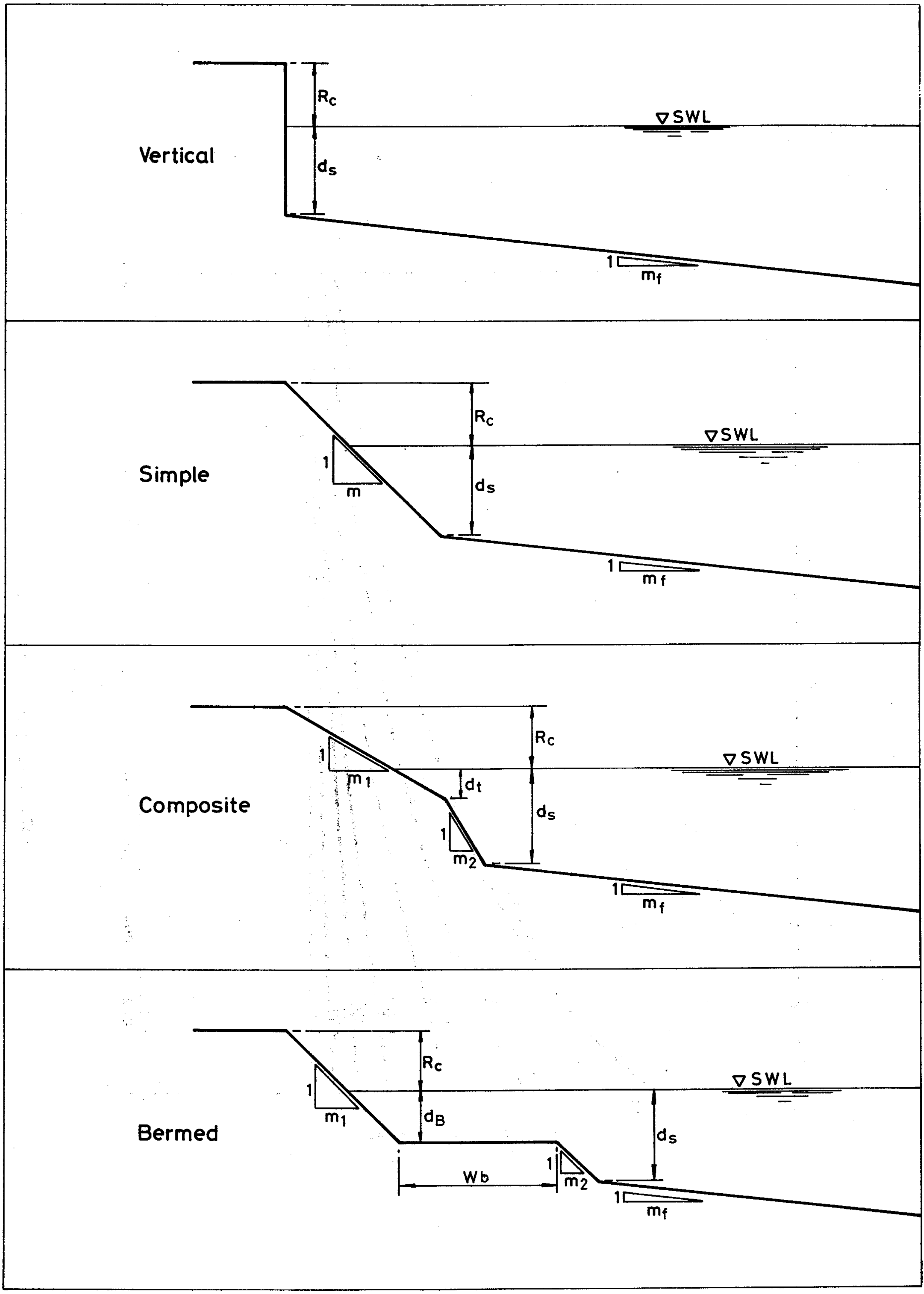
Simple wave refraction and shoaling

Fig.6



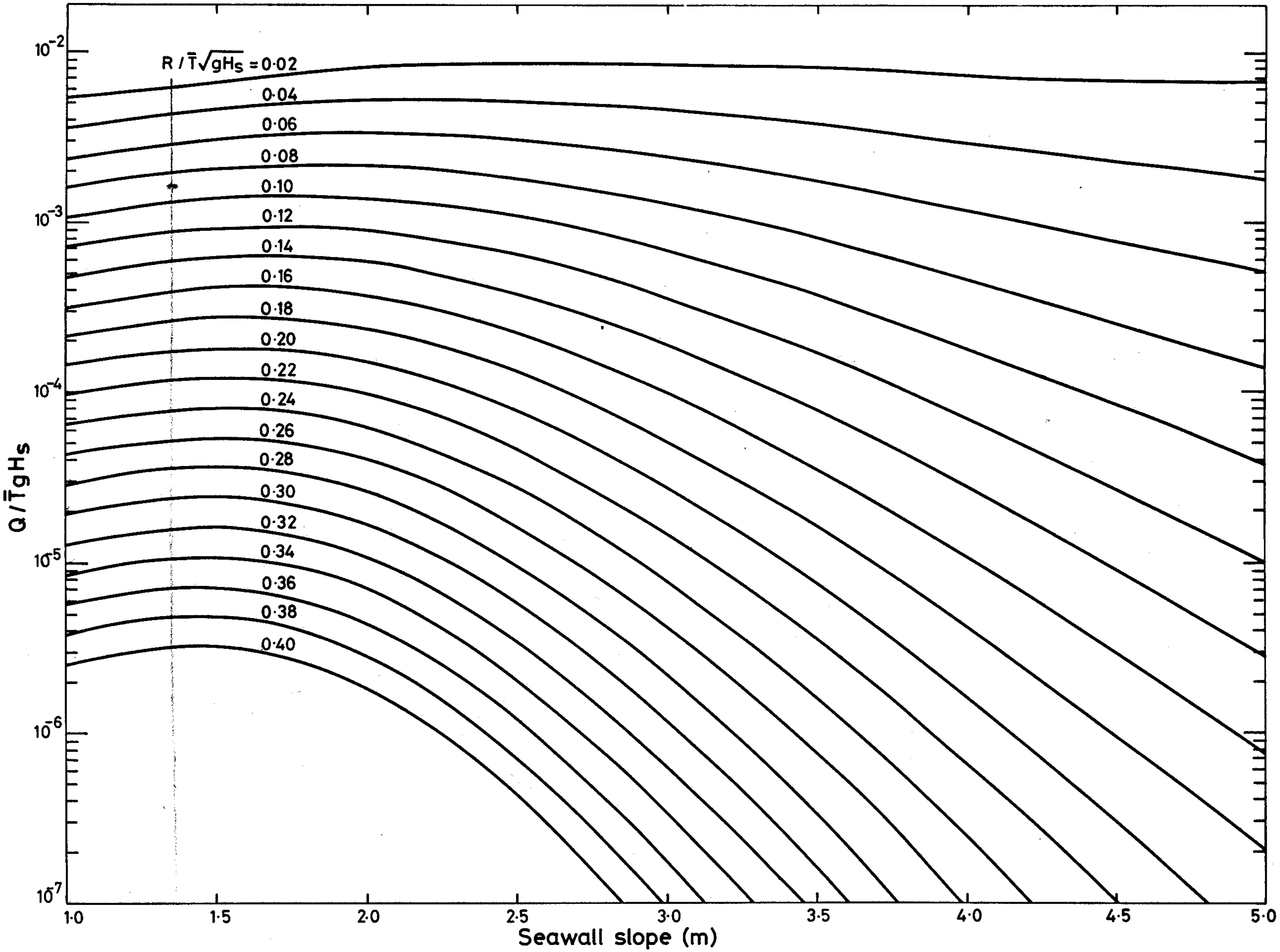
Breaking wave ratio

Fig.7



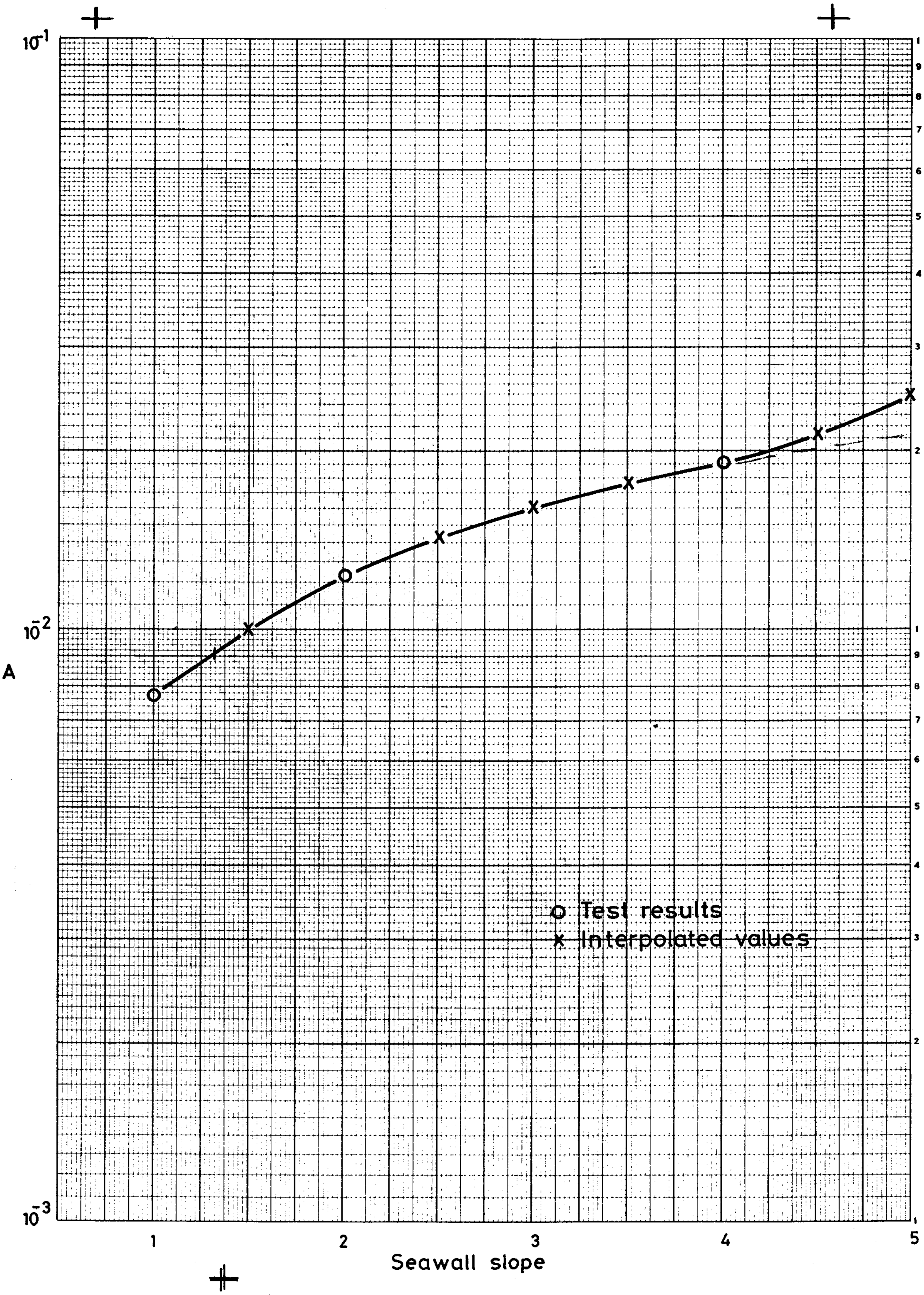
Seawalls of various types

Fig. 8



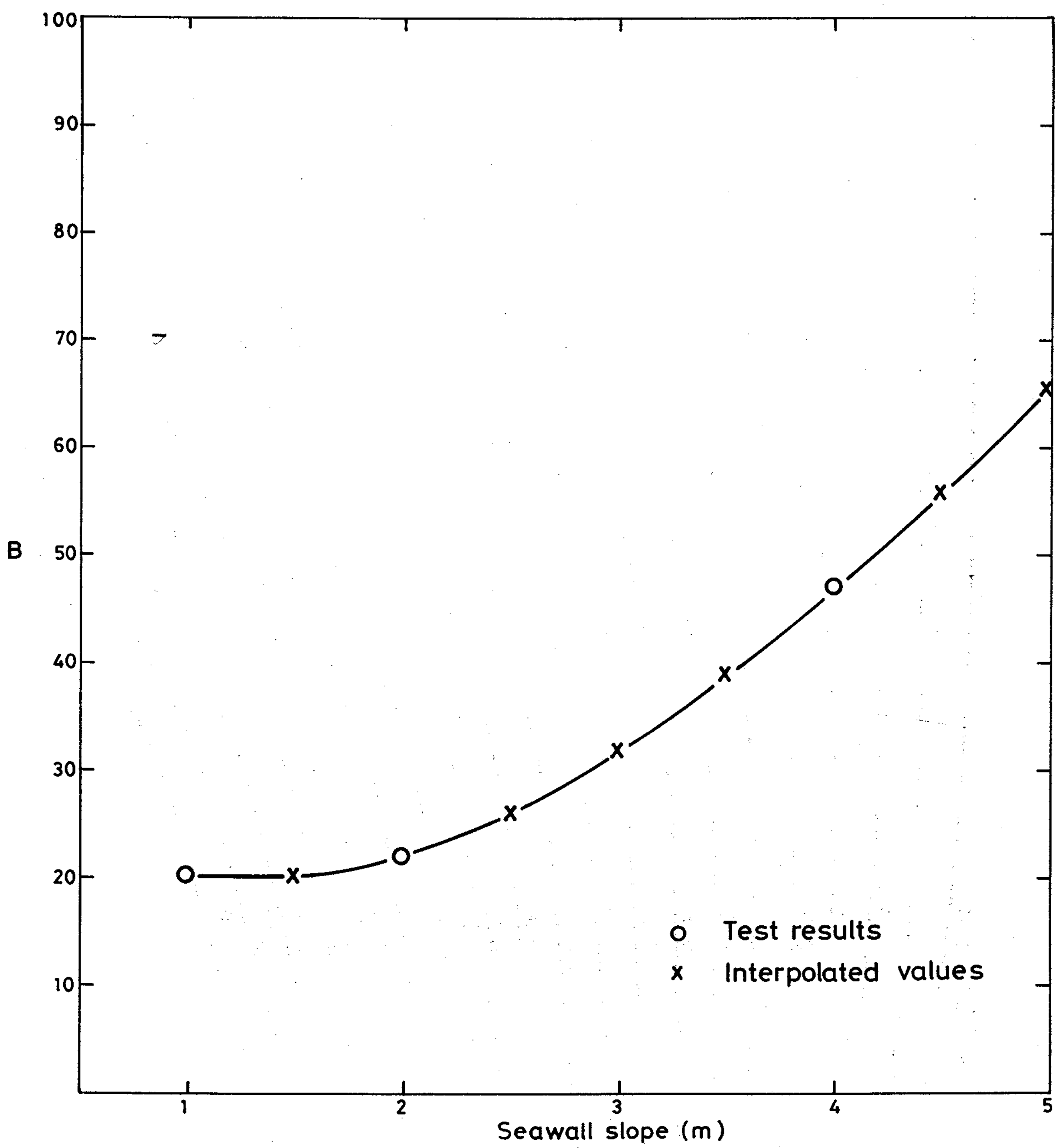
Discharge over simple seawalls

Fig.9



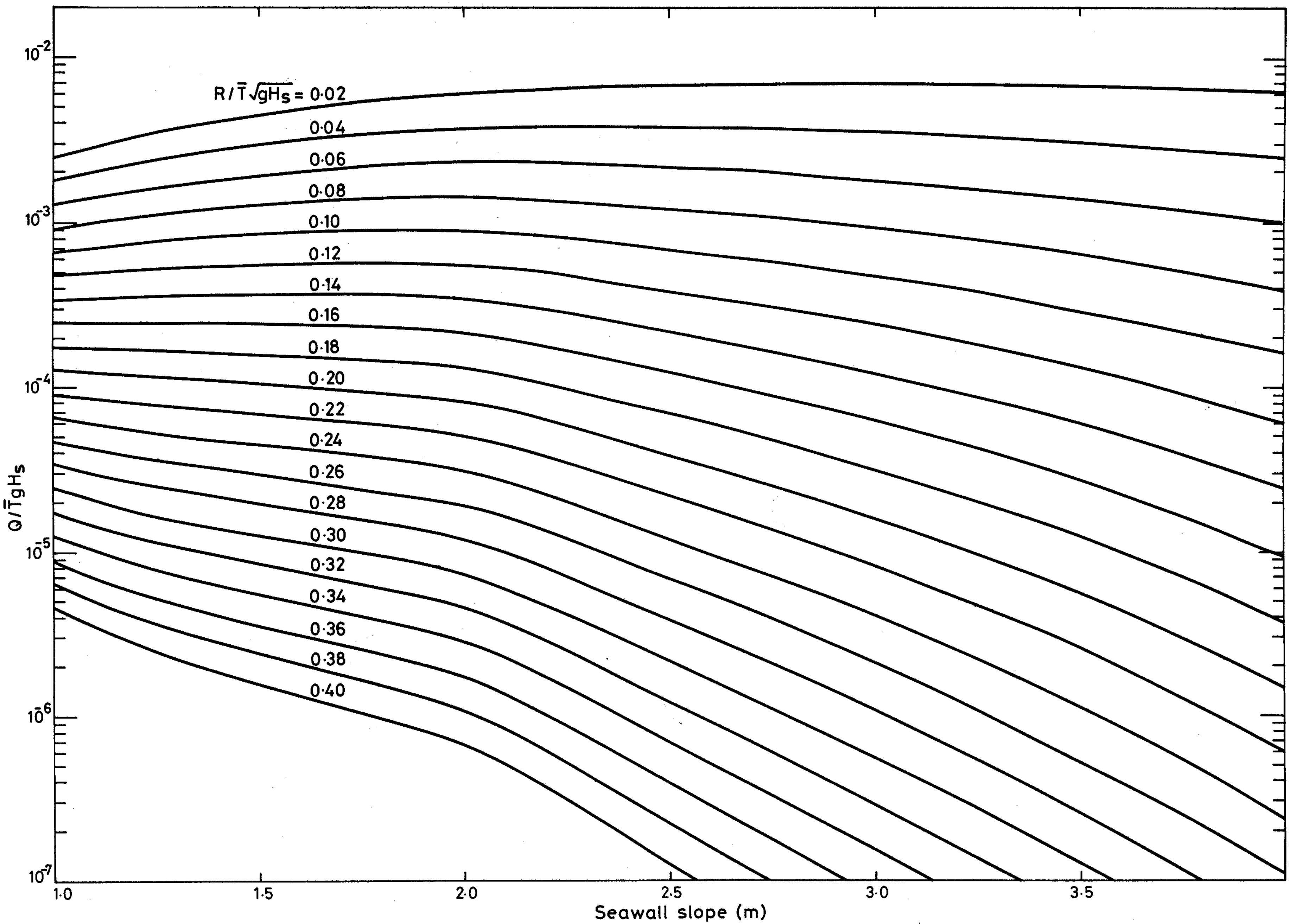
Coefficients A and B for simple seawalls

Fig.10a



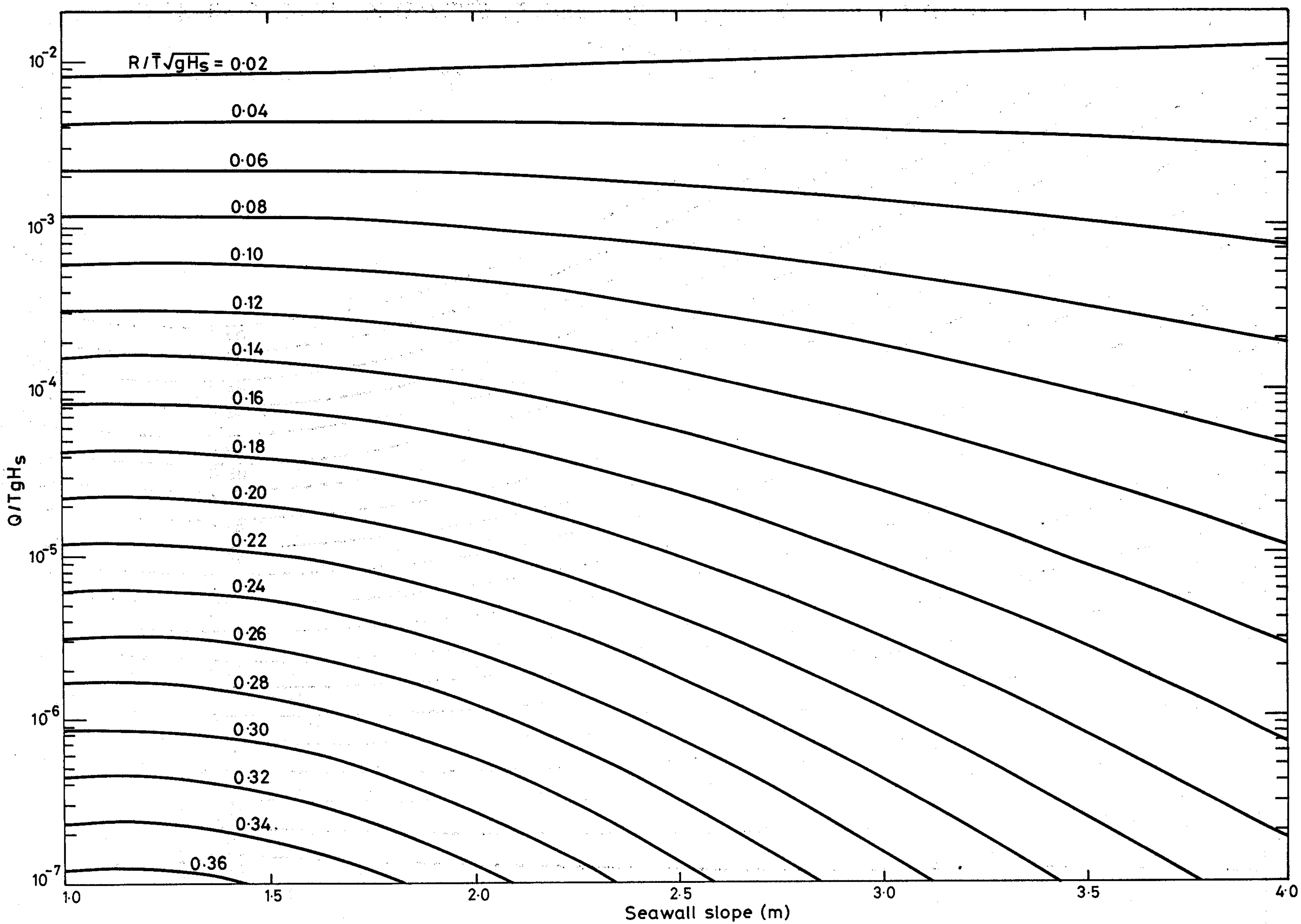
Coefficients A and B for simple seawalls

Fig.10b



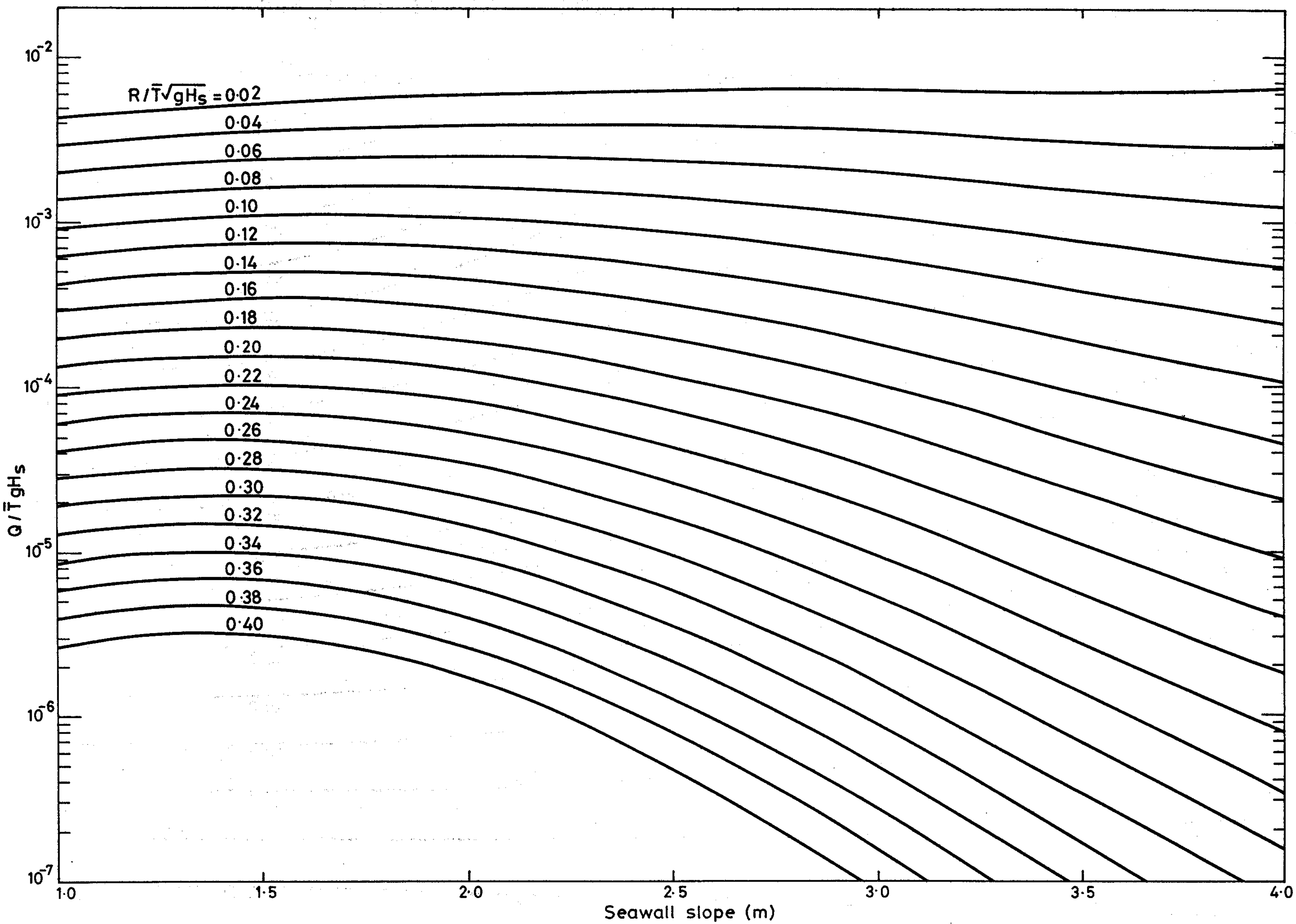
Discharge over seawalls : berm width 5m and elevation 2m below SWL

Fig.11



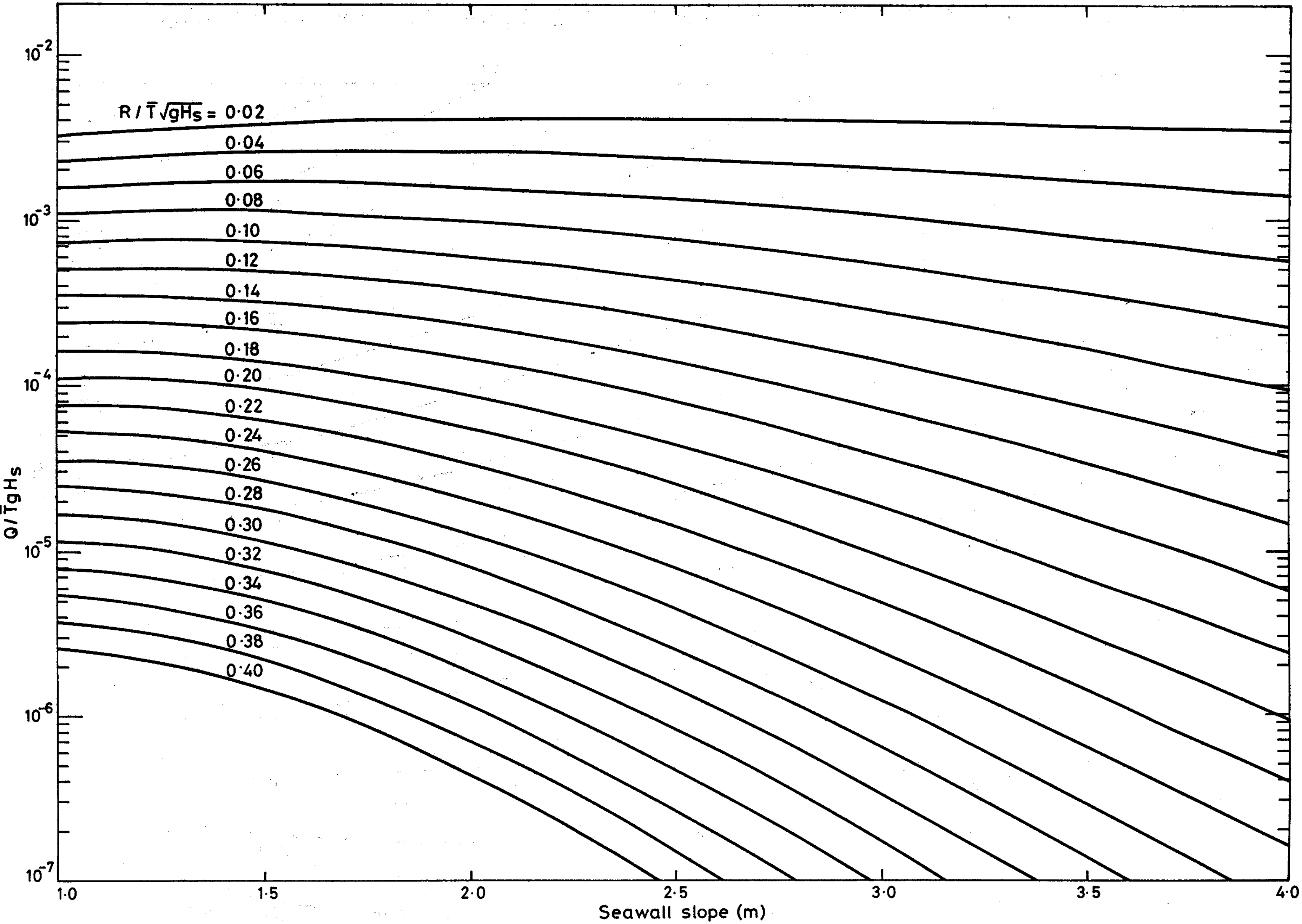
Discharge over seawalls : berm width 5m and elevation 1m below SWL

Fig.12



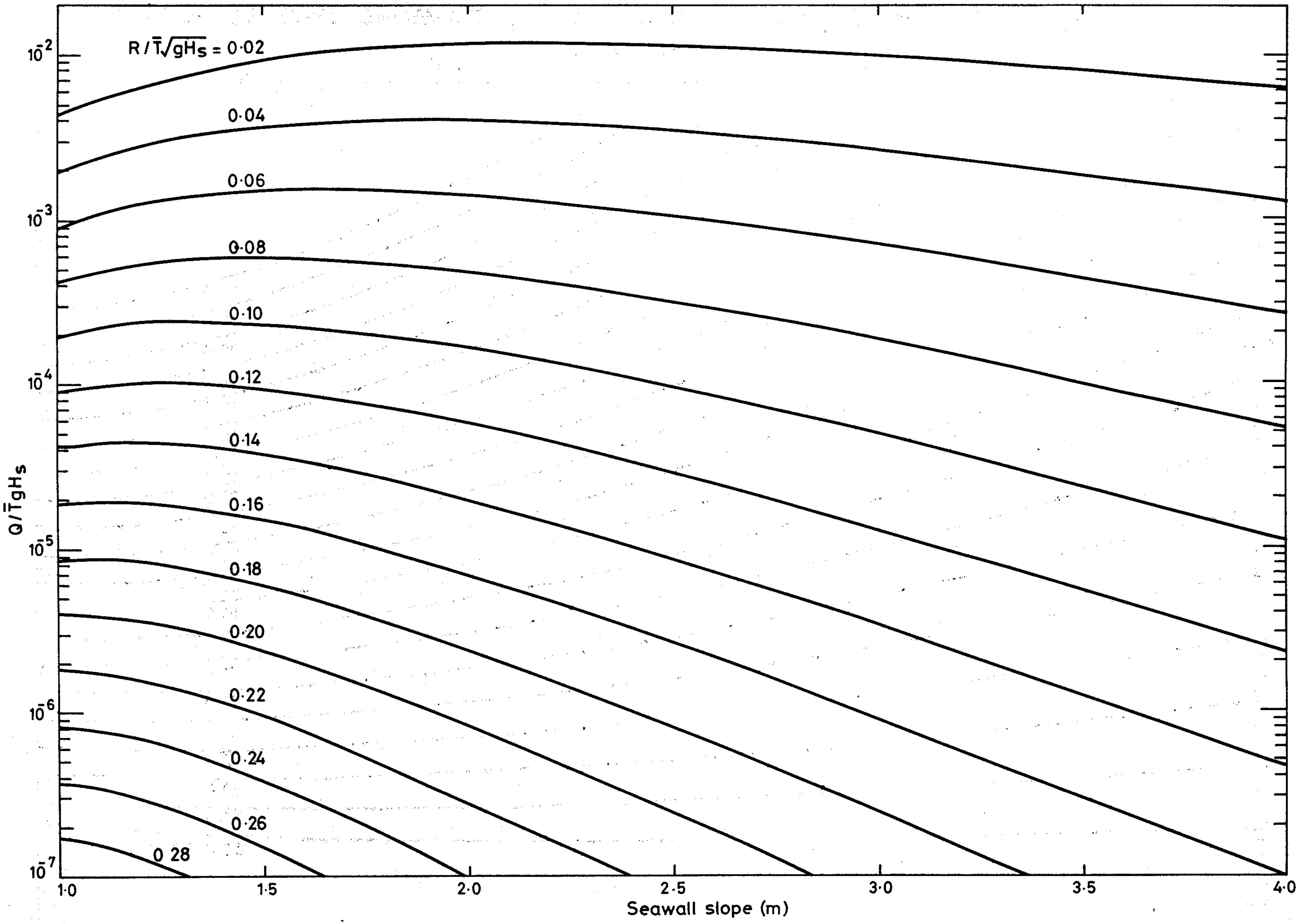
Discharge over seawalls : berm width 10m and elevation 4m below SWL

Fig.13



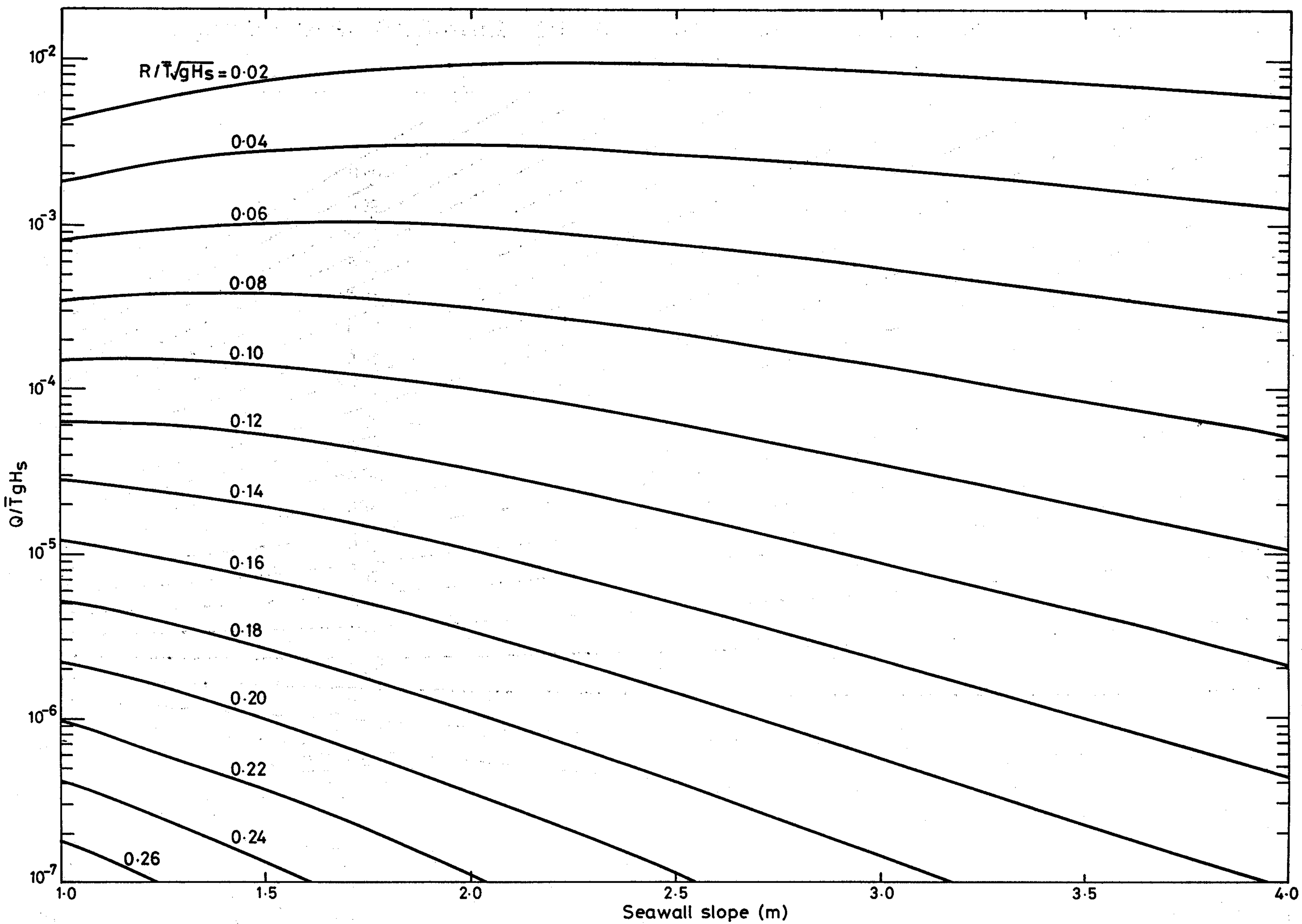
Discharge over seawalls: berm width 10m and elevation 2m below SWL

Fig. 14



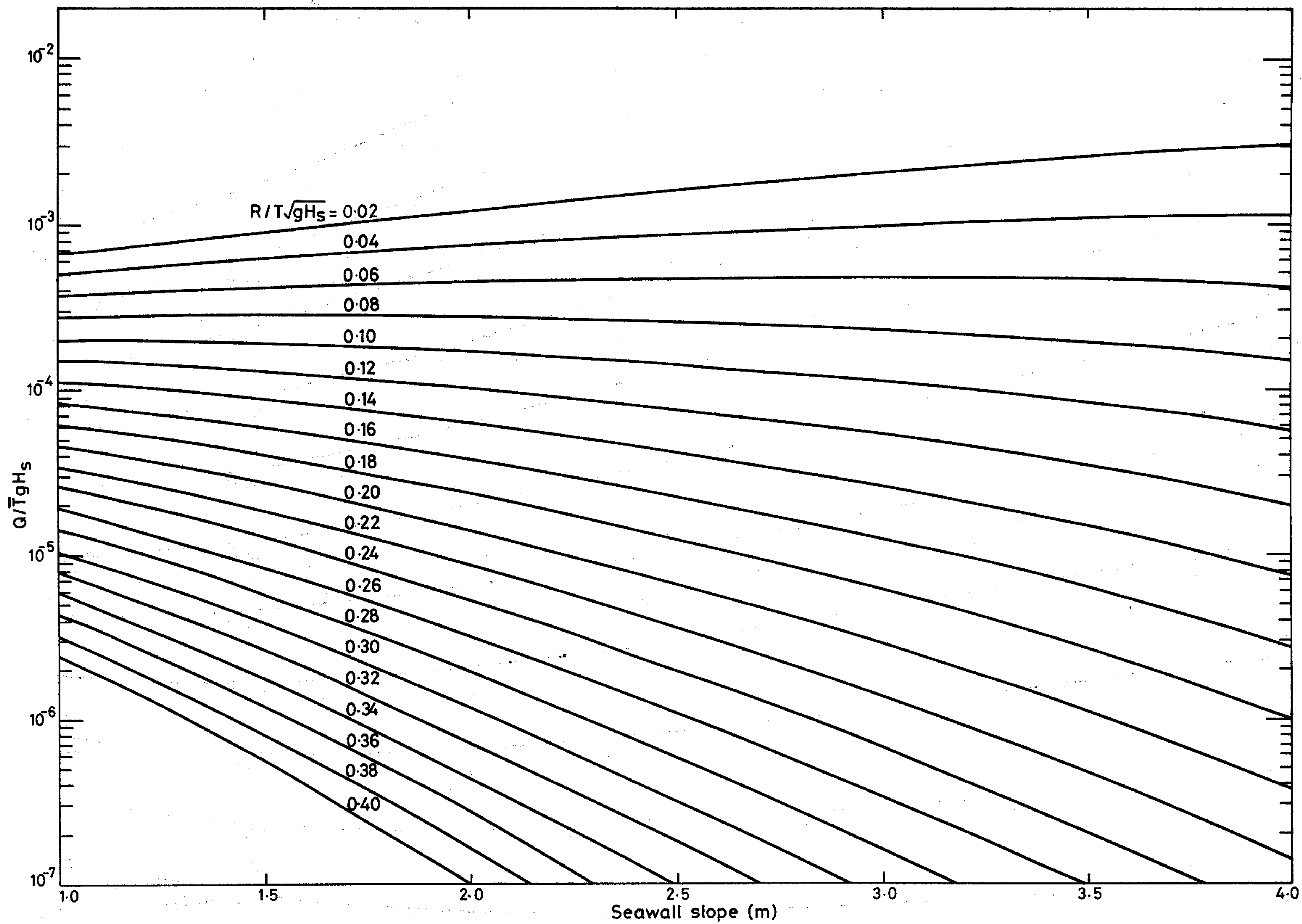
Discharge over seawalls : berm width 10m and elevation 1m below SWL

Fig.15



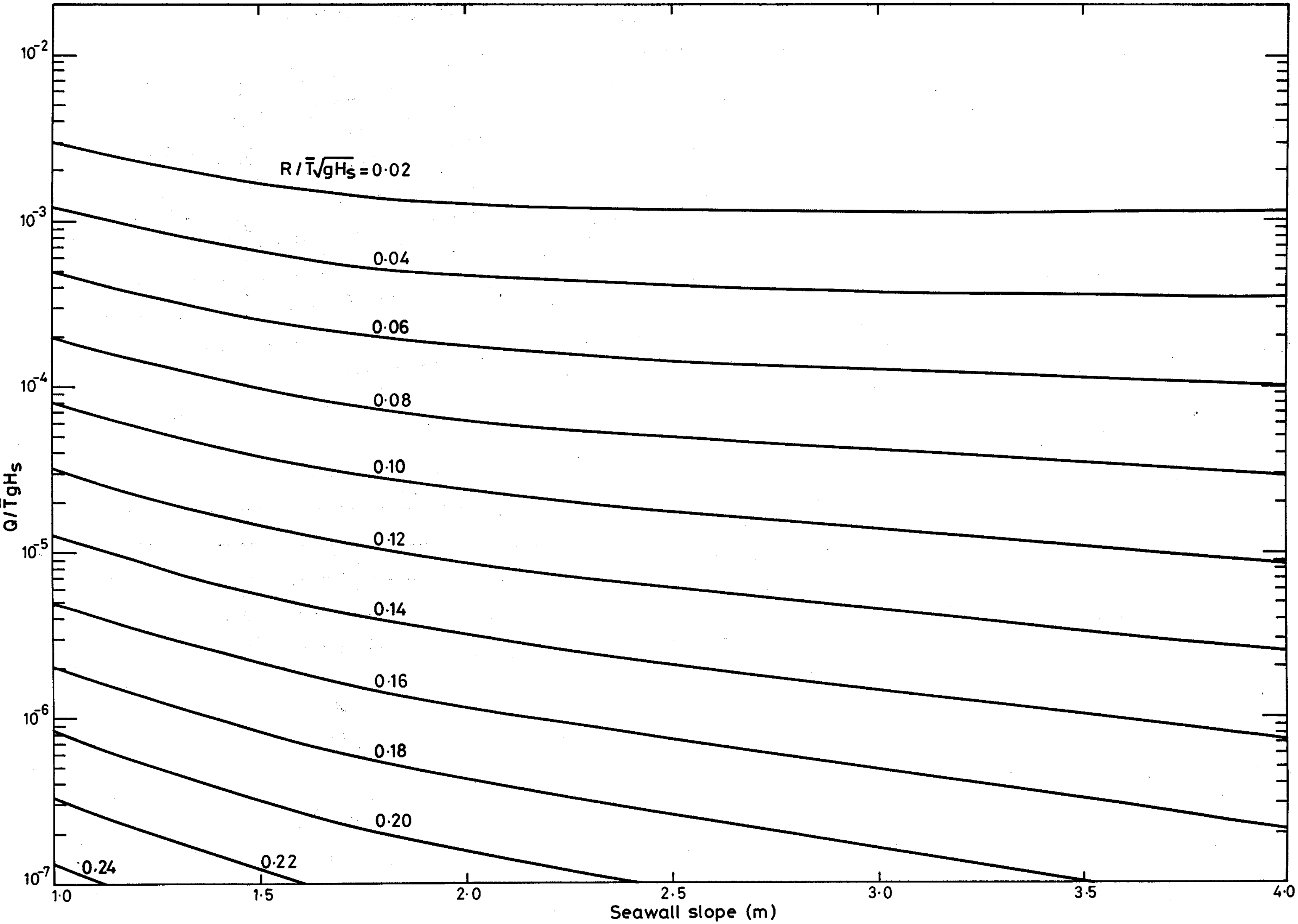
Discharge over seawalls: berm width 10m and elevation SWL

Fig.16



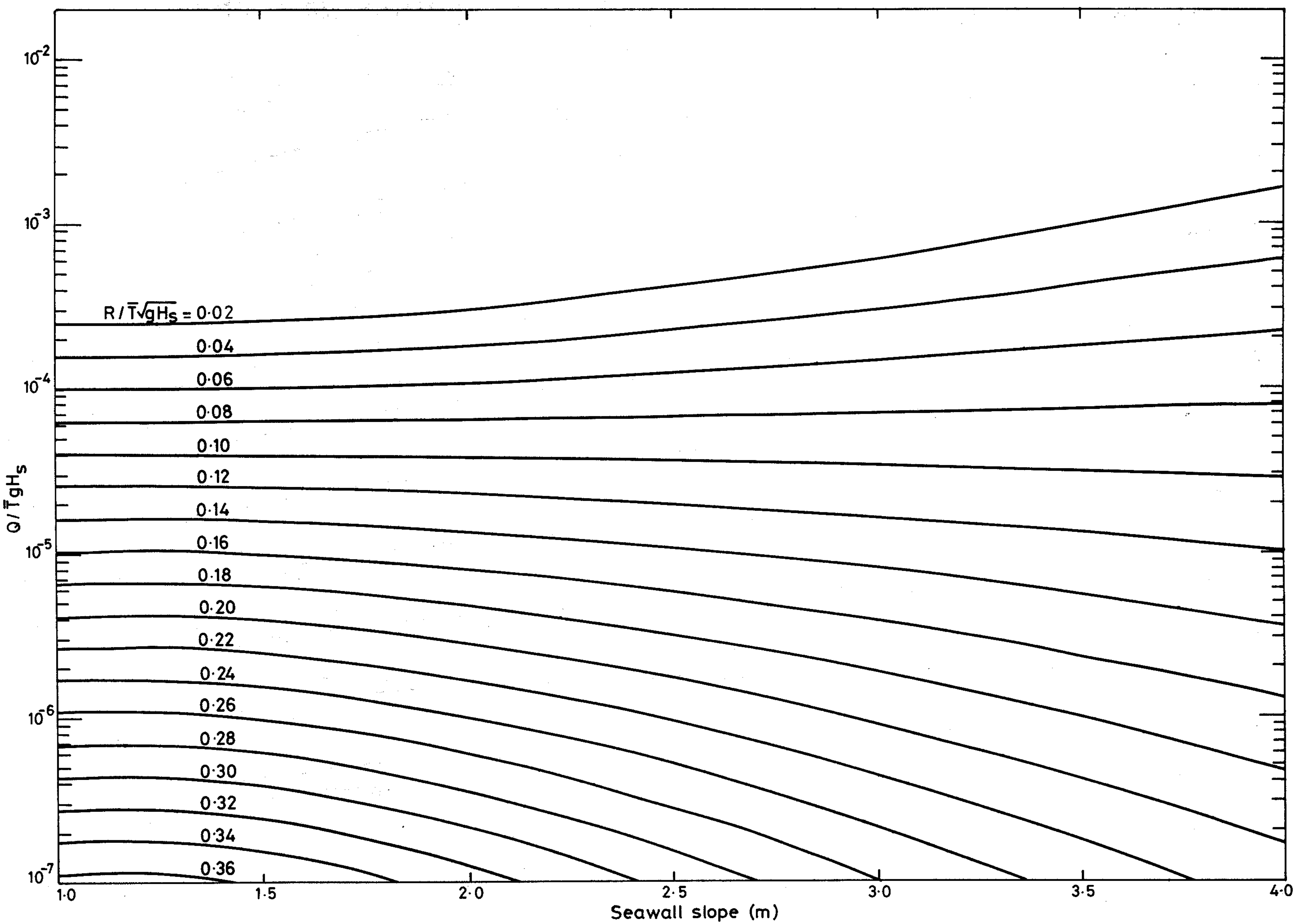
Discharge over seawalls: berm width 20m and elevation 2m below SWL

Fig.17



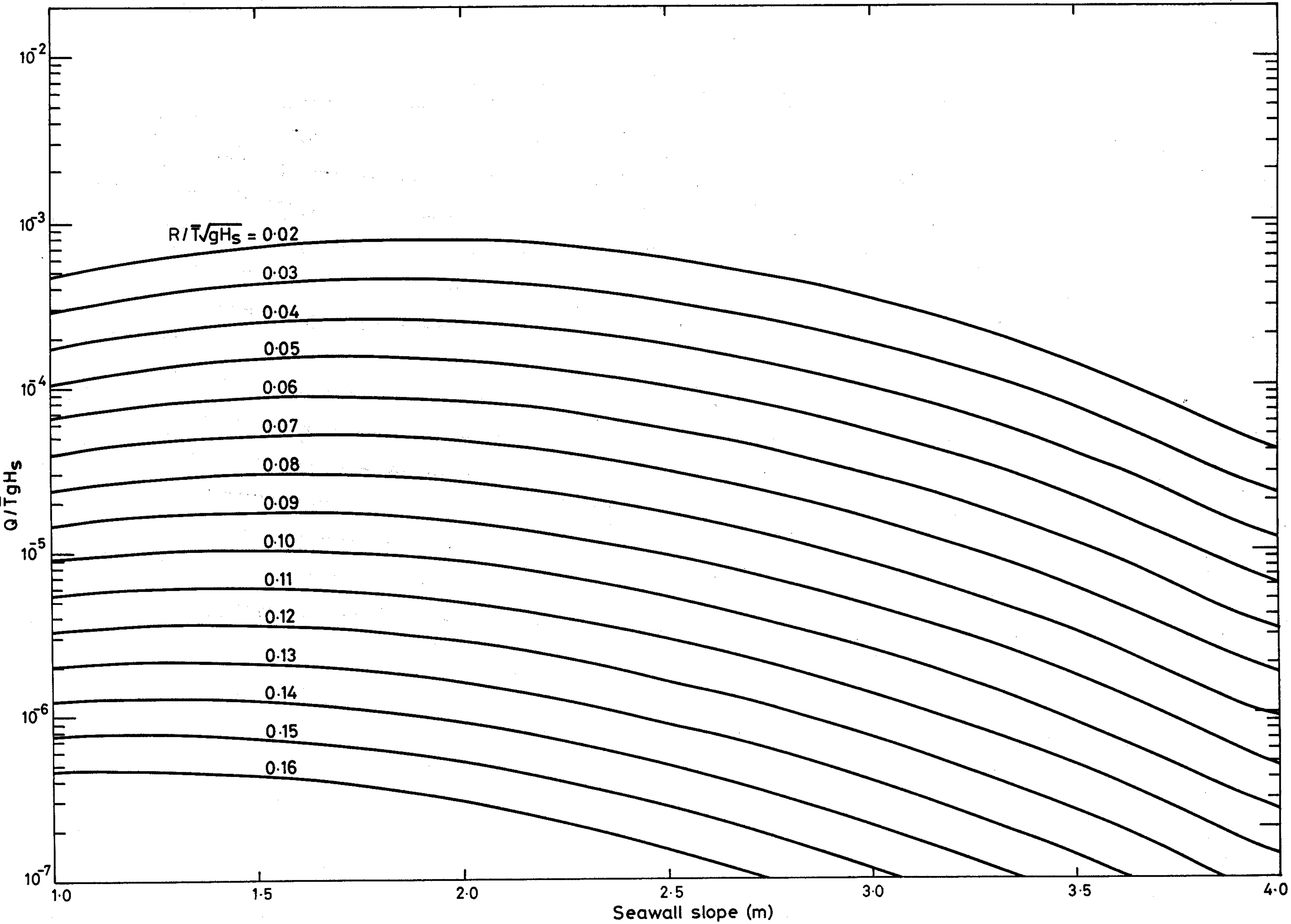
Discharge over seawalls : berm width 20m and elevation 1m below SWL

Fig.18



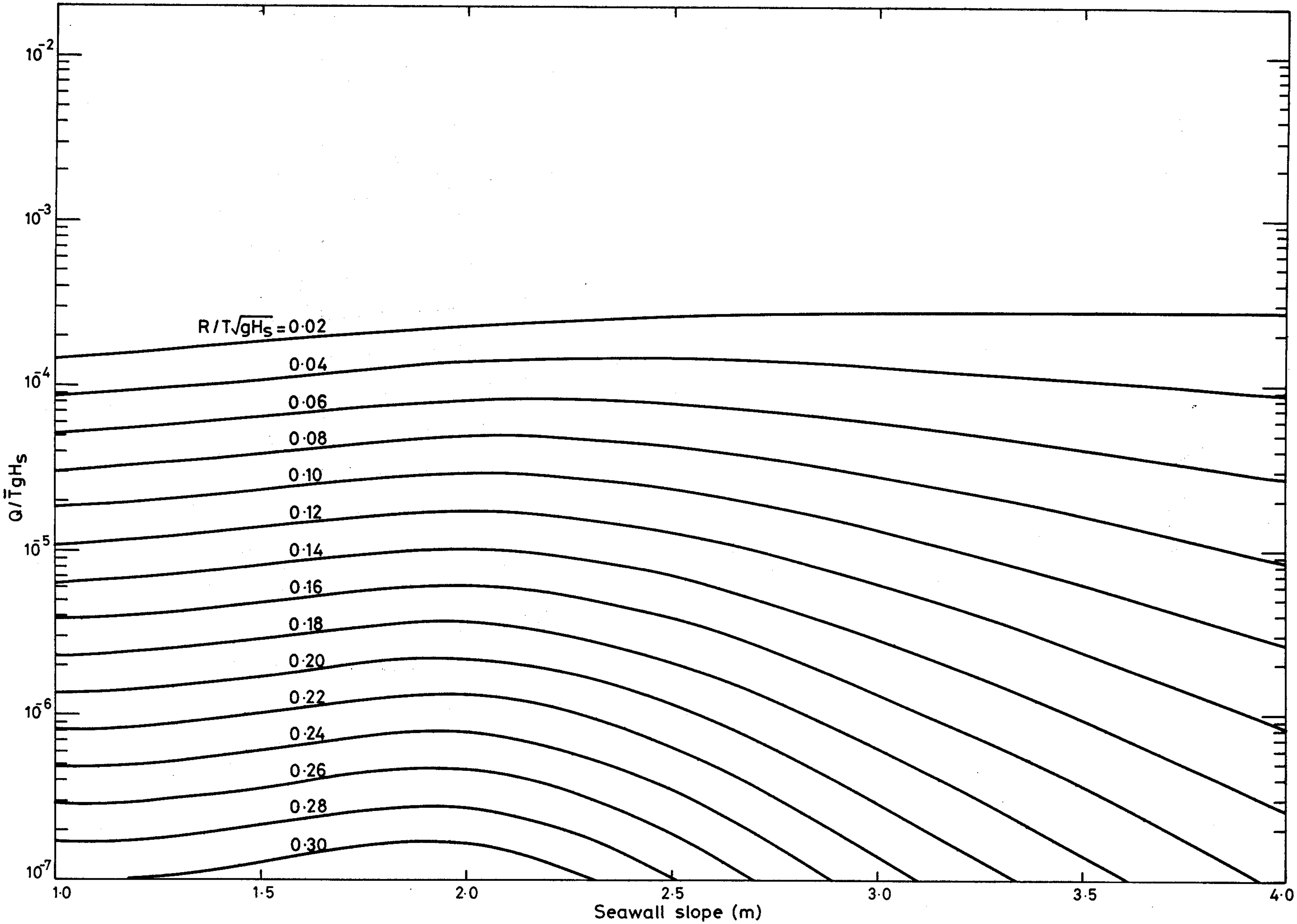
Discharge over seawalls : berm width 40m and elevation 2m below SWL

Fig. 19



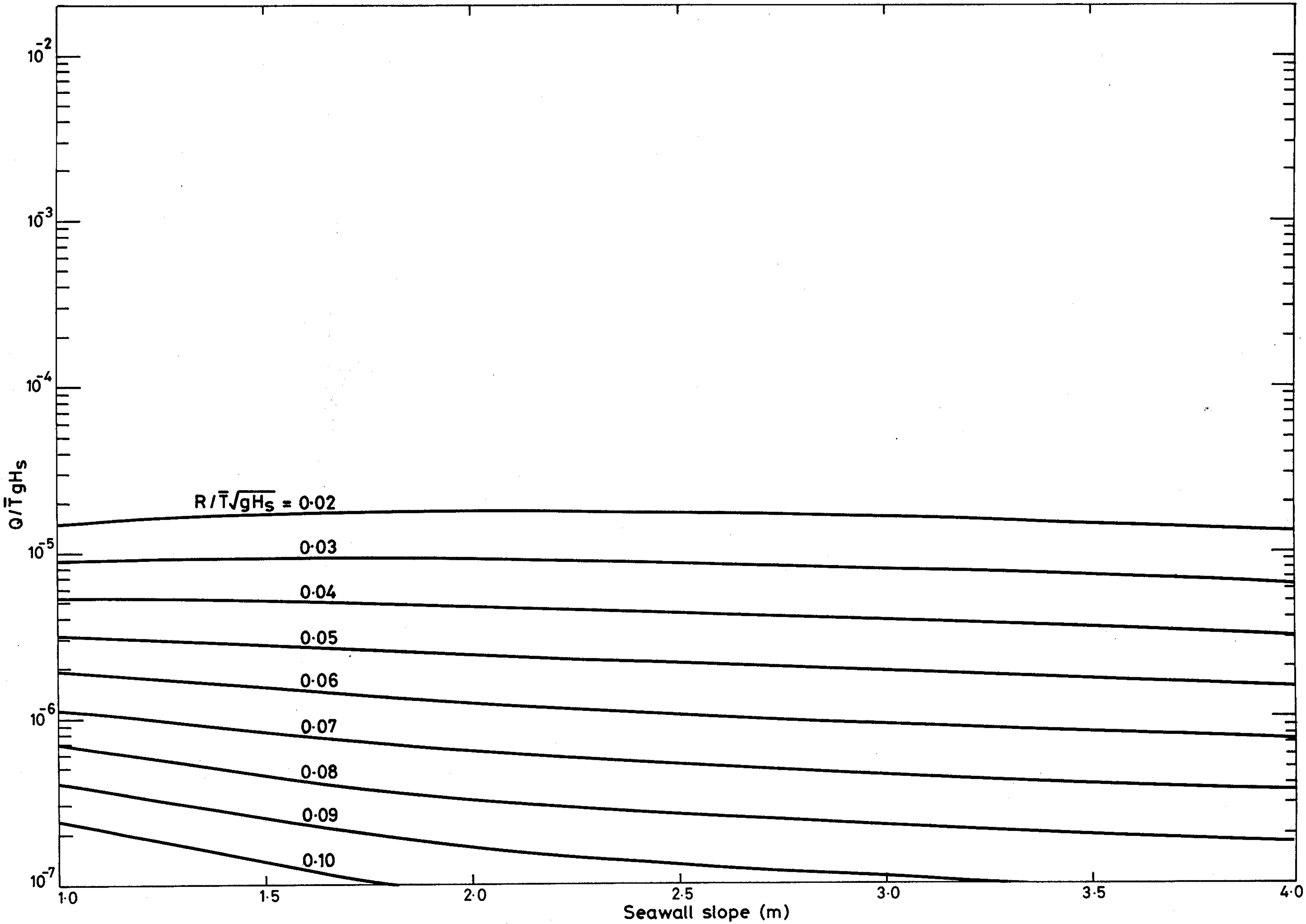
Discharge over seawalls : berm width 40m and elevation 1m below SWL

Fig. 20



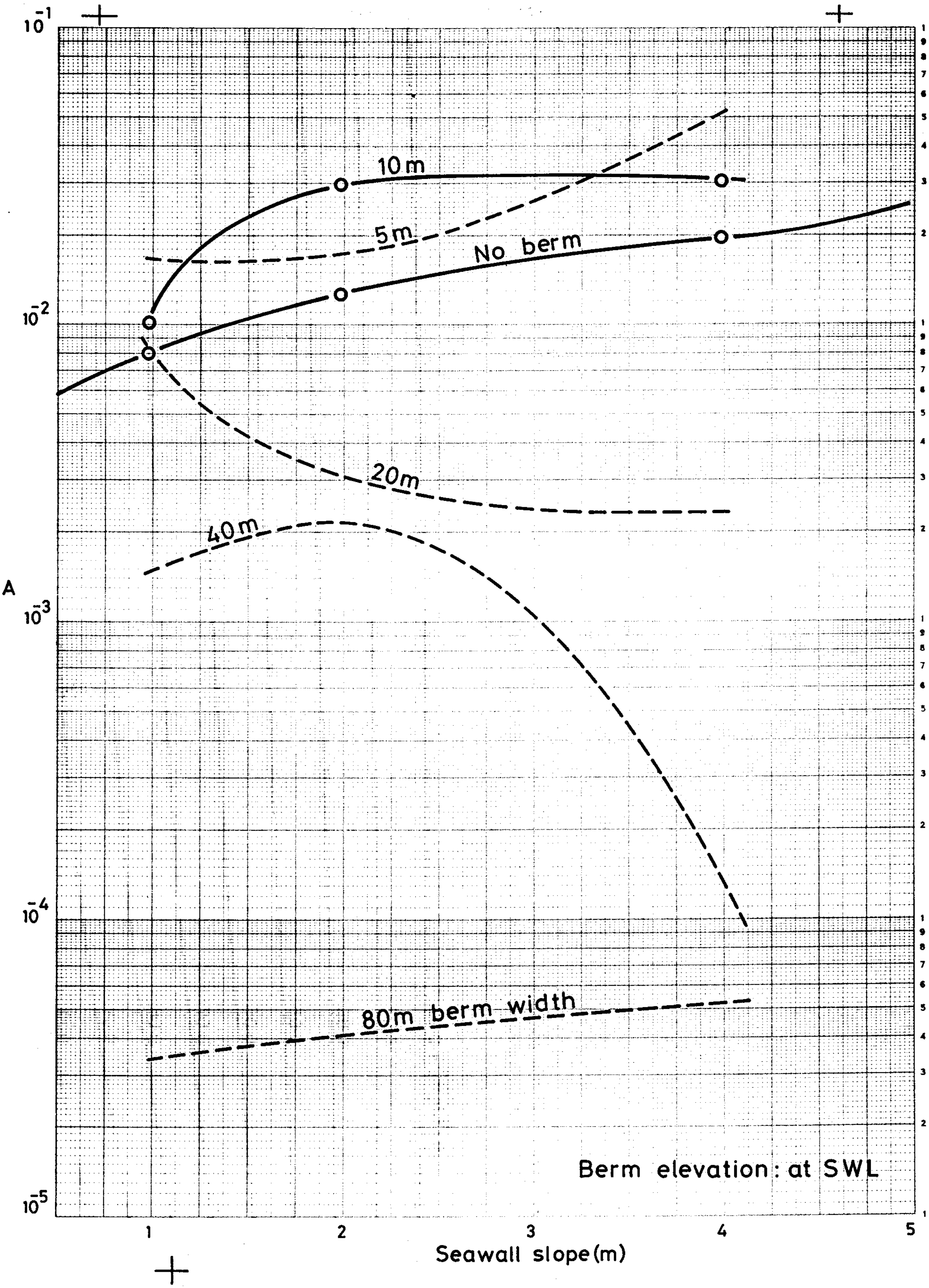
Discharge over seawalls : berm width 80m and elevation 2m below SWL

Fig.21



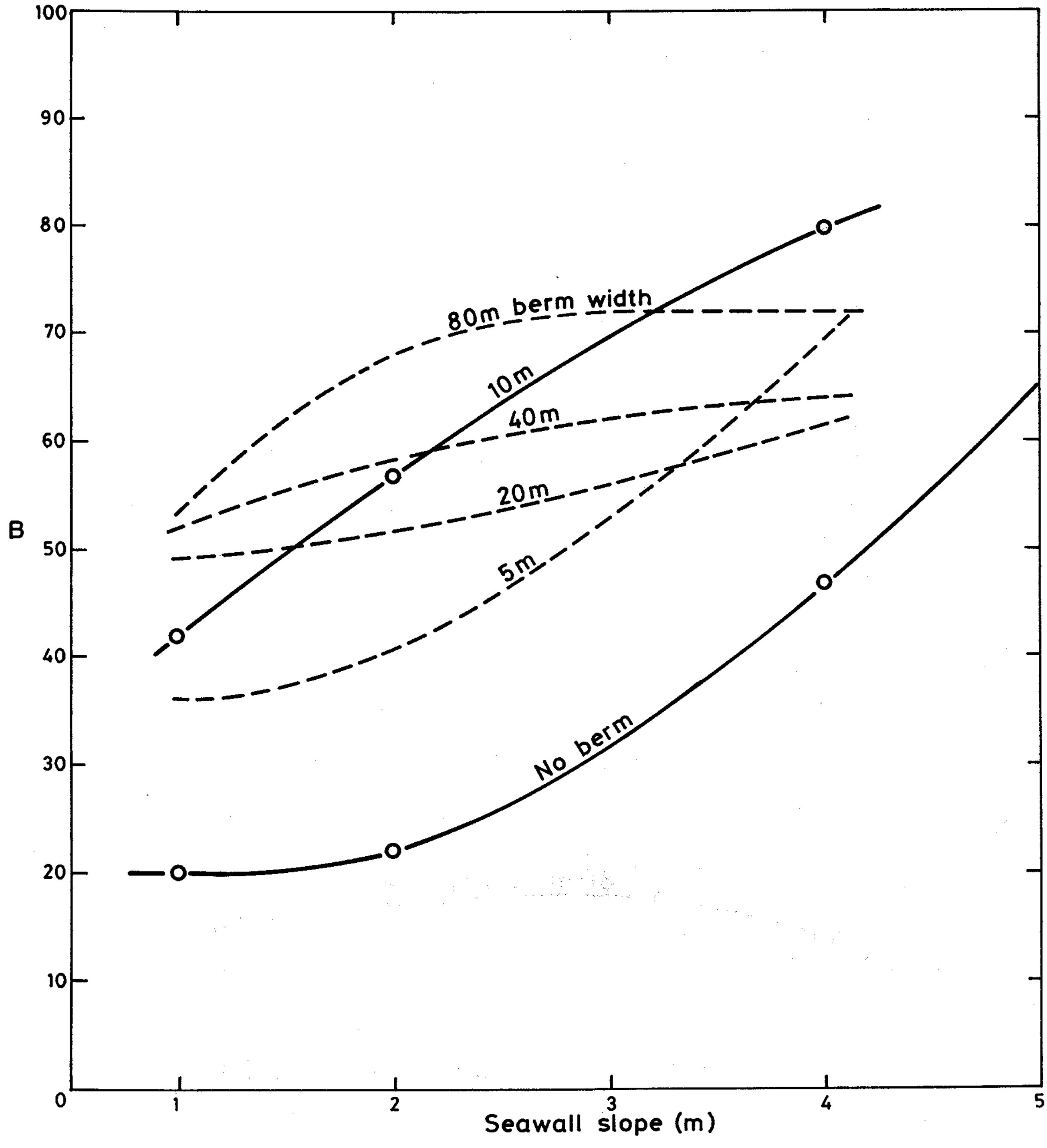
Discharge over seawalls : berm width 80m and elevation 1m below SWL

Fig.22



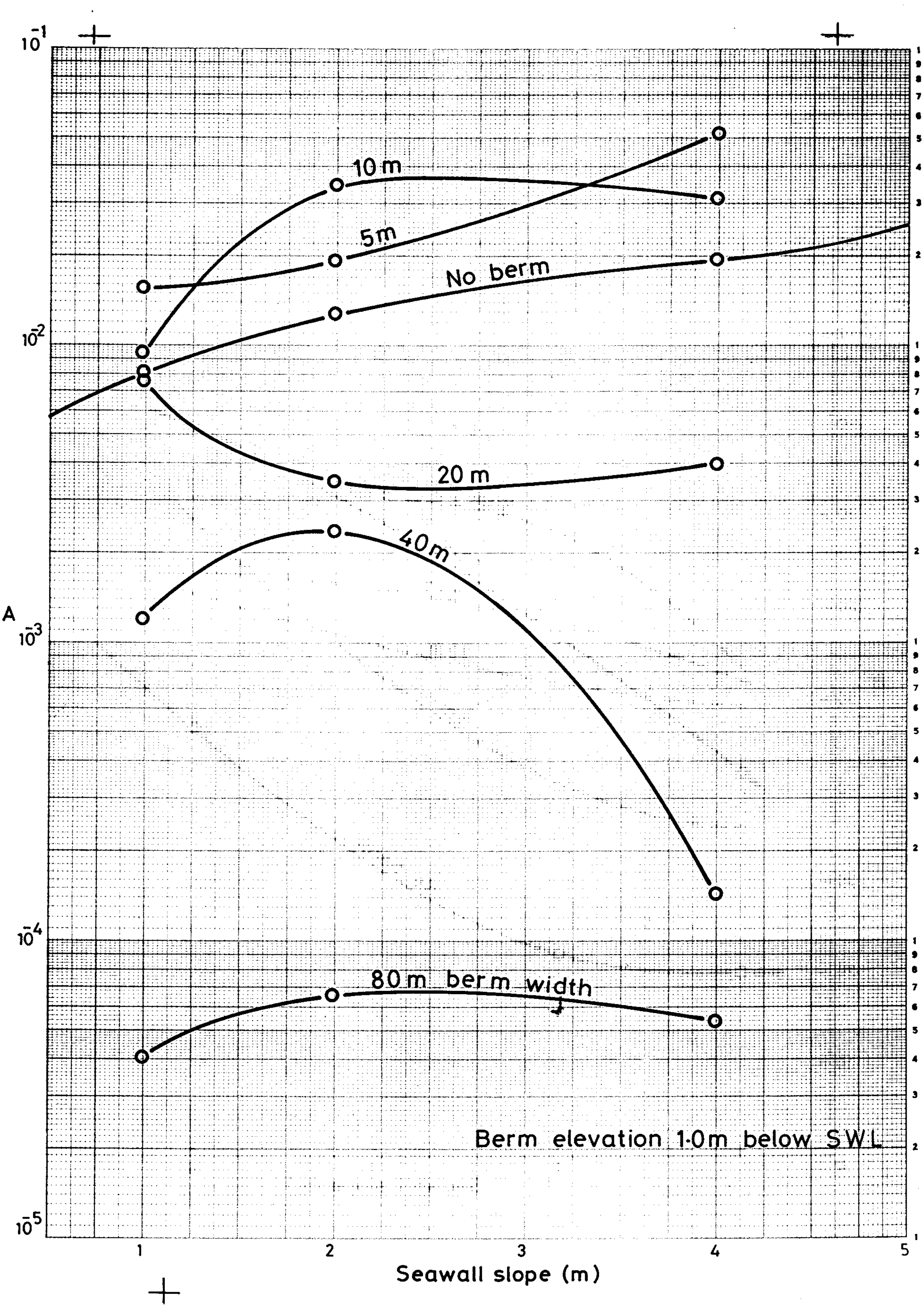
Coefficients A and B : berm elevation at SWL

Fig.23(a)



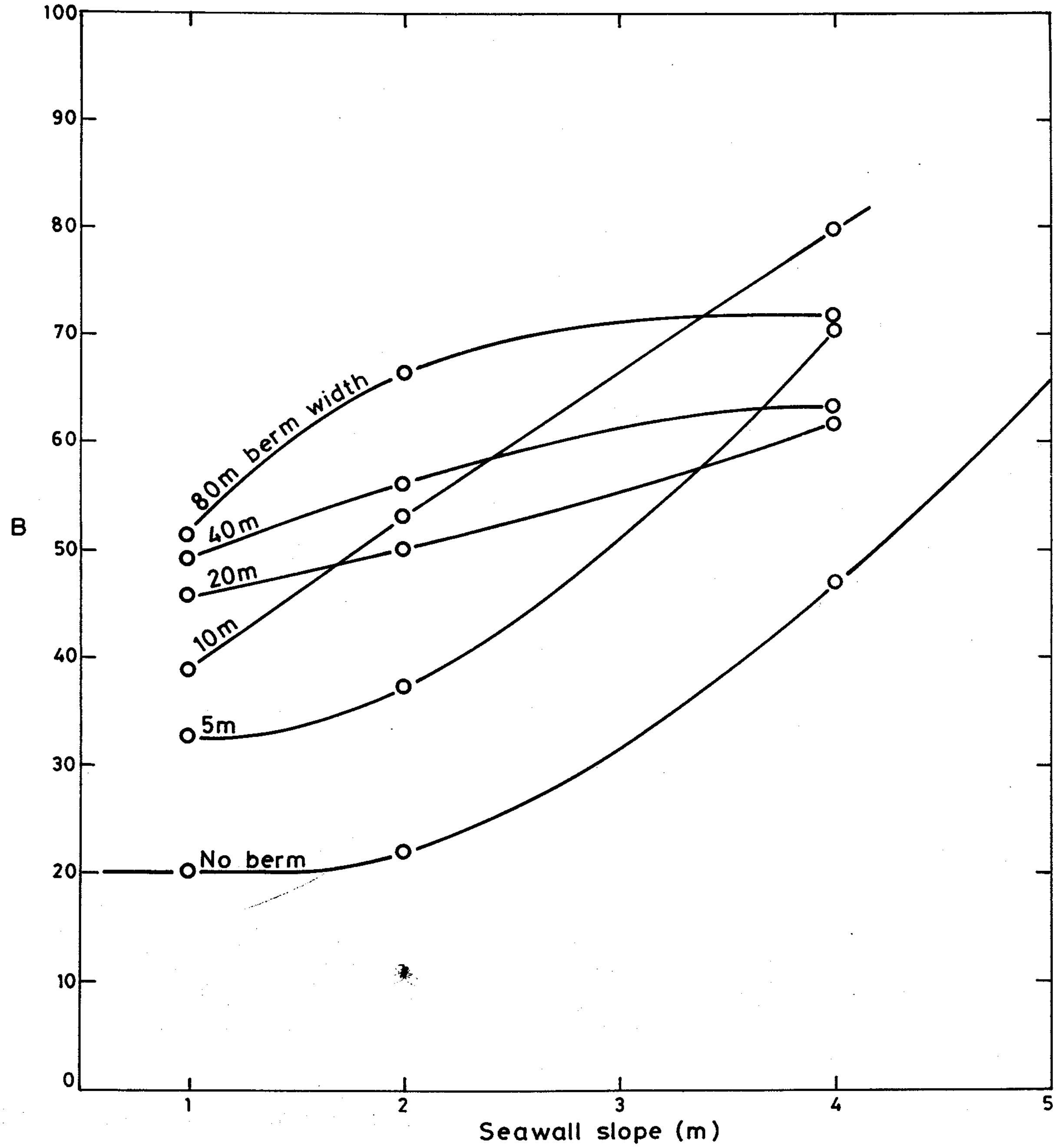
Coefficients A and B : berm elevation at SWL

Fig. 23(b)



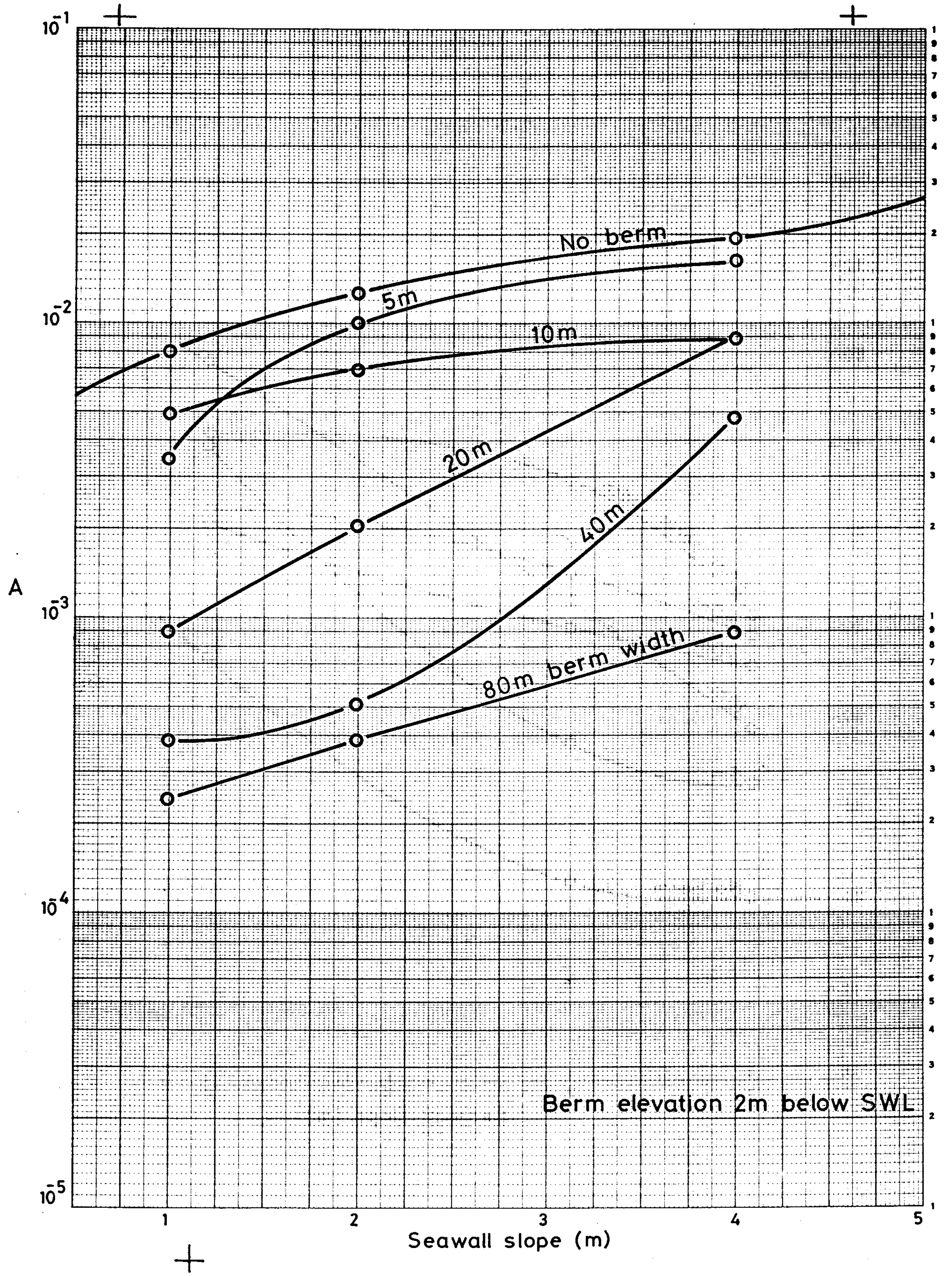
Coefficients A and B : berm elevation at 1m below SWL

Fig.24(a)



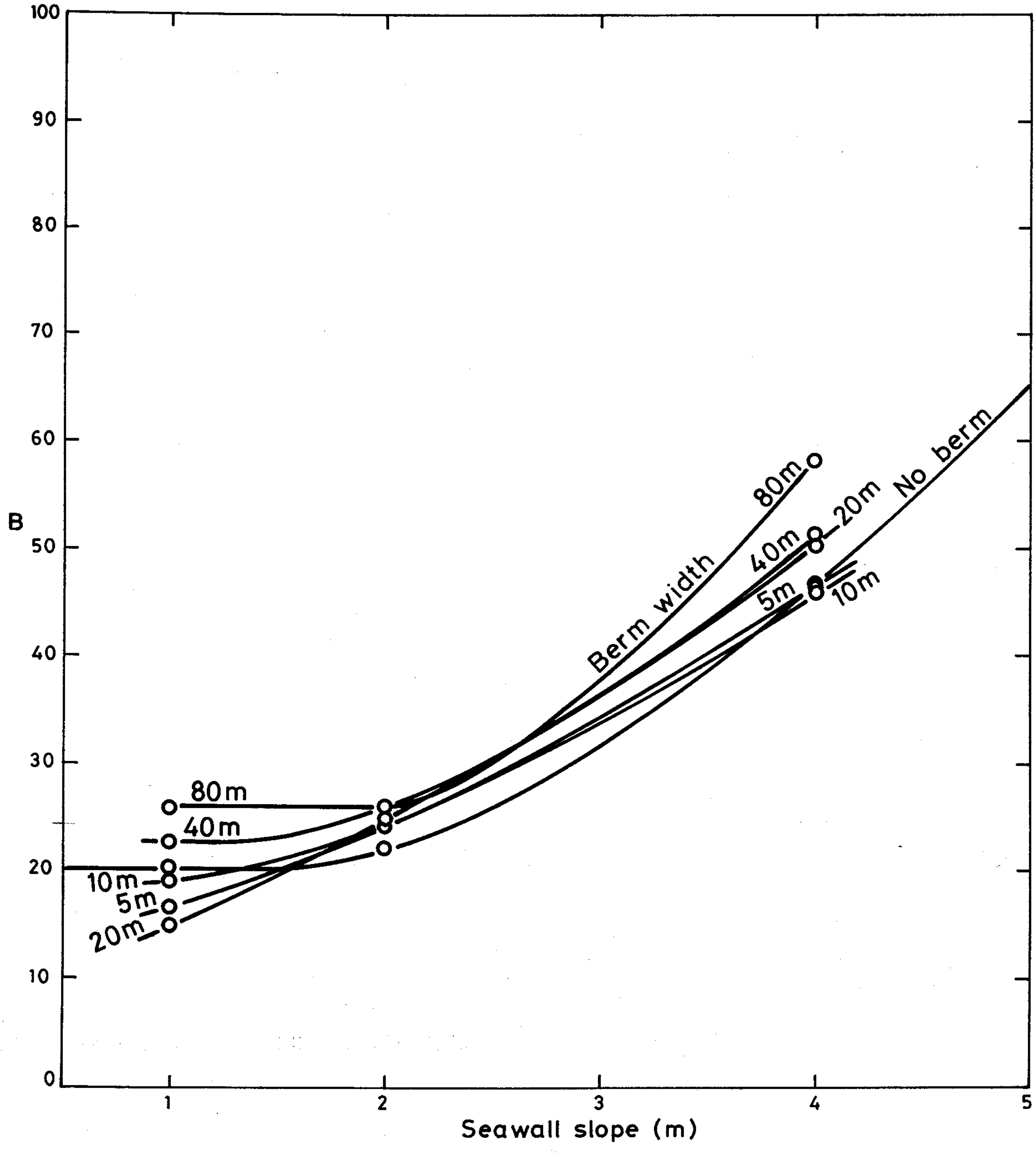
Coefficients A and B : berm elevation at 1m below SWL

Fig.24(b)



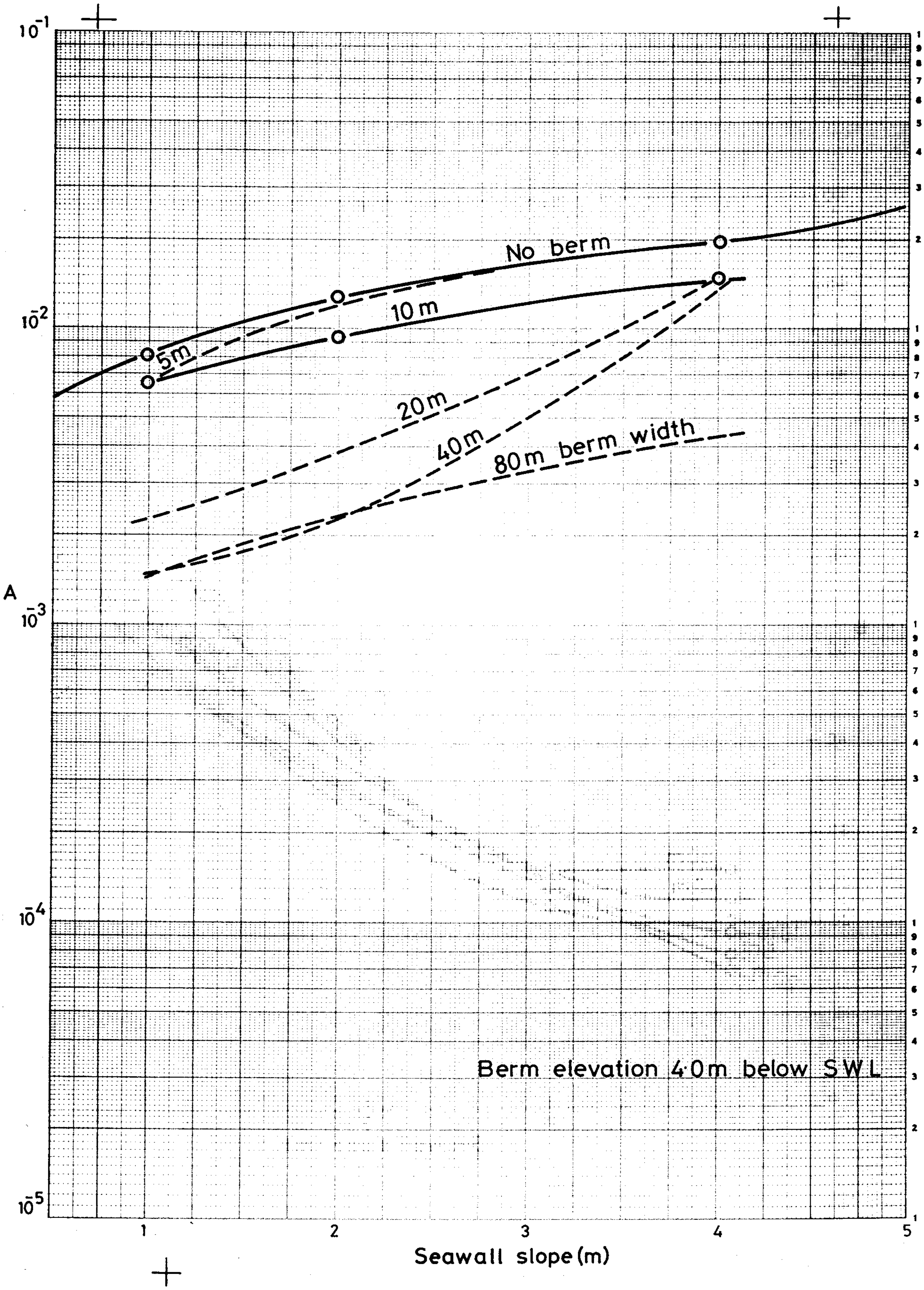
Coefficients A and B : berm elevation at 2m below SWL

Fig.25(a)



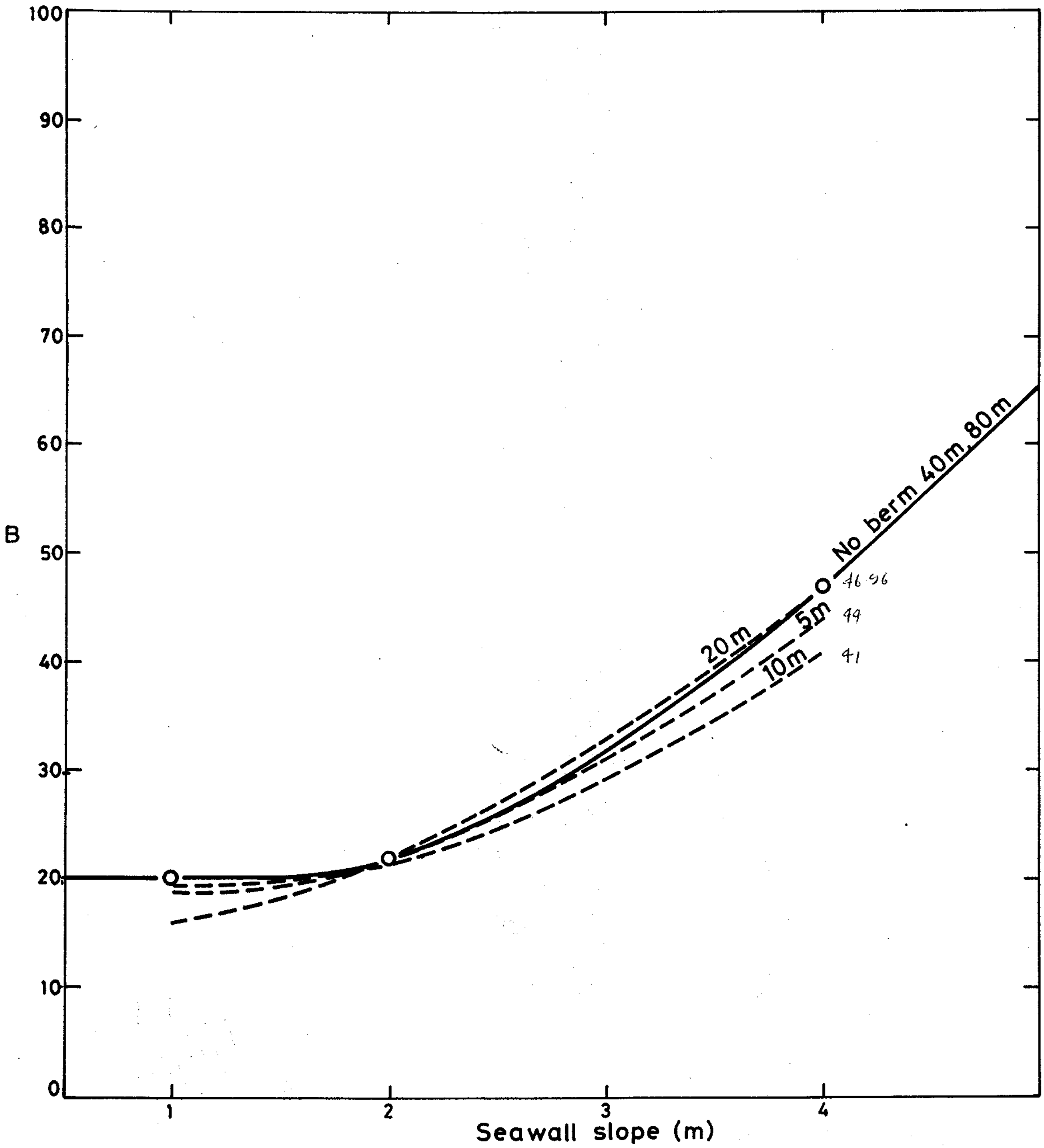
Coefficients A and B : berm elevation at 2m below SWL

Fig.25 (b)



Coefficents A and B : berm elevation at 4m below SWL

Fig.26(a)



Coefficients A and B : berm elevation at 4m below SWL

Fig.26(b)

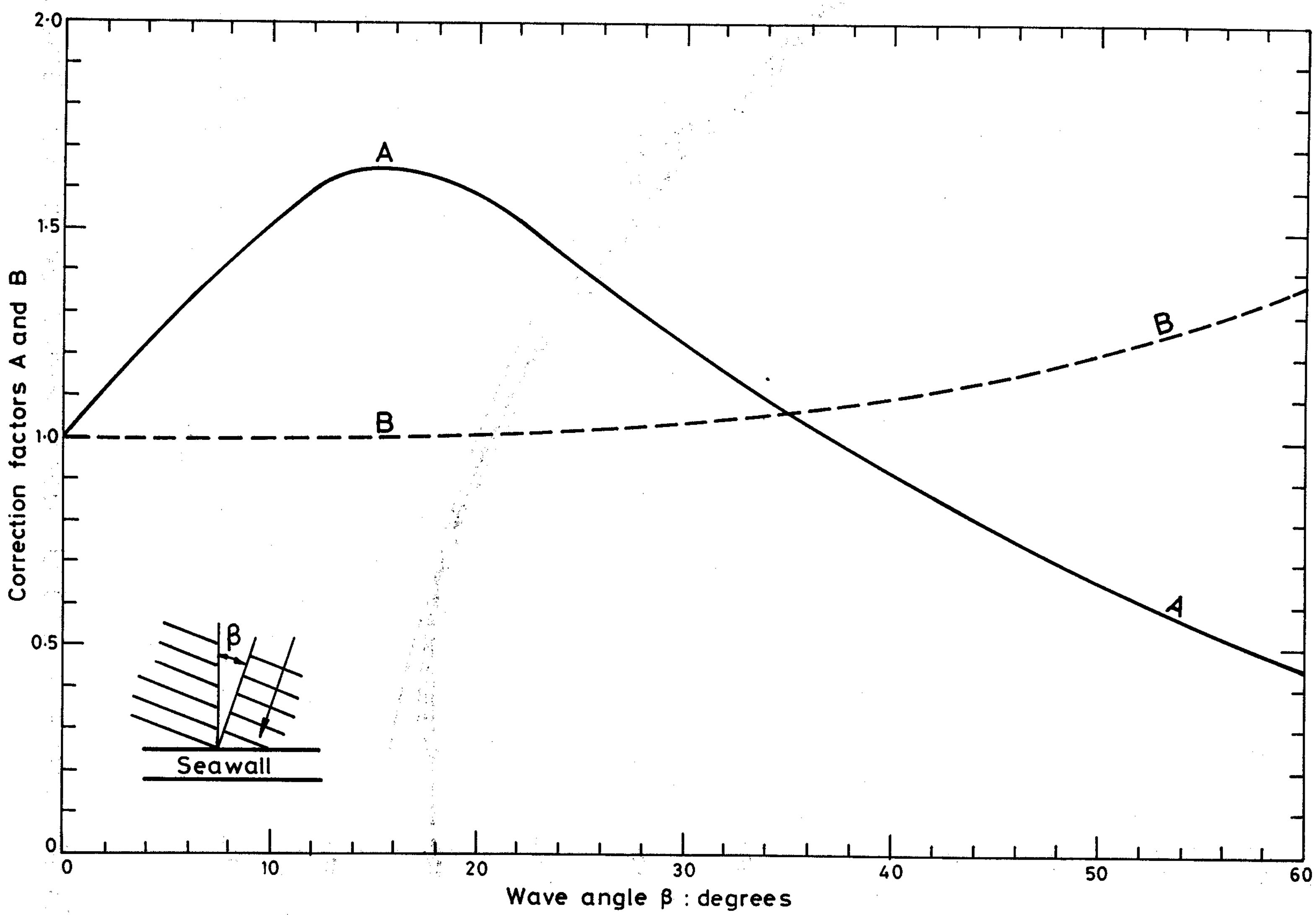
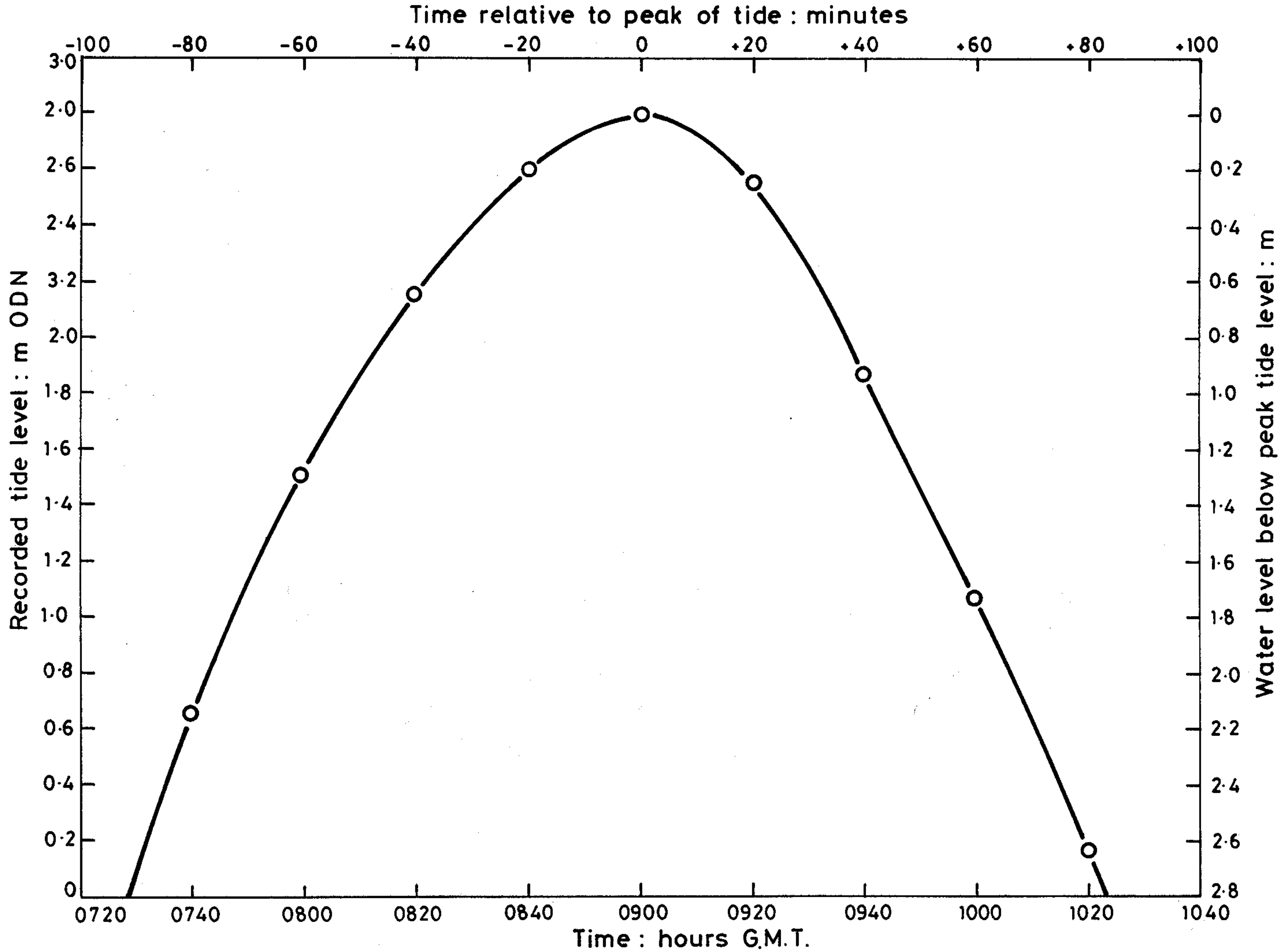


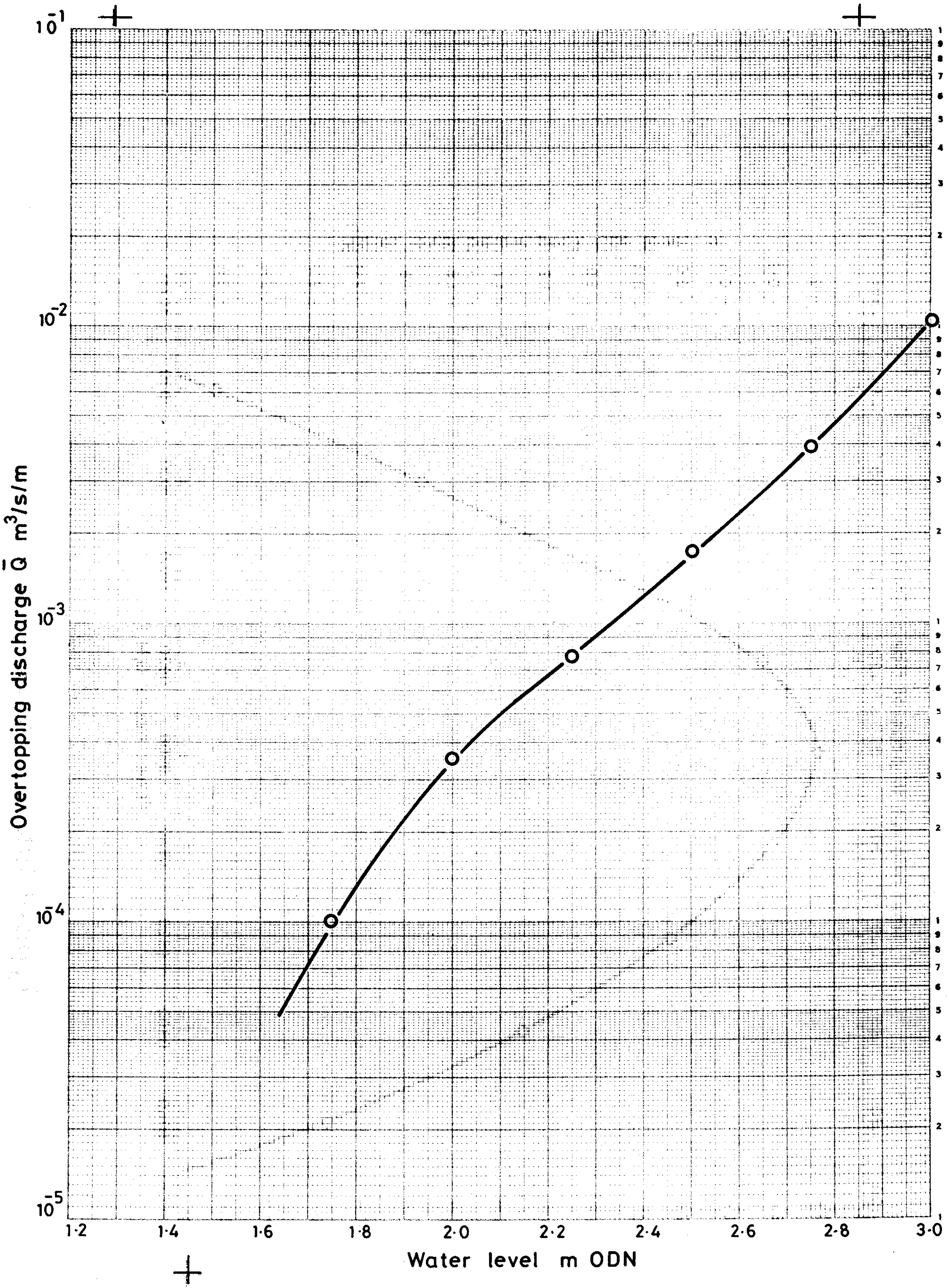
Fig.

Correction factors for angled wave attack



Typical tide curve for Example 8

Fig.28



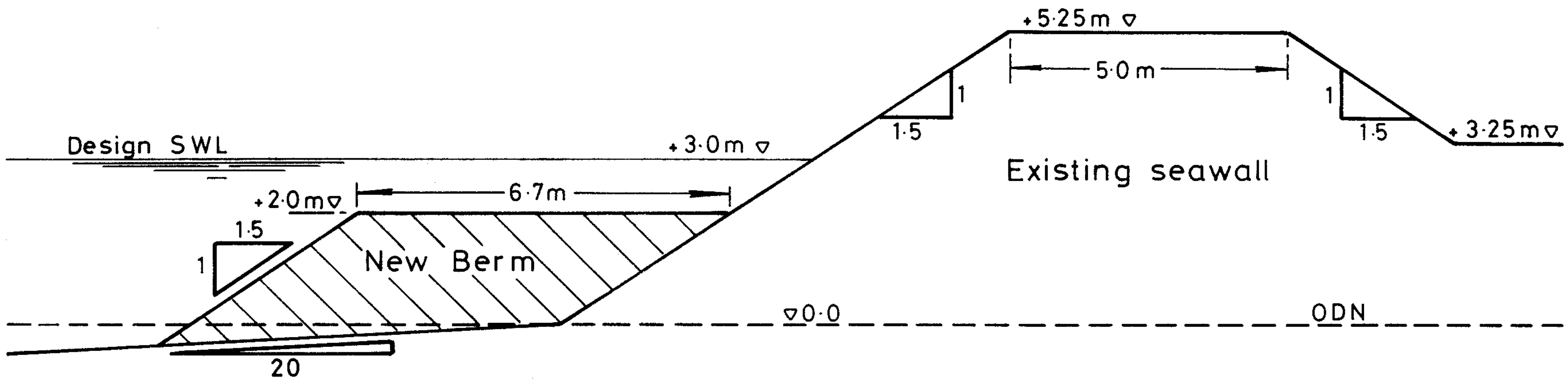
Example 8 : discharge curve

Fig.29

Design storm Still water level +3·0 m O.D.N. Overtopping discharge Existing seawall $2{\cdot}19 \times 10^{-2}$ $m^3/s/m$

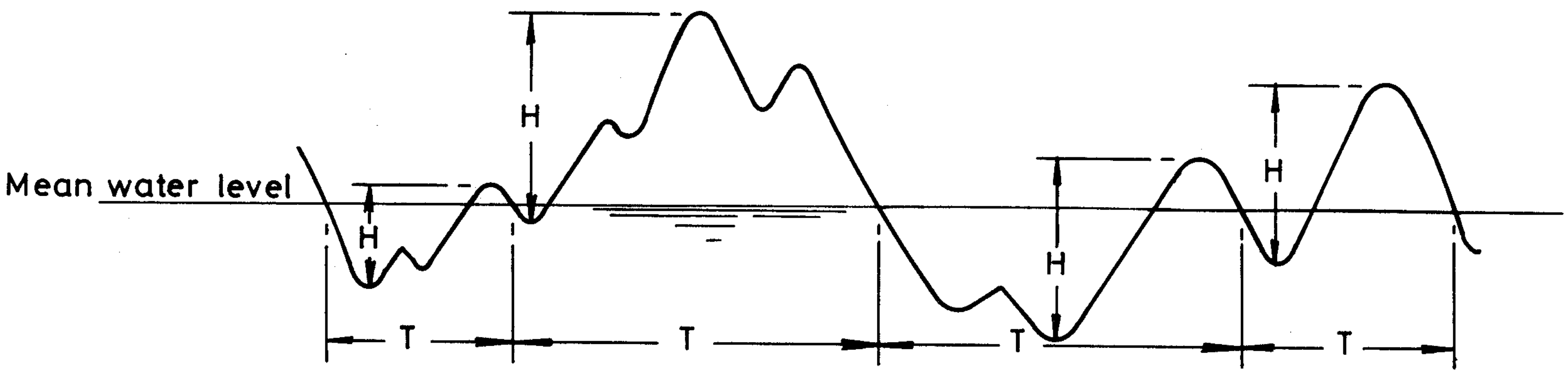
Significant wave height 1·25 m Improved " $2{\cdot}0 \times 10^{-3}$ $m^{-3}/s/m$

Mean wave period 4·0 s



Example 10 : Improved seawall

Fig.30



Definition of zero crossing wave height and period

Fig.3