Wave overtopping of vertical walls

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Contract

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Summary

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Physical model studies have been completed to measure the overtopping characteristics of vertically faced seawalls. A range of wave conditions, water depths, approach bathymetries and seawall elevations were investigated during testing. Based on analysis of the results a design method has been proposed to enable the overtopping discharge for vertical walls to be estimated.

This study forms part of a continuing programme of research into the behaviour of seawalls being carried out at HR Wallingford with support from the Ministry of Agriculture, Fisheries and Food under Commission FDO201, Marine Flood Protection, Sea Defence Structures.

For further information about this study please contact Dr D M Herbert or Dr K A Powell of the Coastal Group, Operations Department.

Notation

A,B	Empirical coefficients defining the discharge curve for a given seawall profile
F^*	Dimensionless freeboard (applicable to rubble mound breakwaters incorporating a crown wall) $R_c^2/(T_m H_s^{1.5} g^{0.5})$
g	Acceleration due to gravity
H_s	Significant wave height
H_{so}	Offshore significant wave height
h	Water depth at toe of seawall
Q	Mean overtopping discharge
Q^*	Dimensionless discharge $Q/(T_m g H_s)$
$Q^\#$	Dimensionless discharge (applicable to vertical walls) $Q/(2g H_{so}^3)^{½}$
R	Freeboard of seawall (the distance of the crest above still water level)
R_c	Distance of the top of a breakwater crown wall above still water level
R^*	Dimensionless freeboard $R/(T_m(g H_s)^{½})$
s_{om}	Offshore sea steepness $2\pi H_{so}/(g T_m^2)$
T_m	Mean wave period

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1 Introduction

Vertical or near vertical seawalls afford protection to a significant length of UK coastline. These seawalls are common in urban areas and are often sited behind shingle or sandy beaches. As part of HR Wallingford's (HR) continued interest in the design of seawalls, model tests have been completed to measure the overtopping discharge performance of vertical walls for a range of approach bathymetries, wave conditions, water levels and wall heights. Previous work completed at HR includes the overtopping of sloping and bermed seawalls (Reference 1) and the effectiveness of recurved wave return walls in reducing overtopping discharges (Reference 2).

This report describes the physical model testing undertaken in order to determine the overtopping performance of vertical seawalls. Chapter 2 outlines the design of the physical model and the measurements made. The analysis of the model data and the results obtained are described in Chapter 3. Finally conclusions to the study are given in Chapter 4.

2 Physical model tests

2.1 Test facility

The model tests outlined in this report were conducted in the wind wave flume at HR. The flume is 50m long by 1.22m wide by 1.1m deep and is equipped with a computer controlled, hydraulically driven, wedge paddle capable of generating any required deep water random wave spectrum. The flume is divided along its length into two channels, the wider of which (0.76m) contains the relevant approach bathymetry. The narrower channel (0.47m) is of constant depth and ends in a gentle sloping shingle spending beach. A more detailed explanation of the flume and its method of operation is given in Appendix A.

2.2 Design of model tests

Work detailing the overtopping performance of vertical walls had previously been completed by Goda (Reference 3). Goda investigated approach slopes of 1:10 and 1:30 and offshore sea steepnesses of s_{om} = 0.012, 0.017 and 0.036. These conditions were considered to be unrepresentative of conditions around the UK coastline where storm sea steepnesses are greater and approach bathymetries are generally shallower. It was the aim of the model tests to confirm and extend the work of Goda so that it was more applicable to UK coasts. Consequently three approach bathymetries of 1:10, 1:30 and 1:100 were used in the model with offshore sea steepnesses ranging from 0.017 - 0.060. Other parameters that were varied included the offshore wave height, the water depth at the toe of the seawall and the freeboard of the wall (the distance of the crest above still water level). These parameters were varied to ensure that the model tests were completed in the zone of interest identified by Goda.

2.3 Wave flume calibration

Prior to testing a series of wave flume calibration tests were completed. These calibrations measured wave conditions in the narrower 'deep water' channel. A spending beach was placed in front of the model structure to ensure that only incident wave energy was measured and reflected wave energy was kept to a minimum. The wave conditions were measured using twin wire resistance type wave probes connected to a Compaq micro-computer, which processed the signals to derive the significant wave height, H_s, and mean zero crossing wave period, T_m, from the wave energy spectrum (see Appendix B).

For this study the JONSWAP form of the wave energy spectrum was used for all tests. A short repeating sequence (in the order of 6-10 minutes) of the relevant spectrum was programmed on the BBC computer controlling the wave paddle and the wave conditions recorded. Recording over a complete wave sequence eliminated any statistical uncertainties in the wave analysis routine. If the significant wave height and period were not within the required accuracy limits, then the settings on the BBC computer were adjusted and the calibration repeated. This iterative procedure was repeated until satisfactory deep water wave conditions were achieved.

2.4 Test procedures

2.4.1 Wave measurements

All testing was completed using a wave sequence length (in the order of several hours) considerably greater than the test length. During testing waves were recorded in the deep water channel using a statistical method of wave analysis (see Appendix B). This method of analysis enabled a range of different wave parameters to be derived.

2.4.2 Overtopping measurements

For each test condition, five overtopping measurements were taken to enable the mean and the standard deviation of discharge to be calculated. Each measurement consisted of collecting all the water which overtopped the seawall during a period of 100 waves (defined as 100 times the nominal mean wave period). In some instances, the quantity of water overtopping the structure during the first measuring interval was very small. Consequently discharges were measured continually over a period of 1000 waves. The resulting depth of water in the collecting tanks was measured, and using previously derived calibration data, the total volume of water was calculated. Further details of these overtopping measurements are given in Appendix C.

3 Results

3.1 Data presentation

Several methods of data presentation were investigated consistent with previous work completed at HR. The first method of analysis is usually used for estimating overtopping discharge rates for simply sloping seawalls (Reference 1). In this method a dimensionless discharge parameter, Q*, and dimensionless freeboard parameter, R*, are defined as follows:

$$Q^* = Q/(T_m\ g\ H_s) \quad (1)$$

$$R^* = R/(T_m\ (g\ H_s)^{1/2}) \quad (2)$$

where Q is the mean overtopping discharge in $m^3/s/m$ run of seawall
T_m is the mean wave period
H_s is the significant wave height
R is the freeboard of the seawall (the distance of the crest above still water level)
and g is the acceleration due to gravity.

These two parameters are connected by the equation:

$$Q^* = A \exp(-B R^*) \tag{3}$$

where A and B are dimensionless coefficients whose values depend on the seawall geometry.

The results of physical model tests for a given seawall profile allow the values of A and B to be determined and hence enable the overtopping performance of the seawall to be determined for a wide range of conditions.

The results of this study were presented in the method outlined above using the offshore significant wave height measured during testing. Although an upper bound line was well defined, the results exhibited considerable scatter and fitting a straight line to the data was thought inappropriate. Further analysis, designed to identify any dependence of this data on other parameters, was unsuccessful.

Work was also completed using wave conditions derived at the toe of the seawall. The empirical method of deriving these wave heights was based on that outlined in Reference 1. Using inshore wave heights reduced, but did not eliminate, the scatter of the results which remained significant. It was therefore concluded that the method of data analysis given by equations (1) - (3) was inappropriate for vertical walls.

Further analysis was undertaken in order to investigate alternative methods of describing the overtopping discharge performance of vertical walls. These included an equation previously used to describe the overtopping performance of a crown wall on top of a rubble slope (Reference 4). This equation states:-

$$Q^* = A F^{*B} \tag{4}$$

where $F^* = R_c^2/(T_m H_s^{1.5} g^{0.5})$
and R_c is the distance of the top of the crown wall above still water level.

However, none of these data presentation methods adequately described the overtopping of vertical walls. It was therefore considered that the most appropriate method of presenting the physical model data was that proposed by Goda (Reference 3). This method involved presenting the data in a non-dimensionalised graphical format. For a given approach bathymetry and offshore sea steepness, s_{om} ($= 2\pi H_{so}/g T_m^2$), a dimensionless discharge, $Q^\#$, was plotted on the y-axis against h/H_{so} on the x-axis where:-

$$Q^\# = \frac{Q}{(2g H_{so}^3)^{1/2}} \tag{5}$$

H_{so} is the offshore significant wave height and h is the water depth at the toe of the wall.

Lines of constant values of R/H_{so} were illustrated on the graph where R is the height of the crest of the wall above still water level. Hence, for a given approach bathymetry, offshore sea steepness, s_{om}, freeboard ratio, R/H_{so}, and non-dimensionalised water depth, h/H_{so}, the value of $Q^{\#}$ and therefore Q can be calculated. Examples of this type of analysis are illustrated in Figures 1 - 12 which have been constructed using the model data measured during the present study. Despite the simplicity of Goda's method it does have a disadvantage in that it is graphical and does not easily lend itself to programming on a computer.

In general the model data gave good agreement with the work of Goda (see Figures 1,2,4 and 5). This was somewhat surprising as Goda used a series of regular wave physical model tests as the basis for his work. Knowledge of the distribution of wave heights for different random sea states then enabled the distribution of overtopping events to be estimated and hence allowed the total overtopping discharge to be calculated.

For a given freeboard ratio, maximum overtopping discharges occurred when $1.4 < h/H_{so} < 2.0$. These conditions correspond to waves breaking immediately seaward of the structure toe. The breaking waves often pass directly over the crest of the seawall. For $h/H_s < 1.4$ waves break before they reach the vertical wall. A considerable amount of energy is dissipated during breaking and hence overtopping is reduced. If $h/H_{so} << 1.4$ the waves break farther offshore and overtopping is further reduced. Conversely, for $h/H_{so} > 2.0$, waves are unbroken when they reach the structure and this also leads to a reduction in peak overtopping discharge. As waves travel into shallower water they steepen (this process is called shoaling) before breaking. When the water depth at the structure is large relative to H_{so} little or no shoaling occurs and overtopping is commensurately lower. For unbroken waves, as the h/H_{so} ratio increases the level of shoaling, and hence overtopping, is reduced. Eventually the effect of water depth at the structure and bed slope will become insignificant and overtopping will be a function of wave height and freeboard only. Therefore, for large vales of h/H_{so}, overtopping discharges will approach a constant value for a given freeboard ratio, R_c/H_{so}.

The graphs illustrated in Figures 1 - 12 exhibited the following characteristics. For given values of offshore sea steepness and freeboard ratio the overtopping discharge generally increased as the approach bathymetry steepened. Steeper slopes produce an increase in shoaling and hence lead to larger waves in the vicinity of the structure. Previous work on simply sloping seawalls (Reference 1) has indicated that longer wave periods result in increased overtopping. The influence of wave period in the proposed analysis method for vertical walls is allowed for through the use of different offshore sea steepnesses, s_{om}. For given conditions peak overtopping discharges (these occurred in the range $1.4 < h/H_{so} < 2.0$) generally increased with decreasing offshore sea steepness (this corresponds to an increase in wave period). However, outside of this range, the influence of wave period on overtopping was less conclusive, especially for large offshore sea steepnesses.

3.2 Use of design method

It is recommended that the design graphs are only used within the range illustrated in Figures 1 - 12 as no physical model data was derived outside of this range. The use of Figures 1 - 12 is self explanatory for the approach bathymetries and values of s_{om} and R/H_{so} illustrated on the graphs. However uncertainties are likely to arise when any of these parameters deviate from the values given above. It is recommended that for approach bathymetries and offshore sea steepnesses not directly given the four graphs straddling these values are used and a sensitivity analysis completed. In the majority of cases, the steepest approach slope is likely to produce the largest overtopping discharge. It is suggested that for unstated values of freeboard ratio linear interpolation is used to derive the required value. This interpolation should be based on the measured distance between straddling values rather than using the values on the logarithmic scale of the y-axis.

For approach bathymetries steeper than 1:10 it is recommended that physical model tests be considered. Steep slopes such as these are likely to cause larger breaking wave heights at the structure toe than those produced by a 1:10 bathymetry and hence overtopping discharges will be greater. Previous research (Reference 5) has indicated that there is little difference in wave transformation between 1:100 and significantly shallower slopes. It is therefore recommended that graphs representing the 1:100 bathymetry can reasonably be used for shallower slopes. All of the graphs detailed in Figures 1 - 12 represent offshore bathymetries of constant slope. Actual bathymetries at prototype structures are likely to be considerably different from these idealised slopes. The designer must therefore derive an overall beach slope. Particular attention should be given to the angle of the bathymetry immediately seaward of the structure toe as this area of the slope will principally define the inshore wave conditions and hence the overtopping discharges.

Two examples are detailed below in the use of Figures 1 - 12 and the associated design method.

<u>Example problem 1</u>

Given: Vertical seawall
Approach bathymetry 1:30
Elevation of seawall toe -1.0m ODN
Crest elevation 4.0m ODN

Find the overtopping discharge when:

Still water level 2.0m ODN
Offshore significant wave height, H_{so} = 2.0m
Offshore mean wave period, T_m = 6.0s

$$s_{om} = 2\pi H_{so}/(g\,T_m^2)$$
$$= 2 \times 3.14 \times 2.0/(9.81 \times 6.0^2)$$
$$= 0.036$$

∴ Use Figure 5

Now $h/H_{so} = 3.0/2.0 = 1.5$
$R/H_{so} = 2.0/2.0 = 1.0$

From Figure 5

$$Q^{\#} = Q/(2g\, H_{so}^{3})^{½} = 3 \times 10^{-3}$$

$$\therefore \quad Q = 3 \times 10^{-3} \times (2 \times 9.81 \times 2.0^{3})^{½} = 0.038 \; m^{3}/s/m$$

Example problem 2

Given: Vertical seawall
Approach bathymetry 1:50
Elevation of seawall toe -1.0m ODN
Crest elevation 4.5m ODN

Find the overtopping discharge when:

Still water level 3.0m ODN
Offshore significant wave height, H_{so} = 2.0m
Offshore mean wave period, T_m = 7.2s

$$s_{om} = 2\pi H_{so}/(g\, T_m^{2}) = 2 \times 3.14 \times 2.0/(9.81 \times 7.2^{2}) = 0.025$$

∴ Use Figures 2, 3, 5 and 6

Now $h/H_{so} = 4.0/2.0 = 2.0$
$R/H_{so} = 1.5/2.0 = 0.75$

Approach bathymetry	s_{om}	$Q^{\#}$	$Q(m^3/s/m)$
1:30	0.017	9×10^{-3}	0.113
1:100	0.017	9×10^{-3}	0.113
1:30	0.036	3.5×10^{-3}	0.044
1:100	0.036	2.4×10^{-3}	0.030

The actual overtopping discharge will therefore be somewhere in the region 0.113 - 0.030 $m^3/s/m$. In order to ensure a conservative solution it is suggested overtopping is taken to be 0.113 $m^3/s/m$. A more accurate assessment of overtopping discharges may be derived through the use of physical model tests.

4 Conclusions

1. A series of physical model tests have been carried out to measure discharges overtopping vertical seawalls. The tests covered a range of approach bathymetries, seawall heights and wave and water level conditions.

2. The measured overtopping discharges generally gave good agreement with the work of Goda (Reference 3). Discharges were greatest for values of $1.4 < h/H_{so} < 2.0$; conditions which produce breaking waves at or just offshore of the structure. For $h/H_{so} < 1.4$ waves break on the approach slope and so wave conditions at the structure are reduced. Conversely, for $h/H_{so} > 2.0$ waves are not large enough to break on the approach slope for a given set of conditions. Steeper approach slopes generally produced greater overtopping discharges.

3. Based on the analysis of the test results graphs have been produced to enable designers to estimate the overtopping discharge performance of vertical seawalls. If a high degree of accuracy is required then it is recommended that physical model tests are undertaken.

5 Acknowledgements

This report describes work carried out by members of the Coastal Group at HR Wallingford. The study was designed and supervised by Dr D M Herbert who also carried out the analysis and the reporting. Mr A R Channell completed the experimental work.

6 References

1. Design of seawalls allowing for wave overtopping." Hydraulics Research Station, EX 924, June 1980.

2. Effectiveness of recurved wave return walls." HR Wallingford, SR 261, ~~February 1991~~.

3. Goda, Y, Kishira, Y, and Kamiyama, Y. Laboratory investigation on the overtopping rates of seawalls by irregular wave. Ports and Harbour Research Institute, Vol 14, No 4, pp 3-44, 1975.

4. Hydraulic performance of breakwater crown walls. HR Wallingford, SR 146, March 1988.

5. Seelig, W N, and Ahrens, J P. Estimating nearshore conditions for irregular waves. CERC, Technical Paper No 80-3, 1980.

Figures

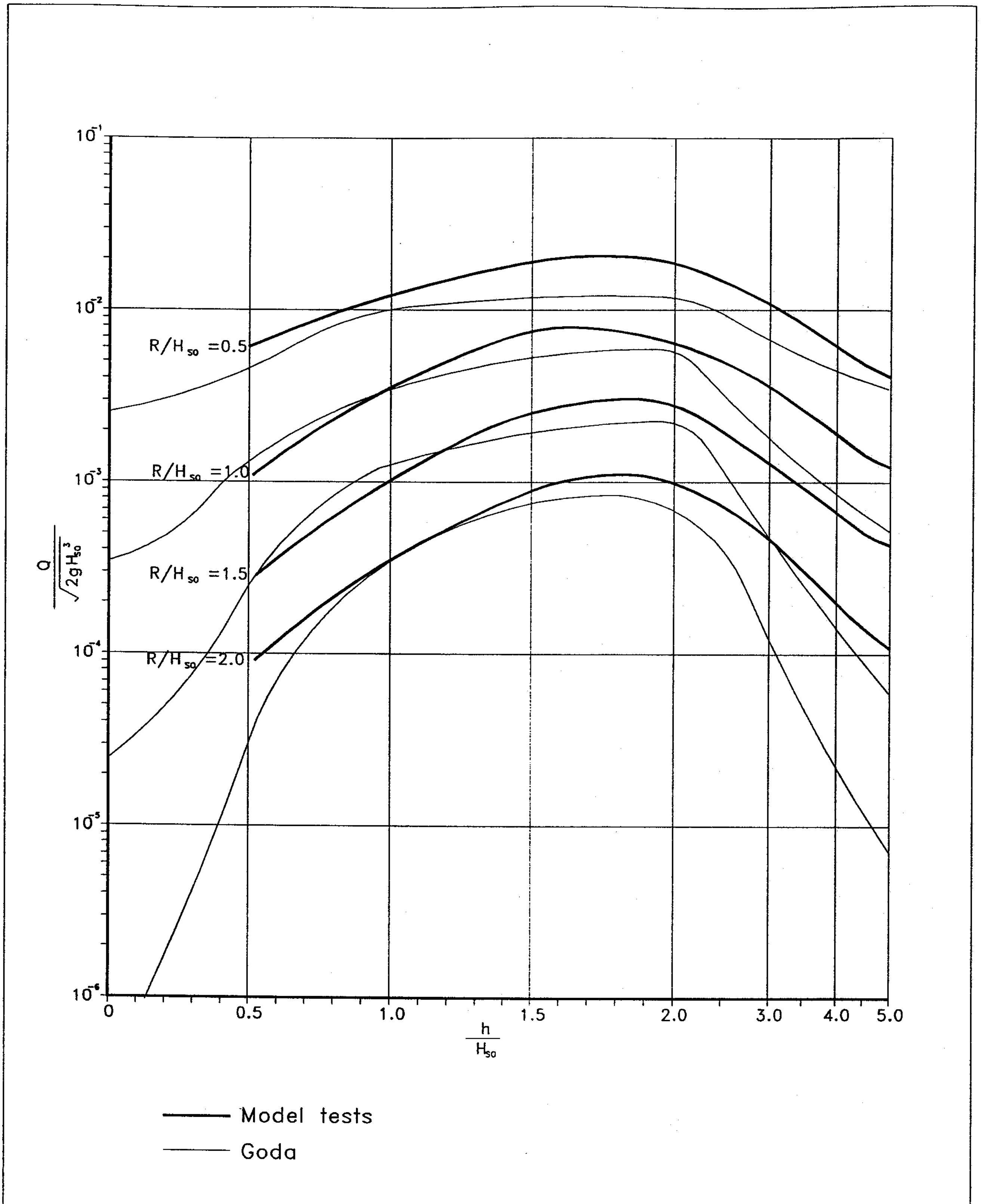

Figure 1 Overtopping discharges, 1:10 slope, Som=0.017

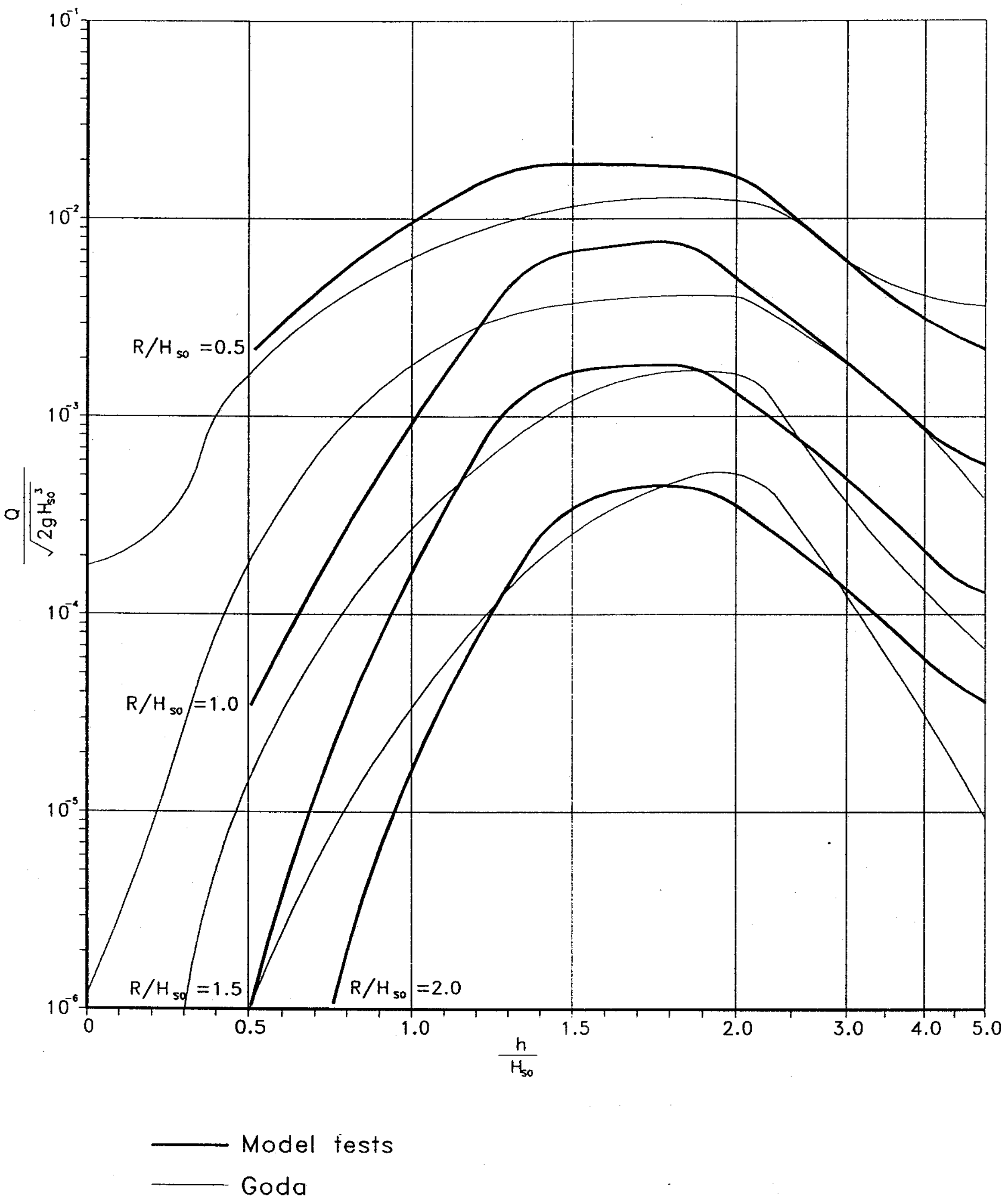

Figure 2 Overtopping discharges, 1:30 slope, Som=0.017

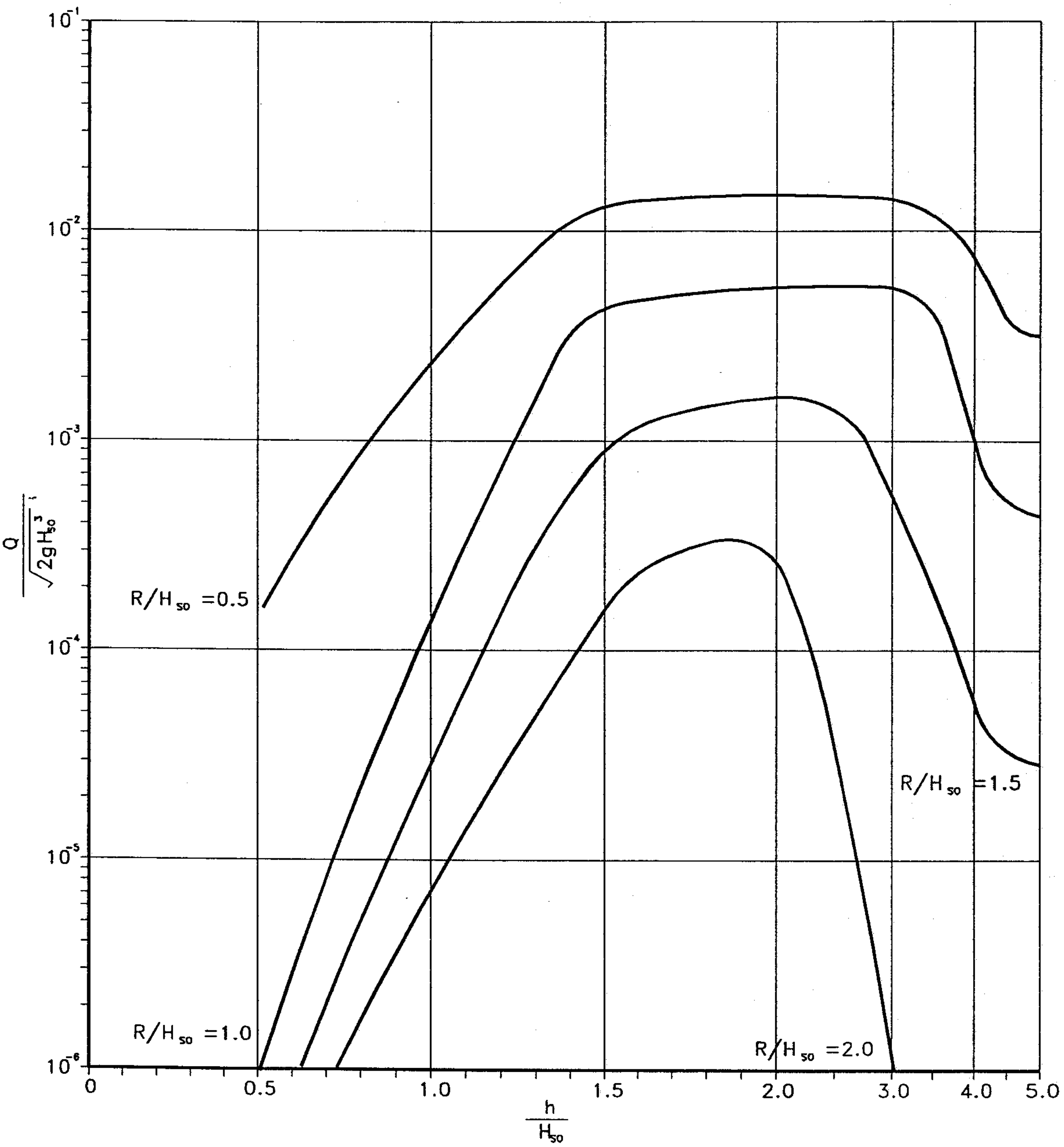

Figure 3 Overtopping discharges, 1:100 slope, Som=0.017

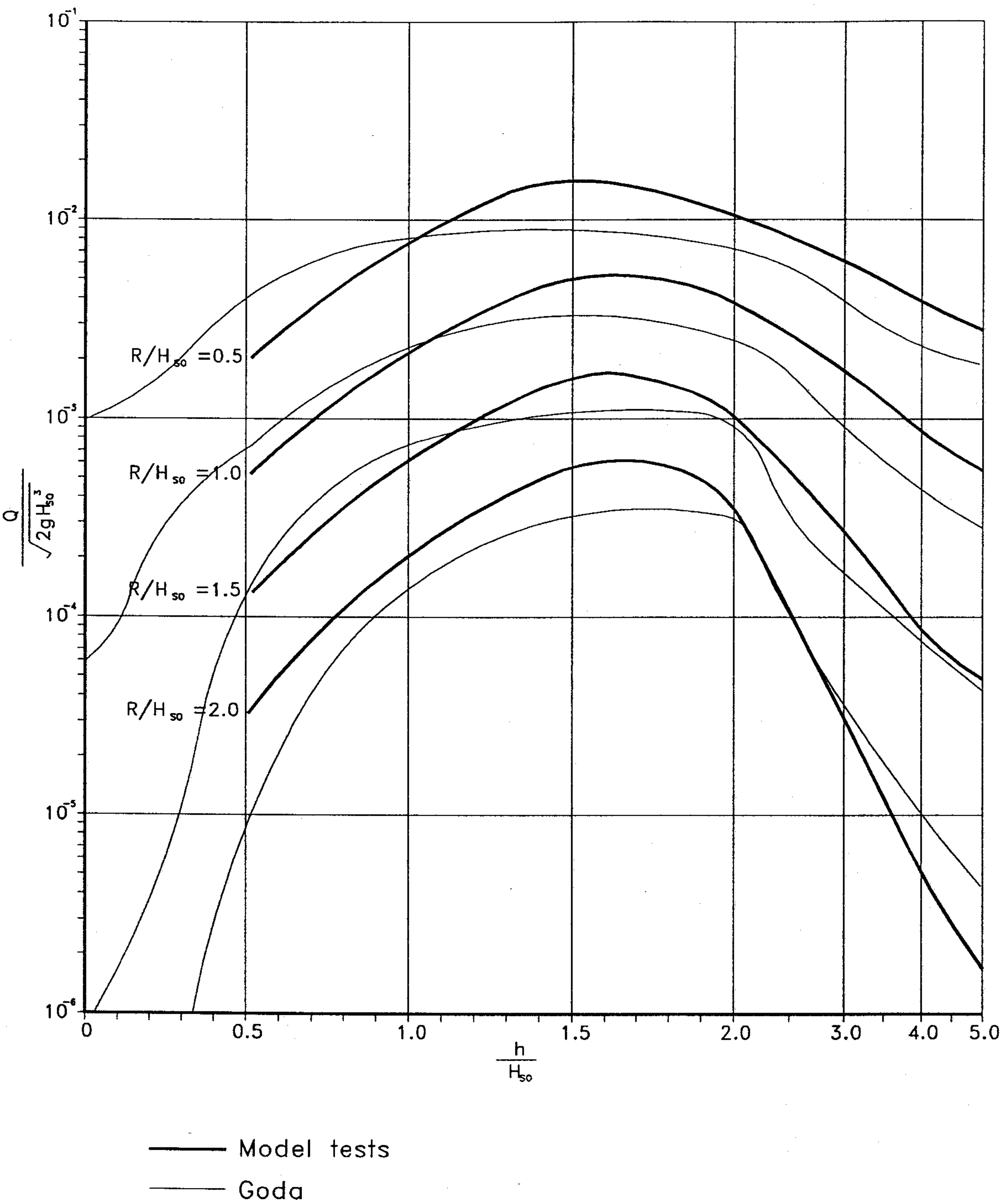

Figure 4 Overtopping discharges, 1:10 slope, Som=0.036

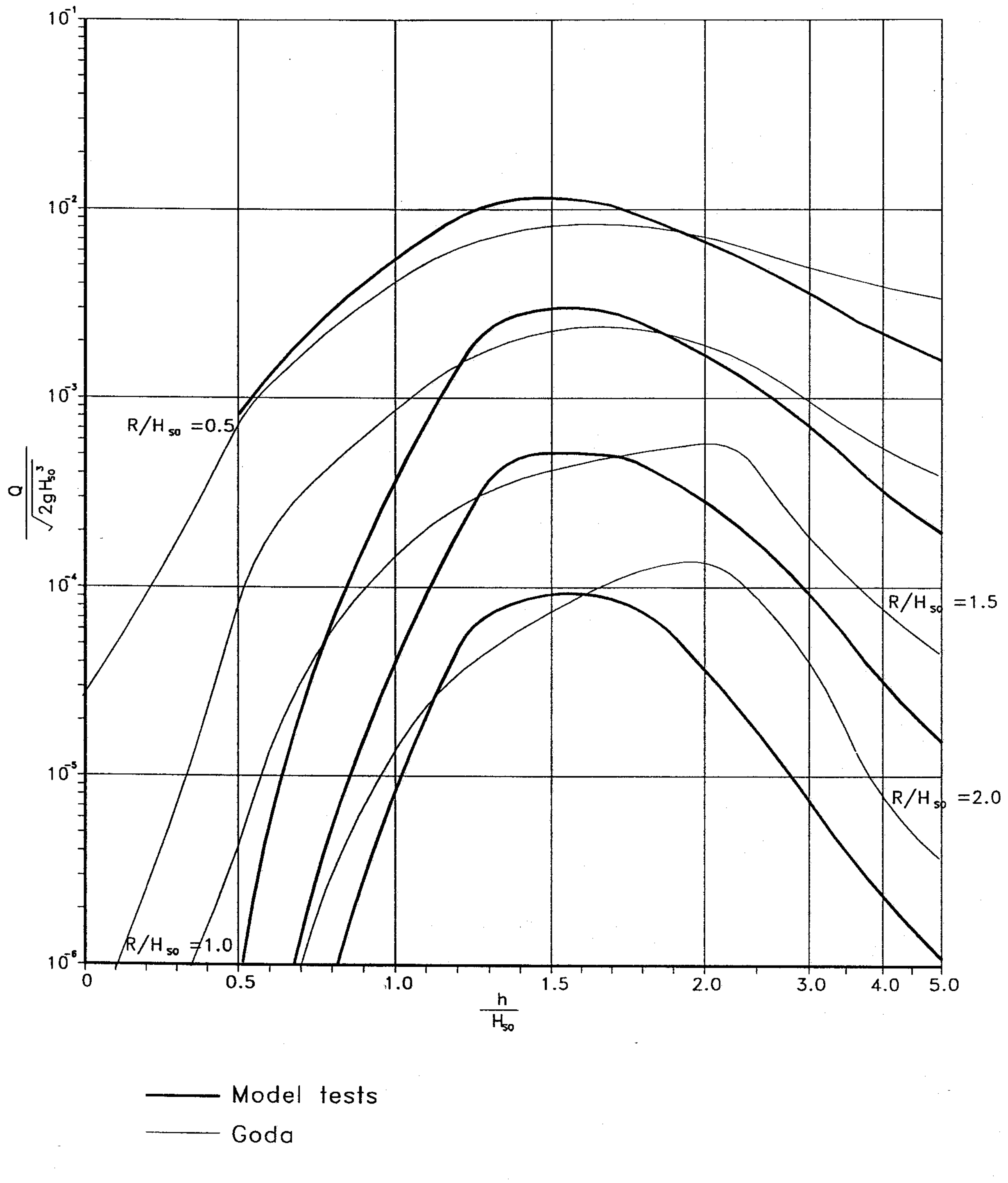

Figure 5 Overtopping discharges, 1:30 slope, Som=0.036

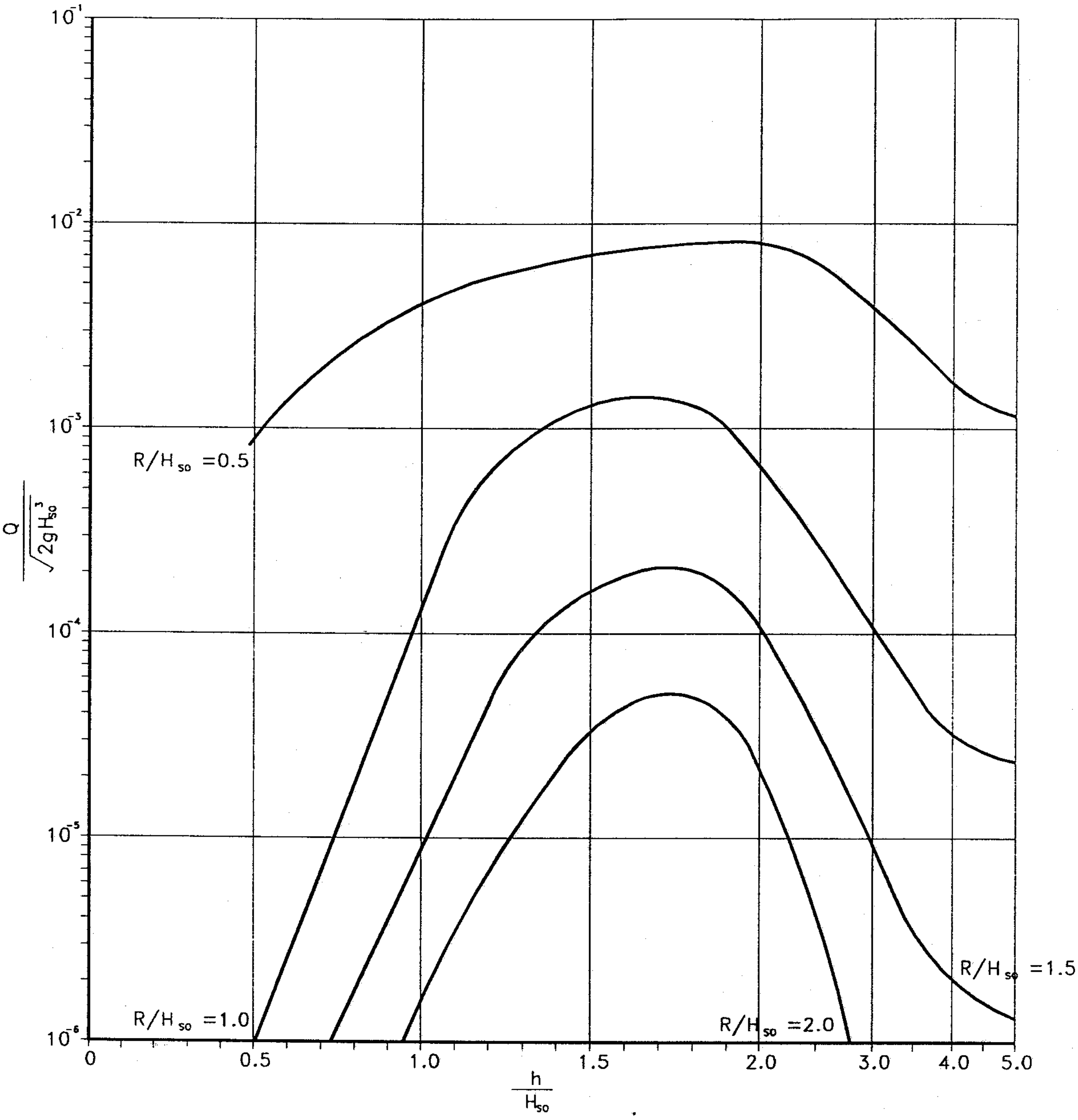

Figure 6 Overtopping discharges, 1:100 slope, Som=0.036

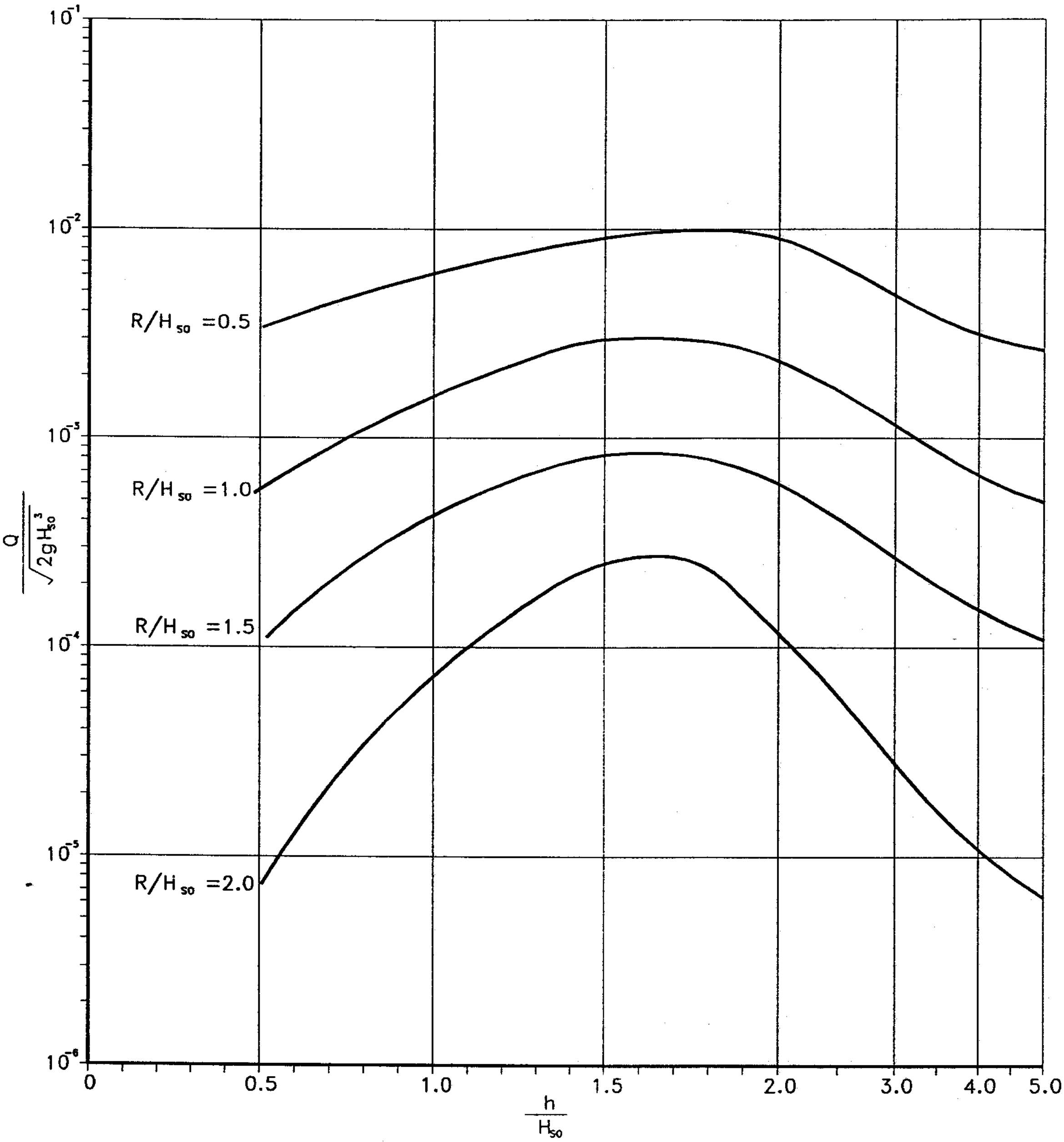

Figure 7 Overtopping discharges, 1:10 slope, Som=0.045

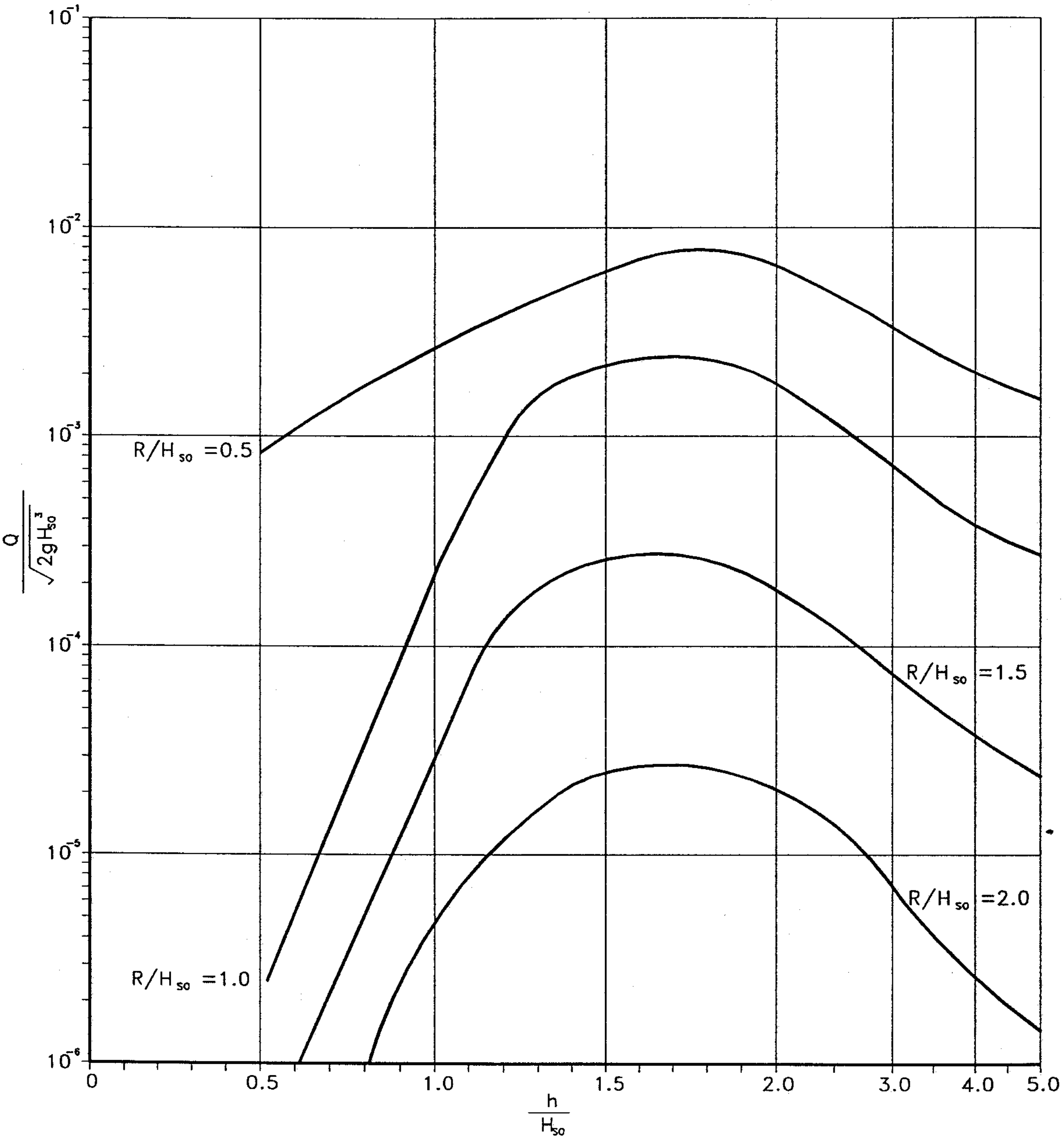

Figure 8 Overtopping discharges, 1:30 slope, Som=0.045

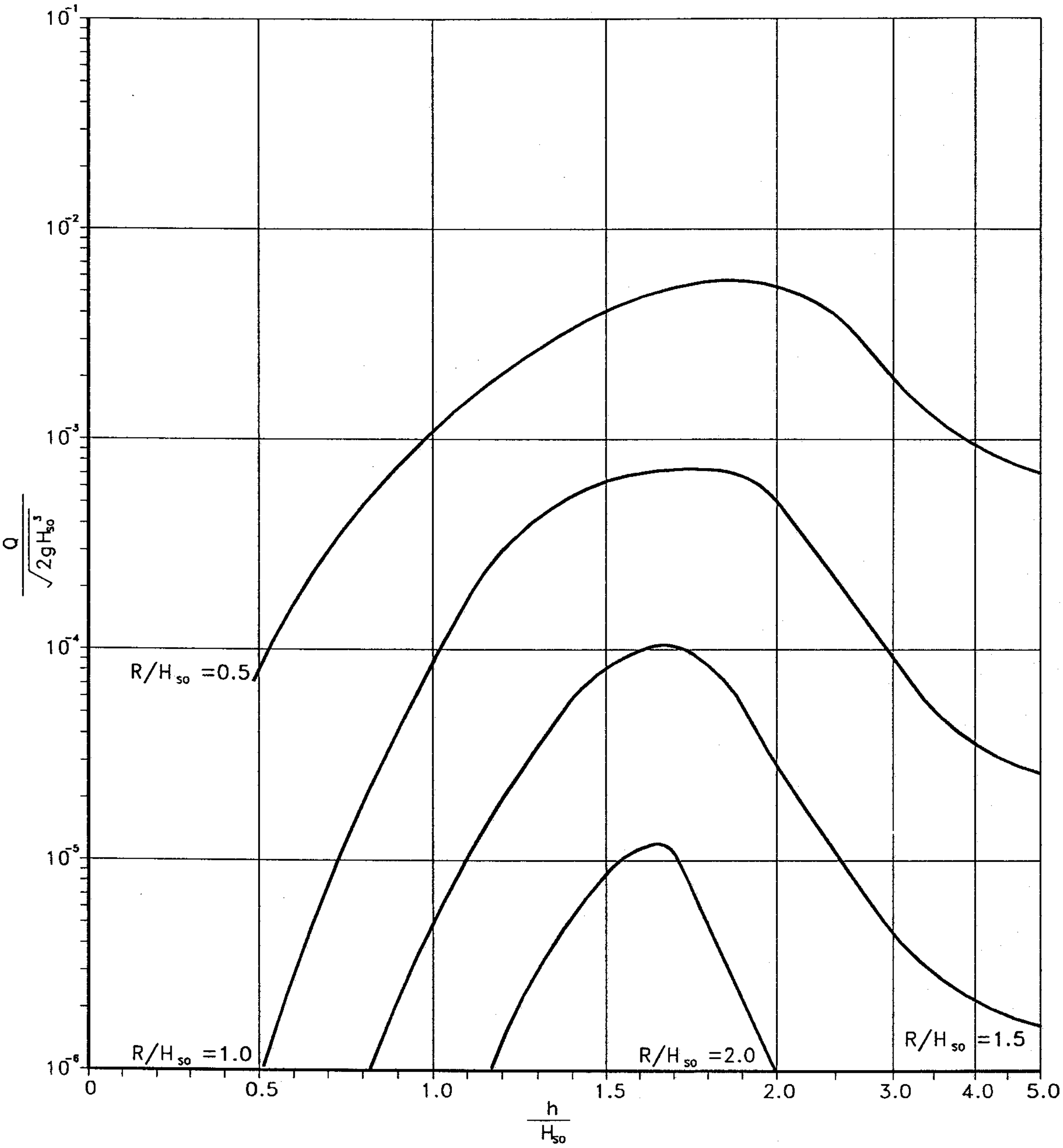

Figure 9 Overtopping discharges, 1:100 slope, Som=0.045

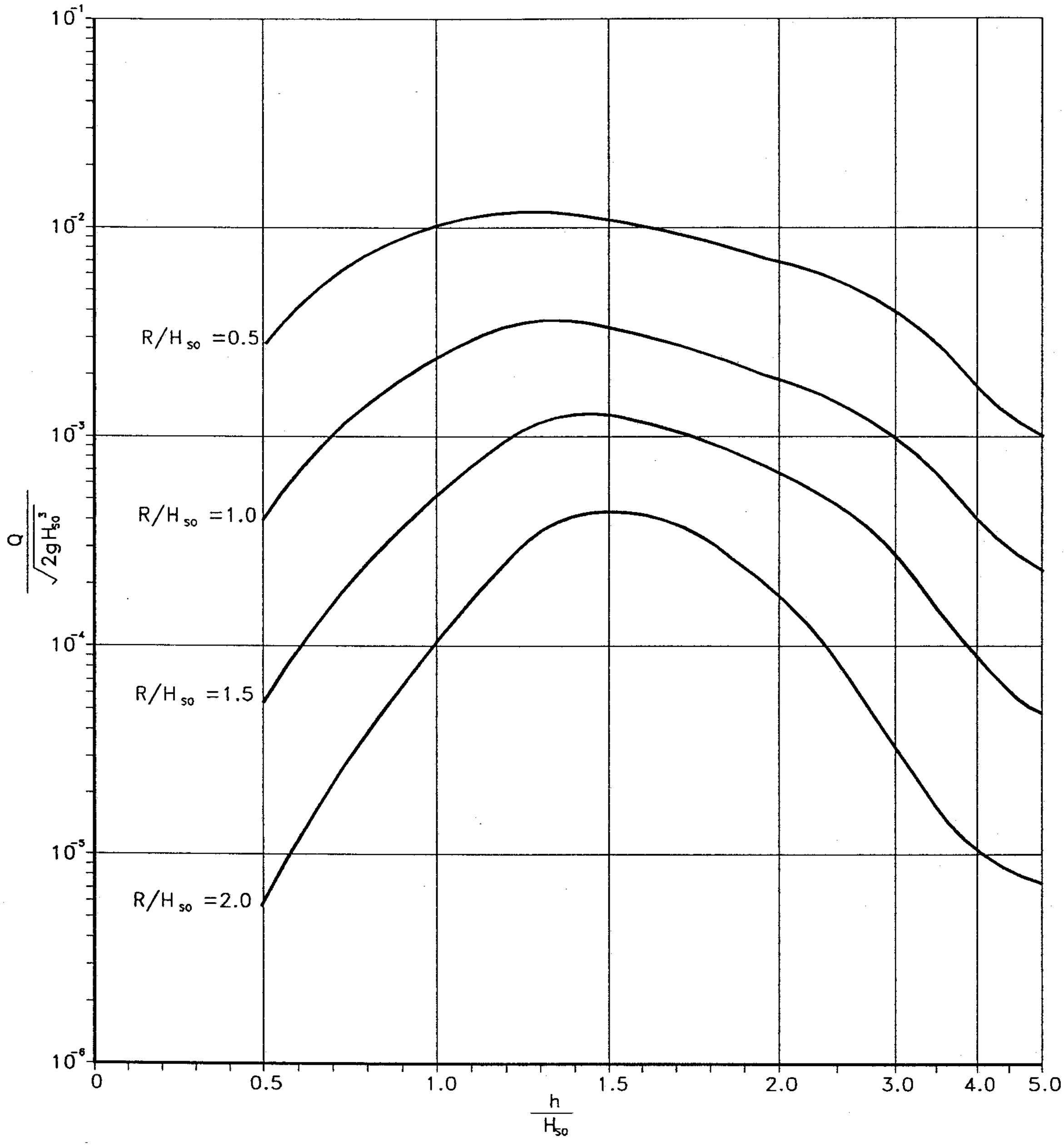

Figure 10 Overtopping discharges, 1:10 slope, Som=0.060

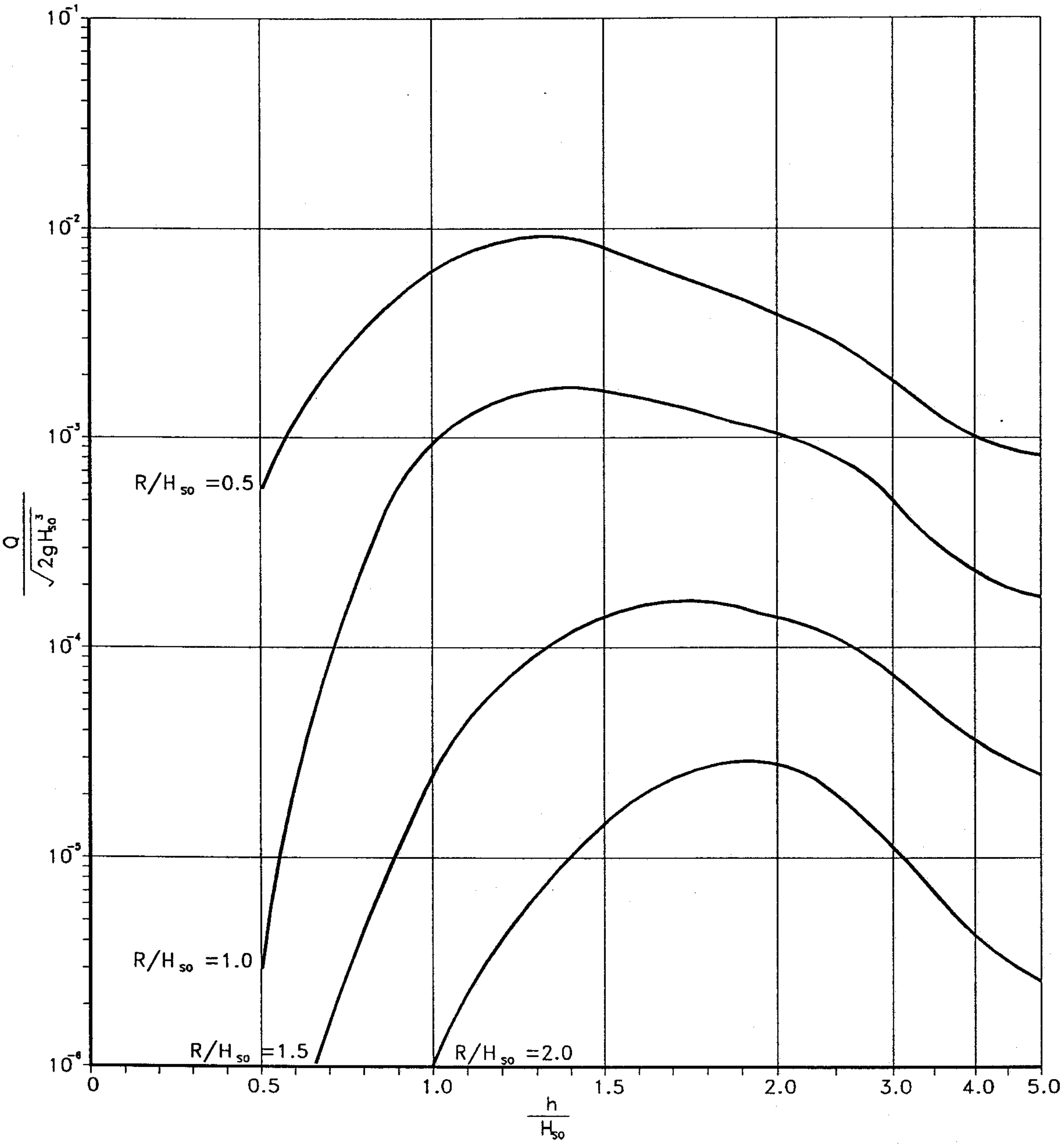

Figure 11 Overtopping discharges, 1:30 slope, Som=0.060

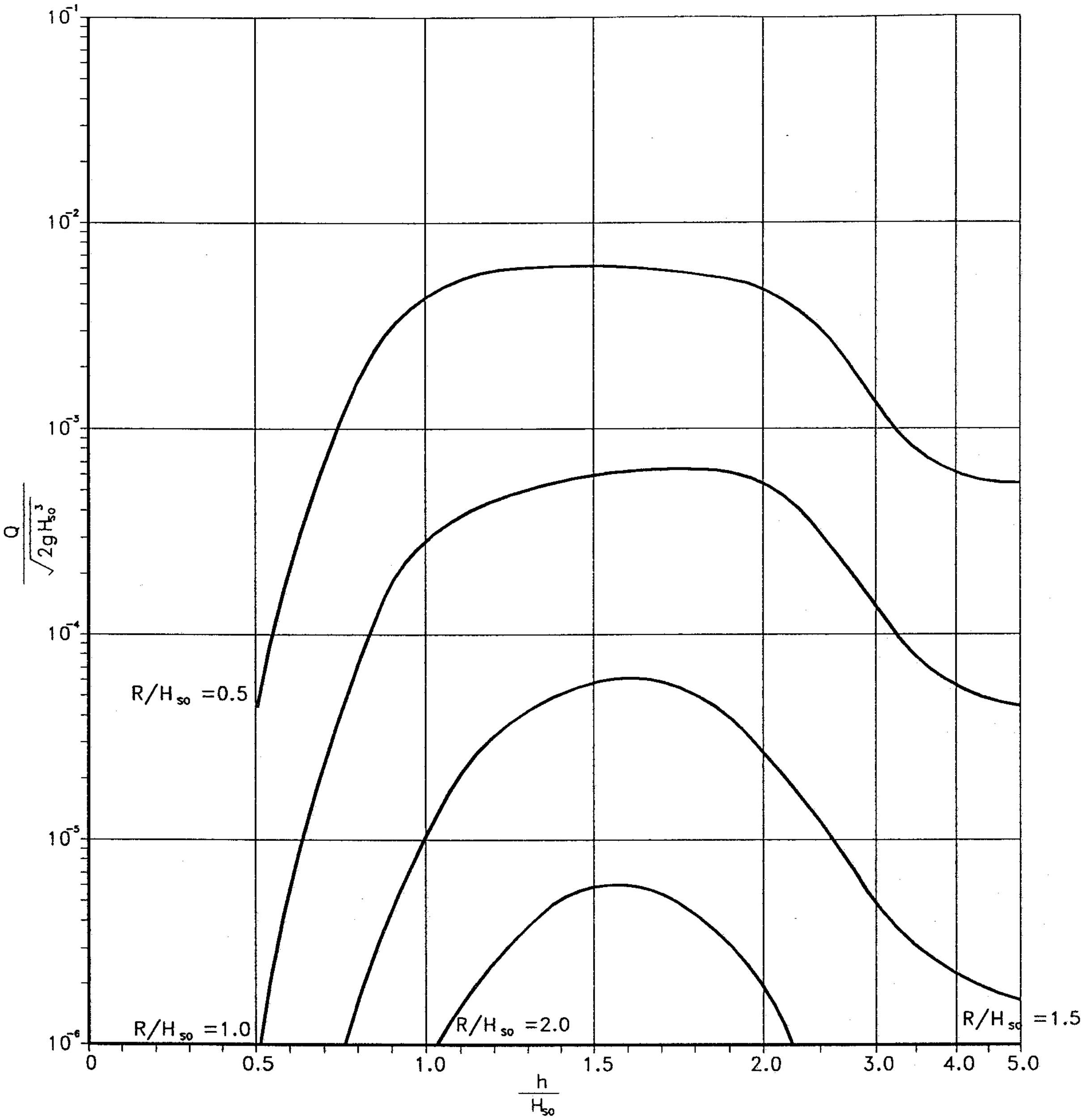

Figure 12 Overtopping discharges, 1:100 slope, Som=0.060

Appendices

Appendix A

Physical model test facility

Physical Model Test Facility

All the model tests for this study were carried out in a wave flume measuring 50m long by 1.22m wide by 1.1m deep and having a nominal working depth of 0.70m. The wave generator is a wedge type random wave paddle powered by a double acting electro-hydraulic ram controlled by a BBC micro-computer. This system was developed at HR from an older "hard wired" wave spectrum synthesizer. This combination of synthesizer and wave generator is capable of producing any required deep water ocean wave spectrum that can be described by 16 spectral ordinates. The BBC micro computer wave spectrum synthesizer produces a random wave spectrum by digitally filtering a white noise signal via a shift register. Varying lengths of wave sequence can be produced on this shift register which is used in conjunction with a clock pulse generator (Reference A1). This allows a repeatable pseudo-random sequence of outputs to be generated creating sequences of waves with repeat times varying from a few minutes to several tens of years depending on the test requirements.

The wave flume is divided along its length into two channels by a vertical splitter wall which increases in porosity as it approaches the generator end of the flume. This porous divide wall helps prevent the generation of cross waves as well as dissipating any energy reflected back from the structure being tested. The smaller of these two channels (0.47m wide) is of constant depth and ends in a shingle spending beach of 1:5 gradient. This channel is used to measure the "deep water" wave conditions produced by the generator. The wider channel (0.75m wide) contained the model and sea bed profile under test.

References

A1. Wave spectrum synthesizers. E&ME Tech Memo 1/1972, Hydraulics Research Station, June 1972.

Appendix B

Spectral analysis and wave counting programs

Spectral Analysis and Wave Counting Programs

The BBC micro computer wave spectrum synthesizer produces a random wave spectrum by digitally filtering a white noise signal via a shift register. Varying lengths of wave sequence can be produced on this shift register which is used in conjunction with a clock pulse generator (Reference B1). This allows a repeatable pseudo-random sequence of outputs to be generated creating sequences of waves with repeat times varying from a few minutes to several tens of years depending on the scaling parameters.

For this study two types of wave sequence were used, ie either a short repeating sequence of about 6 to 10 mins duration or one of between 4 and 5 hours. The former was used for the wave flume calibration checks whilst the long sequences were used for wave overtopping tests.

Two types of wave analysis program were also used, one for each type of wave condition. For the short tests a spectral analysis program was used where data recording takes place over one complete wave generation sequence thus eliminating any statistical uncertainty in the results. The water level at the twin wire wave probe (Reference B2) is recorded by the Compaq micro-computer at every clock pulse of the synthesizer, typically every 0.1-0.2 seconds. A maximum of 4096 data points can be collected from up to 16 probes at one time using this program. The analogue output of the wave probe, representing a displacement relative to a static water level, is first converted to a digital form by an A-D convertor and then to an elevation in prototype metres via the model scale. Hence at the end of sampling a series of water level elevations are known for every clock pulse, ie up to 4096 points. This program then uses a Fast Fourier Transform technique (Reference B3) to convert the time base data into the frequency domain and then splits the data into individual sine waves to extract the energy content of each frequency component. From this data the energy/frequency spectrum can be set up from which values of significant wave height, H_{mo}, and average wave period, T_{ij}, can be estimated using various moments of the spectrum.

The second type of analysis is a wave counting program which is used during the long sequence tests where, as on this study, recording may last up to 60 mins. Here an initial 'mean value of water level' is calculated by sampling the water surface elevation several hundred times during the first few waves of the test, eg a value of 1000 points is not unusual. This value of mean water level is regularly updated throughout the run which then continues with the sampling of waves for analysis where measurements of water surface elevation are made relative to the mean value of the water level. Although the sampling rate of the wave probe is about the same as that of the spectral analysis program, only about 5 points are used to define each 'wave', where a wave is defined as lasting between two successive down crossings of the mean water level. All these values of elevation are squared and summed throughout the recording of each 'batch' or group of waves, the number of which is specified by the user. Typically 5 or 6 batches each of 200 or 300 waves would be used. At the end of each batch 'wave heights', calculated from the sum of the maximum departure above and below the mean in each wave, are sorted into descending order from which statistical values of $H_{1/3}$, H_{MAX}, H_{10} etc can be found. The total number of points recorded in each batch is divided by the number of waves and combined with the sampling rate to give the mean wave period T_{BAR}. Results for this batch are then output and the data discarded before moving onto the next batch.

At the end of sampling the arithmetic mean and standard deviation of $H_{1/3}$ and T_{BAR} over all the batches is calculated and printed out. It is these two values which are then used to represent the wave conditions of the test. The value of $H_{1/3}$ is defined as being the average height of the one third highest waves and is generally quite close to the spectral analysis equivalent of H_{mo}. Similarly T_{BAR} and T_{ij} are also considered to be comparable. This data is followed by two histograms based on wave height giving wave height class and the number of waves in that class. One histogram is accumulated throughout the batch and its totals are added into the other histogram which is accumulated over all batches. Since they are too lengthy to be output during sampling only the histogram for the final batch is output at the end. All the above information applies to each of the wave probes being monitored during the test.

References

B1. Wave spectrum synthesizers. E&ME Tech Memo 1/1972, Hydraulics Research Station, June 1972.

B2. Twin wire wave probe modules. Tech Memo 3/1974, Hydraulics Research Station, October 1974.

B3. The fast Fourier transform with applications to spectral and cross spectral analysis. Hydraulics Research Station, Int Report 100, December 1972.

Appendix C

Overtopping measurement

Overtopping Measurement

The method of measuring overtopping discharge was consistent for all the tests and was based on a standard procedure employed at HR. A set of five overtopping intervals are recorded where water discharging over the seawall is collected in a calibrated tank. Here a magnetorestrictive float monitors the difference in water level from which the volume can be calculated. The overtopping sequences are separated by periods during which water overtopping the wall is returned to the flume. This diversion of water is controlled by a hinged, manually operated flap gate. During these intervals the water collected in the tank is allowed to settle before the level is recorded. If necessary the water is then pumped out and returned to the main flume. Three calibrated tanks are available allowing a range of volumes from 64-556 litres (model) to be recorded in any one interval. The resolution is 0.04 litres (model).

The length of each overtopping and recording interval was based on the nominal mean wave period (T_m) of the wave spectrum being used. Each overtopping interval was $100T_m$ seconds with each recording interval being $200T_m$ seconds. Each test was preceded by a $300T_m$ 'run-in' time. Five overtopping intervals were recorded allowing a mean and standard deviation of volume (and hence discharge) to be calculated. The full test sequence was thus:

$300T_m$: Run-in
$100T_m$: overtopping 1
$200T_m$: recording 1
$100T_m$: overtopping 2
$200T_m$: recording 2
:
:
$100T_m$: overtopping 5
Stop

giving $1600T_m$ seconds for the whole test. With the range of input conditions used these tests lasted between 27 and 60 minutes. In some instances the quantity of overtopping water recorded in the first measuring interval was very small. For these tests, in order to obtain an acceptable resolution, wave overtopping was measured continually over $1000T_m$.