Reservoir Dams: wave conditions, wave overtopping and slab protection

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Summary

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Severe wave action in reservoirs caused by high winds can damage embankment dam faces unless they are adequately protected. Accurate methods of wave prediction, and design methods for blockwork and slabbing are necessary to allow engineers to make reliable assessment of the integrity of the embankment and for the design of new structures.

Wave induced overtopping of dam structures, sometimes termed over-slop, may also cause potential hazards to the crest or downstream slope of the dam, and/or to the area downstream at potential risk of flooding.

This report describes research to simplify and consolidate the prediction of wave action in reservoirs; to calculate wave induced discharges on embankments; and to extend work on blockwork and slabbing protection for dam faces presented by Herbert et al (1995) in the companion report, SR 345.

This report presents a method for the derivation of an appropriate wind speed and fetch length which are used in the Donelan/JONSWAP wave prediction formulae to establish the design wave height and period. It includes a method for determining a crest level based on an acceptable wave overtopping discharge using established formulae. Physical model testing of blockwork and slabbing was undertaken and a new method for designing protection for dam faces was developed. This document describes additional research on the stability of blocks and slabs and provides a guide to design of dam face protection against wave attack.

The report should be used by engineers involved in the inspection, analysis, design and construction of blockwork and slabbing protection against wave attack on faces of embankment dams or similar revetments.

The report is intended to support and extend guidance in SR 345, by Herbert et al (1995), and the 3rd Edition of the Floods and Reservoir Safety Guide, ICE (1996).

If any further information is required on this or related research, pleast contact Professor William Allsop, Manager Coastal Structures at HR Wallingford.

Notation

A,B	coefficients for calculation of wave overtopping
A_f	adjustment factor for calculation of overtopping of recurve walls
A_s	area of slabbing unit
D_f	discharge factor
D_{f15}	15% sieve value for filter layer material
d	depth of water at toe of structure
F	fetch length, in metres
g	acceleration due to gravity
H	wave height
H_D	design wave height
H_{max}	maximum wave height
H_s	significant wave height, average of highest one-third of wave heights
H_{sb}	significant wave height after breaking
Q^*	dimensionless overtopping discharge
q	mean overtopping discharge
q_w	mean overtopping discharge over wave return wall
R_c	dam crest level
R_f	run-up factor
$R_{u2\%}, R_{us}$	2% exceedance or significant run-up levels
R^*	dimensionless crest level of embankment
r	roughness factor
S_a	altitude factor
S_b	empirically derived value relating to stability of blockwork
S_c	empirically derived value relating to stability of slabbing and blockwork
S_d	directional factor
S_f	duration factor
S_s	seasonal factor
S_p	probability factor
S_w	over-water speed-up factor
s_m, s_p	mean or peak wave steepness, $= 2\pi H_s/(gT_m^2)$ or $2\pi H_s/(gT_p^2)$
T_m, T_p	mean or peak wave periods
t_a, t_f	depth of armour layer, slab or block, or depth of filter layer
U	design wind speed, m/s
V_b	basic wind speed, m/s
W_h	height of wave return wall
W^*	dimensionless height of wave return wall
w	gap width between slabbing units
X^*	adjusted dimensionless sea wall crest level
x	empirically derived coefficient for calculation of revetment slope stability
α	angle between the plane of the revetment slope and the horizontal plane
Δ	relative buoyant density of material, eg $(\rho_r/\rho_w - 1)$ or $(\rho_c/\rho_w - 1)$
Δ_s	altitude of site above mean sea level
ξ_m, ξ_p	mean or peak Iribarren number, $= \tan\alpha/s_m^{1/2}$, or $\tan\alpha/s_p^{1/2}$
ρ_r, ρ_c	density of rock, or concrete
ρ_w	density of water
Ω	ratio of gap area to surface area of cover layer units

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1 Introduction

1.1 Background
This report describes research to simplify and consolidate the prediction of wave action in reservoirs; to calculate wave induced discharges on embankments; and to extend work on the stability of blockwork and slabbing protection for dam faces presented in SR345 by Herbert et al (1995). The previous work identified aspects of wind and wave prediction and slabbing stability which required more investigation. HR Wallingford were therefore commissioned by the Department of the Environment in April 1994 to undertake a further programme of research. This document describes the additional research and provides a guide to design of dam face protection against wave attack.

1.2 Objectives of the study
The original objectives of the present study were to:

1. Derive simple wave prediction methods for British inland reservoirs taking account of typical fetch lengths and breadths, and recommend minimum wave allowances.

2. Identify the reduction in armour thickness that may be acceptable by casting large (plan area) slabs to protect embankment dam faces against wave action.

During the course of the study, these objectives were extended slightly to include provision of support by the HR team to the ICE Reservoir Floods Panel in their revision of the Floods and Reservoir Safety Guide, thus to:

3. Derive a design method to find the required crest level of a dam to ensure that wave overtopping is within acceptable limits.

4. Develop a revised design method for both blockwork and slabbing, to optimise the cover layer thickness or, if preferred, the slab area, slope angle or filter thickness may be adjusted.

1.3 Outline of the report
This report looks at three aspects of embankment dam design against wave action:

- The derivation of an appropriate wind speed and fetch length which are used to find a design wave height and period.

- The establishment of the crest level required to limit wave overtopping to acceptable limits.

- The design of blockwork and slabbing to provide sufficient slope protection.

These three areas are covered in Chapters 2, 3 and 4 respectively. Each chapter contains a summary of previous work, an assessment of new contributions and the development of a design method. The full design method with worked examples is given in Chapter 5, and conclusions are drawn in Chapter 6.

The development of the new method for determining blockwork/slabbing thickness was supported by model tests which are described in Appendix 1.

2 Winds and waves

Severe wave action in reservoirs caused by high winds can damage embankment dam faces unless they are adequately protected. Accurate methods of wave prediction are therefore needed to allow engineers to make reliable assessment of the dam integrity and for the design of new structures. This

chapter reviews current methods for wave prediction based on known wind speeds and fetch lengths, and extends and clarifies the method recommended by Herbert et al (1995) in SR 345.

2.1 Previous work

The wave heights in any restricted area of water depend essentially upon the over-water wind speed, the fetch length, and the duration for which the wind blows.

2.1.1 Wind speed derivation

Wind data from a reliable source is required to predict extreme wave conditions. Previous design methods suggest that the Meteorological Office is approached to provide extreme wind speeds and directions from the most suitable weather station. This is indeed valid for large scale projects. For initial work and/or for studies on less important structures, methods are available however to predict the design wind speed at the reservoir site starting with the basic wind speed used by BSI (1995) as shown in Figure 2.1. A method was proposed in Herbert et al. (1995) to calculate the design wind speed at the reservoir site by applying several wind speed factors.

2.1.2 Wave prediction

In the 1980's HR Wallingford completed wind and wave measurements at Megget and Glascarnoch reservoirs in Scotland. This research concluded that the SMB/Saville and Donelan/JONSWAP methods were the most appropriate for wave prediction in reservoirs. This section discusses the two methods in more detail, and recommends one for use in predicting waves on reservoirs.

The SMB/Saville method uses the 1976 SMB wave prediction curves derived for deep water with a modified fetch length proposed by Saville (1954). Saville's method replaced the direct fetch, measured along the wind direction by an effective fetch. The effective fetch was determined by measuring the lengths of individual fetch rays from the dam at intervals within a 90° arc centered on the wind direction and taking the average. This was the method presented in the now superseded Floods and Reservoir Safety Guide, ICE (1978).

JONSWAP was a comprehensive, international field study of wave growth and dissipation held in the late 1960's. The main purpose of this project was to determine the physical processes governing the development of the wave spectrum under the action of the wind. Under east winds, fetch-limited waves were measured along a profile extending 160 km from the Island of Sylt (where the coastline is almost straight) in the German Bight. The results from JONSWAP are given in Hasselmann et al. (1973).

It was found that the principal features of wave growth could be accounted for by non-linear wave-wave interactions. The energy transfer due to these interactions controls the overall balance within the wave spectrum and also accounts for most of the wave growth on the forward face of the spectrum. Measured wave spectra with narrow peaks, overshoot factors and fetch-dependent high frequency tails could be explained in terms of the non-linear energy transfer due to wave-wave interactions.

One aspect of JONSWAP of particular interest to coastal engineers was the form of the wave spectrum at limited fetch and the relation between the integrated properties of this spectrum (significant wave height and average period) with wind speed and fetch. It was found that fetch-limited spectra obeyed the similarity hypothesis and scaling laws proposed by Kitaigorodskii et al (1962), that time and distance are scaled by gt/U and gx/U^2 respectively, where t is time, x is distance, g is the acceleration due to gravity and U is wind speed. It is these relations which are used by Donelan (1980) and recommended here (see section 2.3.2) for application to wave prediction in a reservoir.

The results from JONSWAP have been confirmed by other measurements in the laboratory and in the ocean. In a series of laboratory studies, Mitsuyasu (1968,1969) also showed that the dominant physical term governing the development of waves under the action of the wind was non-linear wave-wave interactions. He gave relations between the wave height and period with fetch using the scaling laws of Kitaigorodskii. These relations are close to those proposed by JONSWAP.

2

Wave measurements by Liu (1981) in Lake Michigan, Kahma (1981) in the Gulf of Bothnia and Donelan (1985) in Lake Ontario have also confirmed, with small differences, the fetch-limited relations first given in JONSWAP. Wave measurements made by the Institute of Oceanographic Sciences, Birch & Ewing (1986) in the QE II storage reservoir at Hersham, Surrey, also give scaled relations from measurements with fetches of about 1 km in agreement with JONSWAP.

The main scientific conclusion from JONSWAP (that wave-wave interactions control the development of the wave spectrum) has lead to important developments in numerical wave modelling. A "third-generation" wave model has been developed by the WAM (Wave Modelling Group) which is now in routine use at several meteorological agencies and oceanographic institutes. The source terms used in present models follow the concepts and ideas from JONSWAP, but with improvements and refinements in the physical terms from theoretical and experimental advances since the JONSWAP study was completed. The work of the WAM group has been published in a recent book on the dynamics and modelling of ocean waves by Komen et al. (1994).

Donelan (1980) gives a method for wave prediction over an area for which the fetch is not uniform, but irregular as in reservoirs and in many coastal locations. In this method, the fetch length is not defined by the wind direction, but is based on the wave direction. Furthermore, the wind speed used for determining the wave properties is that component of the wind along the wave direction.

Wave height prediction has been based on methods derived for coastal water conditions, but adapted for the limited fetch lengths of reservoirs. The series of wind and wave measurements in Megget in 1985/86 and Glascarnoch in 1987/88, have been compared with wave heights predicted by a number of methods, Owen (1987) and Owen & Steele (1988). Further comparison and discussion was made by Herbert et al. (1995). This work concluded that the Donelan/JONSWAP method gave the best estimate of significant wave height, and any errors associated with this method are generally on the conservative side.

2.2 Comparison of wave prediction methods
Further to work carried out by Herbert et al. (1995), comparison has been made between the full Donelan/JONSWAP method and a simplified method as proposed in Section 2.3.2, for three reservoirs namely Glascarnoch, Megget and Grafham Water.

The results showed that there was no significant difference in the results obtained. The simplified method under-predicted the wave height by about 2.4% for Glascarnoch, over-predicted the wave height at Megget by approximately 0.3% and under-predicted by 1.1% at Grafham Water

It is recommended that in line with Herbert et al. (1995), the Donelan/JONSWAP method is used for the prediction of wave conditions on UK reservoirs. It is further proposed that the Donelan/JONSWAP method can be simplified as shown in the following sections.

2.3 Recommended method
This section outlines the recommended method for calculating wave heights and periods on inland UK reservoirs. The method firstly derives a design wind speed using an approach based on that used by BSI (1995) in BS 6399 Part2. This design wind speed is then used as input into the wave prediction method.
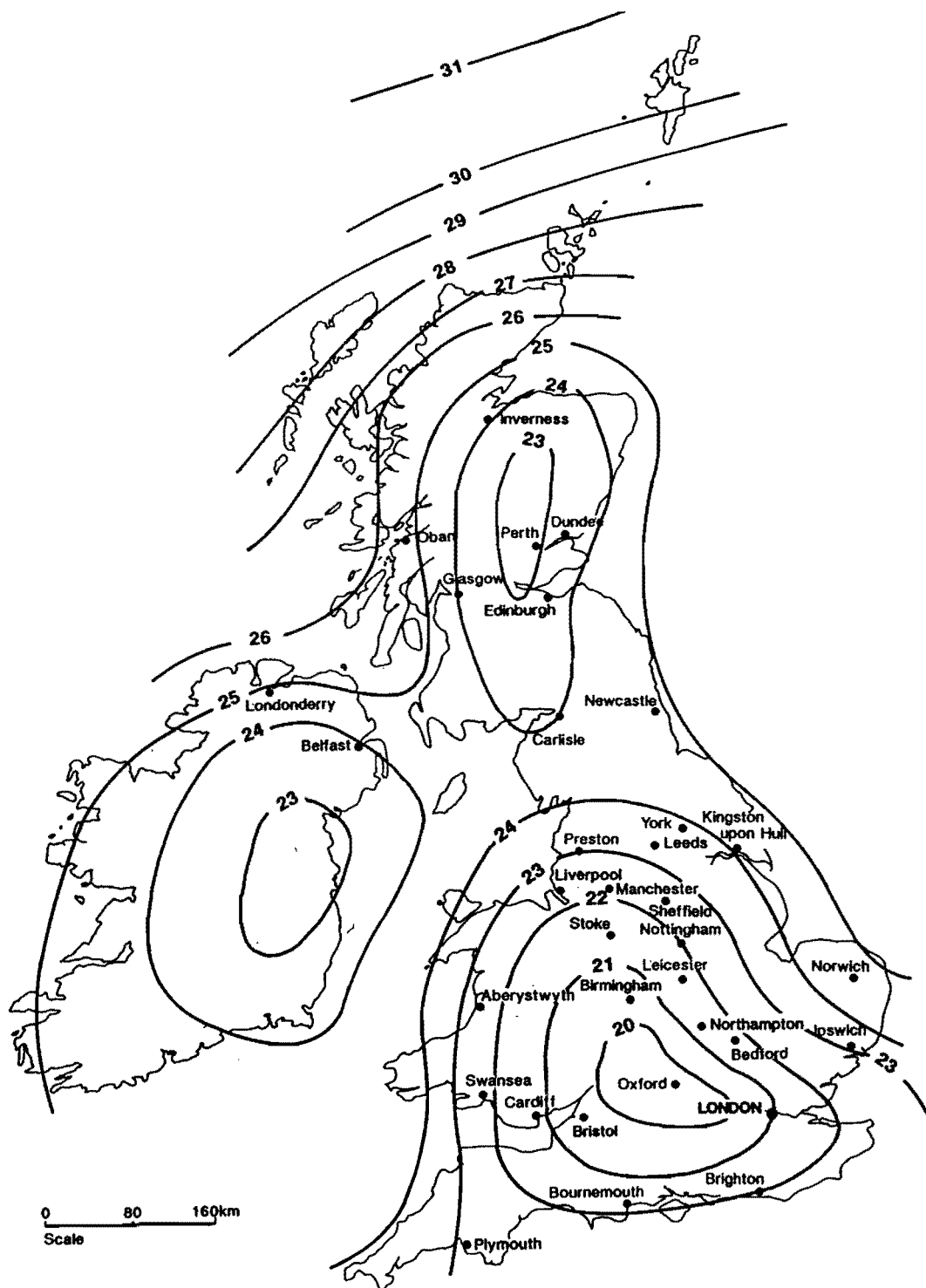
Figure 2.1   *Basic wind speed map*
(source ICE (1996))

2.3.1 Wind speed derivation

The design wind speed U is calculated using a basic wind speed modified by a series of factors:

$$U = V_b S_a S_d S_s S_p \quad\quad (2.3)$$

where V_b is the basic wind speed from Figure 2.1
 S_a is an altitude factor
 S_d is a directional factor
 S_s is a seasonal factor
 S_p is a probability factor

The basic windspeed V_b corresponds to the fifty-year hourly mean windspeed irrespective of wind direction, reduced to a height of 10m over completely flat terrain at sea level. It is assumed that the terrain is of uniform roughness and equivalent to typical open country in the UK.

The problem addressed by these studies differs somewhat from the usual wind loading problem addressed by BSI (1995) in BS 6399 Part2, so two extra factors have been applied to tailor the wind speed prediction formula for use in this application; a duration factor, S_f, and an over-water speed-up factor, S_w. The seasonal factor S_s has been removed from the formula as this is mainly intended for the design of temporary works against spring, summer, or autumn winds only, and is therefore less useful for wave prediction in reservoirs. The wind speed formula therefore becomes:

$$U = V_b S_a S_d S_p S_f S_w \quad\quad (2.4)$$

where each of these factors are described below.

Altitude factor

The altitude factor S_a is applied to adjust the basic wind speed to the altitude of the site above sea level. The following formula is used:

$$S_a = 1 + 0.001 \Delta_s \quad\quad (2.5)$$

where Δ_s is the altitude of the site above mean sea level in metres. This equation compensates for residual topography effects, but if the local topography is considered significant, then S_a should be calculated with reference to BS 6399 Part 2.

One aspect of topography that is considered here is potential wind funnelling along steep sided valleys resulting in the fetch lengths being extended. The use of 'bent' fetches on 'banana' shaped reservoirs is discussed in Section 2.3.2.

Directional factor

The directional factor S_d allows adjustment to be made for any particular direction, resulting in a basic wind speed with the same risk of being exceeded. To be conservative assume $S_d = 1.0$ (factor for a wind direction of 240°N). The values of direction factor for the UK are given in Table 3 of BS 6399 Part 2, and summarised in Table 2.1.

Table 2.1 *Values of direction factor, S_d*

Direction °N	Direction factor S_d
0 North	0.78
30	0.73
60	0.73
90 East	0.74
120	0.73
150	0.80
180 South	0.85
210	0.93
240	1.00
270 West	0.99
300	0.91
330	0.82
Note: Interpolation may be used within this table	

(source BS 6399 Part 2)

Probability factor

The probability factor S_p may be used to change the return period of the basic wind speed, which corresponds to a 50 year return period. Adjustment factors for different return periods derived for the UK are given in Table 2.2.

Table 2.2 *Values of probability factor, S_p*

Return period (years)	Wind speed ratio (Relative to 50 year return period)
1	0.67
5	0.83
10	0.88
20	0.93
50	1.00
100	1.05
200	1.10

(source BS 6399 Part 2)

Duration factor

The duration factor S_t is used to convert the hourly windspeed to a more appropriate duration. For typical UK reservoir lengths, a duration of 10-20mins is usually considered appropriate, so the hourly windspeed should be multiplied by a factor of 1.05. Other duration factors are given in Table 2.3.

Table 2.3 *Values of duration factor, S_t*

Wind Duration	Duration factor
3 sec	1.51
15 min	1.05
30 min	1.03
1 hour	1.00
3 hour	0.96
6 hour	0.93
12 hour	0.87

(source Herbert et al. 1995)

Over-water speed-up factor

The over-water speed-up factor S_w is used to take account of the reduced friction over water. The general advice is that wind speeds derived from Figure 2.1, and modified by the factors discussed above, should be increased by a factor depending upon the fetch length as shown in Table 2.4.
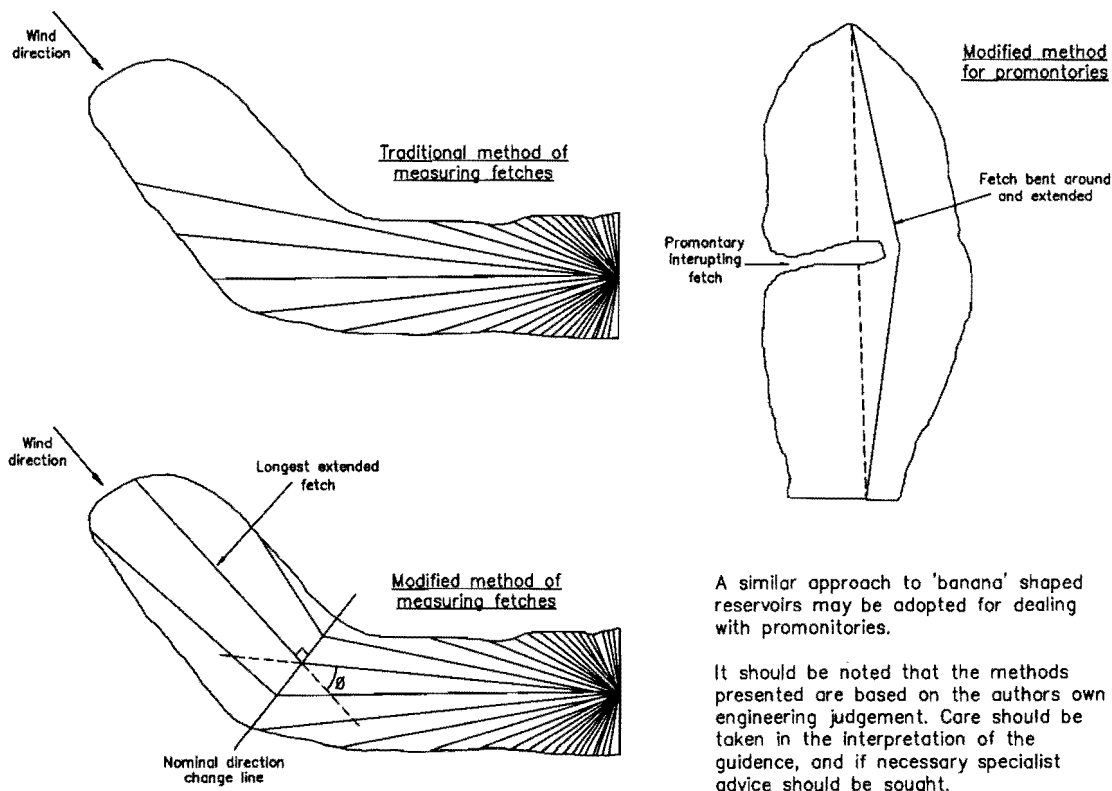
Table 2.4 *Values of over-water speed-up factors, S_w*

Fetch (Km)	Over-water speed-up factor
0.5	1.06
1.0	1.09
2.0	1.16
4.0	1.24
6.0	1.28
8.0	1.30
10.0	1.31

2.3.2   Wave prediction formulae

Fetch derivation

Fetch lengths are usually drawn as straight lines from the centre of the point of interest, here from the dam face to the opposite bank. In certain instances small promontories may interrupt the path of the fetch causing an apparently significant reduction in length. In practice waves may refract and diffract around these promontories causing larger wave conditions to reach the dam face. Guidance is given on the effect of promontories in Figure 2.2. It should be noted that this advice is based on the authors' own engineering judgement.

A similar approach to 'banana' shaped reservoirs may be adopted for dealing with promonitories.

It should be noted that the methods presented are based on the authors own engineering judgement. Care should be taken in the interpretation of the guidence, and if necessary specialist advice should be sought.

If Ø<50° then acceptable to extend fetch, in steep sided valley's as strong winds can be steered by surrounding topography.

Figure 2.2   *Modified method for measuring fetches*

There is anecdotal evidence that wave heights tend to be under-predicted on 'banana' shaped reservoirs where wind blowing over the upper reaches of the reservoir changes direction as the axis of the reservoir changes. It is suggested that the fetch length be bent to follow the axis of the reservoir, thus increasing the overall fetch length. An example of this procedure is outlined in Figure 2.2, which provides guidance as to when the method should or should not be applied. It should be noted that the method of calculating wave heights on 'banana' shaped reservoirs is based on the authors' own engineering judgement and cannot, at present, be supported by scientific data.

Wave height prediction

As previously discussed it is recommended that wave conditions should be determined using the Donelan/JONSWAP method. Research by Owen & Steele (1988) suggests that any errors in this method will generally lead to over-prediction of wave heights thus ensuring a conservative solution.

Calculations have been carried out during this project to compare the full Donelan/JONSWAP approach to the simplified (Donelan/JONSWAP) approach using equations (2.6)-(2.8) shown below. The results showed that for the cases tested there was no significant difference in the results obtained from the two methods. It is therefore recommended that the simplified approach is used.

On reservoirs with several fingers, it may be appropriate to test a number of directions with their appropriate fetch length and factored wind speed. If it is assumed that the direction factor is 1.0 for all directions, then only the greatest fetch direction needs to be calculated.

In its simplified form, the wave prediction method is given by:

$$H_s = 0.00178 U \sqrt{F} / \sqrt{g} \tag{2.6}$$

where H_s is the significant wave height in metres
   U is the design wind speed in metres per second
   F is the fetch length in metres
   g is the acceleration due to gravity (9.81ms^{-2})

and the wave period can be derived using:

$$T_p = 0.07118 F^{0.3} U^{0.4} \tag{2.7}$$

the relationship between T_p and T_m is given as:

$$T_m = 0.82 T_p \tag{2.8}$$

The relationship between wave height, wind speed and fetch length is shown in graphical format in Figure 2.3 for easy reference.

The height, period and direction of the waves generated will depend on the wind speed, duration and direction and the fetch. In most situations, one of either the duration or fetch become relatively unimportant. For example in an inland reservoir even a short storm can produce large wave heights. Because of the small fetch lengths, any increase in the duration of the wind will then cause no extra growth. Thus such waves are described as "fetch limited".

For convenience, wave heights on reservoirs with fetches less than 1km, are given in Table 2.5. These wave heights have been calculated for windspeeds of U = 20, 23 and 25 m/s.
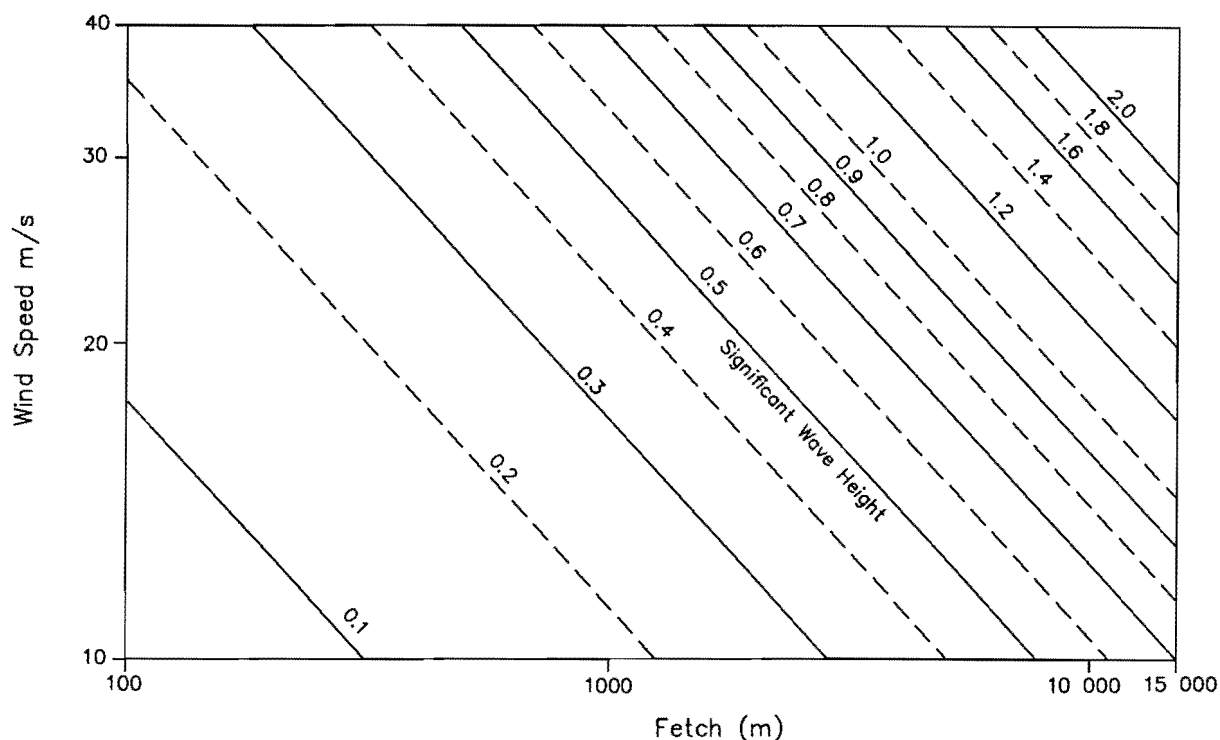
Figure 2.3   *Relationship between wind speed and fetch length*

Table 2.5 *Example wave heights*

Fetch length (m)	U=20m/s	U=23 m/s	U=25m/s
100	0.11	0.13	0.15
300	0.20	0.23	0.25
500	0.25	0.29	0.32
1000	0.36	0.41	0.45

3   Wave overtopping

Large waves at times of high water levels may lead to overtopping of the embankment crest, with potential consequences to access and/or to the stability of the downstream face. This chapter is concerned with wave induced overtopping only, not with overflowing arising when the static water level exceeds the level of the crest. The terminology and notation are generally consistent with coastal engineering practice, see Simm (1991).

3.1 Previous work

The method discussed in section 2.3.2 will predict a significant wave height and wave period for extreme conditions on the reservoir. The design crest level can then be determined using one of

several different methods. Two such methods, namely a wave run-up method, and an overtopping method are discussed in the next two sections.

This section reviews work carried out in the areas of overtopping and run-up. Comparisons are made between the two methods. Recommendations are made as to the most suitable method for determining a crest level.

3.1.1   The run-up method

The run-up method is used to calculate the upward excursion of water on a sloping face. Run-up levels are defined relative to still water level, and vary as do wave heights and lengths in a random sea state. Methods are available to predict the significant run-up level R_{us} or the 2% exceedance level, $R_{u2\%}$ or other exceedance levels. The form of the probability distribution of run-up levels is generally well established. Results of some tests by Allsop et al (1985) suggest that, for simple configurations with slopes between 1:1.33 and 1:2.5, a Rayleigh distribution for run-up levels may be assumed. This distribution may be used to relate the 2% run-up and significant run-up levels giving:

$$R_{u2\%} = 1.4R_{us} \tag{3.1}$$

In order to determine a design crest level using the run-up method factors have to be applied to the significant wave height depending upon the embankment construction. Current practice for checking the crest levels on British reservoirs against potential wave run-up can be found in the ICE guide on Floods, and reservoir safety, (1996).

This method is based on modifying the significant wave height to give a wave surcharge allowance. The extent to which waves can be permitted to go over the crest of a dam depends on the type of the dam. A concrete dam for example is more resistant to erosion than an earth-fill dam. The allowable height, referred to as the design wave height H_D, can be more or less than the significant wave height H_s. The roughness and permeability of the armouring on the upstream slope of the dam also influences the height to which a wave will run up. Factors that can be applied to H_s in order to estimate H_D are given in Table 3.1.

Table 3.1 *Recommended design wave heights*

Dam type	Crest configuration	Design wave height (H_D) metres
Concrete/masonry	-	$0.75H_s$
Rockfill	Surfaced road	$1.0H_s$
Earthfill with reinforced downstream face	Surfaced road	$1.1H_s$
Earthfill with grass downstream face	Surfaced road	$1.2H_s$
Earthfill with grass downstream face	Grass crest	$1.3H_s$
All dam types - no stillwater or wave surcharge carryover permitted		$1.67H_s$

(source ICE (1996))

Once the design wave height has been estimated, Figure 3.1 can be used to determine the run-up factor R_f. Three different dam face surfaces are shown and interpolation can be used between them. The wave surcharge is then simply calculated using the formula below:

$$\text{Wave surcharge} = R_f H_D \tag{3.2}$$

In the ICE guide minimum wave surcharges are given which should be compared to the calculated wave surcharge, and the larger of the two values adopted. The dam freeboard should then be determined, and should not be less than the wave surcharged derived above.

Figure 3.1 *Run-up factor and dam slope*
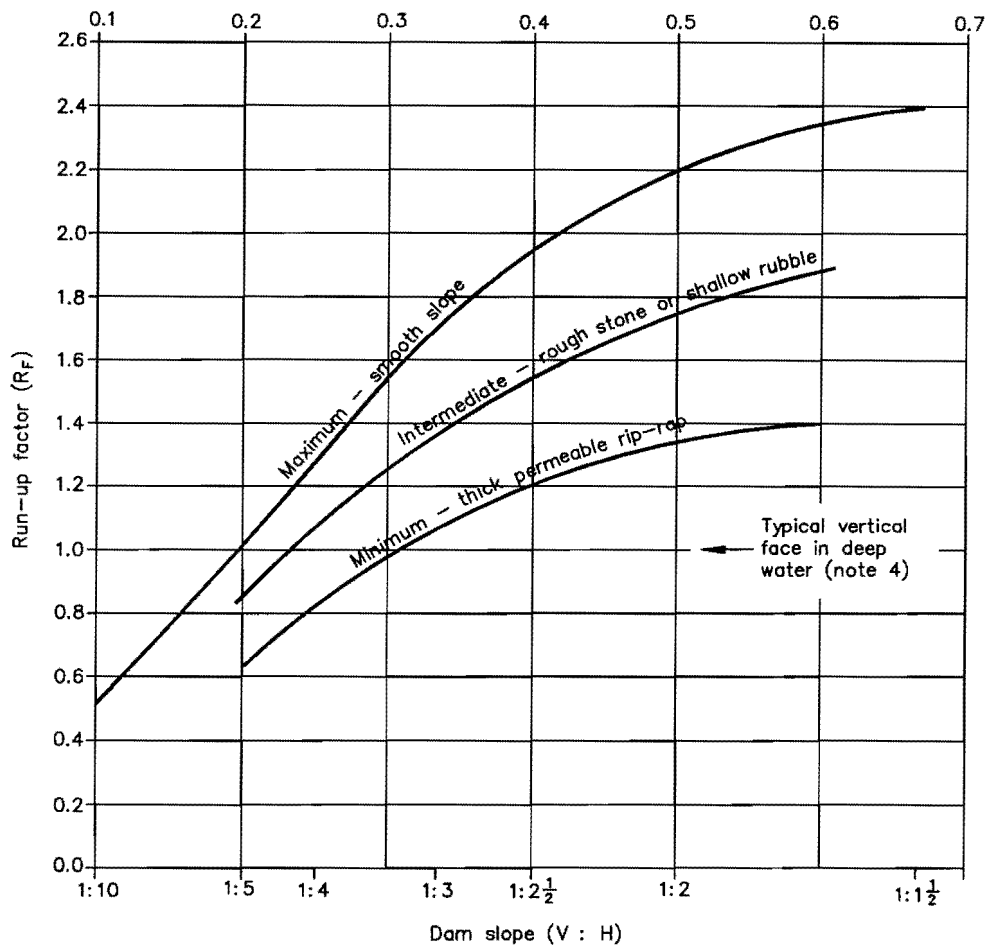
Notes:

1. Maximum line from Saville et al. p.115 for typical wave steepness (height / length) = 0.05

2. Intermediate line is 0.8 x maximum from Technical Advisory Commission on Protection Against Inundation p.69

3. Intermediate line is 0.6 x maximum from Technical Advisory Commission on Protection Against Inundation p.69 and 133

4. For faces off—vertical the run—up ratio rises above unity and can approach 2 in some circumstances where the deep water condition is not fulfilled

3.1.2 Owen overtopping method

During 1978 and 1979 an extensive series of model tests was carried out at the former Hydraulics Research Station, now HR Wallingford, to determine mean wave overtopping discharges for a range of seawall designs subjected to different wave climates.

These model tests enabled empirical methods for evaluating overtopping discharges to be developed by Owen (1980) based on dimensionless parameters for freeboard, crest height and discharge.

The design wave height calculated using the methods presented in Chapter 2 assumes that no wave breaking occurs. In a few instances where the embankment is in relatively shallow water depths, it may not be possible for that wave height to reach the embankment before breaking. In this case the wave height and thus, wave overtopping at the dam will be reduced. A method is described whereby the calculated wave height H_s is replaced by an equivalent post-breaking wave height H_{sb}. Where the underwater ground slope is shallow (less than 1:50), the breaking wave height can be determined using the following equation:

$$H_{sb} = 0.55d \qquad (3.3)$$

where   d is the water depth at the toe of the structure.

if $H_{sb} < H_s$ then H_{sb} should be used in overtopping discharge calculations, otherwise use H_s.

Where approach slopes are steeper, a less simplistic approach will be needed because the breaking wave can be rather larger. Methods used by Owen (1980) or Simm (1991) will be more appropriate.

Once the wave height at the toe of the structure has been determined the wave overtopping discharge can be calculated using the method presented in this section.

The dimensionless freeboard R^*, is determined from;

$$R^* = R_c/(T_m\sqrt{(gH_s)}) \qquad (3.4)$$

where R_c is the crest elevation above still water level
T_m is the mean wave period
g is the acceleration due to gravity
H_s is the significant wave height at the toe of the structure
(allowing for wave breaking)

The dimensionless discharge Q^* is determined from;

$$Q^* = A \exp(-BR^*/r) \qquad (3.5)$$

where   r is a roughness factor
A and B are dimensionless coefficients whose values depend on the embankment geometry

Values of A and B derived by Owen (1980) for simple slopes are given in Table 3.2. To determine the values of intermediate slopes Figure 3.2 should be used, which show the values of A and B in Table 3.2 plotted and joined with smooth curves.

Table 3.2 *Values of A and B for simple slopes*

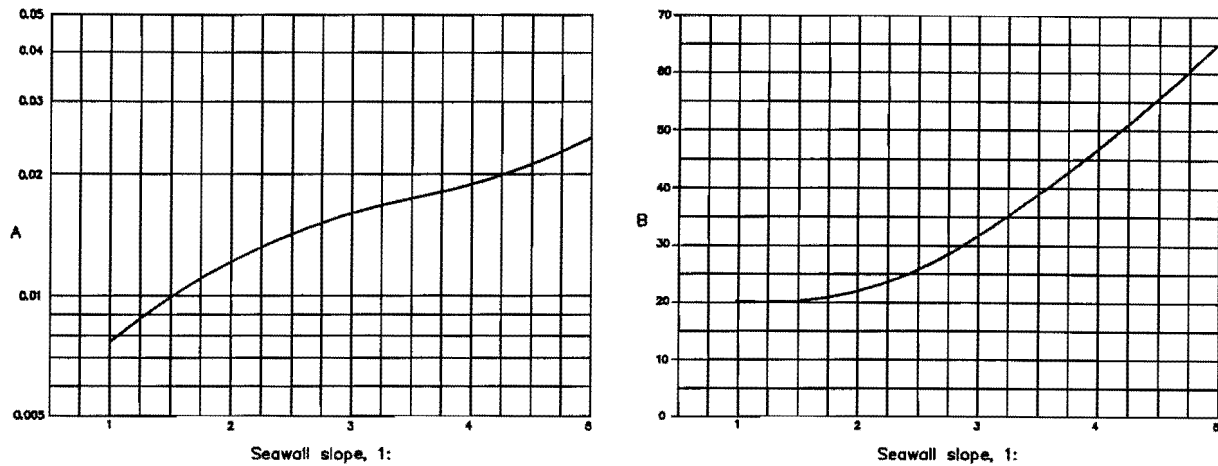| Dam slope	A	B
1:1.5	1.02×10^{-2}	20.12
1:2	1.25×10^{-2}	22.06
1:1.5	1.45×10^{-2}	26.1
1:3	1.63×10^{-2}	31.9
1:3.5	1.78×10^{-2}	38.9
1:4	1.92×10^{-2}	46.96
1:4.5	2.15×10^{-2}	55.7
1:5	2.50×10^{-2}	65.2

(source Owen (1980))

Figure 3.2 Values of A and B for simple slopes

Table 3.3 Values of roughness factor for types of wave protection

Dam construction	Rougness 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 layour of stone rubble on impermeable base	0.8
Stones set in cement, ragstone etc	0.75 to 0.8
Two or more layers or rubble	0.5 to 0.6

(source Owen (1980))

To obtain the mean overtopping discharge due to waves, the expression is:

$$Q^* = q/(T_m g H_s)$$ (3.6)

where   q is the overtopping discharge in m³/s per m run of crest

Allowance can also be made if the crest has a wave return wall built on top. This method involves multiplying the calculated discharge over the dam crest by a factor dependant on the recurve wall height and freeboard. In 1991 HR Wallingford carried out a further series of physical model tests to measure the overtopping discharges of a range of recurved wave return walls, for different wall slopes, water levels and wave conditions, Owen & Steele (1991).

The overtopping discharge passing over the wave return wall crest is calculated from the discharge that would be predicted at the base of the return wall, a dimensionless wall height W*, and adjusted dimensionless freeboard X*.

$$X^* = R^* A_f$$ (3.7)

where A_f is the adjustment factor, and R* is determined as in equation (3.2)

The adjustment factors for various embankment slopes and crest widths are shown in Table 3.4.

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Table 3.4 *Values of adjustment factor, A_f*

$W_h/R_c \geq 0.66$

1:2 slope	0m crest	1.00
1:2 slope	4m crest	1.07
1:2 slope	8m crest	1.10
1:4 slope	0m crest	1.27
1:4 slope	4m crest	1.22
1:4 slope	8m crest	1.33

$W_h/R_c \leq 0.5$

1:2 slope	0m crest	1.00
1:2 slope	4m crest	1.34
1:2 slope	8m crest	1.38
1:4 slope	0m crest	1.27
1:4 slope	4m crest	1.53
1:4 slope	8m crest	1.67

For the wave return wall, the dimensionless wall height is

$$W^* = W_h/R_c \qquad (3.8)$$

where W_h is the wave return wall height
and R_c is the distance between still water level and the base of the wave return wall.

The adjusted dimensionless freeboard X* and the dimensionless wall height W_h, are then used in conjunction with Figure 3.3 to determine the discharge factor D_f.

The mean overtopping discharge, q_w, over the wave return wall is then determined;

$$q_w = D_f \, q \qquad (3.9)$$

where q is the predicted mean discharge at the base of the return wall.
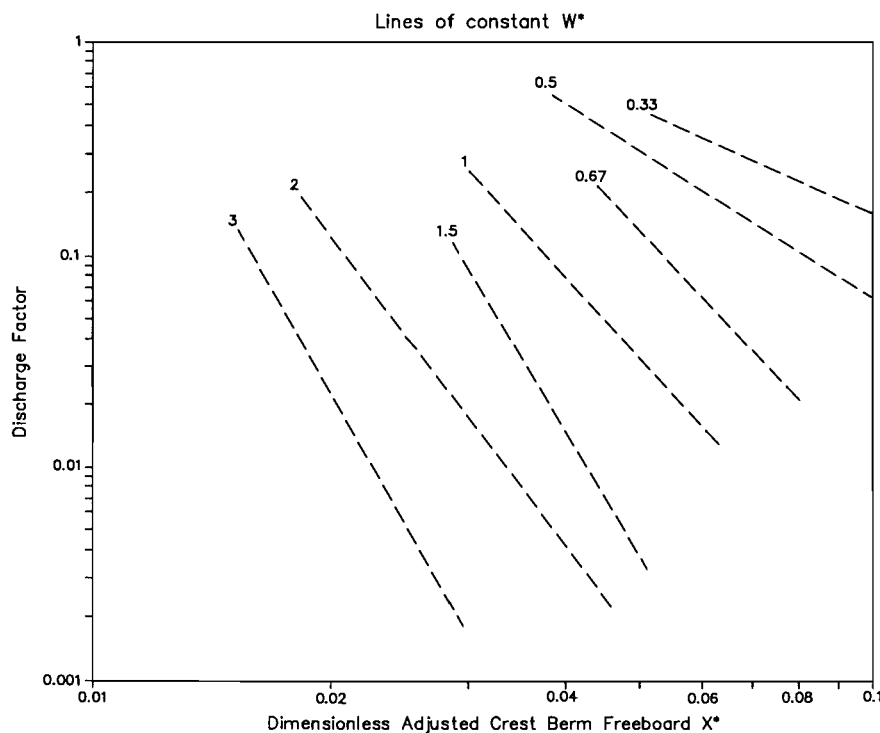
Lines of constant W*

Figure 3.3 Discharge factor for re-curve walls

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3.2 Recommended method

The run-up method recommended by ICE (1996) in Floods and Reservoir Safety Guide, should be used as a simple check of the adequacy of the crest level. It does not however give any indication as to the volume of water which will overtop the dam under wave action.

The Owen overtopping method allows the engineer to check the design crest level based on safe wave-induced overtopping passing over the embankment crest. Assessment is made depending upon the type of embankment construction and the use of the crest, for example vehicle access or pedestrian access. Suggested safe overtopping discharges are shown in Figure 3.4 taken from Franco et al (1994).
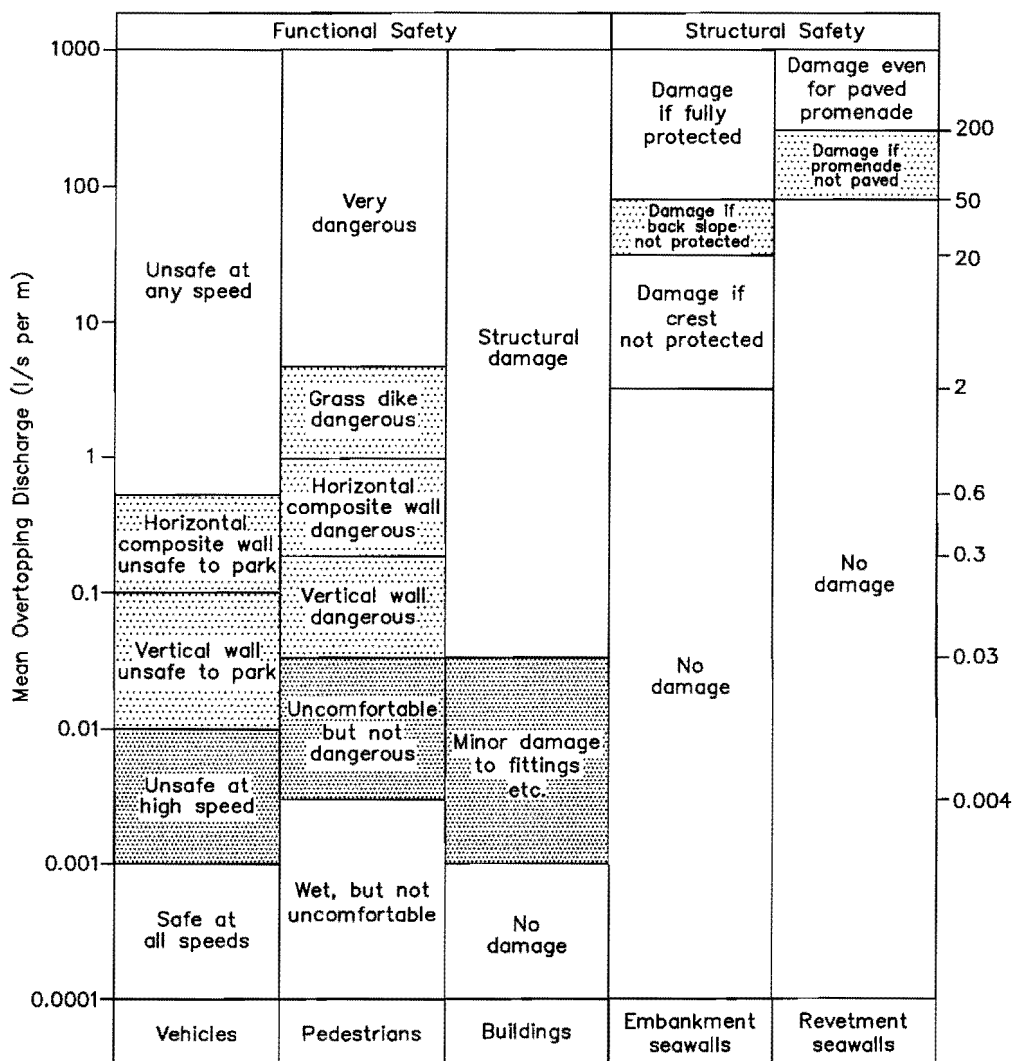

Figure 3.4   *Suggested safe overtopping discharges*

The Owen overtopping method has been incorporated into a software package developed by HR Wallingford, called SWALLOW, which enables the engineer to calculate the overtopping discharge for simple and bermed embankment geometries. It can also be used to determine the minimum crest level required to meet an acceptable overtopping discharge.

3.2.1   Comparison of overtopping and run-up method

Calculations were carried out using both the run-up method and the Owen overtopping formulae. Both methods can be used to predict crest levels. The calculations were carried out for three dam slopes and two roughness factors.

The Owen formulae can be used to set the crest level depending upon the acceptable overtopping discharge required. Three overtopping discharges were used in the calculations and corresponding crest levels derived.

The values obtained were compared assuming the dam has a grass crest. The CIRIA Rock Manual edited by Simm (1991) suggests that the safe overtopping limit for this would be 2 l/s per m. The results of the calculations shown in Table 3.5 suggest that the crest levels derived using the Owen method are significantly lower than those predicted by the run-up method.

Table 3.5 *Comparison of crest freeboards*

Slope	Crest freeboard (m)	
	Run-up method	Owen's method
1:2 smooth	2.76	1.83
1:3 smooth	2.16	1.31
1:4 smooth	1.63	0.93
1:1.5 rock	2.39	1.53
1:2 rock	2.20	1.47
1:3 rock	1.69	1.05

The results above show that for a grass crested embankment the run-up method produces the largest values for dam freeboard. This implies that the run-up method is probably conservative.

The overtopping method outlined above provides a logical, and better supported approach to the derivation of crest levels against wave overtopping. The method is easy to use, consistent with other practice, and more easily understood.

4 Stability of Blockwork and Slabbing

The upstream face on an embankment dam needs protection against wave action to prevent erosion of the underlying fill which could otherwise lead to potential breaching of the embankment. Stone or rough-dressed masonry pitching or blockwork have been traditional means of protection. Since 1945, other measures such as pre-cast concrete blocks and cast in-situ concrete slabbing have been frequently adopted, as has rock armour or rip-rap. Comprehensive details of rock protection can be found in the Rock Manual edited by Simm (1991), and details on the performance of blockwork and slabbing protection in HR Wallingford report SR 345, Herbert et al (1995).

Much existing slope protection has been designed using formulae based on previous experience and hydraulic model tests. Recent model tests have provided greater confidence in the design and assessment of blockwork revetments. Larger slabbing units provide cost savings for protection of new dams, due to improvements in pre-cast and cast in-situ concrete production. Examples of both types of construction are shown in Figure 4.1 Further economies may be achieved by optimising the dimensions and permeability of the slabbing revetments. This chapter briefly reviews existing design methods, outlines new research conducted on both blockwork and slabbing, and suggests a universal method for the design of both types of slope protection under wave action.

4.1 Previous work

A comprehensive review of existing design methods for blockwork and slabbing stability is given by Herbert et al (1995). For convenience a summary of the most relevant recent developments is included here.

Recent design formulae have generally concentrated on predicting armour thickness needed to resist blocks being extracted from the slope by uplift pressures. Pilarczyk (1984) identified eight loading processes that occur as waves reach the structure. The most likely cause of failure is considered to
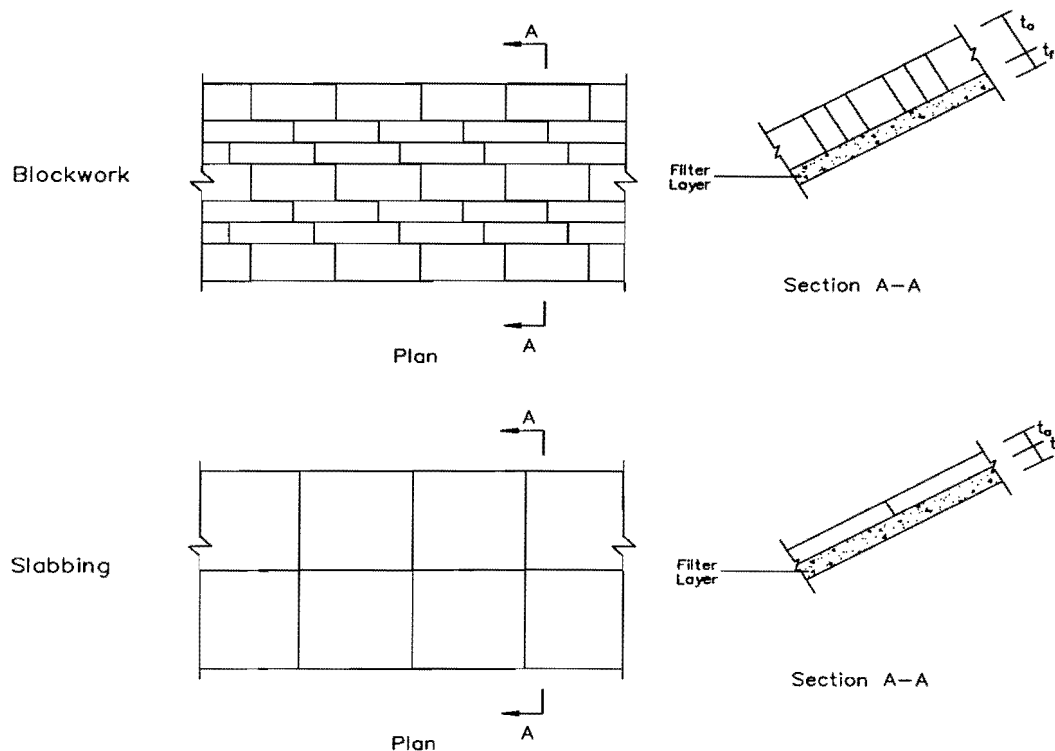
Figure 4.1   *Typical blockwork and slabbing details*

occur during return flow, after wave run-up when water may penetrate between the blocks into the filter layers. Since wave run-up levels are generally greater than the wave draw-down, relative to the still water level, flow into the filter layer can take place over a larger area than flow out of the filter layer. The residual water produces an increase in the phreatic level within the filter layer and consequently an increase in the uplift pressure on the blockwork. This effect is dependent upon the relative permeabilities of the blockwork and filter layers and upon the slope of the face. It is also cumulative for a number of waves.

The method recommended by Herbert et al (1995) was developed by Klein Breteler & Bezuijen (1991) for blockwork revetments. This relates the wave conditions in terms of significant wave height and peak period to the physical properties of the revetment in terms of slope angle, blockwork relative density, and blockwork depth. The equation used may be written;

$$H_s/(\Delta t_a) = S_b \xi_p^x \qquad \text{where } x = -0.67 \qquad (4.1)$$

where H_s is the significant wave height
   Δ is the relative density of the blockwork $(\rho_c/\rho_w -1)$
   ρ_c is the density of the block or slab material
   ρ_w is the density of water
   t_a is the blockwork depth
   ξ_p is the peak Iribarren or surf similarity number
    $(\tan \alpha) / (2\pi H_s / gT_p^2)^{0.5}$
   α is the slope angle
   T_p is the peak wave period
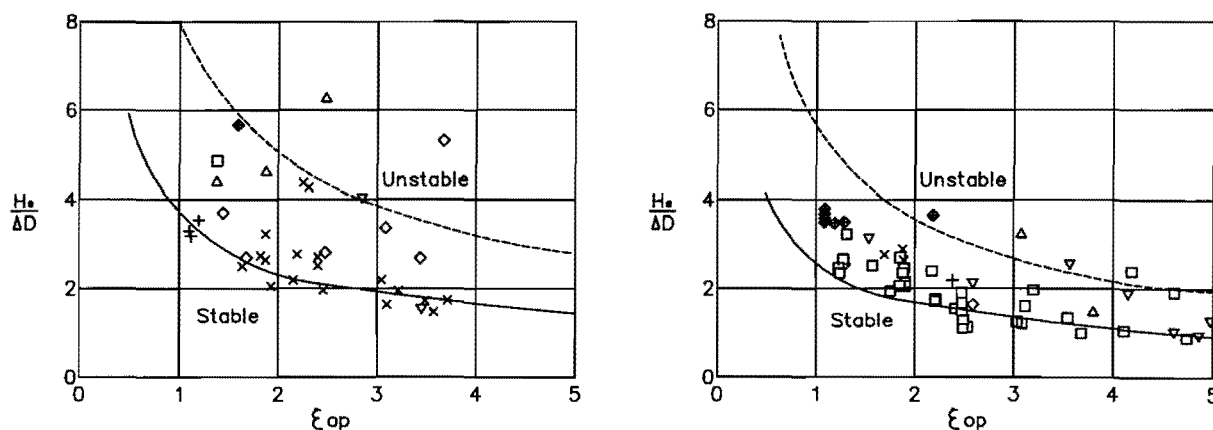 S_b is an empirically derived value

Figure 4.2   Stability curves from Breteler (1991)

The peak Iribarren number is the ratio of embankment slope gradient to peak wave steepness. A high Iribarren number indicates a steep slope or a long wave length. The model testing showed that blockwork was less stable under high Iribarren numbers for a given wave height. This is shown in Figure 4.2, which may be used directly for design purposes, or Equation (4.1) with a suitable value of S_b, as shown in Table 4.1.

A range of values for S_b were found through physical model testing by Breteler & Bezuijen (1991) with regular waves. For loose blockwork on a granular filter layer two ranges are given:

Table 4.1 Range of S_b values

	$S_{b\ min}$	$S_{b\ max}$
low stability (L)	2.6	5.6
normal stability (N)	3.7	8.0

(source Breteler & Bezuijen, (1991))

A slope has "low stability" when the cover layer is thin compared to the filter layer, the cover layer is of low permeability and the filter is of high permeability. Thus Breteler defines "low stability" to be when:

$$D/t_f \leq 0.5, \text{ and } D_{f15} > 0.4 \text{ and } \Omega < 2\% \qquad (4.2)$$

where t_f is the thickness of the filter layer
    D_{f15} is the 15% particle size for the filter layer
    Ω is the ratio of gap area to surface area of the units

For the design of slabbing Herbert et al (1995) suggest the use of equation (4.1) with $S_b = 4.3$. This value was found from a review of embankments protected by slabbing. No slab failure have been recorded after storm events where the calculated value of S_b was less than 4.3.

These design methods pose a number of problems when used in practice. Further guidance for choosing an appropriate value of S_b within the ranges for blockwork embankments is not given by Klein Breteler & Bezuijen (1991) or by Herbert et al (1995) The user is likely, therefore, to use the lowest value of S_b to ensure a conservative design (see Figure 4.2). In most cases this will probably result in an over conservative design.

The criteria for selecting the lowest range of values of S_b is very strict and in most cases results in selection of the "normal" range. In certain cases where one or two (but not all) of the criteria are greatly exceeded, this may result in an over-estimation of the revetment stability.

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The value of S_b = 4.3 for slabbing provides a first approximation of stability, but takes no account of different geometries and permeabilities of revetments. It is not supported by model test data.

4.2 Recommended method

Due to the lack of test data for evaluating slabbing stability, a physical model study was conducted at HR Wallingford. Testing under a range of wave conditions was conducted on various slabbing and blockwork revetment configurations. Full details of the testing method and analysis of results are given by Banyard in Appendix 1.

The new method uses Breteler's general expression for blockwork stability reproduced here for convenience:

$$H_s/(\Delta t_a) = S_b \xi_p^x \tag{4.1}$$

Initially the model test results were compared with the previous work. Values of S_b were found using equation (4.1) with x=-0.67. These were compared to the existing ranges of S_b given in Table 4.1 . The new results were found to be sufficiently consistent with previous work to allow an extension of this method to investigate the stability of slabbing.

Breteler's work was conducted on the stability of revetments under a wide range of Iribarren number, ξ_p. The design method provided may be used for blockwork on dams or on the coast. When considering only dam slope protection the Iribarren number is limited by two physical constraints. The short fetch lengths on reservoirs limit wave development and so only wind waves (wave steepness, s_p greater than approximately 0.03) can be generated. The slope gradient at which blockwork and slabbing is laid is unlikely to exceed 1:1.5 due to the difficulty of placing units at a steeper angle. These two factors result in an Iribarren number limited to $\xi_p < 4$.

The results for all the blockwork and slabbing revetments was re-analysed using only the test data where $\xi_p < 4$. By varying the values of both x and S_b, the best agreements were achieved with x = -1 and a new coefficient S_c. Thus:

$$H_s / (\Delta t_a) = S_c / \xi_p \tag{4.3}$$

The value of S_c varied between 6 and 13 depending on the revetment configuration. Such a wide range is of limited practical use because the lowest value would always be used to ensure a safe design.

The influence of the geometry and permeability of the cover and filter layers on the stability of the revetment were considered to develop a more exact design method.

As the depth and permeability of the filter layer are increased, the in-flow that penetrates under the cover layer just after wave run-up also increases. The wave run-up level is generally greater than the wave run-down relative to still water level. The increase of in-flow due to increased filter permeability during wave run-up is proportionately greater than the increase in out-flow during wave run-down. Up-lift forces on the cover layer are therefore increased.

Conversely, increased permeability of the cover layer allows proportionately greater out-flow during wave run-down than the increase of in-flow during wave run-up. Wider gaps between the cover units therefore increase the slope stability for this particular mode of failure. Wide gaps may however result in loss of fine material from the filter layer, causing under-cutting of the cover units and failure of the slope by subsidence. The loss of fine material will also increase the permeability of the filter and therefore create greater up-lift forces on the cover layer as discussed above.

Increasing the depth of the cover layer influences the permeability due to the increased flow path from the surface to the filter layer. This was examined during analysis of the test data and found to have no significant effect when compared to the increased weight of the unit. The surface area and relative density of the cover units can also be altered to change the effective weight of the unit.

<div align="center">19</div>

An expression for S_c was found:

$$S_c = 3.3 \ln (A_s^{0.5} / t_f) (w / D_{f15})^{0.1}) + 4.0 \qquad (4.4)$$

where A_s is the area of slabbing unit
       t_f is the depth of filter layer
       w is the gap width between slabbing units
       D_{f15} is the 15% sieve value for filter layer material.

This expression may be easily incorporated into spreadsheet calculations, or the design chart in Figure 4.3 may be used. Any five of the six design parameters may be selected and the value of the sixth is found.

Since the expression is derived empirically, there is an uncertainty implicit in any of the results obtained. The expression will give a predicted value which is not the same as the result obtained from the model testing. To allow for this uncertainty a confidence factor may be used by the engineer. When a value is found from Equation (4.4) or from the design chart it should be multiplied by the factor given in Table 4.2. This will give a 95% certainty that the result obtained gives a safe value for design, based on the results of the model testing. For example, the value of t_a from the design chart (Figure 4.3) should be multiplied by 1.3 to find the recommended slab depth.

Table 4.2 *Values of 95% confidence factors*

Parameter	notation	95% confidence factor
slab depth	t_a	1.3
revetment slope	$\tan \alpha$	0.77
slab density	ρ_c	1.3
filter depth	t_f	0.63
slab area$^{0.5}$	$A_s^{0.5}$	1.6
gap width	w	4.1
filter grading	D_{f15}	0.24

No extrapolation beyond the curves should be made. The design method provided is only valid for failure due to extractions of units as a result of high piezometric head difference between the filter layer and the surface of the revetment. Any revetment configuration which lies beyond the limits of the design curves may be liable to some other mode of failure.

Interpolation between each of the four curves relating permeability of the cover layer to permeability of the filter layer (w / D_{f15}) may be made. The highest value of w/D_{f15} given is 1.5. Beyond this point failure due to loss of fine material from the filter layer becomes likely and when w/D_{f15} reaches 6 this becomes the critical mode of failure.

Care should also be taken not to choose individual parameters outside the ranges given in Table 4.3, since scale effects may influence the accuracy of the results:
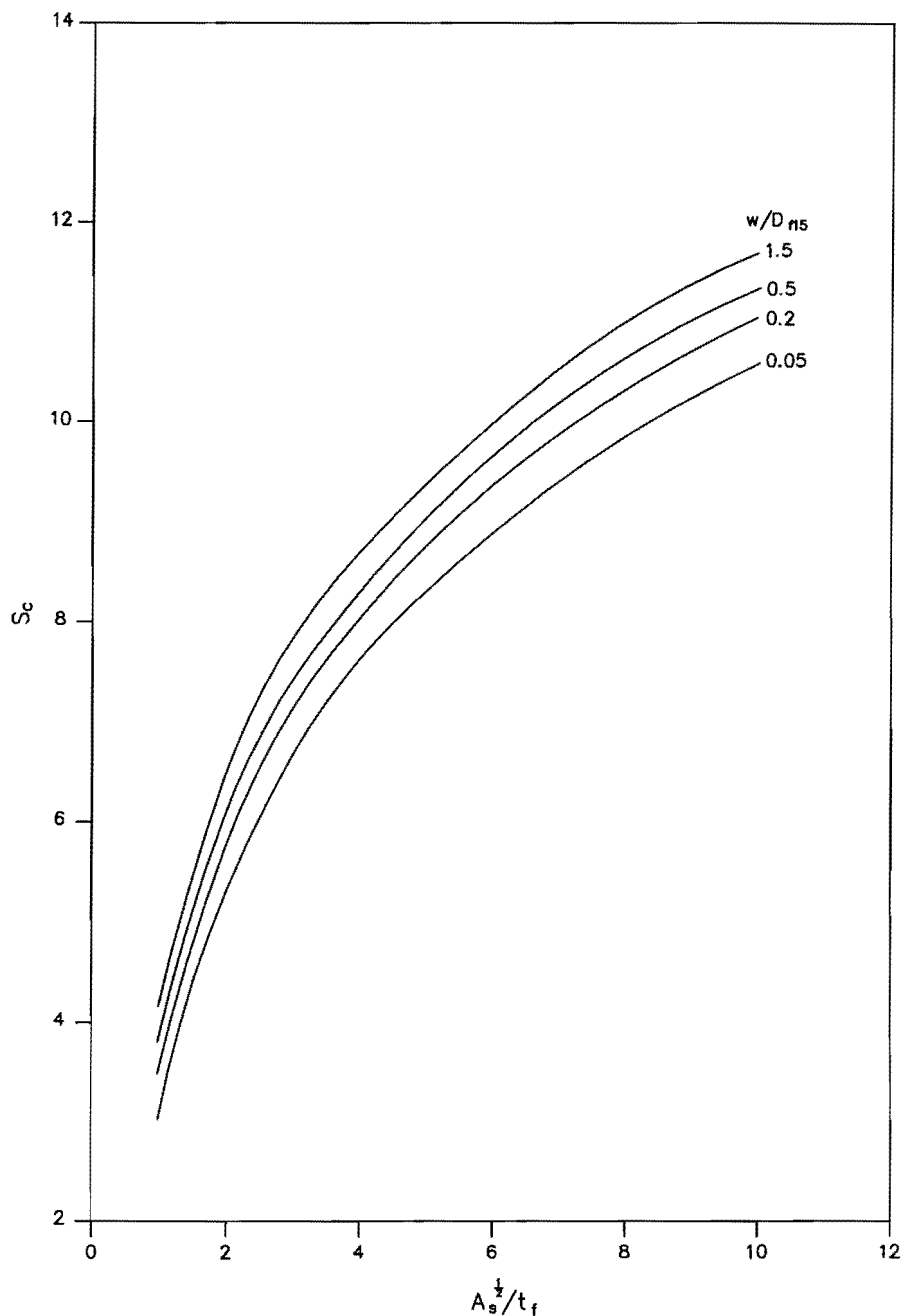
Figure 4.3 *Design chart for slabbing*

Table 4.3 *Range of individual parameters for design of slabbing*

revetment geometry		min	max
slab area (m^2)	A_s	0.07	9.0
slab depth (m)	t_s	0.01	0.4
gap width (mm)	w	0.2	60
filter D_{f15} (mm)	D_{f15}	0.5	100
filter depth (m)	t_f	0.03	0.8
wave conditions			
significant wave height (m)	H_s	0.1	4
peak wave period (s)	T_p	1.0	10.0
surf similarity parameter (-)	ξ_p	1.7	4

It may be possible to work outside these ranges but there is no model data to confirm this.

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It is unlikely that these limits would be exceeded in normal design practice. The upper limits of slabs area and slab thickness should, however, be noted carefully.

The method is not valid for Iribarren numbers ξ_p greater than 4. This would only occur when designing revetments as part of sea defences which are subjected to long period swell waves, rather than reservoir embankments where only short period wind waves are encountered.

5 Summary of design methods

The recommended design methods are summarised in the following examples which are provided to aid the user in the application of the methods.

The first stage in the design process is to derive the design wind speed, in this instance for a 10 year return period, a relatively frequent event.

Box 5.1   Derivation of design wind speed

Given	basic wind speed (from Figure 2.1)	V_b	= 24m/s
	altitude of reservoir	Δ_s	= 200m
	fetch length	F	= 6.4km
	fetch direction		= 240°
	return period		= 10 years

Calculate design wind speed

1.   calculate altitude factor

$$S_a = 1+0.001\Delta_s$$
$$S_a = 1+0.001*200$$
$$S_a = 1.2$$

2.   determine direction factor from Table 2.1

$$S_d = 1.0$$

3.   determine probability factor from Table 2.2 for 10 year extreme

$$S_p = 0.88$$

4.   determine duration factor from Table 2.3.

$$S_f = 1.05$$

5.   Using fetch length of 6.4km and Table 2.4 to find S_w

$$S_w = 1.28$$

6.   calculate design wind speed using equation (2.4)

$$U = V_b S_a S_d S_p S_f S_w$$
$$U = 24 * 1.2 * 1.0 * 0.88 * 1.05 * 1.28$$
$$U = 34.1m/s$$

Then the wave conditions are estimated using the Donelan/JONSWAP method.

Box 5.2   Calculate significant wave height H_s

1.	calculate H_s from equation (2.6)	$H_s = 0.00178u\sqrt{F}/\sqrt{g}$
		$H_s = 0.00178 * 34.1 * 6400^{0.5} * 9.81^{-0.5}$
		$H_s = 1.55\text{m}$
2.	calculate T_p from equation (2.7)	$T_p = 0.07118F^{0.3}U^{0.4}$
		$T_p = 0.07118 * 6400^{0.3} * 34.1^{0.4}$
		$T_p = 4.05\text{s}$
3.	calculate T_m from equation (2.8)	$T_m = 0.82T_p$
		$T_m = 0.82 * 4.05$
		$T_m = 3.32\text{s}$

If wave overtopping is the primary influence on the embankment crest level, Owen's wave overtopping method can then be used to calculate a design freeboard. In this example, a limit on overtopping discharge has been set to avoid damage to grass on the crest and downstream face of the embankment.

At this stage it is important to check that the calculated wave heighty can reach the embankment before wave breaking occurs. If the toe of the embankment is assumed to be in "deep" water, no allowance has to be made. If, however, the embankment is in relatively shallow water equation (3.3) should be used.

Box 5.3   Calculate crest level to limit the wave overtopping

Given	significant wave height at structure	$H_s = 1.55\text{m}$
	mean wave period	$T_m = 3.32\text{s}$
	allowable overtopping discharge	$q = 0.002\text{m}^3/\text{s per m}$
	roughness factor	$r = 1.0$

Calculate design crest level for a 1:3 sloping embankment dam

1.	using equation (3.6)	$Q^* = q/(T_m g H_s)$
		$Q^* = 0.002/(3.32 * 9.81 * 1.55)$
		$Q^* = 3.9618\text{E}^{-5}$
2.	substitute Q^* into equation (3.5)	$Q^* = A \exp(-BR^*/r)$
		$3.9618\text{E}^{-5} = 1.632\text{E}^{-2}\exp^{(-31.9R^*/1.0)}$
		$0.002431 = \exp^{(-31.9R^*)}$
		$-6.021 = -31.9R^*$
		$R^* = 0.18875$
3.	substitue R^* into equation (3.4)	$R^* = R_c/(T_m\sqrt{gH_s})$
		$0.18875 = R_c/(3.32\sqrt{9.81 * 1.55})$
		$R_c = 2.44\text{m}$
4.	The crest level should be 2.44m above top water level to satisfy allowable overtopping discharge by waves.	

In the last stage of this example, the thickness of slabbing is calculated from the predicted wave height and assumed values of other parameters.

Box 5.4   *Design example of slab stability*

Given	significant wave height in deep water	H_s = 1.5m
	peak wave period	T_p = 5.5s

Design slabbing protection for a 1:3 sloping embankment dam

1.   calculate the peak wave steepness

$$s_p = (2 \pi H_s) / (g T_p^2)$$
$$s_p = (2 \pi {}^* 1.5) / (9.81 {}^* 5.5^2)$$
$$s_p = 0.032$$

2.   calculate the surf similarity parameter

$$\xi_p = \tan \alpha / s_p^{0.5}$$
$$\xi_p = 0.333 / 0.032^{0.5}$$
$$\xi_p = 1.87$$

ξ_p is less than 4.0 and so the design method may be applied

3.   Density of concrete for the cover layer ρ_c = 2.35 kg/m^3
and water density ρ_w = 1.00 kg/m^3

4.   Choose slab area and maximum width of gaps between slabs
(With reference to Table 4.3)   3m x 3m slabs with 1mm gaps

5.   Choose filter grading and filter depth D_{f15} = 4mm
(With reference to Table 4.3) t_f = 300mm

6.   Find $(A_s)^{0.5} / t_f$, where A_s is the slab area $(A_s)^{0.5} / t_f$ = 3 / 0.3 = 10
Find w / D_{f15}, where w is the gap width w / D_{f15} = 1 / 4 = 0.25

7.   From design chart (Figure 4.3) find S_c S_c = 11

8.   Calculate depth of slab

$$H_s / (\Delta t_a) = S_c / \xi_p$$
$$t_a = (H_s \xi_p) / (\Delta S_c)$$
$$t_a = (1.5 {}^* 1.87) / (1.35 {}^* 11)$$
$$t_a = 0.19m$$

9.   Apply the probability factor from Table 4.2 t_a = 1.3 * 0.19 = 0.25m

10.   We can be 95% certain that a slab of 250mm depth will be stable under the design conditions

6   *Conclusions and recommendations*

This study has supported and extended guidance in SR345, Herbert et al (1995) and the Floods and Reservoir Safety Guide 3rd Edition ICE, (1996). The study has drawn together areas of best practice, and has derived new design guidance for slabbing protection based on physical model testing carried out as part of the study.

The design method recommended in this study was developed specifically for locally generated wave conditions which occur on inland reservoirs. For coastal and estuarine sites where swell waves may also be significant, the method should be extended to identify the stability of slabbing protection under shallow wave steepnesses.

It has however become clear that there still remain uncertainties in the derivation of a design wind speed, and in the response of different types of embankment protection to wave action.

In order to clarify these uncertainties it is proposed that future work is undertaken in the following areas:

(a) Determine whether "funnelling" down a narrow, steep-sided valley over water does cause a local increase in wind speed.

(b) Investigate further the use of "speed-up" coefficients to take account of valley slope and lower friction over water. The recommendations in this report are based on sound engineering judgement, but they are not well-supported by good data and it is the authors' belief that further work is needed.

(c) Establish if wave propagation can continue along a gently curved valley.

(d) Investigate further the effect of wave diffraction caused by structures in front of the upstream face such as valve towers.

(e) Investigate whether Dutch practice for river bank/coastal protection (geomembrane beneath interlocking blocks with gravel packing between blocks) can be applied to dam faces.

(f) Development calibration and validation of numerical models to calculate uplift pressures acting on blocks or slabs.

7 Acknowledgments

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Appendix 1

Modelling Studies of slabbing and blockwork stability

L S Banyard

Appendix 1   Modelling studies of slabbing and blockwork stability

A.1.1 Model designs

Two sets of physical model tests were undertaken at HR Wallingford to investigate the stability of blockwork and slabbing protection. Tests with regular waves were conducted in a flume 27m long by 0.62m wide by 0.91m deep. Random wave tests were conducted in the wind wave flume which is 50m long by 1.22m wide by 1.1m deep and has a computer controlled wave paddle capable of generating random wave spectra.

The experiments were designed to investigate the influence of various design parameters on the stability of the cover revetment. These parameters were: the depth, plan area, density and permeability of the cover layer, the thickness and permeability of the filter layer, and the slope angle. The arrangement of the test sections is given in Table A.1.

Table A1   *Arrangement of test sections*

Series no.	Slope angle	slab dimensions (mm)	slab density (kg/m³)	gap width (mm)	filter grading (mm)	filter thickness (mm)
S1	1:2.5	120x60x20	2400	0	0.5 - 4	30
S2	1:2.5	120x60x20	2400	0	4 - 6	30
S3	1:2.5	120x60x20	2400	3	4 - 6	30
S4	1:2.5	200x200x11	2400	0	0.5 - 4	30
S5	1:2.5	200x200x11	2400	0	4 - 6	30
S6	1:2.5	200x200x11	2400	3	4 - 6	30
S7	1:2.5	300x300x10	2400	0	0.5 - 4	30
S8	1:2.5	300x300x10	2400	0	4 - 6	30
S9	1:2.5	300x300x10	2400	3	4 - 6	30
H1	1:3	120x84x20	2400	3.5	4 - 6	80
H2	1:3	130x84x20	2400	3.5	10 - 14	80
H3	1:3	130x84x20	2400	6	4 - 6	80
H4	1:3	130x84x20	2400	6	10 - 14	80
M1	1:3	120x84x20	2400	4	10 - 14	80
M2	1:3	200x200x11	2400	4	10 - 14	80
M3	1:3	200x200x11	2250	4	10 - 14	80

A granular filter layer was placed on an impermeable slope. Blockwork or slabbing were laid on the filter material. The required spacing of the cover layer units was achieved by placing metal spacers of known thickness between adjacent units.

A.1.2 Test procedures

<u>Wave calibration</u>
Wave calibration tests were completed to measure the wave conditions in the physical models. A spending beach was installed in the position of the test structures to ensure that only the incident wave energy was recorded and reflected wave energy was kept to a minimum. Wave conditions close to the position of the toe of the test structures were measured using twin wire resistance wave probes.

During calibration of random waves, the wave probes were connected to a computer which recorded the complete test sequence of 1000 waves and determined the significant wave height and the mean period statistically. For regular waves the output from the wave probes was connected to a chart recorder. A short series of regular waves was created for each wave condition and the wave height and period determined.

To relate the stability of blockwork and slabbing under regular waves with wave height H, to stability under random wave attack with significant wave height H_s, it is generally assumed that a simple multiplication factor may be used. In tests conducted by Klein Breteler & Bezuijen (1991) the piezometric pressure on the slope under regular and random wave attack was compared. This comparison indicated that at the threshold of damage:

$$H / H_s = 1.4 \tag{A1}$$

The damage to a revetment under random wave action is caused by the most severe waves of the particular storm event. The maximum wave height may therefore be directly compared to the wave height of a train of regular waves, all of which are equally likely to cause damage. Comparisons of maximum wave height and significant wave height for random waves have been made by Longuet-Higgins (1952) and Goda (1985). Longuet-Higgins provides a formula which predicts the relationship for a given length of wave train.

$$H_{max} = H_s \, ((\ln N)/2)^{0.5} \tag{A2}$$

where   N is the number of waves in the train

For a typical UK storm of 3 hours with mean period in the range 4-8s

$$1.8 < H / H_s < 2.0 \tag{A3}$$

Goda compares H_{max} and H_s as waves approach breaking for a range of wave spectra and wave steepness. In tests using 200 waves in deep water, where $d/H_s < 20$, Goda suggest that a simplification may be made:

$$H_{max} / H_s \sim 1.8 \tag{A4}$$

For the present study the regular wave height was related to an equivalent significant wave height using

$$H / H_s = 1.8 \tag{A5}$$

which is based on Longuet-Higgins' and Goda's equations (A2) & (A4). Since there is a wide range of suggested values for H / H_s, it was appropriate to use a ratio which produces more conservative values of S_b and will therefore result in a conservative design method.

Testing
The random wave tests were conducted for 1000 waves and the regular wave tests for 30 waves. Movement of the cover layer was observed under wave action and the state of the revetment was noted after each test. Failure of the revetment was defined as displacement of one or more of the slabs or blocks from their original position(s).

A.2 Results

A.2.1 Test Observations
Most of the revetment failures observed were due to extractions between still water level and maximum run-down level. In most cases individual units were lifted as waves ran down the revetment slope. When blockwork was tested with longer period waves, (ie Iribarren number between 7 and 15), movement of several blocks together was observed, with extractions occurring after the units had worked loose.

Another mode of failure was observed in one series of tests. This revetment had been constructed with a fine filter layer (0.5-4mm) and relatively wide gaps between the slabs (w = 3mm). Much of the filter

layer was washed out of the slope through the cover layer and the revetment failed prematurely due to under-cutting and subsidence of the armour. It is not recommended that such an arrangement would be used normally for slope protection without some form of geotextile placed between the cover and the filter layers. The results from this test series were not used in the analysis of revetment stability.

A.2.2   Comparison of results of blockwork tests with previous research

The previous method recommended by Herbert et al (1995) was developed by Klein Breteler & Bezuijen (1991) for blockwork revetments. This relates the wave conditions in terms of significant wave height and peak period to the physical properties of the revetment in terms of slope angle, blockwork relative density, and blockwork depth. The equation used may be written;

$$H_s/(\Delta t_a) = S_b \xi_p^x \qquad \text{where } x = -0.67 \qquad (A.6)$$

where H_s is the significant wave height
         Δ is the relative density of the blockwork $(\rho_c/\rho_w - 1)$
         ρ_c is the density of the block or slab material
         ρ_w is the density of water
         t_a is the blockwork depth
         ξ_p is the peak Iribarren or surf similarity number
               $(\tan \alpha) / (2\pi H_s / gT_p^2)^{0.5}$
         α is the slope angle
         T_p is the peak wave period
         S_b is an empirically derived value

The peak Iribarren number is the ratio of embankment slope gradient to peak wave steepness. A high Iribarren number indicates a steep slope or a long wave length. The model testing showed that blockwork was less stable under high Iribarren numbers for a given wave height. This is shown in Figure A.1 a & b, which may be used directly for design purposes. Alternatively Equation (A.6) may be used with a suitable value of S_b. A range of values for S_b were found by Breteler & Bezuijen (1991) through physical model testing with regular waves. For loose blockwork on a granular filter layer two ranges were found:

Table A.2   Range of S_b values

	$S_{b\ min}$	. $S_{b\ max}$
low stability (L)	2.6	5.6
normal stability (N)	3.7	8.0

(source Breteler & Bezuijen, (1991))

A slope has "low stability" when the cover layer is thin compared to the filter layer, the cover layer is of low permeability and the filter is of high permeability. Thus Breteler defines "low stability" to be when:

$$t_a/t_l \leq 0.5, \text{ and } D_{f15} > 0.4m \text{ and } \Omega < 2\% \qquad (A.7)$$

where t_l is the thickness of the filter layer
         D_{f15} is the 15% particle size for the filter layer
         Ω is the ratio of gap area to surface area of the cover layer units

The new test results will be compared with the existing ranges and the agreement between the two sets of results will be discussed.

When the criteria in equation (A.7) were strictly applied, all the armour configurations tested in this study were categorised as having "normal stability" (Table A.3). Revetment configurations H1, H2, H3,

H4 and M1 greatly exceed two of the three criteria for "low stability", but are classified as "normal stability" because they do not meet the third criteria regarding gap area between adjacent armour units. Although Breteler suggests that all three criteria must be met, to ensure a conservative design under these circumstances the engineer would be advised to select the "low stability" factor.

Graphs of $H_s/(\Delta t_a)$ against ξ_p were plotted for each test condition, and curves were fitted to the failure envelope using equation A.6 with x = -0.67, as used by Klein Breteler & Bezuijen (1991) (Figures A.2 a-h). Values of S_b were found for each revetment configuration tested here (Table A.3). Configurations H1, H2, H3, H4 and M1 gave values of S_b between 2.3 and 4.1 which compares well with the "low stability" range from previous research of 2.6 to 5.6. Configurations S1, S2, and S3 gave S_b between 4 and 4.7 which lie within the appropriate "normal stability" range of 3.7 to 8.

Thus the physical model tests on blockwork slopes are consistent with those found in previous research. The results from all the tests were then used to extend the design method for blockwork to include design of slabbing.

A.2.3 Stability of blockwork and slabbing

Graphs of $H_s/(\Delta t_a)$ against ξ_p were plotted for the tests on slabbing revetments. Curves were fitted as before using equation (A.6) with x = -0.67. Although these curves gave a good agreement at high Iribarren numbers ($\xi_p > 4$) they tended to under-predict the value of $H_s/(\Delta t_a)$ for low Iribarren numbers ($\xi_p < 4$). If used for design this would give low values of S_b and result in an over-design. This is, however the only region of the curves that is of interest in the design of dam slope protection. Wave development on reservoirs is limited by the short fetch lengths and so only wind waves (wave steepness greater than approximately 0.03) can be generated. The slope gradient at which blockwork and slabbing is laid is unlikely to exceed 1:1.5 due to the difficulty of placing units at a steeper angle. These two factors limit the Iribarren number to $\xi_p < 4$.

The curve fitting was repeated varying the value of both x and S_b using only the test data with $\xi_p < 4$. The best agreements were achieved with x = -1 (Figures A.3 a-p). So equation (A.6) was revised to give:

$$H_s / (\Delta t_a) = S_c / \xi_p \qquad\qquad\qquad (A.8)$$

The value of S_c varied between 6 and 13 depending on the revetment configuration (Table A.4). Such a wide range is of limited use in practice because the lowest value would always be used to ensure a safe design. The dependence of S_c on the geometry of the revetment was therefore investigated to develop a more exact design method.

Table A.3   Classification of blockwork slopes tested

Series No	Revetment Dimensions					Stability Criteria				
	Slope angle	dimensions (mm)	Ω (%)	Df15 (mm)	t_a/t_f (-)	cover permeability	filter permeability	relative cover thickness	overall stability	S_b
S1	1:1.25	120x60x20	5	0.5	0.67	N	L	N	N	4.7
S2	1:1.25	120x60x20	5	4	0.67	N	L	N	N	4.0
S3	1:1.25	120x60x20	15	4	0.67	N	L	N	N	4.2
H1	1:3	120x84x20	14	4	0.25	N	L	L	N	4.1
H2	1:3	120x84x20	14	10	0.25	N	L	L	N	2.6
H3	1:3	120x84x20	24	4	0.25	N	L	L	N	4.1
H4	1:3	120x84x20	24	10	0.25	N	L	L	N	4.0
M1	1:3	120X84X20	16	10	0.25	N	L	L	N	2.3

Stability criteria	Normal stability (N)	Low stability (L)
Ω = gap area / slab surface area (%)	>2%	<2%
D_{f15} (mm)	<0.4	>0.4
t_a/t_f = depth of cover layer / depth of filter layer (-)	>0.5	<0.5

For overall to be classified as "Low Stability" all three criteria must indicate "Low Stability".

(source Breteler & Bezuijen, (1991))

Table A4   Revetment configurations

Series	Slab dimensions (mm)	D_{15} (mm)	t_a (mm)	t_f (mm)	w (mm)	$a^{0.5}$ (mm)	S_c (-)
S8	300x300x10	4	10	30	0.2	300	11.5
S5	200x200x11	4	11	30	0.2	200	8.5
S9	300x300x10	4	10	30	3	300	10
S2	120x60x20	4	20	30	0.2	85	6
S6	200x200x11	4	11	30	3	200	8.5
S3	120x60x20	4	20	30	3	85	6.3
H1	120x84x20	4	20	80	3.5	100	5.5
H3	120x84x20	4	20	80	6	100	6
S1	120x60x20	0.5	20	30	0.3	85	6
S4	200x200x11	0.5	11	30	0.3	200	11.3
S7	300x300x10	0.5	10	30	0.3	300	13
M2	200x200x11	10	11	80	4	200	8.5
H2	120x84x20	10	20	80	3.5	100	3.5
M1	120x84x20	10	20	80	4	100	3
H4	120x84x20	10	20	80	6	100	4.1

A.2.4   Dependence of stability factor on revetment configuration

To examine which geometrical parameters are important to the stability of the slope the critical mode of failure should be considered. Previous experience has shown that the most common failure occurs when individual units are extracted due to high piezometric head difference across the cover layer. This scenario is consistent with the modes of failure observed in the physical model testing conducted during this study. The maximum piezometric head difference is dependant on the geometry and permeability of the filter and cover layers. The restoring forces are dependant on the weight of the cover layer (ignoring friction forces).

The following expression was developed to incorporate the parameters discussed:

$$S_c \propto (A_s^{1/2} / t_f)\,(w / D_{f15})$$ (A.9)

to be used in the equation A.8

where A_s is the area of a cover layer unit
          t_f is the depth of the filter layer
          w is the gap width between adjacent cover layer units
          D_{f15} is the 15% sieve value of the filter layer material

The values of S_c from all the physical model tests are plotted on Figure A.4. A straight line was fitted to the data and the following expression was found:

$$S_c = 3.3 \ln (A^{0.5} / t_f)\,(w / D_{f15})^{0.1}) + 4.0$$ (A.10)

This expression is dimensionless and so may be used for design blockwork and slabbing arrangements other than those tested in the physical models. A design chart was developed to be used in place of equation (A.10) see Figure A.5.

Since the design method uses dimensionless parameters, it may be applied to proportionately larger revetment configurations than were tested in the model.

Hydraulic scale effects are expected to be insignificant in blockwork and slabbing slopes up to a scale of 1:20. For most of the design parameters a limit of 1:10 is sufficient to allow for the ranges used in normal design practice. The limiting ranges of the design parameters are given in Table A.5

Table A.5   *Range of individual parameters for design of slabbing*

revetment geometry		min	max
slab area (m²)	A₍ₐ₎	0.07	9.0
slab depth (m)	t₍ₐ₎	0.01	0.54
gap width (mm)	w	0.2	60
filter D_{f15} (mm)	D_{f15}	0.5	100
filter depth (m)	t₍f₎	0.03	0.8
wave conditions			
significant wave height (m)	H₍s₎	0.1	4
peak wave period (s)	T₍p₎	1.0	10.0
surf similarity parameter (-)	ξ₍p₎	1.7	4

It is unlikely that these limits would be exceeded in normal design practice.

The upper limits given were found by multiplying the greatest value of each parameter tested in the physical models by a factor of ten. For the depth of cover layer t_a this gives a maximum of 0.2m. For larger slabbing units (above 4m²) the required cover layer depth may exceed this value.

This limiting value may however be increased by considering the failure mechanism of the slabbing. An increase in cover layer thickness has two additional effects on the stability of the revetment. The length of flow path between the surface and the filter layer is increased and the friction between adjacent units is also increased. The change of flow path will decrease the cover layer permeability, which would be expected to decrease the revetment stability. In the regression analysis of the test results, however, the length of flow path had no significance for thin slabs. If deeper cover units are used the increased length of flow path may however become more significant to the stability of the revetment to a small degree.

Any decrease in stability is likely to be offset by the increased friction between blocks. The extraction of deeper units will require proportionately greater up-lift force. A thin slab which rotates as it is lifted under wave action will probably be extracted, but a cube which rotates will tend to jam against adjacent units. The upper limit of cover layer depth was therefore increased to 0.5m.

To evaluate the error implicit in equation (A.10), predictions of the slab depth (t_a) were found, given the other original model test conditions. The average and standard deviation of the error between the predicted slab depth and the actual depth tested were calculated. By assuming that the errors form a normal distribution about the average, the probability that a certain error will be exceeded may be found. When designing the slab depth it is 95% certain that the error is less than ±30%. This process was repeated for the other design parameters, and the results compiled in Table A.6. The slab density and slope angle may be designed with similar accuracy to the slab depth. For slab area and filter depth it is 95% certain that the error is less than ±56%.

Table A.6   *Errors implicit in derived expression*

Parameter	Average error (%)	Standard deviation of error (%)	95% probability limit (%)	Confidence factor
slope angle (tan α)	-2	18	30	0.77
Slab depth (t_a)	-2	18	30	1.30
Slab density (ρ_c)	-2	18	30	1.30
Slab area ($A_s^{0.5}$)	-0	36	60	1.60
Filter depth (t_f)	-0	36	60	0.63
Gap width (w)	17	178	310	4.10
D_{f15}	-17	178	310	0.24

The gap width of the cover layer and the D_{f15} for the filter layer are more sensitive to the uncertainty inherent in equation (A.10). This is due to the 0.1 power factor applied to these two parameters. It is 95% certain that the error is less than ±240%. This error may initially appear to be prohibitively large for design purposes, but when since the gap widths and values of D_{f15} are small, applying such a large error factor does not result in a large absolute difference. For example, given a D_{f15} of 2mm, application of the error factor of 0.3 gives a value of D_{f15}=0.7mm which is not a large difference.

It is unlikely, however, that the gap width between adjacent units would be selected as the main design parameter. The gap width will vary due to movement of the embankment dams during and after construction. Fortunately the influence of the gap width (and the value of D_{f15}) is very small compared to the other design parameters. An uncertainty of a factor of ten in the value of w/D_{f15} will only vary the value of stability factor S_c by a maximum of ±1 (see Figure A.5). The value of gap width used in the design chart should be the minimum spacing of the armour units at the time of placement. If the gap width becomes larger due to dam movement the permeability of the cover layer increases and the stability of the revetment also increases.

The error bounds indicate the improvement of accuracy obtained from use of the new expression for design purposes. Previously the wide range of suggested values of S_c could used result in uncertainty of blockwork depth greater than 100%. The evaluation of probability limits for the results also gives a greater measure of confidence when using the design method.

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Appendix Figures
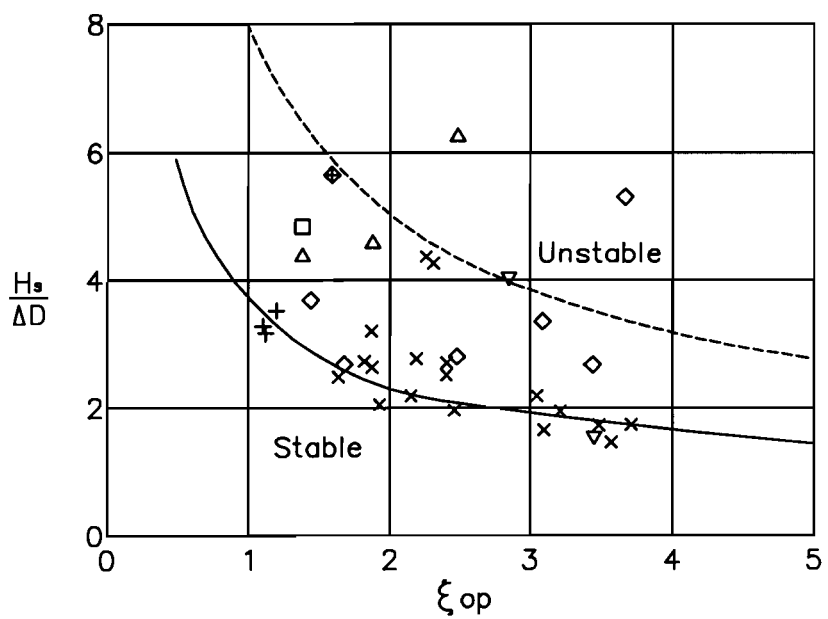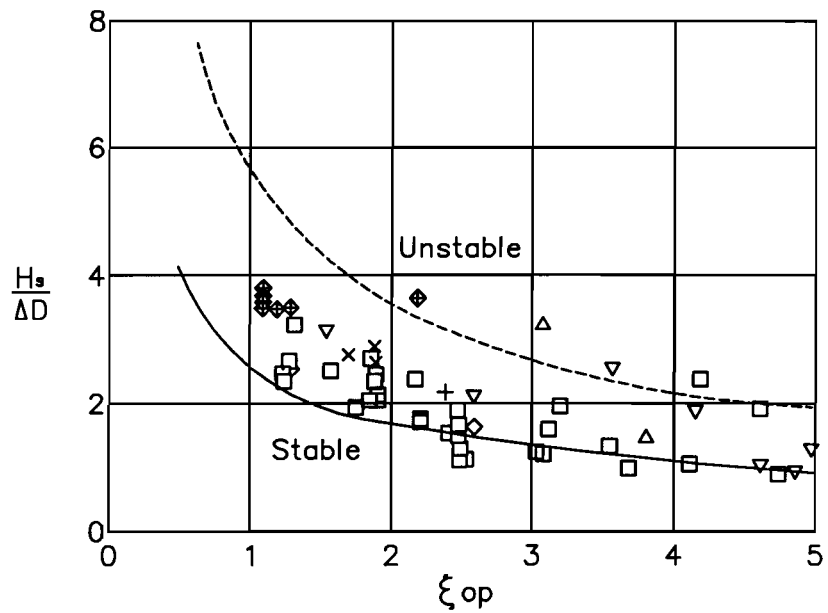
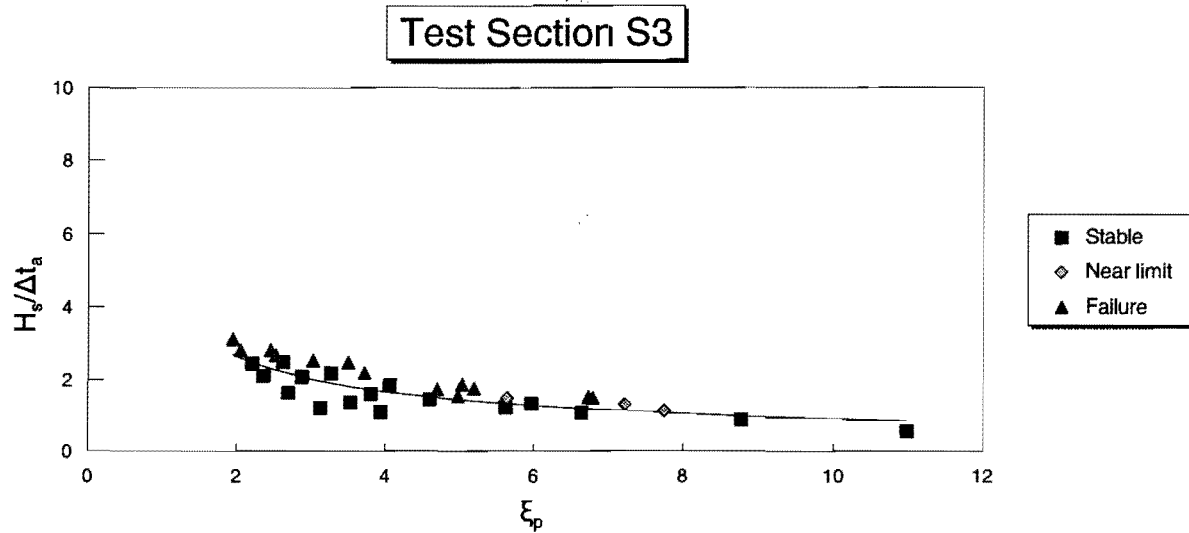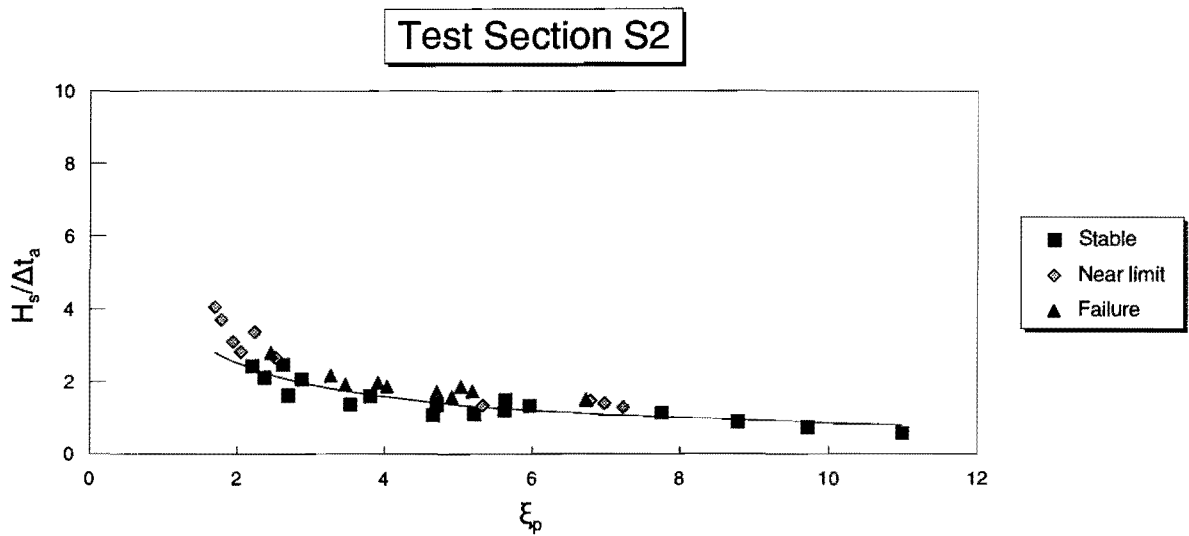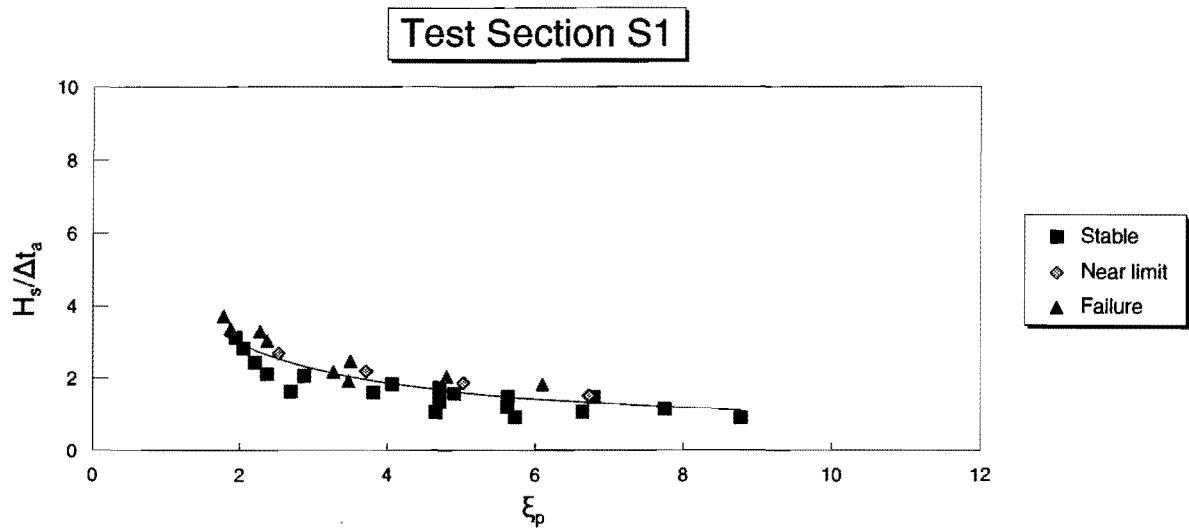**Figure A1a & b**   *Stability curves from Breteler (1991)*

Figure A2a, b & c *Stability curves from model test results*

Test Section H1

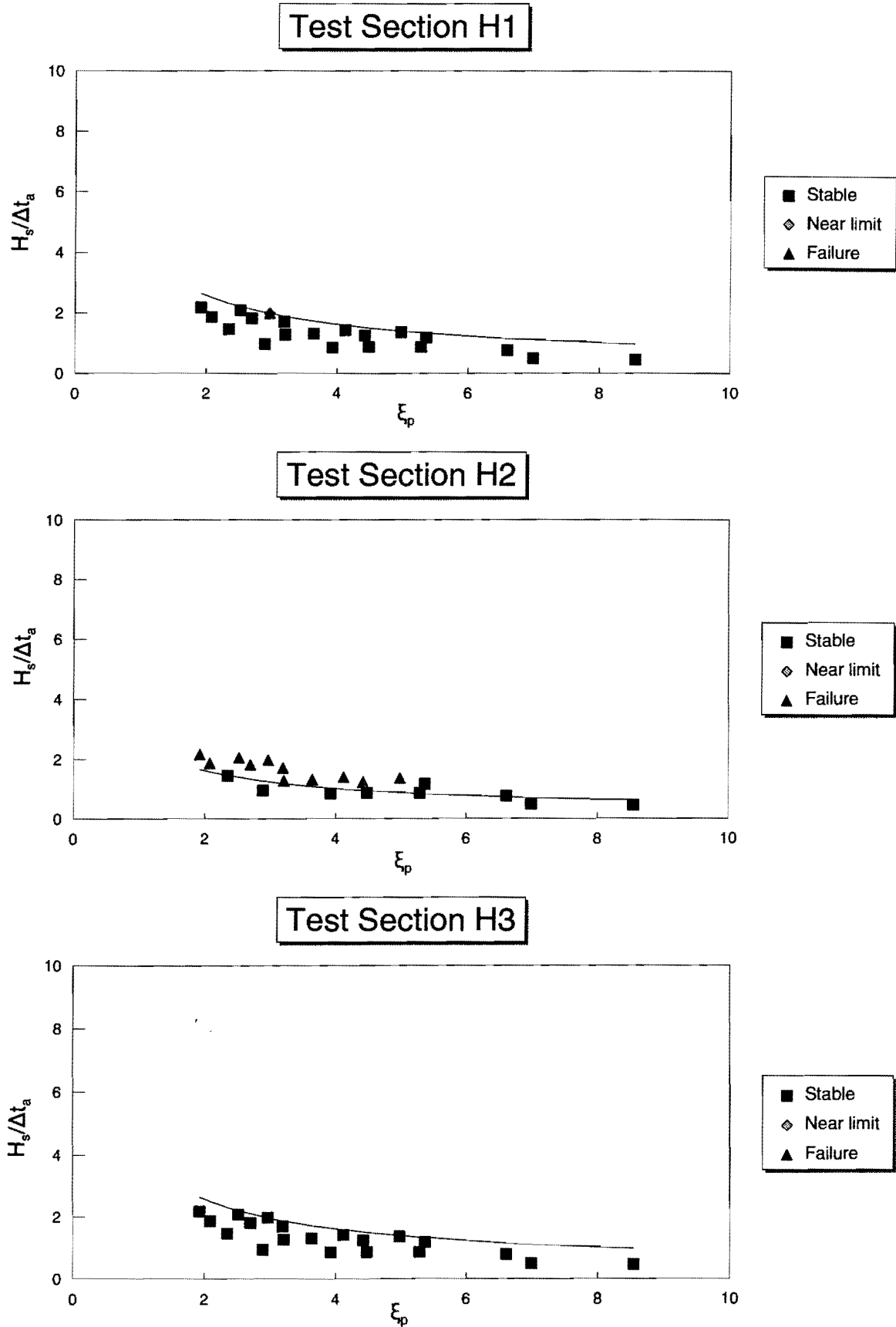## Test Section H2

Test Section H3

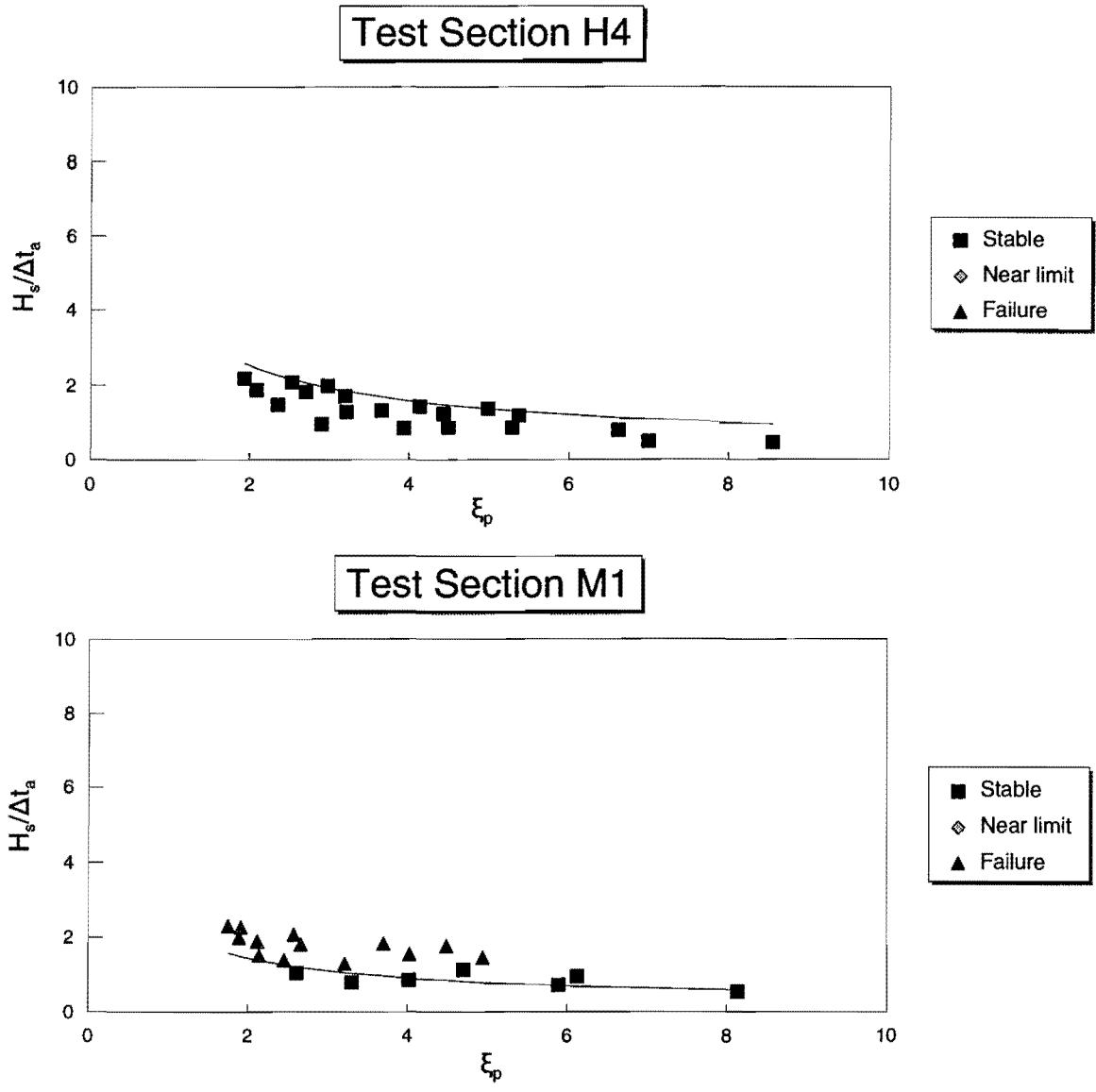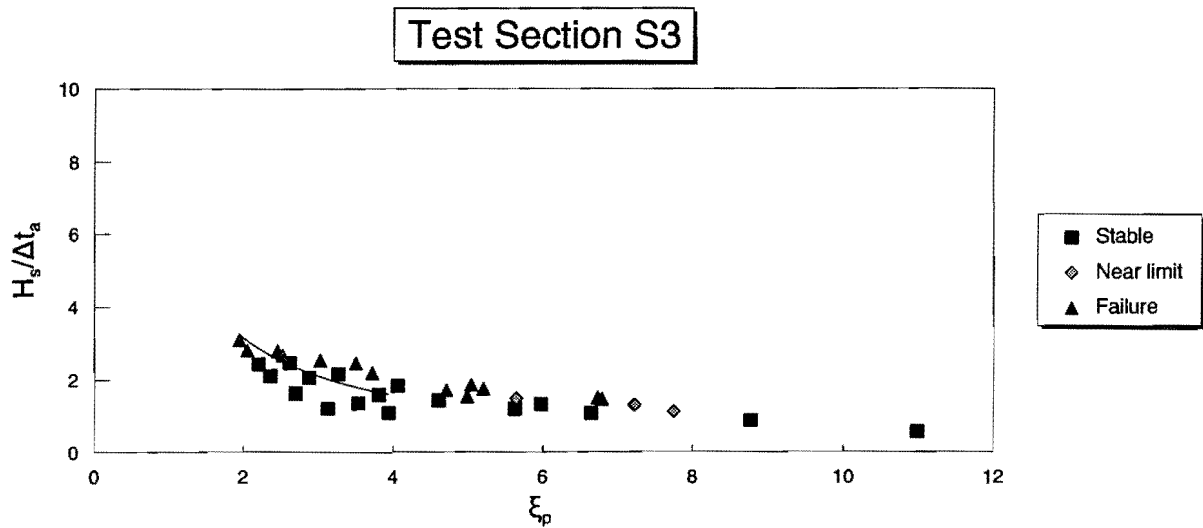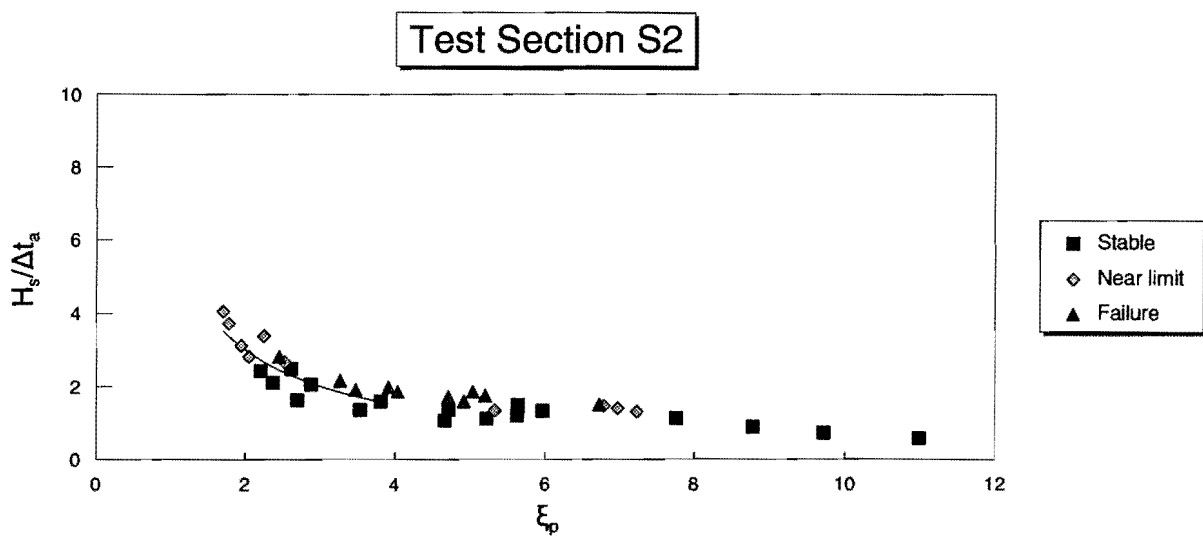**Figure A2d, e & f** *Stability curves from model test results*
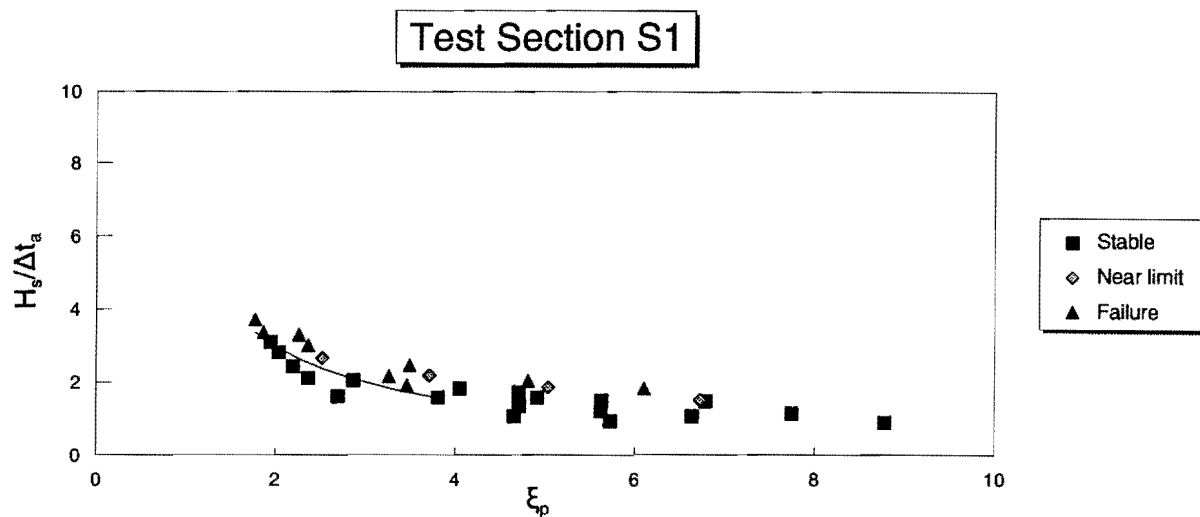
Figure A2g & h *Stability curves from model test results*
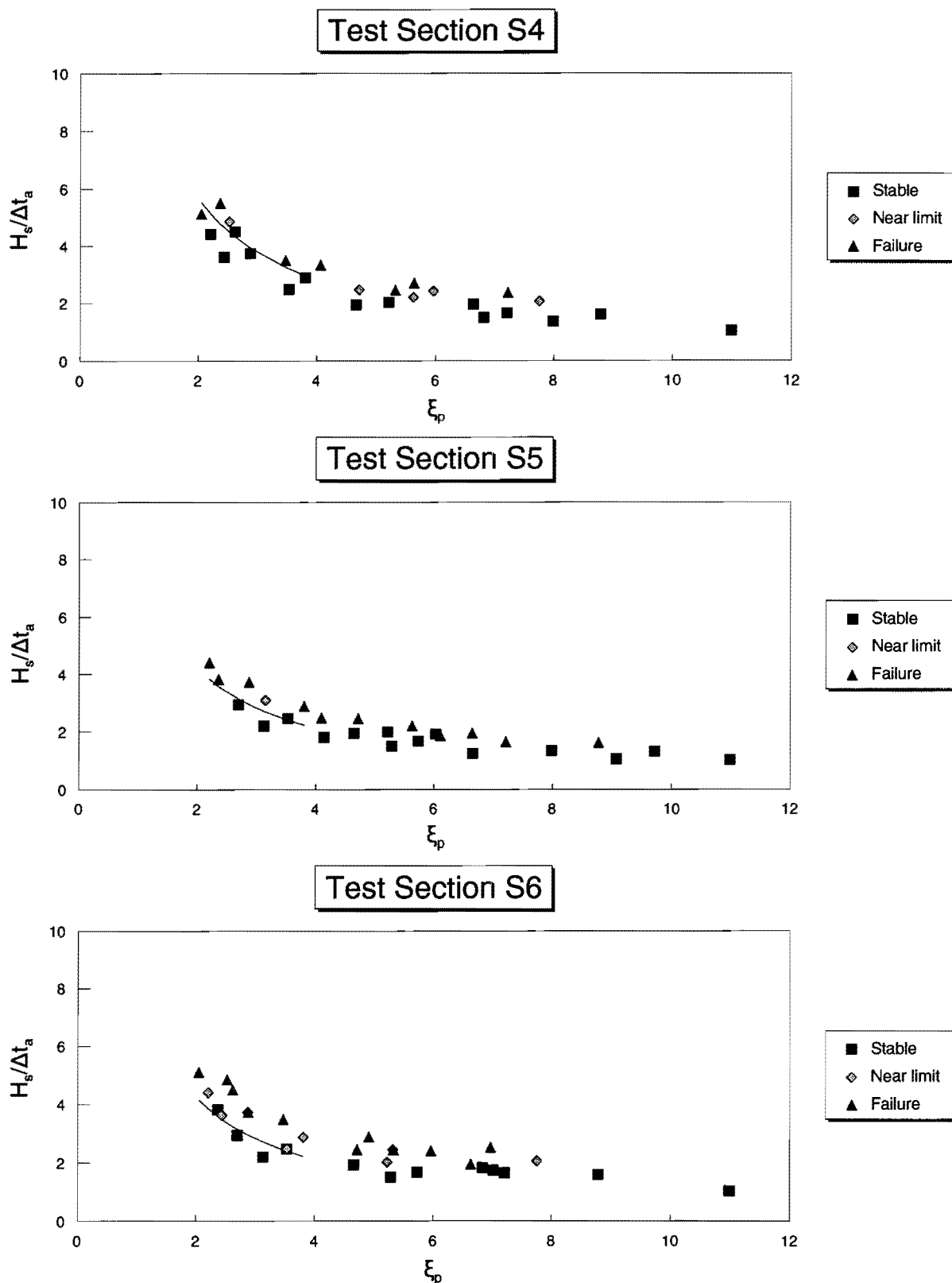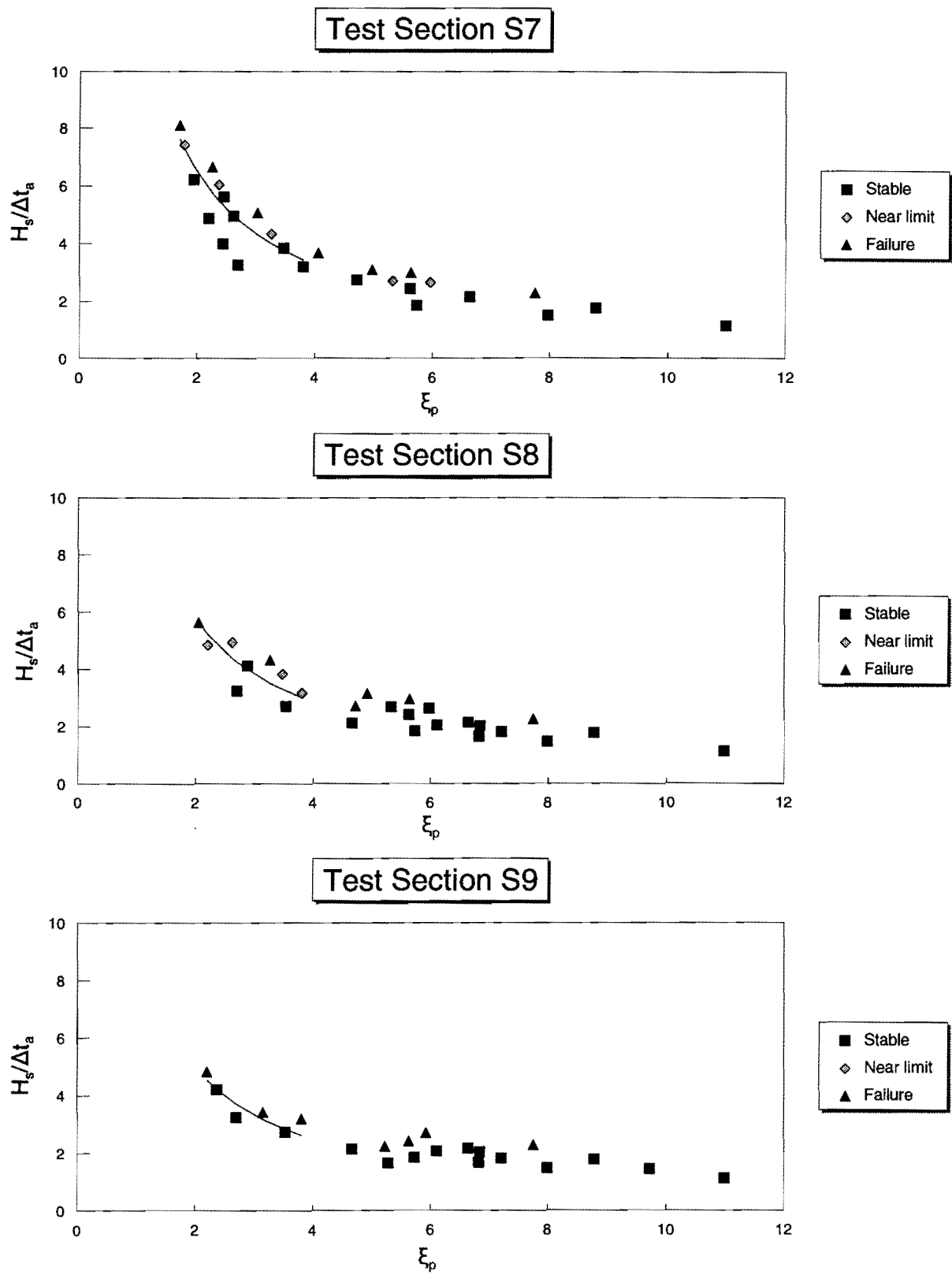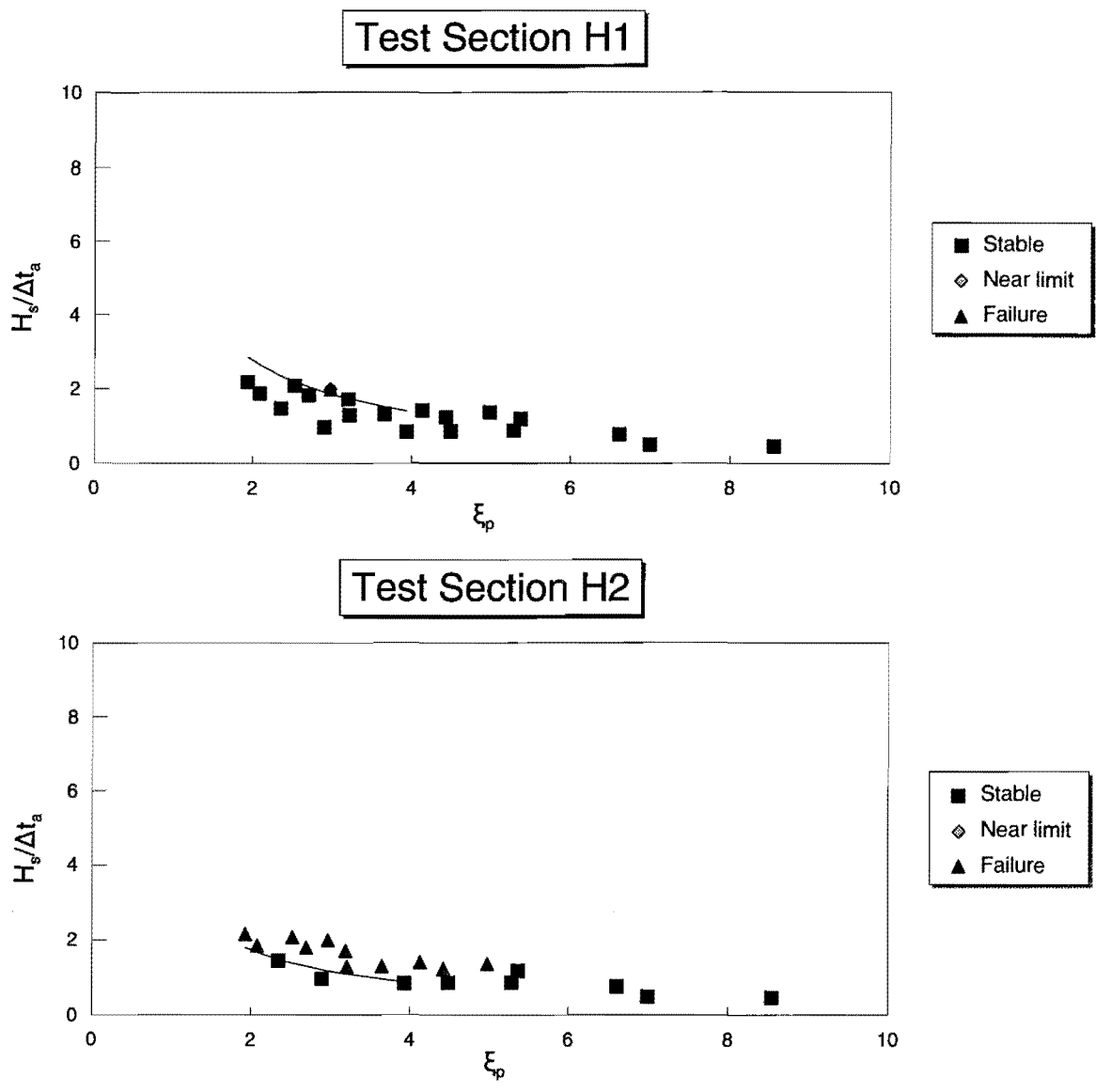
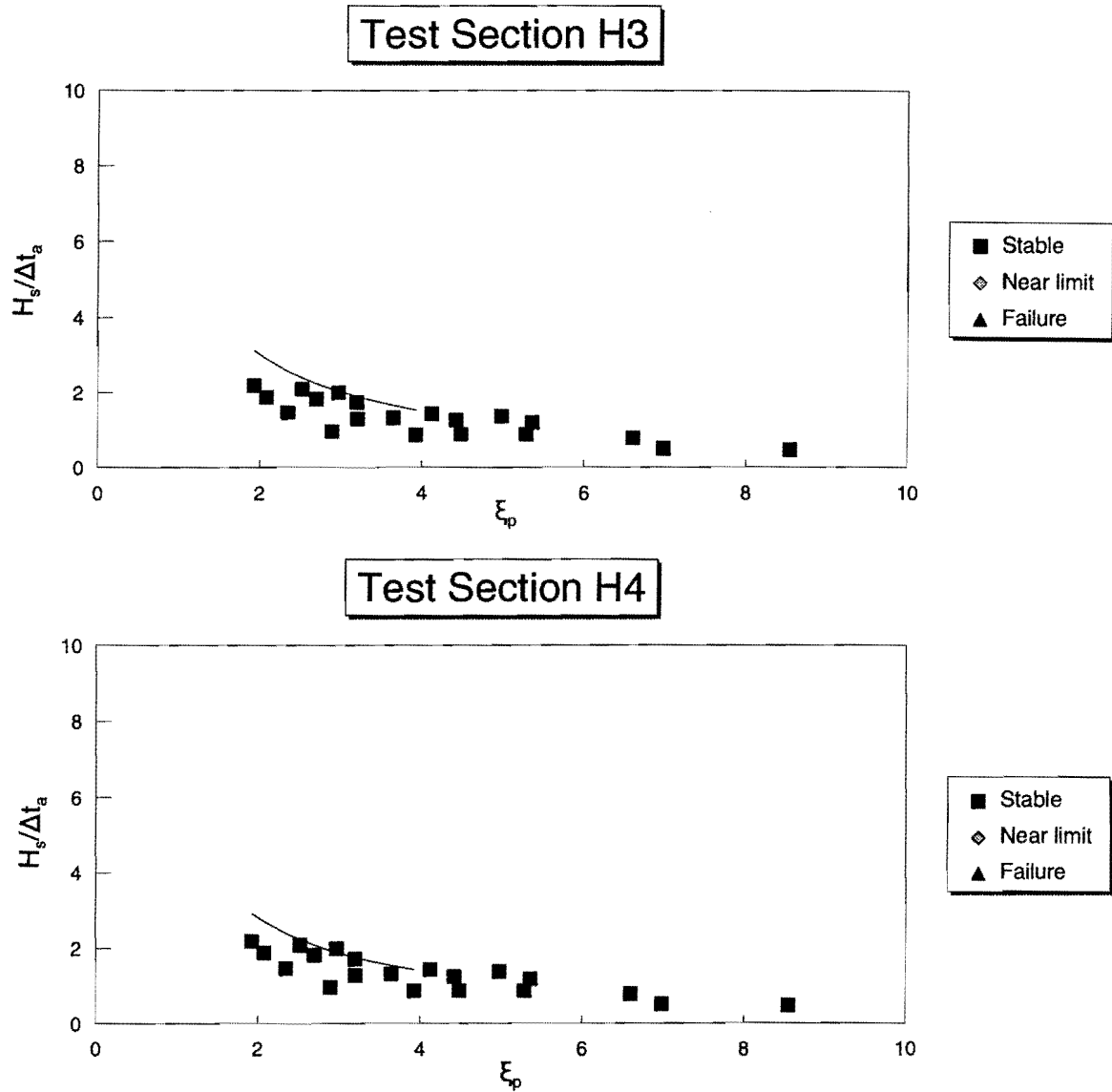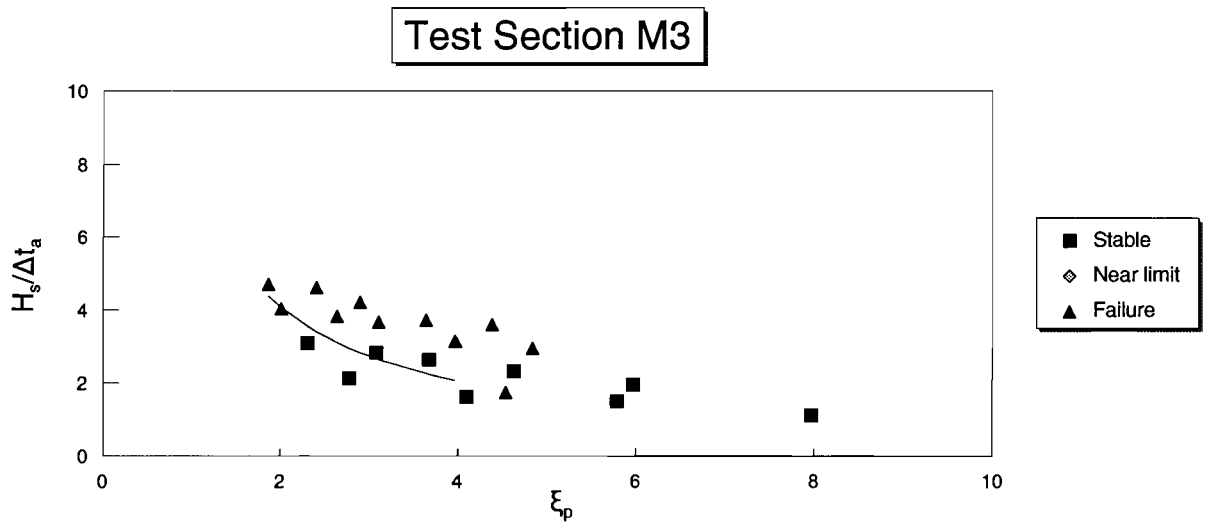**Figure A3a, b & c** *Stability curves from model test results*

Figure A3d, e & f *Stability curves from model test results*

Figure A3g, h & i *Stability curves from model test results*
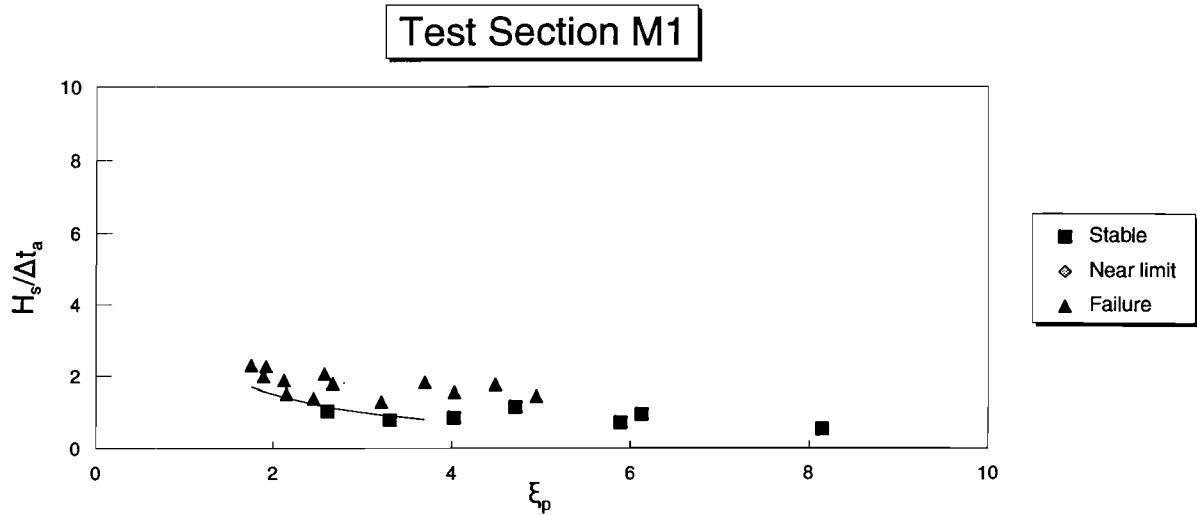
Figure A3j & k   *Stability curves from model test results*

Test Section H3

$H_s/\Delta t_a$

10
8
6
4
2
0

■ Stable
◇ Near limit
▲ Failure

0   2   4   6   8   10

ξ_p

Test Section H4

$H_s/\Delta t_a$

10
8
6
4
2
0

■ Stable
◇ Near limit
▲ Failure

0   2   4   6   8   10

ξ_p

Figure A3l & m   *Stability curves from model test results*

Test Section M1

Test Section M2

Test Section M3
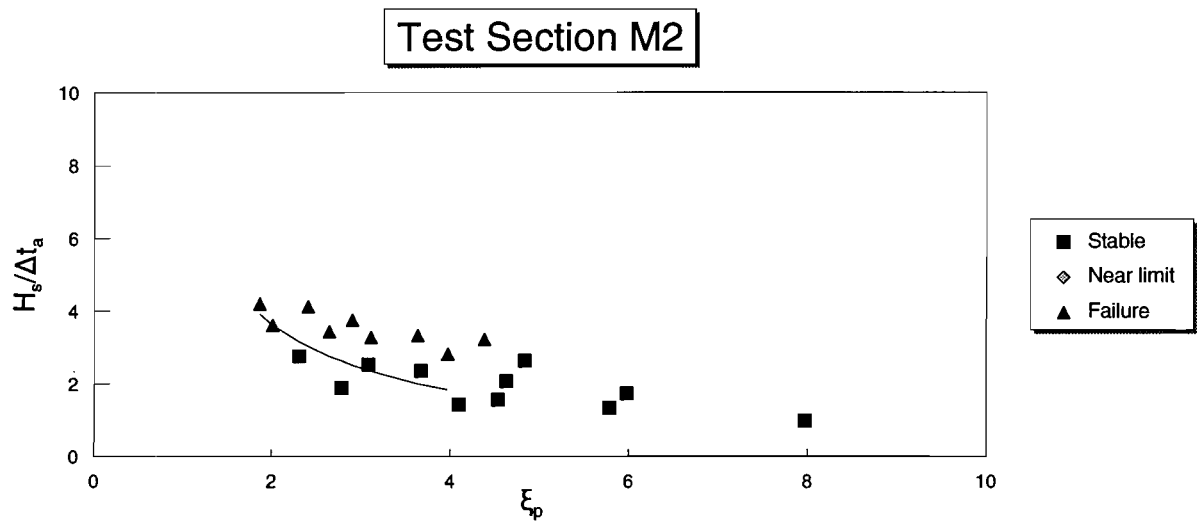
Figure A3n, o & p *Stability curves from model test results*
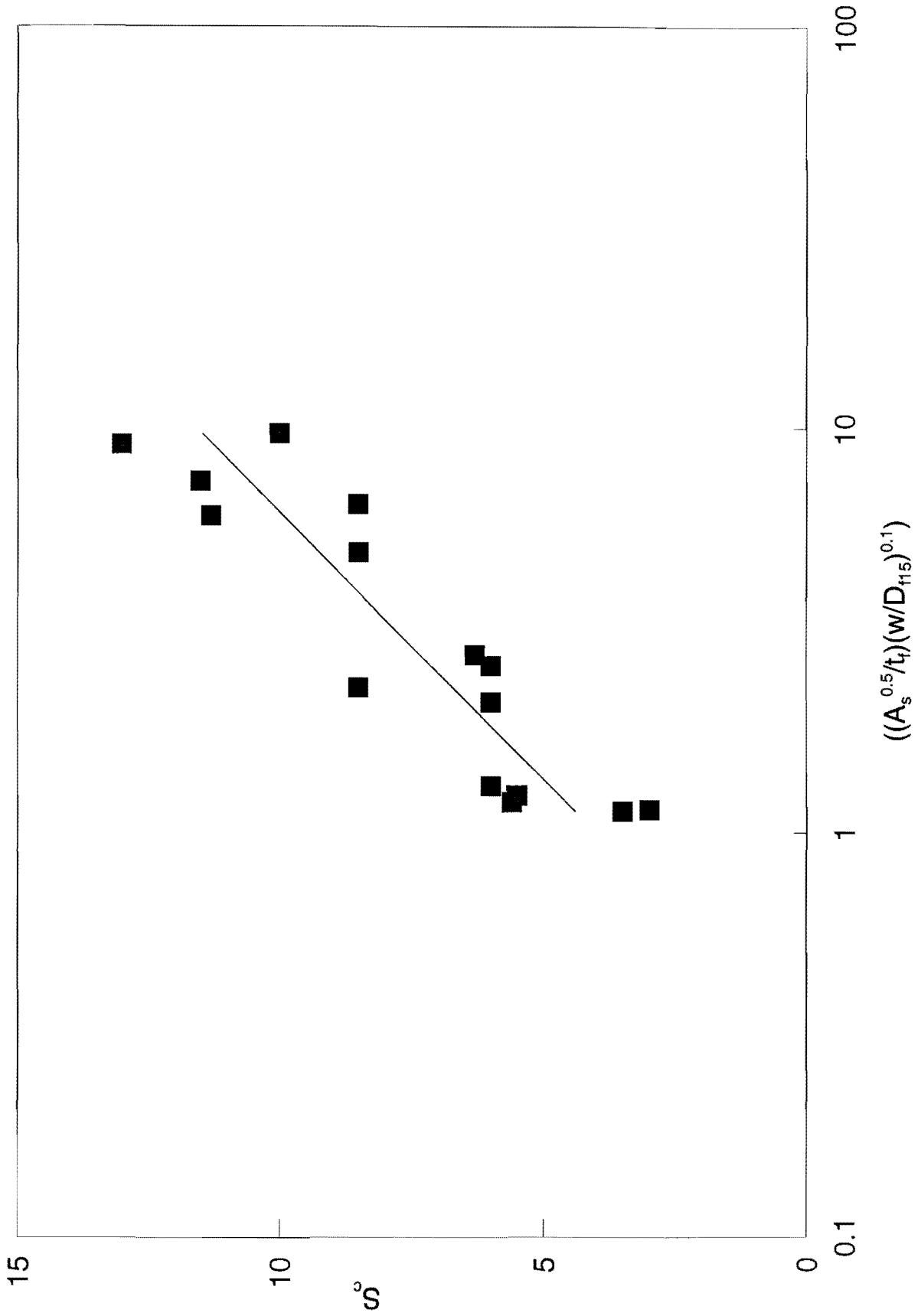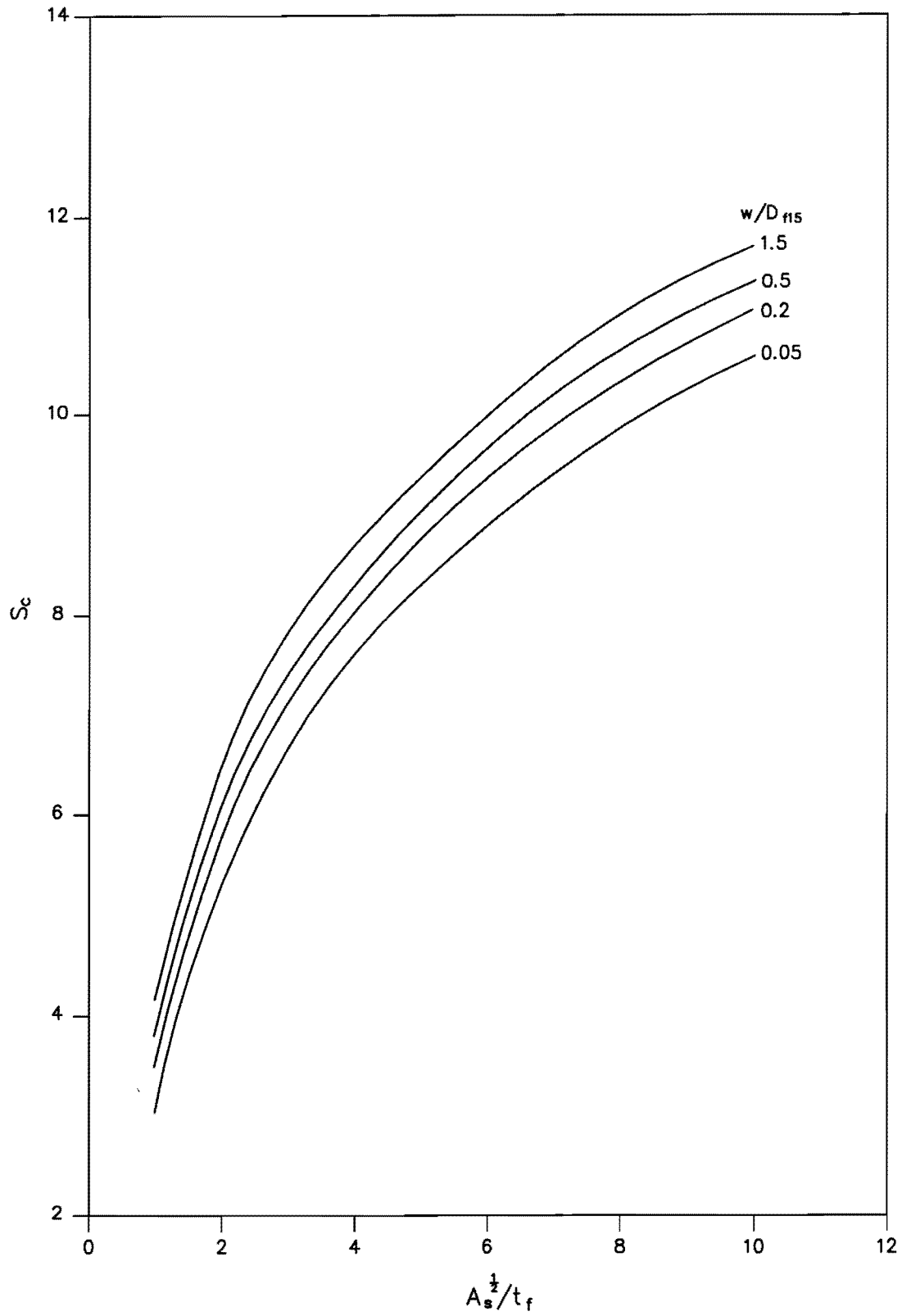
Figure A4   *Values of S_c from the physical model tests*

Figure A5   *Design chart for slabbing*