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SEA WALLS
: A Literature Review

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SUMMARY

The design of sea walls for coast protection or sea defence is a particularly complex task, made more difficult by the site-specific nature of each location, the uncertainties of many aspects of the interaction between waves and the structure, and by a lack of detailed design guidance.

This report brings together, and comments upon, research findings and possible design methods covering all main aspects of sea wall design in the UK. A major part of the review concentrates on the hydraulic aspects of design: wave run-up and overtopping; wave forces and pressures; reflection performance; and the effects of waves on armouring systems. The review also considers other aspects affecting the design of sea walls including the administrative, financial and legal framework for coastal works; design conditions for waves and water levels; coastal processes and geotechnical conditions; typical forms of construction; materials and construction practice. Each chapter includes a detailed list of the references of particular relevance. The report also includes a major bibliography of over 500 items.

The review was conducted as part of CIRIA Research Project 353. This report is intended to accompany the report on Phase 1 of that study, published as CIRIA Technical Note 125.

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NOTATION

a	wave amplitude, or sum of incident and reflected wave heights
A, B	Empirical coefficients
a, b	Empirical coefficients
C_D	Particle drag coefficient
C, C_1, C_2	Empirical or shape coefficients
d	Water depth, usually at structure toe
D	Particle dimension, diameter or thickness
Dn	Nominal particle diameter
Ei, Er	Incident or reflected wave energy
Fc	Structure freeboard, crest level less static water level
g	Gravitational acceleration
H	Wave height
Ho	Offshore, deep water, wave height
Hs	Significant wave height
Hi, Hr	Incident or reflected wave heights
Ir	Iribarren number, defined in equation 5.3
k	Wave number, $2\pi/L$
k'	Wind enhancement factor, equation 5.16
k_β	Wave angularity factor, equation 5.17
K_1	Stability factor, equation 5.33
K_D	Hudson damage coefficient
Kr	Coefficient of wave reflection, equations 5.19-20
L	Structure design life, or wave length
L_o	Deep water wave length, $g\ T^2/2\pi$
N	Number of waves in storm, or proportion overtopping
O_{90}	90 percentile opening size, equation 5.36
p_i	Wave pressure, equation 5.18
P	Notional core permeability factor, equations 5.26-28
P_{fl}	Lifetime probability of failure, equation 2.1
P_{fa}	Annual probability of failure, equation 2.1
q	Volume of overtopping, per wave, per unit length of sea wall
$\bar{Q}$	Mean overtopping discharge, per unit length of sea wall
Q^*	Dimensionless overtopping discharge, equation 5.11
R	Wave run-up level
Rs	Run-up level of significant wave height
R_2	Run-up level exceeded by 2% of run-up levels
R^*	Dimensionless freeboard, equation 5.12
s	Wave steepness, usually H/Lo

Si, Sr	Incident or reflected energy density
S	Damage number, or scour depth
T	Wave period
Tz	Mean zero-crossing wave period
Tp	Peak wave period, of maximum energy density
u	Orbital wave velocity, usually horizontal component
U_{10}	Mean wind speed, 10m above sea surface
W	Armour unit weight
W_{50}	Median armour unit weight
W_f	Wind factor, equation 5.15
α	Sea wall slope angle, to the horizontal (in radians)
α, β	Empirical coefficients
β	Angle of wave attack, normal waves $\beta = 0$
ε	Spectral width
γ	Weight density
γ_w	Weight density of (sea) water
γ_r	Weight density of rock (or concrete)
η_s	Water surface elevation
ρ	Mass density, often of fresh water
ρ_r	Density of rock
ρ_c	Density of concrete
Δ	Relative density ($\gamma_r/\gamma_w - 1$)
ϕ	Angle of repose of granular material

CONTENTS

1 INTRODUCTION

1.1 Purpose of project

The cost of construction and maintenance of sea walls in the United Kingdom amounts to approximately £50 million annually. Present design practice, at its best, seeks to combine broad experience in coastline control with empirical and theoretical approaches.

Due to diverse responsibility for sea walls, there are limited data available nationally on the performance of walls in service and the present level of construction and maintenance costs. In particular, the scale and nature of problem areas which have occurred in the recent past, and which are likely to be encountered over the next (say) 20 years have not been identified nationally.

The need exists for a survey of the present situation, leading to the production of rational guidelines on the design of sea walls along the British coastline, with particular emphasis on the replacement and rehabilitation of existing infrastructure. These guidelines would need to incorporate the improved understanding of the effects of waves and tides upon sea wall structures, and the interaction of these effects with structure foundations and abutting foreshores. In addition, the engineer needs to have an increasing awareness of the environmental impact of coastal schemes.

A research project has therefore been set up under the auspices of the Construction Industry Research & Information Association (CIRIA). The overall object of the project is to establish guidelines for the design of sea walls for coastline control in Great Britain and Northern Ireland.

1.2 Project organisation

In July 1984, CIRIA appointed as research contractor for this project a joint venture between Posford, Pavry & Partners and Lewis & Duvivier. The sub-contract to undertake a 'state of the art' literature review was let to Hydraulics Research in August 1984. A draft of this report was submitted in December 1984. This report constitutes the final report on the literature review conducted by Hydraulics Research for this project. It is intended to supplement the report on phase 1 of the project by Stickland & Haken (Ref 5).

1.3 Extent of the literature review

This review is intended to cover the technical literature relating to the design, performance and maintenance of sea walls, relating principally to the UK, but also considering overseas conditions. In the first instance it was clear that very little technical data was available specifically on the design of such structures. Together with the research contractors, a review of the fundamental information needed in the design process was conducted. From this a series of subject headings were agreed, and these now appear as chapter, section and sub-section headings in Chapters 3-8.

Much of the literature reviewed in this study has been presented to scientific or specialist conferences, in journals and periodicals, or in the proceedings or journals of learned societies, associations and institutions. Some of the fundamental aspects of wave action, and the design of coastal structures, have been covered in a number of text books or manuals. Of these, the most useful are those by Muir Wood & Fleming, Thorn & Roberts, Sorensen, and Goda; and the Shore Protection Manual produced by the US Army (Refs 1, 2, 3, 4, 6).

1.4 Definition of a sea wall

For the purpose of this research project, a sea wall has been defined as a structure whose primary purpose is either protection against erosion, the alleviation of flooding, or a combination of both, in which wave action plays a dominant role. The various types of structure that come within this definition are set out in detail in the CIRIA report of phase 1 of the project (Ref 5). The literature review has considered, where appropriate, information on the following types of sea wall:

(a) Embankment with or without structural crest elements;
(b) Rubble revetment or rubble sea wall;
(c) Armoured revetment;
(d) Vertical, or near vertical wall;
(e) Caisson sea wall, with or without rubble mound.

1.5 Use of references and abbreviations

In common with the practice in some other reviews of this type, reports and papers considered have been referred to in one of two ways. Those articles that have actually been considered in detail, and to which the reader may be referred, have been classified as references. References have been listed at the end of each chapter. They have been identified in the text by a reference number, eg (Ref 8). The branch literature, and many classic papers and reports to which time did not allow further study, have been listed in the bibliography in alphabetical order of author. These items are identified in the text by the use of the author's name and the date of publication, eg Pocklington (1921). Also included in the bibliography are all the references already covered in the text.

In order to simplify the use of this review, the formulae and expressions from the literature have been expressed using a single set of symbols, and in SI units where appropriate. In particular this may mean that some American and Japanese expressions with coefficients having dimensions will have been re-worked. A list of notation used in this review has been given at the start of this report.

In order to reduce unnecessary repetition, especially in the bibliography and reference sections, a number of abbreviations have been used, mainly those commonly used for the names of author or publisher organisations. The main abbreviations are listed below:

ACI - American Concrete Institute;
ASCE - American Society of Civil Engineers;
BHRA - British Hydromechanics Research Association;
BSI - British Standards Institution;
C&CA - Cement and Concrete Association;
CERC - Coastal Engineering Research Centre - US Army Corps of Engineers (now incorporated into WES);
CETA - Coastal Engineering Technical Aids - series of reports published by CERC;
CIRIA - Construction Industry Research and Information Association;
HRS - Hydraulics Research Station - now known as Hydraulics Research, Wallingford;
ICE - Institution of Civil Engineers;
IMCyE - Institution of Municipal and County Engineers
IOS - Institute of Oceanographic Sciences;
JFM - Journal of Fluid Mechanics;
MIAS - Marine Information and Advisory Service;
PIANC - Permanent International Association of Navigation Congresses
SPM - Shore Protection Manual, published by CERC;
TACPI - Technical Advisory Committee on Protection against Inundation, Holland;
WES - Waterways Experiment Station, US Army Corps of Engineers (now includes CERC).

1.6 References

1 Coastal Engineering Research Centre "Shore Protection Manual" Vols I-II. US Government Printing Office. Washington, 4th edition 1984.

2 Goda Y "Random seas and design of maritime structures" University of Tokyo Press, Tokyo, 1985.

3 Muir Wood A M & Fleming C A "Coastal Hydraulics" Macmillan, London 2nd Edition, 1981.

4 Sorensen R M "Basic coastal engineering" Wiley Interscience, New York, 1978.

5 Stickland I W & Haken I C "Sea walls, survey of performance and design practice" Technical Note 125, CIRIA, London, 1986.

6 Thorn R B & Roberts A G "Sea defence and coast protection works - a guide to design" Thomas Telford, London, 1981.

2 SUMMARY OF PRESENT DESIGN PRACTICE

2.1 General

In various countries the design of sea walls may be based upon a design manual, code of practice or standardised design guidelines. Such publications may have mandatory effect or be simply advisory.

In America, the US Army Corps of Engineers publish a design manual, the Shore Protection Manual (Ref 7) and related Coastal Engineering Technical Aids (CETA). These documents are used worldwide for the design of coastal structures. In Holland, the responsible departments of the Dutch government have supported the production of a number of reports or manuals giving design guidelines on certain specific subjects (Refs 16-18). In Germany, recommendations have been produced by the Committee for Waterfront Structures. These are not mandatory regulations and so can be simply up-dated annually if required. Lackner explains the composition and work of this committee, and the publication of its recommendations both in German and in English (Refs 8 and 10).

In the UK, in contrast to the USA where the Corps of Engineers is responsible for many miles of waterway, river and coastline, there is little centralised design or construction of coastal structures. Recently, however, the British Standards Institution (BSI) have issued part 1 of a Code of Practice for maritime structures, BS 6349 (Ref 4), and are in the course of considering the needs for, and the drafting of, various further sections of BS 6349. This Code of Practice is not, however, intended to be of direct use in the design of sea walls and similar structures. Whilst it considers some subject areas in detail, some other aspects of importance in the design of sea walls therefore receive very little attention. Many of the more detailed sections are based closely on the Shore Protection Manual, and reflect that document's essentially regular wave and deterministic philosophies.

2.2 Deterministic design

Virtually all of the design manuals or guidelines considered in this review are based on an essentially deterministic design philosophy. In such design methods, a design storm or wave height of predicted return period, such as 1:50 years, is considered and the structure is designed to resist that event with an acceptable degree of safety. In some designs with rubble sea walls this may represent an acceptable level of damage to the armour layer.

The principal such design manual is the Shore Protection Manual (Ref 7), most other manuals being based upon it, at least in part (Refs 4, 6). Of considerable use in furthering understanding of the processes involved are the text books by Thorn & Roberts, Muir Wood & Fleming, Sorensen, Quinn, and Bruun, all of which are essentially deterministic in philosophy (Refs 5, 12, 13, 14, 19). Deterministic design methods are reasonably simple to use, and require relatively little input data. It is, however, argued by some that deterministic methods often lead to over-design, and that they do not allow the assessment of risk levels of damage or failure.

2.3 Probabilistic design

The use of probabilistic or reliability design methods is not yet incorporated into any of the design manuals or text books covering sea walls. It has, however, been the subject of a number of well argued technical papers covering breakwaters and other coastal structures including sea walls. Bakker & Vrijling argue that deterministic design has given widely differing standards of protection when the risk of failure is assessed from a probabilistic standpoint (Ref 2). A brief outline of a probabilistic method for the design of sea dykes and accompanying sand dunes is given. The authors develop joint probability density functions for storm surge levels and wave conditions. The probability of exceedance of a specific threat (in terms of run-up level) is evaluated by integrating the two-dimensional probability density functions of wave run-up and static water level, for a given sea state.

Dover & Bea (Ref 9) also discuss the theory and use of reliability/risk analysis methods of design. They define reliability as the probability that the resistance of the structure exceeds the loads, conversely risk is defined as the probability of experiencing a failure. The probability density functions of structure loadings and resistance (strength) are defined in general terms. The failure probability, P_f, may be calculated using a closed-form approximation ascribed to Ang & Cornell (1974). The lifetime probability of failure, P_{fl}, may then be related to the annual failure probability P_{fa}, and the structure design life, L:

$$P_{fl} = 1 - \exp(-P_{fa} L) \qquad (2.1)$$

The results of similar calculations are presented by Berry (Ref 3) and by Alcock (Ref 1), who emphasise that the return period is the average time between events exceeding the selected level of severity. In many instances there is significant risk that the event will be exceeded in a period less than the return period.

The use of probabilistic design methods for other major coastal structures is discussed by Stans (Ref 15), and Mol, Ligteringen & Paape (Ref 11).

2.4 References

1 Alcock G A "Discussion on papers 3-4" Developments in breakwaters, ICE, London, 1986.

2 Bakker W T & Vrijling J K "Probabilistic design of sea defences" No WWKZ-80, Vol 12, Rijkwaterstaat, Flushing, June 1980 - also Proc. 17 Coastal Eng. Conf. Sydney, 1980.

3 Berry J "Discussion on papers 8-9" Developments in breakwaters, ICE, London, 1986.

4 British Standards Institution "British Standard Code of Practice for maritime structures, part 1: general criteria" BS 6349: Part 1: BSI, London, 1984.

5 Bruun P "Port engineering" Gulf Publishing, Houston, 1973.

6 Bruun P & Nayak B U "Manual on protection and control of coastal erosion in India" National Institute of Oceanography, Dona Paula, Goa, 1980.

7 Coastal Engineering Research Centre "Shore Protection Manual" Vols I-II. US Government Printing Office. Washington, 4th edition 1984.

8 Committee for Waterfront Structures "Recommendations of the Committee for Waterfront Structures, EAU 1980" Wilhelm Ernst and Sohn, Berlin, 4th English Edition, 1982.

9 Dover A R & Bea R G "Application of reliability methods to the design of coastal structures." Proc. Coastal Structures 79, ASCE, Alexandria, 1979.

10 Lackner E "The West German approach to the Code of Practice for Waterfront Structures" Proc. ICE, Part 1, Vol 76, August 1984.

11 Mol A, Ligteringen H & Paape A "Risk analysis in breakwater design" Proc. Conf. Breakwaters - design and construction, ICE, London, 1983.

12 Muir Wood A M & Fleming C A " Coastal Hydraulics" Macmillan, London 2nd Edition, 1981.

13 Quinn A De F "Design and construction of port and marine structures" McGraw-Hill, 2nd edition, 1972.

14 Sorensen R M "Basic coastal engineering" Wiley Interscience, New York, 1978.

15 Stans J C "Model investigations and probabilistic design" Proc. Coastal Structures 79, ASCE, Alexandria, 1979.

16 TACPI "Wave run-up and overtopping" Technical advisory committee on protection against inundation, Government Publishing Office, The Hague, 1974.

17 TAW "Leidraad cementbetonnen dijkbekledingen" Technische Adviescommissie voor de Waterkeringen, Government Publishing Office, The Hague.

18 TAW "Leidraad voor de toepassing van asfalt in de waterbouw" Technische Adviescommissie voor de Waterkeringen. Holland, January 1984. Now available in English, Rijkswaterstaat, 37/1985.

19 Thorn R B & Roberts A G "Sea defence and coast protection works - a guide to design" Thomas Telford, London, 1981.

3 WATER LEVELS AND WAVE CONDITIONS

3.1 Water levels, tides and surges

3.1.1 General

In this chapter the term "still water level" (swl) is defined as the mean elevation, to a specified datum, of the water surface over a period of time long enough (generally about a minute) to eliminate the high frequency oscillations of surface gravity waves. Water level fluctuations may be classified by the characteristics, and causes, of the types of motion or combinations that take place. These are often given as:- astronomical tides, storm surges, variations in mean sea level, seiches and tsunamis. Tsunamis and seiches are of less direct relevance to sea wall design for the UK than are tides, surges and changes in mean sea level.

In a recent review, Alcock (Ref 5) notes that advice issued by the Ministry of Agriculture, Fisheries and Food (MAFF) to River Authorities in the 1960's suggested the standard of defence should, relative to conditions 50y ahead, protect against a level having a return period of 200 to 300y without permitting any noteworthy flooding, and protect against a level with a 1000y return period without disastrous flooding.

There is a distinction between (a) "noteworthy" and (b) "disastrous" flooding, which could be characterised as

(a) due to the combined level of swl and waves being above the defence level, and

(b) due to the swl itself being above the defence level.

3.1.2 Astronomical tides

The regular fluctuation of water level is caused by astronomical tides. These are generated by the effects of gravitational attraction of various heavenly bodies on the mass of water in the seas and oceans. In practice only the sun and moon give rise to any significant effects. Tides arise due to the relative rotations of the earth and the sun and moon. Simple summaries of tidal theories are given by Macmillan (1966), Muir Wood & Fleming (Ref 51), who also provide a definition of tidal terms, as does BS 6349 (Ref 20) and by Sorensen (Ref 66). Predictions of tide levels may be obtained for many locations around the UK and elsewhere, either by consulting the relevant Admiralty tide tables (Refs

46, 27), or from the Institute of Oceanographic Sciences (IOS). For USA waters the National Ocean Survey of the US Department of Commerce also publish tide tables and Pore & Cummings (1967) give details of a computer program used for official tide predictions around the USA.

For locations not covered by the standard tide tables, tide levels may be available from IOS or may be predicted using basic tidal theory. Some local calibration will be needed to give the local tide constituents. These may be determined from an analysis of a period of water level observations, made ideally over at least a year. Methods of analysis are given in Admiralty Tidal Handbook No 1 and No 3 and by Doodson & Warburg (Ref 27). Methods of water level measurement and suitable gauges are discussed in the British Standard, BS 6349 (Ref 20) and the UNESCO manual (Ref 71). The history and present operation of the UK Tide Gauge Network has been reported by Rae (Ref 62) who describes the two main gauge types, the stilling well and bubbler gauges, and the data collection and processing methods, with appropriate references. Further information is also given in a brief article by Alcock (Ref 4).

3.1.3 Surges

Surges are sudden increases in local water level, not due to astronomical tides, or to seiches or tsunamis. They may be caused by extreme meteorological events. Such surges are often associated with large atmospheric depressions, and hence with strong winds. The water level rise will be due to both a static barometric pressure effect and to dynamic wind effects. The surge may be generated externally or internally to the sea area adjacent to the sea wall. A contribution to the surge level may also be made by wave set-up. This set-up will occur when a strong wind blowing on shore tends to pile the water up against the coast, and may be an important factor in the static water level at a sea wall. It is suggested by James (Ref 47) that shoreline set-up can reach around a fifth of the significant wave height offshore.

The difference in level at any time between that measured and that predicted from the tide tables, or normal tidal theory, is known as the surge residual. In areas of shallow water and/or confined areas, such surges may be significantly magnified. Surges in the North Sea in particular may lead to dramatic changes in sea level as occurred in 1953. Descriptions of the 1953 surge, and its effects at sea and on the coastline of the UK are given by Summers (Ref 69), Grieve (Ref 36) and Pollard (Ref 59). The course of the storm, its effects and the progress of the consequent surge are described. Grieve details the breaches and flooding that occurred in Essex. Pollard's account concentrates principally on the effects on communities in Norfolk and Suffolk, whilst Summers gives a more general account of the floods and their effects.

A chart of estimated storm surge heights has been presented by Crease (Ref 24) and recently updated by Alcock & Flather (Ref 7). It shows 1 in 50 year surge residuals around the UK based on observations and the results of a numerical model.

The calculation of surge levels, together with the contributions of other variations in mean sea level, is discussed in a later sub-section covering secular variations in static water level. We must, however, first consider the causes, and likely order, of changes in mean sea level.

3.1.4 Mean sea level

Variations in mean sea level must be considered in relation to the time period concerned. Over a year, mean sea level will vary due to meteorological changes (from wind stress and air pressure variations) and oceanic changes (from sea water density variations and from changes in ocean currents). In UK waters, meteorological and density (sometimes known as steric) changes are the main contributors to a seasonal cycle of mean sea level, and are of approximately equal importance. Meteorological factors produce a peak in winter, whilst steric changes have maximum effect in later summer. It is generally felt that the steric component is regular, and therefore predictable, year to year. Meteorological effects dominate the short-term variability in mean sea level around the UK.

Over longer periods, of the order of decades, sea level records show a secular or long-term change which can be ascribed to a combination of climatic and/or geological effects. Such long-term trends appear to be due to:

(a) global temperature changes leading to an increase, or decrease, in total water volume;
(b) changes in long-term mean atmospheric pressure;
(c) vertical movements of land masses, such as the post glacial uplift.

For the global sea level rise, data for the period 1930-1980 suggests a rise of 2.3 ± 0.1mm/year, Barnett (Ref 9). There is, however, evidence that rates of sea level rise may have increased considerably in recent years. Studies supported by USA environmental agencies, and the World Meteorological Society, have suggested rises in mean sea level of between 0.5 to 3.5m by AD2100, with a most likely estimate of 1.4-2.2m (Ref 6).

Weggel (Ref 77) discusses the potential effects on sea defences, particularly beach nourishment schemes, of such global increases in sea level, and cites four projections of mean sea-level rise by Hoffman (1984).

Trends (b) and (c) are more local. In the UK it may be noted that the south-east is sinking, whilst places in the north-west are rising. Pugh & Faull (Ref 60) discuss trends for various locations around the UK, estimating overall rises of around 0-200mm/century. It should however be noted that the data sets available are relatively short, and such predictions should be treated with caution.

3.1.5 Predictions of sea level

The prediction of such sea level variations will generally depend upon the availability of a long data set of measured water levels. Among the statistical methods used to predict sea level maxima is that of Jenkinson (1955), known as the general extreme value method. This method has been used by Suthons (1963) and by Lennon (1963) to deduce trends for annual maxima at various locations around the British Isles. More recently Blackman & Graff (Ref 13) and Graff (Ref 35) have used the method of annual extreme level analysis to consider secular variations at various UK ports. For many sites, no significant trends in annual maxima were identified. It was concluded that, where there was a trend, it reflected the secular change in local mean sea level, rather than changes in tides or surge behaviour, and this is supported by Pugh & Faull (Ref 60).

Ackers & Ruxton (Ref 3) have considered the prediction of possible surge levels by computing the surge residual at each high water, using the resulting distribution to make predictions of extreme events. The method uses much more of the available data than does the annual extreme method as approximately two data points are generated per day. The method of high water surge residuals treats the surge and the predicted astronomical events as wholly independent. In practice this may be somewhat conservative, as in general, large surge residuals are very seldom recorded at high water on larger tides, as shown by Pugh & Vassie (Ref 61).

A more sophisticated method is that of joint probabilities of tide level and surge residual. Pugh & Vassie discuss the joint probability method for the computation of extreme sea levels, and contrast it with the method of annual maxima. The joint probability method separates the observed sea levels, recorded at frequent time intervals (say hourly), into components of mean sea level, astronomical tide level (given by the predicted tide) and surge residual. It makes use of all available data, unlike either the method of annual maxima or the method of high water surge residuals. The joint probability method is, however, sensitive to the time accuracy of the tidal recording. A relatively small timing error in the record of observed levels can give the impression of relatively large surge residuals. Further, hourly data must be available and this is generally so only at standard tide gauge locations. Estimates of the extreme levels due to combinations of tide and surge can be adjusted for any identifiable trends in mean sea level.

In its basic form, the joint probability method assumes independence of tide and surge, and this will be the most conservative case. Dependent events may however sometimes be considered. The case of Southend is discussed by Pugh & Vassie and by Pugh & Faull. At Southend the level having a "return period" of 100 years would be over-estimated by 0.5m if full independence were assumed. Walden et al (Ref 74) also use the joint probability method for Portsmouth and Southampton. They found that the assumption of independence between surge residual and astronomical tide was indeed justified for levels at Portsmouth, but not for Southampton, where some variation in surge residual over the tidal cycle is apparent, due to shallow water effects. Walden et al (Ref 75) suggest an adaptation of the joint probability method using a modification proposed by Tayfun (1979) in which the surge is represented as a single event with an intensity dependent on its amplitude and duration. It is noted by Alcock (Ref 5) that the unmodified joint probability approach gives the more conservative estimates of combined surge and tide level, especially where tide/surge interaction has not been allowed for, but that this may be preferable for design purposes.

Alcock (Ref 5) also discusses the difficulties of estimating extreme levels at locations other than those for which tidal records are held. It is noted that a simple ratio may give rise to errors, because it is unstable in areas of shallow water and near amphidromic points. Alcock & Flather (Ref 7) discuss the use of alternative scaling factors to estimate extreme levels at locations where little, or no, tide and surge data is available.

3.1.6 Seiches

Seiches are defined in the SPM (Ref 23) as long period standing waves often persisting after the initial disturbance has passed. The British Standard (Ref 20), however, defines them as oscillations of sea level caused by the passage of an intense depression, or squall line. Seiches as defined by the SPM generally occur in enclosed, or partially enclosed, bodies of water such as basins or harbours. They seldom affect sea wall design, but may be particularly important in harbour design or operations.

3.1.7 Tsunamis

Tsunamis are waves of very long period excited by seismic activity. They are very infrequent around the UK, but may be of great importance elsewhere as they are potentially extremely dangerous. It may be noted that the 1755 Lisbon earthquake led to levels rising at UK by around 3m in a few minutes. Much work on the effects of tsunamis has been published in Japan and the USA, and reference should be made in the first instance to reports published by WES, and to papers in the annual publication, Coastal Engineering in Japan.

3.2 Wave Conditions

3.2.1 Wave prediction

Before embarking on the design of a sea wall or similar coastal engineering structure, it is essential to specify both a set of design water level and incident wave conditions. In order to specify the nearshore wave conditions, it is necessary first to consider wave action in deep water, where the effects of the sea bed on wave propagation are small in comparison with those of the wind. In sections 3.2.2 and 3.2.3 some of the literature describing wave forecasting and hindcasting models will be reviewed. Nearshore wave transformations are then covered in sections 3.2.4 onwards.

The techniques used for both forecasting and hindcasting models are the same, the main difference lies in the information available on the wind conditions. Wave forecasting refers to the calculation of wave conditions at a particular site using forecast wind conditions, whereas wave hindcasting uses recorded wind conditions to calculate wave conditions. The science of wave prediction is developing rapidly, and recent work has produced some very sophisticated numerical modelling techniques. However, a very high degree of certainty of the offshore wave conditions may not be as important to the designer of coastal structures as calculations of the shallow water effects on the incoming waves. A wide range of wave prediction techniques are therefore in common use, varying from complex numerical models that attempt to simulate as many of the physical processes as possible, to simple charts or graphs relating wind speed, fetch length, and possibly water depth, to resulting wave condition. This section of the review has therefore included many methods that, whilst theoretically superseded by more recent techniques, may be in relatively common use for preliminary estimates of wave conditions.

3.2.2 Forecasting/hindcasting models

Waves in deep water may be considered to be of two different types, locally generated wind waves, and distantly generated swell waves. The height (and period) of locally generated waves depends on the wind speed and duration, the effective fetch length, and the average depth of water over that fetch. The height of swell waves depends on all these parameters, as measured at the location where the waves are generated, and at intermediate points on route to the site of interest.

The simplest of the mathematical models used for predicting wind wave conditions assumes that the waves being considered are due entirely to a wind blowing at constant speed and direction for a given duration. The SPM (Ref 23) outlines the work of Pierson, Neumann & James (1955) who introduced this type of wave prediction based on empirical data. This work was extended by Inoue (1966, 1967) who used a differential equation for wave growth based on Miles-Phillips theory. The SPM gives an outline of the original work of Miles (1957) and Phillips (1957) on the mechanism of wind wave generation. Kinsman (Ref 50) also presents much of the original work in wave prediction.

Understanding of the mechanism of wave generation by wind was further advanced by Hasselmann (Ref 39), who suggested a method by which energy was transferred between different frequencies in the wave field. Hasselmann et al (Refs 40, 41) also showed that the energy spectrum, as a function of frequency, of a growing wind sea could be well approximated by a single expression, now known as the JONSWAP formula. The coefficients of this expression are either fitted to an observed spectrum, or are calculated as functions of a dimensionless fetch length. The use of the JONSWAP formula, together with Seymour's (Ref 64) method for restricted fetch, is reported by Hawkes (Ref 42) and Brampton & Southgate (Ref 18). Hawkes describes a numerical procedure, HINDWAVE, developed to hindcast wave conditions. The method may be used to estimate a directionally dependent wave climate distribution, or to extend an existing short record of wave conditions to a longer period. The procedure uses Seymour's method for dealing with restricted fetches, together with the JONSWAP wave prediction equations, to calculate wave conditions from given wind speeds and directions.

The latest edition of the SPM reports that the theory of Hasselmann et al has been used by Resio & Vincent (1977) and Resio (1981) in the WES numerical wave hindcasting model, which is outlined by Resio & Tracy (Ref 63). A model based on these principles was used for the wave climatology study at Sines, see Mynett, de Voogt & Schmeltz (Ref 52).

For shallow water depths, Camfield (Ref 21) has suggested methods for wave estimation where friction levels are high. It is assumed that high friction values can be accounted for by adjustment of the fetch lengths. Camfield's method has subsequently been included in the SPM for calculation of wave propagation over flooded, vegetated land.

The simpler wave prediction models cited above give good results when the wind speed and direction is near constant. However, in situations where the wind speed or direction changes markedly, as during the build up of a storm, a more sophisticated model may be necessary. In these situations, some of the wind wave energy may be transferred to swell waves, and vice versa, and so a more complicated mathematical model representing both wind and swell waves may be required.

One such model is the BRISTWAVE model discussed by Owen (Ref 53), which has been used to hindcast wave conditions in the Severn Estuary and Bristol Channel. This model uses locally obtained wind data and predicts wave conditions for certain selected storms. The wind-wave spectrum along each fetch direction is calculated using the JONSWAP formula, for growing waves, and the Pierson-Moskowitz formula (Ref 58), for fully developed waves. The swell wave spectrum is generated using various formulae whenever the wind waves decrease. The model, which was developed for a restricted fetch area, uses Seymour's method and combines the calculated spectra for each fetch direction into wave height, period and direction at a selected site. Due to its complexity, this model is normally only used for predicting waves generated by particular storms.

Numerical models covering very much larger areas have been developed for predicting wave conditions at many locations due to both wind and swell waves. In particular the NORSWAM model (Ref 76), which covers the northern North Sea, uses a hybrid method for predicting wave conditions from measured wind data for many severe storms in that area. The hybrid method comprises a finite difference solution to a parametrical form of the energy balance equations for the wind wave spectrum, see Hasselmann et al (Refs 40, 41), and a characteristic ray method to deal with the swell wave component. The wave heights and periods are predicted on a regular grid covering the whole area of interest.

More recently, Golding, Ephraums and Francis have described the wave prediction model used by the Meteorological Office to forecast wave conditions offshore of the UK coastline (Refs 29, 30, 33). The model uses a finite difference method to solve a parametric form of the energy balance equations for the wind wave spectrum. The swell wave regime is described using a discrete spectral model. The wind input to this model is provided by the Meteorological Office numerical atmosphere model, described by Gadd (Ref 31).

Several authors have produced reviews of the methods available for wave prediction. Amongst these are Battjes, Cardonne & Ross, and Earle (Refs 10, 22, 28). Detailed reviews of methods for predicting waves for coastal structures are given by Alcock (Ref 5) and Smallman (Ref 65).

3.2.3 Forecasting curves

Whilst it is now more usual for a mathematical model to be used for wave prediction, such a model may be viewed as being too elaborate for a feasibility study or a preliminary design. In such instances an initial assessment of the magnitude of wave conditions may be made by use of simplified forecasting curves.

Such curves have been produced for UK offshore conditions by Darbyshire & Draper (Ref 25). The SPM also gives wave forecasting curves. These are based on the work of Sverdrup & Munk (1947), revised by Bretschneider (1952, 1958), resulting in the SMB method. It should be noted that these curves have been further revised in the current version of the SPM to include the field data of Mitsuayasu (1968) and Hasselmann et al (Ref 40). Wave prediction curves may also be obtained using the JONSWAP formula (Ref 40). Whilst this does not necessarily give a better estimate of wave height and period for all wind conditions, use of the JONSWAP method is nevertheless rapidly replacing the SMB method, mainly because the JONSWAP formula gives a wave spectrum, and not just a simple wave height and period as the SMB method does.

3.2.4 Nearshore wave transformations

In sections 3.2.2 and 3, the effects of the sea bed on the formation, propagation and character of the waves were not considered. These effects are important in the nearshore region where they modify the incident deep water wave conditions before they reach the shore. In the following sections some of the literature available on the various processes altering waves as they approach the shore, through water of decreasing depth, is reviewed. General discussions of the modification of waves as they approach the shoreline are given by Bowers, Goda, and Holmes (Refs 14, 32, 43).

3.2.5 Wave refraction and shoaling

Wave refraction and shoaling are two mechanisms by which the directional spectrum of the waves approaching the shoreline may be altered, without the total wave energy flux or power being decreased. Shoaling may be explained as follows. As waves travel into water of decreasing depth they slow down. If none of the wave energy is dissipated, the energy flux must remain constant, despite the deceleration. As a consequence the energy density of the waves, and hence the wave height, increases. Shoaling occurs when the waves approach the coastline either at normal or oblique incidence. As waves approach a coastline obliquely the decreasing water depth will cause the waves to change direction, turning the wave crests, to become more parallel to the beach contours. This process is known as refraction. Fuller explanations of the mechanisms of refraction and shoaling are given in the SPM, which gives a graphical method for calculating refraction effects, and by Wiegel (Ref 78), Brampton & Southgate (Ref 18) and BS 6349 (Ref 20).

In recent years, a number of different approaches to the mathematical modelling of refraction and shoaling have been investigated. Abernethy & Gilbert (Ref 2) give a very good explanation of the mechanism of refraction of wave spectra. In particular, they establish a ray method for computation of the process of wave refraction which also includes the effects of shoaling. The method is characterised by the following features: the sea bed is represented as depth values on a triangular grid, the wave rays (lines orthogonal to the wave crests) are projected in a reverse direction, running seaward from the point where the inshore conditions are required; and the input deep water waves are specified by a two dimensional energy spectrum in frequency and direction. This back-tracking approach also avoids the problem of caustics arising where rays cross in forward-tracking refraction models.

A ray method intended for computation of the response of harbours to short period waves, but which can also be used to determine nearshore wave conditions, has been developed by Southgate (Ref 67). This is a forward tracking ray model (rays travel in the direction of wave propagation) which, in addition to including the effects of refraction and shoaling, also models wave diffraction around breakwaters and reflection from harbour boundaries. Results obtained from this mathematical model were compared with those from a physical model by Bowers & Southgate (Ref 16), and the two models were found to be in reasonable agreement. Examples of the use of ray models are given by Bowers (Ref 15), who also summarises a number of methods for the transfer of predicted wave conditions in deep water to a point of interest in shallow water.

Hubertz (Ref 44) gives an outline of the forward tracking ray models used by CERC to calculate the effects of wave refraction. In a later paper, Hubertz (Ref 45) also presents a brief description of a mathematical model used by CERC for calculating the refraction of both long and short waves.

In addition to ray methods, several authors have used finite difference and finite element techniques for modelling the effects of refraction. For example, Abbott et al (Ref 1) present a finite difference technique for modelling short waves in shallow water. Berkhoff (Ref 11) also proposes a mathematical model of the effects of refraction and diffraction which uses a finite difference method. Brampton et al (Ref 17) describe a finite difference method for wave refraction which, in addition to modelling refraction effects, also includes the effects of shoaling, viscous friction at the sea bed and breaking. A finite element approach to modelling refraction and diffraction is detailed by Bettess & Bettess (Ref 12). Solutions obtained from this model are used by Southgate (Ref 67) for comparison with his ray model, which was reviewed earlier in this section.

In addition to depth refraction, it is also possible for refraction to be caused by the presence of strong currents. A discussion of the interaction of waves and currents may be found in Peregrine & Jonsson (Ref 56) and Jonsson (Ref 48). The application of the theory of wave-current interaction to computational refraction models is discussed by several authors, amongst these are Jonsson, Christoffersen & Skovgaard, and Southgate (Refs 49 and 68).

3.2.6 Wave diffraction

Wave diffraction is the mechanism by which wave energy is transferred laterally along a wave crest. The term is often used for two distinct effects, that is external and internal diffraction. External diffraction occurs where the water surface is pierced by an obstacle such as a breakwater, and diffraction is the mechanism by which energy is propagated into the lee of the breakwater. Internal diffraction occurs when the sea bed has a rapid change in level or slope, when again the wave energy will travel in a different direction to that of the incident wave train.

In the design stages of a sea wall, diffraction will seldom play a major part. However, where a sea wall is partially in the 'shadow' of a breakwater or headland, diffraction effects may be of some significance. Reference may be made to Wiegel (Ref 78), or the SPM, for preliminary calculations of diffraction effects. Several of the mathematical models in the previous section, for example those by Berkhoff or Southgate, also include diffraction as well as the effects of refraction and shoaling (Refs 11, 67).

3.2.7 Bed friction

Bed friction may be a significant factor in reducing nearshore wave heights. In modelling the energy dissipation due to the effects of bed friction, most authors refer to the work of Bretschneider & Reid (Ref 19). They present a formula which may be used to determine the change in wave length due to bottom friction. Both the SPM and BS 6349 give a series of curves, based on the work of Bretschneider & Reid, which enables the wave height reduction factor due to bed friction to be calculated. The theory of Bretschneider & Reid is evaluated by Grosskopf (Ref 37) by comparison with field measurements. He concludes that Bretschneider & Reid's method is in good agreement with measured data, particularly in cases where the incident wave spectrum is narrow and single peaked.

The theory of Bretschneider & Reid is for viscous dissipation of energy at the sea bed for a monochromatic wave. The process is non-linear and therefore the effect of bottom friction on the whole of a wave spectrum cannot be determined by a simple superposition of the effect on the component waves. Brampton, Gilbert & Southgate (Ref 17) have developed a technique for modelling bottom frictional dissipation of a wave spectrum which is used in their finite difference model of wave refraction described in section 3.2.5

3.2.8 Wave breaking

Wave breaking is a highly complex process, and no satisfactory theory has yet been developed to fully account for the physical processes involved. Various authors have proposed procedures for obtaining reasonable approximations to the energy losses involved in breaking waves. The SPM cites the work of several authors, and gives a number of formulae for calculating the limiting steepness at which waves begin to break. Holmes (Ref 43) gives an outline of some of the methods available for calculating breaking wave height and location. Brampton, Gilbert & Southgate (Ref 17) suggest a simple procedure to approximate the energy losses due to breaking waves. It is based on the assumption that the waves have a Rayleigh distribution, and that they are assumed to break when the ratio of water depth to wave height exceeds a critical value. The energy losses are distributed amongst all the period and directional components of the wave spectrum. More recently a computational scheme for modelling breaking waves has been proposed by Dold & Peregrine (Ref 26), and a useful review of work on the processes of breaking waves has been presented by Peregrine (Ref 57).

3.3 Joint probability analysis

The hydraulic performance of any coastal defence scheme depends critically upon the two major variables considered above, water level and wave conditions. In the past, the design level of a sea wall has often been calculated by allowing a standard freeboard for wave action above the predicted extreme water level. Typical values of this freeboard of around 0.4-1.0m have been quoted by Alcock (Ref 5). Recently, it has been recognised that there may arise combinations of lower than extreme water levels, and more severe wave conditions, that together yield more severe overtopping, and hence possible flooding. It has also been recognised that it may be unreasonable to expect a sea wall to prevent overtopping fully, but that an acceptable risk of overtopping may result from the use of very low values of a mean overtopping discharge. It will be shown in Chapter 5 that the mean overtopping discharge, $\bar{Q}$, for a given sea wall may be described as a function of the freeboard, Fc, (crest level less water level) and the wave height, Hs, and period, Tz, and the incident wave direction, β.

Using physical model tests, Owen (Ref 54) has shown that it is a relatively straightforward task to design the profile of a sea wall for a given set of wave conditions such that the mean overtopping discharge does not exceed a set value. Such a deterministic philosophy does not however allow the easy assessment of the overall performance of the sea wall during its design lifetime. The calculation of mean overtopping discharges under a wide range of conditions may be necessary for cost/benefit studies of the proposed scheme (Ref 55). With the increasing use of such assessment methods to determine the optimum returns, see Thorn & Roberts (Ref 70) and Hardy (Ref 38), it has become more important to estimate the overall performance of a coastal defence scheme during its design lifetime. This generally implies that the overtopping performance, and hence the degree of possible flooding should be estimated for the wide range of circumstances that might be expected within the life of the structure.

It has been shown in section 3.1 that a particular water level may be divided into two components of astronomical tide and surge residual. A wave condition, given by wave height, period and direction may often be determined from a knowledge of wind speeds, duration and direction by the use of the types of mathematical prediction or hindcasting models considered in section 3.2. It should however be noted that any particular range of water levels may in turn be due to a large number of different combinations of tide level and surge residual. Each such combination will be linked to a different probability of occurrence. To assess the probability of occurrence of water levels within a given range, the joint probability density function of tide level and surge residual must be calculated. A number of methods for this have been outlined previously. Similarly an estimate must be made of the joint probabilities of different combinations of water level and wave conditions. This is often presented as a simple task of combining the probability density functions of water levels with those of wave conditions. From the resulting multi-dimensional array of input parameters and attendant probabilities (sometimes called the probability mountain), the probabilities of any chosen level of overtopping may be calculated. Bakker & Vrijling provide a similar example using run-up level rather than overtopping discharge (Ref 8).

However, the task is anything but simple, principally due to the lack of data from which to derive the source probability density functions. Where data is particularly limited, the phenomenon considered may be assumed to have a standard probability distribution. Often the Gaussian distribution is used (Ref 8). This however, presupposes that the phenomenon considered is independent of other phenomena. The resolution of this requires a detailed study of the statistics of the various phenomena involved. This difficulty is one of many complications that are not dealt with in any depth by the literature reviewed. Summaries or explanations of joint probability methods for overtopping discharge are generally confined to site specific and restricted technical reports. Much of the published literature deals almost exclusively with either breakwaters or sand dunes, Graaff, Vellinga and Visser (Ref 34, 72, 73). However, Alcock (Ref 5) has gathered together some of the methods used in the UK and Holland for various site specific studies. In general, a joint probability distribution of wind speed, and direction, and water level is evaluated. Using a manageable number of water level class intervals, the corresponding values of firstly, wave height and period, and secondly overtopping discharge, are then computed.

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4 COASTAL PROCESSES AND GEOTECHNICAL STABILITY

4.1 General

The stability of a sea wall, and indeed its hydraulic performance, will be critically affected both by any nett movement of beach material in front of the structure, and of the land mass immediately behind it. In many instances, a sea wall will only have become necessary as a result of continuing beach erosion, and/or cliff or dune instability. Where a new sea wall or the replacement or refurbishment of an existing structure is proposed, the design of the sea wall must make allowance for the extent and frequency of such movement. Design problems arising from the movement of the beach material in front of the structure are unique to coastal engineering, and are discussed in later sections of this chapter.

Where the land mass behind the sea wall is itself unstable, the sea wall structure may be designed to act as a retaining wall. Many of the design problems for such a wall will be typical of retaining walls and similar structures, and will not be covered in great depth in this review. An area of geotechnical stability that is, however, unique to coastal engineering is that of coastal cliffs. Many of the techniques used for the investigation and stabilisation of cliffs and slopes are typical of those used in the design of cuttings and embankments. It is, therefore, appropriate to discuss some example problems here, although this report does not attempt to present a comprehensive review of the problems of cliff and slope stabilisation.

4.2 Beach movement

The level of a beach, where present in front of a sea wall, will have a critical effect on the wave conditions that reach that wall. The present and possible future beach levels at the wall also significantly affect the design of the toe of any new or refurbished structure. The subjects of beach dynamics, and sediment transport under waves and currents, are highly complex, and are generally beyond the scope of this review. It is however particularly important that the designer of a coastal structure has a clear understanding of such processes, because, not only is a good sand or shingle beach a most effective dissipator of wave energy, but it is probable that it is often the most economic sea defence structure available.

The interaction between a wave and a structure such as a sea wall is a complex phenomenon involving a wide range of coastal processes. The wave itself undergoes changes to its profile and behaviour as it approaches the shore. In order to analyse the hydrodynamics of the wave/structure interaction, it becomes necessary to determine the type of wave acting at the structure. The shoaling of a wave is usually accompanied by refraction, which determines the angle between wall and wave. In addition, as the wave proceeds over the beach it interacts further with the beach profile (itself wave and tide generated) and may break, forming a breaking wave, and giving rise to abrupt pressure fluctuations and turbulent flow.

Once the wave reaches the structure it may be reflected to some extent. This will depend upon properties of both the wave and the structure. Interference between the reflected and incident waves may then occur. Generally, longshore and onshore/offshore currents will arise so that the interaction between the resultant velocity field and the beach sediment will occur in a particularly complex environment.

Changes to a beach may be conveniently divided into the longer and shorter terms. The long term might be considered as a period of the order of the design life of a sea wall, say 20-100 years. The shorter term might be around 1-10 years.

The first assessment of a beach should identify the source of the beach material, the mechanism of beach development, the extent of recent historical changes, and hence estimate the stability of the beach in the longer term. Many papers and articles describing the origin and development of beaches have been presented, usually from a geographical view-point. Many of these have been summarised, or at least referenced, by Bird (Ref 2) and/or King (Ref 14). A summary of shoreline changes around the UK is also provided by Bird & May (Ref 3).

Considering a long stretch of coastline, Clayton, McCave & Vincent (Ref 10) consider the establishment of an overall sand budget, and the implications on the coastal management strategy. They take the example of East Anglia, and estimate present rates of beach material movement, and coastal cliff erosion. They discuss the present and historical state of sandy beaches at many locations around the East Anglian coast. Carr (Ref 8) considers shingle beaches around the UK. He discusses the structure, stability and performance of such beaches, citing nearly fifty references.

It may be noted, however, that relatively few beaches in the UK are in equilibrium in their natural state. The construction of an artificial structure into, or at the back of, a beach and dune system, will further alter the position, often leading to a drop in local beach levels. As part of the design process for the sea wall, the engineer will therefore seek to estimate the possible severity of beach level changes at the structure, over its design life. The most critical such changes might be expected to occur early in the structure life, but in many instances beach erosion appears to have continued throughout the life of the structure. For full understanding of the likely effects of a particular coastal structure, the beach at and either side of the proposed (or existing) structure must be studied carefully. From predictions of local wave conditions, an understanding of beach processes, and a knowledge of the beach characteristics, some estimate may be made of the likely effect of any new or revised structure on beach slopes and levels.

The complex relationships between wave climate, beach form, littoral transport and on/offshore movement have been considered by many authors. Bijker & van de Graff (Ref 1), the Shore Protection Manual (Ref 11), Muir Wood & Fleming (Ref 20), and others including papers edited by Hails & Carr (1975), and Stanley & Swift (1976), discuss general formulae for the estimation of longshore transport. These do not necessarily take any account of the presence of a sea wall. Some of the effects of a sea wall on a beach have however been studied by authors including Russell & Inglis (Ref 24), and Ozasa & Brampton (Refs 22, 23). Ozasa & Brampton discuss the single contour mathematical model of plan shape, mentioned by Price, Tomlinson & Willis (1972), and a two line method described by Bakker (1968). Having concluded that the latter may be particularly difficult to use, Ozasa & Brampton describe a single line model which they calibrate against the results of a physical model. In general, good agreement was obtained.

However, Bijker & van de Graff (Ref 1) consider a number of coastal structures, and their effects on beaches. They conclude that "the construction of a sea wall to prevent further erosion, should be avoided as long as possible"..."usually the depth before the sea wall increases, wave attack increases and stability problems of the wall occur. Moreover, the beach itself will disappear". Based on experience where sand is relatively plentiful, Bijker & van de Graff, however, also conclude that "most types of erosion problems can be solved by proper sand suppletion (sic)".

The use of beach replenishment instead of, or as well as, the traditional "hard" defence has received increasing attention recently, although relatively little design guidance is available. In 1976, Newman (Ref 21) presented a comprehensive review of the use of beach nourishment. Details of schemes in the UK at Bournemouth and Portobello, and in Holland and Goeree, are given, as is a table summarising 24 such schemes world wide. In his paper, Newman concentrates primarily on sandy beaches. Considerable additional detail of the renourishment scheme at Bournemouth is given by Wilmington (Ref 27), together with an assessment of that scheme six years after completion. The calculation of sand sizes and volumes for renourishment schemes is discussed in detail in the SPM (Ref 11), and summarised by Muir Wood & Fleming (Ref 20). Both cite original work by James (1974, 1975).

Recharge of shingle beaches is a regular activity in some areas. At some sites in Kent and Sussex it has been an accepted method of shoreline protection for around 30 years. Shingle recharge at Pett, Walland and Sheerness is discussed by Foxley & Shave (Ref 12), and also by Thorn & Roberts (Ref 26). The monitoring of beach levels and consequent data analysis is described, as are each of the schemes. Certain of the contractual aspects are also highlighted.

4.3 Geotechnical stability

In considering the geotechnical stability of a sea wall, two major structural forms predominate. Many existing sea walls in the UK, often those protecting agricultural land when they were first built, are earth embankments constructed of locally available material, principally clay. The material available for repair or reconstruction of such sea walls is again often clay. Thorn & Roberts (Ref 26) describe modes of failure of clay walls. They summarise some of the methods of slip circle and wedge analysis used to estimate the stability of such structures. They discuss the use of filters for drainage, and weighting berms to stabilise possible slips. The design of clay embankments in general is covered in great detail elsewhere, in standard textbooks such as Terzaghi & Peck (Ref 25); in research papers by Building Research Establishment (Ref 15) and in the British Standard codes of practice for earthworks, BS 6031, 1981, and for site investigation, BS 5930, 1981 (Refs 5, 6).

The other principal form for which geotechnical considerations may be of significance is the solid, vertical (or near) faced wall. This may be designed to serve, in part, as a retaining wall for the material landward. The British Standard on maritime structures, BS 6349 (Ref 7) devotes a complete section to geotechnical considerations of such structures. It gives guidance on site investigations needed for design, and on stability analysis methods. Vertical face, sheet piled, structures are discussed in some detail. A number of design cases for silty clays, sand silts, sand and gravels are considered. In contrast, mass concrete or blockwork structures receive virtually no attention. Moffatt & Nicholl (Ref 18) discuss the properties and use of various soils (gravel, sand, silt and clay) in coastal structures. For further details they refer to Callender and Eckert (in preparation 1983).

It should be noted that data on conditions and material properties at the site will be needed before any calculations of geotechnical stability can be made. Thorn & Roberts give details of site investigation techniques and a comprehensive table of instrumentation, and their use and limitations, for the field measurement of geotechnical properties (Ref 26). Much of this subject is also covered by BS 5930 (Ref 5).

4.4 Cliff and slope stabilisation

The land at or behind a sea wall may be unstable, or potentially so, for various reasons. In some instances the instability of the cliff or slope may endanger an existing or potential coast protection scheme. A number of authors have identified typical mechanisms of slope failure, suitable analysis methods, and possible slope stabilisation strategies. The slopes themselves may be divided simply into friable cliffs and soft slopes, a distinction used by Thorn & Roberts (Ref 26). The types of instabilities commonly seen have been listed by Hutchinson (Ref 13), and by Muir Wood (Ref 19), in turn based upon work by Skempton & Hutchinson (1969). These failure types may be summarised as:- falls (including block subsidence, toppling failures, and some steep translational slides); rotational slides (circular, non-circular and shallow); compound slides; and surface flows (including mud and chalk flows, and mud slides).

The primary reason for cliff and slope failures at the coast is the action of the sea in cutting away the toe support of the cliff or slope. Other contributing factors given by Hutchinson (Ref 13) may be:-

(a) changes in external loads on the toe, storm surges and/or waves;
(b) seasonal variations in the pore water pressures within the cliff;
(c) rising pore pressures associated with clay swelling, in turn due to recent undrained failure leading to unloading of the toe;

(d) undrained (rapid) loading on the upper parts of the slope, perhaps due to a cliff or slope failure further up the slope;

(e) transient pore pressure fluctuations, usually transmitted by a basal aquifer.

The different zones of the slope have been discussed by Muir Wood (Ref 19). Working seawards, he identified the high cliff, the undercliff, foreshore, inshore and offshore areas. The high cliff is in general the zone removed from slips and failures, although it may exhibit slight structural deformations resulting from the movements in the next zone, the undercliff. This is the zone of most obvious movement. It is often filled with the remains of previous slips. The foreshore may be defined as including the intertidal zone and the extent of wave run-up. In this zone the material from slope failures becomes beach material. The inshore area is that of breaking waves where beach material is transported along the shore, and where most of the wave energy is dissipated. The offshore zone is generally seaward of the breaker zone.

The first task when faced with a possible or obvious instability, is to identify the likely causes and mechanisms of anticipated or actual failure. Geological and soil maps, and careful land surveys, together with aerial photographs, will contribute to the first stage of the investigation. Hutchinson (Ref 13) discusses a wide range of such studies, citing examples given in Hutchinson (1965 a, b and c).

The next stage of a thorough investigation will involve the sinking of trial pits and/or driving of boreholes. Such a sub-surface test programme is discussed by Thorn & Roberts, Hutchinson, Muir Wood, and in BS 1377 and BS 5930 (Refs 4, 5, 13, 19, 26). The use of various types of inclinometers and piezometers is covered by Thorn & Roberts, and by Muir Wood. The latter also discusses the importance of drilling boreholes and making measurements to test an hypothesis. Such boreholes may be intended to explore assumptions of stratigraphy, the properties of the ground materials, and the levels of the water tables.

The methods of stability analysis commonly used are those of simple vertical slices, see Terzaghi & Peck (1967) and Janbu (1954), or wedge analysis or circular or other analyses based on the methods proposed by Morgenstern & Price (1965, 1967). These are discussed further in standard textbooks covering geotechnical stability.

Once the likely mechanism of failure has been identified, the remedial measures may be designed.

These will usually involve the grading back of the slope and the provision of drainage. In some instances toe weighting may be required. The possible choice of stabilisation strategies is discussed by Hutchinson (Ref 13), and McGown, Roberts & Woodrow (Ref 16). A number of methods of stabilising clay cliffs are covered by Thorn & Roberts, McGown, Roberts & Woodrow, and Mockridge (Ref 18), all of whom discuss in detail the types of drainage systems that may be used.

It has been suggested that no two coastal landslips are the same. This review has considered accounts of such failures, actual and anticipated, by Chandler & Hutchinson, McGown, Roberts & Woodrow, and Mockridge (Refs 9, 16, 17). The interested reader is referred also to Hutchinson (1965 a, b and c) and to the other references covered by Hutchinson (Ref 13) and Muir Wood (Ref 20).

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5 HYDRAULIC ASPECTS OF DESIGN

5.1 Identification of primary forces

At a sea wall, the primary cause of hydraulic effects is wave action. The flow of water under tidal action seldom induces velocities or forces of significance to the hydraulic design of the sea wall, although tidal effects may be of significance when acting together with wave action, such as in the transport of wave-suspended sediment. In contrast, wave action will lead directly to the generation of a number of forces, and flow conditions, requiring calculation by the designer. The importance of each of these parameters in the design process will depend upon the structure type, the local conditions, and the incident wave climate. The primary parameters for which values may require calculation may be summarised:

(a) wave run-up levels, on the outer surface;
(b) overtopping discharges;
(c) impact pressures and forces on large elements;
(d) effects of drag and impact forces on small elements, particularly the onset and severity of any movements of the elements.

It should be noted that this list, by its very nature, cannot be exhaustive.

In many design situations, only some of these parameters will be calculated. For example, at a rubble sea wall, the wave impact pressures on the rubble slope will be extremely variable, and cannot presently be determined. It may, however, be sufficient for the designer to determine the onset and the severity of wave-induced movement of the rubble armour and supporting layers. Similarly, the design of a mass, or reinforced, concrete sea wall with vertical, or near-vertical, front face, will be dominated by the wave impact forces, wave reflections, and in some instances overtopping.

Similarly, discrimination by researchers implies that some aspects of hydraulic performance have received very little attention. It has been found that work on a particular aspect of hydraulic performance has inevitably been biased towards the structural type for which that force, or flow condition is of greatest importance.

5.2 Run-up and overtopping

5.2.1 Introduction

Coastal structures, such as sea walls subject to wave attack, will experience wave run-up. If the structure crest is lower than the maximum run-up level reached in a particular storm, the structure will suffer overtopping. This may in turn lead to flooding and/or damage to the rearward face of the structure. In the planning and design of coastal structures, especially sea walls, wave run-up and overtopping are often the primary factors dictating the crest level of the wall. As the cross sectional area, and perhaps the cost, of the structure increases approximately with the square of the structure height, a clear understanding of the processes of wave run-up and overtopping is essential to the economic design of such structures.

Historically the designers of sea walls and breakwaters have often attempted to design the crest level of their structure high enough to prevent overtopping. This was done by calculating a maximum run-up level and setting the crest level above it. This does, however, presuppose that a maximum run-up level may be identified. With a fuller understanding of the random nature of wind waves, it has become clear that overtopping cannot always be wholly prevented, although the mean expected overtopping discharge for a design event may be reduced to negligible proportions. Furthermore, for many structures it will be uneconomic to design a crest level above the maximum expected run-up level. The design approach for sea walls has therefore been altered to one of designing for various levels of tolerable discharge under the extreme events considered.

Two major documents present much of the historical work in wave run-up and overtopping. The Shore Protection Manual (Ref 29) is produced in the USA by the Corps of Engineers Coastal Engineering Research Centre (CERC) and is revised periodically, (the latest edition at the time of writing was that published in 1984). The Shore Protection Manual (SPM), as amended and expanded by the Coastal Engineering Technical Aids (CETA), may be taken as summarising current American design practice. In Holland the Technical Advisory Committee on Protection against Inundation (TACPI) published a major review on wave run-up and overtopping in 1974 (Ref 120). This was based on a Dutch language report published in 1972. The TACPI report brings together most work on wave run-up published up to 1972. More recently, a review of methods to calculate wave run-up levels on smooth or armoured slopes under random wave action has been presented by Allsop, Franco & Hawkes (Ref 6).

Methods to calculate wave overtopping discharges under random waves have been presented in the SPM (Ref 29), and by TACPI, Goda, and Owen (Refs 120, 41, 91, 92). These four methods are discussed and compared by Douglass (Ref 32). Whilst most design methods advanced in recent years acknowledge random or irregular wave action in nature; in many instances the data used has been derived from the results of regular wave tests only. However, until superseded by measurements under random waves, such methods remain the only recourse for certain structure configurations.

5.2.2 Wave run-up levels

A wave run-up level is that maximum elevation above static water level, reached by the leading edge of a wave in running up a structure. Under storm wave attack in nature (and random waves in the laboratory) each wave will differ, and will therefore lead to a different run-up level. For any particular sea state a probability distribution, which may be of a standard form such as Rayleigh or Weibull, may be fitted to these run-up levels. Design exceedance levels may be derived from such distributions.

A number of attempts have been made to describe wave run-up from a theoretical rather than empirical standpoint. The major impediment to the evolution of a full description of the behaviour of a wave running up a slope is the highly non-linear wave breaking. The flow processes are extremely complex and highly variable, and theoretical expressions for the turbulent flow in a breaking wave are not yet available in a form useful to the designer.

Run-up on steep slopes has been analysed by many authors. Among them have been Pocklington (1921); Sainflou (1928); Isaacson (1950); Rundgren (1958); Le Mehauté, Koh & Hwang (1968); and Nagai & Takada (1972). Pocklington and Miche produced a simple expression for relative run-up of a wave in deep water:

$$\frac{R}{H} = \left(\frac{\pi}{2\alpha}\right)^{\frac{1}{2}} \tag{5.1}$$

where R is the run-up level above static water,
H is the incident wave height, and
α is the structure slope angle to the horizontal, in radians.

This expression was modified by Sainflou, who argued that the true run-up was somewhat higher. Further modifications have been proposed by Le Mehauté et al and by Nagai & Takada (Ref 89). Their expressions are given by Allsop, Franco & Hawkes (Ref 6).

Run-up on shallow slopes, principally beaches, has also been analysed by many researchers. From the theoretical standpoint, a wave arriving at the beach or structure may be assumed to have kinetic energy in the form of a general particle velocity. The energy of the wave may be determined, and related to a run-up level, using a friction factor to account for the energy losses in friction, turbulence and wave breaking. However, in practice equations given by Freeman & Le Mehauté (1964) and by Bullock (1968) have proved too complex for design purposes. Recently, the wave on a beach has been treated as a bore travelling in shallowing water by Hawkes (1982), and Peregrine & Svendsen (1978). The flow behaviour may be described by complex mathematical models. Whilst development proceeds on this approach, a suitable design method has not yet been produced.

Rather than attempt to derive theoretical expressions for wave run-up, based on an incomplete understanding of the complex hydrodynamics, many researchers have fitted empirical expressions for run-up level to the results of model tests. Many of these results have been incorporated into the prediction curves in the Shore Protection Manual. Re-analysis of many of the early test results by Stoa (Ref 117) has led to the production of revised wave run-up prediction curves (Refs 118, 119).

For sea walls having steep front faces, however, it has been noted by Günbak (Ref 45), Losada & Gimenez-Curto (Ref 74) and by Sawaragi et al (Ref 106), that the classical Hunt formula is only valid for values of the Iribarren number less than about 2.5, where the Hunt formula may be written:

$$\frac{R}{H} = Ir \qquad (5.2)$$

where the Iribarren number, Ir, may be given $Ir = \tan \alpha/(H/Lo)^{\frac{1}{2}}$.

Three expressions have been proposed by Losada (Ref 74) to cover run-up on smooth slopes over the full range of Ir:

$$
\begin{array}{lll}
0 < Ir < 2.5, & R/H = Ir & \\
2.5 < Ir < 4.0, & R/H = 2.5-(Ir - 2.5)/3.0 & (5.3) \\
4.0 < Ir, & R/H = 2.0. &
\end{array}
$$

Again, however, these have not been generalised to random waves, although they do allow a very rapid assessment of possible run-up levels. Further expressions for run-up levels on smooth slopes presented by Tautenheim (1982), Chue (1980) and Kaldenhoff & Gokcesu (1978) have been discussed by Allsop, Franco & Hawkes (Ref 6).

The behaviour of wave run-up on armoured rubble slopes has also been studied with regular waves. Many researchers including Hudson and Savage have reported such tests, and a number of different empirical formulae have been derived to fit the results. Günbak (Ref 45) presents results of run-up measurements on both smooth and armoured slopes plotted as relative run-up, R/H, against the Iribarren number. On the armoured slopes considered, run-up is compared with an expression ascribed to CERC:

$$R/H = 0.8\ Ir/(1 + 0.5\ Ir) \qquad (5.4)$$

Losada & Gimenez-Curto also consider flow conditions on armoured slopes under regular wave attack. They examine many author's measurements of run-up on slopes armoured with various types of armour units. To these results have been fitted a generalised expression for run-up on armoured slopes:

$$R/H = A(1-\exp\ (B\ Ir)) \qquad (5.5)$$

Values of the coefficients A and B are presented for various armour units. Losada & Gimenez-Curto note that run-up on smooth slopes does not follow this general trend, and conclude that it is not therefore correct simply to apply a reduction factor depending only on the type of armour unit.

In a natural sea, wave heights and periods vary randomly. Wave run-up is therefore also a random process. A typical run-up level often adopted for design is that exceeded by 2% of the run-up crests. This run-up level is known as R_2. Other levels may also be used, such as the significant run-up level, Rs or the mean run-up level $\bar{R}$. Before tests with random waves were prevalent, a number of techniques were used to predict the probability distribution of run-up levels and, where random wave results are not available, may still be needed. These techniques used a theory of equivalence to calculate the probability distribution of run-up crests. Expressions for run-up under regular waves, or suitable prediction curves, were used together with a joint probability distribution of wave heights and periods, or lengths. Saville (Ref 105) presents such a run-up prediction method based upon the use of forecasting curves, themselves derived from regular wave model tests.

From these it was concluded that:

$$R_2 \sim 1.3\ Rs \tag{5.6}$$

where Rs was derived using the significant wave height Hs and significant wave period from the prediction curves. The joint distributions of wave heights and periods have also been considered by Goda (1970), Longuet-Higgins (1975), Cavanie et al (1976) and Overick & Houmb (1977).

Thompson (Ref 124) has used an analytical framework for the calculation of run-up distribution, produced by Battjes (1971), for both simple and composite slopes. Thompson considers the two cases when wave heights and lengths are wholly dependent, given by the correlation parameter $\rho = 1$, and wholly independent, given by $\rho = 0$. A value of $\rho = 1$ is thought to apply to a sea in the early stages of growth and implies higher run-up levels. For simple slopes Thompson concluded that:

$$R_2 = 0.6\ Tz\ (g\ Hs)^{\frac{1}{2}} \tan \alpha \text{ for } \rho = 0 \text{ and} \tag{5.7}$$

$$R_2 = 0.75\ Tz\ (g\ Hs)^{\frac{1}{2}} \tan \alpha \text{ for } \rho = 1$$

For fully developed seas in deep water, where $\rho = 0$, and a mean sea steepness, $s = H/L$, of around 0.05:-

$$R_2 \sim 6.7\ Hs \tan \alpha \tag{5.8}$$

Run-up on smooth slopes has also been measured under irregular or random wave attack. For shallow structure slopes, expressions similar to Hunt's formula have been developed by various authors. A modified Iribarren number, Ir' may also be defined:

$$Ir' = \tan \alpha / (2\pi\ Hs/g\ Tp^2)^{\frac{1}{2}}, \tag{5.9}$$

where Hs is the significant wave height, and Tp is the period of peak spectral energy. Van Oorschot & d'Angremond (Ref 90) present the results of tests with irregular waves on smooth slopes of 1:4 and 1:6. They suggested a modified version of the Hunt formula for the 2% run-up level R_2 which may be written:

$$R_2/Hs = (2\pi)^{\frac{1}{2}}\ C_2\ Ir' \tag{5.10}$$

The coefficient C_2 is determined by the spectral width, ε. A single graph of C_2 against ε is presented in the TACPI report (Ref 120).

Grüne (Ref 44) presents results of field work on the German north sea coast in which waves and run-up levels were measured. An equation of the form of 5.10 was fitted to the results. For the 1:4 slope, mean values for C_2 of 0.71 to 0.92 were calculated, somewhat higher than the values usually quoted in the range 0.6 - 0.8.

Relatively steep structure slopes were considered by Kamphuis & Mohamed (Ref 63). They report results of tests on slopes of 1:1, 1.5, 2.0 and 3.0, using limited component irregular waves. They conclude that both wave heights and run-up levels are approximately Rayleigh distributed, but that their run-up results give $R_2/\bar{R} = 2.4$ rather than 2.23 which would be predicted for a Rayleigh distribution. For non-breaking irregular waves they conclude that the Pocklington or Miche expression is valid for irregular waves.

Ahrens (Ref 3) also presents results of wave run-up on a 1:1.5 smooth slope under irregular waves. A good fit to the results is given by the Gamma cumulative distribution function. This distribution describes the higher run-up levels well. The Rayleigh distribution, however, tends to under-predict the more extreme run-up levels. In a further paper, Ahrens (Ref 4) considers run-up caused by both regular and irregular waves. The results of tests with non-breaking regular waves on slopes of 1:1.5 and 4.0 are fitted to a much modified version of Pocklington or Miche's equation. For non-breaking irregular waves, the relative run-up generally follows the trends and conclusions drawn by Kamphuis & Mohamed. Ahrens also examines the statistical distribution of run-up levels. A two-parameter Weibull distribution is found to fit the data very well.

Measurements of random wave run-up on both smooth and armoured slopes are described by Allsop, Wilkinson & Allsop, Allsop, Franco & Hawkes, and Allsop et al (Refs 5, 7, 8, 140). Probability distributions of Rayleigh form were fitted to the run-up measurements. The run-up on the armoured slopes fitted this distribution form with minimal divergence. The run-up on smooth slopes, however, exhibited a similar trend to that identified by Ahrens in his earlier paper (Ref 3), in that the Rayleigh distribution tended to under predict the extreme run-up levels. Allsop, Franco & Hawkes (Ref 7) compared the fit of three-parameter Weibull, Rayleigh and Gamma distributions to run-up measured under random waves on smooth and armoured slopes. They concluded that the Rayleigh distribution generally gave as good a fit as any of the others. They suggest that the coefficient in equation 5.6

should be that given for a Rayleigh distribution, giving:

$$R_2 = 1.4 \text{ Rs} \tag{5.11}$$

5.2.3 Wave overtopping

The basic parameters controlling wave overtopping are essentially those affecting run-up. The commonly accepted measures of overtopping are the mean overtopping discharge, $\bar{Q}$, and the proportion of incident waves overtopping, sometimes known as the rate N. The rate of overtopping, N, is not a particularly good measure of overtopping, and cannot be used directly in design. The mean overtopping discharge, $\bar{Q}$, is of much greater use, and many researchers have attempted to produce prediction methods to calculate $\bar{Q}$ for a variety of input conditions. It should be noted that $\bar{Q}$ is usually given in terms of mean discharge per unit length of sea wall, eg m^3/s m, or l/s m. A volume per wave, q may also be defined, m^3/wave m.

It is not yet possible to predict wave overtopping from an entirely theoretical basis. For some of the simpler profiles of sea walls, the results of sufficient model tests have been assembled and analysed to allow some predictions to be made on the basis of empirical expressions. Much of the literature therefore reports the results of hydraulic model tests. Many more site specific studies of overtopping have been studied, however, but reports of such tests are seldom released. Model tests for sea wall design in Japan and, except the most recent work, in the USA, have generally been conducted with regular waves. Examples of Japanese work have been given by Tsuruta & Goda (1968), Kikkawa et al (1968), Shiraishi et al (1968), Nagai (1970), Goda (1971), Shi-igai & Hsu (1977), Onishi & Nagai (1979), and Nagai & Kakuno (1980). Much of this has been summarised or referred to by Goda (Ref 41) and Douglass (Refs 31, 32). Much of the early work in the USA, such as that presented by Bretschneider (1959), Saville (1962) and Weggel (1976) has been incorporated in the Shore Protection Manual. In Holland work by Battjes and others has been summarised in the TACPI report, and later by Roos & Battjes (Ref 104).

Design methods for UK sea walls have been discussed by Owen (Refs 91-93), who presents a design method for sea walls, accepting that there will always be a finite (albeit small) expected overtopping discharge. Sea walls with and without parapet (or wave) walls are considered. It is concluded that the design of sea walls incorporating wave return walls is so site specific, in that the possible variations of wave wall and cross section are set by local conditions, that generalised design parameters cannot yet be formulated. However, for sea walls without wave walls, a design manual is presented allowing the prediction of overtopping discharge under a wide range of wave conditions and for a wide range of structure variables: crest height, berm level and width, seaward slope, angle of attack etc, (Ref 91). The mean overtopping discharge, $\bar{Q}$, is expressed as a dimensionless discharge:

$$Q^* = \frac{\bar{Q}}{Tz\ g\ Hs} = \frac{\bar{Q}}{(g\ Hs^3)^{\frac{1}{2}}}\left(\frac{s}{2\pi}\right)^{\frac{1}{2}} \tag{5.12}$$

the dimensionless freeboard R* is expressed in terms of the freeboard Fc, the crest level less static water level, as:

$$R^* = \frac{Fc}{Tz\sqrt{g\ Hs}} = \frac{Fc}{Hs}\left(\frac{s}{2\pi}\right)^{\frac{1}{2}} \tag{5.13}$$

Q* and R* are related by:

$$Q^* = A \exp\ (-BR^*) \tag{5.14}$$

where A and B are determined for different values of structure variables.

This method is, however, only directly applicable to sea walls with a plain crest, without a wave return or parapet wall. When incorporated, such return walls are usually designed to suit local conditions, and may therefore vary widely in size and form. In many cases the wave return wall may be vertical or consist of a complex curve. In such instances a simple set of hydraulic model tests may be used, together with the design method given by Owen to provide predictions.

The Shore Protection Manual describes methods for the estimation of overtopping discharge based entirely on the results of regular wave testing, as do Weggel (Ref 132) and Kobayashi & Reece (Ref 68). Most detail is devoted to the calculation of the overtopping discharge under regular wave conditions, although an interim attempt to extend the procedures to account for random wave action is presented. The SPM method requires the calculation of a notional run-up level, R, that would be reached if the structure slope continued upward sufficiently to prevent overtopping. A simple equation is then proposed allowing the calculation of the overtopping discharge per unit length. This equation may be expressed in the notation of this report:

$$Q_{mono} = (BgH^3)^{\frac{1}{2}} \exp\left(-\left[\frac{0.217}{A} \tanh^{-1}\left(\frac{Fc}{R}\right)\right]\right) \qquad (5.15)$$

where Fc is the structure freeboard. Values of the empirical coefficients A and B (given as α and Q_0^* in the SPM) are presented for a wide range of slope types.

Methods suggested by Ahrens (Refs 1-4) are used in the SPM to extrapolate for random waves. It is argued that the overtopping discharge for a sequence of random waves may be given by summing the overtopping contribution of individual run-ups. It is assumed that wave run-up levels fit a Rayleigh probability distribution. This method embodies a number of fairly significant assumptions, and in some instances correction factors are proposed.

Goda (Refs 39, 41) also presents a calculation method for overtopping discharge for vertical walls under irregular wave attack, based upon a theory of equivalence. The total volume of overtopping of a sequence of random waves is calculated as equivalent to the sum of the volume due to each wave separately (as regular waves). This is justified by a comparison with measured overtopping discharges. Generalised overtopping prediction curves are presented for vertical walls and for walls fronted by armour units. All Goda's results appear to be based on regular wave testing.

Very little work has been conducted to compare the predictions of the various methods available, or to calibrate the methods against the results of field measurements. A most useful attempt at this is presented in a review by Douglass (Refs 31, 32), who compares methods given by Battjes (1974), Owen (Ref 91), Goda (Refs 39, 40) and the SPM. Douglass also cites a limited series of field and model measurements by Aaen (1977). It is observed that the prediction methods reviewed showed relatively little overlap. For vertical faced walls, the SPM method estimates higher discharges than does Goda's method, except for very shallow water. For sloping faced structures Battjes' method estimates higher discharges than does the SPM method for shallow slopes. Owen's method similarly gives higher discharges than the SPM for steeper slopes. Douglass concludes that estimates by any of these methods should only be regarded as within, at best, a factor of 3 of the actual overtopping rate. It is considered that Aaen's field measurements appear to suggest that model test measurements may be subject to scale effects. Douglass also notes that the effects of onshore winds on overtopping discharges are also ill-described.

There is considerable difficulty in estimating the amounts of wind induced or assisted overtopping and spray for any given sea wall. This is compounded by the inability of small scale hydraulic models to reproduce correctly spray generation, due principally to surface tension scale effects which control droplet size. Few model studies have successfully used scale wind velocities to assist overtopping. Finally there has been virtually no reliable information reported on the measurement of such overtopping in the field. Without any such field work, it is not surprising that there are no design guidelines. For wind assisted overtopping Horikawa (Ref 57) includes a single graph giving curves of dimensionless discharge against normalised wind velocity, $U/(g\ H)^{\frac{1}{2}}$, after Iwagaki, Tsuchiya & Inoue (1964). The SPM also takes account of enhanced overtopping discharge and suggests an enhancement factor:

$$k' = 1 + Wf\ (\frac{Fc}{R} + 0.1)\ \sin\ \alpha \qquad (5.16)$$

For $U_{10} > 25m/s$, $Wf = 2.0$
$U_{10} = 13m/s$, $Wf = 0.5$
$U_{10} = 0m/s$, $Wf = 0$

Norton & Macha (Ref 88) concentrate principally on wind/wave interactions but they also consider spray entrainment. Gadd et al (Ref 35) discuss qualitative trends in the wind effect, and conclude, as others, that more data is needed to improve on the SPM correction.

It is often assumed that waves attacking a wall with the wave crests plane to the wall, $\beta = 0°$, will give rise to more severe effects than would oblique attack. The Shore Protection Manual does not consider the effect of wave attack at any angle of incidence β other than 0°, that is with the wave crests parallel to the structure. The implicit assumption is that normal wave attack represents the most serious case. A similar general conclusion is drawn in the TACPI report, but is extended to give a reduction factor, k_β, equal to cos β for plain slopes. Two references were considered by the TACPI report, one postulated the reduction factor of cos β, the other, Hosoi & Shuto, presented experimental results for $\beta = 0°$, 30°, 45° and 60° on a 1:2 slope and indicated a lower reduction for most wave steepnesses for $\beta < 45°$ than would be given by multiplying normal run-up by cos β. Hosoi and Shuto's (Ref 58) results lay in the main between two limits:

$$\frac{1 + \cos\ \beta}{2} > k_\beta > \frac{1}{1 + \cos^2\ \alpha\ \tan^2\ \beta}$$

It should be noted that no incident angles within the range 0-30° were considered in any of the above studies. More recent work has however shown that normal wave attack may not give the greatest run-up (or overtopping). Test results, apparently originating from CSIR tests on dolos, and quoted by Günbak (Ref 45) but not referenced, illustrate that for waves of steepness $s = H/L_0$ of 0.03 - 0.04, wave run-up is greater for $\beta \sim 30°$ than for $\beta = 0°$ or 45°. This is not commented upon, in fact Günbak concludes that run-up may be reduced by the cos β factor. Work by Owen (Ref 91) and Tautenhaim et al (Ref 122) has however shown that run-up (and overtopping) may increase over an incident angle range of around 10°-30°. Tautenhaim et al report results of model tests on a 1:6 slope under regular wave attack at incident angles β between 0° and 60°. It is argued, on the basis of the test results, that the effect of oblique incidence is simply to modify the run-up at normal incidence R_n, by a factor k_β where:-

$$k_\beta = \cos \beta \,(2 - \cos^3 2\beta)^{1/3} \qquad (5.17)$$

A similar effect was noticed by Owen in tests measuring the overtopping of sea walls. It was shown that generally the mean overtopping discharge at 15°, and sometimes 30°, exceeded that at $\beta = 0°$.

It seems likely that, whilst the expression given by Tautenhaim was only derived for walls of 1:6 slope, a similar effect may be seen for sea walls of steeper slope. It would appear from these two studies that the angle giving greatest run-up (or overtopping) will be around $\beta = 15 - 20°$. Enhancement factors for wave run-up under oblique attack are discussed by Tautenhaim et al (Ref 122) and Allsop et al (Ref 6). It should be noted that in their recent review paper Gadd et al (Ref 35) appear unaware of the work by Owen and Tautenhaim et al, and suggest reduction factors given by Hosoi & Shuto.

5.3 Wave Forces and Pressures

5.3.1 General

Waves impinging on a sea wall may give rise to severe pressures against the wall, the magnitude of which will be determined by the characteristics of the incoming wave, the history of previous waves, and the shape and construction of the wall. In considering the forces on sea walls a distinction is often made between the effects of breaking and non-breaking waves. The force exerted by non-breaking waves is taken to be predominately hydrostatic, and varies relatively slowly. In contrast breaking (or broken) waves exert a dynamic force, due to the effects of wave momentum, water turbulence and entrapment of compressed air, which may be very much greater than the hydrostatic forces, but may last for much shorter durations. In particular a breaking wave may produce a pressure of very high intensity and short duration, known as a shock (or impact) pressure, followed by a longer period of less intense pressure. The product of the force due to the shock pressure and its duration is usually referred to as impulse, which is a measure of the change of the momentum of the wave as it strikes the wall. For non-breaking waves, the incoming wave is reflected by the vertical wall, forming a standing wave in front of it.

Much of the most useful work on wave forces on vertical walls is of Japanese origin. Horikawa (Ref 57) provides a series of useful illustrations of pressure/time curves showing the transition from non-breaking to breaking wave pressures. Other useful summaries are given by Goda (Ref 41) and Nagai (Ref 85).

5.3.2 Non-breaking waves on vertical walls

Sea walls are often situated where the water depth is such that some waves may break against the structure. However in certain situations, where the wall is located in deep water, the structure may be subjected to non-breaking waves. Therefore a means of calculating the pressures and/or forces due to such waves is required. As there are considerable uncertainties in such calculations, it may be useful to summarise some of the relevant historical methods.

The SPM cites the work of Sainflou (1928), Miche (1944) and Rundgren (1958). Sainflou proposed a method for calculating the pressure distribution on vertical walls due to non-breaking waves based on trochoidal wave theory. Full details of the theory developed by Sainflou and its simplification are given by Horikawa (Ref 57). Whilst the expression due to Sainflou is reasonably easy to apply, it was found by Rundgren (1958) to overestimate the wave forces on a vertical wall for steep waves. Miche (1944) derived a second order theory for calculating the pressure distribution on vertical walls which was found to give better agreement with experimental results. Rundgren extended the work of Miche to include the wave reflection coefficient of the structure. The SPM presents a series of design curves based on the work of Miche and Rundgren which may be used to calculate the forces on vertical walls due to non-breaking waves. BS 6349 (Ref 17) also suggests that the wave pressure distribution on vertical sea walls should be calculated using Sainflou or Miche-Rundgren.

Horikawa (Ref 57) reviews the work of Goda & Kakizaki (1966), which gives a fourth order approximation to the standing wave pressure on a vertical wall, an extension of the work of Tadjbakhsh & Keller (1960). Goda compares the results for pressure obtained from his approximation with measurements from a series of regular wave tests, and finds them to be in reasonably good agreement. Goda also finds that Sainflou's formula generally tends to give an overestimate of the wave pressures. Goda presents design diagrams based on his theory, with some modifications made for the total wave forces using his experimental results.

Nagai (Ref 85) also reviews the theories of standing waves in both deep and shallow water. He compares the calculated pressure distributions and maximum simultaneous pressures at the wall using the above theories, with his own experimental results for various water depths and wave steepnesses. The comparison of theory and experiment lead him to suggest ranges of applicability of the theoretical wave pressure formulae, based on the values of wave steepness, H/L, and relative depth, d/L. The regions of applicability found by Nagai are deep water waves ($d/L > 0.35$) shallow water waves ($0.135 < d/L < 0.35$) and very shallow water waves ($d/L \leq 0.135$ and $H/L < 0.04$). For each of these regions, he presents formulae which may be used to calculate the maximum simultaneous pressure and the force/unit length due to non-breaking waves on verical walls.

5.3.3 Breaking waves on vertical walls

For many situations, a sea wall may be exposed to breaking rather than non-breaking waves at some point in the tidal cycle. A wave breaking on a vertical wall may exert short duration shock pressures which are considerably higher than the forces due to non-breaking waves. Over many years, authors have considered the problem of predicting these forces and pressures using theoretical and experimental means. The results of a few field measurements and observations have also been reported.

The SPM mentions some of the very early work done in this area. More complete reviews are given by both Ramkema (Ref 101) and Stephan (Ref 115). Notable is the work of Bagnold (1939), who conducted experiments to investigate the shock pressures of breaking waves, and proposed a formula for their calculation. Bagnold's theory will predict infinitely high shock pressures if it is assumed that no air is trapped by the breaking wave on impact. Iribarren (1953) proposed a formula for the calculation of shock pressures which does not assume that air must be trapped by the breaking wave. If air is present, its effect is to damp the pressure, and in this situation both Iribarren's and Bagnold's formulae predict similar pressures. Bagnold's work was followed up by Denny (1951) who also conducted a series of experiments to measure the intensity and duration of shock pressures. Extensive model tests to assess the effects of water depth, wave period and bed slope on shock pressures were performed by Ross & Culbertson (1955). Some early field measurements of shock pressures were made by de Rouville, Besson & Petry (1938) on a vertical breakwater wall at Dieppe.

Both the SPM, and BS 6349 (Ref 17), suggest that when calculating forces on a vertical wall due to breaking waves, the method of Minikin (1950) should be used. Minikin based his design procedure on field observations and the experimental results obtained by Bagnold. The SPM, however, notes that Minikin's formula may predict forces which are extremely high and should be used with caution. Several other authors also note limitations of Minikin's method. Horikawa indicates that Minikin's formula predicts a relatively high value for the mean shock pressure averaged over the area affected by the wave pressure. However, the local shock pressure may be underestimated by the value calculated using Minikin's formula. Horikawa also notes that the fairly simple wave pressure formula proposed by Hiroi (1919) often produces results in good agreement with the mean pressure intensity calculated from experimental and field data.

Nagai (Ref 85) gives a summary of theoretical and experimental studies concerned with predicting the forces and pressures due to wave action on composite type breakwaters. Nagai concentrates on deriving formulae to calculate the maximum simultaneous pressure on a vertical wall. For breaking waves, Nagai suggests that, because a whole range of incident wave conditions, from a perfect standing wave to a perfect breaking wave, are covered, a number of different formulae should be used to predict the forces and pressures exerted by such waves on a vertical wall. He proceeds to derive a series of expressions for the maximum simultaneous pressure, each of which is valid for a given range of incident wave conditions and structure dimensions. In each case the expression for the maximum simultaneous pressure is dependent on the maximum local pressure, determined empirically. Nagai gives values for the maximum pressures which represent average values exerted by ordinary breaking waves. The pressures calculated using the various formulae are compared with his own experimental results and the two are found to be in good agreement.

In a later paper Nagai & Kurata (Ref 86) compare the pressures calculated using Nagai's formulae, with estimates of the breaking wave forces which caused sliding of a number of prototype breakwaters in Japan. In addition to Nagai's formulae, the methods of Minikin and Hiroi are also used to give estimates of the pressure. The methods of Minikin and Hiroi are both shown to give poor agreement with the estimated prototype wave forces. It is concluded that Nagai's formulae are sufficiently reliable to produce an optimum design of a composite breakwater.

Kamel (Refs 61, 62) presents the results of an extensive study into the factors affecting the magnitude of wave pressures on vertical walls. Both theoretical and experimental work previously done by a number of authors is reviewed. In addition Kamel derives his own formula, for calculating shock pressures due to breaking waves based entirely on analytic considerations. This formula is compared with results of a series of experiments. The theory proposed by Kamel was found to predict values of the shock pressure very much higher than those measured experimentally. The experimental and theoretical results were also used to determine a probability distribution of the magnitude and duration of shock pressure.

A numerical model for the calculation of the pressure distribution due to breaking waves on a vertical wall is proposed by Weggel & Maxwell (Ref 133) based on the earlier work by Bagnold. The results calculated from the numerical model were compared with pressures measured in a series of physical model tests. The pressures at the vertical wall were measured simultaneously at several adjacent points. Comparisons were made between the numerical and experimental pressure distributions. They were found to be in reasonable agreement.

Goda (Refs 40, 41) describes the evolution of wave pressure formulae for vertical walls which are applicable to a range of incident wave conditions, from non-breaking to breaking waves, with a smooth transition between them. The formulae are based on theoretical considerations, but are dependent on empirically determined coefficients. For all situations, a trapezoidal pressure distribution is assumed, with the maximum pressure, p_1, at static water level. The design wave height to be used in calculating the wave pressures is specified to be the maximum possible wave height at the site of the structure. Goda recommends H_{max} = 1.8Hs for deep water, but discusses the use of both higher and lower multiples. Goda then details the calculation of the maximum wave pressure, p_1, as well as other values sufficient to describe fully the pressure distribution. The formulae do not take into account the very high shock pressures of short duration exerted by breaking waves on a vertical wall, as Goda claims that such high pressures rarely occur in prototype situations. The wave pressure formulae are calibrated with reference to case studies of prototype breakwaters in Japan. The results from these formulae, together with those of Sainflou, Hiroi and Minikin, are compared with estimates of the forces which caused the upright sections of prototype breakwaters to slide, see also Nagai & Kurata (Ref 86). Goda concludes that his own formulae are in the best agreement with prototype results, and should be employed in the design of prototype structures with vertical walls.

A number of authors have reported the results of model tests to measure the pressures due to breaking waves on vertical walls, but these have not generally led to any advances in calculation methods. Measurements of wave forces on solid and perforated caisson structures have been made in regular wave models and are reported by Mitsuyasi (1966), Terrett et al (1968), Marks & Jarlan (1968), and Mogridge & Jamieson (1980).

Model tests have also been used to assess the factors affecting the magnitude of the wave forces on a vertical wall. Hashimoto (Ref 51) conducted model tests to investigate the effect of wave irregularity. The results indicate that the frequency distribution of irregular wave forces is fairly wide in deep water, but becomes narrower in shallow water. Hashimoto used his experimental results to define a relationship between significant wave height and significant wave force. Takezawa (Ref 121) found that the magnitude of the wave forces was influenced by bed slope, wave run-up, and wave breaker type, as well as the depth at the toe of the structure and the incident wave height and length. He presents graphs showing the relationships between these parameters for four different magnitudes of wave force.

Kirkgoz (Ref 66) studied the magnitude, duration and spatial distribution of the shock pressures exerted on a vertical wall by regular waves for different beach slopes. He concluded that the maximum shock pressures occur when a wave breaking directly onto the wall has its front face parallel to the wall at the moment of impact. The greatest shock pressures were produced where the beach had the shallowest slope. Whillock

(Ref 137) conducted a series of experiments to determine the manner in which forces on a vertical wall (and walls of convex form) respond to variations in the angle of wave attack. The results indicated that the maximum forces did not necessarily occur when the waves were exactly at normal incidence. However, subject to this constraint, the pressures and forces in general decreased steadily in magnitude as the angle of wave approach moved significantly away from normal.

The results of model tests may be used to give an indication of the applicability of the theoretical formulae used to calculate wave forces and pressures. There may, however, be problems with scaling of model results to the prototype. It is therefore clearly important, for the purposes of verification, to have reliable field measurements of forces and pressure due to waves breaking on a vertical wall. Many of the early attempts to monitor the forces on vertical walls were limited by the lack of instrumentation capable of measuring very high impact pressures whose duration is short. A review of the early work carried out by authors such as Gaillard (1905), de Rouville (1937) and others is given by Blackmore & Hewson (Ref 15).

Only a limited amount of more recent literature seems to be available in this area. Nagai & Kurata (Ref 86), and Goda (Refs 40, 41) compared their theoretical results with estimates of the forces acting on prototype breakwaters, but no measurements of these forces were actually made. Muraki (Ref 84) gives details of field measurements of wave pressures on a breakwater in Japan. He reports that, whilst shock pressures similar in magnitude to those measured in experiments did occur, their frequency of occurrence was very low. More recently, Blackmore & Hewson (Ref 15) have reported the results of field measurements made on a sea wall on the south coast of England. The sea wall has a re-entrant profile with a bullnose. Several pressure transducers were placed at various points on the wall. Blackmore & Hewson note that high impact pressures were measured but occurred infrequently. In addition, the impact pressures measured were found to be lower than those from previous experiments. This is attributed to the very high percentage of entrained air in the incident waves. Although Blackmore & Hewson's measurements were compared with theoretical and empirical equations for calculating shock pressures, the results are inconclusive. They also suggest a theoretical relationship between the magnitude and duration of impact pressures. This relationship is shown to fit the measured data reasonably well but is dependent on the amount of entrained air, a particularly difficult quantity to estimate.

It would seem that progress on the prediction of pressures and forces due to breaking waves on sea walls can be advanced on several fronts. In particular the mechanism of waves breaking is still not sufficiently well understood and the theory of breaking waves is capable of being extended. Recent work by Dold & Peregrine (Ref 30) seems to be moving in this direction. In addition reliable field measurements of the pressure and forces on sea walls would be useful both in investigating the problems of scaling from models and validating empirical and theoretical expressions.

For vertical or near vertical walls, wave action may give rise to very high local shock pressures of extremely short duration, and to lower pressures of much longer duration. These latter pressures may generally be regarded as those most likely to affect a conventional wall of large elements, of mass or reinforced concrete, or of sheet piling. It seems likely that the method advanced by Goda may provide the most appropriate estimate of such wave pressures. It should, however, be noted that the literature reviewed does not provide the designer with clear guidance. The results of wave pressure calculations for such sea walls should be regarded as subject to considerable uncertainty, and treated with some caution.

Most research appears to suggest that the high shock pressures caused by a wave breaking directly against the wall occur very rarely. However, when a sea wall includes small elements that might be capable of responding to shock pressures of short duration, other calculation methods suggested by the SPM, Kamel (Refs 61, 62), Weggel & Maxwell (Ref 133), Leon Joseph (Ref 72) and by Goda (Ref 41), should also be considered.

5.3.4 Forces on sloping walls

A number of sea walls around the British coastline have sloping front faces. This is particularly so in low-lying and/or agricultural areas, where the sea wall bank is often an embankment structure formed of local materials. Such slopes may be armoured with stone or concrete revetment blocks, or concrete slabs. These and other revetment systems are considered elsewhere in this review. There is, however, relatively little guidance available in the literature to permit the estimation of forces and pressures on such slopes.

The SPM gives an adaptation of the work of Minikin which enables the forces and pressures due to breaking waves to be calculated on sloping forces which are nearly vertical. More recently Stephan (Ref 115) has reported the results of field observations and physical model tests on the effect of shock pressure on sea dykes. In particular he gives an analysis of the damage caused to sea dykes on the north German coast by storm tides with special reference to shock pressure effects. The physical model tests are intended to investigate shock pressure forces and their effect on the core of the dyke. Stephan also illustrates the problems in formulating a model law for shock pressures. He concludes that the maximum shock pressures are sustained about half a wave height below the SWL, that high shock pressures are more likely to occur on steeply inclined slope and that the shock pressures decrease continuously through the core. Stephan also suggests that full scale model tests would be useful in any attempts to derive a model law.

Experiments on a 1:4 slope at scales of 1:10 and 1:1 using regular waves are reported by Führböter (Ref 33). The tests confirm that Froudian scaled model tests may give a slight overestimate of wave pressures on a slope. For practical use, a very simple method is suggested for the calculation of wave pressures. The pressure, p_i, for i = 50, 90, 99 or 99.9% may be calculated by:

$$p_i = Ki \ \rho g H \qquad (5.18)$$

For the median wave pressure, i = 50%, Ki = 2.2; for i = 90%, Ki = 3.0; for i = 99%, Ki = 3.9; and for i = 99.9%, Ki = 4.8.

5.4 Wave reflections

5.4.1 General

Wave energy arriving at a coastal structure may experience a number of processes of concern to the designer. For simplification, these may be considered under three principal headings:

(a) absorption or dissipation;
(b) transmission by overtopping;
(c) reflection.

The first two of these processes are covered elsewhere in this chapter. Energy absorption is dealt with in part in section 5.2 on wave run-up and in sections 5.5-6 covering the effects of waves on armour systems. Energy transmission by overtopping is covered in section 5.2. The estimation, or measurement, of wave reflections, and some of the effects of reflected wave action are discussed in this section.

The degree of wave reflection from a sea wall will depend upon the characteristics of the incident waves, the beach in front of the wall, and of the wall itself. Of these, the parameters having the greatest effect on the reflection performance may be summarised:

(a) wave heights and periods;
(b) angle of wave attack;
(c) beach level;
(d) beach slope and sediment size;
(e) structure front face slope and geometry; and
(f) front face porosity, permeability and roughness.

In particular, it should be noted that the degree of wave reflection will depend critically upon the relations between water level, beach level, and the position of the sea wall. For many sea walls fronted by a beach, the overall reflection performance will therefore change as the tide level rises and falls. This may be illustrated by considering a vertical wall, itself having a high degree of reflection performance, fronted by a shallow sand beach, in turn having a relatively low reflectivity. At water levels at which only the beach is exposed to wave action, the proportion of incident wave energy reflected back outwards will be relatively low. However, at tide levels that allow wave action to reach the wall unmodified by the beach, a high proportion, often approaching 100% of that incident, may be reflected.

5.4.2 Estimation and identification of reflections

Relatively little guidance is available to the designer allowing the estimation of wave reflections. Much of that available is based on work with regular waves, and often assumes the use of linear wave theory. In most instances the reflection performance of a structure is described in terms of a reflection coefficient, Kr. This may be defined in terms of the total incident wave energy, Ei, and the total reflected by the structure, Er, thus:

$$Kr = \left(\frac{Er}{Ei}\right)^{\frac{1}{2}} \qquad (5.19)$$

This is equivalent to the ratio of the reflected to incident wave heights in a regular wave train. For random wave conditions, a coefficient may be similarly defined for each frequency band, in terms of the incident and reflected energy densities, Si and Sr respectively:

$$Kr = \left(\frac{Sr}{Si}\right)^{\frac{1}{2}} \qquad (5.20)$$

The prediction of the level of reflected wave energy is addressed by various researchers, using different approaches. Both analytical and experimental techniques have been reported. In general, however, most methods rely on model tests to determine values of the empirical coefficients used.

Much of the recent work of use to the designer is summarised by Seelig (Ref 107) and, in a longer version, by Seelig & Ahrens (Ref 108). Both present prediction methods for regular wave reflections from beaches, sea walls or breakwaters. For both smooth and rubble structures, Seelig presents a simple prediction equation for Kr in terms of the Iribarren number, Ir, and empirical coefficients α and β, which may be written:

$$Kr = \frac{\alpha\ Ir^2}{\beta + Ir^2} \tag{5.21}$$

For smooth slopes values of $\alpha = 1.0$ and $\beta = 5.5$ are recommended, whilst for rubble structures, with very permeable armour and underlayers, values of $\alpha = 0.6$ and $\beta = 6.6$ might be tried. Seelig also discusses a number of other techniques, and gives design curves and tables for various structure types and configurations. He also compares the use of equation 5.21 with $\alpha = 1.0$ and $\beta = 6.2$ for a smooth slope, with measured data, and the prediction equation given by Battjes (1974):

$$Kr = 0.1\ Ir^2 \tag{5.22}$$

Seelig also discusses the use of an equation adapted from that proposed by Battjes for smooth slopes:

$$Kr = \tanh\ (a\ Ir^b) \tag{5.23}$$

for which values of $a = 0.1$ and $b = 2.0$ are suggested. This equation would appear to allow conservative estimates of the reflection coefficient for structures having simple smooth slopes.

Very few examples of the reflection performance of structures subjected to random wave action have been presented. Allsop (Ref 5), and Allsop et al (Ref 8) show values of reflection coefficient, Kr, against frequency for a number of steep smooth and armoured slopes.

For further analysis of wave reflections it may be necessary to use expressions for wave surface elevations. In general, the choice of a particular wave theory to describe the fluid motion will be determined by the ratio of wave height to wave length and depth to wave length. Le Méhauté (Ref 80) produced a useful diagram showing the range of applicability of various wave theories, since reprinted in the SPM. For shallow water, and anything but the lowest wave, the most appropriate wave will often be either the Cnoidal wave or a higher order Stokes' wave. However, Song & Schiller (Ref 113) provide a comparison of theoretical surface profiles and velocity cycles for linear and second order waves under a range of reflection coefficients. They claim that whilst the envelopes of the linear and second order surface profiles appear to differ greatly, the envelopes of the velocities do not, and the use of linear theory may therefore be justified. This allows the use of a simple expression for the surface elevation, η_s, in the presence of reflections from a vertical wall, $\alpha = 90°$, at incident angle $\beta = 0°$. If the reflection coefficient equals zero, the expression for surface elevation reduces to one for a simple progressive wave. If the reflection coefficient equals unity, a perfect standing wave results.

Methods for the measurement and analysis of incident and reflected waves have been discussed by Kajima (Ref 64), Gilbert & Thompson (Ref 37), Gaillard, Cauthien & Holly (Ref 36), Goda & Suzuki (Ref 42), and Thornton & Calhoun (Ref 128). The original approach by Kajima has been used by Gilbert & Thompson in the development of a computer method for the identification and analysis of both incident and reflected waves measured in either model or prototype. Thornton & Calhoun describe apparently the only successful measurement of the wave reflection performance of a prototype structure.

5.4.3 Effects of wave reflections

Reflected wave action will lead to two principal effects. Firstly the standing wave patterns produced by the interaction of incident and reflected waves will lead to a confused sea in front of the sea wall. Momentarily very steep waves will exist as the two wave trains interact. Whilst probably of great importance to navigators and users of small vessels, this phenomenon does not appear to have received any attention in the technical literature.

The most important effect arising from the reflection of incident energy from a sea wall, or related structure, is scour of the beach material from the area in front of, and close to, the sea wall. Many investigators have devoted considerable effort to attempts to predict the onset and extent of beach scour or erosion at the toe of a sea wall. Some of the more general effects have been covered by work reviewed earlier in Chapter 4. Much of the more detailed work is based on small scale experimental work. It should, however, be noted that much of the experimental work suffers from scale or modelling effects, and in some instances, may lead to contradictory conclusions. It is clear from the literature that the calculation of sediment transport under simple conditions of waves and currents still gives rise to considerable uncertainties. The complications of wave and current interaction with a structure as well as a beach gives rise to further problems, and greater uncertainties. The literature therefore tends to divide into that giving simple empirically-based rules for the design of scour protection, assuming that scour will occur, and that seeking to analyse and describe some element of sediment transport under waves and currents.

General sediment transport theories are discussed by Muir Wood & Fleming (Ref 83), Silvester (Ref 111) and Komar (Ref 70). A detailed account of the interactions of waves and erodible bed sediments is given by Sleath (Ref 112). Details of research into aspects of sediment movement under wave action have been given by Lamb (1932), Shepard (1950), Shepard & Inman (1950), Inman & Bowen (1962), Bowen & Inman (1969), Bowen (1969 a, b), Noda (1969), Natarajan (1969), Carter, Liu & Mei (1973), Yalin & Karahan (1978), Sleath (1978), Du Toit & Sleath (1981), and Perkins & Sleath (1983). Of these Noda (1969) and Carter et al (1973) address problems of mass transport under the standing waves produced in laboratory wave flumes.

Relatively little of the laboratory work published to date is of immediate use to the designer. In a textbook on scour published in 1984, Herbich et al (Ref 53) present the results of various small scale regular wave laboratory experiments, apparently conducted before 1968. From these, general conclusions are drawn on the effect of structure and wave parameters on the scour depth, S, obtained. As much of the data yields results that are expressed in inches, and no discussion on the scaling relationships is included, it would appear that only very general conclusions may be drawn.

Hales (Refs 46-48) has reviewed much of the technical literature on scour problems for a wide range of structure types. He also describes American design and construction practice for scour prevention measures. Hales considers two main studies, those by Herbich & Ko (reported in Ref 53), and by Song & Schiller (Ref 113).

For partially reflecting structures, with a reflection coefficient Kr, Hales suggests that the expression derived by Herbich & Ko may yield the ultimate scour depth, S:

$$S = (d - \frac{a}{2}) \left[(1-Kr)\ u_{max}\ (0.75\ C_D\ \rho \frac{\cot \phi}{D_{50}\ (\gamma_s - \gamma)})^{\frac{1}{2}} - 1\right] \tag{5.24}$$

where a = Hi + Hr;
d is the static water depth;
u_{max} is a maximum particle velocity at the bottom;
C_D is a sand particle drag coefficient, a function of the Reynolds number;
ρ is water density;
γ_s, γ are sand and water specific weights;
D_{50} is the median sediment size; and
ϕ is the angle of repose of the bed sediment.

For impermeable sea walls and, one assumes, those of high reflection coefficient, Hales suggests the expression derived by Song & Schiller for relative scour depth:

$$\frac{S}{Ho} = 1.94 + 0.57 \ln (\frac{X}{X_b}) + 0.72 \ln (\frac{H}{L}) \tag{5.25}$$

where X is the distance from sea wall to the point of interest, and X_b is the distance from the sea wall to the breaker position, and
$\frac{H}{L}$ is the wave steepness.

Finally, Hales (Ref 48) notes that the SPM recommendation is that, in the absence of scour protection measures, a scour depth, equal to the maximum unbroken wave height that could be sustained by the original water depth at the structure toe, should be allowed for.

It will be noted that none of these methods are directly applicable to the design of a sea wall at the back of a beach, subjected to the action of irregular waves and tides.

5.5 Hydro-dynamics of armoured front slopes

5.5.1 General

The design of armoured slopes subject to wave action covers both the stability of the armour on the front face, and the other aspects of hydraulic performance as given by run-up, overtopping, and reflections. In the following parts of section 5.5, the design of armour layers will be considered primarily from the aspect of the stability of the armour system against wave attack. The other aspects of the hydraulic performance of armoured slopes have been covered in earlier sections of this chapter. The design of the crest and rear face armouring of sea walls is considered in section 5.6.

5.5.2 Types of armoured sea walls

The hydro-dynamic performance of the outer armour layers depends not only upon the size, shape, placing patterns and other characteristics of the armour, but also critically upon the hydraulic characteristics of the layers beneath the armour. The flow characteristics of the underlayers and core will, in turn, depend upon the construction materials used, and will vary over a wide range. For example, a sea wall formed with an earth embankment will generally allow no significant flow within, or into, the core over a wave period, and much of the flow, and flow-induced effects, will be confined to the outer layer(s). In contrast, a rubble-mound sea wall may allow considerable volumes of water to flow into and through the armour and under layers, into the core. The hydraulic characteristics of the whole mound must therefore be considered in the design of the armour. The armouring to sea walls has therefore often been considered under a number of separate categories, examples of which have sometimes been termed in the literature; rubble sea walls, rubble revetments, and armoured revetments.

The rubble mound sea wall is designed, and often constructed, on essentially the same basis as a rubble mound breakwater. In fact, such a sea wall may often act as a breakwater in the early stage of construction of a reclamation. For the rubble mound, a central core of quarry rock is placed, usually with side slopes at their natural angle of repose. Filters, or underlayers of suitably sized and graded rock are then laid over the core. On the seaward face, the outer layer is armoured with rock or concrete armour units, usually laid in two layers, although some specialised concrete armour units may be laid in a single layer. The inner face of such a sea wall must be protected by a number of carefully selected filter layers to retain the, often fine, material behind it against the action of waves and tides.

Rubble revetments are built without a large core of quarry rock, but with a number of layers of rock laid against a prepared face of fill or indigenous material. Rock or concrete units may be used as the outer armour layer. In general a rubble revetment will have sufficient depth of porous, pervious construction, usually in a number of layers, to allow a significant level of energy dissipation within those layers. Such a revetment will therefore be relatively pervious to wave and tidal induced flow. Again, carefully selected filters must be used between the revetment and the fill, if the fill material is potentially mobile.

Other revetment systems may have few layers of rock as underlayers and filters. In the extreme, a revetment system may consist of a geotextile acting as a separator and filter, and an array of, usually concrete, close-fitting blocks as armour. The material used in revetment armour is normally stone or concrete, but bricks, gabions, asphaltic or cement concretes may all be used. Revetment protection may be rigid or flexible, the latter consisting of discrete elements, often interlocked or jointed. Revetment blocks are often small enough to be laid by hand and may then be bonded together to form a continuous armour layer. The individual elements of the revetment normally remain in intimate contact with the underlayer, even as it settles in use. Without this flexibility the revetment might fail suddenly over points of settlement.

Of particular relevance to this review are a number of text books, design manuals and technical reviews. The Shore Protection Manual covers rubble structures in some detail, giving basic design guidelines. It should be noted, however, that even the latest edition of the SPM, published in 1984, is based substantially on regular wave test results, and hence regular wave design philosophy. Similar, or abbreviated, design rules are given in the text books by Muir Wood & Fleming (Ref 83), Sorensen (Ref 114), Quinn (Ref 100), and Bruun (Ref 23). A brief summary is given by Thorn & Roberts (Ref 127). Many types of sea wall, including rubble mound and rubble revetment, are described in the review by Bertlin & Partners (Ref 13).

The design of a rubble sea wall may be considered most conveniently from the outer layer inward. The armour layer is dealt with first, as it will set the size of rock in each layer beneath (and their number and thickness), and hence have most effect upon the design of the cross section.

5.5.3 Rock armouring

The armour layer, or layers, to a rubble structure may be composed of narrow size graded rock (sometimes called single size), widely graded rock (often known as riprap) or, in some instances, special concrete armour units. Of these, it is generally agreed that rock should be used where available, for economy and durability. The design of concrete armouring is covered in section 5.5.4 below.

There are three main design methods available for the calculation of rock armour weight. The use of these methods is described by Powell (Ref 98) and may be summarised:

(a) Hudson's formula, used together with values of the empirical damage coefficient, as given in the SPM;
(b) the method of CIRIA 61 for rip-rap armouring, based on model tests by Thompson & Shuttler (Refs 125, 126);
(c) the extension of Thompson & Shuttler's tests by van der Meer (Refs 78, 79).

The design of single size rock armour layers is detailed in the SPM. The classic Hudson formula, Hudson (1958) is used, together with values for the stability coefficient, K_D, to determine the stable rock weight, W, to resist an incident wave height, H. As noted earlier, the Hudson formula and these coefficients have been derived from the results of regular wave tests. There is, as yet, no agreed way of comparing the results of regular and random wave tests. These tests appeared to show no dependence of armour stability on wave period, and the Hudson formula does not therefore take account of wave period. However, it is argued by Bruun & Günbak (Ref 24) quoting Bruun and Johannesson (1976) that "the significance of wave period is clearly demonstrated". They suggest that the surf similarity parameter, or Iribarren number, $Ir = \tan \alpha/(H/L_o)^{\frac{1}{2}}$, should be used to describe flow conditions and stability of the armour on rubble slopes.

Rock of a wider size grading may also be used to armour rubble mound or rubble revetments. Such riprap is often used to provide wave protection to dam faces. Typically the rock sizes in riprap vary between 4 W_{50} and 0.25 W_{50}, the median rock weight being W_{50}. Riprap may be designed to guidelines given in the Shore Protection Manual or by CIRIA (Ref 126). The SPM gives a typical range of rock sizes in riprap as 4.0 to 0.125 W_{50}. The Hudson formula is used for design, with values of K_D (written as K_{RR} for riprap) between 2.2 and 2.5 suggested for the structure trunk, equivalent to the general run of a sea wall. The CIRIA work is based upon a series of random wave tests by Thompson & Shuttler (Ref 125). They tested rock varying in size between 3.4 and 0.3 W_{50}, and used random waves in deep water. From the laboratory results, the onset and degree of damage to a riprap slope may be determined. This allows the design of a suitable size riprap armour for a given degree of damage over its design life. Within the range of the tests, no scale effect was identified, and no correction for scale was recommended.

These laboratory tests were then followed by prototype tests on the trial bank in the Wash, reported by Young, Ackers & Thompson (Ref 142). These field trials showed some differences from the laboratory results, but it was believed that this was principally due to differences in construction of the trial panels. As a result, a further set of model tests were run by Shuttler & Cook (Ref 109). These tests were extended to both higher and lower Reynolds numbers than had been used in the original tests to check whether any correction should be allowed for scale effects. The results of these retrospective tests agreed well with the field trial results. Shuttler & Cook, Pitt & Ackers (Ref 97), and Ackers & Pitt (1983) summarising the conclusions of the field trials, agree with the original belief that no scale effects should be allowed for when using the model test results for full scale design. In contrast, Broderick & Ahrens (Ref 19), and Broderick (Ref 18), still believe that a "scale effect" apparent in regular wave work in the USA should be allowed for in design. They conclude from their work that small models may give zero damage stability numbers, Nzd, up to 20% lower than would full size tests. They have not, however, run large scale random wave tests or prototype field trials.

Van der Meer (Refs 78, 79) has reported a comprehensive set of laboratory tests extending the original work of Thompson & Shuttler. Design formulae have been proposed, derived from the test results. The three main equations, in common with Hudson's method, distinguish between breaking and non-breaking waves.

For breaking waves ($Ir < 2.5\text{–}3.5$);

$$\frac{Hs}{\Delta\, Dn_{50}} = 5.7P^{0.14}\,(S/N^{\frac{1}{2}})^{0.2}\,Ir^{-0.5} \qquad (5.26)$$

For non-breaking waves ($Ir > 2.5-3.5$), $\cot \alpha < 3$;

$$\frac{Hs}{\Delta \, Dn_{50}} = 0.83P^{-0.2} \, (S/N^{\frac{1}{2}})^{0.2} \, (\cot \alpha)^{0.5} \, Ir^{P} \qquad (5.27)$$

For non-breaking waves ($Ir > 2.5-3.5$), $\cot \alpha > 3$;

$$\frac{Hs}{\Delta \, Dn_{50}} = 0.83P^{-0.2} \, (S/N^{\frac{1}{2}})^{0.2} \, 1.73 \, Ir^{P} \qquad (5.28)$$

where

Hs is a design significant wave height
Δ is a relative density = $\frac{\gamma r}{\gamma w} - 1.0$
γr is the weight density of rock
γw is the weight density of water
Dn_{50} is a nominal rock diameter = $(W_{50}/\gamma r)^{1/3}$
W_{50} is the median weight of the armour rock
P is a notional core permeability factor
S is a design damage number
N is the number of waves
α is the structure slope
Ir is the Iribarren number = $\tan \alpha/(Hs/Lo)^{\frac{1}{2}}$
Lo is an offshore wave length = $gTz^{2}/2\pi$

In common with CIRIA 61, the waves used in the model tests were deep water random waves. Thus, again, the design wave conditions used should be those at the toe of the structure.

A number of values for the design damage number, S, are suggested. Powell (Ref 98) comments that the damage criterion chosen at the design stage will effectively determine the maintenance requirements for the structure over its lifetime. In general it may be expected that the majority of structures will be designed to Hudson's zero damage/CIRIA Criterion C/ van der Meer's initial damage.

The main problem when using van der Meer's equations is the assessment of the core permeability factor P. The values of P suggested range from 0.1 for a relatively impermeable core, up to 0.8 for a virtually homogeneous and permeable rock structure. Although this theoretically allows the application of van der Meer's equations to both permeable and impermeable cored structures, the values are only assumed and have not yet been related to the actual core permeability.

In his review of the three main design methods, Powell concludes that, although each of the calculation method discussed has its uses, the Hudson method has important limitations and should only be used to obtain a rough initial estimate of rock size for preliminary design. The method in CIRIA 61 is more restricted than that suggested by van der Meer, but is well tried and tested. The report itself is extremely comprehensive covering all aspects of riprap design. However, there may be circumstances under which rock sizes obtained using CIRIA 61 should only be used as an initial estimate.

Powell suggests that van der Meer's formulae are the most advanced and most widely applicable of the prediction methods currently available, and are based on the widest set of model test data. However, their use requires a subjective estimation of the core permeability of the structure, and it may therefore be advantageous to assess the sensitivity of the calculated armour size to the chosen permeability factor. Nevertheless, for many structures, van der Meer's formulae would appear to offer the best prediction of armour size. It should, however, be noted that the test results presented show noticeable scatter and the author does not give confidence limits to the prediction formulae. It may be noted that the work of both van der Meer (Refs 78, 79) and Timco et al (Ref 130) has illustrated the reduction to the stability of the armour with decreasing core, or underlayer permeability.

Hall, Rauw & Baird (Ref 50) describe the evolution of an unusual sea wall design falling into none of the categories considered above. A number of alternative designs for the protection to a runway extension were considered. The most economic design used a widely graded rock (3.9-19t) in an s-profile. In this configuration the structure performed essentially as an artificial rock beach until it achieved a stable configuration. The tidal range in this area is generally around 1m, up to 2m at extreme tides. This limited tidal range allowed the formation of a stable profile without requiring uneconomic volumes of quarry rock.

A useful summary of the design approach for armour systems for rubble mound structures is given by Baird & Hall (Ref 10). They mention the use of the Hudson formula, but conclude that its limitations preclude its use for other than the preliminary assessment of the dimensions of quarry stone armour units. They recommend that any preliminary design should be model tested in a wave flume to provide the final design.

5.5.4 Concrete armour units

In locations where suitably sized rock is not available, special concrete armour units may be used. Such units are often hydraulically more efficient than rock. They rely not only upon their weight for stability against the disruptive wave forces, but also on interlock and inter-block friction. Incident wave energy is dissipated primarily in turbulence in the voids between the armour units. As the voids ratio increases, so does the stability of the armour layer. Whillock (Ref 138) suggests that the armour stability may increase as the fourth power (at least) of the voids ratio, thus explaining the very high apparent stability of the more sophisticated armour units in model tests. Baird & Hall (Ref 10) describe the introduction and use of concrete armour units, as does Stickland (1983), and they list many of the original references. The SPM and Hudson (Ref 59) also describe many of the concrete armour units available. Those that have been used in sea wall construction around the UK include:- the dolos, Merrifield & Zwamborn (1966); the tetrapod, Danel, Chapus & Dhaillie (1960); the stabit, Singh (1968); the cob, Coode and Partners (1970); the SHED, Wilkinson & Allsop (Ref 140) and Allsop (Ref 5); and the diode, Barber & Lloyd (Ref 11). Of these units, the dolos and tetrapod are normally laid in two layers, the stabit in a quasi-single layer known as brick-wall and the cob, SHED and diode regularly in single layers.

In general, only the preliminary design of armour layers with concrete units is based upon design formulae. It is recommended that the preliminary design should then be subject to model tests to confirm its hydraulic performance. Clifford (Ref 28) summarises the types of tests that a designer might require, and discusses the evaluation of test results. Physical model tests in wave basins and flumes allow the effects of different armour units, armour weights, placement methods and densities, and crest and toe details, to be explored and compared. The use of such models as part of the design process is also discussed by Stickland (Ref 116), Burcharth (Ref 27), Baird & Hall (Ref 9), and Owen & Allsop (Ref 94), and Owen & Briggs (Ref 95).

Hydraulic models have not normally specifically tested the structural design of the armour units. The breakage of concrete armour units at prototype scale, however, is discussed by Burcharth (Ref 25), Magoon & Baird (Ref 75), Silva (Ref 110) and Timco (Ref 129). Various attempts have been made to model or estimate the stresses induced within an armour unit under wave attack. The major problem has been to estimate the loads to which such units may be subjected under prototype conditions. The structural design of the armour unit itself is still therefore a matter for continuing research. A number of design methods have been proposed, principally by Baird & Hall (Ref 9) and Burcharth (Ref 26). To date these approaches to the structural design of the armour unit have generally concentrated on the dolos unit.

5.5.5 Revetment armour

Revetments subject to wave attack may be armoured with open rock or concrete armour units considered above, or by more closely-fitting rock or concrete armour systems. The wide variety of rock and concrete revetment armour systems are covered in general reviews by Moffatt and Nichol (Ref 81), Welsby & Motyka (Ref 134), and McCartney (Ref 76). The last of these lists all types of revetment armour recorded in the USA up to 1976, amounting to 25 categories. McCartney gives brief details and sketches or photographs of the installation. Moffatt and Nichol, and Welsby & Motyka consider the construction and performance of a wide variety of revetment types intended to be of relatively low-cost. In general, little design data is available. Most that is published concerns concrete blockwork.

Concrete blocks have been used successfully for revetment protection for many years. Many kilometres of sea and river bank protection were built in the UK after the disastrous floods of 1953. Much of this reconstruction and improvement still exists today, a tribute to the high standards of workmanship then prevailing. Thorn & Roberts (Ref 127) give examples of blockwork including the interlocking Kent blocks and the W-blocks also described by Whillock (Ref 135). They also give examples of asphaltic jointed concrete blockwork at Pett and Sheerness, and grouted stone blocks at Dymchurch and Sheerness.

Hall (Ref 49) and Giles (Ref 38) report on the results of large scale wave flume tests on various types of concrete block revetments. Hall tested two types of interlocking blocks. Giles tested revetments armoured with standard (American) concrete building blocks. Both sets of tests used regular waves in a single width flume. Scale model tests of interlocking W-blocks are reported by Whillock (Ref 135), who also compared his results with some of Hall's. In later work, Whillock tested small non-interlocking blocks on support conditions equivalent to underlayers of various porosities (Ref 136). These fundamental tests led Whillock to conclude that close-fitting blocks on relatively impermeable foundations are more stable than loosely fitted blocks, or those on porous underlayers.

Moffatt and Nichol also report prototype tests of concrete blocks, including Jumbo and Gobi blocks. Brown (Refs 20-22) reports both site experience, and regular wave model tests at small scale, on a fitted hexagonal pipe revetment unit, the Seabee. Seabees have been produced in a range of sizes from 8kg to 8 tonne, in vitrified clay or concrete depending on size, for different degrees of exposure. Brown presents design formulae based on "blanket theory" for the use of Seabee revetments.

Thomas (Ref 123) reports the installation of a revetment armoured with relatively large (3.5 tonne) double-wedge concrete blocks on a sea wall revetment at Felixstowe, but gives no design guidance.

The recent production of cabled revetment block mats has stimulated considerable interest in a more mechanised style of revetment construction. The first mat of concrete blocks used the Gobi block, special concrete blocks glued to a geotextile. The Gobi mat was tested at CERC under regular waves by McCartney & Ahrens in 1975 (Ref 77). An example of the use of the Gobi mat is given by McCartney (Ref 76).

More recently tests on Armorflex have been conducted in the USA by Weckman and Scales (Ref 131), under regular waves and at small scale, and in Holland in the Delta flume by Lindenberg (Refs 12, 73). Trial lengths of both Armorflex and Petraflex revetment mattresses have been installed at Heacham by the Anglian Water Authority, but the performance has not yet been reported.

Among the various types of flexible revetment systems mentioned in the literature are:-

Armorflex
Amorloc
ACZ - Delta mat
Dycel, Dymex and Dytap blocks
Flexible - slab
Petraflex
Pro-fix
Terrafix
Shiplap.

However, relatively little further has been presented in the technical literature on the design of such mattresses against wave attack.

Recognising this, Powell, Allsop & Owen (Ref 99) have reviewed the hydraulic performance and stability of a wide range of concrete blockwork systems. They describe the stability of blockwork in terms of the dimensionless parameter $H/\Delta D$ used by Pilarczyk (Ref 96). Limits for $H/\Delta D$ are suggested for various block types:-

(a) Tongue and groove blocks,

$$H/\Delta D = 4.1 \quad \text{for slope 1:2} \tag{5.29}$$

(b) Tongue and groove blocks with relief slot to reduce uplift pressures,

$$H/\Delta D = 7.3 \quad \text{for slope 1:2} \tag{5.30}$$

(c) Shiplap blocks with spacers to relieve uplift pressures,

$$H/\Delta D = 5.7 \quad \text{for slope 1:2} \tag{5.31}$$

It is noted that cabled mattresses may afford greater stability than plain or simply interlocked blocks. The actual stability increase due to the cabling is very difficult to assess. Lindenberg (Ref 73) makes the implicit suggestions that such systems should be designed to resist wave action without the cabling. Perhaps the most complete account of cabling systems is given by Wise (Ref 141). This covers the economics and constructional details of such systems, but not, unfortunately, the stability aspects.

Discussing the use of simple stability formulae, Powell et al (Ref 99) note that flexible concrete revetment systems are, however, often specified in terms of mass per unit area, W/A. This parameter may be compared with the notional block thickness D:

$$\frac{W}{A} = C_1 \, \rho_c \, D \tag{5.32}$$

where C_1 is a shape coefficient specific to each type of block, and ρ_c is the mass density of the concrete used (kg/m^3). It is suggested that the stability expression given earlier might then be written in the following general form:

$$\frac{W}{A} = H \cdot C_1 \, K_1 \, \frac{\rho_c}{\Delta} \tag{5.33}$$

where K_1 is a stability coefficient covering the block and underlayer permeabilities, wave period, steepness and slope effects. It might be possible to generalise design information using this approach. However, values of the coefficients are, as yet, not directly available in the literature.

In conclusion, Powell et al note that several authors have suggested that the most stable construction of a revetment using concrete blocks would consist of close-fitting blocks on an impervious foundation.

They note that a revetment with a high degree of interlock/interblock friction will be more stable, but will have a reduced flexibility. It will therefore have a tendency to span voids, which may result from settlement of the underlayers and/or poor construction practice, and will thus be susceptible to wave impact loads. Conversely a flexible 'loose' block system may well follow the contours of the under layers, thus lessening the effect of wave impact loads, but in doing so would present an uneven surface vulnerable to wave induced drag, inertia and lift forces. The stability of such a revetment would therefore be reduced. It follows that the embankment design and preparation must ensure that only very small settlements occur. If large settlements are anticipated, blocks are perhaps not the best choice of protection.

Wave period has also been shown to have an influence upon the stability of revetments. Revetments often suffer more damage at lower wave heights for the longer wave periods than for the shorter wave periods. This effect can be accounted for in revetment design by the use of a minimum stability number.

Aspects of modelling of revetment systems, and the underlayers and foundation materials, are considered by den Boer, Kenter & Pilarczyk (Ref 16), Kostense & den Boer (Ref 71), and Lindenberg (Ref 73). Prototype tests on revetments in a shipping canal are also described by Pilarczyk. The specific aspects of scale effects are discussed by Kostense & den Boer.

5.5.6 Underlayers and filters

Much attention has been paid to armour systems, but until recently relatively little attention has been devoted to the other elements of the rubble mound structure. The Shore Protection Manual gives very little space to the design of the underlayers and core. Recently, Hedges (Ref 52) has reviewed and summarised the various demands made upon the underlayers and core of rubble structures, both in service, and during the construction period. The needs for a particular layer to offer support to the layer above it, and protection to that below it, are discussed. Hales (Ref 48) describes the filtering and separating functions of underlayers in rubble construction. Both Hedges and Hales remind the designer that an underlayer acting as a filter must be many times more pervious than the layer beneath it, in order to allow drainage flow. Further, the filter must be of such a gradation that the base material is retained. The design rules for granular filters as given by Hedges and Hales may be summarised:-

$$4 < \begin{array}{l} D_{15f}/D_{85b} < 5 \\ D_{15f}/D_{15b} < 20 \\ D_{50f}/D_{50b} < 25 \end{array} \qquad (5.34)$$

where the subscript f refers to the filter, and b to the base material below it.

A more severe criteria is suggested by de Graauw et al (Ref 43) who present results and conclusions of fundamental work on flow along and across core/filter interfaces under both steady and cyclic flow. Tests with coarse sands under cyclic flows at periods around 10 seconds (close to many prototype situations), revealed that the critical hydraulic gradients for the onset of sand transport through the filter under cyclic conditions are substantially lower than for the steady flow situation. For a safe design, it is recommended that the ratio D_{50f}/D_{50b} should not exceed 2 or 3 in the case of strong cyclic flow. Van Oorschot (Ref 89) discusses the work of de Graauw et al and also concludes that a safe design rule is :-

$$D_{50f}/D_{50b} < 3 \qquad (5.35)$$

Van Oorschot points out that this implies a relative weight ratio W_{50f}/W_{50b} less than around 25 to 30, a ratio often well satisfied by the armour/underlayer of conventional rubble mounds.

In some instances the installation of a gravel or quarry stone filter system may pose severe problems. In such circumstances a synthetic filter cloth or fabric may be used to substitute for all, or some, of the granular under or filter layers. Dunham & Barrett (1974) describe the early use of filter cloths up to 1974. They detail general design considerations for fabric filters. Typical sections of sea walls incorporating filter cloths are shown. Thorn & Roberts (Ref 127) quote work by Ogink (1975) giving recommendation for the 90th percentile opening of the filter cloth O_{90}, in terms of the D_{90} of the base layer, D_{90b}:-

$$O_{90}/D_{90b} < 1.8 \qquad (5.36)$$

Rankilor (Ref 102) summarises the use and design of both woven and non-woven synthetic filters in many areas of construction, including marine and coastal work. Papers and reports by Calhoun (1972), Ogink (1975) and Zitscher (1975) are discussed. Design rules for the use of synthetic filters under both steady and alternating flow are proposed for a variety of underlying soil sizes and types.

Moffatt and Nichol (Ref 82) also give a comprehensive summary of the types of geotextile available, their physical and chemical proterties, and the typical properties required for use in coastal structures in American waters. The text book by Koener & Welsh (1980) is cited as giving design guidance for a variety of uses of geotextiles. The design criteria required by the Corps of Engineers is based on the work of Calhoun (1972).

Hoare (Ref 55) reviews the range of geotextiles available in the UK, and describes the principal design approaches. He notes that the design of filters for reversing flows is not well understood. He discusses laboratory and site experience, and concludes that, for adequate filtering performance on cohesive base material, the fabric filter should be used in combination with granular filter layers.

The specification of fabric filter type and grade is addressed by Hoare (Ref 56) and Koerner (Ref 69). Both authors discuss the functions needed, the mechanisms of operation, and hence the filter type required. Hoare gives a comprehensive list of references.

A clear general description of the use of geotextiles in revetment construction, and of many aspects of their design is given by Ingold (Ref 60).

5.6 Crest and Rear Slope Armour

The crest and rear slopes of an embankment sea wall may be protected by either of two methods, or a combination of both. If subject to heavy overtopping, the crest and rear slope will commonly be protected with concrete or asphalt paving. When less severe conditions are anticipated, the crest and rear slope, and even some areas of the seaward face, may be adequately protected by grass. Recently combinations of perforated concrete blocks and/or geotextiles or geogrids have been used to reinforce grass. Such reinforced grass has been suggested for certain applications on dam crests and spillways. Under overtopping, the hydraulic conditions on such structures may be very similar to those on an embankment sea wall subject to intermittent overtopping. As, however, there is little design information available for such reinforced grass slopes, a CIRIA research project is being run to study the design of such slopes, subjected to infrequent flow. Interim results of this project have been discussed by Hewlett et al (Ref 54) and Kennard et al (Ref 65).

Relatively little information is available to the designer of protection to the crest and rear slope of an embankment sea wall. There are no recommendations for acceptable levels of overtopping of UK sea walls, except for those contained in two Japanese papers considered by Owen (Ref 91); those by Goda (Ref 39) and Fukuda, Uno & Irie (Ref 34). In particular, Goda recommends the following maximum levels of the mean overtopping discharge, $\bar{Q}$, for the stability of certain levels of protection to crest and rear slope:-

Structure	Max value of $\bar{Q}$:m^3/s
Paved (concrete) crest	0.2
Unpaved (grassed) crest	0.05
Crest and rear slope paved	0.05
Crest, paved and rear slope unprotected	0.02
Crest and rear slope grassed only	0.005

It should be noted that these are suggested values only, and have yet to be corroborated by any reported UK experience. Fukuda, Uno & Irie, present results of field measurements and observations of overtopping. Overtopping discharges were measured, and the effects on structures, vehicles and people analysed.

Bijker (Ref 14) reports that Dutch practice since around 1950 has been to design for a maximum acceptable value of $\bar{Q}$ of $0.002m^3/s$ for grassed slopes. This value was based upon prototype testing of "real grass mats" around 1958. Fukuda, Uno & Irie recommend somewhat lower levels of discharge for the safety of people, vehicles and buildings behind the sea wall.

The use of grass in coast protection and on coastal structures has been considered by various authors. The Shore Protection Manual, Knutson (Ref 67) and Ranwell (Ref 103) consider coastal and sand dunes and marshy areas. They suggest plant types, planting methods and densities for various applications. Ranwell also considers the use of vegetation in cliff stabilisation works, and the uses of shrubs and fencing in dune stabilisation and pedestrian control. None of these authors generally cover the use of grass on clay or earth embankments. That subject area is dealt with by Whitehead, Schiele & Bull (Ref 139) in CIRIA Technical Note 71, Hewlett et al in CIRIA Technical Note 120 (Ref 54), by Thorn & Roberts (Ref 127) and is touched upon by Ranwell. Whitehead, Schiele & Bull, and Hewlett et al, consider many aspects of grassing on hydraulic structures. They concentrate principally on structures subject to fresh water flow, but do also consider briefly sea wall banks, and the effects of sea water. They review site experience and make recommendations covering the grass types, mixtures, seeding rates and maintenance required. Thorn & Roberts summarise experience with grass types and mixtures used for sea defences in Kent.

Whilst a wide variety of perforated concrete blocks, goetextiles or geogrids, have been suggested by manufacturers as suitable for grass reinforcement or slope protection in the coastal environment, little supporting information has been found in the technical literature. Continuing research is described by Kennard et al (Ref 65).

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6 CONSTRUCTION MATERIALS

6.1 Rock

Rock, or stone, may be used in two distinct forms in the construction of sea walls. Selected quarry rock may be used in various sizes for rock armour, underlayers, granular filters and core material. Dressed quarry rock may be used for blockwork, copings, and abrasion resistant facings. The other major form is as crushed rock. In this form it may be used as aggregate in cement or asphaltic concretes, and similar mixes (mortars, grouts, mastics etc). Finally, ground limestone is also often used as a filler in asphaltic mixes. The wide range of rocks that may be used in the first form, as blasted and/or dressed rock, is considered in this section of the review. The use of aggregate is considered in section 6.2 on concrete and in section 6.5 on asphalt.

Not all rocks are suitable for use in the marine environment, nor for use as aggregates in concrete. Soft or faulted rock may deteriorate under wave attack. Reactive aggregates may disrupt concrete. The potential durability of different rocks, and suitable test methods are therefore discussed further below.

The three major types of rock are igneous, sedimentary and metamorphic. Stephenson (Ref 52) classifies 95% of the earth's surface rocks as igneous, with most of the rest sedimentary and some metamorphic rock. Igneous rocks stem from magma ejected from the earth's core and which has solidified at or below the surface. The manner in which the rock originally cooled will have affected both the overall rock structure and the grain size. Igneous rocks may be acid (including granite, rhyolite, diorite and andesite) or basic (including gabbro and basalt). Some sedimentary rocks may be used in marine construction, particularly limestone. Sandstones may also be of use, but this will depend on the degree of cementing of the matrix. Blyth & de Freitas (Ref 9) describe the formation and composition of igneous and sedimentary rocks in some detail. They devote a chapter to the use of rock materials in construction, principally sand, gravel and crushed rock, but also for blockwork or cladding.

Blyth & de Freitas also describe excavation and quarrying. They consider methods of drilling, the drilling characteristics of certain rocks, and describe aspects of blasting. A detailed note on the effects of the blast shock wave is appended. The interested reader is referred for further information on rock blasting to Langefors & Kihlström (1967). The various approaches to, and methods of quarrying rock for, sea defences are discussed by Quinn (Ref 48) and van Oorschot (Ref 42). Quinn describes American practice, but is somewhat dated. Van Oorschot particularly emphasises the need for careful planning and investigation, in the design process, and highlights the use of properly conducted quarry surveys.

The durability of rock in use will depend upon the fundamental properties of the rock, its degree of weathering and treatment in quarrying, production and subsequent handling. The mechanisms and degree of deterioration in use will also depend on the degree of exposure. Fookes & Poole (Ref 21), and Poole, Fookes, Dibb & Hughes (Ref 47) present four zones of exposure. These are reduced to three by Allsop, Bradbury, Poole & Hughes (Ref 6). Fookes and Poole, Poole et al and Allsop et al provide a comprehensive discussion on rock durability in the marine environment, both in UK waters and in the Arabian Gulf, and on the east coast of Australia. Fookes & Poole present preliminary considerations on rock durability, formulated at the start of a research study on rock durability in marine structures. Poole et al (Ref 47) and Allsop et al (Ref 6) summarise and detail, respectively, the final results of the study. During the study, roller mill and notched beam tests were developed and the results for various rocks were compared with characteristics measured by other tests. The authors give recommendations for test methods, (most of the tests are summarised by Allsop et al) and suggest acceptable values for the results, in order to give acceptable durability performance on coastal structures.

Moffatt and Nichol (Ref 35) discuss the use of a wide range of rock types of general application to American coastal structures. The four most significant rock types are: granites, basalts, carbonate rocks, and sandstones. Quarrying, handling and construction practices are all considered. A summary of some of the mechanisms for degradation is given.

The British Standard, BS 6349 (Ref 11) makes recommendations for a limited series of simple tests to check potential durability:-

(a) apparent relative density;
(b) water absorption;
(c) aggregate impact value;
(d) 10% fines;
(e) sodium and magnesium sulphate losses;
(f) aggregate abrasion value.

Details of the sulphate soundness tests are given in appendix B to BS 6349 and by Hosking & Tubey (Ref 30), otherwise readers are referred to BS 812 for details of the other tests, and recommended acceptable limits. Examples of the use of rock quality tests are given by Bradbury (Ref 10).

6.2 Concrete

Concrete has been used successfully in maritime works for many years. Concrete is the principal non-local constituent of most seawalls. It is used as revetment blockwork, crest and rear slope armour, rubble mound armour units, or large pre-cast cladding units. Concrete may also be used in bagwork, in mass concrete, reinforced or plain, placed below or above water. Well designed and placed concrete should be inert and resistant to chemical attack. It will be effectively impermeable to sea water and resistant to impact and abrasion damage. However, the marine environment is particularly aggresive and concrete deterioration often arises as a consequence of the expansive forces due to corrosion of steel reinforcement, chemical attack to cement or aggregate, or to abrasion or impact damage. The use and durability of concrete in the marine environment is discussed by Allen (Ref 3), Allen & Palmer (Ref 4), Cusens (Ref 19), Moffatt & Nichol (Ref 35), and Thorn and Roberts (Ref 56).

Concrete is produced from cement, aggregates, water and, in some instances, admixtures. Reinforcement may be provided by steel reinforcing bars, steel or plastic or glass fibres. BS 6349 (Ref 11) gives four main types of cement that may be used in maritime structures with the appropriate British Standard for each. Both Neville (Ref 40) and Lea (Ref 31) present details of the chemistry and properties of the various cements available.

Aggregates may be specified in accordance with BS 882 and 1201, certain suitable tests are given in BS 812. The quality of rock for maritime construction is also discussed by Neville (Ref 40), Cusens (Ref 19) and Allen (Ref 3). Alkali aggregate reaction arising from the use of reactive aggregates is discussed by Palmer (Ref 44) and Hobbs (Ref 28). The qualities of rock for maritime construction are also covered by Moffatt and Nichol (Ref 35) and Allsop et al (Ref 6).

The quality of mix water (usually specified as suitable for drinking) and the use of admixtures are discussed in BS 6349, Moffatt and Nichol (Ref 35), and by Neville. The occasional presence of various salts in drinking water is noted by Neville, who also covers use of sea water where appropriate.

The use of steel reinforcement bars in concrete is covered in great detail for general concrete usage by

Reynolds and Steedman (Ref 49). Moffatt and Nichol also discuss the use of various grades of (American) reinforcement steel in coastal construction. The use of steel fibres is discussed by Cusens (Ref 19), Godfrey (Ref 23) and by Hoff (Ref 29). Hibbert & Hannant (Ref 27) discuss the influence of both steel and fibrillated polypropylene fibres on the impact resistance of concrete. Morris (Ref 37) gives examples of the use of high-strength plastic mesh in sea walls.

The procedures involved in mix design, and a number of examples, are discussed in considerable detail by Neville (Ref 40). Less detailed summaries of mix design are given by Reynolds & Steedman (Ref 49), and Allen & Palmer (Ref 4). Some specialist aspects affecting marine structures are presented by Browne & Domone (Ref 14). The influence of mix design on durability is discussed later.

The concrete used in sea wall construction is often cast in situ. This aspect of concrete usage is covered by general text books and by Watson (Ref 58), Taylor (Ref 55) and Maquet (Ref 34). However, when a large number of repeatable sections are to be used, pre-casting may be used. Examples of pre-cast sea wall units are given by Paine (Ref 43), Taylor (Ref 55), Cartwright (Ref 17) and by Adaska & Cameron (Ref 1). The production and use of revetment blocks is also discussed by Thorn & Roberts (Ref 56).

Pre-cast caisson sea walls are particularly popular in Japan, but have also been used in Canada, and France. Aspects of their design, construction and placement have been covered by Nagai & Kakuno (Ref 39), Onishi & Nagai (Ref 41), Bertlin & Partners (Ref 8) and by Gerwick (1974).

The durability of both reinforced and plain concrete has received a great deal of attention in the technical literature. A number of research programmes and specialist conferences have concentrated primarily on identifying the mechanisms of degradation and suitable methods of prevention and for repair. The incidence and causes of concrete degradation have been explored from a geologist's viewpoint by Fookes and Poole (Ref 21).

In the UK the Concrete in the Oceans programme, funded in part by the Department of Energy, has given rise to much fundamental research into the mechanisms and rates of concrete degradation. This and related research programmes are reviewed by Sharp (Ref 51). Papers by Leeming (Ref 32), Brook & Stillwell (Ref 12), and Sharp & Pullar-Strecker (Ref 50), summarise some aspects of this research programme. Of the reports on work in this programme, those by Beeby (Ref 7), Browne et al (Ref 13), Wilkins & Lawrence (Ref 59), Paterson Dill & Newby (Ref 45) and Stillwell (Ref 53) are relevant to the performance of concrete in sea wall construction, particularly the corrosion of steel reinforcement.

In the USA, the American Concrete Institute sponsored a conference in 1980 on the performance of concrete in the marine environment. Among the papers edited by Malhotra (Ref 33), are papers by Mehta, Browne, Tuutti, Lin, Paterson, Wiebenga, Heneghan, Holm, and Sharp & Pullar-Strecker (Ref 50), that are of some relevance to the use of concrete in sea walls.

Few, however, of these papers cover the particular problems of abrasion that are common on shingle beaches in the UK and elsewhere. Abrasion of concrete is discussed by Allen (Ref 3), Allen & Palmer (Ref 4), Moffatt & Nichol (Ref 35), Thorn & Roberts (Ref 56), Fookes & Poole (Ref 21), and Allen & Terrett (Ref 5). Thorn & Roberts and Allen & Terrett give the results of abrasion tests conducted to assess the relative durabilities of different concrete mixes under shingle attack.

Durability of concrete depends to a considerable degree upon its permeability to sea water, and particularly its resistance to the ingress of corrosive salts. The permeability of concrete has been studied by Buenfeld & Newman (Ref 15) and Haynes (Ref 25). The influence of initial cube strength on durability is discussed by Cusens (Ref 19) and Deacon & Dewar (Ref 20). These last authors suggest that adequate durability will be ensured by specifying appropriate minimum strength levels. It is generally agreed that correct curing procedures are of critical importance in the development of strength and durability.

Many authors have identified failure at joints, or of the sealing materials, as major reasons for the deterioration or failure of some concrete sea walls. Allen (Ref 3) notes that many expansion joints in sea walls rapidly become inoperative as the sealant is washed out under wave action, and the joint fills with small stones. Allen also notes the continuing maintenance requirements demanded by poor detailing and formation of joints, and discusses preventative measures. Taylor (Ref 55) also gives particular attention to the care needed in producing durable movement joints in site placed mass concrete.

The use and selection of appropriate joint sealants is discussed in BS 6213: 1982 (Guide to selection of constructional sealants) and in the papers to an

American Concrete Institute conference edited by Watson (Ref 57). Moffatt and Nichol (Ref 35) devote 13 pages of their report to joints in concrete, and the various materials available for sealing. They analyse the purpose and mode of action of contraction, expansion, construction and special purpose joints. They include two comprehensive tables of sealant types and properties.

6.3 Timber

Whilst commonly used for dock work and jetties within harbours, and groynes and bulkheads on beaches, timber is not often used in sea wall construction (other than as a temporary component). However, some structures such as timber bulkheads, revetments or wave screens may be constructed as part of a sea defence or coast protection scheme. Timber piling may also be used in the construction of the toe of some structures.

Hester (Ref 26) summarises the use, availability, strength and durability of timber to be used in shore protection, and refers to various publications of the Timber Research and Development Association (TRADA). British Standard 6349 (Ref 11) also discusses the suitability of different timber types to various applications in the coastal environment. BS 6349 describes the mechanisms of degradation commonly found in UK coastal waters, principally fungal decay and the action of marine borers. Both Hester (Ref 26) and the British Standard refer to BS 5589 for details of timber preservation by chemical impregnation.

American practice is covered by Moffatt and Nichol (Ref 35), Graham (Ref 24) and Agi (Ref 2). Moffatt and Nichol present a comprehensive review covering wood properties, strength assessment and typical values, characteristics of common species, decay and degradation mechanisms, preservatives and treatment techniques, and jointing methods. They also devote a section to repair and rehabilitation of degraded members. Agi (Ref 2) also discusses the decay mechanisms for timber in American coastal waters. He suggests inspection methods and instruments (including an ultra-sonic scanner) to detect degradation and borer attack that is not readily visible. Graham describes the design of timber bulkheads (retaining walls) for granular fill. A graph giving design sizes of the various members of a standard design is presented (in imperial units). The design is for a type of bulkhead commonly used in American coastal waters known as the Navy wall. The calculation of curves on the graph was based on a back analysis of existing structures. No allowance for the effects of wave action is apparent.

Pedlingham and Hall (Ref 46), in an article published in 1962, described the production, typical sections, jointing methods and design characteristics of greenheart timber (Ocotea Rodiaei) as commonly used in maritime work.

6.4 Steel

Whilst often used in wharves and docks within harbours, and in groynes on the coast, steel is usually only employed in sea wall construction as toe or cut-off piles, and as reinforcement in concrete.

The Steel Designers Manual (Ref 18) mentions steel piling in marine structures briefly, but cites some references that might be of use to the interested reader. Morley, Waite and O'Brien (Ref 36) discuss the use and design of groynes formed with steel piling. They point out that design sizes of steel sections used in shoreline protection depend almost entirely on the anticipated level of corrosion and abrasion. Rates of material loss for corrosion alone and corrosion and abrasion are given from field measurements. Examples of both sand and shingle abrasion are considered. The recent introduction of an abrasion resistant grade of steel in a beach at Lowestoft appeared to have led to abrasion performance around 50-75% better than the mild steel comparison piles. Moffatt and Nicholl (Ref 35) describe properties of steel and other metals. They mention the introduction of marine grade steel in the USA but note that this affords no improvement in abrasion resistance.

6.5 Asphalt

Asphaltic and bituminous mixes have been widely used in Holland and Belgium in the construction of sloping revetments, sea walls, sea dykes, groynes and breakwaters. The use of such materials in the UK has been generally restricted to paving, some emergency repair work, and as a jointing medium for stone and concrete blockwork. A small number of sea walls in the UK have been built using asphalt grouted stone but little if anything has been published on the methods of construction and subsequent performance. Recently an asphaltic revetment of hog-back profile, similar in form to that used at Vlissingen in Holland, has been constructed at Porthcawl. A revetment of composite slope using both asphaltic concrete and Fixtone has recently been constructed at Prestatyn.

The British Standard, BS 6349 (Ref 11), devotes clause 66 to the composition of bituminous materials. Otherwise relatively little on these materials has been published in the UK. In contrast, a wide variety of publications on this subject has been produced in Holland.

Bituminous materials used for coastal structures may contain varying proportions of rock, sand, fines and bitumen. A detailed specification for each of these components, together with suggested quality tests, are given in the TAW report (Ref 54). The proportion (or absence) of each of these components will determine the properties and use of the resulting mix. Based on the TAW report, Motyka and Welsby (Ref 38) have defined some of the most commonly used mixes:-

(a) Asphaltic concrete
A mixture of stone, sand, fines and bitumen. It should have a voids ratio of 3 to 6% after compaction and is then virtually impermeable. Used as a watertight revetment above high water, or as lining for dams, canals etc. Must be laid and compacted in the dry.

(b) Asphaltic grout
A mixture of sand, fines and with bitumen in sufficient quantity to overfill the voids. Applied hot, by pouring or hand-floating into place. Stone can be added to bulk up the mix, ie when filling in large voids.

(c) Sand asphalt
A mixture of sand with some 3 to 5% of bitumen. It has a permeability very similar to the sand constituent. Can be used as a core material for reclamation bunds, as a filter, or as an underlayer for heavier revetment protection.

(d) Dense stone asphalt
A gap graded mixture of stone, sand, fines and bitumen. The bitumen slightly overfills the voids and hence the material is impermeable. First used at Ijmuiden with very large stones (70 kg max) for breakwater armouring. When lighter stone is incorporated the asphalt can be used as a grout.

(e) Mastic
A mixture of sand, fines and with bitumen in sufficient quantity to overfill the voids. It is naturally dense and requires no compaction. Apart from its use as a lining material above and below water level it is often used as a grout.

(f) Fixtone
A proprietary name for a permeable, so called "open stone" asphalt, developed by Bitumarin BV. The gap graded stone (usually 20/40 mm limestone) is bonded with mastic. It is very much an underfilled mix and hence very permeable. Fixtone can be prefabricated in mattress form, and hence can be laid below the water line, for example, and used as protection against scour. Commonly used

as bank protection in canals etc. Also used as a revetment, generally above the water line. A recent series of tests have been conducted by the Delft Hydraulics and Soil Mechanics Laboratories into the behaviour of a prototype Fixtone slope, and is reported by Burgher & Visser (Ref 16).

Detailed definitions of asphaltic concrete, sand mastic, stone asphalt and lean sand asphalt, and a summary of their use are given in BS 6349. Further details of the composition and use of bituminous mixes are given by Thorn & Roberts (Ref 56) and van Garderen & Mulders (Ref 22). Some detailed example mixes are given, both for asphaltic concrete and for blockwork jointing compounds.

A comprehensive description of asphaltic materials, design, construction and test methods is given by the Dutch TAW report, published in English in 1985 (Ref 54). These guidelines draw on site experience, analytical and empirical expressions to suggest design methods for various load states to which a coastal dyke, or sea wall, might be subjected.

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7 FORMS OF CONSTRUCTION

7.1 General

In their review of the design and performance of sea walls, produced as part of this project, Stickland & Haken (Ref 28) have identified a wide variety of structural forms used for such structures around the UK. They note that many sea walls may serve a multiple function. Their survey confirms that the choice of type of sea wall is strongly influenced by particular considerations of the site, as well as those of function and cost. Little general guidance is available from the technical literature on the selection of different forms of construction, probably because of the highly site-specific nature of the choice. Furthermore, most of the literature concentrates on particular aspects of the design performance, as already described in Chapter 5.

This chapter therefore covers that literature relating directly to the form of the structure, and covering examples of such structures, rather than concentrating on the hydraulic, geotechnical, and material aspects of the design, as discussed in Chapters 4-6.

7.2 Vertical, battered, or re-curved walls

The vertical, battered or re-curved walls favoured for various types of sea wall may be subdivided further, essentially by construction method. Where the structure is to be built largely above the water, at the back of a beach, or protected by a temporary bund or coffer dam, cast in situ concrete may be used, Watson (Ref 34). Shuttering may be temporary or may use pre-cast facing units, Paine (Ref 24) and Cartwright (Ref 6). In similar situations, a vertical faced wall may also be formed with pre-cast concrete panels tied back into granular fill, or held in steel H-section piles, Adaska & Cameron (Ref 1).

Vertical faced sea walls may also be used for reclamations, and in these circumstances caisson sections may be used. Such caissons may be of circular plan shape, as used at Brighton Marina, or rectangular in plan with either solid or perforated front face, Bertlin & Partners (Ref 3); Terrett, Ganly & Stubbs (Ref 29); Llewellyn & Murray (Ref 10); Marks & Jarlan (Ref 11); Onishi & Nagai (Ref 20). Similar sea walls may also be formed from various types of patented stacked interlocking blocks such as the cross-hollow, the Igloo, and the mono-bar, Benassai, Ragone & Sciortino (Ref 2), Shiraishi, Palmer & Okamoto (Ref 27) and Shimada et al (Ref 25).

7.3 Simple and composite slopes

Many non-vertical sea walls are constructed by a number of different methods, and/or at different times. These tend to lead to the construction of composite slopes incorporating changes of slope angle, and/or horizontal berms of varying width. Examples of such types of profile have been considered by many authors including Bertlin & Partners (Ref 3) and Kramer (Ref 9). The influence of wall profile on wave run-up and overtopping has also been studied by many authors including Thompson (Ref 30), Thompson & Shuttler (Refs 31, 32), Owen (Refs 21, 22) and Owen & Allsop (Ref 23).

7.4 Armoured revetments

Revetment slopes may be protected against wave attack by a wide variety of armouring systems. Examples of revetment construction are given by McCartney (Ref 12) and different types of concrete revetment blocks have been dfiscussed by Bertlin & Partners (Ref 3), Whillock (Ref 36), Naidenov & Zozoz (Ref 19), Hall (Ref 8) and Thorn & Roberts (Ref 33). Asphaltic revetments have been used widely in Holland and also in Belgium and France. Examples of such sea walls are given by Bertlin & Partners (Ref 3), McCartney (Ref 12) and Welsby & Motyka (Ref 35).

7.5 Rubble slopes, concrete and rock armour

Whilst the hydraulic behaviour of many examples of armoured rubble breakwaters are considered in the technical literature, in only a few instances have the design and construction of rubble sea walls been reported. The review of sea walls for reclamations by Bertlin & Partners (Ref 3) considers a number of rubble mound sea walls and rubble revetments. Many Japanese papers mention concrete armouring to sea walls, but generally give little detail. Some examples of the use of armouring are given by Nagai (Refs 17, 18) and Shiraishi, Numata & Endo (Ref 26).

7.6 Novel shore protection methods

A wide range of novel or low-cost methods of shore protection have been tried out, particularly in the more sheltered waters in and around the USA and Canada. These methods have included soil cement, sand or mortar filled fabric bags, gabions, scrap car or truck tyres, building block or pipe section revetments, and some asphaltic materials. Many of these methods have been considered elsewhere in this report. However, certain authors have concentrated on describing the use of low-cost protection methods. Motyka & Welsby have considered the use of scrap tyres (Ref 14), sand or mortar filled fabric bags (Ref 15) and gabions (Ref 16). They review experience with these structures principally in American waters. They do not consider that any of the methods considered are suitable for open exposure to wave action, as commonly found on UK coasts.

A major report on many different low-cost coast protection methods has been prepared by Moffatt & Nichol (Ref 13) for the US Army Corps of Engineers. This is the final report of the Shoreline Erosion Control Demonstration Program, designed to test and demonstrate methods of low-cost shore protection for shorelines exposed to light wave action only. McCartney (Ref 12) has also considered a wide variety of revetment armouring systems including:- soil cement, asphaltic concrete, sand or mortar filled bags, Nami rings, and building blocks, as well as more conventional revetment blocks. Experience and design considerations with the conventional revetment blocks and systems reported by McCartney (Ref 12), Hall (Ref 8), Giles (Ref 7) and Brown (Refs 4, 5) have been considered in Chapter 5 above.

Soil cement has been used successfully in various locations in Canada and the USA. Wilder & Dinchak (Ref 37) and Adaska & Cameron (Ref 1) report successful experience with soil cement slopes at Gaspé peninsular, Quebec, and at Bonny reservoir, Colorado. Further details of the use of soil cement to resist wave action on dam faces is given by Nussbaum & Colley (1972) and Holtz & Walker (1962)

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8 ADMINISTRATIVE, LEGAL AND FINANCIAL CONSTRAINTS

8.1 Administrative and legal

The design, construction, and maintenance of sea walls, and other shoreline protection structures, falls within the purview of two Acts. The Land Drainage Act 1976 provides for the arterial drainage of rural areas, urban flood prevention, and sea defence against tidal surges. Most of the work conducted under this Act falls to the Water Authorities, who have specific powers in relation to main rivers and to sea defence. Local authorities, District and County Councils, have the power to prevent and alleviate flooding in urban areas. District Councils are also authorised under the Coast Protection Act 1949 to undertake coast protection works, aimed at preventing land being lost to the sea by erosion.

Trafford & Braybrooks (Ref 14) present a comprehensive review of the administrative, legal and financial framework for the construction of new coast protection and sea defence works around the UK. They summarise the legal and financial framework supplied by the Coast Protection Act 1949, the Land Drainage Act 1976, and the Flood Prevention (Scotland) Act 1961. Examples of the level of grant available from the Department of the Environment (DoE) and the Ministry of Agriculture, Fisheries and Food (MAFF) are given. The division of the coastline and the differing responsibilities of the administering departments, are discussed.

Since 1 April 1985, the Government has transferred responsibility for coast protection in England from the DoE to MAFF. This and other proposed major changes to the legal and administrative framework of coast protection and flood prevention works are discussed in the Government's "Green Paper" of March 1985 (Ref 7). This suggests that Water Authorities would, in general, be responsible for all coastal works, but that District Councils could have delegated powers. Some of these changes are discussed by Capon (Ref 1), who describes the possible effects on local authorities. In particular Capon highlights some of the financial consequences of the proposed changes.

Hardy (Ref 4) summarises some effects of the present administrative, legal and financial constraints from the viewpoint of the district authority. Examples of recent schemes are given and the effects of procedural constraints discussed. It is argued that administrative procedures could be improved, and that the inspector's recommendations should only be overturned in exceptional cases. It is accepted that coast protection schemes can be subject to some form of cost/benefit analysis, but it is argued that this assessment should not become a sole criteria for the approval of such work.

Summaries of the history of the legislative and administrative framework affecting coastal works are given by Thorn & Roberts (Ref 13). Thorn & Roberts also describe the recommendations of the 1953 Waverley Committee, and the effect of revisions to the sea defence standards proposed by the Flood Protection Research Committee in 1978.

8.2 Financial

The costs of much of the sea wall and other coastal works in the UK is met by public funding. Capital funding on drainage and flood prevention schemes is supported by grant aid, provided on schemes approved under the Land Drainage Act 1976, and the Coast Protection Act 1949. In considering applications for grant aid, the Government has to be satisfied that a scheme offers an acceptable level of benefits in relation to the overall costs. Techniques to allow the quantification of benefits and costs have been advanced and discussed by various authors.

The use of cost/benefit studies for the assessment of sea defence works is summarised by Thorn & Roberts (Ref 13). The Ministry of Agriculture, Fisheries and Food (MAFF) support the construction of many sea defences as means of flood alleviation, and MAFF guidelines on the use of cost/benefit studies have been presented by Cole (1973).

The use of cost/benefit assessment techniques for public-funded projects in general has been discussed widely. The use of discounted cash flow methods in civil engineering projects was covered by the Institution of Civil Engineers (1969). The theory, and use, of cost benefit analysis methods is presented in the general textbook by Mishan (Ref 8). This work can be used to identify the many conceptual, technical and operational problems of using project appraisal techniques, such as cost benefit analysis. Further general comments on the use of these techniques, especially on UK public sector projects are given by Kiuper (1971), and in the papers edited by Layard (Ref 5). Whilst now a little dated, these papers, originally written between 1962 and 1972, discuss many of the various assumptions and methods inherent in the use of cost benefit analysis. A more recent assessment is provided by Sugden & Williams (Ref 12).

The particular problems associated with the economic assessment of flood prevention schemes have been addressed principally by the Flood Hazard Research Centre at Middlesex Polytechnic. The problems particularly relevant to sea wall design include those concerned with evaluating the "intangible" scheme costs and benefits, for which market prices do not exist. Other problems include the distributional effects of a project on the different populations affected, and some unintended consequences of the pursuit of economic analysis based on a concept of efficiency. These may mean that it appears to be economically more efficient to protect richer than poorer communities. Further it may appear financially attractive to pass coast protection problems from one community to another, rather than to devise in situ solutions.

Parker & Penning-Rowsell (Ref 9) and Chambers & Rogers (1973) concentrate on the use of cost benefit analysis in the assessment of flood prevention schemes. Estimating of the costs of damage due to flooding may be made using a manual of assessment techniques given by Penning-Rowsell & Chatterton (Ref 11). These costs may then be translated into present value terms. The example of the Whitstable Central Area coast protection scheme is discussed (Ref 9). Recently Parker et al (Ref 10) complement the earlier work (Ref 11) by providing more methods and data for assessing flood damages, particularly the secondary or indirect effects within local, regional and national economies. The identification and evaluation of benefits, and costs, not having an obvious market price are discussed in depth by Green & Penning-Rowsell (Ref 3). Examples are drawn from a study of flood defence proposals for Uphill in Avon. The authors describe many of the reactions of those affected by the possibility of flooding, and draw on new techniques to quantify and weight some of the benefits of a reduction of the possibility of such flooding recurring.

In contrast to flood prevention, relatively little has been written on the economic evaluation of coast protection schemes. In a report produced in 1980, Mackinder (Ref 6) considered some aspects of the assessment of costs and benefits accruing from such works. A procedure is proposed for the economic evaluation of coast protection schemes supported by grant aid by, at the time of the report, DoE. Particular problems of the assessment of intangible and other benefits are discussed. These may include those arising from the possible change of status of undeveloped land threatened by erosion, or the recreational benefits arising from beach nourishment schemes. The evaluation method suggested is considered in relation to case studies on schemes at Peacehaven, Minster, Bournemouth, and Hengistbury Head.

Dunkerley (Ref 2) discusses the administrative and financial considerations of the design of coast protection structures. He gives examples of coast protection projects on the 11km of coastline covered by Blackpool Borough Council. Attention is drawn to the use of finance from the European Regional Development Fund (ERDF), and loans from the European Investment Bank. Dunkerley points out that schemes aided in such a way, and above a certain sum in value, must be advertised in the Official Journal of the European Communities to enable contractors from all countries within the EEC to apply for inclusion on the tender list.

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9 CONSTRUCTION PRACTICE

9.1 General

The practical difficulties of constructing sea defence and coast protection works in the tidal zone subject to the effects of winds and waves can influence both construction methods and programming of the work, and must be considered at all stages of the design.

Watson (Ref 18) describes the way in which prevailing conditions can have a marked effect on construction. For example, tidal channels can determine points of access to a beach and can reduce the working time available. Watson also notes that where the proposed work is the replacement of existing defences, the need to minimise the period without protection may impose restrictions on construction methods.

Barrett (Ref 1) notes that a particular feature of coast protection work is the need to relate design to construction in such a way that each stage of the work is stable under wave action and beach variation.

Duvivier (Ref 4), writing in 1947, illustrated the way in which the design process is influenced by construction requirements. He noted that the limited amount of work which can be carried out in one operation between tides can dictate the lengths of individual sections of work, and the spacing of expansion and construction joints. He advised that sea walls be divided into cells, in order to limit the extent of any storm damage during the vulnerable stages of the construction period.

Thorn (Ref 17), in 1962, discussed work on reconstruction of existing sloping walls, particularly those on the south Kent coast around Dymchurch. He describes work using pressure injection of grout, and discusses the abrasion resistance of alternative designs of concrete mix.

Duvivier emphasised the need for simplicity of design particularly at the lower levels, where speed of construction is a primary consideration. Watson reiterates the need for simplification and standardisation of permanent works design. This will allow temporary works, and formwork to be economical, and construction operations to be quick and simple to perform when working time is limited. Watson adds that, if it is possible to precast some of the concrete elements of construction, this can minimise the amount of work to be done between high tides. Dunkerley (Ref 3) also recommends that standardisation and prefabrication be adopted wherever possible, particularly during operations preparatory to concreting. He makes the point that operations which reduce working time on the beach allow longer setting and curing time for the concrete, and consequently reduce the risk of damage to the 'green' concrete by the tide.

Watson states that the contractor must have sufficient resources of labour and plant, so that the unit of construction can be achieved in a tidal shift. The choice of plant, pumps, shutters, and other equipment are all influenced by the need for speed, efficiency, ease of handling and robustness.

Both Watson and Dunkerley emphasise that, for a contractor, timing of the work is of the utmost importance. Proper planning and programming enable best use to be made of the tides and seasons. Dunkerley noted that if seasonal work is not possible, as in some holiday resorts, then speed and ease of operations, in conjunction with timing, are critical factors.

During the course of the review, a number of papers on the design and construction of particular sea walls were identified in the proceedings of the Institution of Civil Engineers (ICE) and the Institution of Municipal and County Engineers (IMCyE), as well as a few in the technical press. In the main, the IMCyE papers are concerned with a specific location. They typically give a brief history of the coastal erosion and/or flooding in the area, and describe the construction of new works or the repair/replacement of an existing structure. Some papers include a typical cross-section of the wall, and details of costs of the work. A few papers, notably those by Mobbs (Ref 12), and Melville (Ref 10), discuss the design of sea walls, including the merits of different wall types, and give details of the design and construction.

The majority of papers in the ICE proceedings deal in more general tone, usually covering coastal erosion and, occasionally, design. Some give details of specific works. The other periodical articles tend to be site specific in approach, but may give techniques of general interest.

9.2 Historical or local aspects

Some of the papers and articles of historical or general interest are listed (Refs 2, 5-9, 11, 13-16, 19). Others are likely, however, to be of only local interest. These have therefore been listed separately in the Appendix to this report. The references have been grouped into geographical areas, using those given by Stickland and Haken in Figure 30 of CIRIA Technical Note 125. In the Appendix references have been listed in order, following the coastline clockwise, starting at the Scottish/English border near Berwick.

Area 1 is set between the Berwick/Northumberland border and Spurn Head, Yorkshire. General articles in this coastal area are presented by Moore and Seaton. Articles covering more specific sites deal with Blyth, Whitley bay, Tynemouth, South Shields, Sunderland, Hartlepool, Scarborough and Bridlington.

The coastline of Area 2 runs around the Lincolnshire and East Anglian coasts between the Humber and the Thames. A general paper on this stretch of coastline is given by Duvivier. Site specific references include locations at: Hunstanton, Great Yarmouth, Lowestoft, Harwich, Frinton and Clacton.

Area 3 covers the coastline from the Thames, around the south coast of England, to the Exe in south east Devon. Examples of coastal works on the Kent, east Sussex and south east Devon coastlines are discussed by Kemp, Stammers, and Hutton respectively. More particular sites are covered by papers on: Sheerness, Margate, Broadstairs, Ramsgate, Folkestone, Romney Marsh, Hastings, Brighton, Hove, Bognor, Selsey, Burton-on-Sea, Christchurch, Bournemouth, Poole, Weymouth, Lyme Regis, Sidmouth and Exmouth.

Area 4 runs around the Devon, Cornwall and Somerset coasts from the Exe to the Severn. A general article covering the Devon coastline is given in Contract Journal for 15.10.79. More specific references include locations at: Torquay, Carrick, Penzance and Blue Anchor Bay.

Area 5 covers the complete Welsh coastline from the Severn around to the Dee Estuary. Particular coastal locations considered are at: Porthcawl, Aberaeron, Barmouth, Llandudno, Colwyn Bay and Rhyl. A less site specific paper by Irving covers the north Wales coast.

The coastline of Area 6 runs along the Lancashire and Cumberland coasts from the Wirral to the Scottish/English border near Carlisle. Articles covering specific sites include locations at: The Wirral, Wallasey, Blackpool, Fleetwood, Heysham and Silloth.

Area 7 covers the whole of the Scottish coastline from Solway Firth on the Scottish/English border to Berwick. Site specific locations considered are at: Aberdeen, Methil, Burntisland and Edinburgh.

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APPENDIX

Further Papers, Construction and Historical Aspects

This appendix lists papers and articles of local and possible historical interests. The listing is divided into seven areas, and the references are given by the location covered.

Area 1: Berwick to Humber

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