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Coastal Structure and Breakwater Engineering in UK: From Research & Development through Dissemination to Application

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SUMMARY

This paper discusses the development of design methods for coastal structures and breakwaters in the United Kingdom, with particular attention to changes in research activities, publications, technology transfer and the use of research results over the last 200 years. The paper describes some examples of the design and construction of breakwaters and seawalls, methods by which their designs and construction techniques were improved, and how those improvements were promulgated within civil engineering. It discusses problems that arise from recent trends in research funding and publication, and seeks to indicate some possibilities for the future.

The paper draws on the author's experience in research and specialist consultancy at HR Wallingford, published papers and books, but is also guided by lessons from related work in UK and elsewhere.

1. INTRODUCTION

1.1 Geographical context

Great Britain (taken here as including the countries of England, Scotland and Wales) lies on the eastern shore of the Atlantic Ocean, receiving some protection from ocean waves from the island of Ireland, and in turn providing shelter to the soft coastlines of northern France, Belgium, Netherlands and Germany. The form of the main islands of Britain and Ireland, and the adjoining water bodies of the North Sea, Irish Sea, Bristol Channel, and English Channel gives rise to very large and very small tides. Tidal ranges around Britain at mean spring tides shown in Figure 1 vary from 1m to in excess of 12m, with much of the British coastline experiencing tidal ranges in excess of 4m at mean spring tides.

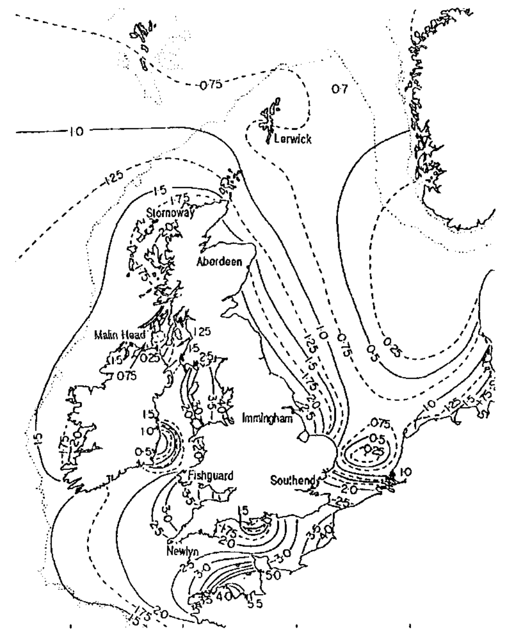

Fig. 1 Tidal amplitudes (50% of range) at mean spring tides (m)

Extreme water levels around the UK may be further elevated in times of severe weather by storm surges. These are greatest in the North Sea, where the 1:50 year surge residual along the open English coast may reach 2.5m, see Fig.2. These contrast

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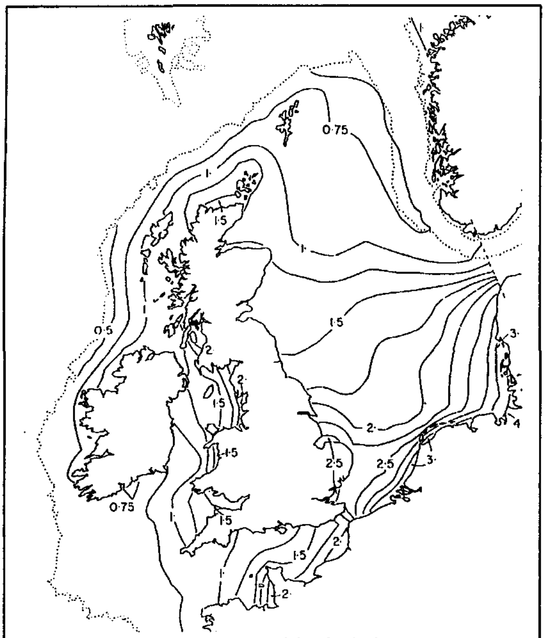

Fig. 2 1:50 year surge residuals (m)

with the coasts of Germany and northern Netherlands where 1:50 year surge residuals can exceed 4m. Around most of the open Scottish coast, surges seldom exceed about 1m, and for Wales surges similarly rise above 1.5m. Within tidal estuaries, surges are further amplified by constricting shorelines, and this often leads to locally increased surge levels.

In all countries around the North Sea, the prevention or amelioration of flooding of low-lying areas by tide, surge and storm has been one of the main motivations of coastal engineering, and continues to be the main economic and social reason for coastal engineering expenditure. In the UK, this has been particularly important since the great floods of 1606 when about 2000 people perished in floods around the tidal River Severn, and in 1953 when 300 died in coastal flooding along the English east coast.

Increased development along the coastline since 1953, particularly around the south and east of England has however raised rather than reduced the number of people at risk. Continuing development has increased pressure on low-lying coastal strips and around the perimeters of estuaries. Increased tensions over development / conservation have led to more procedures to inhibit coastal development, and to constrain schemes which might do damage, see Rendel Palmer & Tritton (1996), Fowler & Allsop (2000) for summaries of recent legislative and administrative procedures.

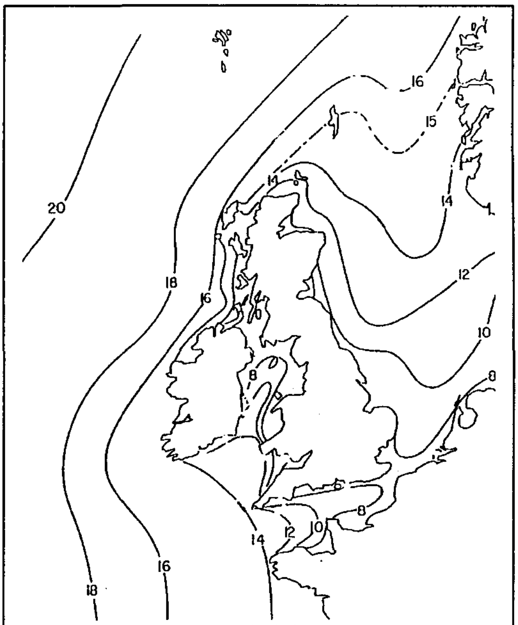

Fig. 3 1:50 year wave heights, H_s, (m)

The shoreline of Britain is also exposed to waves, the largest and longest of which are from the Atlantic, but these impinge mostly on the west coast of Ireland and Scottish islands. Waves in the North Sea and English Channel are generally smaller, see Fig. 3, although 1:50 year storm waves still reach $H_s \leq$ 10-12m for much of the Scottish east coast, English south-west, and $H_s \leq$ 8m along the Welsh coast and English east coast.

Such conditions are not however static. Changes to the North Atlantic Oscillation, and thus to storm patterns around the UK seem to have led to increases in wave heights as well as significant changes in storm directions. Simulated wave heights from wind records shown in Figure 4 illustrate steadily increasing wave heights on the English south coast.

1.2 Historical context

Britain has been a maritime nation since the beginning of recorded history, with trading and military activities ensuring that maritime activities have always commanded attention of politicians and society in general. This was particularly true during the 18th and 19th centuries when naval power was regarded as the key success to military campaigns, active or defensive, and port construction was particularly motivated by threats from France and Spain.

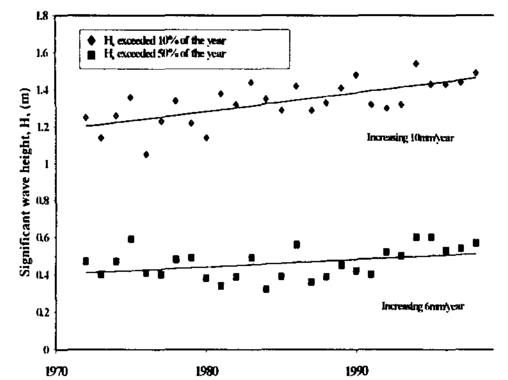

Fig. 4 Wave heights off southern England

Around UK, there was some increase of maritime activities from about 1750-1770, but the most dramatic expansions were between 1830 to 1890, stimulated by real and imagined military threats, and by dramatic increases in volumes of trade, and fishing. The earlier period between 1750 and 1830 saw increased reclamation protection in rural areas, construction of new harbour, and defences of coastal towns. Relatively slow advances in cement / concrete technologies, in diving and construction equipment limited construction, particularly water depths that might be accommodated, and therefore levels of wave attack that might be resisted. As construction materials and techniques developed strongly from about 1830, and demand expanded for reliable trading routes, so British engineers built new harbours or extended old harbours into deeper water and out into more severe waves. Particular examples in UK are harbours at Alderney, Dover, Peterhead, Penzance, Sunderland, Tynemouth, and Whitehaven, see Fig.5

This spirit of that age also drove further protection of coastal towns with new seawalls and promenades, and assistance in reclamation of coastal margins. Taken overall, civil engineers in this period generated a wealth of new infrastructure, and thereby created a number of abiding problems around the UK coastline, with which we now have to deal.

What happened in Britain after 1880-1900? The simplest answer is that the need for naval harbours substantially reduced with the onset of peace, and significant increases in marine trade drove most large harbours to commercial use, e.g. Dover, Peterhead. Very few new coastal harbours have been constructed in the UK since 1890, the notable exceptions being Port Talbot to support the steel industry in south Wales in the 1960s, and Brighton

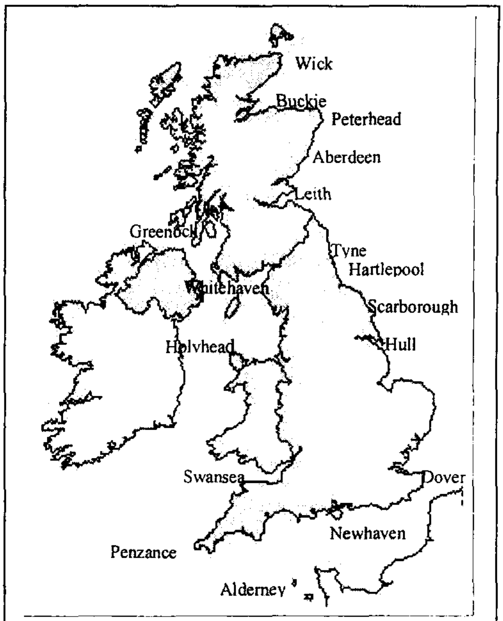

Fig. 5 Port construction, 1830-1890

Marina on the English south coast in the 1970s. Most expansion has been in the estuary ports, e.g. London, Southampton, Felixstowe / Harwich, and in airports.

Along the UK coastline, there has however been considerable recent attention to flood defences and coast erosion protection, leading to substantial attention to the economic, social, and environmental effects of defence options, and to the development and application of a suite of management and strategy plans around the British coastline.

Over the last half of the last century, 1950-1999, British civil engineers have directed considerable attention to overseas markets, where use of new design technologies and practical construction focus has assisted them design and build coast defences, breakwaters and related harbour works around the world.

2. COASTAL STRUCTURES AND BREAKWATERS

2.1 Structure types

Seawalls or breakwaters have been built since the earliest stages in man's development of the coastal zone. The primary purposes of such structures are to defend land against erosion and/or flooding; or to protect areas of water for navigation, anchorage or moorings. Many such structures are however required to serve multiple purposes, some of which may change in time. The main structure types may be summarised:

- coastal seawalls;
- coastal or shoreline revetments;
- nearshore breakwaters, reefs, or sills;
- groynes, bastions, rock headlands and related control structures;
- harbour breakwaters;
- entrance channel breakwaters or training moles.

The structures most commonly discussed in design manuals are simple vertical walls or breakwaters; impermeable and usually smooth revetment slopes; or rubble mound breakwaters or slopes. It is important in describing how these structures operate or fail. to distinguish between structure types which are:

- generally vertical and impermeable;
- sloping but impermeable;
- sloping and permeable.

Breakwaters are generally constructed to protect commercial / naval harbours or marinas; sometimes entrance channels for lagoons or estuaries; or cooling water basins for power stations. Such breakwaters may be constructed in 5 to 50m of water and, where exposed to large waves, may be armoured by special concrete armour units in sizes from 1 to 200 tonnes. [Sizes of concrete armour units above 40 tonne are only used in very rare situations.] Large breakwaters may be formed as rubble mounds protected by rock or concrete armour; or as vertical walls using blocks or caissons, themselves sometimes protected by concrete armour. Smaller breakwaters (usually rubble mounds) may be constructed parallel to the coastline to give local shelter from wave action with the intention of modifying the transport of beach material in their lee. Reefs and sills are essentially smaller versions of nearshore breakwaters used to retain beach material.

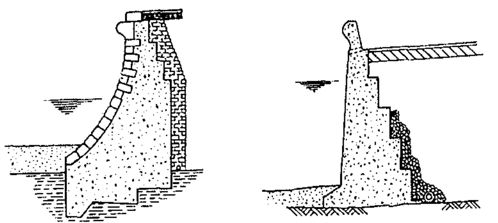

Fig. 6 Typical urban seawalls

Seawalls and revetments are constructed to defend against erosion, in the UK termed "coast protection"; or to reduce the level and/or risk of flooding of low-lying land by inundation from the sea, termed "sea defence". Seawalls may be generally vertical or steeply sloping, see Fig. 6, or they may be embankments protected by armouring. The revetment may in turn be generally smooth and impermeable; or may be rough and porous. Similar construction may be used whether existing land is protected, or is to be formed by reclamation, although there will be detail differences depending upon the hydraulic exposure and the erodibility of the fill material. Seawalls are substantially more numerous than large breakwaters, but many design methods derive originally from studies for breakwaters. Analysis / design methods have therefore usually been developed for larger structures, but are most frequently applied in the design of smaller breakwaters and seawalls. The key exceptions are methods developed to predict wave overtopping discharges, see especially Owen (1980), Allsop (1994) and Besley (1999), where methods derived initially for smooth seawalls have later been extended to rubble mound breakwaters, and then to vertical / composite walls.

The development of many structure types and their relative frequency of use in any particular region are often influenced strongly by historical, economic, and political / legislative conditions. In the UK, Europe, and in the USA, choice of structure types in use around the coastlines have been significantly influenced by central or regional government funding for defence against erosion and/or flooding. Since the early 1980's, increased interest in "soft" defences has started to change the types of schemes proposed and funded.

It is noted that in most highly developed areas, harbour and coastal defence structures have often been constructed in stages, with newer schemes constructed over existing works. Completely new construction is therefore relatively rare in the UK, often substantially complicating design decisions. Analysis of stability / performance and residual life of any existing coastal or harbour defences must therefore also take account of the previous history of the scheme and of its structures.

2.2 Vertical and composite walls

Many simple seawalls or breakwaters were originally formed from stone or concrete blocks laid to steeply battered or vertical slopes. The blocks might be doweled or keyed together, often allowing relative vertical movement, but restricting horizontal sliding. Seawalls of this type were often single-skinned, with beach material excavated from the toe trench being used as fill behind the wall, but most breakwaters used two blockwork walls, filled between with rubble, see Fig. 7. Later, when concrete technology had advanced, some structures used blocks throughout the structure without rubble filling, see example from Dover Harbour in Fig. 8.

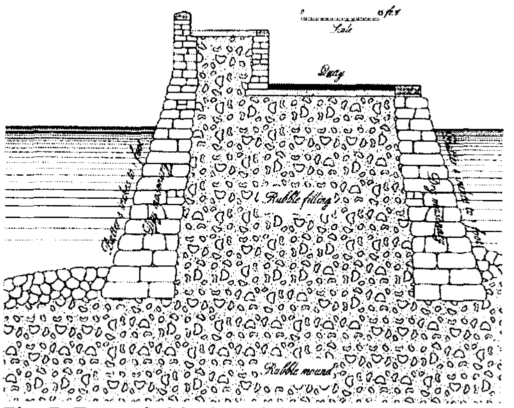

Fig. 7 Example blockwork breakwater, 1856

The main advantages of vertically faced structures lie in their efficient use of space and economy of construction material. Vertical walls have further advantages in harbours, particularly their visibility to navigators, and ease of mooring alongside. Five main types of vertical walled structure can be identified:

- Monolithic structures relying on mass for stability;
- Blockwork structures relying on mass for stability, and on joints or keys, mortar, or ties to maintain integrity;
- In-situ or pre-cast concrete retaining walls with granular fill;
- Sheet pile walls with granular back-fill;
- Screens or walls supported on piles.

It is often convenient to categorise the geometry of the first four types under three headings:

- Full-depth, where the vertical wall extends over the full depth of water, or most of it;
- Vertically-composite, where the wall is formed on top of a rubble mound;
- Armoured or horizontally-composite, where a mound of armour units is placed against the seaward face of the wall.

Design methods for the first two categories are generally addressed together. Those for armoured vertical walls draw on methods for both vertical walls and rubble mounds / slopes. The design of vertical or composite walls is complicated by the influence of any toe mound on wave breaking onto the structure. The type of wave breaking often significantly influences the magnitude of wave loading on the wall. Where the structure is significantly vertical, waves tend not to break onto the wall, and non-breaking or 'pulsating' wave loadings can often be assumed. If however the mound is relatively large, waves may more frequently break over the mound, and wave 'impact' loadings on the wall will give substantially larger forces. Where the mound is further increased, the rubble may serve to dissipate wave energy before it reaches the (crown) wall, and forces will be reduced.

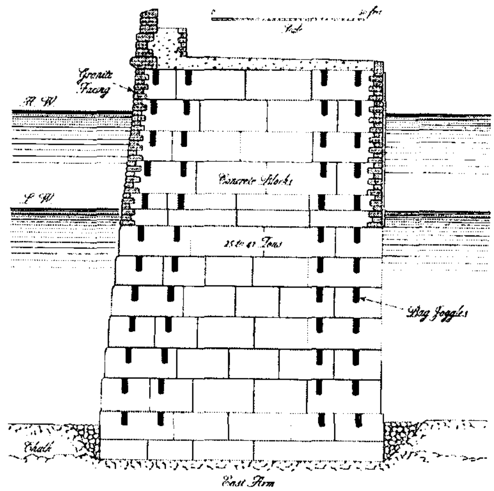

Fig. 8 Dover breakwater, 1898

2.3 Rubble mounds

The principal alternative to the vertical wall is the rubble mound formed from quarry rock or other fine fill material protected by large rock or concrete armour units. Side slopes are generally shallower than 1:1.33 or 1:1.5, so rubble mounds often occupy a greater plan area than vertical walls, and may require more material. In deep water the total volume of a caisson breakwater will often be significantly less than the competing rubble mound. It is also argued that construction with pre-cast caissons can be substantially quicker. Rubble mounds may however be preferred where rock is easily available; where construction plant or conditions are particularly suited to this form of construction; and/or where wave dissipation in the voids of the rubble layers is needed to reduce wave overtopping and/or wave reflections. In some instances, structures of these types can be formed mainly, or sometimes wholly, from concrete armour units, see section 2.4.

Many rubble mounds are therefore formed with a core of quarried rock or locally excavated material. These fine materials are not themselves stable against wave action, so must be protected by armour, perhaps rock armour or rip-rap, concrete armour units, or asphaltic materials. Typical rubble mound breakwaters consists of up to 8 main elements, some of which will be used in any rubble structure:

1. Foundation layer - rock laid to form filter or bedding layers over the natural foundation materials;
2. Scour apron - rock laid to form filter or bedding layers over seabed in front of the toe armour;
3. Core material - quarry-run or locally-excavated material, often with relatively little sorting or processing;
4. Toe armour - generally rock smaller than the main armour, placed at the toe of the front face, or part way up the front face on breakwaters in very deep water, supports the main armour and protects layers beneath;
5. Underlayer(s) - rock processed into (relatively) narrow gradings, laid to form filter or bedding layers over the core;
6. Rear armour - generally rock armour smaller than the main armour, placed on the rear face to resist overtopping waves, and direct and diffracted wave attack within the harbour;
7. Main armour - large rock or concrete armour, sized to resist direct wave attack, usually placed to a constant slope angle and layer thickness;
8. Crown wall elements - pre-cast or in-situ formed concrete elements, often including a roadway and/or a wave wall.

Any armour system must limit wave run-up and/or overtopping, and restrict reflections from the structure. Both of these are assisted by breaking the waves on the sloping face of the structure, and by dissipating wave energy in flow over / within rough and permeable armour layers. The structure elements contributing to these are the main and crest armour, the crown wall, and to a smaller extent the toe armour. The seaward slope angle and crest freeboard generally have the most significant influence on the hydraulic performance. Armour porosity and permeability are particularly important in determining the potential for wave energy dissipation, and hence both influence the armour stability.

The main loads on rubble structures arise from waves, currents, ice, and earthquakes, of which wave effects dominate in most circumstances. Loads due to ice, currents or earthquakes are less frequently damaging. The principal requirement of an armouring system is therefore dissipation of wave energy, and protection of the finer materials in the core. The armour must remain stable under wave attack, and should dissipate energy over and within the voids in the armour and under layer(s), thus limiting wave run-up and overtopping, and reflections. In resisting severe wave action, armoured structures may suffer damage or failure in many different ways. The main failure modes for which functional relationships have been established may be defined:

a) Armour movement on the front face - deemed to include rocking, displacement, and breakage of armour units.
b) Armour movement on the rear face - caused by wave overtopping.
c) Crown wall movement - principally sliding backwards or tilting under wave forces, horizontal and up-lift.
d) Slope failure - geotechnical failure of the mound not due directly to the removal of armour by wave action; includes spreading, settlement, slip or other shear failures.
e) Toe erosion - localised erosion of the foundation material at the toe of the breakwater.

2.4 Concrete armouring

Rock armour is not always available in the sizes needed to resist wave action, or may not be of the quality needed for durability. Rock is seldom available in unit sizes above 20 tonne; is relatively rare above 10 tonne; and realistic yields of rock armour in many areas of the world are limited to below 6 tonne. The development of larger vessels and hence the demand for harbour breakwaters in deeper water and/or exposed to larger waves has accelerated demand for large armour and for improved resistance to waves. In such instances, specialist concrete armour units may be used to replace rock armouring.

The development of specialised concrete units is relatively recent, although there is evidence that a type of concrete cube may have been used as armouring in Mediterranean ports by the Romans. The first concrete armour unit was generally cubic and gave little direct improvement in hydraulic efficiency, but did free designers from limitations in the size of rock armour available. Developments since 1950 of specially shaped concrete armour units like the Tetrapod, Dolos, Stabit, Tribar, Cob / Shed, Accropode, or Core-Loc have aimed to increase armour stability, primarily by increasing interlock between adjacent units, thus spreading the effects of local wave forces. Some units, chiefly Cob, Shed, Diode, and Reef, have also tried to reduce the wave forces acting on the units by increasing unit and layer porosity.

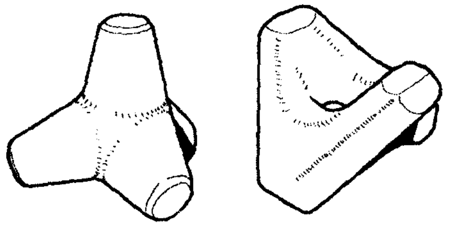

Fig. 9 Tetrapod and Stabit armour units

The simplest type of concrete armour unit is the cubic or rectangular block. On rubble structures, such units may be laid with random orientation to form a rough and permeable armour layer. On steep slopes (cot $\alpha < 2.0$) cubes may show a tendency to slide down-slope to form a less permeable face, thus increasing reflections, run-up and overtopping. This problem may be reduced by laying the armour in an open pattern on a rough underlayer, and by attention to the orientation of the lowest units on the slope.

Many types of concrete armour unit have been developed to give improved stability, and to reduce wave run-up and reflections. One of the first high-stability armour units was the Tetrapod, developed in France by Sogreah in 1950, and patented and licensed around the world. Widespread use in Japan has led to its use on more breakwaters than any other concrete unit. Patenting of this unit however also stimulated development of other types of armour unit, some of which were also patented. The reader should be cautioned that the term Tetrapod has occasionally been misused to mean any types of concrete armour unit. Another high-stability unit, the Stabit was developed and patented by Halcrow in the UK in 1960.

One of the most stable and hence interesting of these types of concrete units is the Dolos, Fig 10, developed in South Africa by the harbour engineer at East London, Eric Merrifield. Its apparently superior stability and hydraulic performance led to its rapid adoption in many designs, particularly in deep water. The inventor chose not to retain patent rights to the unit, so its use was relatively un-controlled and the inventor was not consulted in many uses of the Dolos. These units were therefore used widely by many different designers, and together with Tetrapods and Tribars, were involved on a number of breakwaters which suffered armour failure during the 1970-80s.

A recent development from the Dolos is the Core-Loc, see Fig. 10, devised by the US Army Corps of Engineers to give good interlock with Dolos armour, and thus being used to repair existing Dolos armour layers.

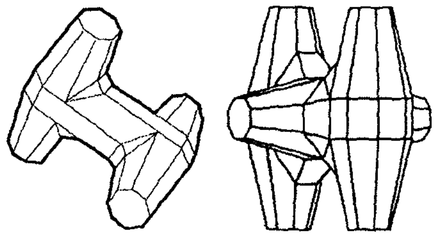

Fig. 10 Dolos and Core-Loc armour units

The units described here are generally un-reinforced, relying on the tensile and torsional strength of plain concrete to resist wave and settlement loads. At small unit sizes, the omission of reinforcement to carry tensile / bending / torsional stresses did not appear to give rise to significant armour breakage, but failures of a number of deep water breakwaters, including the breakwater in 50m of water at Sines in Portugal, during the period 1977 - 1985 revealed critical weaknesses of the more slender armour units under both static and dynamic loads.

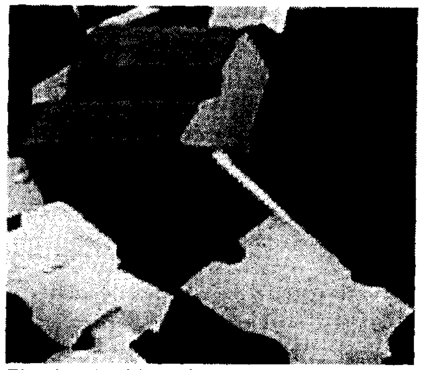

Fig. 11 Antifer cube armour

Many armour units developed since then have been more robust and/or massive. These include the Accropode, patented by SOGREAH as replacement to the Tetrapod, and which by spring 1995 had been used on 50 breakwaters or seawalls. This unit, is similar in appearance to the Gassho block, patented in Japan by Toyo Construction. The Antifer cube, Fig 11, a grooved and tapered cube was developed (but not patented) during the design of the oil port of Le Havre at Antifer, and has been widely used where a simple and robust unit is required, particularly in the Arabian Gulf. The Haro, a registered design by the Belgian consultant Haecon, has been placed on structures in Belgium and in Pakistan.

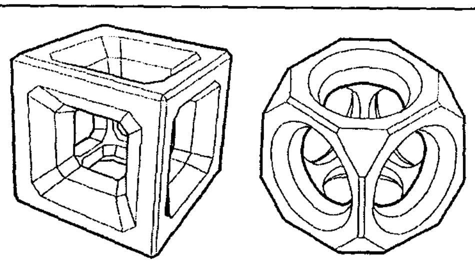

Fig. 12 COB and SHED armour units

The last main category of rubble mound armour units are the single layer, pattern-placed units of high porosity, see Wilkinson & Allsop (1983), Barber & Lloyd (1984), Dunster et al (1988). The COB is a hollow cube of 63% porosity, placed to form a close and regular array in a single layer. The COB demonstrates remarkable stability, as the close placement gives very good interlock with neighbouring units, and the very high porosity gives relatively low wave forces on the armour unit. As a result, the COB has only been used in a single size of 2 tonne, 1.3m side. The SHED, developed from the COB by Shephard Hill, uses an inflated bag to form the central void, and has also been produced only in a 2 tonne size. COBs and SHEDs have been used in about 20 - 30 structures around the UK, in the Mediterranean, the Black Sea, and the Gulf. Practical use of these units was described by Dunster et al (1988), and detailed research studies were presented by Allsop & Jones (1996), Toner & Allsop (1996).

The Tribar, a 3-legged unit, generally used in pattern placement in a single layer, was developed by in Hawaii., and has been used in the USA and Australia. The SeaBee is of hexagonal pipe form, see Brown (1983), and is placed to form a revetment with a relatively smooth surface, and permeability normal to the sloping face. Seabees have been used in Australia and the UK, and can be made in a range of sizes and thicknesses to suit the conditions.

2.5 Flood defences

In the UK, methods to defend against flooding from the sea vary with the local geology, the exposure, and land use. In rural areas, the most common defence is the simple embankment formed by locally-dug clay, usually un-armoured other than by grass. These embankments (generally at 1:2-1:4) were often later armoured by stone pitching, a thin layer of small stones placed tightly, and usually grouted with cementitious or asphaltic material. Later systems used square or rectangular concrete blocks, sometimes with overlap joints, or elsewhere bonded by bitumen joints. In areas where clay materials were rare, the coastal embankment may have been formed over earlier dunes, so the fill material would have been sand. This often led to shallower slopes, 1:3 to 1:6, and the armouring or revetment slope is more commonly formed by concrete using pre-cast elements, see especially Allen (1998), McConnell (1998), and McConnell & Allsop (1999).

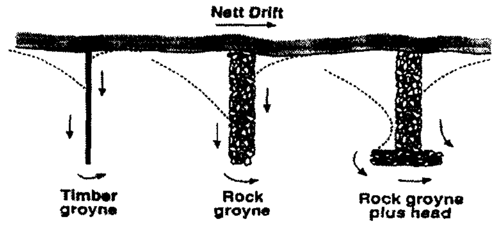

Fig. 13 Influence of groynes on beach processes

Over the last 10-20 years, more attention has been paid to the use of soft defences for flood defence, particularly the role of beaches and dunes or ridges in reducing wave action, and limiting overtopping. Beaches can be constrained or controlled by a range of structures, particularly by groynes or bastions, see Fig. 13, or by nearshore breakwaters. The design of such systems has required the development of new methods to predict beach responses, see for example Ozasa & Brampton (1979) and Powell (1990). The behaviour of the beach in limiting the severity of wave action at any back-beach structure is also of particular research interest, see for example Durand & Allsop (1997).

More information is also needed on the probability of high water levels at the same time as storm waves. Calculations of these joint probabilities, see illustration in Figure 14, are essential to the analysis of the standard of defence that can be provided by a particular defence scheme. Recent refinements to joint probability calculations are described by Owen et al (1997) and Hawkes et all (2000).

The final part in the analysis or design of any flood defence design is to predict the frequency / extent of floods for a wide range of future possibilities. Initially, the modelling used was over simplistic with maximum flood levels simply transferred onto digital ground maps of coastal areas and/or river basins. This gave indicative flood perimeters, but took little or no account of the defences, nor of the temporal nature of flood events. These initial approaches therefore led to substantial over-estimates of the probable flood area for given risk levels.

More complete methods have since been developed to include the probability and extent of any breaching in flood defences, see example in Figure 15 which shows flood probabilities, and to predict the extent of any flood given a time-varying input, say from one or two surge tides at the seaward boundary. These techniques have been discussed by Meadowcroft et al (1996).

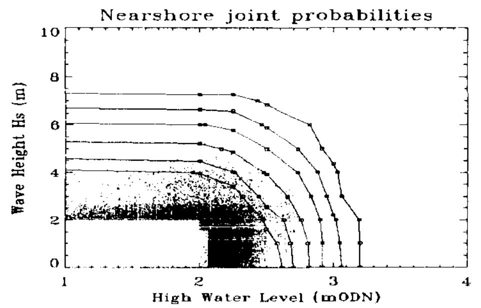

Fig. 14 Example joint probabilities of water level and wave height

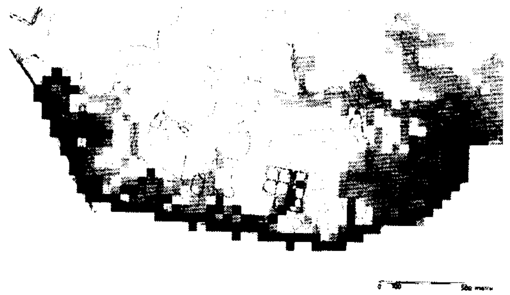
Fig. 15 Flood probabilities

3. RESEARCH AND DEVELOPMENT OF NEW DESIGN TECHNOLOGY

3.1 Observational techniques

Initial methods to develop new construction techniques were based on trial and error, the important aspect of which was that incipient engineers made and recorded observations of performance. This reinforced the need for detailed records of construction detail and technique. By early 1800s, such details were routinely discussed in technical papers or reports, together with observations of performance, see examples cited in section 4.1. The attitude to this approach to engineering is epitomised by the remark by Baker (1881) "If an engineer has not had some failures, ... it is merely evidence that his practice has not been sufficiently extensive, for the attempt to guard against every contingency in all instances would lead to ruinous and unjustifiable extravagance ...".

By the early part of the 1800s, the sciences of geotechnics and hydrodynamics were essentially unborn, and those of measurement and calculations of loads were not well developed, so arguments on the adequacy of particular designs, or their failure, were often hampered by lack of information. Design details like the use of "slicework" at Eyemouth in 1767 were often transferred to other designs, as in a later design for Peterhead. The reader of old texts is however cautioned that practice on site may often have differed from the design, or indeed the designer might be changed. Buchan (1984) shows an elevation of Peterhead North breakwater with masonry placed on the diagonal, implying that this slicework was designed by the Stevenson brothers That design was not however used in the outer breakwaters at Peterhead, but the final construction probably used a design by Sir John Coode with concrete blocks in horizontal courses with joggles on each course.

Design or constructional details were shared and compared from the earliest days, even internationally, as may be seen in description by Routh (1912) of construction between 1661 and 1684 of the main breakwater or Greate Mole to shelter the harbour at Tangier from the Atlantic. The Mole was started in conventional fashion with a rubble foundation mound placed ahead of a blockwork wall. Construction started in August 1663, and the Mole had reached a length of 380 yards (about 350m) by 5 years later! This slow speed was due to adverse wave conditions at the site; to loss of rubble fill into the (soft) sand bed; frequent diversion of the small and occasional workforce to military duties; difficulties in obtaining materials; and delays in payment.

After two year's delay for contract re-negotiation, the breakwater had been breached in two places. The construction method was re-considered, and a new contractor was chosen who proposed a type of construction recently applied in Genoa. The new contractor extended the breakwater using "great wooden chests" bound in iron, an early use equivalent to pre-cast caissons.

Wooden caissons of 500 to 2000 tons (Fig 16) were towed out from England, where each had been named as if a vessel. On site they were sunk into place by being filled with stone bonded with a local mortar. Progress was now quicker and less subject to damage than the earlier blockwork. Work continued until 1678, when Tangier was again attacked. Peace was concluded in 1680. The British occupation was lifted and it was decided that the breakwater should be destroyed lest it provide shelter to a later enemy. This demolition was completed in 1684 with more difficulty than had been anticipated, and marked an apparent halt in significant breakwater construction by British engineers, and certainly in the use of caissons, until at least the late 1700's.

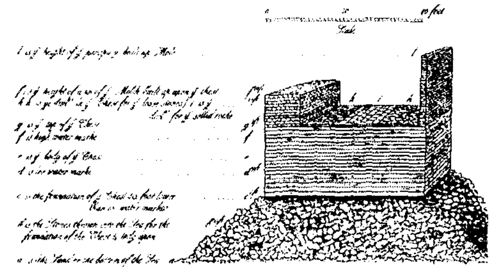

Fig. 16 Tangier caisson, 1677

In many ways, the history of Tangier was repeated many times over the next 200 years. An initial design based on common practice was extended to a new site, without much or indeed any assessment of the differences of wave exposure. {N.B. It was only around 1840-50 that Stevenson developed the first wave prediction formula, see discussion in section 3.2.} First indications of success, or failure, would then become apparent during construction. As this usually ran for some years, there was often ample opportunity for wave action to seek out and expose weaknesses in the design, and perhaps for the designer / contractor to modify the design accordingly. This was also a time of rapid change in construction materials and technology, two of which significantly influenced the development of another landmark breakwater.

The most notable example of design development, even if not leading to a final success, this is the breakwater at Alderney, a small island (2.5 km x 5.5 km) affiliated to UK, but located 13 km off the coast of France. The tidal range at Alderney is 5.2m, but local tidal currents often exceed 7-8 knots. Waves at Alderney are frequently severe, and in depths of 20 - 35m, Atlantic storms reach the breakwater with little reduction. The breakwater shelters Braye Harbour on the north-west side of the island, and was built for the British Navy to monitor the French port of Cherbourg. Construction of Alderney breakwater started in 1847. The initial design by James Walker included a mound to low water, surmounted by blockwork walls with rubble fill. Stone for the mound and walls was to be quarried on Alderney. The breakwater wall would be laid without mortar, and the hearting was not cemented.

Shortly into construction, it became clear that the original design was insufficient to resist wave conditions at Alderney, and geometry and construction were revised. The foundation level for the wall was reduced to about 4m below low water to reduce direct wave attack on the foundation stones. Until then simply placed tightly, these were now laid using mortar made with the recently available Portland cement (patented in 1828, but only commercially available from about 1845). The batter of the wall was steepened to increase gravity loads, and the fill between the walls was concreted. This construction was continued to a length of 823m by 1856, see Fig. 17, but still suffered frequent damage.

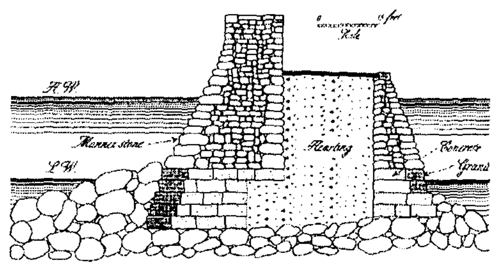

Fig. 17 Alderney breakwater, 1847 - 1870, improved cross-section

After 1856, the section was further revised, again reducing the level of foundation stones, and steepening the seaward face of the wall. Construction of the outer section was completed in 1864, giving a total length of 1430m or 4680 ft.

Even so, the breakwater continued to suffer severe damage every winter between 1864 and 1870, being breached frequently. In 1870, government engineers noted the instability of the mound and made recommendations for improvements. The need for the harbour had however reduced, so the wall was simply protected by stone dumped to maintain the mound and reduce movement of the foundations. Even this proved too costly and from 1873, the outer portion was abandoned to the sea and the wall collapsed leaving a mound crest about 4m below low water. For the inner 870m section, 20,000 tons of stone were dumped annually, and further work was still required to repair breaches in the superstructure. Dumping of rock ceased in 1964, since when the wall has been subject to repeated repair and maintenance. No further work has been expended on the mound.

Responsibility for Alderney breakwater was transferred to the States of Guernsey in 1987. The breakwater wall has continued to suffer repeated damage, and in recent years, a team of 8 men have re-pointed the face of the wall above mid-tide level, filled cracks and replaced damaged masonry each summer. A team of 6 engineering divers have worked on repairs to the toe of the wall, both above and below low water. During winter 1989/90, storms battered the breakwater for six weeks. At its peak on 25/26 January 1990, the storm reached offshore conditions of H_s=10 to 10.5m, and then again 11 and 12 February, again exceeded H_s = 9m. This pounding cracked the masonry facing, and a large cavity breached through the wall in an explosive failure audible around the island. Under planned emergency procedures, repair work started within 10 days at an eventual cost of £1.1 million.

The continuing difficulties of Alderney breakwater have been discussed at great length in government reports, and in Proceedings of ICE, see 4.1 below, Considerable detail on the design and construction of Alderney breakwater has been given by Vernon-Harcourt (1873, 1885), Stoney (1874) and Bishop (1950), and later analysis by Allsop et al (1991) and Allsop & Bray (1994). These analyses identified that some wave effects can be dramatically changed by the form of wave breaking. They assisted identify conditions that lead to violent wave breaking, and hence the importance of either keeping the foundation mound so that waves do not break over the mound onto the wall, as at Dover; or to raise the mound sufficiently to cause breaking before reaching the wall, as at Holyhead; or to remove the vertical wall completely, as at Plymouth.

When considering performance of these early breakwaters, it may be useful to note that the present British Standard (BS6349: Pts 1 and 7) suggests a minimum design life for breakwaters of 60 years, and notes that such structures are often designed for 50-100 years. If designed to present rules, many breakwaters built between 1830 and 1900 would now have served between 1.5 and 3.5 times their design life. In contrast, many designs in recent years for seawalls and breakwaters have quoted 30 – 50 year design life, and a design event return period of 1:50 years. Taken together, these imply a probability of 40-60% over the design life of exceeding the design condition. In this context, it is therefore particularly worth noting recent moves to assess formally the "whole life" costs of a design, allowing greater potential for designs with lower capital cost balancing periodic maintenance / refurbishment costs.

3.2 Theoretical, experimental and empirical methods

The next stage in design methods followed development of new techniques to calculate wave processes / forces, particularly the work of Airey, Rayleigh, and Stokes. These theoretical advances supported development of design methods using calculated wave loads (deterministic) and structural analysis, then calibrated by observations of success or failure on site. The earliest such approaches are probably measurements by Stevenson (1849, 1874) to estimate wave action, and determine wave impact loads acting on breakwaters and lighthouses.

Later stages involved measurements of loads or responses on site, and later in the laboratory. Some of early tests on wave forces with regular waves by Bagnold (1939) followed similitude rules by Froude. The most common generic result of such modelling is however the Hudson (1958) formula for rock armour stability. This empirical equation is significantly less complete than earlier formulae by Hedar, Iribarren and others, and is only valid for regular waves. Yet these weaknesses were significantly out-weighed by wide availability of model test data in technical reports from the US Army laboratory giving stability coefficients. Use of this method was then strengthened by publication of stability coefficients for concrete armour units, Hudson (1974). Continuing use of these coefficients and the method behind them is reinforced by wide availability of the Shore Protection Manual, CERC (1973, 1977, 1984).

The use of random waves in laboratories in Europe from about 1970 provided substantially greater realism in empirical descriptions of armour stability. Tests with random waves on wide-graded rock armour (rip-rap) for protection of dam faces were described by Thompson & Shuttler (1976). These tests all used waves of relatively constant steepness, so no effect of wave period was identified over that of wave height. Van der Meer (1988) later re-analysed Thompson & Shuttler's data with tests at other wave steepness, the effects of wave period, structure permeability, storm duration, and the level of damage were all included in a new method. Van der Meer's advances were rapidly confirmed, and extended by tests on thin armour layers, and different stone shapes described by Bradbury and co-workers (1988, 1990).

Surprisingly, few similar tests have been conducted for concrete armour units, and none that describe the response of more than one class of armour unit. The result of this is that there are no well supported data sets of comparative performance / stability. This lack of knowledge and uncertainty is compounded further when the overtopping performance of an armoured breakwater is considered, as it must be to make rational decisions over its crest level.

A major success of the use of random wave testing to support empirical methods has been the development in UK (and later elsewhere in Europe) of simple but robust methods to predict wave overtopping. With development in the 1940-60s of reliable methods to describe random wave process, and of the variability of tides and surges, came growing understanding that for all practical cases, there is no such design target as "no overtopping". For all realistic situations, a sea defence structure will be subject to stochastically varying tide + surge water levels, with randomly varying storm waves. Even under conditions of depth-limited breaking, any affordable defence structure must permit some (small) overtopping under extreme conditions. This philosophy was most clearly encompassed in the development by Owen (1980, 1982) of an exponential empirical method to predict mean wave overtopping discharges for embankment seawalls of simple and bermed form. Formulae and coefficients were developed for a wide range of wave conditions, structure variations, and relative crest levels, based on test data from random wave tests.

Since their development 20 years ago, these methods have been extended by further studies in UK, the Netherlands and Italy, giving more information on different structures types, and on the physics of the overtopping process. Such methods now constitute the main part of the design process to set crest levels for sea defence structures, and some breakwaters. Even with these advances, the weaknesses of all empirical methods are frequently tested by the sheer variability of structure types / configurations for which design calculations are required. This has driven the development of new generations of numerical models to simulate wave action and hydraulic responses, see section 3.3 below.

In recent years, research projects have become bigger, particularly when a wide range of skills or facilities are needed. Within Europe, two major research projects addressed the design and performance of vertical and composite breakwaters. The MCS-project led by University of Hannover contributed new research information on vertical breakwater and related coastal / harbour structures. Advances were made in identifying and explaining movements of some vertical breakwaters; initial development of dynamic design load criteria and methods; new design methods to dimension foundation mound protection, and to predict the onset of scour. New data were derived on wave overtopping and reflections, and especially on the effect of design modifications intended to reduce reflections, overtopping, and/or wave forces.

Research described by Oumeraci et al (2000) on PROVERBS (Probabilistic Design Tools for Vertical Breakwaters) was in four task areas:

Task 1: Hydro-dynamic aspects;
Task 2: Foundation aspects;
Task 3: Structural aspects;
Task 4: Probabilistic design tools.

Within PROVERBS, Task 1 on hydro-dynamics described by Allsop et al (2000) was intended to produce new guidance on types and magnitudes of wave loads, global and local, short or long. The primary objective of Task 1 was to supply essential data and prediction methods on hydro-dynamic loadings on vertical and composite structures to the other areas of the PROVERBS project. The second objective was to develop new prediction tools to be used more widely by consulting engineers, contractors, and owners in analysis of safety and performance of such structures. Example results from PROVERBS in Fig. 18 show a prediction graph for magnitude of wave impact loads; and in Fig 19 a parameter map to be used to identify types of wave breaking on vertical or composite walls. Other results of PROVERBS on field data measurements, standardised response parameter coverage; new data on performance of perforated caissons; identification of effects of steep bed slopes; and new data on wave induced pressures in rubble foundations, and up-lift forces beneath caissons are described by Oumeraci et al (2000), Oumeraci & Kortenhaus (1999), Belorgy et al (1999) and Bullock et al (1999).

The integration and interpretation of research results is itself a complex task. The example shown in Fig 20 illustrates a single prediction graph developed by McConnell & Allsop (1999) as part of the analysis for the Revetment Manual. The graph draws together a number of different design methods for rock armour, concrete blockwork, and slabbing protection on revetments.

3.3 Numerical modelling

Development of numerical models of wave action and hydraulic responses like wave run-up and overtopping has required some simplifications of wave processes. The simplest such models (now only of historical interest) gave a single response (transmitted or reflected wave heights for example)

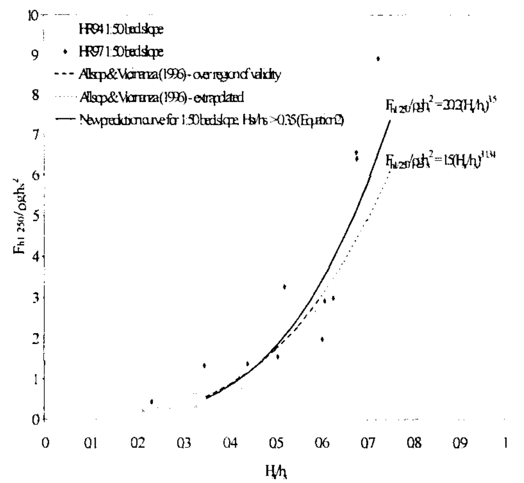

Figure 18: Wave impact forces, 1:50 bed slope

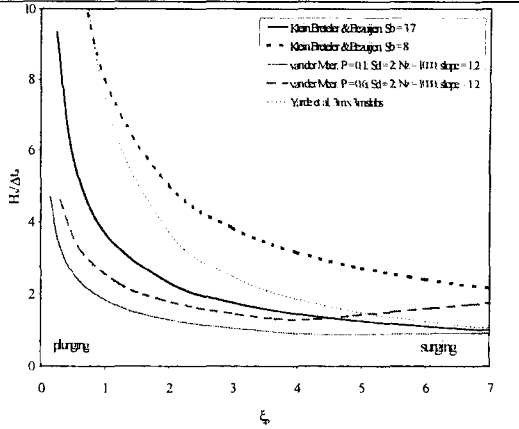

Figure 20: Comparison of design methods for armour layer thicknesses for different revetment materials

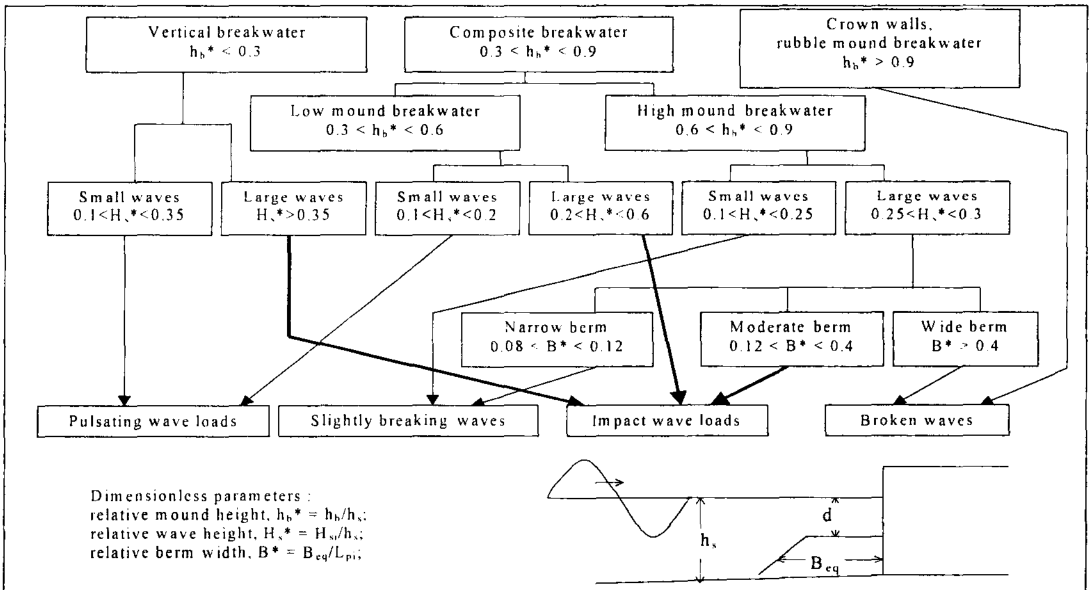

Figure 19 Parameter map to predict occurrence of wave impact loads

for a given wave height and period.

The next class of model calculates wave processes at time steps. Initially limited to a single wave height / period, these models were quickly improved to run trains of waves, but still required simplifications of the wave process. The most common and long-lasting simplification has been the use of one-dimensional depth-averaged (1-dV) flows using the non-linear shallow water wave equations for models such as IBREAK, MBREAK and RBREAK developed Delaware, SLACWAVE and ANEMONE OTT 1-d at Wallingford, and AMAZON at Manchester. These represent waves by a single water depth and averaged velocity at each point. probably first achieved by Packwood & Peregrine (1980), followed by Kobayashi et al (1986) and Allsop et al (1986). Such models are rapid and require minimal computational space, and reproduce bulk wave effects in shallow water up to the point of wave breaking. These models are now being extended to give two-dimesnional models in plan of wave processes in the coastal region. Even with these advances, these model types cannot predict variations of velocities / accelerations with depth, nor can they include the full detail of the processes of wave breaking / overturning. Two further classes of modelling tools may be used where those processes are required.

Boundary Element Models, as developed by Vinje & Brevig (1981) or Boundary Integral Models support the full calculation of velocities and pressures with variation in depth, but require a single continuous air-water surface. These models can be useful where a single deterministic event is required, particularly wave elevation, or wave surface profile / velocity are required to set the elevation of a deck clear of wave impacts. When a wave breaks, this condition is however breached and the model stops calculation. This type of model cannot therefore handle time series where a wave may break, nor can it simulate post-breaking or overtopping processes.

The numerical model type now most frequently adopted to overcome this limitation and thus to cover the full wave breaking process uses the Volume of Fluid method (VOF). Full wave breaking 2-d numerical models based on the VOF technique were first developed by Nichols, Hirt and co-workers (1980) at Los Alamos to model fuel sloshing in aircraft and spaceship fuel tanks. The same or similar techniques have been used by Delft Hydraulics in their SKYLLA model, see van der Meer et al (1992), and others. This model type has been developed to research level at Wallingford & Oxford Brookes University (OBU) in the fully 2-d breaking model NEWMOTICS, by Sabeur et al (1996), Christakis et al (1998) and Waller et al (1999).

VOF models have been successfully developed to research level in many research institutes and universities, but have not yet progressed sufficiently or been validated as reliable design tools. The standard problems with all of these models are the need for clear descriptions of all processes modelled, and of those not included. In the future, these model may be used for computation of wave pressures and forces on quays, jetties, breakwaters and seawalls, including up-lift and impacts; overtopping; and tsunami run-up, but at present, these processes still require more work on validation and in defining safe operation limits for particular models.

4. PUBLICATION AND DISSEMINATION

4.1 Proceedings of Learned Societies and Engineering Institutions

During the main part of the last two centuries, essentially from 1800 to 1950, publication of new knowledge and practice in coastal engineering was limited to (relatively few) proceedings of professional institutions and journals, in the UK chiefly the Transactions or Proceedings of the Institution of Civil Engineers, and the monthly periodical Dock and Harbour Authority. Scientific treatises were published by the Royal Society and by other learned societies. Technical articles, papers and proceedings of meetings described problems, offered and discussed possible solutions, see for example Vernon-Harcourt (1873), Stoney (1874), Howkins (1881), Latham (1905), Wilson (1919) or Bailey (1940).

With time, however, these papers tended to become slowly more theoretical, with fewer practising engineers presenting papers, or travelling to regular meetings and therefore to less detailed discussion. The original types of meetings reported in early transactions now became subsumed into seminars and conferences.

4.2 Research reports

From about 1940 onward, academic and institute researchers became more formally involved in development and application of improved design methods, thus developing a number of new specialisms within coastal engineering. In many countries, this led governments to set up or support specialist hydraulic laboratories or observatories, e.g. those at Wallingford, Bidston and Teddington in UK, Delft in the Netherlands and Vicksburg in USA.

These new institutes like HRS (later HR Wallingford) and the Bidston Observatory (later POL) provided industry-wide guidance on tide and water level predictions, models for wave predictions and transformations, and design of coastal structures. Their research results were published widely in authoritative reports or papers, see especially Barber & Ursell (1948), Darbyshire (1952), Tucker (1963), Draper (1963), Abernethy & Gilbert (1975), Ozasa & Brampton (1975), Brampton (1977). These and later studies on coastal structure responses were then used to support the development of new technical manuals and/or British Standards, see section 4.5.

4.3 Technical Conferences and Papers

The development of major conferences such as the International Conference Coastal Engineering published by the ASCE from 1950, started to increase the rate of publication and widen the flow of research information. These conferences initially attracted practitioners, owners, designers and contractors, but in the last 20-30 years many of these conferences (particularly ICCE) have become dominated by researchers and academics, and attendance by practitioners at those events has declined significantly.

This pressure for academics to publish to meet national research assessments, and the need for new conferences to address more practical needs, have together spawned completely new series of conferences to serve a range of different audiences. The biannual ICCE series continue under the auspices of ASCE, but are now joined by new series on: Coastal Structures ('79, '83, '99); the ICE Breakwaters conferences ('83, 85, '88, '91, '95, '98, and '01) see Clifford (1995) and Allsop (1998); the annual MAFF conference of river and coastal engineers in the UK; Waves '74, '93, '97 and '01, see for example Edge & Helmsley (1997); the conferences on coastal and port engineering in developing countries, COPEDEC; both IAHR and PIANC congresses; or the Antwerp International Harbour Congress.

Each of these conferences have widened the sources of information available, and the opportunities to discuss new advances, but their proliferation has perversely made it significantly more difficult for the practical engineer to find reliable guidance. This difficulty is compounded by the brevity of many papers (some conferences have reduced paper lengths to 8 pages!); by the habit of publishing progress reports as papers, perhaps to give young researchers conference experience; and by the general reduction of time for discussion at some conferences (sometimes reduced to 5-8 minutes per paper). Worse are those occasional conferences without a well-established publisher from whom proceedings can be obtained. Here it is quite possible for a researcher to "hide" a paper well away from the general gaze. On occasions, papers have been printed in conference proceedings before they have been subjected to significant critical review or questioning, and sometimes without presentation, leading to unsupported or even erroneous results being promulgated.

All of these weaknesses of aspects of current research publishing are compounded by the lack of detailed reports or archives for most academic research that could be consulted to give supporting information. Such reports were once written and archives were once commonly held at some universities, but appear no longer to be supported. This lamentable but widespread practice leads to an inability to check published work, for others to re-analyse particular data, and indeed for authors themselves to re-run previous analysis or simulations.

4.4 Technical and academic journals

Originally developed to spread knowledge of technical advances amongst relatively restricted audiences, technical journals have multiplied substantially over recent years in numbers and in cost. Apart from the proceedings of the ICE and ASCE referred to above, and the previous Coastal Engineering in Japan, the seeker after information on research results must now also consult Coastal Engineering, Coastal Engineering Journal, Journal of Coastal Research, Ocean Engineering, Applied Ocean Research, and Ocean & Coastal Management. Even this apparently full list will not however guarantee success for some areas.

Less successful in this area is the Journal of Hydraulic Research, dominated by riverine issues; the Journal of Fluid Mechanics, now dominated by a small group of mathematicians; or the PIANC bulletin with a publication philosophy biased towards "each nations turn".

Scientific journal publishing is a competitive market, but operates on somewhat unusual principles. The key suppliers of the technical material (the authors) are un-paid, indeed some journals in other areas have levied publication charges! The editors (most or perhaps all) are also un-paid, as are the reviewers without whom such a journal will grind to a halt. Those activities are performed for the prestige (real or imagined); a sense of duty to the profession; and (for some) because doing so attracts benefit in academic research assessments. The publisher, however, finds that once established, a technical journal can be one of the most profitable parts of his business.

4.5 Design manuals, and handbooks

At intervals, results of particular work or of the experience of a particular engineer may be summarised in textbooks or guidelines written by practising engineers in harbour or coastal matters. Early examples include Rennie (1854), Stevenson (1874), Vernon-Harcourt (1885), Shield (1895), Minikin (1950, 1963), and Cornick (1969), later ones include Jensen (1984), Goda (1985) or Thoresen (1988). Supporting these were scientific treatises on waves, forces, and materials, for early examples see Airey (1845), Rayleigh (1877), Stokes (1888), Bagnold (1939).

More recently, practical design manuals have been compiled by groups of practitioners and researchers, often under the auspices of the UK Construction Industry Research and Information Association (CIRIA), but now also by research contractors like

HR Wallingford supported by industry-led steering groups. Particular examples of earlier CIRIA reports in coastal engineering are given by Summers & Fleming (1983), Summers (1986), and Stickland & Haken (1986). Larger manuals produced by CIRIA or others include: the Rock Manual edited by Simm (1991), Beach Manual by Simm et al (1996), Scour Manual by Whitehouse (1998), Revetment Manual by McConnell (1998) and Coastal Construction Risk Manual by Simm & Cruickshank (1998).

A recent development in the publication of results of research and practice is the editing of "part-work" manuals by a single editor with chapters contributed by many different authors. Often presented as handbooks or reference books, these offer the opportunity for a number of technical areas to be addressed by individual experts, see for instance Abbott & Price (1994), Allen (1998), Herbich (1990, 2000), Thorne et al (1995), Kobayashi & Demirbilek (1995). At their best, these provide some excellent chapters, that on breakwaters by Burcharth (1994), and that by Hsu et al (2000) on shoreline protection practice in Japan are both excellent examples. Conversely, the lack of integration, editing and checking in the manual on rubble mounds by Bruun (1985) demonstrates what can happen if a publisher fails to maintain control!

The reviewer or user of some of these manuals is led to speculate that selection of authors and topics can sometimes be more strongly influenced by which author is available in the required timescale, and what subject is ready to be covered, than by needs of practitioners for guidance on a particular topic. It is important therefore that editors (and publishers) of handbooks of this type are willing to find, motivate and support the right authors and to select the content on the basis of practitioner need.

4.6 Working Group reports, codes and standards

An ideal way to overcome the problems of such manuals is to co-ordinate the compilation of a manual under a steering committee representing users. The International Navigation Association, PIANC, has for many years provided authoritative guidance in its Working Group reports, see for example PIANC (1985, 1987, 1992, and 1994). These working groups have however suffered a significant problem in recent years, and one that may well get worse. PIANC specialist working groups depend upon considerable levels of goodwill, persuading engineers expert in their field to give (without charge) their time to write, discuss, review and re-write guidance on the subject area of the working group. In previous eras when there may have been less pressure on time, and where experts at universities of government institutes might be able to support such an activity as part of their duties, this might have been reasonable, but that era has passed. University researchers are highly focussed on research assessment targets set for them by academic authorities. Government research institutes have been "privatised", sold off or closed, so their specialists can less easily devote time to "non-core objectives". Consulting engineers compete for work chosen for the lowness of their fees. The results for PIANC has been that working groups often become bogged down as the key authors simply have too many competing pressures on their time, most of which command higher priorities.

The solutions to this problem are not obvious, but probably involve wider co-ordination between PIANC, IAHR, the major conference series and publishers, specialist research contractors and disseminators as described above, national research funders, and the end-user communities.

The final formal stage after design manuals is the development of design standards or codes. In UK, there are no codes for coastal engineering, see Fowler & Allsop (2000), but there is a British Standard for maritime engineering, BS6349, reflecting a bias towards port engineering. As with any code or standard, the design methods and technologies reflected in these codes lag well behind the leading edge procedures covered by the more advanced manuals and guides, and at the time of this paper, considerable difficulties are being encountered in the UK in obtaining funding for writing and/or revising essential parts of BS6349.

5. THE FUTURE

5.1 Research and dissemination

In terms of basic research, there are clear and continuing needs for support for background research on fundamental processes, on materials (existing, adapted and new), and on technological tools and solution methods. More attention is however needed to identify the users of this research, who may in turn be other researchers themselves, and to target dissemination of research results. This may continue to use the accepted academic routes of conferences and journal papers. There would however be considerable advantage in re-instating the good practice at some universities where all substantive pieces of research work were reported in departmental reports, possibly of limited circulation, together with an archive of measurements, or computer code or related outputs, see section 4.3. Web-based tools might assist in widening access to those results.

In applied research, the requirements of stakeholders need to be identified and articulated, be they from industry, local or central government, agencies, utilities or other owners. Within any one nation, this might perhaps be a shared task for professional bodies, representatives of central and local government and their agencies. Internationally, there may be a role for co-operation between national engineer institutions and societies like KSCOE, ASCE, ICE and others.

In response to a more concerted view from the consumers of research, providers of research will need to devise quicker and more efficient ways to deliver improvements in knowledge / technology. In the UK, this task will certainly require substantially greater clarity from both sides on the priority and funding of applied research than is evident at present. Perhaps lessons should be applied from experience in other countries?

In research dissemination and exploitation, there is a need for more clarity on the process and assessment of research; on identification of beneficiaries and the targeting of research results particularly involving technology transfer from researchers to end-users. Those improvements may however be assisted by increased us of international sources of data through the internet, provided that there can be adequate local and international coordination. These data could perhaps then support compilation on an international basis of "best practice guides", probably with access to expert guidance and techniques facilitated by the internet. Such services will not necessarily be without charge, but may be offered on subscription.

5.2 Coastal structure design methods

In coastal scheme design, designers, specialists, and owners will increasingly need to involve relevant stakeholders at earlier stages in scheme concept design. Designers will require modelling tools for the main hydraulic processes to be used at different levels of complexity to support different stages of economic and/or environmental assessments. These models will also need to include simplified measures of environmental health and diversity to be incorporated within environmental models.

In detailed engineering design, the need for numerical models of increasing complexity, and assured reliability, will grow. These tools must reproduce more of the hydro-dynamic processes and should include structure responses. Increasing levels of dynamic response models will be needed for design of caisson breakwaters and related structures or elements vulnerable to wave impact loadings. More data will also be needed on probabilistic descriptions of hydraulic and structural responses, on parameter variability, and on the inherent uncertainties in different design methods.

Numerical models will need to be validated for stated uses, and their limits of operation must be documented. Safe use of such tools will however still require significant levels of training, both of model operators, and of the users and interpreters of their results. Despite this, there will continue to be differences over the values to placed on results from such tools, and physical modelling may still be necessary, probably run in concert with the new numerical modelling tools.

Even so, there still tend to be mistakes in application of numerical models, in use of the wrong model, and in interpretation of results. There will also continue to be mistakes arising simply from not using a model, probably most likely where there is fee competition in design, and/or local pressure to reduce initial costs.

Some of the deleterious consequences of down-sizing and out-sourcing in parts of government (central or local) will be reduced, perhaps forming by partnerships with suppliers of essential data or technologies who can assist retain and interpret key local and historical knowledge in house.

On a more positive note, future advances in "whole life design" will necessarily require growth of reliable methods to evaluate whole life costs, with more attention to the full range of solutions, including those requiring regular maintenance where appropriate.

Taken overall, increasing interest in sharing information and in collaboration across countries and across cultures, will neccesarily improve design methods, and hence will reduce cost and improve safety of coastal engineering. Modern methods are now available to share that information, and there is evidence from recent collaborations of increasing willingness to work together towards shared research and development goals.

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