CHAPTER 3

WAVES USED FOR INTER-TIDAL DESIGN AND CONSTRUCTION

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Introduction

1. The selection of suitable design criteria and methods of construction for works on inter-tidal sand flats has played an important part in a recently completed study (1) which examines the feasibility of storing fresh water in bunded reservoirs built on the foreshore of the Wash bay in the east coat of England (figure 1). Wave, wind and tide conditions have been monitored in the bay for 3 years to provide the information needed for design and construction. Experienced gained during the construction of a 15m high trial embankment (figure 2) located 1m below mean sea level and 5 km from the sea defence bank has enabled us to examine the economy of construction methods. Short and long term observations on the bank will provide data on the performance of rip rap surface protection in field conditions.

2. The Wash is an approximately square bay extending over an area of 600km^2 with an entrance 20km wide. Mean spring tides in the bay have a range of 6.4m and tides with ranges above 8.0m are predicted to occur once or twice a year. The large tidal range is associated with extensive foreshores. At low water of a spring tide approximately half the area of the bay is exposed revealing sand and mud flats which in places extend 8km from the sea defence banks. The problems associated with the accurate measurement of water levels in this area with its extensive sand flats have been described by Pugh and Waller (7).

Wave measurement programmes

3. At the commencement of the study there was very little information available on wave conditions in the Wash. To remedy this situation we initiated two separate programmes of wave observation. Three wave recorders were mounted on the foreshore in an area exposed at low water to record wave conditions over the inter-tidal sand flats. These records were obtained for a 15 minute period each tide close to high water. Anemometers installed on the edge of the bay provided a continous record of wind conditions. The installation and early results from these instruments have been described by Driver and Pitt (3). The second programme of wave observation was the visual estimation of wave height by the skippers of the pilot cutter serving the port of King's Lynn. These seamen observed wave conditions almost every tide at 3 buoys marking the shipping channel which

cuts through the inter-tidal sand flats. Data were obtained from both instrumental and visual programmes for a period of 3 years (1972 - 1975)

4. The aims of the two wave observation programmes were complementary. The storm wave data obtained by the wave recorders were to be used in conjunction with the locally measured wind data and also wind data from long established recording stations to define the wave conditions that should be used for the design of rip rap surface protection for permanent reservoir embankments founded on the foreshores of the southern Wash. The visual estimates of wave height were to show the distribution of typical wave heights experienced in the Wash and the significance of seasonal variations. These results combined with the experience of contractors working in the Wash would enable us to estimate the likely proportion of time wave conditions would limit construction work on the inter-tidal sand flats.

Storm wave conditions on the SE foreshore of the Wash

Wind conditions associated with storm waves

5. An analysis of the operation of the wave recorders during the 3 year study period showed that waves with a significant height of at least 0.5m were recorded on 48 separate tides. The largest waves were as expected associated with NW, N and NE winds since the longest overwater fetches are from these directions. In general, larger waves were associated with stronger winds, though this effect was masked by the influence of tide level on wave height. The largest waves were recorded during storms which coincided with the high waters of spring tides.

6. On every recorded occasion except one a 10m/s wind from the NW, N or NE at high water resulted in 0.5m waves at one or more recorders. The exception occurred at a time when only one recorder (site 1) was operational. There are however many occasions when similar wind speeds and directions occurring at low water failed to generate 0.5m waves at any of the recorders the following high water. This highlighted the fact that waves are generated over the inter-tidal foreshore only when a storm coincides with high water. On such occasions the maximum waves associated with the wind are quickly generated because of the short fetches which exist in almost all directions. For example a NW wind of 20m/s at high water would generate fetch-limited waves at the recorder sites after about $1\frac{1}{2}$ hours.

Analysis of storm wave records

7. The wave recorder output was analysed using the method proposed by Tucker (9) as modified by Draper (2). The data are summarised in figure 3 which shows the significant wave heights and zero crossing periods

recorded at each site. Results have only been included for those occasions (tides) at each site when the recorded significant wave height was at least 0.5m. There were 33 such tides at site 1, 20 at site 2 and 35 at site 3 between September 1972 and May 1975.

8. The wave heights and periods recorded at each site are broadly similar with the highest waves generally being recorded at site 3. The two largest individual values of significant wave height were, however obtained from the recorder at site 1 where significant heights of 2.4m and 1.6m were recorded. The wave heights at site 2 were latterly generally lower than elsewhere. This is thought to result from a 1m increase in bed level during the winter 1973/74.

9. There is only a very weak correlation between the height and period of recorded waves.

10. The wave height distributions (figure 3) cannot be compared in detail because the waves recorded at each site did not all occur on the same occasions. Comparisons may better be made by comparing the differences between the observed waves and those predicted for each occasion by a standard wave-forecasting routine. Although deep-water wave prediction routines cannot be expected to give accurate answers in shallow water, they do provide a standard for comparison taking account of wind speed, duration, direction and fetch length. The differences between the observed and predicted wave conditions will indicate the influence of the shallow and varying water depth on wave generation, refraction, shoaling and friction.

11. Wave hindcasts have been made using the Sverdrup-Munk-Bretschneider (S-M-B) deep water wave forecasting charts (6) for each site on every tide that the significant wave height was at least 0.5m. The waves were hindcast using the average wind speed over a period long enough to generate fetch limited waves. On each occasion the fetch was calculated for high water of a spring tide assuming all wave generation to occur within the confines of the bay. No allowance was made for the effects of refraction in the calculation of fetch.

12. The ratio of observed significant wave height (H_s) to hindcast significant wave height (H_o) has been plotted against water depth for site 1 alone and sites 2 and 3 together on figure 4. The best fit straight lines to each set of wave height data are shown. The results show a rapid increase in wave height ratio with water depth at site 1 but much smaller changes in wave height ratio with water depth at sites 2 and 3. Bounding lines spaced two standard deviations from the mean are also included. Consideration was given to plotting the H_s/H_o rato against water level relative to Ordnance Datum and using this relationship for the subsequent analysis and design instead of one incorporating water depth. The observed reduction in wave heights at site 2 (paragraph 8) following the rise in sea bed level suggested the use of water depth rather than water level. Furthermore in considering

wave heights at different foreshore levels the use of water depth proved to be more convenient than water level. The bed configuration seaward of the reservoir sites does not include banks or bars that would invalidate the use of water depth and the increase in fetch with increasing depth (due to higher water levels) was shown to have no marked effect on the results at site 2.

13. The magnitude of the random errors that have been introducted into the calculation of wave height by the method of analysis of the wave records (paragraph 7) are fairly significant. These random errors alone are estimated to cause a standard deviation in the wave height ratio of around 13%. The measured standard deviation for the observations from sites 2 and 3 is about 20%. The standard deviation of the observations for site 1 is greater at about 27%, due largely to the two occasions when the wave height ratio exceeded 1.4.

14. These two occasions are interesting as these were also the occasions of the largest waves recorded during the study. The operation and calibration of the wave recorded appeared normal on both occasions. The most likely explanation for these exceptional wave heights is that the recorder was situated close to a point where wave energy was concentrated by wave refraction during both storms. The greater sensitivity of wave height ratio to water depth at site 1 when compared to sites 2 and 3 may also be indicative of a concentration of wave energy resulting from wave refraction as such concentrations depend on water depth as well as wave period.

Extreme storm conditions

Extreme wind conditions

15. The installation of anemometers on the shore of the Wash in 1972 provided the first continuous records of winds in the bay. In the past wind records have been collected at Spurn Head (figure 1) which is 50km north of Gibraltar Point. Wind records are also available at several nearby inland sites including a long record at Cranwell (figure 1) 55km west of Gibraltar Point. As the Wash anemometers have only operated for three years there are insufficient data available for them to be used to give reliable estimates of wind speeds occurring less frequently than once in 5 years.

16. The British Meteorological Office was commissioned to examine the relationship between wind speeds at Gibraltar Point and Cranwell with a view to using the long record available at Cranwell to estimate extreme winds at Gibraltar Point. Cranwell was chosen for this comparison as it was the site nearest the Wash with broadly similar wind characteristics and a long record. Initial comparisons indicated that wind direction and strengths at the two sites were usually similar.

17. The results of the comparison of hourly mean wind speeds confirmed that on average the differences in wind speed between Gibraltar Point and Cranwell were fairly small. Despite this apparent similarity the main significance of the regression equations linking the two stations was that the estimates of wind speed at Gibraltar Point using Cranwell data were poor. If corrections were made to the regression equations based on the wind direction, the difference between the observed and estimated wind speed at Gibraltar Point was reduced. Even with this improvement the standard error of estimate was between $1\frac{1}{2}$ and 2m/s. The use of Cranwell data to estimate wind speeds at Gibraltar Point on particular occasions is thus ruled out.

18. The regression equations linking wind speeds at Gibraltar Point with those at Cranwell are considered adequate to estimate the broad characteristics of winds at Gibraltar Point. The evidence from Cranwell suggests for example that gale force winds (speeds 17m/s) from the NE are rare at Gibraltar Point. The examination showed that large errors would result from the use of the regression equations during individual storms. This prevents us estimating extreme wind conditions at one site on the basis of extreme conditions at the other and is due to many complex effects which influence the relationship on particular occasions. These effects may be expected to be especially pronounced during extreme storms.

19. The meteorological office considered that more reliable estimates would be provided from wind maps prepared by Hardman, Helliwell and Hopkins (4). These wind maps have been updated to take account of extreme winds in East Anglia up to June 1975 and the estimates for the Wash (figure 5) taken off these maps show a wind speed of 29.2m/s (57 kt) to have a return period of 50 years. The wind speed frequencies apply to a point in the centre of the entrance to the Wash.

20. The wind directions associated with very high wind speeds cannot be estimated in the same way as the extreme wind speed. Some indication of the most likely directions of extreme winds is provided by the records from Spurn Head from 1922 to 1958. An examination of the directions associated with hourly mean wind speeds of at least 21m/s showed (Table 1 below) that although the most common direction for high winds was W, almost half the occurrences were associated with N or NW winds which are able to generate some of the largest waves in the Wash. There were no recorded occasions of similar NE or S winds at Spurn Head. The large proportion of high winds associated with W, NW and N winds is a feature of observations made along the NE coast of England. Although there may be some differences between the winds at Spurn Head and the Wash, the Spurn Head data are used in the subsequent analysis.

Wind direction	S	SW	W	NW	N	NE	E	SE
Percentage of occasions	0	7	33	30	19	0	7	4

Table 1: Directions associated with mean wind speeds $\geqslant$ 21 m/s at Spurn Head.

Extreme wind speeds at high water

21. The probability that extreme wind conditions will occur during a high water period is lower than the probability of these winds occuring at any state of the tide. There is only a 1 in 4 chance that the maximum hourly mean wind speed during a storm will coincide with a three hour period at high water. From an analysis of storms recorded at Lerwick, Shellard (8) has estimated the ratio of maximum wind speeds averaged over varying numbers of hours to the maximum speeds over one hour. Shellard considered that these ratios could be used to estimate the wind speeds which,when averaged over different numbers of hours,had the same frequency of occurrence each year as the maximum mean hourly wind speed.

22. The wind ratios derived by Shellard (8) for Lerwick have been applied to the Wash data to estimate the frequency of occurrence of combinations of wind speed and duration (figure 5). In our calculation of duration, the wind speed was assumed to be above the average for half the averaging period. The lower wind speeds which occur more frequently for long periods of time have a greater chance of occurring during a high water period. The frequency of occurrence of particular wind speeds at high water may be obtained by combining the frequency of occurrence of the wind speed for a particular number of hours with the probability that this will coincide with high water and summing the frequencies derived for periods up to 9 hours. All periods of 9 hours or more are considered together as they will all coincide with at least one high water period. No account has been taken of periods which overlap two or more high water periods as these rare occasions are unlikely to increase the derived frequency of particular wind speeds by more than 1%. The estimated frequency of occurrence of various wind speeds at high water is also shown on figure 5. The figure shows that for the same frequency of occurrence each year the maximum hourly mean wind speed at high water is about 2m/s lower than the maximum hourly mean speed recorded at any state of the tide.

23. The frequency of occurrence of a given wind speed from a particular wind direction at high water is estimated by combining the results contained in figure 5 with those in table 1 (paragraph 20).

Extreme wave heights

24. The frequency of occurence of a particular large wave height at high water in deep water in the southern Wash is estimated by summing the frequency of occurrence at high water of the winds from W, NW, N and E that are capable of generating the desired wave height. The available fetch is taken as the site 3 fetch within the confines of the bay at high water of a spring tide and the relationship between wind speed and wave height in deep water is taken from the S-M-B wave forecasting charts (6). The deep water wave height frequencies were used with the empirical relationship (figure 4) between wave height and water depth for sites 2 and 3 to obtain shallow water wave frequencies for a range of water depths.

25. The calculations we undertook indicated that N winds would probably cause the largest waves in the SE Wash. The greater frequency of occurence of strong NW winds compared to N winds (Table 1) does not outweigh the effect of the longer N fetch. The longer fetch allows less strong and therefore more frequent N winds to generate large waves more often than NW winds.

26. Up to this point the calculations of wave height frequencies referred to particular water depths. These depths will occur at different foreshore levels as the tides change from springs to neaps. To provide estimates of the frequency of occurrence of large waves at particular foreshore levels, we have combined the probability that high water will reach a particular level with the frequency of occurrence of the required wave height in the appropriate water depth. The results are then summed for all tide levels. The probability that the predicted tide will reach a given level was derived from the annual distribution of predicted tide levels; the assumption has been made that there would be an average tidal residual of +0.5m at high water whenever large waves occurred. This is because strong NW and N winds generally raise water levels in the southern Wash and are quite often associated with North Sea storm surges. The calculations indicated that at foreshore levels above -1m OD large waves are most likely to occur during high spring tides when predicted high water levels are at least +3.3m OD. This is because the frequency of occurrence of wind needed to generate large waves in this depth of water is much higher than that of the wind needed to generate the same size of wave in the shallower water that is more often present.

27. Wave heights which are estimated to have particular frequencies of occurrence at various foreshore levels are shown in figure 6. This figure indicates that at all depths the wave height with a frequency of occurrence of 0.001 each year is only about 2.3 times larger than the wave height that occurs on average once a year. The wave height expected at a foreshore level of -4m OD is about 2.4 times the wave height expected at +2m OD with the same frequency of occurrence.

28. In some areas of the foreshore, wave energy will be concentrated or dispersed by the effects of wave refraction. These local variations in wave energy concentration will change the height of the waves. In areas of severe concentration the wave height may be up to twice that experienced generally. The occurrence of severe concentration of wave energy will be critically dependent on the wave period and water depth on each occasion as well as the offshore bed topography.

Rip rap design for seafacing reservoir banks

29. The extreme wave heights derived for the southern Wash (figure 6) have been used as the basis for design of rip rap surface protection for the Westmark and Hull Sand reservoir shapes (figure 1). The method of relating wave height to surface protection is based on recent model studies by the British Hydraulics Research Station (HRS) (5). The studies indicate that for a given embankment slope, the amount of damage sustained by a randomly placed quarry stone rip rap layer of given stone size depends principally on the significant height and number of waves attacking the embankment. In the model studies the rip rap was placed over suitable filter material in a layer whose thickness was twice the median size of the rip rap. Failure of the modelled slope protection was deemed to occur when a hemispherical foot with a diameter equal to the median size of rip rap under test could just touch the underlying filter layer.

30. The rip rap design for the proposed sea facing reservoir embankments in the Wash was based on the requirement that the surface protection should not require maintenance even after a design storm containing 2000 waves with a significant height estimated to have a probability of occurrence each year of 0.001 at the level of the foreshore upon which the bank was sited. The number of waves in the design storm was calculated as the maximum number likely to attack the bank during the three hour period around high water when water depth would be greatest. The wave heights experienced on the preceding and following high water are likely to be significantly smaller during such an extreme storm because of the improbability of a sufficiently strong wind being maintained for upwards of 12 hours (figure 5). A further reason for confidence in the restriction of the design storm to 2000 waves is that model tests with up to 5000 waves generally showed little additional damage to that caused by 2000 waves (5). The requirement that no maintenance be needed was interpreted as meaning that during the design storm the damage should be less than 15% of that

causing failure,as defined above,of the rip rap protection. The assumption has been made in this definition that the results of the model studies may be directly applied to the proposed Wash reservoirs. From figure 6 the design wave height for an embankment founded close to 0m OD on the SE foreshore of the Wash is 2.3m. Rip rap with a median size exceeding 950mm placed on a 1:4 slope will satisfy the no-maintenance requirement defined above for this wave height.

31. A second requirement for the rip rap design was that in areas subjected to concentrations of wave energy, the protection should not fail, though greater damage requiring maintenance would be permissable. In such areas the significant wave height will increase and greater damage will be experienced during storm with a given frequency of occurrence.

32. For the Westmark reservoir which is founded at and above 0m OD a median stone size of 950mm satisfying the first requirement is considered adequate. This size of stone should withstand, without failure, waves 1.75 times the height experienced generally during the design storm. Such a measure of concentration of wave energy is unlikely to be exceeded on the flat gently shelving foreshore seaward of the reservoir. This area of foreshore has been stable for 100 years and is accreting in a fairly uniform manner.

33. On the line of the Hull Sand reservoir embankment which crosses a series of channels and areas of complex topography, wave heights up to twice the height experienced generally are considered possible. This implies that in some areas during the design storm waves with a significant height of 4.6m could attack an embankment founded at 0m OD. To resist such waves without failure, rip rap laid on a 1:4 slope should have a median size of at least 1050 mm. As a result of the dependence of the wave height on the foreshore level for a storm with a particular frequency of occurrence (figure 6) the size of rip rap used for surface protection should vary with foreshore level. The size of rip rap required for both Westmark and Hull Sand reservoirs at various foreshore levels is shown in Table 2.

Foreshore level (mOD)	Median size of rip rap (mm)		Top level of protection (mOD)	
	Hull Sand Reservoir	Westmark Reservoir	Heavy	Light
+2	750	650	+8.5	+10.0
0	1050	950	+9.75	+12.0
-2	1450	-	+10.75	+14.0
-4	1700	-	+11.5	+15.75

Table 2: Rip rap design for proposed Wash reservoirs (Note: design maximum still water level +5.8mOD)

34. The vertical height to which surface protection should be taken depends on the expected extreme still water level and the height above this which may be damaged during a storm which almost causes failure of the protection. The HRS model studies (5) indicated that at failure of a 1:4 slope damage extended for a vertical distance of 3 median stone diameters above and below the still water level. This finding is applied to the full scale embankment design with an added 0.5m allowance for free board. The recommended top level of rip rap protection (table 2) assumes an extreme water level of +5.8mOD which is estimated to have a frequency of occurrence each year of 0.001 at Roaring Middle.

35. Above the main surface protection, lighter protection is required against wave run up. The HRS model studies (5) indicated that wave run up would occasionally reach a level 1.25 times the significant wave height above still water level. Using this information for reservoir design in the Wash, with a similar allowance for free board and an extreme water level of +5.8mOD gives the top levels of lighter protection (table 2).

The trial bank as a test for the rip rap design

36. The trial bank constructed in the Wash (figure 2) is circular in plan with a diameter of 284m enclosing a small area of foreshore. The bank consists of 412,000m^3 of sandfill won with a cutter-suction dredger from a nearby foreshore borrow pit. The sand fill is protected against wave attack and erosion by quarry stone rip rap and two filter layers of coarse sand and crushed furnace slag from a phosphate process. The rip rap on the seaward side of the bank consists of hard durable limestone with a median size of 660mm laid in a layer 1300mm thick. On the landward side similar though smaller rip rap is used. The protection on the seaward side is taken from sea bed level (-1m OD) to +7.5m OD. Above this level the bank is protected to +9m OD by slag. The top 5m of the bank is grassed.

37. The surface protection used for the trial bank is less heavy than that recommended for the permanent banks of the Westmark reservoir although the trial bank and the proposed Westmark reservoir are founded at similar levels. If the design assumptions used for the proposed reservoir prove to be accurate the rip rap on the trial bank will suffer damage requiring maintenance during storms with a significant wave height exceeding 1.7m. The frequency of occurrence of such waves at the bank site is thought to be about 0.1 each year if wave energy is not concentrated onto the bank.

38. The surface protection of the trial bank should not fail even if attacked by waves with a significant height of 2.7m which are thought to have a frequency of occurrence of about 0.001 each year. If wave energy was heavily concentrated onto the trial bank by wave refraction, failure may occur during storms with a frequency of occurrence of 0.1. Such concentrations of wave energy are thought unlikely as the bed topography seaward of the bank is flat and gently shelving.

39. The long term performance of rip rap on the trial bank will indicate the validity of the design methods we propose for the main reservoir embankment. To this end a continuing programme of wave, wind and tide measurement has been recommended with associated studies on the performance of the surface protection. The performance of the rip rap will be judged on the basis of stone movement.

Visual wave observations to estimate inclement weather

40. Three of the buoys marking the King's Lynn shipping channel were chosen as sites for visual wave observations (figure 1); No 13 indicative of conditions on the upper foreshore, No. 7 indicative of lower foreshore conditions and No. 1 in deep water at the mouth of the channel. Estimates of wave period were not made, although the skippers reported the presence of swell.

41. The wave height frequency distribution is shown in figure 7 which compared the wave height distribution at buoys 13, 7 and 1 for the year June 1974 to May 1975. Large waves are encountered much more frequently at buoy 1 than at the more landward buoys. For example in 1974/5 waves exceeding 0.6m (2ft) were reported on almost 20% of observations at buoy 1, almost 10% of observations at buoy 7 and less than 3% of the observations at buoy 13. At each buoy there were also a large number of occasions when wave heights less than 0.075m (3ins) were recorded. The proportion varied from 10% at buoy 1 to over 20% at buoy 13.

42. The information provided by the skippers of the pilot cutter was of great value to us in assessing typical wave conditions in the Wash, principally because of the regularity of the observations. Over a period of 3 years December 1973 - November 1975 the skippers recorded their wave height estimates at buoy 7 on 92.5% of the 2116 high waters. The wind conditions on the high waters when no observations were made at buoy 7 were compared with conditions on other high waters and found to be very similar suggesting that severe wave and wind conditions did not usually prevent the pilot cutter putting to sea. On the majority of occasions when the pilot cutter remained in port the reason was the absence of shipping requiring pilotage.

43. There were only four recorded occasions in the 3 years when stress of weather was given as the reason for remaining in port or turning back before reaching buoy 7. On 7 of the 9 occasions when waves above 1.5m were noted at buoy 7, however, no readings were made at buoy 1 suggesting that wave height was a contributory factor in causing the pilot cutter to turn back or meet ships before reaching buoy 1. On one of these occasions the cutter reached buoy 1, but the conditions were so severe that the skipper felt unable to estimate the wave height accurately.

44. The seasonal variations in wave height estimates over the 3 year period at buoy 7 are shown in figure 8. This figure shows that the wave height distribution is fairly similar at all seasons of the year with the exception of the 3 summer months of June, July and August when wave heights are noticeably lower, making these months the prime building period.

45. The seasonal estimates of wave height distribution (figure 8) do not indicate for how long the waves remained above a height which could hinder or damage work nor the length of time between periods of inclement weather when building may proceed. This information is conveyed on figure 9 which shows that the number of separate periods each season when waves exceed 0.22and 0.30m remained between 20 and 25 with no significant reduction in the summer. The average persistence of small waves was however about $1\frac{1}{2}$ days longer in the summer than during the remainder of the year. There are also likely to be much longer periods with no waves above 0.45 and 0.6m in summer than at other times of the year. Although the other months generally have longer periods of inclement weather, even these months contain occasional long calm spells.

46. Data from the anemometer at Cranwell indicate that the period of the study was fairly windy. In the two years June 1973 to May 1975 there were 45 percent more hours when the wind speed exceeded 10m/s (UK Beaufort force 6) than the average over the nine years July 1966 to June 1975. During a typical construction period, the amount of time that will be lost due to inclement weather would normally be somewhat lower than the proportion of time indicated by the wave height frequency distribution (figure 7).

47. The effects of variations in wind conditions from year to year may be illustrated by considering the wave and wind data from the spring months of March, April and May during the 3 years 1973 to 1975 (figure 10). During these spring months the weather is often dominated by moderate northerly winds which can generate fairly large waves in the Wash. The proportion of time the wind exceeded 8m/s (UK Beaufort force 5) from the NW, N or NE ranged from 3 to 5% in 1973 and 1974 to 15% in 1975. At the same time wave heights exceeding 0.45m (18ins) were reported on 7 to 8% of tides in 1973 and 1974, but 20% of tides in 1975.

48. Experience gained by contractors using using small craft for survey and building purposes suggested that operations were hindered when wave heights exceeded 0.23m (9 ins).

49. The contractor for the trial bank (HAM Dredging Ltd) imported large quantities of rip rap stone and filter material through an offshore transhipment site in which the materials were moved from coasters to barges by floating cranes. The operation of this site was occasionally affected by the inclement weather experienced in the spring of 1975 (figure 10) which delayed the programme 5 days in 7 weeks. The periods

when weather conditions halted the transhipment operation coincided with reports from the pilot skippers of 0.5m waves at buoy 7.

Conclusions

50. For the design of the surface protection for the reservoir embankments proposed in the recent feasibility study for water storage in the Wash we have attempted to estimate the frequency of occurrence and extent of storm damage. To this end we have measured wave and wind and tide conditions at the site for three years to derive empirical relationships between water depth, wave height and wind speed, direction and fetch.

51. We were advised not to use a direct correlation between local recorded wind conditions and the conditions recorded 55km away at a long term anemometer station as the basis for derivation of extreme winds. Instead extreme wind maps based on all available regional wind data have been used as these are considered more reliable than a direct correlation between two individual stations.

52. To assess the frequency of occurrence of various wave heights on the inter-tidal foreshore we have taken into account the duration and direction of extreme wind speeds and the probable incidence of these winds with high water. Our estimates of wave height, based on the empirical relationship between wind, wave and tide conditions, take account of the variation in high water levels between spring and neap tides. We have been able to estimate wave heights with a given frequency of occurrence at foreshore levels ranging from mean high water springs to mean low water springs in an area with a spring tide range of 6.4m.

53. Our estimates of the frequency of occurrence of wave heights at various foreshore levels have been combined with the results of recent model studies at HRS to produce a rip rap design. The frequency of damage and probability of failure of this surface protection has been estimated.

54. Long term tests on the trial bank rip rap combined with continuing wave wind and tide measurements will indicate the validity of the design methods proposed.

55. Estimates of the amount of time that will be lost during construction of the reservoir bank due to wave activity may be made as a result of regular visual observations of wave height over a three year period. These observations enable the frequency and probable duration of waves likely to hinder building operations to be estimated on a seasonal basis.

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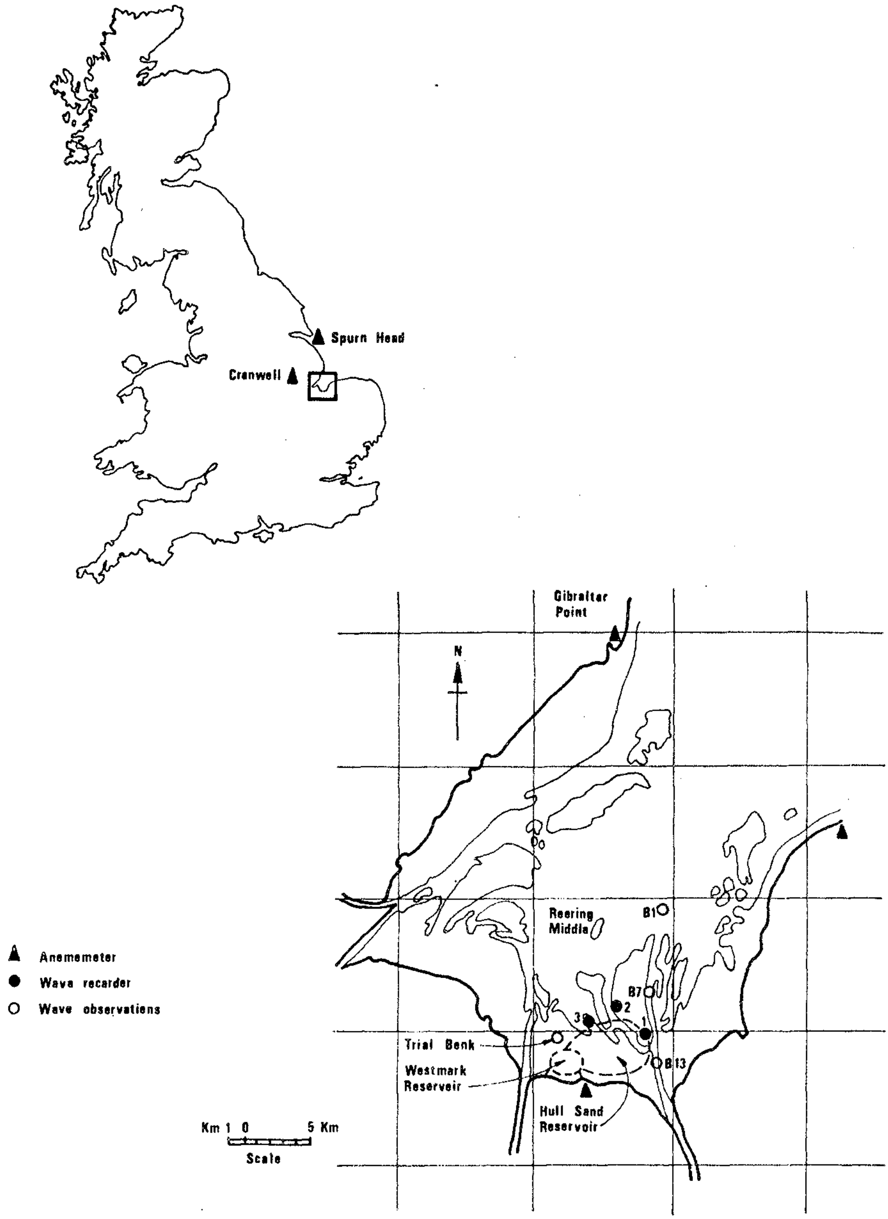

Figure 1. The Wash: General Plan

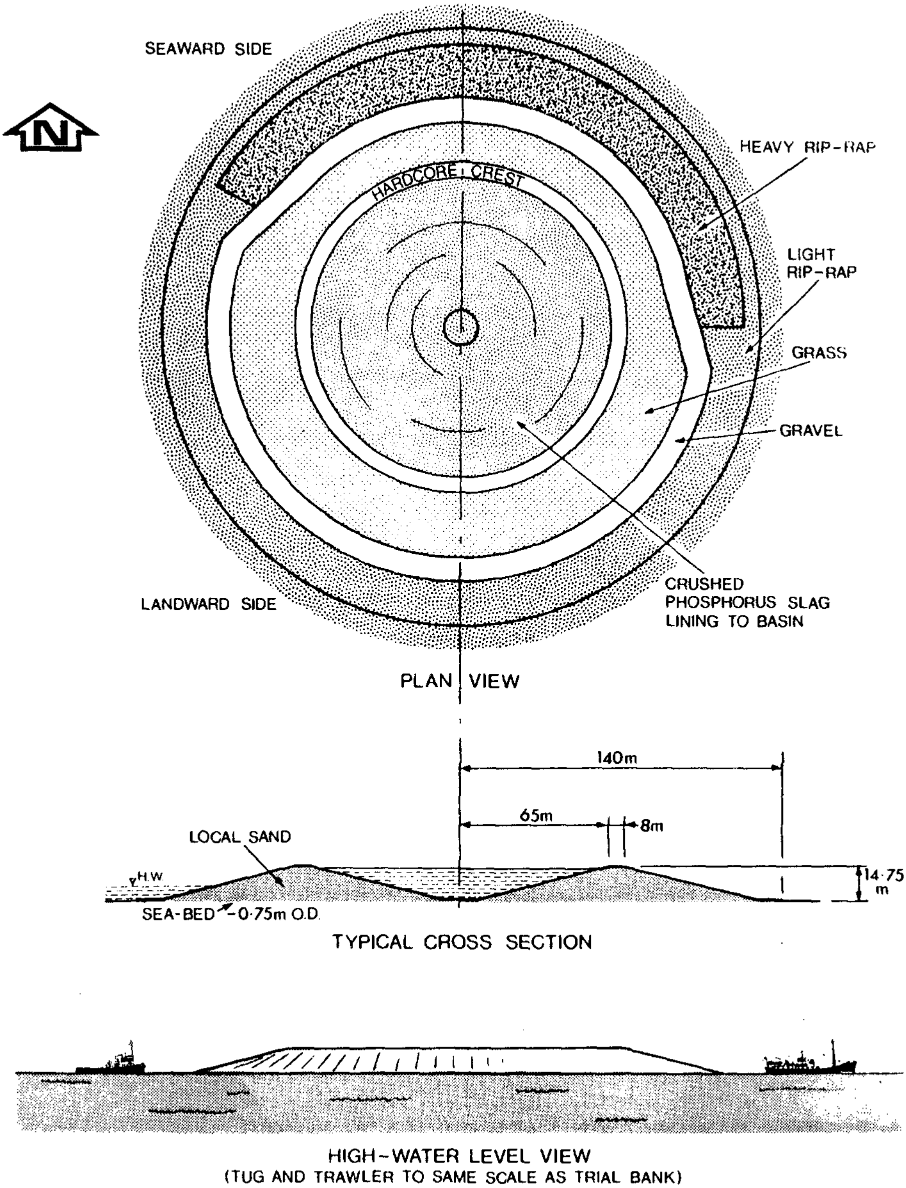

Figure 2. Details of Offshore Trial Bank

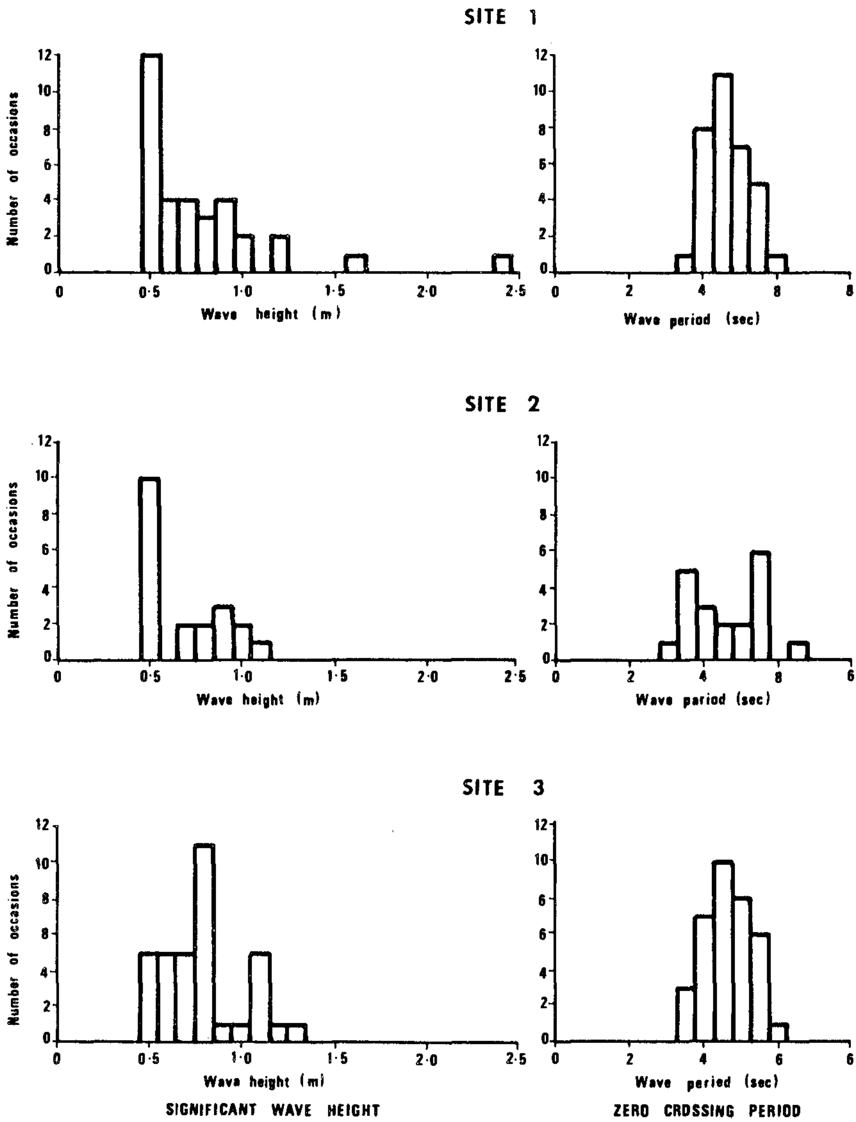

Figure 3. Wave recorder analysis for occasions with wave height 0.5 m or greater

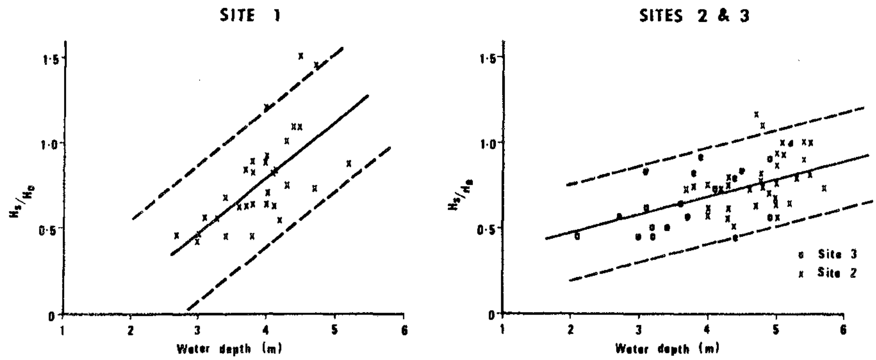

Figure 4. Variation of wave height with water depth

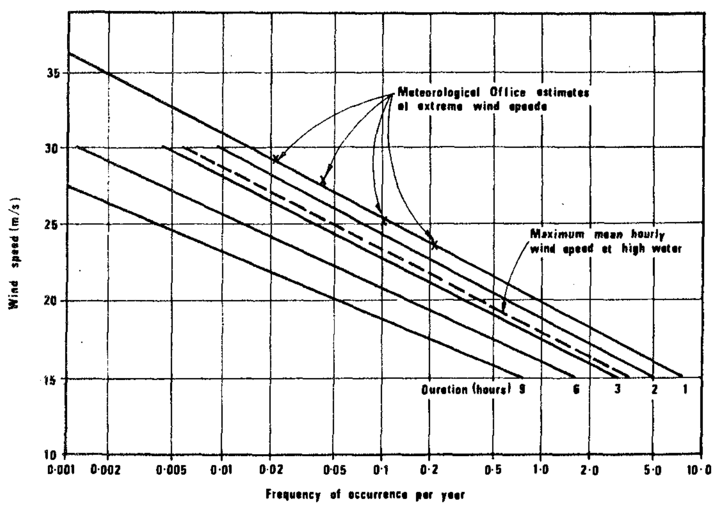

Figure 5. Extreme wind speeds in the Wash

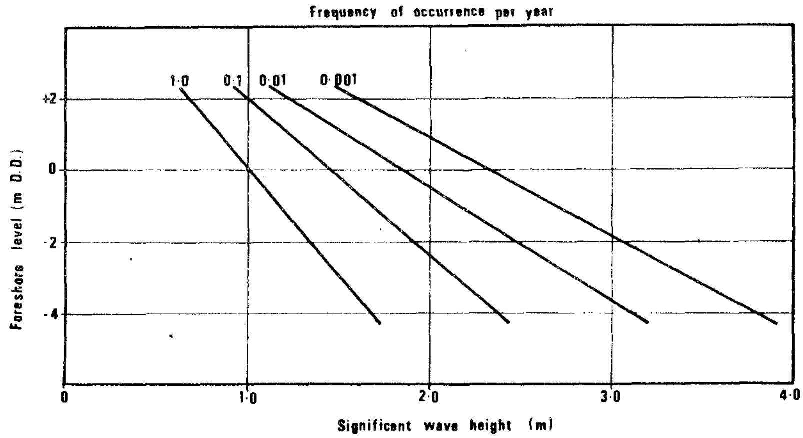

Figure 6. Frequency of occurrence of large waves at various foreshore levels

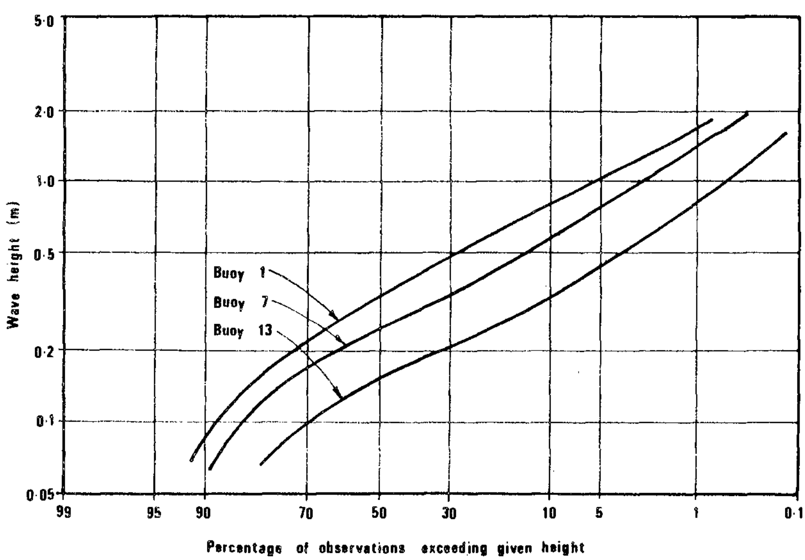

Figure 7. Comparisons of wave heights at Buoys 13, 7 & 1 June 1974 – May 1975

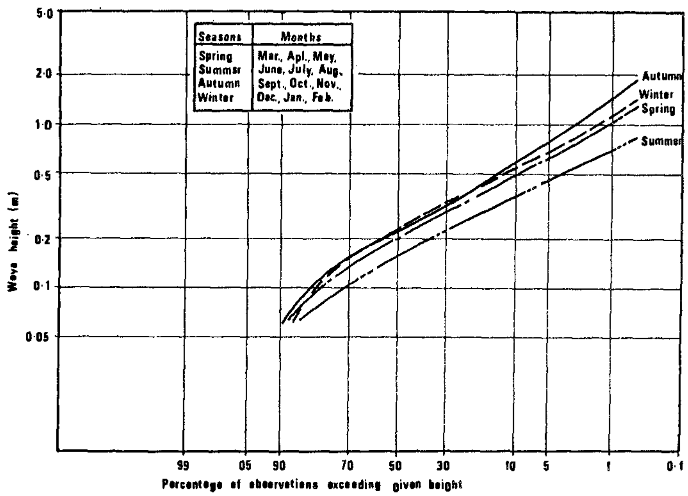

Figure 8. Seasonal variation in wave height distribution at Buoy 7 Dec 1973 – Nov 1975

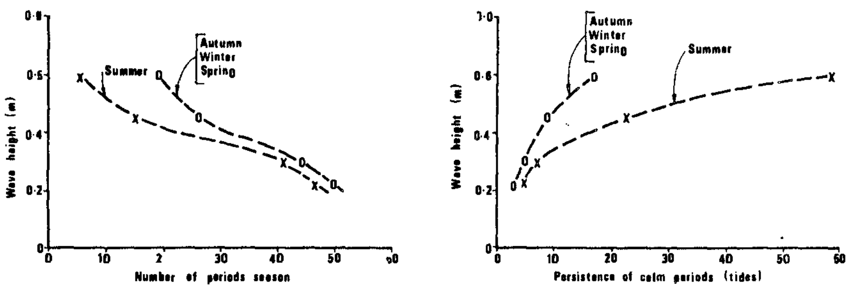

Figure 9. Number and persistence of calm periods at Buoy 7

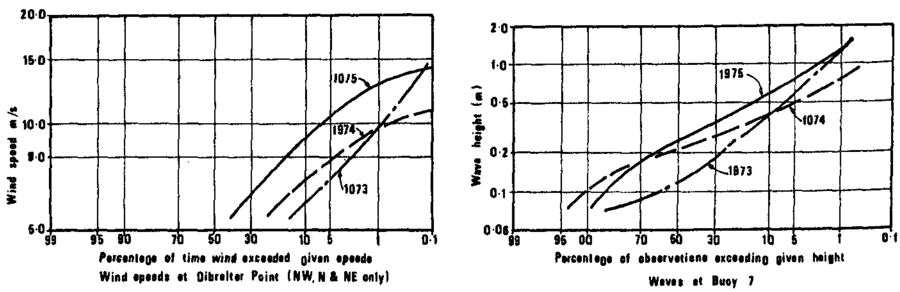

Figure 10. Comparison of wave and wind conditions during spring