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Application of Offshore Breakwaters to the UK: A Case Study at Elmer Beach

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ABSTRACT

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The use of offshore breakwaters in the UK is relatively uncommon with only 6 schemes constructed to date (early 1998). The use of this technology in the UK is examined via a case study of the 8-unit Elmer Beach offshore breakwater scheme which fronts a shingle beach. Profile line surveying and aerial survey data are used to show the development of the planform during the first 38 months and the development is compared to empirical design guidelines. Also an intensive fieldwork program investigated shingle transport inshore of the breakwaters using sediment tracing techniques. The results show that the scheme exhibited rapid changes to planform during the first 6 months, with shingle salients and sand tombolos developing in the lee of the breakwaters. Changes to planform after this time were less marked, although areas of continued erosion/accretion were identified. The sediment transport experiments identified sediment pathways and rates of transport in the lee of the breakwaters. It is suggested that the breakwaters have reduced shingle transport by a factor of at least 2 compared to similar open beaches.

ADDITIONAL INDEX WORDS: *Coastal structures, shoreline evolution, beach change, shingle transport.*

INTRODUCTION

Detached offshore breakwaters began use as a form of coast protection in Europe and the United States in the mid-1960s. However, they proliferated in Japan from the 1970s with some 900 units constructed by 1974 (KOIKE, 1988) and 4,800 units by 1989 (SILVESTER and HSU, 1994). Other schemes have been constructed in Singapore, Israel, Egypt, France, Spain and the USA (COOPER, 1996; CHASTEN *et al.,* 1993).

Offshore breakwater use has been varied, from single units 30-100m metres in length, to segmented schemes comprising many units and covering several kilometres of coastline. Breakwaters are usually shore-parallel but occasionally are positioned obliquely, offering enhanced protection from one particular direction, (BARBER and DAVIES, 1985; CHASTEN *et al.,* 1993). Generally, breakwaters have been deployed as protection measures in response to erosion on sandy beaches and are often combined with beach renourishment. In such cases breakwaters have been deployed to stabilise and retain beach fill material, increasing the period between successive replenishments, (CHASTEN *et al.,* 1993). There are also cases where they have been used to create or enlarge beaches primarily for recreational purposes, (FRIED, 1976; NIR, 1988; ANTHONY and COHEN, 1995; CHASTEN *et al.,* 1993).

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History of Offshore Breakwaters in the UK

The use of detached breakwaters in the UK is still relatively uncommon. To date, offshore breakwaters have only been used at a few sites in the UK, although the number of schemes is expected to rise. The first recorded usage was at Leasowe Bay on the Wirral (near Liverpool), (BARBER and DAVIES, 1985), where two oblique breakwaters were completed in 1982.

Currently, the largest scheme in the UK consists of eight breakwaters constructed at Elmer on the south coast of England (HOLLAND and COUGHLAN, 1994; COOPER *et al.,* 1996b). Other smaller schemes include; a single detached breakwater and rock groyne system at Monk's Bay on the Isle of Wight, which protects the toe of a previous landslip and has also created a new amenity beach, (ISLE OF WIGHT COUNCIL, 1996); two detached breakwaters at King's Parade on the Wirral, which were constructed following the success of the Leasowe Bay breakwaters, (BARBER and DAVIES, 1985), and two oblique breakwaters have been constructed at Sidmouth in Devon, (ANDREWS, 1996).

Additionally, along the east Norfolk coastline a major offshore breakwater scheme, also including beach recharge and sediment bypassing, is currently under construction along the 14km Happisburgh to Winterton coastline, (NRA, 1993; GARDNER and RUNCIE, 1995). Eventually 16 units, around 300m long and situated 200–300m offshore will be constructed. At the current time only 4 of the breakwaters are complete, with the full scheme due to be completed in 2012, (GARDNER and RUNCIE, 1995).

All of these schemes are relatively innovative in the UK, and as such can provide important information regarding the suitability of offshore breakwaters for use in the UK. However, Elmer, with its' eight shore-parallel breakwaters, remains the most widely applicable deployment of offshore breakwaters in the UK and has been subject to a large degree of research interest, (COOPER, 1996; KING *et al.*, 1996).

Application of Offshore Breakwaters to UK Conditions

There are important differences between UK usage of offshore breakwaters and their use in the USA or along Mediterranean coasts. The large tidal range in the UK and their application to shingle beach protection being the main differences. At the Elmer site tides are macro-tidal, and the mean Spring tide range is 5.3m, with a maximum range of just under 7m. This contrasts with the US, Japanese and Mediterranean situations, where tide ranges are generally less than 2m (either meso or micro-tidal), (CHASTEN *et al.*, 1993; LOVELESS and COPELAND, 1991). This large tidal range creates problems in placing the breakwaters. Important design criteria include the average depth of water at the structure and the position of the structure relative to the surf zone. A disadvantage of a breakwater situated within the surf zone may be substantial scour at the toe of the structure, (CHASTEN *et al.*, 1993). Thus in the UK unless the breakwaters are extremely high, they must be positioned closer to shore in order to prevent wave overtopping or submersion at high tides. The large tidal range experienced in the UK may also affect planform development, (DALLY and POPE, 1986).

The large number of shingle beaches in the UK, especially on the south coast of England, diverges from experience elsewhere, where the breakwaters have been used to protect sandy beaches. To the authors' knowledge the Elmer scheme is the first time that offshore breakwaters have been used in conjunction with shingle beaches on such a large scale.

OFFSHORE BREAKWATERS: DESIGN PARAMETERS

Parameters affecting the morphological response of beaches to detached breakwaters include wave climate, water level range, sediment supply and size, in addition to the structural parameters such as length, offshore distance (or water depth at the structure), gap width and the transmission properties of the breakwater. Typically, factors such as length to gap ratio and the transmission properties (porosity and crest elevation) are determined by the required reduction in wave energy inshore of the breakwaters. Other breakwater dimensions are determined by the desired beach response to the structure.

General design guidelines for offshore breakwaters refer to the design wavelength and breakwater layout (length, offshore distance and gap width), (ROSATI, 1990; CIRIA, 1991). Equations 1 and 2 list those guidelines suggested by ROSATI.

$$L \geq 2 \times \lambda_d \quad \text{and} \quad L \approx X \tag{1}$$

$$G \leq \lambda_i \tag{2}$$

where L = breakwater length, G = gap between adjacent breakwaters, X = breakwater distance offshore, and λ is wavelength, (subscript d for design wavelength and i for the wavelength of incident waves). It is, however, important to note that the design guidelines are mainly based on the empirical experience of the US and Japanese schemes, and, as noted above, conditions may differ significantly from the UK.

Tombolo or Salient Development

In the lee of offshore breakwater schemes the typical beach planform exhibits either salients or tombolos, where salients are a 'bulge' in the shoreline in the lee of the breakwater and tombolos are salients which have developed such that they reach the breakwater, (CHASTEN *et al.*, 1993; ABDEL-AAL, 1993). Beach response to offshore breakwater construction can vary from no sinuosity through to permanent tombolo features, (POPE and DEAN, 1986).

Design criteria for offshore breakwaters may call for tombolo development or salient development, although salients are generally the preferred option, (ABDEL-AAL, 1993). Where the longshore movement of sediment is to be preserved then the development of tombolo features is to be avoided, as they block longshore transport shoreward of the structure and may promote offshore sediment losses via rip currents through the breakwater gaps, (CHASTEN *et al.*, 1993; MCCORMICK, 1994). Although rip currents are unlikely to affect a shingle beach, the preservation of longshore transport may still be desirable.

The degree of beach development, from no sinuosity to full tombolo features is generally determined by the non-dimensional ratio of breakwater length, L, to offshore distance, X. Empirical data from several sources suggests that tombolo features (for multiple breakwaters) occur for values greater than about 1.5, (CHASTEN *et al.*, 1993). For salient development, but no tombolo features, a value of between 0.5 and 0.67 is suggested, (DALLY and POPE, 1986).

Gap Erosion

Erosion opposite the gaps between breakwaters may be predicted from the ratio of gap width to offshore distance, G/X, (SEIJI *et al.*, 1987). The possibility of gap erosion is determined by the values given in equations 3 to 5.

$$\text{No gap erosion} \qquad \frac{G}{X} < 0.8 \tag{3}$$

$$\text{Possible erosion opposite gap} \qquad 0.8 < \frac{G}{X} < 1.3 \tag{4}$$

$$\text{Certain erosion opposite gap} \qquad \frac{G}{X} > 1.3 \tag{5}$$

Other Factors

The characteristics of sediment at the site will affect beach development. The transport characteristics of shingle will be different to fine sand, and the resulting beach profiles will differ. For coarse-grained beaches it has been suggested that the structure should be placed in relatively deeper water, (DALLY and POPE, 1986).

The depth of water at the structure is also a factor, (POPE

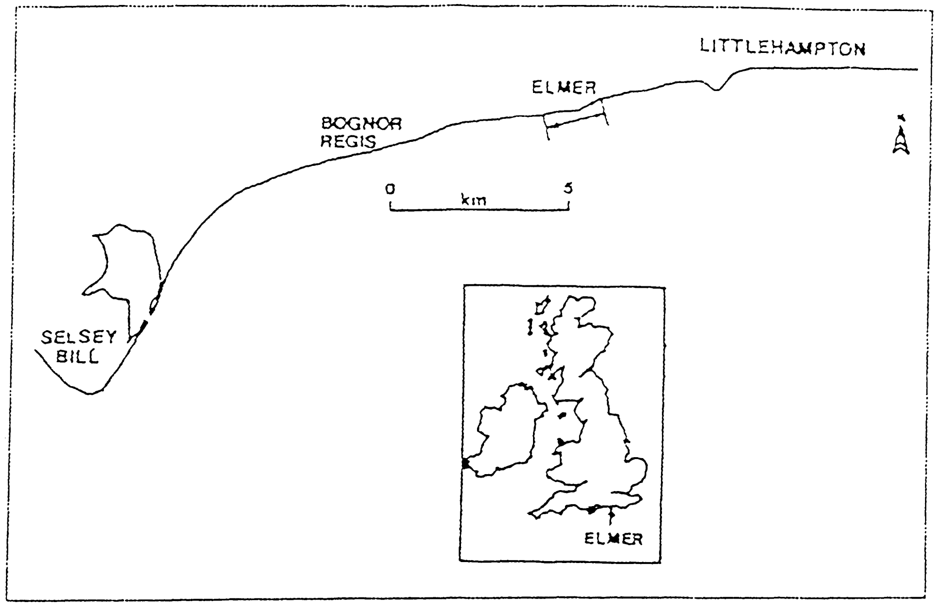

Figure 1. Location map of the Elmer site.

and DEAN, 1986). Breakwaters on an open coast and designed for shore protection are generally placed in water depths ranging from 1 to 8m. It is also suggested that water level fluctuations of over 1.5m will tend to hinder permanent tombolo formation, especially if significant wave overtopping occurs, (DALLY and POPE, 1986).

CASE STUDY: ELMER BEACH, LITTLEHAMPTON, UK

The work reported here investigates the first 3 years of the Elmer site in relation to planform development and sediment transport. It follows earlier work investigating the wave attenuation and transmission properties of the breakwaters, (CHADWICK *et al.*, 1995), and ongoing research is investigating the wave-current interactions in the vicinity of the breakwaters.

Site Details

Elmer is located on the south coast of England between Bognor Regis and Littlehampton, Figure 1, and the current defences were constructed in response to serious storm flooding of the residential hinterland (COOPER *et al.*, 1996b). Initially two offshore breakwaters were constructed in 1990 as 'emergency works' and during the following summer over 11,000m^3 of sand and shingle built up as salients in the lee of these breakwaters, (COOPER, 1996). These two breakwaters were later enlarged and incorporated into the final scheme, (HOLLAND and COUGHLAN, 1994).

The finished scheme comprises eight shore-parallel, intertidal, rock island breakwaters, with a terminal rock groyne at the downdrift end, Figure 2. A beach replenishment of some 200,000m^3 of sand-shingle mix was placed during construction and was an integral part of the scheme. Essentially, the breakwaters were designed to reduce nearshore wave energy and hence retain the nourishment material as the principle form of protection, (HOLLAND and COUGHLAN, 1994). The scheme was completed in August 1993 and is the joint management responsibility of Arun District Council and the Environment Agency.

Hydraulics Research (Wallingford) were commissioned to undertake physical modelling of the Elmer frontage at the design stage, (HR, 1990). Several breakwater layouts were modelled, at a scale of 1:80, using a mobile anthracite beach to simulate the shingle. For the chosen design it was predicted that regular recycling would be initially required, with quantities in the region of 5000m^3 per annum predicted, reducing as the beach stabilised, (HOLLAND and COUGHLAN, 1994). Possible downdrift effects were also considered, however, it was concluded that placement of the nourishment material to an optimum level should result in no reduction in the littoral sediment supply to beaches to the east, (COOPER, 1996).

Breakwater lengths and gaps vary throughout the scheme, with decreasing protection on the beaches to the east. This was intended to produce a smooth transition between the effects of the scheme and the open beach downdrift. The existing rock revetment in bay 5–6 also affected the design. Believing that a lower level of protection was required in this locality due to the presence of the revetment, the breakwaters here were shortened and the gap increased, (COOPER, 1996).

The beach may be described as shingle upper/sand lower, (CIRIA, 1991). The poorly-sorted shingle beach has a pebble size, D_{50}, of 20mm and the sand foreshore has a D_{50} value of 115μm. The mean spring tide range at Elmer is 5.3m (2.9m

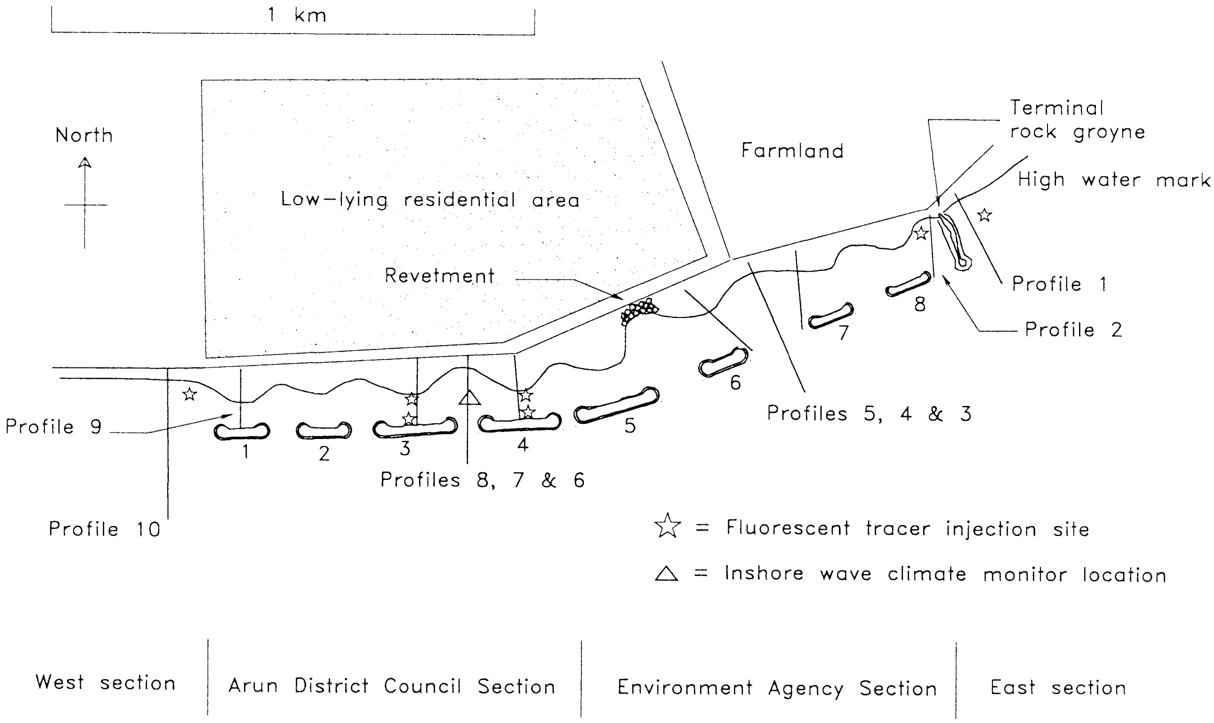

Figure 2. The offshore breakwater scheme layout and experimental details.

on neap tides) and the highest 100-year surge tide is about 3.5m OD. The toe of the breakwaters is exposed on all low tides except extreme neap tides. Local winds are predominantly from the south-west, with 90% of all winds over 11 knots (Beaufort Force 4) from this quadrant, (KING *et al.*, 1996). The net direction of sediment drift is from west to east.

For reference purposes the breakwaters are numbered 1 to 8, in order west to east, and the terminal rock groyne lies to the east of breakwater 8, Figure 2. The bays formed between the breakwaters are referred to by the numbers of the adjacent breakwaters, for example, bay 3–4 is that formed between breakwaters 3 and 4.

The current situation is that well-developed shingle salients have formed and are clearly visible at high tide. At low tide however, it is apparent that sand tombolos have formed, such that as the tide recedes, and not long after high tide, the single expanse of water inshore of the breakwaters becomes a series of individual bays.

ELMER BEACH CASE STUDY—EXPERIMENTAL DETAILS

Medium-Term Beach Profile Monitoring

Monthly surveying of 10 beach profiles was undertaken between April 1994 and April 1996. The profile locations were as shown in Figure 2. Additional profile data, using the same profile lines, were taken during September/October 1996. If the month of scheme completion, August 1993, is designated "month zero", then this monitoring spanned months 8 to 38 inclusively.

Additional analysis was undertaken using aerial survey data obtained from Arun District Council. Data were available for the following dates; August 1993; February 1994; May 1994; September 1994; January 1995 and May 1995 (months 0 to 21 inclusive). The data, in XYZ form, comprised 70 section lines extending from approximately 700m updrift of the scheme to 1400m downdrift.

Short-Term Sediment Transport Study and Beach Surveys

From 18 October to 16 November 1995 intensive fieldwork was undertaken at Elmer, including sediment tracing experiments, daily beach surveys and wave height recording. With the exception of the fluorescent pebble tracing, these experiments were concentrated within bay 3–4.

Wave height measurements were made using the University of Brighton's surface piercing resistive-wire wave sensor, (CHADWICK *et al.*, 1995). It was deployed inshore of the breakwaters in the centre of bay 3–4, Figure 2, and values of significant wave height, (H_{mo}), quoted in the text were measured at this location. Previous research has shown that inshore of the breakwaters wave heights were reduced to between 25 and 62% of their offshore value, Intensive beach surveying was undertaken in order to obtain detailed information on short-term wave induced changes to beach planform. Shingle transport was investigated using both individually-numbered aluminium tracers and indigenous pebbles coated with fluorescent paint.

Unfortunately, during the fieldwork period the wave cli-

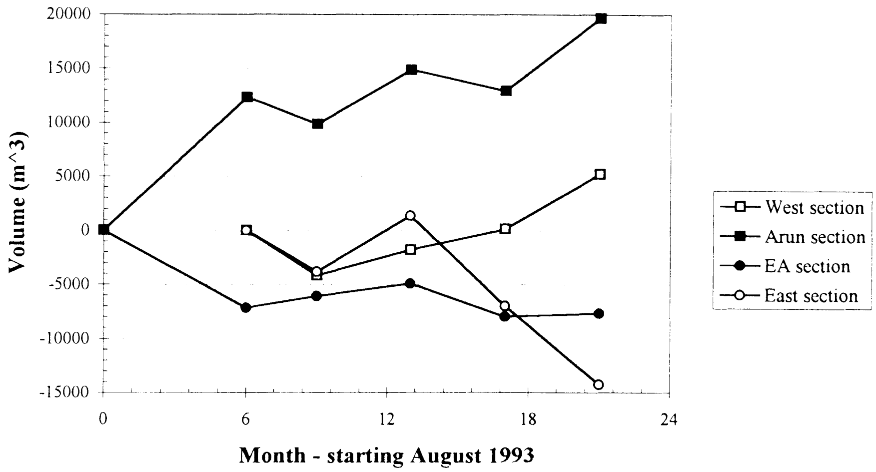

Figure 3. Volumetric analysis of the aerial survey data for the WEST, ARUN, EA and EAST sections.

mate was generally low energy and values of H_{mo} greater than 0.5m were recorded on only 13 occasions. The maximum recorded value of H_{mo} was 1.23m under southerly force 6 conditions. Subsequent analysis of Meteorological Office wind data testifies to the lower than normal wave activity on the south coast during the winter of 1995/1996, (KING *et al.*, 1996).

MEDIUM-TERM BEACH PROFILE MONITORING

Arun DC Aerial Survey Data: Volumetric Analysis

The Arun DC aerial survey data was analysed on a 3D basis to generate a digital terrain model of the Elmer frontage, plus the groyned beaches to the East and West. The frontage was split into four sections to facilitate analysis, these were, WEST, ARUN, EA and EAST, where Arun is the Arun District Council section of the scheme (breakwaters 1 to 4) and EA is the Environment Agency section from breakwater 5 to the terminal rock groyne, (Figure 2). West and East are the groyned beaches to the west and east, respectively, (Figure 2). The West section extends from the first breakwater 600m west of the scheme, and the East section extends from the terminal rock groyne 800m east.

The First 6 Months

From the digital terrain models a volumetric analysis was performed and the results are shown in Figure 3 as volumetric differences from the first available data (August 1993 for Arun/EA sections, February 1994 for the East & West sections). As can be seen in Figures 4 and 5 the renourishment material was placed with some degree of sinuosity in order to replicate the plan shape and level of that obtained from the model testing (1 year geomorphologically averaged condition), (SPENCER, 1995). Despite this, significant changes to planform occurred during this period, Figures 6 and 7, with net volumetric changes of $+12{,}343m^3$ and $-7{,}175m^3$ in the Arun and EA sections, respectively, Figure 3.

A clear trend of deposition in the lee of the breakwaters and erosion in the bays can be observed in Figures 6 and 7, particularly in the lee of breakwaters number 1 and 2. Beach levels changed most dramatically in the Arun section, where, for example, in bay 3–4 the salients/tombolos accreted by up to 2m (in level) and the bay suffered gap erosion of between 1.5 and 2m in places. Significant quantities of material were also lost in the region of the terminal rock groyne, where beach levels fell by up to 4m in places, most probably due to sediment moving through the voids and onto the downdrift beaches.

Months 6 to 21

Between months 6 and 21 there was no significant change in beach planform, although the Arun section, Figure 3, continued to accrete at a rate of roughly $500m^3$ per month. The EA section volume remained relatively stable, despite continuing erosional trends in bay 5–6. This bay, which has a rock revetment as additional protection, required a beach rebuild of $12{,}000m^3$ during October 1994, using material from the salients in the lee of breakwaters 7 and 8. Over this period the main changes were that gap erosion in bays 3–4, 4–5 and 5–6 continued and deposition (of over 0.5m) on the sand tombolos in the lee of breakwaters 3, 4, 5 and 7 also occurred.

During this period the West and East sections gained 5,227 m^3 and lost $14{,}254m^3$, respectively. The loss on the East section was despite replenishment of these beaches by the Environment Agency where material was recycled from downdrift sources, (KING, 1996a).

Overall, between month zero and 21, there was a trend of accretion on the West/Arun sections and erosion on the EA/East sections. These results would suggest that there is insufficient supply of littoral sediment into the EA and East sections from the Arun section.

Profile Line Monitoring

The 10 profile lines were surveyed, using a total station, a total of 29 times between April 1994 and October 1996. Fig-

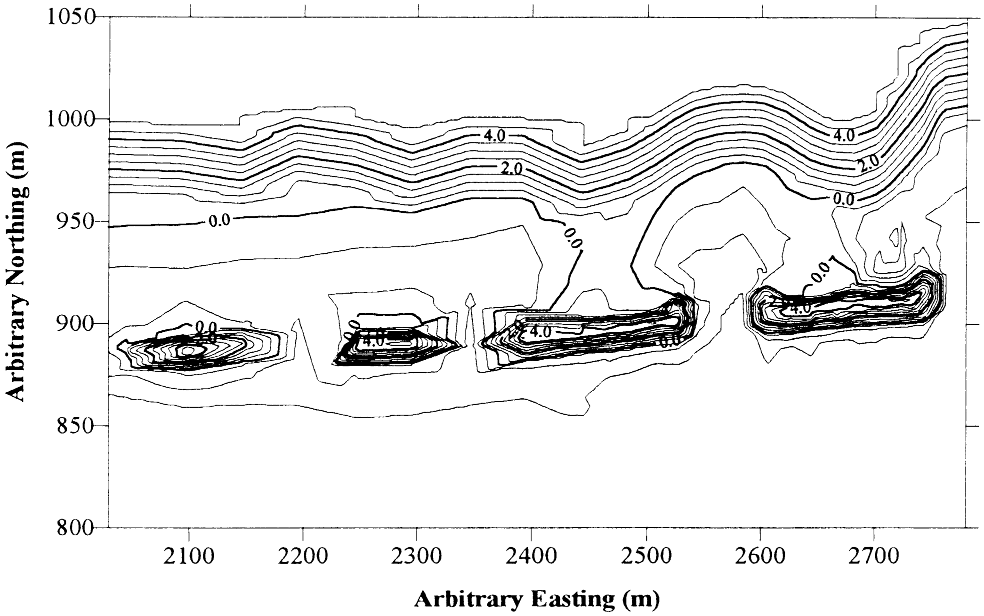

Figure 4. Digital terrain model of the ARUN section (breakwaters 1 to 4), August 1993 (month 0).

ures 8 and 9 show sample results for profiles 7 and 8, located in bay 3–4. Additional profile data was extracted from the Arun DC aerial survey data, giving profiles for August 1993 (month 0) and February 1994 (month 6).

Profile 7, running down the centre of bay 3–4, has changed little over the 38 month period (Figure 8). The main change is the reduction in beach crest width and height between chainages of 10–20m. It should be noted that the origin for this profile is located on the seawall, and immediately behind the seawall lie residential properties, hence beach crest height and width are particularly important here.

Results for the salient profile, profile 8, (Figure 9), show marked accretion during the first 6 months. On-site inspection shows that the accreted material is mainly sand, (D_{50} =

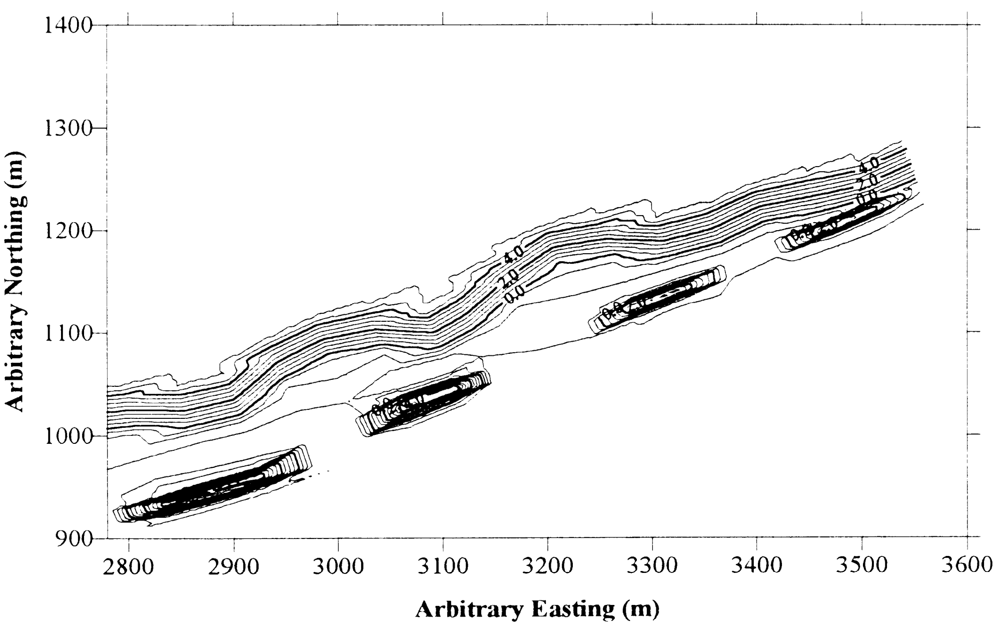

Figure 5. Digital terrain model of the EA section (breakwaters 5 to 8), August 1993 (month 0).

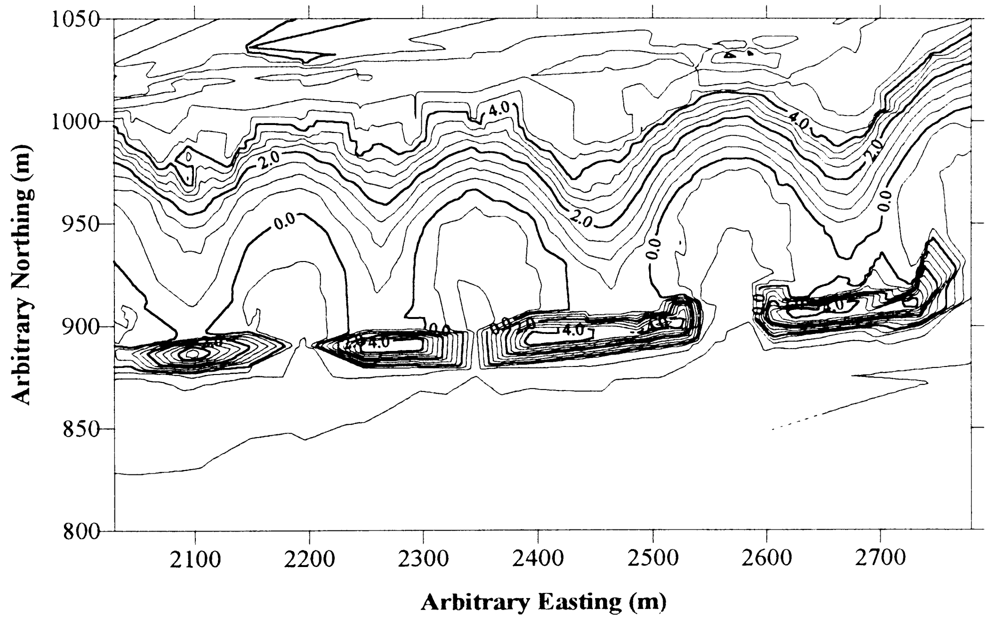

Figure 6. Digital terrain model of the ARUN section (breakwaters 1 to 4), February 1994 (month 6).

0.2mm), below the level of the shingle beach toe, forming a sand tombolo. Again the profile origin is located on the seawall, emphasising the increased beach width at the salients (compared to the centre of bays).

Profile Areas

Profile areas were calculated between specific chainage limits, encompassing the beach crest down to the level of the breakwaters. The profile area data showed that month-to-month variation was significant with respect to the overall trends. Profiles 3, 4 and 5 appeared to be the most volatile, with a large degree of scatter in the results, while the end profiles, 1 and 10, were more stable on a month-to-month basis.

Figure 10 shows the monthly variation in profile area for the two centre of bay profiles, 4 and 7. The data for profile 4

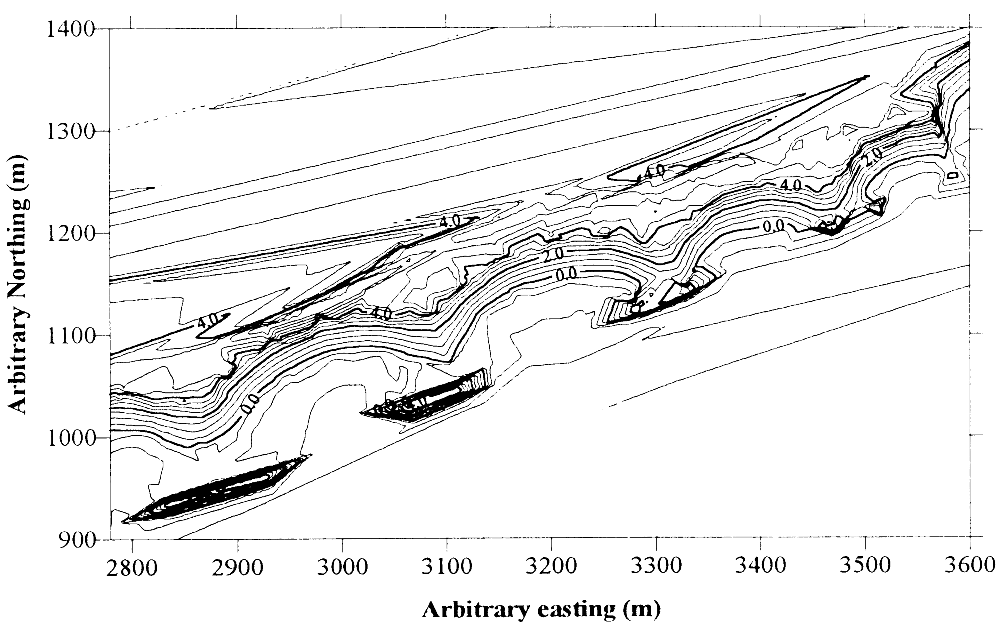

Figure 7. Digital terrain model of the EA section (breakwaters 5 to 8), February 1994 (month 6).

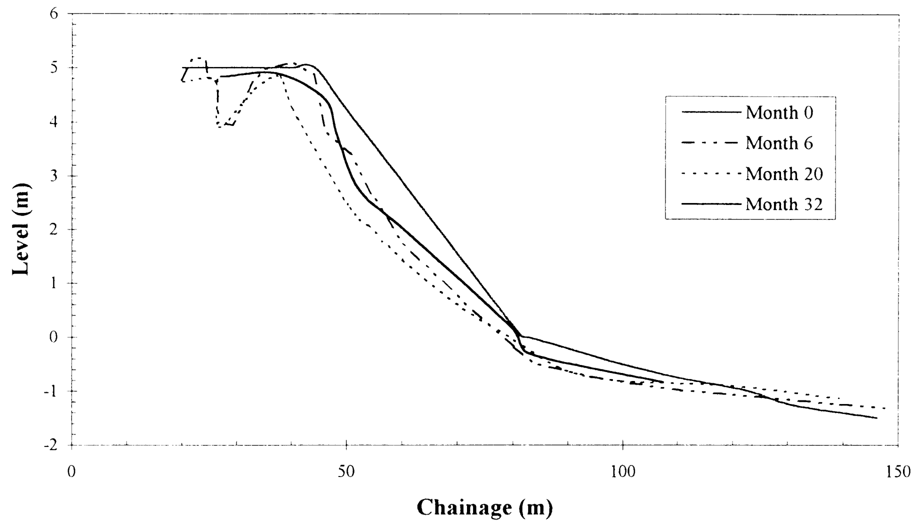

Figure 8. The development of profile line 7 over 32 month period.

exhibit a greater degree of scatter than for profile 7, probably a result of the more exposed location. In both cases the monthly variations are greater than the yearly trend determined from the regression best fit line. Profiles 3 and 5 exhibit similar levels of scatter to profile 4.

Profile areas also exhibited significant variation on the salients/tombolos, Figure 11 shows the results for profiles 6 and 9. The variation along profile 6 (and also 8) was greater than that observed on profile 7. The trends in profile area are listed in Table 1, the regression best fit lines were calculated based *only* on the terrestrially surveyed data between months 8 and 38 (*i.e.* after the initial settling down of the planform).

Overall, Table 1 indicates that the profiles located along the salients (6, 8 & 9) show net accretion. The centre of bay profiles, 4 and 7, show accretion and depletion respectively. At the downdrift end, profiles 1 and 2 indicate depletion.

Beach Crest Width

Beach crest width is defined as the plan distance from the profile origin to the +2.0m OD contour. The +2.0m OD level roughly corresponds to the average high tide level at Elmer (2.33m OD). Using linear interpolation, the chainage associated with the level +2.0m OD was determined for each profile. Thus an increase or decrease in chainage reflects seaward or landward planform movement at that location, respectively.

In general terms, the beach crest width results were sim-

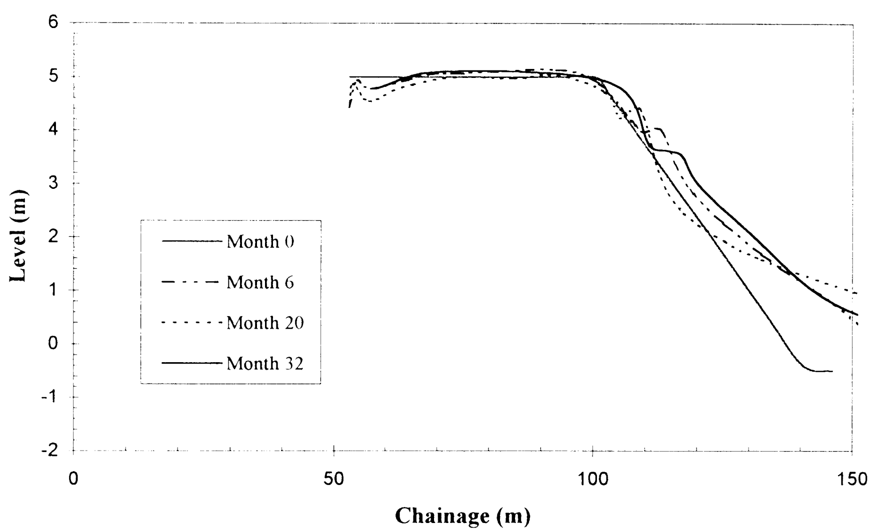

Figure 9. The development of profile line 8 over 32 month period.

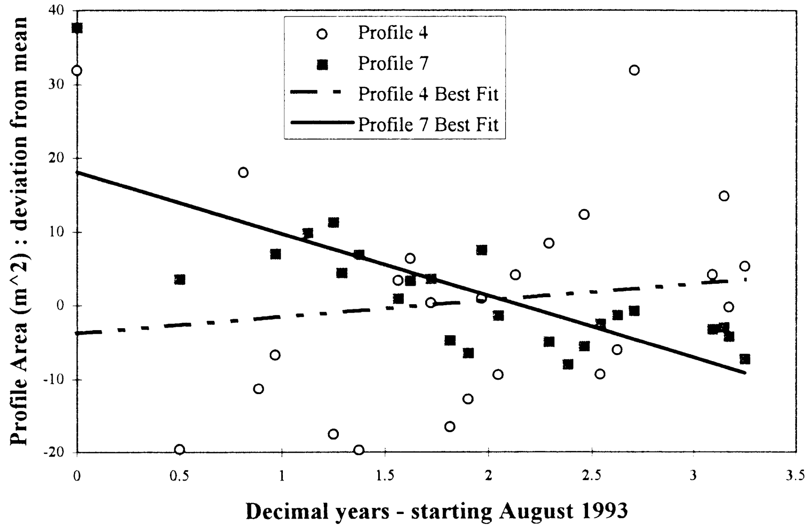

Figure 10. The change in profile area (from mean value) for the centre of bay profiles, (4 and 7).

ilar to those for profile area. Results for profiles 4 and 7 are shown in Figure 12. A large monthly variation, particularly for profiles 3, 4 and 5, was again evident. In bay 3–4 the profiles 6 and 8 exhibited a greater variation than the centre profile (7). The trends in beach crest width, between months 8 and 38, are listed in Table 1.

COMPARISON WITH DESIGN GUIDELINES

The design wavelength for the Elmer scheme, λ_d, was 70m, (COOPER, 1996), the guidelines suggested by ROSATI (1990) therefore indicate that breakwater lengths at Elmer should be around 140m or larger and positioned at a similar distance offshore, (equation (1)). Table 2 indicates that generally they are much smaller than this and were positioned closer to the replenished beach than recommended. This suggests that some advantage in beach sediment retention would be achieved by extending breakwater lengths, especially in the EA section where the differences are greatest.

The second of ROSATI'S (1990) guides, equation (2), states that breakwater gaps widths, G, should be less than the wavelength of incident waves. Only three of the bays fulfil this criteria, based on λ_d, with bays 5–6 and 6–7 exceeding the recommended gap width by a significant factor.

Beach response to the breakwaters, in terms of tombolo or salient development, is predicted by L/X, (CHASTEN *et al.*, 1993; DALLY and POPE, 1986), and 1.5 is the limit stated by

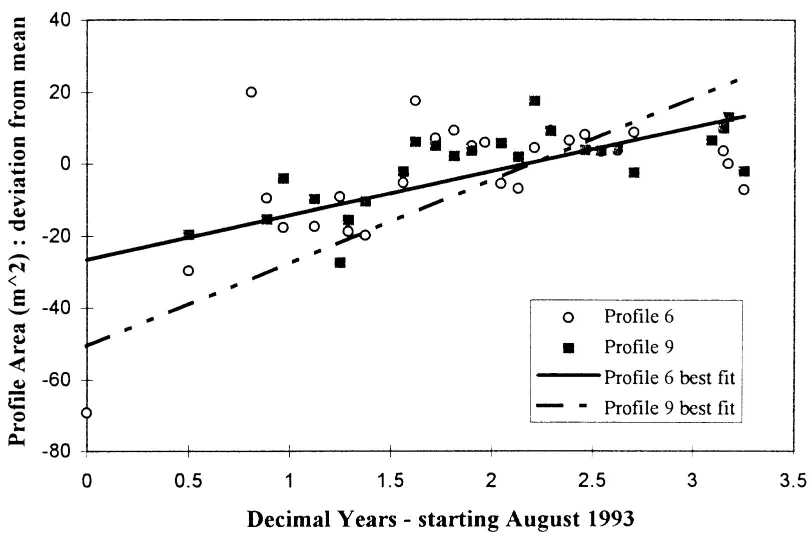

Figure 11. The change in profile area (from mean value) for two salient/tombolo profiles, (6 and 9).

Table 1. *Trends in beach crest width and beach profile area.*

Profile	1	2	3	4	5	6	7	8	9	10
Beach crest width (m/year)	−1.1	−0.2	+0.9	+0.6	−1.3	−1.7	−2.0	+1.2	+2.0	−1.2
Profile area (m^2/year)	−3.3	−2.7	+11.0	+6.2	−7.4	+4.9	−6.1	+3.7	+9.0	+4.6

CHASTEN *et al.* (1993) for tombolo formation. Thus, as illustrated in Table 2, tombolo formation is predicted in the lee of all breakwaters, except 1, 2 and 6. However, in all cases the L/X values exceed the recommended range for salient features with no tombolo formation (0.5 to 0.67), (DALLY and POPE, 1986).

The possibility of gap erosion is predicted by G/X, (SEIJI *et al.*, 1987). For no erosion they suggest a limit of 0.8, and this limit is exceed for all bays at Elmer except those formed by breakwaters 2 to 5, Table 2. It would therefore be expected that any gap erosion would be concentrated in the EA section of the scheme. However, during the first 6 months there was erosion in all bays, with beach levels falling by around 2m. Much of this material was probably lost to the salients and a comparison of gross gains/losses against the net changes also suggests this (KING, 1996b).

SHORT-TERM 3D SURVEY OF BAY 3–4

During the October/November 1995 fieldwork programme, bay 3–4 was surveyed daily with the intention of identifying the areas of erosion and accretion attributable to particular storm events. The survey area was bounded by profiles 6 and 8, the seawall and the line of the breakwaters; an area of approximately 12000m^2. Typically each survey consisted of around 900 data points which were surveyed using a total station. Unfortunately, during the fieldwork period, there were few high energy events and the weather was calm for much of the period.

The results from this exercise indicated that, under the conditions experienced and despite the high number of survey points, no significant changes in planform occurred in the survey region. Observed changes in volume, when divided by the survey area, were of similar magnitude to the survey experimental error for any given point height. Despite this there was evidence that offshore wave direction would influence the beach, creating opposing areas of erosion/deposition. Further details on this work are contained in KING *et al.* (1996).

RESPONSE TO MAJOR STORM EVENT

On the 28 and 29th of October 1996 a major storm occurred on the south coast of England, coinciding with Spring tides of 6.1m range. Gale force south-westerly winds reached 90mph and inshore H_{mo} (bay 3–4) was recorded as 1.96m at high tide. Offshore wave heights, measured at Shoreham, some 30km east of Elmer, were recorded at 3.6m. Despite the extreme conditions no flooding occurred at Elmer and in this respect the scheme fulfilled its design criteria. A survey of the 10 routinely measured profile lines had previously occurred on the 3rd October, and a post-storm survey was undertaken on the 31st October/1st November.

The effects of the storm on profile area and beach crests width can be seen in Figures 10 to 12, where pre-storm data is the penultimate data point, and the post-storm data point is the last point. The effects of the storm on profiles 4 and 7 are shown in Figures 13 and 14. These are compared to a profile taken on the unprotected, downdrift beaches, some 550m east of the terminal rock groyne, Figure 15. The down-

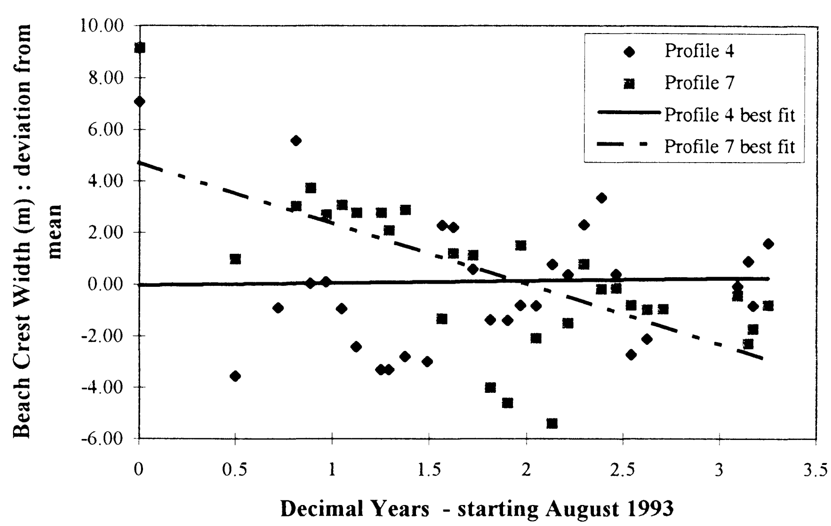

Figure 12. The change in beach crest width, defined as the +2.0m OD level, for the centre of bay profiles, (4 and 7).

Table 2. *Elmer breakwater scheme: breakwater dimensions and design parameters.*

Breakwater	Crest Elevation (OD) (m)	L (m)	G (m)	X (m)	L/X	G/X
1	4.5	90		85	1.06	
			80			1.0
2	4.5	90		79	1.14	
			60			0.8
3	4.5	140		75	1.86	
			60			0.8
4	4.5	140		77	1.82	
			44			0.5
5	4.5	140		88	1.59	
			100			1.4
6	4.5	80		54	1.48	
			140			2.3
7	3.0	80		68	1.18	
			80			1.5
8	3.0	80		38	2.11	

drift profile was clearly affected more by this storm, with a significant movement of material to the lower portion of the profile, causing around 10m of beach width to be moved. Profile 4 exhibited similar trends, although to a lesser extent, whereas profile 7 was relatively unaffected by the storm. The effects of the increased gap width in bay 6–7 can be clearly seen in the differences between profiles 4 and 7 and this accounts for the greater scatter observed in Figures 10 and 12. On all three profiles the sand foreshore, in the inter-tidal region, shows no indication of the effects of the storm.

DETERMINATION OF SEDIMENT TRANSPORT USING TRACING TECHNIQUES

Qualitative Fluorescent Tracer Study

The fluorescent tracer consisted of indigenous pebbles, D_{50} = 40mm, coated with red, blue, yellow or white fluorescent paint. Deployments were of 1000–2000 pebbles per injection site and these sites are indicated in Figure 2. Searches were made during daylight and also at night using a UV lamp, utilising the fluorescent properties of the paint (ZENKOVICH, 1967). Pebble recoveries were limited to those pebbles visible at the surface. Overall the fluorescent tracer was excellent as an indicator of shingle movement, and enabled the aluminium tracer to be deployed more effectively. The main results from these experiments are discussed below.

Experiment (i)

Tracer deployed on the salient headlands, just below the high tide level, in the lee of breakwaters 3 and 4 clearly showed the potential for bay-to-bay longshore transport. S/SE wave conditions on the first tide moved all tracer west into the adjacent bay, with centroid displacements of around 18m. Predominantly SW conditions over the following 12 tides moved the tracer centroids back around the headlands in the next bay at an average rate of 3–4m per tide. Recovery rates ranged between 5 and 15%.

Experiment (ii)

Tracer deployed *immediately* in the lee of breakwaters 3 and 4, opposite the salient tips, showed no movement after 4 tide cycles, during which the wind peaked at a SW force 6 (≈13m/s), and inshore H_{m0} reached 1.23m. 97% of deployed pebbles were recovered, apparently unmoved despite the high energy conditions, indicating that shingle transport in the immediate lee of the breakwaters is unlikely.

Experiment (iii)

Simultaneous experiments were carried out at the east and west ends of the scheme with the aim of comparing shingle transport at either end of the scheme. Three tracer deployments were made, two at the east end of the scheme (either side of the terminal rock groyne) and one at the west end of the scheme. Over the 17 day period, the wind was mostly

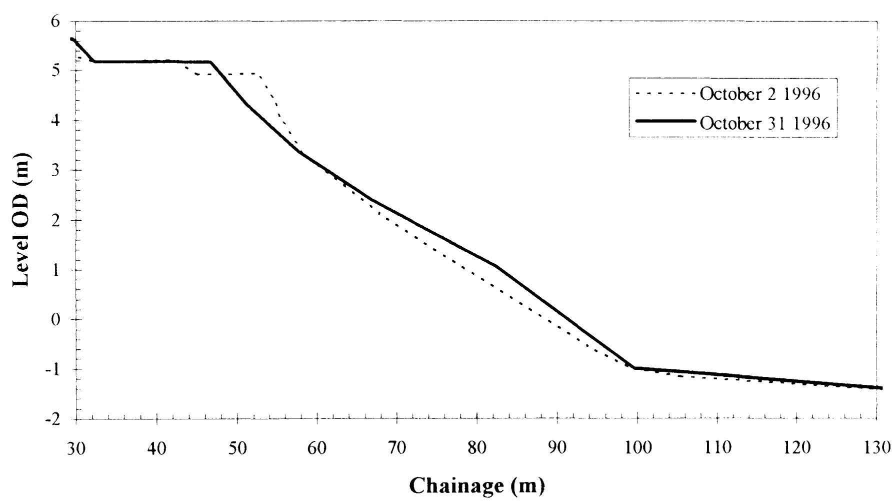

Figure 13. The effects of a major storm on profile 4.

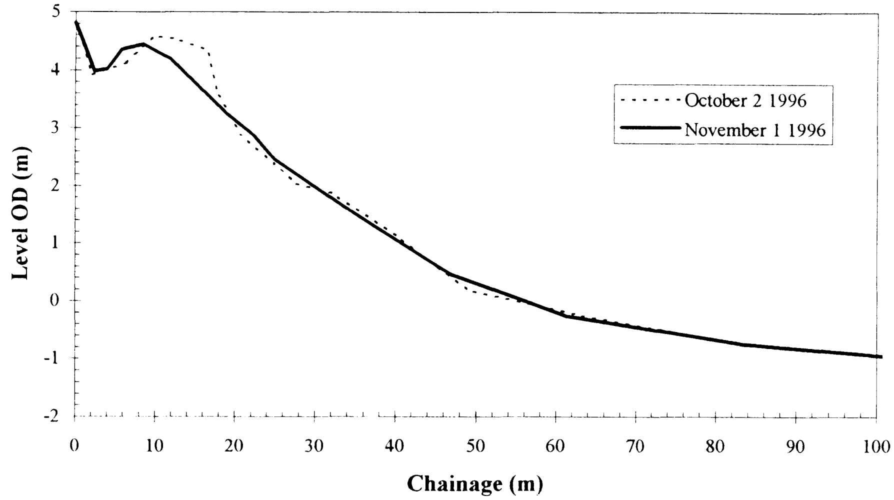

Figure 14. The effects of a major storm on profile 7.

below F3, often from the N/NW direction and the only notable wave activity was 3 days of SW/W F4/5 winds at the end of the experiment.

Despite the calm conditions, tracer deployed updrift of the terminal rock groyne was observed to pass through it and onto the groyned beaches to the east. Tracer deployed downdrift of the terminal rock groyne moved eastwards at a reasonable rate (considering the conditions). However tracer deployed at the west (updrift) end of the scheme indicated no movement into the scheme under the same conditions.

Although the fluorescent tracer technique has now been superseded by aluminium pebble or electronic pebble tracer methods (BRAY *et al.*, 1996; COOPER *et al.*, 1996c), field studies directly comparing fluorescent and aluminium pebbles indicate that the recovered fluorescent pebble distribution closely follows the aluminium pebble distribution, despite the large difference in recovery rates (COOPER *et al.*, 1996a). Recovery rates, of around 10%, compare well with data from the aluminium study which indicates that generally up to 10% of the aluminium pebble tracer is visible at the surface on any given recovery, (COOPER *et al.*, 1996c).

Quantitative Aluminium Tracer Study

The aluminium pebbles were each stamped with a unique number allowing individual pebble movements to be traced daily over a number of days. They were of an average mass of 97.4g, with a b-axis diameter of approximately 37mm. Compared with the indigenous material, the pebbles represent the 84–90th percentile size. Pebbles were deployed in

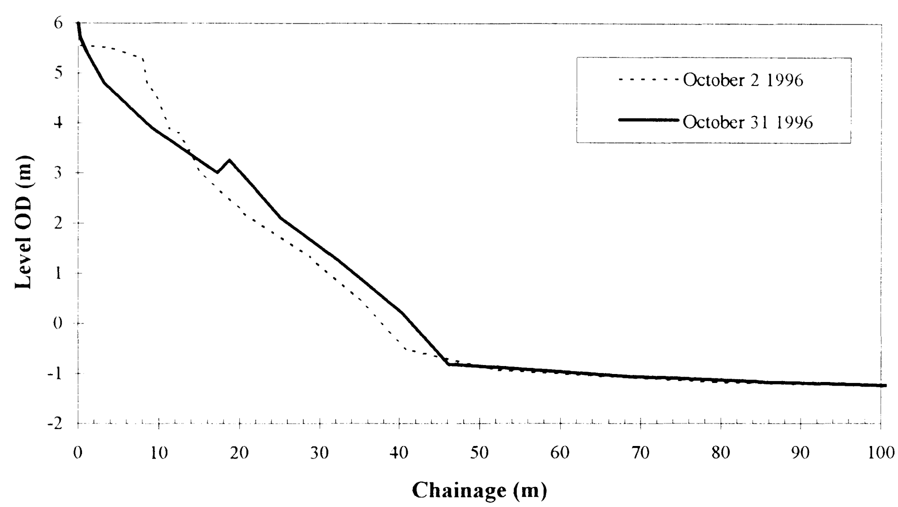

Figure 15. The effects of a major storm on a profile line on an unprotected beach approximately 550m downdrift (east) of the breakwater scheme.

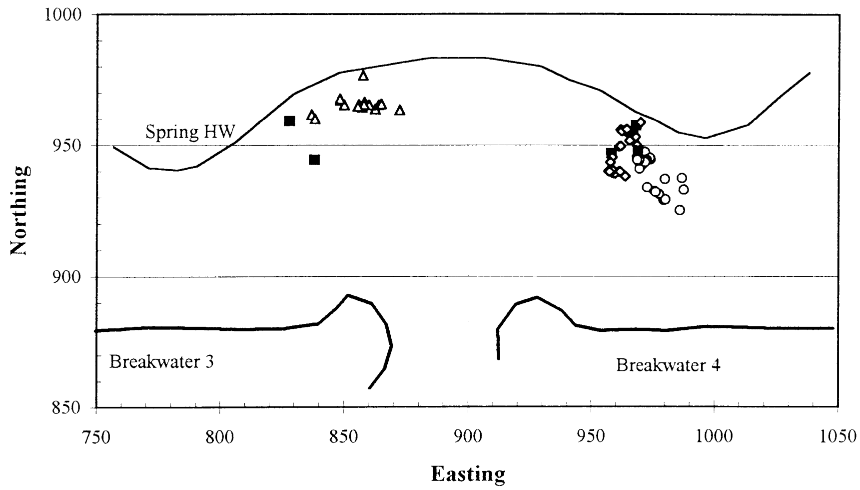

Figure 16. Aluminium tracer injections sites and recovery positions for exposure to medium energy southerly/south-westerly conditions. Data compiled from 3 experiments. Black squares indicate injection positions, open symbols are recovery positions; circles = position after 2 tides; squares = position after 1 tide and diamonds = position after 3 tides.

batches around the bay, with generally 12–24 pebbles per batch. Recoveries were made using metal detectors and recovery rates per search were very good with an average daily rate of 85%. Pebbles were detectable at depths of up to 0.3m.

Sample results are presented for aluminium tracer experiments conducted during October/November 1995, (Figures 16 and 17). Table 3 summarises the wind and wave conditions for each tracer experiment. The E and SE conditions shown in Figure 17 indicate the potential for material movement from bay-to-bay around the salient headlands. The movement around the salient headlands confirms the results from the fluorescent tracer experiments described earlier.

Both aluminium and fluorescent tracer movements were generally confined to the shingle beach, although occasionally pebbles would be rolled down onto the sand foreshore by the falling tide. However the high daily recovery rate suggests that on-offshore movement of shingle is not significant.

Rates of Transport

The centroid of the tracer population was calculated and its longshore movement was obtained both parallel to the local beach contours and as net east-west movement. Positive is defined as "easterly" movement. The net centroid displacement, from the origin, was defined as ΔE for movement in the east-west direction and ΔL for movement parallel to the local beach contours.

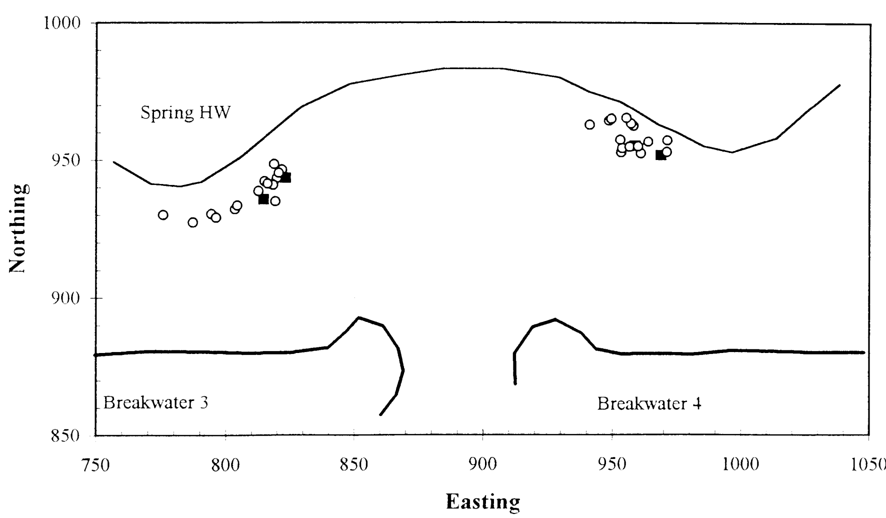

Figure 17. Aluminium tracer injections sites and recovery positions for exposure to 6 tides of low to medium energy easterly/south-easterly conditions. Black squares indicate injection positions, open circles indicate recovery location.

Table 3. *Summary of wind and wave conditions for the aluminium tracer results.*

Figure		Wind	Inshore H_{m0} range (m)
16	Data compiled from 3 experiments under similar conditions	S or SW, Force 4/5/6 (6–14 m/s)	0.51–1.23
17	Data from one experiment only	E or SE, Force 2/3/4 (2–8m/d)	0.10–0.60

The standard method for calculating volumetric transport rates from tracer studies involves identifying a layer of moving sediment from the tracer movements, (BRAY *et al.,* 1996). The volumetric transport rate, Q, is defined by its mean width, (m), thickness, (n), and velocity (U) as shown in equation 6. The mean width is the width of the active shingle beach and the mobile layer thickness may be estimated from the burial distribution of the tracer. BRAY *et al.* (1996) used the mean depth of the deepest 50% of tracers to estimate n. The velocity, U, is obtained from the longshore movement of the tracer centroid per tide.

$$\text{Volumetric transport rate per tide,} \quad Q = m.n.U \quad (6)$$

Table 4 lists the derived volumetric transport rates. The volumetric transport rates were calculated using ΔL, *i.e.* distance measured parallel to the local beach contours.

The maximum rate of transport recorded during the fieldwork was 57m^3/tide, however transport rates were typically an order of magnitude lower than this. These results were for the most protected bay on the Elmer scheme, and therefore transport rates in the other bays could be expected to be higher. The transport of shingle from bay to bay was demonstrated to occur via the salient headlands by the fluorescent tracer. As indicated by Figures 16 and 17, the direction of transport within the bay depends on offshore wave angles, (assuming that wave direction was approximately the same as the prevailing wind direction).

Comparison with Transport Rates on an Unprotected Beach

Previous work on the open shingle beach at Shoreham on the South Coast of England, using the same aluminium tracers, obtained transport rates of around 10m^3/tide for low energy conditions, rising to 150m^3/tide for medium-energy conditions, and a single high energy event recorded transport rates of 3000m^3/tide, (BRAY *et al.*, 1996). A lack of comparable offshore wave measurements makes comparison difficult as the breakwaters significantly reduce wave energy incident on the shingle beach. Also the overall conditions during the Elmer study (October/November 1995) were much calmer than the earlier Shoreham experiments (September 1995). However, if the maximum transport rate obtained during these experiments (just under 60m^3/tide during SW F4/5/6 conditions) is compared to the 'medium-energy' value of 150m^3/tide from BRAY *et al.* (1996) it suggests that transport rates in bay 3–4 are around half of those occurring on open beaches. A similar value is obtained if the 'low-energy' transport rate is compared to the transport rate under E/SE F2/3/4 conditions, Table 4.

Table 4. *Aluminium tracer: centroid displacements and volumetric transport rates.*

Figure	Injection Site	No. of Tides	m (m)	n (m)	ΔE (m/tide)	ΔL (m/tide)	Q (based on ΔL) (m^3/tide)
16	West i	2	26	0.15	14.55	14.59	56.9
	West ii	2	26	0.15	7.46	11.12	43.4
	East i	1	22	0.12	1.44	3.06	8.1
	East ii	1	22	0.12	−1.96	−0.44	−1.2
	East iii	3	18	0.11	2.08	2.95	5.8
17	West i	6	16	0.11	−2.37	−2.40	−4.2
	West ii	6	18	0.11	−1.83	−1.89	−3.7
	East i	6	26	0.11	−1.08	−1.36	−3.9
	East ii	6	24	0.11	−1.08	−1.21	−3.2

DISCUSSION AND CONCLUSIONS

Survey data indicates that the scheme, following rapid adjustment in the first 6 months, is now reasonably stable. The rapid changes that were seen to occur in the first few months emphasize the need for early monitoring of such schemes if the initial settling down is to be recorded. Trends identified from the results of the profile survey indicate that renourishments may be required in some places over the next 5–10 years. If sustained, then the modest rate of erosion determined for the centre of bay 3–4 (profile 7), will become important as the protective beach is relatively narrow at this locality. The lack of storm activity on the south coast during the winter of 95–96 would, if anything, indicate that this rate may actually increase under 'normal' conditions. Continued monitoring of the critical areas, such as bay 3–4 and bay 5–6, is therefore recommended.

The monthly variations in both profile area and beach crest width are also important. Most significantly, for all 10 profiles month-to-month variations were larger than the indicated yearly trend. Thus temporary, but significant, depletion of beach material in critical areas may occur. This may necessitate renourishment of certain areas to act as a 'buffer' against high energy events and is important if pro-active renourishments are planned according to the medium/long term trends in planform.

The scale model tests of the scheme suggested that in-scheme periodic replenishments of 5000m^3 per annum would initially be required, reducing as the beach stabilised, (HOLLAND and COUGHLAN, 1994). These have not been required, although some re-location of material was required in the EA section in October 1994. Overall the scheme appears to be performing well in its capacity as a shingle retention structure.

However this may be at the expense of the downdrift beaches, where both aerial data and the profile data indicate losses over the period. This has necessitated re-cycling work by the Environment Agency, which has been carried out twice-yearly on these beaches since May 1993. At the time of writing approximately 100,000m^3 of material has been re-cycled onto these beaches from downdrift sources, (KING, 1996a). This contrasts with the results of the physical modelling study which predicted that the scheme would have no undue effects on downdrift beaches, (HR, 1990). However, due to scaling criteria, modelling of the sand component at Elmer was not possible. It is the accumulation of sand which has formed the tombolo features, and these features may have caused a reduction in the longshore transport of shingle. Results from the aerial survey data suggest that it is the Arun section that is blocking the supply of littoral sediment, as this section has recorded continued accumulation of material since completion.

The empirical design guidelines suggest that there would be some advantage in increasing breakwater lengths and reducing breakwater gaps in the Environment Agency section of the scheme. This would reduce the effects of erosion in bays 5–6 and 6–7. Also they suggest that all eight breakwaters were constructed closer to the shore than recommended for salient growth without tombolo features. The resulting large (shingle) salient growth in the lee of the three largest breakwaters has resulted in the movement of the protective shingle beach from the centre of the bays (where the beach is narrowest and wave energy highest) to the salients where beach crest width is now much larger than necessary for protection of the hinterland. A less sinuous beach planform would have provided better protection for the areas opposite the gaps (*i.e.* the areas where the protective beach width is at a minimum).

The results, based on the first 38 months of the Elmer scheme, illustrate the potential difficulties in designing schemes of this nature. The shingle salients have developed to an extent not predicted by the initial modelling tests. Also, the ability of the scheme to 'trap' sand and the subsequent development of sand tombolos is now an important feature of the Elmer site. Future schemes based on similar beaches should consider the effects of offshore breakwaters on both shingle and sand components.

Tracer experiments indicate that longshore transport of shingle occurs via the salient headlands. No evidence of shingle transport in the *immediate* lee of the breakwaters was found. Within the bay transport rates vary significantly, and this is dependent on incident offshore wave angle, which subsequently determines inshore wave angles and wave height distribution. Transport rates of up to 57m^3/tide were recorded, but were generally an order of magnitude lower than this. In comparison with open beaches this figure indicates that the breakwaters reduce the rate of transport by a factor of at least 2.

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LITERATURE CITED

ABDEL-AAL, F.M., 1993. Detached breakwaters for beach protection. Coastal zone: Proceedings Symp. Coastal & Ocean Management, 1993, *In:* FABBRI, P. (Volume ed.; Series editor: O.T. Magoon), *Coastlines of the Mediterranean*. New York: ASCE, pp.35–47.

ANDREWS, J., 1996. The Sidmouth phase 2 coast protection scheme. *Proceedings 31st MAFF Conference of River and Coastal Engineers*, Keele University, UK.

ANTHONY, E.J. and COHEN, O., 1995. Nourishment solutions to the problem of beach erosion in France: the case of the French Riviera. *In:* HEALY, M.G. and DOODY, J.P. (eds.), *Directions in European Coastal Management*, Samara, Cardigan, UK, pp.199–206.

BARBER, P.C. and DAVIES, C.D., 1985. Offshore breakwaters—Leasowe Bay. *Proceedings Institution of Civil Engineers*, Part 1, 77, pp.85–109.

BRAY, M.J.; WORKMAN, M.; SMITH, J., and POPE, D.J., 1996. Field measurements of shingle transport using "Electronic" tracers. *Proceedings 31st MAFF Conference of River and Coastal Engineers*, Keele University, UK.

CHADWICK, A.J.; POPE, D.J.; BORGES, J., and ILIC, S., 1995. Shoreline directional wave spectra, part 2, instrumentation and field measurements. *Proceedings Institution of Civil Engineers, Water, Maritime & Energy*, 112, September, pp.209–214.

CHASTEN, M.A.; ROSATI, J.D.; MCCORMICK, J.W., and RANDALL, R.E., 1993. Engineering design guidance for detached breakwaters as shoreline stabilization structures. *Technical report CERC-93-19*, US Army Engineers, Waterways Experiment Station, Coastal Engineering Research Center.

CIRIA, 1991. *Manual on the Use of Rock in Coastal and Shoreline Engineering*. CIRIA Special Publication 83, CUR Report 154.

COOPER, N.J., 1996. The Engineering Performance and Geomorphic Impacts of Shoreline Management at Contrasting Sites in Southern England. PhD thesis, Departments of Geography and Civil Engineering, University of Portsmouth, Portsmouth, UK.

COOPER, N.J.; BRAY, M.J.; KING, D.M., and POPE, D.J., 1996a. Field measurements of fine shingle transport. *Proceedings of Tidal '96*, University of Brighton, pp.42–51.

COOPER, N.J.; KING, D.M., and HOOKE, J.M., 1996b. Collaborative research studies at Elmer Beach, West Sussex, UK. Proceedings of EUROCOAST Littoral '96 *In:* TAUSSIK and MITCHELL (eds.), *Partnership in Coastal Engineering*, Samara, Cardigan, UK, pp.369–376.

COOPER, N.J.; WORKMAN, M., and COATES, T.T., 1996c. Field monitoring of shingle beaches at Shoreham and Elmer, West Sussex. *Report TR8*, HR Wallingford.

DALLY, W.R. and POPE, J., 1986. Detached breakwaters for shore protection. *Technical report CERC-86-1*. US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center.

FRIED, C.E., 1976. Protection by means of offshore breakwaters. *Proceedings 15th Coastal Engineering Conference* (ASCE), pp.1407–1424.

GARDNER, J. and RUNCIE, R., 1995. Planning and construction of four offshore reefs in Norfolk. *Proceedings 30th MAFF Conference of River and Coastal Engineers*, Keele University, UK.

HOLLAND, B. and COUGHLAN, P., 1994. The Elmer coastal defence scheme. *Proceedings 29th MAFF Conference of River and Coastal Engineers*, Loughborough University, UK.

HR, 1990. Coastal defence works, Elmer, West Sussex: Hydraulic model tests. *Report No EX2245*, HR Wallingford.

ISLE OF WIGHT COUNCIL, 1996. Meeting the challenge of coastal defence: Part 1 South Coast. Publicity brochure, published by Isle of Wight Council.

KING, D.M., 1996a. The Elmer offshore breakwater scheme, effects on downdrift beaches: preliminary results and analysis. Hydraulics Engineering Research Unit, Department of Civil Engineering, University of Brighton, UK, Internal report.

KING, D.M., 1996b. Elmer breakwater scheme, West Sussex: Arun District Council aerial survey data. Hydraulics Engineering Research Unit, Department of Civil Engineering, University of Brighton, UK, Internal report.

KING, D.M.; MORFETT, J.C.; POPE, D.J., and COOPER, N.J., 1996. A detached breakwater scheme: sediment transport and beach profile development at Elmer, West Sussex. *Proceedings of Tidal '96,* University of Brighton, UK, pp.52–69.

KOIKE, K., 1988. Japan. *In:* WALKER, H.J. (ed.), *Artificial Structures and Shorelines,* Dordrech: Kluwer, pp.317–330.

LOVELESS, J.M. and COPELAND, G.J.M., 1991. Developments in the design of offshore breakwater schemes. *In:* PEREGRINE, D.H. and LOVELESS, J.M. (eds.), *Symposium on Developments in Coastal Engineering,* University of Bristol, UK, pp.65–77.

MCCORMICK, M.E., 1993. Equilibrium shoreline response to breakwaters. *Journal Waterway, Port, Coastal & Ocean Engineering* (ASCE), 119(6), 657–670.

NIR, Y., 1988. Israel. *In:* WALKER, H.J. (ed.), *Artificial Structures and Shorelines.* Dordrech: Kluwer, pp.253–260.

NRA, 1993. Happisburgh to Winterton sea defences. NRA (Anglian region) leaflet. Published by Public Relations Dept, Reference P25/7/93.

POPE, J. and DEAN, J.L., 1986. Development of design criteria for segmented breakwaters. *Proceedings 20th Coastal Engineering Conference* (ASCE), pp.2144–2158.

ROSATI, J.D., 1990. Functional design of breakwaters for shore protection: Empirical methods. *Technical report, CERC-TR-90-15,* US Army Engineer. Waterways Experiment. Station. Coastal Engineering Research Center.

SEIJI, M.; UDA, T., and TANAKA, S., 1987. Statistical study on the effect and stability of detached breakwaters. *Coastal Engineering in Japan,* 30(1), 131–141.

SILVESTER, R. and HSU, J.C.R., 1994. *Coastal Stabilization: Innovative Concepts,* Englewood Cliffs, New Jersey: Prentice-Hall.

SPENCER, R., 1995. A beach restored: Elmer, West Sussex. Presentation at Seminar on Shoreline Management, Hydraulics Research Ltd., Wallingford, June 1995.

ZENKOVICH, V.P., 1967. *Processes of Coastal Development.* London: Oliver & Boyd.