An Assessment of Processes and Strategies for Management of the Penarth Coast

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ABBREVIATIONS

AMSL Above mean sea level.
AOD Above Ordnance Datum

CBDC Cardiff Bay Development Corporation

CHA Cardiff Harbour Authority
CPA Coastal Protection Authority
CZM Coastal Zone Management

DoC Depth of Closure

EIA Environmental Input Assessment
ICM Integrated Control Management
ICZM Integrated Coastal Zone Management

IPCC Intergovernmental Panel on Climate Change

kn Knots m metre

MHW Mean High Water

MHWN Mean High Water Neaps MHWS Mean High Water Springs

MRI Management Response Indicator

MSL Mean Sea Level

NE Northeast

NGR National Grid Reference

NNE North North East NNW North North West OBM Ordnance Bench Mark OD Ordnance Datum

R² Coefficient of Determination REBAD Reliability Based Design

RNLI Royal National Lifeboat Institution
SBCEG Swansea Bay Coastal Engineering Group

SGCC South Glamorgan County Council SMP Shoreline Management Plan

SW Southwest

VGC Vale of Glamorgan Council

VGCC Vale of Glamorgan County Council

WCB Whole Circle Bearing

ABSTRACT

In 1997, the effects of severe erosion along the Penarth foreshore became apparent when the beach surface fell to critical levels. From consideration of global influences, the physical and cultural environments and anthropogenic activities, including the construction of the Cardiff Bay Barrage, a five-year monitoring programme was devised to assess coastal processes, identify possible causes of the erosion and develop management strategies for protection of the Penarth coast. Approximately 1.5 km of the foreshore, comprising two orientations, NNE and NNW, were surveyed each September and April between 1997 and 2002, to assess summer and winter changes. Results showed in September 1997, sediment transport was southerly in direction whilst from April 2000, there was a consistent return to the traditionally accepted south to north longshore drift; verified by significant differences in longshore gradients. Foreshore analysis provided important regression models representing the variation of the shoreline indicator Mean High Water (MHW) with shoreline position (mean beach level) and gain/loss of beach material. Models represented the temporal variation of MHW and depth of closure with further significant correlation between the two-shoreline indicators (83.85%). Management Response Indicator (MRI) equation provided a simple tool to rapidly assess the health of Penarth beach, MRI = $17.035 + \tan^{-1}(x_{14} - x_6)/240$, whilst temporal variation of longshore gradients, demonstrated an important inter-relationship between beach evolution and orientation (97.37%). Significant temporal and spatial models identified changes in beach formation adjacent to the Barrage breakwater pre and post construction and showed its influence decreased with distance. A further management tool was developed to monitor beach evolution and morphological change in response to this structure. Analysis of ten-year water and wind data identified a projected mean sea level rise of 0.4mm/year and significant increases in easterly wind components between 1995 and 1997, coinciding with significantly higher extreme water levels. Following correlation of these forcing agents with shoreline indicators, supported by documented beach losses of 350 kg/m² subsequent to an easterly storm, it was concluded that the unprecedented erosion of Penarth beach was caused by increased wave attack from the northeast and southeast quadrants. Function Analysis justified foreshore management from a development perspective whilst risk analysis supported management strategies. Initial recommendations of on-going monitoring to assess beach health were supported by developed models and underpinned by management responses detailed for each of three defined beach sections. These included beach nourishment, wave wall construction in conjunction with the proposed Penarth Headland Link and groyne removal adjacent to the Barrage.

CHAPTER 1: INTRODUCTION

1.1 General

The Penarth Coastline (Ordnance Survey Ref: ST189712), location, as shown in Figure 1.1, is approximately 1.5 km of Triassic Marl and Rhaetic Limestone outcropping in the Bristol Channel.

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Figure 1.1: Plan of Penarth coastline. (Source: Ordnance Survey)

The principal interests are recreational and geological, although 93 Ha of the southern part of the beach is designated as a Site of Special Scientific Interest (SSSI). The beach consists predominantly of limestone cobbles and cliff debris overlying marl bedrock. Anthropogenic structures include a pier (*circa* 1894), three slipways, nine groynes, three sea outfalls, a multi-storey car park (demolished autumn 2001), sea defence walls and the remains of an old quay. The beach runs from its southernmost point, approximately NNE for a distance of 1,150 m, where

it rounds Penarth Head to run approximately NNW for a further 350 m, to the site of the Cardiff Bay Barrage (Figure 1.1). Here the breakwater and Barrage join the headland. The Bristol Channel at Avonmouth, has the second highest tidal range in the world (16·4m) and at Cardiff can reach in excess of 13·0 m, although climatic variations can greatly influence the time and height of the tide. Generally, the spring tidal range is accepted as 11·1 m (SBCEG, 1999; Page and Oakley, 2002). The coastal processes have been significantly influenced by human intervention, notably by shoreline reclamation for residential and port development and the construction of flood and erosion defences. According to SBCEG (1999), drift is weak wave induced in a northerly direction although other influences may be affecting the shoreline.

Recent changes along the main length of Penarth Beach had been characterized by loss of beach material and exposure of the marl bedrock. As reported by SBCEG (1999), beach levels dropped to a critical level in 1997/98 and sea defences needed strengthening following undermining of their foundations. There had also been an increased incidence of flooding along the main promenade and it appeared that major changes occurred between 1995 and 1997. This coincided with the completion of two key stages in the construction of the Cardiff Bay Barrage. Firstly, towards the end of 1995, the 600 m long flank embankment was constructed and secondly, in January 1996, a cofferdam, approximately 200 m by 150 m was completed for the construction of the sluices, lock gates, fish pass and associated civil engineering works. Consequently, the Barrage became linked with beach erosion. In addition, other anthropogenic activities, such as offshore marine aggregate dredging, were also considered as possible causes of the beach erosion (Penarth Times, 1998).

The foreshore and specific problems that existed at the start of this research in September 1997 included the following.

1.2 Penarth Foreshore (1997)

1.2.1 South of Lifeboat Slipway

Plate 1.1 characterises loss of beach material along this section.



Plate 1.1: Exposed foundations Yacht Club Slipway.

To the north, marl bedrock is clearly visible, together with exposed foundations and underslab construction of the Yacht Club slipway. This qualitative evidence demonstrates loss of beach cover and there is a 300 mm clearance, from the underside of the foundation, to the beach surface. The new lifeboat slipway, seen behind the first slipway, was completed in 1995.

1.2.2 Lifeboat Slipway to Penarth Pier

A comparison of Plate 1.2 (Bullen 1993a), showing the beach condition in September, 1992 and Plate 1.3 from September, 1997, clearly shows a loss of beach material along this section.

Plate 1.2 has been removed from the digitized thesis for copyright reasons.





Plate 1.3: Penarth beach – September 1997.

Furthermore, it can be seen that there has been a change in the beach composition. This loss of beach material is critical to sea defence stability and management, especially along the promenade, as the seawall is built on marl bedrock. As the effective depth of the sea increases, the likelihood of flooding and structural damage to the Esplanade also increases. The owner of Chandlers Restaurant, situated on the Esplanade, supplied further anecdotal qualitative evidence of change. At the time of an interview in September 1997, the restaurant had been flooded twice in the previous eighteen months compared with twice in the previous twenty years (Rabaiotti, 1997: pers. comm.). Plate 1.4 shows the southern side of the pier construction where it meets the seawall. The clean area at the junction of the

beach and structures indicates a fall in beach level and that this was a relatively recent occurrence.



Plate 1.4: Beach adjacent to Penarth Pier

1.2.3 Penarth Pier to Penarth Head

The multi-storey car park and seawall north of the pier, as shown in Plates 1.5 and 1.6, were in a poor state of repair and in need of urgent attention. Plate 1.7 of the slipway adjacent to the car park shows similar conditions to those seen in Plate 1.1. In both cases, the beach on the southern side had eroded to such an extent that the foundations were exposed. Further north, a comparison of Plate 1.8 (Bullen 1993a), and Plate 1.9, clearly indicates the undermining and loss of beach material adjacent to an old quay.



Plate 1.5: Sea defences below multi-storey car park.

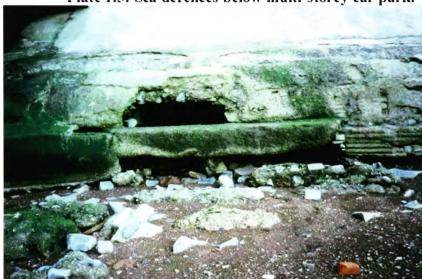


Plate 1.6: Sea defences north of multi-storey car park.



Plate 1.7: Slipway below multi-storey car park.

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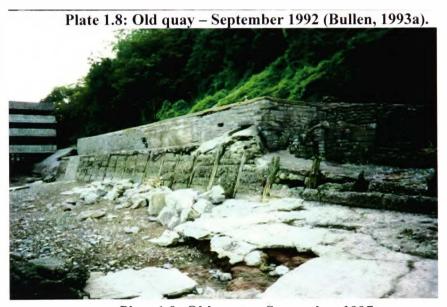


Plate 1.9: Old quay - September 1997.

In July, 1997, the contract to construct a new 1 m diameter sea outfall was completed. This outfall, inside a concrete culvert, replaced twin 600 mm diameter cast iron pipes. The new outfall, had higher beach levels on the northern flank and, at one location, the level difference was 750 mm. According to the Resident Engineer responsible for the contract, on completion, beach levels were left approximately the same on both sides of the outfall (Bright, 1997: pers. comm.).

1.2.4 Penarth Head to the Cardiff Bay Barrage

The cliffs along this stretch are generally unprotected, except for a small length of masonry wall, constructed for an old sewer. This is located where the beach changes orientation from NNE to NNW. The cliffs were in a constant state of erosion (0·2 m per annum: Williams and Davies, 1987; 1990), with frequent losses of material from the cliff face, similar to the situation described by Wallingford (1992) and Bullen (1993a). A manhole cover and slab for an old sewer outfall was approximately 750 mm above the beach in all directions and, as well as indicating erosion, it inferred that there was no net sediment transport in either a northerly or southerly direction.

Continuing NNW and approaching the site of the Cardiff Bay Barrage there are five groynes. The four nearest the barrage are normal to the cliff face and form three bays, whilst only part of the fifth groyne is at 90°. These last four groynes successfully retained beach material with their southern flanks retaining higher levels. Clear of the groyne field, mud covered marl was predominant. The beach along this section was mainly composed of material that had eroded from the cliffs. Plate 1.10 shows the typical beach composition, the cofferdam described in Section 1.1 and the partially completed permeable screen breakwater. In addition, the then site boundary for the barrage works is seen positioned along the northern flank of the final groyne (9).



Plate 1.10: Barrage construction adjacent to groyned beach.



Plate 1.11: Steps northern flank groyne 8.

Average beach levels decreased progressively as the barrage site was approached. Consequently the sea frequently reached the base of the cliffs nearest the barrage site, evidenced by a lack of eroded deposits at the cliff base, coupled with increased cliff erosion. Plate 1.11 shows the steps on the northern face of the penultimate groyne (groyne 8). This clearly indicates undermining of the foundations, also apparent at other groyne locations. Once again, erosion was evident along this coastline. It is a reasonable assumption that, as neither Wallingford (1992) nor Bullen (1993a) indicated beach loss extent, there has been relatively recent changes in the nearshore zone.

1.3 Rationale

It is an accepted axiom that coastal management comprises the integration of a cultural superstructure superimposed on a physical fundament (Sauer, 1963). To this end, detailed analysis of the underlying physical processes in this locality has been undertaken, as effective management depends upon concise, robust, quantitative facts. From this base suggested plans have been formulated for management of the Penarth foreshore. Since 1992 there has undoubtedly been a loss of beach material along the whole of the Penarth coastline. By the utilization of reports, plans, photographs, research papers and publications, discussions with representatives from coastal authorities, companies, developers, agencies and stakeholders, baseline data was established from which past changes could be qualitatively and where possible, quantitatively determined. Establishment of monitoring stations along the beach, enabled future changes and trends to be quantified and evaluated, together with possible causes and effects.

Consequently, the main aims of this study, as set out in 1997, were to:

- determine the cause of the unprecedented erosion along the Penarth foreshore;
- evaluate the consequences;
- develop beach management strategies to protect the coastal environment.

To achieve these aims, in September 1997, a 0.75 km baseline (26 stations at 30m intervals) was established along the NNE foreshore of Penarth beach (Figure 1.2) together with, a 0.72 Ha area of beach (240m longshore by 30m cross-shore rectangular grid), directly in front of the main promenade. In addition, in September 1998, cross-shore profiles were established at two locations to monitor the beach to the depth of closure. These would enable past and future qualitative and quantitative changes in longshore profile, cross-shore profile and sediment transport to be determined.

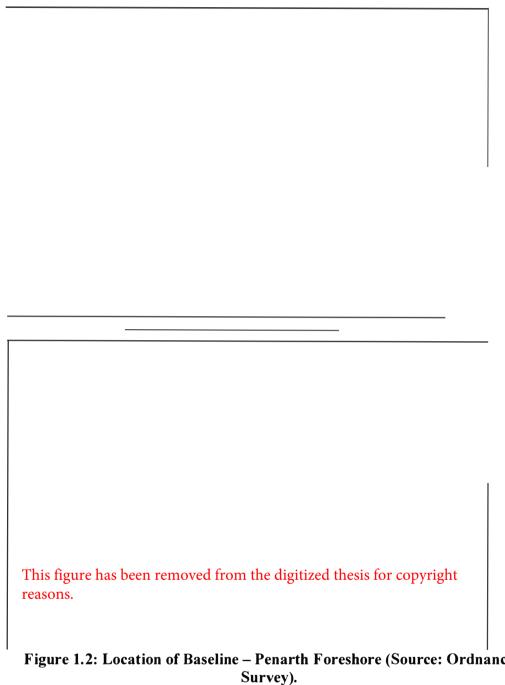


Figure 1.2: Location of Baseline – Penarth Foreshore (Source: Ordnance

Similarly, a 160 m length of coastline, immediately adjacent to the site of the Cardiff Bay Barrage, running SSE towards Penarth Head was established for data collection (Figure 1.3). This length included four groynes (6, 7, 8 and 9) which extended from the cliff face and formed three bays of total area 0.72 Ha. The most northerly groyne (9) was at the approximate location where the breakwater for the Barrage joined the coastline. Consequently, on completion, it was incorporated into the breakwater and appropriate site adjustments were made to the monitoring process. This data would enable past and future qualitative and quantitative changes in beach evolution, pre and post barrage completion, to be determined.

Figure 1.3 has been removed from this digitized thesis for copyright reasons.

To complete the monitoring of the foreshore, a link survey was established between the end of the longshore profile (station 26; 750m longshore) and the start of the groyned beach (mid point groyne 6, southern flank). These results enabled assessment of changes in beach response with orientation. Surveys were undertaken each September and April between September 1997 and September 2002 to assess changes over winter and summer periods.

Evaluation of survey results identified changes in coastal processes and led to the development of temporal and spatial models. These models represented functions of shoreline indicators, Mean High Water (MHW) and beach level, which were subsequently assessed in conjunction with water level and wind data (forcing agents), as well as barrage construction and dredging.

From consideration of Function Analysis, risk assessment and proposed development, the beach models developed in this study were used to produce management strategies for monitoring the foreshore. In conjunction with predicted trends and change, management responses were subsequently proposed for the protection of Penarth's coastal environment. Interestingly, the need to review local management options across the frontage and develop a coastal defence strategy for Penarth was identified by SBCEG (1999), as necessary work to be carried out in the next five years. Therefore, the Shoreline Management Report (SBCEG, 1999) subsequently validated the aims and objectives of this research.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

John and Williams (1980) clearly identified the historical importance of The wide expanses of red Trias clay in the Cardiff district provided raw material for a number of brickworks, including the one established in Penarth in 1854. Penarth Docks opened in 1865 and in 1881 was extended into the hillside, to meet the demands of the burgeoning coal trade. In 1888 the South Wales Portland Cement Company established a works at Penarth to increase the scale of cement manufacture (previously limited) and to exploit the growing domestic building boom. In Glamorgan, there was little ship building activity after the middle of the nineteenth century (John and Williams, 1980). However, the Penarth Ship Building and Ship Repairing Company constructed some iron screw coasters between 1882 and 1890, although it was ship repair that gave continuity of employment, coupled with the minimum of commercial risk. As well as having industrial importance, Penarth was also a desirable seaside resort (Robinson and Millward, 1983; Carradice, 1994). Penarth Station opened in 1878, an elegant park opened in 1880, the town baths, which had two swimming pools, opened in 1885 and the Esplanade Hotel opened in 1887. The major achievement, the Esplanade itself, was built in 1883-4. The construction of Penarth pier commenced in April 1894 as the last item in Lord Windsor's plan to make Penarth an upper-class resort. It was to complement the Esplanade, and almost from its opening in April 1895, was an enormous success (Carradice, 1994). Penarth quickly became a "desirable suburb" of Cardiff, a role that has been reinforced in recent years (Robinson and Millward, 1983; Carradice, 1994).

2.2 Coastal Environment

2.2.1 Beach Evolution

Beaches can be categorised into four principal beach types: shingle; shingle upper/sand lower; shingle/sand mixed and sand (Simm, 1996). However, cobble size at Penarth, depending on location, includes pebbles in the range 100 mm – 500 mm (Bullen, 1993a; Wallingford, 1992). As the definition of the shingle category is 100% passing the 100 mm sieve (Simm, 1996), the cobble beach at Penarth could partially fall into the classification of a rock beach. Therefore, its response to waves falls between that of an armoured mound or slope and that of a shingle beach.

The solid geology of the coastal area and the nearshore seabed, affects beach development. This may be the result of direct effects, for example, by blocking sediment transport pathways, or indirectly, perhaps by modifying the wave or tidal current patterns in the nearshore zone (Simm 1996). Whilst longshore transport is the primary mechanism for changes in beach plan shape, cross-shore transport is the means by which the beach profile changes (Simm, 1996). The response time of beaches to variation in cross-shore transport can be as small as one tidal cycle (during storms) or as large as six months (seasonal variation). Cocco *et al*, (2001) agreed that cross-shore transport, although often negligible in the long term, influences the beach profile, especially after intense storms. Two profiles have been distinguished. The winter profile is commonly associated with offshore transport, whilst the summer profile, formed in calmer weather, is distinguished by the presence of a berm at the wave swash limit.

Beach evolution is due to the combined effect of waves and the tide. Trim *et al* (2002) found the presence of the tide affected beach behaviour differently depending on the wave climate. Under swell conditions on the flood, sediment was eroded once a critical depth had been reached but was

moved onshore in the uprush to form the berm. The tide was shown to increase the size of the berm in a summer profile and push the offshore berm further seaward in the winter profiles. The overall beach slope did not change when subjected to wave attack without the tide but altered significantly under the combined action of the wave and tide (Strahler, 1964; Williams, 1973; Trim *et al*, 2002). The work by Trim *et al* (2002) involved a maximum sediment size of 50mm, and was carried out on two beach slopes (1/7 and 1/9) which were similar to those found along the Penarth foreshore (Bullen, 1993a; Phillips, 1999a and b).

On a longer time scale, beach losses can occur as a result of offshore sediment transport by wind-induced currents. Such losses are not easily quantifiable due to a poor understanding of the physical processes involved and the rapid changes in these forcing parameters with time (Simm, 1996). Anfuso *et al* (2000) showed that different values of disturbance depth have been recorded in beaches exhibiting a broadly similar wave height and that it depends on a combination of factors including prevailing weather, climatic conditions and coastal infrastructure. As beaches are constantly changing in response to the various processes and factors that control them, successful management requires that these processes should be identified and changes should be monitored (Simm, 1996).

2.2.2 Depth of Closure

The depth at which significant sediment motion is absent, is often called the depth of closure, and is a rather vague concept in an oceanic wave environment (Leatherman, 1991). This is confounded when an attempt is made to quantify a relatively confined zone of erosion (e.g. the narrow beach/dune zone) with a broad zone (the shoreface/inner shelf), over which eroding sediment can be thinly spread. Depth of closure is time dependent; the longer the time period being considered the larger the depth of closure (Stive *et al.*,

1992). It can be estimated using a range of techniques including: grain size trends, orientation of offshore contours and wave statistics (Hallermeier, 1981).

For measuring volume change, the seaward limit poses the greatest problem and it is necessary to use the concept of depth of closure (Simm, 1996). It can be defined as the ratio of the change in cross-sectional area divided by the advance or retreat of the high water line (or other convenient contours); and determined from an analysis of beach profiles (providing they continue far enough underwater). Cocco et al (2001), showed a case study where both coastline variation and the volume loss/gain were calculated. The ratio between the two should have yielded the closure depth or some related parameter but, in fact, this was far from reality. Capobianco et al (1997) reviewed the related problems of closure depth and highlighted the difficulties of reconciling theory and practice with evidence. Often, however, insufficient data is available for analysis, and the closure depth has to be estimated by experience using information on beach profiles at various locations along a coast (Simm, 1996). This agrees with the observations of Leatherman (2001) who recognised that in practice, active profile width must be estimated for most developing countries, using expert judgement rather than hard data

It is not unusual to find that a gently sloping shore platform occupies much of the near shore zone, and sometimes the lower part of the inter-tidal foreshore as well (Simm, 1996). These platforms adopt a much shallower slope than mobile sand or gravel under wave action, so that the underwater profile of the beach usually terminates at a more or less distinct line on the platform. In the case of Penarth, the shore platform is generally mud-covered marl and from Plate 2.1, the distinct termination of the limestone cobbles is clearly seen. Therefore, it is argued that for Penarth beach, this represents the depth of closure. Normally, the water depth at the depth of closure is in the range 4 to 6m but the actual water depth will be determined in Chapter 6.



Plate 2.1: Position of depth of closure, Penarth beach.

2.2.3 Wave Climate

"The interactions between waves, currents and coastal structures are complex and not well defined in theory. Local beach response is therefore difficult to predict, even with the use of numerical or mobile bed physical models."

(Simm, 1996: 176)

According to SBCEG (1999), there has been a significant increase in mean wave height since around 1965, increasing at about 2% per year. However, this data is based on the swell component and takes little account of storm-generated waves. No direct data is available to indicate any changes in mean wave height in the Bristol Channel. The effects of surge tides vary greatly depending on the height and direction of waves near the time of high water. Whilst evidence of an increase in wave height offshore appears irrefutable, wind generated waves will, in ocean conditions, be more likely to be duration, rather than fetch limited. (SBCEG, 1999). It should be noted however that wind records on land, may easily show a 10 – 20% reduction in wind speeds due to

increased surface roughness compared to values measured over water (Simm, 1996).

Cocco *et al* (2001), argued it can safely be assumed that the present knowledge on extreme wave evaluation is adequate to the needs of coastal protection. Conversely, they argued that the present understanding of the runup mechanisms including the transformation occurring in waves as they run up the shore, is still not satisfactory, at least as far as complex coastal geometries are concerned. Swash velocities of 1·3 m/s have been recorded on two Californian sand beaches (Schiffman, 1965), whilst Kirk (1970) recorded 1·9 m/s on mixed sand/gravel New Zealand beaches. Williams and Phillips (2000) recorded 4 m/s on the cobble beach at Nash Point, approximately 20 km west of the study area. However, the aspect of Nash Point is such that it is open to the prevailing southwesterly storms (Wallingford, 1992; Bullen 1993b; SBCEG, 1999), whilst Penarth is sheltered from this direction.

2.2.4 Anthropogenic Structures

One of the major impacts on the behaviour of beaches is the long-term cumulative effect of constructing seawalls and groynes, especially those that have been in place for 100 years and more. The construction of seawalls can have a variety of effects on the coastline and adjacent beaches. Walls built in front of cliff-bases not only protect the land at the cliff-tops and reduce sudden cliff-falls, but also prevent the eroding cliff from contributing fresh material to the beach, with the resulting sediment starvation along the downdrift coast (Simm, 1996; Wiegel, 2002). Such a wall has been built at Penarth (Williams and Davies, 1987; 1990).

Four different types of scour/reflection behaviour have been identified for vertical seawalls (Powell and Lowe, 1994). The occurrence of a particular type depends on the local wave conditions and water depth. Their qualitative

model, shown in Figure 2.1, produces four different responses for wave/seawall interaction on coarse sediment beach profiles. These are as follows:

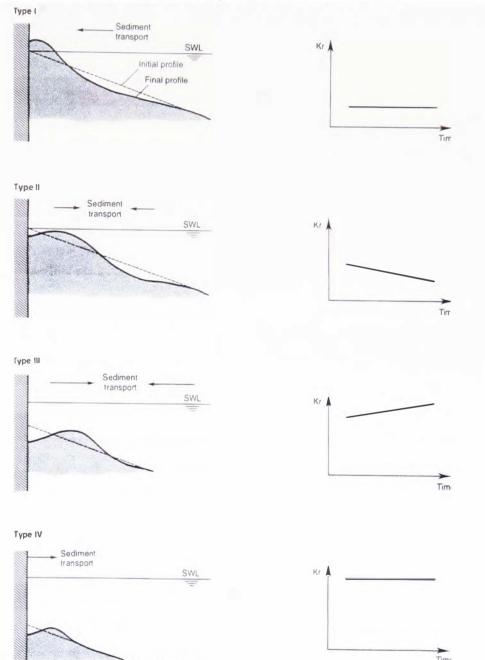


Figure 2.1: Scour/reflection behaviour (Source: Powell and Lowe, 1994)

1. Type I – initial beach level is high relative to the wave height, reflected energy is low, high onshore movement, low offshore movement, reflection coefficient, C, low and constant through time.

The beach responds in much the same way as a natural beach. The seawall plays little part in the beach processes as it is not under direct wave attack.

2. Type II – combination of beach levels and wave heights is such that there is marginal scour, reflected energy is low, predominant onshore transport, some offshore transport, and C decreases through time.

Initially there is toe scour. Over time, the beach can build up due to the predominant onshore movement of material. As the beach level increases so the amount of reflected energy, and hence the proportion of onshore to offshore transport, decreases.

3. Type III – initial beach level is low, relative to wave height, reflected energy is high, low onshore movement, high offshore movement and *C*, is increased through time.

The toe depth increases with time and there is a general lowering of beach levels. As the beach level decreases, so the amount of reflected energy, and hence the proportion of onshore-to-offshore transport increases. Given sufficient time, beach levels can reduce to the Type IV condition.

4 Type IV – initial beach level is very low, relative to the wave height, reflected energy is high, predominantly offshore movement, and *C* is large and constant through time.

Scour is small compared to water depth. The movement of material is low in both onshore and offshore directions due to the large depth of water, relative to wave height. Eventually a dynamically stable situation

is attained from which, regardless of the incident wave conditions, the beach cannot recover its former levels.

Generally, response Types I and II are associated with beach accretion, while Types III and IV correspond to scour. Constructive beach building (Type I and II) occurs at a similar rate to Type III scour with about half of the profile development occurring during the first 100 waves. Type IV scour, being associated with greater depths of water at the wall, occurs at a slower initial rate though for all response types a quasi-equilibrium position is reached quite quickly, that is, within about 3000 waves.

In the short term, wave reflection from seawalls (Plate 2.2) can affect the level of the fronting beach, and causes greater wave attack on the wall itself. The question of whether this leads to a continuing long-term lowering of beach levels is more complex, one not yet fully answered (Simm, 1996). In many situations, for example, beach levels seem to have almost stabilised in front of seawalls, and although the wave reflections are very strong, the beach has apparently adjusted to the new environment.

Groynes are very common on UK beaches (CIRIA, 1990), but are subject to some controversy as to their effectiveness (Fleming, 1990). As a general guideline, groyne systems are appropriate to frontages where the existing shoreline must be maintained and where there is a low net, but high gross drift (Simm, 1996). Groynes on shingle beaches have been more successful than those on sand beaches, and vertical sided groynes have been shown to be efficient only on shorelines that are exposed to moderate or low-energy wave climates (Hs<2m).



Plate 2.2: Wave reflection from Penarth seawall.



Plate 2.3: Vertical side concrete groyne, Penarth foreshore.

Examples can be quoted of existing groyne systems that successfully contradict these guidelines, but often these are on sites, which would have relatively stable beaches with no control structures at all. Therefore, the groynes only serve to reduce short-term variations in beach response (Simm, 1996). However, the Penarth wave climate (Hs = $2 \cdot 1$ m for a 1 : 100 year return

(Wallingford, 1990; SBCEG, 1999)) and beach composition would indicate that vertically sided concrete groynes (Plate 2.3) are appropriate at this location.

Construction of groyne systems can and does tie up beach material resources, especially shingle in one area whilst starving the downdrift beaches of their supply of sediment. The long-term effect of groyning long stretches of coast can also severely reduce the rate of longshore drift (Simm, 1996; Cipriani *et al*, 1999; Wiegel, 2002). Breakwaters similarly contribute to erosion as indicated by Squire (2003) who reported that cliff erosion at Sidmouth had increased, following the construction of two offshore breakwaters. A further impact of groyne construction is the creation of both seaward-flowing rip currents and circulation cells both of which have a length scale similar to that of the groyne spacing. Such currents can lead to offshore losses of fine-grained beach sediment, as well as diminishing the efficiency of the groynes themselves (Simm, 1996).

2.3 Climate Change

It is generally accepted that on the longest timescale of Earth's geological history, trends in solar output have played a major role in shaping the climate (Kelly, 2000; Croner, 2001). Indeed, Lassen and Christensen (1995) reported that they had examined all climate changes in the last five hundred years and found that they coincided precisely with changes in solar activity. Climate fluctuations are normal and have often been dramatic, switching between temperate eras and ice ages lasting thousands of years. Against this background of natural variation, the current debates on global warming centre on predicted changes and human contribution to the process. The 1995 publication of the second climate assessment by the Intergovernmental Panel on Climate Change (IPCC) was the first 1PCC publication to state unequivocally that human activities are having a discernable impact on the Earth's climate (Nature, 2002). However, there are complaints that uncertainties in the science had been played down

and that there is a common reluctance to use probabilities in climate change research. According to Pearce (2000), the natural greenhouse gases in the atmosphere are being boosted by human activities whereas; Christy (2001) argues that even with all our cars, factories and cities, human impact on the weather is too small to measure.

There is no easy way of predicting what future carbon dioxide levels will do to the atmosphere but nonetheless, the speed at which the carbon dioxide concentration is changing, appears to be unparalleled in human history (Croner, 2001). However, low levels of carbon dioxide are associated with cold periods. When the ocean is cold, it can absorb more of the gas and conversely, during warmer periods, carbon dioxide is released and the atmospheric concentration rises. This has led to the argument that the warmer temperatures are the cause of the higher carbon dioxide levels rather than vice versa (Croner, 2001). There are considerable uncertainties surrounding the possible scale, timing and extent of climate change (Gurney, 1999), but there is mounting evidence that the climate is already changing (Environmental Scientist, 1999). Records show that 1997 was the third warmest year on record in the UK (1.1° C above the average between 1961 and 1990). Such a year would be expected once every 17 years but according to the UK Climate Impacts Programme scenarios, by the 2080's such years would occur with increasing frequency. Under a low scenario they would occur every two years and for the other scenarios nearly every year. Thus exceptional conditions are likely to become normal; a prediction possibly supported by the reporting of Matthews (2003) that, on a global scale, 2002 was the second-hottest year ever recorded.

Conversely, according to Matthews (2000), climate change was unsubstantiated and there was evidence contradicting the supposed link between global warming and the floods in the UK during the autumn of that year. He further argued that Britain's weather is naturally capable of much greater extremes than previously thought. This agrees with Christy (2001) who stated that evidence shows a climate of natural variability and that these variations have always occurred, even when humans could not have had any impact. He further argued that hurricanes and tornadoes are not

increasing and that storms and droughts do not show any pattern of increasing or decreasing. It should be remembered that following general global cooling between 1940 and 1975, there were grim prophecies of a return of the glaciers (Berry, 1996) and many journals at that time contained articles predicting a new ice age. It seems that within a generation, environmentalists and climatologists have turned their attention from global cooling to global warming.

Currently the prediction of events involves the use of computer models, which simulate natural processes. Christy (2001) commented that he views computer models with a degree of awe and sense of humour and argues that as no computer model accurately portrays our current weather then how can they be trusted with the future? Matthews (2003) clearly shows the uncertainty in generating credible model-based projections of climate change whilst Nature (2002) highlighted that models simulating long-term climate changes have no comparable reality check. Berry (1996) viewed studies by computer models as often oversimplified and further argued that, as it is much easier to run a computer model than use studies involving rigorous observations of the real world, there is a great danger of bias in methods of study.

2.4 Sea Level Rise

"Changing sea levels have been the driver of shorelines over geologic time."

(Leatherman, 2001: 181)

Sea Level is one of the principal determinants of shoreline position. There is a causal relationship among several factors: sea level, sediment supply, wave energy and shoreline position. Sea level rise induces beach erosion or accelerates ongoing shore retreat for several reasons (Leatherman, 1991). First, higher water level enables waves to break closer to shore. Second, deeper water decreases wave refraction and this increases the capacity for longshore transport. Finally, with higher water level, wave and current erosion processes act further up the beach profile causing a readjustment of that profile. Maintenance of an equilibrium beach/nearshore profile in response to sea

level rise requires an upward and landward displacement of the beach in time and space; this translates to erosion in ordinary terms.

"What makes rising water level so important to humanity is its potential to alter ecosystems and habitability in coastal regions."

(Douglas, 2001a: 1)

Despite uncertainties in the climate change debate, there appears to be an international consensus amongst the coastal and earth science communities regarding the predicted increase of storm activity and sea level rise (Douglas, 2001a). Jensen *et al.* (2001) showed that since about 1960, there had been an increase in frequency and duration of storm floods along the German North Sea coastline; Rivis *et al.* (2002) reported significant increases in storminess between 1948 and 1998, along the coast of Estonia whilst, Vilibic *et al.* (2000), predicted that over the next century there would be an increase in storm surges in the Adriatic. In addition, since at least the 1950's, monitoring has shown an increased frequency of abnormally high tides in the eastern Irish Sea (SBCEG, 1999). Storm surges and tides contribute significantly to beach erosion and lead to considerable damage of coastal infrastructure. Such events will be exacerbated with current predictions of sea level rise.

Crawford and Thomson (1999) reported estimates of sea level rise of between 3 to 9 mm/year for the next 80 years; Douglas (2001a), quoted average rates of 2 to 3 mm/year in the United States with up to 6mm/year or more in some specific areas; Vilibic *et al.* (2000) stated that there had been a mean rise of 180mm over the last century and that it was expected to be about 500mm in the next century; whilst Warrick *et al* (1996), argued that there had been an overall global rate of sea level rise over the last 100 years of nearly 2 mm/year, sharply higher than the average rate during the last several millennia. Weihaupt and Stuart (2000) logically deduced from implications of ancient maps, that the mass reduction of the ancient Antarctic ice sheet could lead to the equivalent of a 4m rise in sea level and although the time-scale for this change is not given, it would have severe implications for the low-lying coastal environments of the world. Therefore existing developments, constructed at or near sea

level, will be at significant risk and activities under threat, will include conservation, industry, real estate and the economy. Consequently, sea level rise coupled with storm surges and high tides will pose severe problems for beach managers and coastal engineers.

"While many factors contribute to shoreline recession, sea level rise is considered the underlying factor accounting for the ubiquitous coastal retreat.

(Leatherman, 2001: 183)

Bruun (1962) long ago proposed that long-termed erosion of sandy beaches was a consequence of sea level rise. His model predicts that beaches will erode by 50 to 200 times the rate of increase of sea level (Douglas, 2001). This relationship of sea level rise to erosion has become known as the Bruun Rule (Leatherman, 2001). It does not suggest that sea level rise actually causes erosion; rather, increased sea level enables high-energy, short-period storm waves to attack further up the beach and transport sand offshore. SBCEG (1999) predicted that the consequences of sea level rise for the Penarth coastline would be increased wave energy impacting existing defences hence causing disruption to infrastructure, property and land. According to Simm (1996) under these circumstances, monitoring should be undertaken.

2.5 Monitoring

Monitoring should be undertaken on any coastline where there is a potential risk of flooding or erosion, which threatens significant property or infrastructure. A rigorous visual site examination should be undertaken that is best performed on foot. It is essential to carry a site plan that is marked up with observations as reconnaissance progresses. Particular attention should be paid to omissions or alternations to the features described on maps or plans, whether they are topographical, geological or manmade. Photographs should be taken and detailed observations made of the nature and characteristics of the beach exposed strata, foreshore or coastal cliff and a preliminary judgement made on the susceptibility of the foreshore strata to erosion

(Simm, 1996). Cocco *et al* (2001) argued that assessment of long-term damage is mainly linked to the possibility of forecasting erosion and that this objective is far from being accomplished, even when the most advanced models and survey technologies are employed. A good qualitative understanding such as can be supplied by geological insight and frequent monitoring activities are still the best available tools.

If insufficient information exists within the study area to facilitate a baseline assessment, original survey work may be required (Simm, 1996). This survey work should take place over a number of years to enable an assessment to be made of any natural fluctuations; to enable the comprehensive prediction of impacts and the assessment of their significances as well as a baseline against which to measure any change.

Beach profiles are necessary for measurement purposes to establish the volume of material on the beach (Simm, 1996) and comprise surveyed section lines either perpendicular or parallel to the shoreline. They can be used to quantitatively establish beach response to storm events, beach recovery rates, long-term volume changes, areas of potential flood or erosion risk and the potential envelope of cross-shore elevations (Simm, 1996). Straightforward extrapolation of the past is of course a possibility, and also a reasonable one; as long as enough coastline measurements of the past are available and no relevant changes of the physical environment have been caused by human action or by extraordinary natural events (Cocco *et al*, 2001).

2.6 Erosion

Schroeder (2000), argued that although storms play a major role in shaping coastal environments, erosion does not generally result in detrimental impacts to unmodified areas and can often provide a net benefit. However, erosion and loss of the existing coastline due to climate change could have major economic consequences, especially as coastal land, often has an inflated value based on location rather than on size (Phillips, 2000). For example, Leatherman (2001) quoted several hundred million

dollars per kilometre for beachfront property in highly urbanised areas in the United States. The Environmental Scientist (2000) reported that strengthening existing coastal and river flood defences could cost £1.2 billion over the next half century in England and Wales, with buildings and infrastructure identified as areas most likely to be affected. Granja and Carvalho (2000), highlighted problems of erosion on the northwest coast of Portugal; Gillie (1999) illustrated the consequences of erosion on the Victoria waterfront, British Columbia and SES (1997) identified erosion as a major issue for the sustainable management of the Severn Estuary. It will have a significant impact on tourism, which according to the World Tourism Organisation (2001) is the worlds largest growth industry. Houston (2002) reported that travel and tourism is the United States largest industry, employer, and earner of foreign exchange and that beaches were the major factor in tourism. He further identified beach erosion as the number one concern of Americans who visit beaches. With 33,000km of eroding shoreline and 4,300km of critically eroding shoreline, the US Army Corps of Engineers (1994) considered it a serious threat to tourism and therefore a major threat to the national economy.

If as stated by Povh (2000), three quarters of the world's population are going to be living within 60 km of the shoreline by 2020, then the consequences of erosion will be even more significant. Population increases in the coastal zone are allometric whilst coastlines are often viewed as stable, permanent assets. In reality, these are dynamic, and respond to natural processes and human activities. The six major spheres of human activity in the coastal zone that were identified by Ketchum (1972): residency and recreation; industrial and commercial; waste disposal; agricultural, aqua cultural and fishing; conservation; military and strategic, still hold true. Indeed all these interests have been represented at Penarth at some time or another (John and Williams, 1980; Carradice, 1994; SBCEG, 1999). These activities are often in conflict with one another as well as with the long-term natural processes. Developments such as ports have demonstrated the tension between cultural and physical environments (Bullen, 1993b; Cipriani *et al.*, 1999) and as argued by Sauer (1963), integration of these factors is essential for effective management. According to Bullen (1993b), the

Institute of Oceanographic Sciences concluded that human intervention, including port developments and seawall construction, had been the main erosion mechanism along the beaches at and near to Port Talbot. Between 1975 and 1991 average beach levels fell between 0.7m at Margam, Port Talbot and 0.43m at Kenfig, approximately 2km further east. In addition, dune recession at the back of the beaches varied between 40m and 20m respectively. Following port developments and associated groynes and breakwaters at Marina di Massa, Tuscany, Cipriani et al. (1999) reported similar findings on the downdrift beaches. Other areas of the UK are experiencing severe erosion. Morgan (2003) reported losses of up to 30 m during September 2002 at Happisburgh, Norfolk; Squire (2003) highlighted the serious problems at Sidmouth, Devon, where erosion rates of 1.7 m per annum are threatening a designated World Heritage Site; and Jones (2003) stated that the beautiful beaches of Wales, including Tenby, are vanishing. SBCEG (1999) reported that beach levels at Penarth, dropped to a critical level during 1997/98, causing the undermining of a section of the large vertical promenade. This agreed with Phillips (1999a) who documented a fall in beach level resulting in an average loss of beach material of 50 kg/m² between September 1997 and April 1998. However, following a survey in September 1998, beach levels had risen with an average gain of beach material of 165 kg/m². This example demonstrates the difficulties faced by coastal managers when deciding on an appropriate response to erosion and this is further complicated because some methods of coastal defence, for example seawalls, themselves cause erosion.

The protection of coastal areas and the planning of their development need a continuous and reliable assessment of the risks deriving from waves and coastal erosion (Cocco *et al*, 2001). A global classification of the erosion risk can be supplied by a clear understanding of the lithological, geological and morphological elements which, given the complexity of the problems, is often of much greater value than any unsophisticated model unsupported by adequate data (Cocco *et al*, 2001).

According to Leatherman (2001), there are three general responses to shore erosion: (1) retreat from the shore (2) armour the coast or (3) nourish the beach. The

choice of response strategy will depend on a number of factors including socioeconomic and environmental conditions. He further argued that the retreat option is preferred for undeveloped or sparsely developed areas whilst for highly urbanised areas; the abandonment option is not politically realistic or economically viable. Therefore, despite damage to Penarth's coastal defences (Section 1.2), the urbanised nature of the promenade would make managed retreat unacceptable.

A further review of pertinent cultural literature, associated with beach management, is given in Chapter 4.

CHAPTER 3: PHYSICAL BACKGROUND

3.1 Introduction

The geology and geography provide the fundament on which the natural and human environments evolve and consequently, management must involve the integration of the physical and cultural environments (Sauer, 1963). From this position, Williams *et al*, (2002) developed the following theoretical structure for understanding the development of landscapes:

$$Ln = \sum f (G, V, C, U \text{ or } X)$$

where the present landscape (Ln) is the result of summation of the geognostic (G), vegetation (V), climatic (C) and unexplained or not understood (U or X) factors, from time zero (t₀) to time present (t_p). This implies that any change in the natural environment, such as climate change, can alter the coastal landscape. Similarly, human activities can have an impact on the natural environment and modify the coastline. For example, many authors, including Bullen (1993b) and Ciprianni *et al* (1999), have shown that beach erosion can be a consequence of port, seawall and groyne construction. As Penarth Beach has evolved from natural and anthropogenic influences, management strategies will need to integrate both the physical (natural) and cultural (human) environments. Consequently, this chapter is considered from physical and cultural environment perspectives.

3.2 Physical Environment

3.2.1 Geology

South Wales is mainly composed of Palaeozoic rocks with some Mesozoic (Bullen, 1993b; SBCEG, 1999). From Ogmore-by-Sea to Penarth there is an unconformity from the Upper Carboniferous until the Upper Triassic and Lower Jurassic sediments and these sediments cover the coastline (SBCEG, 1999). The coastal geology between Lavernock Point and Penarth Head (Figure 3.1a) is dominantly Triassic that passes up conformably into the Rhaetic and Lias.

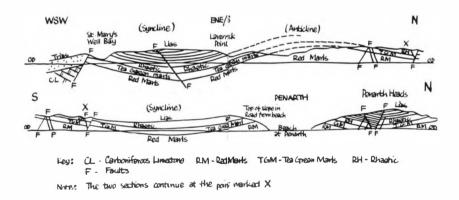


Figure 3.1a: Geological Cross-Section – Lavernock Point to Penarth Head (Source: Owen, 1984; 184 - 185).

The sub-Triassic floor is of folded Carboniferous Limestone which outcrops at Sully and Barry Island. The Mesozoic rocks are also folded into shallow synclines and anticlines, the most obvious being the Lavernock Syncline (Owen, 1984). From Lavernock Point along the beach towards Penarth, the broad downflexure, brings in the greyer Rhaetic and Lias above the red to green Triassic marls. Another syncline occurs in the Penarth Head region, again preserving the Rhaetic and Lias but here there is also considerable faulting. Many small but spectacular faults can be seen along the Penarth – Lavernock section and Owen (1984) argued that it was ideal to demonstrate the tilt of the fault planes, direction of downthrow and combined fault movements which produce faulted troughs. These faults were probably the result of Alpine earth movements which occurred widely

during the middle of the Cenozoic era in the period known as the Miocene. The junction of the Triassic with the Carboniferous Limestone is best seen on Sully Island. From Owen (1984) and SBCEG (1999), Table 3.1 has been compiled to show the full succession in the Penarth-Sully area:

Table 3.1: Geological succession – Penarth / Sully.

Era	Period or System	Age of Base Period (years)	Components	Approximate Thickness(m)
Cenozoic	Quaternary	2,000,000		
	Pliocene	7,000,000		
	Miocene	26,000,000		
	Oligocene	38,000,000		
	Eocene	54,000,000		
	Palaeocene	65,000,000		
Mesozoic	Cretaceous	136,000,000		
	Jurassic	194,000,000	Lias shales (with occasional thin limestones)	
			Blue Lias-shale and limestone alternation	15.250
	Triassic	225,000,000	Watchet Beds - grey marls	2·135
			Langport Beds – pale limestones and shales	0.765
			Cotham Beds – grey to green marls	0.815
			Westbury Beds – black shales with thin sandstones and lime- stones	5.500
			Sully Beds – grey marls	3.965
Palaeozoic	Permian	280,000,000	Tea Green Marls	9.150
			Red Marls passing down into calcareous sandstones	122.000
	Unconformity			
	Carboniferous	345,000,000	Limestone	
	Devonian	395,000,000		
	Silurian	435,000,000		
	Ordovician	500,000,000		
	Cambrian	600,000,000		
Eozoic	Precambrian	Origi n of Earth		

The Rhaetic limestone and shale cliffs at Penarth (Plate 3.1) are constantly eroding (Bullen, 1993a) and recession rates of 0·2 m per year have been recorded (Williams and Davies, 1987; 1990).



Plate 3.1: Cliffs at Penarth (September 1997)

Cliffs north of the pier give a good view of the Keuper marls. These deposits are perhaps best described as fine silts and show thin bands of greenish-grey colour. These green bands increase in frequency upwards and the Red Marls ultimately give way to Tea Green Marls. The colour change is attributed to the state of oxidation of the iron in the sediment due to their accumulation in less arid conditions. It is puzzling to note, as commented upon by Owen (1984), that there are occasional patches and even odd spots of glauconite occurring in the main red marls.

Of particular interest are the thin impersistent horizons of gypsum or alabaster. Many of these masses have fallen on the beach and the salt deposits have always thought to have been deposited in dried-up inland salt lakes in the Triassic deserts. However, according to Owen (1984), it is suggested that very shallow seas were the scene of the salt precipitation and these seas passed into open waters at some considerable distance away, for example, west of the Bristol Channel and into the Irish Sea.

On Penarth beach there are fallen specimens of the overlying Rhaetic formations and particularly noticeable are the slabs of brownish sandstone displaying well defined ripple marks, which also occur on the pale limestone slabs of the Langport Beds. Fossils are common with the black Westbury shales containing the *Lamellibranches: Chlamys* valeniensis and *Rhaetavicula contorta* while *Pleuromya* and *Liostrea* are common in other Rhaetic rocks. Thin biscuit-coloured limestone layers often abound in the small but distinctive *Protocardia*.

Proceeding from Penarth in a southerly direction towards Lavernock, the base of the Rhaetic falls virtually to beach level in the centre of the Lavernock Syncline. The junctions of the grey Sully Beds with the dark Westbury shales is clearly seen nearby. The uppermost marl bed of the Sully Beds is very water worn with 0·150 m deep hollows filled with green marls or black shale. Three bone beds occur in the lowest metre of the Westbury Beds and the middle one can yield fish remains and large reptilian vertebrae (Owen, 1984). At Lavernock Point, the alternating limestones and shales of the Lias appear above the Rhaetic beds.

To complete the geological succession, the latter periods not shown in Table 3.1, The Pleistocene (1.8 million - 10,000 years ago) and The Holocene (the last 10,000 years), will now be considered. It was during the Pleistocene that the most recent episodes of global cooling, or ice ages, took place. Much of the world's temperate zones were alternately covered by glaciers during cool periods and uncovered during the warmer interglacial periods when the glaciers retreated (UCMP, 2004). The Pleistocene also saw the evolution and expansion of our own species, *Homo sapiens*, and by the close of the Pleistocene, humans had spread through most of the world. The Pleistocene was a time during which climates and temperatures shifted dramatically. Today, there is concern about climate change (Section 2.3) and Pleistocene fossils are providing data to understand consequences of future climate change (UCMP, 2004). There were two periods of glaciation during the Pleistocene epoch, the last of which was between 20,000 and 17,000 years ago. Between these two periods, sea level rose in the Bristol Channel area by as much as 10m above its current level (SBCEG, 1999). The glaciation deepened valleys to the west, like the Tawe and Neath

whilst, with retreat, bedrock around Swansea Bay was covered with a stiff pebble-clay till.

Mid-Holocene coasts were radically different in both form and process to those of the early or late Holocene. Relative sea-level (RSL) change and coastal evolution during the period between *circa* 7800 and 4400 yr BP changed rapidly because of a fall in meltwater discharge from the dwindling ice sheets of the northern hemisphere, coupled with a reduced Antarctic contribution (Long, 2001). This period coincided with the formation of sand dunes to the west in Swansea Bay and a series of alluvial fans around the estuaries (SBCEG, 1999). Heyworth and Kidson (1982) evaluated Holocene sea level changes in southwest England and Wales, producing sea level curves as shown in Figure 3.1b.

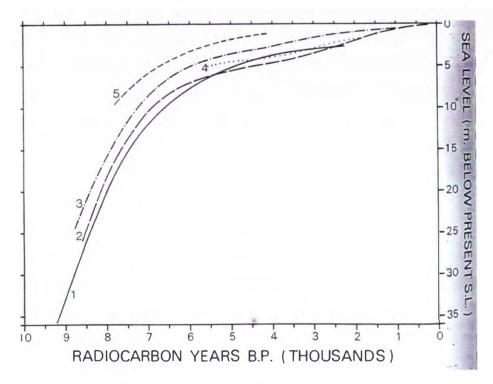


Figure 3.1b: Holocene sea level curves. (Source: Heyworth and Kidson, 1982:110)

Key: 1, Bristol Channel; 2, English Channel; 3, Cardigan Bay; 4 Somerset Levels Trackways; 5, North Wales.

Analysis indicated that with the exception of North Wales (5), no real difference existed between the curves and the apparent separation was due to inherent data uncertainties and the use of present-day MHWS as the common reference datum. Furthermore, in the last 8000 years, the rate and timing of sea level rise was comparable over the whole region and there was no evidence of sea levels higher than the present (Heyworth and Kidson, 1982). Consequently, there has been a Holocene sea level rise of approximately 35m in the Bristol Channel/Severn Estuary.

3.2.2 Coastal Environment

Penarth beach runs from its southernmost point, approximately NNE where it rounds Penarth Head to run approximately NNW, to the site of the Cardiff Bay Barrage. The coastal water is a popular sailing ground, which is used by the yacht club and powerboat/ski club, and these clubs have launching facilities, which are constantly used. The foreshore has generally retreated with a trend to steeper slopes throughout the twentieth century (SBCEG, 1999). As mentioned in Section 3.2.1, recession rates of 0.2 m per annum have been recorded (Williams and Davies 1987; 1990) and as a consequence of hinterland developments, two sections south of the RNLI station have been protected (Bullen, 1993a; Wallingford, 1992; SBCEG, 1999). Shingle beaches, rock platforms, eroding cliffs and a short section of seawall make up the shoreline from Lavernock Point to Penarth Head. Wallingford (1992) argued that the beach is largely supplied with material, which erodes from the cliffs further south. Erosion of the cliffs north of the pier and Penarth Head provides material for the beach between the pier and the Cardiff Bay Barrage.

The area is protected from westerly and southwesterly waves that dominate much of the Bristol Channel. The relevant wave climate is generated from the east and southeast and shore parallel currents are important, with a strong southerly residual carrying sand and fine material into the Bristol Channel. Nearshore banks complicate the wave and tidal

conditions and it is assumed that there is some interchange of material between the banks and the shore (SBCEG, 1999). The tidal currents are unusual in that there is an almost continuous southward flow offshore during the whole tidal cycle (Wallingford, 1992). This is due to a large eddy on the flood tide just before high water between Cardiff Grounds and the coastline from Lavernock Point to Penarth Head. The effect is particularly strong during spring tides but also occurs during neaps. According to Wallingford (1992), this predominantly southward flow will also tend to counteract wave-induced drift of beach material, particularly the finer grained material over the lower part of the inter-tidal foreshore. The tidal regime and wave conditions at Penarth are as follows:

Tidal Regime:	Mean high water springs:	6.0m AOD
	Spring tidal range:	11·0m
	Best estimate of 1 in 100 year water level:	7·9m AOD
Wave	1:1 year return period wave height (H _s):	1·5m
Conditions:	1: 1 year return period wave period (T _z):	5·6s
	1:100 year return period wave height (H _s):	2·1m
	$1:100$ year return period wave period (T_z):	6·1s

The dominant exposure is eastward and wave conditions are predicted for the 5 m CD contour off Penarth (SBCEG, 1999).

According to Wallingford (1992), their previous work defined the directional wave climate 2 km offshore from Penarth. This shows very modest wave heights occurring along this coast; for example, significant wave heights greater than 1·2 m are predicted to occur less than one percent of the time. The most frequent wave direction is from between 190° and 210° true at this offshore location and it can be deduced that, after refraction onto the coast, these waves will produce a northward drift of beach material. This deduction of a net northward drift of beach material agrees with the conclusions of both Bullen (1993a) and SBCEG (1999).

The high energy associated with tides in the estuary has a large effect on the movement of sediments held in suspension and the distribution of bottom sediments. Upstream of a line joining Barry (approximately 10km southwest of Penarth) and Bridgewater Bay (on the opposite side of the estuary), large quantities of fine sediment are redistributed according to the tidal state and range (Figure 3.2).

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Figure 3.2: Severn Estuary – Barry / Bridgewater Bay. (Source: SBCEG 1999)

During the full ebb and flood of spring tides, similar levels of suspended solids, which may be up to 10,000 milligrams per litre, may be found throughout the water column (SES, 1997). It is estimated that the estuary carries over 30 million tonnes of suspended sediment on a spring tide and according to SES (1997) the tidal currents are not the same on both sides of the estuary. At times of maximum ebb flow (HW + 2·8 hours) on a spring tide, just off the tip of Penarth Pier, the current is about 0·4 m/s (Wallingford, 1992).

3.3 Cultural Environment

3.3.1 Historical

At the start of the nineteenth century, the population of Penarth had been 72, but by 1851 it had risen to 273 (Carradice, 1994). With coal providing a major economic impetus, two Acts of Parliament, in 1856 and 1857 respectively, granted permission for the Penarth Harbour and Railway Company to build docks at Penarth. Consequently, in 1859 Penarth Ely Harbour was opened followed in 1865 by Penarth Dock. (John and Williams, 1980). The docks were built in the lee of Penarth Head and this was followed quickly by the development of the town on the plateau top above (Robinson and Millward, 1983). The 1861 census indicated a population of 1923 (Carradice, 1994) which showed a sevenfold population increase in ten years.

Ely Tidal Harbour provided ten tidal coal-shipping berths and an iron-ore discharging stage at a cost of £137,000 whilst Penarth Dock, which opened for traffic in 1865, cost £685,000 (John and Williams, 1980). In 1881, to meet the demands of the burgeoning coal trade, Penarth Dock was extended into the hillside, necessitating the removal of more than one million cubic yards of earth and rock (John and Williams, 1980). This led to peak trade being achieved in 1885 with two and three quarter million tons of coal passing through the port (Carradice, 1994).

Post 1885, Penarth Docks only underwent minor improvements: total expenditure on the entire enterprise from the time the tidal harbour was built until the year 1903, amounted to £1,243,000 (John and Williams, 1980). Although entering something of a decline, Penarth remained for many years, one of the major coal exporting ports in South Wales.

The town of Penarth grew quickly in the wake of its docks. A Coastguard Station opened on the eastern edge of the town in 1864, together

with a Lifeboat Station, virtually on the beach, guarding the approach to Cardiff and Penarth Docks and the wide expanse of the Bristol Channel.

Penarth Station opened in 1878 and thereafter, hundreds of trippers poured into the town every summer's day (Carradice, 1994). The original houses were simple dwellings for labourers employed in building Penarth Docks and, most of the labourers came from Cornwall, Gloucestershire and, in particular, Ireland. Wild and drunken behaviour was common, particularly on pay nights, and the new town quickly won a reputation as a rough place in which to live (Carradice, 1994).

Soon, however, prosperous businessmen from Cardiff, many of them newly rich industrialists connected with the docks, coal or shipping trades, began to build houses for themselves in Penarth, especially Marine Parade. These palatial grey stone houses were elegant and substantial and once the railway link was built, commuters sought to buy less substantial redbrick villas. Thus Penarth quickly became a "desirable suburb" of Cardiff, a role which has been reinforced in recent years (Robinson and Millward, 1983; Carradice, 1994). Consequently, by 1891, the population had risen to 12,414.

The town of Penarth, though coming from an industrial base, was also aware of its position as a seaside resort.

"By the early 1880's complaints were being made about 'the rabble from the hills' monopolising the beach. Some of these unwanted visitors had the temerity to dry their bathing dresses on wire lines along the strand; some of them even bathed naked in the sea!"

(Carradice, 1994: 13.)

With a conscious attempt to upgrade the resort and exclude the workers, and rougher element of the tourist trade, Lord Windsor, the principal landowner in the district, conceived a plan to make Penarth the equal of Brighton.

An elegant park was duly opened in 1880, the town baths, which had two swimming pools, opened in 1885 and the Esplanade Hotel opened in 1887. The major achievement, the Esplanade itself, was built in 1883-4 (Plate 3.2). Linking with Marine Parade and Plymouth Road, it formed a circular carriage drive around Penarth. Thirty-six feet wide, the road ran along the sea front in a north-south direction, providing an elegant promenade for strollers. No longer did visitors have to cross a stony beach; the Esplanade provided a modern flat surface, which would not damage their clothes or shoes. Penarth had gone up-market and ironically, unlike Brighton and other towns on the south coast of England, which had expanded from upper class resorts to take in working people, it was the other way around (Carradice, 1994). In 1894, work started on the construction of Penarth Pier and will be discussed in Section 3.3.2.2.



Plate 3.2: Esplanade (September 1997).

The South Wales ports were at their peak in the years prior to the First World War. Of the total volume of traffic, coal accounted for 90 per cent. After the war, many of the markets for Welsh coal had been lost and other trade had to be rebuilt (John and Williams, 1980). In addition, changes from coal to oil and a decline in the mining industry led to a considerable shrinkage of activity at the ports. As the activity declined, there was a corresponding growth in spare capacity at the major ports and of

unemployment in related occupations, e.g. seamen. This situation necessitated the shutting of Penarth Docks by the mid 1930's.

3.3.2 Built Environment

3.3.2.1 General

Penarth is mainly residential with an important recreational coastal frontage. This includes the low-lying promenade between the high cliffed areas of Cosmeston and Penarth Head. At the southern end it climbed sharply up Cliff Hill which had >40,000 m³ of rubble removed from the hillside in order to make the circuit complete (Carradice, 1994). Today, a road runs just landwards of a narrow promenade, and is protected by a near vertical concrete sea wall. This has a wave return bull nose at its crest and three viewpoints that protrude out from the line of the wall. The only land, which is likely to be at risk from wave overtopping of the defences, is located between the pier and the lifeboat slipway (Wallingford, 1992). This agrees with Bullen (1993a), who noted that at high water on a spring tide, the still water level is within 1 m of the crest of the wall and overtopping during storm conditions appears likely (Plate 3.3). There are private residences located within 15 m of the cliff top on each side of the Esplanade from Cosmeston to Penarth Head. At the southern end of the promenade, there are three slipways. The Yacht and Powerboat/Ski clubs use the most southerly, built post Second World War. The second slipway, completed in 1995, was a new launching slipway for the lifeboat station, originally established in 1864 (Carradice, 1994). The third was the old lifeboatlaunching slipway, which is now in a state of disrepair.



Plate 3.3: Seawall at high tide (September 1997).



Plate 3.4: Columns to multi-storey car park (September 1997).

North of the pier there was a multi-storey car park that was demolished in autumn 2001 (Soltys: Brewster, 2003b) by the Coast Protection Authority (Vale of Glamorgan County Council; VGCC). It was originally closed in 1998, on the advice of structural engineers that it was unfit for public use and Plate 3.4, showed the condition of the supporting columns to the beach surface in September 1997. There is the remains of an old quay structure, which has now all but been destroyed by the sea (Plate 3.5), and the sea defences in the vicinity have suffered severe damage. The extreme northern end of the seawall had been repaired following collapse and much of the rest of the wall has been damaged (Plate 3.6). There is evidence of undermining and repairs are on going, a similar situation to that described by Wallingford (1992).



Plate 3.5: Old quay, north of Pier (September 1997).

There are a variety of outfalls along this section and these have been documented to retain higher beach levels on their southern sides (Wallingford, 1992; Bullen, 1993a). There are nine groynes along the study area from the southern end of the beach to the site of the Cardiff Bay Barrage. These are constructed at 90° to the beach and will be considered in Section 3.3.



Plate 3.6: Damaged sea defences (September 1997).

3.3.2.2 Penarth Pier

The construction of Penarth pier commenced in April 1894 as the last item in Lord Windsor's plan to make Penarth an upper-class resort.



Plate 3.7: Penarth Pier

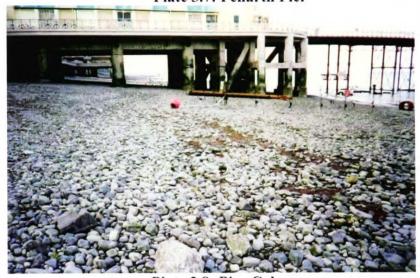


Plate 3.8: Pier Columns

It was to complement the Esplanade (Plate 3.7), and almost from its opening in April 1895 was an enormous success (Carradice, 1994). It was vastly different from almost all other seaside piers and the lack of common amusements was a deliberate ploy by the directors of the pier company and by the wealthy residents of Penarth to deter working class visitors, often termed 'loafers'. At just over 200 m (658 feet) long and 7.6 m (25 feet) wide, it was limited by the original Pier Order, which forbade it reaching too far into the Channel. At the landing stage the width doubled to 15.2 m (50 feet) to facilitate easier docking for boats and it stood approximately 15.2 m (50 feet) above the seabed (Carradice, 1994). The pier has recently (1999) undergone major refurbishment and it currently has a berthing facility for a pleasure paddle steamer, which operates during the summer

months. The pier is supported on circular columns, as shown in Plate (3.8). According to Wallingford (1992) and Bullen (1993a), the pier may have modified the equilibrium of the area, evidenced by the near vertical sea wall behind it, suffering toe erosion. The pier supports and a beam parallel to the contours have produced a sheltered area in which beach material has accumulated. Overall the pier acts as a partially effective groyne (Wallingford, 1992), promoting higher beach levels to the south but causing erosion of the upper beach further north.

3.3.2.3 Penarth Seawall and Esplanade

In 1924 the Penarth Urban District Council purchased the promenade and the pier from the Penarth Pier and Promenade Co. Ltd. (Ryan, 1983). The original seawall was constructed of stone pitching 200 to 300 mm in thickness at an angle of 40° to the horizontal (Figure 3.3). At the top of the wall, the junction with the promenade is made with a small concrete upstand. At the base of the wall, the original ground is horizontal before meeting the cutting slope of the seawall foundation.

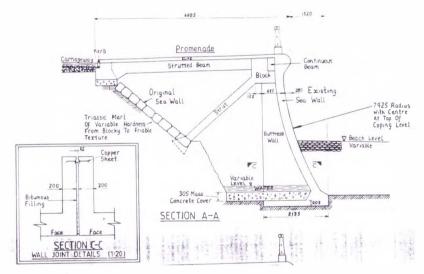


Figure 3.3: Section through Esplanade seawall. (Source: VGCC)

The Esplanade was designed in 1911 using reinforced concrete throughout, when so called "ferro concrete construction" was still in its

infancy. It is flat, straight and level throughout its length (Ryan, 1983; Ordnance Survey) and is divided into five equal sections by bastions that are constructed monolithically with the Esplanade.

The main seawall was constructed during 1926/7 and is counterfort, supported at 3·6 m (average) centres by buttresses and struts. It carries the outer edge of the promenade slab with the inner edge supported at the top of the old seawall (Figure 3.3). The primary function of this wall is to protect the carriageway and seafront properties against the action of tidal and storm damage (Ryan, 1983; Bullen 1993a). The seawall is a constant 250 mm thickness, increasing to 535 mm at the wave return coping. It is concave when viewed from the beach (Plate 3.9), to a circular arc of approximately 7.9 m, the centre of which lies at the coping level. The toe at the base of the wall projects 1·3 m beyond the line of the coping face.



Plate 3.9: Esplanade Seawall

Approximately 2 m below the toe of the original seawall, the underside of the foundations lie on marl bedrock. A layer of mass concrete covers the foundations as a protective layer, and when measured from this level, the internal height to the underside of the concrete slab, is generally in the range 4.6 to 4.8 m. This drops to 4.1 m at one location and these differing internal heights are probably due to varying thicknesses of concrete fill rather than foundation levels. According to Ryan (1983),

externally, the wall is probably a constant height of around 5·3 m from the foundation underside, although this is impossible to determine without excavating the beach.

3.3.2.4 The Cardiff Bay Barrage

In 1985 the Secretary of State for Wales commissioned an investigation into the possibility of a barrage across the mouth of Cardiff Bay. A pre-feasibility study was undertaken to ascertain whether it would be feasible from all technical aspects and this was followed over the next five years by a feasibility study, an initial design and finally a detailed design. Physical and numerical models were used in these processes. The Cardiff Bay Development Corporation (CBDC), an agency of the Welsh Office, was set up to oversee the regeneration of 1,100 Ha of derelict industrial land to the south of Cardiff. This regeneration included the construction of the 1·1 km long Cardiff Bay Barrage to impound the estuaries of the Rivers Taff and Ely and create a 200 Ha freshwater lake with approximately 12·8 km of attractive waterfront.

It was a requirement of the barrage design that, in addition to performing the functional role of impoundment, it should be an aesthetic structure that would encourage public access and use of the area. The resulting design formed a curved "S" shape across Cardiff Bay creating 7 Ha of public open space (Plate 3.10). The inland face of the structure is being landscaped to provide a public park for leisure use and includes steps, groynes and viewing piers. On completion it will be possible to walk along a variated parkland from one end to the other.

Plate 3.10 has been removed from the digitized thesis for copyright reasons.

Plate 3.10: Cardiff Bay Barrage (Source: Balfour Beaty).

An Environmental Impact Assessment was undertaken for the barrage and was a long process beginning with the feasibility study. The identified environmental impacts of the total exclusion barrage included ecology and nature conservation, water quality and water management; fish and fisheries; groundwater and flooding, and recreation and amenity (Hill *et al*, 1996). Environmental Assessment of the Barrage was started in the mid 1980's and Hill *et al* (1996) did not consider environmental impacts on the Penarth coastline. Bullen (1993b) argued that the Barrage could cause outflanking and that long-term changes to the coastline were likely. However, CBDC did commission research into these potential impacts, both at feasibility and design stages, which included the report by Wallingford (1992).

The barrage spans the region between Queen Alexandra Head in the north and Penarth Head in the south. The main structural elements include: embankment, sluices, locks, harbour, fish pass and control tower, the general arrangement as shown in Figure 3.4. The embankment is a rock and sand fill structure, approximately 800 m long, 100 m wide at the base and 25 m wide at the crest, with its seaward face protected from wave action from the southeast by rock armouring. As two rivers, the Taff and Ely are impounded, there is a large fish pass of capacity 10 m³/s (normal fish pass

capacity being 0.5 to 1 m³/s). Furthermore, it has to accommodate a head difference in either direction due to the tides and at high tide there is pumped attraction water of up to 4m³/s (Hunter, (1996). There are five sluice gates, each 9 m wide and 7.5 m high with a total capacity of 2,300 m³/s (500,000 gallons/s) which is more than double the combined maximum flood flow of the Taff and Ely. Therefore they can empty the bay in one tidal cycle.

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Figure 3.4: General Arrangement - Cardiff Bay Barrage

Three locks allow safe passage between the inland bay and the sea and are designed for an ultimate yacht population of 2,000 vessels. The lock chambers are 40 m long and 10·5 m wide between fender faces, and the gates also have to withstand head differences in either direction. The harbour size is approximately 1·8 Ha of water for moorings.

Two breakwaters, each 20 m high and innovative in design, provide a sheltered harbour. The need to avoid excessive wave reflections, coupled with the very deep water at high tide, meant that conventional rubble embankments were inappropriate. The screen breakwaters consist of a permeable seaward screen formed with concrete "planks" with a solid screen on the harbour side (Plate 3.11). Waves hitting the breakwater are partly dissipated when passing through the front screen, and are then reflected from the solid screen. The reflected wave is then out of phase with the incoming wave which maximizes energy dissipation and minimizes the overall wave reflection from the structure. Optimisation of the dimensions of the screen breakwater was carried out in a physical hydraulic model (Hunter, 1996).

The breakwaters were constructed of float-in caissons, one of them being tied into the coastline with a traditional armoured embankment. The structures at Penarth Head and the eastern flank interfere directly with the natural low water channel of the Rivers Ely and Taff respectively. A new channel was designed to realign the flows, and Littlewood *et al* (1996) stated initial modeling highlighted problems of coping with mean annual fluvial flow. The design was modified to overcome this problem.



Plate 3.11: Barrage breakwater (April 1999).

Construction commenced in May, 1994 and the barrage was officially opened in November 1999, at an approximate cost of £200 million. On the 4th November, 1999, the sluice gates of the barrage were officially closed to prepare for the creation of the permanent freshwater lake. The changeover from an estuarine to freshwater environment means there will no longer be a brackish habitat. The lake was not initially freshwater because dredging operations in the impoundment had to be completed as a result of sediment oxygen demand. The mudflats, which are now submerged, were a Site of Special Scientific Interest (SSSI) as they provided winter feeding grounds for migrating wading birds, including Redshank and Dunlin. Cardiff Bay attracted high numbers of these birds because the elevation of the mudflats was higher than other areas, which meant they were exposed for longer during the tidal cycle. In addition, according to Hill et al (1996), the bay provided a sheltered stable environment. The mud banks upstream of the River Ely appear to have changed since the construction of the barrage. There is increasing sedimentation and mudbanks appear to have become more extensive (Plate 3.12). Although this is a qualitative observation, it would agree with that made by Dyrynda (1996), following the construction of the Tawe Barrage.



Plate 3.12: Mudbanks River Ely (April 2000).

The lake has a water level of 4.5 m above Ordnance Datum and this is 4.2 m above the mean tide level. At present it is unclear what correlation there will be between the rise in average water level and the consequent rise in groundwater levels. Therefore, there is a risk of an adverse impact on the existing high density housing in south Cardiff, although legal protection has been granted to potentially affected properties (Heathcote and Crompton, 1996).

Thirty days after initially impounding the bay, the Environment Agency ordered CBDC to return the area to a tidal flow because of concerns regarding the dissolved oxygen content in the bay and, the technical operation of the barrage, including sluices and fish pass (South Wales Echo, 1999; Daily Mirror, 1999). The initial problems were overcome and following the installation of an aeration system and the completion of dredging of the bay, the freshwater impoundment commenced in Spring 2001.

3.4 Interaction of Physical and Cultural Environments

The beach and coastal environment along the study area will now be considered in specific sections.

3.4.1 South of Lifeboat Slipway

Here the sea can reach the cliff base and erosion is taking place quite rapidly between Lavernock Point and Penarth. The beach is marl and pebbles with a little sand in the area near to the slipway (Bullen, 1993a). This rapidly gives way to limestone shingle with the size increasing towards Lavernock Point. The bedrock is red marl, which can be seen beneath the shingle, and the pebble size is in the range of 100 mm – 300 mm. Just south of the lifeboat slipway, concrete seawalls and revetments were installed to reduce cliff erosion. The oldest walls are closer to the main promenade and extend over virtually the whole cliff face (Wallingford, 1992).

Further south, the Vale of Glamorgan Council (VGC) in 1980 introduced a strategy for the removal of 1650 m³ of overburden, together with drainage of the upper cliff face to relieve pore water pressure. Subsequently, as toe protection, a 174 m length of 2 m high wave wall was constructed at the cliff base (Williams and Davies, 1987; 1990). In addition, north of this wall, two concrete groynes were constructed normal to the cliff face to arrest the sediment carried in the longshore current. According to Wallingford (1992), these groynes have had some effect at retaining higher beach levels on their southern flank and there is a sufficient build-up of stones and pebbles south of the groyne field, to help protect the base of the cliff protection wall.

3.4.2 Lifeboat Slipway to Penarth Pier

The seawall described in Section 3.2.1 protects this area. The beach fronting this wall is predominantly limestone shingle and according to Wallingford (1992), the beach level is adequate along the southern part of the wall whilst the semi-circular projections, retain the upper beach material. Approximately 100 m from the wall towards low water level there is only a thin veneer of mobile beach material resting on a solid rock substrate. This marl bedrock is eroding but apparently much less rapidly than the surrounding cliffs (Wallingford, 1992). Because of the different hardness of the rock strata in the shore-platform, the inter-tidal foreshore has a complicated pattern of ridges and troughs. Fine sediment collects in the troughs and in some places is a mixture of sand and mud. Interestingly, where more substantial rock outcrops occur on the otherwise nearly flat platform, mobile beach material gathers in the sheltered areas behind them. One such outcrop lies just beyond the cliff protection works discussed in Section 3.3.1 and another can be found just off Penarth Head.

3.4.3 Penarth Pier to Penarth Head

The beach near the pier is made up of limestone cobbles, shingle and sand. To the north of the pier there is a short length of curved seawall with

a vertical lower section. Prior to its demolition, the multi-storey car park and the seawall further north had suffered serious damage with clear signs of settlement (Wallingford, 1992). Still further north, the remains of an old quay structure has been all but destroyed by the sea (Bullen, 1993a). The end of the seawall has been repaired with a vertical curved wall of concrete filled sandbags. It is interesting that the beach in front of the repaired section is mainly sand and the supply of material to the beach as a result of cliff erosion is prevented by structures. According to Wallingford (1992), the impression is that the frontage is losing more material to the north than it is receiving from the south. At the end of the seawall there are two concrete groynes at 90° to the beach. Only where groynes are present does the beach become healthy but then only on the southern side with a more intense problem immediately to the north (Wallingford, 1992). The cliffs beyond the wall onward and around Penarth Head are unprotected and erosion is causing the undermining of the last groyne. The cliff angle varies between 68° and 78° to the horizontal (Bullen, 1993a) and the beach slope is fairly uniform along this section. It is between 7° and 8° to the horizontal with material forming the beach decreasing in size from the pier to Penarth Head although, there are rock fragments in the size range 150 – 500 mm. Other structures in the vicinity of Penarth Head include the remains of an old World War II searchlight stage and walkway/steps, most of which has collapsed on the beach with the remainder hanging precariously from the cliffs. In this area, water can be seen running down the cliff face contributing to the instability.

3.4.4 Penarth Head to the Cardiff Bay Barrage

Here the beach changes direction from NNE to NNW and is made up of mainly marl/shale gravel and pebbles in the size range 5-25 mm (Bullen, 1993a), with part of the beach being made up of larger rock fragments and limestone cobbles (75-500 mm). At the Barrage end of this section, there are four concrete groynes at right angles to the beach and these have retained a substantial volume of stones and pebbles on the upper beach (Figure 1.3).



Plate 3.13: Accelerated cliff erosion (April 1999).



Plate 3.14: Mudslide adjacent to groyne 8 (September 1997).

There is a marked difference in beach level between the southern and northern sides of the groynes and this is in the region of $1-2\,\mathrm{m}$ (Wallingford, 1992; Bullen, 1993a). The southern sides of the groynes being higher again indicate a net sediment movement in a northerly direction along the beach. However, even in these locations, the cliff face is still eroding and nearest the Barrage is subject to increased erosion due to wave attack at the base (Plate 3.13). On completion of the Barrage the last groyne was incorporated into the breakwater, which formed the outer harbour.

Harder gypsum layers in the cliff, being more resilient than the marl, tend to overhang and when a fall does occur, it is consequently more spectacular. Individual slabs can be in excess of 50 kg (Wallingford, 1992) and there are several examples of erosion due to water running down the cliff-face (Plate 3.14).

CHAPTER 4: BEACH MANAGEMENT

4.1 Introduction

It has been reported that 95% of the world beaches are eroding due to human activities and moving dynamics (Pranzini and Rossi, 1995). According to Cipriani *et al* (2004), this worldwide tendency to coastal erosion may be locally aggravated by some of the very strategies implemented to reverse this pattern. Therefore, as a result of natural phenomena such as tides and winds (physical environment) and social and economic activities (cultural environment), coastal areas undergo transformation. Coastal erosion and the disappearance of beaches as a result of wrong management decisions and lack of effective legislation are consequences that require application of a series of engineering techniques (van der Salm and Unal, 2003).

4.2 Beach Management

According to Nelson and Williams (2004), beach management is a relatively recent sub section of Integrated Coastal Management (ICM). Micallef and Williams (2002) identified that effective beach management may be considered as resulting in a higher financial return due to increased beach use, reduced maintenance and restoration costs, and improved coastal defence and conservation value. An important concept of beach management is that managers should seek to understand and work with natural processes if management plans are to be effective and long lasting. Identification of local characteristics and/or problems and the socio-economic value of resources were also considered necessary.

An understanding of coastal processes including the identification of sediment sources have important repercussions regarding erosion management (Micallef and Williams, 2002). Collection of data on coastal processes and sediment-related

characteristics should include both physical and environmental components and be complemented by subsequent long-term monitoring programmes.

Effective beach management should be considered as a function of socioeconomic pressures and physical environmental constraints. As argued by Micallef
and Williams (2002), analysis in this context is often linked to either problem or preproblem solving exercises in the form of understanding an environmental system,
examining active management if present and in identifying any problems with the
system. Beach management has a spatial and temporal dimension. Spatial measures
with respect to beach management should attempt to identify external developments
which can influence natural beach dynamics, for example, the Cardiff Bay Barrage,
whilst temporal analysis should address timeline variation and causal change, for
example, sea level trends. The collection of quantitative data is often expensive and
time consuming but there is a need for long-term quantitative data related to beach
dynamics (Galofre and Montoya, 1995). From a qualitative aspect there is a need to
identify socio-economic and environmental values associated with beach recreation
and construction.

Because of the problems associated with hard engineering in the near shore zone, not least cost and maintenance, alternative soft engineering techniques that work in conjunction with natural coastal processes are increasingly being used. These include the construction of submerged breakwaters (Aminti et al., 2002), which reduce effective depth offshore thereby reducing wave power and erosion of the beach, and groyne field techniques, which promote sediment deposition, and evolution of the shoreline (Kunz, 1999). A further method for protecting the shoreline is beach nourishment which is a soft engineering solution that has been used primarily for the benefit of the tourism industry. A good example is Miami Beach, which had virtually no beach by the mid 1970's, and as a result, its facilities were run down. Following beach nourishment in the late 1970's Miami Beach was rejuvenated to such an extent that the current annual revenue from foreign tourists alone is \$2.4 billion, about fifty

times the \$52 million cost of the 20 year project (Houston, 2002). Furthermore, foreign tourists that visit Miami Beach, pay more in Federal taxes than the Federal Government spends nationally on beach nourishment. Using the capitalised annual cost of the project, for every \$1 that has been invested annually on the nourishment, Miami Beach has received almost \$500 annually in foreign exchange.

Cipriani et al, (1999) reported that approximately 7km of beaches at Marina di Massa in northern Tuscany, Italy experienced severe erosion as a consequence of the construction of an industrial harbour in the early 1920's. This interfered with longshore sediment transport inducing a sediment deficit to downdrift beaches. Despite the construction of different types of hard structures including seawalls, breakwaters and groynes, as well as the further construction of a submerged breakwater, beach erosion proceeded and the tourism industry suffered. As a consequence of this retreat, a beach nourishment program was introduced where offshore sand was dredged and dumped on the beach, the project being funded by bathing establishment owners and the local authority. The mean grain size of the borrow material was finer than the native beach sediment, resulting in approximately 66% of borrow material disappearing within one year. The beach quality also worsened due to the fine sediments making the beach dusty. Although financially successful in terms of tourist revenue, the process was unsustainable. This led to a loss of faith in a soft engineering solution by private owners, tourist operators and local authorities alike, and this could lead to the construction of additional hard structures. However, had more money been spent initially and the project taken into consideration all prevailing conditions, coarsegrained sediment would have been imported and the nourishment would have been more successful.

Micallef et al, (2001) undertook environmental risk assessments on a proposed beach nourishment project for tourism at St George's Bay, Malta. Two different sized sediments, one fine grained and one coarse grained were modelled utilising potential volumes whilst considering positive and negative performance during both construction and post-construction phases. Findings showed a high probability (0.95)

of a mild impact during the post-construction phase with fine-grained sediment as against a high probability (0.86) of a negligible impact with coarse-grained sediment. These predictions are in agreement with the nourishment problems identified at Marina di Massa.

The lesson to be learned is that beach nourishment can be a sustainable way to manage coastal erosion, provided that coastal processes are appropriately considered. Following nourishment, the new wider beach serves as shore protection from the impacts of storms and increases recreational benefits and new tourism related opportunities (Benassai et al, 2001). For example there currently exists a European sponsored project in the Lazio region of Italy to define technical, environmental and economic problems linked to the borrowing of marine sand for reconstruction and maintenance of littoral coasts under erosion. The main issues relate to the estimation of required sand volumes, the search for sediments on the continental shelf, the environmental compatibility, the extraction technologies and the projected costs (Lupino and Riccardi, 2001). However, beach nourishment is neither a cheap nor a permanent solution and where the coast is protected by seawalls, the reconstruction of a sandy beach is particularly expensive because of the deepening in the near shore profile, induced by wave reflection from the structure itself (Cipriani et al, 2004). It has been shown that a gravel beach may be proposed as an alternative to seawalls, providing a functional beach at a lower cost of implementation and maintenance with Coarse grained sediments show high coastal protection against natural forces. stability both on natural and artificially created beaches. This is due to the greater mass of the single grain and greater porosity and permeability of the beach (Pranzini, However, despite the frequency with which rock is used in the marine 1999). environment, understanding of its behaviour remains incomplete (Surveyor, 2003). In Italy approximately 20 million cubic metres of marine sands have been used for beach nourishment during the last ten years (Cipriani et al, 2004) whilst 20 million tonnes was used in the same period to nourish Britain's beaches (Symes and Byrd, 2003).

4.3 Function Analysis

The growing use of coastal areas affects sustainability since issues become more complex. Needs are created and subsequently attained by the formation of amenities. These result in an alteration of social, economic and environmental functions and interactions are produced (Jorge et al, 2002). Without an element of sustainability and measurable indicators within an integrated coastal management strategy, deterioration of the coastal environment is likely to persist (Charlier and Bologa, 2003). As the coastal environment deteriorates, economic wealth will eventually deteriorate since the economy is dependent on environmental functions or services (Houston, 2002). However, a sustainable conservation or development strategy can be introduced once appropriate indicators are identified. Whilst coastal areas often have varying degrees of conservation value and development potential, it is simplistic to subjectively identify an area that has low conservation value and high development potential and vice versa. Scientific sustainability indicators are needed to accurately determine conservation value and development potential in order to effectively implement a sustainable management strategy. A problem arises when attempting to identify ecological and economic values since these will usually require a unit of measurement which may be difficult to obtain.

Van der Maarel (1978; 1979) and De Groot (1992) approached the assessment of ecological and social values of an environment by considering goods and services provided by various processes and components within that environment. They referred to goods and services (or social values) as the functions of nature, while processes and components represented the natural environmental characteristics of that same environment. This approach is referred to as Environmental Function Analysis. It considers the natural environmental characteristics of an environment and their ability to provide environmental goods and services (environmental functions) and may be employed as a planning and decision making tool (Micallef and Williams, 2003).

Early work by van der Maarel (1978; 1979) later developed by de Groot (1992), culminated in the identification of four main functions, namely:

- Production functions these include all kinds of renewable and non-renewable resources.
- Carrier functions providing space and substrate for human activities.
- Information functions provision by nature of reflection, spiritual and cognitive enrichment and signal functions.
- Regulation functions capacity of ecosystems to regulate essential ecological processes which in turn, contribute to a healthy environment (e.g. clean air, water, soil).

In Environmental Function Analysis, the capacity of a given (semi) natural ecosystem to provide certain goods and services (environmental functions) depends on its environmental characteristics (natural processes and components). In a theoretical consideration of the Function Analysis approach, Cendrero & Fischer (1997) proposed a technique for assessing environmental quality of coastal areas through characterisation of conservation value and use/development potential. This and the technique's ability to effectively integrate scientific evaluation into the decision making process were presented as a direct contribution to coastal planning and management (Micallef and Williams, 2003). The proposed technique involved a four-step approach:

- 1. Definition of area boundaries and selection of homogeneous land-use units within this boundary.
- 2. Identification of characteristic parameters for the environment in question that describe and distinguish between the environmental and socio-economic components of that environment. Based on an exhaustive list of environmental components and characteristics, Cendrero & Fischer (1997) considered aspects specific to coastal areas in order to identify the relevant environmental characteristics that describe the ecological (natural) and socio-economic (human use) components of that environment.

- 3. Allocation of values to the characteristic parameters identified.
- 4. Comparison of natural (ecological) value with human (use/development potential) value to determine sustainable development strategy (Figure 4.1).

Taking into consideration the ethos of integration in Environmental Management, each of the environmental functions identified, that is, Regulation, Carrier, Production and Information functions, should be considered in terms of its social, economical or ecological value. Therefore, Cendrero and Fischer (1997) created a methodological structure that was able to identify coastal environmental quality. Indices that are numerical in nature can be used to compare environmental quality and development potential in coastal regions to aid in decision making processes and develop management strategies.

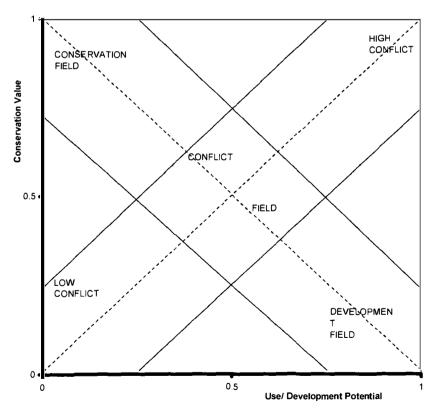


Figure 4.1: Conservation/ Development Matrix. (Source: Cendrero and Fischer, 1997: 742)

The matrix (Figure 4.1) has been used with promising results by Van der Weide *et al*, (1999) to identify appropriate management options for two coastal wetlands in Turkey, and by Micallef and Williams (2003) for bathing beaches in Malta. Coastal areas located on the lower right of the matrix should be managed for development and those at the top left should be managed for conservation. Van der Weide *et al* (1999) proposed that conflicts be resolved by utilising in-depth studies to develop an appropriate management strategy which could be based on analysis of environmental indicators and anthropogenic components (Cendrero & Fischer, 1997, van de Weide *et al*, 1999, Micallef, 2002). The assessment of conservation and development status of coastal areas, is likely to make any management plans more effective.

4.4 Integrated Coastal Management (ICM)

An evaluation of the present day status of a beach is fundamental in deciding whether to actively manage a beach or to refrain from intervention. Concern about the past, or likely future, performance of a beach as a coastal defence is the most common reason for needing to manage a beach (Simm, 1996). In most vulnerable parts of the UK, however, beaches lie in front of built defences. It is important to differentiate between the general health of a beach and its function in coastal defence. A beach may be at a very low level and the volume within it may be reducing, but this does not necessarily mean that any management of the beach to improve the standard of defences is warranted (Simm, 1996). However, Micallef and Williams (2002) argued that very little research work has adequately addressed management needs of beaches and no recent books have been identified that address beach management in a specific or comprehensive manner.

Granja (2001) highlighted that the lack of coastal zone management has irreversibly contributed to the gradual degradation of Portugal's natural heritage, including the landscape that is one of its key elements. She further argued that promoting conservation implies an understanding of geoforms, their generation and longevity and their relationship with the ecosystems dependent on them. Therefore, to

promote conservation and management of the coastal zone, it is important to understand not only present day factors and processes, but also those that have been active in the recent geological past.

"Strategic planning for the management of our coasts relies implicitly upon an understanding of the physical processes responsible for shaping coastal morphology".

(Reeve and Spivack, 2001. p 55)

As the coastal zone has evolved from many natural and anthropocentric factors and processes and in a dynamic perspective, it is logical that management concepts inherent to these factors should be integrated. Therefore Integrated Coastal Management (ICM) could be undertaken using the approach put forward by van der Meulen *et al.* (2001) that it is a cyclic process of problem recognition, planning, implementation and monitoring. This includes recognition of the country's institutional setting in terms of stakeholders and the respective mandate, capacity, commitment and financial potential. It is important to accept the differences amongst all stakeholders and understand potential conflict when deciding who has most rights. Policy and implementation will be influenced by strategic and socio-economic potential in conjunction with available techniques for defending the coastline.

"but increasing populations and development are placing significant stresses on coastal resources; rising sea level is causing land loss, creating a collision course of social and sea level trends."

(Leatherman, 2001. p 181)

The cost and sustainability of technological solutions will be a critical factor as the use of seawalls to protect the coastline is now being questioned on the basis of cost and effectiveness (Bullen, 1993; Cipriani *et al*, 1999; van der Weide *et al*, 2001; Wiegel, 2002). Wiegel (2002) considers that they cause erosion in special circumstances where they prevent erosion of an upland source of sand for a beach downdraft whilst, van der Weide *et al.*, (2001), recognised that engineering works along the coast often cause erosion and accretion. This does not mean that hard engineering solutions will not be used where necessary and Basco (1999) believed that

there are many misconceptions behind the perception that seawalls increase erosion and destroy the beach. However, as advocated by van der Weide *et al.* (2001), to design appropriate mitigating measures, causes should be properly analysed and the technical and economic feasibility of such mitigating measures should be evaluated. Alternative soft engineering techniques in response to erosion are now increasingly used in coastal management. Examples include: groyne-field techniques to facilitate sedimentation (Kunz, 1999); submerged groynes to reduce wave energy in the near shore zone (Jackson *et al*, 2002; Robertson *et al*, 2002) and beach nourishment (Cipriani *et al*, 1999; Benassai *et al.*, 2001; Lupino and Riccardi, 2001; Micallef *et al.*, 2001), which work in conjunction with natural coastal processes.

Granja and Carvalho (2000) highlighted an objection to fundamental conservation and implied that it was unrealistic to believe that all coastlines can be conserved from impending sea level rise and subsequent coastal erosion. They suggest managed retreat and selective conservation of parts of the coast that are important to society and to use technological developments, where possible, to halt inland beach migration. Measures should be taken to attenuate or reduce the effects of erosion but where irreversible loss is evident, society must be persuaded to undertake a gradual and planned retreat from the waterfront. Beach management and sea defences can be justified on socio-economic grounds for a particular region and community, which depend on the beach, even though these may cause the acceleration of coastal erosion further along the coastline. Computer models such as REBAD (Balas and Ergin, 1997), which are based on estimated probability values, can be used as management decisionmaking tools for evaluating appropriate mitigating measures and responses. Consequently, the use of risk assessment tools in conjunction with benefit-cost analyses (Granja and Carvalho, 2000) could be used to justify policies of managed retreat where irreversible loss is evident. Interestingly, these strategies for management of the coastal zone agree with those generally suggested for coping with the negative effects of climate change. According to the Environmental Scientist (1999), they include prevention of loss, tolerance of loss and changing activities and location whilst, the mechanisms to achieve these strategies include institutional, legal, financial and technological aspects. Difficult decisions will need to be taken irrespective of stakeholder interests and there will be economic consequences (Leatherman, 2001). Policies will therefore need to reflect the complex inter-relationships of all activities.

According to Johnson and Scholes (1988), strategic management involves the following:

- 1. Analysis, where one seeks to understand the content of a system, the status of existing management, if any, and whether one should take any action;
- 2. Choice of the different courses of action available;
- 3. Implementation, where the chosen option is put into effect.

The results of research and monitoring will be evaluated in the context of strategic management, and consequently a management strategy will be justified for the protection of Penarth's coastal environment (Chapter 9).

CHAPTER 5: METHODOLOGY

5.1 Introduction

The Penarth Beach monitoring programme followed procedures recommended by Simm (1996) with data collection undertaken using standard civil engineering land surveying techniques (Wilson, 1974), photographs and Met Office statistics.

As many of the attributes of a beach alter seasonally, full evaluation requires information at various times throughout the year (Simm, 1996). Williams and Davies (1997) have argued that an analysis of the range and causes of problems leading to beach degradation was necessary. Therefore, data was collected, analysed and appraised following a carefully planned monitoring programme. The aim was to understand beach processes and responses, identify changes and potential problems, and thereby allow management to be proactive rather than reactive. The programme included:

- recording observations;
- taking photographs;
- measurement of beach profiles and plan changes;
- determination of sediment budget;
- environmental data collection;
- developing management strategies.

5.2 Desk Top Study

Prior to establishing the field programme, a desktop study was undertaken to ascertain the geological and historical development of Penarth Beach. Sources of information included:

- 1. Geological memoirs and maps (Bullen, 1993a and b; Owen, 1984; Robinson and Millward, 1983; SES, 1997; Wallingford, 1992; Williams and Davies, 1987; Williams and Davies, 1990).
- 2. Published research papers (Bullen, 1993a and b; Williams and Davies, 1987; Williams and Davies, 1990; SBCEG, 1999 etc).
- 3. Ordnance Survey sheets (1:1250 digital Super plan).
- 4. Previous beach surveys (Bullen, 1993a; Wallingford, 1992; VGCC).
- 5. Engineering drawings and records (VGCC; CBDC; Met Office; Carradice, 1994; Hill *et al*, 1996; Hunter, 1996; Littlewood *et al*, 1996; SES, 1997; Bullen, 1993a and b).
- 6. Historic photographs (Bullen, 1993a; Carradice, 1994; VGCC).
- 7. Press reports and local knowledge (RNLI; VGCC; CBDC; Penarth Times, Bright, 1997; Rabaiotti, 1997).

5.3 Site Inspection

A site inspection was undertaken along approximately 1.5 km length of Penarth Beach, designated as the study area (Figures 1.2 and 1.3). This included all anthropogenic structures starting south of the first groyne (near RNLI slipway) and heading NNE for a distance of 1150 m to Penarth Head; then NNW for a further 350 m to the final groyne (No. 9) located on the site boundary of the Cardiff Bay Barrage. Anecdotal evidence suggested that coastal processes had changed and that the erosional condition of the beach was giving cause for concern. The results of this inspection were documented in Section 1.2.

Photographic evidence was collected relating to beach changes, including beach levels against seawalls and groynes and exposure of the marl shore-platform in the intertidal zone (Plates 1.1 and 1.7). Damage to structures was recorded including spalling and abrasion of concrete walls (Plates 1.5, 1.6 and 1.9) and exposure of structure foundations (Plates 1.1 and 1.7).

From the desktop study and initial site investigation, a strategy was adopted to achieve the aims of the monitoring programme identified in Section 5.1. This involved dividing the beach into two separate areas directly related to their bearing (NNE and NNW) as well as the type of monitoring to be undertaken. The rationale and assessment strategy underpinning these surveys will be discussed in the following sections.

5.4 Longshore Profile – Penarth Beach

A 750 m baseline was established on the beach foreshore, starting approximately 110 m SSW of the RNLI slipway and finishing approximately 300 m NNE of the pier. This length included all man-made structures fronting the foreshore. Figure 1.2 shows the 26 stations, each 30 m apart, and comprised the baseline used for monitoring the longshore profile. It was positioned using standard surveying techniques (Wilson, 1974), with a Sokkisha 30" electronic theodolite, steel tape and survey pins.

Profile lines for general monitoring should not be taken immediately adjacent to structures and a separation of about 5-10 m is recommended (Simm, 1996); whilst the baseline had to be tied to independent control points to enable them to be reestablished for successive surveys (Wilson, 1974). However, as can be seen in Figure 1.2, the whole length of the foreshore is influenced by anthropogenic structures and the variation of the longshore profile in relation to these structures was deemed necessary. This would enable any localised short-term and/or longer-term trends to be identified.

Therefore, stations 6, 10 and 16 were fixed to permanent control points on the backshore by triangulation (Wilson, 1974), and using the theodolite, the baseline was projected at 30 m intervals, either side, as far as stations 1 and 26 respectively.

Consequently, the full 750 m longshore baseline was positioned, at a minimum distance of 10 m on the seaward side of the seawall. It should be noted that the pier and multi-storey car park are supported on columns and are clear of the beach surface. The levels of the stations were obtained using a Leica automatic level and staff, tying into the Ordnance Survey Bench Mark on the Esplanade (9·01 m AOD). This enabled the longshore profile to be determined.

The first survey took place in September 1997 and the last in September 2002, a total of eleven surveys. These were carried out at the beginning of April and September each year to ascertain summer and winter profiles. Although not strictly "summer" and "winter" periods, storms are common during the months of October to March inclusive, whilst they are rare April to September (Met Office data sheets). Therefore, it is argued that the survey in April each year will show the cumulative winter effects on the longshore profile, whilst the survey in September will show the cumulative summer effects. According to Simm (1996), vertical accuracy is usually \pm 20 mm for topographic surveys and the range for plan position, is between \pm 0·2 m and \pm 1 m. However, every levelling survey closed to \pm 5 mm.

Photographs of the beach were taken in conjunction with the surveys. They were used to record beach development adjacent to structures and in areas not covered by the fixed cross-shore profiles detailed in following Sections. Furthermore, immediately after a significant storm event, additional photographs were taken. Therefore, any rapid change as a result of wave and current interactions with the beach structures, and/or if beach response was varied, would be supporting evidence to make the formal programme of measurements more effective. Results of these surveys are tabulated and considered in Chapter 6: Penarth beach.

5.5 Foreshore Grid – Esplanade (stations 6 to 14)

A 0.72 Ha area of foreshore was surveyed to establish a fixed grid of 240 m x 30 m, sectioned into 60 m longshore x 10 m cross-shore rectangles (Figure 4.1). The longshore limits of this grid are stations 6 and 14 fronting the Esplanade, whilst the

cross-shore limits are 10 m on the western (seawall) side (negative distance) and 20 m on the eastern (seaward) side (positive distance). This area is bounded by the old lifeboat slipway to the South and the Pier to the North. This was chosen due to the potential consequences of increased erosion and flooding risk with subsequent impacts on valuable real estate (Rabaiotti, 1997; Penarth Times 1998; SBCEG, 1999). Using standard surveying techniques (Wilson, 1974), the grid was established using a Sokkisha 30" electronic theodolite, steel tape, optical square and survey pins. Levelling was carried out as for the longshore profile (Section 5.4).

According to Simm (1996), the beach plan shape is normally characterised by one or more longshore lines defined by tidal levels such as MHW. Therefore, this grid allowed variations in the MHW level to be monitored, long-term volume changes of sediment to be determined and areas of potential risk to be identified. In addition, the grid comprised four shore parallel and five cross-shore profiles which improved plan shape definition.

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Figure 5.1: Survey grid – Penarth foreshore

The first survey was undertaken in September 1997 and the last in September 2002. Surveys were carried out concurrently with the longshore profile (Section 5.4) and results tabulated and evaluated in Chapter 6: Penarth beach,

5.6 Beach Profiles – Penarth Foreshore

Following a review of the monitoring at the end of the first year (September 1998), it was decided that the programme should be extended. Consequently two cross-shore beach profiles were established along the foreshore fronting the Esplanade. These were extensions of the grid profiles located at stations 6 and 14 (Section 5.5; Figure 4.1). The cross-shore limits for each profile were the back beach and depth of closure. Using standard surveying techniques (see Sections 5.4 and 5.5) each profile was levelled at 10 m cross-shore intervals. The marl bedrock occupies much of the near shore zone and lower part of the inter-tidal foreshore with the mobile beach material terminating at a distinct line on the shore-platform (Section 2.2.2). Although it was argued that this represented the depth of closure, the profiles were extended seaward of this line in April 1999 and April 2000 to evaluate any variation. However, it will be seen in Chapter 6, that throughout the survey period, there was insignificant seaward variation in beach level beyond the depth of closure.

The first survey was undertaken in September 1998 and the last in September 2002 (Appendix 1). All surveys were carried out concurrently with foreshore grid surveys and this addition to the monitoring programme enabled better evaluation of cross-shore movement. This improved the analysis of long-term volume changes, sweep zones and plan shape definition. In conjunction with other survey results, evaluated in Chapter 6, better quantitative estimates of sediment transport volumes, rates and direction could be achieved.

5.7 Link Survey

Between station 26 and the mid point of the south face of groyne 6 (Figures 1.2 and 1.3), a 567m survey line was established to assess beach level change between the two orientations (NNE and NNW). This foreshore length is the least influenced by anthropogenic construction (Section 3.4) and the survey comprises three longshore ground positions: station 26 (750m), mid point (E) of manhole at orientation change (1245m) and mid point south face groyne 6 (1k; 1317m; Plate 5.1). These surveys would indicate differences in beach response to external change (natural or anthropogenic). A total of eight surveys were carried out between September 1997 and September 2002, coinciding with times of longshore profile and groyned beach surveys. Groyne 5 is located between the manhole and groyne 6. During the first surveys, this groyne, which is less substantial than groynes 6, 7, 8 and 9, did not demonstrate a level variation between north and south faces. It appeared that its construction had been linked to the sewer works (Plate 5.2) and consequently was not considered part of the groyned beach. Results are tabulated and evaluated in Chapter 6: Penarth beach.



Plate 5.1: North face groyne 6 (September 1997).



Plate 5.2: Sewer outfall construction (September 1997).

5.8 Groyned Beach – Penarth Head

The length of coastline monitored was a 160 m section immediately adjacent to the site of the Cardiff Bay Barrage. The bearing of this coastline is NNW and four groynes (6, 7, 8 and 9) extend from the cliff face to form three bays which successfully retain beach material (Figure 1.3). The Ordnance Survey plan did not show groynes 6, 8 or 9 and consequently their locations were established using traditional surveying techniques (Wilson, 1974). The groyne bays were of different widths: Bay 1 (45 m); Bay 2 (58 m), and Bay 3 (57 m). In addition, the groynes were of different lengths: groyne 6 (45 m), groyne 7 (44 m), groyne 8 (49 m) and groyne 9 (42 m).

As recommended by Simm (1996), lines were established either side of the groynes, together with the centre of the bays, in order to monitor beach response. Each survey comprised nine points per bay, i.e. top, middle and bottom of the beach, adjacent to each groyne and in the centre of the bay. Their positions were located using ranging rods and steel tape whilst the levelling was undertaken using a Leica automatic level and staff.

A temporary benchmark was established on a survey pin driven into the centre line of groyne 8. The level of this pin was determined from the Ordnance Survey Bench Mark on the Esplanade (9·01 m AOD) and reduced to 5·810 m AOD (± 5 mm). The initial survey of the groyne field itself was carried out in September 1997 and the final survey in September 2002. The survey period included the continuing construction and completion of the Barrage and breakwater (Plate 3.10), which ultimately resulted in the Rivers Ely and Taff being impounded.

A total of ten surveys were carried out at different times throughout the year. The varying survey times were caused by the construction of the Barrage and breakwater, which meant that access to the beach, was restricted. The location of the breakwater was such that on completion of the construction, groyne 9 became incorporated into the breakwater itself. Subsequent surveys were corrected for the new bay width of 43·3 m. In addition, Bay 3 was used to stockpile material during the construction process and 150 m³ was left on the beach. This volume, which was factored into the analysis, was made up of 120 m³ of sand/gravel and 30 m³ of larger rock fill (Boese, 1999. pers comm.). All results from the surveys are tabulated and considered in Chapter 7: Groyned Beach – Penarth Head.

5.9 Water Level and Wind Data

Although large amplifications of tides occur in the Severn Estuary/Bristol Channel (Spring Tidal Range up to 14 m), the basic driving forces of tidal movements are astronomical and therefore entirely predictable (Simm, 1996). Consequently, predicted tide levels were obtained from the Admiralty Tide Tables – ATT Port Ref. 514 (Cardiff).

The assessment of extreme water levels provides appropriate criteria for the design of coastal defences etc. As Simm (1996) recommended working from reliable published data, extreme water levels, including future trends, were taken from POL (1995) and SBCEG (1999). Actual sea level data was supplied by the British

Oceanographic Data Centre (BODC) as part of the function of the National Tidal and Sea Level Facility, hosted by the Proudman Oceanographic Laboratory and funded by the Department for Environment, Food and Rural Affairs (DEFRA) and the Natural Environment Research Council (NERC). This data was taken from a tidal gauge located at Newport whilst wind data, supplied by the Met Office, was taken from weather stations at Rhoose (NGR 3066E, 1677N) and St Athan (NGR 2998E 1683N). Monthly wind records, from January 1993 to December 2002, included mean wind speed and direction and maximum gust wind speed and direction. Wave conditions, identified by Simm (1996) and SBCEG (1999), completed information sources used to identify change and correlate environmental data to measured beach response. All data is tabulated and considered in detail in Chapter 8: Water and Wind.

5.10 Function Analysis

The methodology adopted was a modification of Cendrero & Fischer's (1997) procedure for assessing the environmental quality of coastal areas for planning and management. The adaptation by van der Weide et al (1999) and Micallef and Williams (2003; Section 4.3) was aimed at formulating a management strategy for increasing environmental quality and improving sustainability. Furthermore, it was argued that comparisons could be made between areas as part of an integrated coastal management strategy and that the method could be a simple tool to identify change. However, van der Weide et al (1999) identified weaknesses, which included a lack of available quantitative data on some of the parameters such as ecology, water quality and pollution. Consequently, to obtain a more reliable quantitative assessment and to reduce subjectivity, the present study along the Penarth coastline incorporated data from other agencies as part of an integrated management approach. These included Unitary Authority Structure Plans for planning and development designations, Shoreline Management Plans for land use and coastal environment, Blue Flag beach status and Good Beach Guide as indicative measures of beach and water quality, environmental designations such as AONB, cSAC, Heritage Coast and SSSI for

physical and ecological parameters and the Environment Agency for air pollution. Economic values were determined from potential for use and human well-being.

The assessment of the values was expressed numerically using an evaluation system. As Anastassova (1996) concluded, a qualitative Delphi approach is a forward thinking procedure which can use specialist opinion and data collection for good practice and evaluation. This technique was used to value the environmental and socioeconomic indicators. Van der Weide et al (1999) and Micallef and Williams (2003) used a scoring system of 1 to 3 for each indicator; 1 being the lowest (worst) and 3 being the highest (best). Both used the indicators established by Cendrero & Fischer (1997) but, only considered those that were relevant to the respective coastal environments. The methodology utilised by Van der Weide et al (1999) and Micallef and Williams (2003) involved normalising the score of the individual environmental (Table 5.1) and socio-economic components (Table 5.2) to give a parameter score between 0 and 1. Comparison is made by graphical illustration of normalised scores, allocated environmental and to human components. Overall conservation/development/use values were normalised by dividing the collective score of the parameters by the total possible score. The two figures can be plotted position graphically determine the distinct of Penarth on the Conservation/Development Matrix (Figure 4.1).

Sauer (1963) identified components of landscape whilst Cooke and Doornkamp (1990) evaluated Leopold's (1969) seminal work on landscape assessment where values ranging from a minimum score of 1 to a maximum score of 5 were allocated to environmental components. These considered the physical, biological and human-use functions of the natural environment in a similar manner to the methodology adopted in this work. Leopold's (1969) methodology for quantifying landscape, enabled for the first time, an economic value to be assigned to the environment. This could be subsequently weighted against development, on a cost-benefit basis (Tietenberg, 1994). Assessment methodologies developed for other areas of coastal management such as dune management (Davies *et al*, 1993 and Williams *et al*, 1995) and beach

aesthetics (Ergin *et al*, 2002 & 2003; Williams *et al*, 1993 & 2000; Leatherman, 1997) have also used a range of 1 to 5 to assess environmental components.

Most checklist and rating schemes used for coastal assessment are open to criticism with regard to subjectivity and weighting, particularly in aesthetics where the viewer's preferences and priorities tend to dominate (Leatherman, 1997). However, Leatherman (1998) later used a checklist approach to assess 'Americas Best Beaches'. Checklists can be extremely helpful, and may be used as an aid to planning and decision making (Leopold, 1969).

The Function Analysis adaptation by Van der Weide et al (1999) and Micallef (2003),and Williams was used as the method of conservation/development status of the Penarth coast. The environmental indicators assessed, are shown in Table 5.1 and the socio-economic indicators, in Table 5.2. These had been modified as appropriate for Penarth and a 1 to 5 scoring system applied to each indicator. The modifications included considering minerals and fishing as socio-economic components instead of environmental components. These latter changes helped redress a disadvantage highlighted by van der Weide et al (1999), that the methodology was weighted too heavily towards the environmental components. Results and evaluations are considered in Chapter 9.

Table 5.1: Environmental Indicators (Penarth)

Environment Component	Characteristic	Indicators	Characteristic Evaluation
Air	Pollution	Gravity	
		Visibility	
		Effect On Humans	
	Noise	Intensity	
Coastal Waters	Quality	Microbiological Pollution	
		Chemical Pollution	
	Aesthetic Condition	Turbidity	
		Litter	
Fresh Water	Supply	River Flow	
		Pollution	
		Turbidity	
		BOD	
Terrestrial Biota	Quality	Natural Vegetation Cover	
	Quantity	Biological Productivity	
		Biological Diversity	
		Species of Special Interest	
Marine Biota	Quantity	Biomass	
		Biological Productivity	
	Diversity	Biological Diversity	
		Species of Special Interest	
Geological & Topographical Features		Lithological	
		Geomorphology	
		Quality	
		Size of Bathing Area	
Hazards		Coastal Erosion	
		Coastal Flooding	
		Storms	
		Sedimentation	
		Cliff/ Slope Instability	
		Soil erosion	
		River Flooding	
Resources	Non Renewable	Soil (Fertility)	
	Renewable	Fish	
	Landscape	Visual Quality	
	Lanuscape	Uniqueness	
	<u> </u>	Omqueness	<u> </u>

Table 5.2: Socio-economic Indicators (Penarth)

Socio- economic Component	Characteristic	Indicator	Characteristic Evaluation
Human Aspects	Potential For Use	Historic, Artistic, Archaeological sites	
		Minerals, Rocks, Construction Materials, Fuels	
		Fishing	
		Recreation Facilities	
		Hotel, Restaurants	
		Utilities	
		Parking	
		Accessibility	
		Land Use – different types	
		Land Ownership	
		Extent of Development	
		Population Density	
		Intensity of Use	
	Human Well-being	Public Health	
		Unemployment	
		Education	
		Crime	
		Development Potential	

5.11 Risk Assessment

A hazard can be defined as something with the potential to do harm and risk is the probability of that harm occurring. The sea is a definite hazard to coastal infrastructure and consequently, a risk assessment was carried out on the Barrage and breakwater embankments. The method of risk assessment was the Reliability Based Design (REBAD) model (Balas and Ergin, 1997). Coastal structures should be evaluated to determine the sensitivity of the design to various parameters and to test

the survival probability of the structure over an appropriate time period and under given environmental conditions (Williams *et al*, 1998). Therefore, it is essential to evaluate the safety of rubble mound revetments by using probabilistic methods in which the design parameters are modelled as random variables. Balas and Ergin (1997) suggested such an approach with their REBAD model.

The model evaluates the safety and serviceability of coastal structures by modelling random resistance variables with probability distribution at the limit state (Balas *et al*, 1995). The serviceability limit state is implemented for safety evaluations of rubble mound breakwaters, as the exceedance of the failure damage level may not result in complete breakdown of the structure, but an interruption in the achievement of its functions (Ergin and Balas, 1998). The reliability of rubble mound breakwaters is evaluated in REBAD by employing the limit state equations generated from Hudson and van der Meer failure functions (van der Meer, 1988). Implementation of REBAD can be for both design and safety evaluations.

The variables used in deterministic design are:

- 1. Slope of the armour unit (cot α);
- 2. Mass density of the armour unit (ρ_s) kg/m³;
- 3. Mass density of sea water (ρ_w) kg/m³;
- 4. Stability Coefficient (K_D);
- 5. Damage Level (DL) %;
- 6. Placement of armour blocks;
- 7. Dimensions of armour blocks m;
- 8. Structure of armour blocks;
- 9. Breaking condition;
- 10. Design significant wave height (Hs) m;
- 11. Normal diameter of armour unit (D_{n50}) m;
- 12. Median weight of armour blocks (W₅₀) tonne.

These variables were obtained for both the Barrage and breakwater and input into the model. Output was discussed and interpreted as part of a management strategy for the groyned beach (Chapter 9).

5.12 Statistical Analysis

Data was analysed using both parametric and non parametric techniques. Non parametric tests can be used on interval data but are slightly less likely to detect a statistical difference or relationship if present, compared with parametric tests (Wheater and Cook, 2000). However, if data sets are small there is an inherent hazard of a type I error, that is, rejecting the null hypothesis when in fact it is true,

5.12.1 Parametric tests

Variables that are measured on continuous scales are often normally distributed (Wheater and Cook, 2000).

$$s = \sqrt{\frac{\sum (x - \overline{x})^2}{n - 1}} \text{ or } s = \sqrt{\frac{\sum x^2 - \frac{(\sum x)^2}{n}}{n - 1}}$$

where x is each data point

$$\frac{1}{x}$$
 is the mean of data $\left(\frac{\sum x}{n}\right)$

n is the sample size.

Data can be judged to be approximately normally distributed if it satisfies all the following conditions:

(i) the frequency distribution of the data looks similar to a normal distribution;

- (ii) about 68% of the data points lie in the range $\bar{x} \pm s$;
- (iii) almost all of the data lie within the range $\bar{x} \pm 3s$.

(Wheater and Cook, 2000: 56)

The t statistic for the paired t test was determined from

$$\overline{d} = \frac{\sum d}{n}$$

where d is the difference between each pair of data points n is the number of pairs of data points.

The standard error of the mean difference (SE_d) was subsequently found:

$$SE_{d} = \sqrt{\frac{\sum d^{2} - \frac{\left(\sum d\right)^{2}}{n}}{n(n-1)}}$$

The t statistic was determined from:

$$t_{calc} = \frac{\bar{d}}{SE_d}$$

Finally, the calculated t statistic (t_{calc}) for both independent and paired t tests was compared with tabulated t values (t_{tab}), according to degrees of freedom df in a test for significance, as follows:

If t_{calc} < t_{tab} the null hypothesis was accepted and there was no significant difference, but if t_{calc} > t_{tab} the null hypothesis was rejected and there was a significant difference.

5.12.2 Non parametric tests

Non parametric tests are used when data is not normally distributed. The Wilcoxon matched-pairs test involves calculating the differences between pairs of data points and ranking the differences in order of magnitude from small differences to large ones, ignoring the sign of the difference and any values of zero difference. The sign of the difference is then attached to the rank and the positive and negative values are summed separately (denoted T₊ and T₋ respectively). To test the null hypothesis, the lowest rank of the sums is compared to a table value of critical values and if it is equal or less than the critical value then the null hypothesis can be rejected. Another non-parametric test, the Mann-Whitney U test (U₁, U₂), was also used for data analysis. This test can be applied to normally distributed data but will be less powerful than the t test with the higher likelihood of accepting the null hypothesis when in fact it is false (type II error). To test the null hypothesis, the lowest rank is compared to a table value of critical values and if it is equal or less than the critical value then the null hypothesis can be rejected.

CHAPTER 6: PENARTH BEACH

6.1 Introduction

Chapter 1 identified changes along the Penarth coastline and provided the aims and objectives of the study. The literature review (Chapter 2) discussed current thinking with respect to the issues raised; whilst the physical background (Chapter 3) set the work in context of the integration of both physical and cultural environments. From a review of beach and coastal zone management (Chapter 4), in conjunction with the rationale, current thinking and physical background, the methodology (Chapter 5) was logically developed for monitoring the Penarth coast. This monitoring programme provided a tool to quantify and assess changes and to systematically ascertain future changes and trends. Effectively, 1,464m of the foreshore was monitored between September 1997 and September 2002, in three sections:

- Penarth beach;
- link survey;
- groyned beach adjacent to the Cardiff Bay Barrage.

This chapter analysed the results for Penarth beach and link survey and comprised:

- the longshore profile;
- foreshore grid;
- cross-shore profile;
- link survey.

The rationale for these surveys was discussed in Chapter 5 and results evaluated in the following sections.

6.2 Longshore Profile – Penarth Beach

Plates 6.1, 6.2 and 6.3 are aerial photographs of the study area whilst Figure 6.1, from the OS map, identified the locations of the twenty-six stations, at 30m intervals, which formed the 750m baseline. Surveys were carried out each April and September between September 1997 and September 2002, a total of eleven surveys. Table 6.1 gives the results of these surveys, detailing the beach level for

each station/longshore position along the baseline, and examples of the field data collection sheets are included in Appendix 1. The first survey in September 1997 provided information against which anecdotal perceptions of changes to the Penarth foreshore were evaluated as well as baseline data against which future changes would be monitored.

Plate 6.1: Aerial photograph south of lifeboat slipway (Source: Getmapping plc)

Plates 6.1, 6.2 and 6.3 have been removed from the digitized thesis for copyright reasons.

Plate 6.2: Aerial photograph lifeboat slipway to Penarth Pier (Source: Getmapping plc)

Plate 6.3: Aerial photograph north of Penarth Pier (Source: Getmapping plc)



Plate 6.4: Groyne 1 at high tide, September 1997.

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Figures 6.1 and 6.2 have been removed from the digitized thesis for copyright reasons.

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Figure 6.1: Stations 1 to 26, longshore profile, Penarth foreshore. (Source: Ordnance Survey)

Figure 6.2: MHW groynes 1 and 2, south of lifeboat slipway. (Source: Ordnance Survey)

Table 6.1: Temporal variation of longshore profile.

Sta- tion	Dist- ance (m)	Beach level (m) AOD												St Dev
(No)		Sep-97	Apr-98	Sep-98	Apr-99	Sep-99	Apr-00	Sep-00	Apr-01	Sep-01	Apr-02	Sep-02	Level (m)	(m)
1	0	4.695	4.255	4.495	4.760	4.345	4.050	4.415	4.530	4.655	4.775	4.850	4.530	0.248
2	30	4.585	4.210	4.13	4.155	4.130	4.015	4.075	4.210	4.305	4.355	4.435	4.237	0.169
3	60	4.000	3.735	3.615	3.575	3.785	3.710	3.675	3.770	3.700	3.735	3.740	3.731	0.109
4	90	3.315	3.370	3.330	3.705	3.440	3.325	3.290	3.560	3.440	3.54	3.615	3.448	0.139
5	120	4.395	4.435	4.000	3.755	3.900	3.645	3.740	3.565	3.505	3.475	3.470	3.808	0.345
6	150	4.125	4.090	4.025	3.795	3.835	3.565	3.710	3.575	3.535	3.555	3.595	3.764	0.227
7	180	3.710	3.790	3.870	3.775	3.775	3.505	3.675	3.645	3.605	3.675	3.610	3.694	0.104
8	210	3.495	3.475	3.570	3.620	3.615	3.660_	3.660	3.420	3.500	3.605	3.665	3.571	0.085
9	240	3.335	3.170	3.320	3.500	3.320	3.450	3.475	3.290	3.255	3.375	3.365	3.350	0.098
10	270	3.115	2.970	3.000	3.165	2.980	3.060	3.175	3.045	3.150	3.125	3.130	3.083	0.075
11	300	2.850	2.620	2.755	2.845	2.675	2.760	2.870	2.795	2.815	2.870	2.925	2.798	0.090
12	330	2.480	2.345	2.550	2.660	2.545	2.625	2.635	2.595	2.705	2.720	2.710	2.597	0.113
13	360	2.195	2.130	2.195	2.330	2.255	2.495	2.410	2.395	2.560	2.525	2.475	2.360	0.149
14	390	2.005	1.885	2.180	2.010	1.980	2.070	2.110	2.105	2.180	2.200	2.215	2.085	0.106
15	420	1.905	1.840	1.930	1.935	1.870	1.900	1.905	1.930	1.950	1.985	2.105	1.932	0.069
16	450	2.110	2.005	2.075	1.905	1.870	1.855	1.880	1.960	1.895	1.955	2.010	1.956	0.085
17	480	1.550	1.500-	1.580	1.540	1.605	1.590	1.475	1.550	1.600	1.625	1.630	1.568	0.050
18	510	1.205	1.155	1.275	1.300	1.410	1.345	1.235	1.450	1.480	1.475	1.505	1.349	0.124
19	540	1.035	1.000	1.155	1.290	1.395	1.210	1.155	1.325	1.365	1.390	1.410	1.248	0.147
20	570	1.395	1.285	1.460	1.3 15	1.510	1.455	1.375	1.540	1.495	1.555	1.625	1.455	0.105
21	600	1.125	1.050	1.130	1.080	1.170	1.085	1.190	1.230	1.295	1.315	1.340	1.183	0.101
22	630	1.140	1.015	1.120	1.085	1.335	1.225	1.205	1.255	1.275	1.325	1.405	1.217	0.118
23	660	1.080	0.955	1.085	1.100	1.130	1.035	1.010	1.125	1.300	1.495	1.615	1.175	0.208
24	690	1.740	1.160	1.265	1.355	1.305	1.250	1.220	1.300	1.540	1.720	1.955	1.437	0.261
25	720	2.560	2.340	2.520	2.275	2.495	2.370	2.300	2.265	2.365	2.530	2.625	2.422	0.127
26	750	2.520	2.275	2.430	2.130	2.425	2.390	2.290	2.460	2.385	2.445	2.575	2.393	0.124

Table 6.1: Temporal variation of longshore profile (Continued).

Sta- Dist-	Beach level (m) AOD												St Dev
(No) (m)	Sep-97	Apr-98	Sep-98	Apr-99	Sep-99	Apr-00	Sep-00_	Apr-01	Sep-01	Apr-02	Sep-02_	Level (m)	(m)
Time (Months)	0	7	12	19	24	31	36	43	48	55	60		
Mean O/A	2.603	2.464	2.541	2.537	2.542	2.486	2.506	2.534	2.571	2.629	2.677	2.554	0.063
Mean Stns 1-5	4.198	4.001	3.914	3.990	3.920	3.749	3.839	3.927	3.921	3.976	4.022	3.951	0.113
Mean Stns 6-15	2.922	2.832	2.940	2.964	2.885	2.909	2.963	2.880	2.926	2.964	2.980	2.924	0.045
Mean Stns 16-26	1.587	1.431	1.554	1.489	1.605	1.528	1.485	1.587	1.636	1.712	1.790	1.582	0.104

South of the old lifeboat slipway, stations 1 to 6 (0 to 150m longshore), Figure 6.1 showed the line of the Mean High Water (MHW) and its alignment along the foreshore. A section of this OS map between stations 1 and 6, shown in Figure 6.2, clearly demonstrated that the beach level, south of groyne 1 was higher than between groynes 1 and 2. Wallingford (1992) stated:

"There are two concrete groynes south of the slip and all three structures have had some effect at retaining higher beach levels on their southern flank."

(Wallingford, 1992: 3)

However, comparison with Plate 6.4 (September 1997), showed groyne 1 at high tide, and this clearly demonstrated beach levels south of the groyne were lower than to the north. Therefore, the position of the MHW had changed.

Figure 6.3, produced from Table 6.1, showed the September 1997 longshore profile. At distance 0 to 30m the profile showed rising southward beach levels, opposite to the OS line of the MHW between groynes 1 and 2, whilst Plate 6.5, the northern flank of groyne 1, illustrated a transition in the beach formation process. Section 1.2.1 highlighted erosional problems along this section with the exposed foundations of the southern flank of the Yacht Club slipway. Clearly the survey provided evidence of change along this section of the beach and suggested a reverse of the accepted south to north longshore drift.

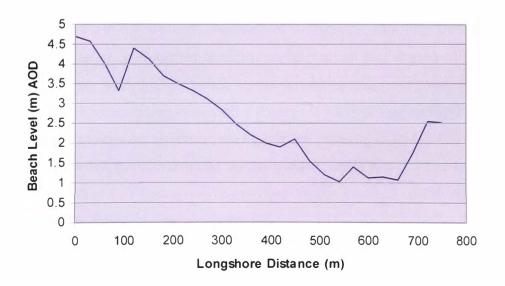


Figure 6.3: Longitudinal Profile, September 1997.



Plate 6.5: Northern flank groyne 1, September 1997.

The longshore profile for the next beach section (Figure 6.3), the old lifeboat slipway to Penarth Pier, stations 6 to 15 (longshore distance 150m to 420m), showed a northerly fall in level of 2.220m between the slipway and pier (Table 6.1). If the slipway was acting as a groyne, increased beach level at the slipway would again be consistent with a southern regime of sediment transport. Section 1.2.2 demonstrated a loss of beach material between 1992 and 1997 as well as a change in composition of beach material. The effects of this loss are highlighted in Plate 6.6 with the exposure of a Victorian cast iron storm water sewer. In addition, Wallingford (1992) reported that the pier acted as a partially effective groyne with higher beach levels to the south, consistent with the traditional south to north longshore drift. Figure 6.3 showed that although the pier still acted as a groyne, beach levels rose by 0.205m (stations 15 to 16; 420 to 450m longshore) between the southern and northern sides (Table 6.1). Once again this length of coastline had undergone change and similar to south of the slipway, a north to south sediment regime was indicated.



Plate 6.6: Exposed Victorian cast iron sewer

North of Penarth Pier, large areas of red/green marl were exposed, as documented in Section 1.2.3. Once again beach material to the south had eroded, evidenced by the exposed foundations of the slipway beneath the multi-storey car park and undermining by the sea of the columns and seawall. Figure 6.3 indicated a relatively uniform beach level along this section (600 to 660m longshore), explained by no beach cover to the exposed marl bedrock. The concrete culvert for the 1m diameter sea outfall (Plate 6.7) had higher beach levels on the northern flank (maximum level difference 0·750m) but, when completed in July 1997, beach levels were left approximately the same on both sides (Bright, 1997: pers comm). Further reference to Section 1.2.3 illustrated loss of beach material fronting an old quay but the threat of the wall being outflanked (Wallingford, 1992) was reduced due to a build-up of material to the north of groynes 3 and 4 (Table 6.1). Plate 6.8 shows the remains of the timber groyne 3 and concrete groyne 4. The pebble covering to the beach, north of the pier, was generally very thin, the exception being between the outfall and groyne 4, approximately 700 to 750m longshore.



Plate 6.7: Concrete culvert for 1m diameter sea outfall.



Plate 6.8: Groynes 3 and 4.

Examination of groyne 4 (Plate 6.9) showed higher beach levels on the northern flank and similar to groyne 1, there was a change in position of the MHW. Once again both Bullen (1993a) and Wallingford (1992) agreed that this section of the coastline was subject to a net northward drift of beach material with frequent

references made to retention of material on southern flanks of structures. Wallingford (1992) and Bullen (1993a) both further highlighted damage suffered by coastal defences along this section. There was no doubt that increased loss of beach material was further undermining sea defences and evidence justified perceptions of change. Similar to the rest of the beach, a southward drift of beach material had been shown.



Plate 6.9: Groyne 4, September 1997.

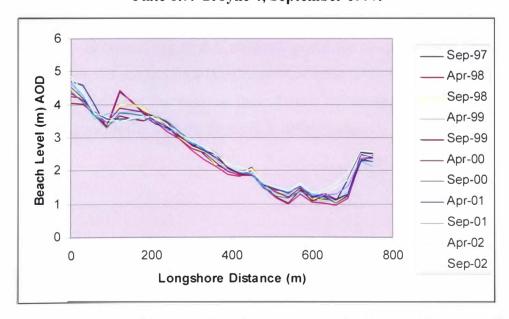


Figure 6.4: Temporal variation of longshore profiles (Sep 1997-Sep 2002).

Consequently, the results of this first survey demonstrated clear changes and a *reverse* of accepted longshore sediment transport between 1992 and 1997 with anecdotal evidence further reducing the timescale of change from 1995. Table 6.1 detailed all survey results, mean beach levels, and standard deviations. These survey results, shown graphically in Figure 6.4, identified temporal variations relative to the profile and data established in this first survey. However, to identify trends over the five-year monitoring period, it was considered more appropriate to evaluate the beach in three distinct sections:

- south of the old lifeboat slipway;
- old lifeboat slipway to Penarth Pier;
- Penarth Pier to groyne 4.

6.2.1 South of the old lifeboat slipway

This section of the beach, is shown in Figure 6.1 and Plate 6.1 and included stations 1 to 6 (0 to 150m longshore). Figure 6.5, produced from Table 6.1, further detailed the beach profiles for the eleven surveys where it was clearly seen that the beach level at station 2 (30m), between groynes 1 and 2 (Plate 6.10), was at its highest in September 1997. At that time beach levels were similarly high at station 5 (120m), between the old and new lifeboat slipways but conversely, near their lowest at station 4 (90m), the location of the exposed foundations of the Yacht Club slipway, (Section 1.2.1). However, Figure 6.5, although useful for an overview of changing profiles, did not facilitate identification of trends. Consequently, as the first survey provided the baseline data, Figure 6.6 compared each successive longshore profile with the September 1997 position.

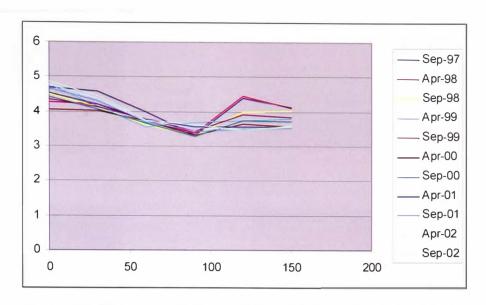


Figure 6.5: Temporal variation of longshore profile stations 1 to 6.



Plate 6.10: Station 2, September 1997.

Between September 1997 and April 1998, beach levels fell between stations 1 and 4 (0 to 90m) whilst remaining relatively constant to station 6 (150m), with a consequent average loss of 0·197m (Table 6.1). By September 1998, overall beach levels along the section had fallen still further to an average 0·284m below September 1997 but more significantly, had decreased between stations 4 and 6 (90m to 150m). In April 1999, mean beach levels had risen to 0·208m below September 1997 and the level peak and trough between stations 3 and 6 (60 to 150m) was replaced by a more uniform profile. Between stations 1 and 3 (0 to 60m) groynes 1 and 2 influenced longshore transport but the rising south to north gradient between stations 3

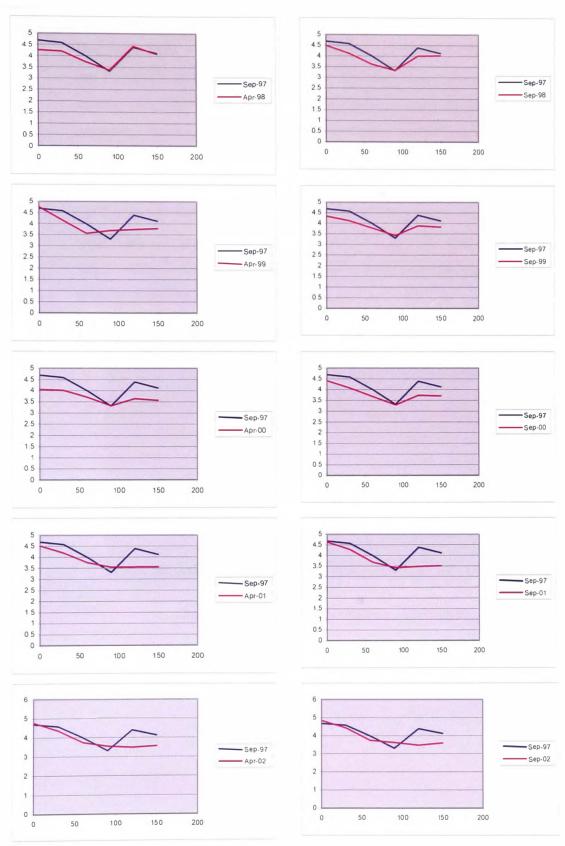


Figure 6.6: Temporal comparison of profiles (varying timescales) with September 1997 (0 to 150m).

and 6 (60 to 150m) may have indicated a return to a south to north longshore drift. The effects of increased beach levels at station 4 (90m) can be clearly seen in Plate 6.11 where the foundations of the Yacht Club slipway were no longer visible. The return to a south to north sediment transport system was further supported by beach levels being clearly higher on the southern flanks of groynes 1 and 2 respectively (Plates 6.12 and 6.13) and Plate 6.14 the old lifeboat slipway looking northwards towards the Esplanade. Plate 6.15 of the northern flank of groyne 1, when compared with Plate 6.5, highlighted definite beach loss and this was such that the steps on the northern flank of the groyne were suspended approximately 0.150m above the beach surface (Plate 6.16).



Plate 6.11: Yacht Club slipway, April 1999.



Plate 6.12: Groyne 1, April 1999.



Plate 6.13: Groyne 2, April 1999.



Plate 6.14: Old lifeboat slipway, April 1999.



Plate 6.15: Northern flank groyne 1, April 1999.



Plate 6.16: Steps groyne 1, April 1999.



Plate 6.17: Yacht Club slipway, September 1999.



Plate 6.18: Exposed slipway foundations, September 1999.

However, by September 1999, average beach levels along this section had once again fallen to 0.278m below September 1997. Most significantly, the loss of beach material at station 4 (90m) led to the re-exposure of the foundation of the Yacht Club slipway (Plates 6.17 and 6.18). At this time the barrage breakwater had been completed and work had started on the final closure of the Cardiff Bay Barrage. By April 2000, average beach levels along this section had fallen to their lowest point during the monitoring period, 0.449m below September 1997 (Table 6.1). September 2000 saw average beach levels increase to 0.359m below September 1997 and this was the start of a consistent rise in average beach level: 0.271m (April 2001), 0·277m (September 2001), 0·222m (April 2002) and 0·176m (September 2002) below September 1997 levels. However, it was not until April 2001 that the foundations to the Yacht Club slipway were once again covered. Further examination showed that from April 2001, there was less profile variation. It appeared that beach processes had become more uniform and by September 2002, as measured over the five-year period, the beach level at station 1 was at its highest, whilst the level at station 6 (150m) was almost at its lowest value, (Table 6.1). Plates 6.19 and 6.20 show that beach levels were once again higher on the southern flanks of groynes 1 and 2 respectively whilst Plate 6.21 showed the marl bedrock exposure on the northern flank of groyne 2. The Yacht Club slipway foundations were no longer exposed (Plates 6.22 and 6.23) and Plate 6.24

demonstrated the effects of the overall reduction in level at stations 5 and 6 (120m and 150m longshore). This resulted in a more uniform gradient across the old lifeboat slipway which, by then, had partially collapsed. Furthermore, over the five years, the beach level at station 5 (120m) varied the most compared with any other station over the whole 750m baseline, that is, 0.925m with the next largest variation 0.535m at station 23 (660m: Table 6.1). Therefore, by September 2002, beach profiles and photographic evidence were more consistent with a return to the traditional south to north sediment transport rather than the north to south movement shown in the earlier surveys.



Plate 6.19: Groyne 1 September 2002.



Plate 6.20: Groyne 2 September 2002.



Plate 6.21: Exposed marl northern flank groyne 2, September 2002.



Plate 6.22: Upper Yacht Club slipway, September 2002.



Plate 6.23: Lower Yacht Club slipway, September 2002.



Plate 6.24: Old lifeboat slipway, September 2002.

To test whether there was any statistical significance to these qualitative observations, the independent t test was carried out on the beach levels in September 1997 and September 2002. This test was appropriate as all stations were at approximately the same cross-shore position. Results of the analysis showed that there was no significant difference between the beach levels in September 2002 compared with September 1997 (t = 0.840; df = 10; p > 0.05) and there was no quantitative evidence to support the qualitative changes.

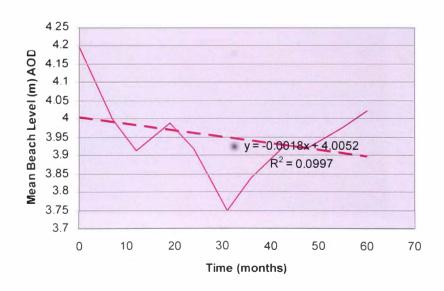


Figure 6.7: Temporal variation in mean beach level - 9/97 to 9/02

Consequently, temporal trends were evaluated for this part of the beach and Figure 6.7, produced from Table 6.1, illustrated the temporal variation of The regression equation y = -0.0018 x + 4.0052the mean beach level. indicated a trend of falling beach levels over the monitoring period but the regression analysis did not explain much of the temporal variation (Coefficient of Determination, $R^2 = 9.97\%$). However, over the five years there was evidence of a change in direction of longshore transport and from Figure 6.7 this appeared to coincide with the results of the April 2000 survey (Month 31). This survey date was also significant in that it was the first survey undertaken following completion of the Cardiff Bay Barrage (4th November, 1999). Therefore, Figures 6.8 and 6.9 represents the variation of the mean beach level between September 1997 and April 2000, and April 2000 and September 2002 respectively. From Figure 6.8, the regression equation y = -0.0114x + 4.1382 explained a high percentage (R²) = 78.09%) of the temporal decrease in mean beach level over that time period. The same analysis for the period April 2000 to September 2002 (Figure 6.9) gave a regression model y = 0.0085x + 3.5167 which explained an even higher percentage ($R^2 = 93.11\%$) of the temporal increase in mean beach level over that period.

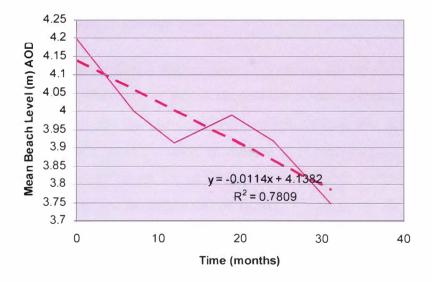


Figure 6.8: Temporal variation in mean beach level - 9/97 to 4/00

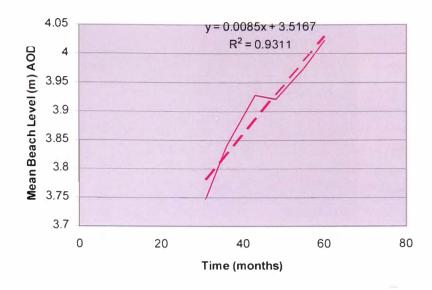


Figure 6.9: Temporal variation in mean beach level - 4/00 to 9/02

Both analyses provided quantitative evidence to support a change in direction of longshore sediment movement and due to the significance of the temporal regression models, suggested that during each of these periods there was relative uniformity of coastal processes. Furthermore, there was a possibility that the completion of the barrage had an impact on coastal processes along this section of the baseline.

6.2.2 Old lifeboat slipway to Penarth Pier

This section included stations 6 to 15 (150m to 420m) and is shown in Figure 6.1 and Plate 6.2. Figure 6.10, produced from Table 6.1, detailed the beach profiles for the eleven surveys which showed that the largest variations in beach level occurred at stations 6 (150m) and 13 (360m). Once again, although useful for an overview of changing profiles, Figure 6.10 did not facilitate identification of trends. Similar to the process adopted in Section 6.2.1, each successive survey was compared to the September 1997 profile and these comparisons were shown in Figure 6.11.

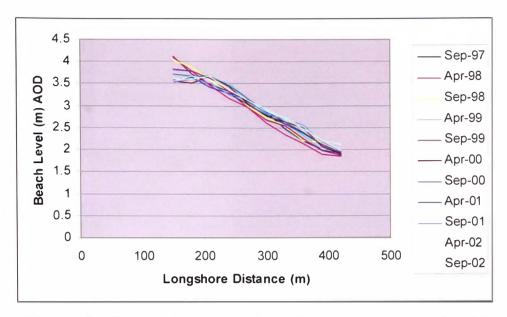


Figure 6.10: Temporal variation of longshore profile stations 6 to 15.

Between September 1997 and April 1998, with the exception of station 7 (180m), beach levels fell along this part of the foreshore by an average of 0·090m (Table 6.1). This coincided with the lowest recorded mean beach level during the five-year investigation (2·832m). Plate 6.25, at that time, showed the further exposure of the cast iron storm water sewer seen in Plate 6.6 and the Penarth Times (1998) highlighted concerns, reporting that Councillors believed that dredging may be to blame. However, by September 1998, overall beach levels along this section had *risen* to an average 0·018m above September 1997 (Table 6.1).

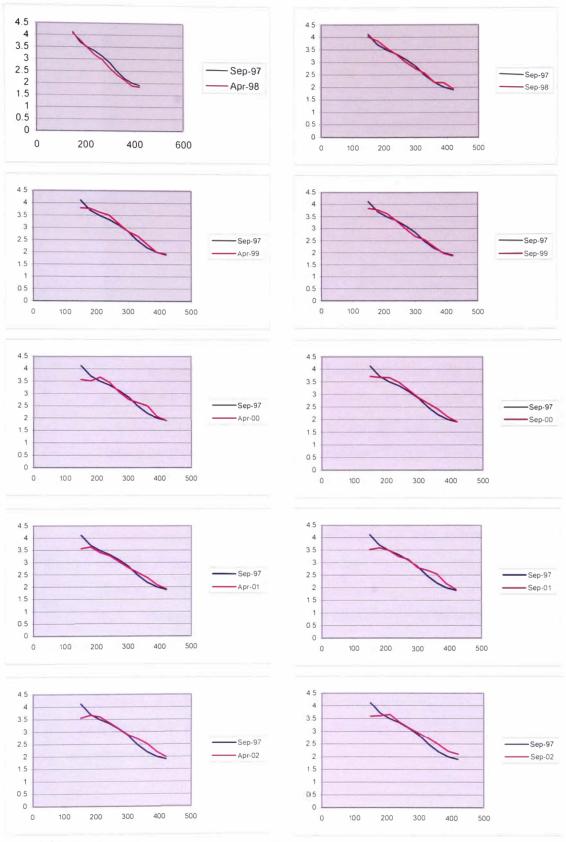


Figure 6.11: Temporal comparison of profiles (varying timescales) with September 1997 (150 to 420m)



Plate 6.25: Victorian sewer, April 1998.

In April 1999 mean beach levels had risen still further to 0·042m above September 1997 but significantly beach levels had fallen by 0·330m at station 6 (150m). The effects of this fall were seen when comparing Plate 6.26 (September 1998) with Plate 6.27 (April 1999). Conversely, at the northern end of this section, beach levels had risen and once again, comparison of Plates 6.28 (September 1997) and 6.29 (April 1999) showed increased nourishment on the part of the beach that had caused the initial erosion concerns. Similar to what had occurred south of the lifeboat slipway, there was an indication of a return to a south to north longshore drift. This situation highlighted the problems faced by beach managers and decision-makers in avoiding hasty decisions when developing strategies to manage the coastal zone. Although anthropogenic activities previously blamed for erosion of the beach were ongoing, that is, the barrage construction and marine aggregate dredging, the beach indicated signs of recovery.



Plate 6.26: Station 6, old lifeboat slipway, September 1998.

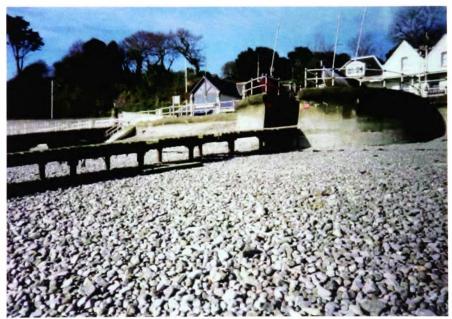


Plate 6.27: Station 6, old lifeboat slipway, April 1999.



Plate 6.28: Esplanade steps, station 14, September 1997.

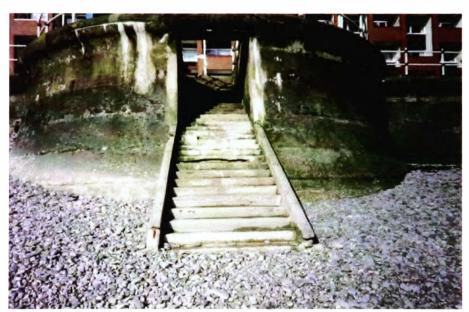


Plate 6.29: Esplanade steps, station 14, April 1999.

However, by September 1999, average beach levels had once again fallen to 0.037m below September 1997 (Table 6.1). Comparison of photographs of the beach adjacent to the old lifeboat slipway clearly showed increased beach levels at this location. Plates 6.30 (April 1999) and 6.31 (September 1999) showed the beach adjacent to the old lifeboat slipway and clearly demonstrated increased beach levels at this location. Note particularly the base of the mast adjacent to the seawall. This change was similar to that

discussed south of the old lifeboat slipway (Section 6.2.1) and as beach levels between stations 10 and 15 (270m to 420m) had fallen significantly during this period (average 0·100m/station: Table 6.1), it indicated a possible return to a southerly sediment transport.

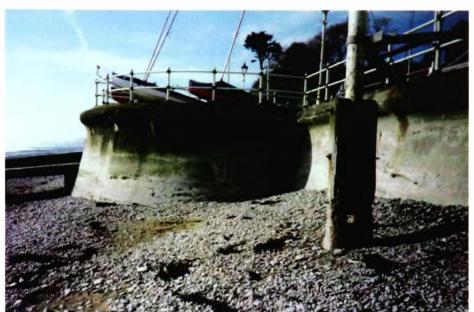


Plate 6.30: Old lifeboat slipway, April 1999.



Plate 6.31: Old lifeboat slipway, September 1999.

April 2000 saw beach levels increase to 0.013m below September 1997 (Table 6.1). This was the first survey following completion of the barrage and from Figure 6.11 it was seen that although beach levels had

decreased at station 6 (150m), since September 1997 there was a more uniform distribution of increased beach levels along the foreshore. In September 2000 beach levels had increased to 0·041m above September 1997 but by April 2001 they had once again fallen to 0·042m below September 1997 values (Table 6.1). Although there was an overall loss since September 2000, the most significant losses occurred between stations 6 to 12 (150m to 330m longshore), the southern part of the section.



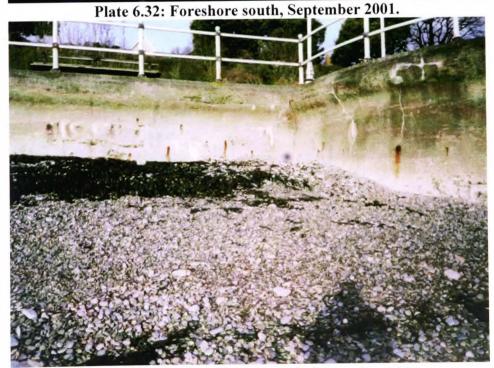


Plate 6.33: Back beach station 10, September 2001.



Plate 6.34: Foreshore north, September 2001.



Plate 6.35: Esplanade steps, station 14, September 2001.

September 2001 once again saw an increase in mean beach level to 0.004m above September 1997, whilst by April 2002 there were significant increases between stations 10 and 15 (270m to 420m longshore: Figure 6.11; Table 6.1). Plate 6.32 showed the results of decreased levels at station 6 (Figure 6.11) which was similar to beach conditions in April 1999 (Plate 6.30). The increased beach levels between stations 10 and 15 (mean level 0.042m above September, 1997) were illustrated in Plate 6.33, taken

adjacent to station 10 (270m), Plate 6.34 looking northwards towards the pier, and Plate 6.35 at the steps adjacent to station 14 (390m). These provided stark contrast to the beach condition in September 1997 (Plate 6.28).



Plate 6.36: Penarth foreshore, September 2002.

The final survey in September 2002 saw the mean beach level increase to the highest recorded during the five-year monitoring period; 0.058m above September 1997 (Table 6.1). Interestingly, comparison of Plate 6.36 with Plate 1.2 (Bullen, 1993a) indicated that the beach was recovering, conditions being more like September 1992 rather than September 1997. Further examination of Figure 6.11 indicated that between September 1997 and September 2002, beach levels had increased over the northern half of this section, which further supported a return to the traditional south to north longshore sediment movement.

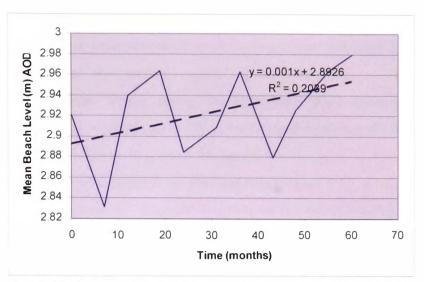


Figure 6.12: Temporal variation in mean beach level - 9/97 to 9/02

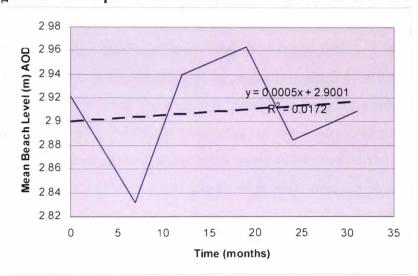


Figure 6.13: Temporal variation in mean beach level - 9/97 to 4/00

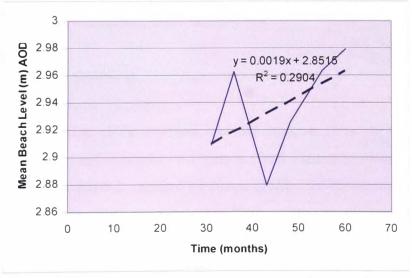


Figure 6.14: Temporal variation in mean beach level - 4/00 to 9/02

These qualitative observations were tested statistically using the independent t test, between beach levels in September 1997 and September 2002. Results of this analysis again showed there was no significant difference between beach levels in September 2002 compared with September 1997 (t = 0.201; df = 18; p > 0.05) and consequently, there was no quantitative evidence to support the qualitative changes. As with south of the slipway, temporal trends were evaluated and Figure 6.12, produced from Table 6.1, illustrated the temporal variation of the mean beach level. The regression equation y = 0.001x + 2.8926 indicated a trend of rising beach levels over the monitoring period but regression analysis did not explain much of the temporal variation ($R^2 = 20.39\%$). Subsequently, Figures 6.13 and 6.14 were produced representing the variation of the mean beach level between September 1997 and April 2002, and April 2000 and September 2002 respectively. From Figure 6.13 the regression equation y = 0.0005x +2.9001 explained little of the temporal increase in mean beach level over that time period ($R^2 = 1.72\%$) and indicated that there was little consistency in prevailing coastal processes. The same analysis for the period April 2000 to September 2002 (Figure 6.14) gave a regression model y = 0.0019x +2.8515, which explained a higher percentage ($R^2 = 29.04\%$) but still did not reflect consistency. Both analyses provided evidence of increased beach levels contrary to the September 1997 position but, unlike south of the slipway, there was no firm indication that the completion of the barrage may have had any influence on the beach levels.

6.2.3 Penarth Pier to Groyne 4

This section of the baseline included stations 15 to 26 (420m to 750m longshore) and is shown in Figure 6.1 and Plate 6.3. Figure 6.15, produced from Table 6.1, detailed the beach profiles for the eleven surveys between September 1997 and September 2002. It was clearly seen that the largest variations in beach level were at station 19 (540m), approaching the multi-storey car park and station 23 (660m) mid-way between the end of the car park and sewage outfall, 0·410m and 0·660m respectively (Table 6.1). Similar to the beach fronting the Esplanade, the survey in April 1998

documented the lowest beach levels, whilst the final survey in September 2002 indicated the highest recorded beach levels and potential beach recovery. Although it gave an overview of changing profiles, Figure 6.15 did not enable detailed analysis of trends. To facilitate analysis, as for the two previous sections of the baseline, each successive survey was compared to the September 1997 profile and shown in Figure 6.16.

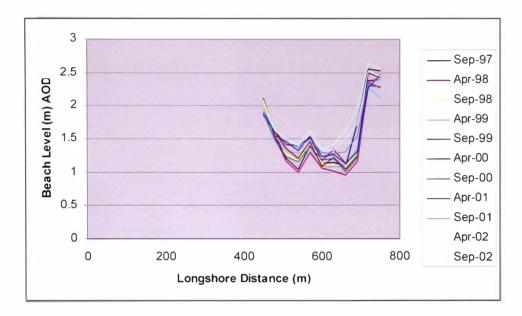


Figure 6.15: Temporal variation of longshore profile stations 15 to 26.

Without exception, between September 1997 and April 1998, beach levels fell along the whole of this section of the coastline by an average of 0·156m to 1·431m (Table 6.1), the lowest recorded during the five-year investigation. Immediately north of the pier, Plate 6.37 indicated effects of beach loss with the exposure of the 0·300m diameter surface water outfall. Similar to the other storm water sewers, these Victorian pipelines were originally constructed to discharge below the surface of the beach. Plate 6.38 looked northwards from station 15 (420m) and detailed lack of pebble covering beneath the pier, whilst Plate 6.39 shows the multi-storey car park and damaged sea defences, exposed marl bedrock and the concrete sewer culvert constructed the previous year.

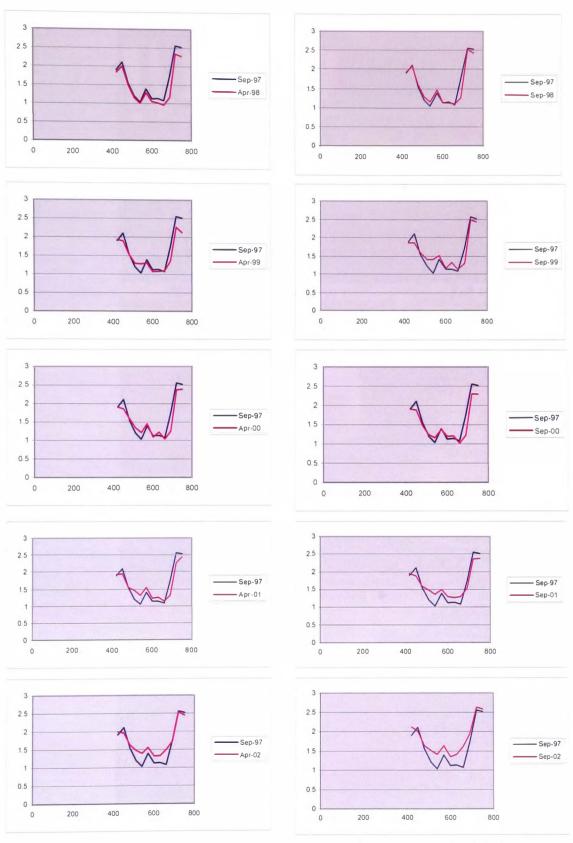
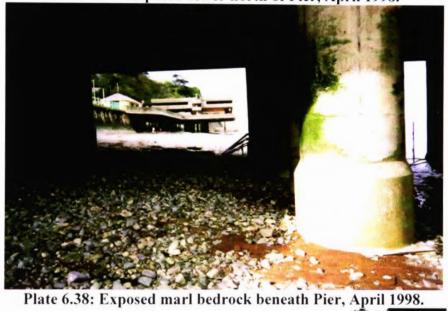


Figure 6.16: Temporal comparison of profiles (varying timescales) with September 1997 (420 to 750m)



Plate 6.37: Exposed sewer north of Pier, April 1998.



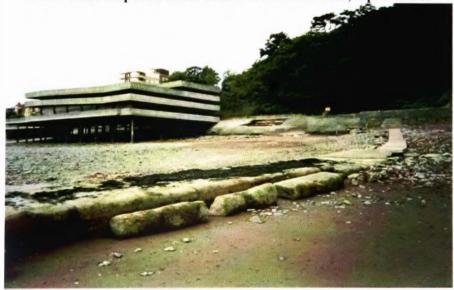


Plate 6.39: Damaged sea defences and concrete culvert, April 1998.

Moving northwards to the end of the baseline, the old timber groyne 3 and concrete groyne 4, similar to conditions in Plate 6.8 beach covering was good and afforded protection to the cliffs, evidenced by the accumulation of weathered material at the cliff toe. By September 1998, average beach levels had risen to 0.033m below September 1997 (Table 6.1) but fell once again in April 1999 to 0.098m below September 1997 (Table 6.1). Significantly, however, beach levels north of the pier (station 16; 450m) were now lower than immediately south (station 15; 420m). This agreed with conditions recorded by Wallingford (1992) and indicated a possible return to a south to north longshore drift. At that time, beach levels (Table 6.1) reached their lowest values at stations 25 (720m) and 26 (750m).

In September 1999, contrary to what had occurred with the previous two sections of the baseline, average beach levels increased to 0.018m above September 1997 (Table 6.1). However, at either end of the section, that is, stations 15 to 16 (420m to 450m) and stations 24 to 26 (690m to 750m) beach levels had fallen (Table 6.1). Comparison of Plate 6.37 with Plate 6.40 showed the effects of this beach loss with the further exposure of the storm water sewer, which was suspended above the beach surface. Conversely, between stations 17 and 23 (480m to 660m) there was a significant rise in beach level either side of the multi-storey car park (Plate 6.41). Similar to the previous sections, this possibly indicated a return to a north to south sediment movement. April 2000 saw a fall in average beach levels to 0.059 below September 1997 (Table 6.1), although once again beach levels south of the pier were higher than to the north. Average beach levels in September 2000 again fell to 0·102m below September 1997, caused mainly by losses between stations 24 and 26 (690m to 750m), whilst from this time onwards, average beach levels continued to rise with successive surveys 0m (April 2001); + 0.049m (September 2001); +0.125m (April 2002) and + 0.203m (September 2002) relative to September 1997 (Table 6.1). These latter surveys showed that beach levels generally increased along the length of this section.



Plate 6.40: Exposed sewer north of Pier, September 1999.



Plate 6.41: Beach to the north of Pier, September 1999.



Plate 6.42: Exposed sewer north of Pier, April 2002.



Plate 6.43: Beach to the north of Pier, April 2002.

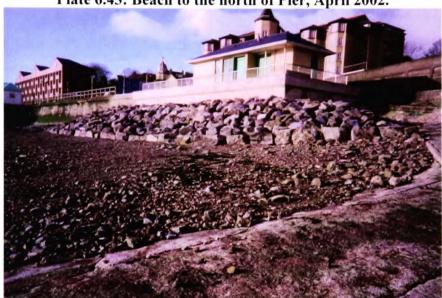


Plate 6.44: Old multi-storey slipway looking south, April 2002.



Plate 6.45: Old multi-storey slipway looking north, April 2002.

Furthermore, beach levels south of the pier were higher than to the north, which was the case for the other structures. This qualitatively indicated a return to a traditional south to north longshore transport of beach material.

Plate 6.42 (April 2002) showed increased beach levels near the storm sewer, shown in Plate 6.40, whilst Plates 6.43, 6.44 and 6.45 showed the beach following the demolition of the multi-storey car park at the end of 2001. Significantly, there was little indication of the exposed marl bedrock which had been predominant in September 1997. In September 2002, beach levels had further recovered, as evidenced beneath the pier by a comparison of Plates 6.46 and 6.38 and Plate 6.47 of the surface water sewer compared with Plate 6.40. Plate 6.48 of the sewer outfall and Plate 6.49 of groyne 4 compared with Plate 6.9, showed higher beach levels on southern flanks.



Plate 6. 46: Beach beneath Pier, September 2002.

As for the two other sections of the baseline, qualitative observations were tested statistically using the independent t test, between beach levels in September 1997 and September 2002. Results of the analysis showed that there were again no significant differences between the beach levels in September 2002 and September 1997 (t = 1.038; df = 22; p > 0.05).

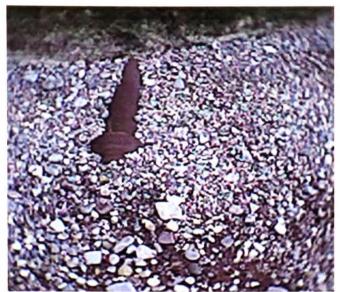


Plate 6. 47: Exposed sewer north of Pier, September 2002.



Plate 6.48: Concrete culvert, September 2002.



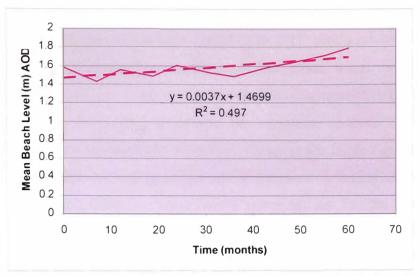


Figure 6.17: Temporal variation in mean beach level - 9/97 to 9/02.

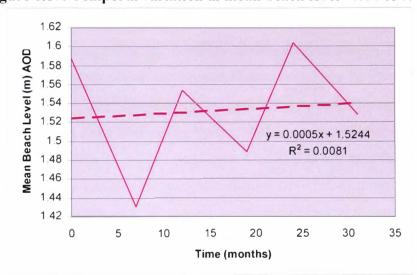


Figure 6.18: Temporal variation in mean beach level - 9/97 to 4/00.

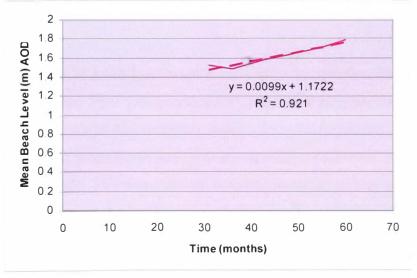


Figure 6.19: Temporal variation in mean beach level – 4/00 to 9/02.

Subsequently, as for the two previous sections, temporal trends were evaluated for this part of the beach and Figure 6.17, produced from Table 6.1, illustrated the temporal variation of the mean beach level. The regression equation y = 0.0037x + 1.4699 indicated a trend of rising beach levels over the monitoring period and the regression analysis explained nearly half of the variation ($R^2 = 49.7\%$). This was the highest correlation along the baseline but was probably due to this section having the lowest beach levels and thinnest overall beach covering. As for the previous sections, Figures 6.18 and 6.19 were produced representing the variation of mean beach level between September 1997 and April 2000, and April 2000 and September 2002 respectively. From Figure 6.18, the trendline y =0.0005x + 1.5244 explained little of the temporal increase in mean beach level over that time period ($R^2 = 0.81\%$) although interestingly, it was the same gradient as this period for the old slipway to the pier (0.0005: Figure 6.13). However, little consistency in prevailing coastal processes was indicated. From April 2000 to September 2002 (Figure 6.19) the regression model y = 0.0099x + 1.1722 explained a significant percentage of the variation of mean beach level along this section ($R^2 = 92.1\%$). Furthermore this reflected consistency of coastal processes during this time, which coincided with the completion of the barrage. All analyses along this section had indicated a trend of increased beach levels.

6.2.4 Overview

Changes in beach level were evident over the five-year monitoring period (September 1997 to September 2002) but a paired t test comparison for the beach levels at the twenty-six stations between September 1997 and September 2002 showed that there was no significant difference (t = 0.250; df = 25; p > 0.05). This was expected because the same analysis on each of the three sections of the baseline had yielded similar outcomes. However, there was qualitative and verifiable evidence of a north to south longshore drift in September 1997 when concerns of erosion on Penarth beach were first raised. Subsequently, by September 2002 there was again qualitative

and verifiable evidence of a return to the traditionally accepted south to north longshore transport (Wallingford, 1992; Bullen 1993a; SBCEG, 1999). Overall temporal analyses of beach level trends were undertaken for the full 750m baseline length. Figure 6.20 produced from Table 6.1 showed the five-year variation of mean beach level. The regression equation y = 0.0016x + 2.5047 showed a trend of increasing beach levels, although this model did not explain much of the variation ($R^2 = 26.03\%$). For uniformity, the same analysis was undertaken between September 1997 and April 2000, and April 2000 and September 2002, represented by Figures 6.21 and 6.22 Figure 6.21 showed that during the earlier part of the respectively. monitoring period, there was a trend of falling beach levels given by y = -0.0018x + 2.5562, which was mainly influenced by results south of the old lifeboat slipway. The Coefficient of Determination ($R^2 = 17.1\%$) indicated that coastal processes were variable during this period, which was in contrast to the latter period (Figure 6.22) which indicated that the temporal model, y = 0.0066x + 2.269, represented a significant percentage of the variation in mean beach level ($R^2 = 96.9\%$).

In all analyses from April 2000, trends of increasing beach levels were represented by temporal regression models which explained a significant percentage of the variation and thus indicated more uniformity in prevailing coastal processes. Although Wallingford (1992) predicted that the completed barrage would have little effect on Penarth beach, the completion of the structure coincided with these uniform trends. Therefore, the barrage influence will be further considered in Chapter 7 (Groyned Beach). Baseline analysis considered the same relative crossshore position, and was not able to identify cross-shore trends. Cross-shore analysis would likely be significant due to the qualitative changes identified during the longshore evaluation. Concern over critical beach levels, as identified fronting the Esplanade in 1997 (SBCEG, 1999), has often been related to seawall foundations and overtopping in severe storms (Simm, 1996). Evaluation of such scenarios would require beach profile response and cross-section area analysis to identify the significance of change and consequently these were undertaken in the following section.

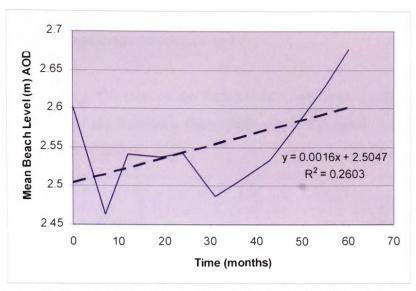


Figure 6.20: Temporal level variation longshore profile – 9/97 to 9/02.

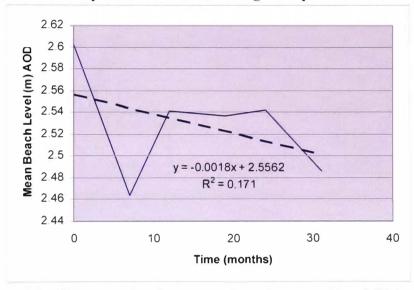


Figure 6.21: Temporal level variation longshore profile – 9/97 to 4/00.

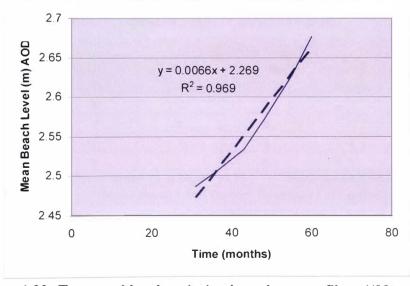


Figure 6.22: Temporal level variation longshore profile -4/00 to 9/02.

6.3 Foreshore Grid – Esplanade (stations 6 to 14)

Figure 6.26, an OS map of the Penarth foreshore fronting the Esplanade, shows the location of the foreshore grid, whilst Table 6.2 gives a diagrammatic representation of survey points.

Figure 6.23 has been removed from the digitized thesis for copyright reasons.

Figure 6.23: Location of Foreshore Grid fronting Esplanade. (Source: Ordnance Survey)

Table 6.2: Diagrammatic representation of foreshore grid

Cross-shore	Longshore Distance (m)							
Distance (m)	te (m) Station 6 Station 8 Station 10 150m 210m 270m		Station 12 330m	Station 14 390m				
-10 m	X	х	X	X	X			
0 m	X	X	X	х	Х			
+ 10 m	X	х	X	Х	Х			
+ 20 m	X	х	X	X	X			
	Mean Beach Level Stns 6 to 10 Mean Beach Level Stns 10 to 14							
	Mean	Beach Level O	verall Foreshor	e Grid stations	6 to 14			

x indicates position of surveyed beach level.

Beach levels and mean values, for all eleven surveys carried out each September and April between 1997 and 2002, are shown in Tables 6.3 to 6.13 inclusive, with examples of the field data collection sheets, included in Appendix 2.

Table 6.3: Beach Levels Survey 1 – September, 1997

Cross-shore						
Distance (m)	Station 6	Station 8	Station 10	Station 12	Station 14	
	150m	210m	270m	330m	390m	
-10 m	5.990	5.000	4.265	3.715	3.080	
0 m	4.125	3.495	3.115	2.480	2.005	
+ 10 m	2.715	2.200	1.830	1.340	1.090	
+ 20 m	1.605	1.015	0.705	0.380	0.245	
	Mean Beach Level = 2.956m Mean Beach Level = 1.994m					
	Me	an Beach Level	Overall Forest	nore Grid = 2·47	75m	

Table 6.4: Beach Levels Survey 2 – April 1998

Cross-shore	Longshore Distance (m)						
Distance (m)	Station 6	Station 8	Station 10	Station 12	Station 14		
	150m	210m	270m	330m	390m		
-10 m	6.050	4.980	4.105	3.600	2.920		
0 m	4.090	3.475	2.970	2.345	1.885		
+ 10 m	2.840	2.355	1.800	1.315	1.040		
+ 20 m	1.745	1.155	0.715	0.335	0.280		
	Mean Beach Level = 2.984 m Mean Beach Level = 1.913m						
	Me	an Beach Level	Overall Forest	nore Grid = 2·44	19m		

Table 6.5: Beach Levels Survey 3 – September 1998

Cross-shore	Longshore Distance (m)						
Distance (m)	Station 6	Station 8	Station 10	Station 12	Station 14		
	150m	210m	270m	330m	390m		
-10 m	5.550	4.915	4.430	3.890	3.135		
0 m	4.025	3.570	3.000	2.550	2.180		
+ 10 m	2.660	2.350	1.835	1.475	1.280		
+ 20 m	1.695	1.165	0.775	0.510	0.505		
<u></u>	Mean Beach Level = 2.972m Mean Beach Level = 2.097m						
	Me	an Beach Level	Overall Forest	nore $Grid = 2.53$	35m		

Table 6.6: Beach Levels Survey 4 – April, 1999

Cross-shore	Longshore Distance (m)						
Distance (m)	Station 6	Station 8	Station	10	Station 12	Station 14	
	150m	210m	2701	n	330m	390m	
-10 m	4.890	4.900	4.550		4.050	4.095	
0 m	3.800	3.600	3.17	5	2.895	2.565	
+ 10 m	2.590	2.350	1.90	0	1.525	1.340	
+ 20 m	1.520	1.145	0.785		0.570	0.700	
	Mean Beach Level = 2.940m Mean Beach Level = 2.292m						
	Me	an Beach Level	Overall F	orest	nore Grid = 2·61	6m	

Table 6.7: Beach Levels Survey 5 – September, 1999

Cross-shore	Longshore Distance (m)					
Distance (m)	Station 6	Station 8	Station 10	Station 12	Station 14	
	150m	210m	270m	330m	390m	
-10 m	5.040	4.945	4.375	3.845	3.065	
0 m	3.780	3.625	2.920	2.460	2.110	
+ 10 m	2.555	2.245	1.940	1.375	1.030	
+ 20 m	1.515	1.100	0.770	0.350	0.310	
	Mean Beach Level = 2.903m Mean Beach Level = 2.010m					
	Me	an Beach Level	Overall Fores	hore Grid = 2·45	57m	

Table 6.8: Beach Levels Survey 6 – April 2000

Cross-shore							
Distance (m)	Station 6	Station 8	Station 10	Station 12	Station 14		
	150m	210m	270m	330m	390m		
-10 m	4.805	4.980	4.390	3.925	3.355		
0 m	3.650	3.640	3.095	2.585	2.030		
+ 10 m	2.495	2.425	1.770	1.345	1.050		
+ 20 m	1.515	1.135	0.710	0.405	0.300		
	Mean Beach Level = 2.911m Mean Beach Level = 2.043m						
	Me	an Beach Level	Overall Fores	hore $Grid = 2.4'$	77m		

Table 6.9: Beach Levels Survey 7 – September 2000

Cross-shore	Longshore Distance (m)						
Distance (m)	Station 6 150m	Station 8 210m	Station 10 270m		Station 12 330m	Station 14 390m	
-10 m	4.790	4.975	4.6	10	4.065	3.325	
0 m	3.710	3.660	3.1	75	2.635	2.110	
+ 10 m	2.655	2.480	1.89	90	1.520	1.075	
+ 20 m	1.515	1.110	0.750		0.360	0.310	
	Mean Beach Level = 2.969m Mean Beach Level = 2.124m						
	Me	an Beach Level	Overall	Forest	ore Grid = 2·54	17m	

Table 6.10: Beach Levels Survey 8 - April, 2001

Cross-shore	Longshore Distance (m)						
Distance (m)	Station 6	Station 8	Station 10		Station 12	Station 14	
	150m	210m	270	m	330m	390m	
-10 m	4.690	4.735	4.5	35	4.070	3.475	
0 m	3.565	3.475	3.1	20	2.650	2.140	
+ 10 m	2.500	2.240	1.8	35	1.460	1.120	
+ 20 m	1.455	1.100	0.775		0.400	0.325	
	Mean Bea	Mean Beach Level = 2·834m Mean Beach Level = 2·122m					
	Me	Mean Beach Level Overall Foreshore Grid = 2.478m					

Table 6.11: Beach Levels Survey 9 - September, 2001

Cross-shore	Longshore Distance (m)						
Distance (m)	Station 6	Station 8	Statio	n 10	Station 12	Station 14	
	150m	210m	270	m	330m	390m	
-10 m	4.615	4.825	4.58	35	4.050	3.510	
0 m	3.535	3.500	3.15	50	2.705	2.180	
+ 10 m	2.520	2.360	1.90	00	1.505	1.110	
+ 20 m	1.465	1.225	0.805		0.390	0.330	
	Mean Beach Level = 2.884m Mean Beach Level = 2.151m						
	Me	an Beach Level	Overall	Forest	nore Grid = 2·51	8m	

Table 6.12: Beach Levels Survey 10 – April, 2002

Cross-shore		Longshore Distance (m)							
Distance (m)	Station 6	Station 8	Station	10	Station 12	Station 14			
	150m	210m	270r	n	330m	390m			
-10 m	4.565	4.870	4.62	5	4.040	3.580			
0 m	3.555	3.605	3.12	5	2.720	2.200			
+ 10 m	2.550	2.415	1.87	5	1.475	1.125			
+ 20 m	1.505	1.240	0.780		0.410	0.340			
	Mean Bea	Mean Beach Level = 2.916m Mean Beach Level = 2.152m							
	Me	an Beach Level	Overall F	orest	nore Grid = 2·53	34m			

Table 6.13: Beach Levels Survey 11 – September 2002

Cross-shore	Longshore Distance (m)						
Distance (m)	Station 6	Station 8	Station 10		Station 12	Station 14	
	150m	210m	270)m	330m	390m	
-10 m	4.560	4.985	4.6	40	4.015	3.660	
0 m	3.595	3.665	3.1	30	2.710	2.215	
+ 10 m	2.575	2.455	1.8	65	1.465	1.145	
+ 20 m	1.570	1.260	0.785		0.420	0.345	
	Mean Bea	Mean Beach Level = 2.952m Mean Beach Level = 2.155m					
	Me	an Beach Level	Overall	Foresh	ore Grid = 2.55	54m	

As explained in Section 5.5, this section of the beach is bounded on three sides by the old lifeboat slipway, the Esplanade seawall and Penarth Pier (Figure 6.23). Furthermore, there are three viewing areas along the promenade which protrude from the line of the seawall. As documented by Phillips (1999a) it appeared that the cross-shoreline at the mid point of the foreshore grid, acted as an axis of rotation for the surface plane of the beach. This was further supported in Section 6.2.2 (longshore profile) where it was seen that rising levels adjacent to the pier coincided with falling levels at the old lifeboat slipway and vice versa. Therefore, as the cross-shore grid profile at station 10 (270m longshore) influenced both northern and southern sides of the foreshore grid, this required consideration when quantifying mean beach levels. A method of overcoming this eventuality, used by civil engineers and quantity surveyors when determining bulk earth movements on construction contracts, is to introduce a system of weighting. method gives a better approximation of the average height of a grid of levels, by assigning a weighting to each grid point. For example, if a level influences only one part of the grid, it is given a weighting of 1; if it influences two parts of the grid, weighting 2 and similarly for four parts of the grid, weighting 4. The average level is then determined as follows:

Average level
$$\bar{x} = \frac{\sum dw}{\sum w}$$

where d is each individual beach level
w is the weighting of that level.

x is the average level of the grid.

The average level between stations 6 and 10 for Survey 1 (Table 6.3) was calculated as follows:

Level D	Weighting w	d x w
5.990	1	5.990
4.125	2	8.250
2.715	2	5.430
1.605	1	1.605
5.000	2	10.000
3.495	4	13.980
2.200	4	8.800
1.015	2	2.030
4.265	1	4.265
3.115	2	6.230
1.830	2	3.660
0.705	1	0.705
	$\Sigma = 24$	$\Sigma = 68.230$

Average Level =
$$\frac{68 \cdot 230}{24} = 2 \cdot 956$$
m

The weighted values for all surveys were similarly determined.

6.3.1 Mean beach level

The variation of mean beach level from Tables 6.3 to 6.13 inclusive is shown in Table 6.14.

Table 6.14: Temporal variation of mean beach levels (Foreshore Grid).

Survey	Date	Time	Mean Beach Level (m) AOD				
		(Months)	Stations 6-10	Stations 10-14	Overall Grid		
1	9/97	0	2.956	1.994	2.475		
2	4/98	7	2.984	1.913	2.449		
3	9/98	12	2.972	2.097	2.535		
4	4/99	19	2.940	2.292	2.616		
5	9/99	24	2.903	2.010	2.457		
6	4/00	31	2.911	2.043	2.477		
7	9/00	36	2.969	2.124	2.547		
8	4/01	43	2.834	2.122	2.478		
9	9/01	48	2.884	2.151	2.518		
10	4/02	55	2.916	2.152	2.534		
11	9/02	60	2.952	2.155	2.554		

6.3.2 Volume and Mass Change

Mean beach levels from Table 6.14 were converted into volume and mass change between surveys, i.e. gain or loss in volume and mass. From Coduto (1999) and Bell (1993), for the combination of beach material, that is, marl and limestone cobbles, an estimate of 2,200 kg/m³ was adopted. Consequently, volume change between surveys was determined as per the following example, using data from Table 6.14:

Mean level stations 6-10, Survey 1 = 2.956m

Mean level stations 6-10, Survey 2 = 2.984m

 \therefore Change in average level between surveys = +0.028m

Survey area = $(270 - 150) \times 30 = 3600 \text{m}^2$

 \therefore Volume change between surveys = +100.8m³

This equates to a gain of beach material of 100.8 x 2,200 kg

= 221,760 kg = 221.8 tonnes.

The survey area between stations 10 and 14 (270m to 390m longshore) was similarly 3,600m² and overall 7,200m² (stations 6 to 14; 150m to 390m longshore). Therefore, all surveys were analysed and intersurvey volume/ mass gain or loss of beach material determined (Table 6.15).

Table 6.15: Inter-survey Gain/Loss (+/-) of Beach Material.

	Volume gain/loss (+/-) m ³				Mass/gain/loss (+/-) tonnes			
Surveys	Stations	Stations	Overall	Cumulative	Stations	Stations	Overall	Cumulative
	6 – 10	10-14			6-10	10-14		
1 and 2	+ 100.8	- 291·6	- 190·8	- 190·8	+ 221.8	- 641.5	- 419.8	- 419·8
2 and 3	- 43.2	+ 662.4	+ 619.2	+ 428.4	- 95.0	+1457-3	+1362-2	+ 942·4
3 and 4	- 115·2	+ 702.0	+ 586.8	+ 1015.2	- 253·4	+1544.4	+1291.0	+2233.4
4 and 5	- 133·2	-1015-2	-1148·4	- 133·2	- 293·0	-2233·4	-2526.0	- 292·6
5 and 6	+ 28.8	+ 118.8	+ 147.6	+ 14.4	+ 63.4	+ 261.4	+ 324.7	+ 32·1
6 and 7	+ 208.8	+ 291.6	+ 500·4	+ 514.8	+ 459.4	+ 641.5	+1100.9	+1133.0
7 and 8	- 486	- 7.2	- 493.2	+ 21.6	-1069-2	- 15.8	-1085-0	+ 48.0
8 and 9	+ 180	+ 104.4	+ 284.4	+ 30.6	+ 396·0	+ 229.7	+ 625.7	+ 673.7
9 and 10	+ 115.2	+ 3.6	+ 118.8	+ 424.8	+ 253.4	7.9	+ 261.4	+ 935·1
10 and 11	+ 129.6	+ 10.8	+ 140·4	+ 565·2	+ 285·1	23.8	+ 308.9	+1243·4
Totals	- 14.4	+ 579.6	565.2	565.2	- 31.7	+1275·1	+1243.4	+1243.4

6.3.3. Longshore Gradients

Mean and central longshore gradients were determined as follows:

For Survey 1 (Table 6.3)

Average Level station 6 (150m) =

$$\frac{5 \cdot 99 + 4 \cdot 125 + 2 \cdot 715 + 1 \cdot 605}{4} = 3 \cdot 609$$
m

Average Level station 14 (390m) =

$$\frac{3 \cdot 08 + 2 \cdot 005 + 1 \cdot 090 + 0 \cdot 245}{4} = 1 \cdot 605m$$

:. Mean longshore gradient =

$$\frac{1 \cdot 605 - 3 \cdot 609}{240} \times 100\% = -0.835\%$$

Note that negative gradients indicate falling gradients in a northerly direction. This process was repeated for all surveys (Table 6.16).

Table 6.16: Temporal variation of longshore gradient with crossshore distance

		Cross-shore distance (m)						
Date	+ 10	0	- 10	- 20	gradient			
	(%)	(%)	(%)	(%)	(%)			
9/97	- 1.213	- 0.835	- 0.677	- 0.567	- 0.823			
4/98	- 1.304	- 0.909	- 0.750	- 0.610	- 0.896			
9/98	- 1.006	- 0.769	- 0.575	- 0.596	- 0.712			
4/99	- 0.331	- 0.514	- 0.216	- 0.342	- 0.427			
9/99	- 0.823	- 0.696	- 0.635	- 0.502	- 0.664			
4/00	- 0.604	- 0.675	- 0.602	- 0.506	- 0.597			
9/00	- 0.610	- 0.667	- 0.658	- 0.502	- 0.609			
4/01	- 0.506	- 0.594	÷ 0·575	- 0.471	- 0.536			
9/01	- 0.460	- 0.565	- 0.588	- 0.473	- 0.521			
4/02	- 0.410	- 0.565	- 0.594	- 0.485	- 0.514			
9/02	- 0.375	- 0.575	- 0.596	- 0.510	- 0.514			

6.3.4 Cross-shore Gradients

Mean and central cross-shore gradients were determined as follows:

For Survey 1 (Table 6.3)

Average Level back beach (-10m) =

$$\frac{5 \cdot 99 + 5 + 4 \cdot 265 + 3 \cdot 715 + 3 \cdot 08}{5} = 4 \cdot 410$$
m

Average Level front beach (+20m) =

$$\frac{1 \cdot 605 + 1 \cdot 015 + 0 \cdot 705 + 0 \cdot 38 + 0 \cdot 245}{5} = 0 \cdot 790$$
m

∴ Mean cross-shore gradient =

$$\frac{4 \cdot 410 - 0 \cdot 790}{30} \times 100 \% = 12 \cdot 1\%$$

Note that positive gradients indicate rising gradients towards the back of the beach. This process was repeated for stations 6, 8, 10, 12 and 14 and for all surveys with the results tabulated in Table 6.17.

6.3.5 Evaluation

Over the winter period, September 1997 to April 1998, overall beach levels fell by 0.026m (Table 6.14), equivalent to a loss of beach material of 190.8 m³ (419.8 tonnes: Table 6.15). This equated to an average loss of beach material of approximately 58 kg/m² over the 0.72 Ha foreshore grid (Section 5.5) correlated with a steepening negative longshore gradient (Table 6.16) and reduction in average cross-shore gradient (Table 6.17). Furthermore, from Table 6.15, it was seen that the volume of beach material between stations 6 and 10 increased by 100.8 m³ (221.8 tonnes) whilst between stations 10 and 14 the volume of beach material decreased by 291.6 m³ (641.5 tonnes) equivalent to + 61.6 kg/m² and - 178.2 kg/m² Therefore, the results provided quantitative evidence of the north to south longshore drift qualitatively indicated in Section 6.2 and verified a change from the accepted south to north longshore drift. Due to evidence provided in Chapter 1 and Section 6.2, it is argued that this change was not an isolated occurrence and was an on-going cause of the erosion problems experienced at that time along the Penarth foreshore.

By September 1998 average beach levels had increased by 0.086m (Table 6.14) equivalent to a gain in volume of 619·2 m³ (Table 6.15). This correlated to a reduction/average in longshore gradient (- 0.712%; Table 6.16) and fall in cross-shore gradient at stations 6 and 8 (12·8% and 12·5%; Table 6.17) further evidenced by the corresponding loss of material between stations 6 and 10 of 43·2 m³ and gain between stations 10 and 14 of 662·4 m³ (Table 6.15). This trend continued over the winter period to April 1999, with a further increase in mean beach level of 0.081m (Table 6.14), corresponding to a net gain in beach volume of 586·8 m³ (Table 6.15).

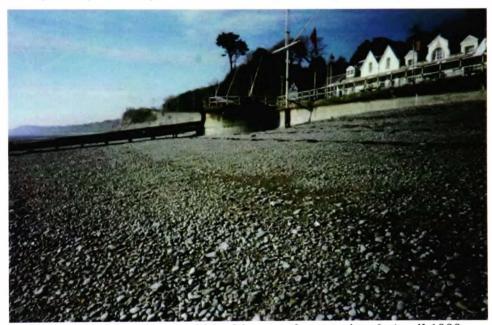


Plate 6.50: Foreshore grid looking south to station 6, April 1999.



Plate 6.51: Foreshore grid looking north to station 14, April 1999.

Table 6.17: Temporal variation of cross-shore gradient with longshore distance.

		Mean	Standard				
Date	Station 6-150m (%)(°)	Station 8-210m (%) (°)	ngshore distance (Station 10-270m (%) (°)	Station 12-330m (%)(°)	Station 14-390m (%)(°)	Average (%)(°)	Deviation (+/-)
9/97	14.6 (8.3)	13.3 (7.6)	11.9 (6.8)	11.0 (6.3)	9.5 (5.4)	12.1 (6.9)	1.77 (1.01)
4/98	14.4 (8.2)	12.8 (7.3)	11.2 (6.4)	10.9 (6.2)	8·7 (5·0) 8·7 (5·0)	11.6 (6.6)	1.48 (0.83)
9/98	12.8 (7.3)	12.5 (7.1)	12.1 (6.9)	11·2 (6·4) 11·6 (6·6)	11.3 (6.4)	11.8 (6.7)	0.60 (0.31)
4/99	11.2 (6.4)	12.5 (7.1)	12·6 (7·2) 12·0 (6·8)	11.7 (6.7)	9.2 (5.3)	11.5 (6.6)	1.21 (0.67)
9/99 4/00	11.8 (6.7)	12.8 (7.3)	12·3 (7·0)	11.7 (6.7)	10.2 (5.8)	11.6 (6.6)	0.92 (0.53)
9/00	10.9 (6.2)	12.9 (7.4)	12.9 (7.4)	12.4 (7.1)	10.1 (5.8)	11.8 (6.7)	1·14 (0·66) 0·81 (0·45)
4/01	10.8 (6.2)	12.1 (6.9)	12.5 (7.1)	12·2 (7·0)	10.5 (6.0)	11.6 (6.6)	0.86 (0.48)
9/01	10.5 (6.0)	12.0 (6.8)	12.6 (7.2)	12·2 (7·0) 12·1 (6·9)	10.8 (6.2)	11.6 (6.6)	0.95 (0.54)
4/02	10.2 (5.8)	12·1 (6·9)	12.9 (7.4)	12.0 (6.8)	11.1 (6.3)	11.6 (6.7)	0.99 (0.58)
9/02 Mean and	11.65	12.53	12.35	11.72	9.95	11.66	
Standard Deviation	1.53	0.40	0.49	0.49	0.92	0.16	

Beach angle in brackets given in degrees.

There was a further reduction in longshore gradient (- 0.427; Table 6.16) and important changes in cross-shore gradients at the longshore boundaries of the foreshore grid. At station 6 (150m) the gradient further reduced from 12.8% to 11.2%, whilst there was a significant increase at station 14 (390m) from 8.7% to 11.3% (Table 6.17) which further validated consideration of the foreshore grid in two halves. Movement of material across the foreshore resulted in an overall gain of material of 586.8 m³, which comprised a loss of 115.2 m³ between stations 6 and 10 and gain of 702 m³ between stations 10 and 14 (Table 6.15). The effects of these changes in gradients and volume movements were shown in Plates 6.50 (station 6, 150m) and 6.51 (station 14, 390m). It now appeared that the longshore drift had returned to the traditionally accepted south to north direction (SBCEG, 1999; Bullen, 1993a; Wallingford, 1992) and further supported the problems identified in Section 6.2.2, regarding the development of beach management strategies.

The 18th May, 1999, was the second day of easterly storms where, at high tide, wave energy caused flooding along the Esplanade. Plate 6.52 showed the section of seawall backing stations 10 to 14 (270 to 390m longshore), whilst Plate 6.53 showed the easterly driven waves between stations 6 and 10 (150m to 270m longshore). Sediment motion was observed with pebble sizes greater than 150m diameter, being freely displaced in a southerly direction. Consequently, the survey in September, 1999 produced some interesting results. From Table 6.14, it was seen that the overall beach grid levels had fallen by 0.159m, equivalent to a loss of 1,148·4m³ (2,526 tonnes) of beach material. This loss comprised 133·2m³ between stations 6 and 10 and 1,015·2m³ between stations 10 and 14. The average longshore gradient had once again steepened to 0.664% (Table 6.16) whilst since April 1999 the cross-shore gradient at station 6 (150m) had increased by 0.6% to 11.8% and at station 14 (390m) had decreased by 2.1% to 9.2% (Table 6.17). These results supported the predictions of Wallingford (1992), discussed in Section 2.2.3, that the largest waves at Penarth occur at high water, arrive from 50 to 90 degrees true and produce an element of southward drift.



Plate 6.52: Easterly storm stations 10 to 14, 18th May 1999.



Plate 6.53: Easterly storm stations 6 to 10, 18th May 1999.

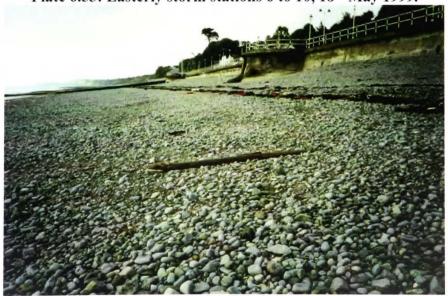


Plate 6.54: Exposed Victorian sewer near station 12, September 1999.

Plates 6.54 and 6.55 once again showed the consequences of the loss of beach covering with the re-exposure of the Victorian cast iron surface water sewers. These conditions were similar to those of September 1997 (Plate 6.6) and April 1998 (Plate 6.25), whilst the base of the steps, shown in Plate 6.29, further illustrated the change from April 1999. Plate 6.56 showed the tide line along the foreshore and indicated the rising plane of the beach in a southerly direction. Therefore it is concluded that the total loss of beach material from this part of the beach face was in a southerly direction, that is, a north to south sediment transport.



Plate 6.55: Exposed Victorian sewer near station 14, September 1999.



Plate 6.56: Incoming tide, looking north from old lifeboat slipway, September 1999.

Beach recovery demonstrated in April 1999 had been reversed by September 1999. Anthropogenic activities blamed for the beach erosion, such as marine aggregate dredging and the construction of the Cardiff Bay Barrage had been ongoing throughout. Consequently, these last two surveys further supported the conclusion of Phillips (1999a) who argued that changes in weather conditions between 1995 and 1997, and an increased incidence of easterly waves, may have been the most likely explanation for beach levels falling to critical levels along the Penarth foreshore (SBCEG, 1999). This hypothesis will be further considered in Chapter 8.

The winter of 1999, following the completion of the construction of the Cardiff Bay Barrage, saw a slight recovery in beach levels by April 2000 with an average rise over the foreshore grid area of 0.020m (Table 6.14). This equated to an overall gain of material of 147.6 m³ (Table 6.15) and reflected a reduction in steepness of the longshore gradient to -0.597% (Table 6.16). Furthermore, the cross-shore gradients again changed with a reduction in steepness to 11.0% at station 6 and increase to 10.2% at station 14 (Table 6.17). By September, 2000, further recovery had taken place, demonstrated by an overall increase in beach level of 0.080m to 2.547m (Table 6.14). Once again beach volumes had increased between both stations 6 and 10, and 10 and 14; 208·8m³ and 291·6m³ respectively (Table 6.15). There was little difference in longshore and cross-shore gradients between the April and September surveys (Table 6.16 and 6.17), probably explained by the similar longshore gains of material.

In April, 2001, mean beach levels had fallen over the whole survey area: 0·135m, stations 6 to 10; 0·002m, stations 10 to 14; 0·069m overall (Table 6.14), which translated to a loss of beach material: 486m³ stations 6 to 10; 7·2m³ stations 10 to 14; 493m³ overall (Table 6.15). There was a reduction in steepness of the falling longshore gradient from -0·609% to -0·536% (Table 6.16) and the reduction was common at all cross-shore locations. Furthermore, cross-shore gradients reduced at all stations with the exception of the northermost grid location (station 14) which increased from 10·1% to 10·5%. These results were consistent with a south to north

longshore drift and could be explained by a higher incidence of southeasterly waves. This will also be further analysed in Chapter 8. Results from the final three surveys, September 2001, April 2002 and September 2002, were similar to results from the longshore profile (Section 6.2) in that they reflected more consistency in coastal processes during that time period. The foreshore gained material in each of the surveys, evidenced by increased overall levels: +0.030m, September 2001; +0.016m, April 2002, and 0.020m, September 2002 (Table 6.14), equivalent to volume increases of: +284·4m³, September 2001; + 118·8m³, April 2002 and 140·4m³, September 2002 (Table 6.15). There was a consistent reduction in steepness of the falling longshore gradient, and by September 2002, the average gradient was at its minimum over the five-year monitoring period: -0.514% (Table 6.16). The cross-shore gradients were consistent in each of the three surveys at stations 8, 10 and 12 (Table 6.17). However, there was an overall decrease from 10.5% to 10.0% at station 6, whilst station 14 experienced a corresponding overall increase from 10.6% to 11.1%. These results again indicated a return to a south to north sediment transport and supported the outcomes from the analysis of the longshore profile surveys (Section 6.2).

6.3.6 Assessment of Change

Survey data collected over the five-year monitoring period was subjected to qualitative and quantitative analysis to determine patterns and trends to justify identified changes. Figure 6.24, produced from data given in Table 6.14, illustrated the temporal variation for stations 6 to 10 (150 to 270m longshore), stations 10 to 14 (270 to 390m longshore) and for the foreshore area overall, i.e. stations 6 to 14 (150 to 390m longshore). During the five years, the southern half of the foreshore decreased in average level by 0.004m (Table 6.14), equivalent to a loss of beach material of 14.4m³ (31.7 tonnes: Table 6.15). However, from Figure 6.25, which showed the temporal variation of mean beach level along the foreshore (stations 6 to 10: Table 6.14), it was seen that the overall trend was of falling beach levels, indicated by the negative gradient (-0.0011).

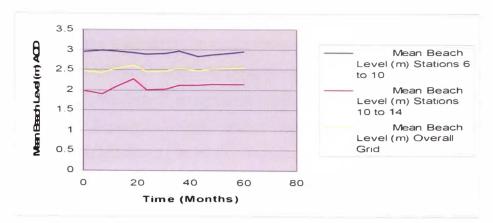


Figure 6.24: Temporal variation of mean grid beach levels.

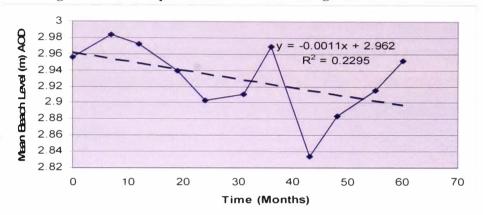


Figure 6.25: Temporal variation beach level stations 6 to 10.

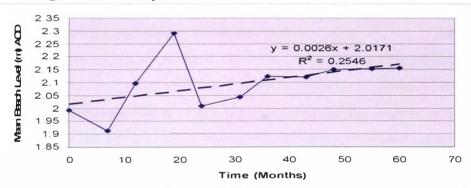


Figure 6.26: Temporal variation beach level stations 10 to 14.

Overall Grid (Stations 6 - 14)

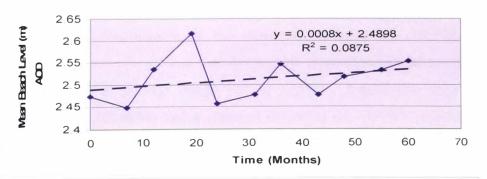


Figure 6.27: Temporal variation beach level stations 6 to 14.

Conversely, the northern half of the foreshore increased in average level by 0·161m, equivalent to a gain in beach material of 579·6m³ (1,275·1 tonnes: Table 6.15). As would be expected, Figure 6.26, which illustrated the temporal variation of mean beach level between stations 10 and 14 (Table 6.14), showed an overall trend of increased beach levels, indicated by the positive gradient (+ 0·0026). Figure 6.27 showed the trend for the temporal variation of mean beach level for the complete foreshore area (stations 6 to 14; 150 to 390m longshore) and was indicative of an overall rise in beach level (+ 0·0008 gradient). The rise of 0·079m (Table 6.14) translated into a gain of beach material of 565·2 m³ (1,243·4 tonnes: Table 6.15). Therefore, over the five-year monitoring period, there was evidence of change in coastal processes along the foreshore which further supported results from Section 6.2, of a change in direction of longshore drift from a southerly to northerly direction.

Changes in beach level along the foreshore grid were evident between September 1997 and September 2002. However, using data from Tables 6.3 and 6.13, a paired t test comparison for the beach levels at these times, showed no significant difference (t = 1; df = 19; p > 0.05). Analysis of the longshore gradients (Table 6.16) further identified change. It was seen that the falling northerly longshore gradients had reduced at all cross-shore locations between the start and end of the survey period. Consequently, a t test was applied to the longshore gradients at those times (September 1997 and September 2002) and results showed, at the 95% confidence level between surveys 1 and 11 there was a significant difference in longshore gradients (t = 2.664; df = 6; p < 0.05).

Further examination of the temporal variation of cross-shore gradients (Table 6.17), showed that the average cross-shore gradients were very similar for all surveys (range 11.5 to 12.1%). Indeed, the mean of these gradients (11.66%) had a standard deviation of \pm 0.16%, verifying overall consistency which supported Phillips (1999a) argument that station 10 acted as the axis for beach face rotation. Interestingly, the progressive fall in cross-shore gradient at station 6 (150m longshore) throughout the

five-year survey period was matched by a progressive increase at station 14 (390m longshore). This variability was shown by the respective standard deviations ($\pm 1.53\%$ and $\pm 0.92\%$) whilst the deviations at stations 8, 10 and 12 were very similar ($\pm 0.4\%$; $\pm 0.49\%$ and $\pm 0.49\%$ respectively). Analysis had once again supported the hypothesis that there had been a change in direction of longshore transport from a southward drift in September, 1997 to a northwards drift in September, 2002. Obviously, both cross-shore and longshore gradients are functions of beach morphology, and these aspects will be further considered in Section 6.3.7.

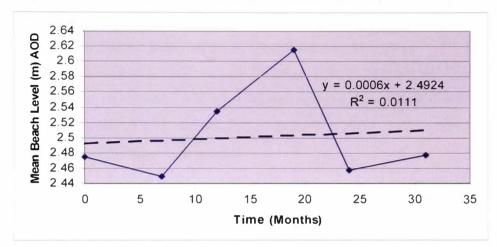


Figure 6.28: Temporal trends mean grid level - 9/97 to 4/00.

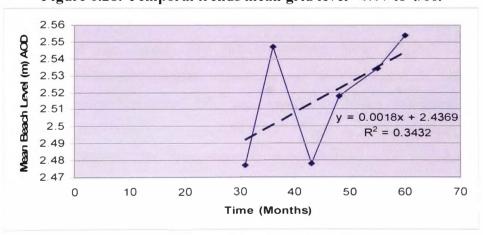


Figure 6.29: Temporal trends mean grid level - 4/00 to 9/02.

Similar to the evaluation process undertaken for each section of the longshore profile (Section 6.2), temporal trends of beach levels were analysed between September 1997 and April 2000, and April 2000 and September 2002 (Figures 6.28 and 6.29 respectively). From Figure 6.28, the regression equation y = 0.0006x + 2.4924 explained little of the temporal

increase in mean beach level over that time period ($R^2 = 1.11\%$) and indicated variability in coastal processes. The same analysis for the period April 2000 to September 2002 (Figure 6.29) gave a regression model y = 0.0018x + 2.4369, which explained a higher percentage correlation of increased beach levels with time ($R^2 = 34.32\%$). Whilst demonstrating more consistency in coastal processes, it was not certain whether the completion of the Cardiff Bay Barrage had any influence on beach levels or that natural processes were becoming more settled. Interestingly, the analysis of the foreshore grid showed agreement with results from this section of the longshore profile (Section 6.2.2). Comparison of Figures 6.13 and 6.14 with Figures 6.28 and 6.29 showed similar gradients (0.0005 and 0.0019 to 0.0006 and 0.0018) and coefficients of determination ($R^2 1.72\%$ and 29.04% to 1.11% and 34.32%). Although the longshore baseline formed part of the foreshore grid, the relative agreement of these results indicated that cross-shore changes were similar.

6.3.7 Changes in Beach Morphology

Tables 6.3 to 6.13 inclusive were imported into a Dplot package to produce two and three-dimensional survey representations of the foreshore grid area of the beach. The two-dimensional plots (Figure 6.30) demonstrated the chronological change in spatial-distribution of beach material between September 1997 and September 2002. It must be remembered that there is a scaling factor of ³⁹⁰/₃₀:1 or 13:1 longshore to cross-shore, that is, 1m cross-shore is equivalent to 13m longshore. Therefore, changes in cross-shore position are exaggerated from natural scale plan position, but this magnification enabled change to be more easily identified. The consequences of material movement along the foreshore were subsequently evaluated from these plots.

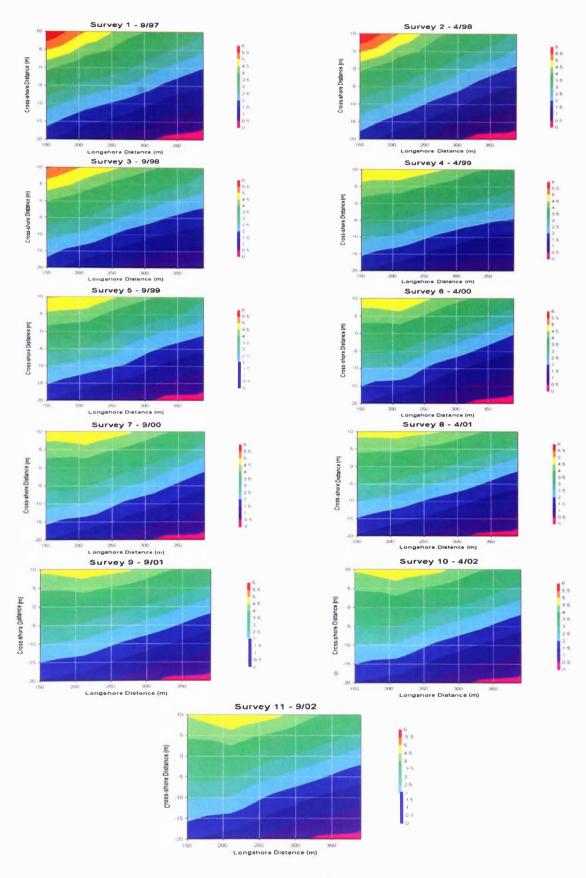
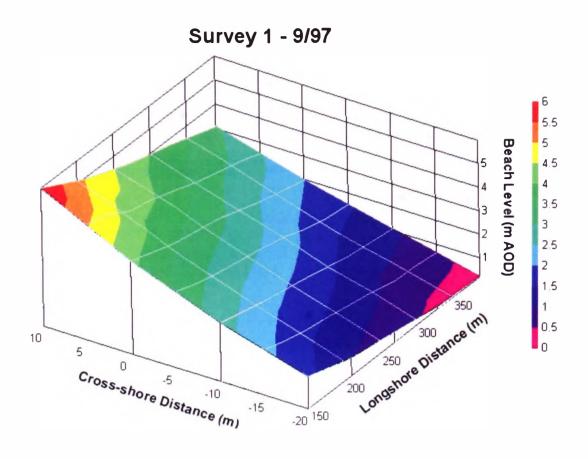


Figure 6.30: Foreshore Grid - Chronological change of beach contours

Considering the position of the 4m contour along the back beach line, it was seen from the first two surveys of Figure 6.30 (September 1997) and April 1998) that this contour was displaced in a southerly direction (reduction in longshore distance). This corresponded to an overall loss of beach material of 190.8m³ (Table 6.15). Surveys 3 and 4 (September 1998 and April 1999) saw gains in beach volume of 586·8m3 which had the corresponding effect of moving the 4m contour in a northerly direction (increase in longshore distance: Figure 6.33). The loss of material documented in September 1999 (- 1,148·4m3; Table 6.15), once again moved this contour in a southerly direction (Figure 6.30). For all intersurvey results of beach volume gain, shown in Table 6.15, the 4m contour moved in northerly direction (increased in longshore distance) whilst conversely, whenever there was an overall loss of beach material, it moved in a southerly direction. This would suggest that beach erosion resulted from a north to south sediment transport, whilst the traditional south to north longshore drift resulted in beach accumulation.

To further demonstrate changes that had occurred between the first and last surveys, three-dimensional views were prepared, and shown in Figure 6.31. Comparison of these plots illustrated the changes in longshore and cross-shore gradients, discussed in Section 6.3.6. The reduced steepness of longshore gradients between September 1997 and 2002 was clearly illustrated, whilst there was verification of reduction in cross-shore gradient at station 6 (150m longshore) and increase at station 14 (390m longshore). The back beach profile in September 2002, shown in Figure 6.31, identified a build-up of material between 150m and 250m longshore, and reflected beach conditions shown in Plate 6.57. This provided evidence that the survey results and extrapolation appropriately represented beach conditions.



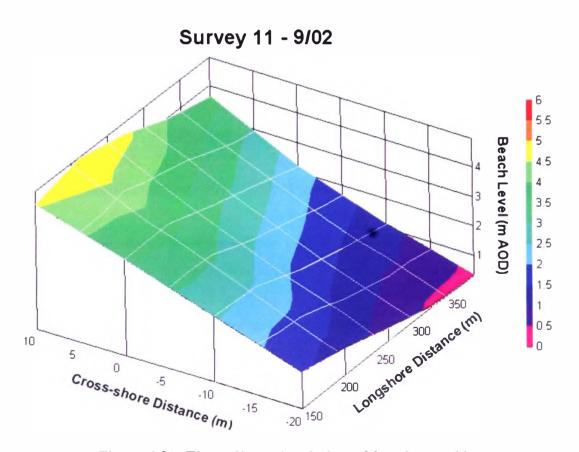


Figure 6.31: Three-dimensional plots of foreshore grid



Plate 6.57: Build up of material back beach, September 2002.

In Section 6.2, it was clearly shown that the position of the MHW in September 1997 had changed significantly from its traditional alignment shown on OS maps. Therefore, as the MHW position was an indicator of change, the changing position of the MHW line for each of the surveys was determined. The height of the MHW (average of MHWS and MHWN) for Penarth is 4.500m AOD (Page and Oakley, 2002; Hunter, 1996). From the OS map of the foreshore (Figure 6.23) the WCB of Esplanade seawall is 17.035°. As the longshore line of the foreshore grid was parallel to the seawall, then 17.035° was also the longshore WCB of the survey grid. Further analysis of the Dplot package provided co-ordinates for the 4.5m contour, from which its angle to the foreshore grid was determined. If the rotation of the 4.5m contour (MHW) was anti-clockwise to the foreshore grid, then the angle was deducted from the WCB of the foreshore grid to obtain the WCB of the MHW (Table 6.18). As previously shown from the behaviour of the 4m contour throughout the survey period, there should be a relationship between the variation in line of the MHW and gain or loss of beach material. Consequently, Table 6.18 also included data on volume gain/loss from Table 6.15.

Table 6.18: Variation of MHW and beach volume

Survey	Date	Time (Months)	Angle of MHW (°)	WCB MHW (°)	Change in WCBMHW	Volume Gain/loss (m³)
1	9/97	0	4.539	12.496		
2	4/98	7	4.850	12.185	- 0.311	- 190.8
3	9/98	12	3.484	13.551	+ 1.366	+ 619.2
4	4/99	19	1.633	15.402	+ 1.851	+ 586.8
5	9/99	24	2.375	14.660	- 0.742	- 1148.4
6	4/00	31	1.420	15.615	+ 0.955	+ 147.6
7	9/00	36	1.198	15.837	+ 0.222	+ 500.4
8	4/01	43	0.749	16.286	+ 0.449	- 493·2
9	9/01	48	0.427	16.608	+ 0.322	+ 284.4
10	4/02	55	0.260	16.775	+ 0.167	+ 118.8
11	9/02	60	0.208	16.827	+ 0.052	+ 140.4

Figure 6.32 was subsequently produced from Table 6.18 which showed the relationship between change in angle of the MHW line and gain/loss of beach material. There was a positive correlation, and the regression equation y = 512.6x - 165.49 accounted for a significant percentage ($R^2 = 52.54\%$) of the change in angle of MHW with gain/loss of beach material. Furthermore, it quantified the qualitative observation that erosion corresponded to a reduction in WCB MHW and accumulation resulted in an increase in WCB MHW.

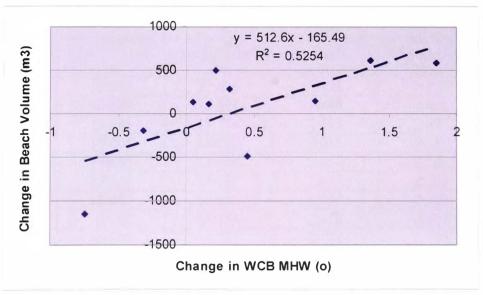


Figure 6.32: Variation of beach volume with WCB MHW.

However, it must be remembered that this section of the beach is bounded on three sides by the lifeboat slipway and Penarth Pier longshore and the seawall, back beach. In Section 2.2.4 it was seen that wave reflections in

the short-term can affect the level of the fronting beach (Simm, 1996) and therefore this may have had an influence on the correlation significance. Consequently, this analysis will be further examined in Section 6.4.

According to Crowell *et al* (1997), rates of shore erosion can be determined by monitoring the location of a representative *shoreline indicator*, usually the high water line over a specified time frame. They further argued that the measured shoreline movement, divided by the temporal span of the data, gives the rate of shoreline change. Therefore, it is logical that the average grid level represented the measured shoreline movement and/or erosion and the bearing of the MHW was the shoreline indicator. This was in line with discussions in Section 2.2.2 where the definition of depth of closure involved the advance or retreat of high water line (Simm, 1996). Consequently the average angle of the MHW and the average foreshore grid level will provide a model to evaluate shoreline change. Data from Tables 6.14 (mean beach level) and 6.18 (WCB MHW) was used to produce Table 6.19, from which Figure 6.33 was generated.

Table 6.19: Variation of angle of MHW with beach level

Data	WCB of MHW	Cumulative Average WCB of MHW	Overall Beach Level	Cumulative Average Beach Level
	(°)	(")	(m) AOD	(m) AOD
9/97	12.496	12.496	2.475	2.475
4/98	12.185	12.341	2.449	2.462
9/98	13.551	12.774	2.535	2.486
4/99	15.402	13.409	2.616	2.519
9/99	14.660	13.659	2.457	2.506
4/00	15.615	13.985	2.477	2.502
9/00	15.837	14.249	2.547	2.508
4/01	16.286	15.504	2.478	2.504
9/01	16.608	14.737	2.518	2.506
4/02	16.775	14.942	2.534	2.509
9/02	16.827	15.113	2.554	2.513

The regression equation, y = 44.411x - 97.149 showed a strong positive correlation between the WCB MHW and average beach level. This further confirmed that erosion of the beach would cause a reduction in the WCB MHW and vice versa. The model explained a high percentage of

the variation (R² = 61·61%) and it is argued that over a longer time frame, the model could be refined with a consequent increase in its precision (coefficient of determination). Validation of this model can be qualitatively shown from a consideration of Plate 1.2 (Bullen, 1993a) which showed a well-nourished foreshore in September 1992. Although no beach levels were given, it is clear that they were higher than in September, 1997. From the model, it would be predicted that the WCB MHW was correspondingly higher. From Figure 6.23 it was seen that the line of the MHW on the OS map was nearly parallel to the seawall, that is, WCB approaching 17°. This analysis provided supporting evidence that the model would also represent conditions prior to the start of this research.

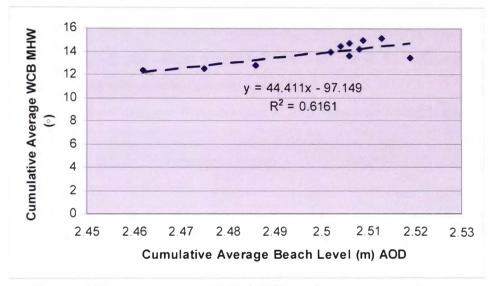


Figure 6.33: Variation of WCB MHW with average beach level.

The major concern associated with the erosion of Penarth beach was the apparent suddenness of the loss of beach and that there was no forewarning of the problem. The refinement of this model with future monitoring, will provide new trends and a reduction in the gradient would be an indicator of the onset of change. As was demonstrated, a reduction in the WCB MHW indicated beach erosion, with movement of material from the foreshore in a southward direction. However, the cause of the changes had yet to be determined as there was no direct correlation of change with the construction of the Barrage or ongoing marine aggregate dredging. Consequently, these analyses will be further considered in conjunction with

forcing agents (sea level and wind) in Chapter 8. However, further work was undertaken on the Penarth foreshore, considering beach profile response, as documented in the following section.

6.4 Beach Profiles – Penarth Foreshore

In Section 2.2.1 it was seen that whilst longshore transport is the primary mechanism for changes in beach plan shape, cross-shore transport is the means by which the beach profile changes (Simm, 1996). Section 5.6 detailed the rationale of the survey, whilst Figure 6.26, the OS Map of the foreshore, showed the location of the profiles at stations 6 and 14 (150m and 390m longshore). In Section 2.2.2, it was argued that the distinct termination of the limestone cobbles was the position of the depth of closure (Hallermeier, 1981). As this line varied along the foreshore, its distance seawards from the longshore profile (Section 6.2) was measured at stations 6 and 14 (150m and 390m longshore), the location of the cross-shore profiles for each survey with results shown in Table 6.20.

Table 6.20: Cross-shore distance from longshore profile to depth of closure.

Survey	Cross-shore distance (m)					
Date	Station 6 (150m)	Station 14 (390m)				
Sep 1998	55	20				
Apr 1999	53	26				
Sep 1999	49	21				
Apr 2000	42	28				
Sep 2000	45	30				
Apr 2001	35	40_				
Sep 2001	30	46				
Apr 2002	30	49				
Sep 2002	29	50				

Note: Distances were measured to the nearest metre.

Consequently, the mean \pm standard error of all distances in Table 6.20 was calculated as the seaward position of the depth of closure, that is, $37.667 \pm 2.588m$ (range 35.079 to 40.255m). However, according to Leatherman (1991) the depth of closure is the depth at which significant sediment motion is absent. As it was shown that the variable cross-shore limits at stations 6 and 14 were a maximum 55m and 50m respectively (Table 6.20), sediment motion was demonstrated to these limits. Therefore it is argued that with little error the cross-shore position of the depth of closure may be taken as 50m seaward of the line of the longshore

profile. Furthermore, despite its importance, there is generally uncertainty in its cross-shore position, which varies with location, and estimates are usually made (Section 2.2.2). Beach levels for all nine surveys, carried out at stations 6 and 14, each September and April between 1998 and 2002, are shown in Tables 6.21 and 6.22 respectively. These were used as the basis of the following analyses.

6.4.1 Depth of closure

To determine the mean beach level at the position of the depth of closure, beach levels 50m cross-shore (stations 6 and 14) from Tables 6.21 and 6.22, were averaged for all survey dates. Consequently, the mean \pm standard error beach level for the eighteen results was -1.017 ± 0.074 m AOD (range -0.943 to -1.091m AOD). As the height of the MHW is 4.500m AOD (Section 6.3.7) the water depth at the depth of closure for Penarth beach is in the range 5.443 to 5.591m, within the normally accepted water depth 4 to 6m (Section 2.2.2).

6.4.2 Cross-shore Profiles

Using data from Tables 6.21 and 6.22, Figures 6.34 and 6.35 detailed the temporal variation of the cross-shore profiles at stations 6 (150m) and 14 (390m) respectively. The minimum length of each profile was 100m (90m seaward and 10m landward of the longitudinal profile). In April 1999 and April 2000 the surveys were extended 170m seaward, approaching the physical position of the low water line during the spring tidal cycle. They were undertaken following two winter periods to ascertain whether there were significant changes in beach level beyond the depth of From Figures 6.34 and 6.35 there was little variation, closure (50m). Further examination of although station 14 (390m) was more variable. Figure 6.34 (station 6) showed that there was little variation in beach level beyond the depth of closure (50m cross-shore) and the largest variation, as would be expected, was at the back beach position (-10m: 1.490m: Table 6.21). Evaluation at

Table 6.21: Temporal variation of cross-shore beach levels – Station 6 (150m longshore)

Cross- shore Distance	Beach Level (m) AOD										
(m)	Sep-97	Apr-98	Sep-98	Apr-99	Sep-99	Apr-00	Sep-00	Apr-01	Sep-01	Apr-02	Sep-02
-10	5.990	6.050	5.550	4.890	5.040	4.805	4.790	4.690	4.615	4.565	4.560
0	4.125	4.090	4.025	3.800	3.780	3.650	3.710	3.575	3.535	3.555	3.595
10	2.715	2.840	2.660	2.590	2.555	2.495	2.655	2.500	2.520	2.550	2.575
20	1.605	1.745	1.695	1.520	1.515	1.515	1.515	1.455	1.465	1.505	1.570
30			0.615	0.570	0.590	0.470	0.445	0.410	0.470	0.495	0.525
40			-0.195	-0.210	-0.080	-0.225	-0.230	-0.185	-0.260	-0.260	-0.250
50			-0.715	-0.600	-0.575	-0.730	-0.795	-0.745	-0.775	-0.780	-0.750
60			-1.230	-1.210	-1.155	-1.180	-1.210	-1.185	-1.205	-1.290	-1.280
70			-1.560	-1.835	-1.745	-1.630	-1.705	-1.620	-1.610	-1.715	-1.745
80			-2.130	-2.145	-2.155	-2.175	-2.160	-2.045	-2.140	-2.150	-2.160
90			-2.525	-2.560	-2.555	-2.600	-2.650	-2.485	-2.540	-2.595	-2.600
100				-2.880		-2.900					
110				-3.200		-3.265					
120				-3.395		-3.365					
130				-3.610		-3.635					
140				-3.775		-3.805					
150				-3.925		-3.950					
160				-4.050		-4.080					
170	<u></u>			-4.230		-4.250					

Table 6.22: Temporal variation of cross-shore beach levels – Station 14 (390m longshore)

Cross- shore Distance	Beach Level (m) AOD										
(m)	Sep-97	Apr-98	Sep-98	Apr-99	Sep-99	Apr-00	Sep-00	Apr-01	Sep-01	Apr-02	Sep-02
-10	3.080	2.920	3.135	4.095	3.065	3.355	3.325	3.475	3.510	3.580	3.660
0	2.005	1.885	2.180	2.565	2.110	2.030	2.110	2.105	2.180	2.200	2.215
10	1.090	1.040	1.280	1.340	1.030	1.050	1.075	1.120	1.110	1.125	1.145
20	0.245	0.280	0.505	0.700	0.310	0.300	0.310	0.325	0.330	0.340	0.345
30			-0.210	-0.095	-0.385	-0.305	-0.285	-0.305	-0.300	-0.290	-0.295
40			-0.705	-0.675	-0.925	-0.865	-0.820	-0.850	-0.775	-0.695	-0.650
50			-1.350	-1.210	-1.455	-I.420	-1.435	-1.360	-1.300	-1.205	-1.I10
60			-1.865	-1.725	-1.945	-1.915	-1.920	-1.950	-1.780	-1.720	-1.695
70			-2.150	-2.115	-2.280	-2.245	-2.250	-2.295	-2.235	-2.225	-2.150
80			-2.480	-2.430	-2.500	-2.565	-2.540	-2.550	-2.515	-2.500	-2.440
90			-2.740	-2.730	-2.745	-2.765	-2.695	-2.660	-2.615	-2.590	-2.625
100				-2.860		-3.020					
110				-3.100		-3.250					
120				-3.285		-3.380					
130				-3.460		-3.575					
140				-3.655		-3.720					
150				-3.760		-3.830					
160				-3.880		-3.980					
170				-4.100		-4.235					

station 14 (Figure 6.35) similarly showed relatively little variation beyond 50m cross-shore with the largest variation back beach (-10m: 1·175m: Table 6.22).

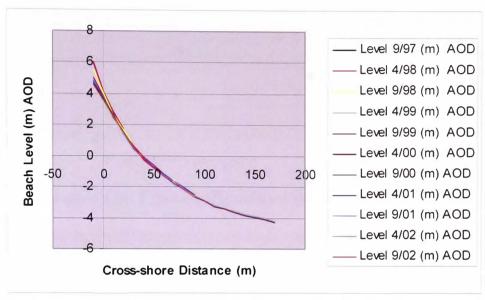


Figure 6.34: Temporal variation of cross-shore profiles, station 6.

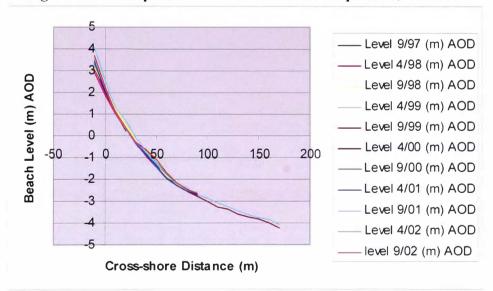


Figure 6.35: Temporal variation of cross-shore profiles, station 14.

To assess the changes in the cross-shore profiles between September 1998 and September 2002, comparisons of the profiles at these times were produced as Figures 6.36 and 6.37, for stations 6 (150m) and 14 (390m) respectively. It was apparent that beach levels at station 6 (150m) had fallen over the four years, whilst at station 14 (390m) they had generally increased.

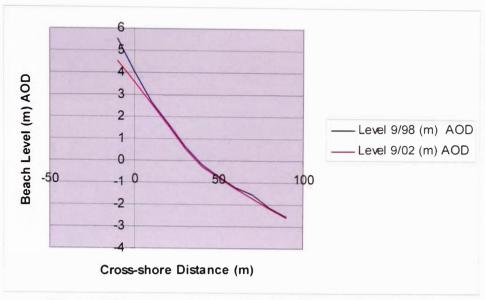


Figure 6.36: Cross-shore profiles 9/98 and 9/02, station 6.

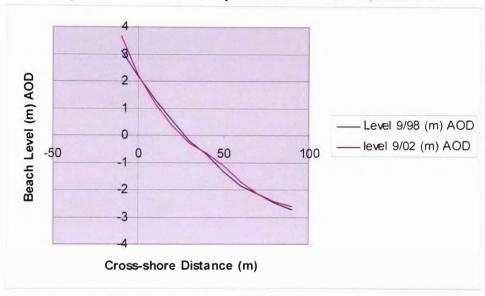


Figure 6.37: Cross-shore profiles 9/98 and 9/02, station 14.

Plate 6.58 showed the foreshore in September 1998 and the plan position of the depth of closure, the cross-shore boundary of the pebble beach, was clearly seen. Distances from the longitudinal profile were 55m cross-shore at station 6 and 20m cross-shore at station 14 (Table 6.20). Plate 6.59 showed the foreshore in September 2000, the mid-point of this particular survey period when the cross-shore distances to the boundary of the pebble beach were 45m at station 6 and 30m at station 14 (Table 6.20). Finally, Plate 6.60 from September 2002, showed the effects of increased beach levels at station 14, with the more uniform strandline across the length of the foreshore.



Plate 6.58: Esplanade foreshore, September 1998.



Plate 6.59: Esplanade foreshore, September 2000.



Plate 6.60: Esplanade foreshore, September 2002.

At that time, the line of the seaward edge of the pebble beach from the longshore profile was 29m cross-shore at station 6 (150m) and 50m cross-shore at station 14 (390m). These results were consistent with a change in longshore drift from a southerly to northerly direction. Therefore, these results further validated the conclusions made in Sections 6.2 and 6.3.

6.4.1 Changes in beach morphology

Tables 6.21 and 6.22 were imported into a Dplot package to produce two and three-dimensional survey representations of the pebble beach. The plots were limited in cross-shore distance to correlate with the seaward boundaries of the depth of closure detailed in Table 6.20. The twodimensional plots (Figure 6.38) illustrated the chronological change in spatial distribution of the beach material between September 1998 and September 2002. Similar to the foreshore grid in Section 6.3.7, there is a scaling factor, this time $^{390}/_{60} = 6.5$: 1, that is longshore to cross-shore, 1m cross-shore is equivalent to 6.5m longshore. Again, cross-shore position was exaggerated from natural scale plan position but this magnification enabled change to be more easily identified. As for the analysis of the foreshore grid, when the beach gained volume (Table 6.15), contours moved in a northerly direction, whilst movement in a southerly direction coincided with beach loss. To further identify changes that had occurred between September 1998 and September 2002, three-dimensional views were prepared and shown in Figure 6.39. Comparison of these plots illustrated over this time period:

- a reduction in steepness of longshore gradients;
- reduction in steepness of cross-shore gradients at station 6 (150m);
- increased steepness of cross-shore gradients at station 14 (390m);

This agreed with findings in Section 6.3.6 and further reference to Plates 6.58, 6.59 and 6.60 reinforced these trends. Using the seawall as a reference, the gradual build-up of beach material, longshore towards station 14 (390m), culminated in a clearly defined strandline in Plate 6.60 (September 2002).

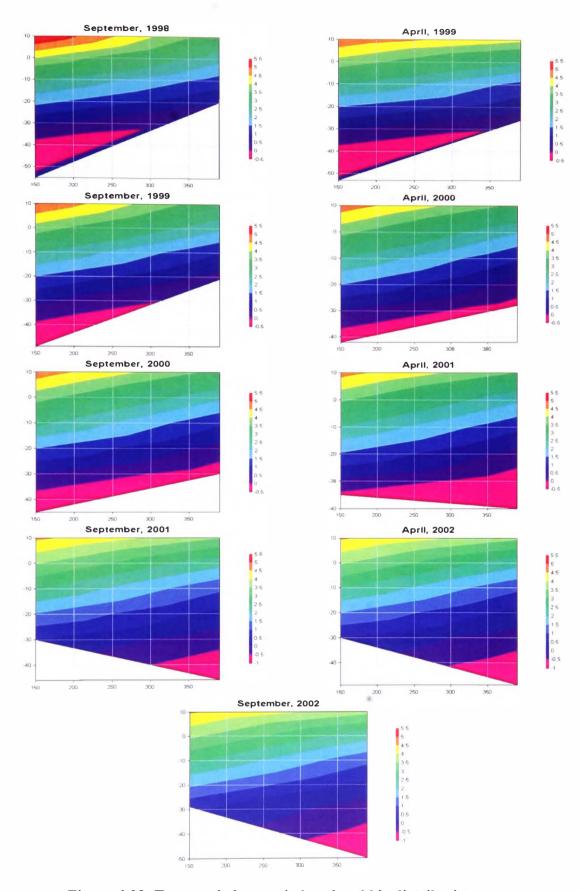
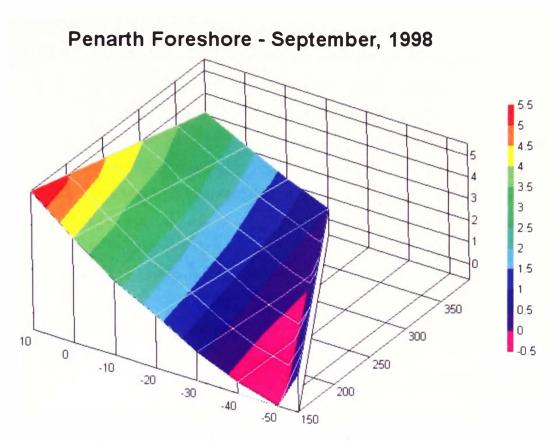
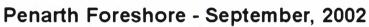


Figure 6.38: Temporal changes in beach pebble distribution.





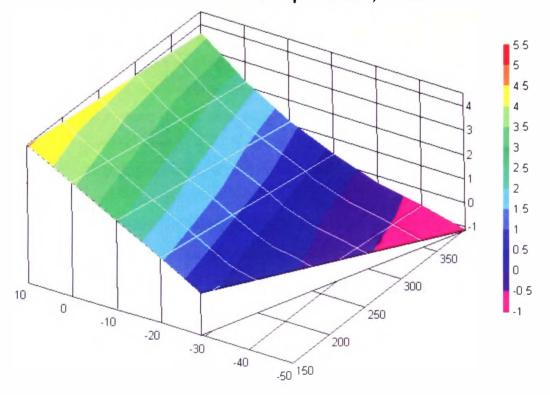


Figure 6.39: Three-dimensional projections of pebble beach.

The limitation of generating plots from two cross-shore profiles, separated by such a large distance (240m), is that interpolation between known points is coarse. For example, Figure 6.31 was generated from the foreshore grid levels and had sufficient refinement to identify isolated accumulations of beach material (Plate 6.57) whilst Figure 6.39 was unable to do so. Importantly, Figure 6.38 showed the progressive change of direction of the seaward limit of the pebble beach, the position of the depth of closure. As well as being a function of longshore drift, it is clearly related to cross-shore transport and change in beach profile. These critical functions will be analysed in the next section.

6.4.2 Assessment of change

From Figure 6.38 and Table 6.20, the WCB of the seaward boundary of the pebble beach, which will be referred to as the depth of closure (DoC) in the following analyses, was calculated for all surveys. As for the foreshore grid, the WCB of the longshore profile/back beach was 17.035°. Therefore, depending on whether the angle of the DoC to the line of the back beach was clockwise or anticlockwise, this dictated whether it would be added to or subtracted from the WCB of back beach line. For example, in September 1998, this angle was $tan^{-1} \frac{35}{240} = 8.297^{\circ}$. As this was anticlockwise to the line of the back beach, the angle was subtracted from 17.035° to give the WCB DoC = 8.738° . In September 2002, the angle was $\tan^{-1} \frac{21}{240} = 5.001^{\circ}$. This time the angle was clockwise to the line of the back beach and was added to 17.035° . This made the WCB DoC = 22.036° at that time. The intervening surveys were similarly evaluated and the results shown in Table 6.21. Furthermore, as the WCB DoC could be interpreted as another shoreline indicator, its relationship to the WCB MHW warranted evaluation. Therefore, surveyed values of the WCB MHW from Table 6.18 were incorporated into Table 6.21.

Table 6.23: Temporal variation of shoreline indicators

Date	Time	WCB MHW	WCB DoC
	(Months	(°)	(°)
Sep1998	12	13.551	8.738
Apr1999	19	15.402	10.616
Sep1999	24	14.660	10.381
Apr 2000	31	15.615	13.696
Sep 2000	36	15.837	13.459
Apr 2001	43	16.286	18-228
Sep 2001	48	16.608	20.849
Apr 2002	55	16.775	21.561
Sep 2002	60	16.827	22.036

Note: Time started at 12 months for alignment with other surveys.

25 y = 0.061x + 13.50620 $R^2 = 0.8559$ Angle (Degrees) **WCBMHW** 15 WCB DoC · Linear (WCBMHW) 0.3103x + 4.196610 $R^2 = 0.9522$ Linear (WCB DoC) 5 0 80 0 20 40 60 Time (Months)

Figure 6.40: Temporal variation of WCB MHW and WCB DoC.

Figure 6.40, produced from Table 6.23, showed the temporal variation of the shoreline indicators (MHW and DoC). Both the WCB MHW and WCB DoC increased over the survey period and in Section 6.3.7, it was seen that increased WCB MHW correlated with increased mean beach level. Therefore, it is logical that increased WCB DoC would also correlate to increased mean beach levels. It was seen that the temporal variations of the shoreline indicators intersected at approximately 40 months, that is, January 2001. It was significant that the erosion in September 1999 (time 24 months), documented in Table 6.15 and Sections 6.2 and 6.3, resulted in a reduction of the WCB of both MHW and DoC (Table 6.23). Conversely the loss of beach material documented in April

2001 (time 43 months: Table 6.15) did not cause similar angle reduction whilst from Table 6.23 the WCB of both MHW and DoC actually increased. In September 1999 the loss of material was in a southerly direction, whilst in April 2001 it moved in a northerly direction (Section 6.3.5). This would indicate that the beach level does not undergo as severe change when sediment movement is in the traditionally accepted direction from south to north. The regression equations y = 0.061x + 13.506 (MHW) and y = 0.3103x + 4.1966 (DoC) both significantly represented the temporal variation of angle of the shoreline indicators, $R^2 = 95.22\%$ and 85.59% respectively. Both provided valid models for assessment of change and verified increasing trends with positive gradients + 0.061 (MHW) and + 0.3103 (DoC).

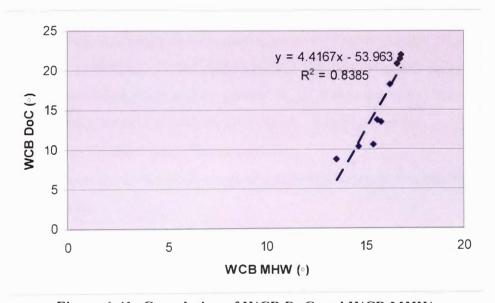


Figure 6.41: Correlation of WCB DoC and WCB MHW.

The correlation between angle of MHW and DoC was evaluated in Figure 6.41. The regression equation y = 4.4167x - 53.963 demonstrated significant correlation ($R^2 = 83.85\%$). In Section 6.3.7, it was shown that there was a direct significant correlation between the WCB MHW and mean foreshore beach level. Therefore there must be the same significant correlation between the WCB DoC and the mean foreshore beach level. It is therefore argued that this relationship will provide a simple management tool to assess beach health. By measuring the distance from the longshore

profile to the seaward limit of the pebble beach at stations 6 (150m) and 14 (390m), the WCB DoC can be calculated from the following equation:

WCB DoC° =
$$17.035 + \tan^{-1} \frac{(x_{14} - x_6)o}{240}$$

where 17.035° = WCB longshore profile

 x_{14} = distance (m) from longshore profile to seaward limit of pebble beach – station 14.

 x_6 = distance (m) from longshore profile to seaward limit of pebble beach – station 6.

240 = longshore distance station 6 to station 14 (m).

A reduction in WCB DoC would indicate a fall in mean bench level, an estimate of which may be calculated from y = 4.4167x - 53.963 (Figure 6.41) and y = 44.411x - 97.149 (Figure 6.33). The benefit of this equation is that it provides a rapid current assessment on the health of the Penarth foreshore and comparison over any time period. It can be used to:

- assess the short-term effects of storms:
- indicate the resultant direction of longshore transport over any time period;
- provide an early warning of potential problems;
- give an estimate of change in the WCB MHW (Shoreline Indicator);
- give an estimate of mean beach level (Shoreline Movement);
- compare time now with past records, e.g. Table 6.23.

The two causes of shoreline recession are sediment loss and an increase in sea levels (NSW, 1990). The best indication of shoreline movement is obtained from a volumetric sediment budget survey where with care and careful interpretation, acceptable estimates of long-term recession trends can be obtained. The Dplot package gave volumes of material below the plane of the beach for each survey plotted (Section 6.4.3). However, recession estimates are based on temporal changes in the

volume of beach sediments, normally m³/m. In this analysis the beach length was 240m. Consequently, the survey volumes from the Dplot package were divided by 240 and the results shown in Table 6.24.

Table 6.24: Temporal variation of beach volume

Date	Time	Beach Volume	Beach volume per m run
	(Months)	m³	m³/m
Sep 1998	12	21,606	90.025
Apr 1999	19	21,461	89.421
Sept 1999	24	19,181	79.921
Apr 2000	31	19,021	79.254
Sep 2000	36	19,815	82.563
Apr 2001	43	19,517	81.321
Sep 2001	48	20,067	83.613
Apr 2002	55	20,478	85-325
Sep 2002	60	20,554	85.642

Note: Time started at 12 months for alignment with other surveys.

Figure 6.42, produced from Table 6.24 illustrated the temporal variation in the volume of the pebble beach. The regression equation y = -0.0571x + 86.2, whilst not explaining much of the variation ($R^2 = 5.94\%$), indicated a trend of beach recession. The analysis was repeated but this time the significant erosion event, measured in September 1999 (month 24) was omitted. The resulting shoreline trend is shown in Figure 6.43. The regression model y = 0.1787x + 74.936 indicated a trend of accumulation of beach material (gradient 0.1787) which significantly represented the temporal variation of beach volume ($R^2 = 85.63\%$). Whilst the timeframe of both these recession evaluations are too short for meaningful predictions of shoreline recession, they did highlight the effects of storm events, on beach volume (month 24 and month 43) discussed in Section 6.3.

The final analysis in this section, considered the relationship between the inter-survey change in volume calculated for the foreshore grid (Table 6.15) and the inter-survey change in volume to the seaward limit of the pebble beach (Table 6.24). The resulting values were included in Table 6.23.

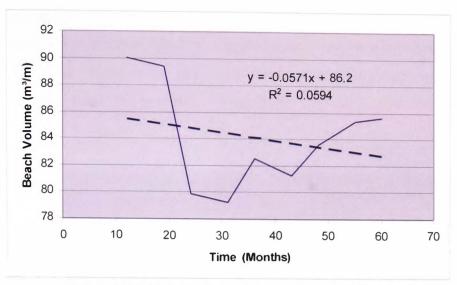


Figure 6.42: Temporal variation of beach volume.

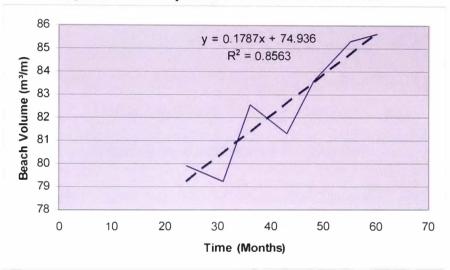


Figure 6.43: Shoreline trend – September 1999 to September 2002.

Table 6.25: Comparison of Inter-survey Gain/Loss (+/-) of beach volume

Date	Time	Volume Gain/Loss (+/-) m ³				
	(Months)	Foreshore Grid	Seaward limit (DoC)			
Apr 1999	19	586.8	- 145			
Sep 1999	24	- 1148·4	- 2280			
Apr 2000	31	147.6	- 160			
Sep 2000	36	500.4	794			
Apr 2001	43	- 493.2	- 298			
Sep 2001	48	284.4	550			
Apr 2002	55	118.8	411			
Sep 2002	60	140.4	76			

Initial examination of Table 6.25 indicated that there may be little correlation between the two assessments of change in volume. This was

unexpected as the total foreshore area included the foreshore grid, that is, the variables were interdependent. As previously mentioned in this section, the foreshore results to the DoC were based on two cross-shore profiles which may have led to coarse interpolation over the 240m length of foreshore.

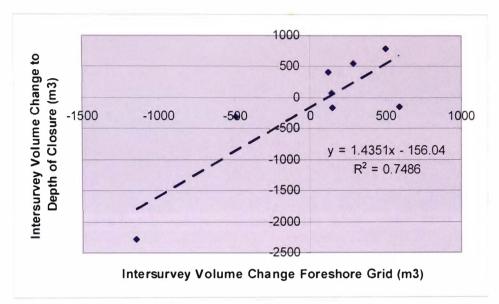


Figure 6.44: Beach volume comparisons.

However, when Figure 6.44 was produced from Table 6.25 the results showed not only a positive correlation (1·4351) but that the regression equation y = 1·4351x - 156·04 demonstrated a significant relationship ($R^2 = 74·86\%$) between the two variables. Therefore, this analysis further validated the quantitative assessments made for the shoreline indicators: WCB MHW, WCB DoC and mean beach level.

6.5 Link Survey

Plate 6.61 was photographed from Penarth Pier, looking northwards towards Penarth Head, whilst aerial photographs, Plate 6.62 and 6.63 and Figure 6.45 (OS map) show the study area. As identified in Section 5.7, this length of beach links the end of the longshore profile with the groyned beach, adjacent to the Cardiff Bay Barrage. A total of eight surveys were carried out between September 1997 and September 2002, omitted surveys coinciding with restricted access to the beach area (Section 5.8), and Table 6.26 gives the results of the surveys.

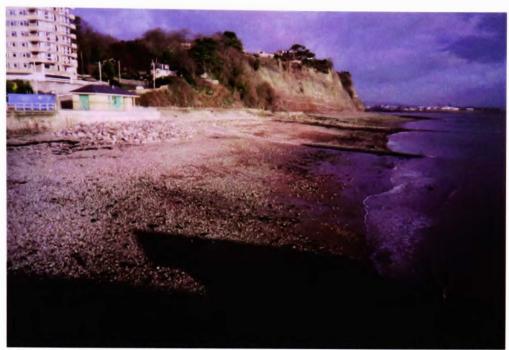


Plate 6.61: Penarth beach, northwards to Penarth Head – April, 2002.

These plates have been removed for copyright reasons.

Plate 6.3: Aerial photograph link survey (S). (Source: Getmapping plc)

Figure 6.45 has been removed for copyright reasons.

Figure 6.45: Link survey (Source: Ordnance Survey).

Figure 6.46 produced from Table 6.26, detailed the temporal variation of longshore profiles over this 567m length of foreshore (750 to 1,317m longshore). It was clear that the beach rose in level from station 26 to the manhole at Penarth Head from where it fell, with the exception of the final survey, to the site of the Barrage. This was consistent with the report of Bullen (1993a) and supported by Phillips (1999b), who argued that the change in beach heading from approximately NNE to NNW, represented a transition in beach formation processes. To aid in the assessment of change, the longitudinal gradients were subsequently calculated from the levels and distances in Table 6.26 and shown in Table 6.27.

Table 6.26 showed that the inter-survey variation of beach level at both 750m and 1,317m longshore, were inversely related to the variation at 1,245m longshore, that is, as beach levels fell at 1,245m they rose at 750m and 1,317m, and vice versa. Furthermore, the greatest variation in mean beach level was adjacent to the manhole at Penarth Head, 1,245m longshore; $(4.503 \pm 0.155m)$. In September 2002, the beach reached its lowest level at this location during the monitoring period (Table 6.24). Figure 6.47, produced from Table 6.26, compared the profiles at the start (September 1997) and end (September 2002) of the survey period and illustrated that in September 2002, for the first time, there was a rising gradient between 1,245m and 1,317m.

Table 6.26: Beach levels – Link Survey

			Beach Level (m) AOD						
Survey	Date	Time (Months)	Station 26 (750m longshore)	M/H Penarth Head (1,245m longshore)	Groyne 6 (1k) (1,317m longshore)	Mean (750m to 1,245m)	Mean (1,245m to 1,317m)	Mean (750m to 1,317m)	
1	Sep 97	0	2.520	4.400	4.270	3.460	4.335	3.730	
2	Sep 99	24	2.425	4.655	4.095	3.545	4.375	3.725	
3	Apr 00	31	2.390	4.640	4.145	3.650	4.379	3.725	
4	Sep 00	36	2.290	4.675	4.060	3.483	4.368	3.675	
5	Apr 0I	43	2.460	4.510	4.245	3.555	4.378	3.738	
6	Sep 01	48	2.385	4.525	4.165	3.455	4.345	3.692	
7	Apr 02	55	2.445	4.375	4.235	3.410	4.305	3.685	
8	Sep 02	60	2.575	4.240	4.450	3.408	4.345	3.755	
Mean x			2.436	4.503	4.208	3.496	4.354	3.716	
Standard Deviations			0.087	0.155	0.122	0.083	0.026	0.028	

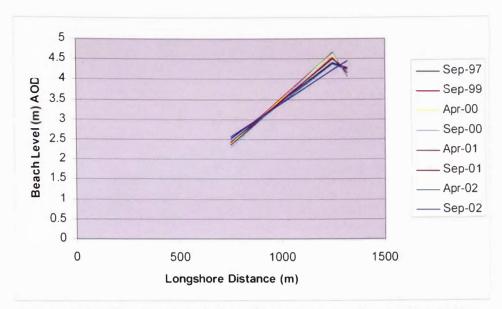


Figure 6.46: Temporal variation of longshore profile – 750 to 1317m.

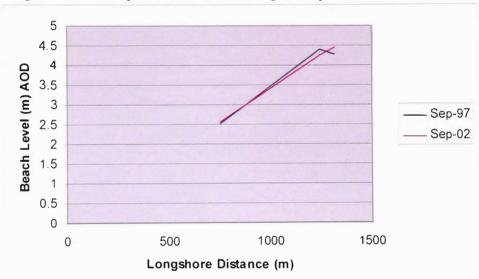


Figure 6.47: Change in longshore profile between 9/97 and 9/02.

Table 6.27: Longshore gradients

Survey	Date	Gradient %				
		750m to 1,245m	1,245m to 1,317m			
1	Sep 97	+ 0.380	- 0.181			
2	Sep 99	+ 0.450	- 0.778			
3	Apr 00	+ 0.454	- 0.688			
4	Sep 00	+ 0.482	- 0.854			
5	Apr 01	+ 0.414	- 0.368			
6	Sep 01	+ 0.432	- 0.500			
7	Apr 02	+ 0.390	- 0.194			
8	Sep 02	+ 0.336	+ 0.292			



Plate 6.64: Manhole Penarth Head (1245m) – September 1997.





From Table 6.24, the mean survey beach levels for the different longshore ranges: 750 to 1,245m; 1,245 to 1,317m and 750 to 1,317m showed standard deviations of 0.083m, 0.026m and 0.028m respectively, which indicated little overall variation. Plates 6.65 and 6.66 taken at the times of surveys 1 and 2 respectively show the manhole and cover slab located at Penarth Head (1,245m longshore). In Plate 6.64 the cover was 0.750m clear of the beach whilst in Plate 6.65 it was 0.500m clear. Although a general rise in beach levels at this location were indicated, beach levels were still below those at the time of the sewer construction. Plate 6.66 shows the manhole in September 2002, the time of the last survey. The years of wave and pebble attack had finally caused failure of the concrete slab and the remaining slab was approximately 0.9m above the beach surface. Loss of beach material was apparent although the overall fall in beach level of 0.160m, between September 1997 and September 2002 (1,245m) coincided with a gain of 0.180m at 1,317m longshore (Table 6.26). This increase at groyne 6 will be further considered in Chapter 7.

In Section 6.2, temporal trends were evaluated for each part of the longshore profile. A similar analysis was undertaken for this length of coastline, which was divided into two separate elements according to direction, that is, 750m to 1,245m (NNE) and 1,245m to 1,317m (NNW), a decision justified by the gradients documented in Table 6.25.

6.5.1 Temporal Trends – 750m to 1,245m longshore

Figure 6.48 produced from Table 6.26, shows the temporal variation of mean beach level along this section. The regression equation, y = -0.0015x + 3.5515 indicated a trend of falling beach levels over the monitoring period but the analysis did not explain much of the temporal variation ($R^2 = 12.19\%$). In Sections 6.2.2, 6.2.3 and 6.2.4 the temporal analysis of the longshore profile also considered two distinct time periods, that is, September 1997 to April 2000 and April 2000 to September 2002. These coincided with pre and post Barrage completion and Figures 6.49 and

6.50, produced from Table 6.26, represented these periods for the link survey.

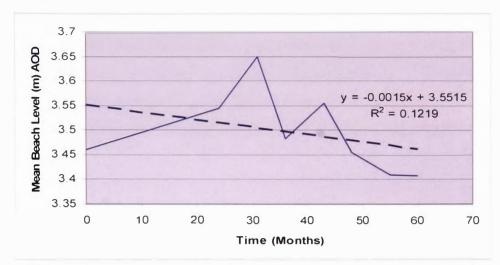


Figure 6.48: Temporal variation in mean beach level – 9/97 to 9/02.

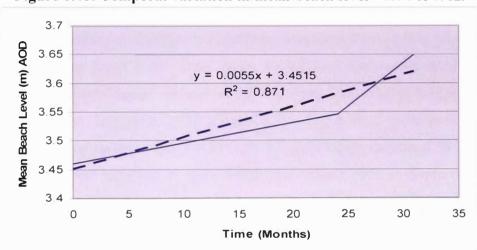


Figure 6.49: Temporal variation in mean beach level – 9/97 to 4/00.

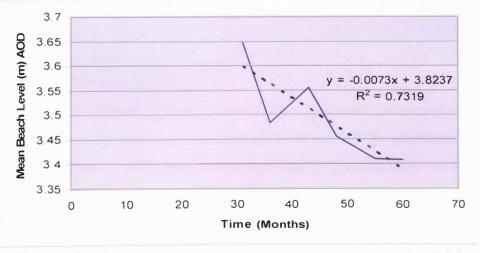


Figure 6.50: Temporal variation in mean beach level -4/00 to 9/02.

From Figure 6.49 the regression equation y = 0.0055x + 3.4515 explained a significant percentage of the temporal increase in mean beach level ($R^2 = 87.1\%$) between September 1997 and April 2000. The same analysis for the period April 2000 to September 2002 (Figure 6.50) gave a regression model y = -0.0073x + 3.8237, which explained 73.19% of the temporal reduction in beach level. Therefore the trend of increasing beach levels with time, prior to completion of the Barrage, was reversed on completion.

6.5.2 Temporal Trends – 1,245m to 1,317m longshore

Using data from Table 6.26, Figure 6.51 shows the temporal variation between September 1997 and September 2002, from which the regression model y = -0.0003x + 4.3649 only explained 4.91% of the falling trend of mean beach levels. Once again the variation was re-evaluated in two distinct time frames: September 1997 to April 2000 (Figure 6.52) and April 2000 to September 2002 (Figure 6.53). The first of these periods produced a regression model y = 0.0015x + 4.3358 which explained 98·2% of the temporal variation in mean beach level. April 2000 to September 2002 was represented by y = -0.0019x + 4.4404 which, although accounting for 56·77% of the variation again demonstrated change since April 2000. As with the previous length (750 to 1,245m longshore) trends of increasing beach levels to April 2000 were subsequently reversed.

6.5.3 Evaluation

In both analyses of temporal trends, there appeared to be consistency in coastal processes between September 1997 and April 2000; demonstrated by the significant coefficients of determination of the regression models (87·1% and 98·2%). This was opposite to results for the longshore profile in Section 6.2, which indicated variability during this period. However, with the exception of south of the lifeboat slipway (Section 6.2.1), there was an agreed trend of increased beach levels. Between April 2000 and September 2002 the regression models for the link survey both indicated falling trends

of beach levels with less consistency in coastal processes ($R^2 = 73.19\%$ and 56.77%). The trend of falling beach levels differed from the results of Section 6.2, where all sections of the baseline indicated increased beach levels over this time period.

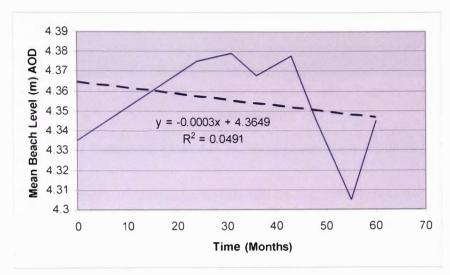


Figure 6.51: Temporal variation in mean beach level – 9/97 to 9/02.

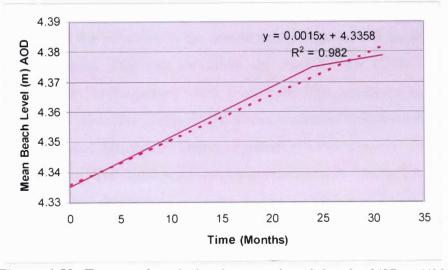


Figure 6.52: Temporal variation in mean beach level – 9/97 to 4/00.

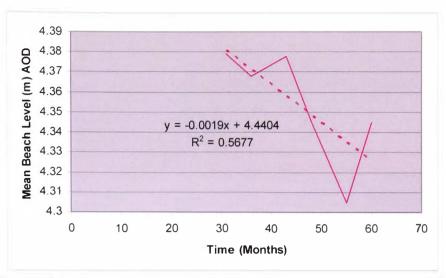


Figure 6.53: Temporal variation in mean beach level – 4/00 to 9/02.

This length of coastline within the survey boundaries, as well as comprising two different headings, was the longest without any human construction (Section 5.7). The temporal models between September 1997 and April 2000 were only based on three surveys, so their significance may be misleading. However, the models for April 2000 to September 2002 may have reflected the influence of the completed Barrage as well as changing coastal processes. This was especially the case for the NNW section (1,245m to 1,317m longshore) where from April 2000, gradients increased in a northerly direction (Table 6.27). These outcomes may have resulted from the changed direction of longshore transport when trends of increased beach levels to April 2000 were subsequently reversed. This is correlated with the changed direction of sediment transport which would be a logical cause.

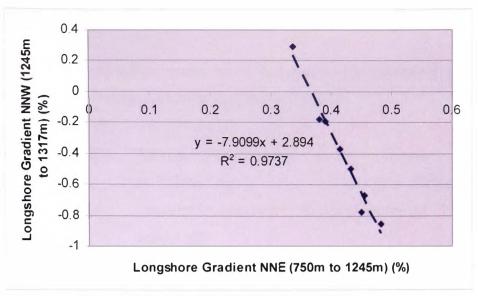


Figure 6.54: Correlation of longshore gradients.

To determine whether there was a relationship between the NNE and NNW beach headings, the longshore gradients for each survey (Table 6.27) were plotted as Figure 6.54. The regression equation y = -7.9099x + 2.894 showed an extremely significant correlation between the NNE and NNW longshore gradients ($R^2 = 97.37\%$). Therefore, it must be concluded that beach formation processes along the Penarth foreshore do not change with beach heading and although beach responses differed depending on direction, they were interrelated irrespective of external factors. This was demonstrated with:

- pre and post barrage construction, (control of flow of the Rivers Taff and Ely),
- tidal and storm conditions (including September 1999 which showed a southerly loss of beach material from the foreshore) and
- changes in direction of longshore drift from north to south and south to north.

6.6 Summary

This Chapter analysed the results, for the five-year monitoring period, of the four surveys undertaken along 1,245m of the Penarth foreshore.

These included:

• Longshore Profile: Stations 1 (0m) to station 16 (750m) inclusive;

• Foreshore Grid: A 7,200m² area of foreshore fronting the Esplanade,

bounded by station 6 (150m) and station 14 (390m)

longshore;

• Beach Profiles: Variable area of foreshore fronting the Esplanade and

bounded by station 6 (150m) and station 14 (390m)

longshore and -10m line to longshore profile and

depth of closure, cross-shore;

• Link Survey: Longshore profile between station 16 (750m) and

mid point of groyne 6, denoted 1k, (1,245m).

Conclusions drawn from the analysis of these surveys are summarised in the following sections.

6.6.1 Longshore Profile

This was considered in three parts:

- South of the old lifeboat slipway;
- Old lifeboat slipway to Penarth Pier;
- Penarth Pier to groyne 4.

Surveys were undertaken every September and April between September 1997 and September 2002. The first survey in September 1997 clearly showed changes in beach processes since 1992 and these had resulted in alterations to the beach conditions described by Wallingford (1992) and Bullen (1993a). Anecdotal evidence had narrowed the timescale of this change to between 1995 and 1997. Evaluations of photographic evidence, OS maps and survey data clearly showed a reversal of the

accepted northerly longshore drift (Wallingford, 1992; Bullen, 1993a; SBCEG 1999).



Plate 6.67: Groyne 1 at high tide, September 2002.



Plate 6.68: Groyne 2 at high tide, September 2002.

Although there was no statistical significance in the change of beach levels between the start and end of the monitoring period, there were other significant outcomes. Temporal analysis of beach levels indicated variability in coastal processes between September 1997 and April 2000. This was particularly highlighted in September 1999 by reductions in beach

level following indications of recovery. However, between April 2000 and September 2002, all temporal models, including the full profile, significantly represented a trend of increased beach levels with time (y = 0.0066x + 2.269; $R^2 = 96.9\%$: stations 1 to 26; 0 to 750m longshore). Furthermore, it had been demonstrated that there was more uniformity in beach evolution over this time period and that longshore drift had returned to the accepted northerly movement. This was clearly shown on comparison of Plates 6.67 and 6.68 (September 2002), when beach levels were higher on the southern flanks of groynes 1 and 2, with Plate 6.4 (September 1997) when they were lower.

6.6.2 Foreshore Grid

Surveys were undertaken every September and April between September 1997 and September 2002. Analysis indicated that the north to south longshore drift identified to April 1998 was not an isolated occurrence and was considered the on-going cause of the foreshore erosion. Evaluation of the September 1999 survey highlighted the potential effects of an easterly storm in May 1999 with recorded inter-survey losses of beach material of $1,148\cdot 4m^3$ (2,526 tonnes), equivalent to $0\cdot 160m^3/m^2$ (350 kg/m²) of beach surface. These observations would be further evaluated in conjunction with sea level and wind data (Chapter 8). Analysis of longshore gradients between September 1997 and September 2002 at the four cross-shore positions of the foreshore grid, showed significant differences (t = $2\cdot 664$; df = 6; p < $0\cdot 05$). The gradients in September 2002, were not as steep in a northerly direction (Table 6.16) and this was verified by a reduction in cross-shore gradients at station 6 (150m) and increase at station 14 (390m; Table 6.17).

Spatial consideration of the foreshore grid in two halves: station 6 (150m) to station 10 (270m) and station 10 (270m) to station 14 (390m), was justified. Temporal evaluations of changes in beach level agreed with conclusions from the analysis of the longshore profile. Variation in beach levels between September 1997 and April 2000 indicated variability in

coastal processes whilst between April 2000 and September 2002 they indicated that coastal processes had become more settled. Furthermore, it was demonstrated that there was a trend of increasing beach levels over the five years and that longshore trends were similar, irrespective of cross-shore location. Importantly, evaluation of the foreshore grid confirmed the change of a southerly sediment transport regime in September 1997 to a northerly direction in September 2002. In addition, this change was shown to have occurred by April 2000.

Survey data was imported into Dplot ground modelling software to represent beach contours. From analysis of changes in beach morphology, it was shown that erosion induced a southerly movement of beach contours and vice versa, whilst photographic evidence demonstrated that use of the software was valid. It was subsequently justified that as the contour MHW was a shoreline indicator and beach level represented shoreline position, then there was a relationship between change in WCB MHW and gain/loss of beach material. The model representing this relationship y = 512.6x - $165.49 (R^2 = 52.54\%)$ quantitatively verified that reductions in WCB MHW resulted in beach loss and vice versa. Furthermore, a quantitative relationship existed between the average WCB MHW and average beach The regression model, y = 44.411x - 97.149 (R² = 61.61%) once level. again confirmed that a reduction in WCB MHW would correspond to erosion of the beach. Furthermore, this model was validated by apparently representing conditions, prior to the start of the research.

6.6.3 Beach Profiles - Penarth foreshore

Surveys were undertaken each September and April between September 1998 and September 2002. The cross-shore position of the depth of closure was 50m seaward of the longshore profile, at a water depth in the range 5·443 to 5·591m. Consideration of the cross-shore profiles, at stations 6 (150m) and 14 (390m) showed that the greatest temporal variation in beach level was at the back beach with little variation beyond the depth of closure. The temporal variations in cross-shore profiles further

supported conclusions from the analysis of the longshore profile and foreshore grid. By September 2002 beach levels had reduced at station 6 (150m) and increased at station 14 (390m) which corresponded to reductions and increases at these locations, in the seaward edge of the pebble beach. These results not only confirmed the change in direction of longshore drift, from a southerly to northerly direction by September 2002, but also the effects of cross-shore transport on the beach profiles. It was shown that erosion, as part of a northerly sediment transport, did not have as severe an effect on beach morphology as erosion in a southerly direction.

Significant temporal models represented the variation of the shoreline indicators MHW and DoC. Furthermore, there was significant correlation between the variation of both indicators. This model v =4.4167x - 53.963 (R² = 83.58%) combined with WCB DoC = $17.035 + tan^{-1}$ $(x_{14} - x_6)/240$ produced a simple tool to rapidly assess the health of the Penarth foreshore. In addition, the significant relationship between the WCB MHW and mean beach level would enable estimates to be made of the shoreline indicator (WCB MHW) and shoreline movement (mean beach level). Simply by measuring the distance from the longshore profile to the seaward edge of the pebble beach at station 6 (150m) and station 14 (390m) and substituting, the WCB DoC is obtained. Qualitatively, increases in WCB DoC indicate increases in mean beach level and vice versa which could be determined quantitatively, by substitution into the significant regression models relating WCB DoC to WCB MHW and subsequently into the model correlating WCB MHW to mean beach level. Results could then be compared with past data for interpretation of significance. Further benefits of this simple approach to assess beach health over any time period is that the WCB DoC can be used to quickly:

- assess the short-term effects of storms;
- indicate changes in longshore transport over any time period;
- provide an early warning of problems;
- compare time now with past records.

As such this equation is a management tool and indicator. It is therefore suggested that the initials MRI (Management Response Indicator) should be adopted for WCB DoC. Therefore, the equation becomes:

MRI =
$$17.035 + \tan^{-1} \frac{(x_{14} - x_6)}{240}$$
 o

Although this equation is Penarth specific, the general form could be tested at other beach locations

MRI = WCB Back of beach +
$$tan^{-1} \frac{(x_1 - x_2)}{240}$$

where

- Back of beach may be dune line, seawall, etc
- x₁ and x₂ are fixed distances along back of beach.

A temporal analysis of sediment budgets indicated beach recession but when repeated subsequent to the erosion event in September 1999, beach accumulation was indicated. Whilst the timeframe for both evaluations was too short for meaningful evaluations of trends, they did highlight the impact of a storm event. Comparisons of inter-survey volume changes between the foreshore grid and pebble beach overall, bounded longshore by stations 6 (150m) and 14 (390m), showed significant positive correlation.

6.6.4 Link Survey

Eight surveys were undertaken along this section due to limited access to the beach during the construction of the barrage breakwater. This length of foreshore included both NNE and NNW headings. Analysis of the NNE heading, 750 to 1,245m longshore, showed an overall trend of falling beach levels. Interestingly, to April 2000 the trend was of increased beach levels only to be subsequently reversed. The same trends were found on analysis of data for the NNW section of the survey. The temporal variation of beach levels between September 1997 and April 2000 and April 2000 to September 2002, produced significant relationships. Increased beach levels along this section would be consistent with a north to south sediment

movement, whilst reductions in level would indicate the transport mechanism was reversed. From an analysis of the temporal variation of longshore gradients between the two beach headings (NNE and NNW), it was found that there was significant correlation between the two: y = -7.9099x + 2.894 ($R^2 = 97.37\%$). This meant that irrespective of direction of longshore transport and external factors such as storms and anthropogenic activities, beach responses to coastal processes along this section were interrelated.

The remaining length of monitored foreshore still to be considered is the groyned beach adjacent to the barrage breakwater. This will now be evaluated in Chapter 7.

CHAPTER 7: GROYNED BEACH - PENARTH HEAD.

7.1 Introduction

This Chapter records and discusses the results and implications of the ten surveys undertaken between September 1997 and September 2002, on the groyned beach adjacent to the main breakwater for the Cardiff Bay Barrage. The rationale of these surveys was discussed in Section 5.8.

7.2 Survey Results

Plate 7.1 shows an aerial view of the groyned beach, and Figure 7.1 is a diagrammatic representation of the locations of beach levels that were taken in each survey along this section.

Plate 7.1 has been removed for copyright reasons.

Plate 7.1: Aerial photograph of groyned beach (Source: Getmapping plc).

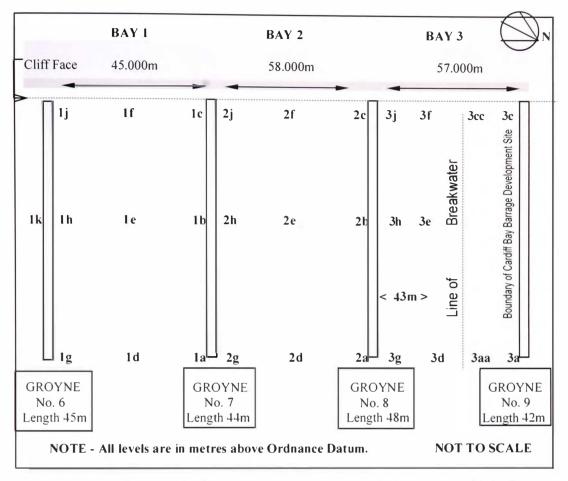


Figure 7.1: Diagrammatic representation of groyned beach

Figure 7.2 is an OS map of the beach and shows the location of groyne 9, which was to become incorporated into the main breakwater for the Barrage. Beach levels and mean values for all surveys are shown in Tables 7.1 to 7.10 inclusive with examples of the field data collection sheets included in Appendix 3. As the groynes varied in length from 44m to 49m, the cross-shore distance was limited to 44m for each bay and levels were taken at 0, 22 and 44m. This was repeated centrally and each side of the groyne, in all bays, as indicated, whilst longshore distance was dictated by the length of each bay. Note that all levels are in metres above Ordnance Datum Newlyn.

Figure 7.2 has been removed for copyright reasons.

Figure 7.2: Location plan of groyned beach. (Source: Ordnance Survey)

Table 7.1: Beach Levels Survey 1 – September, 1997.

		Bay 1			Bay 2			Bay 3	
	6.570	6.495	6.690	5.910	6.595	6.895	5.815	6.370	6.145
4.270	3.775	3.760	5.045	3.560	4.160	4.220	3.190	3.440	3.410
	1.130	0.840	1.130	1.165	0.870	1.160	1.160	1.070	0.810
	Mea	n Level 3.9	57m	Mea	n Level 3.8	37m	Mean	Level = 3.	490m
			Mean I	Beach Lev	vel Bays 1, 2	2 and $3 =$	3·761m		

Table 7.2: Beach Levels Survey 2 – November, 1997

		Bay 1			Bay 2			Bay 3			
	6.940	6.800	6.660	5.970	6.435	6.380	5.602	6.100	6.245		
3.975	3.415	3.515	4.700	3.160	3.575	4.280	3.485	*	3.460		
	1.090	1.005	1.190	1.170	0.970	1.235	1.220	1.145	0.995		
	Mea	n Level 3.	924m	Mea	n Level 3.	686m	Mea	n Level 3.	534m		
			Mean I	Beach Level Bays 1, 2 and $3 = 3.715 \text{ m}$							
	*Stockpile of material/spoil from Breakwater Construction										

Table 7.3: Beach Levels Survey 3 – February 1999

		Bay 1			Bay 2			Bay 3		
	7.205	7.040	6.925	6.025	6.775	7.065	5.795	6.675	6.655	
4.790	3.765	3.755	4.140	3.195	3.245	4.110	3.485	3.050	3.710	
	1.405	I·145	1.395	1.300	0.745	1.275	1.275	1.615	1.225	
	Mea	n Level 4	086m	Mea	n Level 3.	748m	Mea	n Level 3.	721m	
			Mean	Beach Level Bays 1, 2 and 3 = 3.852m						
	Nev	v Breakwa	ater in pos	ition (43.3	m northe	rly from g	groyne 8)			

Table 7.4: Beach Levels Survey 4 – September 1999

		Bay 1			Bay 2			Bay 3	
	6.385	6.745	6.805	6.040	6.345	6.840	5.800	6.005	6.560
4.095	3.645	3.660	3.755	3.210	3.485	4.385	3.665	3.350	3.910
	1.300	0.975	1.175	1.110	0.895	1.545	1.575	1.645	1.345
	Mea	n Level 3.8	327m	Mea	n Level 3.	762m	Mean	Level = 3	·761m
			Mean B	Beach Level Bays $1, 2$ and $3 = 1$			3·783m		

Table 7.5: Beach Levels Survey 5 - April 2000

		Bay 1			Bay 2			Bay 3		
	6.775	6.835	6.970	5.840	6.445	7.085	5.730	6.180	6.755	
4.145	3.485	3.655	3.860	3.125	3.510	4.290	3.620	3.625	4.330	
	1.265	1.035	1.220	1.220	0.860	I·460	1.505	1.935	2.075	
	Mea	n Level 3.	900m	Mea	Mean Level 3·759m Mean Level = 3·973m					
			Mean I	Beach Lev	el Bays 1,	2 and 3 =	3·877m			

Table 7.6: Beach Levels Survey 6 – September 2000

	Bay 1			Bay 2			Bay 3		
	7.095	7.000	7.370	6.130	6.250	6.560	6.065	6.590	6.690
4.060	3.375	3.470	3.920	3.035	3.450	4.255	3.655	3.575	4.355
	0.985	0.710	0.920	0.925	0.890	1.425	1.505	1.765	1.625
	Mea	n Level 3.	872m	Mea	n Level 3.	658m	Mean	Level = 3	·981m
			Mea	Beach L	evels 1, 2	and $3 = 3$	837m		

Table 7.7: Beach Levels Survey 7 – April 2001

		Bay 1			Bay 2		Bay 3				
	6.620	6.665	7.085	6.160	6.135	6.585	6.085	6.660	6.915		
4.245	3.635	3.800	4.135	3.370	3.545	4.315	3.635	3.570	4·160		
	1.245	1.050	1.265	1.190	0.970	1.510	1.510	1.695	1.770		
	Mea	n Level 3.9	944m	Mean Level 3·753m Mean Level = 4·000m							
	Mean Beach Levels 1, 2 and 3 = 3.899m										

Table 7.8: Beach Levels Survey 8 – September 2001

		Bay 1			Bay 2		Bay 3		
	6.715	6.755	6.825	5.970	6.665	6.985	6.100	6.635	6.890
4.165	3.685	3.865	4.185	3.075	3.245	4.320	3.510	3.605	4.465
	1.195	1.025	1.235	1.205	0.905	1.445	1.485	1.625	1.705
	Mea	n Level 3.	943m	Mean Level 3·757m Mean Level = 4·002m					
			Mea	an Beach Levels 1, 2 and 3 = 3.901m					

Table 7.9: Beach Levels Survey 9 – April 2002

		Bay 1			Bay 2			Bay 3		
	6.705	6.735	7.060	5.875	6.570	7.150	6.095	6.620	6.900	
4.235	3.560	3.705	4.205	3.290	3.315	4.300	3.530	3.650	4.445	
	1.205	1.105	1.225	1.195	0.870	1.390	1.470	1.605	1.720	
	Mea	n Level 3.9	945m	Mean Level 3·773m Mean Level = 4·004m					004m	
			Mea	n Beach L	evels 1, 2	and $3 = 3$	907m			

Table 7.10: Beach Levels Survey 10 – September 2002

		Bay 1			Bay 2			Bay 3	
	6.645	6.820	6.860	6.080	6.595	7.110	6.175	6.665	6.930
4.450	3.545	3.620	4.270	3.265	3.290	4.290	3.510	3.675	4.420
	1.345	1.115	1.245	1.180	0.770	1.340	1.440	1.590	1.745
	Mea	n Level 3.	941 m	Mea	n Level 3.	769m	Mean	Level = 4	·017m
			Mea	n Beach Levels 1, 2 and 3 = 3.909m					

7.2.1 Mean Beach Level

The variation of mean beach level from Tables 7.1 to 7.10 inclusive, are shown in Table 7.11

Table 7.11: Temporal variation of mean beach levels in Groyned Beach.

Survey	Date	Time		Mean Beach L	evel (m) AOD	
		(Months)	Bay 1	Bay 2	Bay 3	Overall
1	9/97	0	3.957	3.837	3.490	3.761
2	11/97	2	3.924	3.686	3.534	3.715
] 3	2/99	17	4.086	3.748	3.721	3.852
4	9/99	24	3.827	3.762	3.761	3.783
5	4/00	31	3.900	3.759	3.973	3.877
6	9/00	36	3.872	3.658	3.981	3.837
7	4/01	43	3.944	3.753	4.000	3.899
8	9/01	48	3.943	3.757	4.002	3.901
9	4/02	55	3.945	3.773	4.004	3.907
10	9/02	60	3.941	3.769	4.017	3.909

7.2.2 Volume and Mass Change

Mean beach levels from Table 7.11 were converted into volume and mass change between surveys, that is, gain or loss in volume and mass. According to Coduto (1999) the unit weights of soil samples obtained in the laboratory are very misleading, especially gravelly soils, as they are affected by sample disturbance. He further recommended not to attempt unit weight measurements on these soils but to use typical ranges of unit weights. Therefore, from Coduto (1999) and Bell (1993), for the combination of beach material, that is, marl and limestone cobbles, an estimate of 2,200 kg/m³ was adopted.

Volume change between surveys was determined from table 7.11 data, as per the following example:

```
Mean level Bay 1, Survey 1 = 3.957m

Mean level Bay 1, Survey 2 = 3.924m

Change in mean level between surveys = 3.924-3.957=-0.033m

Area of Bay 1 = 45 \times 44 = 1,980m^2

\therefore Volume change between surveys = -65.3m^3
```

This equates to a loss of beach material of $65.3 \times 2,200 \text{ kg} = 143,660 \text{ kg}$ = 143.7 tonnes.

The area of Bay $2 = 44 \times 58 = 2,552\text{m}^2$ and Bay 3: $44 \times 57 = 2,508\text{m}^2$ for Surveys 1 and 2, and subsequently $44 \times 43 = 1,892\text{m}^2$ due to the completion

of the breakwater. Surveys were analysed and gain or loss of beach material determined (Table 7.12).

Table 7.12: Gain/Loss (+/-) of Beach Material between Surveys.

	V	olume gain	/loss (+/-) n	n³	M	ass gain/los	s (+/-) tonn	ies
Surveys	Bay 1	Bay 2	Bay 3	Overall	Bay 1	Bay 2	Bay 3	Overall
I and 2	- 65.3	- 385.4	+ 110.4	- 340·3	- 143.7	- 847.8	+ 242.9	- 748·6
2 and 3	+320.8	+ 158·2	+ 353.8	+ 832.8	+ 705.7	+ 348.0	+ 778·4	+1832·1
3 and 4	- 512.8	+ 35.7	+ 75.7	- 401.4	-1128-2	+ 78.5	+ 166·5	- 883·2
4 and 5	+ 144.5	- 7.7	+ 401·1	+ 537.9	+ 318.0	- 16.9	+ 882·4	+1183.5
5 and 6	- 55.4	- 257.8	+ 15.1	- 298·1	- 122.0	- 567-2	+ 33.3	- 655.9
6 and 7	+ 142.6	+ 242.4	+ 35.9	+ 420.9	+ 313.6	+ 533.3	+ 79·1	+ 926.0
7 and 8	- 2.00	+ 10.2	+ 3.8	+ 12.0	- 4.4	+ 22.4	+ 8.3	+ 26.3
8 and 9	+ 4.00	+ 40.8	+ 3.8	+ 48.6	+ 8.8	+ 89.8	+ 8.3	+ 106.9
9 and 10	- 8.00	- 10.2	+ 24.6	+ 6.4	- 17.6	- 22.4	+ 54·1	+ 14·1
Net	- 31.6	- 173.8	+1024-2	+ 818-8	- 69.8	- 382·3	+2253.3	+1801-2

Note: corrections have been made to volumes in line with the reduction in bay 3 width.

7.2.3 Longshore Gradients

Mean and central longshore gradients were determined as follows:

For Survey 1 – Bay 1 (Table 7.1)

Mean level northern side
$$= \frac{6 \cdot 690 + 5 \cdot 045 + 1 \cdot 130}{3} = 4 \cdot 288 \text{m}$$

Mean level southern side
$$= \frac{6 \cdot 750 + 3 \cdot 775 + 1 \cdot 130}{3} = 3 \cdot 885 \text{m}$$

Length Bay 1
$$= 45m$$

∴ Mean longshore gradient Bay 1 =
$$\frac{4.288 - 3.885}{45} \times 100\% = +0.9\%$$

Central grid level northern side = 5.045m

Central grid level southern side = 3.775m

Length Bay 1 = 45m

$$\therefore$$
 Central longshore gradient Bay 1 = $\frac{5 \cdot 045 - 3 \cdot 775}{45} \times 100\% = +2 \cdot 8\%$

Note that positive gradients indicate rising gradients in a northerly direction.

This process was repeated for each bay and for all surveys (Table 7.13)

Table 7.13: Central and mean longshore gradients

		Gradient (%)		
Survey	Date	Bay 1	Bay 2	Bay 3
1	9/97	2.8 (0.9)	1.1 (0.9)	0.5 (0.2)
2	11/97	2.9 (0.8)	1.9 (0.9)	- 0.1 (0.3)
3	2/99	0.8 (0.1)	1.6 (1.1)	0.5 (0.8)
4	9/99	0.2 (0.3)	2.0 (1.4)	0.6 (0.6)
5	4/00	0.8 (0.4)	2.0 (1.5)	1.7 (1.8)
6	9/00	1.2 (0.6)	2.1 (1.2)	1.6 (1.1)
7	4/01	1.1 (0.9)	1.6 (1.0)	1.2 (1.3)
8	9/01	1.1 (0.5)	2.1 (1.4)	2.2 (1.5)
9	4/02	1.4 (0.8)	1.7 (1.4)	2·1 (1·5)
10	9/02	1.6 (0.6)	1.8 (1.3)	2·1 (1·5)

Mean values are shown in brackets.

7.2.4 Cross-shore Gradients

Mean and central cross-shore gradients were determined as follows:

For Survey 1 – Bay 1

Mean Level back beach
$$= \frac{6 \cdot 750 + 6 \cdot 495 + 6 \cdot 690}{3} = 6 \cdot 645$$
m

Mean Level front beach
$$= \frac{1 \cdot 130 + 0 \cdot 840 + 1 \cdot 130}{3} = 1 \cdot 033 \text{m}$$

Cross-shore distance, Bay 1 = 44m

$$\therefore \text{ Mean cross-shore gradient} = \frac{6 \cdot 645 - 1 \cdot 033}{44} \times 100\% = +12 \cdot 8\%$$

Central level back beach = 6.495m

Central level front beach = 0.840m

Cross-shore distance Bay 1 = 44m

$$\therefore \text{ Central cross-shore gradient } = \frac{6 \cdot 495 - 0 \cdot 840}{44} \times 100\% = +12 \cdot 8\%$$

Note that positive gradients indicate rising gradients towards back of beach.

This process was repeated for each bay and for all surveys, and the results tabulated in Table 7.14.

Table 7.14: Central and mean cross-shore gradients

		Gradient (%)		
Survey	Date	Bay 1	Bay 2	Bay 3
1	9/97	12.8 (12.8)	13.0 (12.3)	12.0 (11.6)
2	11/97	13.2 (13.0)	12.4 (11.7)	11.3 (11.1)
3	2/99	13.4 (13.0)	13.7 (12.5)	11.5 (12.2)
4	9/99	13·1 (12·5)	12.4 (11.9)	9.9 (10.5)
5	4/00	13.2 (12.9)	12.7 (12.0)	9.6 (10.0)
6	9/00	14.3 (14.3)	12.2 (11.9)	11.0 (10.9)
7	4/01	12.8 (12.7)	11.7 (11.5)	11.3 (11.1)
8	9/01	13.0 (12.8)	13·1 (12·2)	11.4 (11.2)
9	4/02	12.8 (12.9)	13.0 (12.2)	11.4 (11.2)
10	9/02	13.0 (12.6)	13.2 (12.5)	11.5 (11.4)

Mean values are shown in brackets.

7.3 Evaluation

7.3.1 *Survey 1 – September 1997*

This survey provided the baseline data against which changes along the groyned beach were measured. Wallingford (1992) reported a difference in level of over 1m, whilst Bullen (1993a) noted a difference of 1 – 2m across these groynes and both agreed that the southern flanks were higher. From Table 7.1, it can be seen that the differences in level across groynes 6, 7 and 8 were 0·495, 1·485 and 1·030m respectively. These differences, at fixed locations that were not necessarily the maximum, initially indicated that there had been little change along this section.



Plate 7.2: Groyne 8, bays 2 and 3 (September, 1997 see Figure 7.2).

Plate 7.2 showed groyne 8 and bays 2 and 3, where from Table 7.1, it can be seen that the average level in bay 2 was 0·347m higher. The implications of this level difference meant that eroded cliff material remained at the cliff toe in bay 2. The difference in beach material distribution between the bays is clear, as seaward of the groyne limits, mud covered marl was predominant and the groynes successfully retained beach material. The proximity of houses at the cliff top (approximately 15m) may result in future problems when considered in conjunction with Plate 7.3.

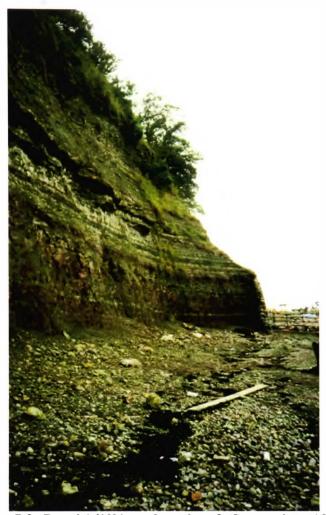


Plate 7.3: Beach/cliff interface, bay 3 (September, 1997).

Here there was evidence of accelerated cliff erosion in bay 3. Jones and Williams (1991) concluded that erosion was controlled by the extent of beach deposits at the cliff base with the volume of beach face material being the dominant explanatory variable. Therefore, the lack of beach face material in bay 3 may well have contributed to the increased erosion along this section of cliff. However, according to Weigel (2002) and Basco (1999), increased erosion can be caused by sea walls acting as a groyne or preventing erosion of an upstream cliff. North of groyne 9 (Figure 7.2) the cliff had been faced with masonry and consequently, a combination of seawall, groyne and flanking effects would also have contributed to this increased erosion.

Plate 7.4 showed the bottom of bay 3, its relation to the Barrage cofferdam construction, the breakwater caissons and groyne 9, which was

the site boundary of the Cardiff Bay Barrage. The beach at the top was mainly weathered red and green marl from the cliff face whilst larger limestone pebbles were retained towards the southeast of the bay. Table 7.13 indicated that there was a reduction in longshore gradients across the centre line of the three bays, that is, 2.8% (Bay 1), 1.1% (Bay 2) and 0.5% (Bay 3), whilst Table 7.14 showed a similar reduction in mean cross-shore gradients, that is, 12.8% (Bay 1), 12.3% (Bay 2) and 11.6% (Bay 3). Again this was consistent with lower beach levels adjacent to the barrage.



Plate 7.4: Barrage breakwater construction (September 1997).

Evidence of change from conditions reported by Wallingford (1992) and Bullen (1993a) are illustrated in Plate 7.5. The northern face of groyne 8 was clearly being undermined, as were other locations along the groyne (Plate 1.11). This incidentally indicates the uncertainty as to the depth of the marl bedrock below the beach surface as the exposure of the marl under the steps is at least 3m higher than that exposed immediately seaward of the groynes. Plate 7.6 shows the southern side of groyne 8 and would indicate a loss of beach covering. Unfortunately no past profiles along this coastline stretch were available for comparison purposes. Consequently another factor for consideration would be the location of the cofferdam and caissons which might have caused a localised change in tidal and current patterns which contributed to the loss of beach material. This would help to explain

the longshore and cross-shore changes approaching the barrage, that is, a consistent reduction of 60% (longshore) and 0.3° (cross-shore).



Plate 7.5: Northern flank, groyne 8 (September 1997).



Plate 7.6: Southern flank, groyne 8.

7.3.2 Survey 2 – November, 1997

The reasons for the relatively short time interval of two months between surveys 1 and 2 were two-fold. Firstly, the beach area was to be closed to public access for at least a year whilst construction of the main breakwater was undertaken. Secondly, the contractor had started stockpiling spoil and armouring in bay 3 (Figure 7.2), in preparation for the construction. From Table 7.2, it can be seen that no measurement was possible at the mid point of the bay grid, as the spoil heap covered a plan area of approximately 200m² with a height in excess of 4m (approximate volume 270m³). Table 7.12 showed that bays 1 and 2 lost over 450m³ of beach material with bay 2 suffering a loss greater than 385m³, corresponding to a fall in average beach level of 0·151m (Table 7.11). Conversely, bay 3 gained 110m³ of beach material corresponding to a rise in average beach level of 0·044m.

It appeared that tidal action on the stockpile in bay 3 contributed to the rise in mean beach level, as the location of the material coincided with increased beach levels. At other locations, beach levels fell, similar to levels in bays 1 and 2 and the underlying trend corresponded to a net loss of beach material, compared with survey 1, of 340m³ or approximately 750 tonnes (Table 7.12).

7.3.3 Survey 3 – February, 1999

This survey was undertaken on completion of the breakwater construction and subsequent available access to the beach. Groyne 9 (Figure 7.2) was no longer in existence having been incorporated into the breakwater construction. Consequently, the width of bay 3 reduced from 57m to 43m, a loss of 14m. From Table 7.11 there was an average increase of beach level of 0·137m which corresponded to an overall increase in volume of 833m³ (>1,830 tonnes; Table 7.12). This was clearly shown in Plate 7.7 where the beach had extended beyond the extremity of groyne 8 and provided a stark contrast to conditions shown in Plate 7.2. This was

further evidenced by the average increase of beach level at the groyne limits of bay 3 of 0.252m (Tables 7.2 and 7.3). Plate 7.8 shows the location where the breakwater joins the cliff face. Comparisons with Plate 7.3 indicated that the effects of increased beach levels in bay 3 meant weathered material remained at the cliff toe.



Plate 7.7: Groyne 8, bays 2 and 3 (February, 1999).



Plate 7.8: Beach/cliff interface, bay 3 (February, 1999).

Similarly, when Plate 7.9 was compared with Plate 1.11, the increased beach levels provided protection to the steps on the northern face of groyne 8. The barrage construction was approaching completion and Plate 7.10 showed the varying level of the beach adjacent to the completed breakwater. Comparisons with Plate 7.4 once again illustrated the changes that occurred between Surveys 1 and 3.



Plate 7.9: Steps northern flank, groyne 8 (February 1999).



Plate 7.10: Barrage breakwater (February 1999).

In addition, in May 1998, the cofferdam was removed on completion of the construction of the locks, sluices and fish pass. This removal, in addition to stockpiling material for the construction of the rock spit between

the caisson breakwater and Penarth cliffs, could have introduced large quantities of material into the coastal zone and influenced beach levels. In an attempt to quantify this component, contact was made with CBDC (client), Bechtel (consultants) and Gibb (contractors). In a written response, included in Appendix 4, Boese (1999) detailed the stockpile as a mixture of limestone rockfill and dredged sand. As the fines content was too high for the grading requirements of the rockfill to the core of the rock spit, the material was placed on the beach for tidal action to remove the fine material. During construction it was estimated that 600m³ was stockpiled on the beach from which 60 to 120m³ (10 to 20%) of sand and gravel was left on the beach. In addition, a small amount of the larger rockfill was left bringing the maximum volume of material left on the beach to around 150m³. This would equate to approximately 330 tonnes and an overall rise in beach level of 0·023m across all bays or 0·079m solely in bay 3.

7.3.4 Survey 4 – September, 1999

During the inter-survey period, work had started on the final closure of the barrage (Plate 7.11), which would ultimately lead to tidal exclusion and form the freshwater lake.



Plate 7.11: Barrage closure (September 1999).

This survey demonstrated the continuing rise in mean beach level of bay 3 (3.761m), which was now approximately the same as that in bay 2

(3.762m; Tables 7.4 and 7.11). Both these bays gained material between surveys 3 and 4 as illustrated in Table 7.12 (approximately 111m³/240 tonnes in total). However, bay 1 reduced in mean level by 0.259m (Table 7.11). Therefore, the groyned beach lost in total approximately 401m³ (833) tonnes) of beach covering (Table 7.12). During the five year monitoring programme, this was the largest loss of material in bay 1 over an intersurvey period (512.8m³; 1,128.2 tonnes) and corresponded to the lowest longshore gradient 0.2% (Table 7.13) and mean cross-shore gradient 12.5% (Table 7.14). The loss of material along this section of coastline mirrored what had happened on the foreshore during the same period (Section 6.3.5) Therefore, it appeared that storms from an easterly direction also caused significant erosion at this location and that breakwater influence prevented similar losses in bays 2 and 3. In addition, this was the first September survey to identify what was to become a typical trend that over a summer period, that is, April to September, bay 1 would consistently suffer a loss of beach covering.

7.3.5 Survey 5 – April, 2000

On the 4th November 1999, the barrage sluice gates were officially closed to prepare for the creation of the permanent freshwater lake, which will be maintained at 4·5m AOD. Thirty days after initially impounding the bay, the Environment Agency ordered CBDC to return the area to a tidal flow due to concerns regarding the technical operation of the barrage that included the sluices and fish pass. There were also concerns about compliance with mandatory dissolved oxygen levels in the bay which were only met by the manual input of oxygen into the lake and once weekly overnight emptying and refilling of the bay (Phillips and Williams, 2000). However, despite the initial problems with the barrage operation, tidal conditions adjacent to the groyned beach had now permanently changed for at least the next century (barrage design life 125 years). Therefore, this survey was the first following significant changes to tidal movement patterns. From Tables 7.5, 7.11 and 7.12 it can be seen that the mean level in bay 3 rose significantly by 0·212m, equivalent to a gain in beach material

of $401\,\text{m}^3$ (882 tonnes). This corresponded to the largest gain during the monitoring period and resulted in the largest increase in cross-shore gradient to 1.7% (Table 7.13).



Plate 7.12: Sediment accumulation at cliff toe (April 2000).



Plate 7.13: Strandline and beach/cliff interface bay 3 (April 2000)

These increases allowed further accumulation of material at the cliff toe (Plates 7.12 and 7.13) whilst the strandline was located a minimum 5m from the cliff face. Plate 7.14 showed bay 3, groyne 8 and the rock spit joining the breakwater to the cliffs. A comparison with Plate 7.7 showed once again that the material composition clear of the groyne field had changed, similar to conditions in Plate 7.2, despite there being an overall increase in beach level.



Plate 7.14: Groyne 8, bays 2 and 3 (April, 2000).

Further investigation revealed that since September 1999, cross-shore levels on the northern flank of groyne 8 had fallen by an average of 0.061m, whilst centrally and along the breakwater they had risen by an average of 0.247m and 0.448m respectively (Tables 7.4 and 7.5). In addition, from Table 7.11, it can be seen that for the first time, mean beach levels were higher in bay 3 than either bays 1 or 2. Consequently the completion of the barrage appeared to have had a significant impact on adjacent beach levels. However, over the same period beach levels had fallen by an average of 0.003m in bay 2 and risen by an average of 0.073m in bay 1 (Table 7.11). Overall, this resulted in a gain of 537.9 m³ (1,183.5 tonnes) for the groyned beach (Table 7.12). Furthermore a trend was once again established for bay 1 where, typically over a winter period, that is, September to April, this part of the beach experienced rising levels with consequent increased covering, the opposite to what had happened over the summer period.

7.3.6 *Survey* 6 – *September*, 2000

This survey once again reinforced previous trends. Firstly, there was a further increase in mean level of bay 3, which was again higher than mean levels of both bays 1 and 2 (Table 7.11). Secondly, there was again a loss of

beach material over the summer period in bay 1, i.e. 55·4m³ (122 tonnes) (Table 7.12). However, levels in bay 2 fell by an average 0·101m, which equated to 257·8m³ (567·2 tonnes; Tables 7.11 and 7.12). Examination of both longshore and cross-shore gradients (Tables 7.13 and 7.14) did not indicate any significant change to correlate this loss. Overall beach levels for the groyned beach fell by an average 0·040m, equivalent to a loss of 298·1m³ (655·9 tonnes) and was consistent with observed changes between April and September surveys.

7.3.7 Survey 7 – April 2001

From Tables 7.11 and 7.12 it can be seen that mean beach levels rose in all three bays over the winter period by an average of 0.062m, equivalent to 420.9m³ (926 tonnes). The greatest gain was in bay 2, where beach material increased by 242.4m³ (533.3 tonnes), reversing what had happened in the previous survey. Levels in bay 3 increased by 0.019m (Table 7.11) and again demonstrated that this part of the beach was the least volatile and was being subjected to relatively modest level changes. More importantly, by comparing Tables 7.6 and 7.7, increases in the back beach levels in bay 3 accounted for a significant part of the level increase which in turn, afforded greater protection to the toe of the cliffs.

7.3.8 Surveys 8, 9 and 10 – September 2001, April 2002 and September 2002.

From Tables 7.8 to 7.14 inclusive, there was relatively little difference in individual levels, mean beach levels, volume movement, longshore and cross-shore gradients between these surveys. Two years on, it appeared that dynamic equilibrium had been reached on the groyned beach between the tidal processes and the completed barrage. Consequently, these surveys are considered in tandem. Although there was little variation in mean beach levels compared with the earlier surveys, some trends were reinforced. Firstly bay 1 continued to fall in level with consequent volume loss between April and September, whilst rising in level and gaining material during the winter period September to April. Secondly, bay 3

continued the trend demonstrated throughout the monitoring period of increased mean beach levels in successive surveys. Overall, between September 1997 and September 2002 the most significant changes occurred in bay 3 where the mean beach level increased by 0.527m (Table 7.11), equivalent to a gain of beach material of 1,024·2m³ (2,253·3 tonnes; Table 7.12).

For completeness, the following photographs provide a record of the groyned beach on completion of the monitoring survey in September 2002. The stepped access across the rock spit onto the beach (bay 3), shown in Plate 7.15, suffered damage in just three years (Plate 7.16). This provided a timely reminder of the hostile environment and the power of the natural forces at work. Plates 7.17, 7.18 and 7.19 shows the breakwater from different positions along bay 3 and indicate variation in beach composition. The cross-shore gradient of 11.5% (Table 7.14), equivalent to a beach angle of 6.6° , could be seen and comparison of Plates 7.10 and 7.18 showed beach changes where the rock spit joined the caissoned breakwater.



Plate 7.15: Stepped access - main breakwater (September 2002).



Plate 7.16: Tidal damage to steps (September 2002).



Plate 7.17: Beach composition end of rock spit, bay 3 (September 2002).



Plate 7.18: Variation in beach covering, bay 3 (September 2002).



Plate 7.19: Back beach, bay 3 (September 2002).

Tidal processes had removed the sandy construction material that remained following cofferdam removal and rock spit completion. Plate 7.20 once again shows the accumulation of material at the cliff toe which reinforced the effects of increased beach levels when comparison was made with Plate 7.3 (September 1997). Plate 7.2 showed accumulations of material at the toe of the cliffs in bay 2, which, in September 1997 had a mean back beach, level of 6·467m (Table 7.1). The mean back beach level in bay 3 in September 2002 was 6·590m (Table 7.10) from which it can be concluded that the level increases in bay 3, also highlighted in Surveys 3 and 5, meant that tidal action no longer reaches the toe of the cliffs. This was further evidenced by Plate 7.21, which showed the corner of bay 3 adjacent to groyne 8. This was the lowest part of the back beach where there was a reduction in material at the toe of the cliffs.



Plate 7.20: Accumulations back beach, bay 3 (September 2002).



Plate 7.21: Back beach groyne 8, bay 3 (September 2002).



Plate 7.22: North view - groyne 8, bays 2 and 3 (September 2002).

Plate 7.22 showed groyne 8 and bay 3. If a line was drawn diagonally from the top of groyne 8 to the bottom of the rock spit, fine material was to the north and larger pebbles to the south. This was the same as Phillips (1999b) reported following the first survey in September 1997. Plate 7.23 showed the bottom of groyne 8 looking southwards across bay 2.



Plate 7.23: South view - groyne 8, bays 2 and 3 (September 2002).

Plates 7.24 and 7.25 show typical material distribution and accumulation of weathered material at the cliff toe in bay 2. The top of groyne 7 in Plate 7.26 had not been attached to the cliffs where the irregular marl face meant that there was a gap of approximately 0.6m. The mean level of this bay had fallen by 0.068m over the monitoring period (Table 7.11) whilst the mean level of the back beach had risen by an average of 0.128m (Tables 7.1 and 7.10). Observations made in bay 1 are illustrated in Plates 7.27 and 7.28. Accumulations of material were seen once more at the toe of the cliffs. The mean beach level had fallen by 0.016m since September 1997 (Table 7.11) although, as in the case of bay 2, the mean level of the back of the beach had risen by an average of 0.190m (Tables 7.1 and 7.10).



Plate 7.24: Beach compositon, bay 2 (September 2002).



Plate 7.25: Accumulations at cliff toe, bay 2 (September 2002).



Plate 7.26: Interface of cliff and groyne 7 (September 2002).



Plate 7.27: Beach composition, bay 1 (September 2002).



Plate 7.28: Groyne 6 and bay 1 (September 2002).

Therefore, in all three bays, the average back beach level had risen between September 1997 and September 2002. Surveys and Plates have shown that some conditions were similar to those described in Section 3.2.1, in that the Rhaetic formations along this section of the Penarth coast were still eroding, evidenced by cliff toe accumulations.

7.4 Assessment of Change

Survey data collected over the five-year monitoring period was subjected to qualitative and quantitative analysis to determine patterns and trends and consequently to justify identified changes. This analysis included both parametric and non-parametric tests as appropriate (Section 5.12 – Methodology).

7.4.1 Qualitative Analysis

Figure 7.3 produced from data given in Table 7.11, illustrated the temporal variation of mean beach level for all three bays and for the groyned beach overall. During the five years, bay 3 continually increased in level whilst bay 1 was more volatile and decreased in level by an average 0.061 m (Table 7.11). This was equivalent to a loss of beach of 31.6m³ (69.8 tonnes) (Table 7.12). Levels in bay 2, although less volatile between surveys decreased by an average 0.068m (Table 7.11), equivalent to 173.8m³/382·3 tonnes (Table 7.12). This meant that between September 1997 and September 2002, the groyned beach gained 818·8m³ (1,801·2 tonnes) overall (Table 7.12). Therefore, qualitatively, it appeared that there were significant changes in beach formation processes in bay 3 and these changes were not evident in the other two bays.

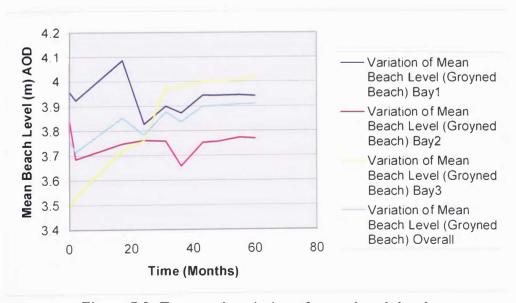


Figure 7.3: Temporal variation of mean beach level.

At time 2 months (Survey 2) beach levels increased in bay 3, probably due to spoil stockpiling (Section 7.3.2) whilst levels fell in the other two bays. It was possible that this stockpile masked a general trend of erosion. Time 17 months (Survey 3) saw levels increase in all three bays. Material left on completion of the breakwater (Section 7.3.3), if evenly distributed across all three bays, would be equivalent to an average rise in beach level of $150/(44 \times 146) = 0.023$ m. Conversely, if this material contributed only to bay 3, this would be equivalent to a rise of 150/(44 x 43) = 0.079m. From Table 7.11, average beach level rise of this period was 0.137m whilst bay 3 increased by 0.187m. Further examination of Plate 7.9 (Time 17 months) showed a build-up of material where the rock spit joined the caissoned breakwater, which was not apparent in Plate 7.19 (Time 60 months). This material was clear of the groyne field and unless the contractor seriously underestimated the amount of spoil entering the coastal environment on breakwater completion, then it is judged that this did not significantly contribute to rising beach levels. It is recognised that although this material was clear of the survey area, it may have affected erosion in the groyned beach by locally modifying the wave climate.

Over the next survey period, work had begun on the final closure of the barrage (Section 7.3.4). By September 1999 (time 24 months) levels rose in bays 1 and 2 but fell in bay 3. The tidal exclusion barrage was completed in November 1999 (Section 7.3.5) and by the next survey (time 31 months) bay 3 experienced the greatest single rise in level, 0·212m (Table 7.11). Levels also rose in bay 1 by 0·073m but fell slightly in bay 2 by 0·003m. Therefore, it appeared that the completed barrage had a significant effect on the adjacent beach. The next five surveys saw a continuing rise in level in bay 3 which now appeared to be more uniform than previous rises, 0·044m in total (Table 7.11). Consequently it is argued qualitatively that modifications to the coastal environment caused by the barrage and breakwater construction have contributed to increased beach levels in bay 3.

During the same period, that is, April 2001 (time 31 months) to September 2002 (time 60 months), beach level changes in bays 1 and 2 were not uniform. However, beach levels rose in both bays by a total of 0.041m in bay 1 and 0.010m in bay 2 (Table 7.11). Conversely previous surveys had seen falls in mean beach levels of 0.057m in bay 1 and 0.078m in bay 2 (Table 7.11). Although other coastal processes to be discussed in Chapter 8, influenced beach levels, it is argued that modifications to processes in the nearshore zone, caused by the completed barrage, contributed to the increased beach levels in these bays (mean level range subsequent to barrage construction was -0.057 to 0.041m = 0.098m (bay 1) and -0.078 to 0.010m = 0.088m (bay 2)).

7.4.2 Temporal Quantitative Analysis

Results were subsequently analysed quantitatively to ascertain the validity of perceived qualitative changes. Firstly, references have been made concerning the accumulation of material at the cliff toe in bay 3 (Sections 7.3.3 and 7.3.8), the first following Survey 3 (Section 7.3.3). At the start of monitoring, the mean back beach level in bay 3 was 6·110m (Table 7.1) whilst at the time of survey 3 it had risen to 6·375m (Table 7.3). By the end of five years it had progressively risen to 6·590m. According to Page and Oakley (2002) the MHW Springs for Cardiff was 12·300m relative to Chart Datum which itself is –6·300m relative to Ordnance Datum Newlyn, that is 6·000m AOD. Furthermore, the MHW at Cardiff is 4·500m AOD (Section 6.3.7). Furthermore, as Crest Nicholson (2002) predicted that high tide levels for Penarth only reach 12·500m Chart Datum (6·200m AOD) once a month for ten months of the year, the reasons for the recent accumulations and why they increased in quantity northwards along the bays were evident.

The quantitative process consisted in part of comparisons between surveys 1 and 10, that is, the start and end of the monitoring period. This was because there was no previous data and consequently no prior trends could be identified for comparison.

From Table 7.1 detailing results for Survey 1:

For bay $3 \times = 3.490 \text{ m}$; s = 2.090 m; n = 9

Analysis of the results above for $x \pm s$ gave a range of 5.580 to 1.400m. As only three of the nine data points lay within this range (33%), the data did not comply with conditions of normality (Section 5.12) and consequently, beach levels were not normally distributed. However, if data is not normally distributed, the data set should be transformed, if possible, to allow analysis using parametric tests. Justification for the groyned beach is argued on the data set being related to an arbitrary datum, that is, Ordnance Datum Newlyn and not to the depth of mobile beach covering. As identified in Section 7.3.1, the level of the shore platform in bay 3 varied cross-shore along groyne 8. Therefore, it is the temporal level difference that will determine changes in mobile material on the surface of the beach. The paired t test is appropriate where differences between pairs of data are normally distributed but the data itself does not necessarily have to be normally distributed. Consequently, this test was applied to bays 1, 2 and 3 between the start and end of the monitoring programme, that is, surveys 1 and 10. The t statistic for the paired t test was determined from survey results and Tables 7.15, 7.16 and 7.17 were produced from Figure 7.2 and Tables 7.1 and 7.10. The null hypothesis in each case was there was no significant difference in beach levels between surveys, that is, survey 1, September 1997 and survey 10, September 2002.

For the level differences d, $\bar{x} = -0.0167$ and s = 0.325, consequently the range $\bar{x} \pm s$ was -0.3417 to +0.3083 and $\bar{x} \pm 3s$ was -0.9917 to +0.9583. Therefore, as 89% of data points fell within $\bar{x} \pm s$ and 100% within $\bar{x} \pm 3s$, the data complied with the requirements for normal distribution (Wheater and Cook, 2000) and the paired t test was appropriate.

Table 7.15: Paired t test Bay 1 – Surveys 1 and 10

Location	Beach Level Survey 1 (9/97)	Beach Level Survey 10 (9/02)	d (Survey 10 –	d ²
	(m)	(m)	Survey 1)	
la la	1.130	1.245	0.115	0.0132
1b	5.045	4.270	- 0.775	0.6006
lc	6.690	6.860	0.170	0.0289
1d	0.840	1.115	0.275	0.0756
le	3.760	3.620	- 0.140	0.0196
lf	6.495	6.820	0.325	0.1056
lg	1.130	1.345	0.215	0.0462
] 1 h	3.775	3.545	- 0.230	0.0529
lj	6.750	6.645	- 0.105	0.110
Total Σ=			- 0.150	0.954

$$\overline{d} = \frac{\sum d}{n} = \frac{-0.150}{9} = -0.0167$$

Standard error of mean difference

SEd =
$$\sqrt{\frac{0.954 - 0.0025}{72}} = \sqrt{0.0132} = 0.115$$

$$t_{calc} = \frac{\overline{d}}{SE_d} = \frac{-0.0167}{0.115} = -0.145$$

From t tables, with degrees of freedom df = n-1 = 9-1 = 8, for p = 0.05 and 0.01, then $t_{tab} = 2.306$ and 3.355 respectively. As $t_{calc} < t_{tab}$ (-0.145 < 2.306 and 3.355) the null hypothesis is accepted and there was no significant difference in beach levels in bay 1 between surveys 1 and 10 (paired t test: t = -0.145; $d_f = 8$; p > 0.05). The negative sign for t_{calc} is purely arbitrary with respect to the tables but indicated that the levels in bay 1 had fallen between surveys.

Table 7.16: Paired t test Bay 2 – Surveys 1 and 10

Location	Beach Level Survey 1 (9/97) (m)	Beach Level Survey 10 (9/02) (m)	d (Survey 10 – Survey 1)	d^2
2a	1.160	1.340	0.180	0.0324
2b	4.220	4.290	0.070	0.0049
2c	6.895	7.110	0.215	0.0462
2d	0.870	0.770	- 0.100	- 0.0100
2e	4.160	3.290	- 0.870	0.7569
2f	6.595	6.595	0	0
2g	1.165	1.180	0.015	0.0002
2h	3.560	3.265	- 0.295	0.0870
2j	5.910	6.080	0.170	0.0289
Total ∑ =			- 0.615	0.9665

As $\bar{x} = -0.0683$; s = 0.321; $\bar{x} \pm s = 0.2527$ to -0.3893; $\bar{x} \pm 3s = +0.8947$ to -1.0313, the data set complied with the requirements of a normal distribution. Therefore, $\bar{d} = -0.0683$; $SE_d = 0.113$; $t_{calc} = -0.604$ and t_{tab} (P = 0.05) = 2.306; t_{tab} (P = 0.01) = 3.355 $(d_f = 8)$.

From the above, the null hypothesis was accepted and there was no significant difference in beach levels in bay 2 between surveys 1 and 10 (paired t_{test} : t = -0.604; $d_f = 8$; p > 0.05) and once again levels had fallen between surveys.

Table 7.17: Paired t test Bay 3 – Surveys 1 and 10

	Beach Level	Beach Level	d	d ²
Location	Survey 1 (9/97)	Survey 10 (9/02)	(Survey 10 –	
	(m)	(m)	Survey 1)	
3aa	0.810	1.745	0.935	0.874
3bb	3.410	4.420	1.010	1.020
3cc	6.145	6.930	0.785	0.616
3d	1.070	1.590	0.520	0.270
3e	3.440	3.675	0.235	0.055
3f	6.370	6.665	0.295	0.087
3g	1.160	1.440	0.280	0.078
3h	3.190	3.510	0.320	0.102
3j	5.815	6.175	0.360	0.130
Total $\Sigma =$			4.740	3.232

As $\bar{x} = 0.527$; s = 0.286; $\bar{x} \pm s = 0.241$ to 0.813; $\bar{x} \pm 3s = -0.331$ to 1.385, the data set complied with the requirements of a normal distribution.

Therefore,
$$\bar{d} = 0.527$$
; $SE_d = 0.101$; $t_{calc} = 5.218$
and $t_{tab} (P = 0.05) = 2.306$; $t_{tab} (P = 0.01) = 3.355$

From the above, the null hypothesis was rejected and there was a significant difference at the 99% confidence level in beach levels in bay 3 between surveys 1 and 10 (paired t test; t = 5.218; $d_f = 8$; P < 0.01) with levels rising significantly.

Non parametric tests can be used on interval data but are slightly less likely to detect a statistical difference or relationship if present, compared with parametric tests (Wheater and Cook, 2000). It was decided, for completeness, to conduct the Wilcoxon matched-pairs non parametric test on the same data for the bays between surveys 1 and 10. Tables 7.18, 7.19 and 7.20 were produced using data from Tables 7.15, 7.16 and 7.17 respectively and in each case the null hypothesis was that there was no significant difference in beach levels.

Table 7.18: Wilcoxon Matched-Pairs Test Statistic Bay 1 – Surveys 1 and 10

Location	Differences d	Rank of	Signed Ranks
	(m)	Differences	
la	0.115	2	2
1b	- 0.775	9	- 9
1c	0.170	4	4
1 d	0.275	7	7
1e	- 0.140	3	- 3
1f	0.325	8	8
lg	0.215	5	5
1h	- 0.230	6	- 6
lj	- 0.105	1	- 1

$$T_{+} = 2 + 4 + 7 + 8 + 5 = +26$$

$$T_{-} = 9 + 3 + 6 + 1 = +19$$

Taking lower value of $T_{calc} = 19$

For
$$p = 0.05$$
 $T_{tab} = 5$ and for $p = 0.01$, $T_{tab} = 1$ $(n = 9)$.

As $T_{calc} > T_{tab}$ the null hypothesis was accepted and there was no significant difference in beach levels in bay 1 between Surveys 1 and 10 (T = 19; n = 9; P > 0.05).

Table 7.19: Wilcoxon Matched-Pairs Test Statistic Bay 2 - Surveys 1 and 10

Location	Differences d (m)	Rank of Differences	Signed Ranks
2a	0.180	5	5
2b	0.070	2	2
2c	0.215	6	6
2d	- 0.100	3	- 3
2e	- 0.870	8	- 8
2f	0	*	
2g	0.015	1	l _
2h	- 0.295	7	- 7
2j	0.170	4	4

^{*} Ignore zero hence n = 8

$$T_{+} = 5 + 2 + 6 + 1 + 4 = 18$$

$$T_{-} = 3 + 8 + 7 = 18$$

Taking lower value of $T_{calc} = 18$

For
$$p = 0.05$$
, $T_{tab} = 3$ and for $p = 0.01$, $T_{tab} = 0$ (n = 8)

As $T_{calc} > T_{tab}$ the null hypothesis was accepted and there was no significant difference in beach levels in bay 2 between Surveys 1 and 10 (T = 18; n = 8; P > 0.05).

Table 7.20: Wilcoxon Matched-Pairs Test Statistic Bay 3 - Surveys 1 and 10

Location	Differences d (m)	Rank of Differences	Signed Ranks
3aa	0.935	8	8
3bb	1.010	9	9
3cc	0.785	7	7
3d	0.520	6	6
3e	0.235	1	1
3f	0.295	3	3
3g	0.280	2	2
3h	0.320	4	4
3i	0.360	5	5

$$T_{+} = 8 + 9 + 7 + 6 + 1 + 3 + 2 + 4 + 5 = 45$$

$$T_{-}=0$$

Taking lower value of $T_{calc} = 0$

For
$$p = 0.05$$
, $T_{tab} = 5$ and for $p = 0.01$, $T_{tab} = 1$ $(n = 9)$

As $T_{calc} < T_{tab}$, the null hypothesis was rejected and there was a significant difference with 99% confidence in beach levels in bay 3 between Surveys 1 and 10 (T = 0; n = 9; P< 0.01).

Therefore, both the paired t test and Wilcoxon matched pairs test statistic agreed that there was no significant difference in beach levels in bays 1 and 2 over the monitoring period. However, at the 99% confidence level, they both agreed that there was a significant increase in beach levels in bay 3 over the same period. Reference to Figure 7.3 shows the temporal variation of mean beach levels in bay 3 throughout the survey period. Having shown that there was a significant difference to the beach levels in bay 3, a correlation and regression analysis was undertaken to ascertain whether a valid temporal model could be determined. From Table 7.11 the data used to determine the correlation and regression for bay 3 was transformed as Table 7.21.

Table 7.21: Correlation and Regression for Temporal Variation Bay 3

Tim (Mon			Beach Level (m)	
X	X ²	y	y ²	$\mathbf{x} \times \mathbf{y}$
0	0	3.490	12.180	0
2	4	3.534	12.489	7.068
17	289	3.721	13.846	63-257
24	576	3.761	14.145	90.264
31	961	3.973	15.785	123·163
36	1,296	3.981	15.848	143-316
43	1,849	4.000	16.000	172.000
48	2,304	4.002	16.016	192.096
55	3,025	4.004	16.032	220-220
60	3,600	4.017	16.136	241.020
Totals Σ 316	13,904	38.483	148-477	1,252.404

Correlation Coefficient (r) =
$$\frac{n \sum xy - \sum x \sum y}{\sqrt{(n \sum x^2 - (\sum x)^2)(n \sum y^2 - (\sum y)^2)}}$$

$$r_{calc} = \frac{10 \times 1252 \cdot 404 - (316 \times 38 \cdot 483)}{\sqrt{(10 \times 13904 - 316^2)(10 \times 148 \cdot 477 - 38 \cdot 483^2)}}$$

$$r_{calc} = 0.938$$

For P = 0.05,
$$r_{tab} = 0.632$$
 and for P = 0.01, $r_{tab} = 0.765$ ($d_f = n - 2 = 8$)

$$\therefore$$
 Coefficient of determination = $r^2 = 0.938^2 = 0.88$ ($R^2 = 88\%$)

As $r_{calc} > r_{tab}$ (P = 0.01) there was a strong positive correlation between beach level and time (r = 0.938, $d_f = 8$, P< 0.01) with the two variables having 88% of their variation in common.

The regression line y = mx + c was subsequently determined as follows:

m =
$$\frac{\sum xy - \frac{\sum x \sum y}{n}}{\sum x^2 - \frac{\left(\sum x\right)^2}{n}}$$
=
$$\frac{1252 \cdot 404 - \frac{316 \times 38 \cdot 483}{10}}{13904 - \frac{316^2}{10}}$$
=
$$0.00927$$
c =
$$\overline{y} - m\overline{x}$$
=
$$\frac{38 \cdot 483}{10} - \left(0.0093 \times \frac{316}{10}\right)$$
=
$$3.5554$$

Therefore the regression equation was

$$y = 0.00927x + 3.5554$$

The significance of this regression line was determined by analysis of variance (ANOVA).

$$SS_{total} = \sum y^{2} - \frac{(\sum y)^{2}}{n}$$

$$= 148.477 - \frac{38.483^{2}}{10}$$

$$= 0.383$$

$$SS_{regression} = \frac{\left(\sum xy - \frac{\sum x \sum y}{n}\right)^{2}}{\sum x_{2} - \frac{(\sum x)^{2}}{n}}$$

$$= \frac{\left(1252 \cdot 404 - \frac{316 \times 38 \cdot 483}{10}\right)^{2}}{13904 - \frac{316^{2}}{10}}$$

$$= 0.337$$

$$SS_{residual} = SS_{total} - SS_{regression} = 0.383 - 0.337 = 0.046$$

These results were used to produce the ANOVA Table.

Table 7.22: ANOVA Results Table – Regression Line

	df	Sums of Squares	Mean Squares	F Value	PV	alue
<u> </u>		(SS)	(MS)		P 0.05	P 0.01
Regression	1	0.337	0.337	56.17	5.32	11.3
			SS _{regression}	MS reg		
			1 regression	MSSresiduall		
			,			
Residual	n-2=8	0.046	0.006			
			(SS _{residual})			
			n - 2			
Total:	n-1=9	0.383				

As the calculated value of F is much higher than either critical value for P = 0.05 or 0.01, the null hypothesis is rejected and a highly significant amount of the variation of beach level in bay 3 is accounted for by passage of time.

The coefficient of determination for the regression analysis was given by

$$r^2 = \frac{SS_{regression}}{SS_{total}} = 0.88 \quad (R^2 = 88\%) \quad .$$

Consequently, there was a significant relationship between beach level and time in bay 3 (y = 0.00927x + 3.5554; $F_{1.8} = 56.17$; P< 0.01). The regression model explained a high percentage of the variation in beach level ($R^2 = 88\%$).

Figure 7.4 produced from Figure 7.3, showed data points instead of lines for the temporal variation in each bay, along with the EXCEL

projected trendlines, equations and coefficients of determination. It was clearly seen that the computer regression results for bay 3 were virtually identical to those above, i.e. y = 0.0093x + 3.5552 ($R^2 = 0.8789$) (EXCEL) and y = 0.00927x + 3.5554 ($R^2 = 0.88$) (calculated). Therefore, EXCEL was subsequently used to determine all further trends.

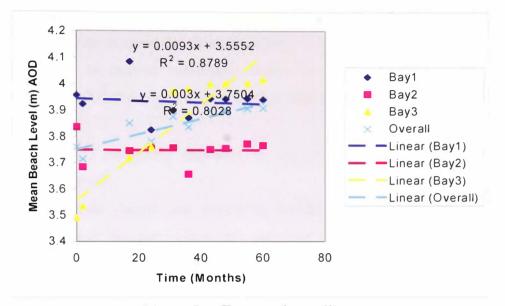


Figure 7.4: Temporal trendlines

From Figure 7.4 the regression equations for bays 1 and 2 highlighted that virtually none of the variation in mean beach level was explained by the passage of time, that is, $1\cdot18\%$ and $0\cdot01\%$ respectively. However, the regression model for the overall mean beach level for the groyned beach ($y = 0\cdot003x + 3\cdot7504$) once again indicated that variation in time explained a high percentage of the variation in mean beach level ($R^2 = 80\cdot28\%$). The shallower gradient of this regression model, compared with that for bay 3, was indicative of trends in bays 1 and 2 being integrated into the overall mean beach level model. Therefore, it could be argued that the relatively larger increase of mean beach level in bay 3 was distorting the overall mean beach level for the groyned beach and that this model purely reflected changes in bay 3. However, in the qualitative analysis (Section 7.4.1) an argument was made that the completed barrage contributed to increased beach levels in bays 1 and 2. Therefore, the temporal models were

reviewed by examining changes in mean beach levels subsequent to the completion of the breakwater and barrage.

For this analysis beach levels in bays 1 and 2 were evaluated separately from those of bay 3 and overall, due to previously discussed significant and insignificant relationships. Survey 5 in April 2000 was the first following completion of the barrage and Table 7.11 showed the significant rise in mean beach levels in bay 3. Therefore, it was deemed appropriate to analyse subsequent trends in all three bays between September 2000 (Survey 6) and September 2002 (Survey 10). This was for three reasons:

- firstly, immediate responses to the environmental change had occurred;
- secondly, nearly one year (>10 months) had elapsed since the barrage became operational and this time would have given opportunity for coastal processes to become settled with the new tidal regime;
- thirdly, operational problems that had required frequent emptying of the freshwater lake (Phillips and Williams, 2000) were resolved.

Figure 7.5 was subsequently produced from Table 7.11 and showed mean beach levels in bays 1 and 2 between September 2000 and September 2002. It was seen that the regression equation for bay 1 was y = 0.0024x + 3.8131 with a coefficient of determination (R²) equal to 50.9%. Therefore, the regression model explained this percentage variation in mean beach level with time and indicated a positive correlation to increased beach levels, following the completion of the barrage. For bay 2, the regression equation y = 0.0041x + 3.5418 explained a higher percentage of the variation in mean beach level (R² = 67.97%) and similarly there was a positive correlation to increased beach levels over time following the completion of the barrage.

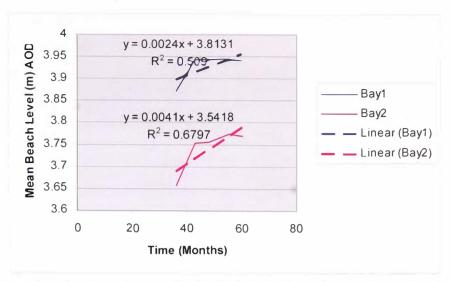


Figure 7.5: Temporal variation bays 1 and 2 (post barrage completion)

Figure 7.6 similarly produced from Table 7.11 showed the variation in mean beach level for both bay 3 and the groyned beach overall.

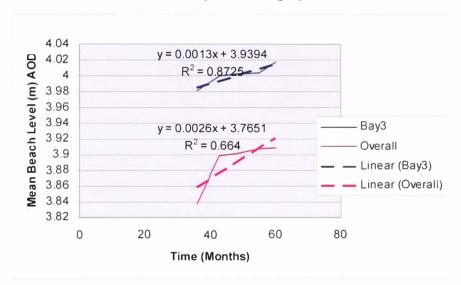


Figure 7.6: Temporal variation bay 3 and beach (post barrage completion)

Table 7.23: Regression of Mean Beach Level with Time – Calculating the Residuals

Location	x (Months)	Predicted y from equation (m)	Measured y (m)	Residual y (Measured y – Predicted y)
Bay 1	-	y = 0.0024x + 3.8131	1	(m)
Day 1	36	3·900	3.872	- 0.028
	43	3.916	3.944	
	48	3.928		+ 0.028
			3.943	+ 0.015
	55	3.945	3.945	0
	60	3.957	3.941	- 0.016
Bay 2		y = 0.0041x + 3.5418	ı	
	36	3.689	3.658	- 0.031
	43	3.718	3.753	+ 0.035
	48	3.739	3.757	+ 0.018
	55	3.767	3.773	+ 0.006
	60	3.788	3.769	- 0.019
Bay 3	_	y = 0.0013x + 3.9394		
-	36	3.986	3.981	- 0.005
	43	3.995	4.000	+ 0.005
	48	4.002	4.002	0
	55	4.011	4.004	- 0.007
	60	4.017	4.017	0
Overall		y = 0.0026x + 3.7651		
Groyned	36	3.859	3.837	- 0.022
Beach	43	3.877	3.899	+ 0.022
	48	3.890	3.901	+ 0.011
	55	3.908	3.907	- 0.001
	60	3.921	3.909	- 0.012

The regression equation, y = 0.0013x + 3.9394, accounted for an even more significant percentage ($R^2 = 87.25\%$) of the increased beach levels compared with bays 1 and 2 and also demonstrated a positive correlation with increased beach levels, subsequent to the completion of the barrage. The overall result for the groyned beach y = 0.0026x + 3.7651 as expected, demonstrated a positive correlation to increased beach levels and explained a high percentage ($R^2 = 66.4\%$) of the overall variation in beach level. Therefore, these models appear to provide a useful tool for predicting changes in mean beach level over time. Furthermore, in association with projected longshore and cross-shore gradients (Tables 7.13 and 7.14) estimates could be made for individual levels at the respective grid locations.

However, assumptions were made in this analysis that required verifying in order to validate the results. These included that there was a

linear relationship between beach level and time; that the independent variable (x) is fixed by the researcher, that is, for each value of x, the value of the dependent variable is free to vary and finally; if multiple readings are taken, the variance should be the same along the regression line (Wheater and Cook, 2000).

- The first was confirmed by the results.
- The second was clearly met in that the time intervals between surveys were fixed and beach level free to vary.
- To test the final assumption, an analysis of residuals was undertaken as shown in Table 7.23. Results showed that in all cases, the residuals were randomly positioned with respect to x and, therefore, the use of linear regression was justified.

Further evaluation of the linear models showed that each time a statistical technique was applied, the relationship for mean beach levels in bay 3 proved most significant. This reinforced earlier findings that there was a significant increase in beach levels in bay 3 over the five-year monitoring process. Consequently, as well as a temporal relationship, results appeared to indicate that a spatial relationship also existed. Therefore further analysis was undertaken to determine the validity of this hypothesis.

7.4.3 Spatial Quantitative Analysis

Groynes, constructed to retain beach material at the cliff toe, by definition interfere with longshore transport processes. This factor was considered in the analysis to determine trends in longshore variation in beach level. Beach levels across the groynes along the groyned beach displayed consistently higher levels on the southern flanks and maintained support for the south to north longshore movement of material identified by Wallingford (1992) and Bullen (1993a). From Tables 7.1 to 7.10 inclusive, level differences at the mid-grid location across groynes 6, 7 and 8 were determined and shown in Table 7.24.

Table 7.24: Temporal Differences in Beach Level across Groynes 6, 7 and 8 (Figure 7.2).

Survey	Date	Level dif	ference across gro	ynes (m)
		Groyne 6	Groyne 7	Groyne 8
1	9/97	0.495	1.485	1.030
2	11/97	0.560	1.540	0.795
3	2/99	1.025	0.945	0.625
4	9/99	0.450	0.545	0.720
5	4/00	0.660	0.735	0.670
6	9/00	0.685	0.885	0.600
7	4/01	0.610	0.765	0.680
8	9/01	0.480	1.110	0.810
9	4/02	0.675	0.915	0.770
10	9/02	0.905	1.005	0.780

These values were analysed statistically to determine the range of the mean within 99% confidence limits and are tabulated, as follows:

Table 7.25: 99% Confidence Limits – Mean Level Difference across Groynes.

Location	x (m)	S (m)	SE (m)	t _{0·01 (n-1)}	$x \pm t_{0.01} (n-1) SE(m)$
Groyne 6	0.655	0.176	0.056	3.25	0.655 ± 0.182
Groyne 7	0.993	0.299	0.095	3.25	0.993 ± 0.307
Groyne 8	0.748	0.116	0.037	3.25	0.748 ± 0.120

It was found that 70% of the level differences across groynes 6, 7 and 8 were within the 99% confidence limit range of the mean. Furthermore as the groyne data complied with the requirements of a normal distribution, independent t tests were carried out to determine whether there was a significant difference between the retained levels across each groyne. The t test comparisons were: groynes 6 and 7 (t_{calc} = 3·081; t_{tab} 0·05 (n-1) = 2·262); groynes 6 and 8 (t_{calc} = 1·402; t_{tab} 0·05 (t_{tab} 0·05 (t

Analysis continued to further identify spatial beach level changes with respect to the barrage. To determine spatial mean beach levels, the cross-shore levels in each bay were transformed with respect to distance from the barrage breakwater. Throughout this work, traditional surveying

practice had been used which meant that positive distance was in a northerly direction. Consequently, as the barrage breakwater was at the northern limits of this research, distances from the breakwater were considered negative. Therefore, the longshore distances from the breakwater, corresponding to cross-shore beach levels in the surveys were as follows: 0m (adjacent to breakwater; -21·5m (mid point bay 3); -43m (groyne 8); -72m (mid point bay 2); -101m (groyne 7); -123·5m (mid point bay 1) and –146m (groyne 6). Mean cross-shore levels were then determined, at each longshore position, for each of the ten surveys. From Figure 7.3, these were calculated as follows: 0m (3aa + 3bb + 3cc)/3; -21·5m (3d + 3c + 3f)/3; -43m (3g + 3h + 3j + 2a + 2b + 2c)/6; -72m (2d + 2c + 2f)/3; -101m (2g + 2h + 2j + 1a + 1b + 1c)/6; -123·5m (1d + 1e + 1f)/3 and -146m (1g + 1h + 1j + 1k)/4. The calculated cross-shore levels for the survey period were determined from Tables 7.1 to 7.10 inclusive and shown in Table 7.26.

Table 7.26: Variation of Mean Beach Level with Distance from Barrage Breakwater.

Distance from					Mean bead	ch level (m))			
barrage	Survey	Survey	Survey	Survey	Survey	Survey	Survey	Survey	Survey	Survey
breakwater	1	2	3	4	5	6	7	8	9	10
(m)	9/97	11/97	2/99	9/99	4/00	9/00	4/01	9/01	4/02	9/02
0	3 455	3.567	3 863	3 938	4 387	4.223	4.280	4.353	4 355	4 365
- 21 5	3 627	3.623	3 780	3.667	3.913	3.977	3 975	3 955	3 958	3 977
- 43.0	3.740	3 700	3 834	3.968	3.948	3.911	3 940	3.974	3.989	3.978
- 72·0	3 895	3.660	3.588	3 575	3.605	3.530	3 550	3 605	3 585	3 552
-101.0	3.917	3 808	3.830	3 683	3.706	3.717	3 868	3.749	3.808	3-817
-123·5	3.698	3.773	3 980	3.793	3.842	3.727	3.838	3 882	3 848	3 852
-146·0	3.981	3 855	4.291	3.856	3.918	3.879	3 936	3.940	3 926	3.996

Figure 7.7 was subsequently produced from Table 7.26 and it can be clearly seen qualitatively, that over the monitoring period, the longshore profiles had changed significantly: the closer to the barrage, the greater the change.

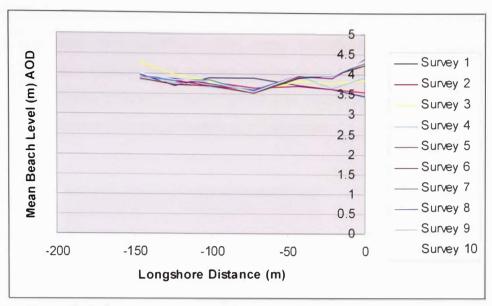


Figure 7.7: Variation in mean beach level with longshore distance.

This once again validated previous findings that beach levels in bay 3 had undergone the most significant change with progressive rises in mean beach level. To compare overall change, Figures 7.8 and 7.9 show the longshore profiles and trendlines at the start (Survey 1) and end (Survey 10) respectively of the five-year research period. Regression equations for both surveys were calculated manually and compared with computer results as a check. For survey 1 manual calculations gave the regression analysis y = -0.0027x + 3.563 (R² = 60.86%) whilst survey 10 gave y = 0.0021x + 4.086 $(R^2 = 22.29\%)$. Therefore, the validity of accepting EXCEL projections for this section of the analysis was appropriately demonstrated. Analysis of the longshore profile from survey 1 showed decreasing beach levels towards 0m (the proposed location of the barrage breakwater). Further reference to Plate 7.4 indicated that material would be free to continue moving in a northerly direction. Examination of the regression equation indicated that longshore distance explained a significant percentage of the variation in mean beach level ($R^2 = 61\%$) and the negative gradient provided supporting evidence for the net south to north longshore drift identified by Wallingford (1992) and Bullen (1993a). The profile also illustrated the longshore influence of the groynes, especially groynes 6 and 7 (-146m and -101m respectively). However, groyne 8 (-43m) appeared to have little influence on the profile. The observed effects of groyne 7 again appeared more influential and validated previous findings that level differences across this groyne were more significant than across the others. It was logical to assume that without these groynes the beach gradient would have been more uniform, but results also confirmed their locations were appropriate to provide increased beach protection of the shoreline.

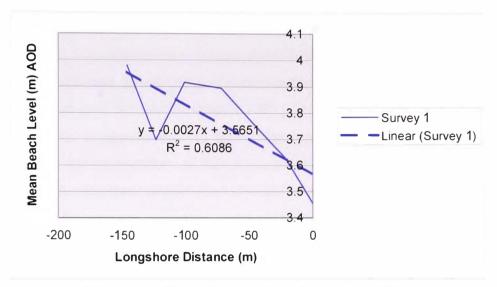


Figure 7.8: Longshore profile - September, 1997.

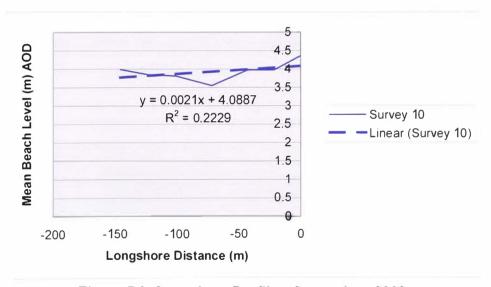


Figure 7.9: Longshore Profile - September, 2002.

The regression equation for Survey 10 had a positive gradient, which indicated rising beach levels approaching the barrage. Although the equation does not initially appear to explain a significant percentage of the variation in beach level ($R^2 = 22\%$), it demonstrated a change over the five

years, which can be directly correlated to the construction of the barrage breakwater. However, on further examination of the longshore profile, it seemed that this trendline had to model the effects of two significantly different process, that is, mean beach levels appeared to fall to the mid point of bay 2 (-72m) before rising towards the breakwater (0m).

In Section 7.4.2 it was argued that the completed barrage and breakwater had contributed to increased beach levels. Therefore, following the same rationale, an analysis was undertaken on the longshore variation in mean beach level between September 2000 (survey 6) and September 2002 (survey 10). Longshore profiles from these surveys (Figure 7.10) further reinforced the interpretation of falls in mean beach level to the mid point of bay 2 with subsequent steeper rises towards the barrage breakwater. Levels from the five surveys were then averaged and the mean longshore beach profile between September 2000 and September 2002 produced (Figure 7.11). The equation of the regression trendline (y = 0.0023x + 4.0735) was very similar to that for survey 10 (Figure 7.9; y = 0.0021x + 4.0887) and although the coefficient of determination had increased slightly (28.84% as opposed to 22.29%) this still did not explain a high percentage of the variation in mean beach level.

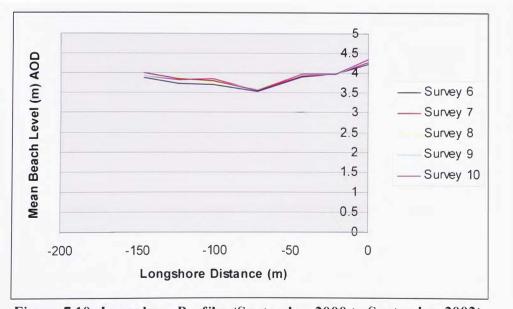


Figure 7.10: Longshore Profiles (September 2000 to September 2002).

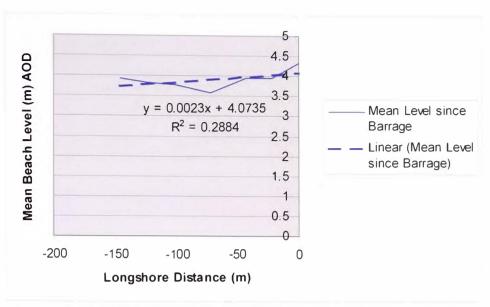


Figure 7.11: Mean Longshore Profile (September 2000 to September 2002).

Consequently, to examine the two trends identified the profile shown in Figure 7.11 was divided into two sections, that is, groyne 6 to mid point of bay 2 (-146m to -72m) and the mid point of bay 2 to the barrage breakwater (-72m to 0m; Figures 7.12 and 7.13 respectively).

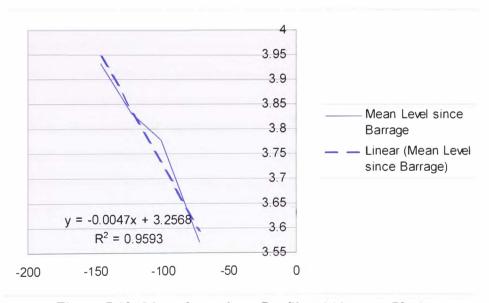


Figure 7.12: Mean Longshore Profile (-146m to -72m).

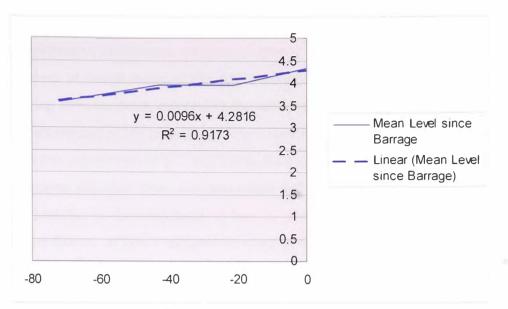


Figure 7.13: Mean Longshore Profile (-72m to 0m)

From the trendlines, it was clearly seen that a significant relationship existed in both parts of the beach. The regression equation from groyne 6 to the mid point of bay 2 (-146m to -72m) was y = -0.0047x + 3.2568 and indicated a clear trend of falling beach levels towards the breakwater. The model explained a highly significant percentage of the variation in beach level ($R^2 = 95.93\%$; Figure 7.12). Furthermore, the regression equation from the mid-point of bay 2 to the barrage breakwater (-72m to 0m), i.e. y = 0.0096x + 2.816, similarly indicated a clear trend, this time of rising beach levels towards the breakwater. Once again the model explained a highly significant percentage of the variation in mean beach level ($R^2 = 91.73\%$; Figure 7.13) and further showed that the rising gradient along this section was approximately twice the falling gradient in the previous section (0.0096 to 0.0047).

This analysis provided further support to the temporal models developed in Section 7.4.2. The significance of these models increased the closer to the barrage breakwater, i.e. bay 3 (87·25%), bay 2 (67·97%), bay 1 (50·9%) and overall (80·28%). It is hypothesised that the two significant longshore trends are inter-related. Results for survey 1 (Figure 7.8) showed a reduction in mean beach level towards the barrage. Although the groynes interfered to some extent with net longshore transport (south to north),

material was moving northwards away from the groyned beach. Following completion of the barrage, analysis of surveys 6 to 10 inclusive, indicated a build-up of beach material that had probably resulted from the northerly moving material being retained at the breakwater. In the new tidal regime, this accumulation led to increased beach levels south of the breakwater. It is therefore argued that at the time of the final survey in September 2002, the longshore influence from the breakwater had extended a distance of 72m southwards, to the mid-point of bay 2. This was further supported by accumulations of beach material seaward of the groyned beach which had changed from September 1997 (compare Plate 7.2 with Plates 7.23 and 7.24). Therefore, over time, this influence is expected to continually extend further southwards.

Linear regression has demonstrated relationships with respect to mean beach level and its variation with time, longshore and cross-shore distance. As Crowell, *et al* (1997) concluded that linear regression provided the best forecast of shoreline trend, it is argued that these models can be used to predict future shoreline trends for the groyned beach. By implication, these relationships must have had an effect on beach morphology and to complete this analysis morphological change was evaluated.

7.4.4. Changes in Beach Morphology

Tables 7.1 to 7.10 inclusive, were imported into a Dplot package to produce two and three dimensional survey representations of the groyned beach. The two dimensional plots (Figure 7.15) demonstrated the chronological change in spatial-distribution of beach material between September 1997 and September 2002. The first image of Figure 7.15 showed the beach distribution in September 1997 and demonstrated that there was no uniformity between the three bays. Further examination showed a cusp in bay 3 where central beach levels were lower than those adjacent to the groynes (8 and 9). The fourth and fifth survey plots (September 1999 and April 2000) clearly identified a change in the beach

distribution as the groyned beach adjusted to conditions following completion of the barrage and breakwater. Therefore, this qualitative change can be directly correlated to the completion of the impenetrable tidal barrier. Remaining plots showed the temporal evolution of the beach as dynamic equilibrium was established with the tidal patterns and geophysical barrier. It was seen that there was more similarity between profiles from September surveys than those in April. This was possibly due to calmer summer conditions although, bay 1 consistently lost beach volume between April and September. The final plot of survey 10 (September 2002), showed that there was a relatively uniform distribution of beach material across all three bays. To further demonstrate the change that had occurred between the first and last surveys, three-dimensional plots were prepared as shown in Figure 7.15. By comparison, changes were evident but especially in survey 10 there was a similarity in material distribution with consistent rising longshore gradients in all three bays. At the back beach location, the uniform gradients, rising from south to north, were completely different to those of September 1997. From Section 7.4.2, the level of the MHW in 2002 was 4.500m AOD. Consequently, to quantify the relative uniformity between the bays in survey 10, the WCB of the MHW was determined.

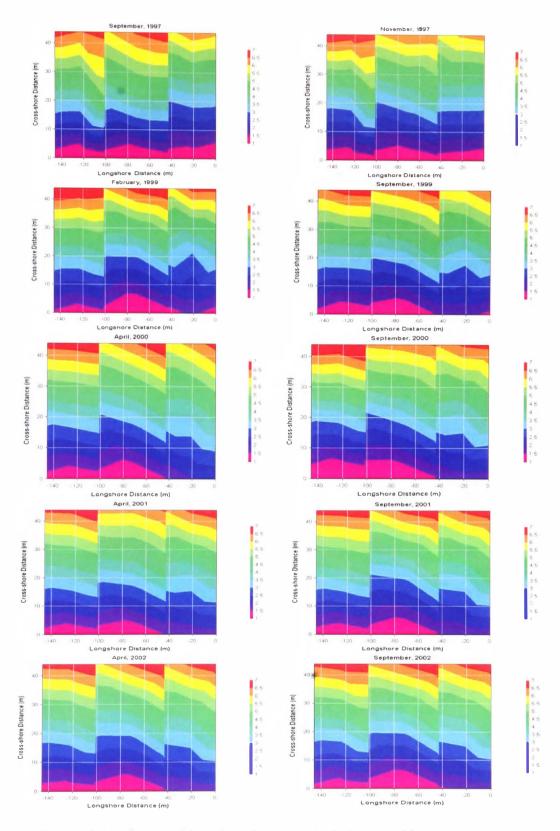
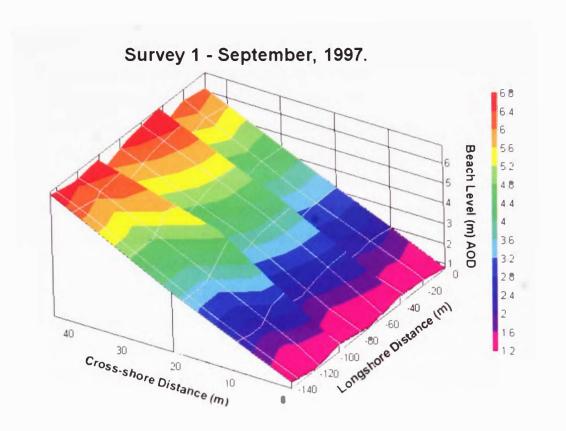


Figure 7.14: Groyned beach – chronological change of beach contours.





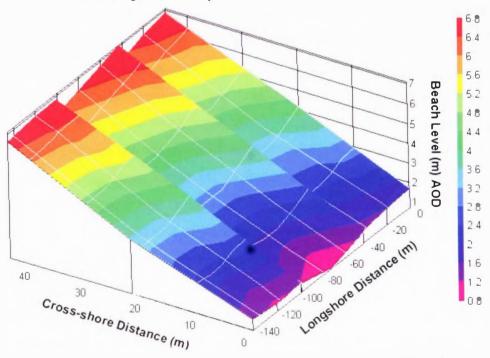


Figure 7.15: Three-dimensional contour plots of groyned beach.

Using rectangular co-ordinates (eastings and northings) from the OS map (Figure 7.2) the mean WCB of groynes 6, 7, 8 and 9 was 63.5°. From the two-dimensional Dplot co-ordinates, the mean line of the 6m contour

was: 63·6° anticlockwise to groyne 8 (bay 3); 64·9° anticlockwise to groyne 7 (bay 2) and 77·2° anticlockwise to groyne 6. Therefore the WCB of the MHW in each bay of the groyned beach was: 346·3° (bay 1); 358·6° (bay 2) and 359·9° (bay 3). From this analysis it was seen that bays 2 and 3 were very similar but there was a 12·3° change between bays 1 and 2. These results further supported findings that the barrage influence was less significant on beach formation processes in bay 1 compared with bays 2 and 3. However, it is also argued that beach evolution in bay 1, shown in Figure 7.15, is following a similar pattern to that demonstrated in the other two bays. This was justified by Figure 7.11, which showed that the variation in longshore beach levels was far less volatile following completion of the barrage extends southwards, beach contours in bay 3 will be similar to those in bays 2 and 3.

7.5 Summary

Mean beach levels for the groyned beach increased overall throughout the five-year monitoring period. Although mean levels in bays 1 and 2 had demonstrated an overall loss between surveys 1 and 10, as soon as the barrage and breakwater had been completed, beach levels increased in each of the bays. Bay 3 provided the major contribution to increased mean beach levels, which resulted in a net gain of beach covering equivalent to 818.8m³ (1,801.2 tonnes). Post completion of the barrage, both longshore and cross-shore gradients became less volatile and photographic evidence qualitatively demonstrated that the increased beach levels in bay 3, prevented tidal action at the cliff toe. This was a significant change from the start of the survey programme and was verified quantitatively by parametric and non parametric tests at the 99% confidence level. These identified significant differences in beach levels between the start and the end of the five years but this was not the case in either bays 1 or 2. Regression analysis determined that there was a significant relationship between mean beach level and time and further analysis of the post barrage results identified valid temporal models for mean beach levels in each of the three bays and for the groyned beach overall. These were: bay $1(y = 0.0024x + 3.8131; R^2 = 50.9\%);$ bay $2(y = 0.0041x + 3.5418; R^2 = 67.97\%;$ bay 3 (y = 0.0013x + 3.9394; R² = 87.25%) and overall for the groyned beach (y = 0.0026x + 3.7651; R² = 66.4%). Residual analysis showed that the use of linear regression was justified and this conclusion was further justified by the work of Crowell *et al* (1997) who argued that this type of model provided the best forecast of shoreline trend.

Results from the September 1999 survey (Section 7.3.4) showed an intersurvey loss of 512·8m³ (1,128·2 tonnes; Table 7.12). This equated to 569 kg/m² and over the same period, stations 10 to 14 of the foreshore grid showed a loss of 1015·2 m³ (2233.4 tonnes; Table 6.15) equivalent to 620 kg/m². These results supported conclusions in Section 6.6.4 that beach response to influences such as storms are inter-related, irrespective of beach orientation.

Spatial analysis identified two highly significant longshore trends with respect to distance south of the barrage breakwater. From 146m to 72m distance from the breakwater, the regression equation y = -0.0047x + 3.2568 ($R^2 = 95.93\%$) modelled a trend of falling beach levels. This was a continuation of the pre-barrage trend indicating falling beach levels along the whole of the groyned beach, which was caused by the south to north sediment transport. Conversely, from 72m to 0m (the location of the barrage breakwater) beach levels increased at a rate greater than twice the fall in the previous section and was represented by y = 0.0096x + 2.816 ($R^2 = 91.73\%$). Therefore, influence of the barrage decreased with distance. These trends were further supported by results that indicated significantly greater level differences across groyne 7. Furthermore, these longshore trends contributed to the projected accuracy of the temporal models and highlighted why the model for bay 1 explained the lowest percentage of the variation in mean beach level.

Impacts of the temporal and spatial trends, especially subsequent to the completion of the barrage, were clearly seen in the changed beach morphology. The distribution of ground contours was similar in bays 2 and 3 (WCB MHW 358·6° and 359·9° respectively). Whilst bay 1 (WCB MHW 346·3°) did not completely mirror the other two bays, there was clear evidence of change between September 1997 and September 2002. Therefore, in time, as breakwater influence extends

downstream, it is expected that contour orientation in all three bays will be the same.

Groyne 7 was built circa 1970, whilst groynes 6, 8 and 9 were constructed They were obviously part of a management strategy to retain beach material at the cliff toe and from Plate 7.2, this had been, to some degree, successful. However, in September 1997 there were concerns due to the proximity of properties on the cliff top regarding beach levels in bay 3 and the consequent accelerated erosion of the cliffs. By September 2002, increased beach levels had provided protection to these cliffs. Therefore, the temporal model developed for bay 3 (y = 0.0013x + 3.9394) could be used as a management tool to diagnose effects of potential changes in coastal processes such as increased incidence of storms and sea level rise. Levels would simply be taken annually each September in bay 3 for actual measurements to be compared with predicted. Further substitution into the temporal models for bay 2 and overall in association with field data collection biannually, could be used to extend predictions for the groyned beach and further refine the models. In conjunction with longshore gradients determined from the spatial analysis, changes in trends could be identified and the rate of change of the barrage's influence to the south could be monitored. These results will be further evaluated in Chapter 9, as part of an overall management strategy for the Penarth coast.

CHAPTER 8: WATER AND WIND

8.1 Introduction

The changes identified along the Penarth coastline in Chapters 6 and 7 were functions of external environmental factors, such as tidal water level and storm surges. In practice the term storm surge is used loosely to include the astronomical tidal component and other meteorological effects, for example, variations in atmospheric pressures and subsequently generated winds (Simm, 1996). The roles of waves, tides and currents are therefore inextricably linked in determining the unique character of every beach. Furthermore, as discussed in Section 2.4, sea level is one of the principal determinants of shoreline position (Leatherman, 2001). Consequently, both tidal water level data and wind data were analysed and subsequently evaluated with respect to the measured changes along the Penarth foreshore (Chapters 6 and 7).

8.2 Analysis of tidal water level

The nearest tide gauges to Penarth, available as part of the National Tidal and Sea Level Facility, are at Mumbles and Newport. Newport is approximately 15 km east of Penarth whilst Mumbles is approximately 80 km west. As Newport is closer to Penarth, and both are located on the Severn Estuary and sheltered by land from the prevailing southwesterly winds, it is argued that sea conditions should be more similar between Penarth and Newport. Consequently, monthly mean and extreme sea level data, between April 1993 (earliest available) and December 2002 was obtained for the Newport tide gauge (Site History and raw data included in Appendix 5). This data was supplied by the British Oceanographic Data Centre (BODC) as part of the function of the National Tidal and Sea Level Facility, hosted by the Proudman Oceanographic Laboratory and funded by the Department for Environment, Food and Rural Affairs (DEFRA) and the Natural Environment Research Council (NERC). Each of these data sets were analysed separately to identify trends and subsequently related to the study area.

8.2.1 Monthly mean sea levels

The datum reference for the tide gauge readings is Admiralty Chart Datum which is related to the Lowest Astronomical Tide, and this varies from coast to coast. Consequently, data for the monthly mean sea levels was transformed relative to Ordnance Datum for Newport (-5·81m) and recorded in Table 8.1.

Table 8.1: Monthly Mean Sea Levels - Newport (Source: BODC).

					Mean S	sea Level	(m) AOD)			
Month	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002	10 Year Monthly Mean
Jan	N/A	0.295	0.379	0.355	0.175	0.313	0.392	0.162	0.334	0.315	0.302
Feb	N/A	0.200	0.390	0.183	0.383	0.267	0.133	0.273	0.067	0.385	0.253
Mar	N/A	0.240	0.257	0.134	0.144	0.303	0.212	0.130	0.234	0.187	0.205
Apl	0.218	0.114	0.132	0.165	0.124	0.318	0.222	0.277	0.112	0.187	0.187
May	0.209	0.195	0.189	0.208	0.245	0.150	0.213	0.199	0.054	0.262	0.192
June	0.226	0.172	0.121	0.140	0.291	0.271	0.208	0.186	0.098	0.192	0.191
July	0.209	0.163	0.234	0.200	0.186	0.269	0.198	0.313	0.181	0.183	0.214
Aug	0.173	0.234	0.181	0.253	0.267	0.213	0.281	0.236	0.223	0.187	0.225
Sept	0.330	0.277	0.293	0.211	0.253	0.326	0.399	0.355	0.357	0.220	0.302
Oct	0.279	0.325	0.370	0.372	0.311	0.402	0.320	0.392	0.428	0.326	0.353
Nov	0.209	0.338	0.372	0.349	0.464	0.298	0.248	0.508	0.175	0.445	0.341
Dec	0.387	0.421	0.229	0.206	0.346	0.281	0.402	0.486	0.139	0.259	0.316
Annual Average	0.249	0-248	0.262	0.231	0.266	0.284	0.269	0.293	0.200	0.262	0.256

From Page and Oakley (2002) tidal ranges at Newport were as follows:

Mean High Water Springs
12·1m
Mean Low Water Springs
0·2m
Mean High Water Neaps
9·0m
Mean Low Water Neaps
2·9m

$$\therefore \text{ Mean Sea Level} = \frac{12 \cdot 1 + 0 \cdot 2 + 9 \cdot 0 + 2 \cdot 9}{4}$$

$$= 6 \cdot 05 \text{ m ACD}$$

$$= 6 \cdot 05 - 5 \cdot 81$$

$$= 0 \cdot 240 \text{ m AOD}$$

From Table 8.1 it can be seen that in 1996 and 2001 the annual average monthly mean sea level was lower than the predicted astronomical values (0.240; 20% of the time period). Therefore, qualitatively, there appeared to be a change in actual mean sea levels compared with predicted values. To test the significance of this observation, the paired t test was

undertaken (Table 8.2), as individual data sets were not normally distributed.

Table 8.2: Paired test statistics for actual and predicted annual mean sea level.

Year	Mean S	ea Level	Diff erence		Rank of	Signed
	Actual	Predicted	d	d ²	Difference	Ranks
1993	0.249	0.240	0.009	0.000081	2.5	+ 2.5
1994	0.248	0.240	0.008	0.000064	1	+ 1
1995	0.262	0.240	0.022	0.000484	4.5	+ 4.5
1996	0.231	0.240	- 0.009	0.000081	2.5	- 2.5
1997	0.266	0.240	0.026	0.000676	6	+ 6
1998	0.284	0.240	0.044	0.001936	9	+ 9
1999	0.269	0.240	0.029	0.000841	7	+ 7
2000	0.293	0.240	0.053	0.002809	[10 [+ 10
2001	0.200	0.240	- 0.040	0.001600	8	- 8
2002	0.262	0.240	0.022	0.000484	4.5	+ 4.5
Total Σ =			0.164	0.009056		

$$\begin{array}{lll} s & = & 0.0252 \\ \overline{d} & = & 0.0164 \\ SCd & = & 0.00841 \\ t_{calc} & = & \frac{\overline{d}}{0.00841} & = 1.950 \end{array}$$

$$t_{tab}(P = 0.05; df = 9) = 2.262$$

As $t_{tab} > t_{calc}$, the null hypothesis was accepted and consequently, at Newport, from 1993 to 2002 inclusive, there was no significant difference between actual and predicted annual mean sea levels. However, the data set does not fully comply with the requirements of a normal distribution, i.e. only 60% and not 68% of the data points lie in the range $\bar{\times} \pm s$ (Wheater and Cook, 2000). Also there was a possibility of a Type II error, i.e. the probability of accepting the null hypothesis when in fact it was false. Therefore a non-parametric test, i.e. the Wilcoxon matched-pairs test statistic was determined and from Table 8.2

$$T_{+} = 2.5 + 1 + 4.5 + 6 + 9 + 7 + 10 + 4.5 = 44.5$$

 $T_{-} = 2.5 + 8 = 10.5$

Taking the lower value $T_{calc} = 10.5$, from tables, T_{tab} (P = 0.05; n = 10) = 8. Therefore, as $T_{tab} < T_{calc}$, there was no significant difference and once again the null hypothesis was accepted and there was no significant difference

between actual and predicted mean sea levels over this time period (1993 to 2002 inclusive).

8.2.2 Extreme monthly sea levels

Although the monthly mean sea levels showed no significant change, the more important values, the extreme monthly sea levels, were subsequently evaluated. In most situations, wind set-up is a small component of the water level, wave set-up is very localised, and in the UK, tsunami are so rare that they are not significant in beach assessment (Simm, 1996). Where storm surges and seiches are significant, they are automatically included in tidal measurements. Therefore, when planning coastal infrastructure, extreme values are used in the design. According to Simm (1996) it was recommended that reliable published extreme water levels should be converted to the site of interest. Therefore, a probability distribution of extreme water levels at Penarth, where only basic astronomical tidal information was available, was correlated with Newport for which both tidal data and actual extreme water level data was available. The monthly extremes (Appendix 5) relative to Admiralty Chart Datum were converted to Ordnance Datum, as detailed in Table 8.3.

Table 8.3: Monthly Extreme Sea Levels - Newport (Source: BODC)

					Extreme	Sea Lev	el (m) AC)D			
Month	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002	10 Year Monthly Mean
Jan	N/A	6.832	6.846	7.090	7.130	7.147	7.687	6.807	6.459	6.835	6.981
Feb	N/A	7.523	7.677	6.947	7.825	7.486	6·259 ·	6.820	7.006	7.406	7.217
Mar	N/A	7.366	7.210	7.045	7.441	7.666	6.747	6.737	7.382	7.306	7:211
Apl	5.957	7.239	7.229	6.634	7.060	7.291	5.903	6.731	7.029	7.347	6.842
May	6.516	6.937	7.036	6.621	6.928	6.793	6.942	6.682	6.208	6.629	6.729
June	6.389	6.696	6.842	6.639	6.604	6.456	6.637	6.620	6.113	5.826	6.482
July	6.674	6.561	7.207	7.230	6.774	6.429	6.697	6.763	6.569	5.989	6.689
Aug	7.123	6.551	7.033	7.310	7.356	6.765	6.649	7.095	7.056	6.633	6.957
Sept	7.574	7.169	7:051	7.439	7.532	7.523	7.216	7.185	7.292	7.316	7.330
Oct	7.272	7.192	7.422	7.710	7.496	7.411	7.310	7.086	7.349	7.364	7.361
Nov	6.996	7.326	7.154	6.503	7.071	7.532	7.096	6.840	6.269	7.292	7.008
Dec	6.896	7.778	7.353	6.592	6.368	6.830	7.736	7.276	6.073	6.709	6.961
Annual Mean	6.822	7.098	7-172	6.980	7-132	7-111	6.907	6.887	6.741	6.888	6.974

Correlation of this data with Penarth was achieved by assuming that the ratio, identified by Graff (1981) was the same for both sites, i.e.

Extreme Level – mean high water spring (MHWS) level
Spring tide range (MHWS – MLWS)

The following astronomical data was obtained from Page and Oakley (2002):

Cardiff/Penarth Newport

MHWS
$$6.000 \text{m AOD}$$
 MHWS 6.290m AOD

MLWS -5.100m AOD MLWS -5.610m AOD

$$\therefore \frac{E_p - 6.000}{6 - (-5.100)} = \frac{E_N - 6.290}{6.290 - (-5.610)}$$
 $11.9 (E_p - 6.000) = 11.1 (E_N - 6.290)$
 $11.9 E_p = 11.1 E_N + 1.581$
 $E_p = 0.933 E_N + 0.133$

where E_p represents the extreme monthly water level at Penarth and E_N represents the measured extreme monthly water level at Newport.

Therefore, monthly extreme sea levels for Newport (Table 8.3) were converted using the above relationship, to produce monthly extreme sea levels for Penarth (Table 8.4).

Table 8.4: Monthly Extreme Sea Levels - Penarth.

					Extreme	Sea Lev	el (m) A(DD			
Month	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002	10 Year Monthly Mean
Jan	N/A	6.507	6.520	6.748	6.785	6.801	7.305	6.484	6.159	6.510	6.647
Feb	N/A	7.152	7.296	6.615	7.434	7.117	5.972	6.496	6.669	7.043	6.866
Mar	N/A	7.005	6.860	6.706	7.075	7.285	6.428	6.419	7.020	6.949	6.861
Apl	5.691	6.887	6.878	6.323	6.720	6.936	5.640 .	6.413	6.691	6.988	6.517
May	6.212	6.605	6.698	6.310	6.597	6.471	6.610	6.367	5.925	6.318	6.411
June	6.094	6.380	6.517	6.327	6.295	6.156	6.325	6.309	5.836	5.569	6.181
July	6.360	6.254	6.857	6.879	6.453	6.131	6.381	6.443	6.262	5.721	6.374
Aug	6.779	6.245	6.695	6.953	6.996	6.445	6.337	6.753	6.716	6.321	6.624
Sept	7.200	6.822	6.712	7.074	7.160	7.152	6.865	6.837	6.936	6.959	6.972
Oct	6.918	6.843	7.058	7.326	7.127	7.047	6.953	6.744	6.990	7.004	7.001
Nov	6.660	6.968	6.808	6.200	6.730	7.160	6.754	6.515	5.982	6.936	6.671
Dec	6.567	7.390	6.993	6.283	6.074	6.505	7.351	6.922	5.799	6.392	6.628
Annual Average	6.498	6.755	6.824	6.645	6.787	6.767	6.577	6.559	6.415	6.559	6.639

At the start of the monitoring period, September 1997, beach levels on the Penarth foreshore had fallen to critical levels (Phillips, 1999a;

SBCEG, 1999). In Chapters 2 and 3 it was shown that accepted longshore drift along the Penarth coastline was from south to north (Bullen 1993a; Wallingford 1992; SBCEG 1999). However, in Chapter 6 it was demonstrated that from September 1997 to September 1998, net longshore drift had changed direction from north to south, a process that had apparently been ongoing. Therefore, change had occurred between 1993 and 1997. To further refine the time period, anecdotal evidence was used. In 1995, the new lifeboat slipway for the RNLI was completed and in an interview with the owner of Chandler's Restaurant on the Esplanade, who was also a Town councillor and member of the RNLI, it was stated that the restaurant had been flooded twice since the end of 1995 compared to twice in the previous twenty years (Rabaiotti, 1997; pers comm). At that time, local fishermen believed that the beach had undergone change since the end of 1995 and blamed this on the barrage construction. The connection was made due to the completion in December 1995, of the 600m flank embankment (Plate 8.1) and in January 1996, the completion of the cofferdam (200m × 150m) for the sluice gate and lock structures. Furthermore, during 1997, this study also identified falling beach levels along the foreshore (Chapters 6 and 7).

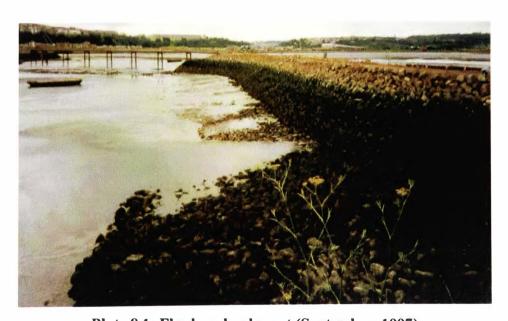


Plate 8.1: Flank embankment (September, 1997)

As water levels play a significant role in beach formation processes, analysis was undertaken on the annual mean extreme sea levels between 1995, when change in longshore drift was identified, and 1998 when longshore drift returned to accepted patterns (Section 6.2). This was to identify whether there was any significant change in extreme water levels during this period which could then be correlated to the change in beach formation processes. Variables such as sea level measured on continuous scales are often normally distributed (Wheater and Cook, 2000). Therefore, an independent t test was undertaken for annual mean extreme sea levels between 1995 and 1998 inclusive, compared with 1993 – 1994, and 1999 to 2002 inclusive (Table 8.4), coinciding with north to south and south to north longshore drift respectively. The null hypothesis was that there was no significant difference between annual mean extreme water levels at Penarth during these time periods (Table 8.5).

Calculation of t test statistic - Annual mean extreme sea **Table 8.5:** levels

Year	Annual Mean (m) AOD	Year	Annual Mean (m) AOD
1993	6.498	1995	6.824
1994	6.755	1996	6.645
1999	6.577	1997	6.787
2000	6.559	1998	6.767
2001	6.415		
2002	6.559		

$$n_1 = 6$$
; $x_1 = 6.561$; $s_1 = 0.103$

$$t_{calc} = 3.305$$

$$n_1 = 6$$
; $x_1 = 6.561$; $s_1 = 0.103$
 $n_2 = 4$; $x_2 = 6.756$; $s_2 = 0.067$

From statistical tables, for df = 8, $t_{tab} = 2.306$ (P = 0.05) and 3.355 (P = 0.01). Consequently the null hypothesis was rejected and the annual mean extreme sea levels between 1995 and 1998 inclusive were significantly higher at the 95% confidence level than during two years prior and four years subsequent to this period (t = 3.305; df = 8; P < 0.05). Indeed, the significance was approaching the 99% confidence level. However, because of the small sample numbers and the inherent hazard of a type I error, i.e. rejecting the null hypothesis when in fact it was true, a nonparametric test (Mann-Whitney U test) was also applied to the data. This test can be applied to normally distributed data but will be less powerful than the t test with the higher likelihood of accepting the null hypothesis when in fact it is false (type II error).

Table 8.6: Calculations of Mann-Whitney U test statistic Annual mean extreme sea levels.

Year	Annual mean extreme sea level	Ascending Order	Rank	
2001	6.415	1	_11	
1993	6.498	2	2	
2000	6.559	3	3.5	
2002	6.559	4	3.5	
1999	6.577	5	5	
1996	6.645	6	6	
1994	6.755	7	7	
1998	6.767	8	8	
1997	6.787	9	9	
1995	6.824	10	10	
	$n_1 (1995 - 1998) = 4$	$n_2 (1993-1994; 1999-2002) = 6$		

$$\therefore U_1 = 1$$

$$\therefore$$
 U, = 23

Taking smaller value of U

$$U_{calc} = 1 : n_1 = 4 : n_2 = 6 : U_{tab} (P = 0.05) = 2 : U_{tab} (P = 0.01) = 0$$

As $U_{calc} < U_{tab}$, the null hypothesis was rejected at the 95% confidence level, and the annual mean extreme sea levels between 1995 and 1998 inclusive, were significantly higher than during the two years prior and four years subsequent to this period ($U = 1 : n_1 = 4 ; n_2 = 6 : P < 0.05$). However, if the change in beach levels at Penarth had started in 1994, and logically the consequences had not become apparent until some years later, then annual mean extreme sea levels between 1994 and 1998 required analysis. The independent t test and Mann-Whitney U test were applied to years 1994 to 1998 inclusive ($n_1 = 5$) and 1993 with 1999 to 2003 inclusive ($n_2 = 5$). It was found, with 99% confidence for both parametric and non-parametric tests, that the annual mean extreme sea levels between 1994 and 1998 were significantly higher (t = 6.200; t = 8 ; P < 0.01 and $t = 0 : n_1 = 5 ; n_2 = 5 : P < 0.01$).

Although a conversion factor for mean sea levels at Penarth, similar to that for extreme sea levels was not available, there would obviously be a strong correlation between mean sea levels at Newport and Penarth. Therefore, the same tests for 1994 to 1998 and 1993/1999 to 2002, were applied to the annual mean sea levels for Newport, shown in Table 8.1. It was found that the independent t test (t = 0.187; df = 8; P > 0.05) and the Mann-Whitney U test (U = 11.5; n_1 = 5, n_2 = 5; P > 0.05) both confirmed the null hypothesis that there was no significant difference in annual mean sea levels for these years. Interestingly, from Table 8.1, it was noted that the highest and lowest annual mean sea levels occurred in 2000 and 2001 respectively, whilst these years corresponded to the third lowest and lowest annual mean extreme sea levels.

Correlations often arise between components of meteorological origin such as storm surge, wind set-up, wave set-up and even seiches. As extreme sea levels are significant in coastal infrastructure design and beach formation, it was qualitatively shown that significant differences in mean annual extreme sea levels coincided with significant changes on the Penarth foreshore, including beach loss and change in direction of longshore drift. Therefore it is argued that the changes at Penarth were as a result of components of meteorological origin. According to the Environmental Scientist (1999), 1997 was the third warmest year on record over the UK (1·1°C above the average between 1961 and 1990) and this was the year that Penarth beach levels fell to critical values (SBCEG, 1999). The Environmental Scientist (2000) further reported that the 1990s was the warmest decade on record with wetter winters and drier summers.

If these changes to the Penarth foreshore had been a common occurrence, e.g. cyclical, then there was likely to have been historical records of this. Available literature (Bullen 1993a and b; Wallingford 1992; SBCEG 1999) only referred to net south to north longshore drift and none highlighted, prior to 1997, any concerns with beach levels fronting the Esplanade. Indeed, the position of the MHW level, shown on OS maps

followed accepted trends along the foreshore. Therefore, it is argued that had the extreme sea levels from previous years been available and compared with 1994 to 1998, then there would have been the same significant difference. Consequently, it is probable that this period (1994 – 1998) of significant change to processes on the Penarth foreshore was unique and was directly related to a significant change in extreme sea levels as a result of components of meteorological origin. However, it has only been qualitatively shown that a period of significantly higher extreme sea levels coincided with significant losses of beach material and change in direction of longshore drift along the Penarth foreshore (Chapter 6). To determine whether there was a quantitative relationship between these changes, the extreme sea level data (Table 8.4) was further evaluated in conjunction with the analysis of wind data (Section 8.3) and related to the quantified beach changes shown in Chapters 6 and 7. Firstly however, for completeness, the sea level data was further examined to identify trends.

8.2.3 Sea Level Trends

8.2.3.1 Mean Sea Level

The annual average mean sea levels at Newport (Table 8.1) were plotted against the relevant years to produce Figure 8.1:

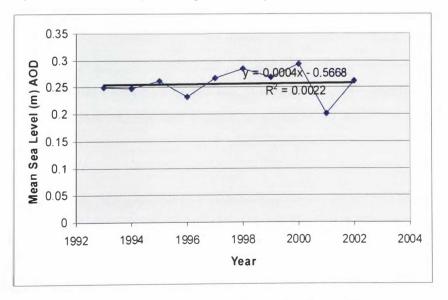


Figure 8.1: Annual variation of mean sea level.

According to Wheater and Cook (2000):

"Most computer programs use ANOVA to test the significance of the regression line. Some also display the results of a t test. If both sets of results are given, you only need to record the results of the ANOVA. If you are using a program which only calculates a t test, you should record the t value, number of degrees of freedom (n-2), and the probability (P), in addition to the regression equation (the intercept and slope) and R²."

(Wheater and Cook, 2000; 109)

In Section 7.4.2, it was shown by comparison of manual calculation (Table 7.22) and computer program (Figure 7.4) that EXCEL uses ANOVA to test regression line significance. Therefore, it is argued that only the regression equation and R² are required for this and subsequent analyses.

The scatter of data points (Figure 8.1) is typical when analysing temporal mean and extreme sea level trends (Douglas, 1992; Douglas 2001b; Jensen *et al*, 2001; Jensen and Mudersbach, 2004; Shennan and Woodworth, 1992). The regression equation y = 0.0004x - 0.5668, although not producing a significant predictive model ($R^2 = 0.0022$), showed a trend of increasing mean sea levels, illustrated by a rising gradient of 0.0004. This is equivalent to a projected rise in mean sea level of 0.4mm/year, considerably lower than the predictions of 2mm/year discussed in Section 2.4. For the UK and Europe, Shennan and Woodworth (1992) showed that global sea level rise, determined from tide gauge data, was 1.0 ± 0.15 mm/year.

Douglas (2001b) reported that authors have used sea level records as short as ten or twenty years in their analyses but he considered that this was inadequate. He argued that at least 60-year records were needed (Douglas, 1992) and later found more precise results for global sea level rise by using records longer than 70 years (Douglas, 1997). In these circumstances, the length of data set would improve the Coefficient of Determination (R²) and

produce a more reliable predictive model. Therefore, according to these works, no conclusions should be drawn from the analysis of this ten-year record. However, as the PSMSL records at Newport only started in 1993, a longer data-set was unavailable. In addition, the tide gauge data has been compared with the current Harmonic Constants library and residuals checked. Records have been visually inspected for spikes, datum shifts and timing errors (BODC; Appendix 6). Consequently, although short in timescale, it is argued that these results can be examined for a valid indication of trends. Tide gauges respond well to familiar semi-daily tides where the amplitude is typically of the order of a metre. This is 500 or more times the amount of yearly sea level rise (Douglas, 2001b). As the tidal range at Newport is 11.9m (Page and Oakley, 2002), and tide gauges are designed to filter out changes of water level occurring over periods of a few seconds due to waves, it is possible that the true rise in mean sea level at Newport is larger than 0.4 mm/year (Figure 8.1) and this trend may be masked by the large tidal range.

Heyworth and Kidson (1982) highlighted errors in estimation of tidal levels with large tidal ranges and noted that in these circumstances, distribution of wave energy is as variable as tidal range. However, from analysis of sea-level curves (Figure 3.1b), Cardigan Bay and Bridgewater Bay (Figure 3.2) are almost identical which suggested that in the last 8000 years, the rate and timing of sea level rise is comparable over the whole region (Heyworth and Kidson, 1982). According to Allen (1991), from an analysis of deposits in active marshes of the inner Severn Estuary, the average rate of rise of sea level ranged from 0.4mm/yr between the later Roman Period and the Medieval Period to 4.65mm/yr since *circa* 1945. Whilst this latter figure is >11 times that predicted in Figure 8.1, the work of Haslett *et al* (1998) questioned previous models and found from clay deposition, that relative sea level change was 0.41-0.82mm/yr. Therefore, although only based on a 10-year record, the trend of 0.4mm/yr determined from tide gauge data, agreed with some analyses of Holocene sea level rise.

8.2.3.2 Extreme Sea Level

The annual average extreme sea levels at Penarth (Table 8.4) were plotted against the relevant years to produce Figure 8.2:

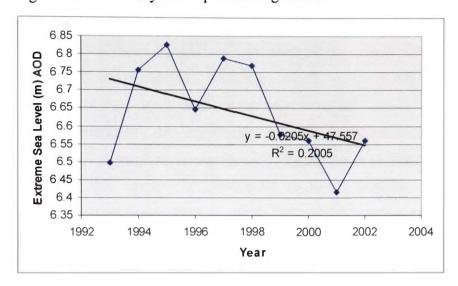


Figure 8.2: Annual variation of extreme sea level.

It was seen that over ten-year period, opposite to the temporal mean sea level trend, the extreme sea level trend was of falling sea levels, defined by the -0.205 gradient. This was not in agreement with the findings of Jensen and Mudersbach (2004) that showed increasing trends of extreme water levels following analysis of forty years of tide gauge data from twelve stations on the German North Sea. As in the previous section, the problem of the limited time-scale meant that no definitive conclusions could be drawn. However, it is argued that the significantly higher extreme water levels between 1995 and 1998 (Section 8.2.2), caused the apparent falling trend and further supported the argument that during these years, there was a change in beach formation processes along this section of coastline.

8.3 Analysis of Wind Data

On a long time scale, beach losses can occur as a result of offshore sediment transport by wind induced currents, whilst beach evolution is due to the combined effect of waves and tides (Section 2.2.1). In Section 2.2.3 it was seen that information on increases in wave heights in the Bristol Channel was based on the swell component and took no account of wind generated waves. Wave conditions at

Penarth are given in Section 3.2.2 and are statistically determined. Furthermore, according to Simm (1996) locally generated waves, which give rise to the largest wave heights at Cardiff, are uncorrelated with high water levels. This is opposite to findings in the Outer Bristol Channel, where swell waves from the west give rise to the largest wave heights and these correlate to high water levels. Consequently in the Penarth area, wind or storm generated waves are significant in providing the largest wave heights. Although wind data can be used to predict wave-climate where no such information exists, this is generally used for the design of coastal structures (Simm, 1996). Furthermore, it was shown by Maraghei, et al (2002) that generating wave data in this way can be problematic, leading to predicted wave heights of double the observed values. Phillips (1999a) alluded to changing weather conditions and wind direction between 1995 and 1997 as the cause of changing coastal processes at Penarth beach. Rivis, et al (2002) and Kont, et al (2004) highlighted extensive erosion and alteration to depositional coasts in Estonia, caused by a statistically significant increasing trend in annual storminess and high sea levels, especially during the 1990s. It has already been shown in Section 8.2.2 that extreme water levels were significantly higher between 1995 and 1998. Consequently, wind data was analysed to evaluate whether there was any significant changes in storminess and hence wave attack along the Penarth coast over this time period. Wind data was provided by the Met Office.

8.3.1 Mean Wind Speed and Direction

Monthly mean wind speed and directions between January 1993 and July 2002 were supplied for Rhoose, National Grid Reference (NGR) 3066E 1677N, which is located at an altitude of 65m Above Mean Sea Level (AMSL). This data is included in Appendix 7. Although it was recognised in Section 2.2.3 that wind speeds over land can show a 10-20% reduction compared to values over water, direction is apparently unaffected. Rhoose was the nearest station with available wind data and due to its proximity to Penarth (approximately 12 km west), as in Section 8.2 with respect to water levels at Newport, the data was considered valid for interpretation in this analysis. The monthly mean wind speeds and directions were transformed into annual values and shown in Table 8.7.

Table 8.7: Annual Mean Wind Speed and Direction (Rhoose)

Year	Mean Wind Speed (kn)	Standard Deviation ± (kn)	Mean Wind Directive	Standard Deviation ± (°)
1993	10.333	2.014	184.667	43.219
1994	11.083	1.801	194.417	37.649
1995	10.167	1.572	187.750	36.369
1996	10.333	1.886	169-917	41.981
1997	11.083	1.605	166.667	31.454
1998	9.667	1.434	191-250	19·464
1999	9.667	1.748	190.083	24.676
2000	9.750	1.963	187-917	32.556
2001	9.167	1.344	187.083	25.960
2002	10.714	1.906	201.000	34.422

From Table 8.7, it was seen that the mean wind speed between 1993 and 1997, was on average, 0.8 kn higher than the average between 1998 and 2002. However, wind direction is very important with respect to incoming wave attack on beaches. In Section 8.2.2 it was shown that extreme sea levels were significantly higher between 1995 and 1998 than for the periods either side (1992-1993 and 1999-2002). Therefore, for comparison, the independent t test was applied to the annual mean wind directions for these years.

In 1998, there was evidence of a return to the traditional south to north longshore drift (Section 6.3) whilst, in Section 1.1, the timeframe of beach loss had been identified as being between 1995 and 1997. Consequently, the independent t test was also undertaken for the annual mean wind directions between 1995 and 1997 inclusive, compared with 1993 to 1994 and 1998 to 2002 inclusive (Table 8.7). The null hypothesis in both analyses was that there was no significant difference in the annual mean wind directions during these time periods (Table 8.8).

Table 8.8: Calculation of t test statistics – Annual mean wind directions

Year	Annual	Year	Annual	Year	Annual	Year	Annual
	Mean (°)		Mean (°)		Mean (°)		Mean (°)
1993	184.667	1995	187.750	1993	184.667	1995	187·750
1994	194.417	1996	169-917	1994	194.417	1996	169-917
1999	190.083	1997	166-667	1998	191-250	1997	166-667
2000	187-917	1998	191-250	1999	190.083		
2001	187.083			2000	187-917		
2002	201.000			2001	187.083		
	1			2002	201.000		
$n_1 = 6$	$\bar{x}_1 = 190.861$	s ₁ =	5.437	$n_1 = 7$	$\bar{x}_1 = 190.91$	7 :	$s_1 = 5.035$
$n_2 = 4$	$x_2 = 178.896$	$s_2 =$	10.734	$n_2 = 3$	$\bar{x}_2 = 174.77$	'8 <u>s</u>	$s_2 = 9.268$
	t _{calc} =	2.361			t _{calc} =	3.675	

From statistical tables; for df = 8 then t_{tab} = 2·306 (P = 0·05) and 3·355 (P = 0·01), so in both cases the null hypothesis was rejected. Therefore, annual mean wind directions between 1995 and 1998 inclusive were significantly lower, i.e. increased easterly component at the 95% confidence level than during the two years prior and four years subsequent to this period (t = 2·361; df = 8; P < 0·05). There was now an agreement that at the time of severe erosion along the Penarth foreshore (1995 to 1997) there were significant differences in extreme water levels, mean annual wind direction and higher average annual wind speeds. Furthermore, the significance increased to 99% confidence limits for lower mean annual wind directions during the period 1995 to 1997 (t = 3·675; df = 8; P < 0·01). Interestingly, although the mean wind speeds had been higher during this period, the increases were found not to be significant (t = 1.188; df = 8; P > 0.05).

The WCB of Penarth beach is 17·035° (Section 6.3.7) which means that wind directions between 17·035° and 197·035° approach the beach from the sea. The lower the mean annual wind direction, the greater the incidence of easterly winds that comprise the mean value. Further reference to Table 8.7 showed that between 1993 and 1996 inclusive, the standard deviations for the mean wind direction were greatest which indicated more variability in wind direction during this period. To investigate the correlation between extreme annual water levels and mean annual wind directions, yearly data

for extreme water levels from Table 8.4 was plotted against wind direction values from Table 8.7, to produce Figure 8.3.

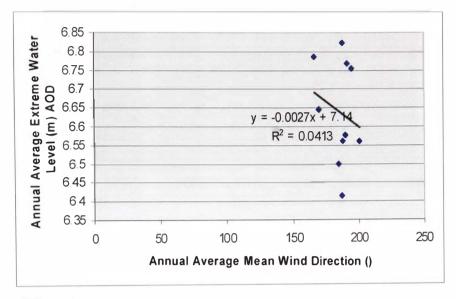


Figure 8.3: Variation of extreme water levels with mean wind direction.

There was a negative correlation and whilst the regression equation y = -0.0027x + 7.14 did not explain much of the variation ($R^2 = 4.13\%$) the trend was interesting. As extreme water levels reduced, the mean wind directions increased. Newport and Penarth are sheltered by land from the prevailing southwesterly winds and consequently it was shown that higher extreme water levels correlated with lower wind angles, that is, winds towards the north-east and south-east quadrants.

Using data from Table 8.7, the annual variation of mean wind speed and direction were plotted in Figures 8.4 and 8.5 respectively. Verification of the lower average annual wind speeds from 1998 onwards, was indicated by the negative gradient in the regression equation y = -0.0938x + 197.62 (Figure 8.4).

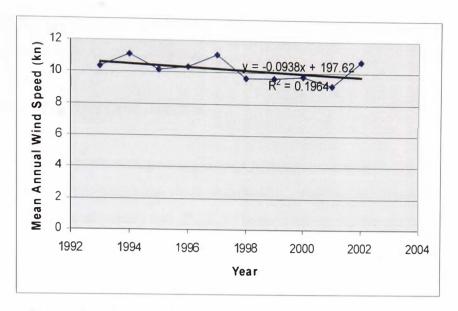


Figure 8.4: Temporal variation of annual mean wind speed.

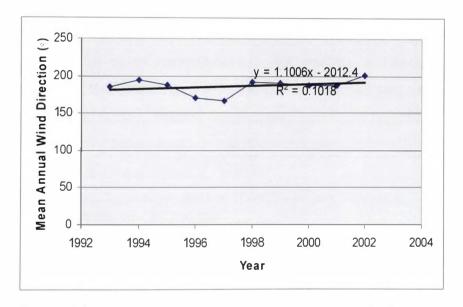


Figure 8.5: Temporal variation of mean annual wind direction.

From Figure 8.5 it was seen that the positive gradient in the regression equation, $y = 1 \cdot 1006x - 2012 \cdot 4$ indicated a trend of increasing mean annual wind direction, that is, towards the south-westerly quadrant. This would suggest that the recovery of Penarth beach from 1998 onwards, documented in Sections 6.2 and 6.3, coincided with more frequent winds from the prevailing south-westerly direction. However, it is the maximum wind speed and direction that would indicate storm conditions and the larger waves thus generated could have had most impact on beach

morphology and caused major changes. Therefore, in the following section these wind parameters were analysed.

8.3.2 Maximum Wind Speed and Direction

Discussions were carried out over a two-year period with the Met Office, for data on maximum wind speeds and direction. In September 2004, maximum gust speeds and direction were obtained for the period 1st January, 1993 to 31st December, 2002. However, five years of data (1993 to 1997) were from Rhoose (NGR 3066E 1677N) whilst between 1998 and 2002, data was supplied for St. Athan (NGR 2998E 1683N) which is located at an altitude of 49m AMSL. On contacting the Met Office it appeared that no data was available for Rhoose after 1997. St. Athan is approximately 18 km west of Penarth and as in Section 8.3.1, the data was considered valid for direct interpretation in this analysis. It comprised approximately 3,650 daily records (106 A4 sheets) of maximum gust speeds and direction and examples from both locations are included in Appendix 8. The daily records of maximum gust speeds and directions were transformed into mean annual values and shown in Table 8.9.

Table 8.9: Mean Annual Maximum Gust Speed and Direction (Rhoose and St. Athan)

Year	Mean Gust Speed (kn)	Standard Deviation (kn)	Mean Gust Direction (°)	Standard Deviation (°)
1993	25.715	9.865	186.510	100-255
1994	26.359	10.083	206.521	93.057
1995	25.477	9.377	195.945	99.269
1996	24.574	9.113	176.284	96.786
1997	25.880	9.386	179.327	93.254
1998	24.299	9.454	218-219	85-198
1999	23.742	9.444	232.438	84.701
2000	24.380	10.180	210.714	82.949
2001	22.627	7.915	200.331	96.331
2002	25.000	9.622	200.744	89.509

From Table 8.9 it was seen that the mean maximum gust speed between 1993 and 1997 was on average, 1.591 kn higher than the average between 1998 and 2002. However, as for the mean wind speeds, the difference between 1995 and 1997 was not significant compared with the periods either side (t = 1.011; df = 8; P > 0.05). Similar to the analysis of

mean wind speed and direction, the independent t test was also applied to the mean annual maximum gust directions between 1995 and 1997 inclusive and compared with 1993 to 1994 and 1998 to 2002 inclusive (Table 8.9). The null hypothesis was that there was no significant difference in the mean annual maximum gust directions between these time periods (Table 8.10).

Table 8.10: Calculation of t test statistic – Annual mean maximum gust direction

Year	Annual Mean (°)	Year	Annual Mean (°)
1993	186.510	1995	195.945
1994	206.521	1996	176.284
1998	218-219	1997	179-327
1999	232.438		
2000	210.714		
2001	200-331		
2002	200.744		
n ₁ = 7	$\bar{x}_1 = 207.925$ $s_1 = 13$		_ 2 700
	_	τ	$t_{\rm calc} = 2.790$
$n_2 = 3$	$x_2 = 183.852$ $s_2 = 8.$	641	

From statistical tables, for df = 8 then t_{tab} = 2·306 (P = 0·05) and 3·355 (P = 0·01). Consequently, once again the null hypothesis was rejected and from Table 8.10, the mean annual maximum gust direction between 1995 and 1997 inclusive was significantly lower at the 95% confidence level than during the two years prior and four years subsequent to this period (t = 2·790; df = 8; P<0·05). Similar to the results of the annual mean wind direction, the mean annual maximum gust direction for these significant years (1995 – 1997) meant the wind approached the beach from the sea. There was again a larger easterly component and from Table 8.9, the largest standard deviations in wind direction occurred between 1993 and 1997.

To investigate the correlation between extreme annual water levels and mean annual maximum gust directions, yearly data for extreme water levels from Table 8.4 was plotted against direction values from Table 8.9, to produce Figure 8.6. Once again there was a negative correlation and whilst the regression equation y = -0.0007x + 6.7842 did not explain much of the variation ($R^2 = 0.82\%$) the gradient showed that higher extreme water levels

correlated with lower maximum gust wind directions, i.e. winds from the north-east and south-east quadrants.

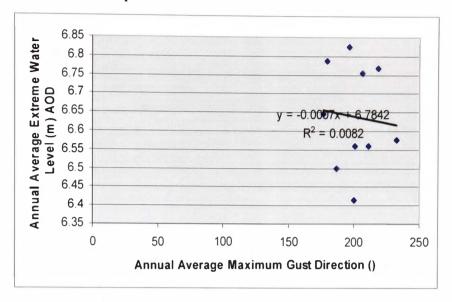


Figure 8.6: Variation of extreme water levels with maximum gust direction.

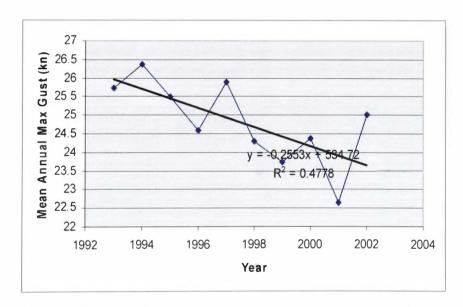


Figure 8.7: Temporal variation of annual maximum gust speed.

Using data from Table 8.9, the annual variation of maximum gust speed and direction were plotted in Figures 8.7 and 8.8 respectively. The regression equations y = -0.2553x + 534.72 and y = 2.218x - 4229.8 respectively, indicated a decrease in mean annual maximum gust and increase in mean annual gust directions from 1993 to 2002. Although they

only explained 47·78% and 14·89% of the variations, the trends agree with those of the mean annual wind speed and direction, that is, decreased wind speeds and increased WCB.

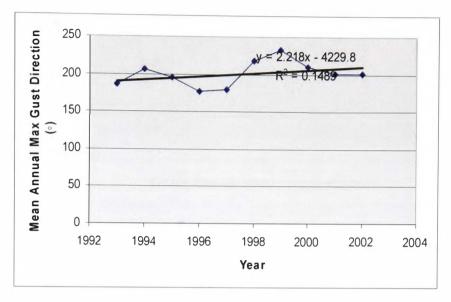


Figure 8.8: Temporal variation of annual maximum gust direction.

Table 8.11: Variations of annual average extreme sea levels with annual mean wind and mean maximum gust speeds

Year	Mean Extreme Sea Levels (m) AOD	Mean Wind Speed (kn)	Mean Maximum Gust speed (kn)
1993	6.498	10.333	25.715
1994	6.775	11.083	26.359
1995	6.824	10.167	25.477
1996	6.645	10.333	24.574
1997	6.787	11.083	25.880
1998	6.767	9.667	24.299
1999	6.577	9.667	23.742
2000	6.559	9.750	24.38
2001	6.415	°9·167	22.627
2002	6.559	10.714	25.000

For completeness, as there appeared to be much agreement in the temporal trends of wind speed and direction, relationships between annual average extreme water levels and annual average mean and maximum gust wind speeds were evaluated. Table 8.11 was produced using data from Tables 8.4, 8.7 and 8.9 and used to construct Figures 8.9 and 8.10.

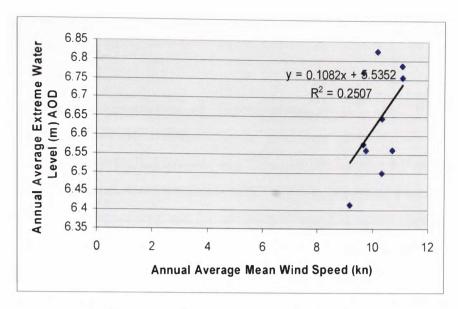


Figure 8.9: Variation of extreme water levels with mean gust speed.

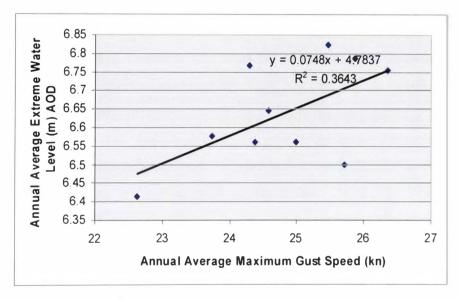


Figure 8.10: Variation of extreme water levels with maximum gust speed.

As was expected both Figures 8.9 and 8.10 showed a positive correlation between increased extreme sea levels and increased mean and maximum gust speeds. Although the correlation was stronger between the maximum gust speed than the mean wind speed (36·43% compared with 25·07%) the regression equations only explained some of the variation. This

may have been caused by the timing of the maximum gusts not coinciding with high water levels.

Section 6.3.5 documented an easterly storm on 18th May, 1999 (Plates 6.53 and 6.54) and reference to the wind data showed that the maximum gust speed recorded that day was 35kn and its direction 40° (Met Office, 2004). This wind speed was equivalent to 64 kph, measured 8 on the Beaufort Scale and was within the classification of a gale (Turner, 2003). Larger pebbles were moving freely over the Penarth foreshore and the September 1999 survey showed a loss of 2,526 tonnes from the foreshore, equivalent to 350 kg/m² (Table 6.15) whilst the groyned beach experienced a loss of 883 tonnes, equivalent to 275 kg/m² (Table 7.12). Table 8.12 (Rowlett, 2001) gives details of the Beaufort Scales of wind speed with equivalent conditions at sea and on land.

The lower end of Beaufort 5 is 17kn but examination of the wind data showed that over 90% of the recorded maximum gust speeds included or exceeded this value. Therefore, in the analysis of the maximum gust speeds, winds above this range had been effectively included. As swash velocities on pebble beaches had been documented as 4m/s (Williams and Phillips, 2000), it was decided to evaluate maximum gust speeds when pebbles would be freely moving over the beach surface. Therefore, maximum gust data for near gale, Beaufort 7 (28kn) and above was analysed. Monthly records of these winds between January 1993 and December 2002 were abstracted and analysed in the following section.

8.3.3 Maximum Gust Wind Speeds and Directions (≥ 28kn)

Table 8.13 shows the monthly mean maximum gust speeds, greater than or equal to 28kn, for the ten year data set (1993-2002). The directions of these gusts were similarly averaged and the results presented in Table 8.14.

Table 8.12: Beaufort Scales (Wind Speed).

Force		Speed	l	Name	Conditions	Conditions
	knots	kph	mph	Паше	at Sea	on Land
0	< 1	< 2	< 1	Calm	Sea like a mirror.	Smoke rises vertically.
1	1-3	1-5	1-4	Light air	Ripples only.	Smoke drifts and leaves rustle.
2	4-6	6-11	5-7	Light breeze	Small wavelets (0.2 m). Crests have a glassy appearance.	Wind felt on face.
3	7-10	12-19	8-11	Gentle breeze	Large wavelets (0.6 m), crests begin to break.	Flags extended, leaves move.
4	11-16	20-29	12-18	Moder ate breeze	Small waves (1 m), some whitecaps.	Dust and small branches move.
5	17-21	30-39	19-24	Fresh breeze	Moderate waves (1.8 m), many whitecaps.	Small trees begin to sway.
6	22-27	40-50	25-31	Strong breeze	Large waves (3 m), probably some spray.	Large branches move, wires whistle, umbrellas are difficult to control.
7	28-33	51-61	32-38	Near	Mounting sea (4 m) with foam blown in streaks downwind.	Whole trees in motion, inconvenience in walking.
8	34-40	62-74	39-46	Gale	Moderately high waves (5.5 m), crests break into spindrift.	Difficult to walk against wind. Twigs and small branches blown off trees.
9	41-47	76-87	47-54	Strong gale	High waves (7 m), dense foam, visibility affected.	Minor structural damage may occur (shingles blown off roofs).
10	48-55	88- 102	55-63	Storm	Very high waves (9 m), heavy sea roll, visibility impaired. Surface generally white.	Trees uprooted, structural damage likely.
11	56-63	103- 118	64-73	Violent storm	Exceptionally high waves (11 m), visibility poor.	Widespread damage to structures.
12	64+	119+	74+	Hurri- cane	14 m waves, air filled with foam and spray, visibility bad.	Severe structural damage to buildings, wide spread devastation.

Table 8.13: Mean Monthly Maximum gust speeds (≥ 28kn).

Month					Average	Maximum (Gust (kn)				
	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002	Ten Year Monthly Mean
Jan	42.7 (21)	40.2 (19)	41.8 (19)	34.8 (18)	31.9 (7)	42.6 (14)	35.9 (20)	38.0 (10)	35.2 (9)	38.2 (13)	38.1 (15)
Feb	30.4 (5)	34.4 (12)	38·1 (18)	36·1 (14)	43.5 (17)	32.8 (6)	34.3 (13)	35.7 (17)	32.4(5)	39.3 (24)	35.7 (13.1)
Mar	30.4 (8)	39.6 (20)	36.7 (19)	33.8 (12)	32.8 (10)	36.5 (10)	41.8 (5)	33.1 (12)	31.7 (11)	36.8 (10)	35.3 (11.7)
Apl	33·1 (9)	38.9 (14)	36.2 (5)	29.3 (4)	33.8 (4)	30.4 (10)	35.3 (8)	32.9 (7)	35.7 (9)	34.7 (10)	34.0 (8.0)
May	No data	32.6 (10)	29.0 (4)	31.3 (16)	30.9 (12)	30.5 (4)	31.2 (6)	32.6 (10)	36.5 (2)	33.5 (11)	32.0 (8.3)
June	No data	32.2 (9)	29.9 (7)	31.0 (3)	34.5 (11)	29.0 (5)	31.8 (4)	32.0 (4)	33.0(3)	32.3 (4)	31.7 (5.5)
July	No data	31.5 (2)	31.8 (6)	32.8 (5)	31.4 (5)	30.9(8)	30.5 (2)	30.3 (3)	31.0 (3)	31.5 (4)	31.3 (4.2)
Aug	38.0 (3)	32.4 (10)	34.3 (7)	33.2 (5)	35.4 (9)	34.4 (5)	32.0(2)	28.5 (2)	29.6 (5)	No data	33.1 (5.0)
Sept	34.9 (7)	33.0 (9)	33.6 (9)	34.8 (8)	31.7 (6)	32.4 (10)	32.1 (8)	31.8 (5)	33.8 (8)	30.7(3)	32.9 (7.3)
Oct	29.3 (4)	32.3 (10)	36.9 (8)	38.7 (13)	34.3 (11)	36.9 (17)	34.8 (10)	38·1 (18)	33.4 (12)	43.0 (8)	35.8 (11.1)
Nov	36.3 (8)	32.6 (5)	32.7 (9)	37-1 (15)	31.9 (14)	35.5 (11)	33.9 (8)	37.4 (18)	34.4 (8)	33.5 (16)	34.5 (11.2)
Dec	42.1 (26)	40.7 (20)	35.9 (8)	38.8 (11)	39.8 (15)	38·1 (11)	39.9 (21)	38·1 (17)	34.6 (10)	37.4 (10)	38.5 (14.9)
Annual Mean	35.2 (10.1)	35.0 (11.6)	34.7 (9.9)	34·3 (10·3)	34·3 (10·1)	34·2 (9·3)	34.5 (8.9)	34.0 (10.3)	33.4 (7.1)	35.5 (10.3)	34·5 (9·8)

Figures in brackets indicate the number of winds contributing to the mean.

Table 8.14: Monthly Mean Maximum Gust Direction

Month				Avera	ge Maximu	m Gust Dir	ection ≥ 28	kn (°)			
	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002	Ten Year Monthly Mean
Jan	264.0	267:4	253.7	118-9	67·1	260.0	234.0	252.0	166.7	245.4	212.9
Feb	268.0	116.7	252.8	158.6	247.6	265.0	276.9	265.9	198.0	241.7	229-1
Mar	187.5	274-5	258.9	79-2	198.0	267.0	260.0	230.8	169·1	195.0	212.0
Apl	185.6	217-9	152.0	200.0	267.5	185.0	226.3	132.9	260.0	187.0	201.4
May	No data	106.0	167.5	163·1	198-3	160.0	186.7	205.0	170.0	232.7	176.6
June	No data	250.0	157-1	296.7	117.3	266.0	265.0	252.5	263.3	250.0	235.3
July	No data	175.0	106.7	192.0	270.0	268-8	175.0	280.0	136.7	267.5	208.0
Aug	280.0	196.0	230.0	242.0	183-3	232.0	280.0	165.0	258.0	No data	229.6
Sept	147.1	162-2	184.4	158.8	261.7	249.0	235·1	242.0	262.5	226.7	213.0
Oct	90.0	207.0	225.0	234.6	170.9	259.4	214.0	262-2	233.3	200.0	209.6
Nov	165.0	246.0	174-4	244.0	195.0	257-2	250.0	231.6	263.4	221.3	224.8
Dec	258.4	240.5	61.3	143.6	206.7	242.7	251.9	181.8	254.0	170.0	201·1
Annual Mean	205·1	204-9	185:3	185-9	198.6	242.7	237-9	225·1	219.6	203·1	210-8

Note: Where no data is indicated monthly records were incomplete.

Directions in bold indicate an approach to the beach over water.

Table 8.14 was examined for incidences where the wind directions approached the Penarth foreshore from the sea, that is, between $17\cdot035^{\circ}$ and $197\cdot035^{\circ}$ (Section 6.3.7). Between January 1993 and December 1997, $^{28}/_{57}$ or 49% of the time, the maximum gust direction approached from the sea, whilst for the same time period between January 1998 and December 2002, the equivalent result was $^{14}/_{59}$ or 24%. Therefore, between 1992 and 1997, Penarth beach experienced over twice the number of near gale plus winds from the sea. The mean and standard deviation of the annual mean gust direction over the ten-year period was $210\cdot820 \pm 18\cdot920^{\circ}$. Interestingly, the same analysis of the annual mean gust speed (Table 8.13) gave a mean and standard deviation of $34\cdot510 \pm 0\cdot584$ kn, which showed that whilst there was a large variation in mean gust directions over the ten years, the mean wind speed showed little variation, irrespective of direction.

An independent t test was applied to the maximum gust directions (Table 8.14) between 1995 and 1997 inclusive and compared with 1993 to 1994 and 1998 to 2002 inclusive. The null hypothesis was that there was no significant difference in the annual maximum gust directions between these time periods (Table 8.15).

Table 8.15: Calculation of t test statistic – Annual mean maximum gust direction (Gusts ≥ 28 kn)

Year	Annual Mean (°) Year	Annual Mean (°)
1993	205·1	1995	185-3
1994	204.9	1996	185.9
1998	242.7	1997	198.6
1999	237.9		
2000	225·1		
2001	219.6		
2002	203·1		
$\mathbf{n_1} = 7$	$\bar{x}_1 = 219.800$ s_1	= 15·100 t _{cale}	s = 3·226
$n_2 = 3$	$\bar{x}_2 = 189.933$ s	₂ = 6·133	

From statistical tables, for df = 8 then t_{tab} = 2·306 (P = 0·05) and 3·355 (P = 0·01). Consequently the null hypothesis was rejected and from Table 8.15, the direction of the annual mean maximum gust \geq 28kn

between 1995 and 1997 inclusive was significantly lower at the 95% confidence level than during the two years prior and four years subsequent to this period (t = 3.226; df = 8; P<0.05). Indeed it was approaching the 99% level. Therefore, in the three analyses of wind direction (annual mean, mean annual maximum gust and annual maximum gust \geq 28kn) there was a significant difference in wind direction and in all cases; the direction of the wind was from the sea. Interestingly, with refinement of the wind data, the mean values of direction between 1995 and 1997 inclusive, for the three separate analyses, increased towards the southwest, i.e. 174.778° (mean); 183.852° (gust) and 189.933° (gust \geq 28 kn). This meant that during these years, there was a general change in wind direction with more easterly components across the range.

Using data from Tables 8.13 and 8.14, the annual variation of gust speed and direction were plotted in Figures 8.11 and 8.12 respectively. The regression equations y = -0.0697x + 173.73 and y = 2.9333x - 5648.5 respectively, similar to the earlier examples, indicated a decrease in maximum gust and increase in maximum gust direction between 1993 and 2002. Although they only explained 11.76% and 19.83% of the variations, they mirrored what had become a pattern for the wind analysis over the ten years.

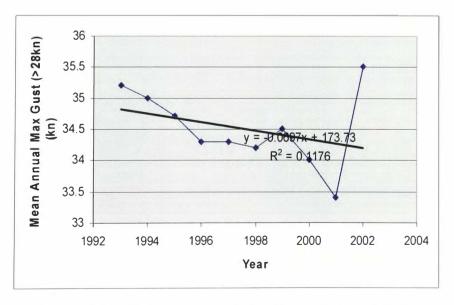


Figure 8.11: Temporal variation of annual gust \geq 28 km.

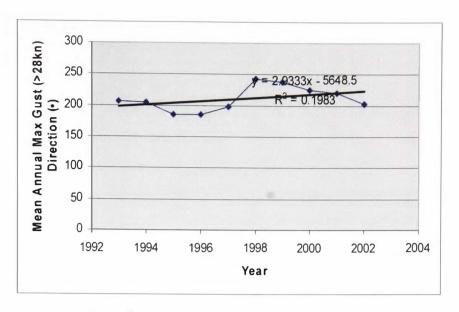


Figure 8.12: Temporal variation of annual gust (≥ 28 kn) direction.

To investigate the correlation between extreme annual water levels and the maximum gust and direction, yearly data for extreme water levels from Table 8.4 was plotted against gust speeds from Table 8.13 and gust directions from Table 8.14. The results are shown in Figures 8.13 and 8.14. Once again they confirm trends in earlier examples that extreme sea levels increased with increased wind speed and decreased with increased wind angle from north.

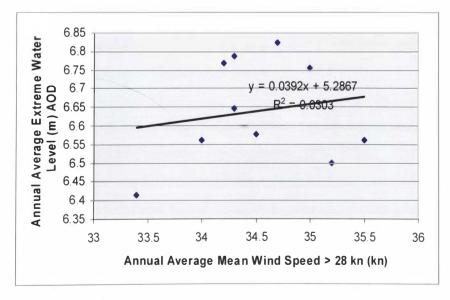


Figure 8.13: Variation of extreme water levels with maximum gust ≥ 28 kn.

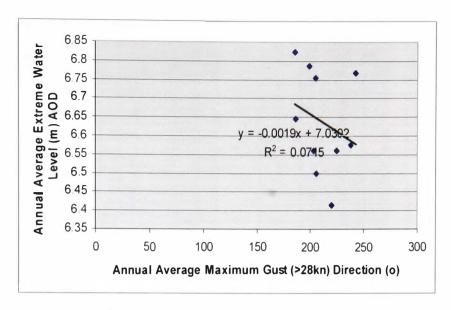


Figure 8.14: Variation of extreme water levels with maximum gust (≥ 28 kn) direction.

The evaluation of wind speeds and direction has shown some, if not strong correlation with extreme water levels. However, this was expected as the largest wave heights at Cardiff, are uncorrelated with high water levels (Simm, 1996). In all wind analyses, there was strong quantitative evidence to support an increased incidence of easterly wind components between 1995 and 1997 inclusive; the period which coincided with severe erosion on Penarth beach. The wind data between 1993 and 2002 was again examined to identify the frequency of gusts greater than and equal to 28 kn that came from the northeast quadrant, i.e. 0° to 90° true. Table 8.16 documents the incidence of these winds over the ten-year period.

Out of a total of 216, 74% ($^{160}/_{216}$) of these easterly winds occurred between 1993 and 1997 inclusive. Furthermore, 22% ($^{47}/_{216}$) of these occurred in 1996. For consistency with the previous wind analyses, an independent t test was applied to the number of easterly winds between 1995 and 1997 inclusive and compared with 1993 to 1994 and 1998 to 2004 inclusive (Table 8.16). The null hypothesis was that there was no significant difference in the number of easterly winds between these time periods and the results are shown in Table 8.17.

Table 8.16: Frequency of winds \geq 28 km (WCB 0 to 90°)

		N	umber	of days	with wi	nds ≥ :	28 kn fr	om NE	quadra	nt	
	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002	Total
Jan	0	0	0	10	7	1	0	0	4	0	22
Feb	1	7	0	4	0	0	0	0	1	1	14
Mar	3	0	1	10	0	0	0	$\overline{2}$	4	3	23
Api	3	4	3	1	0	3	1	4	0	4	23
May	5	8	2	7	4	2	2	3	1	0	34
June	0	1	3	0	7	0	0	0	0	0	11
July	0	1	5	2	0	0	1	0	1	0	10
Aug	0	4	2	0	3	0	0	1	0	0	10
Sept	2	4	3	4	0	1	0	0	0	0	14
Oct	3	2	0	0	5	0	2	0	l	3	16
Nov	2	0	1	2	3	0	0	0	0	0	8
Dec	1	1	8	7	4	0	0	4	1	5	31
Total	20	32	28	47	33	7	6	14	13	16	216

Table 8.17: Calculation of t test statistic – Incidence of winds from the northeast quadrant

Year	Annual Mean (No)	Year	Annual Mean (No)
1993	20	1995	28
1994	32	1996	47
1998	7	1997	33
1999	6		
2000	14		
2001	13		
2002	16		
$\mathbf{n}_1 = 7 \qquad \mathbf{x}_1 = 15$	•428 $s_1 = 8.139$		
		$t_{\rm calc} = 3.67$	4
$\mathbf{n_2} = 3 \qquad \mathbf{x}_2 = 36$	$s_2 = 8.04$		

From statistical tables, for df = 8 then t_{tab} = 2·306 (P = 0·05) and 3·355 (P = 0·01). Consequently, the null hypothesis was rejected and from Table 8.17, the number of winds from the northeast quadrant between 1995 and 1997 was significantly higher at the 99% confidence level (t = 3.674; df = 8; P < 0.01). As well as confirming earlier results this had further significance in that the trends of increasing wind direction for the mean wind and gust analyses were as a result of decreasing numbers of winds from the northeast quadrant. Figure 8.15, produced using data from Table 8.16, further illustrated the decreasing trend in numbers of these winds by the negative gradient of the regression equation (-2·3515).

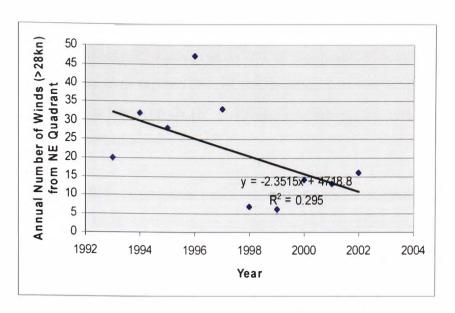


Figure 8.15: Temporal variation of incidence of NE winds.

To investigate the correlation between the incidence of winds from the northeast quadrant and extreme annual water levels, yearly data for extreme water levels from Table 8.4, was plotted against incidence of winds from Table 8.16. Results are shown in Figure 8.16.

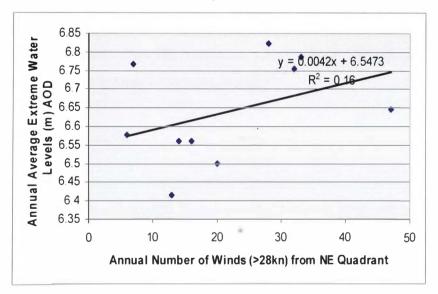


Figure 8.16: Variation of extreme water level with incidence of NE winds.

The gradient of the regression model, y = 0.0042x + 6.5473 confirmed that an increased incidence of winds from the NE quadrant would cause a rise in extreme water level. The total number of gust records

over the ten years (1993 to 2002), was 3652. From these, only 216 were from the northeast quadrant, equivalent to 6% of the time. Although the model initially appeared to explain little of the variation between incidence of winds and extreme water levels ($R^2 = 16\%$), this relationship was now more valid with respect to trend.

Matthews (2003) highlighted problems with statistical significance in that the methods used were prone to exaggerate real significance. The concerns were with respect to the length of data sets on which significance was tested and that *P* values take no account of the inherent plausibility of the hypothesis being tested. The water level and wind data sets each comprised 120 monthly results, and each of these was synthesised from daily records. Even the data set of the incidence of maximum gusts, equal to or greater than 28 kn, comprised 216 results. The hypothesis tested in each analysis was plausible as the question asked was basic to inferential statistics, that is, two means were compared and the null hypothesis was that there was no significant difference between them. Therefore, it is argued that the conclusions of the statistical analyses of the wind and water level data were valid. The next stage was to evaluate the effects of the analyses of extreme water levels and wind on Penarth beach.

8.4 Consequences for Penarth beach

It has been well documented throughout the Chapters in this thesis that the timeframe of severe erosion along Penarth beach occurred between 1995 and 1997. No problem had been apparent prior to 1995 and despite periodic episodes of erosion, recovery of the beach from 1998 onwards, has been shown (Sections 6.2, 6.3, 6.4 and 6.5). Erosion can be naturally or human induced. Anthropogenic activities identified as possible causes of the erosion, such as the construction of the Cardiff Bay Barrage and marine aggregate dredging required consideration. In Section 7.5, it was shown that the completion of the barrage stabilised the adjacent beach leading to increased beach levels whilst, from Sections 6.2, 6.3 and 6.4, the recovery of the beach from September 1998, occurred during the construction

phase of this structure. Therefore, it was concluded that the construction of the barrage was unlikely to have caused the severe erosion of Penarth beach.

Similarly, marine aggregate dredging operations were ongoing between 1993 and 2002. Indeed applications were received for increased extraction licences in the Bristol Channel (Wallingford, 1997). Brampton and Evans (1998) argued that because the marine environment is very difficult to predict, it was vital to fully consider potential impacts of nearshore dredging on the surrounding coast. However, studies of eroding beaches in the lee of dredging operations at Port Eynon and Horton, Gower, approximately 100 km west of Penarth, showed that beach erosion there, appeared dependent on storm frequency and that erosion and accretion were independent of dredging operations (Phillips and Lees, 2003; Phillips and England, 2001). Furthermore, as for the barrage construction, dredging activities were on-going during the beach recovery documented in September 1998. Consequently, it was concluded that marine aggregate dredging operations were unlikely to have caused the severe erosion of Penarth beach.

As anthropogenic impacts have been discounted, with the exception of the debated causes of perceived climate change (Section 2.3), then natural events are likely to have been responsible for erosion of the foreshore. It has been shown that there were significant changes in wind direction between 1995 and 1997 inclusive. Pack and Hartmann (2004) undertook studies on the risk of drowning in the Eastern Mediterranean and identified wind direction as critical in current generation. They hypothesised that a change in prevailing wind direction and the consequent currents generated in the 1990's, was a cause of increased drowning incidents. According to Environmental Scientist (1999), 1997 was the third warmest year on record over the UK whilst the 1990's were the warmest decade on record, with wetter winters and drier summers (Environmental Scientist, 2000). In Section 2.4 there were many examples given of increased storm activity and changes in weather patterns over recent years. Therefore, there is supporting evidence of the changed meteorological conditions which led to the significant changes in wind direction between 1995 and 1997. Furthermore, according to SBCEG (1999), the analysis of wind data from Rhoose between 1st January, 1960 and 31st October, 1986, showed that the

maximum wind speed in the northeast quadrant was 10 m/s, 19·5 kn or Beaufort 5, much lower than the wind speeds documented in Section 8.3.3.

In 1997, surveys along Penarth beach evidenced a reverse of the accepted longshore drift to a north to south regime (Sections 6.2 and 6.3) and it is recognised that increased storm frequency can have a major impact upon sediment distribution and beach morphology (Woodroffe, 2002). As the heading of the foreshore is approximately 17° to 197° true, wave attacks from the northeast quadrant would move material from the beach in a southerly direction, as evidenced in May 1999 (Section 6.3.5). Neves (2004), reporting on two decades of observations along the Portuguese coast, found that there was only erosion in the surf area when the wind came from the sea towards the land and accumulation when the wind approached the sea from the land. The prevailing wind direction along the Penarth coastline between 1st January, 1960 and 31st October, 1986, was from the southwest quadrant (41·1% between 180° and 270° true; SBCEG, 1999) then according to Neves (2004) these winds would not lead to erosion of the foreshore. However, there was an increase of winds with an easterly component between 1992 and 1997 and this increase became significant between 1995 and 1997 inclusive, the period of severe erosion along the foreshore. Therefore, it is qualitatively concluded that the cause of beach levels falling to critical values in 1997 (SBCEG, 1999) was an increase in wave attack from the northeast quadrant, generated by significant changes in wind direction.

A principal research objective in coastal geomorphology is to define trends of phenomena but it is recognised that there are problems inferring long-term trends from short-term data (Leatherman, *et al*, 1997). As the erosion of Penarth beach had occurred by the start of the monitoring period, September 1997, no physical measurements of prior beach levels had been made. Therefore, it was not possible to correlate changes in beach morphology between 1995 and 1997, with measured wind or water level data. Although storm events can cause short-term change, beach evolution is generally the resultant effect of coastal processes over time. In Section 6.3.7, it was argued that the average level of the foreshore grid represented the shoreline movement whilst the WCB of the MHW was the shoreline indicator. Consequently, these indicators were analysed in conjunction with the extreme

water level and wind data to determine quantitative relationships. Monthly extreme sea levels from Table 8.4 were averaged for the inter-survey periods, that is September to March inclusive, prior to the corresponding April survey and April to August inclusive, prior to the September survey.

Data for the monthly mean wind direction (Appendix 7), the maximum wind gust direction (Appendix 8) and maximum gust direction for winds greater than or equal to 28 kn (Table 8.14), was similarly analysed. Survey results over the five-year monitoring period, of mean beach levels for the foreshore grid (Table 6.14) and WCB of the MHW (Table 6.18) were incorporated into Table 8.18 alongside the corresponding extreme water levels and wind directions.

Table 8.18: Water, Wind and Shoreline indicators

Survey Date	Mean Beach Level (m) AOD	WCB MHW	Mean Extreme Sea Level (m) AOD	WCB Mean Wind Direction	WCB Mean Maximum Gust Direction (°)	WCB Mean Maximum Gust (>28kn) Direction (°)
9/97	2.475	12.496	6.612	170-400	195.038	207-280
4/98	2.449	12.185	6.899	172.857	191-840	232·329
9/98	2.535	13.551	6.428	201-400	220.915	222.360
4/99	2.616	15.402	6.796	192-571	226.462	254-171
9/99	2.457	14.660	6.259	179.000	200.654	226.600
4/00	2.477	15.615	6.760	196.714	247.089	242.810
9/00	2.547	15.837	6.457	170.600	197·124	207.080
4/01	2.478	16.286	6.695	179-571	199.770	207:342
9/01	2.518	16.608	6.286	193.400	210.061	217.600
4/02	2.534	16.775	6.601	194.857	207.762	242·186
9/02	2.554	16.827	6.183	209.750	223.791	234·300

Mean beach level and angle of the MHW were plotted against the intersurvey extreme sea levels as Figures 8.17 and 8.18. Both confirmed previous trends, that is, mean beach level and direction of the MHW were inversely correlated with extreme sea levels. However, both regression equations did not provide accurate models of the variation ($R^2 = 1.96\%$ and 14.48% respectively).

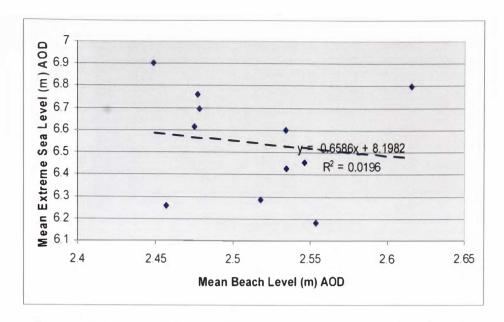


Figure 8.17: Variation of extreme sea level with mean beach level.

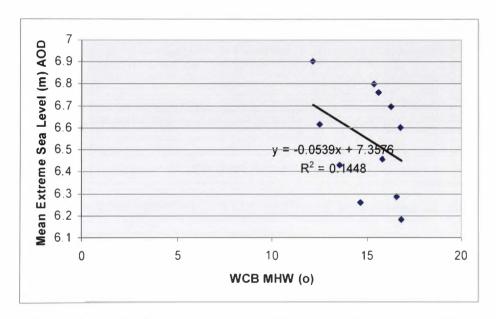


Figure 8.18: Variation of extreme sea level with bearing of MHW.

Figures 8.19, 8.20 and 8.21 showed the variation of mean beach level with mean wind direction, maximum gust direction and maximum gust (\geq 28 kn) direction respectively. In all cases there was a direct correlation between beach level and wind direction. However, the regression equations did not explain enough of the variation for a significant model ($R^2 = 24.28\%$, 14.84% and 14.91% respectively).

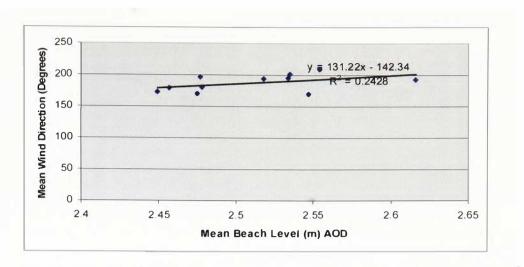


Figure 8.19: Variation of mean beach level with mean wind direction.

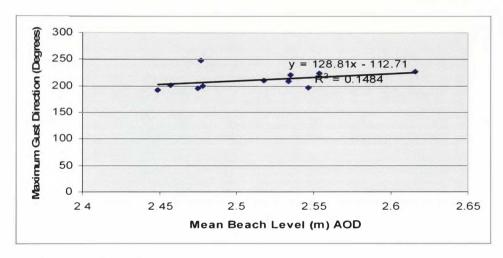


Figure 8.20: Variation of mean beach level with maximum gust direction.

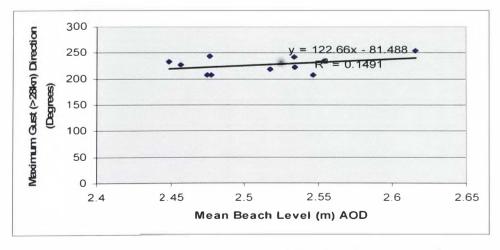


Figure 8.21: Variation of mean beach level with gust (≥ 28 km) direction.

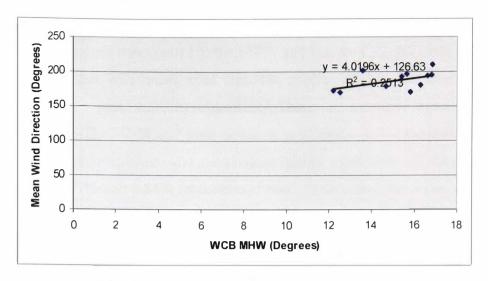


Figure 8.22: Variation of MHW with mean wind direction.

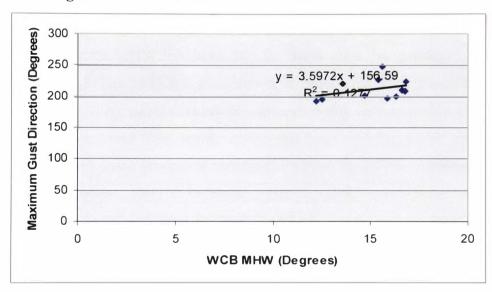


Figure 8.23: Variation of MHW with maximum gust direction.

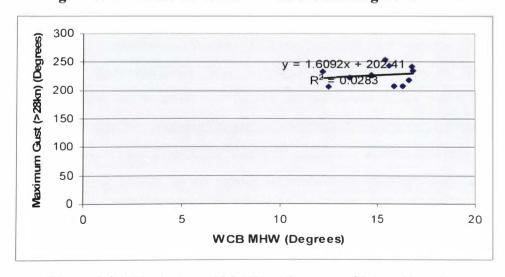


Figure 8.24: Variation of MHW with gust (≥ 28 kn) direction.

The same analysis was applied to the MHW and Figures 8.22, 8.23 and 8.24 illustrated its variation with mean wind direction, maximum gust direction and direction of maximum gust (\geq 28 kn) respectively. There was a positive correlation between the WCB of the MHW and wind direction in all cases. As for the previous analysis, the regression equations did not produce highly significant relationships ($R^2 = 25\cdot16\%$, $12\cdot77\%$ and $2\cdot83\%$ respectively) but in both cases, the mean wind direction produced the more reliable model ($24\cdot28\%$ mean beach level; $25\cdot16\%$ MHW). Again, it is likely that many of the maximum gusts occurred during periods of low water which would have had little influence on the pebble beach. Future work must include correlation of winds with state of tide.

The WCB of the MHW represents the plan beach shape (morphology) and the mean beach height represents the plan position. In Section 6.4 it was shown that there was a strong positive correlation between these shoreline indicators (61·61%). It has now been confirmed from survey results that lower extreme water levels correlate to increased beach levels and increased WCB of the MHW. Conversely increased wind direction, that is, increased angle to north, is correlated with increased beach levels and increased WCB of the MHW. These results support the conclusions drawn where significant differences in extreme water levels and even more significant differences in wind direction caused the severe erosion of Penarth beach between 1995 and 1997.

As discussed earlier in this Section (8.4), there is difficulty inferring long-term trends from short-term data. The consistent correlation between shore indicators (MHW and beach level) and forcing agents (water levels and wind) indicated that there may be more significant models than those shown in Figures 8.17 to 8.24 inclusive. At any given time, shoreline position is the cumulative result of the action of forcing agents. As the survey intervals were relatively short, it is reasoned that for each survey, the determined shoreline position was the cumulative result of forcing agents to that time. Therefore, for each successive survey, all results for the water level and wind directions were transformed to the survey time by averaging. For example, in Table 8.18, the mean wind direction in September 1999 was 179.000°. This was transformed to:

$$\frac{170 \cdot 400 + 172 \cdot 857 + 201 \cdot 400 + 192 \cdot 571 + 179 \cdot 000}{5} = 183 \cdot 246 ^{\circ}$$

(the average of results to that time); and the process repeated for all wind and extreme sea level data. This transformed data was included in Table 8.19 for comparison with the WCB MHW.

Table 8.19: MHW and transformed extreme sea level and wind data

Survey Date	WCB MHW (°)	Cumulative Average Mean Beach Level (m) AOD	Cumulative Average Extreme Sea Levels (m) AOD	Cumulative Average Mean Wind Direction (°)	Cumulative Average Max Gust Direction (°)
9/97	12.496	2.475	6.612	170.400	195.038
4/98	12.185	2.462	6.756	171.629	193-439
9/98	13.551	2.486	6.646	181-552	202.598
4/99	15.402	2.519	6.684	184-307	208·564
9/99	14.660	2.506	6.599	183·246	206.982
4/00	15.615	2.502	6.626	185-490	213·367
9/00	15.837	2.508	6.601	183-363	211.303
4/01	16.286	2.504	6.613	182-889	209.862
9/01	16.608	2.506	6.577	184.057	209-988
4/02	16.775	2.509	6.579	185-137	209-672
9/02	16.827	2.513	6.543	187-375	210.955

Figures 8.25, 8.26 and 8.27 showed the variation of the WCB MHW with cumulative extreme sea levels, cumulative mean wind directions and cumulative maximum gust directions respectively. The regression equation for the extreme sea levels, y = -20.004x + 147.57 (Figure 8.25) identified that 47.69% of the variation in the WCB MHW, was accounted for by the variation in extreme sea levels, and showed once again, an inverse correlation.

The regression equations shown in Figures 8.26 and 8.27, y = 0.2703x - 34.014 (mean wind) and y = 0.0329x - 32.985 (maximum gust), both highlighted that a significant amount of the variation in the WCB MHW was accounted for by the cumulative variation in wind direction, 79.26% and 85.53% respectively. In addition, they reinforced previously established trends of direct correlation (Figures 8.22 to 8.24).

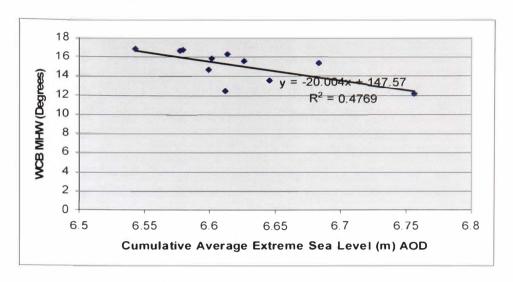


Figure 8.25: Variation of MHW with cumulative extreme sea level.

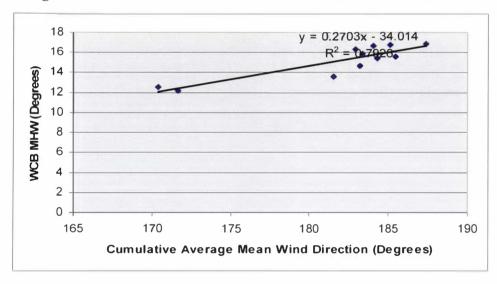


Figure 8.26: Variation of MHW with cumulative mean wind direction.

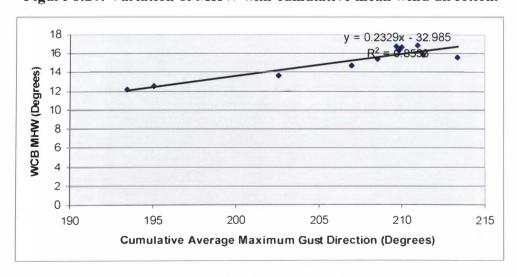


Figure 8.27: Variation of MHW with cumulative maximum gust direction.

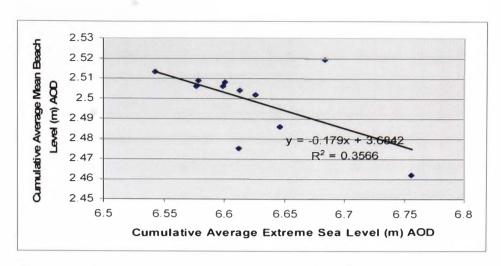


Figure 8.28: Variation of cumulative mean beach level with cumulative extreme sea level.

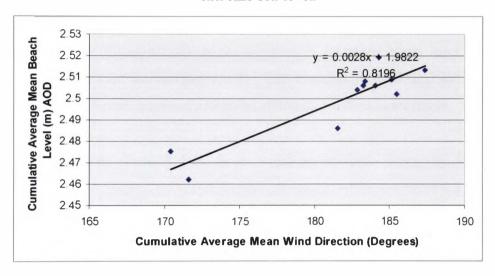


Figure 8.29: Variation of cumulative mean beach level with mean wind direction.

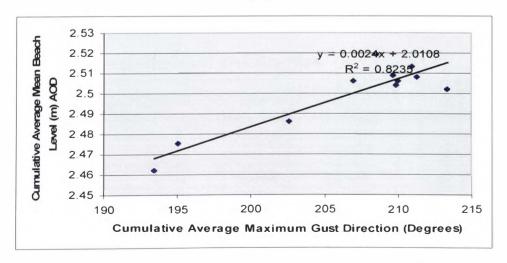


Figure 8.30: Variation of cumulative mean beach level with maximum gust direction.

To compare the variation of mean beach level with extreme sea level and wind direction, the cumulative average beach levels from Table 6.19 were included in Table 8.19. Figures 8.28, 8.29 and 8.30 showed the variation of the WCB MHW with cumulative extreme sea levels, cumulative mean wind direction and cumulative maximum gust directions respectively. The regression equation for the extreme sea levels, y = 0.179x + 3.6842 (Figures 8.28) identified that 35.66% of the variation in the mean beach level, was accounted for by the variation in extreme sea levels and confirmed previously established negative correlations.

The regression equations for wind direction, Figures 8.29 and 8.30, y = 0.0028x + 1.9822 (mean wind) and y = 0.0024x + 2.0108 (maximum gust) both highlighted that a significant amount of the variation in the mean beach level was accounted for by the cumulative variation in wind direction, 81.96% and 82.35% respectively and reinforced previously established trends of direct correlation.

The link survey (Section 6.5), gave longshore gradients from the 567m length of beach, uninterrupted by anthropogenic structures whilst Sections 6.2 and 6.3 documented measured changes in direction of longshore drift. As longshore gradients reflect sediment movement, directions of mean wind, maximum gust and maximum gust \geq 28kn were evaluated in conjunction with the surveyed longshore gradients. Table 8.20 was generated from longshore gradient data (Table 6.27), separated into the two beach headings, NNE and NNW, with corresponding wind direction data from Table 8.18.

Table 8.20: Longshore gradients and wind directions

Survey Date	Longshore Gradients (%)		WCB Mean Wind Direction	WCB Mean Maximum Gust	WCB Mean Maximum Gust (>28kn)
	750m - 1245m	1245m - 1317m	(°)	Direction (°)	Direction (°)
9/99	0.450	-0.778	179.000	200.654	226.600
4/00	0.454	-0.668	196.714	247.089	242.810
9/00	0.482	-0.854	170.600	197·124	207.080
4/01	0.414	-0.368	179.571	199.770	207·342
9/01	0.432	-0.500	193-400	210.061	217-600
4/02	0.390	-0.194	194.857	207.762	242.186
9/02	0.336	0.292	209.750	223.791	234.300

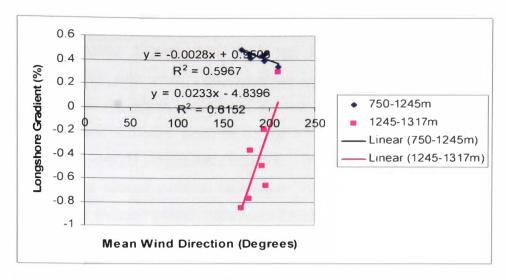


Figure 8.31: Variation of longshore gradients with mean wind direction

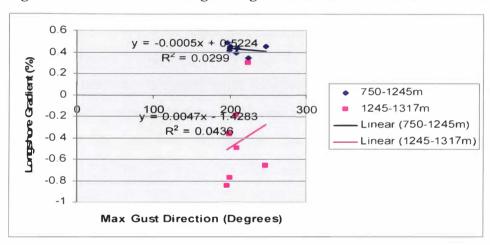


Figure 8.32: Variation of longshore gradients with maximum gust direction

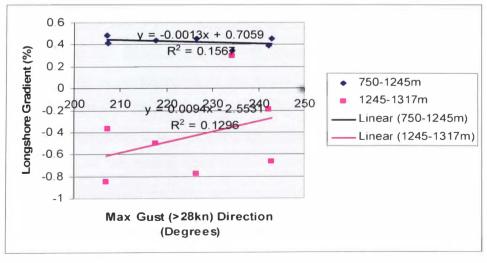


Figure 8.33: Variation of longshore gradients with maximum gust (≥ 28 kn) direction

Subsequently, Figures 8.31, 8.32 and 8.33 showed the variation of longshore gradient with mean wind, maximum gust and maximum gust ≥ 28 km directions. The regression equations showed inverse correlations between longshore gradient and wind direction for the NNE heading (750m to 1245m): -0.0028 (Mean), -0.0005 (Maximum Gust) and -0.0013 (Maximum Gust ≥ 28 kn). Conversely, they showed positive correlations for the NNW heading (1245m to 1317m): 0.0233 (Mean), 0.0047 (Maximum Gust) and 0.0091 (Maximum Gust \geq 28 kn). It had already been shown in Section 6.5, that there was a 97.37% correlation between the NNE and NNW longshore gradients and this analysis demonstrated that winds from the northeast quadrant would result in steeper longshore gradients along both beach headings, as a consequence of loss of beach material. This would explain the erosion of the beach adjacent to the manhole located on the transition between beach headings (Section 6.5). The significant increase in easterly winds between 1995 and 1997 would have caused loss of material in both directions from this location and resulted in the cover slab being supported above the beach surface. This effect was demonstrated on a smaller scale in Sections 6.3 and 7.3.4 when the main foreshore and groyned beach both suffered a loss of beach material following the northeasterly storm of 18 May, 1999.

From Figure 8.31, both regression equations: y = -0.0028x + 0.9508 (750m to 1245m) and y = 0.0233x - 4.8396 (1245m to 1317m) showed that the mean wind direction explained much of the variation of longshore gradient, 59.67% and 61.52% respectively. This means that the models would be valid for future management projections. Interestingly, the models for maximum gust (Figure 8.32) and maximum gust ≥ 28 kn (Figure 8.33) did not produce similar significant relationships. Therefore, it can be concluded that beach formation at Penarth is more dependent on the general wind climate as usually, most storms would come from the prevailing south-westerly direction, thereby approaching the beach over land.

Valid quantitative models have now been established from the results of the five-year monitoring of the Penarth foreshore, with respect to the shoreline indicators (MHW and beach level) and forcing agents (sea level and wind).

Although these models could be used to predict changes in the WCB MHW and mean beach level, other models developed in Sections 6.2, 6.3 and 6.4 would be easier to implement. However, their significance is clear. As they were generated from the results of physical monitoring and demonstrated significant relationships between shoreline indicators and forcing agents, they have provided a quantitative link.

In Section 8.3 it was qualitatively shown that the severe erosion of Penarth beach between 1995 and 1997 was caused by significant changes in wind direction, exacerbated by significant increases in extreme sea levels. Therefore, it has now been shown both qualitatively and quantitatively, that the erosion of Penarth beach between 1995 and 1997 was caused by increased wave attack from the northeast and southeast quadrants, generated by changes in wind speed and significant changes in wind direction. Furthermore, the critical change with respect to the foreshore over this period was the increased frequency of storm winds from the northeast quadrant (Table 8.16). As identified in Sections 2.2.3 and 6.3.5, these winds would have led to the largest waves with a consequent southward drift of beach material.

8.5 Summary

The water level and wind analyses clearly showed the reasons for the severe erosion, between 1995 and 1997, along the Penarth foreshore. The main findings of the general evaluation were as follows:

- The annual mean extreme sea levels between 1995 and 1998 were significantly higher (95% confidence limits) and between 1993 and 2002 there was an overall trend of falling extreme sea levels (-0.0205);
- The annual mean wind direction between 1995 and 1998 was significantly lower (95% confidence limits) and between 1995 and 1997 the significance reached 99% confidence limits. In this context, as direction is measured from North, then lower values indicate more easterly components. However, the overall trend between 1993 and

- 2002 was of increasing wind direction, i.e. movement towards the southwest quadrant (+ 1.1006);
- Between 1993 and 2002 there was a trend of reducing annual mean wind speeds although this was small (-0.0938);
- The annual mean maximum gust direction between 1995 and 1997, was again significantly lower (95% confidence limits) and similar to mean wind direction, between 1993 and 2002, there was an overall trend of increasing wind angle towards the southwest (+ 2·218);
- Between 1993 and 2002, there was an overall trend of reducing annual mean maximum gust speeds (-0.2553);
- The annual mean maximum gust direction for winds ≥ 28 kn between 1995 and 1997, was again significantly lower (95% confidence limits). Similar to the other two wind evaluations, between 1993 and 2002, there was the largest overall trend of increasing wind direction towards the southwest (+ 2.9333);
- Between 1993 and 2002, again similar to the other wind evaluations, there was an overall trend of reducing annual mean maximum gust speeds ≥ 28 kn (-0·0697). However, this was the lowest overall trend and indicated that there was little change in the intensity of storms, only the direction;
- The number of maximum gusts generated from the northeast quadrant between 1995 and 1997, were significantly higher (99% confidence limits). This was supported by the trend, between 1993 and 2002, of reducing incidences of winds from this quadrant (-2·3515);
- There was a positive correlation between extreme water levels and incidence of easterly winds.
- The projected rise in mean sea level based on records between 1993 and 2002, was 0.4 mm/year, significantly lower than other forecasts.

Following application of sea level and wind data to the survey results collected whilst monitoring the foreshore, the conclusions drawn were as follows:

- Shoreline indicators, mean beach level and direction of the MHW, were inversely correlated to extreme sea levels, that is, higher sea levels resulted in a loss of beach material and a reduction in the bearing of the MHW;
- The direction of the MHW was positively correlated with wind directions (mean, maximum gust and maximum gust ≥ 28 kn), that is, the WCB of the MHW increased with the WCB of the wind;
- There was a positive correlation between mean beach level and wind direction (mean, maximum gust and maximum gust ≥ 28 kn), that is, mean beach levels increased with WCB of the wind. Therefore, beach material was lost when winds came from the northeast quadrant and accumulated when winds were from the southwest quadrant;
- Significant models were determined for the variation of the MHW line and cumulative wind direction. These were y = 0·2703x 34·014 (mean wind) and y = 0·0329x 32·985 (maximum gust) and they explained 79·26% and 85·53% of the variation respectively;
- Models were determined for the variation of cumulative mean beach level with cumulative wind direction. These were y = 0.0028x + 1.9822 (mean wind) and y = 0.0024x + 2.0108 (maximum gust) and both highlighted that a significant amount of the variation in the mean beach level was accounted for by the variation in wind direction, 81.96% and 82.35% respectively;
- The regression equations: y = -0.0028x + 0.9508 (750m to 1245m) and y = 0.0233x 4.8396 (1245m to 1317m) showed that the mean wind direction explained much of the variation of longshore gradient, 59.67% and 61.52% respectively.
- In general, mean wind direction was more significant than maximum gust direction, to beach formation processes.

During the period 1995 to 1997 inclusive, there were statistically significant differences in extreme sea levels and wind direction. Furthermore, between 1993 and 2002 inclusive, there were definite trends of reducing extreme sea levels, wind speeds (mean and maximum gust) and increasing WCB of mean wind and maximum gust directions (resulting in winds approaching the beach over land). Whilst it is acknowledged that wind speeds over land may be 10 to 20% lower than over water (Simm, 1996), direction is unaffected. Relationships showed that there was correlation between the forcing agents, extreme sea level and wind direction and the shoreline indicators, mean beach level and direction of MHW. Higher extreme water levels resulted in lower mean beach levels and reduced WCB MHW, the same outcomes for wind angles approaching the beach over water. In Section 8.3 direct relationships were shown between extreme sea level and wind direction whilst, in Section 6.3, it was shown that there was a direct relationship between WCB MHW and mean beach level. The reliability of the statistical significance of all changes in wind direction and extreme sea level between 1995 and 1997 was established whilst possible anthropogenic causes of erosion were discounted. There was supporting evidence for the changes in meteorological conditions at that time whilst there was little change in wind speed, only direction. Conclusions that erosion of the foreshore coincided with winds approaching the beach over water, were supported by changes in prevailing wind direction. Normally, winds from the southwest approach Penarth beach over land but the erosive effects of a sea approach were illustrated by northeasterly generated storms. These findings were further validated by the work of Neves (2004) who concluded that there was only erosion when the wind approached the beach over water. Furthermore, valid quantitative models were developed from the results of the five-year monitoring of the Penarth foreshore, with respect to the shoreline indicators (MHW and beach level) and forcing agents (sea level and wind). Therefore, supported by qualitative and quantitative evidence, it was concluded that the erosion of Penarth beach between 1995 and 1997, was caused by increased wave attack from the northeast and southeast quadrants, generated by changes in wind speed and significant changes in wind direction and extreme sea levels.

The consequences and effects of these analyses, in conjunction with the survey results and models developed for the foreshore and groyned beach (Chapters

6 and 7), will be used to produce management strategies for the Penarth coast (Chapter 9).

CHAPTER 9: MANAGEMENT STRATEGIES FOR THE PENARTH COAST

9.1 Introduction

With respect to management strategies, the words of Leonardo da Vinci seem appropriate:

'Although nature begins with the cause and ends with the experience, we must follow the exact opposite course, namely begin with the experience and by the means of it investigate the cause.'

(Leonardo da Vinci, Notebooks, 1512)

Beaches are used for recreation and tourism more than any other habitat in the coastal zone which, in turn, causes the value of adjacent land to increase (Clark, 1996). However, traditional hard structures such as breakwaters, seawalls and groynes may be cost effective but often the structure survives and the beach is lost (Cipriani *et al*, 2004). The monitoring of beach erosion and its implications to and from coastal tourism are among the many issues of planning and management of the coastal zone. Therefore adopted strategies should be considered within a wide technical and political scenario

In Chapter 4, theoretical management strategies were considered with appropriate response options for the protection of the coastal environment. Beach management has a spatial and temporal dimension and from a five-year monitoring programme (September 1997 to September 2002) of 1,464m of the Penarth coast, trends and changes in coastal processes have been evaluated. Models have been developed to represent beach evolution enabling predictions to be made. Future comparisons with time now, over any timescale as part of on-going monitoring, would identify change and provide a basis for decision-making (Chapters 6 and 7).

Beach management should identify external developments which can influence natural beach dynamics and the effects of the Cardiff Bay Barrage and offshore marine aggregate dredging were evaluated. Temporal dimensions should address timeline variations and causal change which led to evaluation of water level and wind data.

Analysis of sea levels and wind data showed that when compared with other years between 1993 and 2002, there were significant differences in these forcing agents between 1995 and 1997. As this period (1995 to 1997) coincided with severe erosion of the foreshore and quantitative relationships had been established between forcing agents and shoreline indicators (MHW and mean beach level), it was shown that the severe erosion along the foreshore was due to significant changes in wind direction from the NE quadrant during these years (Chapter 8).

According to Johnson and Scholes (1988) the first requirement of strategic management, is analysis, where one seeks to understand:

- the content of a system,
- the status of existing management, if any, and
- whether one should take any action.

Beach measurements and analysis have led to an understanding of coastal processes whilst existing management has resulted in the stabilised shoreline (seawalls, groynes and breakwater). It was highlighted by SBCEG (1999) that there was a need to review local management options across the frontage and to identify a coastal defence strategy for Penarth. It is argued that implementation of the beach models presented in this thesis indicates whether action should be taken. Therefore the first requirement of strategic management has been met.

Figure 9.1 produced with data from Tables 6.1, 6.24 and 7.26 shows the temporal variation of longshore profile between September 1997 and September 2002, for the full monitored length of shoreline (0m station 1: 1,464m barrage breakwater). The level variation is the cumulative result of changed direction of longshore drift (Chapter 6) storm events (Chapters 6, 7 and 8) and construction of the Cardiff Bay Barrage (Chapter 7). Consequently, management options will need to respond to these functions of beach evolution.

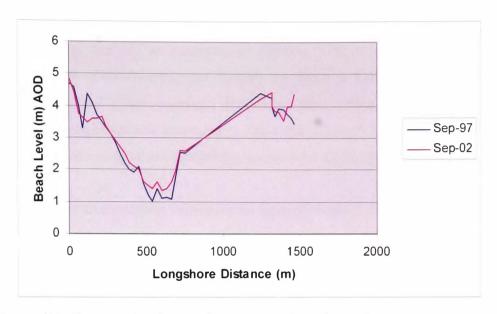


Figure 9.1: Temporal and spatial variation of beach levels between September 1997 and September 2002.

With regard to the second and third requirements of strategic management (Johnson and Scholes, 1988), that is, choice of different courses of action available and implementation of chosen option, Sauer (1963) considered that environmental management was the integration of the physical and cultural environments. Effective beach management is usually considered as a response to a specific interaction of cultural influences with the physical environment (Williams and Davies, 1997) and should be dynamic in approach (Nelson *et al*, 1997). From a qualitative aspect there is a need to identify socio-economic and environmental values associated with beach recreation and construction. The cultural environment will be assessed from the application of function analysis to the Penarth environment and the consideration of risk of failure of existing management. From these evaluations, including consideration of future developments, beach management strategies will be proposed and implementation justified. Finally, future work will be discussed to enhance this study and refine management responses.

9.2 Function Analysis

Function analysis as discussed in Section 5.10, was applied to the total surveyed length of the Penarth coast. Results are shown in Tables 9.1 (Environmental Indicators) and 9.2 (Socio-economic Indicators) from which normalised parameter scores were determined and presented in Table 9.3. Figure 9.2, produced from Table 9.3, illustrates the comparison between the normalised scores of all parameters.

The low characteristic evaluation for litter (2: Table 9.1) was due to significant accumulations of persistent marine debris on the groyned beach, following barrage completion. This is despite vast quantities of litter being prevented from entering coastal waters from riverine sources. Four hundred tonnes of debris was collected in the first ten months operation of a boom installed across the River Taff to prevent litter entering the freshwater lake behind the barrage (The Post, 2002). Previously, this had freely entered the coastal environment. In the methodology of both van der Weide, *et al* (1999) and Micallef (2002), mineral resources were considered as an environmental or conservation component. However, in this analysis, due to offshore marine aggregate dredging, it was considered as a socio-economic or development component. This change helped further balance the environmental and socio-economic components in line with the recommendations of van der Weide, *et al* (1999). Furthermore, high employment or high unemployment could be argued as being more beneficial for

Table 9.1: Environmental Indicators.

Environment Component	Characteristic	Indicators	Characteristic Evaluation PENARTH
Air	Pollution	Gravity	4
		Visibility	5
		Effect On Humans	4
	Noise	Intensity	4
Coastal Waters	Quality	_Microbiological Pollution	4
		Chemical Pollution	4
	Aesthetic Condition	Turbidity	1
		Litter	2
Fresh Water	Supply	River Flow	3
		Pollution	4
		Turbidity	3
		BOD	5
Terrestrial	Quality	Natural Vegetation Cover	2
Biota	Quantity	Biological Productivity	2
		Biological Diversity	2
		Species of Special Interest	2
Marine Biota	Quantity	Biomass	1
		Biological Productivity	1
	Diversity	Biological Diversity	1
		Species of Special Interest	1
Geological & Top	pographical Features	Lithological	5
		Geomorphology	3
		Quality	3
		Size of Bathing Area	4
Hazards		Coastal Erosion	2
		Coastal Flooding	3
		Storms	1
		Sedimentation	2
		Cliff/ Slope Instability	2
		Soil erosion	2
		River Flooding	N/A
Resources	Non Renewable	Soil (Fertility)	N/A
	Renewable	Fish	3
	Landscape	Visual Quality	3
	F	Uniqueness	3

Table 9.2: Socio-economic Indicators.

Socio-economic Component	Characteristic	Indicator	Characteristic Evaluation PENARTH
Human Aspects	Potential For Use	Historic, Artistic, Archaeological sites	3
		Minerals, Rocks, Construction Materials, Fuels	N/A
		Fishing	4
		Recreation Facilities	3
		Hotel, Restaurants	4
		Utilities	3
		Parking	3
		Accessibility	4
		Land Use – different types	3
		Land Ownership	5
		Extent of Development	5
		Population Density	5
		Intensity of Use	3
	Human Well-being	Public Health	5
		Unemployment	3
		Education	5
		Crime	4
		Development Potential	4

Table 9.3: Normalised scores for parameters at Penarth.

Parameters	Normalised Score		
Air	0.85		
Coastal Waters	0.55		
Fresh Water	0.75		
Terrestrial Biota	0.4		
Marine Biota	0.2		
Geological & Topographical Features	0.75		
Hazards	0.4		
Resources	0.6		
Environmental Total	0.55		
Human Aspects	0.78		

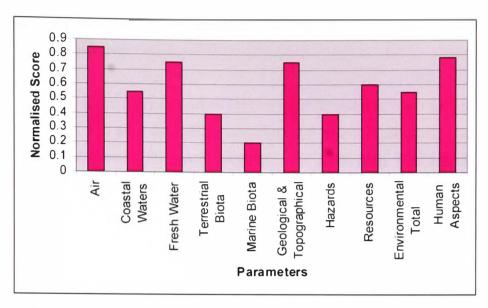


Figure 9.2: Graphical illustration of normalised scores allocated to environmental and human components.

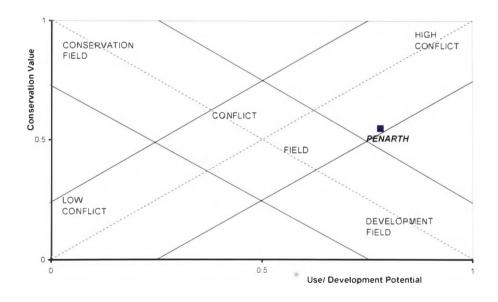


Figure 9.3: Conservation/Use/Development Matrix for Penarth.

development. In this instance it did not matter, as for Penarth it was considered to be the mid point 3 (Table 9.2). As the total length of the foreshore was evaluated as a single unit, there was a freshwater component due to discharges from the Rivers Taff and Ely whilst if the study had been limited to fronting the Esplanade, this would not have been considered.

The function analysis work of De Groot (1992) and adaptation by Cendrero and Fisher (1997), had been designed to accommodate regional differences and enable a more rapid assessment for beach managers. In discussions with van der Meulen (2004, pers comm.) conclusions were that the changes made in the technique for this thesis were deemed to be valid. In general, van der Meulen (2004, pers comm.) felt that as long as the study was consistent, issues such as employment and minerals could justifiably be argued from both perspectives and that a holistic approach should be applied to the coastline. Furthermore, it was his opinion that there was an inbuilt flexibility in the methodology, which recognised that circumstances change, and that environmental considerations were not fixed. This was essential in assessing current status and being responsive to change. His original co-authored work (van der Weide, et al 1999) was itself a pilot study for the assessment of conservation and development conflict and he welcomed studies of this nature as part of the evolutionary process (van der Meulen, 2004; pers comm.).

The total normalised score for environmental indicators were subsequently plotted against the total normalised score for the socio-economic indicators (Figure 9.3). This was the Conservation-Development matrix developed by Cendrero and Fischer (1997) and analysis showed that Penarth was on the borderline of the development and conflict field. Therefore, it was concluded that management of the Penarth coast could justifiably be more focussed towards maintaining and developing socio-economic interests.

9.3 **Risk**

As a result of, and in recognition of the hazards implicit in building along the coast, coastal development should be controlled through land use management (Crowell, *et al*, 1997). The Penarth foreshore is artificially stabilised by a seawall

fronting the Esplanade, which limits natural shoreline change. *Circa* 1990, the void behind the seawall, detailed in Section 3.3.2.3, was filled with foamed concrete by the then Coastal Protection Authority (CPA), South Glamorgan County Council (SGCC). The work was funded by CBDC and was linked to infrastructure improvements to the Esplanade with the pavement and road being integrated into the seawall construction (Morgans, 2003 – pers comm.). However, a section of the vertical promenade seawall was undermined in 1997/98 when the beach fell to critical levels (SBCEG, 1999). Consequently, there is a risk of failure that needs to be managed.

The beach from 750m to 1,464m longshore (Station 26 to the barrage breakwater) is backed by constantly eroding cliffs (Section 3.2.1). The risk in this instance is to properties at the top of the cliffs and management may be necessary in the future. The rubble mound barrage breakwater, at the longshore limit of the beach (1,464m), will be integral to management proposals due to its influence on beach evolution (Section 7.4.3) and the design life of the barrage and breakwater is 125 years (Wallingford, 1990). However, to assess risk, the breakwater was checked post-construction with as-built construction data using the REBAD model (Section 5.11).

Table 9.4: Statistical properties of design variables in Hudson performance function.

Variable	Probability	Mean	Standard Deviation
(\mathbf{x}_i)	Distribution	(μ_{xi})	(σ_{xi})
Y	Normal	1,00	0,180
			(% 18)
D_{n50}	Normal	1,50 m	0,1 m
		·	(% 7)
H _s	Fisher-Tippet I	1,63 m	0,5 m
			(% 30)
Δ	Normal	1,53	0,1
_			(% 7)
Cot θ	Normal	1,50	0,1
			(% 7)

Table 9.5: Exceedance probability of 0-5(%) failure level (P_f) and the values of the design variables in the performance function at the limit state.

Method	Reliability index β	$P_{\rm f}$	$1-P_f$	Y	D_{n50}	Δ	Cot θ	Hs
		(%)	(%)		(m)			(m)
Second Moment	1,5060	6,60	93,40	0,821	1,494	1,464	1,478	2,024
Method								
(Direct method)								
Second Moment	1,4260	7,69	92,31	0,862	1,505	1,475	1.491	2,189
Method								
(İterative solution)								
Monte Carlo Simulation	-	3,07	96,39	-	-	-	-	-

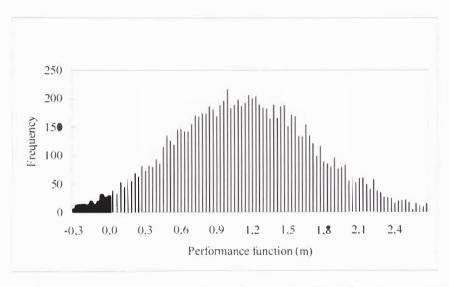


Figure 9.4: Occurrence probability of Hudson Performance function values at the limit state.

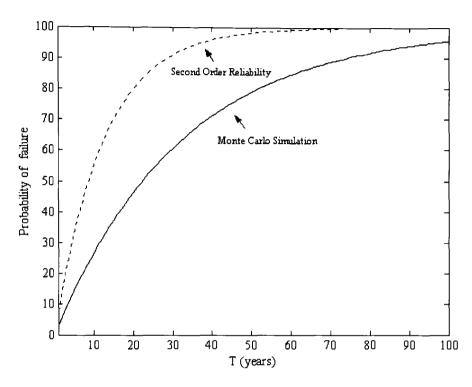


Figure 9.5: Exceedance probability of 0-5 (%) damage level in the lifetime of the rubble mound structure.

The remaining input data for the primary and secondary armour was obtained from the Cardiff Harbour Authority (CHA) and is included in Appendix 9 (Jones, 2002; pers comm.). Results of the REBAD simulation are given in Tables 9.4 and 9.5 and Figures 9.4 and 9.5. Table 9.4 detailed the statistical properties of the input design variables with respect to performance in limit state design whilst Table 9.5 evaluated the design performance of the design variables at the limit state. In these evaluations high tide was the most critical which was as expected, that is, risk assessment is in response to a hazard. The reliability was assessed in three ways: second moment direct; second moment iterative and Monte Carlo simulation (Table 9.5). The Monte Carlo simulation is more refined as parametric probabilities are used and, therefore, gives the actual distribution of exceedance probability (P_f) and reliability (P_f). Figure 9.4 shows the normal distribution at the limit state where effectively performance function was the reliability and the black shading on the graph indicated failure. The resultant graph (Figure 9.5), showed the Monte Carlo simulation output for the rubble mound

breakwater which represented the probability of a 0-5% damage level to the structure. Effectively, for T (50 years) the probability of a 0 to 5% failure/damage occurring would be 80%, whilst for T (100 years) the probability of a 0 to 5% failure/damage occurring would be 95%. Therefore, another way of considering the output is that in 100 years there would be a 95% chance of 0 to 5% damage to the breakwater. Therefore, the REBAD model results from the as-built breakwater data confirmed computer modelled projections (Wallingford, 1990) and consequently, the breakwater is considered safe and the risk of failure acceptable.

9.4 Penarth Headland Link

The Vale of Glamorgan is proposing to build a car free connection between Penarth Esplanade and the Cardiff Bay Barrage. The new Penarth Headland Link (Plate 9.1) would be a 5·5m wide high-level walkway/cycleway, approximately 700m in length. According to Soltys: Brewster (2003a), the Link would be located at a "safe distance" from the cliff to avoid dangers of rock falls and at an appropriate height above the beach (Plate 9.2) such that users would not be affected by tidal conditions. The Cardiff Bay Barrage Act 1993 permitted CBDC to construct a promenade linking the two destinations but for various reasons was never implemented by CBDC (Soltys: Brewster, 2003b).

Plate 9.1 has been removed for copyright reasons.

Plate 9.1: Proposed Penarth Headland Link (Soltys: Brewster, 2003a)

Plate 9.2 has been removed for copyright reasons.

Plate 9.2: Schematic of elevated walkway Penarth Headland Link (Taylor, 2003)

Justification for the scheme is for Penarth to fully benefit from the growing visitor attraction of Cardiff Bay and the Barrage; an estimated 50,000 people per month visiting the Barrage (Soltys: Brewster, 2003b). To do nothing would leave Penarth at an economic disadvantage to the more accessible Cardiff Bay and its arc of entertainment (Soltys: Brewster, 2003a; 2003b). According to Soltys: Brewster, (2003a; 2003c) the intertidal habitats have limited abundance and diversity of species across almost all areas and were of limited value for over-wintering wading birds. They concluded that the development would have no significant impacts on the ecology and nature conservation features of the site and immediate surrounding area. Whilst Soltys: Brewster (2003c) recognised the need to understand coastal processes to enable determination of construction design criteria, and that the coastal character was influenced by, amongst others, the protected shoreline of vertical seawall and revetment, there was no reference to the critical erosion along the foreshore in 1997/98. This was very surprising, especially as the EIA had been produced in consultation with the Shoreline Management Partnership. Should this development be implemented, it would have big implications for beach management.

9.5 Management Strategies

Penarth beach is not a resort beach, rural beach or bathing beach, although water quality is monitored in accordance with EC Bathing Water Standards (Soltys: Brewster, 2003a). It is popular for water sports, sailing, fishing and promenade

strolling. Indeed, the Esplanade was originally constructed so that walkers would not damage their clothes and shoes, walking on a stony beach (Carradine, 1994). Without an element of sustainability and relevant indicators within an integrated coastal zone management strategy, deterioration of the coastal environment is likely to persist (Charlier and Bologa, 2003).

In Section 9.2, function analysis was used as a means of sustainably assessing the foreshore conservation and development potential and it was concluded that management should be focussed towards maintaining the human environment. The assessment of risk (Section 9.3) highlighted potential undermining of the seawall fronting the Esplanade, whilst REBAD verified the stability of the barrage breakwater. The proposal for an elevated walkway that will link the Esplanade to the barrage (Section 9.4) was justified on socio-economic and competitive grounds so that Penarth would not lose out on the increased tourism generated by the Cardiff Bay development. Therefore, as the local environment is not deemed significant for nature conservation (Section 9.4) then management of the Penarth physical environment from an anthropogenic perspective, is further justified.

In Section 4.4, ICM was considered in response to coastal change. Granja and Carvalho (2000) suggested managed retreat and selective conservation of parts of the coast that are important to society. Where possible technological developments should be used and measures taken to attenuate or reduce the effects of erosion. Bijlsma, *et al* (1996) working on coastal adaptation options had already identified similar responses, that is, protect, accommodate or retreat and these are integrated within the management policies for coastal protection and flood defence for the Severn Estuary (SES, 2001). To manage the coast implies an understanding of present day factors and processes (Granja, 2001) whilst Reeve and Spivack (2001) argued it was implicit upon the understanding of the physical processes responsible for shaping coastal morphology. Evaluation of management strategies is regarded as a crucial component of vulnerability assessment but there have been concerns that adaptation has emphasized a protection oriented response rather than consideration of the full range of adaptation

options (Climate Change, 2001). In answer to these concerns, Klein *et al* (2000) developed a four-step methodology for the process of coastal adaptation:

- Information collecting and awareness raising;
- Planning and design;
- Implementation;
- Monitoring and evaluation.

Again these proposals are in line with the corporate strategy advocated by Johnson and Scholes (1988). As managed retreat and advancing the line are not feasible solutions for the Penarth coast, holding the line is the strategic option and consequently, implementation will require some form of protection. However, as shown in Chapters 6 and 7, each part of the coastline had different physical characteristics, which required that beach management be considered in three distinct sections:

- Penarth beach;
- Penarth Pier to groyned beach;
- Groyned beach.

9.5.1 Penarth Beach

Sections 6.2, 6.3 and 6.4 evaluated shoreline indicators (MHW and mean beach level) fronting the Esplanade, whilst Section 8.4 considered beach evolution in response to forcing agents (sea levels and wind). Significant changes in wind direction between 1995 and 1997 and the consequent effects on wave climate caused erosion of the foreshore, whilst in Section 9.3 it was demonstrated that there was a risk of the seawall being undermined. In response to the strategic option of holding the line, it is implicit that there is a need to protect the seawall. Plate 9.3, taken in December 2004 illustrates the pebble beach fronting the seawall and line of depth of closure. It could be argued that in comparison with September 1992 (Plate 1.2) beach levels are still relatively low. However, it must be remembered that in September 1999,

following easterly storms, a loss of 1,148·4 m³ (2,526 tonnes) of beach material was recorded from the foreshore grid area (Section 6.3.5). Therefore, although unquantified, the loss of beach due to the significant increases in storm activity from the northeast quadrant between 1995 and 1997 (Table 8.16), must have been considerable. It can take years for a beach to recover from one storm event so it is possible that it will be many years before the beach reaches the state of nourishment demonstrated in September 1992.

In the short-term it is recommended that no protection measures be introduced but monitoring should be continued on a seasonal basis. In Section 6.6 it was argued that any of the models developed, could be used to indicate change ahead of a considered and planned management response but, that the quickest model to implement would be:

MRI =
$$17.035 + \tan^{-1} \frac{(x_{14} - x_6)o}{240}$$
 (Section 6.4.4)

The formula is the end result of a huge amount of quantitative fieldwork (Chapters 6 and 8). It was shown that this model accurately reflected changing processes along the beach and is a simple formula for a manager to assess beach movement. The collection of quantitative data is often expensive and time consuming but there is a need for long-term quantitative data related to beach dynamics (Galofre and Montoya, 1995). Effectively this formula reflects quantitative assessment but will not be time consuming or expensive to implement. With reference to Plate 9.3, for a guide, this would involve measuring the cross-shore distances, x, from station 14 (approximately 1m south of the steps) and station 6 (adjacent to the old lifeboat slipway) to the seaward boundary of the pebble beach.



Plate 9.3: Esplanade foreshore - December 2004.

Interpretation of results, as discussed in Section 6.4.4, showed that an inter-survey reduction in MRI would indicate losses of beach material by a southerly longshore transport mechanism and vice versa.

It must be remembered that the erosion events of 1995 to 1997 inclusive had not been previously experienced along the Penarth foreshore and also, not since. However, with predictions of increased storminess in the Severn Estuary (SES, 2001) it is possible that, at some future time, the seawall will need protecting. This could be achieved with a hard engineering solution such as increasing the size of the seawall but this could cause further problems (Section 2.6). Therefore, in line with current thinking, beach nourishment could be the preferred soft engineering solution (Karasu *et al*, 2004).

A successful example of sustainable beach nourishment involved the construction in 1995 of a coarse clastic beach to prevent erosion and protect a main highway, on the Victoria waterfront, British Columbia, Canada. The

beach was modelled, designed and constructed using groynes and imported stone, the size of which had been calculated for the wave climate and prevailing coastal processes. The scheme was an unqualified success resulting in the nearby highway, which previously had suffered frequent annual flooding, no longer being flooded subsequent to the beach construction. Furthermore, in the subsequent four years, there had been no appreciable loss of material (Gillie, 1999). This approach has found further support in Italy in recent years, with the construction of several gravel beaches (Cipriani et al., 2004). It was found that they provided a cost effective management strategy to protect coastal settlements, prevented shoreline retreat and provided a surface for recreation. Therefore, with proper planning and design, it has been shown that beach nourishment, as a management response, would be an appropriate solution for Penarth beach. It is also suggested that a feasible source of beach material would be the pebble deposits adjacent to Aberthaw Power Station, approximately 15 km west.

Another beach nourishment alternative would be to construct a sandy beach fronting the Esplanade. Before 1880, although the beach itself was a pebble one, there was a flat sandy strip below the high-water mark (Carradice, 1994). A feasibility study would need to be undertaken similar to that for St. George's Bay, Malta. Micallef et al (2001) undertook environmental risk assessments on a proposed beach nourishment project for tourism at St. George's Bay Malta. Two different sized sediments, one fine grained and one coarse grained were modelled utilising potential volumes whilst considering positive and negative performance during both construction and postconstruction phases. Findings showed a high probability (0.95) of a mild impact during the post-construction phase with fine-grained sediment as against a high probability (0.86) of a negligible impact with coarse-grained sediment. Several decades ago that bay contained sufficient sand to make it suitable for sun bathing and swimming but over time the sand was lost (Ebejer, 2004). The nourishment project was completed by July 2004 and at peak times 800 Maltese and tourists used the replenished beach. It was concluded that all commercial establishments, as well as property owners in the area, benefited from the public sector project (Ebejer, 2004). Another successful example of nourishment is Miami Beach which had virtually no beach by the mid 1970's, and as a result, its facilities were extremely run down. Following beach nourishment in the late 1970's Miami Beach was rejuvenated to such an extent that the annual revenue from foreign tourists alone is \$2.4 billion, about fifty times the \$52 million cost of the twenty-year project (Houston, 2002).

Conversely, there are examples where beach nourishment has not been successful (Ciprianni *et al*, 1999). At Marina di Massa, Italy, the use of an incorrect mean grain size as nourishment, resulted in approximately 66% of borrow material disappearing within one year. This occurrence further validated the results of the assessments for St. George's Bay undertaken by Micallef *et al* (2001). Performance of a nourishment project is generally determined by percentage of remaining material, that is, time dependent, the remaining volume to the volume used. This ratio depends on various wave, coast and material parameters (Karasu *et al*, 2004). Whilst full analysis is outside the current study, the benefits of undertaking it at Penarth are clear. In essence two beach nourishment solutions, pebbles and sand, have been proposed.

Table 6.25 compared inter-survey gains and losses of beach material from the foreshore grid and beach to the seaward limit of depth of closure. These were subsequently converted to annual volumes (Table 9.6).

Table 9.6: Annual beach gain/losses Penarth foreshore

Date	Volume Gain/Loss (+/-)m³			
	Foreshore Grid	Seaward Limit (DoC)		
Sep 1909	- 562	- 2,425		
Sep 2000	648	634		
Sep 2001	- 209	252		
Sep 2002	259	487		

Considering worse case scenario of a five-year erosion at 2,425 m³/year, then quantities required for beach replenishment would be 12,125 m³, approximately 50 m³/m run of beach (stations 6 to 14; Figure 1.2). From back beach to DoC, approximately 60m, would mean an average increase in beach height of 0.833m. This would be equivalent to an increase back beach of approximately 1.6m.

Quantitative comparisons of Plates 1.2 and 1.3 indicate that a 1.6m height increase back beach on the September 1997 position would reflect beach conditions in September 1992. Therefore, the estimated volume is feasibly correct. However, as the total length of the foreshore, old lifeboat slipway to Penarth Pier is approximately 270m, then total nourishment volume requirement is $50 \times 270 = 13,500 \text{ m}^3$. Consequently, estimated cost of beach nourishment for the two options are as follows:

Option 1 – Pebbles

According to Williams *et al* (2002), 0·200 to 0·250m clean stone at Newton beach, Porthcawl cost £12 a tonne, whilst from engineering experience, labour and plant costs are generally 30% of material costs. Therefore, including inflation, estimated placement costs are £18 a tonne. Estimated overall beach cost, using 2,200 kg/m³ as unit weight (Coduto, 1999; Bell, 1993), is therefore $13,500 \times 2.2 \times £18 = £534,600$, say £550,000.

Option 2 – Sand

Quoted cost of marine sand was £16.90 a tonne plus VAT (South Wales Sand). Therefore, including labour and plant the estimated placement rate is £22 a tonne (VAT ignored as a bulk order discount). The estimated sand nourishment costs are $13,500 \times 2.2 \times £22 = £653,400$. In the case of pebble nourishment, the cross-shore beach edge could be left at the angle of repose. However, for sand, there is a need for edge groynes (stations 6 and 15; Figure 1.2) to retain material and preferably a third at mid beach location (station 10). Costs per groyne are likely to be £1,000 per metre run, resulting in a total groyne cost of £150,000. Therefore, the estimated cost of a sand beach for the Penarth foreshore is £653,400 + 150,000 = £803,400 say £800,000.

Nourishment at St. George's Bay, Malta was for a 200m wide beach and involved 5,000 to 6,500 m³ of sediment at a cost of Lm 200,000, approximately £350,000 (Ebejer, 2004: Micallef *et al*, 2001). Consequently, proposed costs derived for Penarth are in line with actual costs for St. George's Bay. Furthermore, at Southerndown, approximately 30 km west of Penarth, beach users utilise the pebble beach for sitting, as well as the sand/rock platform areas during the tidal cycle. Therefore proposals for Penarth follow this precedent.

In Section 9.4, it was shown that the proposed Penarth Headland Link was being justified on socio-economic grounds to take advantage of tourists visiting Cardiff Bay. Should the construction of a sandy beach of the correct grain size be feasible there may be locally available sand from marine sources that, as argued by Symes and Byrd (2003), provides a natural symmetry and harmony by combating coastal erosion with ballast won from the sea. Consequently, the Esplanade could become a desirable destination, not just the journey, with the two projects becoming interrelated to generate increased tourism revenue.

9.5.2 Penarth Pier to Groyned Beach

The distinct character here is the relatively uniform nature of the constantly eroding cliff face (Bullen 1993a), some 30m in height. "It is the most natural section of this coastline, the rock face having a strong grain and presence and is fronted by a rocky foreshore with long tidal range and a green clifftop of mature trees" (Taylor, 2003). If the Penarth Headland Link goes ahead, then there would be modifications to the coastal environment, whichever of the five options was chosen. Soltys: Brewster (2003a) argued that with appropriate mitigation measures in place, local effects of the built scheme on coastal processes on the shore, are likely to be minor. However, monitoring would need to be undertaken and change could be identified by comparison of longshore gradient relationships from the link survey (Section 6.5.3; Figure 6.45). The significant model (Figure 6.54):

$$y = -7.9099x + 2.894$$

represented the temporal variation in longshore gradients on comparison of the two beach headings (y NNE(%): x NNW(%)). These gradients, from the section least influenced by anthropogenic activities, reflected natural changes in direction of longshore drift (Section 6.2) and significant changes in wind direction (Section 8.4). Therefore, non-compliance would indicate changes due to the Penarth Headland Link and comparison with the survey results between 1997 and 2002, would indicate effects. Should the consequences result in undermining of the cliff, management response could be toe protection via an appropriate length of 2m high wave wall, as had been previously used, south of the lifeboat slipway (Williams and Davies, 1987; 1990). If the Headland Link does not go ahead, the same monitoring and response procedures are recommended.

9.5.3 Groyned Beach

Analysis of the groyned beach showed significant temporal and spatial relationships with respect to mean beach level (Section 7.4.2 and 7.4.3 respectively). Changes were shown in beach morphology (Section 7.4.4) and all analyses demonstrated that the barrage construction had modified near shore coastal processes. In Section 7.5 it was argued that temporal and spatial models could be used to predict and assess changes in mean beach levels. Specifically, the temporal model for bay 3 (Figures 7.2 and 7.6):

$$y = 0.0013x + 3.9394$$

would be the quickest indicator of changes in mean beach levels with time (y mean beach level (m); x time (months)). It can be used in conjunction with the other temporal and spatial models to produce further refinement and increase accuracy (Section 7.5). The models will also quantify the increasing updrift influence of the breakwater on beach evolution. Plate 9.4 from December 2004, shows the beach conditions in bay 3 (Figure 7.2). Comparison with Plate 7.21 from September 2002, indicated that beach levels, as predicted in Section 7.5, had increased. The steps from the breakwater to the foreshore, shown in Plate 9.5, had failed, a function of the hostile environment highlighted in Section 7.3.8 and Plate 7.17. Therefore, as increased beach levels provide cliff-toe protection, it is suggested that on-going monitoring, utilising the temporal and spatial models, should continue.

The construction of groynes 6, 7 and 8 were a beach management response to retain beach material along this part of the shoreline, but following the construction of the breakwater it is argued that they are now redundant. Consequently, as management strategies for this section of coastline should beach levels continue, as expected, to increase, it is suggested that these groynes should be removed. This would have the further benefit of removing



Plate 9.4: Groyned beach (Bay 3) - December 2004.



Plate 9.5: Failure of breakwater steps to foreshore, - December 2004.

groyne effects (Table 7.24), and thereby allowing equalisation of beach levels with the south to north longshore drift and breakwater influence. In time, the

effect of the breakwater will reach the manhole at Penarth Head (1,245m longshore) and result in a uniform longshore gradient along the NNW beach heading.

An estimated 50,000 people per month visit Cardiff Bay and the Barrage (Soltys: Brewster, 2003b), irrespective of whether the Penarth Headland Link is constructed (Section 9.4). Therefore the groyned beach could be developed to take advantage of this tourism market. Advertising the existence of the adjacent beach in the barrage car park, could offer barrage visitors the opportunity of a nearby picnic area, possibly with refreshment facilities on the approach to the breakwater steps (Plate 9.5). Cipriani et al (2004) showed that following seawall removal, several gravel beaches had been These proved cost-effective in restoring the created in Italy in recent years. waterfront and providing a surface for recreation, which had re-united people with the waterfront. Groyne removal therefore, as well as contributing to increased beach levels along the foreshore could provide a similar recreation surface. However, the constantly eroding cliffs (Bullen 1993a; Williams and Davies 1987: 1990) and potential rock falls of harder gypsum layers (> 50 Kg; Wallingford 1992) presents a health and safety problem. Whilst the costs of ameliorative measures are outside the scope of this thesis, a possible solution would be to apply geo-textile/steel netting to the cliff-face (similar to motorway cuttings) and provide a 5m exclusion zone from the cliff-face, protected by appropriate fencing. This would be important, as according to Nelson et al (2000), in a beach-user study, intrinsic beach safety was the second highest priority.

9.6 Future Work

It is suggested that foreshore monitoring be continued to maintain the relevance of this work and refine the beach models as appropriate. Whilst evaluations of sea level and wind data yielded significant results, correlation of maximum gust winds with state of tide was not possible. Therefore, it is proposed that the times of maximum gusts should be obtained from the Met Office which would allow refinement of these wind models. The likely outcome of subsequent analysis would be an even more significant correlation between storms and resultant beach levels.

Work will be required evaluating the study in conjunction with management recommendations in the new Shoreline Management Plan and Implementation of ICZM in Europe (COM/2000/545). Furthermore, implications of the EU Water Framework Directive (DIRECTIVE 2000/60/EC) and its holistic catchment approach, will need consideration.

Policies focussed towards sustainable coastal management are advocated in the Strategy for the Severn Estuary (SES, 2001). Whilst the application of function analysis assessed the Penarth coast from a conservation development perspective, goods and services were considered from their functions. The resultant outcome is considered sustainable from the overall holistic approach, that is, anthropocentric development at Penarth would be balanced by conservation management elsewhere. According to Jorge, et al (2002), needs are created and subsequently attained by the formation of amenities. This results in an alteration of socio-economic and environmental parameters and interactions are produced. However, historic development along the foreshore and recent developments at Cardiff Bay prevent a truly sustainable management evaluation; for example, managed retreat in response to erosion at Penarth is not an option. Therefore, future work should evaluate the sustainability of the management strategies advocated.

Finally, and importantly, the relevance of the models developed in this study, should be tested at other locations, to assess their relevance to the wider coastal community.

9.7 Summary

The first requirement of strategic management was met by an understanding of coastal processes generated by the five-year monitoring of the Penarth coast. Function analysis justified managing the foreshore from a socio-economic perspective whilst risk analysis highlighted the potential failure of the Esplanade seawall and stability of the barrage and breakwater. These evaluations were critical to the formulation of beach management strategies for Penarth. Implications of the Penarth Headland Link to foreshore management were evaluated and research showed that the EIA for this development did not consider the erosion of the foreshore in 1997/98.

Management strategies and responses were considered and justified for three distinct sections of the 1,464m length of monitored foreshore (Figures 6.23, 6.45 and 7.2):

- Penarth Beach On-going monitoring was recommended to identify trends of erosion or beach recovery. Beach models developed in Sections 6.3 and 6.4 were proposed for use in the monitoring programme, specifically WCBDC = 17·035 + tan⁻¹ (x₁₄ x₆)/240°, due to its rapid evaluation and significance of result. In the short-term, the *do nothing* option was recommended, whilst in the event of future erosion of the foreshore, beach nourishment was the management solution. Two nourishment options were considered, pebbles and sand. The former (estimated cost £550,000) was the more likely option although the feasibility of a sand beach (estimated cost £800,000) should be investigated, due to the potential tourism appeal and synergy with the proposed development of the Penarth Headland Link.
- Penarth Pier to Groyned Beach Again monitoring was recommended using the beach model developed in Section 6.5.3, y = -7.9099x + 2.894. This would identify changes in the two beach headings (NNW and NNE) caused by either changes in coastal processes or the construction of the Penarth Headland Link. Management solutions in both cases should cliff toe erosion occur, was toe

- protection via an appropriate length of 2m high wave wall as had been used south of the lifeboat slipway.
- Groyned Beach As for the rest of the foreshore, on-going monitoring was recommended to evaluate the continued influence of the barrage and breakwater construction on mean beach levels. Bay 3 was specifically identified and the model y = 0.0013x + 3.9394 (Section 7.5) would enable rapid assessment of change. Whilst continued increases in beach level were forecast, it was recommended that groynes 6, 7 and 8 should be removed to allow equalisation of beach levels, in line with changed nearshore processes. This could provide a picnic area for tourists provided safety measures were introduced.

Suggested future work included correlating maximum wind values with state of tide and evaluating the study in conjunction with the new Shoreline Management Plans and European legislation and initiatives. Whilst decisions were justified from a socioeconomic perspective, it was recommended that the sustainability of the management strategies and options be evaluated and the work tested for wider relevance.

Beach management for the Penarth coast was considered as a function of socio-economic pressures and physical environmental constraints. Furthermore, according to Nelson and Williams (2004), for successful beach management, it is imperative that one takes cognisance of preferences and priorities of the beach user. It is argued that due to the usage of the beach (Section 9.5) the preferences and priorities of the beach user have been accommodated in the management strategy. Different courses of action were considered and consequently the second requirement of strategic management has been demonstrated. The third implementation, although justified, is the prerogative of the Coastal Protection Authority (Vale of Glamorgan County Council).

CHAPTER 10: CONCLUSIONS

In 1997 the effects of severe erosion along the Penarth foreshore became apparent when the beach fell to critical levels. Published reports showed change had occurred since 1992 and anecdotal evidence identified the onset from 1995. This coincided with construction of two key stages for the Cardiff Bay Barrage and consequently, as well as offshore marine aggregate dredging, this structure became linked as the cause. From surveys of 1464m of coastline comprising two separate orientations (NNE and NNW), carried out each September and April between 1997 and 2002 to assess summer and winter changes, spatial and temporal trends were established. Evaluation of these trends produced important regression models which described the variation of shoreline indicators, MHW, mean beach level and depth of closure, as well as gain/loss of beach material. Analysis of a ten-year (1993 to 2002) record of sea level and wind data produced significant relationships which were applied to developed beach models. Results in conjunction with hindcasting showed the cause of the unprecedented erosion of the beach. Conclusions from analyses and models were subsequently used to justify monitoring and management strategies for the Penarth coast.

Evaluation of photographic evidence, OS maps and survey data clearly showed a reversal of accepted northerly longshore drift. Temporal analysis of foreshore beach levels indicated variability in coastal processes between September 1997 and April 2000 whilst between April 2000 and September 2002 significant models for the 750m longshore profile represented increased beach levels with time. Furthermore it was shown that sediment transport mechanisms had returned to the accepted south to north movement.

Analysis of beach levels fronting the Esplanade highlighted consequences of an easterly storm in May 1999, with monitored inter-survey losses of 1148·4m³ (2526 tonnes), equivalent to 350 kg/m² of beach surface. A return to a northerly sediment movement was further supported by significant changes in longshore gradients (95% confidence limits), between September 1997 and 2002, irrespective of cross-shore location, further verified by reductions in cross-shore gradients at station 6 (150m) and increases at station 14 (390m). Analysis of changed beach morphology showed that erosion induced a southerly movement of

beach contours and vice versa. MHW as a shoreline indicator and mean beach level to represent shoreline position was justified. There was a strong positive correlation between change in WCB MHW and gain/loss of beach material with the two variables having 52·54% of their variation in common. Furthermore a highly significant quantitative relationship existed between the average WCB MHW and average beach level, both variables having 61·61% of their variation in common. These relationships verified that a reduction in WCB MHW was directly correlated to erosion.

The cross-shore position of the depth of closure was 50m seaward of the longshore profile (approximately 60m from the seawall) at a water depth in the range 5·443m to 5·591m. Results agreed with Hallermeier (1981) and observations on active profile width by Leatherman (2001) and Stive *et al* (1992), who argued that the longer the time period considered, the larger the depth of closure. Cross-shore profile results showed greatest temporal variation back beach with little variation beyond the depth of closure and these variations further supported conclusions from analysis of the longshore profile and foreshore grid. Significantly for Penarth, it was shown that erosion as part of a northerly sediment transport did not have as severe an effect on beach morphology as erosion in a southerly direction.

Significant temporal models represented the variation of shoreline indicators MHW and DoC and there was an extremely strong positive correlation between them, that is, both variables had 83·85% of their variation in common. Synthesis of foreshore results led to the development of a simple management tool to rapidly assess the health of Penarth beach. The use of the beach specific version of the model, MRI = $17\cdot035 + \tan^{-1}(x_{14} - x_6)/240^\circ$, enables assessment of: consequences of storms, longshore transport, gain/loss of beach material, potential problems and varying timescale changes. Although evaluations of sediment budgets were over too short a timescale for meaningful interpretation, they highlighted the consequences of storms and further validated the quantitative assessments made for shoreline indicators WCB MHW, WCB DoC and mean beach level. The link survey was important as demonstrated that irrespective of direction of longshore transport and external factors such as storms and anthropogenic activities, there was a significant correlation in the variation of the

longshore gradients (97·37%). Therefore, irrespective of orientation (NNE or NNW), beach response to coastal processes were inter-related.

Overall mean levels in the groyned beach increased throughout the five-year monitoring period. Although net levels initially fell in bays 1 and 2, following Barrage completion, they increased, whilst bay 3 consistently increased in levels throughout the five-year monitoring period. Importantly, material gain in bay 3 by September 2002 was 1,024·2m³ (2,253·3 tonnes: 1,191 kg/m²) and this significant change (99% confidence limits) resulted in increased cliff toe protection. Strong temporal correlation was shown with respect to mean beach level in all three bays and valid temporal models were developed. Spatial analysis identified two highly significant longshore trends with respect to distance from the barrage breakwater. Both models reflected strong relationships between mean beach level and distance, and showed that the breakwater influence decreased with distance.

The first modelled beach response to longshore transport (95·93%) whilst the second reflected beach evolution under the combined interaction of breakwater and sediment transport (91·73%). Residual analysis showed that the use of regression analysis was justified and according to Crowel *et al* (1997), this type of model provided the best forecast of shoreline trend. Impacts of spatial and temporal trends resulted in a progressive change in beach morphology in the three bays. In time, as the influence of the breakwater extends southwards, it was predicted that beach contours will become more uniform. This was supported by measured beach response to the easterly storm in May 1999 where erosion of 569 kg/m² was documented in bay 1 compared with an accumulation of 30 kg/m² (bay 2) and 88 kg/m² (bay 3). Furthermore, similarity of losses in bay 1 compared with the foreshore grid, provided support to link survey conclusions that beach response did not change with foreshore orientation.

Analysis of a ten-year (1993 to 2002) record of sea level and wind data (forcing agents) provided the cause of the unprecedented erosion, between 1995 and 1997, along the Penarth coast. There was a projected mean sea level rise of 0.4 mm/year, much lower than other forecasts but conversely there was an overall trend of falling extreme water levels.

However, between 1995 and 1998, extreme sea levels were significantly higher (95% confidence limits) than in other years. Corresponding evaluations of mean wind, maximum gust and storm gust data, showed that between 1995 and 1997, there were significant increases in easterly components of all winds. Furthermore, in those years there was a significant increase (99% confidence level) in the incidence of easterly storm winds, similar to the May 1999 occurrence.

Subsequent evaluation of wind and water level results, in conjunction with relationships generated from field data collection, showed that the shoreline indicators were inversely correlated with extreme sea levels and positively correlated to wind direction. Further significant quantitative models represented the response of shoreline indicators to forcing agents. Mean wind direction was found to be more important than maximum gust direction to normal beach formation processes and it explained approximately 60% of the variation of longshore gradients. Resultant changes in wind direction meant that between 1995 and 1997 more winds approached Penarth over water which provided the necessary conditions for erosion of the foreshore (Neves, 2004: Wallingford 1992). As anthropogenic activities such as barrage construction and offshore marine aggregate dredging, previously linked with the erosion of Penarth beach had been discounted, it was concluded that increased wave attack from the northeast and southeast quadrants, generated by changes in wind speed and significant changes in wind direction and extreme sea levels, had caused the unprecedented erosion.

The underlying principle that coastal management comprises the integration of a cultural superstructure, superimposed on a physical fundament, was adopted in the development of management strategies for the foreshore. Application of Function Analysis justified management of the foreshore from a socio-economic development/use perspective and risk analysis highlighted potential failure of the Esplanade seawall. However, using asbuilt data, application of the REBAD model verified that the barrage and breakwater were stable. Initial recommendations of on-going monitoring to assess beach health were supported by developed models and underpinned by management responses detailed for each of three defined sections. These included two options of beach nourishment, pebbles

(estimated cost £550,000) and sand (estimated cost £800,000) fronting the Esplanade, cliff toe protection to manage potential changes caused by the proposed Penarth Headland Link and groyne removal should beach levels progressively increase south of the barrage breakwater. Future work should include testing the MRI equation at other locations for relevance to the wider coastal community.

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Appendix 1: Data sheets – longshore profile surveys

(Two examples from possible eleven)

SURVEY BASELINE -PENANTH BEACH (STATIONS 1-26 INC)

BS	IS	FS	HC	RL	Remarks
1.125	. 0		10.135	9.010	(DEM TOLLET VALL (CIPCENADE)
0.425		3.780	6.780	6.355	CP
2.765		3.720	5.825	3.060	STN 10 (CP)
	2.375			3.450	C 472
	2.165			3660	STN 8
13.075		2.320	6.580	3.505	STN 7 (CP)
	3.015			3565	57 26
1.050		2.935	4.695	3.645	STN 5 (CP)
	1.370			3.725	STN 4.
2.840		0.585	6.55o	3710	STN 3 (CP)
L	2.535			1.015	STN 2
L	2.500			4.050	1 472
3.660		0120	3.900	6.240	ICP
1.630		1.430	601.01	8.470	l ce
		1.085		2.015	oan estanade
11.120			10.130	9.010	OBM ESTIMABLE
0.510		3.765	6.875	6.365	CP
	3.725			3.120	STN 10
11.915		3.605	5.185	3.270	CP
	2425			2.760	STN 11
<u> </u>	2.50			2.625	STN PZ
	2690			2.495	STN 13
1.675		3.115	3.745	5.60	STN 14 (CP)
<u> </u>	1.845			1.500	STN 15
<u> </u>	1.850		_	1.855	STN 16
1.645		2.155	3,235	1.590	STN 17 (CF)
	068.1			1.345	STN 18
	2.0 25			1.210	STN 19
	1.780		2 0 5 5	1.455	577 20
2.170		2.150	3.552	1.085	STN 21 (CP)
	2.030			1,225	STN 22
	2.220			1.035	57 N 33
<u> </u>	2.005			1.250	5-4 54
	0.885			2.370	STN 25
-	0.865			2.390	7-4 SF
2.415		1.425	4.24.5	1.830	CP
3.540		0.090	7.695	4.155	CP.
2.965		0.315	10345		CP
1.820		2.000	10.165		
1.655	· ·	1.710	10.110	8.455	
		1.095		9.015	ORM ESPURADE

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SURVEY BASELINE - PONARTH BEACH (STATIONS 1-26 INC)

BS ·	IS	FS	HC	RL	Remarks
i. 200			10.210	9.010	OBM Esplande (Toilet)
6.580		3.845	6.945		CP
0.690		2-550	5.085	L:395	TBM BOTTOM STEP (RHS)
	2.040			3.045	57210
	2.290			2.795	STNII
1.835		2490	4430	2.595	STN12 (CP)
	2.035			2:395	57113
	2.325			2.105	STN14
	2.500	ļ		1930	STNIS
1.585		2.470	3.545	1.960	STN 16 (CP)
	1.99S			1.550	5717
ļ	2095			1,450	STNIE
	2.220			1.325	ST N 19
0.825		2.005	2.365	1.540	57N20 (CP)
	1.135			1.230	STN 21
	1.110			1.255	57N.22
100	1.240	1	- C: -	1.125	57 N 23
2.540	1 . 24	1.065	3.840	1.300	STN 24 (CP)
	1.380			5.00	STN 26
1.0.55	1.5 75	2 2.	- 5,5	2.265	STN 25
1.355	<u> </u>	2710	2.485	1.130	CP
2.300	<u> </u>	1.270	3.515	1.215	CP
1.3i5	<u> </u>	1.11,0	3.696	2.375	CP
2.965	<u> </u>	1.000	5.655	2.690	CP TOM DOTAL (TOO)
1.610	2 = 1	1.260	6.005	4.395	TOM BOTTOM STEP (RHS)
	2.715			3.250	9 NT2
1 7 7 7	2.585	206=	<u> </u>	3.420	8 472
1.775	i C	5.360	2.450	3-64-5	STN 7 (CP)
2 5 5 5	1.845	: 056	1 1:5		57N 5 (CP)
3.050	2 255	1.855	6.615	3.565	57 N L
2 1 0 5	3.055.	1 2 01 5	(1) 1 m m		STN 3 (CP)
3-430	2.590	2.845	7.200	3.770	STN 2 (CP)
	í]		4.530	STNI
2.790	2.670	0.055	9.935	7.145	CP
		 	10:135	8.465	CP
1.730		1470	10/103	9.005	OBM Esplande (Tailet)
	•	1190		\ <u>\`\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\\</u>	- 21 1 Marie (19:181)
			1		
					:
	<u> </u>	<u></u>	l		

Appendix 2: Data sheets – foreshore grid and cross-shore profile Surveys

(Four examples from possible twenty-two)

SURVEY BEACH PROFILES + GRID LEVELLING -PONANTIA BEACH

BS	IS	FS	HC	RL	Remarks
1175	. 6		10-185	9.010	OBM TOILETUGUL (EFLAME
261.0		3.715	6.665	6.470	ce
	2.14.0			4.525	TRM 30TTOM STEP
3.660		0.195	10.130	6470	CP
	<u> </u>	1.115		9.015	OBM Tower usin
	<u>(</u>	ų.			
0.182	· · · · · · · · · · · · · · · · · · ·		おりし	4525	GOTT CON TEE MET
<u> </u>	0.160			4-550	STN 10 (-10 m)
	1.535	ļ		3.175	57Njo (On)
1	1.5.810	·		1.900	STN 10 (+10m)
0.036	1	3.925	6.812	0.785	STN 10 (+20m) (CP)
	0.562		ļ	021.0-	STN10 (+30m)
<u></u>	1:575		_	-0.760	STNIO (+con)
<u> </u>	2.010		1	261.12	5-N10 (-50m)
	2-620			-1.805	57N10 (+60m)
	3.015	1	 	-5.500	STN10 (+70m)
1	3.275	1	<u> </u>	-2.460	(m08+) 01 MTZ
	3.620	<u>[</u>	1	-2.805	(mo@+) 01 Ni2
	3820	1]	-3.005	57 N 10 (+100 m)
	4.050			-3.235	57N10 (+110n)
3470	1	0.330	3.955	0.485	CP
	1.060	1	1	2.895	STN12 (On)
	2.430			1.252	STN12 (+10n)
<u> </u>	3.385	1	1	6.570	STN 12 (+20m)
5.01-2	1	1.135	16865	2.820	CP (
 	0.815			4.020	21 W 12 (-10n)
	0770	1	1	260.7	57N/L (-10m)
	2.300			2.565	STN16 (00)
6 : 4 6	3.525	2 6 5 5	1 100 -	1.340	57~14- (+10~)
0.160	1	3 855	1.170	1.010	Ce
	0.470			0700	57414 (+20~)
 	1.265			-0.095	57 14 (+30m)
 	1.845			13.0	5-70 in (+1,0m)
<u> </u>	2.380			-1.210	57N14 (750m)
	2.895			-1.725	57N14 (+60m)
<u> </u> 	3.582	<u> </u>	<u> </u>	-2.115	STN 14 (+70m)
1 3 = -	3.600	3 71 5	1.265	-2.430	57N 17 +80m)
1.305	112.0	3.765	-1.250	- 2.595	CP (202)
	1.640			-2.730	£ \
	1.573			-2.860	5-N14 (+100m)
	1.810	<u> </u>		-3.100	STN 14 (+110m)
	1.995			-3.285	STN 14 (+1200)
	2.170			-3.4.60	STN 14 (+130m)

GRID LEVELLING - CONTINUED

BS	IS	FS	HC	RL	Remarks
	2.365		-1.290	-3.655	STN 14 (+1400)
	2470			-3.760	(m021+) 41 WT2
	2.590			-3.880	57N 14 (+160m)
	2.810			-4:100	STN 14 (+170m)
3.645		26L.0		-2.085	CP
3.900		0.175		1.385	(P
		0.750		4.535	TOM BOTTOM STOP
6750			5.285	4.535	TOM BOTTOM (TEF
	0.385			4.500	STN & (~10m)
	1.685				STN 8 (0m)
	5.532			2.350	STN & (+10m)
1.085		3.500	2.870	1.785	
	1725			1.145	500 8 (+20-)
	1.350			1.520	S-NG (+20m)
	2.300			0.570	5-N ((+30m)
	3.080	1		-0.210	STN 6 (+40m)
	3. 670]		-0.600	STN 6 (+50m)
2.205		3710	1.365	1-0.820	CP
	2.575	1		1-1-210	5-N6 (+60m)
	3.200			1-1.835	51N6 (+70m)
	3.510			1-2.145	
3.865		10.435	4795	0.930	
	2.205			2.590	5726 (+10m)
	0.993			3.800	STN 6 (Om)
2400	1	0.435	6.760	4360	Ce
<u> </u>	1.870				STN 6 (-10m)
		2.220		12.540	TBN BUTTOM STEP
					<u> </u>
	T				
					
					
					

SURVEY BEACH PROFILES + ERID LEVELING-PENENTH BEACH

BS	IS	FS	HC	RL	Remarks
1125			10.135	5.010	OBM TOLETUALL (ETPENADE)
0.225		3.770	6.550	6.365	CP
	2.220			4370	TBM DETTER (TRE
3.800		0.240	10.17.0	6.350	Ce
		1.145		200.0	GBM ESPLANADE
				· · · · · · · · · · · · · · · · · · ·	
6.9 55			25.3	4:370	TBM BOTTOM STEP
	1.400			3.925	STN 12 (-10m)
1.235		2.740	3.820		STN 12 (On) (CP)
	12.475			1.345	
	3.415			0.405	STN 12 (+10m)
	1.790			2.030	S=~ 14 (0m)
	0.465			_	STN 14 (-10M)
	12.776			1.050	STN 14 (+10m)
0.075		3.520	0.375	0.300	STN 14 (+20m) (CP)
	0.680			-0.305	STN 14 (+30m)
	1.240			-0.865	STN 14 (+40~)
	1.795			-1.420	STN 14 (+50m)
	2.290			-1.915	5TN 14 (+60m)
	2.620			-2.245	57N 14 (+70m)
0.900		2.940	-1.665	-2,565	
	1.100			-2,765	57~14 (+50~)
	1.355			-3.020	STN 14 (+100m)
	1.585			-3.250	1 -
	1.715			-3.380	STN 14 (+120m)
	1.910			-3.575	STN 14 (+130m)
	2.055			-3.720	STN 14 (+140m)
	2165			-3.830	510 14 (+1500)
	2.315				STN 14 (+1600)
	2.570		<u> </u>	-4-235	STN 14 (+170~)
3.735		0.950	1.120	-2.615	CP
3.840		0.490	4.470	0.630	CP
		0.095		4375	TIBM BOTTOM STEP
			<u> </u>		
					,
				<u> </u>	

SURVEY BEACH PROFILES + GRID LEVELLING - PENATEH BOACH

BS	IS	FS	HC	RL	Remarks
0.535			2064	4.370	TOM BOTTOM STEP
	0.515			4390	STN 10 (-iem)
	1.810			3.095	STN 10 (0m)
0.365		3.135	2135	1.770	STN 10 (+10m) (CP)
1	1.425			0.710	STN 10 (+20m)
	5.352			~0.190	(molt) 01 miz
<u></u>	2.840			-0.705	57 N 10 (+40m)
0.320		3.350	-0·395	-1.215	57N10 (+50m) (CP)
	0.240			71.835	5+10 (+600)
	1.310				STN 10 (770m)
	1.6 25			-2.520	
	1.970	_		-2.865	STN 10 (+90m)
	2.130			-3.025	STN 10 (+100m)
	2.380			-3.275	STN10 (+110m)
3.820		0.060	2.935	-0.955	<p< th=""></p<>
<u> </u>	1800			1.135	STN8 (+20m)
3.295	1	0.510	5.720	2.425	STN & (+10m) (CP)
<u>[</u>	5.080				1 N S (cm)
<u> </u>	0.740			1.280	STN 8 (-10n)
<u> </u>	216.0			L805	STN 6 (-10m)
<u> </u>	5.000	1 _	_	3.650	(mo) 2 MTZ
0.225	1 .	3.552	2.720	2,495	STN 6 (+10m) (CP)
<u> </u>	1.502			1.515	STN 6 (+200)
<u>[</u>	2.250			0,470	57N 6 (+30m)
	5.345		— (–	-0.225	STN 6 (+40m)
0-170	1 -	3-4-50	-0.560	-0.730	57~ 6 (+50m) (CP)
<u>[</u>	0620	1	<u> </u>	-1.180	STN 6 (+60~)
<u> </u>	1.070		<u> </u>	-1.630	STN 6 (+70m)
<u> </u>	1.615		<u> </u>	-2.175	STN6 (+80m)
	2.040	<u> </u>	<u> </u>	-2.600	5746 (+902)
	2.340	20.6	2	-2.900	17W6 (+100m)
1.265	1 2 4 6	2.705	-2.000	-3.265	STN 6 (+1102)
	1.365	<u> </u> 	<u> </u> }	-3.365	STN 6 (+1202)
<u> </u>	1.635			-3.635	STN6 (+130m)
}	1.805	<u> </u>		-3.850	STN 6 (+1400)
	1.950	<u>[</u>	<u> </u> 	-3.950	
	2.080			-4.080	STN 6 (+160n)
7.0.6	2.250	0 5	0.770	-4250	57~ 6 (+170n)
3.245 3.865	}	0.515	0.730	-2.515	CP
3. 8L5		0.145	430	2.880	CP CP
2.620		1.550	5.500		
	<u></u>	1.130	[4.370	TOM BOTTOM STOP
		<u> </u>	<u> </u>		

Appendix 3: Data sheets – groyned beach surveys

(Three examples from possible ten)

1/11/97 OVORCAST + CHILLY

SURVEY 3 BAYL BETWEEN GROTNES 6,7,8 AND 9, ADJACTI TO BANKAGE

BS	IS .	FS	HC	RL	Remarks
0.510	<u> </u>		6.720	5.810	TBM NAIL GROTNE 8
	2.41.0			4.280	26
	-0.500			6.920	2c
	0.582			6435	2f
	3.145			3.575	2e
	6.750			5.970	2;
3.540	.]	3.560	6.700	3.160	2.h
	0.01-0			6.660	1c
	2000			12700	1b
<u> </u>	3.185	<u> </u>		3.515	le
	-0.100			6.800	14
	-0.400			7.100	113
	7.285	1	Ì	3.415.	16
	12.725]		3.975	
0.540		3.765	3.475	2.935	CP
	2.385			1.050	la
	2.470)	·	1.005	lg ld
	2.285			1.150	la
	2.305	1		1.170	2a
	2.505			6.970	2 d
3.880	<u> </u>	2.240	5.115	1.235	2a
	2685	1	1	1.220	39
	3.570			1.145	3a
	4.120			0.995	39
	1.655			3.460	<u>3P</u>
3.140	<u> </u>	6.785	7.470	4.330	CP
	3.585		<u> </u>	3.485	36
	1.225			6.245	36
	1.020			6.450	6209NF-9
	1.745			5725	Every 9
	1.370		<u> </u>	6-100	36
	1.725		ļ	6.145	35
		1.655		S.815	TOM NAIL EROTHE 8
				<u> </u>	1

^{*} Be centre Bay 3 could not be measured as there was a stockpile of spoil from the Barrage dumped in the Bay

SURVEY 3 BAYS COTHER CREAMES 6,7,8 AND S, ADJACENT TO BARRAGE

BS	IS	FS	HC	RL	Remarks
0.100			5.910	5,810	TEM NAIL GRETNES
	2,860			3.050	30(3e)
	2.200			3.710	3bb
3.760		2-1+25	7:245	3.485	13h
	228.0			6,390	3;
	0.570			6.675	3£
	0.890			6.655	3cc
0.850		3.870	4.225	3.375	CP
	3.000			1.225	3aa
\	2.610			1.615	3d
	2.950			1.275	3g AND 2a
6.810	<u> </u>	0.745	4.290	3.480	C.P.
	081.0			4.110	26
<u> </u>	1.045			3.245	2e
	3.545			0.745	2d
	2.990			1.300	29
3.585		1.095	6.780	ZC1.5	2 _q 2 _h
	0.755			6.025	2; 2f
	0.005			6775	2 ξ
1.740		3.175	5.345	3.602	CP
	1.205			4.140	l b
	3.950			2esi	la
	4.500		`	1.145	ld
	3.940			1.405	la
	1.580			3.765	1h
	0.222			4.790	1k
3.955		1.590	סוריר	3.755	le
	0.345			7.365	ij
	0.670			7.040	1 f
	0.785			6.925	10.
2.705		2.465	7.950	5.245	CP
	0.130			7.820	2c
		2.135		5.815	TBM NAIL GROYNIC 8
		<u></u>	,		ļ

^{* 3}a, 3b and 3c could not be measured in BAT 3 as the line of the breakwater and its construction have covered groyne 9 and its immediate vicinity.

SURVER 3 BATS BETWEEN GROWNES 6, 7, 8, AND D ADJACENT TO BARREE

BS	IS	FS	HC	RL	Remarks
1.250	6		7.060	5.810	TBM NAIL GROTNE 8
	3.425			3.635	36
	0.345			6.715	3;
	0.145			(.915	366
	0.400			6.660	3t
	3.1.90			3.570	38
0.610		2.900	4.770	4.160	366 (CP)
	3.600			1770	3a a
	3.075		·	1.695	34
ļ	3.260			1.510	36
3.520		3.260	5.030	1.510	2a (cp)
	0.715			4:315	26
	3.840			001.1	2a
	1.1.85			3.51-5	<u> 2ē</u>
<u> </u>	1.860	_		3.370	2h
2.675		3.250	3.755	1.080	CP
[2.785			0.570	24
3.795		2 490	5 060	1.265	la (cp)
<u> </u>	0.925			4135	1.18
<u>[</u>	1.260			3.800	i e
	1.425			3.632	l h
	0.815			4.245	1k
<u> </u>	3.815		,	1.245	19
<u> </u>	4.010		٠.	1.050	lid
3.375		0.215	8.550	1.845	CP
	1.465			6.755	i; if
	1.555			6.665	1 f
	11.135			7.085	1c
<u> </u>	2.060		1	6.160	2;
	2.085			6:135	2.f
<u> </u>	1.055			7.165	,
		2.415		2.802	TBM NAIL GROTNE 8
			<u> </u>		
	ļ	<u> </u>			
		<u></u>			

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