Dynamics of gravel and mixed, sand and gravel, beaches

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Notation

A	:	Empirical coefficient (-)
B	:	Bimodality parameter (-)
B_h	:	Berm elevation (m)
B_s	:	Set-up at the shoreline (m)
B_∞	:	Asymptotic inland over-height (m)
C	:	Proportionality constant
C_g	:	Wave celerity (m/s)
C_R	:	Frequency dependant coefficient of reflection (-)
C_{gravel}	:	Speed of propagation through gravel sediment (m/s)
C_{mixed}	:	Speed of propagation through mixed sediment (m/s)
$D_{x,}$ d_x	:	Grain size for which x% by weight is finer (mm)
D_g	:	Geometric mean diameter (mm)
f	:	Frequency (Hz)
f_B	:	Frequency bandwidth (Hz)
fp	:	Peak frequency (Hz)
g	:	Acceleration due to gravity (m/s^2)
h	:	Head of water (m)
H	:	Wave height (m)
H_0	:	Deep-water wave height (m)
H_b	:	Breaking wave height (m)
H_{m0}	:	Spectral significant wave height (m)
Hs	:	Significant wave height (m)
i	:	Hydraulic gradient (-)
k	:	Specific or intrinsic permeability (m^2) *or* wave number ()
K	:	Coefficient of permeability or Hydraulic conductivity (m/s)
K_D	:	Coefficient of dissipation (-)
K_R	:	Coefficient of reflection (-)
L_0	:	Deep-water wave length (m)
m_0	:	Zero-th order moment
Nav	:	Number of points used to derive the average (-)
n	:	Porosity (-)
nw	:	Number of waves (-)
p	:	Head of water (m)
pr	:	Relative head of water (m)
ptpos	:	Position of the pt measured from bottom of the flume (m)
s	:	Ratio of densities of soil and water (-)
s_i	:	Calibrated measurement for pt i (m)
Snn	:	Spectral density (m^2/Hz)
S_D	:	Dissipated spectral density (-)
S_I	:	Incident spectral density (-)
S_R	:	Reflected spectral density (-)
T	:	Wave period (s)
Tm	:	Mean wave period (s)
Tp	:	Peak wave period (s)
u, V	:	Velocity (m/s)
w	:	Velocity (m/s) in the vertical direction
W_S	:	Sediment fall velocity (m/s)
z_i	:	Zero for pt i (m)
z_{step}	:	Step elevation (m)
γ	:	Unit weight (kN/m^3)
β_1	:	Skewness
β_2	:	Kurtosis

η	:	Surface elevation (m)
$\overline{\eta}_m$	:	Set-up at the shoreline (m)
$\overline{\eta}_\infty$	:	Asymptotic inland over-height (m)
λ	:	Wave length (m)
μ	:	Dynamic viscosity (kg/ms)
ξ	:	Iribarren number (-)
ρ	:	Density (kg/m^3)
σ_g	:	Geometric standard deviation (mm)
ν	:	Kinematic viscosity (m^2/s)
$\tan\beta$	:	Beach slope (-)
$\tan\beta_F$	:	Beachface slope (-)
Δt	:	Data sampling interval (s)

Abbreviations

bpt	:	buried pressure transducer
GR	:	Gravel Beach
GWK	:	Grossen Wellen Kanal (Large wave flume)
HF	:	High frequency (0.05-0.5Hz)
IC	:	Imperial College
LF	:	Low frequency (0.004-0.05Hz)
MX	:	Mixed Beach
pt	:	pressure transducer
std	:	standard deviation
SHF	:	Super high frequency (0.2-0.5Hz)
SWL	:	Still Water Level
SPG	:	Slope-parallel gradient
VG	:	Vertical gradient
wg	:	wave gauge

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Chapter 2. BACKGROUND INFORMATION ON MIXED BEACHES

It is necessary to review the sediment characteristics of a range of mixed beaches in order to have an idea of their typical morphology. This review is carried out in the first part of this chapter, after a definition of mixed beach is given and discussed.

In the second part of this chapter a review is carried out on the characteristics that differentiate mixed beaches from sand and gravel beaches.

Finally, a review of the existing methodologies to study the morphodynamics of mixed beaches is carried out.

2.1 Definition and characterisation of mixed beaches

2.1.1 Definition

Mixed sediments are usually classified according to the modified Folk scheme (BGS, 1987), whereby the proportions of mud (<62.5μm), sand (62.5 to 2000μm) and gravel (2 to 64mm) are expressed as the ratio of sand to mud and the percentage of gravel. However, the present research concentrates on beaches formed of mixed sand and gravel sediments only.

Coates and Damgaard (1999) defined mixed beaches as those including sediment sizes ranging over three order of magnitude from fine sand (100μm), through gravels (2-64mm) right up to small boulders (>256mm). Sediment distributions may vary across the beach profile, along the shore and with depth below the beach face, as well as with time. This type of beach is frequent around the world, including the Mediterranean, New Zealand and South Atlantic coasts. Mixed beaches are also found when renourishment schemes have used materials that are significantly different to the indigenous one in terms of size and distribution. These nourishment schemes are often used in preference to hard defences, particularly in areas that are valued for coastal recreation. Also, the use of shingle has been proposed as an engineering solution to mitigate erosion (Muir-Wood, 1970)

Throughout this thesis, different terms will be used and are specified below:

- Sand beaches: those composed of sediment ranging from 62.5 to 2000μm
- Shingle or gravel beaches: those composed solely of gravel sediment (2-64mm)
- Coarse grained beaches: this term includes both gravel beaches and mixed (sand and gravel) beaches. Other authors have used the term coarse clastic beaches to refer to coarse-grained beaches.

2.1.2 Characterisation

Although mixed beaches are present in many parts of the world, both naturally and artificially, characterisation of their morphology and processes is scarce. A review of relevant work done around the world on the characterisation of mixed beaches follows.

Greece

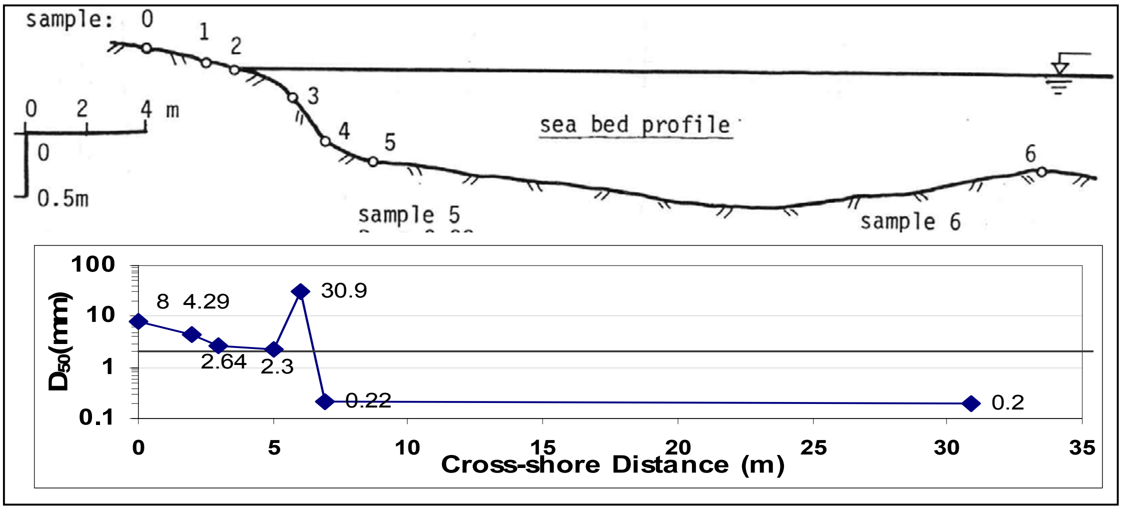


Figure 2-1. Cross-shore changes in sediment grain size in Marathonas beach (modified from Moutzouris, 1991).
(Numbers in graph show median grain size).

Moutzouris (1991) gives an overview of the typical sand-gravel mixed beaches found in Greece, based on an extensive surveying exercise, concluding that the non-uniformity of grain-size distributions across-shore is an important factor that should not be ignored in models of nearshore processes. The tideless mixed beaches in Greece show significant differences in the cross-shore distribution of sediment size. A regular sediment sorting due to the differing hydrodynamic loading was found: grains were coarser and less well-sorted in zones of increased wave energy than in zones with lower energy levels. For that reason the "plunge step" (station 3 on Figure 2-1) was always found to be composed of the coarsest and worst sorted material (see Figure 2-1).

New Zealand

Single and Hemmingsen (2001) summarised the work of Kirk and collaborators in the characterisation of New Zealand mixed sand and gravel barrier beaches. These beaches are mainly meso-tidal with a moderately steep lower foreshore (see Figure 2-2) where wave breaking occurs. They contain a wide range of sediment sizes with an across-shore zonation, having a bimodal composition in the lower foreshore with a peak at 3-5mm and another one around 18mm. The profile is considered to be dominated by swash and backwash processes, and alongshore transport involves two different processes:

a) seaward of the nearshore step where sand is transported by currents and
b) in the swash zone, where it is driven by breaking wave and swash processes.

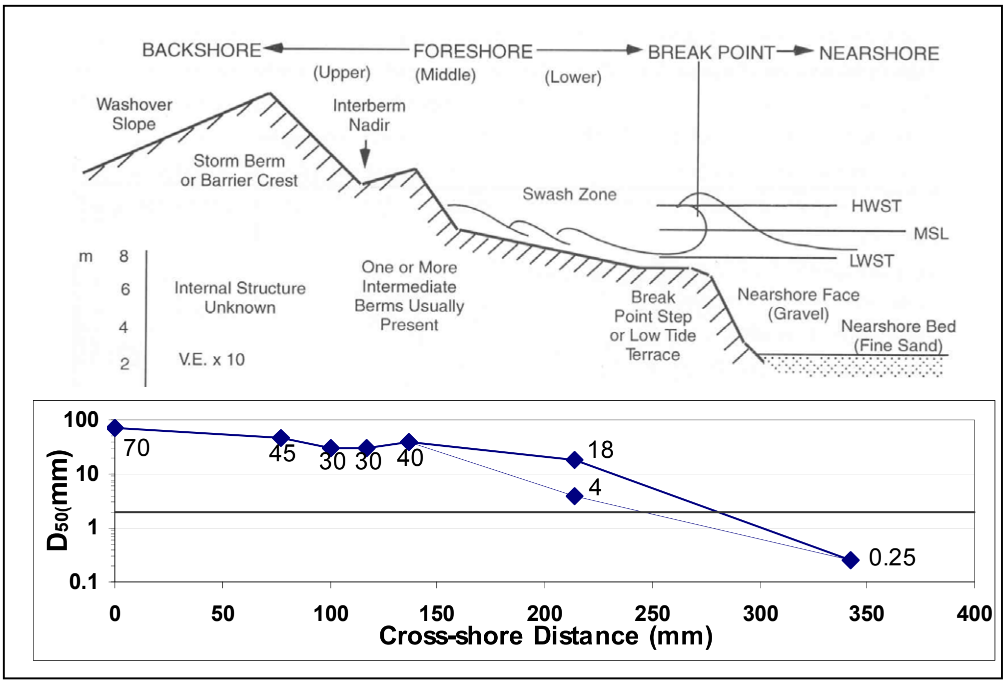


Figure 2-2. Cross-shore changes in sediment grain size on a typical New Zealand tidal beach (modified from Single and Hemmingsen, 2001).

(Numbers in graph show median grain size).

UK

Two types of mixed beaches are found in the UK according to Mason (1997):

- Homogeneous mixtures of shingle and sand, with varying proportions of each, both cross-shore and long-shore. The low water sandy region is exposed only during spring tides, if at all.

- Beaches with a largely shingle ridge towards the high tide region, and a sandy inter-tidal terrace, i.e. a sand beach with fringing shingle, often referred to as a composite beach.

Both types of beach may have a noticeable break of slope between the mixed/shingle and sand sections.

Coates and Mason (1998) reported sand contents in mixed beaches from various locations, as summarised in the following table.

Location	**Sand content***	**Author**
Suffolk, UK	30%	Pontee (1995)
Carnsore, SE Ireland	<20%	Orford and Carter (1985)
Mixed barrier, Canadian Beaufort Sea coast	53% sand (8% silt and clay)	Hill (1990)
Canterbury, New Zealand	68% to 48% along the coast	Kirk (1980)
Palliser Bay, North Island, New Zealand	Between 17 and 63%	Matthews (1983)
Several beaches and barriers, Washington State, USA	Between 15 to 50%	McKay and Terich (1992)

Table 2-1. Reported sand content in mixed beaches around the world.
(*the balance being gravel)

Figure 2-3 shows some examples of mixed beaches around Europe, including the UK. It is clearly seen that this type of beaches has a great recreational value (e.g. Sicily). Also, that the processes giving rise to erosion are not always understood and well managed (e.g. Ventimiglia).

The beaches on which this study concentrates are those composed of a mixture of shingle and sand in an “un-regulated” manner (as opposed to composite beaches). The term “un-regulated” is used because the mixture found in the beach is rarely homogeneous. The properties of the sediment mixture change with depth, across and alongshore as well as in time.

Regarding the temporal variability, Pontee (1995) found that one of the most noticeable features of mixed beaches studied along the Suffolk coastline (UK) was the great variability on a day to day, month to month time-scale under apparently similar conditions. The reasons proposed for this variation are:

1. Rapid beach responses to changing conditions.
2. Possible existence of feedback effects from previous beach and nearshore conditions.
3. The differential deposition of sand and gravel under different conditions.

Playa del Albir, Spain

Sicily, Italy

Ventimiglia, Italy

Amorgos Island, Greece

Hayling Island, UK

Seaford, UK

Figure 2-3. Examples of mixed beaches around Europe.

2.2 What is different about mixed beaches?

The following factors arise when considering the differences between mixed, shingle and sand beaches:

- A differentiating characteristic of mixed and shingle beaches, compared with sandy beaches is their relatively higher permeability. Coarser grains in shingle beaches result in large pore sizes, but not necessarily a change in porosity, allowing flow between the grains. But to what extent are mixed beaches permeable relative to shingle beaches, because of the fact that finer grains fill the voids between coarser grains? How do the grading characteristics influence the permeability of the mixtures?

- Generally, beaches have been divided into those which are dissipative and those which are reflective beaches, with different morphodynamics and sediment transport related to each one. These types of beach can also be associated with sediment size, so that sandy beaches are normally dissipative and gravel ones reflective, with a range of possibilities in between. Are mixed beaches dissipative, reflective or something in between? What are the possible causes of this?

- Propagating waves create dynamic water pressures on the surface of the seabed. Several authors have studied the wave motion percolating through a permeable bottom and its influence on the wave-induced forces on hydraulic structures supported on or extending into the bottom. In the present case, changes of pressure associated with the wave motion and the wave set-up, producing a flow of sea-water within the beach itself, are of interest. How can this flow of water be quantified and what is its relevance?

- The permeability will influence the pressure propagation within the bed, as drainage is allowed, and consolidation will influence the stability of the bed as a whole. Is the mixed sediments bed response to water waves immediate due to the elasticity of the soil or does it take time to propagate into the soil? During the passage of a wave trough, a vertical pressure gradient would exert an upward force on the top layer of the sediment. If this pressure gradient is sufficiently large, the sediment may become fluidised over a short period of time. Do these large pressure gradients really occur; under what circumstances and if so what are the implications?

- Groundwater flow has increasingly been considered important in the morphodynamics of beaches. Beach drainage systems have been implemented as soft protection measures throughout the world. To what extent is the groundwater flow important in mixed beaches?

- In the study of the behaviour of mixed beaches, not only do the sediment size distributions have to be considered but also the hydrodynamics and sediment transport. There are several processes that need special attention in order to derive the morphodynamic behaviour of the mixed beaches. One of the first questions that comes to mind when considering different sizes of sediment on a bed is if this bed will be more easily transported: Will the different size grains inhibit initiation of motion compared with a single size bed? Will the particles be more easily entrained? Will the seepage flow into/out of the bed restrict/enhance initiation of motion? What is the influence on the boundary layer? How will these flows influence the sediment transport?

These issues are addressed in the following sections, where a literature review of the differentiating characteristics of mixed beaches is carried out.

2.2.1 Permeability

Definition

The permeability, K (m/s), or hydraulic conductivity, is a measure of the ease with which a fluid flows through a permeable material. It is a function of both the sediment properties and the fluid flowing through it. The sediment properties controlling permeability are porosity, n, sediment size and grading; the fluid ones are the density, ρ, and viscosity, μ.

Empirical evidence and intuitive reasoning indicate that permeability increases with increasing particle size. British Standards give typical values of permeability for fine sand (0.0001m/s), medium sand (0.001m/s), coarse sand (0.0065 to 0.01m/s) and gravel (0.04 to 0.16m/s). Barnes (1995) gives typical values for gravel (0.01-10m/s) and sand and sand-gravel mixtures (0.00001 to 0.01m/s).

Permeability is sometimes quoted as a coefficient of permeability or "intrinsic permeability". The relation between permeability, K, and intrinsic permeability, k(m^2) is:

$$k = \frac{K\mu}{\rho g} \quad \text{...(2-1)}$$

where

ρ is the density of the fluid (kg/m^3) and

μ is the dynamic viscosity (kg/ms) given by $\mu = \rho\nu$ where ν is the kinematic viscosity (m^2/s)

Effects of grain size and its distribution on permeability

Grain size distribution will affect the permeability of a sample because the permeability is related to the amount of free space there is for fluids to flow. If the fluid path is blocked due poorly sorted sediment sizes, the effect will reduce the amount of fluid that can flow through. This is accounted for mathematically by a "tortuosity constant".

In terms of the sediment, permeability depends on grain size, sorting, grain shape and packing.

The relationship between porosity and permeability is expressed through the Carmen-Kozeny relationship that states that permeability is related to porosity as well as the grain specific area (see Table 2-2). Several authors have given formulations for the permeability in terms of sediment size characteristics (see Table 2-2). The Krumbein and Monk (1942) formulation takes into account a measure of sorting as well as particle size.

Author	Permeability	
Hazen (1911)	For sands: $K=CD_{10}^2$ For graded sand filters: $K=CD_{60}\,D_{10}$	C=0.01-0.015 D_{10} and D_{60} in mm K in m/s
Kozeny (1927)	$K=\dfrac{Cd^2n^3}{(1-n)^2}$	n: porosity d: particle size C: empirical constant
Kozeny (1927) – Carmen (1937)	$K=\dfrac{\rho_w g}{\mu}\dfrac{n^3}{(1-n)^2}\dfrac{d_m^2}{180}$	n : porosity d_m : representative grain size
Krumbein and Monk (1942)	$K=(760d_w^2)\exp(-1.31\sigma_\psi)$	d_w : geometric mean diameter in mm σ_ψ: std of the ψ distribution function ($\psi=-\log_2 d$; d in mm)

Table 2-2. Hydraulic conductivity formulations.

In soil mechanics, it is recognised that small particles mixed into a soil will decrease its permeability. Kachi and Suezawa (1937) proposed an empirical equation for the permeability of such a mixture:

$$\frac{K}{K_0}=C\exp(as) \quad \text{(2-2)}$$

where

K_0 is hydraulic conductivity of the original soil

s is the volume of small particles

C,a are constants

An estimation of the permeability can also be made with laboratory permeameter tests. However, Darcian flow is assumed in the permeameter tests and therefore is not valid for coarse sediments.

Porosity

Porosity, n (-), is defined as the ratio of voids to the total volume, which is also a measure of the volume of particles contained in a defined volume space.

Domenico and Schwartz (1997) give typical values of porosity for gravel (0.24-0.38), coarse sand (0.31-0.46) and fine sand (0.26-0.53).

The factors affecting porosity are:

- Grain size: In and of itself, grain size has no effect on porosity. Well-rounded sediments that are packed in the same arrangement generally have a porosity from 26% to 48% depending on the packing. The influence of the packing is straight-forward when calculating the porosity of solid spheres with equal diameter as an approximation, Fetter (1994):
    - Cubic packing (each sphere sits directly on the crest of another sphere): n=0.48
    - Rhombohedral packing (the spheres lie in the hollows formed by four adjacent spheres of the underlying layer): n=0.26
- Sorting: Well sorted sediments generally have higher porosity than poorly sorted sediments for the simple reason that if a sediment contains a range of particle sizes then the smaller particles may fill in the voids between the larger particles. Sorting is measured as the ratio of the larger to smaller particle sizes in the sediment. This measure is called a uniformity coefficient,D_{60}/D_{10} where D_{60} is the grain size below which 60% of the sediment is finer and d_{10} is the grain size below which 10% of the sediment is finer
- Grain shape: Irregularly shaped particles tend not to pack as neatly as rounded particles, resulting in higher proportions of void space.

Porosity of mixed sediments

Liao (1998) derived an empirical formula to calculate the porosity of graded sediments from the geometric mean diameter, D_g, and the standard deviation of the grain size distributions, σ_g, as

$$n = 0.3531 + 0.1229\log_{10} D_g - 0.6852(\log_{10}\sigma_g)^{1.754} \quad \text{......................................(2-3)}$$

where:

$$D_g = \sqrt{D_{84.1}D_{15.9}} \quad \text{..(2-4)}$$

$$\sigma_g = \sqrt{\frac{D_{84.1}}{D_{15.9}}} \quad \text{.......(2-5)}$$

where $D_{84.1}$ and $D_{15.9}$ are the grain sizes for which 84.1% and 15.9% by weight are finer, respectively. The ranges of the sediments used in his experiments were: D_g = 0.295 to 4mm and σ_g = 1.34 to 1.98mm.

Permeability of coarse-grained beaches

Several coastal researchers agree on the importance of the permeability in coarse-grained beaches, which could extrapolate to mixed beaches. Whereas the permeability of sandy beaches is low and the permeability of gravel beaches is of significant magnitude and importance; the permeability of mixed beaches depends on many factors. These factors are:

- the type of sediment mix, gap-graded or well-graded,
- the sizes of the sediments and
- the percentage of these sediments.

When the coarser grain interstices are full of smaller material or there is a small amount of coarse material, the permeability will be quite low, close to that of the small particles. When these interstices are not full, because of a limited amount of smaller material or because of its sizing, the permeability will be closer to that of the greater grains. Moreover, as the swash zone is very energetic and variable, sediments are not highly consolidated and water content is variable, both influencing the permeability.

Holmes *et al.* (1996) carried out some laboratory experiments with fine sand (0.5mm), coarse sand (1.5mm) and bimodal (50:50 mix) beaches under random waves in order to study the principal factors in the evolution of the bed. The profile evolution of bimodal beaches was found to be similar to that of fine sands (as found by Quick and Dyksterhuis, 1994), although considerable sorting was observed. They found that the permeability was largely controlled by the finest 10% of sediment within the mix (as Hazen, 1992) and concluded (in agreement with Quick and Dyksterhuis, 1994) that the permeability is the dominant controlling factor.

Mason (1997) conducted several experiments in order to determine the permeability of sand/shingle mixtures. The mixtures consisted of fine beach shingle of D_{50}=4mm with 10% incremental proportions of fine (D_{50}=0.159mm), medium (D_{50}=0.356mm) and coarse (D_{50}=0.846mm) well-sorted sand. Results are shown in Figure 2-4.

The inclusion of sand in the shingle had a significant influence on the permeability, compared to the permeability of the shingle-only:

- a mixture of only 20% medium sand and shingle reduced the permeability of the shingle by 65%. When mixed with 20% of fine sand this decrease was almost 90%. Mason explains this reduction of permeability with increasing sand content due to the fact that the pore spaces between the shingle grains are filled with the sand grains.
- an interesting finding was the fact that the addition of between 10% and 60% of shingle to coarse and medium sand actually reduced the permeability of the mixture, which was explained in terms of the tortuosity of the fluid path through the mixture.

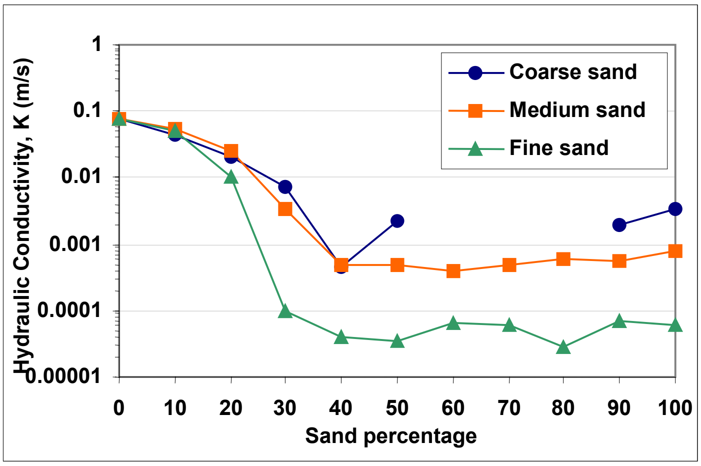


Figure 2-4. Mason (1997) hydraulic conductivity results.

Flow of water through a porous medium

The flow of water through a porous medium will depend on the permeability of the medium (Bear, 1972). If the flow is laminar (for Re[1]<10), the relationship of the flow velocity to the pressure gradient driving the flow is given by Darcy's law:

$$V = K\frac{dh}{dz} = Ki \quad \text{.......(2-6)}$$

For Re>10, the intergranular flow may be turbulent and the relationship is given by the Forchheimer equation:

$$i = aV + bV^2 \quad \text{.......(2-7)}$$

where, for consistency, a=1/K. van Gent (1993) gives a summary of expressions and values for a and b.

[1] Reynolds number defined as Re=uD/v, where the length scale D is equal to the D_{50} of the sediment

Packwood and Peregrine (1980) calculated the critical velocities for various materials for which the linear Darcy's law is valid; their results given in the table below.

Sediment	**D (mm)**	**K (m/s)** (K=6500D^2(m) from Bear, 1972)	**Critical velocity,u_c (m/s)**	$\frac{u_c}{K}$
Gravel	5.0	0.16	0.002	0.0125
	2.5	0.04	0.004	0.1
Coarse sand	1.25	0.01	0.008	0.8
	1.00	6.5×10^{-3}	0.01	1.5
Medium sand	0.4	1.0×10^{-3}	0.025	25
Fine sand	0.125	0.1×10^{-3}	0.08	800

Table 2-3. Critical velocity for Darcy's law validity (modified from Packwood and Peregrine 1980).

2.2.2 Wave Reflection

An important aspect of beaches is their effectiveness in dissipating wave energy, especially when treating beaches as a coast protection structure.

Definition

When a wave encounters a beach, part of the wave energy is dissipated. The remaining energy is reflected seaward and/or transmitted through the beach. The coefficient of reflection can be defined as:

$$K_R = \sqrt{\frac{S_R}{S_I}} \qquad (2\text{-}8)$$

where S_I is the incident energy and S_R is the energy reflected.

Similarly, the coefficient of dissipation can be defined as

$$K_D = \sqrt{1 - \frac{S_R}{S_I}} \qquad (2\text{-}9)$$

so that the relation between both is given by:

$$K_D = \sqrt{1 - K_R^2} \qquad (2\text{-}10)$$

On natural beaches, Waddel (1980) showed that reflection is strongly dependent on wave frequency for a given beach slope, so that the beach acts as a selective absorber/ reflector. The beach can be considered as acting as a "low-pass filter", damping high frequency waves and allowing low-frequency waves to propagate inland and/or to be reflected.

Background

Sandy micro-tidal beaches have been long recognised to form a spectrum of types from dissipate to reflective with various intermediate states (Short, 1999 as a background and update on this subject). However, although it is recognised that dissipation processes in gravel and mixed (sand and gravel) beaches could be important, very little work has been carried out on the measurement and estimation of reflection coefficients on such beaches.

Variability with beach slope

As a rule of thumb it has been suggested that the slope of the beach can be used to determine the reflection coefficient of gravel beaches. For a slope of tanβ, the reflection coefficient will be of the order of β%. Benoit and Teisson (1994) found the ratio of slope to coefficient of reflection, K_R/tanβ to be of the order of 0.5 for slopes ranging from 1:2 to 3:4 based on laboratory tests.

Variability with Iribarren Number or Surf Similarity Parameter

Battjes (1974) obtained an empirical formula for the reflection coefficient on a sloping bottom as:

$$K_R = 0.1\xi \qquad \text{(2-11)}$$

where ξ is the Surf Similarity Parameter or Iribarren Number defined by

$$\xi = \frac{\tan\beta}{\sqrt{H/L}} \qquad \text{(2-12)}$$

However, this equation cannot be applied to all sizes of beach material because of the different dissipation processes.

From an extensive data set of laboratory beaches, Chessnutt (1978) found that many factors influence the magnitude of the reflection coefficient, suggesting that

$$K_R = \frac{\alpha\xi^2}{\xi^2+\beta} \text{ with } \beta=5.5 \qquad \text{(2-13)}$$

Seelig and Ahrens (1981) suggest using α=1.0 for conservative estimates and α=0.5 for predictions of “average” reflection.

Variability with wave steepness

Miche (1951) calculated a limiting wave steepness for obtaining complete reflection from a sloping beach:

$$\left(\frac{H_0}{L_0}\right)_{\max} = \sqrt{\frac{2\beta}{\pi}}\frac{\sin^2\beta}{\pi} \qquad \text{(2-14)}$$

where $\tan\beta$ is the beach slope. The reflection coefficient for the sloping beach becomes:

$$K_R = \frac{\left(\frac{H_0}{L_0}\right)_{\max}}{\left(\frac{H_0}{L_0}\right)} \qquad \text{if } \left(\frac{H_0}{L_0}\right) > \left(\frac{H_0}{L_0}\right)_{\max} \quad \text{...(2-15)}$$

$$K_R = 1 \qquad \text{if } \left(\frac{H_0}{L_0}\right) \leq \left(\frac{H_0}{L_0}\right)_{\max} \quad \text{...(2-16)}$$

This formula leads to an overestimate when K_R is nearly unity.

Powell (1990) found from laboratory studies on gravel beaches that, generally, in excess of 90% of the wave energy is dissipated by a gravel beach. Also, the dissipation capacity of a beach profile is not significantly affected by its development in time. He found a relationship between the coefficient of dissipation, K_D, and the incident “sea state steepness”, H_s/L_m (where H_s is the significant wave height and L_m the mean wavelength). From this relationship, for a steepness of 0.02, K_D is constant and equal to 0.99 (K_R=0.14) and for smaller values of steepness the dissipation is reduced (K_R>0.1). Surprisingly, both the material size and the effective beach thickness had small contributions to the overall trend. This was ascribed to the flow within the body of the beach having a limited influence on the overall dissipation of wave energy.

Wave Reflection on Mixed Beaches

Little work has been carried out to date in the estimation of wave reflection for mixed beaches. Davidson *et al.* (1994) pointed out the importance of the tidal variation in the coefficient of reflection. As the slope of the mixed beach varied with the tidal stage, higher reflection coefficients were obtained from the steeper shoreface gradients. Mason (1997) also observed an increase in reflection coefficients with rising tidal levels, although this increase was only for the swell component of the incident wave field.

There are two aspects influencing reflection on mixed beaches:

- The milder beach slope in comparison with a gravel beach will reduce reflection on a mixed beach
- However, the greater permeability of gravel beaches in comparison with mixed ones may influence the reflection by reducing the amount of energy available for reflection (as some of that energy is transmitted into the beach and absorbed in it).

An analysis of the influence of the slope and permeability characteristics of beaches is necessary to understand their joint influence on wave reflection. Mason *et al.* (1997)

concluded that a mixed sediment profile will reflect more energy than both a sand beach (due to a steeper gradient) and a shingle beach (due to less energy dissipation through infiltration).

2.2.3 Internal pressures and flow within the beach

The interaction between propagating waves and the seabed has received extensive attention, wave-induced liquefaction in particular. A primary application of the wave induced liquefaction research was for offshore structures, because of the possible damage that pipelines, platforms or breakwaters can suffer. However, studies into the relation between wave-induced liquefaction and scour, littoral drift and sediment transport are now more numerous.

Definitions

Excess pore pressure and liquefaction criteria

In response to the wave-induced pressures, excess pore pressures are produced in the seabed (excess defined as difference from the hydrodynamic pressure). There are two mechanisms causing the excess pore pressure build up under waves (Zen and Yamazaki, 1990):

- Oscillatory excess pore pressures that occurs transiently and periodically, corresponding to each wave. They are produced by the spatial difference of pore pressures in the seabed, so that the key element is the vertical gradient of the oscillating pore pressure near the bed surface. The pore pressure distribution brought about by the damping and phase lag in the propagation process of pressures is considered to be the reason for the oscillatory wave-induced liquefaction. Vertical pressure gradients induce flow into the bed under wave crests and upwards out of the bed under the wave through (see Box 2-1, page 37).

- Residual excess pore pressure, due to the continuous build up of pore pressures induced by the cyclic wave loading. The pore pressure generated will depend on the relative density of the soil, the induced cyclic shear stress ratio and the existing pore pressure.

The total excess pore pressure is the superposition of both the oscillatory and residual excess pore pressure. However, residual excess pore pressure for coarse-grained sediments is not believed to be of importance as drainage is sufficient to relieve it. Moreover, in the case of the swash zone, in contrast to deeper depths, the generation of excess pore-pressure results from the dynamic or oscillatory condition in contrast to a continuous build-up. Therefore this thesis considers only the oscillatory excess pore pressure.

The excess pore pressure can cause deformation of the grain skeleton or even of the fluid. Considering that the grains themselves are very stiff and their volume change under load can be ignored, the volume change is due to:

- Compaction: or expulsion of air from the soil.
- Volume changes in water. Normally it can be assumed that under saturated conditions the compressibility of the water can be disregarded, whereas for unsaturated conditions, the compressibility of the air/water mixture will be highly dependant on the saturation rate.
- Consolidation: volume changes in the grain skeleton as drainage through a permeable boundary occurs. The amount of change will depend both on the geometric details of the soil profile and on the permeability and compressibility characteristics of the soil. The volume changes in the soil skeleton are due both to the rearrangement of the soil particles and to the elastic strains in the particles.

There may also be an increase (decrease) of the effective stresses as deposition (erosion) of the soil occurs

In Box 2-1 a summary of the excess pore pressures generated by waves and the liquefaction criteria is given. It is clearly seen that during the passage of a wave trough, the vertical pressure gradient exerts an upward force on the top layer of the sediment. If this pressure gradient is sufficiently large, this upward force may exceed the overburden, and the soil could become liquefied for a short period of time

Governing equation for oscillatory pore pressure

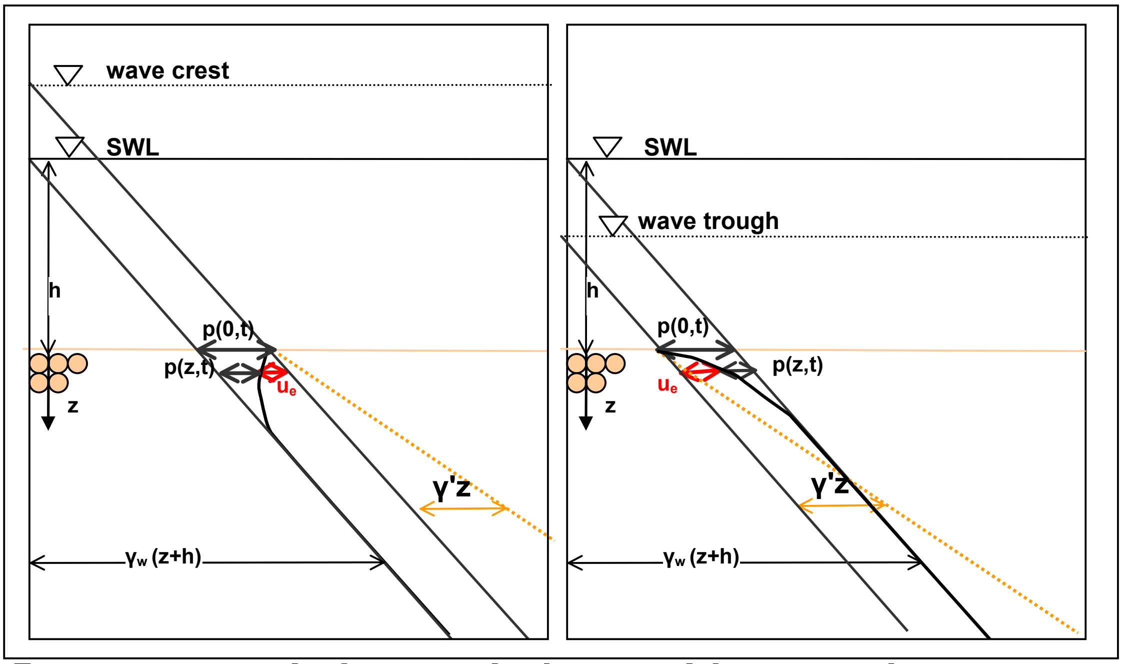
It is usually assumed that the flow of pore water in the soil is governed by the steady state form of Darcy's law. Considering the conservation of mass of pore water, one can deduce the governing equation of excess pore pressure (e.g. Zen and Yamazaki, 1993 for derivation). This equation reduces to the Consolidation equation when the compressibility of the water is considered to be negligible.

Box 2-1 Excess pore pressure and Effective stress

Considering the oscillation of the wave-associated pressure and the motion of the seabed as one-dimensional and omitting the cyclic shear stresses induced by the waves, the following pore pressures and effective stresses can be determined. {(z,t) represents the pressure at a given time at an arbitrary depth z, whereas (0,t) represents the pressure at the seabed surface}

	Initial	**At a time t after a water level change**
Pore water pressure	$u(z,0)$	$u(z,t) = u(z,0) + p(0,t)$ *p(0,t): pressure change imposed on the seabed surface*
Total vertical stress	$\sigma(z,0)$	$\sigma(z,t) = \sigma(z,0) + p(z,t)$ *p(z,t): pore pressure change from the initial state of the hydrostatic pressure*
Effective vertical stress	$\sigma'(z,0)=\sigma(z,0)-u(z,0)=\gamma' z$ *γ'z is the effective overburden*	$\sigma'(z,t) = \sigma'(z,0) + p(0,t) - p(z,t)$ For convenience, it is usual to define $p_b=p(0,t)$ and $p_m=p(z,t)$
Effective stress change	$\Delta\sigma'(z,t) = p(0,t) - p(z,t)= p_b-p_m$	
Excess pore pressure	$u_e(z,t) =- \Delta\sigma'(z,t) = -[p(0,t) - p(z,t)]=-[p_b-p_m]$	

The figure below represents this for the case of a wave crest and a wave through



Excess pore pressure development under the crest and the wave trough.

Liquefaction Criteria

Liquefaction is induced when the vertical effective stress is zero or less:

$\sigma'(z,t) \leq 0$. As we defined $\sigma'(z,t)= \sigma'(z,0) + p_b - p_m$ the liquefaction criteria is met when:

$\sigma'(z,0) \leq -[p_b - p_m] = u_e(z,t)$

which is saying that liquefaction will occur when the excess pore pressure exceeds the effective overburden pressure.

Seepage Flow

An additional pressure force acting on the bed particles called seepage pressure will be present due to these vertical flow velocities through the bed. This force is defined as

$$j = -\frac{\partial p_m}{\partial z}$$

Reported infiltration / exfiltration velocities in natural beaches

A number of authors have reported on infiltration/exfiltration velocities, w, as calculated from Darcy's law (assuming laminar flow) from pressure measurements taken in the same vertical at a given beach position:

$$w = -K\frac{dh}{dz} = -Ki \qquad (2\text{-}17)$$

where K is the hydraulic conductivity (m/s)
i is the hydraulic gradient, dh/dz (-)

The flow can be consider laminar for Reynolds number < 10 (Bear, 1972). The reported velocities are summarised in the Table 2-4 below.

Author	Beach	Beach characteristics	Infiltration velocities	Exfiltration velocities
Butt *et al.*, 2001	Perranporth Beach, UK	tanβ=0.0143 D_{50}=0.24mm	w/K≈0.2	w/K≈0.2
Turner & Nielsen, 1997	Assateague Island, Maryland, US	tanβ=0.05 D_{50}=0.4mm K=0.002 **(1)**	w/K≈0.2 downwards	
Horn *et al.*, 1998 Blewett *et al.*, 1999	Candford Cliffs, Poole, UK	tanβ=0.055-0.027 D_{50}=0.24mm K=0.00225m/s (0.00036 to 0.01179m/s) **(2)**	w/K≈0.3-0.47 (in swash uprush) w/K≈1.2-1.5 (when uprush first onto dry sand)	w/K≈-2. (in backwash)
Blewett *et al.*,2001	Seaford, UK	tanβ=0.1 upper: gravel (D_{50}=6-40mm) lower: gravel/ coarse sand mix	w/K≈1.8-2.2 (max.)	w/K≈0.06

(1) Calculated hydraulic conductivity from Krumbein and Monk (1942)
(2) Measured hydraulic conductivity

Table 2-4. Infiltration/ exfiltration velocities in natural beaches.

It is clearly seen that in coarse beaches infiltration is more important than in sand beaches. However, the velocities calculated are greater than the critical velocity for the validity of the linear Darcy flow (see Table 2-3), implying that the flow is turbulent and the velocities should be calculated accordingly.

The authors quoted in Table 2-4 have questioned the potential for fluidisation to occur from the exfiltration velocity calculations for sandy beaches. These are summarised in the next section. However, one has to note that not only are the reported velocities different, but also the condition for fluidisation varies between these different authors, as seen in next section.

Reported liquefaction events

Tsuruya and Korezumi (1991) predicted that the state of liquefaction in the top layer of sand beds in the surf zone is easily formed just posterior to the breaking wave crests. The measured sand concentrations near the bed frequently showed an abrupt increase in accordance with the occurrence of liquefaction.

Horn *et al*. (1998) and Blewett *et al.* (2001) show in their field measurements what they name "unloading" events, characterised by a large upward-acting hydraulic gradients in the top layer of a sand bed after a swash event. According to the Packwood and Peregrine (1980) criterion (Table 2-5), they are significant enough to induce the fluidisation and hence possible enhanced erosion (providing the destabilising gradients are in-phase with the velocity profile, that there is a head of water above the beach surface and that the exfiltration flow does not change the boundary layer properties of the flow and hence the shear-stress).

Baird and Horn (1996) carried out preliminary model tests that indicated that the pressure reversal (due to the loading/unloading cycle of swash/backwash) is a mechanism for potentially-enhanced sediment transport under swash action.

These unloading/reversal events can be explained as follows:
As swash advances over the beachface, there will be a rapid increase in pore water pressures below the beach flow. During swash, when the sediment is saturated the pressures will propagate rapidly through the sediment (also, movement of water is limited, as changes in the sediment skeleton are minimal). When this swash retreats, there will be a release of pressure on the beach face producing large hydraulic gradients acting vertically upwards, immediately below the sediment surface.

However, much discrepancy exists in this subject, Butt *et al.* (2001) and Turner and Nielsen (1997), not finding that the conditions for fluidisation are met. Table 2-5 summarises the different liquefaction criteria used.

Care has to be taken when using these criteria because of the assumptions made in their development. None of them has been shown to be valid for coarse sediment or mixed sediments.

Author	Criteria	Validity / Remarks
Madsen (1974)	$\frac{u}{K} > \frac{\gamma_s}{\gamma} \tan\theta \approx 0.5$	For horizontal pressure gradients. Assumed θ≈35º
Packwood and Peregrine (1980). Used by Horn *et al.*, 1998 and Blewett *et al*. (2001)	$\frac{w}{K} > \frac{\gamma_s}{\gamma} \approx 0.7 to 1$	Sands and fine gravels
Turner and Nielsen (1997)	$\frac{w}{K} > (s-1)$	
Soulsby (1997)	$w_{crit} = \frac{\upsilon}{d}\left\{\left(10.36^2 + 1.049 n^{4.7} D_*^3\right)^{1/2} - 10.36\right\}$	For laminar and turbulent flow
Soulsby (1997)	$w_{crit} = 5.75x10^{-4}(s-1)\frac{gD_{50}^2}{\upsilon}$	For grains smaller than 0.8mm
Butt *et al.* (2001)	$\frac{w}{K} > \frac{s-1}{a} \approx 3.2$	Assuming *a*=0.5 as suggested by Martin and Aral (1971) experiments **(1)**

(1) Martin and Aral (1971) experiments were with sands (0.18-0.72mm) and glass beads (0.1-0.3mm)

a : ratio between the seepage force acting in the surficial layers of sediment and that acting within the bed (Martin, 1970)
υ : kinematic viscosity of the water (m^2/s)
θ : internal friction angle
ρ : density (kg/m^3)
γ : unit weight (kN/m^3) γ=ρg

$$D_* = D\left(\frac{g(s-1)}{\upsilon^2}\right)^{1/3}$$

n: porosity (-)
s: ratio of densities of grain and water (-)

Table 2-5. Criteria for fluidisation.

Flows within a permeable beach

The beach system is driven by the physical energy induced by waves and tides, the water flow through the beach body being of great importance. This water flow is not only important for the accretion-erosion process it results in but also because it controls the vertical and horizontal, chemical and biological gradients and nutrient exchange in the beach (McLachlan, 1989). His measurements showed that the volume of water filtered through the beach depended on the beach type (reflective / dissipative as ends of the spectrum).

Simplifying matters, assuming a tide-less permeable beach, four regions can be defined (see Figure 2-5):

- Region 1: where the mean water depth is assumed to be constant. The waves propagate with no significant change.
- Region 2: the mean water depth changes within the profile. This region is dominated by the set-down (depression of the mean sea level below SWL), which is maximum near the breaking point.
- Region 3: the broken wave propagates over the beach slope. Set-down diminishes and becomes set-up. The maximum set-up defines the dynamic shoreline position, whereas the SWL defines the static shoreline position.
- Region 4: dominated by the wave uprush.

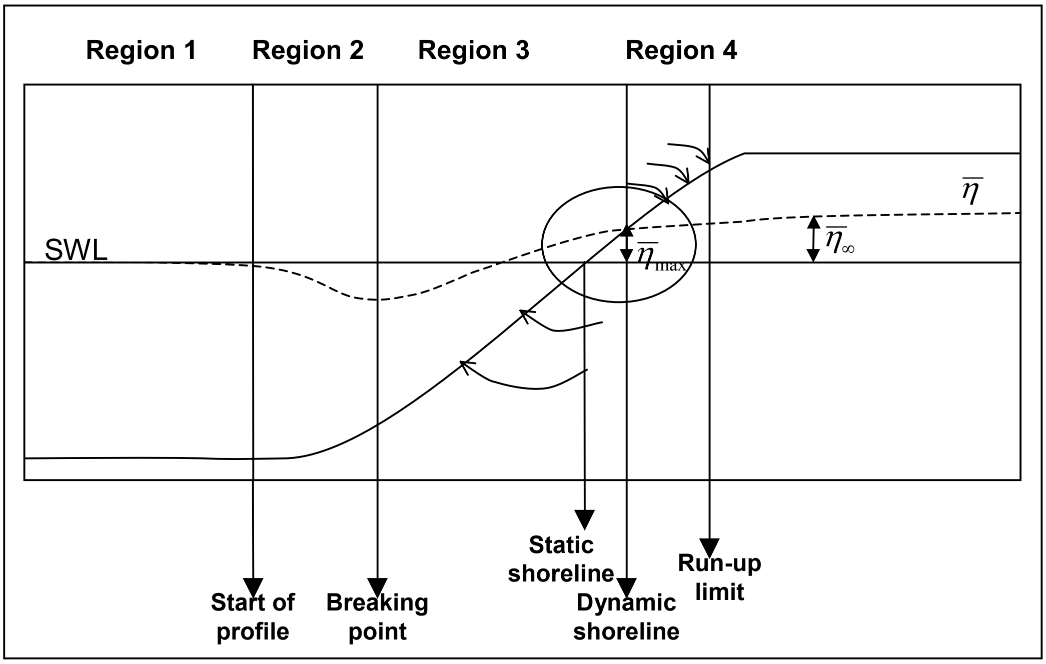


Figure 2-5. Regions within the beach (modified from Massel and Pelinovsky, 2001).

These regions are dominated by different flows:

- Region 3: dominated by the exfiltration (sometimes named as groundwater circulation by Nielsen*,* 1997 and Li and Barry, 2000).
- Region 4: dominated by the instantaneous infiltration produced by the run-up. This run-up contributes to the raising of the coastal water table, as will be seen in the next section.

However, looking closer at the area near the static and dynamic shoreline (defined by the SWL and the maximum set-up, respectively) that has been marked as a circle in Figure 2-5, one could expect a different flow pattern from the general one described above. Because of the up-rush and backwash the free surface intersects the beach at a different position and infiltration/exfiltration patterns could reverse from the general pattern described above. Exfiltration within the swash zone may occur when the swash depth is zero, given a driving head within the body of the beach.

Massel (2001) presented a theoretical attempt to predict the groundwater circulation induced by the set-up in a sandy beach. He mentioned two systems of circulation, related to different gradients of the set-up height:

- Offshore gradient: the horizontal excess pressure gradient completely swamps the viscous forces in the boundary layer and carries the flow in the offshore direction
- Closer to the shore, the pressure gradient is reversed (at the static shoreline, the external excess pressure reaches its maximum value) and the resulting flow moves shorewards. This system is smaller than the first one.

In his deduction, Massel (*op.cit.*) assumed shallow water, Darcian flow, fully saturated flow (no air), rigid grain skeleton and grains (with these assumptions, the pressure obeys the Laplace equation), permeability given by Hazen (1911) and local accelerations to be zero.

Turner and Masselink (1998) state that the transport of sediment within the swash zone is determined by the wave characteristics, beach slope, bed characteristics and swash in/exfiltration. They measured in/exfiltration flows of the order of 10^{-3}m/s, affirming the occurrence of laminar flow and the validity of the Darcian law assumed for the calculations. They also found, at the time scale of single swashes, the vertical pore pressure gradients within the beach face to be of several orders of magnitude greater than the horizontal pore pressure gradients.

Turner and Nielsen (1997) stated that the stress exerted normal to the bed due to the exfiltration of groundwater across the beach face is proportional to the vertical pore pressure gradient. The total force acting on a given volume is then the summation of gravity and pressure gradient. They then calculated the fluidisation velocity (given by the fluidisation criteria that the total force has to be zero) for different sand grain sizes, assuming well sorted sands to calculate the permeability. The conclusion was that for the full range of fine to coarse sands, exfiltration flow speeds in excess of 50mm/s would be sufficient to fluidise the bed.

2.2.4 Groundwater

Nielsen (1997) defined the water table as the mean surface within the beach material where the mean pore water pressure is zero. The elevation and shape of the beach water table depend on:

- The characteristics of the beach material, such as size and shape range, porosity and permeability, and
- The hydraulic conditions. Not considering rainfall, the waves and tide determine these hydraulic conditions.

Although the tidal response of the water table is quite important (rising steeply with a flooding tide, and falling more slowly during the ebb), this is not considered in this thesis, where only the effects of the waves are studied.

Definitions

The role of the waves in modifying groundwater elevation in the coastal zone is, according to Turner *et al.*(1997), two-fold:

1. Set-up at the shoreline, which results in a raising of the mean water surface at the shoreline
2. Run-up of waves across the beach-face, which further elevates the potential zone of seawater inflow.

However, a clarification of the terminology used by Turner *et al.* (*op.cit.*) is required. Run-up is usually considered as composed of a steady super-elevation above the SWL called set-up and a fluctuating component called swash. The role of the waves in modifying groundwater elevation should then be the result of:

1. Set-up at the shoreline
2. Swash of waves across the beach-face

These concepts are illustrated in Figure 2-5 and are explained in the next section.

Set-up at the shoreline

The set-up at the shoreline is the maximum set-up. Bowen *et al*. (1968) proposed a simple linear model of set-up at the shoreline, B_s or $\overline{\eta}_{max}$, as a function of wave height for impermeable, smooth and constant slopes:

$$B_s = B_{\min} + \frac{3}{8}\gamma H_b \quad \text{(2-18)}$$

where

B_{min} is the maximum set-down immediately prior to breaking

γ is the wave height to water depth ratio at breaking

H_b is the breaker wave height

As waves usually break at a depth approximately 1.2 times their height, this makes the set-up at shoreline to be about 25% of the height of the breaking wave. Fredsoe and Deigaard (1992) suggest:

$$B_s = \frac{3}{8}\gamma H_b \approx 0.2 H_b \quad \text{(2-19)}$$

However, further studies (Bowen *et al.*, 1968) suggest that the mean water surface close to the shoreline rises even further. An extensive investigation by Hanslow and Nielsen (1993) suggests the empirical relationship:

$$B_s = 0.048\sqrt{L_0 H_{0rms}} \quad \text{(2-20)}$$

which means that the set-up on natural beaches will raise the mean water level at the beach face by approximately 40% of the H_{rms} offshore.

Hanslow and Nielsen (1993) also found the beach slope to be important but only on steeper beaches:

$$B_s = 0.45\sqrt{L_0 H_{0rms}} \tan\beta_F \quad \text{for } \tan\beta_F > 0.06 \quad \text{(2-21)}$$

However, there is a discrepancy in these formulas, as for tanβ=0.06, the maximum set-up calculated with Eq.2-20 and 2-21 is $B_s = 0.048\sqrt{L_0 H_{0rms}}$ and $B_s = 0.027\sqrt{L_0 H_{0rms}}$ respectively.

Other measurements of set-up on natural beaches have been compiled in Table 2-6. It can be appreciated that all these beaches are sandy and relatively flat.

Location	**Slope**	**Set-up**	**Author**
Torry Pines, California	tanβ=0.02	$\overline{\eta}_m = 0.17 H_{os} = 0.24 H_{orms}$	Guza and Thornton (1981)
Pacific coast of Japan	tanβ=0.017	$\overline{\eta}_m = 0.052 H_{os}^{0.8} L_{os}^{0.2}$	Yanagishima and Katoh (1990)
North Devon, UK	tanβ=0.014 to 0.025	$\overline{\eta}_m = 0.12 H_{orms}$	King *et al.* (1990)
Tideless beach on Georgian Bay	Flat profile (tanβ=0.015) with a steeper inshore bar (tanβ=0.031 to 0.047)	$\overline{\eta}_m = 0.19 H_{os} = 0.25 H_{bs}$	Greenwood and Osborne (1990)

Table 2-6. Wave set-up at shoreline measured on natural beaches.

Set-up at the shoreline on coarse-grained beaches

Powell (1990) measured the set-up at the shoreline induced in laboratory experiments, concluding that generally the degree of wave set-up is between 10-30% of the significant wave height, with a pronounced wave steepness dependency, and proposing the following relationship:

$$\frac{B_s}{H_s} = 0.31 - 3.5\frac{H_s}{L_m} \quad \text{(2-22)}$$

where B_s/H_s is the dimensionless set-up at the shoreline.

Run-up of waves

The wave run-up results in the mean height of the water table being above mean sea level, which has been termed as "over-height" of the water table, $\overline{\eta}$. The asymptotic value of this over-height, $\overline{\eta}_\infty$ or B_∞, has been correlated to the surf similarity parameter by Kang *et al.* (1994) based on from laboratory studies with regular waves, with the relationship:

$$\overline{\eta}_\infty = 0.62 \tan\beta_F \sqrt{H_i L_0} \quad \text{(2-23)}$$

where

$\overline{\eta}_\infty$ is the asymptotic inland watertable over-height

β_F is the beach-face slope

H_i is the wave height

L_0 is the deep water wavelength

When calibrated for field conditions (Nielsen and Kang, 1995) the relationship reads:

$$\overline{\eta}_\infty = 0.55 \tan\beta_F \sqrt{H_{0rms} L_0} \quad \text{(2-24)}$$

A relationship between maximum set-up and asymptotic over-height can be obtained by combining equations 2-20 and 2-21 with 2-24 to obtain, respectively:

$$\frac{\overline{\eta}_\infty}{B_s} = 11.46 \tan\beta_F \quad \text{(2-25)}$$

$$\frac{\overline{\eta}_\infty}{B_s} = 1.2 \text{ for } \tan\beta_F > 0.06 \quad \text{(2-26)}$$

Note that the discrepancy at the limit $\tan\beta_F$=0.06 raised with the formulas 2-20 and 2-21 is still present in formulas 2-25 and 2-26.

Role of permeability in groundwater

Based on laboratory experiments with regular waves on an equilibrium beach profile, Gourlay (1992) concluded that the higher the permeability of the sediment, the lower the wave-induced over-height of the water table. Gourlay (*op.cit.*) found the groundwater level for fine sand of low permeability to be almost equal to the run-up height.

This appears to contradict the empirical formulation, shown above, where the over-height depends only on the wave conditions and beach-face slope, and not on the permeability. However, if the beach slope is regarded as a consequence of the permeability of the beach, the influence of the latter parameter is indirectly included in the empirical equation.

Response of beach groundwater level to waves

Is the response of the groundwater level individual and rapid to individual waves? Several hypotheses have been drawn in this respect, Hegge and Masselink (1991) considering that the beach water table responds to a combination of:

a) Pressure forces created by breaking waves and transmitted through the sediment. If the sediment is completely saturated, the velocity of transmission will be that of the speed of sound in water. In this case the rise of the groundwater could precede the arrival of swash.

b) Water input. Although it is usually assumed that the infiltrating water will not cause a rapid fluctuation of the water table, the presence of a capillary fringe above the water table could cause the reversed Wieriengermeer Effect (Turner and Nielsen, 1997). Within this capillary fringe, the sediment pores approach 100% saturation, this moisture being held by capillary forces. The pore water pressure in the capillary fringe is lower than atmospheric pressure (whereas at and below the water table is at and greater than atmospheric pressure). Adding a small quantity of infiltrating water can make the water table rise through the sand by 0.3m in 15s (Gillham, 1984). Laboratory experiments (Nielsen *et al.*1988) have also shown the opposite: a rapid fall in the water table followed by a small amount of drainage.

Hegge and Masselink (1991) observed that pressure forces (a) are likely to dominate seaward of the seepage zone, whereas infiltration (b) is more important landwards.

2.2.5 Sediment transport

In order to study the physical processes involved in the sediment transport on mixed beaches, the first issue to consider would be the initiation of grain motion. This will have an important influence on the entrainment of the finer sediments into suspension and the bed-load transport of the coarser sediments. The ability of a certain flow component to transport sediment is mainly a function of the shear stresses it induces at the bed, Nielsen (1994). This bottom shear stress depends on the boundary layer formed just above the seabed and will be an important issue both for bringing the sediment into suspension and for the vertical distribution of the current. Lastly, the influence of seepage on both the initiation of motion and the boundary layer need to be considered. A review of all these issues has been given in Blanco (2001) and has been summarised in the flow chart given in Figure 2-6.

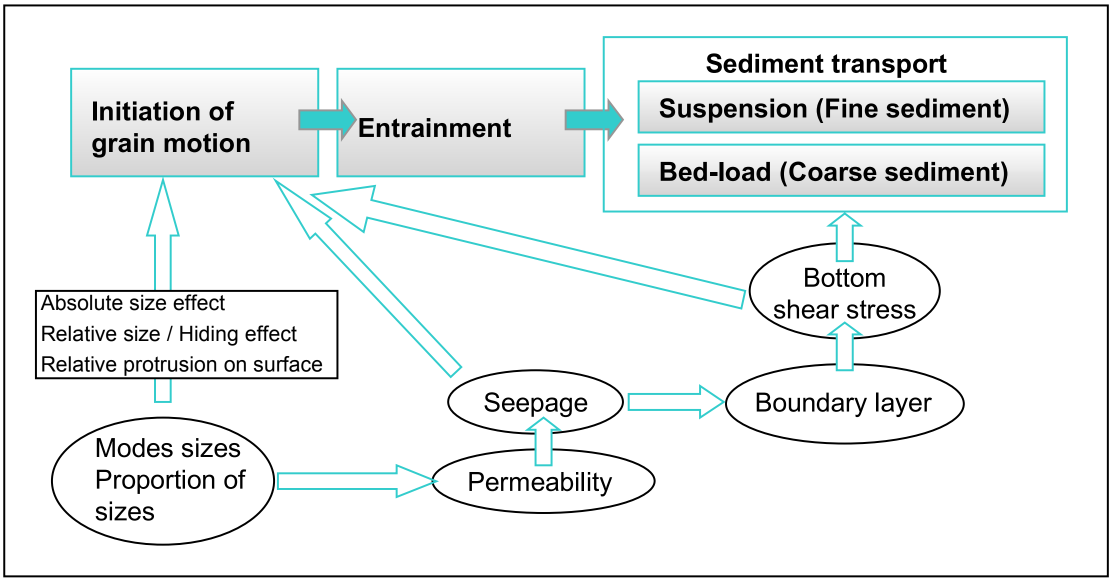


Figure 2-6. Sediment transport of mixed sediments.

The review undertaken here was devised as a mean of understanding the different findings on the processes to date and is not intended to be a study of the processes itself. It is clear that further detailed research is required to quantify these processes in the oscillatory flows of the surf and swash zones.

2.3 Existing predictive methods for mixed beaches

In order to develop a management strategy for a section of coastline, it is important to consider the likely response of beaches both under long term and storm conditions. The prediction of the profile development during storms can be carried out using either physical models or numerical models.

2.3.1 Physical models

Physical models are divided into flume and wave basin models, each providing valuable information. Available facilities and costs normally limit the use of physical models for shoreline management.

Mobile bed modelling at small scales is further limited by the necessity to work out a compromise between the requirements of the different processes that must be simulated.

When modelling any -gravel, sand- beach sediment the three main requirements to reproduce are, according to Powell (1990):

- Beach permeability;
- Threshold of sediment mobility; and,
- Relative onshore/ offshore movement.

(An additional requirement, not mention by Powell (*op.cit.*) would be bedload/ suspension load).

Although these requirements need to be satisfied, there are only two main characteristics the sediment particles have:

- Size,
- Specific gravity.

Therefore, some compromise is always necessary in the selection of the optimum characteristics for the model material. These factors are further complicated by the fact that there is only a limited range of specific gravities among the available material. Moreover, for sand beaches, an additional problem is the lower limit in size for non-cohesive material.

Although acceptable approaches to scaling narrow graded gravel beaches have been used for over twenty-five years (Powell, 1990), small scale modelling of beaches with a mixture of sand and gravel is limited because of the incompatibility of having both fractions within the same model.

2.3.2 Numerical models

Numerical approaches are broadly divided into parametric models, often derived from observed results in the laboratory or from field measurements, and physics based models which attempt to account explicitly for the main physical processes active across the foreshore. Each of these approaches has weaknesses and strengths, but none is able to simulate all of the important and complex processes influencing mixed beaches. Most numerical models have been derived for sand beaches and have then been extended to include a wider range of grain sizes. The main problems with respect to extending these models to mixed beaches are set out in Coates and Mason (1998) and Blanco *et al*. (2000), as follows:

- Assuming a simplistic description of beach sediment, usually defining the complete beach by a single D_{50} value or another simple parameter.
- Assuming that beaches do not vary in composition across-shore, along-shore, vertically or over time.

- Assuming an impermeable surface and ignoring flows within the beach face (infiltration and exfiltration).
- Finally, assuming a simple threshold of motion based on the defined grain size.

Predictive tools for longshore transport in use at present, both those based on numerical simulation of appropriate physical processes, Damgaard *et al.*, (1996) and those based on equations such as the CERC formula, Brampton and Motyka (1984) or Damgaard and Soulsby (1996), provide reasonable results. In contrast, cross-shore numerical models have had limited success – mainly due to a lack of knowledge of the governing physical processes and/or an inability to model the processes adequately. Hence no process-based model is available to predict the response of a coarse grained or mixed beach to a given hydrodynamic forcing.

HR Wallingford is currently working on the extension of OTTP-1D (one-dimensional swash zone model with a porous layer) towards a morphological capability (Clarke and Damgaard, 2002).

2.3.3 Parametric models

At present the coarse grained profile empirical or parametric models available are those of Powell (1990) known as SHINGLE, and BREAKWAT, by van der Meer (1988), based on extensive scaled laboratory flume tests (small scale with anthracite in Powell's and large and small scale with gravel in van der Meer's). The main factors influencing gravel beach profiles are, according to Powell (1990) and van der Meer (1988), wave height, wave period, wave duration, beach material and angle of wave attack.

Parametric models for mixed-grained beaches do not exist at the moment.

2.4 Summary and thesis hypothesis

In this Chapter, a review of the main differences of mixed beaches with respect to the relatively well known sandy and gravel beaches has been carried out. In summary, it can be concluded that the majority of these differences are ultimately due primarily to the permeability of the beach.

In fact, one could consider all beaches as an envelope of processes behaviour, so that the permeability is the cause of the spread of behaviour, Figure 2-7.

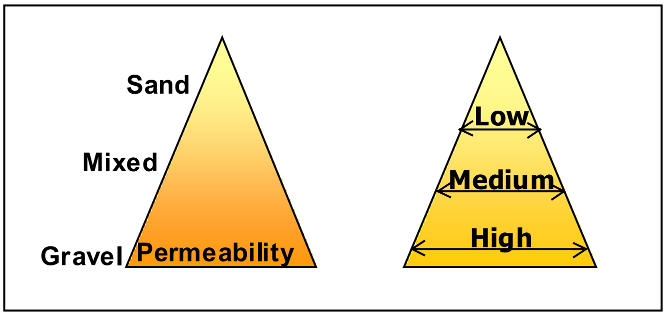


Figure 2-7. Beach envelope classification.

However, one has to keep in mind that permeability is not a "bulk" parameter for the whole beach and cannot be obtained from a single sample of beach sediment. This is especially important in the case of the mixed beaches where the measured permeability can vary by orders of magnitude depending on the cross-shore location of the sample and its depth within the beach.

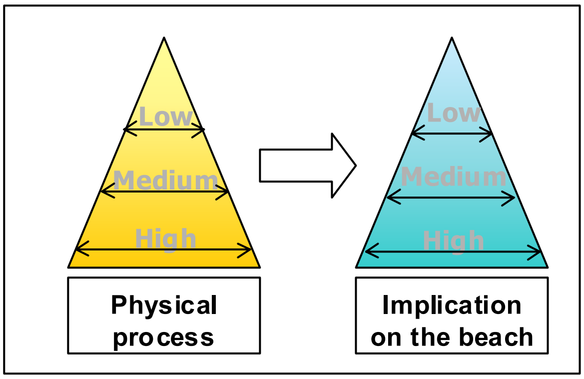


Figure 2-8. Colour-coding in Conceptual Model.

This is the hypothesis taken throughout the thesis and it will be the base for constructing the conceptual model, Figure 2-7, that will be presented in Chapter 10. The different blocks in the conceptual model are colour-coded according to what they represent, as indicated in Figure 2-8. The triangles are used to convey how each component varies with beach composition (permeability) i.e.: high, medium or low. Open arrows represent cause-effect relationships.

Chapter 3. CHARACTERISATION OF MIXED BEACHES ON THE UK COAST

In order to understand the dynamics of mixed sediment beaches on UK coastlines it is necessary, as a first step, to define the size and grading characteristics of such beaches. To assist in this, a brief review of the sediment samples from a number of UK mixed beaches, available at HR Wallingford, has been undertaken and is described in this chapter. Trends in the distribution across, along-shore and in depth were looked for.

3.1 Characterisation parameters

From this review, it became clear that most of the beaches found had a bimodal composition, one mode being in the sand fraction (<2mm) and the other in the gravel or shingle fraction (≥2mm). It then seemed appropriate to characterise the beaches according to this bimodality in contrast to a grading coefficient.

3.1.1 Sediment characterisation

According to the sediment properties of the bed a number of parameters are needed to characterise it:

- For so-called uniform sediments (standard deviation < 2.4) – the sediment can be characterised by the mean sediment size, D_{50}
- For non-uniform sediments (standard deviation > 2.4) – depending on the sediment distribution, the sediment can be defined as
    - Unimodal – the sediment bed is characterised by the mean sediment size, D_{50}, and the standard deviation, σ.

- Bimodal – the sediment bed is characterised by the mean sediment size of the coarse fraction, d_{50sh}, the ratio, f, of the mean sediment size of the coarse (≥2mm) fraction and the fine (<2mm) fraction, $f = d_{50sh}/ d_{50sa}$, and the percentage of sand in the mixture, %Sa.

3.1.2 Bimodality

Necessity of assessing bimodality

It is obviously expected that when trying to evaluate certain hydraulic parameters for sediment mixtures a dependency on the mixture itself would be found. At one extreme, the hydraulic parameter will be defined as that for coarse sediments whereas at the other extreme it will be defined as that for fine sediments. In between, it will depend on the parameters stated in the section above, which will give a bimodality index.

One of the problems encountered when attempting to assess the importance of grain size bimodality results from the difficulty of defining this property. The author has not found in the general statistical literature any parameter that defines the degree of bimodality of a given sample.

Bimodality parameters

Wilcock (1993) proposed a parameter to characterise the degree of bimodality of a sediment. This index of bimodality, B, is defined as

$$B = \left(\frac{D_c}{D_f}\right)^{1/2}\left(F_c + F_f\right) \qquad (3\text{-}1)$$

where

D_c and D_f are the grain sizes of the coarse and fine modes, respectively

F_c and F_f are the proportion of sediment contained in the coarse and fine mode, respectively. Each mode width is one phi unit, ϕ, according to the Wentworth scale where $\phi = -\log_2 d$; d in mm. (This definition is the reason for $F_c+F_f \neq 1.0$). Figure 3-1 shows a representation of the modes in a bimodal mixture.

The square root of ratio of the size of the two modes represents mode separation, the proportion of sediment in the modes helps to distinguish between mixtures with two prominent modes and those with a strong and a weak mode. Wilcock (*op.cit.*) concluded from flume and field data that only when B>1.7 did the bimodality of the sediment influence entrainment and transport properties. Thus sediments with a value of B<1.7 can be treated as unimodal.

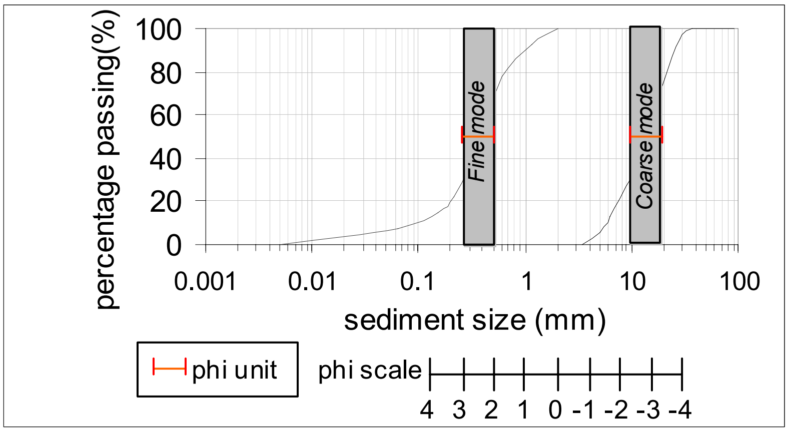


Figure 3-1. Representation of the modes in a bimodal mixture according to Wilcock (1993).

Sambrook Smith *et al.*(1997) derived another factor in order to account for the relative magnitude of the modes:

$$B^* = \left|\phi_2 - \phi_1\right| \frac{F_2}{F_1} \quad \text{(3-2)}$$

where F is the proportion contained in each mode of size ϕ.

The subscript 1 refers to the primary mode, and in the case both fractions were equal,to the coarser. Therefore:

- For a dominant coarse mode:

$$B^* = \left|\phi_f - \phi_c\right| \frac{F_f}{F_c} \quad \text{(3-3)}$$

- For a dominant fine mode:

$$B^* = \left|\phi_f - \phi_c\right| \frac{F_c}{F_f} \quad \text{(3-4)}$$

Sambrook Smith *et al.*(1997) refers to 1.5-2.0 as the cut-off for unimodal/bimodal sediments The author decided to use the index given by Wilcock (1993) for its simplicity and because it was also used and referred to in his experiments.

3.2 Sediment samples review

A total of 158 sediment gradings were considered, sampled from the following beaches, shown in Figure 3-2, in the years indicated in brackets:

- Hengistbury Head (1979) and Highcliffe (1993) in the South
- Hastings (1989) and Isle of Sheppey (2000) in the Southeast
- Hunstanton (1997) in the East

For each sediment distribution, the following values were calculated:

- values of the D_{50} for the gravel and sand fractions, $D_{50\ gravel}$, $D_{50\ sand}$,
- sand percentage, %Sand
- ratio, f ($f=D_{50\ shingle}/\ D_{50\ sand}$),
- proportion of sediment contained in each mode, P_{gravel} and P_{sand}
- the parameter of bimodality B and
- standard deviation for each fraction, σ_{gravel} and σ_{sand}.

These values can be found in Blanco (2001).

The statistical results of this review are summarised in Table 3-1 in terms of D_{50} of each fraction, sand percentage and f ($f=D_{50\ shingle}/\ D_{50\ sand}$).

	$D_{50shingle}$ (mm)	D_{50sand} (mm)	f	B	Sand %
Average	21.73	0.35	52.53	3.12	15.89
Mean	16.50	0.30	52.53	3.09	15.89
Standard deviation	9.94	0.07	28.00	1.79	17.57
Maximum	57.17	0.72	198.82	11.62	96.00
Minimum	2.36	0.07	3.29	0.79	0.00

Table 3-1. Statistical summary of the sediment samples review.

Sand distributions are, in general, uniform and symmetrical, whereas shingle distributions are usually less uniform and slightly skewed. The overall mean diameter for the sand fraction is 0.3mm and 16.50mm for the shingle fraction. Sand content is highly variable, covering the range from 0 to nearly 100%. No definite trends could be found in the sediment distribution along, across-shore and in depth.

Further examples of mixed beaches taken from along the Welsh coast show the proportion of coarse gravel and cobbles is much higher than those shown here.

Figure 3-2. Map of location of beaches with sediment samples reviewed.

Hunstanton Beach

Samples were taken in 1997 along two different profiles both of the surface and 150mm below it.

The distributions show a clear bimodal distribution of sediment for all of the samples (only two have a B<1.7 due to the fact that the coarse mean fraction is quite small).

Only one sample does not contain sand at all, the sand content in general being quite high, at least 20%, its mean being 38%. There does not seem to be a tendency for sand content to change with depth or alongshore.

The D_{50} for the sand is of the order of 0.35mm and is not very well sorted (average σ_{sand} = 1.8), whereas for the gravel D_{50} is 9.7mm, being better mixed (average σ_{gravel} = 2.5). No clear trend can be seen in the mean fraction distribution over the profile or in depth.

Isle of Sheppey

Samples were taken during October 2000 along a profile on Sheppey Beach along which 4 pits were dug. Samples were taken at the surface, and at 0.5, 1.0 and 2.0m depths. A clear bimodal distribution was found for all the profiles (all the values for B are well above 1.7).

The sand content of the samples is usually small, with an average of 14%. This sand percentage increases with depth and decreases from crest to toe of the beach. The sand median size is 0.32mm, with a standard deviation of the order of 2.2. The gravel median size is around 14.5mm (average standard deviation of 1.8), the coarser materials being found in the surface, diminishing in size with depth. No clear trend is seen in the distributions along the profile.

Hastings Beach

A total of 9 samples were taken from an undisturbed section of the beach midway between groynes 52A and 53 in 1989, along a line perpendicular to the seawall. Three different positions along the profile were analysed, with samples taken at the surface, and at depths of 0.5-0.6m and 0.8-1.1m.

A clear bimodal sand/gravel structure can be seen for all the samples (B is always greater than 1.7), where the sand percentage is usually quite small, with an average of 18%. The sand content increases from crest to toe and from surface to deeper positions. The D_{50} for the coarse fraction is 12.1mm (average standard deviation of 2.5), and coarser material is found at greater depths and from toe to crest of the beach. The sand fraction median value is 0.24mm, with a smaller standard deviation (1.8).

Highcliffe Beach

A very extensive sampling procedure was carried out at Highcliffe during 1993 as part of a MAFF funded study on Recharged beaches (Coates and Bona, 1997). A total of 105 samples were studied along 6 different profiles. The first 3 profiles were located East of groyne H1, at the middle of groyne bay 1-2 and west of groyne 2, whereas the second 3 profiles were located East of groyne H7, at the middle of groyne bay 7-8 and west of groyne 8. A total of 5 positions were sampled (1: crest at surface, 2: mid-beach, at surface, 3: mid-beach at sub-surface 0.5m, 4: toe, at surface and 5: toe, at sub-surface 0.5m). Sampling was carried out during January, March, April and June.

Most of the samples show gravel only content, but when the sand is present, its fraction is quite variable, from 11% to 75%. Sand content is usually zero at the surface (except for a proportion of 74% in April in one of the samples). Sand percentage at the subsurface varies from 0% to 57%, its average value being 18%.

Mean gravel sediment size is 22.7mm, the beach getting coarser from toe to crest as well as from West to East. The mean sand sediment size is 0.4mm, being quite uniform. No specific trend in time can be seen for any of the parameters

Hengistbury Head

Data from sampling in April and September 1979 at Hengistbury was found in terms of D50 for the sand and shingle components as well as the percentages. A total of 8 samples were summarised, four being located at the MLWS level and four at MHWS.

The mean for the shingle fraction is around 15mm but quite variable, whereas the sand mean is around 0.29mm. The proportions have a mean at MLWS of 84% sand-16% shingle and at MHWS 31% sand-69% shingle.

3.3 Typical mixed beach

Although a good number of samples along different profiles and at different depths was analysed, a clear trend could not be found. One of the reasons is the fact that positions alongshore in different profiles are not usually comparable, because of the different baselines adopted. Moreover, the samples are usually taken, for practical reasons, high in the beach, with few samples at the toe of the beach where there is usually a thin layer of gravel over a sand beach.

General trends can be outlined, but it must be noted that variability is very high. In general, mixed beaches along the UK coast can be said to have the following characteristics, (see Figures 3-3 and 3-4):

1. They can be characterised as bimodal, with one mode in the sand fraction and another one in the gravel. Wilcock's bimodality parameter B is always found to be greater than 1.7 and quite variable, with values as high as 11.

2. There is generally a predominance of gravel content, being of the order of 80%[2].

3. Sand content usually increases with depth, of the order of 10% more at about 1.5m deep below the beach-face.

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[2] Note that Figure 2-4 shows permeability for 20% sand reduced by a factor of 10 depending on sand size.

4. The gravel mode usually covers a wide range of sizes from about 5 to 30mm, the D_{50} varying from beach to beach between 10 and 20mm. $D_{84/16}$ for the gravel is of the order of 4.5.

5. The deeper samples usually contain coarser gravel.

6. The size range of sand is usually narrower, covering a range between 0.250 to 0.5mm, the D_{50} being of the order of 0.3mm. $D_{84/16}$ of the sand fraction is of the order of 3.5.

7. The sand fraction is more uniform than the gravel one, its distribution not being very variable in depth or along the profiles.

8. Also, there is a very great longitudinal, transverse and temporal variability. An idea of this variability is shown in Figure 3-5.

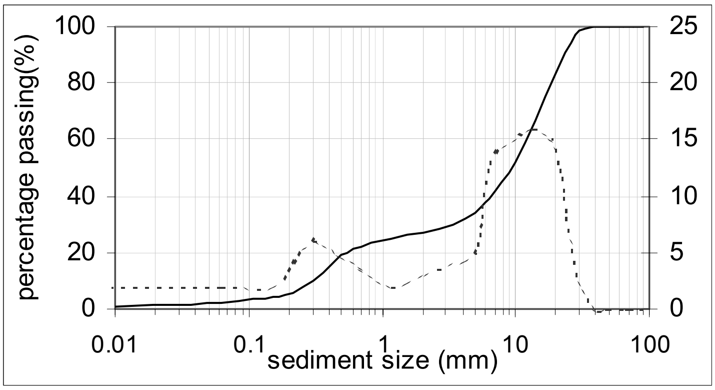

Figure 3-3. Mixed Beach Characteristics - Summary.

(Full line shows the cumulative sediment distribution, with the percentages given in the left-hand axis; dotted line shows the sediment size distribution with the percentages given in the right-hand axis).

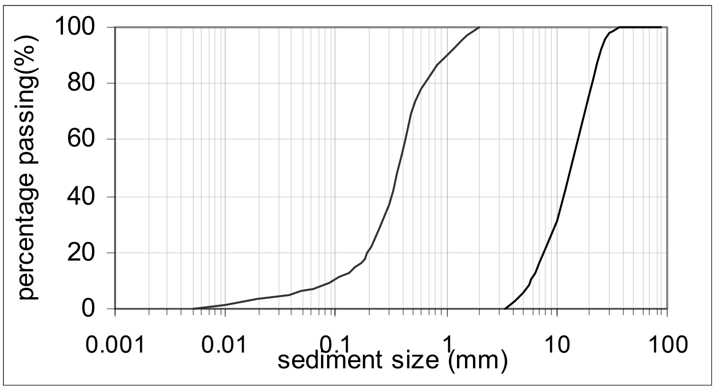

Figure 3-4. Typical sediment distribution for each mode or fraction of a mixed beach.

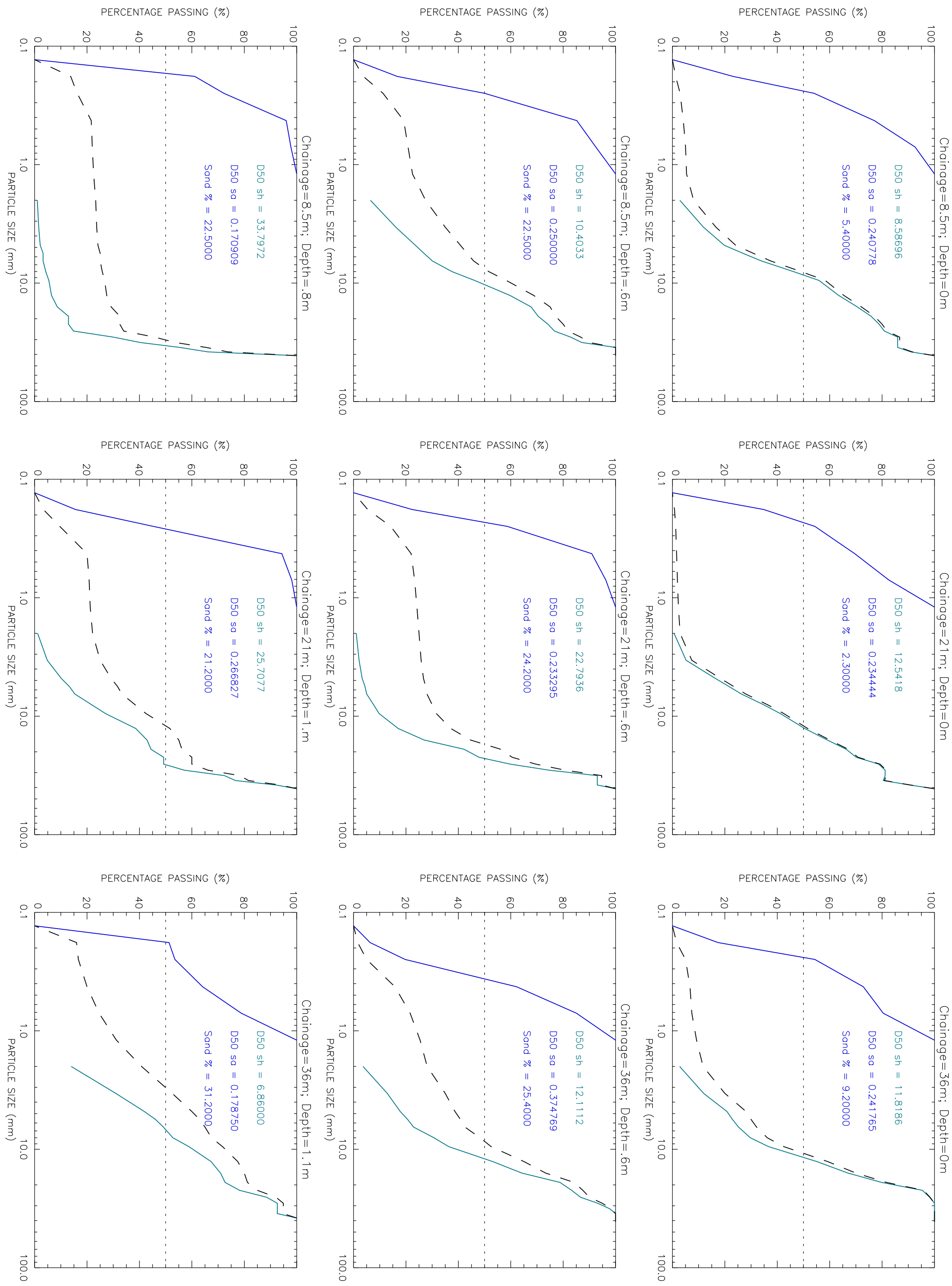

Figure 3-5. Example of longitudinal, transverse and vertical variability. Hastings, UK. (Solid lines: grading of fine and coarse fractions; dashed line: total grading).

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