9. MORPHODYNAMIC RESPONSE OF SHINGLE AND MIXED SAND / SHINGLE BEACHES IN LARGE SCALE TESTS – PRELIMINARY OBSERVATIONS

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Abstract

Full-scale physical model investigations have been conducted in the Large Wave Channel (*GWK*) of the Coastal Research Centre (*FZK*), Hanover. The morphodynamic response of both shingle and mixed sand with shingle beaches have been tested under random wave conditions. Results have been used to examine whether significant scale effects are present in small-scale model investigations of shingle beaches.

9.1 Introduction

Shingle (*or gravel*) and mixed sand / shingle beaches are widespread in many parts of Europe: they are particularly prevalent in the southeast of England. These beaches are highly efficient dissipators of wave action and they can provide excellent natural or managed defence systems. Shingle beaches are particularly efficient since their high permeability enables energy loss through percolation within the beach.

Beach management schemes within the UK are becoming increasingly frequent and beach recharge is regularly implemented, using both shingle and mixed sand with shingle. Annual expenditure on such schemes within the UK typically exceeds 60 million euro per year. As a result there is significant demand within the shoreline management community for robust predictive beach management tools. There is, perhaps surprisingly, extremely little guidance available to the engineer on the design and performance of such beaches.

Some limited guidance is available for the assessment of the cross-shore profile response of shingle beaches (*Powell (1990), van der Meer (1989)*). Inadequate field data is available to provide calibration or validation of the shingle models, however. Pure shingle beaches are relatively scarce when compared with mixed beaches, but predictive tools are non-existent for mixed beaches; this is probably reflective of the difficulty of scale model testing of such beaches. No well-developed tools are available for prediction of cross-shore profile response in storm conditions. This is a particular problem for beach managers, since the hydraulic discharge methods used to construct beach recharge schemes cause artificial mixing of the recharge sediments, thereby reducing the beach permeability and resulting in an unpredictable initial profile response. Although natural beach sorting will eventually occur, as the beach becomes reworked in the natural environment, the initial beach response may be unfavourable and result in accelerated beach erosion arising from reflections from the low permeability beach. The reduced permeability of mixed beaches presents complications that cannot be dealt with at small model scales. No empirical frameworks have previously been developed for these beaches and the programme discussed in this paper is believed to be the first to investigate the response of these beaches in a systematic manner.

9.2 Beach characterisation

The composition characterisation of coarse-grained shingle and mixed sand with shingle beaches is highly variable. Shingle beaches are characterised by coarse material with a mean grain size >6mm: the sediment size distribution is normally unimodal but can be highly variable. Steep slopes characterise the surface emergent section of the beach; this shape is governed, to some degree, in response to the high permeability of the beach in combination with wave action. By contrast, mixed sand and shingle beaches comprise a mixture, in which sand fills the interstices between the shingle; this results in a significantly less permeable beach and consequently the dynamic equilibrium beach slope tends to be somewhat flatter than on a shingle beach, for similar hydrodynamic conditions. The grading is typically characterised by a bimodal distribution of sand and shingle (***Figure 9-1***).

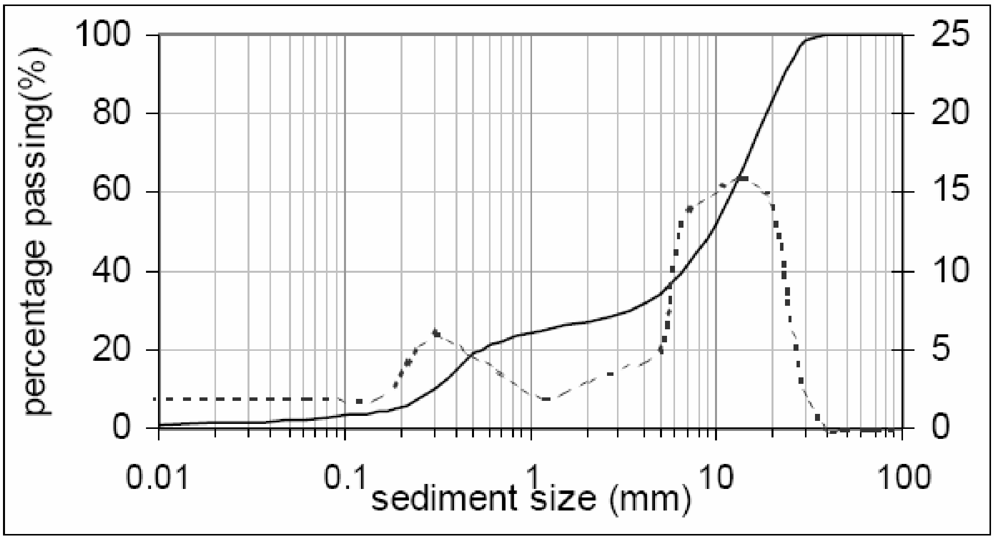

Figure 9-1. Typical bimodal distribution for a mixed beach (from Blanco et al 2002)

Field experiments (*Mason and Coates, 2001*) have indicated that the morphodynamic response of a mixed beach becomes very similar to a sand beach when the sand composition reaches 40% of the total material. The conceptual classification of beach composition and permeability suggested by Blanco *et al* (*2002*) is shown (***Figure 9-2***).

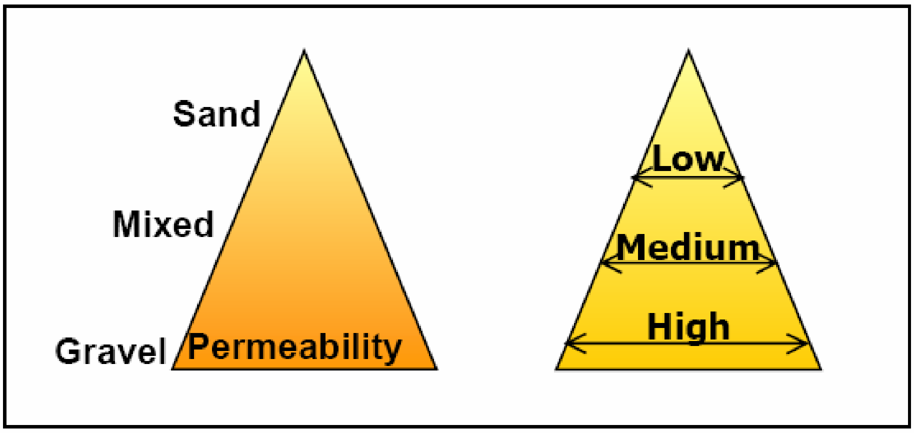

Figure 9-2. Classification of beaches – conceptual variation of permeability with sediment composition. (from Blanco et al 2002)

9.3 **Previous investigative approaches to modelling coarse sediment**

Advances in predictive techniques for the management of shingle beaches have largely been confined to parametric predictive models developed from small-scale model tests (*Powell, 1990; van der Meer, 1989*) and more recently to physics based numerical models (*Clarke et al, 2002*). Both approaches require validation. There are two main approaches to scaling of these beaches in physical models. The first uses Froudian similitude when the material is scaled geometrically; this typically results in a reduced model permeability and a consequent flattening of profile response, relative to full scale. The alternative approach is based upon a technique originally developed by Yalin (*1963*), in which lightweight sediments with distorted geometry are used to represent the sediment. The theoretical techniques outlined below provide a modelling solution based upon independent solution of three key performance criteria.

(i) Permeability

Beach slope is governed by permeability, and indirectly by grain size. A method of scaling of shingle beaches, which allowed both the correct permeability and drag forces to be reproduced in the model, has been described (*Yalin, 1963*). This approach suggests that the percolation slope must be identical, in both model and prototype, to ensure that the permeability is reproduced correctly, in an undistorted model:

$$J = \frac{\mathrm{k}(\mathrm{Re_v})\mathrm{v}^2}{gD_{10}} \qquad \text{[E1]}$$

Where:

J = percolation slope
k = permeability, $k = f(Re_v)$
$\mathrm{Re_v}$ = voids Reynolds number, vD_{10}/υ
υ = water velocity through the voids
D_{10} = 10% undersize of the sediment
v = kinematic viscosity
λ = Froude scale ratio

For identical percolation slopes, in the model and at full-scale, this gives:

$$\frac{\lambda_v{}^2 \lambda_k}{\lambda_D} = 1$$

Where λ is the model scale (*full-scale/model scale ratio*). Assuming that the model is operated according to Froude's Law then $\lambda_v{}^2 = \lambda$ (*the geometric scale*) so that:

$$\frac{\lambda \lambda_k}{\lambda_D} = 1 \qquad \text{[E2]}$$

As permeability is a non-linear function of Re_v, Yalin (*1963*) proposed a steady-state flow law and generated a recommended curve of *k* against Re_v. This curve can be approximated by the expression:

$$Log\, k = 3.17 - 1.134 Log\, \mathrm{Re}_v + 0.155 Log^2\, \mathrm{Re}_v \qquad \text{for the range } 1 < \mathrm{Re}_v < 200$$

With such a non-linear expression, the scaling law will depend on the representative value of the full-scale (*prototype*) permeability. If this is designated k_p and the Reynolds number is Re_p then:

$$\lambda_k = \frac{k_p}{k_m} = \frac{\lambda_D}{\lambda} \quad \text{and} \qquad \lambda_D = \frac{\lambda k_p}{k_m}$$

Now, $k_m = k\,(Re_m)$. Where Re_m is the model Reynolds number, so

$$k_m = k(\frac{\mathrm{Re}_p}{\lambda_v \lambda_D}) = k(\frac{\mathrm{Re}_p}{\lambda^{1/2}\lambda_D})$$

By substituting this expression, the implied equation for λ_D is obtained as

$$\lambda_D = \lambda k_p / k(\mathrm{Re}_p \big/ \lambda^{1/2}\lambda_D) \qquad \text{[E3]}$$

Assuming that k_p, Re_p and the form of the function $k(Re_v)$ are known, then this equation can be solved by successive approximation - to define the particle size for the model sediment, which will achieve the correct permeability within the model.

(ii) Onshore/offshore movement

It has been postulated that the relative tendency for sediments to move onshore or offshore depends upon the dimensionless parameter $H_b/W_T\,T_{m,}$ (*van der Meer, 1988*). In this expression, H_b is the breaking wave height, T_m is the mean zero crossing period and W_T is the settling velocity of the sediment particles. If $H_b\,/W_T\;T_m < 1$, then sediment moves onshore; if $H_b\,/\,W_T\,T_m > 1$, offshore movement occurs. In physical terms, the parameter represents the ratio between wave height and the distance which the sediment particle can settle during a single wave period. Therefore, for the correct reproduction of the relative magnitudes of onshore and offshore movement, the model scales must be such that:

$$\frac{\lambda_{Hb}}{\lambda_{WT}\lambda_{Tm}} = 1$$

Under Froudian model scaling $\lambda_T = \lambda^{1/2}$; assuming that the beach slope is modeled correctly then, $\lambda H_b = \lambda$, which gives $\lambda_{WT} = \lambda^{1/2}$

The general form of the settling velocity is given by:

$$W_T^{\,2} = \frac{4gD(\rho_s - \rho_f)}{3C_D\rho_f}$$

where ρ_r and ρ_f are the specific gravities of the sediment and the fluid, respectively, and C_D is the drag coefficient for the settling particles.

For modeling purposes,

$$\lambda_{WT} = \frac{\lambda_D^{1/2}\lambda_\Delta^{1/2}}{\lambda_{CD}^{1/2}} = \lambda^{1/2} \text{ and } \lambda_\Delta = \frac{\lambda\lambda_{CD}}{\lambda_D} \qquad \text{[E4]}$$

where Δ is $(\rho_s-\rho_f/\rho_f$

C_D is also a non-linear function; in this case, a function of the sediment particle Reynolds number ($Re_T = W_T D/\upsilon$). Thus, the actual scaling depends upon a typical value used for the prototype drag coefficient. Denoting this prototype value as C_{DP} and applying the appropriate Reynolds number Re_{TP} ,

$$\lambda_{CD} = \frac{C_{D_p}}{C_{D_m}} = \frac{C_{D_p}}{C_D(\mathrm{Re}_m)} = \frac{C_{D_p}}{C_D(\mathrm{Re}_p / \lambda_{WT}{}^{1/2} \lambda_D)}$$

$$\lambda_{CD} = \frac{C_{D_p}}{C_D(\mathrm{Re}_p / \lambda^{1/2} \lambda_D)} \qquad \text{[E5]}$$

If C_D and Re_p are known and λ_D has also been determined (*for example from the permeability scaling*) then equation [*E5*] can be solved for λC_D. The derived value can be inserted then into equation [*E4*] to derive ρ_s, the specific gravity of the model sediment. If both model and prototype sediments are coarse-grained (*i.e. greater than 4mm*) then $\lambda C_D \sim 1$ giving $\lambda_\Delta \sim \lambda/\lambda_D$

(iii) Reproduction of the threshold of motion

Komar and Miller (*1973*) have proposed that, for sediment sizes above 0.5mm and under oscillatory flow, the threshold of movement is defined by the expression

$$\frac{U_m{}^2}{\Delta g D} = 0.46\pi\left(\frac{d_o}{D}\right)^{1/4}$$

where U_m is the peak value of the near-bed wave orbital velocity at the threshold of motion, and d_o is the near-bed orbital diameter. Since the mean diameter in both the model and prototype materials lie above 0.5mm, this method would seem to be appropriate for application to shingle size sediment.

Since $U_m = \pi d_o/T_m$, this expression can be rewritten as

$$\frac{U_m{}^{7/4}}{(\Lambda D^{3/4} T^{1/4})} = 0.46\pi^{3/4} g$$

The maximum near bed orbital velocity, to the first order, is given by

$$U_m = \frac{\pi H}{T_m Sinh(2\pi d / L)}$$

where L is the wavelength

Substituting this expression and rearranging gives the threshold, in terms of wave height and period, as:

$$\frac{H^{7/4} A^{7/4}}{(\Delta D^{3/4} T^2)} = \frac{0.46g}{\pi}$$

where A is the depth attenuation factor (*1/Sinh (2πd/l))*

For the appropriate modelling of the threshold of motion, therefore, the following expression is given

$$\frac{\lambda_H^{7/4} \lambda_A^{7/4}}{(\lambda_\Delta \lambda_D^{3/4} \lambda_T^{\ 2})} = 1$$

In a Froudian model $\lambda_H = \lambda_L = \lambda_d = \lambda$ and $\lambda_T = \lambda^{1/2}$; therefore, $\lambda_A = 1$, which gives:

$$\lambda_\Delta \lambda_D^{3/4} = \lambda^{3/4} \qquad \text{[E6]}$$

Solution of each of the criteria results in conflicting results for the density and size grading of the model beach material for any scale, except unity, and the modelling solution is therefore an approximation. When combined some relaxation of the rules is required to arrive at a practical, but flawed, modelling solution. Permeability is undoubtedly extremely important and controls internal flow within the beach. The limited range of lightweight materials presents the physical modeller with a series of compromises when modelling shingle at small scale. Doubt has been cast upon the validity of the lightweight modelling approach, but the empirical model derived from this technique (*Powell, 1990*) is widely used in the design and management of shingle beaches. Particular concerns have been expressed at the rate of evolution of the dynamic equilibrium profile of the beach, wave run-up and also the evolution of the key beach descriptors, such as the crest, the step and the base of the profile.

Powell's parametric model provides a predictive framework based upon a series of empirically derived dimensionless equations used to identify a defined set of profile descriptors. Although some limited attempts have been made to validate this modelling approach in field investigations (*Bradbury, 1998*), the complexity of installation of field instrumentation (*Blanco et al, 2002*) and the lack of ability to control conditions within a systematic framework mean that large scale modelling provides the best possibility for the investigation of the morphodynamic response of these complex systems.

9.4 **Project aims**

The key project aims are stated below.

- Improve the understanding of morphodynamic and hydrodynamic processes on gravel and mixed sand / shingle beaches through the use of large scale physical model tests.Provide validation of existing parametric models of shingle beach profile response based on small scale model tests
- Provide calibration data for hydrodynamic numerical models of wave beach interaction
- Examine internal flow patterns within shingle and mixed beaches
- Develop acoustic techniques for measurement of bed motion of coarse sediments

- Develop predictive management tools
- Disseminate information within shoreline management community

This paper is confined to examination of the morphodynamic response of the beach, and validation of the small-scale model impacts of the beach response. Complementary investigations were also conducted to examine the internal flow within the beach (*Blanco et al 2002*) and also development of hydrodynamic numerical modelling techniques (*Clarke et al 2002*). Investigations were also conducted to develop the used of acoustic techniques to quantify bed motion of coarse sediments.

9.5 **Model layout and test programme**

Powell's (*1990*) predictive parametric model of profile response has formed the basis of the design of the large-scale experimental programme. The full-scale model framework has been developed to replicate some of the small-scale (*1:20*) tests conducted by Powell. (***Table 1***). Experiments were conducted of two model test sections at a scale of 1:1. The test programme provided the opportunity for comparison of full-scale model tests with small-scale tests of shingle beaches and for direct comparison with the response of mixed beaches

The experiments were carried out at the Grossen Wellen Kanal (*GWK*), The flume is 342m long, 7m deep and 5m wide, with a 1:6 permanent asphalt slope, over which the sediment was placed to an initial 1:8 slope, and to a minimum depth of sediment of 2m. The test section is shown in Figure 3. The beaches were allowed to develop naturally following initial construction: no regrading was undertaken following initial construction. Full details of the model test procedures are presented in Blanco and Holmes (*2002*).

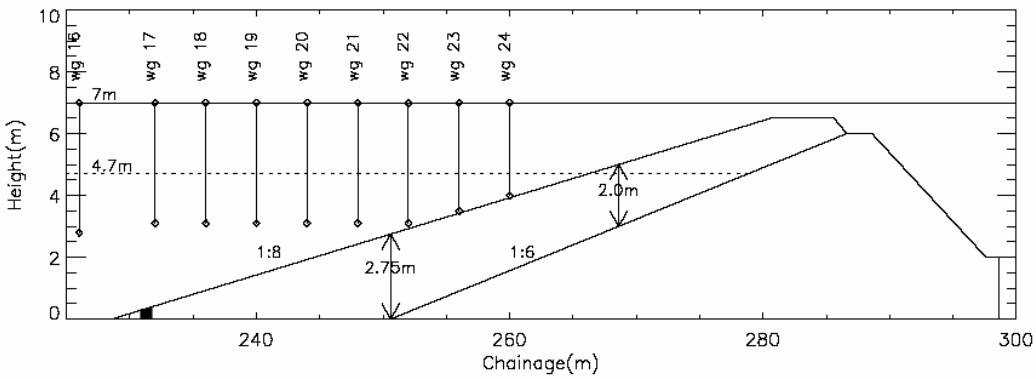

Figure 9-3. Model test section (from Blanco et al 2002)

The model test sections were constructed with materials described below.

- • Shingle Beach: 16- 32 mm narrow grading, with a D_{50} =21mm; and,
- • Mixed Beach: bimodal mix of shingle (*as above*) and sand, with a D_{50} = 300µm.

The percentage of sand in the mixture was around 30%. The sediment was thoroughly mixed both prior to testing.

The experimental programme was replicated on both beaches; this comprised a total of 5 wave spectra, with significant wave height ranging from 0.6m to 1.1m, wave steepness (H_s/L_m) from 0.01 to 0.05. Tests were run in a series of sequenced batches of waves, described by the same wave spectra, but of different sequence length time- series and duration; this enabled the profile development to be investigated incrementally with time.

Table 1. Experimental Core Programme

Beach	Test	Hs (m)	Tp (s)	H/L (-)	Sequencing of Batches (number of waves)						
					A	B	C	D	E	F	G
Gravel	Test 1 & 1R	0.6	3.22	0.05	50	100	500	1000	1500	3000	
Gravel	Test 2	1.0	4.14	0.05	50	500	1000	2000			
Gravel	Test 3	1.2	4.48	0.05	50	500	1000	2000			
Gravel	Test 4	1.0	5.29	0.03	50	500	1000	2000	3000		
Gravel	Test 5	1.0	7.74	0.015	50	500	1000	2000	3000		
Mixed	Test 1 & 1R	0.6	3.22	0.05	50	100	500	1000	1500	3000	4500
Mixed	Test 2	1.0	4.14	0.05	50	500	1000	2000	3000		
Mixed	Test 3	1.2	4.48	0.05	50	500	1000	2000	3000		
Mixed	Test 4	1.0	5.29	0.03	50	500	1000	2000	3000		
Mixed	Test 5	1.0	7.74	0.015	50	500	1000	2000	3000		

Experimental procedures included measurements of wave conditions along the flume, profile response, surf and swash velocities, internal pressures, run-up, and sediment distributions.

9.6 Test results

Preliminary observations made during the test programme indicate that the crest evolution occurs very rapidly, initially, particularly on the Shingle test section. The crest approaches a dynamic equilibrium elevation after approximately 3000 waves. A significant proportion of the profile evolution occurs after a period of 500-1000 waves. Evolution appears to occur at a similar rate to the small-scale model tests (***Figure 9-4***); these results were consistent for all conditions tested. Although the run-up crest elevation was typically slightly higher than the SHINGLE model suggests, small-scale tests appear to provide an adequate description of the beach response for this variable.

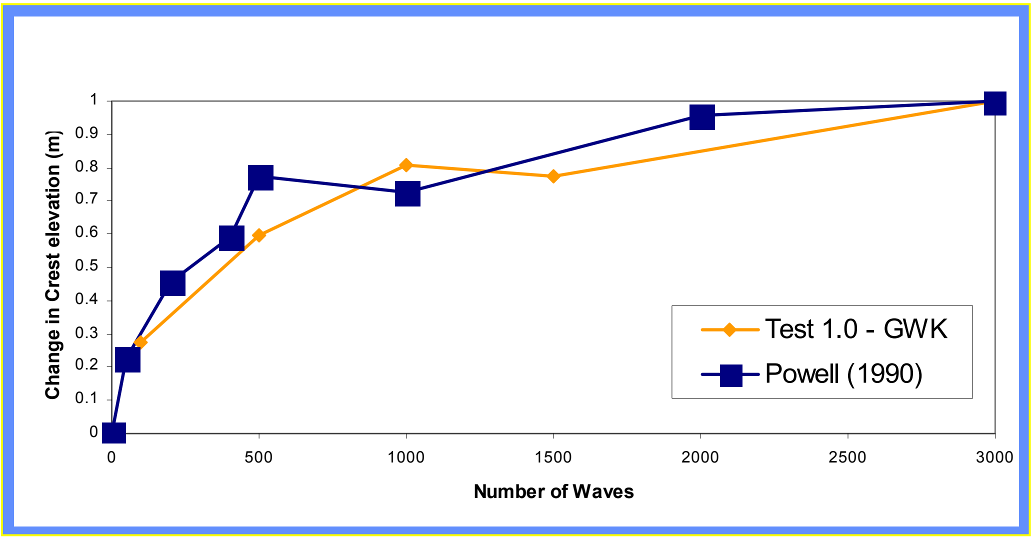

Figure 9-4. Typical run-up crest evolution on the shingle test section

Comparisons of measured shingle test section profiles with the SHINGLE model (*Powell, 1990*) generally indicate that the small-scale models have provided a reasonable simulation of profile response. Certain of the variables are better represented in the lightweight sediment model than others. Typical responses are shown for one hydrodynamic condition (***Figure 9-4***); these are compared also with the mixed beach (***Figure 9-4***), which provides a somewhat different response. It is clear that the crest elevation is lower and the location is further to landwards on the mixed beach than the shingle beach, for comparable hydrodynamic conditions; this has major design implications for beach recharge solutions. Currently some designers revert to tools developed for shingle beaches to aid the design of mixed beach recharge schemes. It is clear that such application of shingle models to a mixed beach situation will result in under-prediction of the location of the crest position relative to still water level.

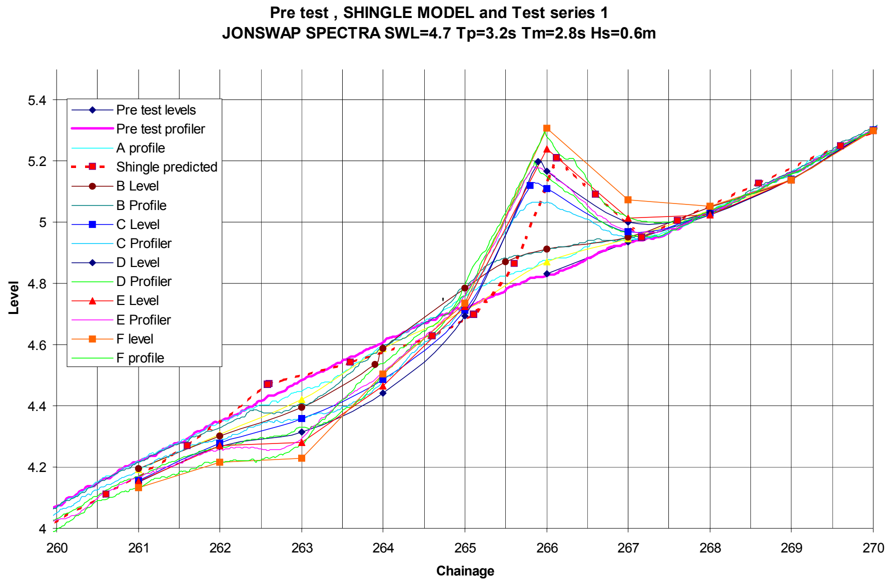

Figure 9-5. Evolution of shingle beach profile and comparison with SHINGLE model.

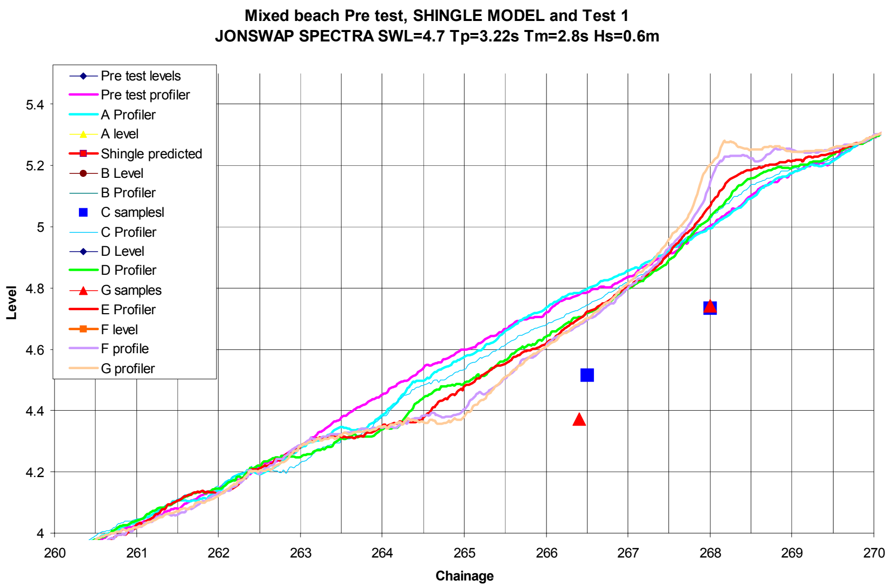

Figure 9-6. Evolution of mixed beach profile

The comparison between the full-scale shingle test data and the SHINGLE model (***Figure 9-5***) is generally favourable. Isolation of each of the predictive variables suggests that certain of the parameters are better represented than others for the shingle beach. In particular the shingle profile- base and step are less well reproduced than the crest. These variables are highly susceptible to the last few waves prior to measurement however; this observation is consistent with those made by Powell (*1990*). Although the crest elevation is slightly under predicted, the crest position appears to be well represented in the SHINGLE model.

By contrast to the shingle model tests, the response of the mixed beach presented less certain results and the small quantity of tests make conclusive observation difficult to ascertain. Model effects appear to be present, resulting in uneven profile response from one side of the flume to the other; this is thought to be a function of differential beach settlement and reflection from flume walls, accentuated by the low beach permeability. The test grading condition adopted is not necessarily representative of any real sites; these are highly variable in terms of grading width, sand composition and D_{50} grain size. It is clear that further development of empirical predictive solutions are required for mixed beaches.

9.7 Conclusions

This paper presents preliminary findings of large-scale investigations of the response of shingle and mixed beaches to wave action. Early indications suggest that the methodology previously adopted for small scale testing of shingle beaches is adequate to describe cross-shore profile response under normally incident wave conditions. Considerable further analysis is required to develop predictive techniques for mixed beaches, but there is currently inadequate data available to develop a robust empirical profile response model for such beaches.

9.8 References

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9.9 **Acknowledgements**

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