Influence of coastal structures on equilibrium beach

M. González, R. Medina
Ocean & Coastal Research Group. Departamento de Ciencias y Técnicas del Agua y del Medio Ambiente. Universidad de Cantabria. 39005 Santander, Spain.

J.J. Muñoz-Pérez
Departamento de Física Aplicada. Universidad de Cádiz. CASEM, Polígono Río S. Pedro, s/n. Puerto Real, Cádiz, Spain.

ABSTRACT

An equilibrium beach profile model for a beach affected by a coastal structure is presented. The model is based on the well-known energy flux approach proposed by Dean (1977). The effect of the structure is taken into account by considering the modification that the structure generates on the wave energy flux. The model is then applied to several cases that are usually found along the littoral, namely a perched beach and a reef-protected beach. Several field and laboratory data are used to analyze the merit of the proposed model for describing the equilibrium condition of a beach profile affected by a coastal structure. A good comparison is obtained.

1 INTRODUCTION

It is well known that coastal structures may modify both beach shoreline and beach profile. Actually many coastal structures are built in order to change the existing shoreline configuration and to generate a broader dry beach. Several authors have studied the shoreline response due to offshore breakwaters using a numerical approach (e.g. Hanson and Kraus 1989, 1990 and Gravens et al., 1991) or an empirical approach (e.g. Hsu and Silvester 1990, Gonzalez and Medina 1999). However, the effect of coastal structures on the modification of the beach profile has received much less attention.

Investigators have long recognized that beach profiles can be complex and may exhibit series of bars and troughs. However, in overall form they tend to be concave upwards and have a progressively decreasing slope as the water depth increases in the offshore direction. This regularity has inspired several attempts to develop mathematical expressions to describe the profile shape.

One of the most important approaches used for the determination of profile shape is that of the equilibrium profile of beaches. In a broad sense, the equilibrium beach profile is the result of the constructive and destructive forces acting in a beach profile. The hypothesis behind the equilibrium beach profile is that beaches respond to wave forcing by adjusting their form to an equilibrium or constant shape attributable to a given type of incident wave or sediment characteristic.

Various expressions have been proposed over the years (see González et al 1997 as a general reference). The most widely used formulation, very simple and easy to apply, is the 2/3-power profile shape proposed by Bruun (1954) and Dean (1977). Both authors concluded that the beach profile shape could be adequately represented by:

$$h = Ax^{2/3} \qquad (1)$$

where h is the total water depth, A is a dimensional shape parameter that depends on the grain size (Moore, 1982) and x is the horizontal distance from the shoreline. However, none of the presently proposed equilibrium beach profile expressions are able to adequately represent the interaction between the beach profile and a coastal structure.

The aim of this paper is to develop a general formulation of equilibrium for a beach profile affected by a coastal structure. First the fundamentals of the model are presented. The starting point of the model is the well-known energy flux approach proposed by Dean (1977). The effect of the structure is taken into account in the energy flux balance. Different hypotheses are made in order to consider different kind of coastal structures. The modification of the energy flux balance yields a new expression for the beach profile configuration. The model is then applied to several cases that are usually found along the littoral, namely a perched beach and a reef-protected beach. All the fitting parameters needed in the developed expressions are calibrated using field and laboratory data.

2 FUNDAMENTALS OF THE MODEL

Several approaches have been pursued in an attempt to characterize equilibrium beach profiles. One of the most popular is to consider the time-averaged wave energy flux equation for straight and parallel contours:

where F is the net shoreward energy flux per unit width and ε is the energy dissipation rate per unit area. An extension of equation (2) for cases in which refraction or diffraction processes are important can be found in Gonzalez et al (1997).

Equation (2) involves three variables, namely: the wave height, H, the water depth, h, and the energy dissipation, ε. Several authors have used equation (2) to develop a model for the wave energy dissipation by means of field or laboratory data of wave height and water depth (e.g. Battjes and Janssen 1978, Thornton and Guza 1983, Dally et al 1985). Many numerical models take advantage of equation (2) to solve the wave height modifications during its propagation cross-shore, given the water depth and an energy dissipation model. In the equilibrium beach profile problem one seeks the water depth and, consequently, an appropriated energy dissipation model and wave height variation across the profile must be provided. The key point of the present work is that the equilibrium beach profile of a beach affected by a coastal structure can be obtained if the adequate energy dissipation model and wave height variation across the profile is used in the energy flux balance.

3 PERCHED BEACHES

Introduction

The basic concept of the perched beach is to reproduce the existing profile to some convenient seaward point and then intersect this profile with a submerged toe structure to retain the beach in a perched position, see Figure 1.

Since the main hypothesis behind the equilibrium beach profile model is that the wave energy flux dissipated along the beach profile, the equilibrium beach profile for a perched beach will depend on the amount of energy flux that is transmitted over the toe structure. Consequently, the design of perched beaches requires an understanding of how waves are reflected at the submerged structure and how they are dissipated as they travel across the structure-top.

A large body of literature is available on wave transformation on a reef breakwater (see Ahrens (1987) as a general reference). An important conclusion of these studies, related with the stability of the perched beach, is that, although the submerged breakwater may induce wave breaking, it occurs well beyond the breakwater crown (Grilli et al, 1994).

An estimation of the structure width for the waves to break on the structure can be obtained from Gourlay (1994). In this work, Gourlay (1994) studied the wave transformation of waves approaching a fringing reef with a steep face and outer reef-top slope gently decreasing in the landward direction. From Gourlay's results it can be concluded that, at least, at a distance from the edge of the reef, l, of $l \approx T\sqrt{gd}$, the wave height can be greater than the incoming wave and the wave energy flux can exceed the stable value of wave energy flux given by the constant breaker-to-depth ratio $\gamma = 0.8$ for that particular depth, d. Furthermore, the breaking process will take a distance (one or two wave lengths) to reduce this wave energy flux to a stable value. This result agrees with Muñóz et al (1998) field data which show that for a natural reef-protected beach to exist, the reef width must exceed three wave lengths.

According to these studies it is clear that in wide natural reefs, wave breaking over the reef limits the amount of energy reaching the beach profile and it is the most important factor affecting the beach profile shape (Muñóz-Pérez, 1996). In most of the man-made structures, however, wave breaking over the structure can be neglected since the breakwater crest width is usually much smaller than the wave length and breaking occurs at the perched beach. Furthermore, frictional damping over the breakwater can also be neglected and, consequently, wave reflection at the breakwater is the main process that determines the beach profile shape in a perched beach.

Energy Flux Balance

In order to solve equation (2) for a perched beach, the beach profile is divided into three regions as shown in Figure 2.

Region 1 is the offshore part of the profile. In this area Dean's (1977) hypothesis of constant breaker-to-depth ratio and uniform wave energy dissipation per unit water volume is assumed. According to Dean's (1991) work, a beach profile located in front of a seawall can be adequately represented by a profile extending behind the seawall as if the seawall did not exist. In other words, the reflection of the seawall, F_{er}, does not modify the beach profile shape. Consequently, beach profile in region 1 is not affected by the submerged structure and can be determined using the virtual x-origin (see figure 1). This profile defines the water depth at the seaward side of the structure, h_e, and also the amount of energy flux reaching it, F_e (see Figure 2).

Region 2 is the breakwater domain. This region is defined by the breakwater crest width, B, the water depth over the breakwater, d, and the water depth at the seaward and leeward, h_e and h_i respectively. The above mentioned parameters (B, d, h_e, h_i) determine the energy flux, F_i, which reaches the perched profile.

Region 3 is the leeward part of the profile. In this domain Dean's (1977) hypothesis, stated in region 1, is also assumed. Consequently, the perched beach profile shape in region 3 is defined by the parabolic equation (1).

Notice that the perched beach profile is completely defined if h_i is given which, due to the constant breaker-to-depth ratio, can be determined if F_i is known. In order to calculate the value of F_i, the energy balance in region 2 must be solved:

where F_{er} is the wave energy flux reflected by the structure.

Assuming linear shallow wave theory, constant breaker-to-depth ratio and that only the oscillatory (non-breaking) part of the wave contributes to the reflected flux of energy, Baquerizo (1995), equation (3) can be written as:

where R is the reflection coefficient ($R = H_r / H_e$), H_e is the incoming wave and H_r is the reflected wave height.

The solution for the reflection coefficient, R, for the wave propagation over an impermeable step problem, can be found in several previous works (eg. Losada et al. (1992)). The main characteristics of the solution are also showed in Figure 3, where the reflection coefficient, R, is plotted as a function of the dimensionless breakwater crest width, B/L, for different values of the dimensionless water depth, d/h_e. In this figure, the well-known feature of wave resonance is clearly observed. Due to this phenomenon, the reflection coefficient goes to a value close to zero for some particular sets of parameters (d/h_e, B/L).

In order to solve equation (4) an iterative process must be carried out, in which a first estimation of h_i is specified to solve the reflection coefficient. This reflection coefficient is then used to determine a new value for h_i.

Using the above mentioned procedure values of the water depth ratio h_i/h_e are plotted in Figure 4 versus the dimensionless water depth, d/h_e, for different breakwater crest widths, B/L. From Figure 4 it can be concluded that for dimensionless water depths d/h_e greater than 0.5 minor benefits are achieved with the construction of a submerged breakwater ($h_i \sim h_e$). A considerable reduction in h_i/h_e is obtained for d/h_e less that $d/h_e < 0.1$. In this area, however, resonance effects may modify this picture resulting in an inefficient structure.

Model Validation

Several lab data sets can be used in order to analyze the merit of equation (4) for describing the equilibrium condition of a perched beach. Chatham (1972) carried out two-dimensional studies to determine the amount of sand which would be lost seaward over the submerged toe structure by normal and storm wave action. The model beach was subjected to test waves until equilibrium was reached for a wide range of wave conditions. The values of parameters tested are listed in Table 1. Calculated values of the ratio d/h_i are also presented in Table 1.

Sorensen and Beil (1988) conducted two-dimensional experiments of perched beach profile response to storm waves. Five test cases were investigated. The first consisted of a nourished profile without a toe structure. The remaining four cases were perched beach conditions with a sill located at various depths. A complete list of the parameters studied are listed in Table 2. Calculated values of the ratio d/h_i are also presented in Table 2. From Tables 1 and 2, it can be concluded that equation (4) gives an adequate estimation of the equilibrium water depth at the breakwater for a perched beach.

h_e(m)	d (m)	h_i (m)	T (s)	L(m)	B(m)	B/L	d/h_e	d/h_i (measured)	d/h_i (calculated)
5.72	3.43	5.55	7.0	50.2	2.3	0.046	0.600	0.610	0.606
8.23	5.72	8.19	7.0	55.8	2.3	0.041	0.695	0.698	0.697
7.10	4.60	7.00	7.9	60.9	2.3	0.038	0.648	0.657	0.651
6.28	3.43	6.20	10.0	75.2	2.3	0.031	0.546	0.553	0.551
7.43	6.63	7.35	10.0	81.1	2.3	0.028	0.892	0.902	0.892
6.29	3.43	6.10	16.0	123.6	2.3	0.019	0.545	0.562	0.548
7.43	7.20	7.35	16.0	133.9	2.3	0.017	0.969	0.980	0.969

h_e(m)	d (m)	h_i (m)	T (s)	L(m)	B(m)	B/L	d/h_e	d/h_i (measured)	d/h_i (calculated)
0.187	0.091	0.18	1.6	2.06	0.06	0.029	0.490	0.506	0.494
0.140	0.046	0.14	1.6	1.81	0.06	0.033	0.330	0.330	0.340
0.130	0.023	0.12	1.6	1.80	0.06	0.033	0.180	0.192	0.205
0.140	0.001	0.07	1.6	1.81	0.06	0.033	0.007	0.015	0.016

4 REEF-PROTECTED BEACHES

Introduction

There are many locations in which the entire beach profile is not sand rich and areas of hard bottom or mud are encountered (e.g. coral reefs, perched barriers). Many characteristics and informative details about these kinds of beaches, which will be denoted as reef-protected beaches, have been previously studied (see Muñoz-Pérez 1996 as a general reference). In a special way, wave breaking and wave attenuation over submerged horizontal shelves have been considered (Horikawa and Kuo, 1966; Gourlay, 1994; Nelson, 1994).

It is well known that the spilling-wave breaking assumption with a constant wave height to water depth ratio, γ, is not adequate for waves breaking on a shelf. Horikawa and Kuo (1966), computed theoretical curves that have a consistent agreement with experimental data in the case of wave transformation on a horizontal bottom. The ratio between the local wave height and the mean water depth decreases from 0.8, at the initial wave breaking point, to become almost constant, about 0.5, in the inner zone.

Several wave-decay expressions have been proposed (e.g. Dally et al, 1985; Andersen and Fredsoe, 1983). Fredsoe and Deigaard (1992), for example, gave the following exponential decay:

where h_r is the water depth over the reef, H is the wave height and l is inshore distance from the edge of the shelf (see Fig. 5). From eq. (5) it can be concluded that the wave height that reaches the sandy beach toe, which is located at the depth h_r, is less than the wave height that would reach that particular depth in a beach without the hard shelf. Consequently, the total amount of energy that has to be dissipated by the sandy profile is minor.

Energy Flux Balance

The beach profile form of a reef-protected beach can be determined by means of Larson and Kraus's (1989) derivation of the equilibrium profile, taking into account the available wave energy at the toe of the beach and assuming the dissipation model of Dally et al (1985). The resulting beach profile form will be given by an expression similar to eq. (1). However, for the same grain size, the profile shape parameter for a reef-protected beach will not be the same as the A value used in the usual Dean equilibrium profile in eq. (1) due to the shelf wave-decay dependence.

A simple relationship between the shape parameter for reef-protected beaches, hereafter denoted as A_{rp}, and non-reef-protected beaches can be obtained considering that the energy flux, Ec_g, at h_r must be dissipated along the beach profile in both cases.

Assuming linear wave theory and that eq. (1) is valid along the entire profile, it yields:

where H is wave height, W is the total length of the profile and the subscript $(\ )_{rp}$ indicates the reef-protected beach (see Figure 6).

Since H_{rp} at h_r is less than H at the same depth, the total length of the profile for the reef-protected beach will also be less than the non-reef-protected beach and, consequently, the beach profile slope will be steeper.

Equation (7) can also be written in terms of the breaker-to depth ratio as:

EQUACION(8)

where Γ is the breaker-to-depth ratio for a reef-protected beach (e.g. equation (5)) and γ is the breaker-to-depth ratio in a non-reef-protected beach. For a wide shelf ($l \approx \infty$), typical values of Γ range between 0.55 to 0.35 (Nelson, 1994). Values of γ depend on beach slope and wave steepness, and have a wider range of variability. Kaminsky and Kraus (1993) compiled a large database of wave breaking parameters and showed that for typical field beach slopes (1/30 to 1/80) most of γ values are encountered in the range 0.65 to 1.1 with an average value of 0.79.

Introducing equation (1) in equation (8), a relationship between the shape parameters can be found as:

where A_{rp} is the shape parameter for the reef-protected beach and A is the non-reef-protected beach shape parameter.

Model Validation

Using the set of field data compiled by Gomez-Pina (1995), beach profile data from reef-protected beaches along the Spanish coast have been collected to verify the proposed model. Over 50 profiles from seven beaches have been. The main characteristics of these beach profile data are shown in table 3. It is noted that the values of A_{rp} listed in Table 3 have been determined by best fitting and the values of A by means of Moore's (1982) relationship.

The predicted values of A_{rp} using equation (9) and the best-fitted values listed in Table 3 are compared in Figure 7. The predicted values are computed using Fredsoe and Deigard's (1992) model for Γ. It is seen in Figure 7 that equation (9) provides a good representation of the beach shape parameter A_{rp}. The asymptotic best fit for a wide shelf ($l/h>60$) is $A_{rp}=1,48$ A which corresponds to a value of $W_{rp} = 0.56$ W.

5 CONCLUSIONS

This study has focused on the equilibrium shape of a beach profile affected by a coastal structure. The new shape has been obtained on the basis of the well-known energy flux approach proposed by Dean (1977). The main assumption of the present work is that the equilibrium beach profile of a beach affected by a coastal structure can be obtained if the adequate energy dissipation model and wave height variation across the profile is used in the energy flux balance. In this way the effect of the structure can easily be taken into account in the energy flux balance by considering the modification that the structure generates on the wave energy flux.

The model has been applied to a perched beach and a reef-protected beach. Several field and laboratory data have been used to analyze the merit of the proposed model for describing the equilibrium condition of a beach profile affected by a coastal structure. A good comparison has been obtained

For the case of the perched beach it is concluded that the maximum advance of the perched beach shoreline depends substantially on the dimensionless water depth. Minor advance is obtained for large d/h_e (>0.5) while a considerable advance is achieved for d/h_e less than 0.1. In this area, however, resonance effects induced by the structure may modify the final picture resulting in an inefficient structure depending on the value of B/L.

For the case of the reef-protected beach it is concluded that although the resulting beach profile form is similar to the one proposed by Dean (1977), the shape parameter is not the same as the A value used in the usual Dean (1977) equilibrium profile due to wave decay dependence. A simple expression has been proposed for the shape parameter A_{rp} for reef-protected beaches based on Andersen and Fredsoe's (1983) wave decay model.

ACKNOWLEGDEMENTS

This study has been partly funded by the European Communities in the framework of the Marine Science and Technology Programme (MAST), under the No MAS3-CT97-0081 project "Surf and Swash Zone Mechanics" (SASME). The field data have been kindly provided by the Dirección General de Costas (Ministerio de Medio Ambiente) and its support is hereby gratefully acknowledged

REFERENCES

ANDERSEN, O.H. and J. FREDSOE, 1983. Transport of suspended sediment along the coast. Progress Report No. 54 Inst. of Hydrodynamics and Hydraulic Eng. ISVA. Tech. Univ. of Denmark. pp. 33 - 46.

DALLY, W., R. DEAN and R. DALRYMPLE, 1985. Wave height variation across beaches of arbitrary profile. J. Geophys. Res., Vol. 90, No. C6.

FREDSOE J. and R. DEIGAARD, 1992. Mechanics of coastal sediment transport. Advanced Series on Ocean Eng. Vol. 3. World Scientific, 369 pp.

GÓMEZ-PINA, G., 1995. Análisis de perfiles de playa en las fachadas cantábrica y atlántica de la costa española y su aplicación a proyectos de regeneración. Universidad de Cantabria E.T.S.I.C.C.P. Dpto. de Ciencias y Técnicas del Agua y Medio Ambiente.

GONZÁLEZ, M., R. MEDINA and M.A. LOSADA, . Equilibrium beach profiles: effect of refraction and difraction. Proc. Coastal Dynamics=97. In pressGOURLAY, M.R., 1994. Wave transformation on a coral reef. Coastal Eng., 23: 17-42.

Beach	D_{50} (mm)	l (m)	h_r (m)	A ($m^{1/3}$)	A_{rp} ($m^{1/3}$)
Ondarreta	0.33	200	4.50	0.12	0.18
Sta. María	0.42	500	8.50	0.14	0.21
Torregorda	0.25	330	3.50	0.11	0.16
Victoria	0.32	470	4.00	0.13	0.20
Arroyo Hondo	0.25	750	5.30	0.10	0.15
Regla	0.25	380	3.50	0.11	0.16
Fuentebravía	0.27	740	5.50	0.10	0.15

HARDY T. and I. YOUNG, 1996. Field study of wave attenuation of an offshore coral reef. Journal of Geophysical Research. Vol. 101, No.C6 pp. 14311 - 14326.

HORIKAWA, K. and C.T. KUO, 1966. A study on wave transformation inside surf zone. Proc. 10th Coastal Eng. Conf. Tokyo, pp. 217 - 233.

KAMINSKY, G.M. and N.C. KRAUS, 1993. Evaluation of depth-limited wave breaking criteria. Porc. Ocean Wave Measurement and Anal., Waves 93. ASCE, pp. 180-193.

LARSON, M. and N.C. KRAUS, 1989. SBEACH: Numerical model to simulate storm-induced beach change. Army Corps of Engr., Waterway Expt. Station., Tech. Rpt. CERC-89-9.

MOORE, B., 1982. Beach profile evolution in response to changes in water level and wave height. M.S. Thesis, University of Delaware.

Ahrens, J.P., 1987. Characteristics of reef breakwaters. U.S. Army Corps of Engineers, Coastal Engineering Research Center, Tech. Rep. CERC-87-17.

Baquerizo, A., 1995. Reflexión del oleaje en Playas. Métodos de evaluación y de predicción. Ph. D. Dissertation, Universidad de Cantabria, Santander, Spain, 180 p.

Bruun, P., 1954. Coast erosion and the development of beach profiles. Beach Erosion Board, Technical Memorandum, Nº 44.

Chatham, C.E., 1972. Movable-bed model studies of perched beach concept. Proc. 13th Int. Conf. Coastal Eng. ASCE, pp. 1197-1216.

Dean, R.G., 1977. Equilibrium beach profiles: U.S. Atlantic and Gulf coasts. Department of Civil Engineering, Ocean Engineering Report Nº. 12, University of Delaware, Newark, Delaware.

González, M., R. Medina and M.A. Losada, 1997. Equilibrium beach profiles: effect of refraction. Proc. Coastal Dynamics 97 (in press).

Gourlay, M.R., 1994. Wave transformation on a coral reef. Coastal Eng. Elsevier, 23: 17-42.

Grilli S.T., M.A. Losada and F.L. Martín, 1994. Characteristics of solitary wave breaking induced by breakwaters. Journal of Waterway, Port, Coastal and Ocean Eng. Vol. 120, N. 1, pp. 74 – 92.

Losada, I.J., M.A. Losada and A.J. Roldán, 1992. Oblique incident wave past vertical thin barriers. Applied Ocean Research, Vol. 14, Nº 3, pp. 191-200.

Muñóz-Pérez J.J.,, 1996. Analisis de la Morfologia y la variabilidad de playas apoyadas en lajas. Phd Thesis (in Spanish)

Sorensen, R.M. and N.J. Beil, 1988. Perched beach profile response to wave action. Proc. 21st. International Conference on Coastal Eng. ASCE, pp. 1482-1491.

Battjes, J. A., and Janssen, J. P. F. M. 1978.Energy loss and set-up due to breaking of random waves. Proc. 16th Int. Conference on Coastal Eng. ASCE. Vol. 1, pp. 569-587.

Thorton, E. B., and Guza, R. T., 1983. Transformation of wave height distribution. Journal of Geophysical Research. Vol. 18, pp. 5925-5938.

Hanson, H., and Kraus, N.C. 1989. GENESIS: Generalized model for simulating shoreline change. *Technical Report CERC-89-19*, U.S. Army Engr. Wtways. Experiment Station, *Coast. Engrg. Res.* Ctr., Vicksburg, Miss.

Gravens, M.B., Kraus, N.C., and Hanson, H. 1991. GENESIS: Generalized model for simulating shoreline change, report 2, workbook and user's manual. *Technical Report CERC-89-19*, Report 2 of a series, U.S. Army Engr. Wtrways. Experiment Station, Vicksburg, Miss

Hsu, J.R.C., and Silvester, R. 1990. Accretion behind single offshore breakwater. *J. Wtrway., Port, Coast. and Oc. Engrg*., ASCE, 116(3), 367 - 380.

Gonzalez, M and Medina R. 1999. Equilibrium Shorelina behind a Single Off-Shore Breakwater. Proc. Coastal Sediments'99. ASCE. pp844-859.