

FORCES INDUCED BY BREAKERS ON PILES

by

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ABSTRACT

This paper consists of three parts. The first part presents a method for analyzing the forces exerted by breaking waves on a circular pile. The force consists of two components, a slowly varying force and a much larger but very short duration quasi-impact force (probably of the order of 1/100 of a second). The second part is concerned with breaker characteristics, with emphasis being given to the few field data that have been measured. The third part consists of a presentation of some available data on surf zone bottom profile variations with time.

Information on all three of these parts is needed for the proper design of a pile supported structure in the surf zone. If the bottom along the site of a proposed pier is sand, an estimate of the variability with time of the profile must be made. The effect of bottom depth and configuration on the height of waves moving shoreward, and the effect of this, in turn, on the wave loading is important in the calculations of wave-induced moments about the bottom. The ability of the structure to withstand these horizontal loads depends in part upon the depth of penetration of the piles. If the bottom varies with time, then calculations of wave characteristics and wave-induced loads on the piles should be made for appropriate bottom configurations.

INTRODUCTION

A large number of pile-supported structures have been built that extend from a beach through the surf zone (Simison, Leslie and Noble, 1978), and many more will be built in the future. The forces exerted on the piles and the ability of the structure to withstand these forces depend upon a number of factors such as pile shape and material, structure configuration, breaker type, beach configuration, tides, currents, and wave climate. The beach configuration changes with changing wave conditions, which further complicates the problem, as there is a relatively strong interaction between breaker characteristics and bottom configuration. Much work has been done in regard to the "design wave," but almost no work is available to the practicing engineer on the "design bottom profile." Both are needed for a well designed structure. In addition, the effect of possible local scouring at the piles or along the entire pier, must also be considered (U.S.

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BREAKER FORCES ON PILES

Only a few measurements are available of the forces exerted by breaking waves on piles located in the surf zone. These measurements show a slowly varying force similar to that exerted by a nonbreaking wave, together with a very short duration rather large force that occurs just as the wave breaks at the pile, Figure 1. Some field measurements of forces induced on a circular pile by breaking waves in the surf zone are given in papers by Snodgrass, Rice, and Hall, 1951; Morison, Johnson and O'Brien, 1953; Miller, Leverette, O'Sullivan, Tochko and Theriault, 1974. Results of laboratory studies have been given by Hall, 1957; Goda, Haranaka and Kitahata, 1966; Watanaba and Horikawa, 1974. It is difficult to measure this "impact" force reliably owing to its very short duration combined with the dynamic characteristics of the test system.

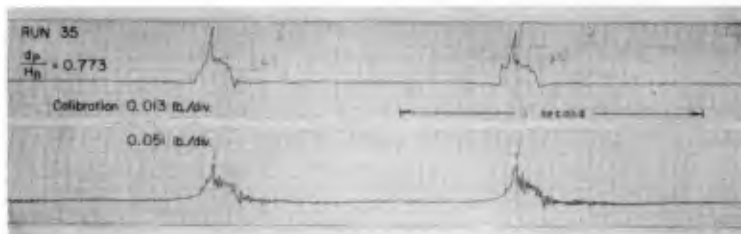


Figure 1. Force Record (From Hall, 1957)

The cause of the "impact" force can be understood by considering the analogy of the wave loading of a horizontal circular cylinder that is alternately in the air and then immersed by a passing wave crest (Kaplan and Silbert, 1976; Faltinsen, Kjaerland, Nottveit and Vinje, 1977; Sarpkaya, 1978). In this problem, one force term is $\dot{\eta}(\partial m_y / \partial t)$, where $\dot{\eta}$ is the time rate of change of the water surface in the vertical direction, and m_y is the vertical component of added mass. $\dot{\eta}(\partial m_y / \partial t)$ can be given by

$$\dot{\eta}(\partial m_y / \partial t) = \dot{\eta}^2 (\partial m_y / \partial y) \quad (1)$$

Here, $\partial m_y / \partial y$ is the change in the vertical component of added mass with immersion by the wave of the horizontal structural member. This term has been calculated by Kaplan and Silbert (1976) and is shown in Fig. 2 as a function of the relative immersion z/r , where r is the radius of the circular cylinder and z is the immersion, and ρ is the mass density of the fluid. Note for $z/r = 0$, just as the circular cylinder is first immersed, the coefficient has the value of π and then decreases rapidly.

Additional papers of interest in regard to the problem described above are Attfield, 1975, Dalton and Nash, 1976, and Kjeldsen and Myrhaug, 1979.

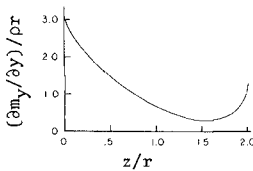


Fig. 2. $(\partial m_y / \partial y) / \rho r$ vs. z/r
(From Kaplan and Silbert, 1976)

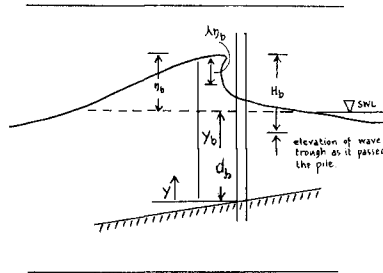


Fig. 3. Definition Sketch

The approach described above can be used to calculate the force due to a breaking wave acting on a vertical circular pile, using $\partial m_x / \partial x$ in place of $\partial m_y / \partial y$, where x is the horizontal coordinate and $\partial m_x / \partial x$ is the change in the horizontal component of added mass with immersion of the vertical pile by the breaking wave (Wiegel, 1979). One can calculate the horizontal component of the slowly varying wave-induced force using the Morison equation from $y = 0$ to $y = y_b - \lambda \eta_b$ (see Fig. 3). Then, one can compute the "impact" force on the section of the pile that is struck by the vertical or nearly vertical portion of the breaking wave (or, "foam" line or bore), $\lambda \eta_b$. (See the variation of dynamic pressure measurements with distance beneath the surface by Miller, et al., 1974 for the reason that this can be done).

$$F_s \approx \eta^2 (\partial m_x / \partial x) \approx \frac{1}{2} \rho C_s D C_b^2 \quad (2a), (2b)$$

Here, the maximum value of $C = \pi$, D is the diameter of the pile, C_b is the velocity at which the breaking wave crest moves, and F is the "impact" force per unit length of the pile subject to this force, $\lambda \eta_b$. This equation is of the same form as the one developed by Goda, Haranaka and Kitahata, 1966. (See, also, Watanabe and Horikawa, 1974). λ is a function of the

type of breaker. For the range of wave steepnesses used in these tests and bottom slopes of $1/100$ and $1/30$, λ was less than 0.1 . These were for spilling breakers. For plunging breakers on a steep slope ($1/10$), λ was as great as 0.5 . These values of λ were obtained indirectly, and the original papers should be consulted for details. Analysis of some field data by the writer showed that η_b/H_b ranged from 0.55 to 0.95 .

The interval of time during which C_s is about at its highest value is very short, with C_s decreasing from this value. Consider a design breaking wave height of 10 m, and a 1.0 m diameter pile. From other studies $C_b = 1.12 (2g \times 10)^{-1/2} = 15.7$ m/sec. The time it would take for the breaker to immerse completely the $\lambda \eta_b$ section of the pile would be $1/15.7$ sec (about 0.06 sec). The value of C_s would be decreasing from the initial value of π during this time. Thus, the "impact" force would occur during a short time, substantially less than 0.06 sec., probably of the order of $1/100$ sec. The more slowly varying wave-induced force and moment from $y=0$ to $y=y_b$ calculated using the Morison equation would probably be used by most design engineers to control the design. However, the effect of the large loadings of short duration on the fatigue life of the pile must be taken into consideration (see Attfeld, 1975 for a discussion of a similar problem).

It appears from the work of Snodgrass, et al, 1951; Hall, 1957, 1958; and Watababe and Horikawa, 1974, that the Morison equation is adequate for waves that have not yet broken, insofar as any individual pile is concerned. A wave breaking in the plunging manner at a pile induces a greater force in the $\lambda \eta_b$ portion of crest (see Miller et al, 1974). "Foam lines" (bores) resulting from plunging breakers exert a very large force on a pile in the same manner, Figure 4. Sketches are given in Fig. 5, which show the various types of breakers designated in Fig. 4. An example of breaking waves passing the test structure taken from the individual frames of a motion picture film is given in Fig. 6. Similar results were found in the field tests of Miller et al, 1974, for a vertical flat plate of small width and thickness.

BREAKER CHARACTERISTICS

Values of breaker characteristics are needed for the calculation of wave-induced forces on a pile. These are η_b , λ , H_b , d_b , and C_b (forward speed of the wave at time of breaking, or bore speed after breaking), together with the water particle velocities and accelerations within the wave.

One important parameter used in predicting breaker characteristics is the relationship between the type of breaker that occurs and the two primary factors; deep water wave steepness and the slope of the beach (Patrick and Wiegel). They observed this relationship during an extensive field study, followed by a hydraulic laboratory study. For later laboratory studies, see, for example, Calvin, 1968, Weggel, 1972, Battjes, 1974, Singamsetti and Wind, 1980. The most important factor has been found to be the ratio of beach slope to the deep water wave steepness:

$$\xi = \tan \theta / (H_o/L_o)^{1/2} \quad (3)$$

where θ is the angle between a plane beach and the horizontal, H_o is the deep water wave height, and L_o is the deep water wave length. This ratio has been given the name "surf similarity parameter." It is sometimes

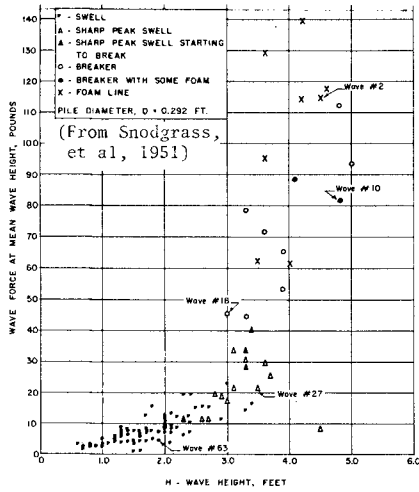


FIGURE 4. WAVE FORCE AT MEAN WAVE HEIGHT VS. WAVE HEIGHT

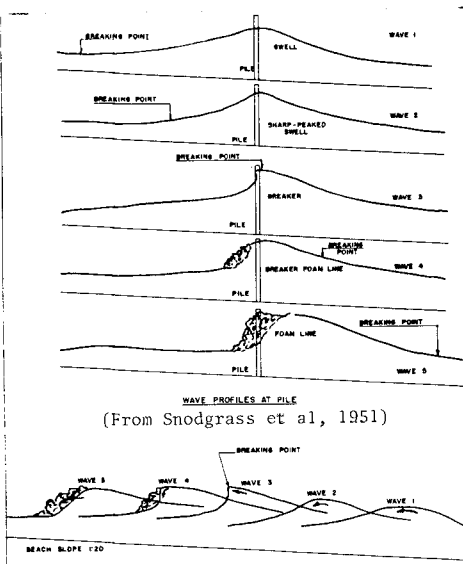


Fig.5. Breaker Type and Wave Transformation



FIGURE 6. - WAVE NO. 2 - FOAM LINE (From Chalmers et al.)

designated the "Iribarren Number," when used in regard to breaker types at a rubblemound breakwater. Numerical values of this ratio and their relationship to breaker type have been reported for plane rigid impervious beaches. A set of calculations has been made to show this relationship, and the results are presented in Table 1.

TABLE 1
Relationship Between Surf Similarity Parameter
(Iribarren Number)($\tan\theta/\sqrt{H_0/L_0}$) and Breaker Type. *

Beach Slope $\tan \theta$	1/5	1/10	1/15	1/20	1/25	1/30	1/40	1/50	1/75	1/100
Wave steepness H_0/L_0	0.2	0.1	0.0667	0.05	0.04	0.0333	0.025	0.02	0.0133	0.01
1/10 0.1000	.64	.32	.21	.16	.13	.10	.08	.06	.04	.03
1/15 0.0667	.77	.39	.26	.19	.16	.13	.10	.08	.05	.04
1/20 0.0500	.90	.45	.30	.22	.18	.15	.11	.09	.06	.04
1/25 0.0400	1.00	.50	.33	.25	.20	.17	.13	.10	.07	.05
1/30 0.0333	1.09	.55	.36	.27	.22	.18	.14	.11	.07	.05
1/35 0.0286	1.18	.59	.39	.30	.24	.20	.15	.12	.08	.06
1/40 0.0250	1.26	.63	.42	.31	.25	.21	.16	.13	.08	.06
1/45 0.0222	1.34	.67	.45	.34	.27	.22	.17	.13	.09	.07
1/50 0.0200	1.42	.71	.47	.35	.28	.24	.18	.14	.09	.07
1/60 0.0167	1.55	.78	.52	.39	.31	.26	.19	.16	.10	.08
1/70 0.0143	1.67	.83	.56	.42	.33	.28	.21	.17	.11	.08
1/80 0.0125	1.79	.89	.60	.45	.36	.30	.22	.18	.12	.09
1/90 0.0111	1.90	.95	.64	.48	.38	.32	.24	.19	.13	.09
1/100 0.0100	2.00	1.00	.67	.50	.40	.33	.25	.20	.13	.09
1/125 0.0080	2.24	1.12	.75	.56	.45	.37	.28	.22	.15	.11
1/50 0.0067	2.45	1.23	.82	.61	.49	.41	.31	.25	.16	.12
1/175 0.0057	2.65	1.32	.88	.66	.53	.44	.33	.27	.17	.13
1/200 0.0050	2.83	1.41	.94	.71	.57	.47	.35	.28	.19	.14
1/225 0.0044	3.00	1.50	1.00	.75	.60	.50	.37	.30	.20	.15
1/250 0.0040	3.16	1.58	1.06	.79	.63	.53	.40	.32	.21	.16
	Surging Breakers	Plunging Breakers						Spilling Breakers		

* Breaker classes shown are based upon the following assumption.

Surging Breaker $\tan\theta/\sqrt{H_0/L_0} > 2.0$
Plunging Breaker 0.4 to 2.0
Spilling Breaker < 0.4

Many laboratory measurements have been made of breaker characteristics, nearly all for simple plane bottoms, rather than for the more complex bathymetry of natural beaches. The design engineer needs data on more realistic models of bottom configuration, with tests being made with wave spectra as the input. An even greater need, however, is for field data of the type obtained by Suhayda, 1974, Thornton, 1979, and Wright, et al, 1979, 1982.

In nature, sand beaches are not plane, nor are they impervious,

nor are they uniform in the direction along the beach; in addition, the bottom configuration changes with changing wave conditions and tide range. In general, the combination of steep beaches and relatively flat waves (swell) result in a relatively highly reflecting wave condition (called a surging breaker). The combination of steep waves (local storm waves) and a relatively flat beach result in spilling breakers. Plunging breakers occur for intermediate values of the parameter.

As mentioned above, for the combination of a very steep beach and waves of relatively small steepness, much of the wave energy is reflected. The surging type of breaker is of this type, and the runup of the waves on the beach is relatively great, and the drawdown is fast. Often, edge waves are formed under these conditions, and these appear to be associated with the formation of cusps along the beach. For flat beaches and steep waves, the waves spill over a wide surf zone. Most of the wave energy is dissipated in this process, and the wave runup on the beach is rather small. Through a complicated process there is a set-down at and just seaward of the region of breaking, and a setup of the water level within the surf zone, landward of the breakers (for example, see Saville, 1962; Longuet-Higgins, 1970; Komar and Guaghan, 1972; Battjes, 1974; Jonsson and Buhelt, 1978; van Dorn, 1978). On ocean beaches this setup varies with time, exhibiting long-period oscillations known as surf beat (most commonly in the 60 to 150 second period range). Rip currents exist, as a part of a complicated littoral current system created by the breaking waves. Between these two types of breakers are the plunging breakers, which occur for intermediate combinations of beach slopes and wave steepnesses. Strong rip currents are also observed under these conditions. Owing to the complexity of natural beaches with one or more bars present, breakers which have plunged often run as a bore in water which is somewhat deeper landward of the location at which the wave breaks, with the bore dissipating most of its energy before finally moving up a small amount onto the beach face.

Three equations sometimes used to calculate breaker speeds, C_b , needed in Eq. 2 are:

$$C_b = K_1(2g\eta_b)^{1/2}, \quad K_2(g\gamma_b)^{1/2}, \quad \text{or} \quad K_3(2gH_b)^{1/2} \quad (4)$$

The writer analyzed the field measurements made by Snodgrass, et al, 1951. In these field tests a marker pole was set 6.04 m seaward of the test pile. The wave crest velocity was determined by counting the number of frames of the motion picture film for the wave crest to pass from the marker pole to the test pile (see Fig. 6). The film speed (in frames per second) was determined from a clock which appeared in the film. During the tests thirty-one waves broke at the test pile, or were foam lines (bores) or sharply peaked swell (these broke just landward of the test pile). The average value of the ratio of the measured wave crest velocity to $(2gH_b)^{1/2}$ was 1.12. The average value of the ratio of the measured wave crest velocity to $(g\gamma_b)^{1/2}$ was 1.14. Here, both H_b and γ_b were measured values at the test pile, obtained from the motion picture film. γ_b may be calculated for some beaches using the equation

$$\gamma_b = d_b + \eta_b \quad (5)$$

where d_b is the water depth at breaking and η_b is the height of the

crest of the breaking wave above the still water level. η_b was found to be approximately equal to $0.8H_b$ from another set of field measurements, Figure 7. The average value of η_b/H_b is in agreement with the laboratory data of Wiegel and Beebe, 1956, and van Dorn, 1978. Values of η_b are needed in Eq. 5, in the first of Eqs. 4, and in the application of Eq. 2 (to calculate $\lambda\eta_b$).

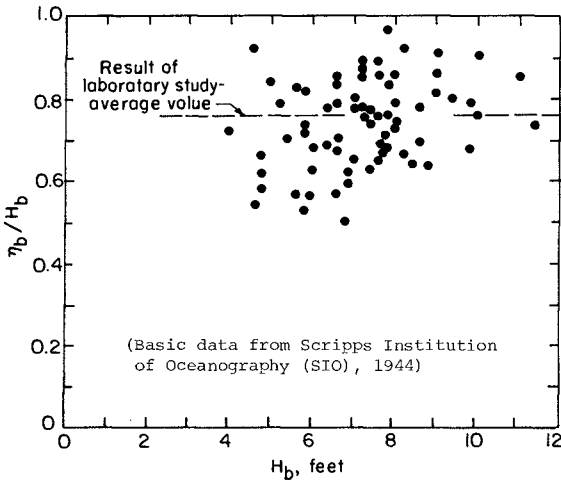


Fig. 7. η_b/H_b , Field Data, SIO Pier, CA.

The relationships among beach slopes, breaker heights, and the the water depths in which the waves break have been studied in the laboratory by a number of people, each of whom has developed a slightly different semi-empirical equation. The general forms of the equations used to predict breaker heights and the depth of water in which waves break are

$$H_b/H_o = A(\tan \theta)^B (H_o/L_o)^C \quad (6)$$

$$H_b/d_b = \gamma_b = D(\tan \theta)^E (H_o/L_o)^F \quad (7)$$

Here, d_b is the mean water depth at breaking (including setdown) and A, B, C, D, E and F are empirically determined coefficients. These, and other formulae have been compared by Singamsetti and Wind, 1980, for several beach slopes. They suggest the use of

$$H_b/H_o = 0.575 (\tan \theta)^{0.031} (H_o/L_o)^{-0.254} \quad (8)$$

with the range of validity being $0.02 < H_o/L_o < 0.065$ and $1/40 < \tan \theta < 1/5$. This curve is the "best fit" curve by Singamsetti and Wind through the other suggested curves. Note the relatively insensitivity to beach slope, except for very steep beaches $[(\tan \theta)^{0.031}]$ is 0.93 for $\tan \theta = 1/10$, 0.91 for $\tan \theta = 1/20$, 0.89 for $\tan \theta = 1/40$, and 0.87 for $\tan \theta = 1/100$.

The formula for H_b/d_b recommended by Singamsetti and Wind is

$$H_b/d_b = 0.937 (\tan \theta)^{0.155} (H_o/L_o)^{-0.130} \quad (9)$$

with the range of validity being $0.02 < H_o/L_o < 0.06$, and $1/40 < \tan \theta \leq 1/5$. They state that the results for the 1/5 slope differ considerably from this, and that the following formula is a better approximation for the flatter slopes

$$H_b/d_b = 0.568 (\tan \theta)^{0.107} (H_o/L_o)^{-0.237} \quad (10)$$

with the range of validity being $1/40 < \tan \theta < 1/10$. A formula similar to the one presented above (due to J.A. Battjes) is

$$H_b/d_b = 1.16 [\tan \theta / (H_o/L_o)^{1/2}]^{0.22} \quad (11)$$

Some measurements have been made of the breaker "bore heights" as the bores (also called "foam lines") move shoreward over the bottom which has a rather flat slope. These measurements were made over the shoreward 150 meters of a wide surf zone (about 400 to 500 meters) at a beach just west of the entrance to The Coorong in South Australia (east of Adelaide). The beach is composed of fine sand (mean diameter of 0.13 mm), with the bottom slope of the 150 meter wide section where the studies were being made, of about 1/100. The beach is exposed to long high-energy southwesterly swell. During the experiment the significant breaker height was about 3 meters and the wave period in the range of 12-15 seconds (hence, ξ - see Eq. 3- in the range of 0.085 to 0.11). The average spring tide range is only 0.8 meter and the neap tide range is only 0.4 meter. No rip cells or beach cusps were present during the experiment. The measurements of the bore heights (i.e., "foam lines" after breaking) were made in the shoreside 150 meter wide portion of the surf zone. The relationship between significant bore heights and local water depth, d , was measured and the data presented in the report. The average value of $\gamma (H_{bore}/d)$ was found to be only 0.42.

Van Dorn discovered a rather simple equation for the horizontal component of water particle velocity, u , along a vertical line directly under the crest of a wave just as it starts to break. It appears to be reasonably reliable for beaches flatter than 1/25. This equation is

$$u/C_b = 0.2 + \frac{0.10(y/y_b)}{1.125 - (y/y_b)} \quad (12)$$

A reasonable approximation for calculating C_b is either

$$C_b = 1.12(2gH_b)^{\frac{1}{2}} \quad (13a) \quad \text{or} \quad C_b = 1.14(gy_b)^{\frac{1}{2}} \quad (13b)$$

where H_b is the breaker height. y_b may be calculated for some beaches by Equation 5.

Much work has been done in the laboratory on the water particle velocity (see, for example, Hedges and Kirkgoz, 1981) and acceleration fields in breaking waves, while little field data are available (see, for example, Miller and Ziegler, 1964; Thornton, 1979; Stive, 1980). Space limitations prevent going into this problem in detail herein.

The main difficulty in applying the several equations presented above for breakers is that they have not been checked sufficiently by measurements of ocean waves with the variable bottom slopes that exist on ocean beaches. Furthermore, the laboratory studies have nearly all been for plane rigid impervious beaches, and these are not a good representation of natural beaches.

A very useful study is that of the field data measured at the Scripps Institution of Oceanography pier (Scripps Institution of Oceanography, May 1944 and October 1944; U.S. Navy Hydrographic Office, 1944). A number of characteristics of breakers (such as H_b/d_b , the ratio of the breaker height to the "still water depth" at the point of breaking) were found to depend upon the configuration of the bottom. In the report on field measurements it is stated that during the period that observations were made, two types of nearshore underwater slopes occurred: Type 1, which was a reasonably plane bottom with a slope of about 1/40; Type 2, which has an outer portion with a slope or about 1/25, then a bar, with a slight trough shoreward of this, called the middle portion. The breakers were classed into two groups: Group 1, those that broke over the outer portion, and Group 2, those that broke over the middle portion. Values of H_b/d_b were considerably smaller for the Group 2 waves breaking on the Type 2 bottom configuration than for the other three combinations.

BOTTOM CONFIGURATION

Sand is moved along a beach by the action of waves breaking at an angle to the beach, by the movement of water rushing up and down the beach in a zig-zag motion, and by the longshore currents generated in the surf zone by the breaking waves. The littoral currents are often three dimensional, with rip currents moving out to sea at intervals along a beach, with a number of "cells" existing along a beach (Shepard and Inman, 1952; Sonu, 1972; Sasaki, Horikawa and Hotta, 1976; Mei and Liu, 1977). Sand is moved up and down the beach, and in and out of the surf zone in an onshore-offshore direction by wave action (Wiegel, 1964) and by the currents generated by winds blowing onshore, with return currents near the bottom flowing towards the offshore (King, 1959; Kraai, 1969; Sonu and van Beek, 1971; Sonu, 1972). Neglecting the wind generated currents, and their effects, there is evidence that the parameters that describes the direction of the net motion of sand is (Hattori and Kawamata, 1980):

$$\begin{aligned} (H_o/L_o) \tan \theta / (w_s/gT) &< \quad (\text{onshore transport, accretive}) \\ &= 0.5 \quad (\text{neutral, equilibrium profile}) \\ &> \quad (\text{offshore transport, erosive profile}) \end{aligned} \quad (14)$$

where w_s is the fall velocity of the median diameter sand grains. It can be seen from this, that it is difficult to apply this to actual beaches owing to the variation of sand size along a profile.

Relatively steep waves cause a net motion of sand off a beach face into the surf zone, often forming a bar. Waves of low steepness generally move sand from the bar and other portions of the surf zone onto the beach. These mass movements of sand onto and off the beach cause a modification in the characteristics of the breaking waves, the runup and rundown of the waves on the beach face, and the littoral currents, as mentioned previously. There is a strong relationship between the type of breakers and the beach slope (Patrick and Wiegel, 1955; Wiegel, 1964; Sonu, 1972; Wright, Chappell, Thom, Bradshaw and Cowell, 1979).

In planning the installation of a pier (or a pipeline) across a beach and through the surf zone, one must consider that the action of waves, currents, and winds continuously change the beach configuration, both on the shore and in the nearshore region. In calculating the bending moments exerted on piles by waves during the "design storm" one must consider the associated problem of the actual water depth along the section of a pier in the surf zone during the design storm (Wiegel, 1979), as well as the waves. An example of the variation of the bottom profile of a beach subject to heavy wave activity is shown in Figure 8. An extremely difficult problem is that of estimating where the bottom will be under various conditions (Wiegel, 1980). One must estimate the "design bottom profile."

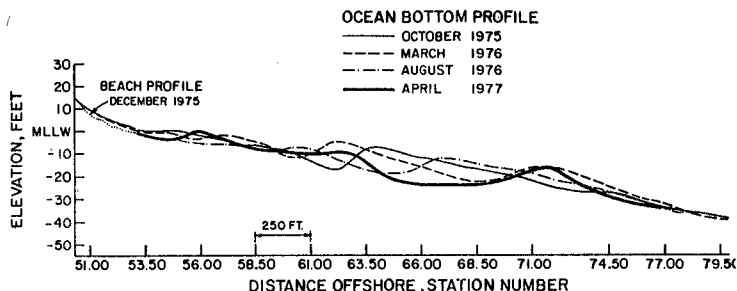


FIGURE 8. BEACH AND NEARSHORE PROFILES, SAMOA, CALIFORNIA, OCTOBER 1975-APRIL 1977 (FROM WINZLER & KELLY, 1976).

There is no simple answer to the question of how much change can be expected to occur as a result of a severe storm. It depends upon the recent past histories of the waves, tides (i.e., spring tide, neap tide, etc.) and beach profile of the particular site in question.

One of the longest set of records of beach and nearshore profiles is that obtained by Shepard, 1950. He made measurements from a pier at La Jolla, California. His data are summarized in Fig. 9. This region is not as greatly exposed to waves as are many other regions, such as the northern California coast or the southeast coast of Australia.

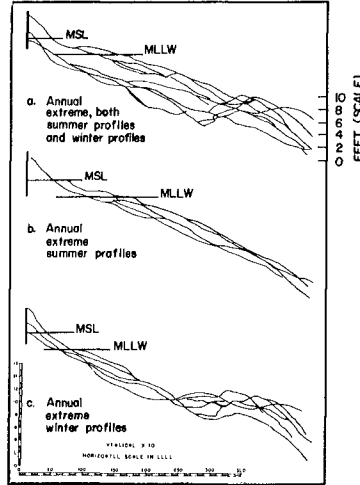
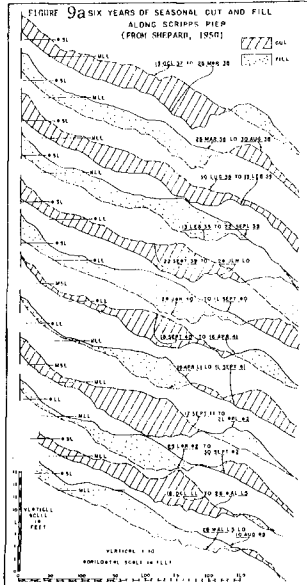


FIGURE 9b - ANNUAL EXTREME PROFILE CONDITIONS FOR 6 YEARS, DATA FROM SHEPARD (JULY, 1950) SCRIPPS PIER.

Shepard studied in detail the bar crest depths (bar depth) and bar trough depths (trough depths). It was found that the bar at the pier seemed to disappear on some occasions. Whether the bar did disappear, or was simply missing along the pier was not determined. Bars are not continuous along a beach, and there are gaps in them. That this is so can be seen by observing waves breaking on a beach where a bar is present offshore. A line of breakers (breaking over a bar) can be followed for some distance, separated by a gap, followed by another line of breakers. The waves move in through the gap (where the water is deeper, likely about the same depth as the trough) and break closer to shore. In considering the data given by Shepard, note that the ratio of bar depth to trough depth for the large bars measured at very exposed locations along the Oregon and Washington coast is about 2 or so, rather than the smaller values at other locations, Figure 10.

Much additional work is needed on the variations of bars with wave conditions. These appear to be perturbations about some mean bottom profile on sand beaches. Dean, 1977, has found that a mean bottom profile curve can be approximated by the following formula, neglecting bars:

$$h = Ax^{0.67} \quad (15)$$

Here, h is the vertical distance below the mean low water level, x is

the horizontal distance out from the origin (taken as the intersection of the beach with mean low water), and A is a coefficient to be determined empirically. As an example, consider one profile across Ninety-mile Beach, Victoria, Australia, Figure 11. Using the survey data, and an average value through the survey, it was found that $A = 0.235 \text{ ft}^{1/3}$. It is important to note that Dean found 70% of the values calculated for 502 beach profiles were between 0.1 and 0.2, and that another 18% were between 0.2 and 0.3.

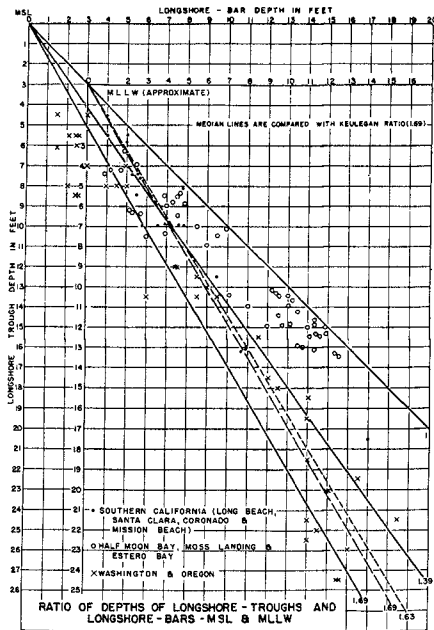


FIGURE 10. (FROM SHEPARD, 1950).

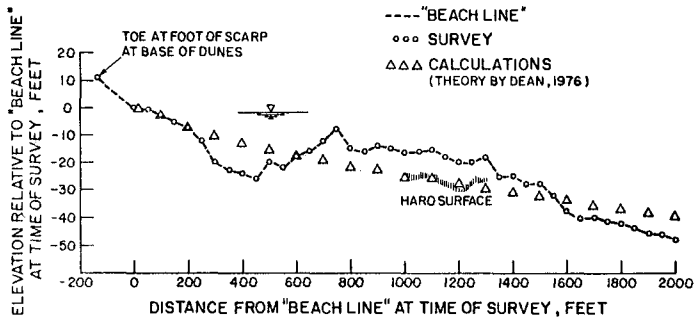


FIGURE 11 SURVEY ALONG PROPOSED "SNAPPER" PIPELINE, 29 SEPTEMBER 1976, NINETY-MILE BEACH, VICTORIA, AUSTRALIA (FROM WIEGEL, 1976).

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