1988

An investigation of perched beach profile response to wave action /

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AN INVESTIGATION OF PERCHED BEACH PROFILE RESPONSE TO WAVE ACTION

by

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A Thesis

Presented to the Graduate Committee

of Lehigh University

in Candidacy for the Degree of

Master of Science

in

Civil Engineering

Lehigh University

1987

This thesis is accepted and approved in partial fulfillment of the requirements for the degree Master of Science.

December 7, 1987

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Symbols and Notation

A	- Dimensionless scale parameter
C_u	- Uniformity coefficient
d	- Water depth below still water level
dsub	- Submergence depth of the toe structure crest elevation below still water level
D_{10}	- Particle diameter with 10% of the sample passing
D_{50}	- Median particle diameter
D_{50m}	- Median particle diameter, model
D_{50p}	- Median particle diameter, prototype
D_{60}	- Particle diameter with 60 % of the sample passing
ER_l	- Linear Rate of Beach Profile Erosion
Hmo	- Zero-th spectral moment wave height
Ho	- Monochromatic Wave Height
Hs	- Significant wave height
Hs_m	- Significant wave height, model
Hs_n	- Neutral significant wave height
Hs_p	- Significant wave height, prototype
Kt	- Wave height transmission coefficient
MLW	- Mean low water level
N_h	- Horizontal scale ratio
N_T	- Wave induced water surface elevation
N_t	- Time scale ratio

N_v - Vertical scale ratio

N* - Number of water surface elevation readings

p:m - Scale ratio, prototype to model

SWL - Still water level

T - Monochromatic wave period

Tp - Significant wave period

Ts_m - Significant wave period, model

Ts_p - Significant wave period, prototype

V_f - Fall Velocity

X - Horizontal offshore distance

Y - Beach profile elevation

ABSTRACT

The perched beach concept is an alternative to traditional methods of beach stabilization. The method utilizes a submerged toe structure built offshore and parallel to the shore. The structure has dual purposes - to protect the beach from erosive wave action and to retain the beach fill material.

Two-dimensional irregular storm wave tests were conducted on a typical non-perched beach and a perched beach (consisting of a rubble-mound toe structure and a nourished beach profile). The position of the toe structure was varied along the beach profile to provide a range of structure crest submergence depths equal to 1.0, 0.5, 0.25, and zero incident significant wave heights.

Each of the four perched beach test cases was subjected to an identical storm wave climate as the non-perched beach test case, to facilitate comparison. Beach profiles were recorded at selected time intervals during each test run for comparison and evaluation of the effectiveness of the concept. Resulting profile data are presented and discussed.

Plots of the successive beach profiles indicate typical beach profile response to storm wave attack. This response is characterized by a cutback in the

location of the beach face and the formation of an offshore bar. The parameter ER_1 (the linear rate of beach face recession at the SWL) was introduced to compare the rate of the beach face retreat between test cases.

Test results indicate that the submerged toe structure triggered the larger individual storm waves to break. In addition, as the submergence depth of the structure's crest elevation approached zero, the horizontal distance from the submerged toe structure to the beach face decreased. In comparing ER_1 values, one-third to two-third's of the overall beach face recession occurred during the first six-hour storm simulation - ER_1 values decreased substantially after the first storm period thus decreasing the incremental beach face recession.

Equilibrium conditions were achieved for the perched beach tests where the structure crest elevation was near the SWL (see Test Cases 4 & 5). Resulting beach profiles indicate that the perched beach concept is a viable alternative to shoreline stabilization provided there is sufficient distance (40-50 Hs as found in this study) between the desired berm crest location and the submerged toe structure to account for the resulting storm induced beach face cutback. In addition, a savings of fill material can be realized provided the nourished beach fill slopes landward and seaward of the submerged toe structure approximate equilibrium conditions for typical storm wave attack.

1.0 INTRODUCTION

1.1 Background

Several locations along the New Jersey shore, as well as much of the shoreline of other coastal states, are subject to beach erosion. One cause of natural shoreline retreat is the disruption of the supply of alongshore sediment transport by natural or man made features. Another cause is the relative rise in sea level. This retreat can reach proportions of 15 meters/year or more (2). As beaches erode, shoreline structures as well as other coastal features such as houses, boardwalks, and transportation facilities may be endangered. In some cases, damage to or failure of these features can result in substantial economic, environmental, and recreational loss.

A common solution for the beach erosion problem has been the periodic placement of sand fill to nourish the beach. Often, structures are constructed to help stabilize the fill. Examples of such structures include groins and various types of offshore breakwaters. Groins are usefull for preventing the loss of sand owing to unbalanced longshore transport, but they have little effect in preventing the offshore transport (and effective loss) of sand that is commonly caused by storms. Offshore breakwaters, built parallel to the shore and generally outside the surf zone, intercept waves and offer protection to the beach. They are, by necessity, constructed in relatively deep water, resulting in massive expensive structures (costing several to ten thousand dollars per foot in the open ocean).

The perched beach concept is a variation on the use of offshore breakwaters for stabilizing nourished beaches and protecting shore structures. A low submerged toe structure is constructed offshore and parallel to shore. Sand fill is placed landward of the structure, creating the "perched " beach (see Figure 1). The toe structure retains the perched beach, greatly reducing the volume of sand otherwise required for a non-perched beach which would have to extend much further seaward to maintain the same profile.

In addition to retaining the sand fill, the toe structure triggers the breaking of the larger (more destructive) waves, dissipating much of their energy before they reach the beach face. It has little effect on the normal, day-to-day lower wave activity so recreational aspects of the beach are not diminished. If beach nourishment is not present, the structure may still function to stabilize the natural beach and protect shore structures.

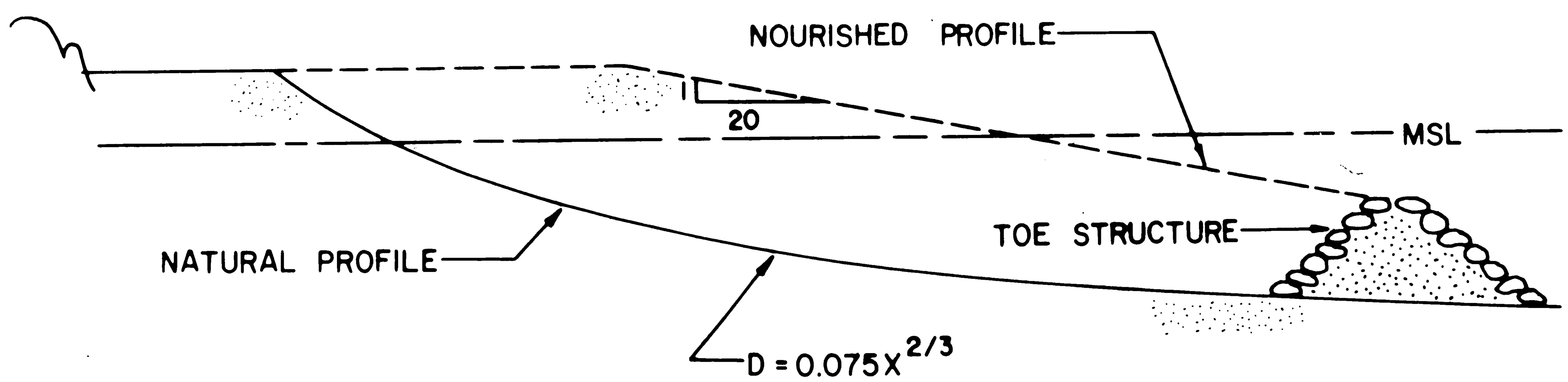

FIGURE 1. SCHEMATIC PROFILE - PERCHED BEACH CONCEPT

A thorough evaluation of the perched beach concept requires a two-phase effort: the first to investigate stability of the toe structure and the second to investigate the response of the nourished beach profile to wave attack. The first phase was completed by Givler (1).

The second phase, presented in this study, focuses on the response of the non-perched and perched beach profiles during storm wave attack. Experiments were conducted in a two-dimensional wave tank having a wave generator with the capability of producing irregular spectral waves.

Initially, a typical natural beach profile was constructed with a nourished section placed at a 1:20 slope and no toe structure (see Figure 1). This profile was subjected to a series of storm and calm wave conditions. The profile was measured at specific time intervals to determine it's response to the wave action and to establish a base test case for later comparisons to tests with a toe structure in place.

The second part of this study focused on the effects of a toe structure located at selected positions along the nourished profile (see Figure 1). Tests were conducted to determine beach profile response and a possible "best structure location", i.e. a location resulting in minimal erosion of the perched beach profile.

1.2 Objectives

Considering the impact of beach erosion in the state of New Jersey and the surrounding region, the need for the evaluation of alternative beach protection schemes is evident. The relative lack of information concerning the perched beach concept and it's processes required further research to evaluate the concept and to provide guidance to designers.

Specific objectives for this portion of the perched beach study are:

1. Evaluate the effectiveness of the perched beach concept for shoreline stabilization.
2. Determine the relative effect of the toe structure location on the nourished beach profile response.
3. Propose design guidance as to the overall function and capability of the perched beach concept.

1.3 Scope of Work

The following sections include a review of the literature pertaining to the perched beach concept and a description of the experimental setup and apparatus used herein. For the later, items such as the wave tank, wave generator, beach sand characteristics, and wave characteristics are discussed. This is followed by an overview and a step-by-step summary of the test procedures used. Briefly, two-dimensional irregular wave tests were conducted first on a nourished non-perched beach profile to establish base test conditions and then on several nourished perched beach profiles each of slightly different geometry. The differences in geometry resulted from varying the position of the toe structure in order to provide a range of structure crest submergence depths equal to 1.0, 0.5, 0.25 and zero incident significant wave heights for storm conditions. Each nourished perched beach test case was subjected to the same wave climate as the base test case, to facilitate comparison.

Based upon this study and the results presented by others, recommendations and design guidance for the use of the perched beach concept as a means of shoreline protection are presented. The results reported are normalized so they can be applied to any location, although the test parameters were developed for typical east coast conditions.

2.0 LITERATURE REVIEW

To date, perched beaches have been constructed in the United States at a few sheltered locations (e.g. Delaware Bay and Cape Cod), but not in the open ocean. At Slaughter Beach, Delaware, the Corps of Engineers (3) constructed a perched beach with sill (toe structure) sections made of sand bags, wood sheet piling, and Longard tube. The toe structure's crest was located about 1 meter below MLW and coarse sand was pumped into the enclosure formed by the sill. The monitoring data from this project was presented by Douglass and Weggel (4). The offshore sill structure and associated shore return structures altered the beach planform and bathymetry much as would be expected behind a submerged breakwater: the shoreline accreted at the downdrift end and receded at the updrift end. However, available data was insufficient to completely analyze the effect of the sill structure on cross-shore sediment movement.

On Cape Cod (5), sand bag sills were built at four sites. These sills were placed such that their crest elevations ranged from 0 to 1.5 meters above MLW. It was hoped the structure would trap sand naturally. Gutnam (5) reports the failure of 3 out of 4 of these installations due to severe storms, surrounding structures which inhibit longshore transport, and high tidal ranges. No evaluation of the fourth structure was presented due to insufficient data being available.

The perched beach concept has not been effectively evaluated for open ocean conditions. In the 1960's, a plan was developed to build a freeway along the coast at Santa Monica, California. This plan called for the construction of a perched beach to widen the existing beach to provide right-of-way for the freeway (6).

The proposed 10-kilometer-long beach widening was to be constructed using approximately the same profile as the existing beach from the berm crest to about the 8-meter depth contour, where a submerged rubble-mound breakwater type structure was to be constructed to hold the new beach in a permanent position. A stone rip rap apron would be constructed along the shoreward edge of the breakwater to prevent offshore transport of beach material due to wave agitation near the breakwater.

A brief series of movable bed model studies was conducted at the U.S. Army Waterways Experiment Station (7,8) to:

1. Estimate the amount of sand which might be lost seaward over the toe structure due to normal and storm wave actions.

2. Determine the optimum crown elevation of the submerged toe structure and length of the stone apron required to reduce seaward migration of sand to a minimum.

A distorted two-dimensional, movable bed model was constructed in a flume having a monochromatic wave generator. Using coal (specific gravity of 1.30) as the model beach material, average beach slopes for the Santa Monica area were constructed. Hindcasting methods were employed to determine required prototype test waves which had periods ranging from 11 to 17 seconds and heights ranging from 2.43 to 4.28 m. This data was used to test several "plans" (8) which varied the offshore location and height of the toe structure. The ratio of significant wave height to water depth over the structure crest ranged from 0.4 to 1.4 m. A stone apron shoreward of the structure was installed in some of the plans.

Test results indicate that normal wave action (waves that occur a high portion of the time) caused no appreciable loss of beach fill. However, larger storm waves of sufficient duration caused a large seaward loss of the perched beach fill (8). The best results occurred with the structure located approximately one significant wave height below SWL and a 30.48 m stone apron in place shoreward of the structure. This "plan" significantly reduced the amount of beach fill lost seaward. Hence, the model study found the perched beach concept feasible for the Santa Monica area.

3.0 EXPERIMENTAL SET-UP

3.1 Experimental Apparatus

The wave tank investigations were conducted at the H. R. Imbt Hydraulics Laboratory, Lehigh University. The tests were performed in a concrete wave tank having a glass observation section and equipped with a programmable wave generator capable of producing spectral waves. During testing, water surface time histories were measured by a parallel wire wave gage. An analog signal digitizer was employed to record measured surface elevation data.

The modeling sand used in the tests was a fine, uniformly graded silica. The submerged toe structure was constructed with plywood and installed independently of the wave tank to facilitate variation of it's crest elevation and location. Beach profiles were recorded by a movable point gage at sufficient time and space intervals to assure adequate data monitoring for analysis.

The following sections describe in detail the various components of the experimental apparatus.

3.1.1 The Wave Flume

Tests were performed in a concrete wave flume

32.66 m by 0.91 m by 0.91 m as shown in Figure 2. A plywood bulkhead spanning the width of the flume served to retain the beach sand. The beach section and profiled distance extended 15.24 m seaward from the bulkhead to an intersection with the bottom of the flume. Stone was placed behind the plywood to counteract the hydrostatic pressure and the dead load of the beach sand.

A 2.44 m long glass wall test section is located 0.61 m seaward of the bulkhead (see Figure 2). The glass wall section permitted observations of onshore/offshore transport mechanisms and wave run-up patterns.

A steel rail mounted on each flume wall carried a carriage which supported the beach profiling apparatus (discussed in Section 3.1.7). In addition, a base line (referenced to the bulkhead located at 0.00 m) was affixed to one of the steel rails for beach profiling (see Figure 5, section 3.1.7).

3.1.2 The Toe Structure

The toe structure was constructed of plywood to the dimensions shown in Figure 3. Once installed at the proper location, the structure was ballasted with lead bricks to insure stability and the sides adjoining the

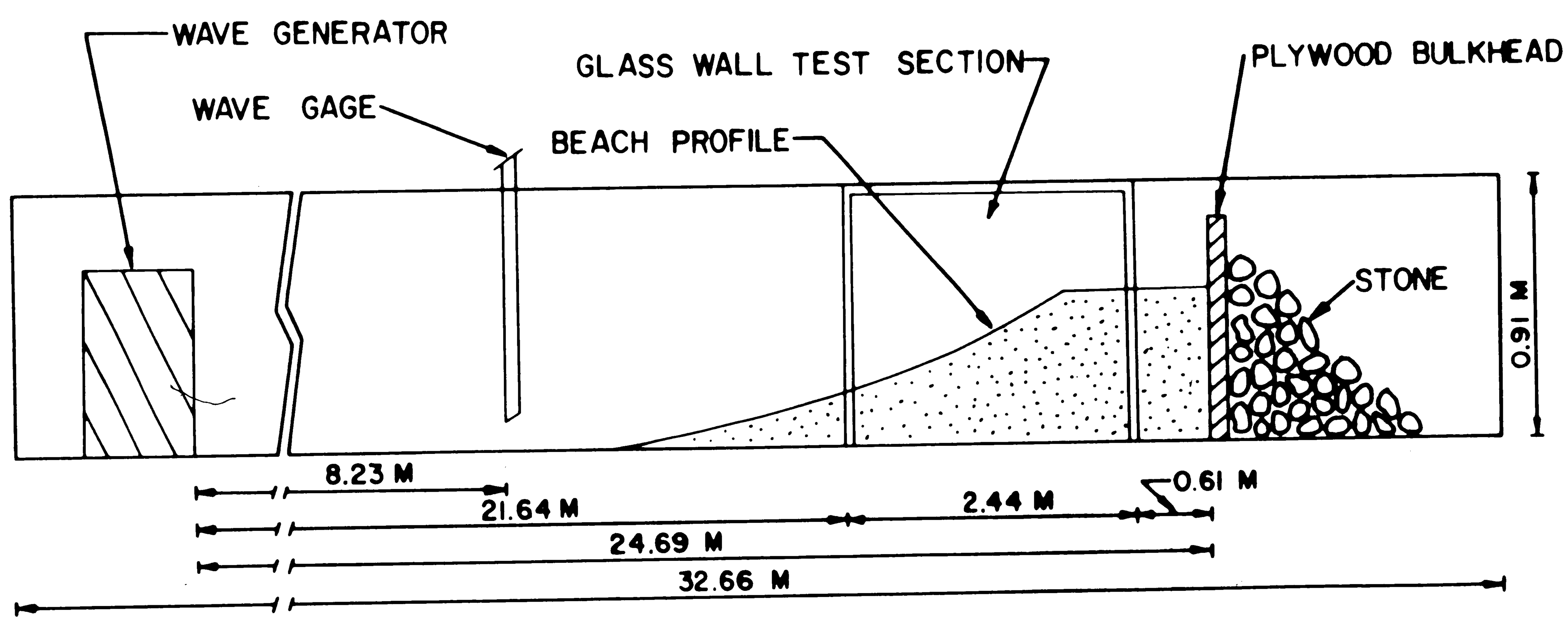

FIGURE 2. THE WAVE FLUME SET-UP
(NOT TO SCALE)

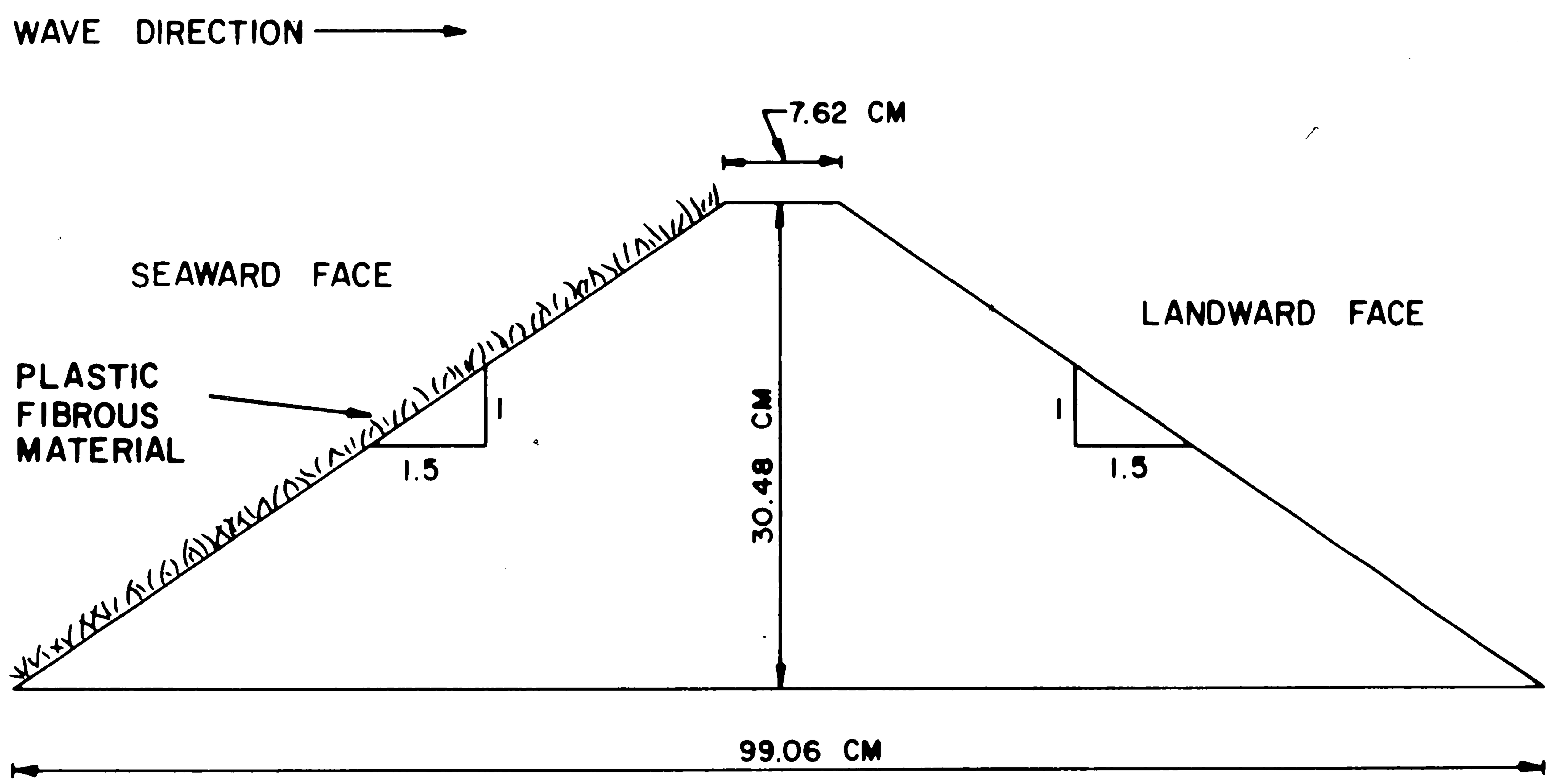

FIGURE 3. CROSS SECTION OF THE TOE STRUCTURE

wave flume were sealed to prevent sand loss between the structure and the walls of the flume. The front, or seaward slope of the structure was covered with a plastic artificial turf material to reduce wave reflection. Prior to testing, the structure was surveyed with the point gage to both level the structure and assure accurate location with respect to the beach profile and still water elevation.

3.1.3 The Wave Generator

Irregular wave spectra were generated by a pair of pneumatically driven piston wave generators. The characteristics of the spectra were controlled by a programmable signal generator which governed the movement of the pistons. The signal generator is capable of creating monochromatic waves or a variety of wave spectra. Only spectral waves were used in this study. (Input to these generators was calculated by a computer program discussed in section 3.1.5, The Simpulse Program.)

The wave generator system is equipped with adsorption capability, i.e., reflected waves were sensed by the generator and subtracted from the current wave being generated. Thus, a truer representation of the inputted wave spectrum was obtained, i.e. reflected wave effects were reduced to a minimum.

3.1.4 Wave Gages and Recorder

To measure the generated wave spectra, a parallel wire resistance wave gage was installed 8.23 m from the generator (see Figure 2). The wave gage was located a sufficient distance from the generator to insure complete formation of the generated waves and seaward of the toe of the sand beach. Output from the wave gage was recorded on chart paper (see Figure 4) to yield a visual record of the water surface time history. Output was also recorded in digitized form by an analog signal digitizer for analysis.

3.1.5 The Analog Signal Digitizer

The analog signal digitizer digitizes and records voltage levels output from the wave gage. A range of digitizing frequencies is available; a frequency of 0.12 hz or 8.33 digitized values per second was used.

Prior to use, the wave gage and digitizer must be calibrated to establish the relationship between the voltage readings and the water surface elevation. (An explaination of this calibration can be found in section 4.3, Data Collected.)

3.1.6 The Simpulse Program

The Simpulse Computer Program is a BASIC program

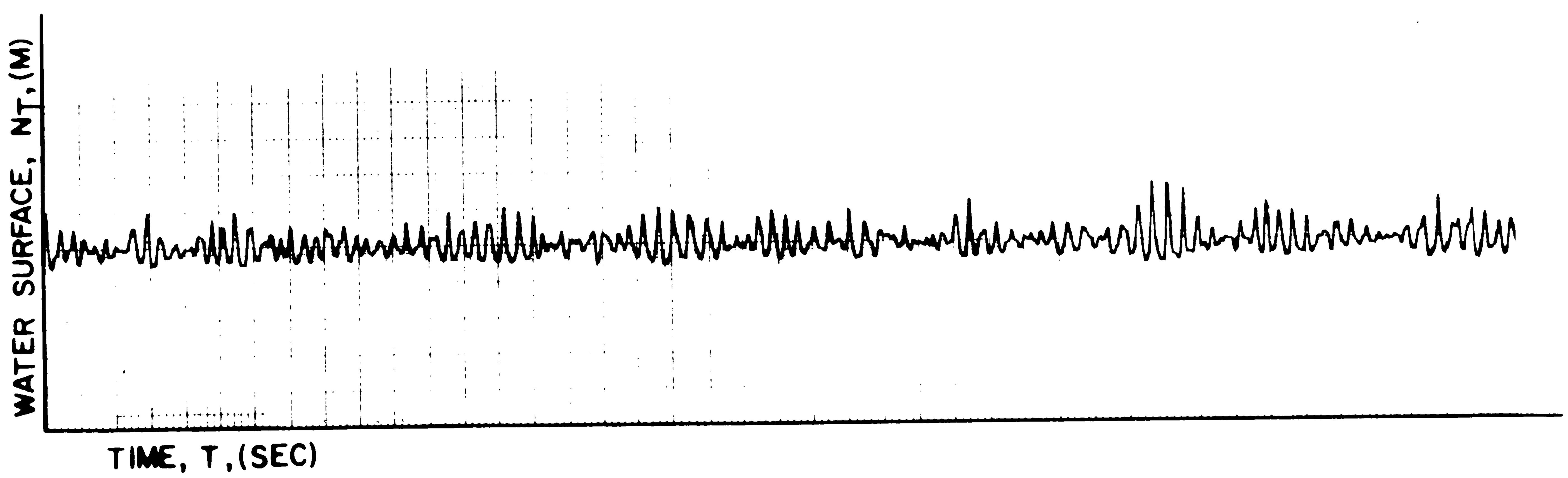

FIGURE 4. TYPICAL WATER SURFACE TIME HISTORY FROM WAVE GAGE AND RECORDER

that calculates input settings necessary for the wave generator to produce the desired irregular wave spectrum. A JONSWAP wave spectrum (18) was used in this study.

Input data required for the Simpulse Program consists of:

1. Water depth and depth to the paddle bottom.
2. Model scale.
3. Maximum frequency of spectrum (usually 3 to 4 times the peak spectral frequency).
4. The selected spectral type (Darbyshire, I.T.T.C., Pierson Moskowitz, or JONSWAP).
5. The selected significant wave height, Hs, and the peak wave period, Tp.

The generated spectral shape remains constant for a selected peak period and a range of significant wave heights, (i.e., the wave spectrum need not be recalculated for a variation in wave height). The variation in significant wave height was achieved by varying the generator gain setting. Thus, by increasing the gain, a spectrum with a greater significant wave height but the same basic shape and peak period can be generated.

3.1.7 Bottom Profiling

The beach profiling system consisted of a point gage mounted on a movable carriage assembly and a base line affixed to the flume (see Figure 5). Beach profiles were recorded by taking point gage readings every 0.03 m along the baseline for the entire length of the profile. Averaging across the profile was deemed unnecessary due to careful initial beach profile construction and the small amount of lateral assymetry that developed in any of the tests. The vertical datum was obtained by taking an average point gage reading on the floor of the flume. The vertical datum was established at 0.21 m, (i.e., the average point gage reading on the floor of the wave flume was 0.21 m).

3.2 Beach Sand Characteristics

To duplicate prototype behavior in a scaled model, the beach material should be small enough in diameter to satisfy scale requirements yet large enough so as to behave as cohesionless material. Some authors, e.g. Noda (20), recommend the use of a fine uniformly graded sand as the model beach material. This appears to to be the best compromise even though exact sand size scaling is not maintained. Fine uniform sands range from 0.074 mm to 0.425 mm in grain size and have uniformity

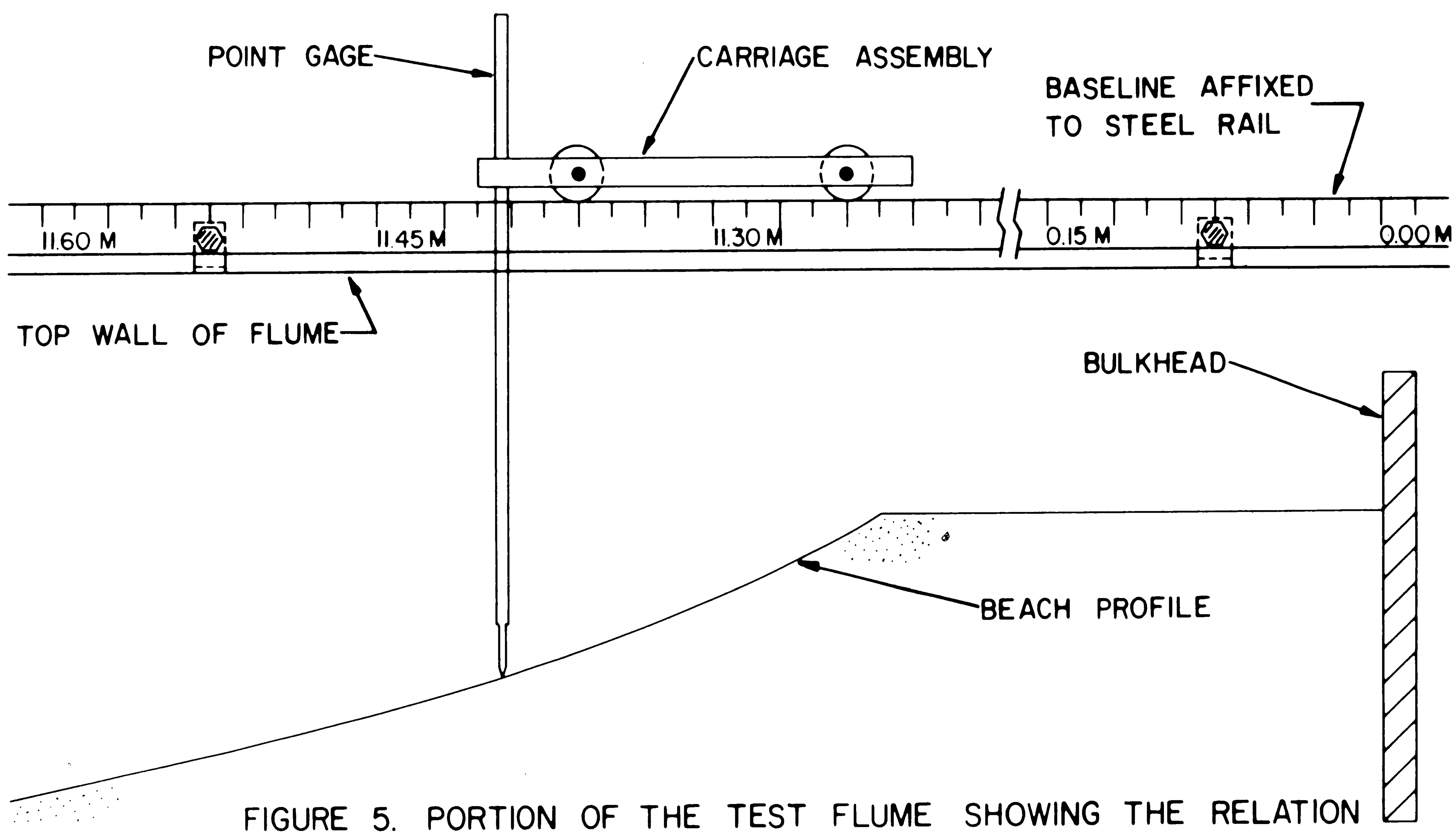

FIGURE 5. PORTION OF THE TEST FLUME SHOWING THE RELATION OF BULKHEAD TO BASELINE AND THE BOTTOM PROFILING APPARATUS

coefficients near unity (9).

For this study, eight tons of sand was supplied from two different sources; "George Silica" and "Hardy Sand Company". A sieve analysis was conducted to determine the size distribution and uniformity of each sample. From this analysis, the median particle diameter, D_{50}, and the uniformity coefficient, Cu, were obtained (C_u is defined as D_{60}/D_{10}). Figure 6 is a cumulative logarithmic plot of the grain size distribution for both samples of the model beach sand. The median particle sizes and uniformity coefficients for each sample are shown in Table 1.

Table 1. Beach Sand Characteristics

Source	Mean Particle Diameter, D_{50}	Uniformity Coefficient, C_u
Georgia Silica	0.146 mm	1.44
Hardy Sand Co.	0.145 mm	1.42

It is obvious from Table 1 that both samples meet the requirements of a fine, uniformly graded sand. Furthermore, the sieve analysis indicates the samples are almost identical in makeup, indicating no further

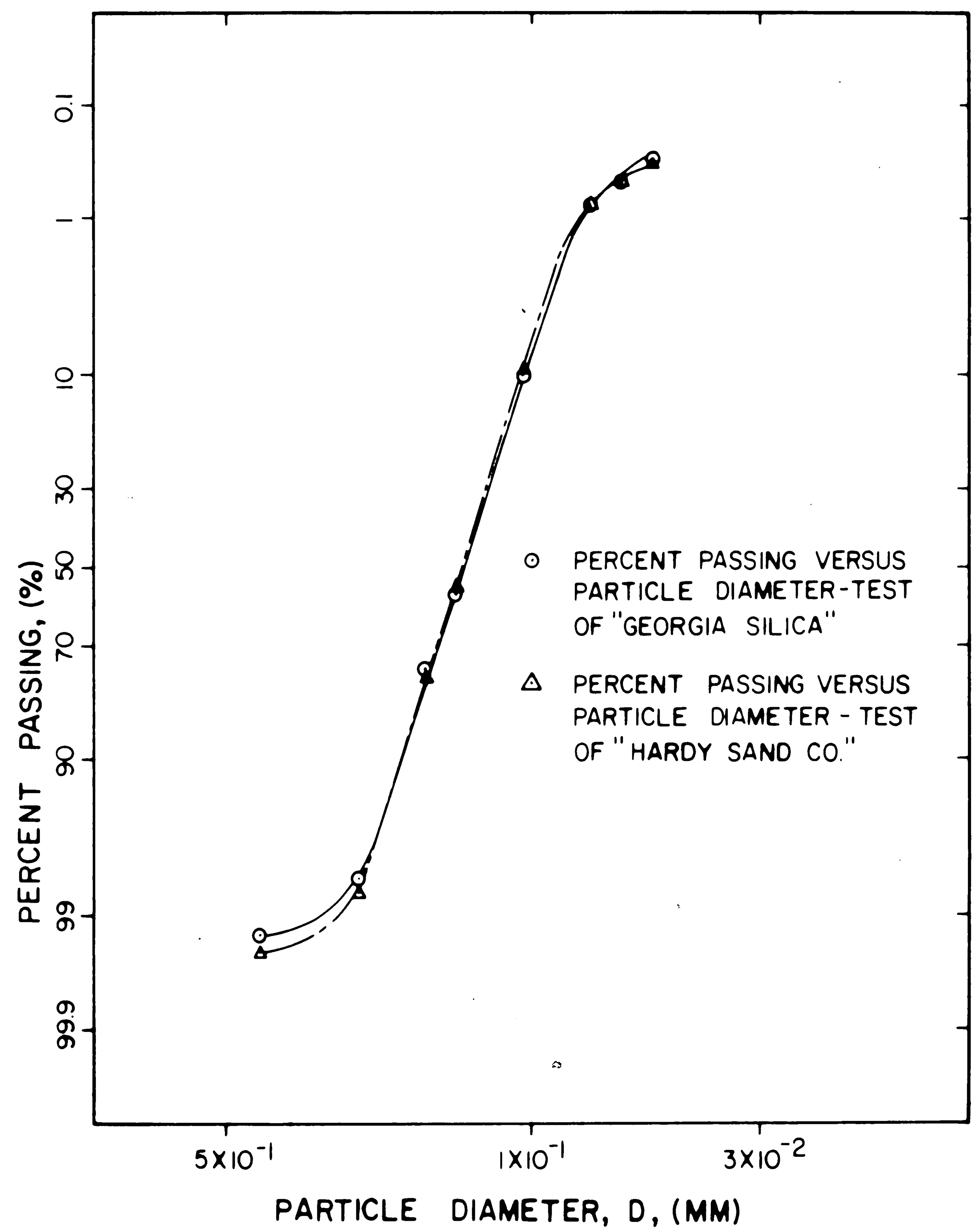

FIGURE 6. SIEVE ANALYSIS RESULTS FOR "GEORGIA SILICA" AND "HARDY SAND CO."

need to distinguish between the two samples (see Figure 6). Thus, D_{50} was established as 0.145 mm for the model beach sand.

In considering the motion of littoral materials, one meaningful parameter is the particle fall velocity, V_f. The Shore Protection Manual (10) defines V_f as "the vertical velocity attained by an isolated solid grain settling due to gravity in a still, unbounded, less dense fluid".

The fall velocity was calculated using the method prescribed in the Shore Protection Manual (10) with the following assumptions:

1. Specific gravity of sand = 2.65
2. Median particle diameter, D_{50} = 0.145 mm.
3. Fluid's kinematic viscosity (water temperature at 20° C) = 1.00×10^{-6} m^2/s.

This yielded a fall velocity equal to 0.018 m/s.

3.3 Model Scales

The modeling scales chosen for this investigation were based on Froude scale modeling, the physical limitations of the flume/generator, and typical east coast storm wave characteristics. Table 2 presents the resulting wave parameters and the model scales used in this investigation.

Table 2. Wave Parameters and Model Scales

Wave Parameters

	significant wave height	significant wave period
prototype	2.44 m	8.0 s
model	0.1 m	1.6 s

Model Scales

scale type	value (p:m)
Horizontal, N_h	25:1
Vertical, N_v	27:1
Time, N_t	5:1

For a reasonable water depth (0.45 m to 0.60 m), the generator is capable of producing a significant wave height, Hs_m, of 0.1 m (see Table 2). Thus, the characteristic vertical length of the model was established as 0.1 m. By scanning a series of winter month storm observations for Atlantic City, New Jersey (11), the significant wave height of the prototype, Hs_p, was taken to be 2.44 m (see Table 2). This is the characteristic vertical length of the prototype. From the same source (11), the significant wave period, Ts_p, was found typically to be 8.0 seconds (see Table 2).

For this study, $N_v = Hs_p/Hs_m$ or 25:1 (see table 2). Froude modeling requires that the vertical scale, N_v, and the time scale, N_t, are related as follows:

$$N_t = (N_v)^{0.5} \qquad \text{Eq. 1}$$

Thus, N_t = 5:1, resulting in Ts_m = 1.6 seconds (see table 2).

Hallermeier (12) suggests a relationship between the horizontal and vertical scales based on simulated requirements for beach profiles models (too complicated to be presented here). Using his formulas and a typical Atlantic Coast beach sand size D_{50p} value of 0.25 mm (10), the required horizontal scale, N_h, was calculated to be 27:1 (see Table 2). Thus, for a prototype having a mean particle diameter of 0.25 mm, this model is essentially undistorted hydraulically , i.e. N_h approximately equals N_v. The sand size is, of course, distorted, having a prototype to model size ratio of 0.25/0.145 or approximately 1 to 1.

3.4 Initial Beach Profiles

3.4.1 Beach Geometry

Figure 7 illustrates the terminology used to describe the beach profile and the wave action in the nearshore zone. The features described in the figure are referred to throughout this investigation.

3.4.2 The Natural Profile

The natural or equilibrium beach profile is an idealization of conditions which occur for particular sediment characteristics and steady wave conditions (2). Although beaches may never attain an equilibrium profile (due to changing wave climates) the concept is useful in the design of beach nourishment schemes.

Dean (13) analyzed beach profiles along the East and Gulf Coasts of North America and found the profiles could be described by:

$$d = A * x^{2/3} \qquad \text{Eq. 2}$$

in which x is the distance offshore to a water depth d and A is a dimensionless scale parameter which depends on sediment size. Moore (14) determined the scale parameter, A, as a function of sand diameter. For D_{50m}

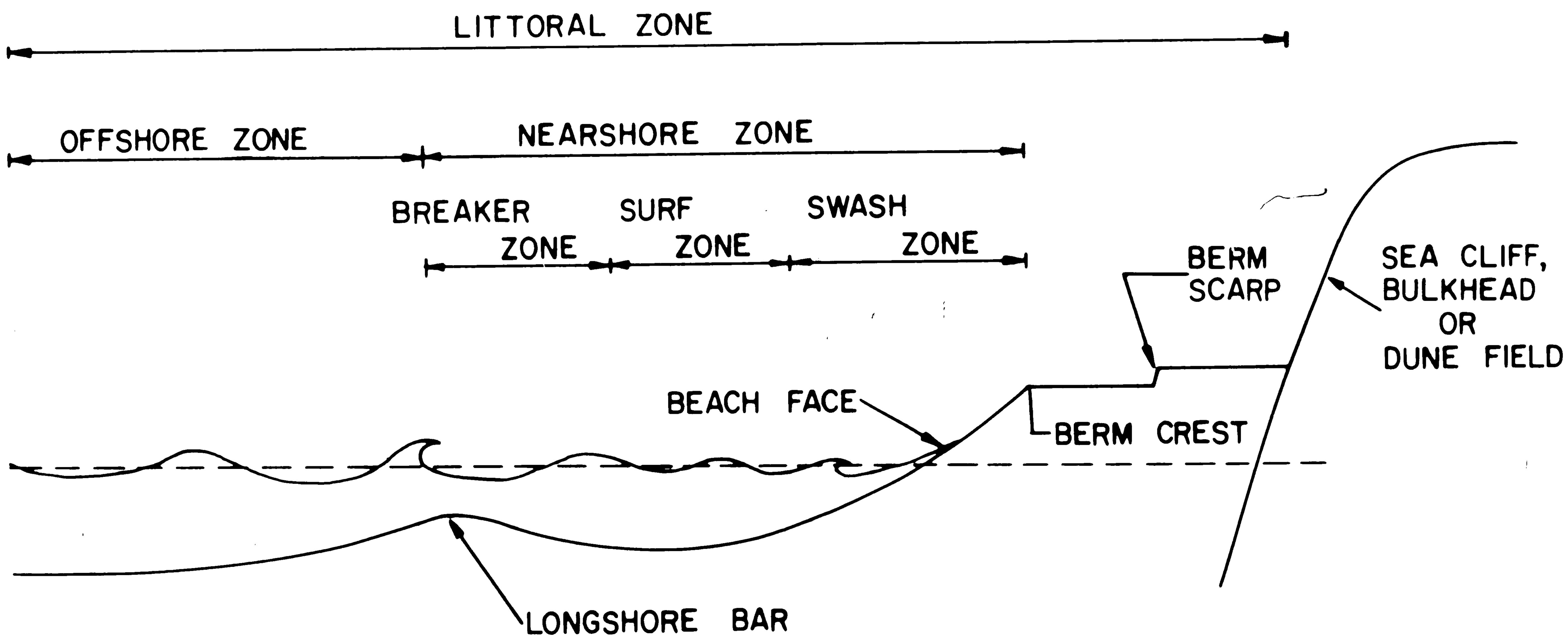

FIGURE 7. TERMINOLOGY USED TO DESCRIBE THE BEACH PROFILE AND WAVE ACTION IN THE NEARSHORE ZONE (AFTER KOMAR [16])

= 0.145 mm, the scale parameter was found to be 0.075 (2). A plot of the resulting natural beach profile is given in Figure 8.

The natural beach profile is commonly used to estimate the beach geometry of a particular site. Fill is then placed over the natural beach to form the nourished beach - discussed in the following section.

3.4.3 The Nourished Profile

A review of numerous beach nourishment projects indicates a typical fill slope for the nourished profile is 1 on 20 (15). Hence, a 1 on 20 slope was chosen as the nourished beach slope for this investigation. A berm extension of 2.44 m was installed to insure that the beach profile did not erode back to the bulkhead. Erosion to the bulkhead would disrupt the profile formation.

Figure 8 depicts the nourished profile and it's relationship to the natural profile and the wave flume. The origin of the natural profile was located at the bulkhead. The nourished profile intersected the natural or equilibrium profile 12.80 m seaward of the bulkhead (see Figure 8). Beyond this point of intersection, the natural profile was approximated by a 1 on 45 slope

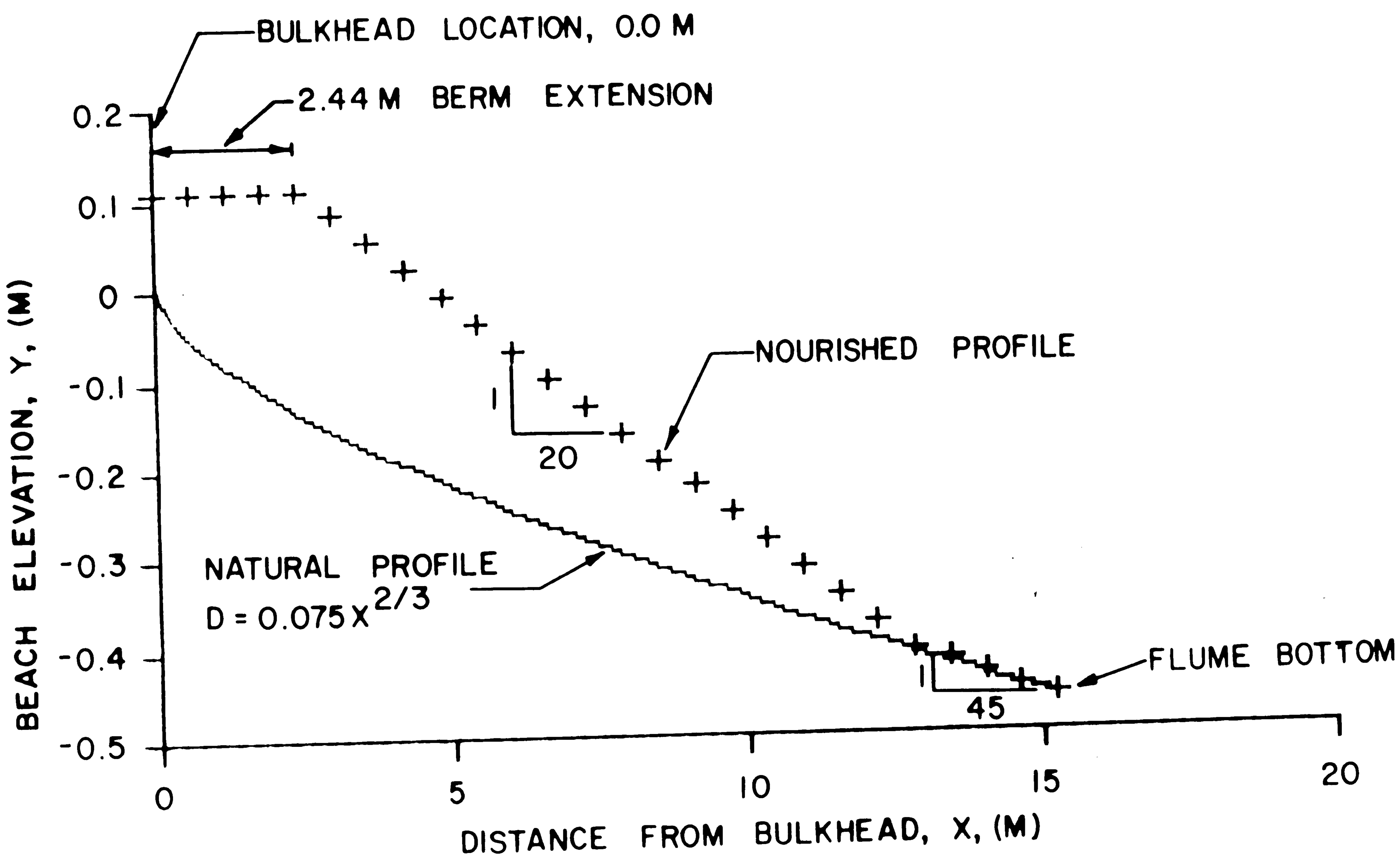

FIGURE 8. PLOT OF THE NOURISHED BEACH PROFILE AND IT'S RELATIONSHIP TO THE NATURAL BEACH PROFILE AND THE WAVE FLUME

until it intersected with the floor of the wave flume (see Figure 8). Thus, the total length of the nourished beach profile was 12.80 m.

3.5 Wave Characteristics

3.5.1 Erosive and Accretive Wave Climates

Aside from wind and currents, beach processes are largely governed by the predominant wave conditions. If the wave attack results in a "cutting back" of the beach, wave conditions are classified as erosive or stormy. If, however, the beach accumulates, accretive or calm wave conditions exist.

The type of wave climate present depends on wave characteristics such as the significant wave height, Hs, and the significant wave period, Ts, as well as the fall velocity of the beach sand. Kriebel et al. (17) suggest the following relationship to define the dividing zone between erosive or accretive conditions:

$$\frac{Ho}{V_f * T} = 2.0 \text{ to } 2.5 \qquad \text{Eq. 3}$$

taking the fall velocity as 0.018 m/s (constant for this investigation), and 2.25 as the average of the right side of Eq. 3, a linear relationship was developed between the significant wave height, Hs, and the significant wave period, Ts. This relationship is as follows:

$$Hs = 0.0405 * Ts \qquad \text{Eq. 4}$$

For this investigation (Ts = 1.6 sec), the significant wave height dividing the erosive and accretive wave climates - referred to as the neutral significant wave height, Hs_n, is 0.064 m. Therefore, a spectrum generated with a significant wave height greater than 0.064 m should be erosive, while a significant wave height less than 0.064 m indicates an accretive wave climate.

To check Eq. 4 for this experimental set-up and the storm and calm waves used in the experiments, a beach with a 1 on 20 slope was constructed in the wave flume. The profile was subjected to 10 hours of storm waves, then 15 hours of calm waves. The significant wave heights, calculated for the storm and calm conditions, were 0.076 m and 0.030 m respectively. Beach profiles were recorded at 0, 10, and 25 hours to observe changes in profile characteristics. The results of this test are shown in Figures 9 and 10.

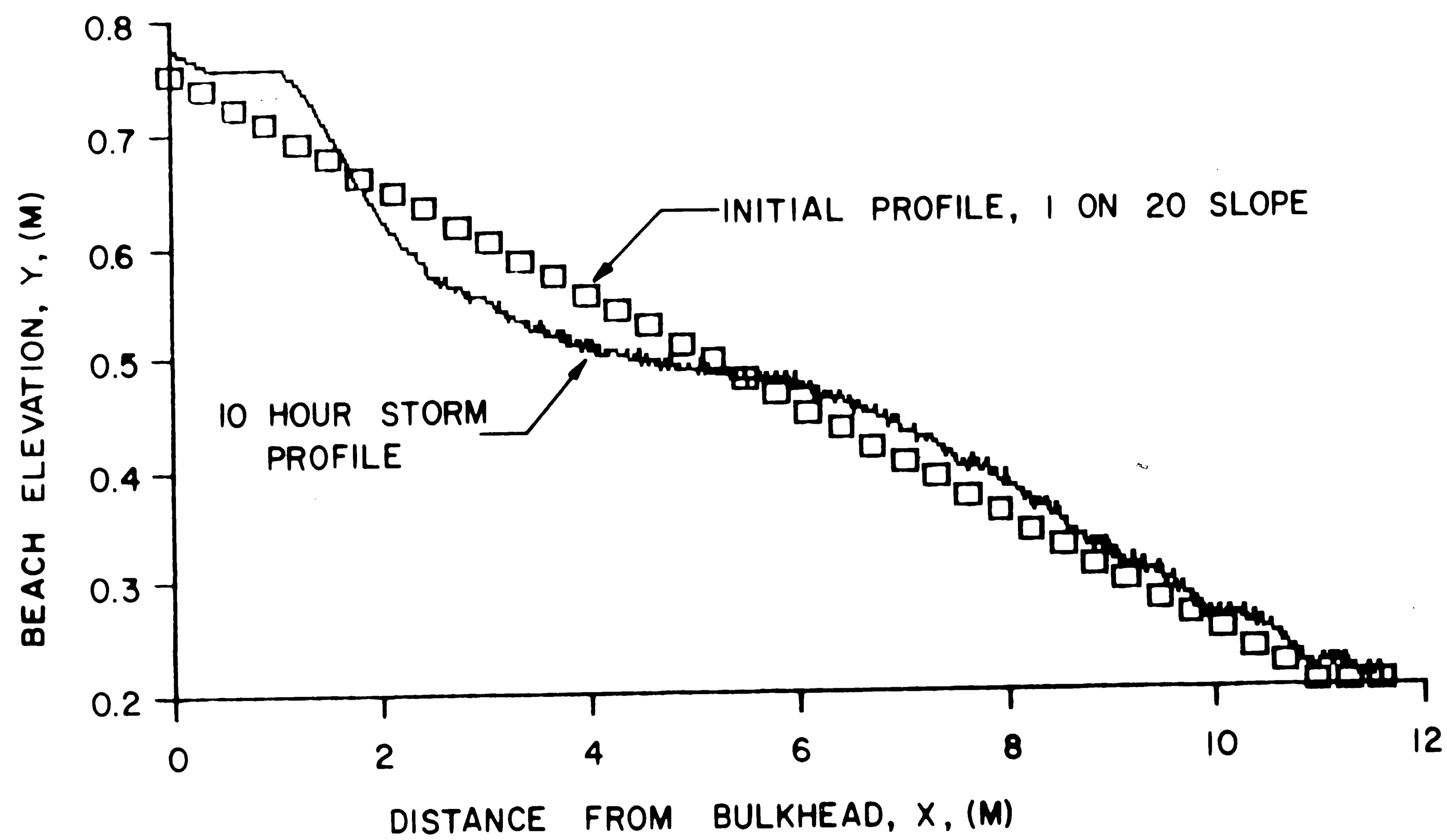

FIGURE 9. PLOT OF THE INITIAL AND 10 HOUR STORM PROFILES – THEORY VERIFICATION

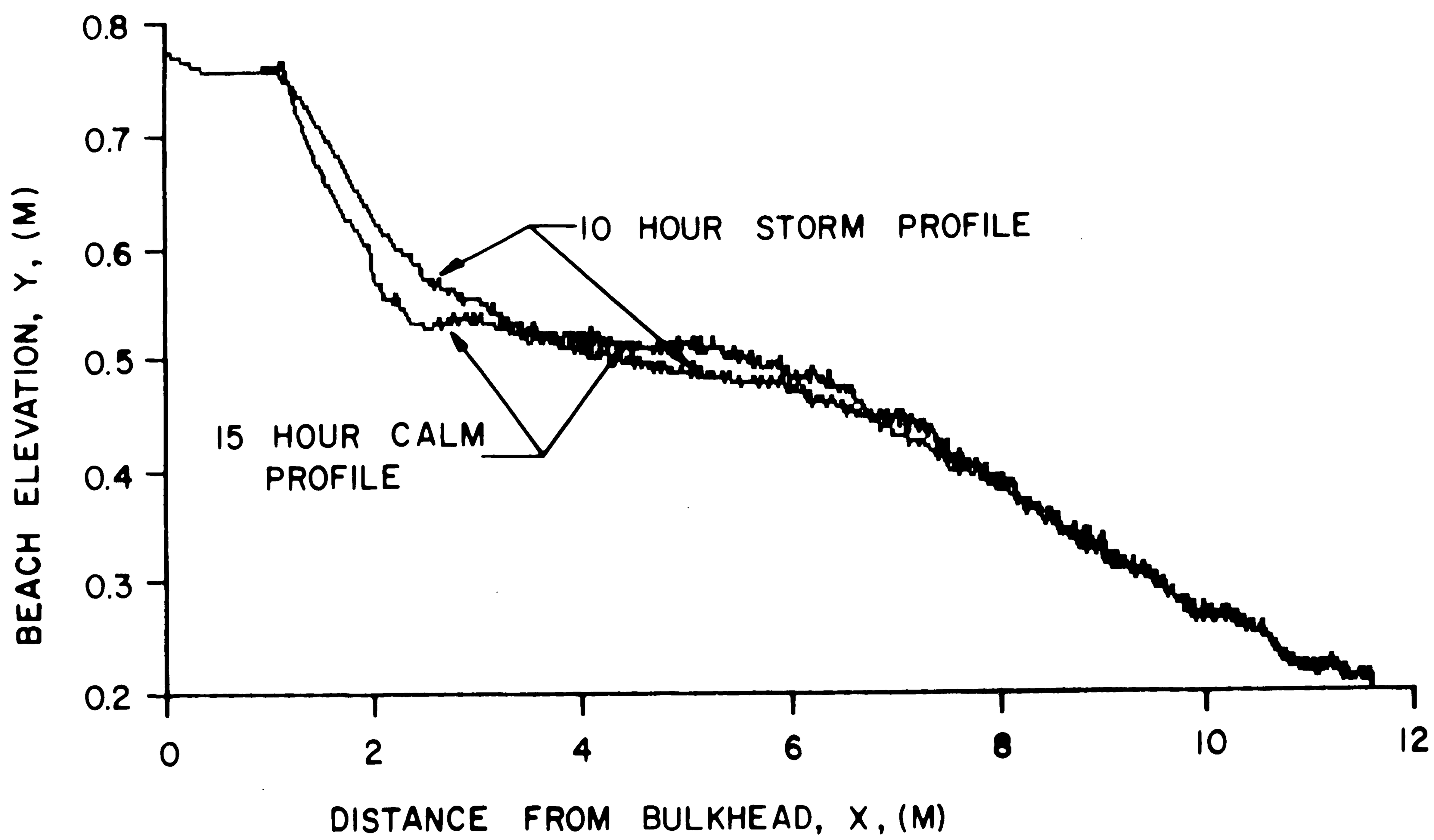

FIGURE 10. PLOT OF THE 10 HOUR STORM AND 15 HOUR CALM PROFILES – THEORY VERIFICATION

Figure 9 shows the beach profile after 10 hours of storm simulation: the rectangular symbols depict initial conditions. The storm simulation resulted in a steepened beach face which "cut back" towards the bulkhead. In addition, a longshore bar formed. These features indicate a typical storm beach profile.

Figure 10 shows the 10-hour storm profile and the resulting profile after 25 hours of simulation; 10 hours of storm conditions and 15 hours of calm conditions. During the calm test, the beach face steepened further with no additional cutback in berm location. Also, the longshore bar moved toward the beach face. These two features are typical of rebuilding or accretive beaches.

3.5.2 The Wave Spectrum, 0.457 m Depth

An initial base test was conducted to observe the nourished profile response to a period of storm waves followed by a period of calm waves. JONSWAP wave spectra representing these wave climates and having significant wave heights of 0.030 m and 0.091 m were chosen for the calm and storm wave climates respectively. The significant wave period was chosen at 1.6 seconds (see Table 2). A standing water depth in the wave flume of 0.457 m was selected to insure that the above wave heights could be generated.

The testing program consisted of three consecutive 6-hour storm wave runs followed by three consecutive 10-hour calm wave runs. The longer calm runs were necessary to fully observe the rebuilding process - onshore/offshore transport rates of accretive wave conditions are by nature slower than that of erosive conditions. An initial benchmark profile was recorded to verify the accuracy of the nourished beach profile construction. In addition, individual profiles were recorded after each storm/calm event for observation and discussion purposes. These profiles are shown in Figure 11.

The 0.457 m depth testing program was terminated after the second 6-hour storm run. During the storm wave runs, steepening of the beach face and longshore bar formation occurred, but the profile did not cutback into the berm crest (see Figure 11). This was due to limited wave run-up and overtopping. Wave run-up and overtopping occur frequently during storm as evidenced by a narrowing of the berm width. Hence, the 0.457 m water depth was deemed inadequate - an increase in the still water level (as typically occurs during storms) was necessary to better duplicate prototype beach profile behavior.

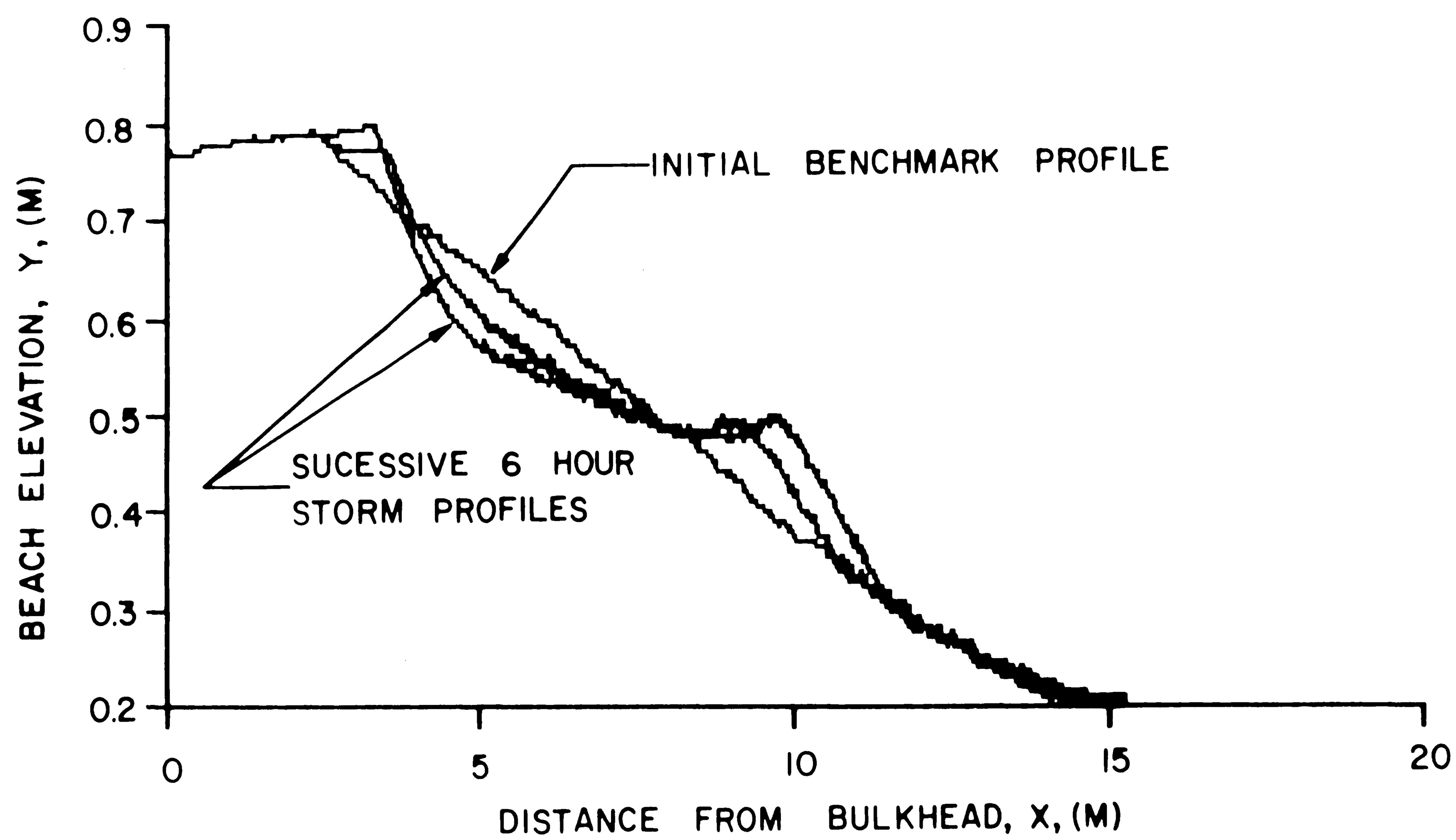

FIGURE 11. BEACH PROFILES FOR THE 0.457 M
WATER DEPTH TEST

3.5.3 The Wave Spectrum, 0.483 m Depth

The 0.457 m water depth of section 3.5.2 was selected to insure that the 0.030 m and 0.091 m significant wave heights could be generated (a shallower still water depth would not allow the wave train to fully develop, i.e., some individual waves would break upon the generator's paddles). During a storm event, the SWL rises due to wave setup and storm surge. To account for storm surge, the SWL was increased by 0.026 m to a depth of 0.483 m. The Simpulse Program was rerun for a water depth of 0.483 m, the other input parameters remained unchanged from those listed in section 3.5.2. The 0.483 m depth was selected as the operating depth for the storm wave runs while the 0.457 m depth was used for the calm wave runs.

3.5.4 Verification of the Wave Generator Program (Simpulse)

3.5.4.1 Hs Values

Due to energy loses at the generator and wave reflection at the toe structure, the generated spectrum may not be identical to the JONSWAP spectrum calculated by the Simpulse Program. Thus, the actual spectrum must be verified to assure it's properties are within acceptable limits of the JONSWAP spectrum calculated by Simpulse.

The Simpulse Program calculates the required settings for the selected significant wave height, Hs. Hence, calculating Hs from a portion of the generated wave record and comparing it to the Simpulse input value serves as a check on the accuracy of both the generated spectrum and the wave gage.

For each test spectrum, 5000 water surface elevation readings (N_T) were recorded over five 2-minute time intervals. These values of N_T were plotted with respect to time to obtain a wave record. Individual wave heights were determined using the zero- upcrossing method and the highest 33 % were averaged to yield a significant wave height for each test spectrum as shown in Table 3. Also shown in the table is the inputted significant wave height of the Simpulse Program.

Table 3. Significant Wave Heights for the Individual Test Spectra

spectrum	generated Hs	calculated Hs (Simpulse)
storm wave climate	0.073 m	0.091 m
calm wave climate	0.029 m	0.030 m

3.5.4.2 Hmo Values

Another representative wave height is the zero moment height, Hmo. This wave height is based upon the average energy of the wave train rather than an average of individual wave heights as used in the Hs definition. Hmo can be expressed by the following formula (10):

$$Hmo = 4(\sum N_T{}^2/N*)^{1/2} \qquad \text{Eq. 5}$$

N* is the number of water surface elevation readings.

Hmo values were calculated from the same wave record used in the significant wave height section. The results are shown in Table 4. Also listed in this table are Hs values which correlate to the Hmo values (field observations indicate Hs is approximately 0.95 Hmo for deep water waves [17]). These values of Hs correlate well to those presented in Table 3. Thus, Hs can be considered approximately equal to Hmo in this investigation.

Table 4. Zero Spectral Moment wave Heights and Correlated Significant Wave Heights for the Individual Test Spectra

spectrum	Hmo	Hs (0.95 * Hmo)
storm wave climate	0.079 m	0.075 m +
calm wave climate	0.030 m	0.029 m +

\+ indicates correlated Hs values - see Table 3, generated Hs for comparision

4.0 EXPERIMENTAL PROGRAM AND DATA

4.1 Overview

The testing program was conducted in two phases. The first phase investigated the response of the nourished, non-perched beach profile while the second phase investigated nourished beach profile response with a toe structure of set positions resulting in four test case comparisons. Initial and intermediary beach profiles were recorded at specific time intervals to facilitate comparison. All tests were conducted using the irregular wave spectra in Sections 3.5.2 and 3.5.3. An in-depth description of the testing program follows.

4.2 Step by Step Procedure

4.2.1 Non-Perched Beach - Base Test Case

The nourished profile described in Section 3.4.3 was constructed by the string-line grading method. This initial or benchmark profile was verified and recorded by a point gage to assure accuracy.

The wave spectra of Sections 3.5.2 and 3.5.3 represented the prototype calm and storm wave conditions respectively. The testing program - the number and duration of calm and storm wave attack test periods -

was developed as the base test case progressed. Six hour (30 prototype hours) storm wave attacks were repeated until the beach profile exhibited a trend toward an equilibrium condition. These were followed by a series of 18 hour (90 prototype hours) and 36 hour (180 prototype hours) calm wave attacks. Again, a trend towards beach profile stabilization defined the required number of calm wave repetitions.

The final testing program for the base test (non-perched beach) case and each of the perched beach test cases is as follows:

1. Four 6-hour storm wave test periods
2. One 18-hour storm wave test periods
3. Two 18-hour calm wave test periods
4. Three 36-hour calm wave test periods

4.2.2 Perched Beach Test Cases

The nourished profile used in the perched beach test cases is shown in Figure 12. Note the variation in the position of the toe structure as shown by the solid and dashed toe structure outlines. This results in four test cases; see Table 5, cases 2 through 5 (four toe structure locations at 1.0, 0.5, 0.25, and zero incident significant wave heights respectively). Landward of the

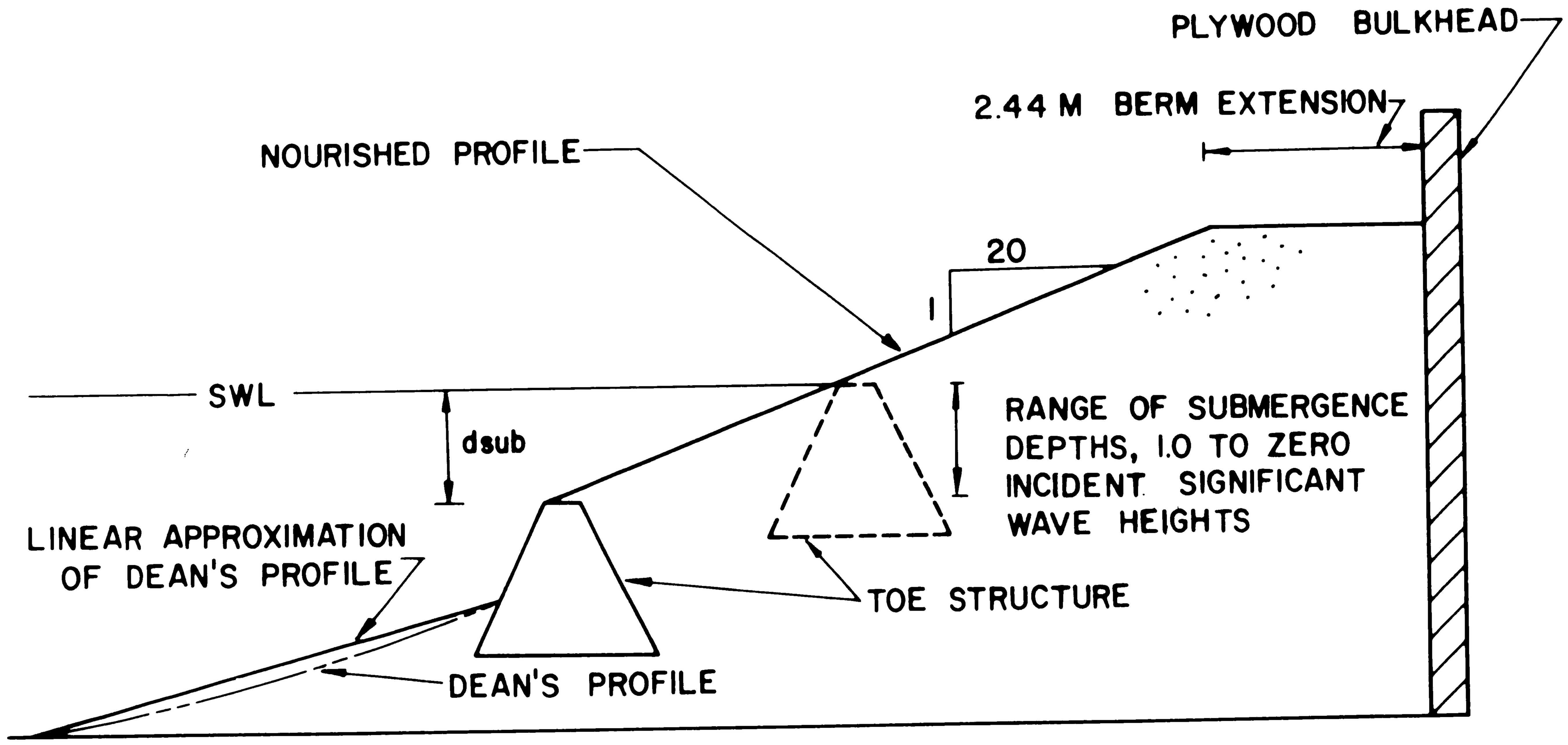

FIGURE 12. THE NOURISHED BEACH PROFILE FOR THE PERCHED BEACH TEST CASES

(NOT TO SCALE)

toe structure, the nourished profile is identical to that discussed in Section 3.4.3. Seaward, the nourished profile consists of a linear approximation of Dean's profile (see Figure 8).

Table 5. Description of Test Cases

test case no.	beach condition	toe structure crest submergence	testing completed storm waves	testing completed calm waves
1a	perched	--	X	---
1b	perched	--	---	X
2	non-perched	1.0 Hs	X	---
3	non-perched	0.5 Hs	X	---
4	non-perched	0.25 Hs	X	---
5	non-perched	0.0 Hs	X	---

The testing program for the perched beach test cases was identical to the base test case. Initially, the toe structure's crest was located at 1.0 incident significant wave height below the SWL. During the storm wave attacks, poor performance (rapid perched beach erosion) was noted. Based on the poor performance and the lengthy time required for the rebuilding (calm) conditions to reach equilibrium, the submergence depth of the toe structure was decreased to 0.5 incident significant wave height below SWL. Thus, the calm wave conditions of the testing program were not conducted for Test Case 2. Similarly, calm wave conditions were omitted from Test Cases 3, 4, and 5.

4.3 Data Collected

Results of the testing program are presented in the form of successive beach profiles for the time intervals given in Section 4.2.1. Figures 13 through 18 depict the resulting beach profiles for the non-perched beach test cases (tests 1a & 1b) as well as the perched beach test cases (tests 2 through 5).

Note that these figures are only a portion of the entire beach profile. However, they depict the primary focus of interest - that portion of the beach profile extending seaward from the plywood bulkhead to the submerged toe structure. Each figure represents one test case. Beach profiles are plotted and labelled successively to illustrate the erosion/accretion occurring over time.

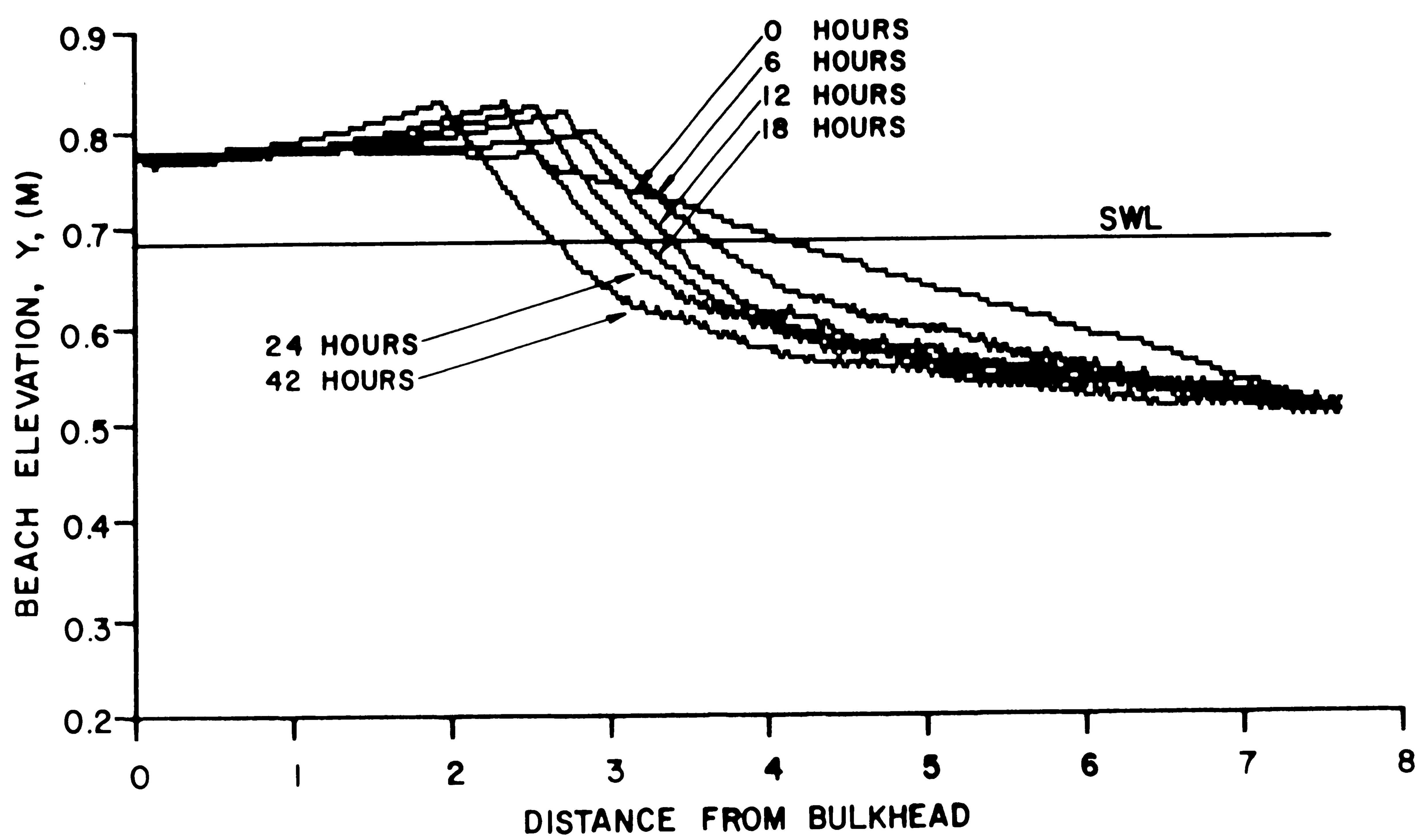

FIGURE 13. CUMULATIVE BEACH PROFILE, TEST Ia

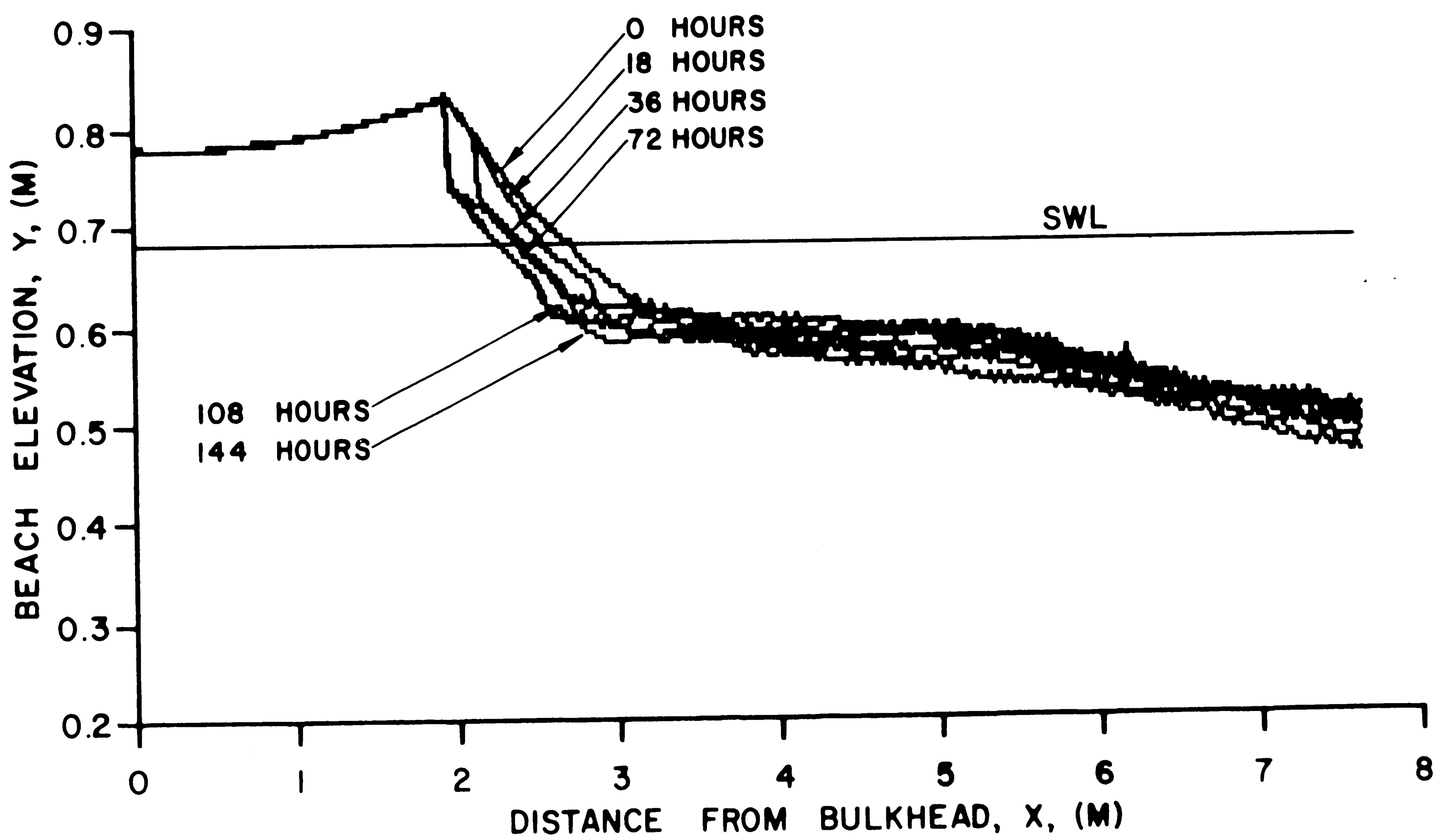

FIGURE 14. CUMULATIVE BEACH PROFILES, TEST Ib

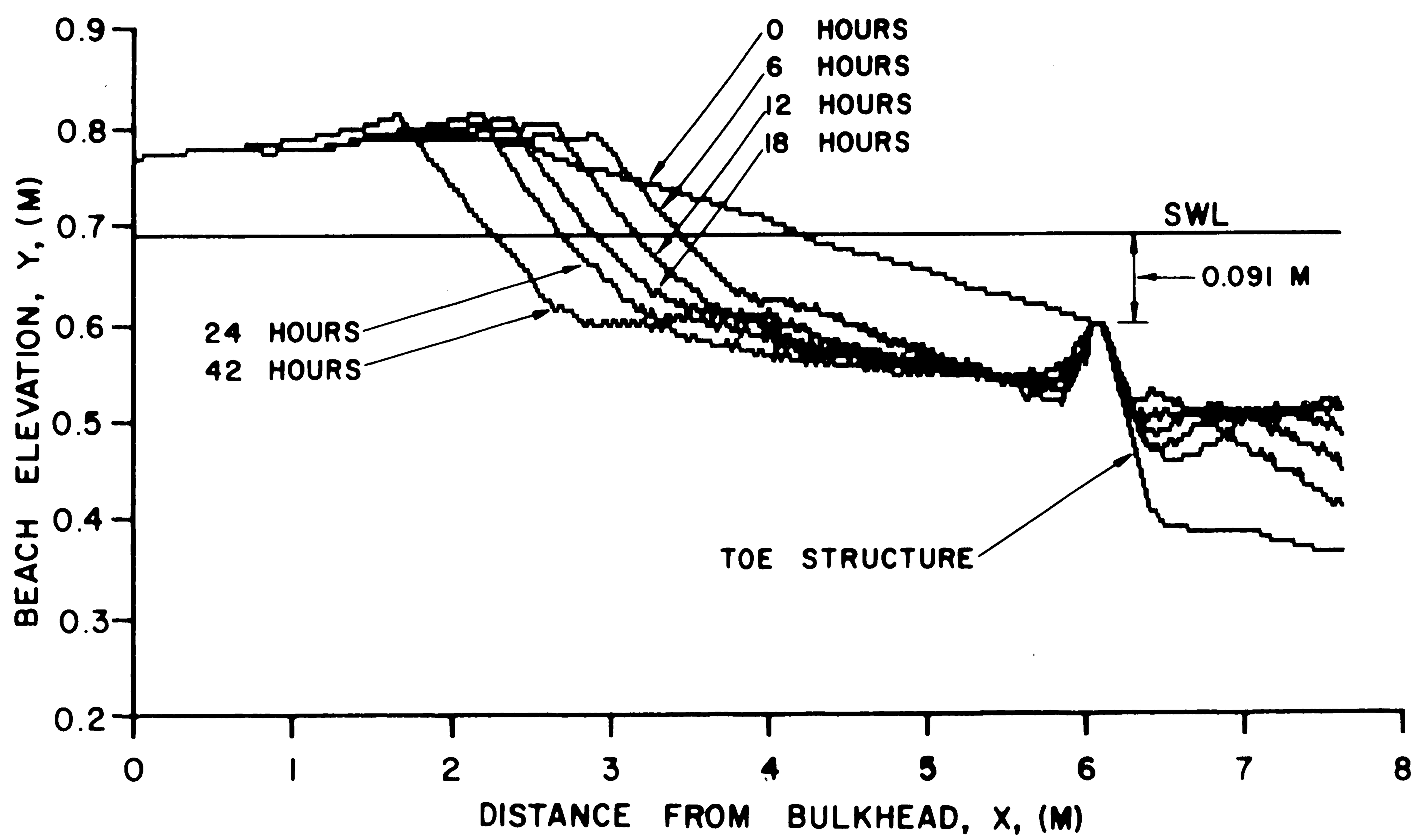

FIGURE 15. CUMULATIVE BEACH PROFILES, TEST 2

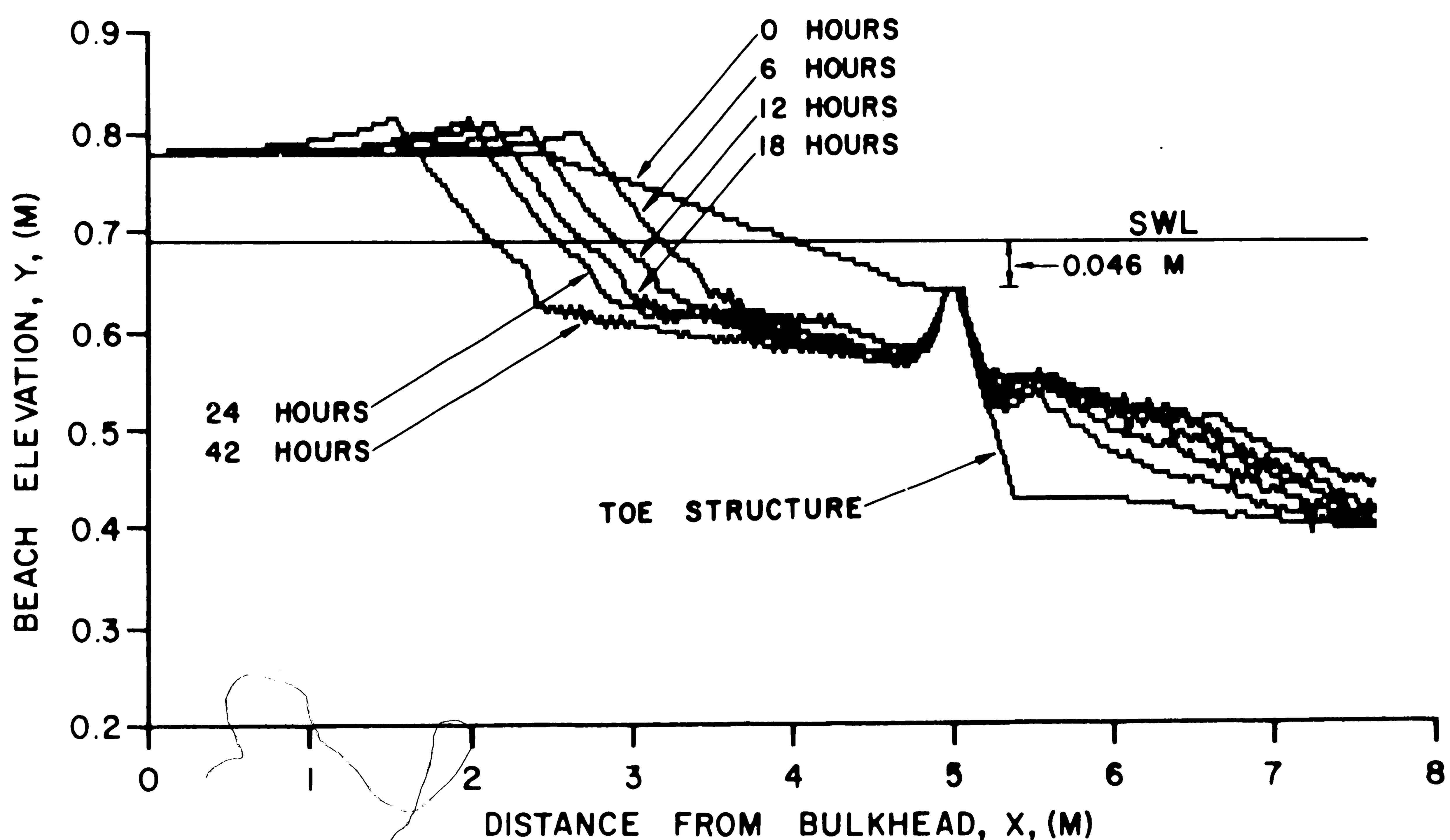

FIGURE 16. CUMULATIVE BEACH PROFILES, TEST 3

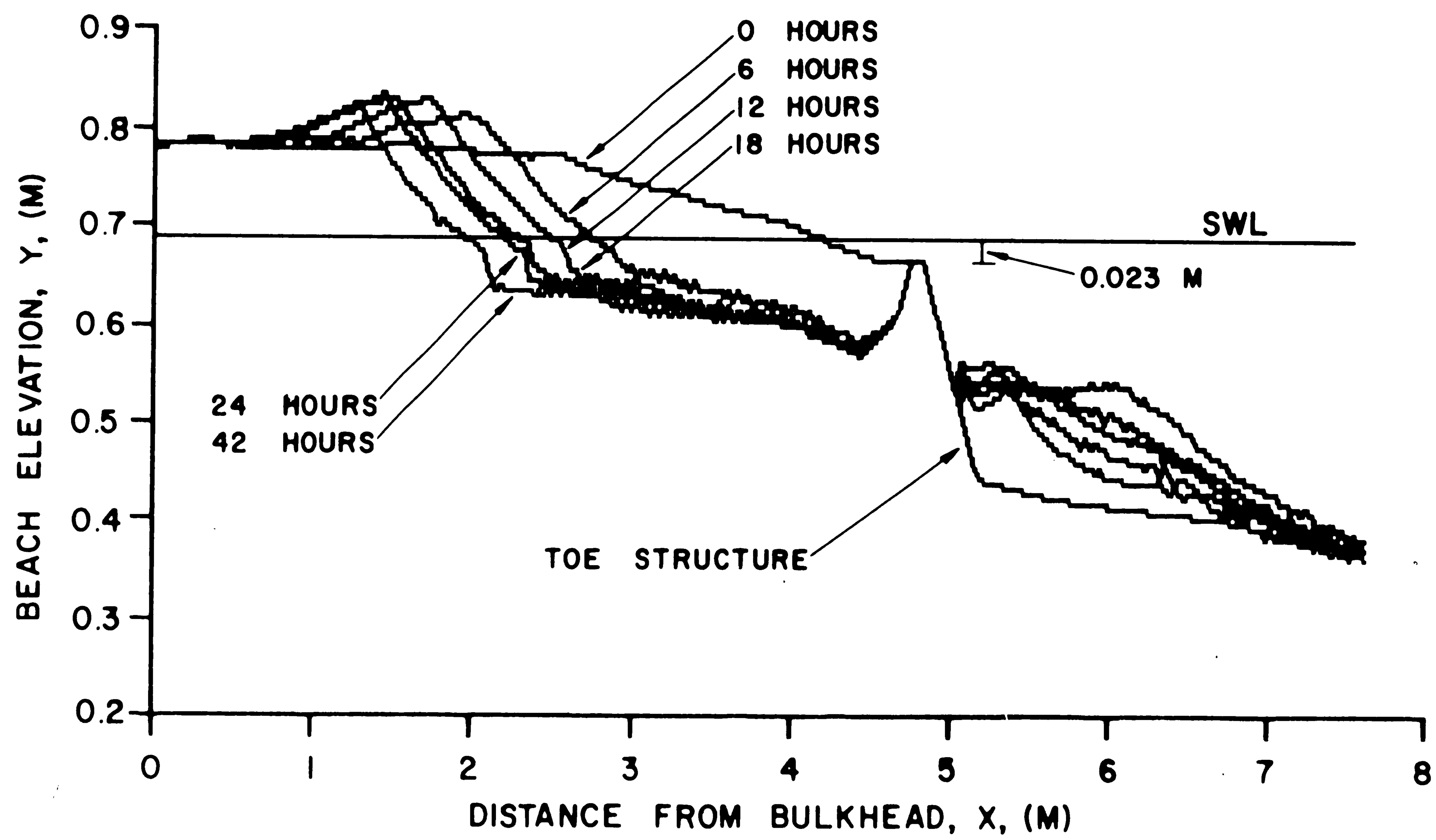

FIGURE 17. CUMULATIVE BEACH PROFILES, TEST 4

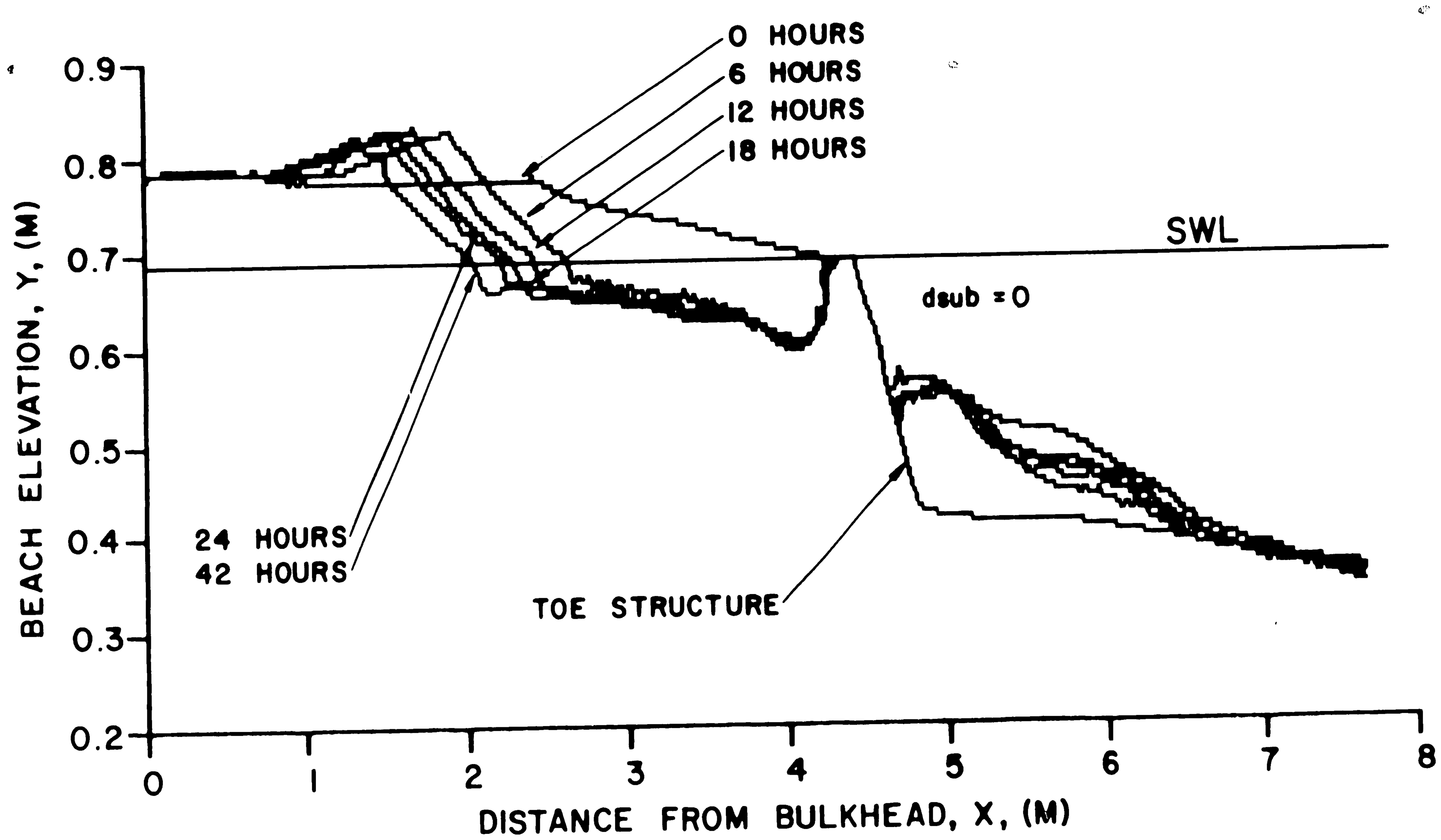

FIGURE 18. CUMULATIVE BEACH PROFILES, TEST 5

5.0 DATA ANALYSIS AND DISCUSSION

5.1 Accuracy of the Initial Profile Construction

The initial beach profile should be constructed as close as possible to the specified profile (see Section 3.4.3). Vertical deviation of the specified and constructed beach profiles was calculated and the average difference was found to be 3.1 mm. The maximum vertical deviation was found to be 14.1 mm (in isolated cases). Hence test results presented hereafter require no adjustment due to variation of initial test conditions.

5.2 Beach Profile Erosion

5.2.1 Linear Rate of Erosion

Beach profile erosion is defined, for this study, in terms of the rate of horizontal profile retreat, ER_l. More specifically, it is the horizontal distance between successive beach profiles measured at the SWL, divided by the model time (6 or 18 hours) required to generate the particular profiles. The SWL was chosen because it is a meaningful vertical datum on the active portion of the profile and it allows consistant comparison between test cases.

Table 6 lists the linear rates of erosion for each successive storm profile for Tests 1a through 5. As dsub approaches zero, the beach profile retreat occurring during the first 6-hour storm simulation increased from 35% (Test 1a) to 70% (Test 5) of the overall beach profile retreat (see Table 6). Thus, a large portion of the total beach profile erosion occurs during the first 6-hour storm wave attack.

After the initial 6-hour storm simulation, ER_l is substantially reduced for all test cases with the greatest reduction occurring in Test Case 5 (see Table 6). The lowest values of ER_l (after the initial 6-hour storm simulation) occur when dsub equals 0.25 Hs and zero Hs, Test Cases 4 and 5 respectively. This implies that these test cases are nearing equilibrium, while test cases 1a, 2, & 3 require additional storm wave attack (and additional beach profile erosion) to reach equilibrium.

Figure 19 depicts the final beach profile (i.e., the 42-hour storm profile) for each of the test cases. Note that the berm crest retreats farther landward for each successive decrease in dsub. This is due, in part, to the non-equilibrium conditions in Tests 1a, 2, & 3. However, the horizontal distance from the submerged toe structure to the beach face (measured at the SWL)

TABLE 6. LINEAR RATES OF EROSION OF EACH SUCESSIVE STORM PROFILE FOR TESTS Ia, 2, 3, 4, & 5.

TEST CASE	dsub	ERI (M/HR) 0-6 HRS.	ERI (M/HR) 6-12 HRS.	ERI (M/HR) 12-18 HRS.	ERI (M/HR) 18-24 HRS.	ERI (M/HR) 24-42 HRS.
Ia	—	0.083	0.039	0.031	0.033	0.020
2	1.0 Hs	0.131	0.044	0.045	0.036	0.023
3	0.5 Hs	0.136	0.043	0.037	0.027	0.023
4	0.25 Hs	0.238	0.041	0.036	0.012	0.014
5	0.0 Hs	0.277	0.036	0.020	0.011	0.007

NOTE: TABULAR VALUES OF ERI REPRESENT MODEL RATES OF EROSION MEASURED FROM A FIXED REFERENCE POINT

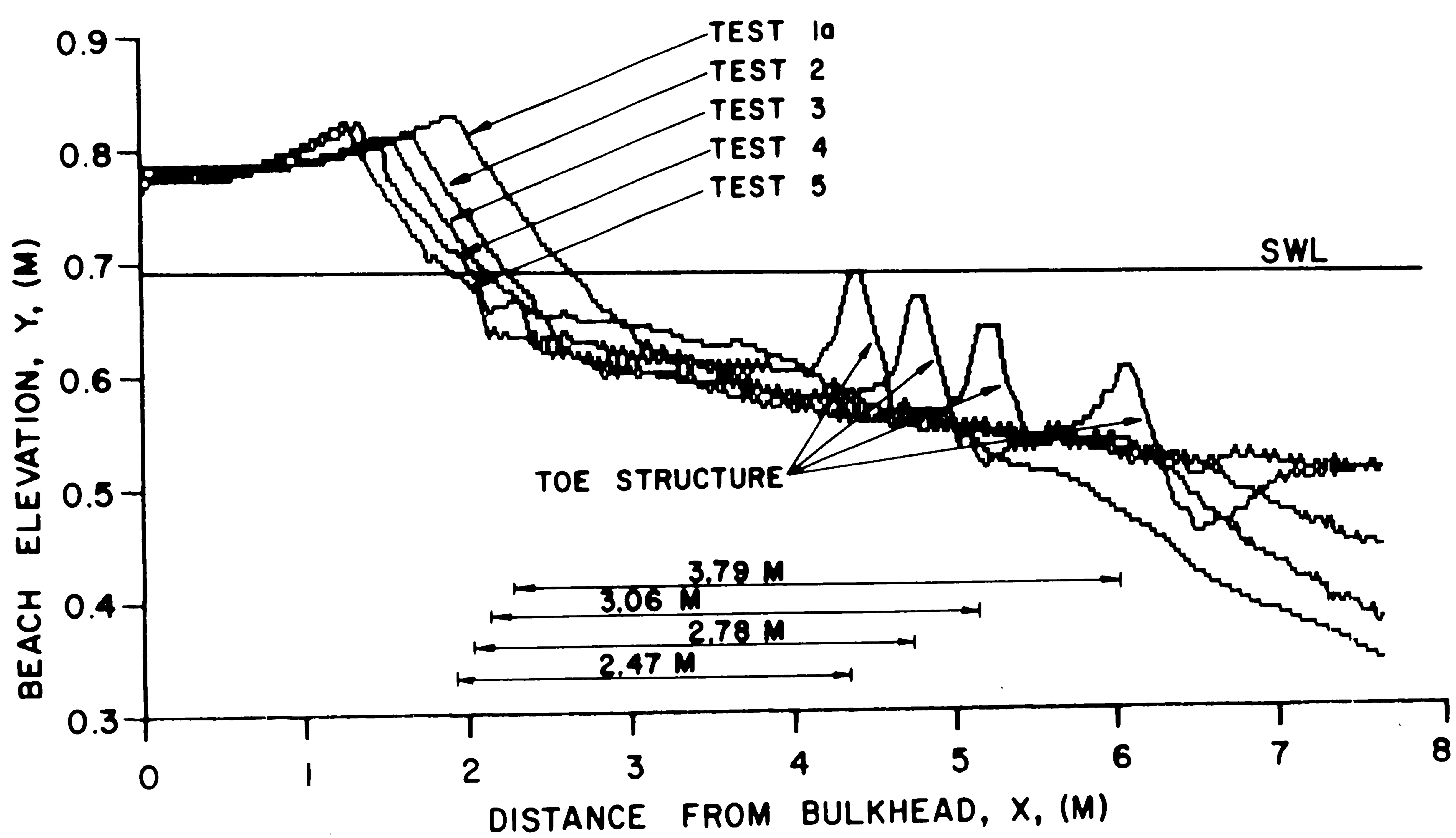

FIGURE 19. A COMPARISON OF THE 42-HOUR STORM PROFILES FOR TESTS 1a, 2, 3, 4, & 5

decreased slightly as dsub approached zero. The total beach face retreat was found to be 3.79 m for Test Case 2 (dsub = 1.0 Hs) while the total retreat for Test Case 5 (dsub = 0.25 Hs) was 2.47 m. In earlier test cases, more retreat is likely due to the additional storm simulation period required to achieve equilibrium. Hence, a toe structure located near the SWL will produce the least amount of beach face retreat from the structure.

5.2.2 Beach Profile Immediately Landward of the Submerged Toe Structure

5.2.2.1 Influencing Factors

A stilling well installed landward of the submerged toe structure recorded no appreciable wave set-up (i.e. the water level within the stilling well coincided with that of the SWL) for all of the test cases. Wave overtopping of the submerged toe sructure was not directly measured in this study. However, the greatest amount of overtopping was observed in Test Case 1a (at the maximum value of dsub).

Recently, Seelig determined wave height transmission coefficients, Kt, for numerous submerged structures (19). Typical Kt values when dsub equals

1.0, 0.5, and zero Hs are 0.78, 0.72, and 0.33, respectively. Seelig's data indicates that Kt, and thus the transmitted significant wave height, decreases as dsub approaches zero. The above Kt values correspond to transmitted significant wave heights of 0.071, 0.066, and 0.030 m for Test Cases 2, 3, and 5, respectively (Test Case 4 could not be correlated to Seelig's data).

5.2.2.2 The Resulting Beach Profile

The effect of the above factors on the beach profile can be seen in Figure 19. The beach profiles (for all the perched beach test cases) immediately landward of the submerged toe structure can be divided into three sections, each with common features. Since geometries of these features remained about the same for all test cases, average values were calculated for each and are reported below. Progressing landward from the toe structure, there was:

1. A consistant water depth immediately landward of the toe structure of approximately two-third's of the incident significant wave height.
2. A gentle upward-sloping (approximately 1:45) submerged plateau to the steeper sloping beach face.

3. An abrupt increase in the beach profile slope to approximately 1:5 on the beach face up to the berm crest.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

Based on general observations, resulting beach profiles, and the discussion above, the following conclusions were reached:

1. The location of the submerged toe structure - expressed as a submergence depth of the structure's crest elevation, dsub - had some effect on the final beach profile. As dsub approached zero, the horizontal distance from the submerged toe structure to the beach face (measured at the SWL) decreased. Thus, an optimal location for a structure of this type is near the SWL.

2. One-third to two-third's of the overall beach profile retreat occurred during the first six-hour storm simulation period. The subsequent storm periods exhibited a marked decrease in beach profile retreat. Equilibrium conditions were achieved for Test Cases 4 & 5.

3. As dsub approached zero, the time required for the beach profile to reach equilibrium decreased. Test Cases 4 & 5 support this as ER1 values decreased rapidly (an ER_1 value of zero indicates equilibrium conditions have been achieved for that particular test run).

6.2 Recommendations

Recommendations for further testing:

1. Additional testing is required in Test Cases 1, 2, and 3 to fully evaluate the equilibrium storm profiles of each submerged toe structure location.

2. For a given toe structure location, say dsub equals zero, simulate a series of storm periods in which the significant wave height and peak wave period are varied. For example, let Hs = 0.091 m (as in this study) and vary the peak wave period, from 2.5 to 3.5 seconds, or let the peak wave period equal 1.6 seconds and vary the significant wave height from 0.07 to 0.11 m. The objective of these tests should be to develop a relationship between the wave climate and the resulting beach profile erosion (specifically the horizontal distance from the toe structure to the beach face, measured at the SWL) for the optimum toe structure location.

3. Constructing the nourished beach profile as described in Figure 20 and setting dsub equal to zero, repeat the testing program described in this study. The objective is to verify that the nourished profile mentioned above is indeed an optimal one.

Design Recommendations:

1. The perched beach concept can be successfully implemented for shoreline stabilization. Figure 20, which is based on the resulting beach profile immediately landward of the submerged toe structure (see Section 5.2.2.2), depicts the optimal location and geometry for the elements of the perched beach concept. A discussion of Figure 20 is located at the end of this section.

2. Based on the 42-hour storm profiles and Figure 20, the submerged toe structure's crest elevation should be located near the SWL. This results in a minimum of beach profile retreat while maximizing the savings of beach fill material used in constructing the nourished profile.

3. An additional constraint on the toe structure's location is imposed by the total beach face retreat shown in Figure 20. Given the design criteria of berm width and berm crest elevation, the submerged toe structure must be a sufficient distance seaward of the berm crest to accommodate the expected beach face retreat (40 to 50 Hs as found in this study) in addition to having it's crest near the SWL.

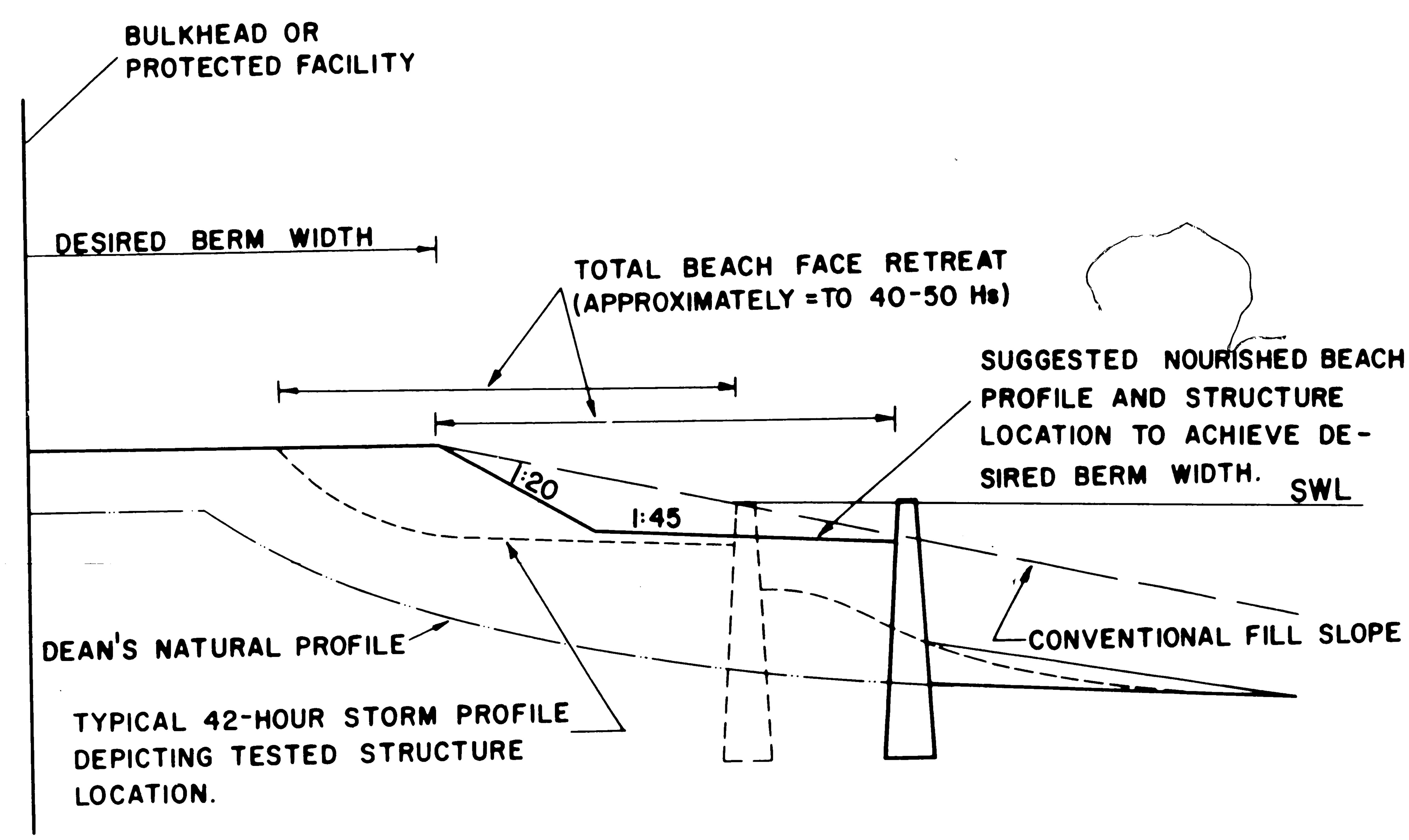

FIGURE 20. SUGGESTED NOURISHED PROFILE AND SUBMERGED TOE STRUCTURE LOCATION, PERCHED BEACH CONCEPT.

4. The nourished beach profile (above) results in a savings of beach fill material from three areas:

   - The volume of the submerged toe structure.
   - The volume seaward of the submerged toe structure (that difference in volume between conventional fill methods and the linear approximation of Dean's profile used in this study).
   - The volume landward of the submerged toe structure (see Figure 20).

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APPENDIX A

Comprehensive Beach Profiles

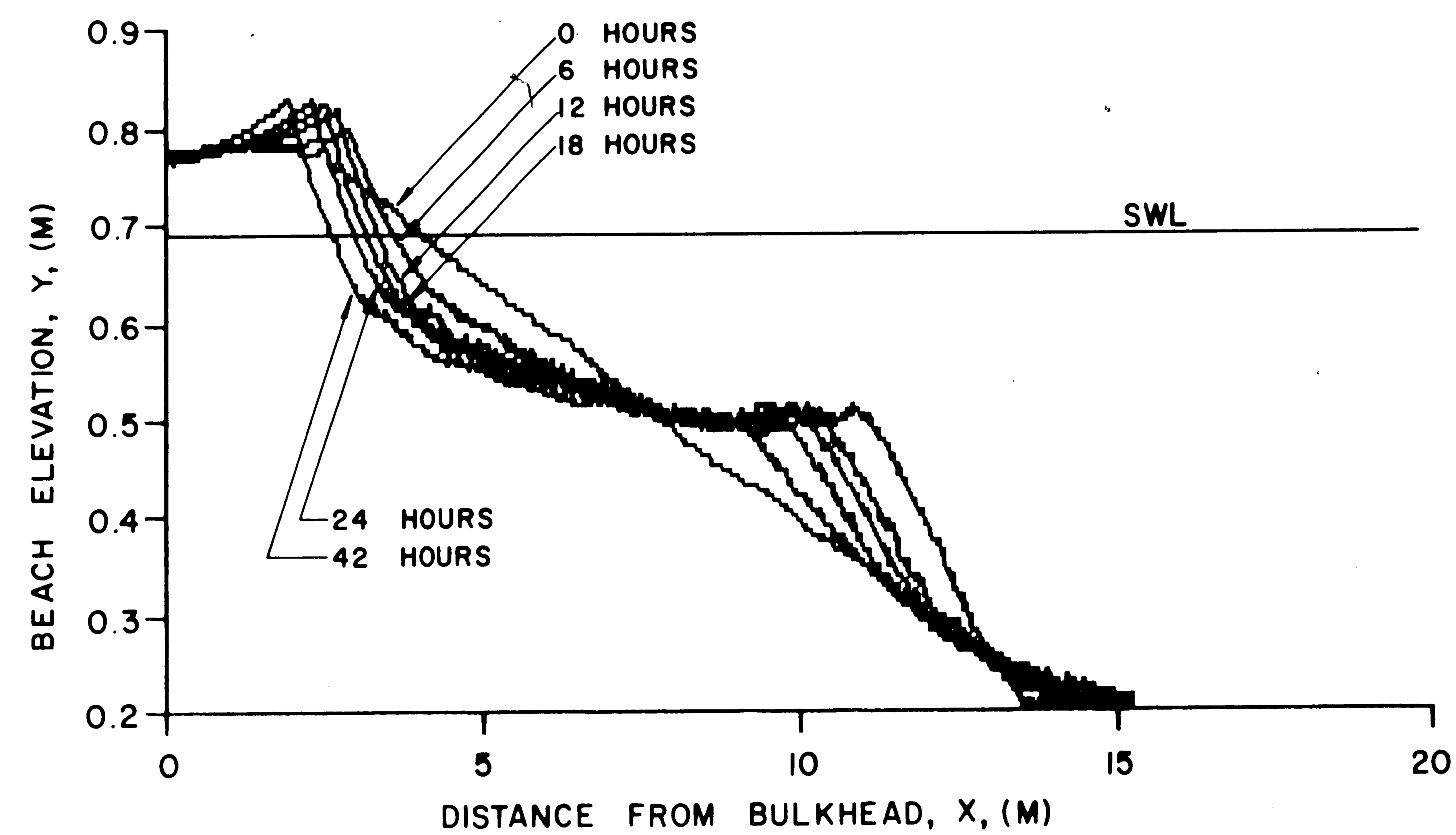

FIGURE B-1. CUMULATIVE BEACH PROFILES CORRESPONDING TO 0, 6, 12, 18, 24, & 42 HOURS OF STORM WAVE ATTACK, TEST 1a

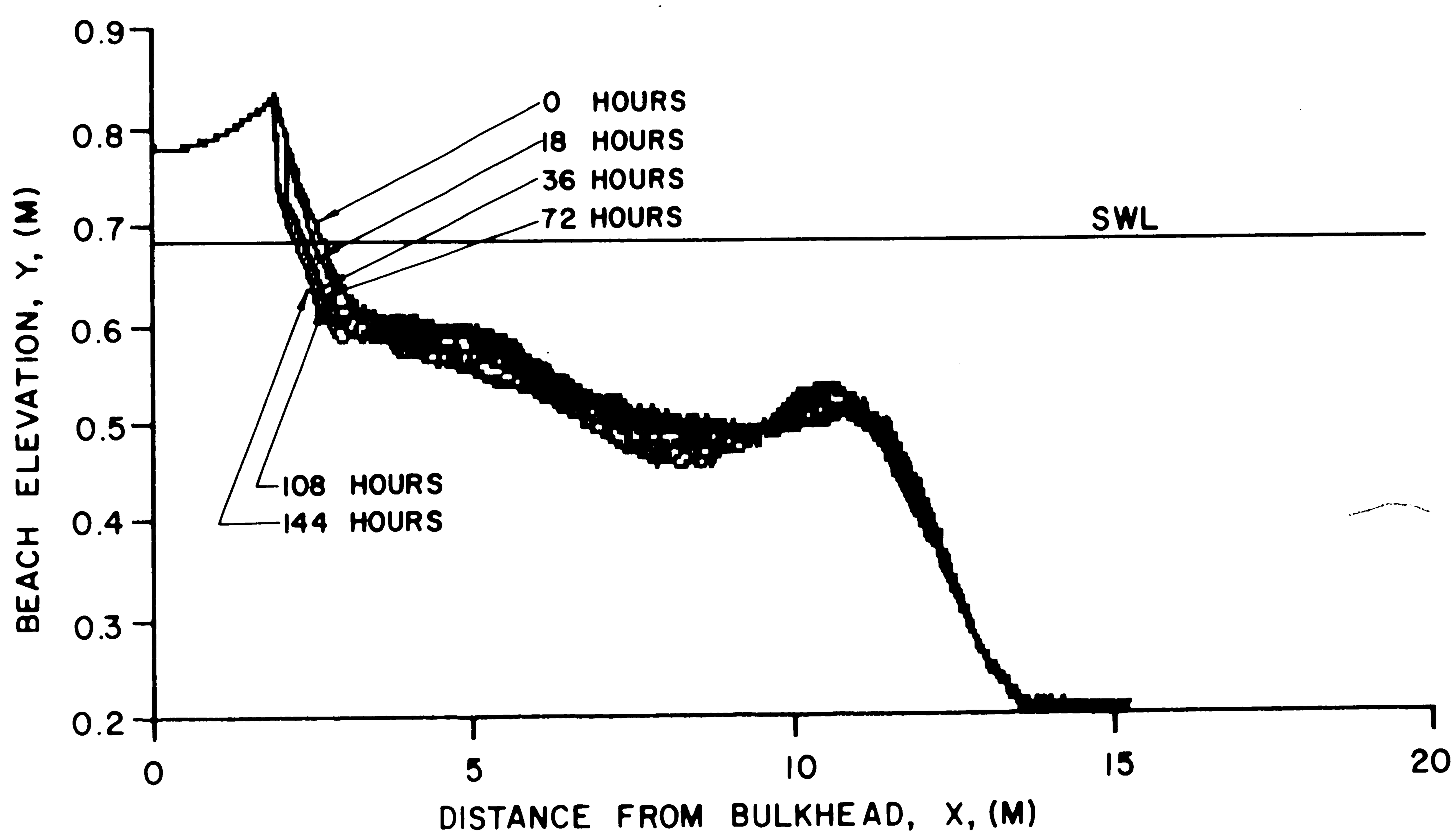

FIGURE B-2. CUMULATIVE BEACH PROFILES CORRESPONDING TO 0, 18, 36, 72, 108, & 144 HOURS OF CALM WAVE ATTACK, TEST Ib

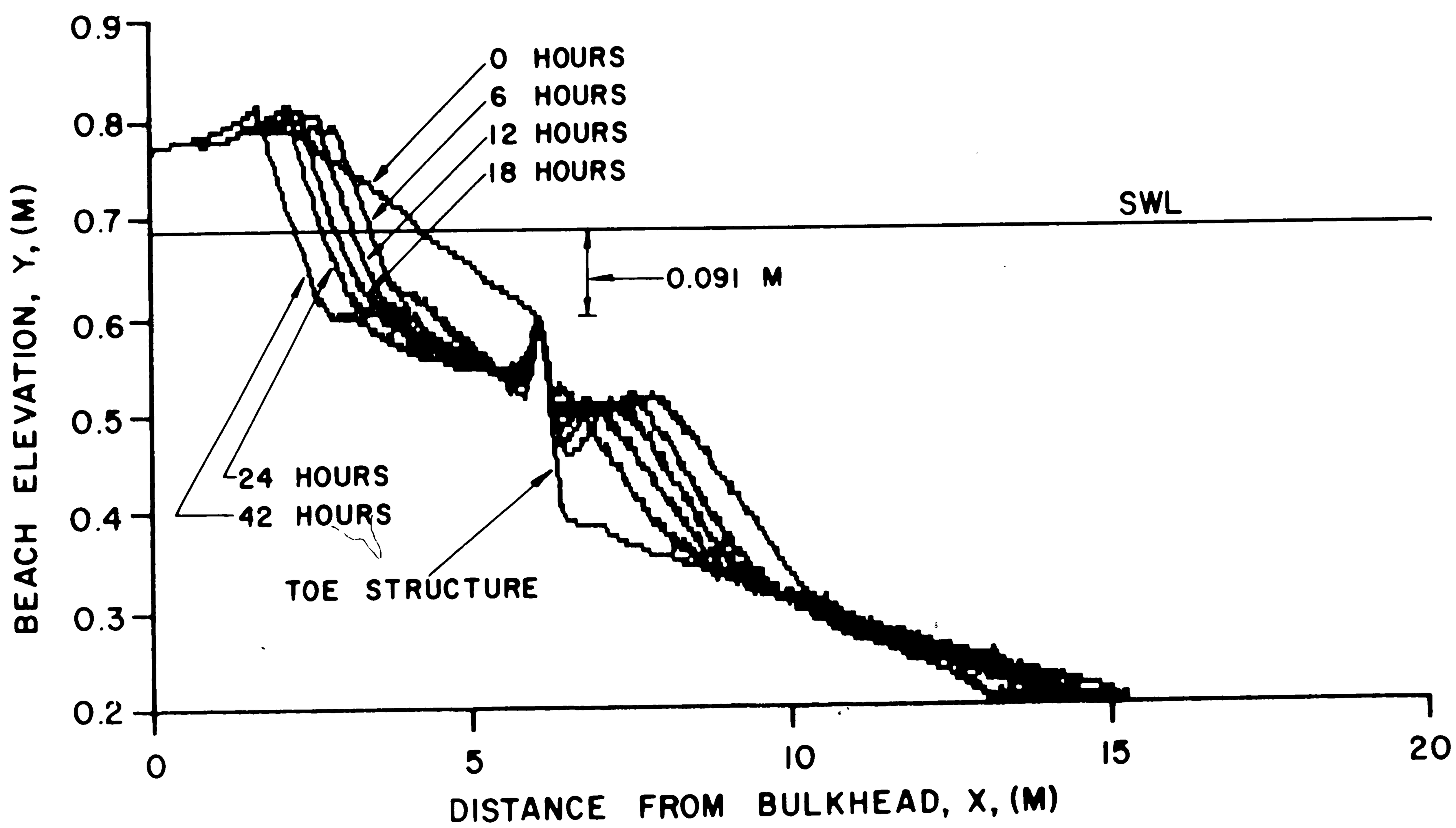

FIGURE B-3. CUMULATIVE BEACH PROFILES CORRESPONDING TO 0, 6, 12, 18, 24, & 42 HOURS OF STORM WAVE ATTACK, TEST 2

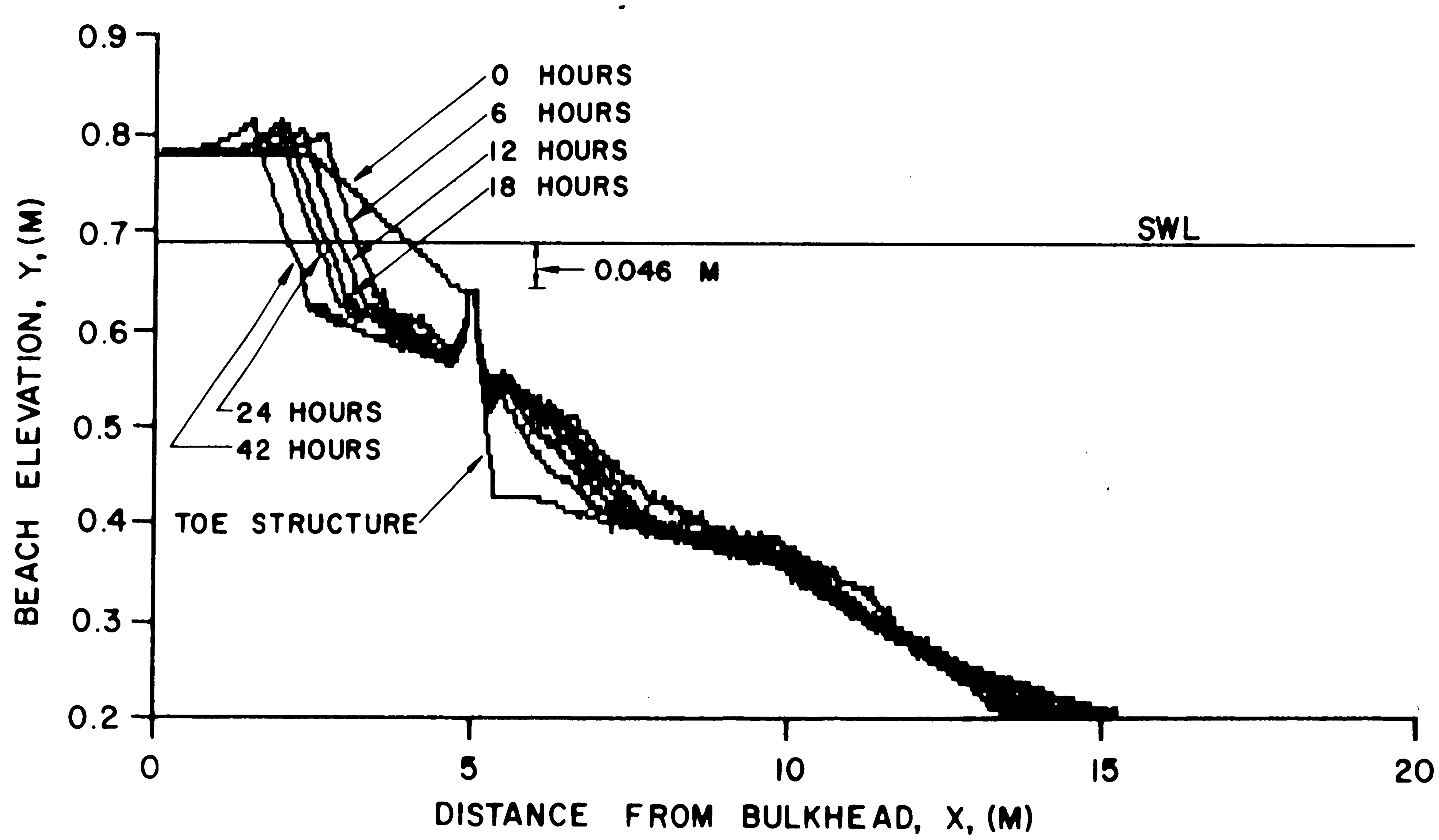

FIGURE B-4. CUMULATIVE BEACH PROFILES CORRESPONDING TO 0, 6, 12, 18, 24, & 42 HOURS OF STORM WAVE ATTACK, TEST 3

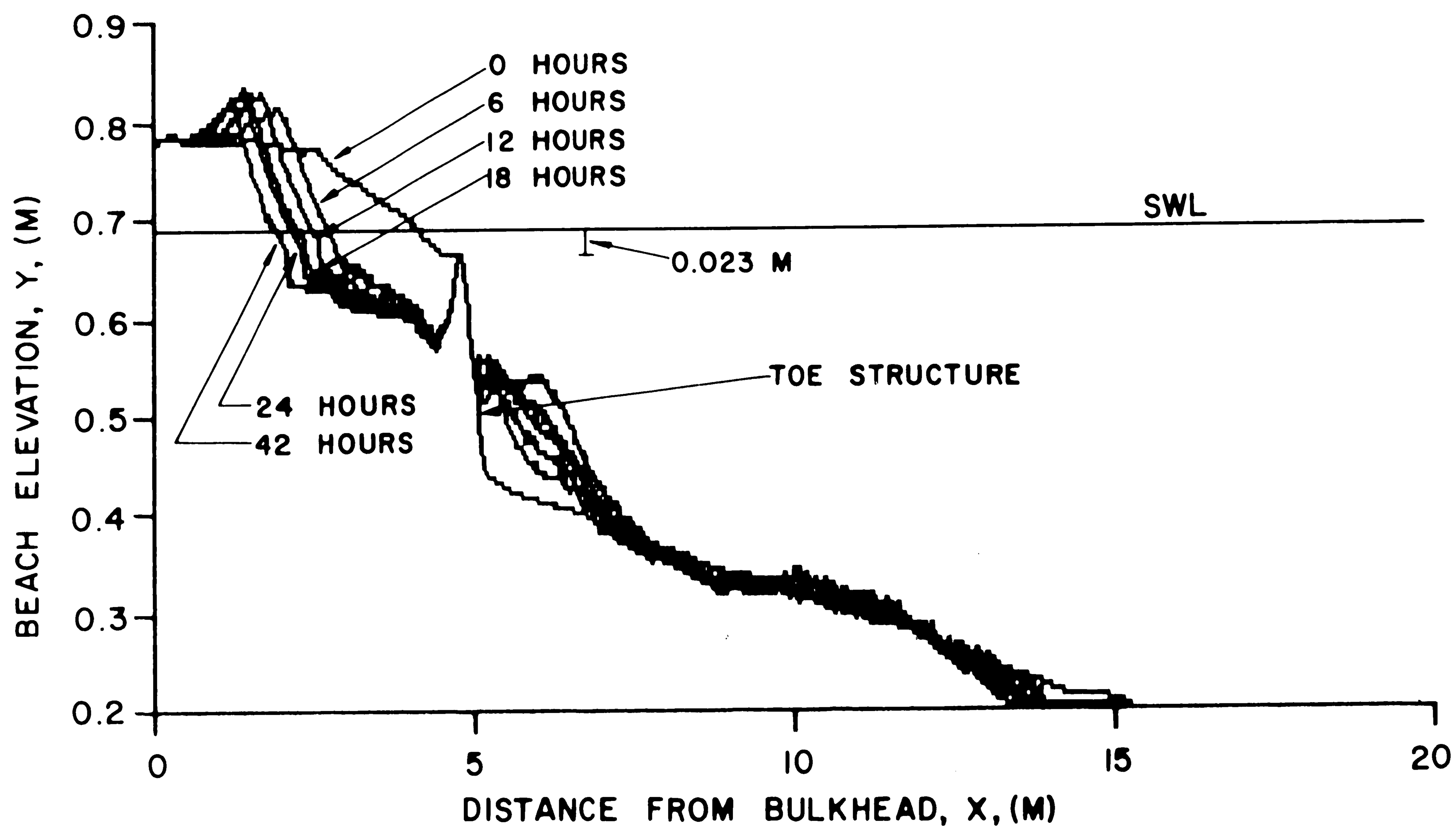

FIGURE B-5. CUMULATIVE BEACH PROFILES CORRESPONDING TO 0, 6, 12, 18, 24, & 42 HOUR STORM WAVE ATTACK, TEST 4

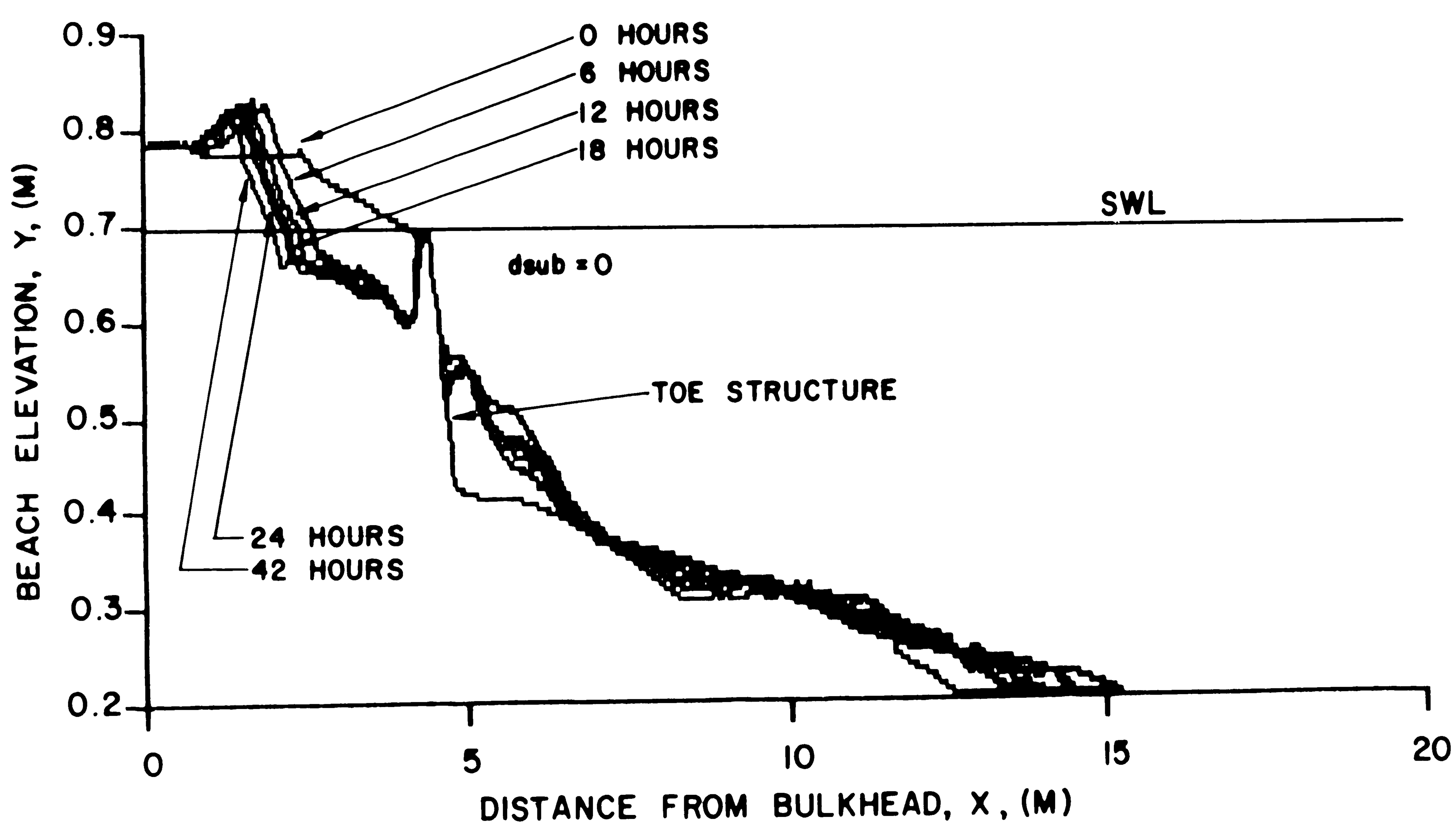

FIGURE B-6. CULMULATIVE BEACH PROFILES CORRESPONDING TO 0, 6, 12, 18, 24, & 42 HOUR STORM WAVE ATTACK, TEST 5

APPENDIX B

Vita

NATHAN J. BEIL

PLACE AND DATE OF BIRTH

Allentown, Pennsylvania
March 5, 1961

PARENTS NAMES

Bert H. and Naomi P. Beil

EDUCATION

M.S. in Civil Engineering, Lehigh University (1987), Primary Field of Study: Hydraulics, Hydrology and Coastal Engineering.

B.S. in Civil Engineering, Lehigh University (1983), Primary Field of Study: Civil Engineering, Hydraulics and Water Resources.

Completed Various FHWA Training Courses in Hydrology, Road Design, and Public Relations while employed at Delaware Department of Transportation (1983-1986).

PROFESSIONAL EXPERIENCE

1/88 - Present	Senior Project Engineer, Kidde Consultants, Incorporated, Towson, Md. Project Engineer/Manager on various State and Federal aide highway projects. Served as a member of the Hydraulics Group within the Highway Design Section.
9/86 - 12/87	Research/Teaching Assistant, Lehigh University, Bethlehem, PA. Performed wave tank experiments and developed a master's thesis on the Perched Beach Profile Response Project. Teaching responsibilities included recitation and grading assignments for an introductory hydraulics course.

PROFESSIONAL EXPERIENCE CONTINUED

2/87 - 12/87	Hydraulic Consultant, Borough of Hellertown, Hellertown, PA. Developed a scaled street map for the borough. Organized and updated the water supply and distribution system and the sanitary sewer system. Drafted these systems onto the scaled street map. (Part-time while pursuing M.S. degree)
8/83 - 9/86	Civil Engineer III, Department of Transportation, State of Delaware. Project Engineer on various Federal Aid and State Funded road design projects. Duties included all phases of project design, plan development, and cost estimation. Headed the hydraulics group responsible for design of roadway drainage systems (open channel, closed system, and stormwater management facilities). Trained entry level engineers in design and drafting procedures. Served as Road Design's representative to the computer support section.
6/83- 8/83	Hydraulics Lab Technical Assistant, Lehigh University, Bethlehem, Pa. Constructed the model test flume for the Monksville Dam stepped spillway hydraulic model investigation. Assisted in experimental test procedures and data aquisition.
6/82 - 8/82	Engineering Technician, GS-04, U.S. Army Corps of Engineers, Picatinny Arsenal, Dover, NJ. Field Engineer responsible for the administration of military construction projects. Included inspection and supervision of construction personnel.

PUBLICATIONS

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REGISTRATION

Engineer-In-Training (Pennsylvania, 1983).

MEMBERSHIPS

American Society of Civil Engineers
American Water Resources Association