STAMFORD-NEW HAVEN DISPOSAL OPERATION MONITORING SURVEY REPORT

#7

January 30, 1980

Submitted to:

New England Division
U.S. Army Corps Of Engineers
Waltham, MA

Submitted by:

Ocean Science & Technology Division
Science Applications Inc.
Newport, RI

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INTRODUCTION

The disposal of Stamford spoils in the Central Long Island Sound Disposal Site, and the subsequent capping of this material with spoil from New Haven harbor has been carefully monitored as part of the DAMOS program since January, 1979. The results of the disposal project were reported in a series of interim reports (#1-5), and post disposal conditions as of August, 1979 were described in report #6, all of which have been published by the New England Division. A brief summary of the operation and the resulting monitoring data are presented in the following section.

BACKGROUND

Prior to disposal of Stamford spoil, two specific sites were designated for dumping; one, .5 NM south of the 1974 disposal mound, which would be capped with silt and a second, .5NM north of the original mound, that would be capped with sand. A total volume of 34,200 m^3 of Stamford spoils were deposited at the southern site and slightly more than 26,000 m^3 were dumped at the northern site (Reports # 3 & 4). Following disposal of Stamford material, spoils from the inner harbor at New Haven were used to cap the deposit until the total volume of material at the southern site was approximately 118,000 m^3 (erroneously reported in report #6 as 155,000 m^3). At the northern site, sand from the outer breakwater of New Haven Harbor was used as capping material, and a total volume of 59,000 m^3 was deposited by June, 1979.

Following the completion of the disposal operation in June, a monitoring survey was conducted in August to evaluate short term changes in the spoil mounds. As a result of this study, the estimated loss of volume from the south site was on the order of 900 m^3 while the loss from the north site was approximately 1700 m^3. These changes in volume are close to the error estimates of the procedure and must be considered insignificant. Furthermore, specific topographic features could be reproduced by comparison of vertical profiles, indicating no major changes in the surface of the spoil mounds. These losses, although insignificant, could certainly be accounted for through settling and consolidation of the spoil mound.

SURVEY RESULTS

On November 7, 1979, further monitoring of the southern site was undertaken to evaluate changes from the addition of approximately 6000 m^3 of Stamford material resulting from clean up operations in the harbor. Analysis of the survey data indicated that a major change in the topography of the spoil mound had occurred. Profiles across the Center of the mound revealed a flat surface at approximately 19 meters (Figure 1) rather than the previous rounded profile with a minimum depth on the order of 17 meters. This flat upper surface is also readily apparent on the contour chart presented in Figure 2.

These measurements indicate that approximately 2 meters of spoil had been removed from the top of the spoil mound. Some of that material appeared to be present on the northeast corner of the mound, however, the apparent build-up of material in that area could not account for all of the missing spoils. Volume difference calculations were made between the August and November surveys (Figure 3) and a total volume loss of 10,000 m^3 was observed. Some of this loss may be attributed to continued settling and consolidation of the spoil mound, however, the flat topography of the surface and the amount of volume lost suggest that this was not the primary reason for the volume difference. Furthermore, the development of the contour difference chart (Figure 4) showed that slumping of the spoils occured on both the north and south slopes of the pile as there was an increase of material in both areas. Although divers observed the surface of the spoil mound, they were unable to detect any differences in the spoil surface because of the recent disposal of Stamford material.

When the results of this survey were presented, it became obvious that further investigations were necessary to determine

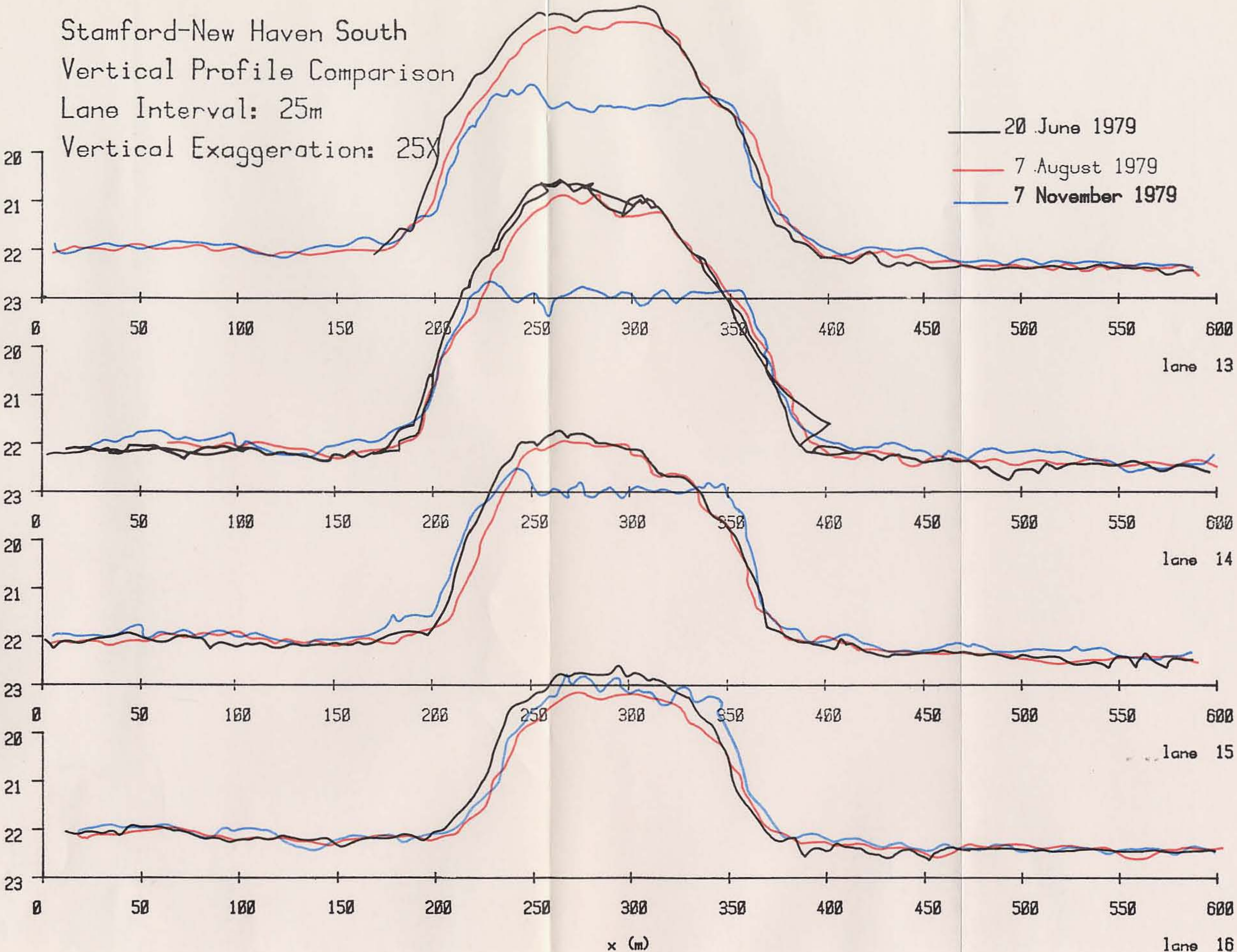
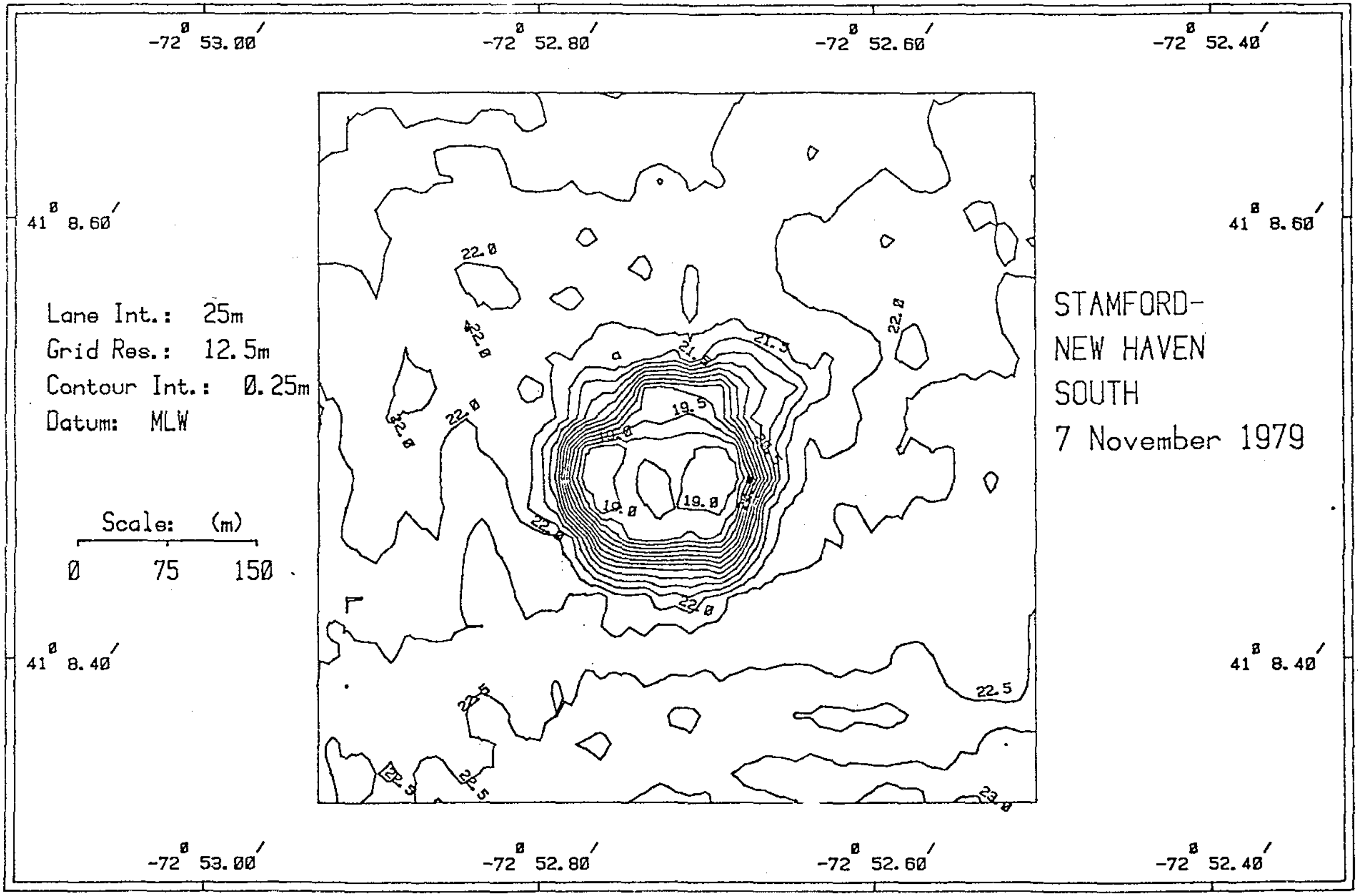

Stamford-New Haven South
Vertical Profile Comparison
Lane Interval: 25m
Vertical Exaggeration: 25X
20 June 1979
7 August 1979
7 November 1979
20
21
22
23
0
50
100
150
200
250
300
350
400
450
500
550
600
lane 13
lane 14
lane 15
lane 16
x (m)

FIGURE 1



-72° 53.00'
-72° 52.80'
-72° 52.60'
-72° 52.40'
41° 8.60'
41° 8.40'
Lane Int.: 25m
Grid Res.: 12.5m
Contour Int.: 0.25m
Datum: MLW
Scale: (m)
0 75 150
STAMFORD-
NEW HAVEN
SOUTH
7 November 1979
22.0
21.5
19.5
19.0
22.5
23.0

FIGURE 2

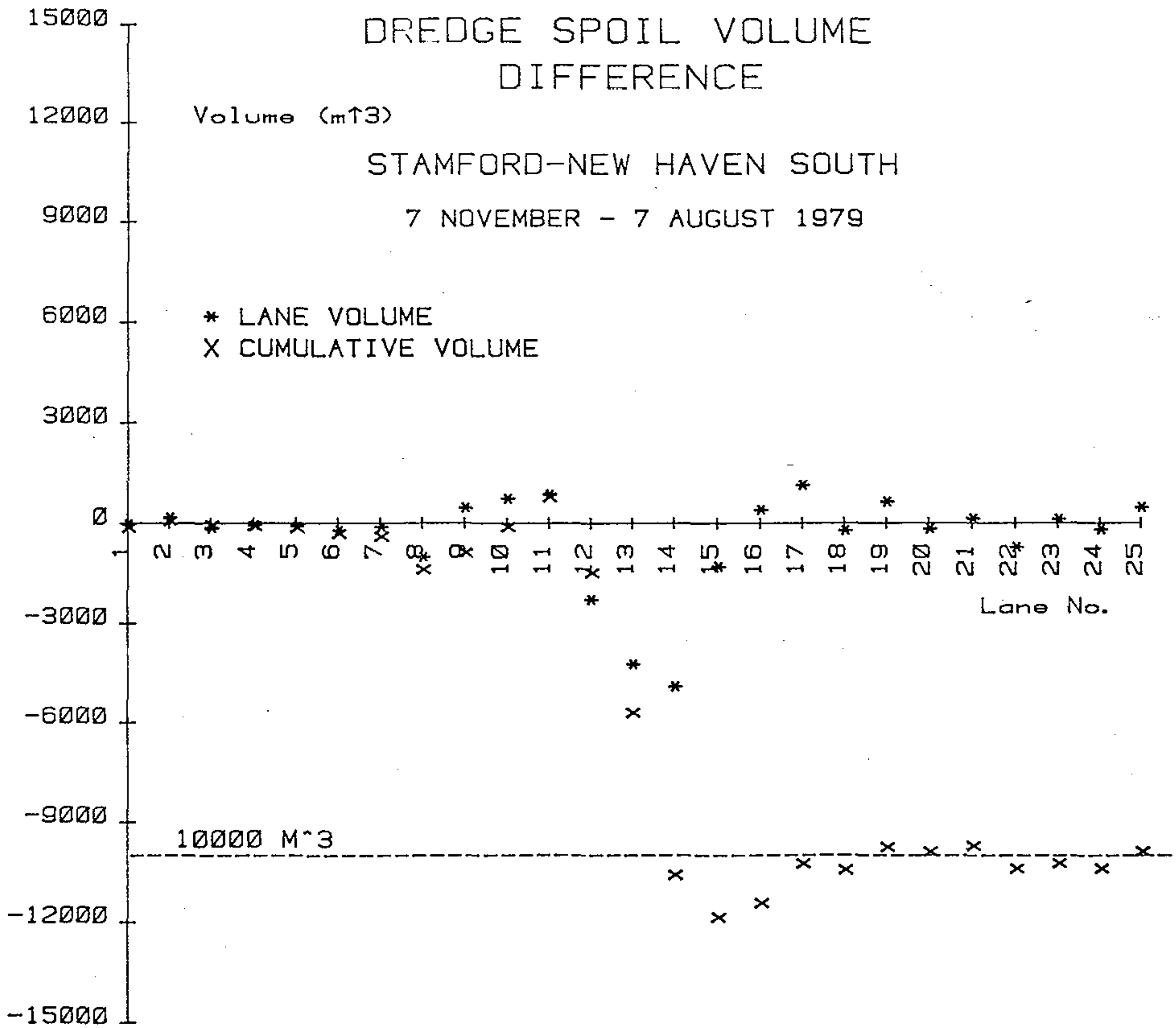

DREDGE SPOIL VOLUME
DIFFERENCE
Volume (m↑3)
STAMFORD-NEW HAVEN SOUTH
7 NOVEMBER - 7 AUGUST 1979
* LANE VOLUME
X CUMULATIVE VOLUME
15000
12000
9000
6000
3000
0
-3000
-6000
-9000
-12000
-15000
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25
Lane No.
10000 M^3

FIGURE 3

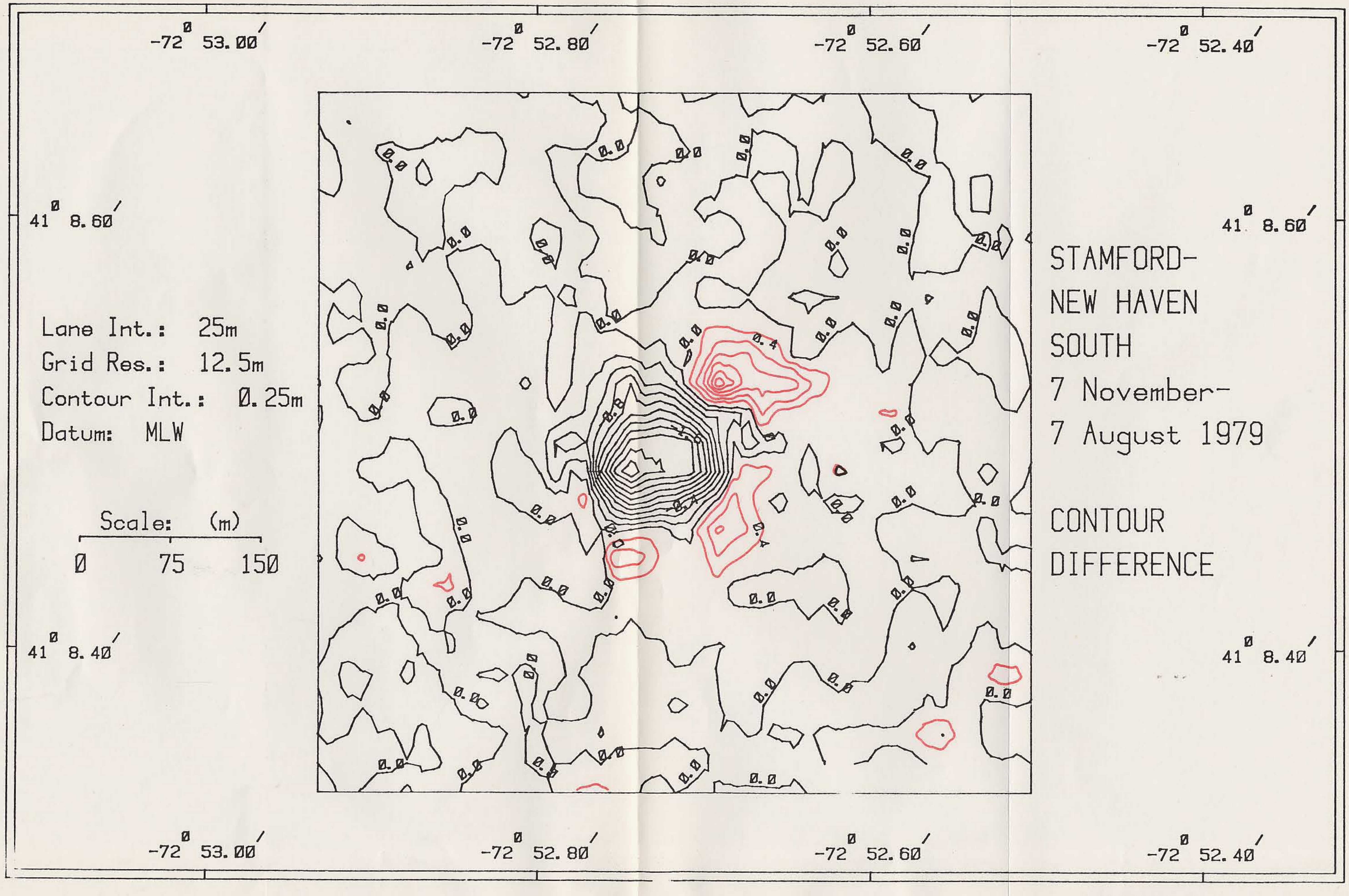

-72° 53.00′
-72° 52.80′
-72° 52.60′
-72° 52.40′
41° 8.60′
41° 8.40′
Lane Int.: 25m
Grid Res.: 12.5m
Contour Int.: 0.25m
Datum: MLW
Scale: (m)
0 75 150
STAMFORD-
NEW HAVEN
SOUTH
7 November-
7 August 1979
CONTOUR
DIFFERENCE

FIGURE 4

the causes of spoil movement and to evaluate conditions at the other sites. The flat topography of the spoil surface at a constant depth suggested that wave action was most likely responsible for the movement of material and the passage of Hurricane David through the area on September 6 provided an energy source to create the wave motion required. Consequently, additional work was authorized to survey the other disposal sites, to sample the surface of the south site and to determine the potential stress exerted on the spoil mounds as a result of the hurricane.

STORM SURVEY RESULTS

The additional survey work was accomplished the following week, during adverse weather conditions. Problems developed in the survey of the north site, when incorrect set-up data were introduced to the computer which caused a shift in the starting point of the survey 88 meters east and 17 meters morth of the original survey grid. Consequently, although a contour chart of the disposal area can be drawn, the profiles can not be accurately reproduced and volume differences cannot be calculated. The survey of the 1974 New Haven disposal mound was conducted without problems, however, further work to inspect the southern site was hampered by rough seas and the presence of an atmospheric inversion layer that degraded navigation precision.

The results of the north site survey are presented in Figures 5 and 6. The contour chart reveals no major changes in the shape of the spoil mound since the August survey and minimum depths on the order of 17 meters are still present. When the profile charts are offset east-west and aligned with the next farther north transect from the August survey the agreement of topographic features is quite consistant and no obvious loss of material is apparent since minimum depths and horizontal extent of the mound are relatively unchanged.

The survey of the disposal mound created by the 1974 dredging of New Haven Harbor was made by establishing a grid with 25 meter lane spacing, arranged to coincide with the 50 meter lane spacing of previous surveys. In this manner detailed comparison of the spoils with previous surveys could be made. The contour

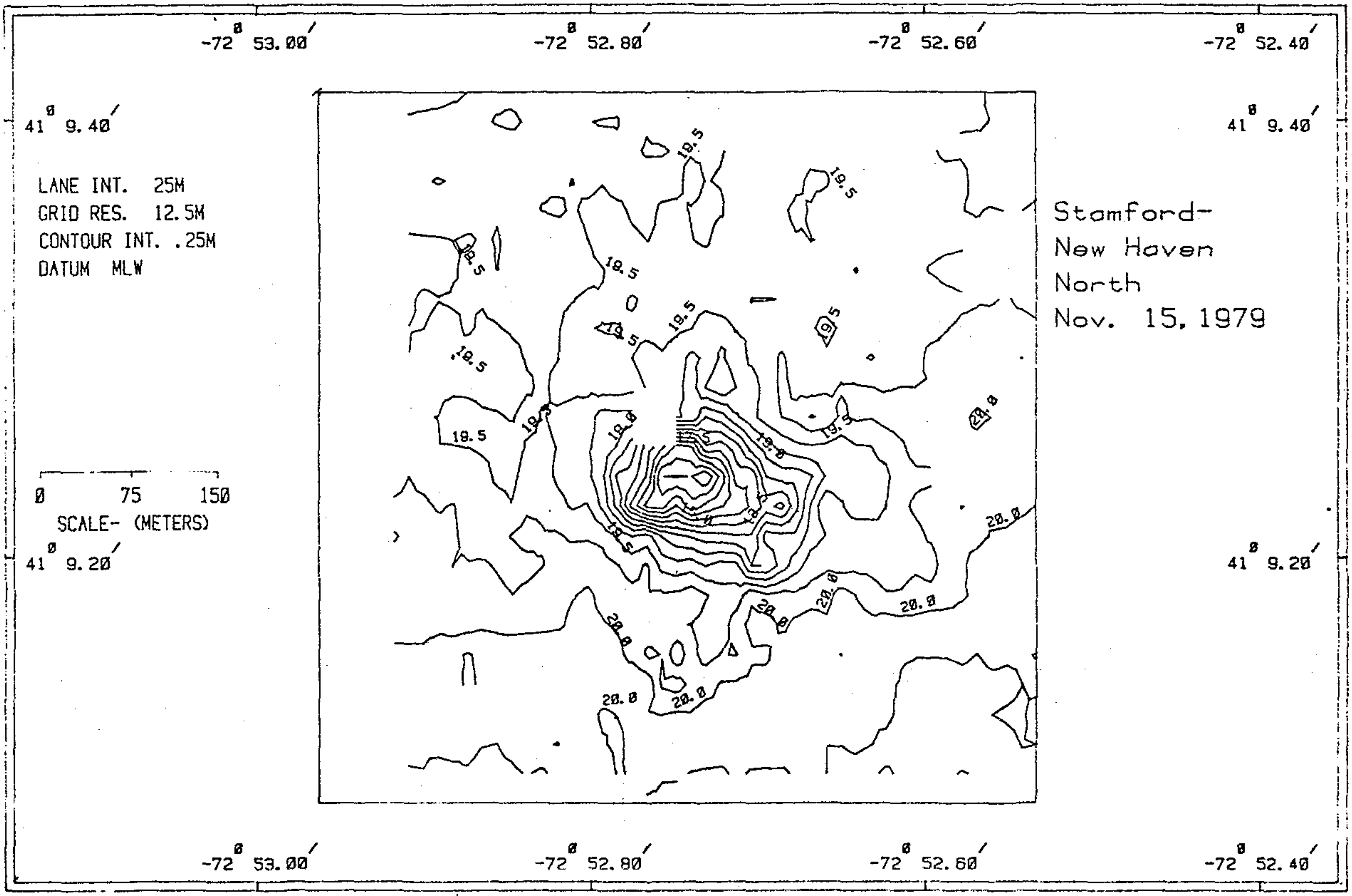

-72° 53.00'
-72° 52.80'
-72° 52.60'
-72° 52.40'
41° 9.40'
41° 9.20'
LANE INT. 25M
GRID RES. 12.5M
CONTOUR INT. .25M
DATUM MLW
0 75 150
SCALE- (METERS)
Stamford-
New Haven
North
Nov. 15, 1979
19.5
20.0

FIGURE 5

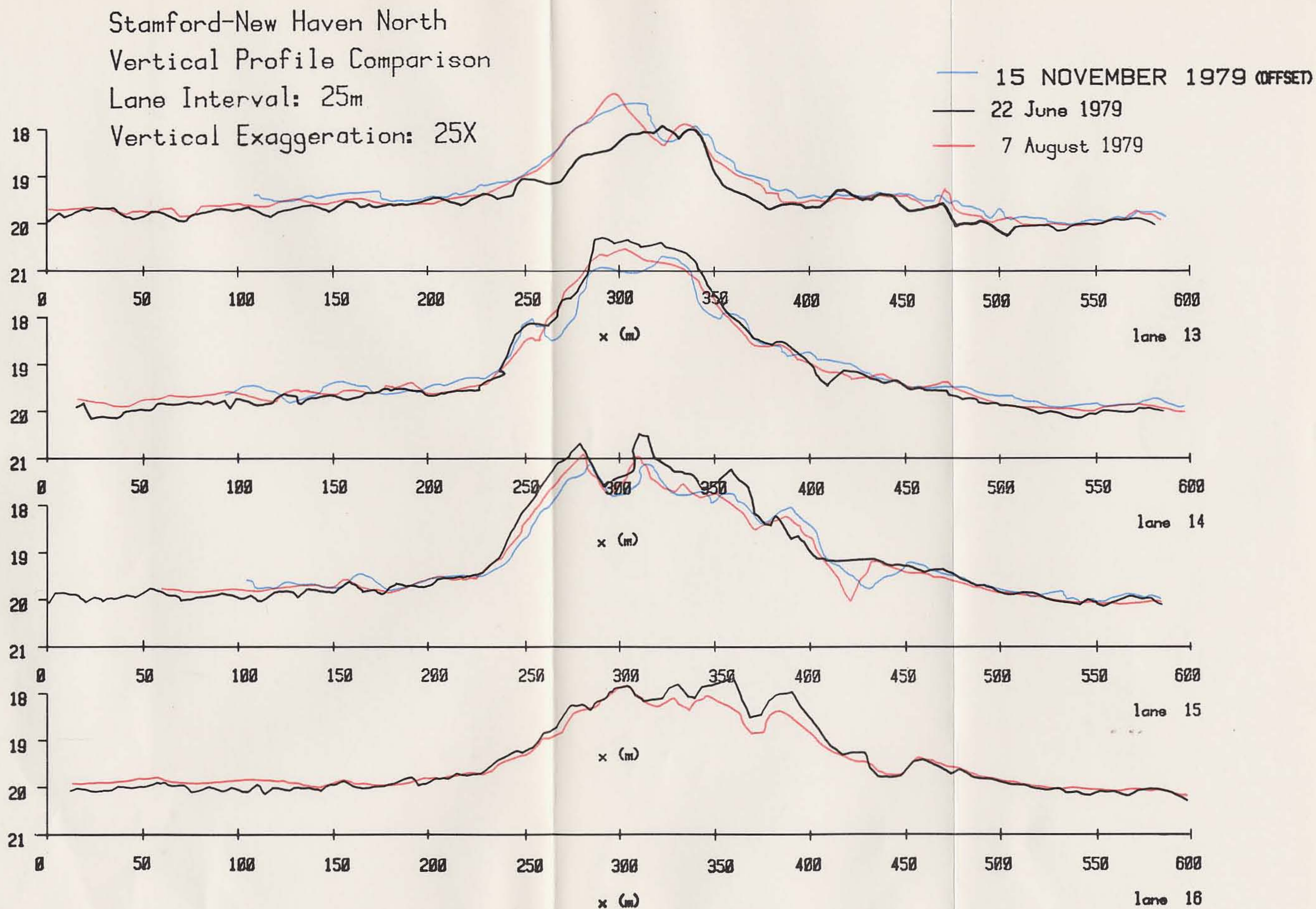

Stamford-New Haven North
Vertical Profile Comparison
Lane Interval: 25m
Vertical Exaggeration: 25X
15 NOVEMBER 1979 (OFFSET)
22 June 1979
7 August 1979
18
19
20
21
0 50 100 150 200 250 300 350 400 450 500 550 600
x (m)
lane 13
lane 14
lane 15
lane 16

FIGURE 6

chart of the resulting survey is presented in Figure 7. The minimum depth of less than 15 meters and the similarity of topographic profiles to those of January, 1979 (Figures 8 a&b) indicate no significant changes have occurred on this mound.

It is important to note that while both the Stamford/New Haven North and the 1974 New Haven deposit have minimum depths that are less than that of the southern site, neither has evidence of significant changes during the period that the southern site was affected. Since these three mounds are all within a mile of each other, on a comparatively flat bottom, it is highly unlikely that one site would experience markedly different environmental stress exerted by currents or wave action than would be expected at the other sites. Therefore, an explanation for the loss of material from the southern mound must account for the lack of movement at shallower depths through differences between the physical and lithological properties of the spoil mounds.

The Stamford/New Haven North and the 1974 New Haven spoil mounds can be distinguished from the southern site since they are both capped with a fine sand material which is probably thicker on the newer spoil mound. This lithology is in sharp contrast to the cohesive silt surface of the southern mound which is characterized by clumps of cohesive clay interspersed within a fine silty matrix.

In addition, the slopes of the sand covered mounds are more gentle than those of the southern site, although all three sites exhibit angles less than 5° and should be within a stable angle of repose for the sediment.

There are three possible means for providing sufficient energy to the spoil mounds to cause movement of spoil material. Such movement could be accomplished either through stress induced by tidal currents, by wave motion or by a combination of the two. Calculations of stress exerted on the sediment by currents are essentially straight forward, those accounting for stress by wave motion more sophisticated and a combination of the two extremely complex.

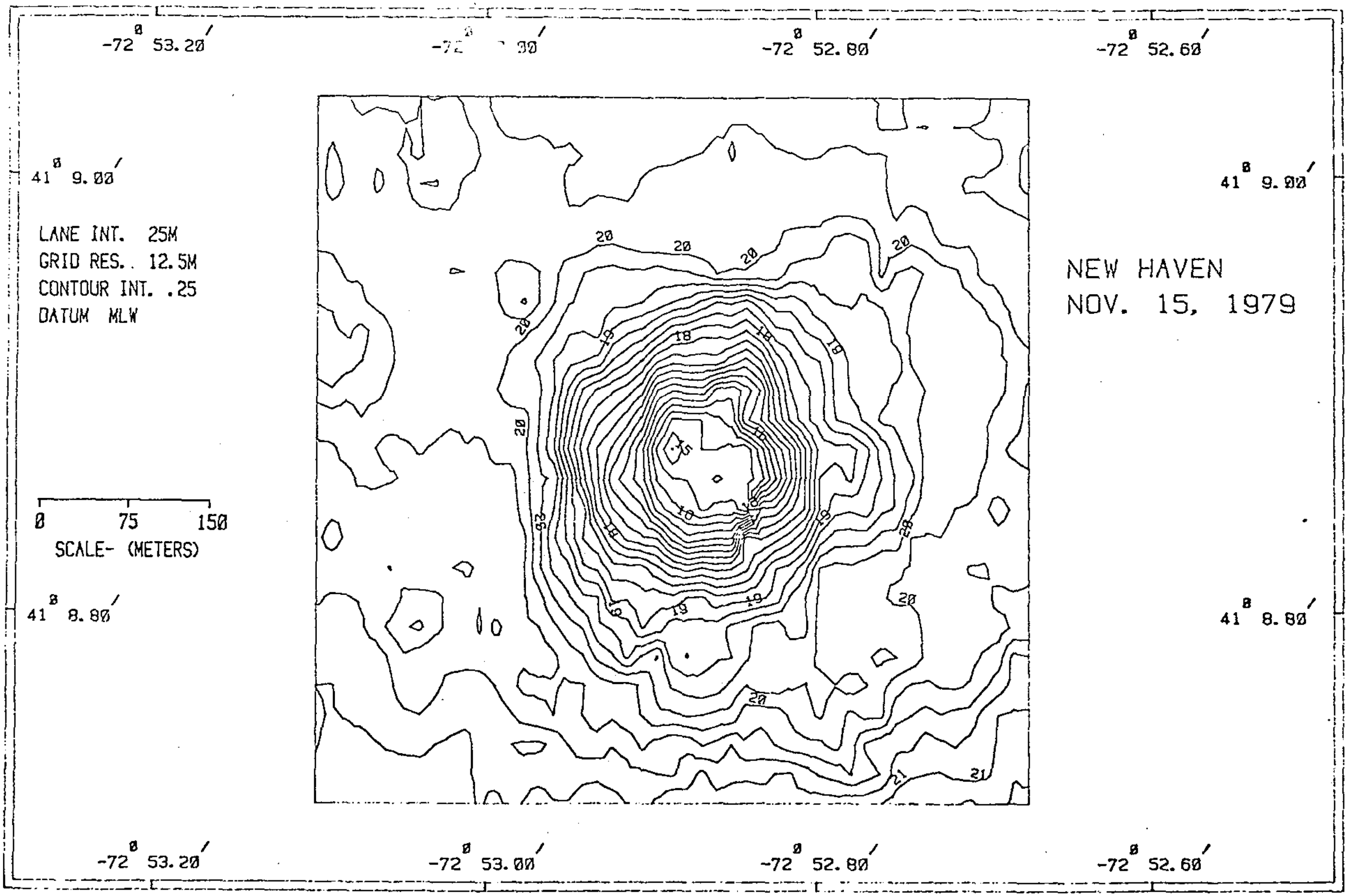

-72° 53.20'
-72° 52.80'
-72° 52.60'
41° 9.00'
41° 8.80'
LANE INT. 25M
GRID RES.. 12.5M
CONTOUR INT. .25
DATUM MLW
0 75 150
SCALE- (METERS)
NEW HAVEN
NOV. 15, 1979
-72° 53.20'
-72° 53.00'
-72° 52.80'
-72° 52.60'

FIGURE 7

VERTICAL PROFILE COMPARISON

NEW HAVEN
JANUARY 17, 1979
LANE INTERVAL: 50M
VERTICAL EXAGGERATION: 25X

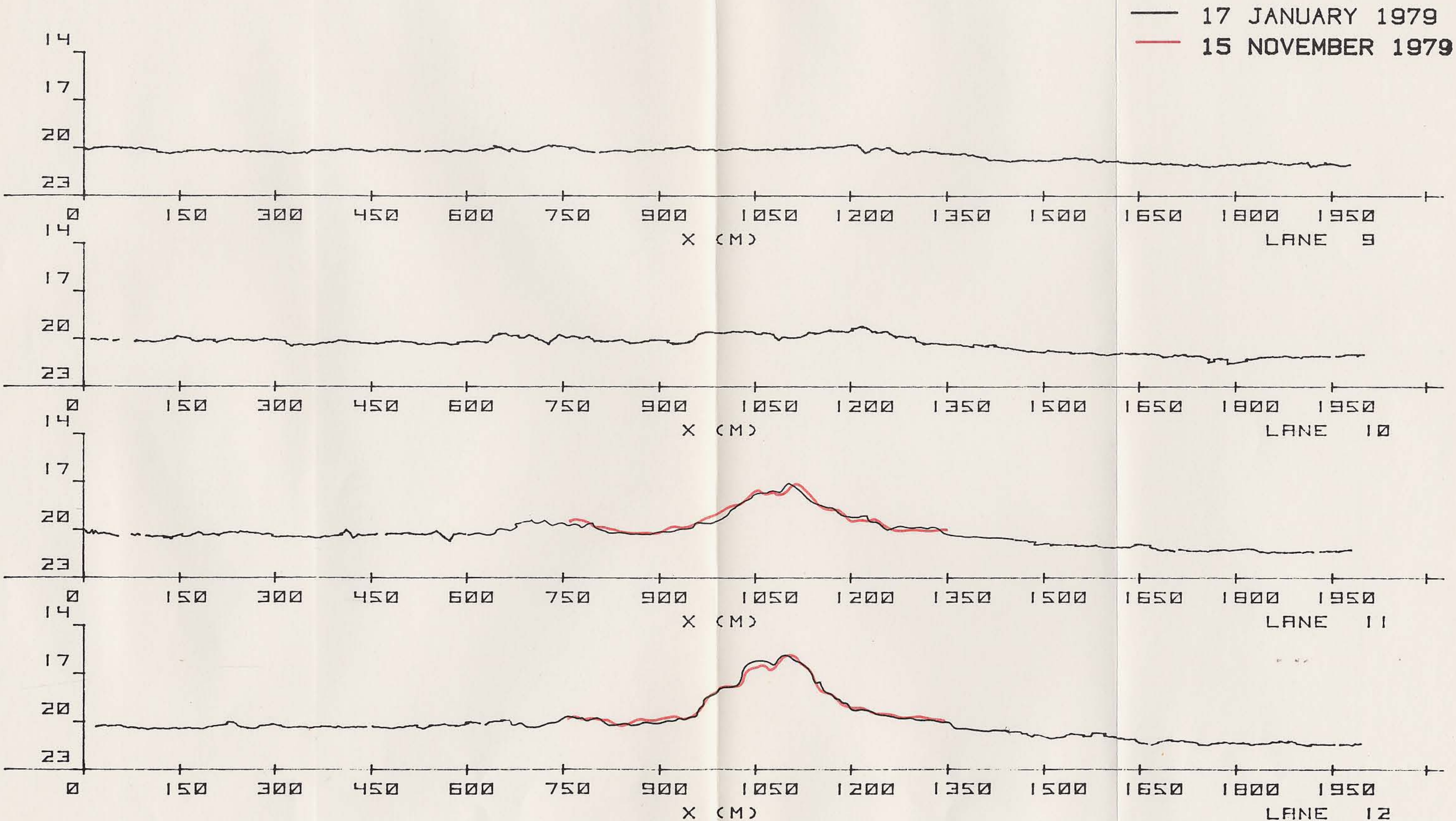


FIGURE 8A

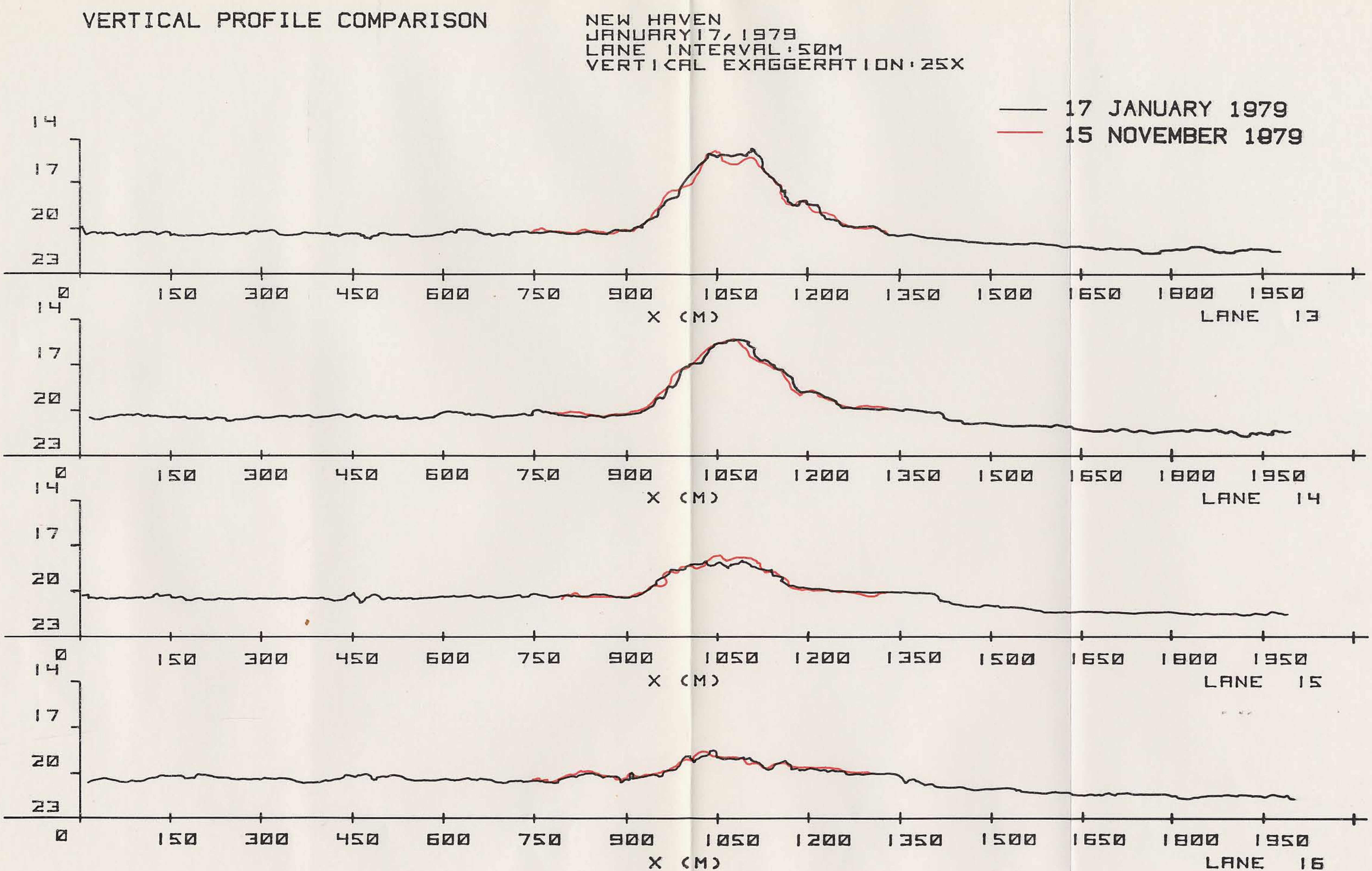

VERTICAL PROFILE COMPARISON
NEW HAVEN
JANUARY 17, 1979
LANE INTERVAL: 50M
VERTICAL EXAGGERATION: 25X
17 JANUARY 1979
15 NOVEMBER 1979
14
17
20
23
0
150
300
450
600
750
900
1050
1200
1350
1500
1650
1800
1950
X (M)
LANE 13
LANE 14
LANE 15
LANE 16

FIGURE 8B

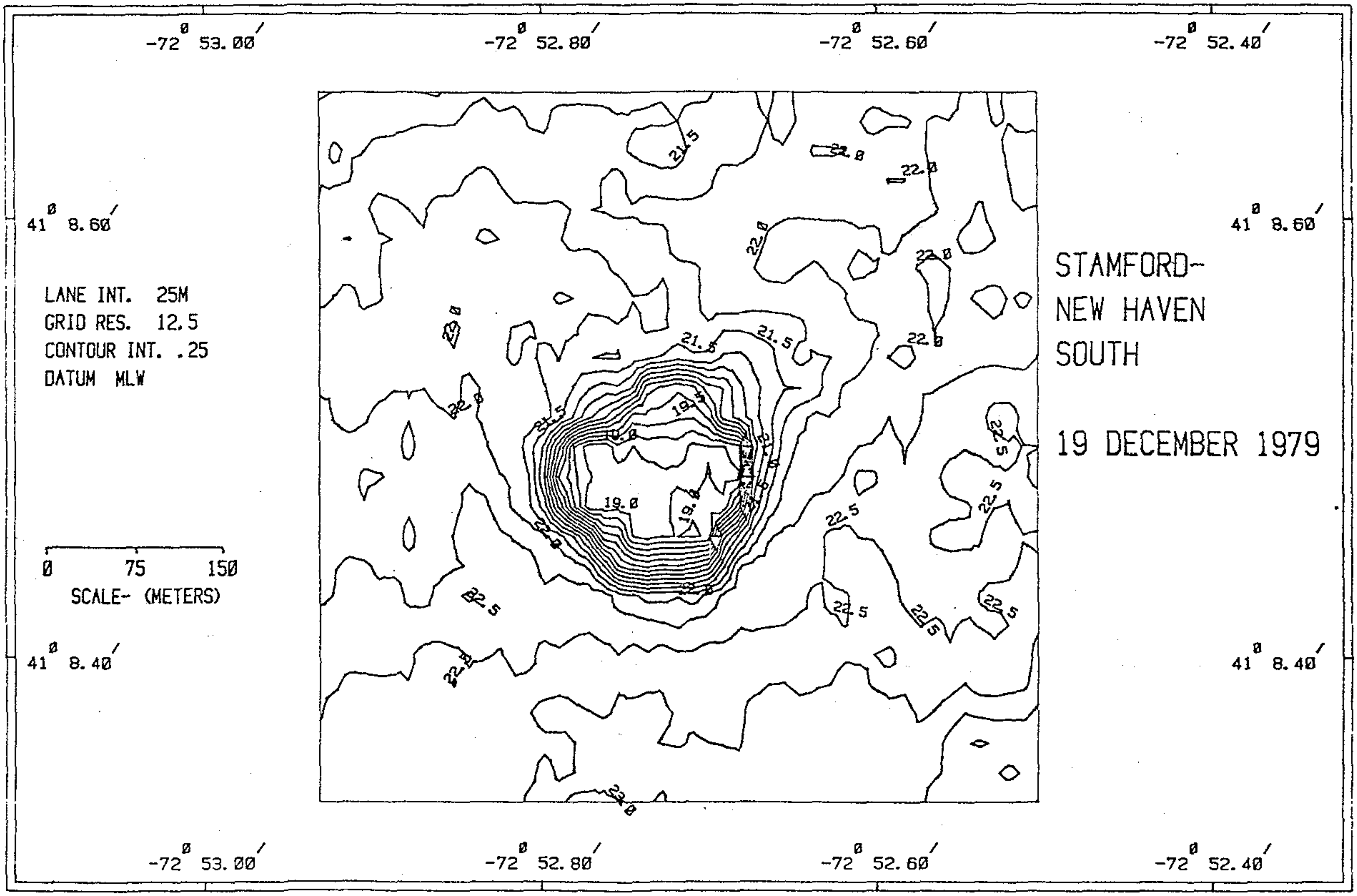
There are several reasons to suggest that normal tidal currents are not responsible for the movement of spoils in this case. First, there has never been any previous indication of significant movement of spoils in this area, either on earlier disposal mounds or during this disposal operation. Second, although the motion of the tidal currents is in an east-west direction at this site, the only observed shift of material is in a north and south direction. Finally, a subsequent survey of the disposal site conducted on December 19, 1979 (Figure 9) indicated that no further changes in the topography had occurred during the month following the original detection of spoil loss. This December survey was also made under such severe weather and atmospheric conditions that direct comparision with other surveys would not be meaningful. Since tidal currents are not likely to initiate sediment motion, the most logical explanation would be the stress exerted on the spoils by wave action or a combination of waves and currents. Because Long Island Sound is a relatively protected area, the generaations of long period waves that are capable of affecting sediment at depths greater than 18 meters must be a rare occurrence. However, the passage of Hurricane David may be just such a situation and may have provided sufficient stress to initiate sediment motion.

Therefore, a comprehensive study to examine the probability that the observed loss of material on the southern site resulted from stress exerted by waves generated during the hurricane was initiated. The results of that study are presented in the following section.

THEORITICAL ANALYSIS OF SEDIMENT MOTION

Determination of Waves

Near bottom oscillatory currents, caused by surface waves, may develop shear stresses capable of causing sediment motion. Evidence of such movement can be seen in the formation of symmetrical bottom bedforms (oscillation ripples) which have been found to depths as great as 200 meters on the Oregon continental shelf (Komar et al. 74). Motions at this depth are reasonable under the very long surface waves (periods of about 20 seconds) developed



-72° 53.00′
-72° 52.80′
-72° 52.60′
-72° 52.40′
41° 8.60′
41° 8.40′
LANE INT. 25M
GRID RES. 12.5
CONTOUR INT. .25
DATUM MLW
0 75 150
SCALE- (METERS)
STAMFORD-
NEW HAVEN
SOUTH
19 DECEMBER 1979
21.5
22.0
22.5
19.0
19.5

FIGURE 9

during Pacific winter storms. In the nearshore zone, where shorter period waves are competent to initiate motion, sediment is made available for transport by steady (wind driven, geostrophic) or slowly varying (tidal) current. These currents would not by themselves be sufficient for entrainment; however, sediment, thus mobilized, is swept away from the region of entrainment to be deposited elsewhere.

The material that was lost from the top of the spoil mound at the New Haven south site may have been mobilized by the waves, or waves combined with currents and transported beyond the border of the disposal site. The purpose of this study is to investigate the reasons why the top of the mound dredge pile was affected during the passage of Hurricane David (September 6, 1979), while adjacent spoil sites covered with fine sand were apparently unaffected. Calculations will be made of the shear stress developed by the waves over the rough surface of the south mound and compared to the shear stress developed over the smooth sand bed. Comparisions will also be made between the developed shear stress and the critical shear stress for initiation of sediment motion under a variety of conditions. Since emperical data on wave and sediment parameters necessary to make the calculation are, for the most part, lacking, it will, be necessary to hindcast waves and to develop the theory based on available evidence of the erosion and the literature of sediment transport in the coastal zone.

The prediction of waves from knowledge of the wind field is presently a very active field of investigation. Various methods have been proposed which provide a significant wave height ($H_{1/3}$) and period ($T_{1/3}$) of deep water waves if the investigator knows the wind speed, duration, and size of the fetch area. These methods have been summarized in Ippen (1966) and in the Shore Protection Manual (SPM, 1973). Other methods which provide a prediction of the spectrum of wave height and period have been developed and tested in the North Sea (Barnett, 1972) , the Great Lakes and in other ocean regions (Liu, 1971). These have been summarized by Seymour (1977) who evaluates the prediction of wave spectra in coastal regions of limited fetch. None of the available methods have been systematically tested on Long Island Sound. Bokuniewicz et al. (1977) used a method of hindcasting developed by Seville, McClendon and Cochran (SCM) which was reported by Linsley and Franzini (1972) as being appropriate for enclosed bodies of water. The system used only fetch and wind velocity to determine wave height and period, while wave length is calculated for the deepwater condition. This technique must provide a less accurate approximation for shallow bodies of water (relative to wave length), since the wave generated may be in the intermediate to shallow water range, hence consideration of the water depth and resulting bottom friction over the fetch area is important.

The method of Seymour (1977) has been used by Knowles (1978) to predict the wave spectrum on Pamlico Sound, North Carolina with relatively good success. Bohlen (1980, pers. comm.) has used Knowles' method to predict the wave spectrum on Long Island Sound and found reasonable agreement. Although further testing is required, this prediction method suggested by Seymour (1977) shows promise for forecasting spectral estimates on the Sound and will be used here.

The method of Seymour (1977) takes as its basis the shallow water wave prediction equations of Bretschneider as reported in the SPM (1973). The coefficients of the equation have been slightly modified based on a published addendum to the manual (Knowles, 1980 pers. comm.). The corrected prediction equation for significant wave height ($H_{1/3}$) is:

$$H_{1/3} = \frac{0.283 \quad U^2}{g} \tanh\left[0.530\left(\frac{gh}{U^2}\right)^{0.75}\right] \tanh\left[\frac{0.0125 \; \frac{gF}{U^2}^{0.42}}{\tanh\left[0.530\left(\frac{gh}{U^2}\right)^{.75}\right]}\right]$$

where U is wind velocity, h is average water depth over the generating area, F is fetch distance and g is the gravitational constant.

Analysis by Knowles (1980, pers. comm.) indicates that significant wave height estimates are difficult and do not correspond to measured values as well as estimates of period.

Significant wave period is also estimated using the Bretschneider equations corrected as before. The significant perid, $T_{1/3}$ is:

$$T_{1/3} = \frac{7.54^{U}}{g} \tanh\left[0.833\left(\frac{gh}{U^2}\right)^{0.375}\right] \tanh\left\{\frac{0.077\left(\frac{gF}{U^2}\right)^{0.25}}{\tanh\left[0.833\left(\frac{gh}{U^2}\right)^{.375}\right]}\right\} \quad (2)$$

where variables are the same as above. Experiments have shown that wave period is quite well predicted by equation (2) (Seymour, 1977).

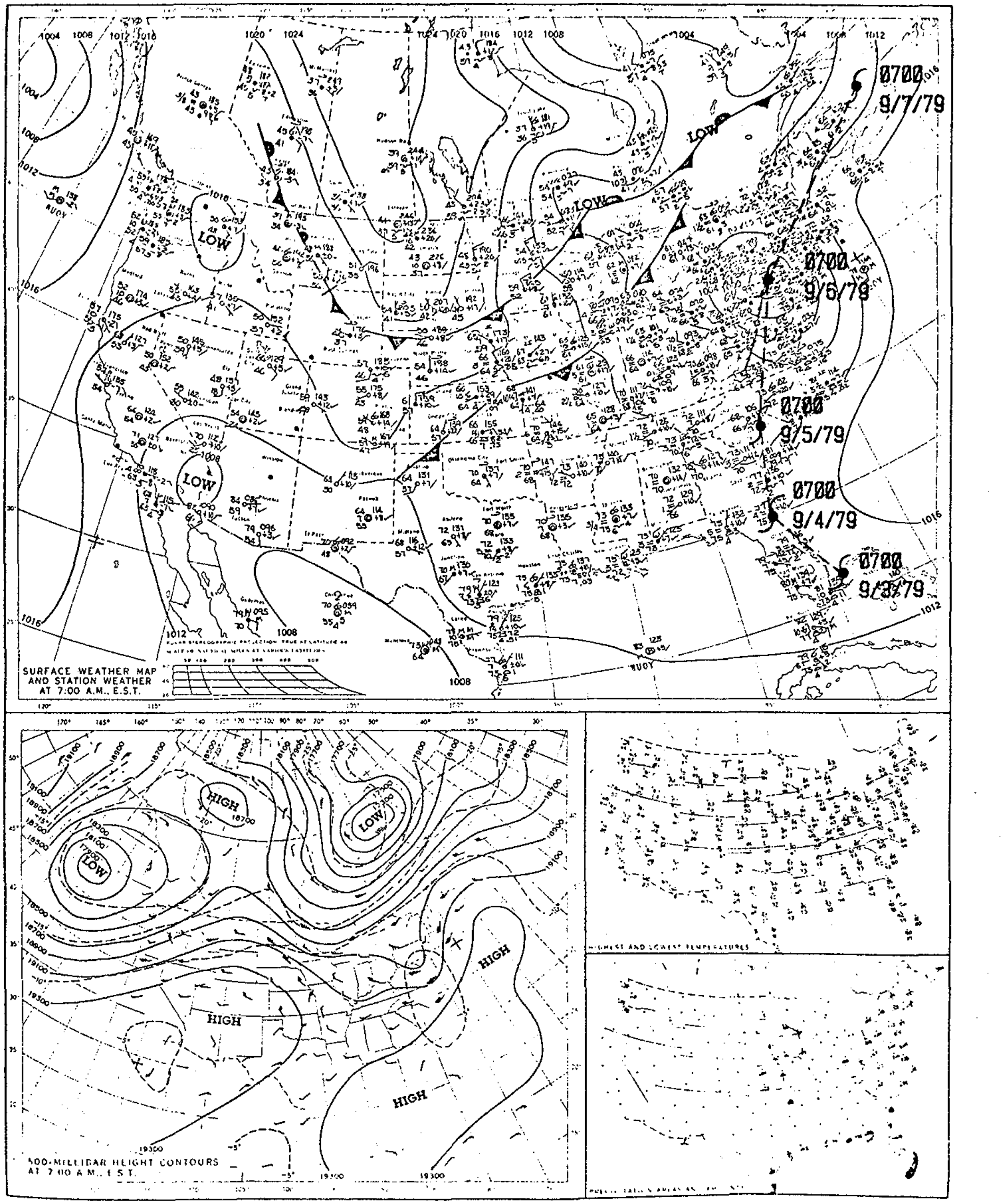
Spectral Estimates

A single wave height and period are not sufficient to characterize the wave field which is composed of waves with a range of wave heights and periods. Estimates of the spectral width and energy have been developed but are untested on the Sound. Though the spectrum of wave energy is certainly important to the development of bottom shear stress and resulting transport, the state-of-the-art has not yet developed to the point that it can be used to obtain these estimates. Calculations of shear stress must be made using single values of wave height and period such as are determined from equations 1 and 2. A reliable prediction method for the wave field on the Sound due to local winds would have great engineering value and should be developd.

Determination of Parameters

Knowledge of the local winds is required since velocity is needed directly for equations 1 and 2, while direction is used to determine the fetch, and duration is used to estimate whether a fully arisen sea has been attained. A record of the loca winds along the Connecticut shore was obtained for the period of passage of Hurricane David on 6 September 1979. The track of the storm from September 3 through 7, 1979 (Figures 10) shows clearly that

THURSDAY, SEPTEMBER 6, 1979



TRACK OF HURRICANE DAVID, SEPT. 3 – 7, 1979

FIGURE 10

the hurricane passed west of the disposal site during the morning of September 6. Data were collected from four coastal stations and are presented as wind vector diagrams in Figure 11. Hourly values were recorded from 0400 - 1800 at the Bridgeport airport with slightly shorter records available from other coastal stations. The winds show coherence in magnitude and direction along the entire shoreline for the period measured indicating that the winds are regional over the Sound rather than local. As the center of the storm passed west of New Haven and continued on a northeasterly course, the winds rotated clockwise from south to west.

Observations from the Bridgeport station were used as the wind input to the wave prediction model. Local winds were averaged over four hour periods from 0600 to 1800 in order to obtain magnitude and direction as shown in Table 1. The four hourly average was selected since it is the approximate time required to develop a fully arisen sea (Bokuniewicz et al. 1977, Figure 4).

Fetch Determination

The fetch is the distance across the water over which the wind blows in order to generate the waves. The simple fetch, F_s is simply the distance from the shoreline to the site measured along the axis of the wind. In regions of restricted fetch, however, the effective fetch, F_e, is a more appropriate measure. This distance takes into account the generation of waves from a 90° segment of arc centered on the wind azimuth. The effective fetch for each average direction was calculated using the procedure in SPM (1973). Limitations caused by fetch width were considered. The determined values of F_e are also shown in Table 1. These are considerably smaller than the value of 41.8 reported by Bokuniewicz et al. (1977, pg.25) which may have been determined using a different method.

Water Depth Determination

Estimates of the mean water depth over the fetch area were obtained from NOS chart number 12354 for the three fetch directions and are also shown in Table 1.

TIME	DIR. ^{o}T	VELOCITY KNOTS	GUST VELOCITY	AVERAGE DIRECTION	AVERAGE VELOCITY m/sec	EFFECTIVE FETCH dm	AVERAGE DEPTH m	CALC. WAVE PER. T,sec	CALC. WAVE HT. H,m
0600	180	21	30						
0700	140	25	36	165	13	20.1	20	3.93	1.11
0800	180	30	50						
0900	160	28	60						
1000	182	32	40						
1100	200	30	42	198	14	20.7	20	4.15	1.22
1200	210	25	35						
1300	200	26	36						
1400	210	26	32						
1500	270	22	30	260	11.5	33.8	10	3.90	1.05
1600	270	25	38						
1700	290	21	28						

Table 1. Local winds recorded at Bridgeport Airport, 6 September 1979.

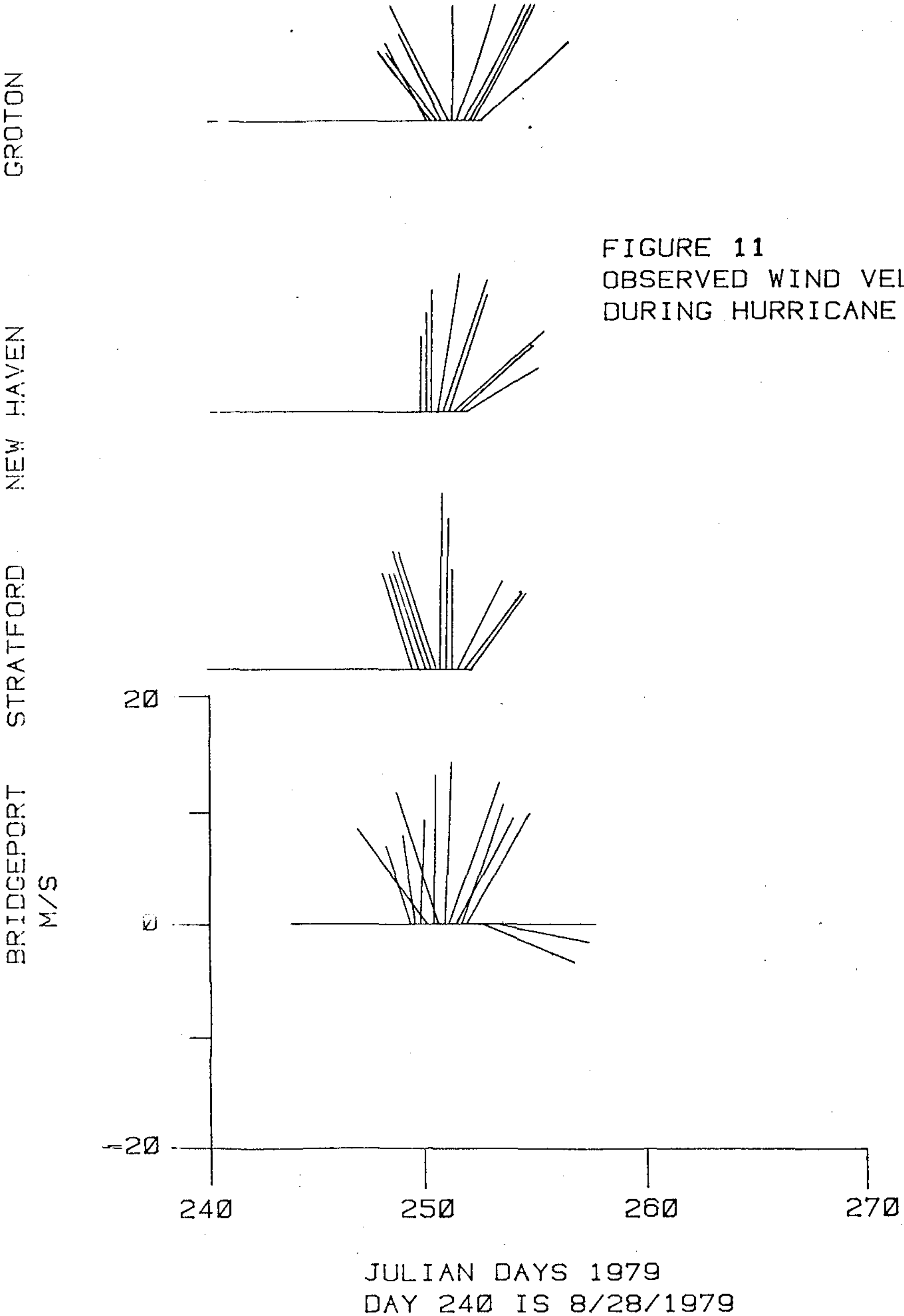


FIGURE 11
OBSERVED WIND VELOCITY
DURING HURRICANE DAVID

Selection of the Design Wave

The parameters developed in the previous sections were used in equations 1 and 2 to calculate values of wave period and height (Table 1). The fetch distance selected by Bokuniewicz et al. (1977) was also used to determine period assuming 20 meter water depth and winds from 198°T, the maximum average winds. The period determined was, T = 4.84. Since this value agrees closely with those of Table 1 , wave periods between 4.0 and 5.0 seconds will be used for estimating the remaining shear stress parameters. Wave heights and periods were not measured during the storm period, however, observations of waves were made at New London, resulting in estimated heights of 2-2 1/2 meters. These heights may be greater than expected at the disposal site since the fetch distance to New London is greater. They do indicate, however, that the predicted waves (Table 1) are probably somewhat small. These data combined with the previous questions concerning the reliability of $H_{1/3}$ calculations has led us to accept the period predictions, but to perform stress calculations for a range of wave heights.

Determination of Sediment Motion

The flow conditions under which sediment particles begin to move have been studied for both unidirection and oscillatory flow conditions for a number of years. The method of determination of critical flow most widely accepted is that of Shields (1936). The Shields' Criterion,

$$\gamma = \frac{\tau_0}{(s-1)\ \rho\ gD} \qquad (3)$$

where τ_0 is bottom shear stress, s is sediment specific gravity, ρ is fluid density, g is the gravitational constant and D is grain diameter, essentially expresses a ratio of entraining to stabilizing forces. When the ratio is exceeded, motion is initiated. The Shields' curve is usually represented as a function of a Reynolds number of the type:

$$R = \frac{U_* D}{\nu}$$

in which $u_* = {\tau_0/\rho}^{\frac{1}{2}}$, the so called friction velocity, and ν is the kinematic viscosity. The traditional Shields diagram is difficult to use because shear stress (τ_0) appears in both the abscissa and ordinate. The solution of critical flow for a given grain size must, therefore, be iterative. Data from a large number of experiments dealing with initiation of motion of unconsolidated sediment under unidirectional flow conditions were recently reevaluated by Miller et al. (1978). They showed only slight changes from Shields (1936) original curve, but also presented other curves of Shields' parameter versus more easily used parameters.

It has been shown (Komar and Miller, 1974) that the Shields Criterion is completely general and adequately (within existing data) predicts initiation of motion under oscillatory as well as unidirectional flows. The developed bottom shear stress in either case is determined by the quadratic stress law:

$$\tau_0 = \tfrac{1}{2}\ \rho f u^2 \tag{5}$$

in which f is a friction factor which depends on bottom roughness. This factor may be evaluated graphically for oscillatory conditions using the curves of Jonsson (1967). The maximum instantaneous stress is obtained when u is the maximum orbital velocity near the bottom as determined from linear wave theory.

Several authors have evaluated the available data on initiation of motion due to waves and have devloped empirical relationships that more or less fit the Shields curve over the experimental range (Komar and Miller, 1975, Dingler, 1979). The relationship of Dingler (1979) was used to evaluate the threshold condition because it is a single formula extending over a range of grain sizes from .018 to .145 cm. The method of Komar and

Miller (1975) predicts very closely as well, but is somewhat more difficult to use for our purposes. Dingler's initiation criterion in nondimensional form is:

$$\frac{\gamma_s T^2}{\rho\ D} = 290 \left(\frac{d_o}{D}\right)^{4/3} \left(\frac{\rho \gamma_s \quad D^3}{\mu^2}\right)^{-1/9} \tag{6}$$

where $\gamma_s = \left(\rho_s - \rho\right) g$ is the dynamic viscosity and d_o is the bottom orbital diameter determined from linear wave theory. Equation (6) was rearranged to give:

$$H = \left[\sinh\left(\frac{2\pi\ h}{L}\right)\right] \left[\frac{2gh}{g\ \tanh\left(\frac{2\pi h}{L}\right)}\right]^{3/4} \left(\frac{C^3}{D}\right)^{-1/2} \tag{7}$$

where sinh and tanh are hyperbolic sine and tangent, respectively, L is wave length determined by iteration from the dispersion relationship for each given period and C is a constant 0.1757 in cgs units.

Equation (7) was used to prepare diagrams of wave height versus period for initiation of motion of given grain sizes in water depths of 14, 16, and 18 meters (Figures 12a-c). Vertical lines drawn at wave periods allow a prediction of the necessary wave height to mobilize unconsolidated sediment of a given grain size. Previous DAMOS studies have shown the representative grain diameter on the north spoil mound and the 1974 New Haven mound to be about 0.025 cm, a size between fine and medium sand. The crossing of this grain size with the selected period indicates the wave height necessary to cause motion. It should be noted that the initiation of motion curves were prepared from experiments in which clean, well sorted, unconsolidated, non-cohesive sediments were measured under laboratory conditions. None of these conditions are likely to prevail in the natural environment, hence, it is expected that the shear stress required to initiate motion in natural sediments is greater than that predicted from laboratory studies (Krone, 1972,

UNCONSOLIDATED SEDIMENT THRESHOLD OF MOTION
AS A FUNCTION OF WAVE PARAMETERS AT 14 M

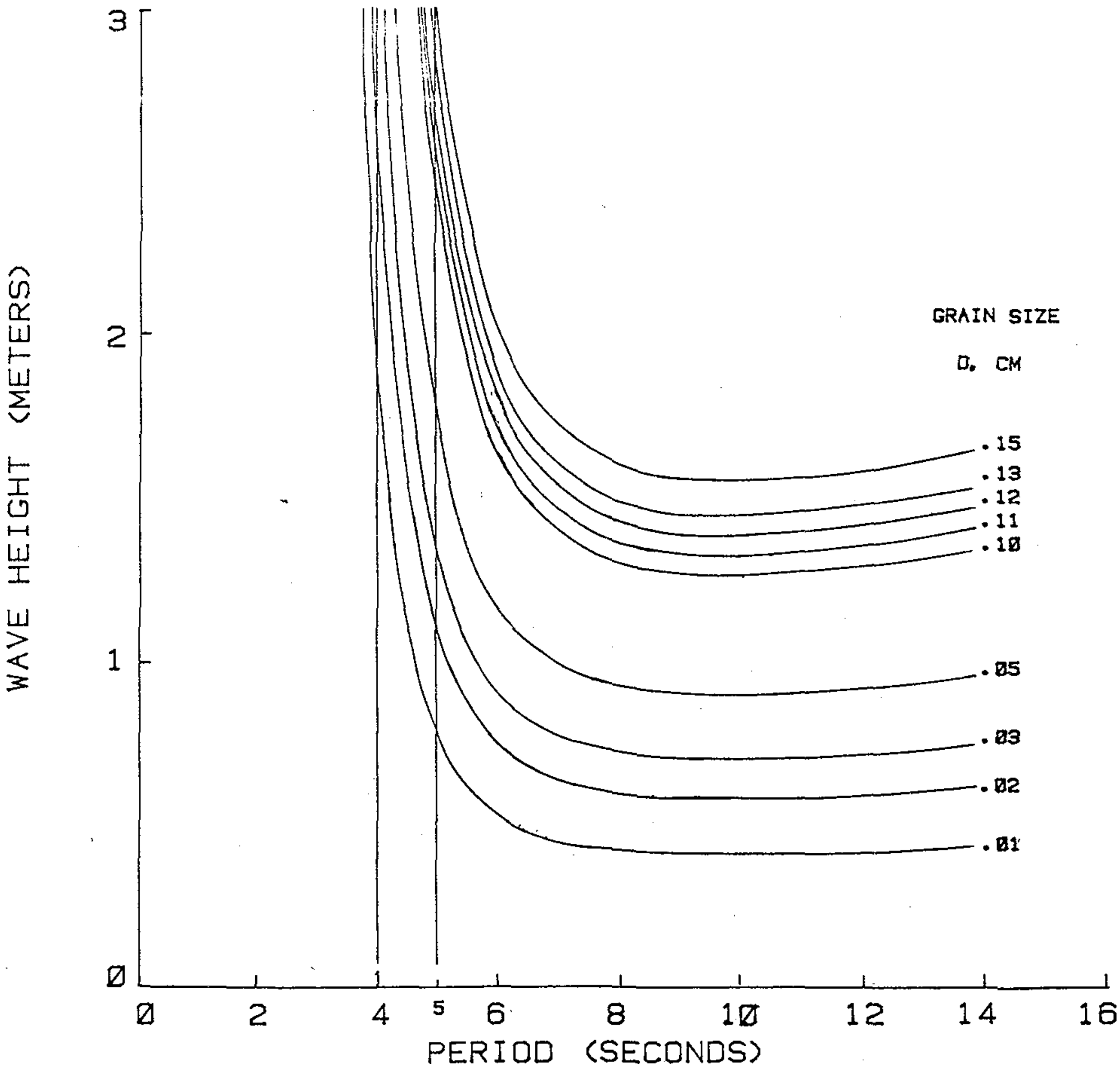
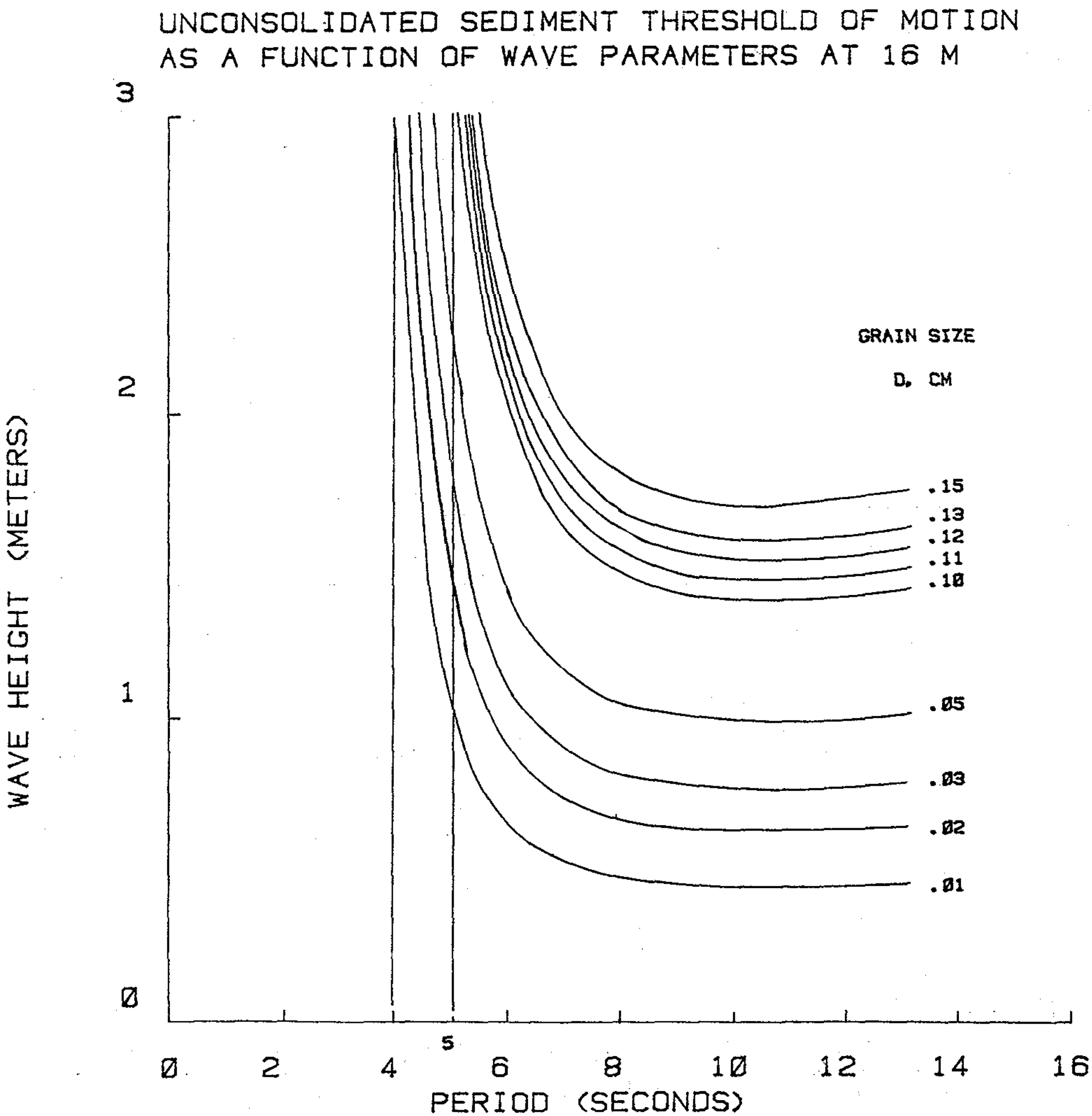


FIGURE 12A



UNCONSOLIDATED SEDIMENT THRESHOLD OF MOTION
AS A FUNCTION OF WAVE PARAMETERS AT 16 M
WAVE HEIGHT (METERS)
3
2
1
Ø
GRAIN SIZE
D, CM
.15
.13
.12
.11
.1Ø
.Ø5
.Ø3
.Ø2
.Ø1
5
Ø
2
4
6
8
1Ø
12
14
16
PERIOD (SECONDS)

UNCONSOLIDATED SEDIMENT THRESHOLD OF MOTION
AS A FUNCTION OF WAVE PARAMETERS AT 18 M

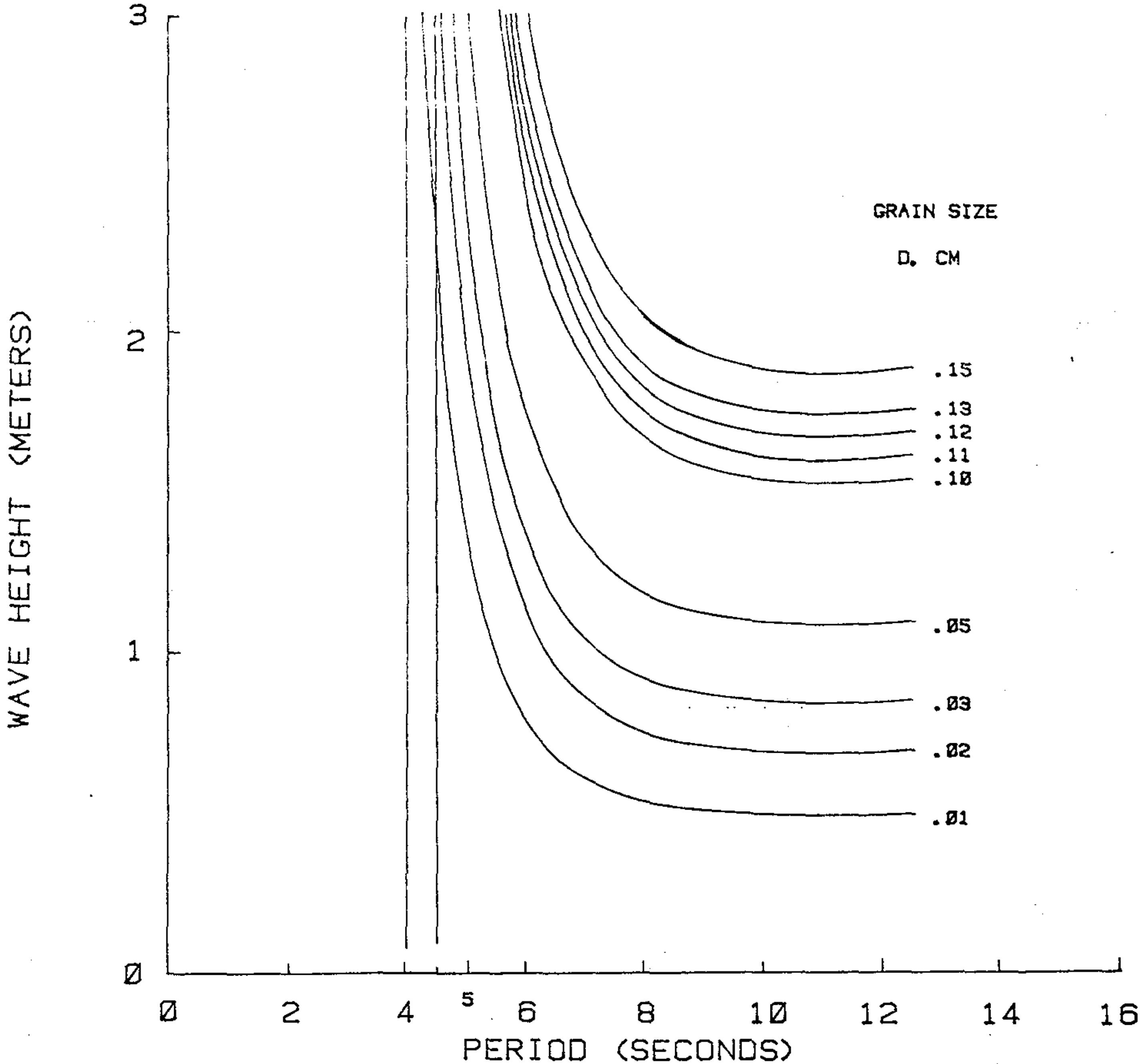


FIGURE 12C

Southard et al, 1971). Figure 12 a, b show that the nominal threshold was exceeded in water depths of 14 and 16 meters for 4.5 and 5.0 second waves of sufficient height, however, the wave height in 18 meters depth must exceed 2 meters with a period of 5 sec to meet the nominal threshold condition. The wave hindcast makes development of such long period waves seem unlikely. However, since failure of the top of the 18 meter south pile was observed, estimates of the developed shear stress were made and compared.

Determination of Shear Stress

The spoil mounds differ in depth of water, composition, shape, and surface roughness. The south pile is composed of dumps of consolidated clay material surrounded by a fine silty clay matrix. These clumps protrude into the near bottom flow and will therefore develop shear stress due to form drag as well as skin friction. The size of these elements, estimated from bottom photographs and relatively undisturbed grab samples, is approximately 20 cm. Side slopes of the southern pile approach 5^{o} which is slightly greater than the low mounds in shallower water depths. The latter mounds have been covered with fine-medium sand and have a roughness estimated from the grain size to be about 0.025 cm.

The developed bottom shear stress (τ_o) for waves of period 4.0 and 4.5 seconds was calculated for a water depth of 18 meters and for wave heights from 1 to 3.5 meters in half meter increments. The friction factor was determined using the method of Jonsson (1967). Results are shown in Table 2 for estimated bottom roughness of 0.025 cm (sand size) and 20 cm (block size). The friction factor, f, depends on the relative bottom roughness, $(d_o/2)/K$ and the flow Reynolds number $((U_m d_o)/2)/\nu$. The maximum value of relative roughness for which data are available is 1.0. Therefore, the ratio of the bottom roughness to Reynolds number was taken as 1.0 when it fell below that value. Since friction factor increases with decreasing relative roughness, at a given Reynolds number, it is expected that this procedure will yield a conservative

h = 1800 cm L$_1$ = 2500 cm sinh kh = 46.14 h/L = .7202 k_1 = .025
T = 4 sec h/L$_o$ 0.72 k_2 = 20 cm

H (m)	d_o (cm)	U_m (cm/sec)	$\frac{U_m\, d_o/2}{\nu}$	$\frac{d_o/2}{k_1}$	f_1	$\frac{d_o/2}{k_2}$	f_2	τ_1	τ_2	Ψ_1	Ψ_2
1	2.17	1.70	$1.84.10^2$	43.3	.1	.05	.60	.29	1.73	.007	.04
1.5	3.25	2.55	$4.15.10^2$	65.0	.09	.08	.53	.30	1.77	.007	.043
2	4.33	3.40	$7.39.10^2$	86.7	.07	.19	.5	.41	2.97	.01	.072
2.5	5.42	4.26	$1.15.10^3$	108.	.059	.13	.5	.55	4.66	.031	.112
3	6.50	5.11	$1.66.10^3$	130.	.048	.16	.5	.64	6.70	.16	.162
3.5	7.59	5.96	$2.26.10^3$	152.	.040	.19	.5	.73	8.31	.018	.200

Stress parameters at 18 meter depths with a wave period of 4 seconds and roughness elements of .025 and 20 cm.

h = 1800 cm L$_o$ = 3158. sinh kh = 18.05 h/L = .5709 k_1 = 0.025 cm
T = 4.5 h/L$_o$= 0.57 k_2 = 20 cm

H (m)	d_o (cm)	U_m (cm/sec)	$\frac{U_m\, d_o/2}{\nu}$	$\frac{d_o/2}{k_1}$	f_1	$\frac{d_o/2}{k_2}$	f_2	τ_1	τ_2	Ψ_1	Ψ
1	5.54	3.87	$1.07.10^3$	111.	.06	.14	.5	.46	3.85	.011	.093
1.5	8.31	5.80	$2.41.10^3$	166	.04	.21	.5	.69	8.64	.017	.208
2	11.08	7.74	$4.29.10^3$	222	.035	.28	.49	1.078	15.07	.026	.363
2.5	13.85	9.67	$6.70.10^3$	277	.024	.35	.49	1.152	23.53	.028	.567
3	16.62	11.60	$9.64.10^3$	332	.020	.42	.49	1.382	33.86	.033	.815
3.5	19.39	13.54	$1.31.10^4$	388	.016	.48	.49	1.506	46.13	.436	1.11

Stress parameters at 18 meter depths with a wave period of 4.5 seconds and roughness elements of .025 and 20 cm

Table 2

estimate of shear stress.

The Shields Criterion, Ψ, was also determined and plotted on a modified Shields diagram extracted from Madsen and Grant (1976). Figures 13 and 14 show the Shields Criterion for rough and smooth conditions plotted for grain diameter D = 0.025 cm ($S_* = 23.97$) as a function of wave period. This is as general as the traditional Shields diagram, but is more easily used since the shear stress has been eliminated from the abscissa (x axis).

The calculated shear stress values for large roughness height are near or exceed the critical value for all tested wave heights at both periods. In contrast, the shear stress developed over the surface of smaller roughness never exceed the critical value. Consequently we can conclude that the high roughness factor resulting from the clumps of cohesive sediment on the south site create a greater stress and cause sediment motion under storm wave conditions, while the smoother surfaces of the other spoil mounds produce significantly smaller stress values thus insuring the stability of the spoils even at shallower depths. This conclusion is supported, of course, by Figure 12c.

Shear Stress Under Combined Wave and Currents

Methods for determination of shear stress under combined waves and currents have recently been developed by Grant and Madsen (1979) and Smith (1977). The former assumes the interaction of waves and currents in a wave dominated environment whereas the latter assumes a current dominated environment. Measurements of tidal currents in the vicinity of the disposal site indicate that maximum orbital velocities and currents are of equal magnitude at least during part of the tidal cycle. Wave and currents combine over rough bottoms to generate shear stress that is a non-linear summation of the individual components. Though the shear stress under these conditions has not been calculated, it would be greater than that due to waves alone. Further work is needed to evaluate the wave current interactions with spoil material in Long Island Sound.

MODIFIED SHIELDS DIAGRAM (MADSEN & GRANT, 1970)
WAVE PERIOD T = 4.0 SEC.

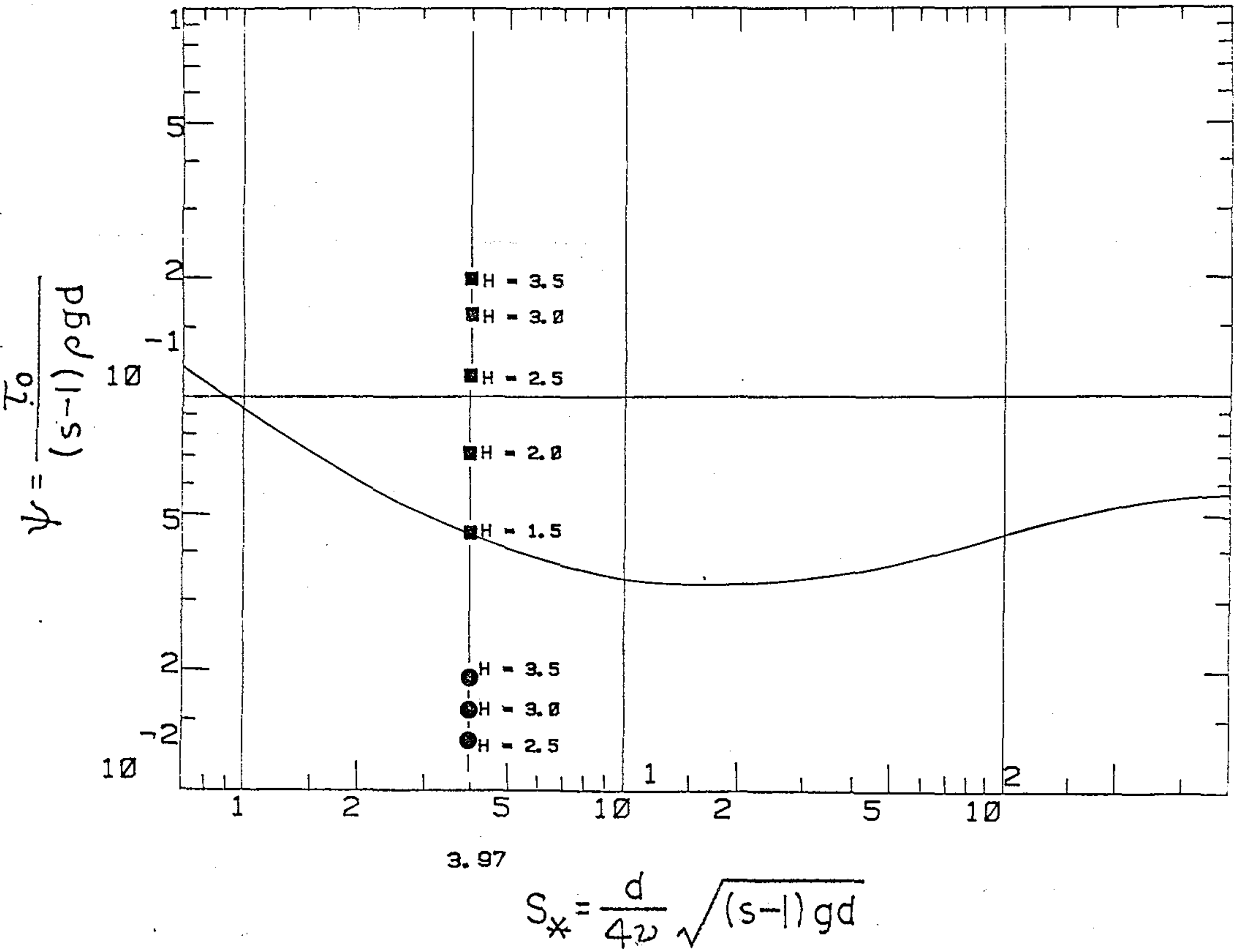
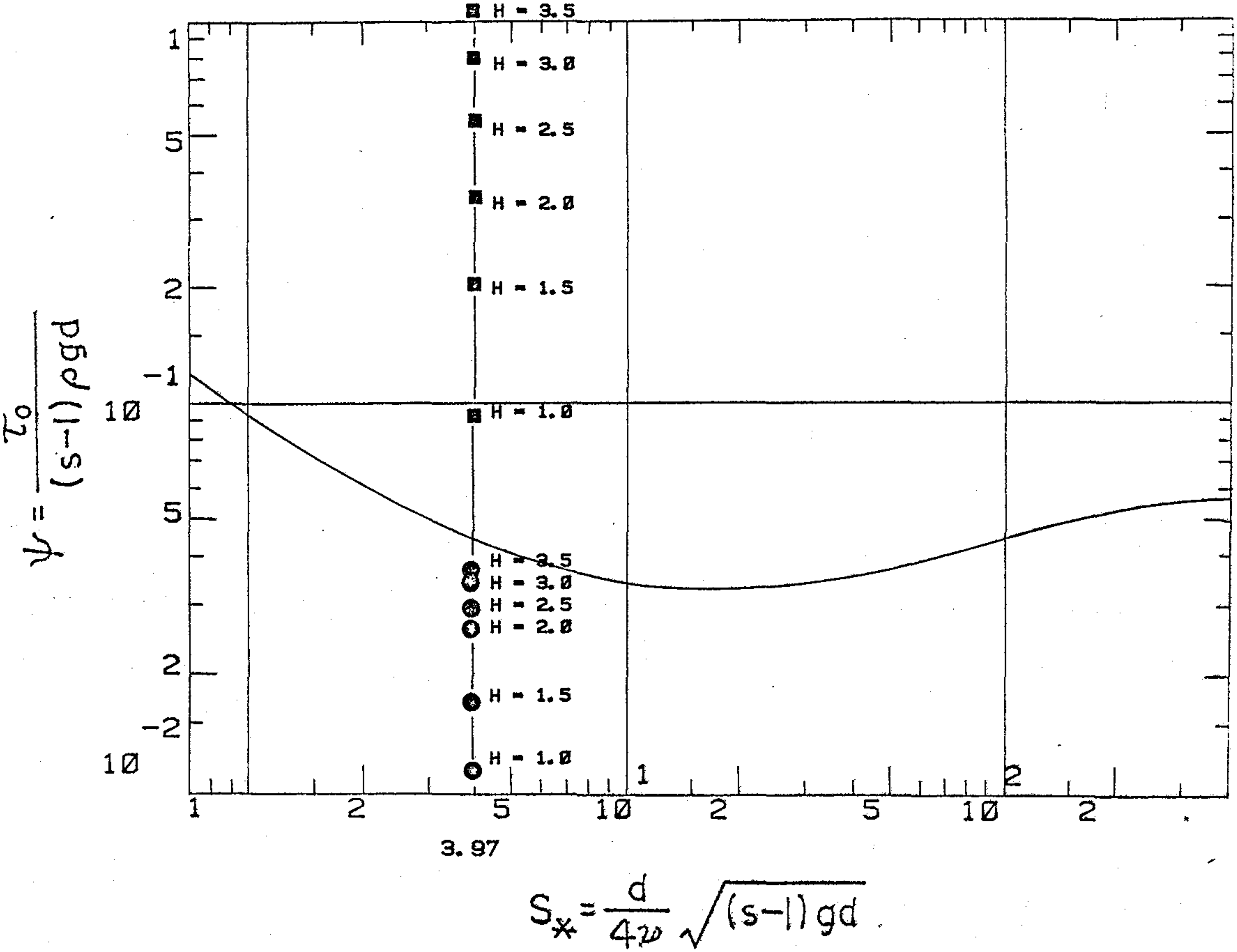


FIGURE 13

MODIFIED SHIELDS DIAGRAM (MADSEN & GRANT, 1970)
WAVE PERIOD T = 4.5 SEC.



Discussion

The preceeding analysis has shown that for waves with the periods and heights likely to have been generated by the passage of Hurricane David there are considerable differences between the shear stresses generated on the spoil mounds which are due to the surface roughness. Though the calculations show that this difference could have been the cause of the preferential erosion of the south pile, some factors affecting the accuracy of the results must also be considered.

The calculated shear stress due to waves over the relatively smooth surfaces may be done with some confidence since the relative roughness values are within the range of experimental observation. However, the determination of the stress over a surface of very great relative roughness must be considered more of an estimate. Without field observations under these conditions, we do not know how the stress is partitioned between skin friction, which may cause erosive failure of the block, and form drag, which may physically move the block or cause eddies which entrain interstitial material. Furthermore, actual Shields criteria for consolidated sediments are essentially unknown and can only be estimated as substantially greater than unconsolidated sediments.

Further investigation should be done in order to determine:

- The mode of failure of the blocky material under conditions of high shear stress.
- The degree of consolidation and cohesion of the bottom sediments (dredge pile, sand cover, block). and the effect of these parameters on erodability of spoils.
- The partitioning of shear stress over beds of large roughness under waves and currents.

This investigation has concentrated on the influence of storm generated waves on spoil stability and has provided an explanation for the selective erosion of spoils from the Stamford-New Haven south site. The impacts of the results of this study will be discussed in the final section.

SUMMARY

The field surveys conducted under the DAMOS program to monitor the Stamford New Haven disposal operation have been successful in identifying changes in the spoil mounds indicative of spreading of material from the disposal site. The loss of material from the New Haven south site amounted to 10,000 m^3 or approximately 12% of the total capping material. However, since all of this material was lost from the upper surface of the mound, no exposure of Stamford material has occurred. This was confirmed by sediment sampling on December 18, 1979., when the only Stamford material observed was located at the site of recent disposal and at the previously reported errant barge dump 200-250 meters west of the mound.

Observations of the other spoil mounds in the Central Long Island Sound Site have shown no measureable changes in spoil volume or distribution, even though these deposits have more shallow minimum depths than the southern site.

An explanation for the selective movement of spoils on the south site has been proposed based on the interaction of storm waves resulting from Hurricane David and the roughness parameters of the cohesive New Haven spoils. This explanation has been substantiated through theoretical calculations and discussions with scientists working on the study of sediment motion. (F. Bohlen, W. Grant, J. Ianiello and C. Knowles).

The implications of these conclusions are important to future disposal and/or capping operations. Consolidated, cohesive spoils are common in the New England area, and clamshell dredges which preserve the cohesive nature of the spoils must be used to reduce suspended load and spreading of spoils at both the dredging and disposal sites. Consequently, most spoil mounds will have surface roughness comparable to the New Haven south site for a period of time after disposal. These features have been observed at the New London site, but the cohesive clumps have broken down over a period of time premarily due to biological activity, but also as a result of fracturing and erosion (Stewart, 1978).

From the results of this study, it is apparent that the stress created by the roughness factor associated with these clumps under storm wave conditions is more important than the depth of the spoil surface, the strength of currents or the cohesive nature of the sediment in determining the stability of spoils. The occurrence of a major storm such as Hurricane David, before the surface of the spoil mound has been smoothed by natural forces thus creates a potential for large scale erosion and transport of material.

Future disposal operations might, therefore, consider methods to produce a smooth spoil surface at the conclusion of the dumping procedure. Such methods could include:

- capping with sand material, as was done at the other New Haven sites
- dredging and disposal of less cohesive sediments from the mouth of the estuary near the end of the operation
- disposal of cleaner material from the mouth of the estuary after artificially increasing the water content of these spoils to break down cohesion.
- artificially smoothing the surface through dragging.

Obviously more work needs to be done to determine if this is in fact necessary and to evaluate the potential for recurrence of the effects observed at the New Haven site. This problem is being addressed to some extent under the DAMOS program through a combination of bottom turbulence and spoil erosion studies, however, the phenomena observed at the Central Long Island Sound Site emphasize the importance of understanding the interaction of the energy regime with spoil material.

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MEMO

FROM: R. Morton

TO: R. Semonian

DATE: 6 February 1980

RE: Frequency of occurence of storms with intensity levels of Hurricane David.

1) The most recent Stamford/New Haven Disposal Operation, Monitoring Survey Report (#7) discusses the impact of Hurricane "David" on the spoil mounds in the Central Long Island Sound Disposal Site. Since this storm did cause significant changes in the southern spoil mound, it is important to determine the frequency of such tropical depressions to develop a management approach to minimize future impacts and to assess the potential for reoccurence of such an event.

2) Data on the frequency of storms and tropical depressions were obtained from the National Climatic Center in Asheville, North Carolina. For western and eastern Long Island Sound the percentage risk of a tropical storm occurring in any given year has been determined for three levels of intensity:

• Tropical Storms	Winds $<$ 72 MPH	11% or 1 every 9 years
• Hurricanes	Winds $>$ 72 MPH	6% or 1 every 17 years
• Great Hurricanes	Winds $>$ 72 MPH with extensive damage	2% or 1 every 50 years or greater

The intensity of Hurricane David on Long Island Sound was at the lower end of the Hurricane Scale, and therefore, a conservative estimate of recurrence of such storms would be on the order of once every 12-13 years.

3) Further data are available on all storms, throughout the year, that are not necessarily tropical depressions, but are classified according to damage. During the period from 1901 - 1955, there were fourteen storms that caused significant damage in the Long Island Sound Area; an average of one major storm

every four years. It is doubtful that all of these storms would have been as intense as Hurricane David, therefore, a conservative estimate of the frequency of occurence of such a storm either resulting from a tropical depression or from seasonal weather patterns would be on the order of once every eight to ten years.

R W Morton

cc: Carl Hard