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CEERD-HZA 19 November 2015

MEMORANDUM FOR Commander, U.S. Army Engineer District, New England District, CENAE-PP-C Mr. Edward O'Donnell, 696 Virginia Road, Concord, MA 01742.

SUBJECT: Evaluating Sediment Mobility for Milford, CT Nearshore Placement Letter Report, ERDC/CHL LR-15-7

1. The U.S. Army Engineer District, New England, requested that the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL) investigate the sediment mobility at two nearshore placement sites near Milford, CT. The two proposed nearshore placement sites have been evaluated to estimate the potential sediment mobility frequency and sediment migration direction for each location under typical and storm wave conditions. The attached letter report, ERDC/CHL LR-15-7, describes the findings of the study.

2. If you have any questions, please contact Dr. Brian McFall at (601) 634-2056, or Dr. Katherine Brutsché at (601) 634-4174.

Encl

FOR

JOSÉ E. SÁNCHEZ, PE, SES
Director

ERDC/CHL LR 15-7
November 2015

Evaluating Sediment Mobility for Milford, CT Nearshore Placement

by Brian C. McFall, Cheryl E. Pollock, Katherine E. Brutsché

INTRODUCTION. This letter report consists of a preliminary engineering study to approximate the sediment mobility and direction of two potential nearshore placement sites in Milford, Connecticut. Strategic placement of dredged material in the nearshore is a technique commonly used for material that may contain more fine material than is allowed for placement directly on the beach. Transportation costs can be reduced by placing material in the nearshore region close to the dredging location rather than an offshore disposal site. Placing the dredged material in the nearshore region can also keep the material in the littoral system, nourishing the beach and protecting the beach from large, erosive storm waves.

Milford Harbor is located approximately 13 km west of New Haven Harbor at the mouth of the Wepawaug River on the north shore of Long Island Sound. The New England District (NAE) of the U.S. Army Corps of Engineers (USACE) is currently proposing to hydraulically dredge approximately 20,000 cubic yards (cy) [approximately 15,000 cubic meters] of sandy material from shoaled areas within the entrance channel of the Milford Harbor Federal Navigation Project (FNP). There are currently two proposed placement sites for the material. The first option is placement in the nearshore region near Bayview and Pond Point Beaches, shown in the red rectangle in Figure 1 with an average water depth of 4 m. The second option is placement along Gulf Beach, shown in the blue cross-hatched rectangle with an average depth of 3 m.

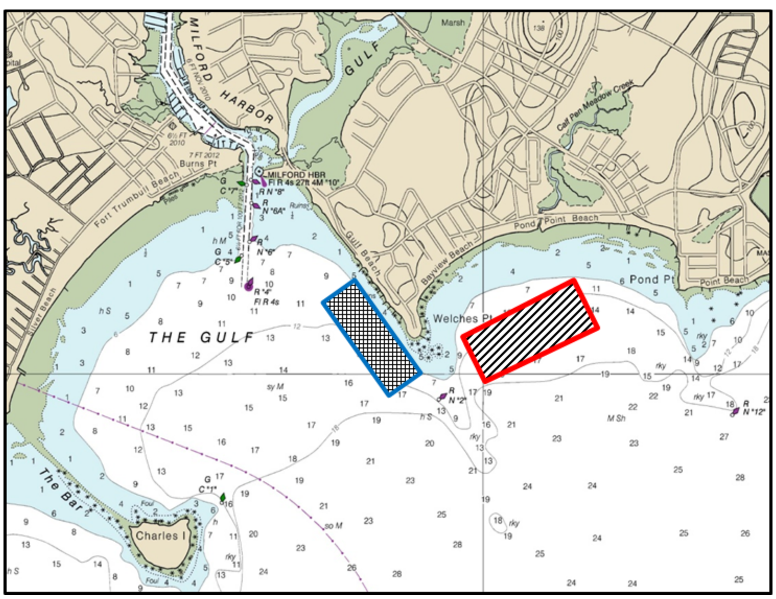

Figure 1. Proposed placement sites near Milford, Connecticut. Site 1 is delineated by the red, striped rectangle. Site 2 is shown by the blue, cross-hatched rectangle (Figure modified from maps provided by NAE).

The 2014 sediment sampling and testing report by the USACE for the dredging project took four samples in the proposed dredging site and found the material to consist of 81 to 95% fine sand with an average median grain size (d_{50}) of 0.21 mm for the four samples (USACE, 2014). The median grain size diameter (d_{50}) is defined as the diameter which 50% of the grains by mass are finer. Similarly, 10% of the grain size diameters are finer than d_{10}, and 90% are finer than d_{90}. The d_{10} to d_{90} for the samples ranged from 0.1 to 0.5 mm, therefore the critical thresholds of sediment mobility are calculated for this range.

Wave and current information for this study was obtained from two sources. Typical wave conditions were estimated using wave characteristics measured by the University of Connecticut Department of Marine Sciences MYSound wave buoy located approximately 22 km southeast of New Haven Harbor in a water depth of 27 m. These wave conditions were transformed into the project regions using the conservation of energy flux and Snell's Law. The hourly significant wave height, dominant wave period, and average wave period are available from 2002 to 2003 and are representative of typical wave conditions.

The Coastal Hazards System (CHS) (USACE, 2015a) contains storm wave and current conditions from the North Atlantic Coast Comprehensive Study (NACCS) (USACE, 2015b). CHS is a coastal storm hazard data storage and mining system that stores NACCS's comprehensive, high-fidelity storm response computer modeling results from 1,050 synthetic tropical storms and 100 extratropical storms (nor'easters) including climatology, surge, total water levels, waves, and currents. The ADCIRC circulation model was coupled with the STWAVE wave model to output the necessary storm wave and current information for this analysis. The storm wave and current data in the placement site from the high-fidelity models is representative of storm wave conditions.

EVALUATION OF SITE 1.

This site is located offshore of Bayview Beach in approximately 4 m water depth, as seen in Figure 1.

Site 1 - Sediment Mobility.

Sediment Mobility Introduction

Sediment mobility is estimated using the procedure described in McFall et al. (2015). This procedure is intended to approximate sediment mobility for planning purposes. Two methods are applied to estimate the sediment mobility for a selected location, water depth, and sediment profile of the material to be placed. Method 1 analyzes the bed shear stress from local wave and current conditions and compares it with the critical thresholds for various median grain size diameters. Assuming wave steepness (height/wave length) is small, Method 1 employs linear wave theory to calculate the near-bed wave orbital velocity and the resulting bed shear stress. Method 2 analyzes the near-bed velocity and compares the critical near-bed velocity to locally generated velocities. Method 2 uses nonlinear stream function wave theory to calculate the near-bed wave orbital velocity, which generally produces larger velocities than linear wave theory. By using both methods, a range of mobility is shown.

Site 1 - Method 1: Bed Shear Stress

The critical shear stress (τ_{cr}) is estimated from the Shields diagram following a procedure given by Soulsby (1997) and Soulsby and Whitehouse (1997) as

$$D_* = d_{50} \left(\frac{g\left(^{\rho_s}/_{\rho} - 1\right)}{\nu^2} \right)^{1/3} \tag{1}$$

$$\theta_{cr} = \frac{0.30}{1 + 1.2\, D_*} + 0.55[1 - \exp(-0.020\, D_*)] \tag{2}$$

and

$$\tau_{cr} = \theta_{cr}\, g\, (\rho_s - \rho)\, d_{50} \tag{3}$$

where D_* is the dimensionless grain size, g is the gravitational acceleration, ρ_s is the sediment density, ρ is the water density, ν is the kinematic viscosity, θ_{cr} is the critical Shields parameter, and τ_{cr} is the critical shear stress. The critical shear stress is the threshold stress for which the sediment can be expected to be dislodged from the seabed for all greater shear stresses.

The bottom skin shear stress is calculated using a method described by Soulsby (1997) and Myrhaug (1989) for currents and waves. Form shear stress is not included in the calculations. The current-induced shear stress (τ_c) is calculated as

$$\tau_c = \rho \left(\frac{\bar{U}\, \kappa}{\ln\left(\frac{z}{z_0}\right)} \right)^2 \tag{4}$$

where $\bar{U}$ is the estimated mean current velocity, κ is the von Karman's constant (κ = 0.4), z_0 is the bed roughness length ($z_o = d_{50}/12$ for flat sand), and z is the assumed height of the current velocity above the bed (z = 1 m). A current velocity of 0.1 m/s is assumed for the typical wave conditions whereas the current velocity for the storm conditions is obtained from the CHS database. The wave-induced shear stress (τ_w) is given as

$$\tau_w = \frac{1}{2}\, \rho\, f_w U_w^2 \tag{5}$$

where f_w is the wave friction factor and U_w is the bottom wave orbital velocity as determined by Soulsby (1997), which integrates the linear wave theory orbital velocity across the Joint North Sea Wave Observation Project (JONSWAP) wave spectra.

The maximum shear stress (τ_{max}) from the waves and currents shear stresses is calculated as

$$\tau_m = \tau_c \left[1 + 1.2 \left(\frac{\tau_w}{\tau_c + \tau_w} \right)^{3.2} \right] \tag{6}$$

and

$$\tau_{max} = [(\tau_m + \tau_w \cos\phi)^2 + (\tau_w \sin\phi)^2]^{1/2} \tag{7}$$

where τ_m is the mean bed shear stress and ϕ is the angle between the wave and current directions.

Figure 2 shows a histogram of maximum bed shear stresses using typical wave conditions and storm conditions for Site 1. The vertical dashed lines show the critical bed shear stress for several grain sizes. The legend shows the various median grain sizes (d_{50}), critical shear stress (τ_{cr}), and frequency of mobility (f_M).

a)
b)

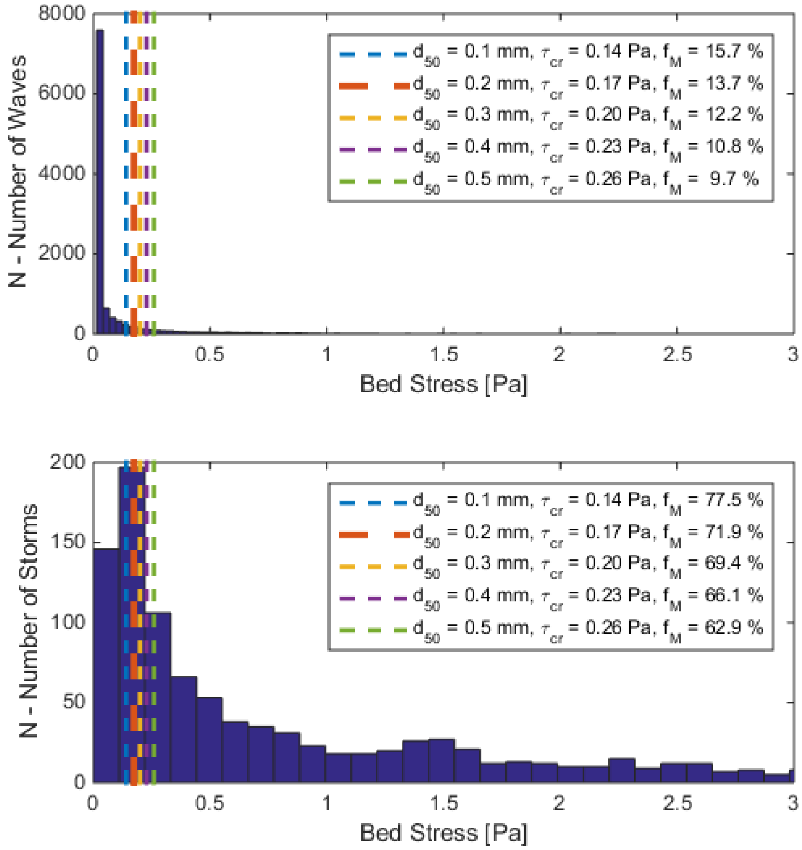

Figure 2. Site 1: Histogram of the calculated maximum bed shear stress using linear wave theory at a depth of 4 m for a) typical wave conditions and b) storm wave conditions. The critical bed shear stress for the median grain sizes is noted with the respective vertical dashed lines. The measured median grain size is d_{50}=0.21 mm. N is the number of typical waves or storm wave events in each shear stress bin. Values to the right of the vertical dashed lines mobilize the sediment.

The bed stress indicates the sediment will be mobilized by 10 to 16% of the waves under typical wave conditions and 63 to 78% of storm events for grain sizes from 0.1 to 0.5 mm. The measured d_{50} of 0.21mm from the sediment samples is predicted to be mobilized by 13.6% of the typical waves and 71.4% of the storm waves for Site 1.

Site 1 - Method 2: Near-bed Velocity

The critical near-bottom velocity (u_{cr}) using nonlinear stream function wave theory is calculated with a procedure given by Ahrens and Hands (1998), which is based on research by Hallermeier (1980) and Komar and Miller (1974), as

$$u_{cr} = \sqrt{8\, g\, \gamma\, d_{50}} \qquad \text{for } d_{50} \leq 2.0\ mm \tag{8}$$

and

$$u_{cr} = [0.46\, \gamma g\, T^{1/4} (\pi d_{50})^{3/4}]^{4/7} \qquad \text{for } d_{50} > 2.0\ mm \tag{9}$$

where T is the peak wave period and $\gamma = (\rho_s - \rho)/\rho$, where ρ_s is the sediment density, and ρ is the water density. Ahrens and Hands (1998) used Dean's (1974) stream function wave theory table (SFWT) to derive the following equations for the near-bottom wave induced velocity from the wave crest ($u_{\max crest}$) and trough ($u_{\max trough}$) as

$$u_{\max crest} = \left(\frac{H}{T}\right)\left(\frac{h}{L_o}\right)^{-0.579} \exp\left[0.289 - 0.491\left(\frac{H}{h}\right) - 2.97\left(\frac{h}{L_o}\right)\right] \tag{10}$$

and

$$u_{\max trough} = -\left(\frac{H}{T}\right)\exp\left[1.966 - 6.70\left(\frac{h}{L_o}\right) - 1.73\left(\frac{H}{h}\right) + 5.58\left(\frac{H}{L_o}\right)\right] \tag{11}$$

where h is the water depth, H is the significant wave height, and L_0 is the offshore wave length given by $L_0 = (g\,T^2)/2\,\pi$. The maximum near-bottom velocity was taken as $u_{max} = \max(|u_{\max crest}|, |u_{\max trough}|)$.

Histograms of the maximum near-bottom velocity using typical wave conditions and storm conditions at Site 1 are shown in Figure 3. The critical near-bed velocity for several grain sizes are noted with vertical dashed lines.

a)

b)

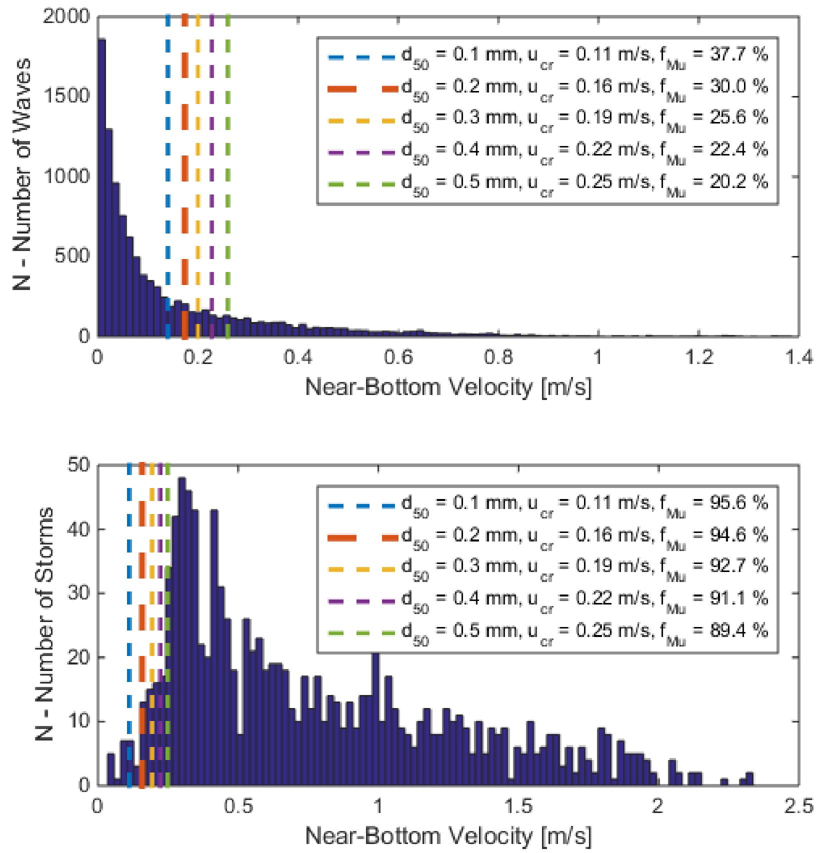

Figure 3. Site 1: Histogram of the calculated maximum near-bottom velocity using nonlinear stream wave theory at a depth of 4 m for a) typical wave conditions and b) storm wave conditions. The critical velocity for the respective median grain sizes are noted with the vertical dashed lines.

The near-bed velocity predicts the sediment to be mobilized by 20 to 38% of the typical waves and 89 to 96% of storm events at Site 1 (Figure 3). The measured d_{50} of 0.21mm from the sediment samples is predicted to be mobilized by 29.5% of the typical waves and 94.4% of the storm event waves.

Site 1 - Sediment Mobility Conclusions
Using the extremes of both methods, the sediment at Site 1 is predicted to be mobilized 10 to 38% of the time for the various grain sizes and 14 to 30% for the measured d_{50} of 0.21 mm under typical waves. The various grain sizes are predicted to be mobilized by 63 to 96% of the storm waves and the measured d_{50} is estimated to be mobilized by 71 to 94% of the storm event wave and current conditions. Table 1 shows the correlation between the average storm recurrance interval and the induced bed shear stress and near-bed velocity at Site 1.

Average Storm Recurrence Interval (yr)	Bed Shear Stress (Pa)	Near-Bed Velocity (m/s)
1	0.46	0.48
2	1.38	0.90
5	2.55	1.29
10	3.31	1.48
20	4.06	1.62
50	4.99	1.75
100	5.64	1.82

Table 1. Site 1: Bed shear stress and near-bed velocity induced by the average storm for various recurrence intervals.

It can be seen in Table 1 that the 1-year storm is predicted to mobilize the grain sizes in the proposed placement site by exceeding the critical bed shear stresses shown in Figure 2 and critical near-bed velocities shown in Figure 3. This result is appropriate considering the various sediment sizes are predicted to be mobilized by 71 to 96% of the storm events.

The National Oceanic and Atmospheric Administration (NOAA) predicts the annual average tidal range for Milford Harbor to be approximately 2 m for 2015 (NOAA, 2015). The sediment mobility calculations were performed for the proposed placement depth of 4 m. When the calculations were rerun at depths of 3 and 5 m, or ±1 m of the original depth, the frequency of sediment mobility generally increased and decreased by 10%, respectively. Thus, the frequency of mobility can vary ±10% due to tidally-induced depth variations.

Site 1 - Sediment Mobility Direction. Larson and Kraus (1992) hypothesized that artificial nearshore berm behavior should be similar to natural sand bars and studied the onshore and offshore migration of the offshore bar in Duck, North Carolina from 1981-1989. The dimensionless Dean number is generally used to determine bar migration and is given as

$$D = \frac{H_0}{\omega\, T} \tag{12}$$

where H_0 is the offshore wave height, ω is the sediment fall speed, and T is the wave period. Dean Number, D, values greater than 7.2 were found to induce erosive, offshore bar migration and values less than 7.2 resulted in accretionary, onshore bar migration.

The sediment fall speed is dependent on the grain size diameter and was calculated with the equations derived by Hallermeier (1981). The Dean Number is calculated for each wave record and the predicted sediment migration results are shown in Table 2 below. The finer sands are generally transported offshore during storm waves, while the coarser sands are transported towards shore.

d_{50} (mm)	**Typical Waves Predicted Sediment Migration**	**Storms Predicted Sediment Migration**
0.1	83% Erosive, Offshore Migration	97% Erosive, Offshore Migration
0.2	60% Accretion, Onshore Migration	52% Erosive, Offshore Migration
0.21	**63% Accretion, Onshore Migration**	**52% Accretion, Onshore Migration**
0.3	84% Accretion, Onshore Migration	74% Accretion, Onshore Migration
0.4	96% Accretion, Onshore Migration	91% Accretion, Onshore Migration
0.5	99% Accretion, Onshore Migration	99% Accretion, Onshore Migration

Table 2. Site 1: Predicted sediment migration for various sediment sizes exposed to typical wave conditions and storm conditions using the Dean Number.

In Table 2 the d_{50} of 0.21 mm was accretionary under the majority of typical and storm wave conditions. The sediment was accretionary under 52% of the storms; therefore 48% of the storms resulted in erosive migration. Although the majority of storm wave conditions produce onshore migration, there is not a strong signal of onshore migration dominance. A transition point from offshore to onshore sediment migration under the majority of storm wave conditions appears to be between grain sizes of 0.2 and 0.21 mm. Figure 4 shows a wave rose of the storm wave direction and wave height to estimate the dominant axis onshore and offshore migration. A wave rose is not created for the typical wave conditions because the wave direction is not recorded in the dataset used for typical wave conditions.

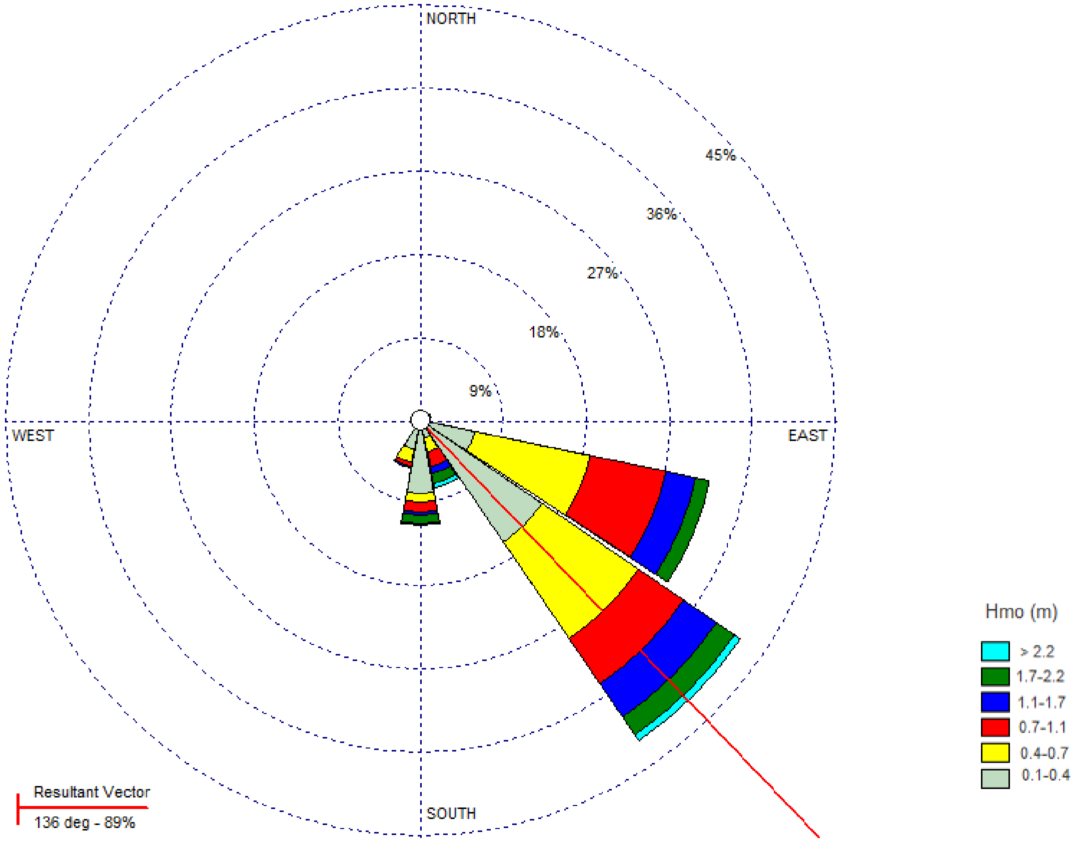

Figure 4. Site 1: Wave rose of the wave direction and zero moment wave height (H_{m0}) at the proposed site of the nearshore berm from storms using the meteorological direction convention. The resultant wave direction vector is 136° from north, propagating from the southeast.

The resulting storm wave vector is 136° from north, indicating that the material from the placement site will generally migrate towards Bayview Beach under accretionary waves. It is important to note that this migration direction only considers storm event waves and does not include typical wave conditions or the influence of tidally induced currents. This analysis only considers movement and direction of the material in the nearshore, and does not apply to the surf-zone and beach-water interface (swash-zone). In order to determine sediment behavior within the surf-zone and swash-zone, more robust numerical methodologies that consider waves, tidal currents, and wave-current interaction would be required.

Summary for Site 1. Table 3 summarizes the predicted sediment frequency of mobility and migration direction for the various grain sizes measured in the sediment samples using wave and current conditions for Site 1.

	Typical Waves		Storm Events	
d_{50} (mm)	**Frequency of Mobilization**	**Predicted Sediment Migration**	**Frequency of Mobilization**	**Predicted Sediment Migration**
0.1	16 – 38%	83% Offshore	78 – 96%	97% Offshore
0.2	14 – 30%	60% Onshore	72 – 95%	52% Offshore
0.21	**14 – 30%**	**63% Onshore**	**71 – 94%**	**52% Onshore**
0.3	12 – 26%	84% Onshore	69 – 93%	74% Onshore
0.4	11 – 22%	96% Onshore	66 – 91%	91% Onshore
0.5	10 – 20%	99% Onshore	63 – 89%	99% Onshore

Table 3. Site 1: Summary of the predicted sediment mobilization frequency and sediment migration directions for various grain sizes under typical wave conditions and storm wave conditions.

The average depth in the site is 4 m. The average median grain size (d_{50}) from sediment samples is 0.21 mm. The d_{50} of 0.21 mm is predicted to move predominantly onshore under typical and storm wave conditions. The coarser material is more likely to move onshore, while the finer material is more likely to move offshore. The sediment is predicted to be moved by 14 to 30% of the typical significant wave heights and 71 to 94% of the storm wave heights. The wave rose shows the resultant storm wave direction to be 136° from north, making the dominant migration axis for storm events towards Bayview Beach during accretionary waves. Tidal- and watershed-induced currents may modify this migration axis.

EVALUATION OF SITE 2. The alternate proposed nearshore placement site is closer to the ship channel than Site 1 (Figure 1). The water depth is approximately 3 m at this location, which is 1 m shallower than Site 1, resulting in larger shoaling waves and increased sediment mobility. The dredged material will originate from the same place as Site 1, and therefore has the same grain size characteristics. The same equations and methods used for the analysis at Site 1 were used for the analysis at Site 2.

Site 2 - Sediment Mobility.

Site 2 - Method 1: Bed Shear Stress

Similar to Site 1, Equations 1-7 are used to calculate maximum bed shear stress using both typical wave conditions as well as storm conditions. Critical shear stress is estimated from the Shields diagram following Soulsby (1997) and Soulsby and Whitehouse (1997) (Equations 1-3). The current induced shear stress is calculated using Equation 4, and the wave induced shear stress is calculated using Equation 5. Finally, the maximum bed shear stress was calculated using Equations 6 and 7. The maximum bed shear stress is compared to critical thresholds for various median grain size diameters.

Figure 5 shows a histogram of maximum bed shear stresses for Site 2, with vertical dashed lines indicating critical bed shear stress for several grain sizes. At Site 2, the bed stress indicates that sediment will be mobilized 13 to 21% of the time during typical wave conditions and 73 to 88% during storm wave conditions. The measured d_{50} of 0.21mm from the sediment samples is predicted to be mobilized by 17.1% of the typical waves and 82.2% of the storm waves at Site 2. Overall, the frequency of mobility percentages were higher for Site 2 than for Site 1.

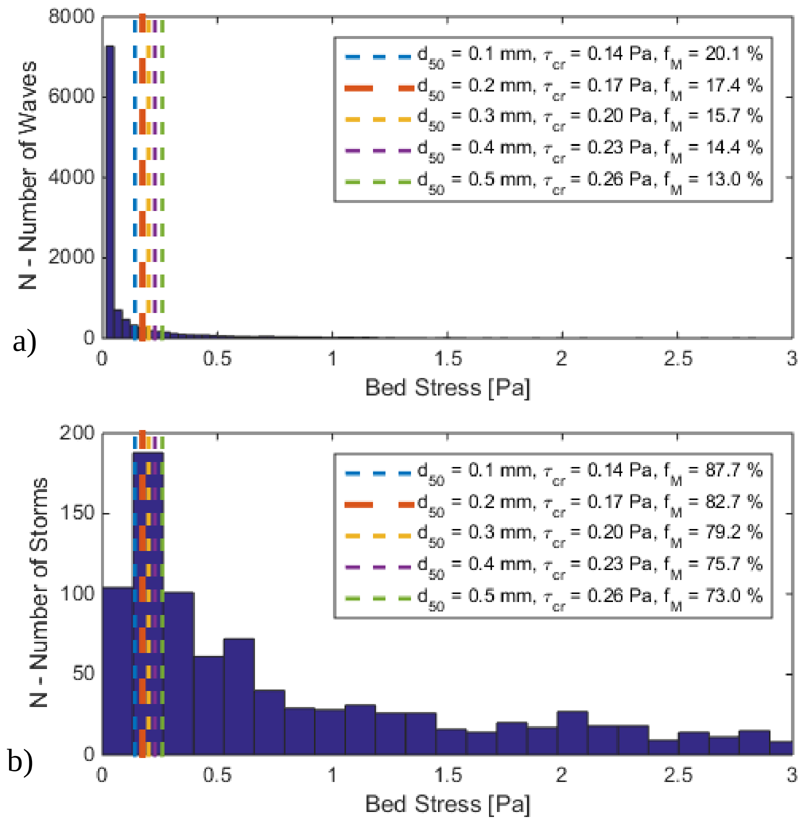

Figure 5. Site 2: Histogram of the calculated maximum bed shear stress using linear wave theory at a depth of 3 m for a) typical wave conditions and b) storm wave conditions. The critical bed shear stress for the respective median grain size noted with the vertical dashed lines.

Site 2 - Method 2: Near-bed velocity

The near-bed velocity method described in detail for Site 1 is also used for Site 2. The critical near-bottom velocity is calculated using Equations 8 and 9, where the wave-induced velocity from the wave crest and trough is quantified using Equations 10 and 11. The maximum near-bottom velocity is taken as $u_{max} = \max(|u_{\max crest}|, |u_{\max trough}|)$.

The histogram shown in Figure 6 describes the near-bed velocity and critical near-bed velocity for certain grain sizes under typical wave conditions and storm wave conditions. Using this method, it is predicted that under typical wave conditions, sediment will move 24 to 44% of the time. Under storm wave conditions, sediment will move 93 to 97% of the time. The measured

d_{50} of 0.21mm from the sediment samples is predicted to be mobilized by 34% of the typical waves and 96% of the storm waves at Site 2. Similar to the bed shear stress method, the percentages of mobility were higher for Site 2 than for Site 1.

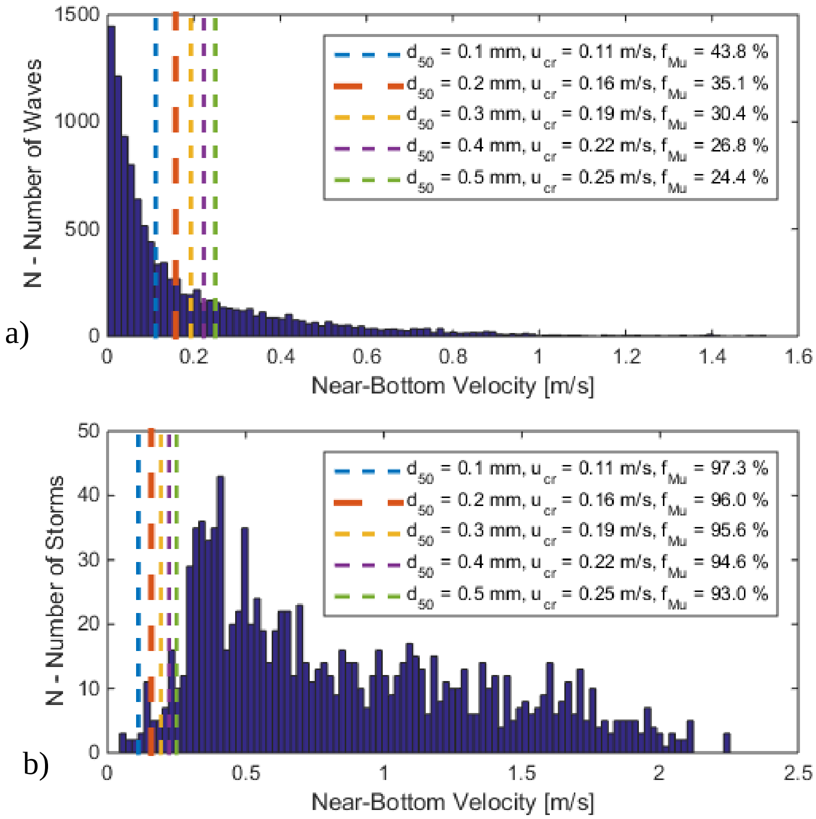

Figure 6. Site 2: Histogram of the calculated maximum near-bottom velocity using nonlinear stream wave theory at a depth of 3 m for a) typical wave conditions and b) storm wave conditions. The critical velocity for the respective median grain sizes are noted with the vertical dashed lines.

Site 2 - Sediment Mobility Conclusions

The sediment at Site 2 is predicted to be mobilized 13 to 48% of the time for the various grain sizes and 17 to 34% for the measured d_{50} of 0.21 mm under typical waves. The various grain sizes are predicted to be mobilized by 73 to 97% of the storm waves and the measured d_{50} is estimated to be mobilized by 82 to 96% of the storm event wave and current conditions. Table 4 shows the correlation between the average storm recurrance interval and the induced bed shear stress and near-bed velocity at Site 2.

Average Storm Recurrence Interval (yr)	Bed Shear Stress (Pa)	Near-Bed Velocity (m/s)
1	0.78	0.63
2	2.13	1.09
5	3.62	1.46
10	4.59	1.64
20	5.43	1.76
50	6.40	1.86
100	7.08	1.91

Table 4. Site 2: Bed shear stress and near-bed velocity induced by the average storm for various recurrence intervals.

The average 1-year storm is predicted to mobilize the grain sizes in Site 2 by exceeding the critical bed shear stresses shown in Figure 5 and critical near-bed velocities shown in Figure 6. This result is appropriate considering the various sediment sizes are predicted to be mobilized by 73 to 97% of the storm events.

Site 2 - Sediment Mobility Direction. Similar to Site 1, the Dean number (Equation 12) is used to calculate onshore or offshore migration of the sediment at Site 2. Dean number values greater than 7.2 are considered erosive, and values less than 7.2 are considered accretionary. Table 5 shows the predicted sediment migration for the range of sediment sizes in the channel to be dredged, including the measured d_{50} of 0.21 mm, exposed to typical and storm wave conditions.

d_{50} (mm)	Typical Waves Predicted Sediment Migration	Storms Predicted Sediment Migration
0.1	83% Erosive, Offshore Migration	97% Erosive, Offshore Migration
0.2	60% Accretion, Onshore Migration	52% Accretion, Onshore Migration
0.21	**63% Accretion, Onshore Migration**	**55% Accretion, Onshore Migration**
0.3	84% Accretion, Onshore Migration	79% Accretion, Onshore Migration
0.4	96% Accretion, Onshore Migration	97% Accretion, Onshore Migration
0.5	99% Accretion, Onshore Migration	99% Accretion, Onshore Migration

Table 5. Site 2: Predicted sediment migration for various sediment sizes exposed to typical wave conditions and storm conditions using the Dean Number.

The onshore/offshore sediment migration is the same for both placement sites under typical wave conditions. This is because the typical offshore waves were transformed to the placement sites using conservation of energy flux which assumes the wave period remains constant, therefore the Dean's Number remains constant. Additionally the wave directions are not measured at the offshore buoy so the waves are assumed to be propagating towards shore and are transformed into the placement sites with direct shoaling, rather than taking into account the topographic and bathymetric anomalies like the small peninsula between Bayview and Gulf Beaches.

The storm sediment migration is slightly more accretionary for the measured d_{50} in Site 2 than in Site 1 (55% and 53%, respectively). The high fidelity models used to calculate the storm wave characteristics account for topographic and bathymetric anomalies and the nonlinear effects that can alter the Dean Number.

A wave rose with the storm wave height and direction at the Site 2 is shown in Figure 7 below. Similar to Site 1, a wave rose for typical wave conditions was not created since the wave direction is not recorded in the dataset used. For Site 2, the resulting storm wave vector is 144º from north, indicating that material will likely migrate towards the northern portion of Gulf Beach, near the Milford Harbor under accretionary conditions. Due to the orientation of the placement at Site 2 relative to the dominant wave direction, there is likely to be a strong alongshore component of transport as compared to Site 1. As is the case with Site 1, it is important to note that this migration direction only considers storm waves and not typical wave conditions. Tidal influences are also not considered in this analysis.

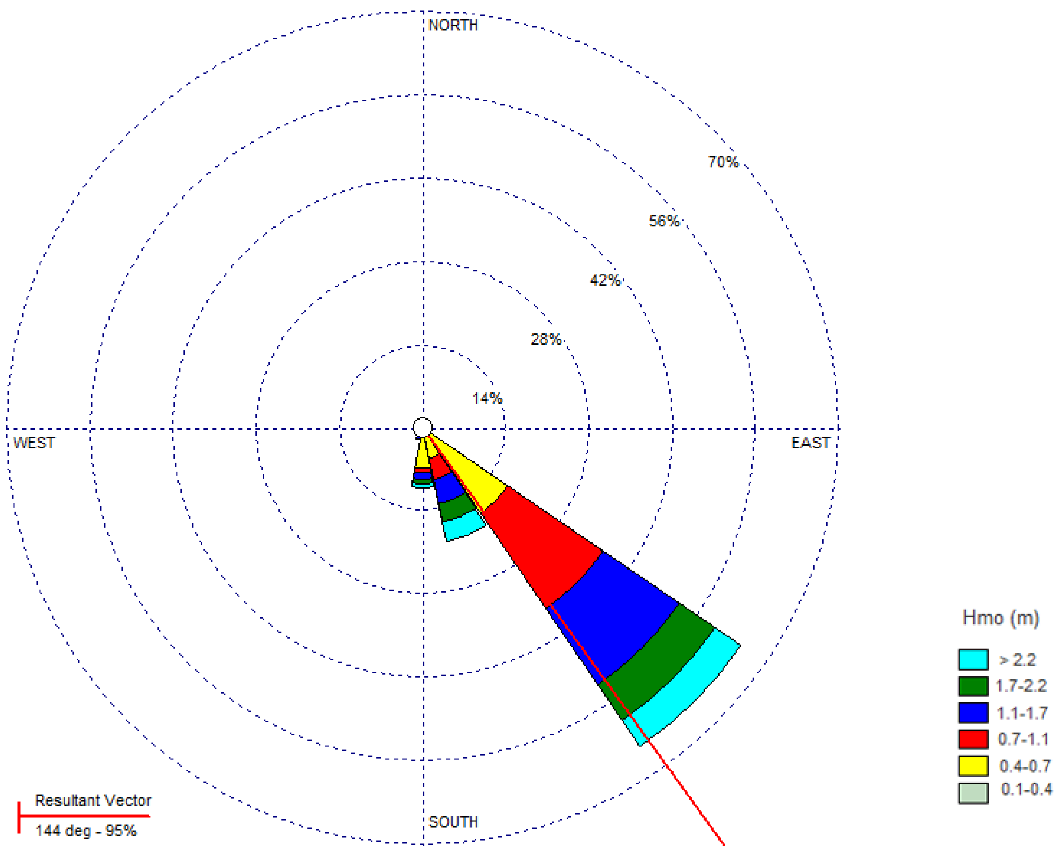

Figure 7. Site 2: Wave rose of the wave direction and zero moment wave height (H_{m0}) at the proposed site of the nearshore berm from storms using the meteorological direction convention. The resultant wave direction vector is 144° from north, propagating from the southeast.

Summary for Site 2. Table 6 summarizes the predicted sediment frequency of mobility and migration direction for the range of grain sizes in the channel to be dredged under typical and storm wave conditions for Site 2.

	Typical Waves		Storm Events	
d_{50} (mm)	**Frequency of Mobilization**	**Predicted Sediment Migration**	**Frequency of Mobilization**	**Predicted Sediment Migration**
0.1	20 – 44%	83% Offshore	88 – 97%	97% Offshore
0.2	17 – 35%	60% Onshore	83 – 96%	52% Onshore
0.21	**17 – 34%**	**63% Onshore**	**82 – 96%**	**55% Onshore**
0.3	16 – 30%	84% Onshore	79 – 96%	79% Onshore
0.4	14 – 27%	96% Onshore	76 – 95%	97% Onshore
0.5	13 – 24%	99% Onshore	73 – 93%	99% Onshore

Table 6. Site 2: Summary of the predicted sediment mobilization frequency and sediment migration directions for various grain sizes under typical and storm wave conditions.

As is the case for Site 1, the median grain size (d_{50}) from the sediment samples is 0.21 mm. This grain size is predicted to move dominantly onshore for both typical waves and storm events. As would be expected, storm events create a higher frequency of mobility (82 – 96%) than do typical waves (17 – 34%). The resultant storm wave direction shown in the wave rose for Site 2 is 144° from north, making the wave dominant migration axis for storm events towards the northern portion of Gulf Beach, near Milford Harbor under accretionary conditions.

SUMMARY. Two proposed nearshore placement sites in Milford, Connecticut, have been evaluated to estimate the potential sediment mobility frequency and bar migration direction for each location. The sediment mobility has been estimated using linear and nonlinear wave theories to provide a range of mobility frequency. Linear wave theory provides a more conservative estimate of the sediment mobility due to the smaller wave-induced velocity compared to nonlinear stream function wave theory. Nonlinear wave theory is appropriate when the wave steepens and becomes asymmetric in the nearshore region. Sensitivity analysis of the water depth indicates the frequency of sediment mobility can vary by ±10% within the annual average tidally-induced depth variation. The onshore or offshore sediment migration has been estimated using the critical Dean number. Wave roses have been created with storm wave height and direction to show the dominant axis of storm wave induced migration for each site. The axes of migration do not consider typical wave conditions, tidally influenced currents, or riverine and watershed runoff induced currents. An overall comparison of the two sites is shown in Table 7.

	Site 1		Site 2	
Wave Condition	**Typical**	**Storm**	**Typical**	**Storm**
Frequency of Mobilization	14 – 30%	71 – 94%	17 – 34 %	82 – 96%
Predicted Sediment Migration	63% Onshore	52% Onshore	63% Onshore	55% Onshore
Migration Direction	N/A	136º	N/A	144º

Table 7. Overall comparison of the two potential placement sites for typical wave conditions and storm event wave conditions.

Overall, Site 2 has a higher frequency of mobility than Site 1, likely due to the shallower placement location inducing larger shoaling waves and therefore increased sediment mobility. Both sites are expected to experience onshore migration of the sediment under 63% of the typical waves. The majority of storm wave conditions at both sites produce onshore migration (52% for Site 1 and 55% for Site 2), but there is not a strong signal of onshore migration dominance. Although slightly different at each site, overall, the dominant storm wave direction is from the southeast propagating northwest for both sites. Due to its orientation, the southeast to northwest wave propagation will result in a relatively large alongshore component at Site 2 compared to an onshore component for Site 1. The dominant storm wave axis of migration under accretionary waves is towards Bayview Beach from Site 1 and is towards the northern portion of Gulf Beach, near Milford Harbor, for Site 2.

ADDITIONAL INFORMATION. This Coastal and Hydraulics Engineering Letter Report(CHELR) was prepared for the New England District (NAE) by Dr. Brian C. McFall, Cheryl E. Pollock, and Dr. Katherine E. Brutsché, U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL), Vicksburg, MS. Questions pertaining to this CHELR may be directed to Brian McFall (Brian.C.McFall@usace.army.mil).

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