Wave-Induced Sediment Movements

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WAVE-INDUCED SEDIMENT MOVEMENTS

In 1969, a small but intense storm, Camille, struck the Mississippi delta area off the Louisiana coast, a prolific oil and gas producing area. In the aftermath of the storm, one platform was found toppled and two other platforms were damaged severely, all in the So. Pass area of the Mississippi delta (Bea, 1971). Subsequent investigations showed that the damage occurred not as a result of wind and wave forces on the platforms, but rather from wave-induced mudslides in the soft, underconsolidated sediments prevalent in the area. Although platform damage and pipeline breakages were well known in the area prior to Camille, this was the first major destruction of a large platform, and it sparked a multitude of studies on the topic of wave seabottom interaction. One of the first studies reported was by Henkel (1970) who used standard slope stability analysis with the forcing function being wave-induced bottom pressures. The only sediment properties required in this analysis were unit weight and undrained shear strength. Subsequently, Wright and Dunham (1972). developed a finite element approach which took into account the actual sediment properties at So. Pass 70, the location where the platform was destroyed during Camille. Other analytical studies have been conducted, many of which used Biot's formulation for poro-elastic bodies. In addition, a significant number of geological studies were conducted which developed the extent and magnitude of seafloor slides in the Mississippi delta area, as well as their morphology (e.g. Coleman et al., 1978 and 1980).

The study reported herein utilized results from research conducted for Chevron, Gulf and Mobil oil companies in response to needs for design of mudslide resistant platforms in the So. Pass 57-77 area. This research took three main paths: a) the development of constitutive relationships for the soft sediments involved, b) determination of sediment drag forces on platform members, mostly for piles, and c) development of the governing equations for sediment movement and numerical solution of these equations. The end product was a computer program which provided cyclic and downslope movement of the sediments in response to wave forces and calculated the drag forces of the sediment on platform members.

Sediment Constitutive Relationships

The constitutive relationship study was initiated by Stevenson (1973), who used a miniature vane rotated at four different rates in the sediment. A vane was chosen for the following reasons: the samples were often too soft to stand on their own, consequently they could not be trimmed or transferred to more sophisticated testing devices without causing significant disturbance, and the insitu vane which had recently become available showed promise for obtaining the same information insitu. Figure 1 is a typical torque-rotation plot. Calculations from these data are then plotted as shown in Fig. 2. These types of plots clearly show that the sediment behaves in a viscoelastic manner; the slope of the sets of lines in Fig. 2 represents the viscoelastic rate constant, n. King (1976) conducted a companion study using a long cylindrical vane installed along the axis of sediment samples which were contained in their original sampling liners. Studies by

Marti (1976)and later by Riggins, (1980) who used a large simple shear device, further confirmed that the sediments behaved as nonlinear viscoelastic materials.

Subsequent tests on large numbers of Gulf of Mexico sediment samples established that the value of the constant, n, ranged from about 0.06 to 0.15 with the majority of values between 0.09 and 0.12. A relationship between the liquidity index of the sediment and n was developed. This relationship is coded into the computer program; alternatively, the actual values of n obtained by tests on sediment samples can be entered.

<u>Sediment Drag Forces</u>

The drag forces of moving sediment on platform elements is responsible for platform distress and destruction. Although the PODS study is concerned with the analysis of seafloor slides, the calculation of drag forces on structures provides a convenient method of determining the severity of the slide movement. Conventional geotechnical practice dictates a standard bearing capacity approach to this problem, usually with a bearing capacity factor of 9 or 10 applied to the frontal area of the structural element. However, the problem is more complicated than this as soft sediment, similar to a liquid, can actually flow around a structural shape. This requires an approach more akin to the classical Morison equations used for fluid flow around an object.

Marti (1976) studied this problem using experimental and theoretical approaches. He conducted an extensive set of experiments with model instrumented piles of various diameters buried in a soft, prepared sediment. The experimental equipment was capable of applying confining pressures up to 30 psi to the pile-sediment mass, it allowed the pile to move in the sediment at controlled velocities, and since the testing equipment was also a large scale simple shear device, it allowed for sediment movement against the model piles.

Based on a plastic limit analysis of the problem and the experimental results, Marti determined that the drag force per unit length of pile was:

$$F = 11.42(125.9n)^n (V/D)^n C_{uo} D$$

Where Cuo = vane shear strength determined at the standard rotation rate
V = velocity of sediment movement with respect to the pile
D = pile diameter
n = viscoelastic constant

Some time later, this approach was further validated during an AGA-sponsored project on sediment drag forces on buried pipelines (Schapery and Dunlap, 1984).

<u>Mathematical Solution to Wave-Seabottom Interaction</u>

Armed with knowledge of the sediment constitutive behavior, Schapery (1976) developed the governing equations for determining sediment movement induced by storm waves. After developing these equations, a computer program was written to solve the equations (Raju and Schapery, 1976). It is emphasized that this is not a finite element

approach but rather a numerical solution of the closed form equations. The computer program is capable of providing:
a. horizontal and vertical sediment movements at desired locations below the mudline
b. downslope movement if the sediment is on a slope
c. drag forces against a pile of specified diameter at desired depths below the mudline
d. wave degeneration, i.e. distance required for a wave to lose a percentage of its height due to energy lost while traveling over a deformable bottom.

Key input parameters to the program are:
a. wave characteristics – length, height and period
b. sediment characteristics – undrained shear strength, Cu; viscoelastic rate parameter, n; unit weight; strength factor, G1/Cu; all with respect to depth
c. water depth and water unit weight
d. subbottom slope
There are other input parameters relevant to the internal operations of the program including an error tolerance applicable for certain iterative operations while solving the governing equations. An initial stress ratio – shear stress/shear strength – must also be entered; a reasonable guess will hasten the iterative process.

The strength factor, SF, is defined as the viscoelastic modulus, G1, divided by the undrained shear strength, Cu, determined by a vane shear at the standard rotation rate, i.e. SF = G1/Cu. Although actual values determined by test can be input into the program, early correlations established this ratio as 32. Later research using the large scale simple shear device (Riggins, 1980) determined that a value of 120 seemed more appropriate, especially for large strains.

The original program used a parallelogram model for cyclic stress-strain behavior of the sediment. Later a somewhat more realistic hyperbolic stress-strain equation was used to relate the maximum shear stress and strain. This was followed by the addition of a Ramberg-Osgood model. The user may specify which option is desired in the program; the hyperbolic law is generally preferred.

The sediment properties must be entered with respect to depth by assuming constant properties for each of several successive layers. Although there is no practical limit to the number of layers that can be used, the program in its present state utilizes a maximum of 10 layers plus a "hard bottom". This hard bottom is not necessarily perfectly rigid, but it is expected to be a zone that is significantly stronger and more rigid than the overlying sediments.

Finally, the program does not consider a change (degradation) in sediment properties with loading cycles although this could easily be done. The present procedure is to successively replace the original properties with new ones based on cyclic load test and rerun the program.

Program Validation

As with other wave-seabottom interaction programs, this program has not been tested in nature from the standpoint that no platforms designed using this approach have been subjected to large storms. There is, however, anecdotal evidence that supports its validity. One platform designed using the program was found at a later date to have more than 20 ft. of sediment that had accumulated against the legs since construction, but with no distress to the platform. This greatly exceeds normal sedimentation rates in the area and it must have been caused by sediment movement. But it is unknown whether this was a single slide or an accumulation of smaller slides. One validation approach is to test the program against the behavior of the one platform that is known to have failed during a storm event – So. Pass 70 B. The company records of this failure are not available, however, there are ample data available in the public records to reconstruct the important parameters.

Extensive studies were conducted on sediment strength using both laboratory and in situ vane measurements. Bea and Arnold (1973) reported a somewhat idealized strength profile which considered in situ vane measurements (Fig. 3). This shows an often observed "cutback zone" of strengths at sediment depths of 60 to 85 ft. Hooper (1980) suggested reasons why these zones of lower strength occur. If these cutback zones occur within the subbottom depth where maximum shear stresses due to waves occur (0.16L for an elastic material) the subbottom movements can be greatly magnified.

Wave parameters at the platform location were also reported by Bea and Arnold (1973) as follows:
Wave height = 65 ft.
Wave length = 1000 ft.
Wave period = 14 secs.
These are hindcast values and do not reflect API design standards.

Other sediment parameters such asn, strength factor and unit weight were varied over typical ranges with little effect on the results, probably because the shear strengths and the location of the cutback zone largely governed the magnitude of the movement. The results are illustrated in Figs. 4 and 5. Maximum downslope velocity was determined as 22.6 fps. These results do not consider a reduction in strength due to cyclic movement. The calculated cyclic movements (Fig. 4) are larger than those reported by others (Arnold, 1973, Bea and Arnold, 1973), however, they did not calculate continuous downslope velocities. Comparison of pre-and post-Camille soil borings by Sterling and Strohbeck (1973) showed significant strength reductions at the 70-80 ft. range after the storm which they interpreted as the depth to which the sediment was remolded as it translated downslope. Although this compares favorably with the wave-seabottom program results (Fig 4) this is not too surprising since the cutback zone occurs at roughly the 70-80 ft range.

One interesting aspect is that the lateral loading on the pile always occurs on the upslope side of the pile (Fig. 5) even though the cyclic sediment movement switches from one side of the pile to the other in concert with the waves. Running the program with the same parameters as before but using a 0% slope results in the lateral loading shown in Fig. 6. Obviously, the significant downslope velocity and the resulting drag forces on the pile overwhelm any lateral loads caused by cyclic upslope movement of the sediment.

In summary, the wave-seabottom interaction program calculates movements and forces which are consistent with failure of the So. Pass 70B platform. Significant downslope velocities and large cyclic sediment movements were predicted.

Parametric Study

A parametric study was conducted to provide guidance for estimating seafloor sliding potential. There are too many variations of near surface sediment properties with depth to be able to handle even a small number of them. The approach taken here was to use a range of properties consistent with the Gulf of Mexico but which are probably valid for a number of other near-surface cohesive sediment locations.

The single most important sediment property governing wave-seabottom interaction is shear strength. In slide prone areas in the Gulf of Mexico, surface sediments have an undrained shear strength of approximately 50 psf which then increases with depth depending on the state of consolidation: 5 psf for highly underconsolidated clays, 8 psf for lightly underconsolidated clays and 11 psf for normally consolidated sediments. These strength were used in the parametric study. The thickness of soft sediments prone to sliding also varies widely but it ranges generally from about 75 ft to 175 ft. Sediment unit weight, while obviously important , varies only slightly from 100 pcf for the soft sediments. As discussed earlier, an n value of 0.09 is an average value. Finally, a strength factor of 120 was used.

Water wave characteristics were selected from API guidelines for maximum height and length, and a period of 13 seconds was used. Water depths of 100, 200 and 300 ft. were used as shown below:

Water depth, ft	Maximum wave height, ft	Wave length, ft
100	57	648
200	66	794
300	68	845

Bottom slopes were 0, 0.5, 1.0 and 1.5%. Sediment forces were calculated on a 4 ft. diameter pile.

Results of the parametric study are presented in Figs. 7, 8 and 9. For the 75 ft. sediment thickness and 100 ft. water depth, large cyclic displacements and downslope velocities occurred for the highly underconsolidated and lightly underconsolidated sediments, while normally consolidated sediment exhibited only minor movement. However, this movement is probably still large enough to create cyclic strength degradation. In 200 ft water depth, only the highly underconsolidated sediment exhibited significant cyclic and downslope movement. Insignificant movements occurred with all states of consolidation in the 300 ft. water depth.

Increasing the sediment thickness up to 175 ft. had a relatively small effect (roughly a

10% increase) on cyclic displacement. Downslope movements were increased somewhat more, but again it was not a major change. Increasing the water depth had a more significant effect than increasing the sediment thickness.

Obviously, the potential exists for infrastructure damage in the 100 ft. water depth for underconsolidated sediments, and in the 200 ft. depth for highly underconsolidated sediments, but damage potential is very slight in 300 ft. water depth for even very weak sediments. This considers only linear increases of strength with depth, not cutback zones of lower shear strengths in the strength-depth profile. The calculated displacements should be considered as a snapshot in time. Continual movement, particularly downslope, may be modified by changes in slope, and by loss of strength due to large strains and mixing with moisture. The latter could lead to true debris flows with potentially higher velocities.

Attempts to place these results in dimensionless graphs have so far been unsuccessful, probably because of the highly nonlinear nature of the problem. However, interpolation between graphs should be fairly successful.

Conclusions

Shallow water storm waves of major proportions can cause significant movements in soft sediments. Two types of movements can occur: a cyclic back and forth movement in response to bottom pressures which can occur even if the bottom is flat, and a downslope movement if the bottom is sloped. Highly underconsolidated and slightly underconsolidated clays will suffer large movements whereas normally consolidated clays will be only marginally affected, although continued wave loading in water depths less than 200 ft. may cause cyclic reduction of strength which can lead to more significant movements. These conclusions are based on shear strengths that increase fairly linearly with depth. If the strength-depth profile contains cut back zones where the strength decreases at depth, the sediment movements can become much larger. This depends greatly on the depth of the cutback zone in relation to the wave length.

Graphs have been presented which can be helpful as a first estimation of whether sediment movement will be a problem.

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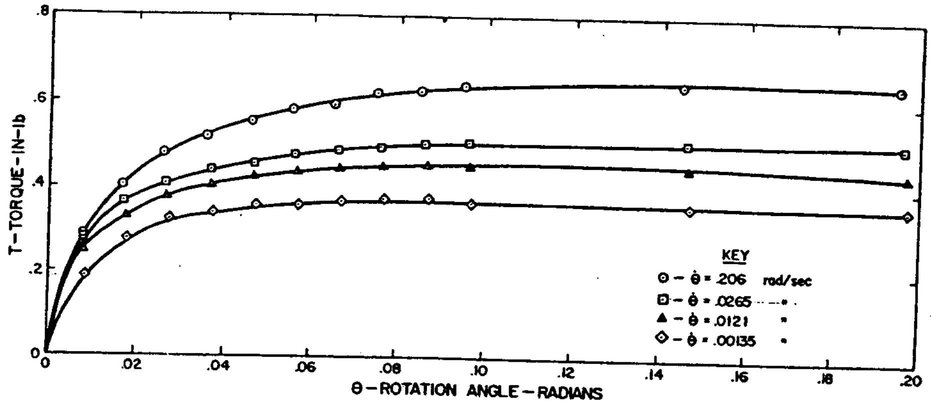


Fig. 1 – Typical torque vs. rotation data

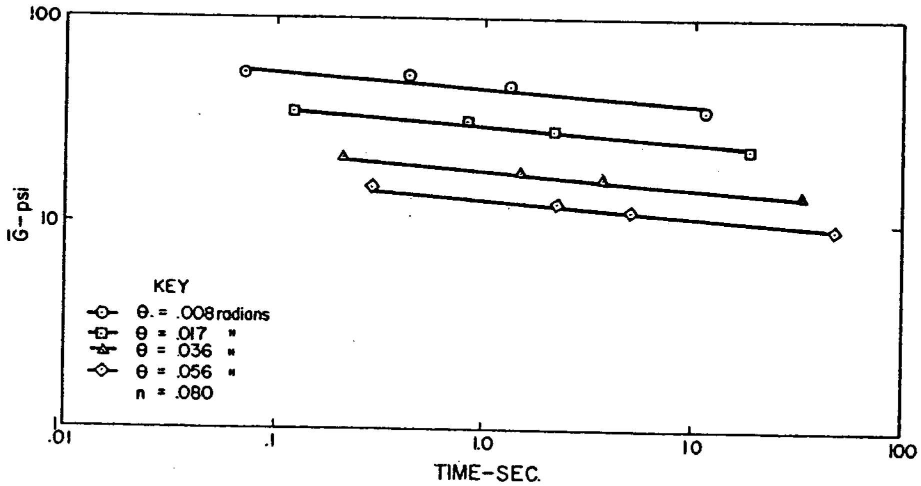


Fig. 2 – Relationship between stiffness G and time for results shown in Fig. 1

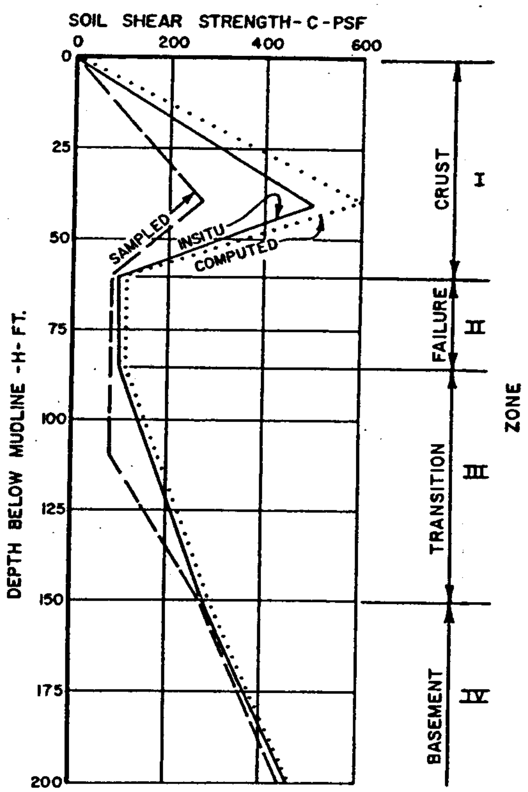


Fig. 3. Idealized shear strength profile for So. Pass 70B platform. In situ strength was used for analysis

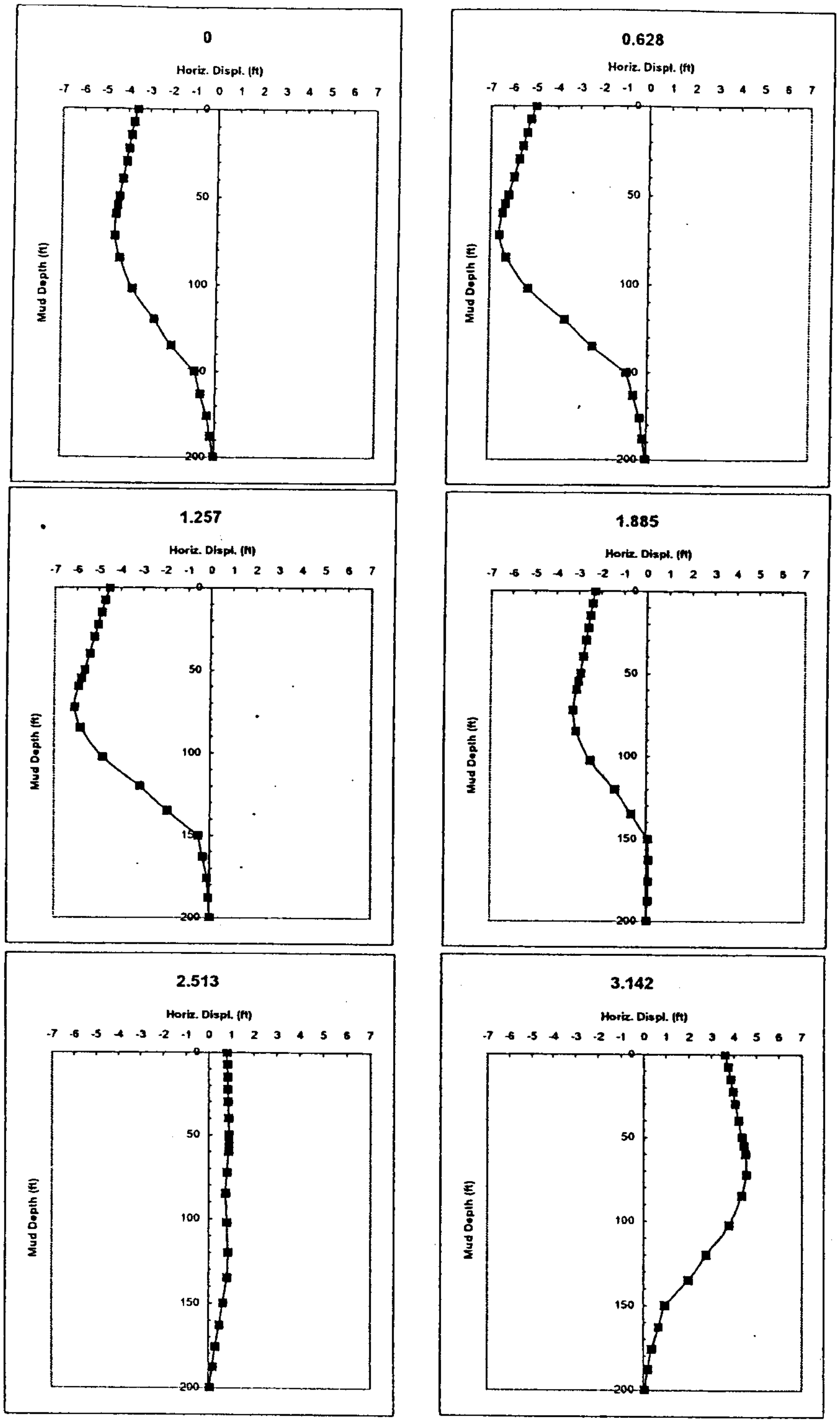


Fig. 4. Calculated cyclic sediment movements for So. Pass 70B platform. Bold numbers on top of each graph indicate distance along the wave length of 2pi

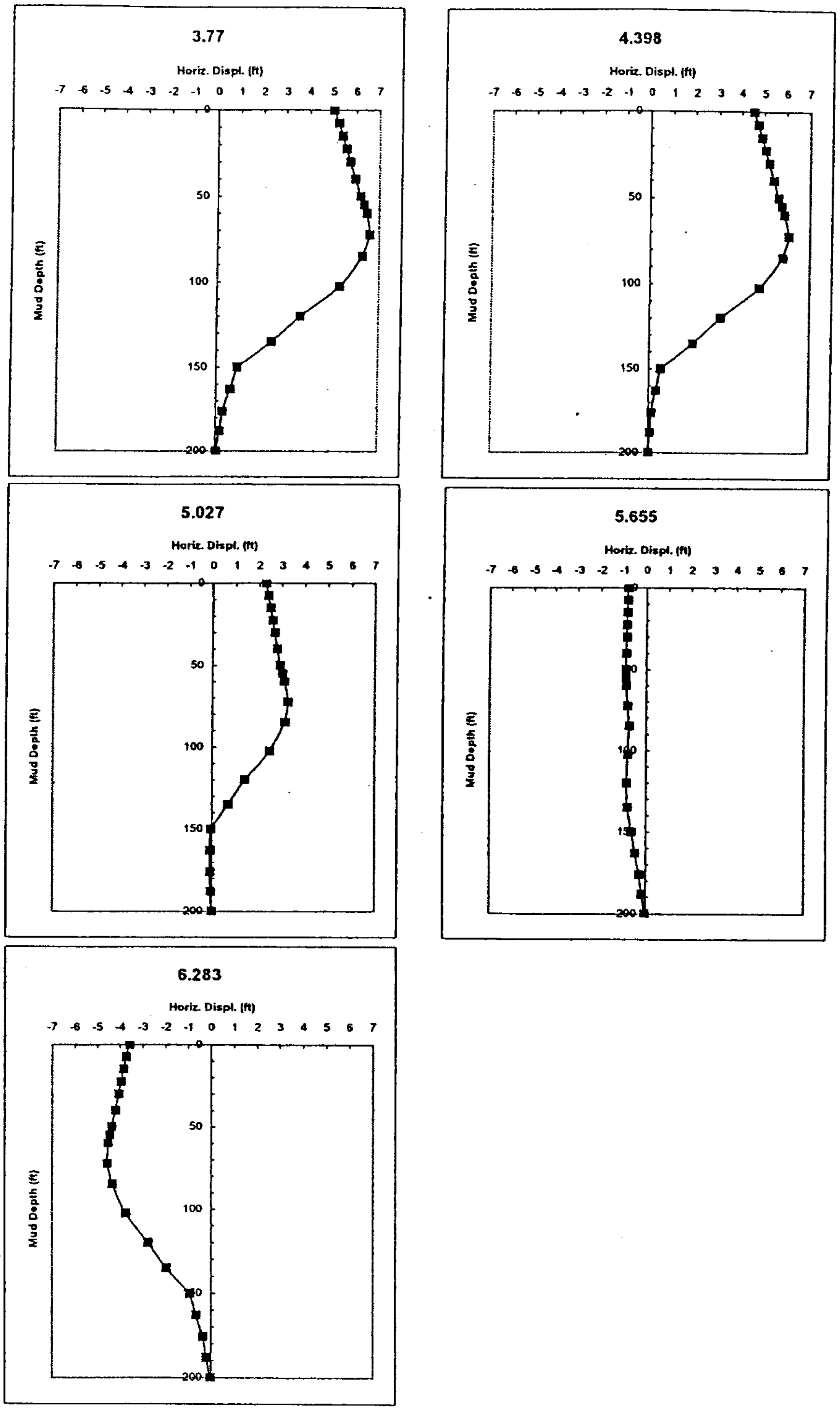


Fig. 4. Continued

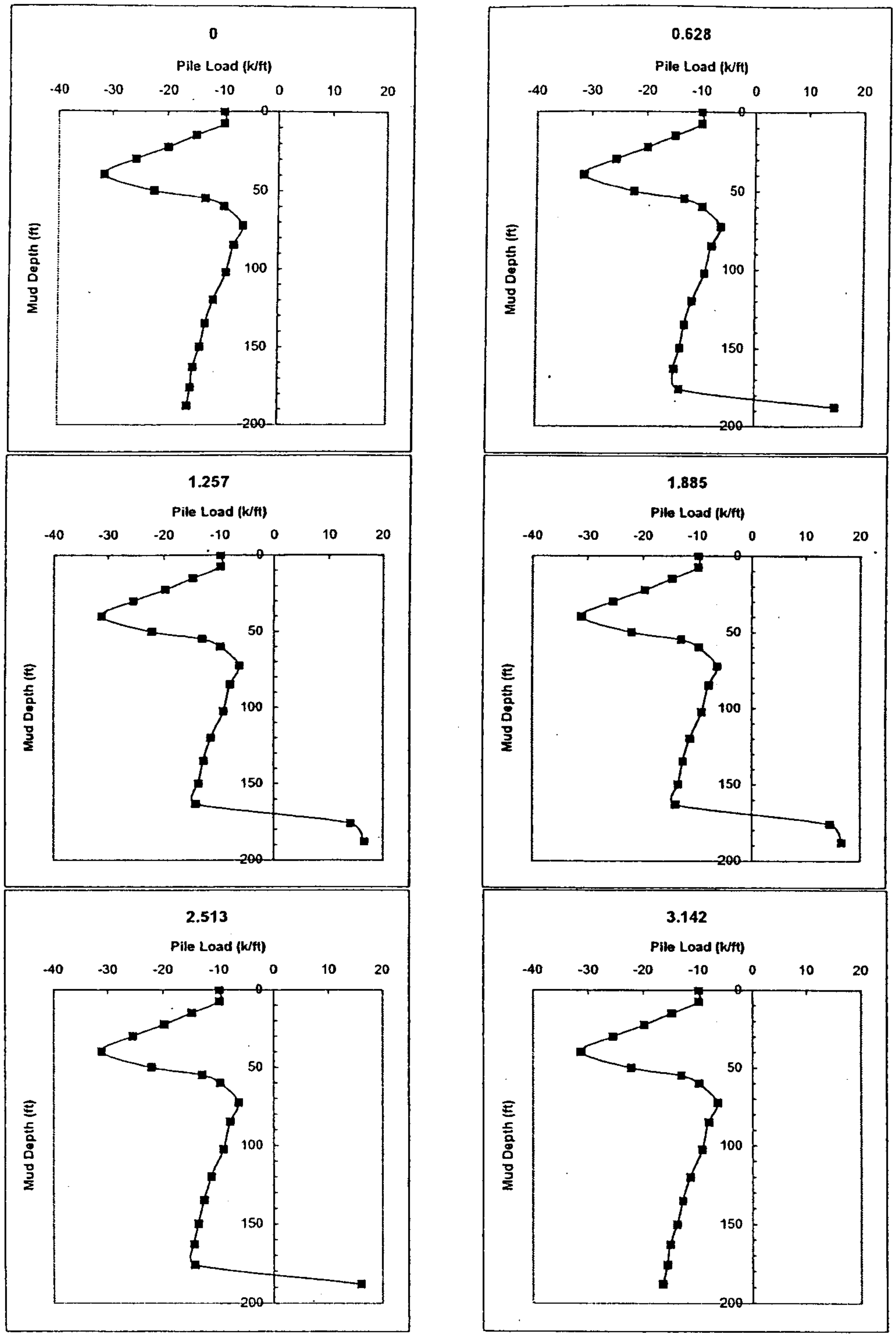


Fig. 5. Calculated sediment forces along 4 ft. diameter pile with 1% bottom slope for So. Pass 70B platform

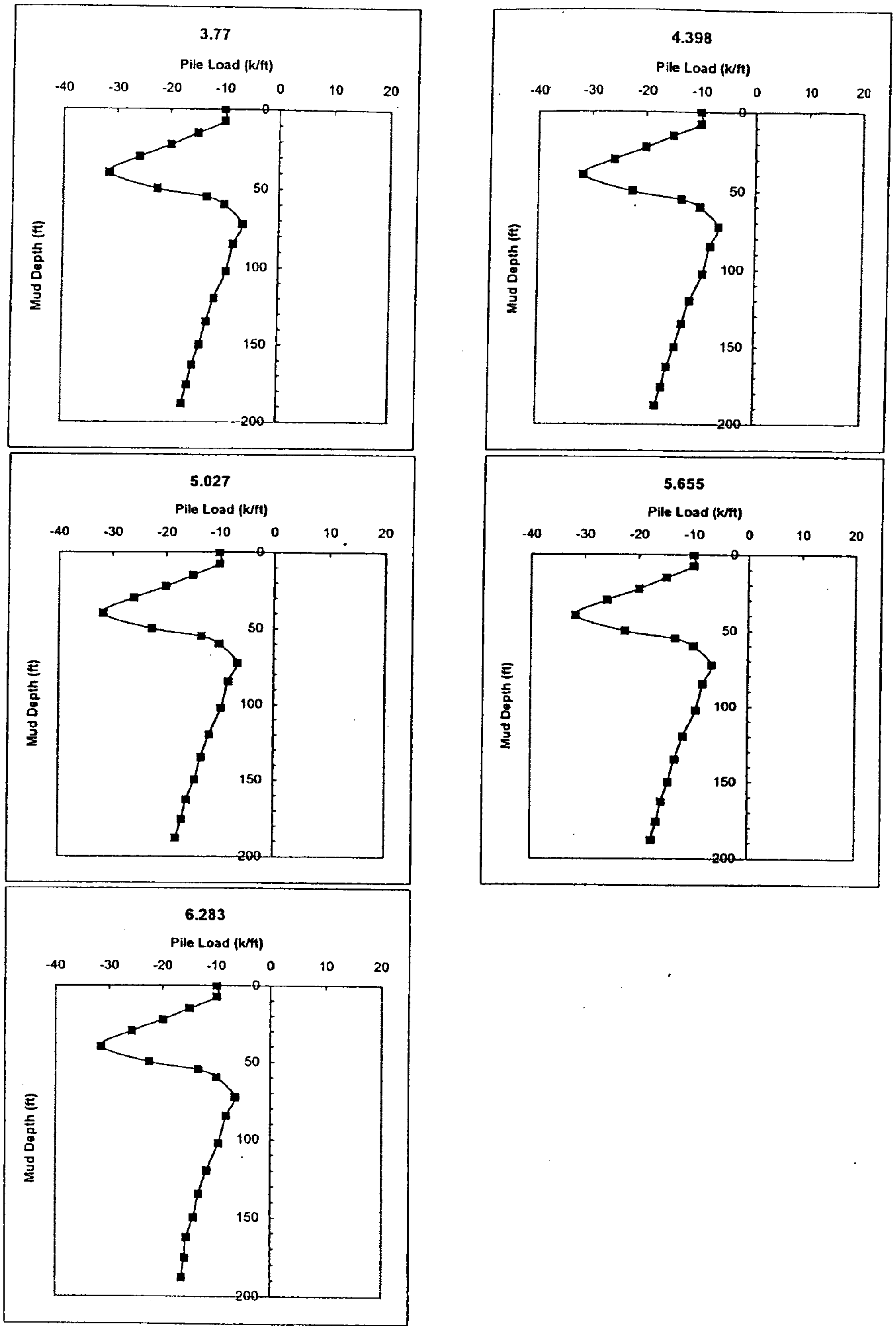


Fig. 5. Continued

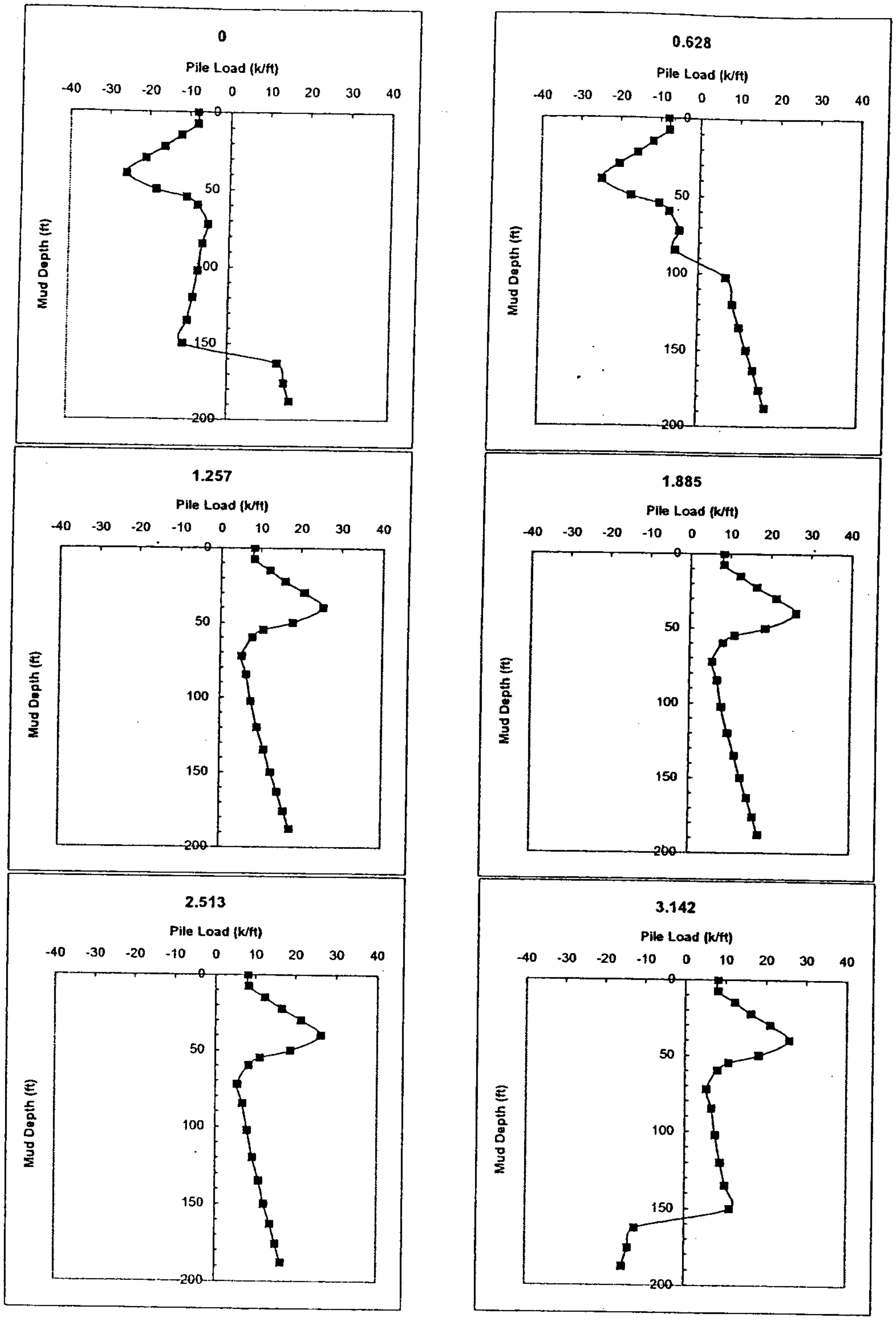


Fig. 6. Calculated sediment forces along 4 ft. diameter pile with 0% bottom slope for So. Pass 70B platform

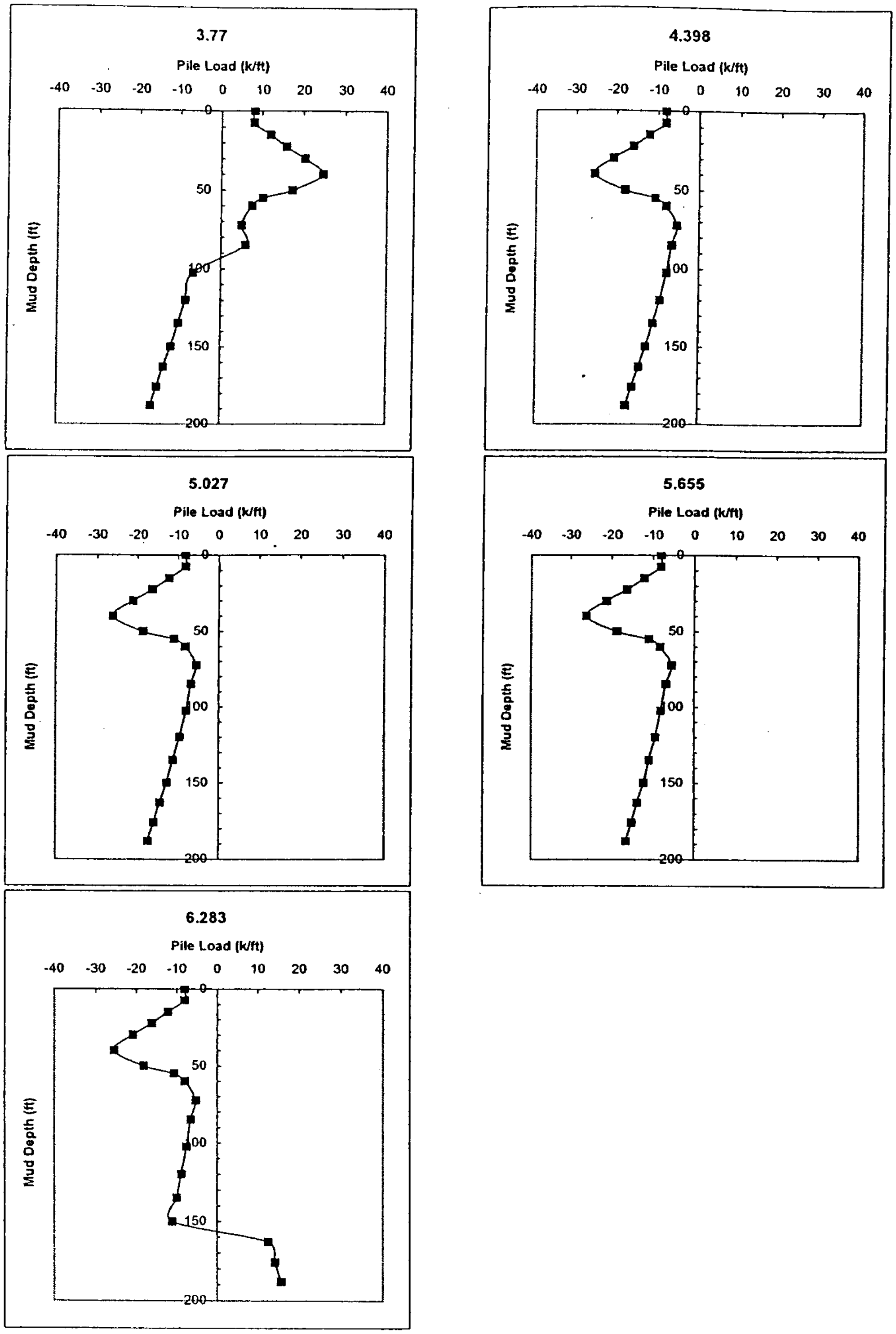


Fig. 6. Continued

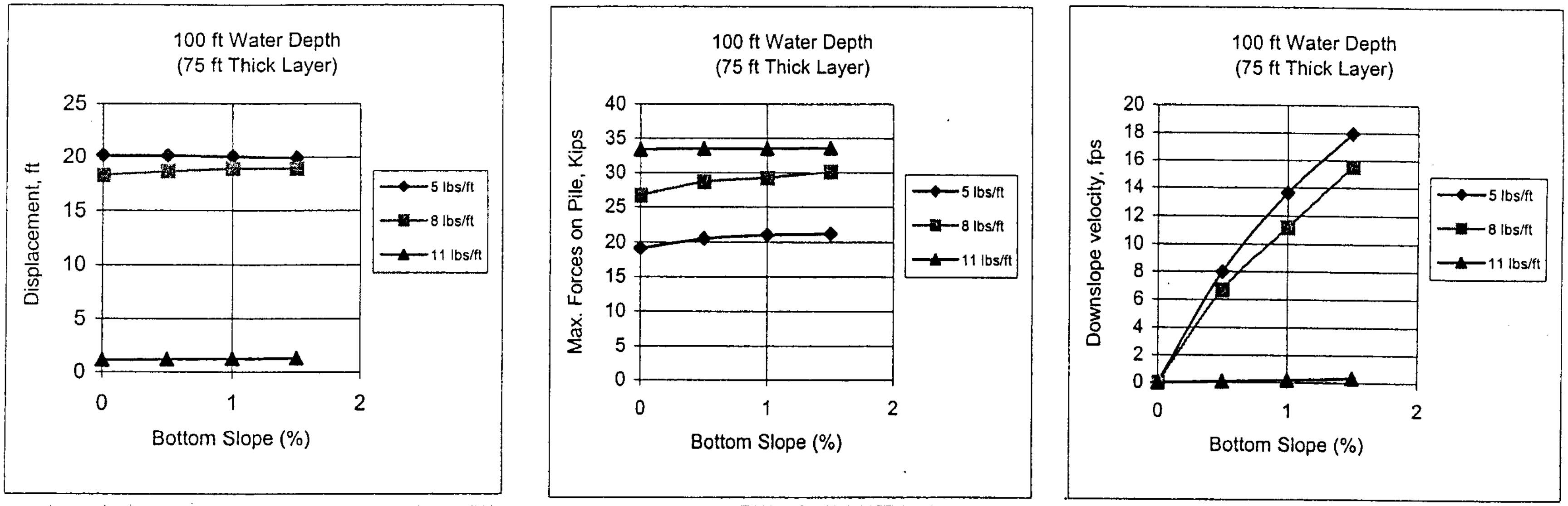


Fig. 7a. Sediment movements and pile forces for 100 ft. water depth and 75 ft. sediment thickness

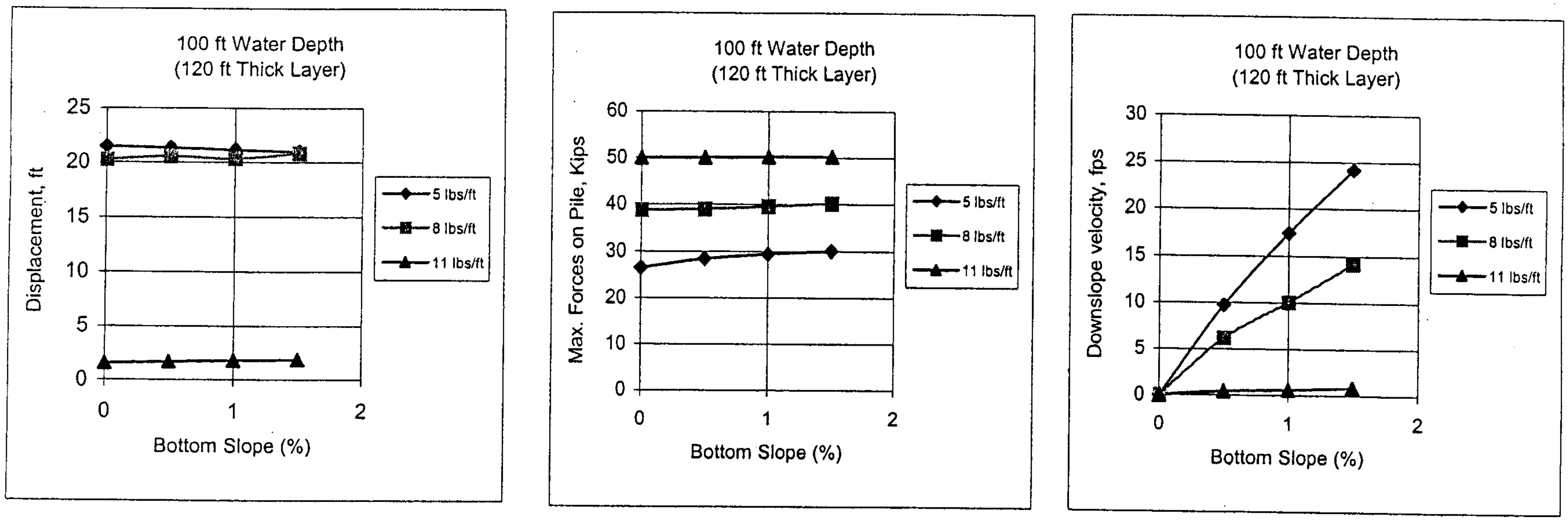
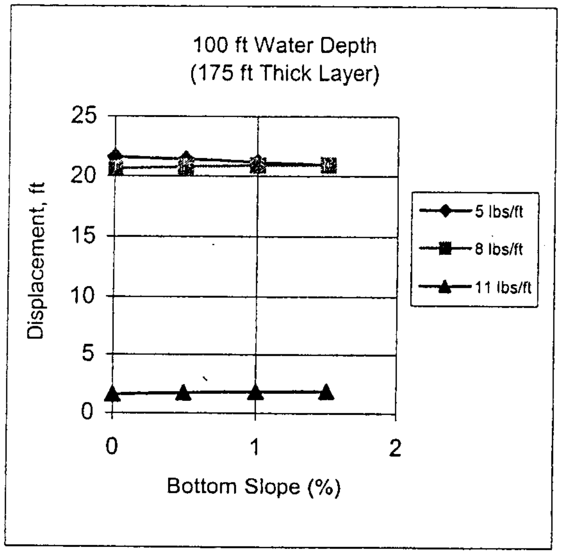
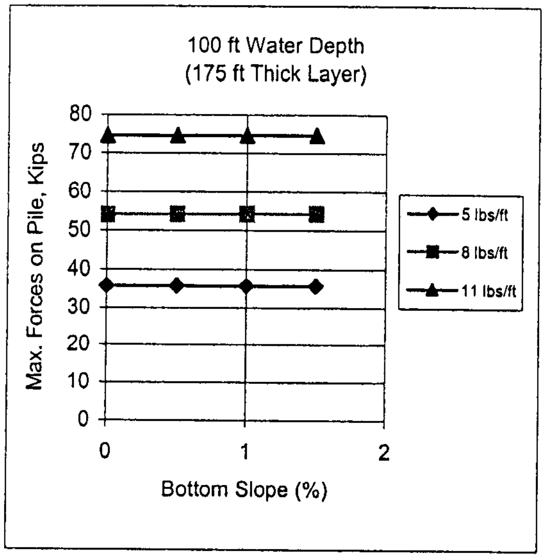


Fig. 7b. Sediment movements and pile forces for 100 ft. water depth and 120 ft. sediment thickness





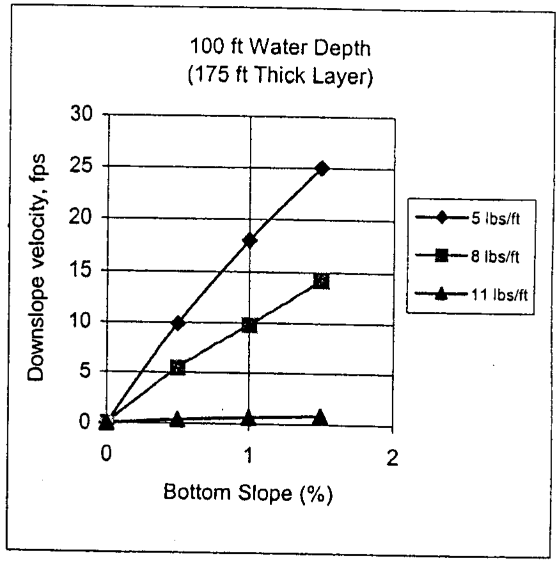


Fig. 7c. Sediment movements and pile forces for 100 ft. water depth and 175 ft. sediment thickness

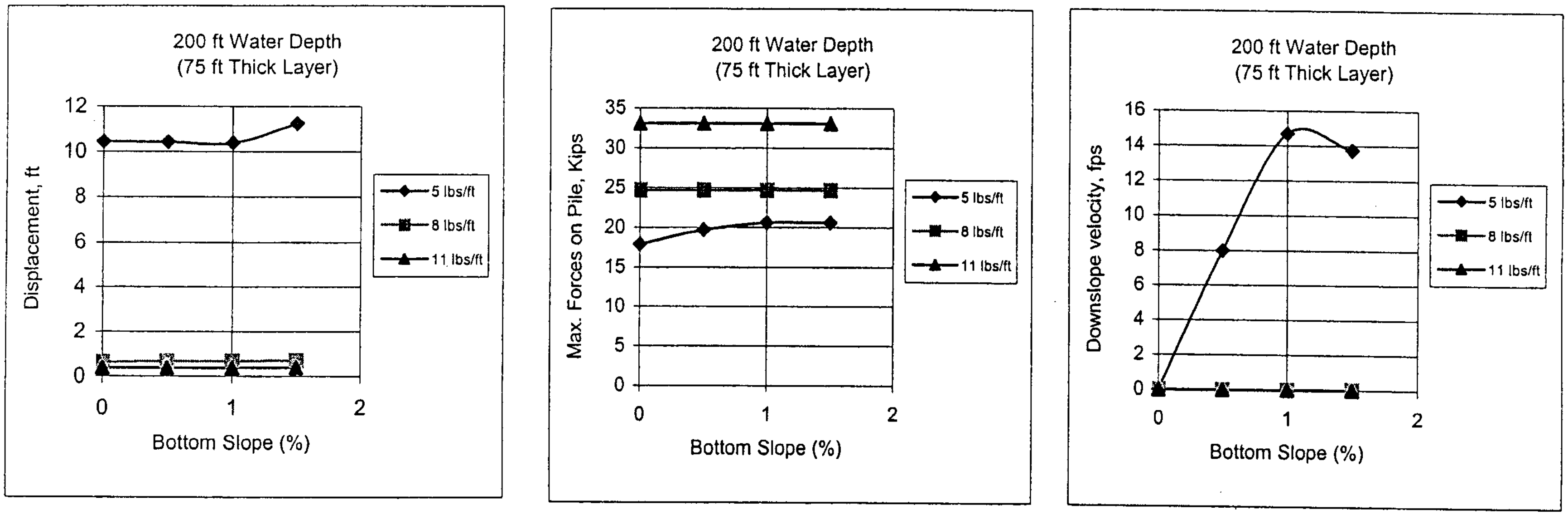
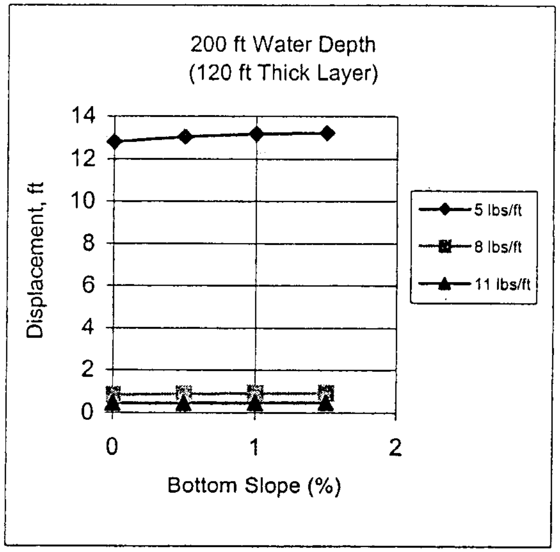
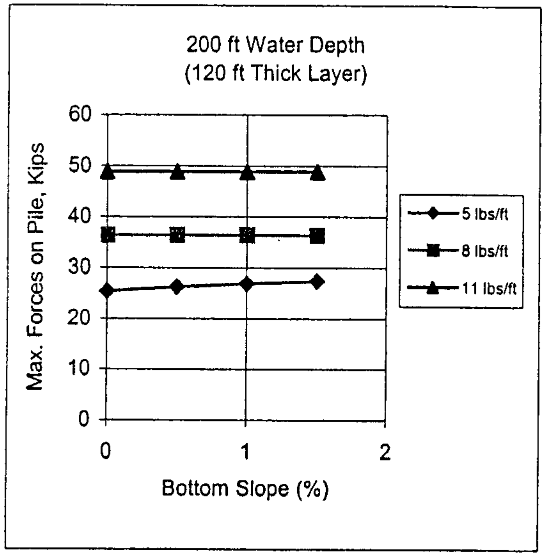


Fig. 8a. Sediment movements and pile forces for 200 ft. water depth and 75 ft. sediment thickness





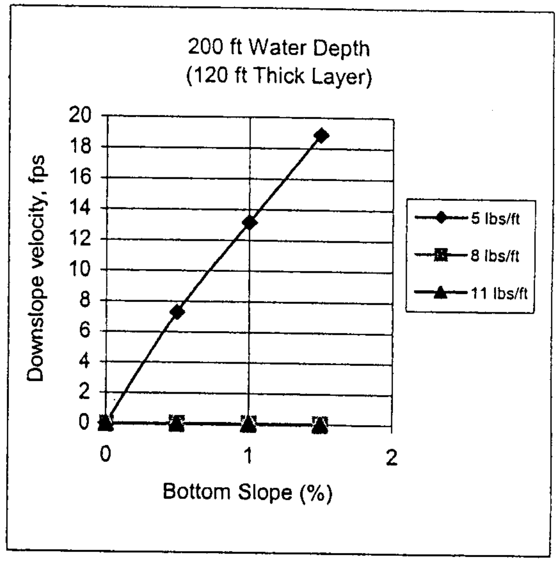


Fig. 8b. Sediment movements and pile forces for 200 ft. water depth and 120 ft. sediment thickness

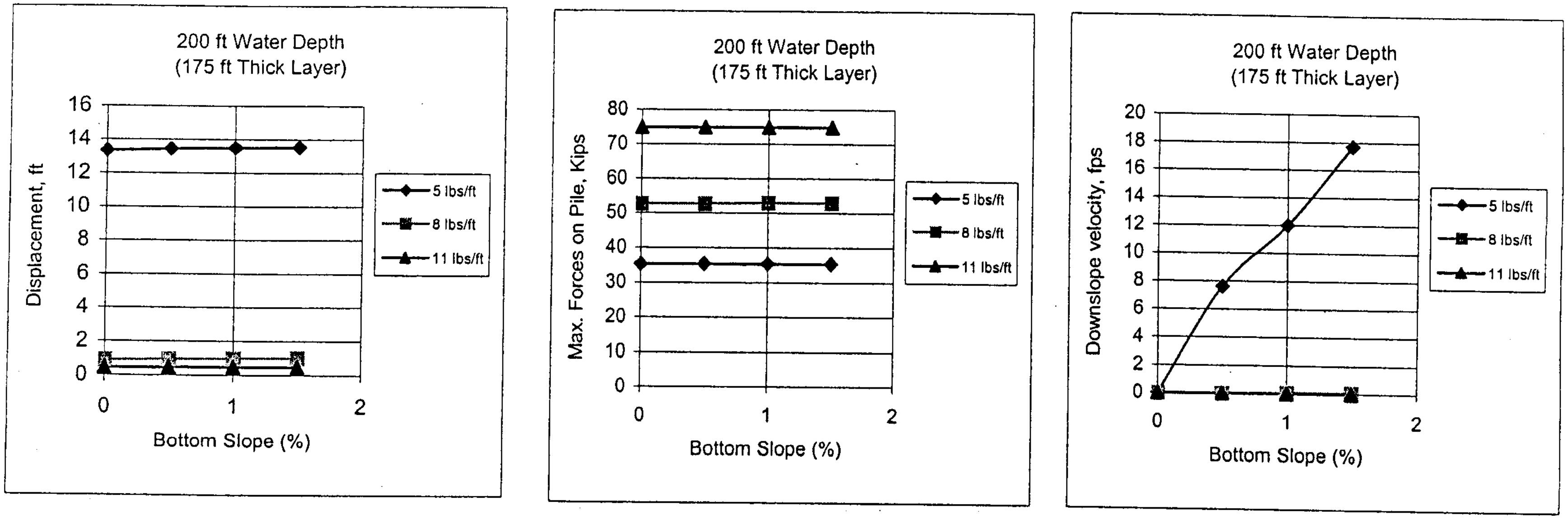


Fig. 8c. Sediment movements and pile forces for 200 ft. water depth and 175 ft. sediment thickness

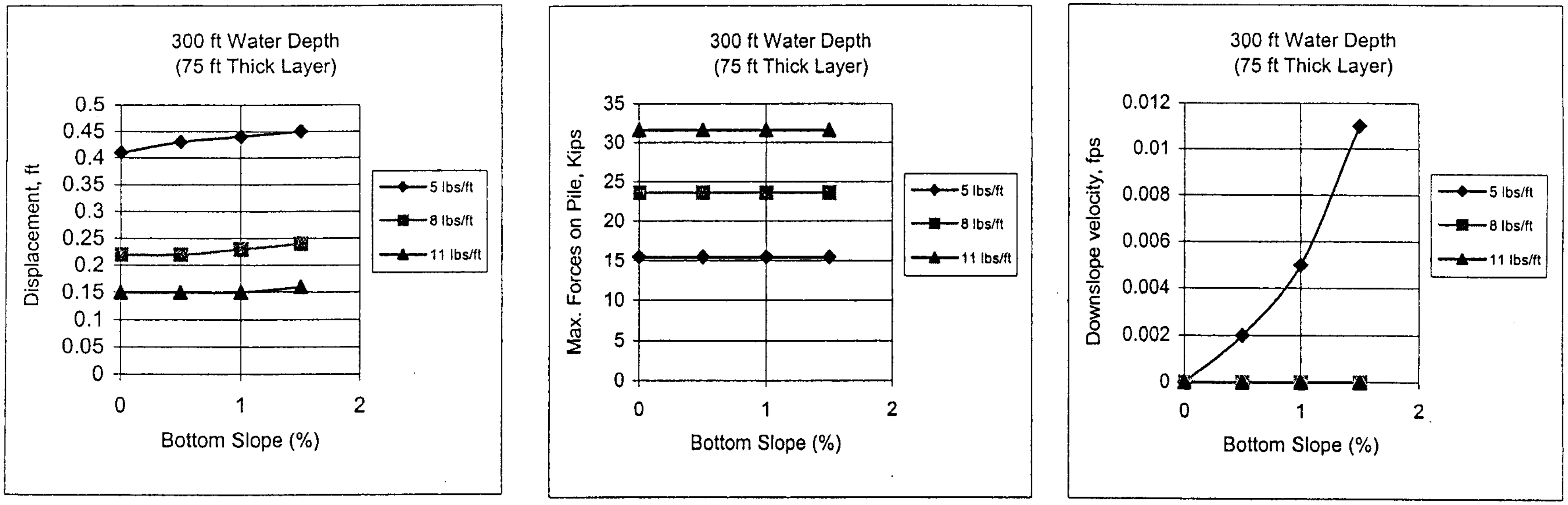


Fig. 9a. Sediment movements and pile forces for 300 ft. water depth and 75 ft. sediment thickness

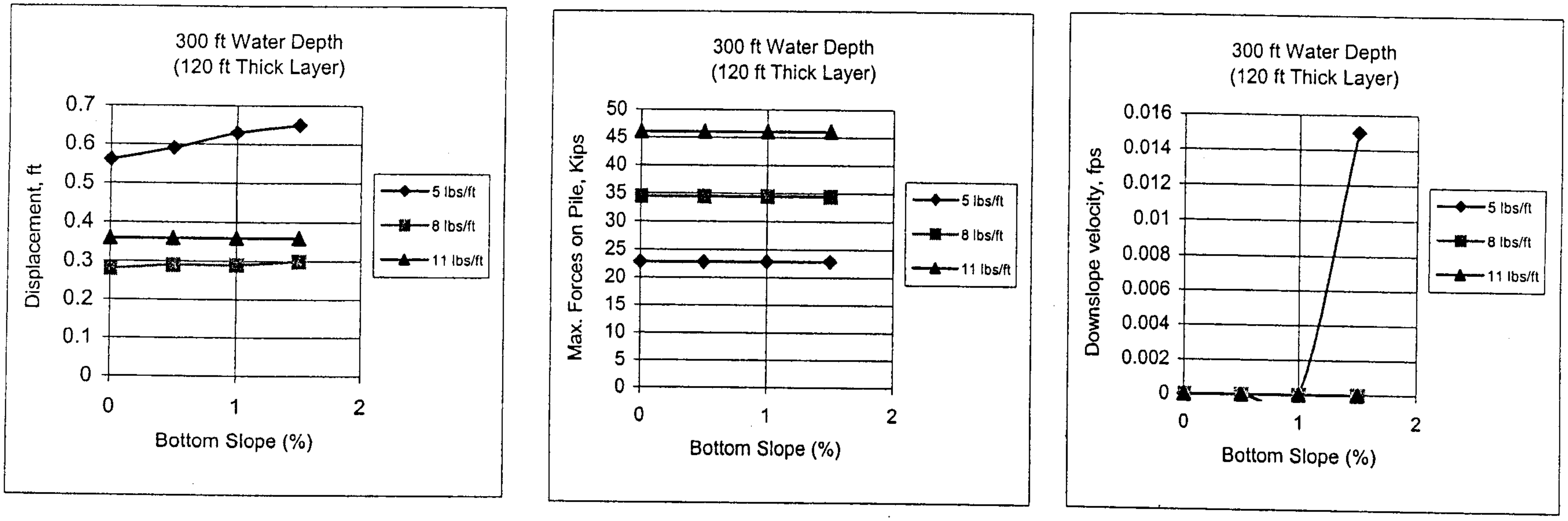


Fig. 9b. Sediment movements and pile forces for 300 ft. water depth and 120 ft. sediment thickness

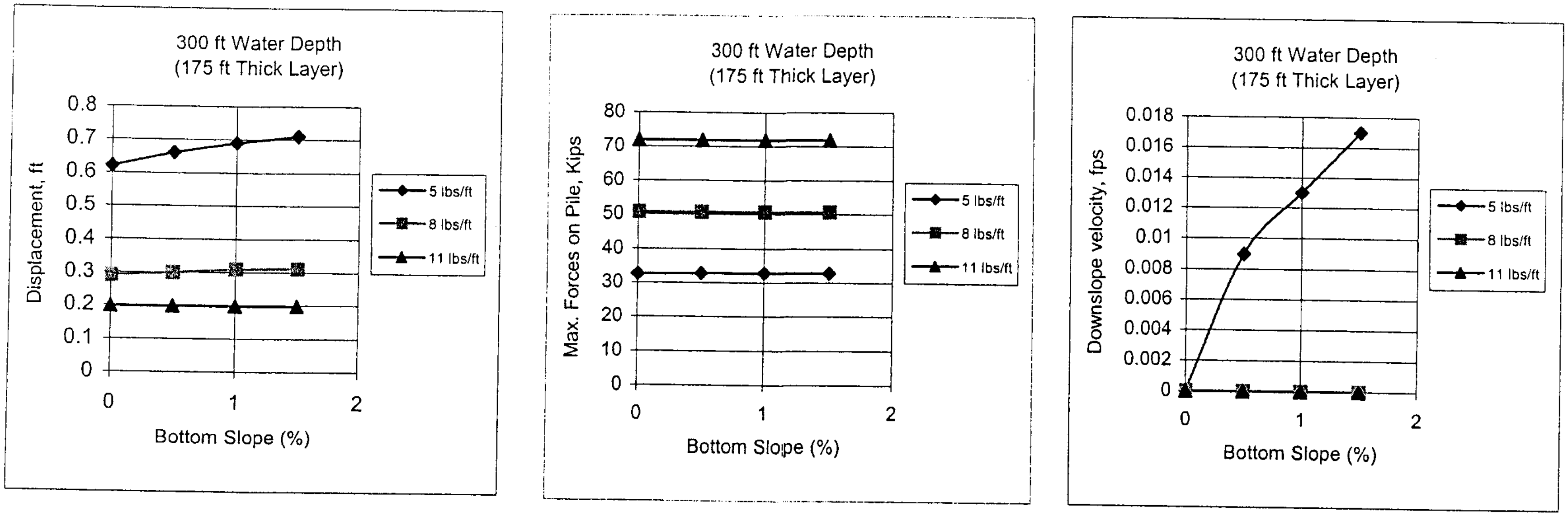
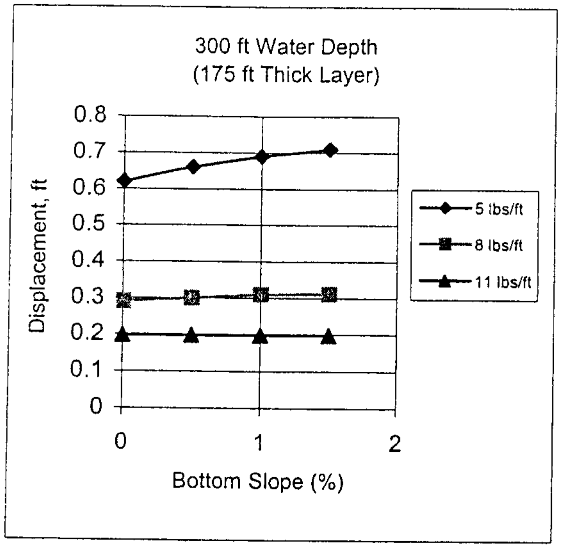
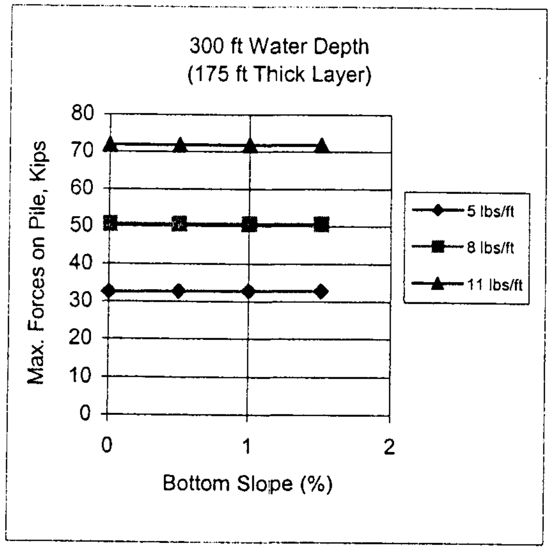


Fig. 9c. Sediment movements and pile forces for 300 ft. water depth and 175 ft. sediment thickness





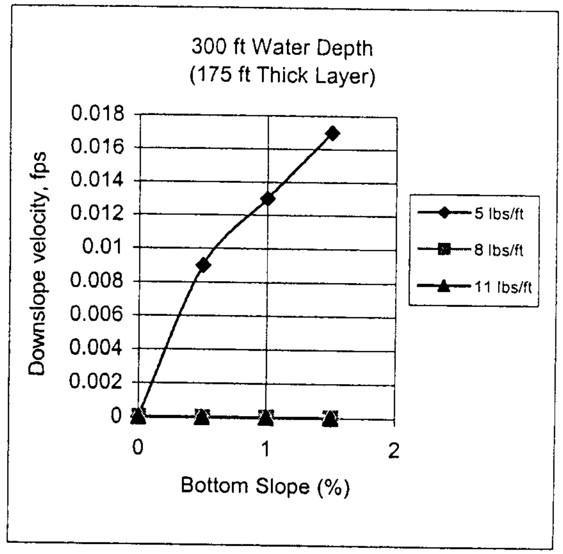


Fig. 9c. Sediment movements and pile forces for 300 ft. water depth and 175 ft. sediment thickness