UNITED STATES DEPARTMENT OF THE INTERIOR

GEOLOGICAL SURVEY

Geotechnical characterization and mass-movement potential of the United States North Atlantic Continental Slope and Rise

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Open-File Report 85-123

Prepared in cooperation with
MINERALS MANAGEMENT SERVICE
(formerly U.S. Bureau of Land Management)
under Memorandum of Understanding
AA851-MU0-18
and Interagency Agreements
AA851-IA1-17 and AA851-IA2-26

1. Woods Hole, MA 02543

1984

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INTRODUCTION

On a regional basis the Georges Bank Continental Slope (fig. 1) has experienced widespread mass movement: evidence of past instabilities is present throughout much of the area and is abundant in specific locales. Aaron and others (1980) estimate, based on high-resolution seismic-reflection data, that up to 37 percent of the Georges Bank Slope may have been affected by this type of slope failure. In addition to being widespread, the variety of features possibly derived from mass-movement processes is impressive. These features, identified on sidescan-sonar images by O'Leary and Twichell (1981), include rotational and translational slide scarps, toreva blocks, debris flows, rubble fields, allochthonous blocks, wavelike distortion of beds, and general small-scale surface irregularities.

The geologic setting that has favored the mass movement has been extensively investigated and its general character is well known. The area is dominated by the presence of numerous submarine canyon systems (Emery and Uchupi, 1972) which, in themselves, are testimony to large-scale slope degradation. Regional slope gradients can be 7^{o}-10^{o} (Keller and others, 1979; Aaron and others, 1980); locally slopes exceed 20^{o} (O'Leary and Twichell, 1981). Pleistocene/Holocene sediments have been most extensively affected by mass wasting; along parts of the upper slope these sediments are more than 300 m thick (Valentine and others, 1980). Analyses of the surficial sediment (piston core samples) by Doyle and others (1979) showed that silts and clays dominate, although a wide variety of grain sizes exist, including sand and clasts of probable glacial origin. This is supported by the work of Keller and others (1979), whose data indicate that the surficial sediments are composed, on average, of more than 80 percent fine-grained sediment. Further, Hathaway and others (1979) found thick sections of fine sediments in several locales during drilling operations in the area.

Sedimentation rates have probably varied considerably over recent geologic time. It has been pointed out (Emery and Uchupi, 1972) that during lowstands of the sea, depositional rates may have been enhanced both by greater proximity to drainage systems and by greater runoff from glacial melt. Thus, although the Pleistocene sedimentation rate for the area may have averaged about 20 cm/1,000 yr (Emery and Uchupi, 1972) and may currently be at a similar rate (based on a determination by Doyle and others, 1979), much higher rates could have existed at times of lowstand, particularly in local areas on the upper slope.

Also a part of the geologic setting, especially as it pertains to slope failure and related phenomena, are the forces acting on or within the sediment section that affect stability. These forces include those from external sources, such as tectonic and oceanographic phenomena, and those generated internally, such as from gas. Earthquakes, as one manifestation of tectonic activity, are infrequent in the vicinity of the Georges Bank area of the Continental Slope compared to some of the renowned seismically active areas of the world. However, the area is not aseismic. Since the emplacement of seismometer networks in the northeastern United States, numerous shocks of $m_b>2$ have been recorded off the southern New England coast (see, for example, fig. 2 in Yang and Aggarwal, 1981). In addition, the historical record (summarized by Sykes, 1978) shows that two major quakes occurred on the Massachusetts coast near Cape Ann in 1638 and 1755, and a third occurred on

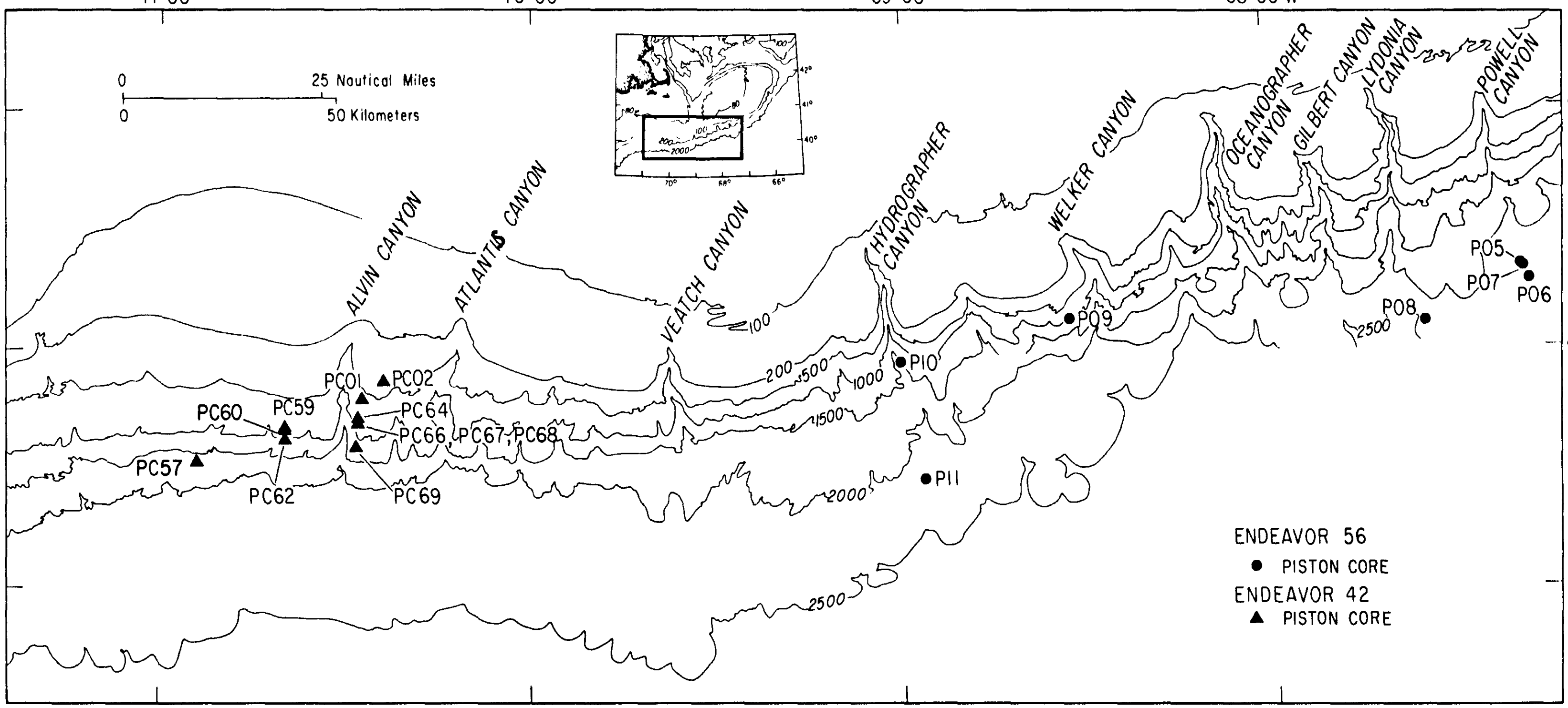


Figure 1. Piston-core station locations.

the Grand Banks in 1929. It has been inferred that all of these had magnitudes (m) greater than 6 and the last was probably over 7. Sykes also infers that the New England seamount chain, which trends to the east off the southeastern portion of Georges Bank, may be seismically active at times.

Oceanographic forces include currents and waves. Several current measurements have been made at the outer shelf (Butman and others, 1982), and in some of the numerous submarine canyons (Keller and Shepard, 1978), but little data is available for the slope proper. Keller and Shepard (1978) made a single measurement near Alvin Canyon (see fig. 1) and reported a peak current velocity of over 70 cm/s. At Continental Slope depths (200-2,000 m), wind waves, even from major storms, do not exert a significant force on the sea floor. It is possible, however, that internal waves may influence the bottom sediments (Southard and Cacchione, 1972). No field data are available on internal wave forces.

Forces which may act from within the section to affect slope stability are primarily those which are induced by excess pore pressures. Rapid deposition, shallow gas, and artesian systems may create such pressures. Within the area of investigation, however, average rates of deposition (20 cm/1,000 yr) are not high enough to elevate pore pressure (i.e., trap pore water) on a regional basis, nor does interstitial gas seem to be widespread, although it is present locally (D. O'Leary, oral commun., 1983). Finally, the absence of fresh or brackish water below the sea floor at the shelf edge (fig. 1c in Hathaway and others, 1979), suggests an absence of a fresh water artesian system.

Given this geologic setting, along with the abundant and diverse evidence of mass movements and related phenomena, it was our purpose to (1) establish the current stability of the slope and the geologic conditions which may promote further slope failures, and (2) establish a framework to judge the more likely causes of mass movement in the past.

Because knowledge of the geotechnical properties of sediments is one of the chief prerequisites for understanding the various aspects of mass movement, these properties formed the primary data base for the investigation. In addition, the geotechnical data serve to describe the engineering characteristics of the sediment. Accordingly, such a description is also included in this report.

METHODS

Shipboard

Sampling/analytical

The piston cores used for the geotechnical aspect of the study were collected aboard the R/V ENDEAVOR in August 1979 and October 1980. The coring system was modified from the conventional design in order to obtain cores with minimal mechanical disturbance because many geotechnical properties, especially those related to strength, are vulnerable to the effects of disturbance. Details of the modifications are presented in Booth and others (1981a). Eighteen cores up to approximately 8 m long were recovered during the two cruises in the area. Station data are shown in table I and locations are shown in figure 1.

Table I

Station Data

Piston Cores

Core	Latitude (N.)	Longitude (W.)	Water depth (m)	Core length (m)
PC01	39°54.12'	70°27.52'	697	5.20
PC02	39°56.47'	70°23.55'	534	3.13
PC57	39°46.24'	70°53.71'	1,593	1.00
PC59	39°50.49'	70°39.64'	851	1.71
PC60	39°50.21'	70°39.65'	856	1.20
PC62	39°49.35'	70°39.70'	937	0.96
PC64	39°51.89'	70°27.85'	813	3.00
PC66	39°51.35'	70°27.88'	835	0.65
PC67	39°51.37'	70°27.90'	837	1.40
PC68	39°51.38'	70°27.87'	834	2.45
PC69	39°48.06'	70°28.06'	1,195	0.40
P05	40°10.47'	67°19.34'	2,190	7.92
P06	40°08.52'	67°17.37'	2,375	4.71
P07	40°10.08'	67°18.57'	2,235	3.90
P08	40°03.26'	67°34.95'	2,420	4.22
P09	40°04.39'	68°32.50'	800	4.29
P10	39°59.05'	69°01.39'	1,200	3.32
P11	39°44.35'	68°56.26'	2,225	7.57

As was the case during sampling operations, avoiding disturbance was the prime consideration during core processing and storage. Once on board, the cores were cut into 1.5-m-long sections by using a tube cutter to sever the liner and a wire saw to part the sediment. Up to three subsections were also cut for later triaxial and consolidation testing. All subsections, which were generally taken from near the bottom portions of the cores, were X-radiographed in order to judge the condition of the sample; only apparently undisturbed samples were retained for later testing. Finally, the subsections were capped, taped, sealed with wax, and stored upright at $4^{o}C$ in specially fabricated boxes padded with foam rubber. The remaining core sections were split lengthwise: one part of each section served as the archive half, the other as a "working" half, which was taken to the shipboard laboratory for description, strength testing, and subsampling.

After a cursory description, "undisturbed" undrained shear strength was measured with a four-bladed, 12.7 mm-square laboratory vane at intervals of 0.50 m and at lithologic changes. Obvious sand layers, which are cohesionless and, therefore, inappropriate for this type of test, were avoided. The blade was inserted normal to the long direction of the core and buried at least 20 mm into the section to be tested. In order to guard against sample drainage during the application of torque, a rotation rate of 0.0262 radians/s (90^{o}/min) was used. This relatively high rate of speed also maximizes measured shear strength and, because of possible detrimental effects of ship motion and vibrations, allows a test to be completed quickly to minimize disturbance. It is assumed on the basis of previous experience (e.g., Booth, 1979) that strength reduction due to the release of in situ stresses and mechanical disturbance may generally be kept to less than 30 percent if care is taken. Remolded strength (strength of thoroughly kneaded sample) was also determined with the vane apparatus. The precision of the vane shear measurements is ±0.30 kiloPascals (kPa).

Subsamples for index property testing were taken at the points of strength measurements, placed in plastic bags, and sealed in cans for later laboratory testing. These subsamples, and those samples taken for triaxial and consolidation testing, were transported to the laboratory in a refrigerated ($4^{o}C$) van.

Laboratory

Index properties

The suite of geotechnical index property tests (water content, liquid and plastic limits, and grain-specific gravity) was conducted according to procedures recommended by ASTM (1982), with two exceptions. Grain-specific gravity was measured with an air comparison pycnometer and all water content data were corrected for salt content. Precisions were: water content ±3 percent (relative); liquid limit, ±3 percent (absolute); plastic limit, ±2 percent (absolute), and grain-specific gravity, ±1 percent (relative). Derived from this basic data set were plasticity index, liquidity index, bulk density, and porosity.

In addition, results of textural and mineralogical analyses were made available by L. J. Poppe (unpublished data, 1980; 1982).

Triaxial Testing

Consolidated undrained triaxial tests with pore pressure measurements were conducted in accordance with procedures given by Bishop and Henkel (1957). In sum, for each set of tests three or four (depending on availability of suitable material) specimens were cut from the prime core sample, trimmed to a right cylinder (50 mm I.D. x 100 mm), and placed in triaxial cells. After the specimens were saturated they were consolidated to 0.75 (if a fourth specimen were available), 1.0, 2.0, and 4.0 times the assumed in situ overburden pressure. When consolidation was complete, the specimens were sheared; generally at a rate of 0.015 mm/min. Data from all phases of the tests, including axial force, change in specimen length, pore pressure, change in volume, and time, were logged by an automatic data acquisition system. From this basic data set the angle of internal friction with respect to effective stress (ϕ'), cohesion with respect to effective stress (c'), percent strain at failure, and undrained strength to effective overburden stress ratio (S_u/σ'_{vo}) were determined.

Consolidation testing

The constant rate of strain (CRS) method was used for consolidation testing. In this method a 63.5-m I.D. x 25-mm-thick sediment disc is confined in a ring (one-dimensional test) and shortened (consolidated) at a constant rate. The increase in stress is monitored along with the change in length and pore pressure. The strain rate must be slow enough so that effective stress remains equal, or nearly so, to applied stress. Drainage is permitted. Wissa and others (1971) give details of procedure. Derived from the test are preconsolidation pressure (P_c), which is the maximum past pressure experienced by a sample (Casagrande (1936) method used to determine P_c), the coefficient of consolidation (C_v), the compression index (C_c), and the coefficient of permeability (k).

GEOTECHNICAL CHARACTERIZATION

Cohesion

The variety of sea-floor conditions represented by the cores is reflected by the range of nearly two orders of magnitude in the vane shear strength (cohesion) values (table II). Thus, the overall mean value of 11.0 kPa is not necessarily representative of specific locales. Further, it is higher than the range of mean cohension values reported from other studies (Keller and others, 1979; McGregor and others, 1979; Lambert and others, 1981; and Booth and others, 1983a) in this and proximal slope and rise areas, which may reflect the greater core length (which can result in the recovery of stiffer material) and the intentional sampling of erosional areas. With the exception of core PC57 (S_u~85 kPa), which was recovered from a slide scarp, the surface sediment is classified as having a very soft consistency.

No significant trend is judged to be present between the cohesion data and water depth: Correlation analysis of 13 core-averaged cohesion values with depth yielded r values of -0.35, which is statistically insignificant. However, correlation of cohesion and longitude (i.e., slope-parallel trend) is significant: r=+0.58. Accordingly, 34 percent of the variability in cohesion is apparently explained by this relationship. A tendency for shear strength to increase in a westward direction is thus indicated. Cores which were taken from erosional surfaces or mass-movement scarps were eliminated from the analysis so that only the Holocene/Pleistocene section of apparently continuous accumulation was considered in the analysis.

Downcore cohesion profiles show, in general, the expected increase in shearing resistance with depth below the sea floor (figs. 2a-m). Cohesion values of cores with a "PC" prefix represent corrections of previously released values (shear strength data in Booth and others, 1981a). In some cores, however, relatively stiff sediment was encountered within 0.5 m of the top of the core. Through erosion, mass movement, or other degradational processes, the shearing resistance at the top of these cores (PC01, PC57, PC59, PC68) is high for surface material. Each of these cores was collected in the vicinity of Alvin Canyon. In addition, cores P05 and P07 (fig. 2g, i) show relatively high strengths near the surface, but these high values do not continue downcore: significant reductions in strength are found beneath the anomalous surface values. Note that in figure 2, only cores which have at least two vane measurements are plotted.

As a final comment on the cohesion data, it must be pointed out that our attempts to characterize trends in the present, continuous depositional surface are, to a degree, biased - despite eliminating data from apparent erosional surfaces. Similarly, the mean cohesion value is, to a degree, biased. Topography, bottom-sediment type, and, perhaps less conspicuously, corer design, are factors in introducing the bias. A full discussion of this subject is beyond the scope of this report.

Sensitivity

Sensitivity is a measure of the amount of strength loss in a sediment due to remolding. As shown in table II, these sediments have an average sensitivity of 4.8, and range from 1.4 to 15. According to the classification developed by Rosenqvist (1953), these sediments, on average, would be considered very sensitive, and their sensitivities range from slightly sensitive to slightly quick. In other terms, these sediments could lose up to 90 percent of their strength through dynamic loading (e.g., earthquakes), strain softening, or other means, and the average strength loss would be over 80 percent. The highest sensitivity value was associated with PC57, which was also the core with the highest measured cohesion.

As was the case for analysis of spatial trends in the cohesion data, analysis of trends in sensitivity data was done after eliminating cores from erosional or otherwise exhumed surfaces. No significant trends were identified in either the downslope or across-slope directions.

Downcore sensitivity profiles are shown in figures 2a-m. These profiles are irregular and there are no apparent trends. An immediate increase in sensitivity downcore is common, however, which may indicate greater disturbance at the core tops or that sensitivity values within the upper half meter or so is partially controlled by small changes in overburden. Bioturbation, which tends to remold the sediment and hence decrease sensitivity, may also be a factor.

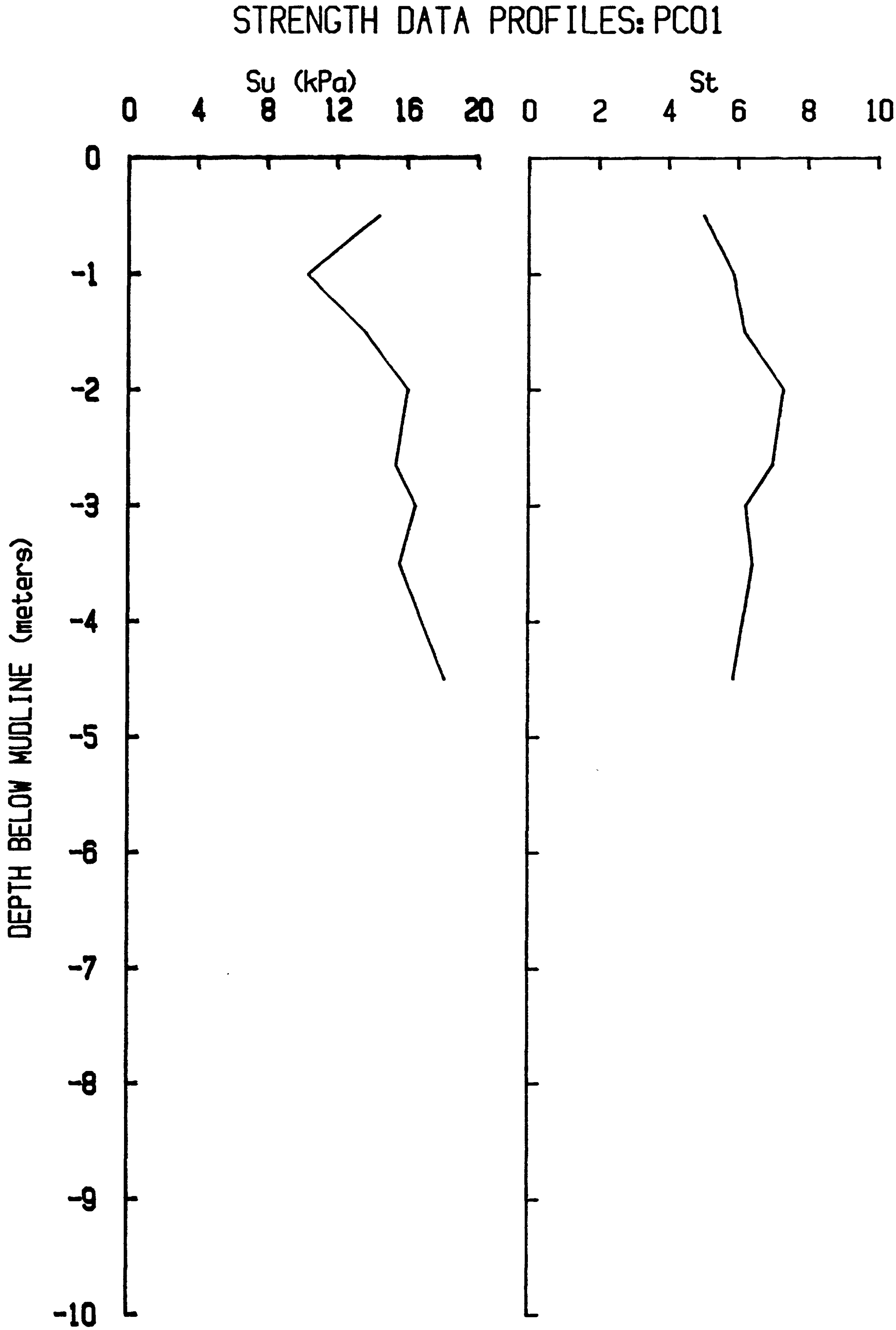


Figure 2a. Cohesion (S_u) and sensitivity (S_t) vs. depth in core PC01.

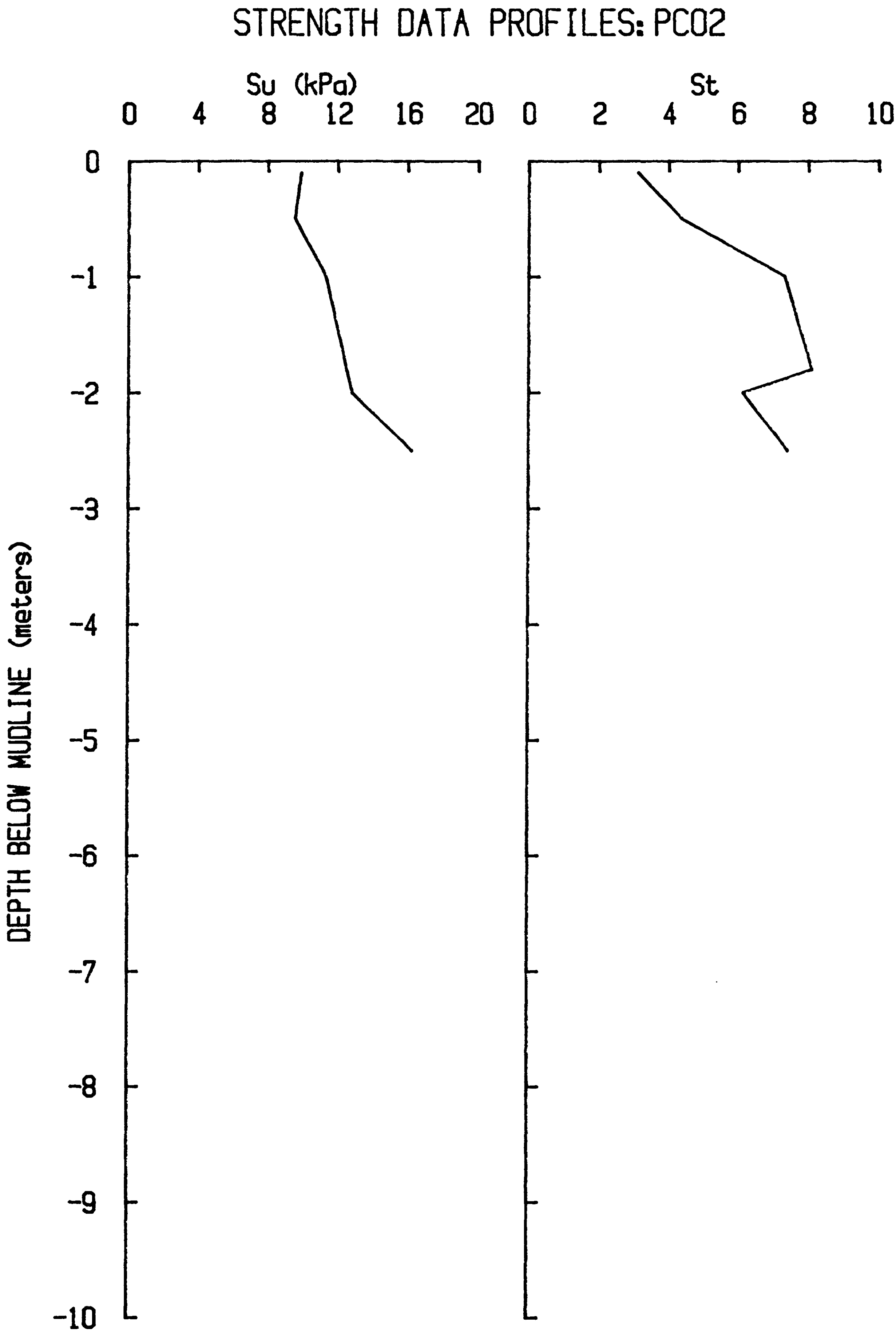


Figure 2b. Cohesion (S_u) and sensitivity (S_t) vs. depth in core PC02.

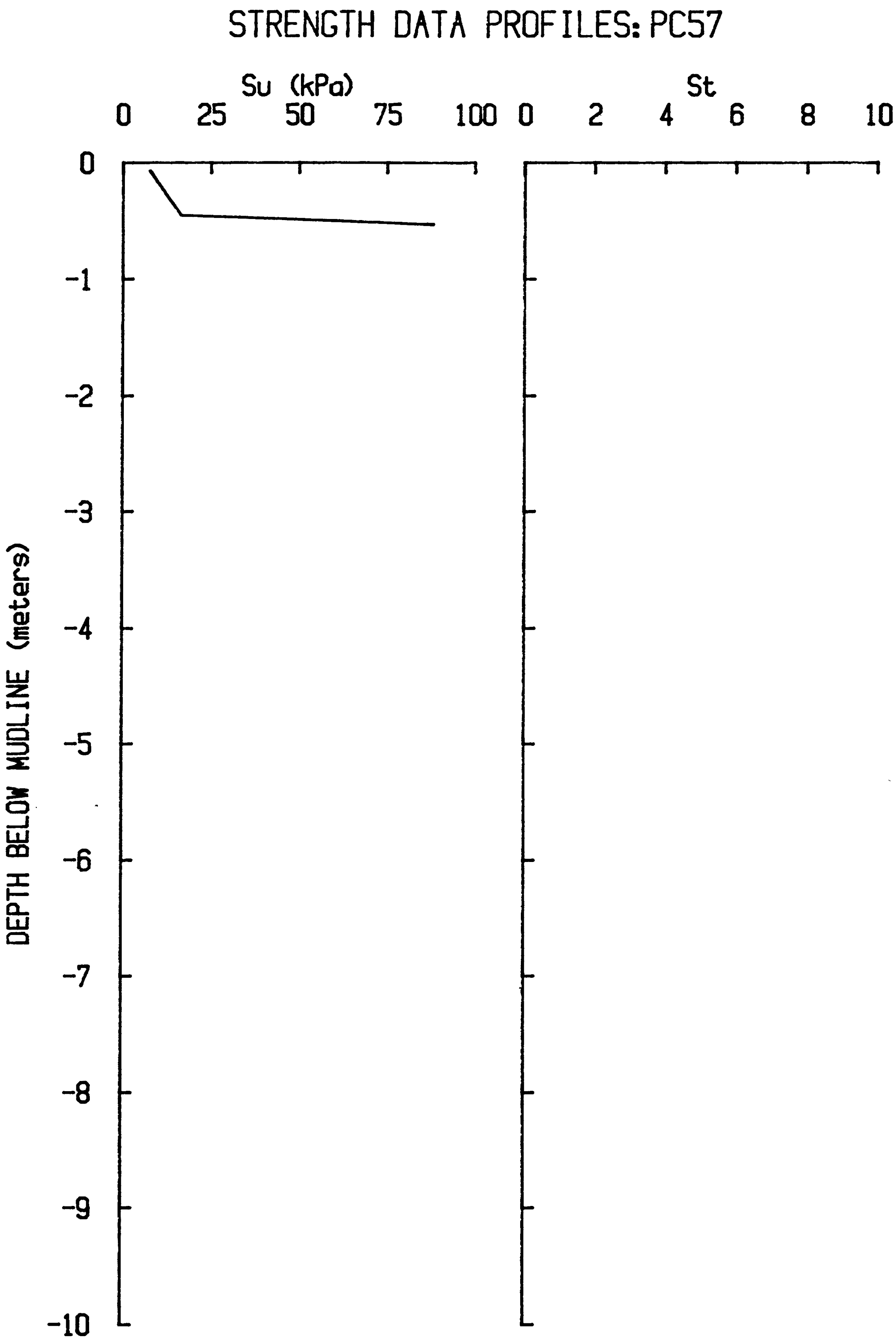


Figure 2c. Cohesion (S_u) and sensitivity (S_t) vs. depth in core PC57.

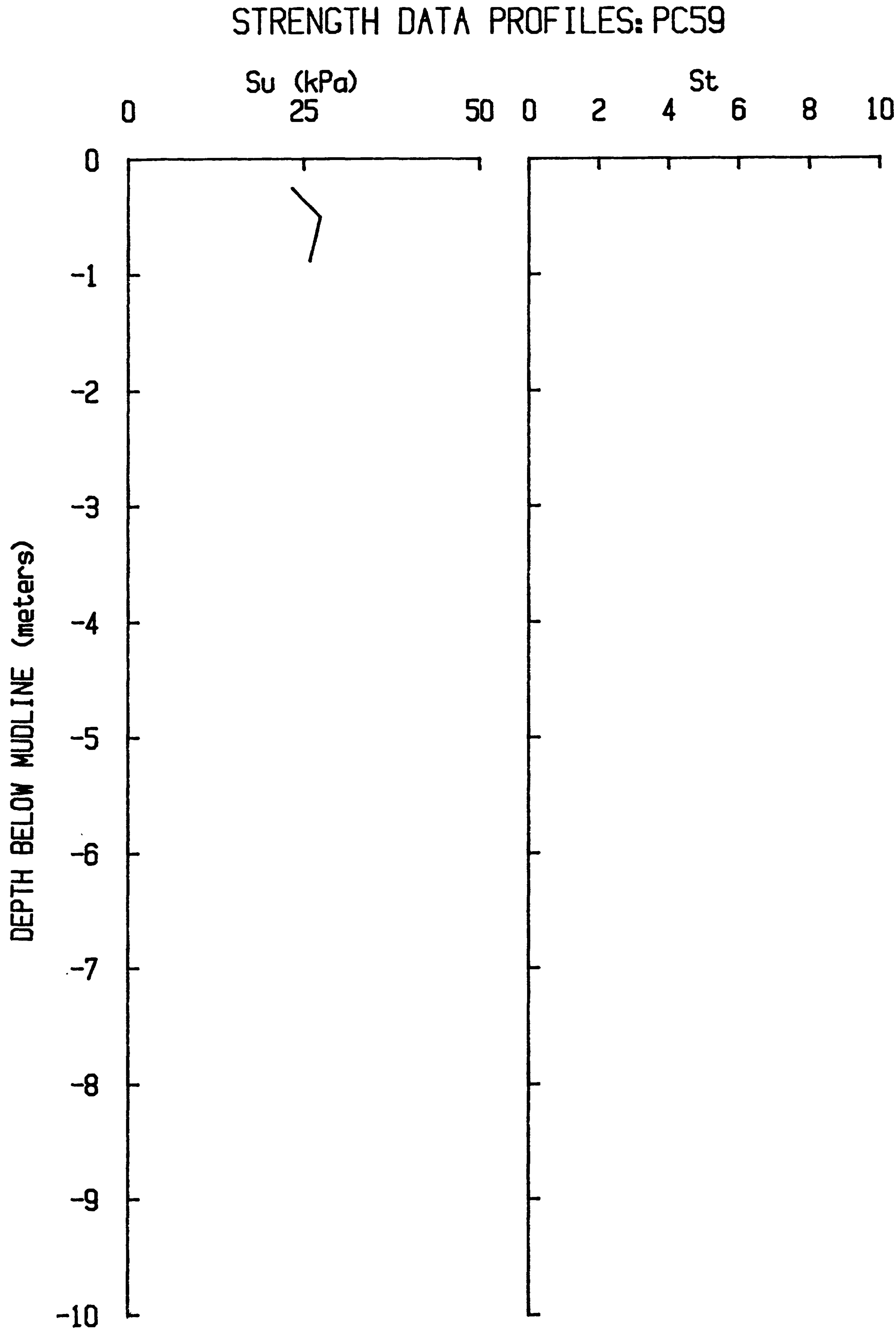


Figure 2d. Cohesion (S_u) and sensitivity (S_t) vs. depth in core PC59.

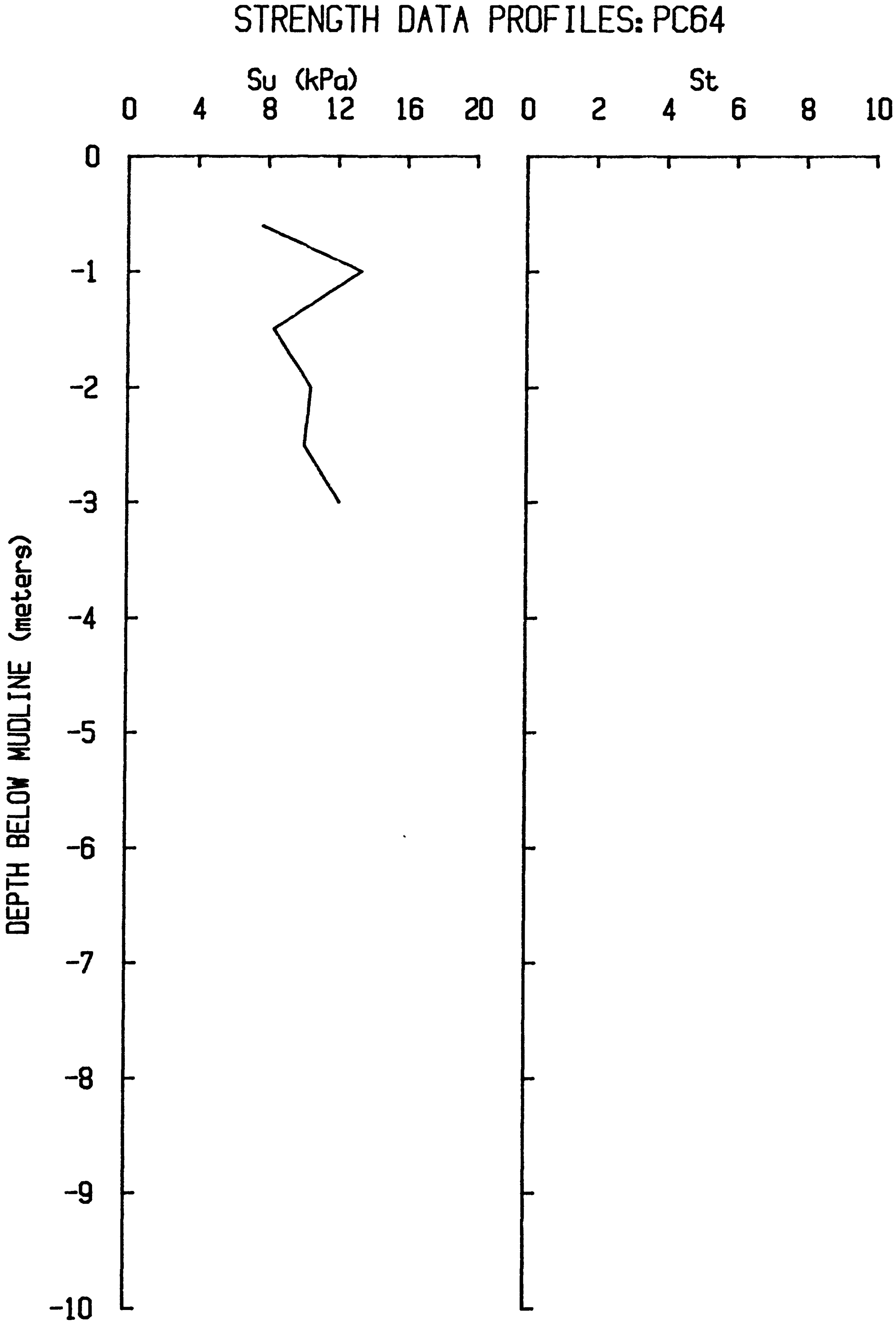


Figure 2e. Cohesion (S_u) and sensitivity (S_t) vs. depth in core PC64.

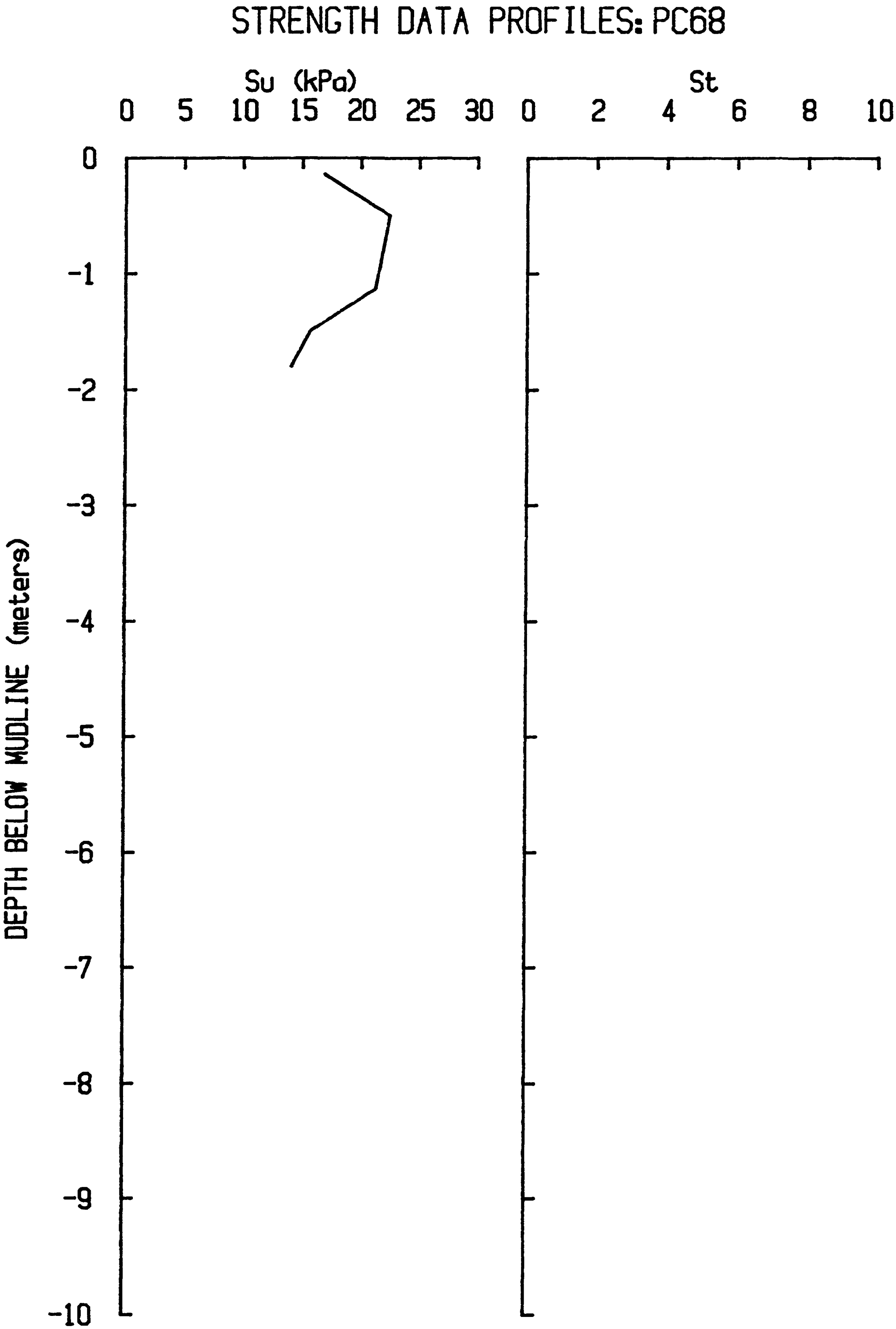


Figure 2f. Cohesion (S_u) and sensitivity (S_t) vs. depth in core PC68.

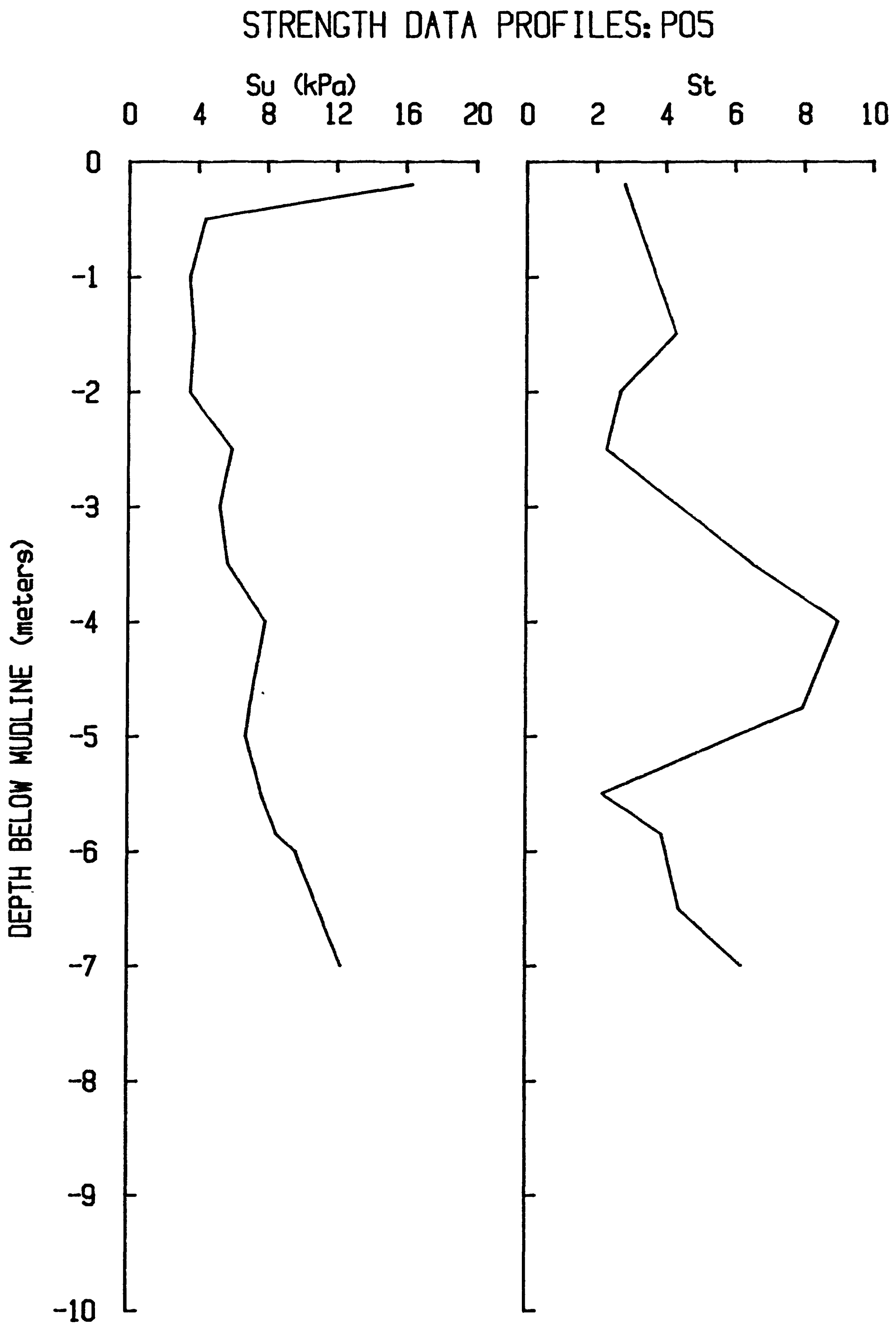


Figure 2g. Cohesion (S_u) and sensitivity (S_t) vs. depth in core P05.

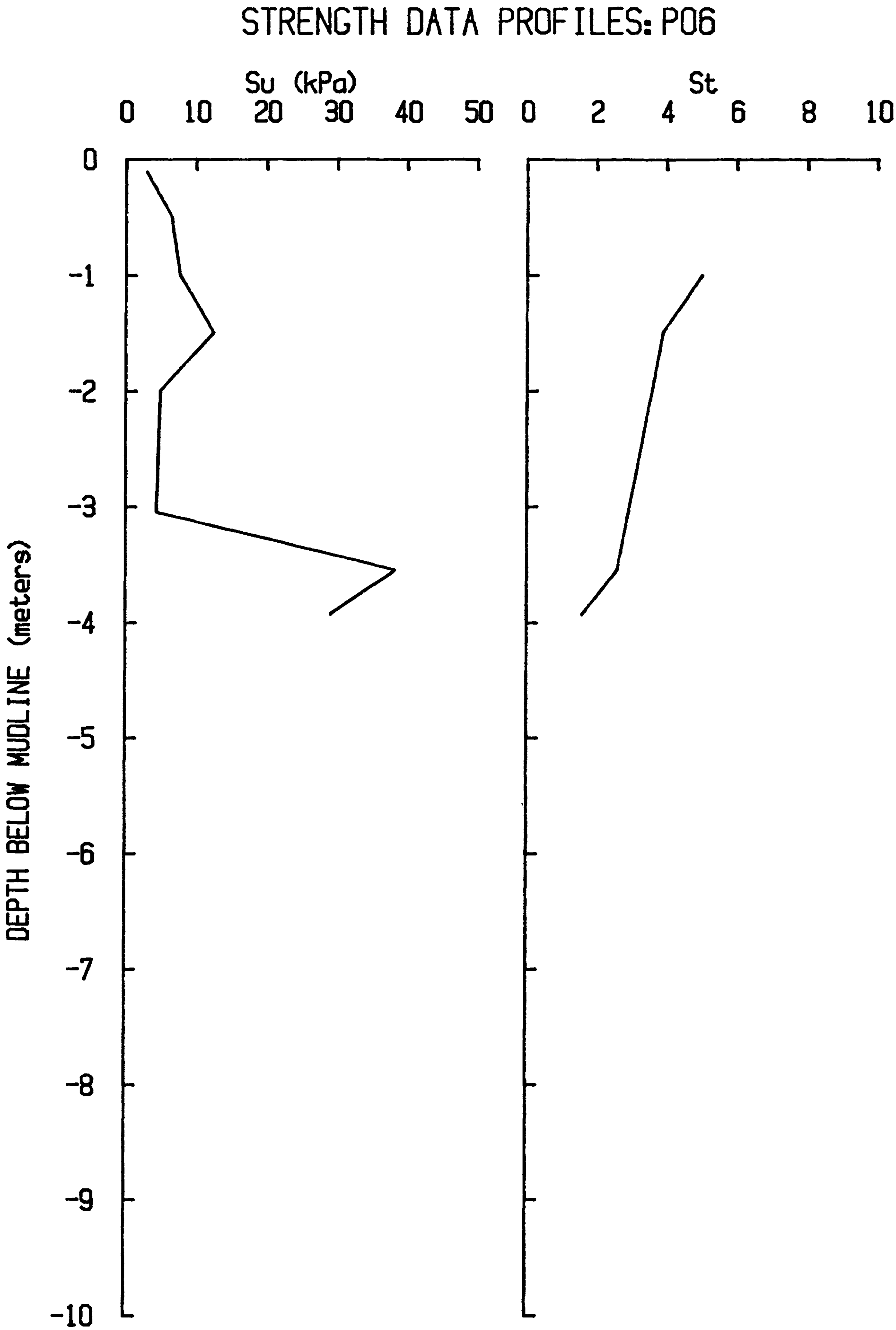


Figure 2h. Cohesion (S_u) and sensitivity (S_t) vs. depth in core P06.

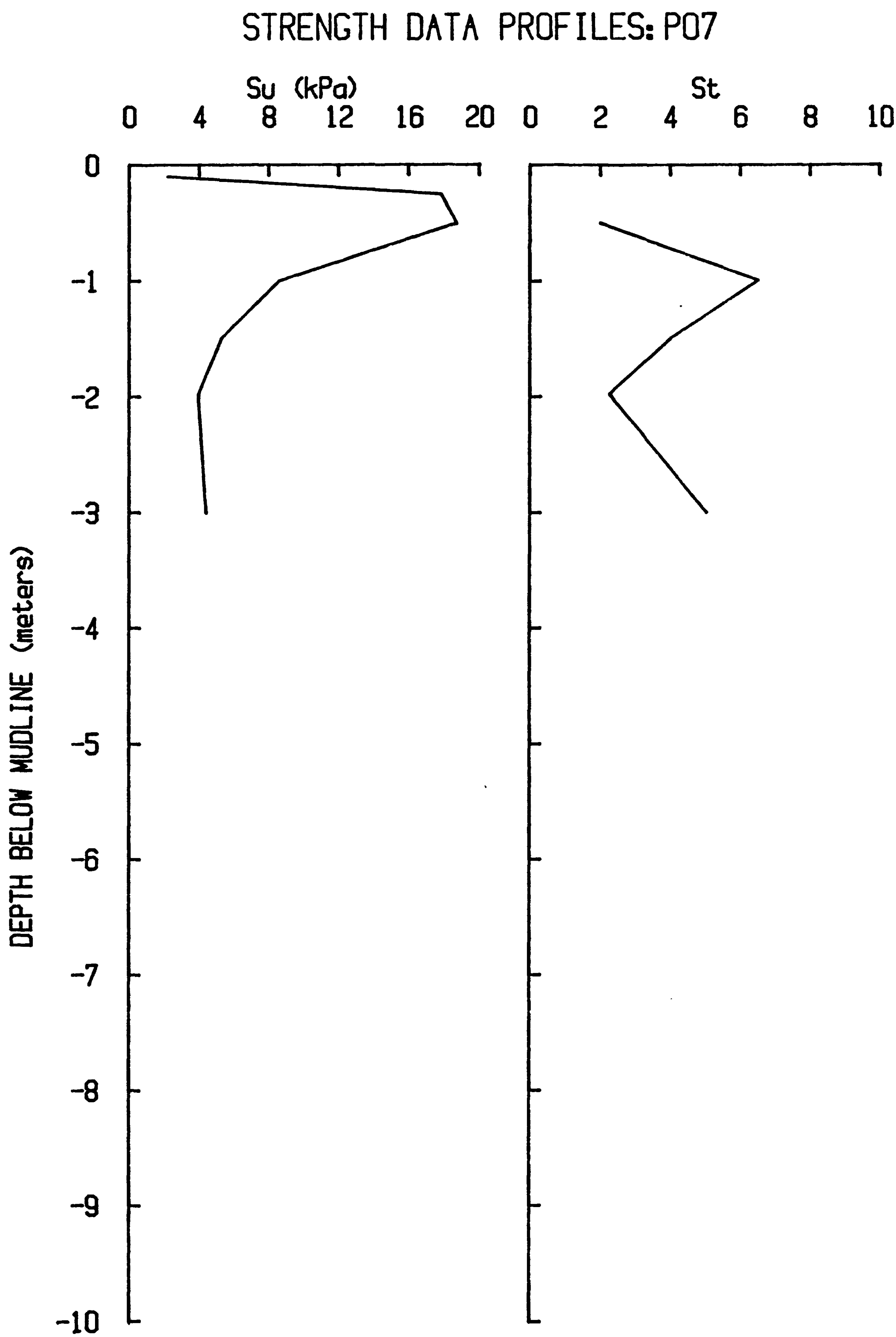


Figure 2i. Cohesion (S_u) and sensitivity (S_t) vs. depth in core P07.

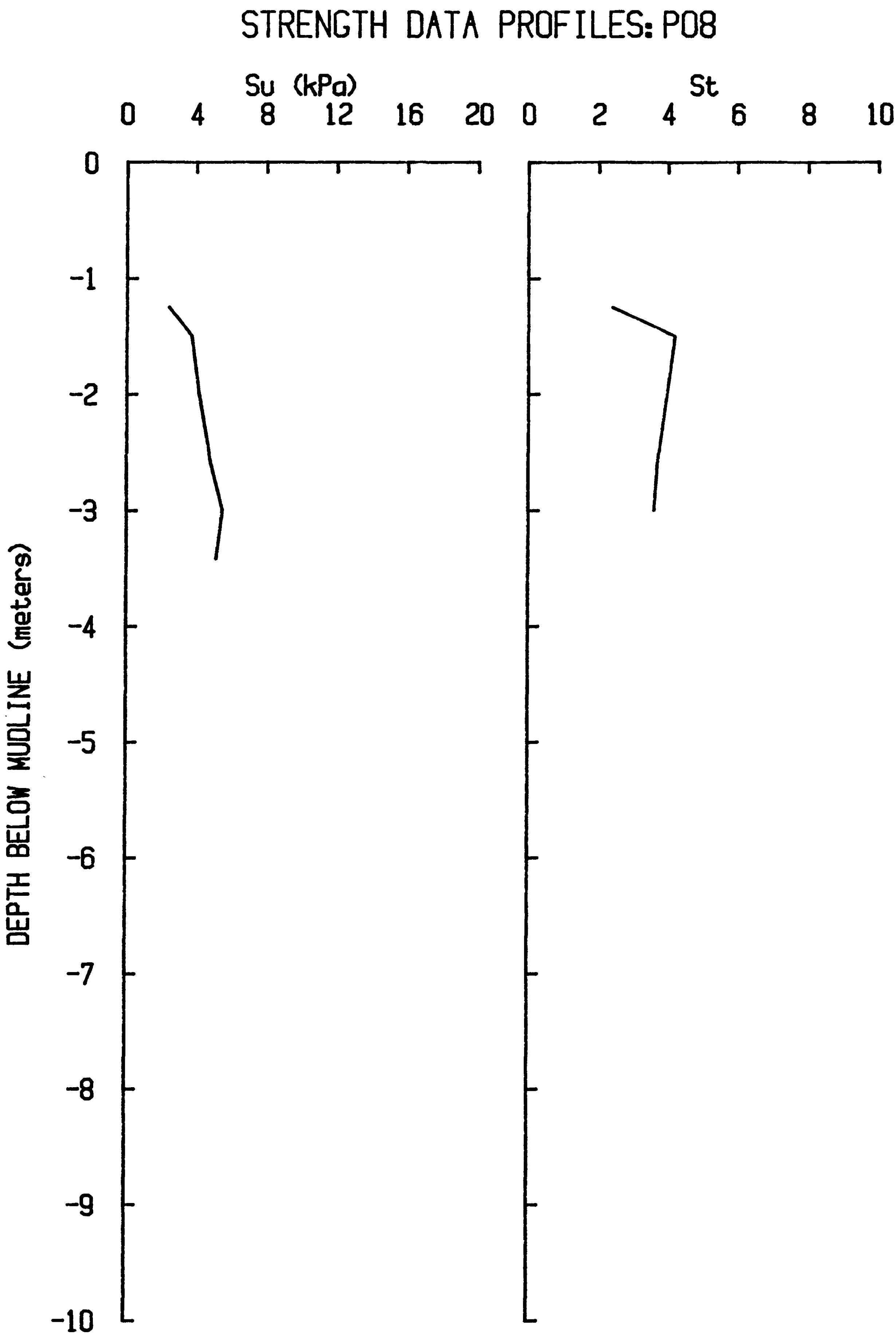


Figure 2j. Cohesion (S_u) and sensitivity (S_t) vs. depth in core P08.

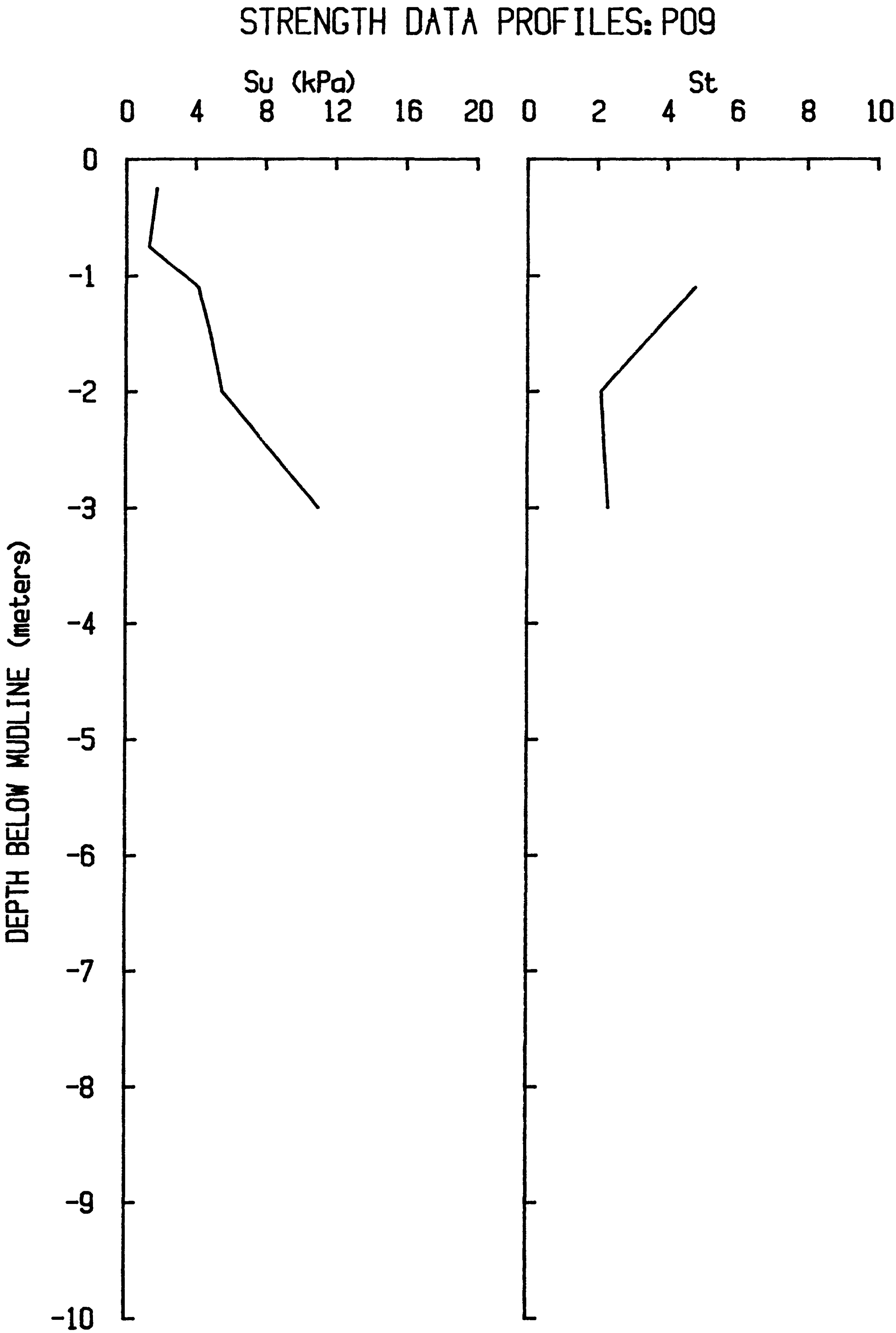


Figure 2k. Cohesion (S_u) and sensitivity (S_t) vs. depth in core P09.

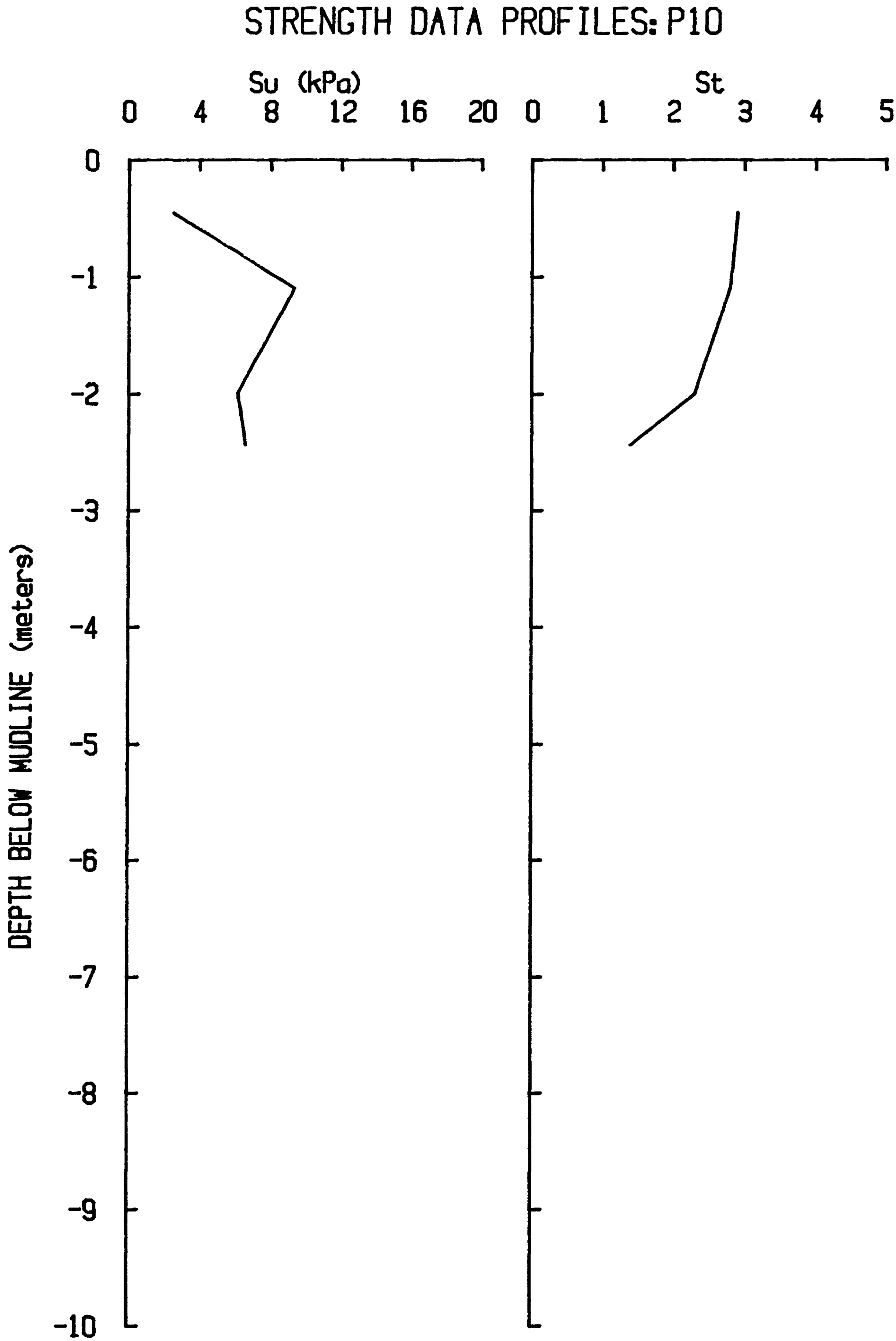


Figure 21. Cohesion (S_u) and sensitivity (S_t) vs. depth in core P10.

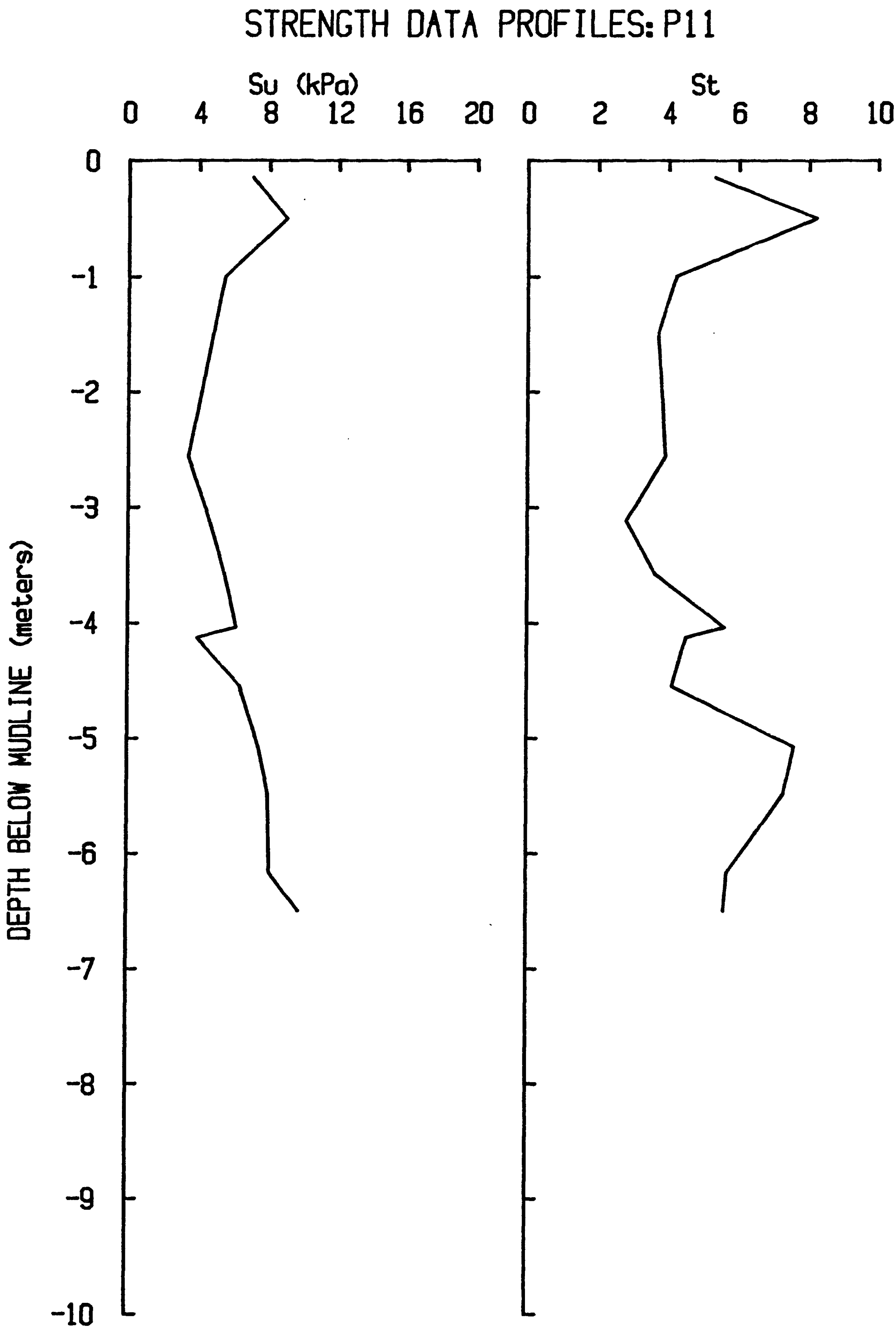


Figure 2m. Cohesion (S_u) and sensitivity (S_t) vs. depth in core P11.

Table II

Cohesion and Index Property Data Summary

Property	Number of measurements	Min.	Avg.	Max.
Natural shear strength (kPa)	98	1.3	11.0	88
Sensitivity	62	1.4	4.8*	15.0*
Water content (%)	131	30	59.7	113
Bulk density (g/cc)	118	1.46	1.72	1.98
Porosity (%)	120	45	60	73
Liquid limit (%)	127	33	57.4	108
Plastic limit (%)	130	16	25.1	40
Plasticity index (%)	127	12	32.5	70
Liquidity index	124	.15	1.12	2.33
Grain-specific gravity	94	2.65	2.70	2.76

*Because many samples were too weak to measure (cohesion below threshold value) for vane-shear apparatus after remolding, these values are minimums.

Sensitivities determined in this study are slightly higher than those reported from some other studies in this and proximal slope areas (Keller and others, 1979; McGregor and others, 1979), although they are similar to those reported by Lambert and others (1981) and Booth and others (1984). Marine fine-grained sediments typically have sensitivities of four or less, thus at 4.8, these sediments are slightly more sensitive than normal. Mitchell (1976) presents a discussion of the factors which contribute to a high sensitivity. An open, metastable fabic, cements, thixotropic hardening, and the presence of dispersing agents (such as certain types of organic substances) may all be causes. In these sediments, however, the high percentage of silt suggests that an open fabric may not be of prime importance regarding sensitivity. A poor correlation (r=0.3; n=21) between mean grain size and S_t also would seem to indicate a lack of control of fabric on the values, at least as far as fabric and grain size are related. Further, the presence of cements is not obvious in these sediments. The other factors have not been investigated.

Sensitivity may also be used as a crude indication of sample quality. In the general case, disturbance leads to a reduction in "natural" shear strength, but remolded strength remains the same. Therefore, the sensitivity value (undisturbed S_u/remolded S_u) decreases. Thus, although there are many factors which may control sensitivity (as discussed previously), a strong positive correlation between natural cohesion and sensitivity may be one indication that systematic disturbance has occurred. We found no such correlation: in fact, r=0.04 in the sample population (n=58) used for the determination. The fact that our sensitivity values tend to be slightly higher than those reported from other studies conducted in the general area also suggests, insofar as sensitivity may be used in this manner, that the sample quality is relatively good.

Bulk-density group

Bulk density and related properties vary considerably. Table II shows the minima, maxima, and mean value for this group, which, along with bulk density (γ_t), includes water content (w), grain-specific gravity (G_s), and porosity (n). Of these properties, only grain-specific gravity is independent of the effects of the compaction process: the others tend to change in concert with relative compaction and, thus, one another.

The mean value of bulk density (1.72 g/cc) along with the mean values for water content (60 percent) and porosity (60 percent) are anomalous for surficial marine sediment. This combination of rather high bulk density and appropriately low water content and porosity may be attributed to either the sampling of overconsolidated material or a texture that is atypically coarse in comparison with other continental slopes and rises. Both contribute. As discussed previously, several cores were recovered from erosional surfaces. The exhumed sediment in these areas is relatively compact in contrast to areas experiencing uninterrupted sediment accumulation. Thus, the properties reflect this history. In addition, the sediment sampled in this study contains significantly more silt than other studies have reported from this and adjacent continental margin areas. This also tends to increase bulk density and decrease water content and porosity. For data comparisons see Keller and others (1979), McGregor and others (1979), Lambert and others (1981), and Booth and others (1984).

Specific gravity values (table II) are similar to those reported in the aforementioned publications, although slightly lower. On average, the grain-specific gravities are also slightly less than for most terrigenous marine sediment.

Despite the shortness of some of the cores and the presence of samples from overconsolidated sediment, the properties in question do exhibit the expected downcore trends. The profiles shown in figures 3a-m indicate, in general, that bulk density tends to increase, and water content and porosity tend to decrease, downcore. Specific gravity shows no trend. As with some of the cohesion and sensitivity profiles, a sawtooth pattern is present in these plots. This suggests a complexity in depositional and post-depositional processes.

On a regional basis, bulk density tends to decrease and water content and porosity tend to increase in both a downslope and, possibly, along-slope (to the northeast) direction. The former observed trend is in agreement with the findings of Keller and others (1979).

Plasticity

The plasticity characteristics of a sediment (i.e., liquid limit, plastic limit, plasticity index, and liquidity index) provide a basis for classification, an indication of certain other intrinsic properties (e.g., texture, mineralogy), and partial insight into its stress behavior.

Classification is traditionally accomplished by using a chart devised by Casagrande (1948). The chart is divided into fields that represent different soil types. Figure 4 is a plot of samples from this study on such a chart. There is a considerable spread of points along and above the A-line, which indicates a wide range in the plasticity characteristics of the sediment. However, only two soil types are represented in the analyzed samples. The average sample is classified as an inorganic clay of high plasticity (Unified Soil Classification: CH). According to Wagner (1957), this material would be considered relatively undesirable as a foundation material, although it is typical of marine sediments in such a setting and is a common terrestrial soil type.

Vertical profiles of plastic limit, liquid limit, and water content are shown in figure 5a-o. Downcore fluctuations in values of the two limits (e.g., P11, fig. 5o) are intimately associated with changes in texture. An increase generally means finer material is present and a decrease indicates the presence of coarser material. Mineralogy, which may also have a major effect on liquid and plastic limit, is fairly constant in these cores (L. J. Poppe, unpublished data, 1980) and therefore does not represent an important source for variation. The profiles also show that, except for some of the short cores (particularly those short cores associated with erosional surfaces), water contents tend to be greater than liquid limit (see fig. 5i). The average liquidity index of 1.1 (table II) reflects this relationship. This circumstance is common on the upper few meters of marine fine-grained sediments and implies that upon remolding from dynamic loading, or other source of vibration or disturbance, these surficial sediments would behave more as a viscous liquid than as a plastic material. The fact that some of the water contents are less than the liquid limits in some of the

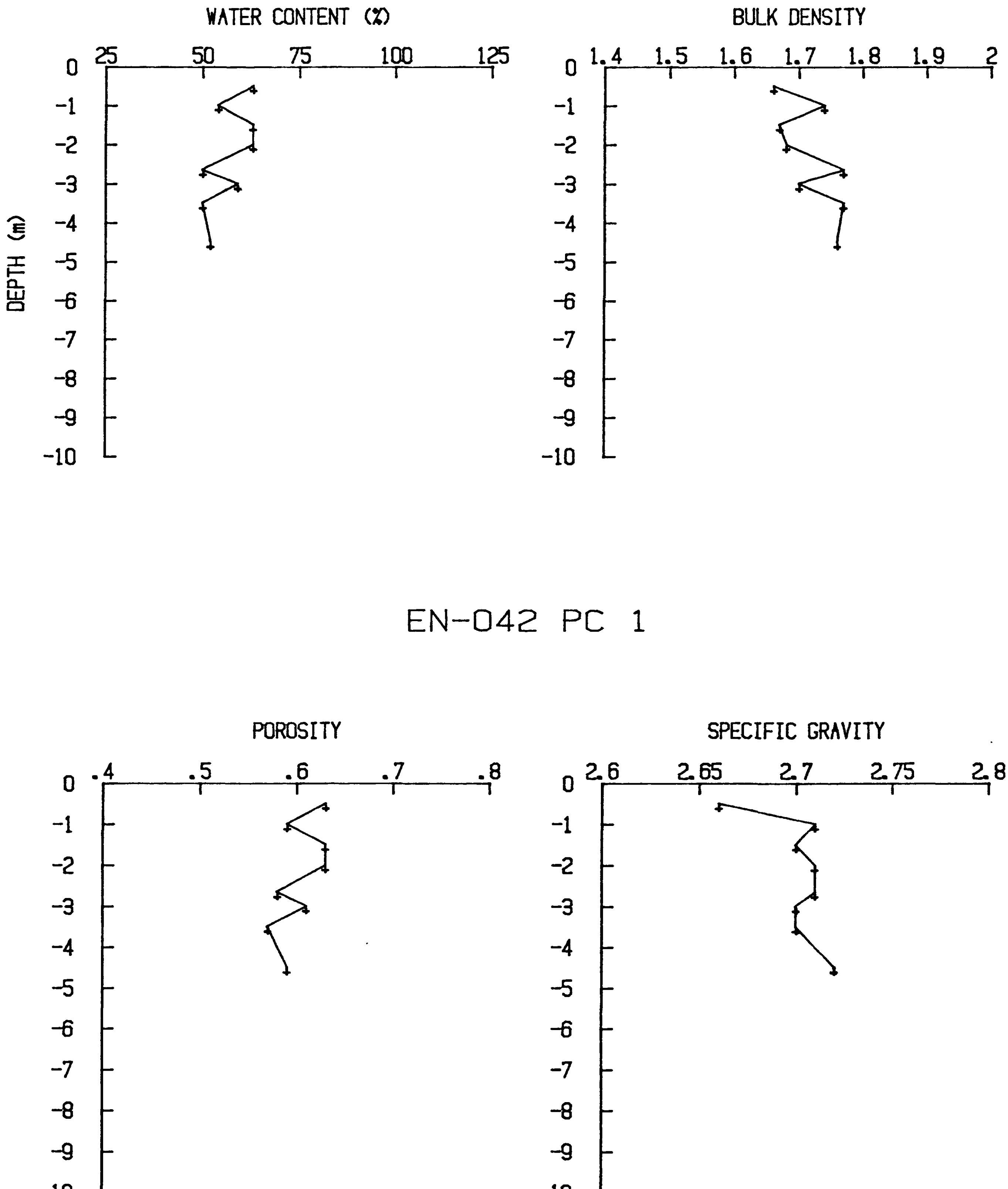


Figure 3a. Water content, bulk density, porosity, and grain-specific gravity vs. depth in core PC1.

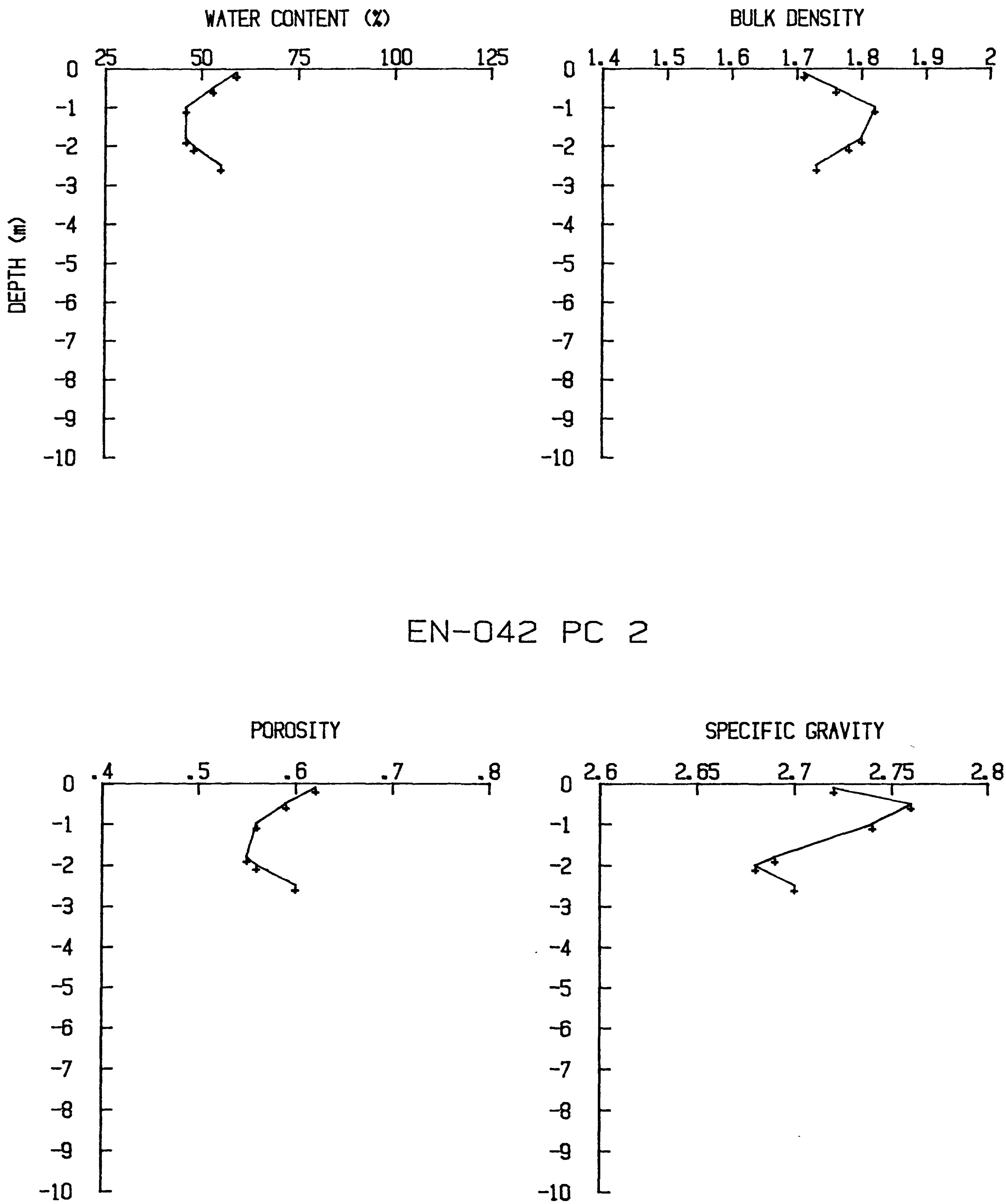


Figure 3b. Water content, bulk density, porosity, and grain-specific gravity vs. depth in core PC2.

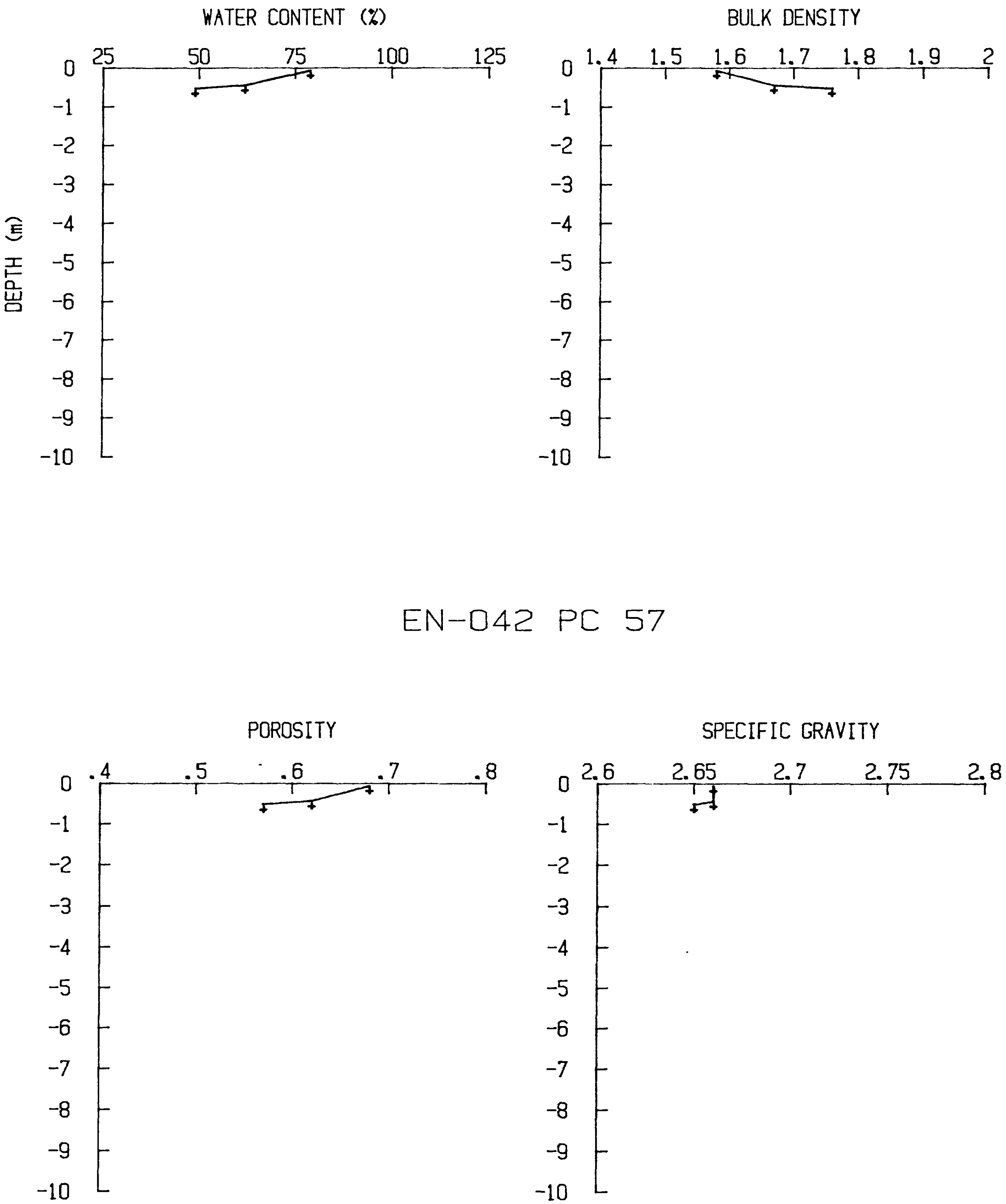


Figure 3c. Water content, bulk density, porosity, and grain-specific gravity vs. depth in core PC57.

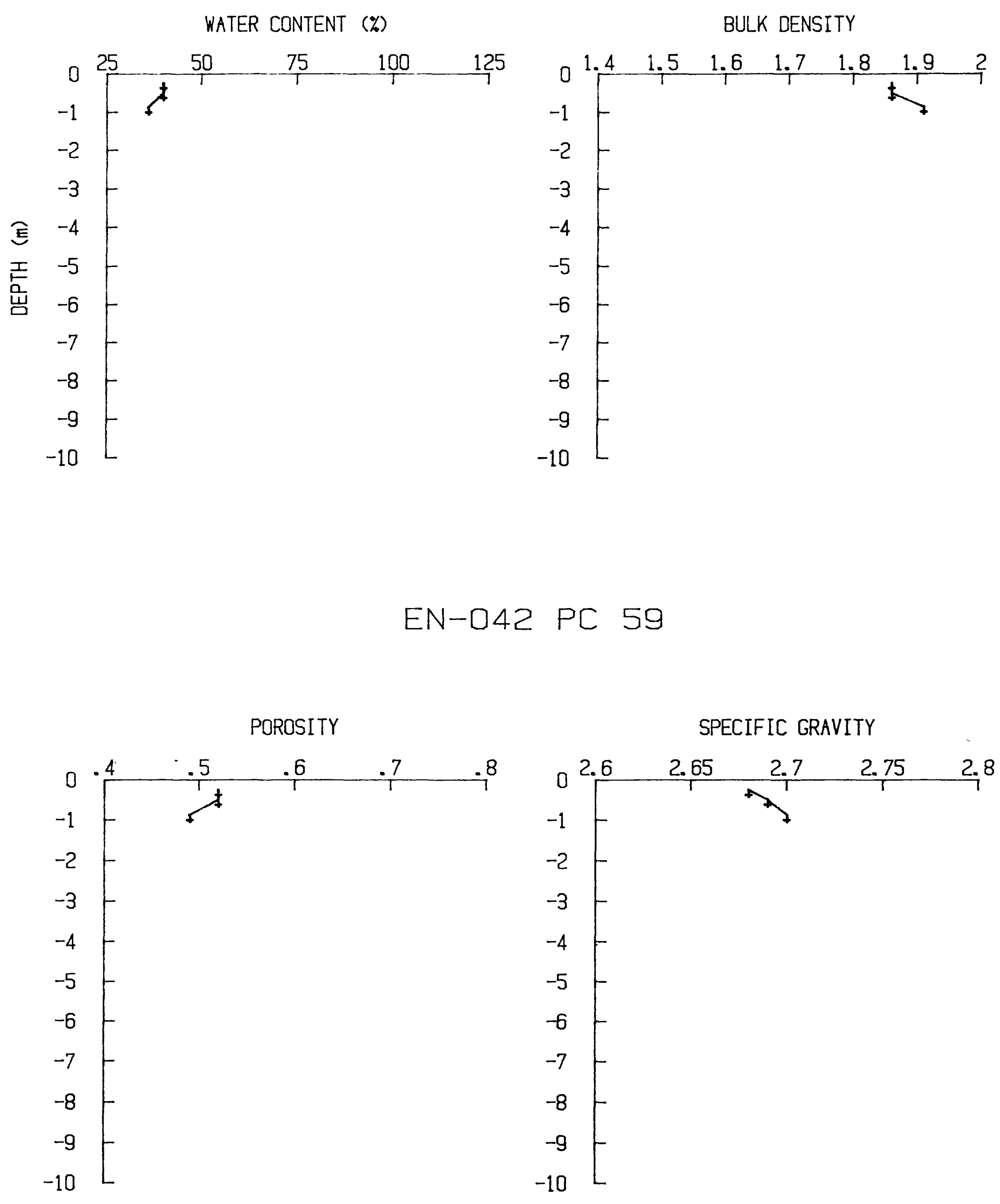


Figure 3d. Water content, bulk density, porosity, and grain-specific gravity vs. depth in core PC59.

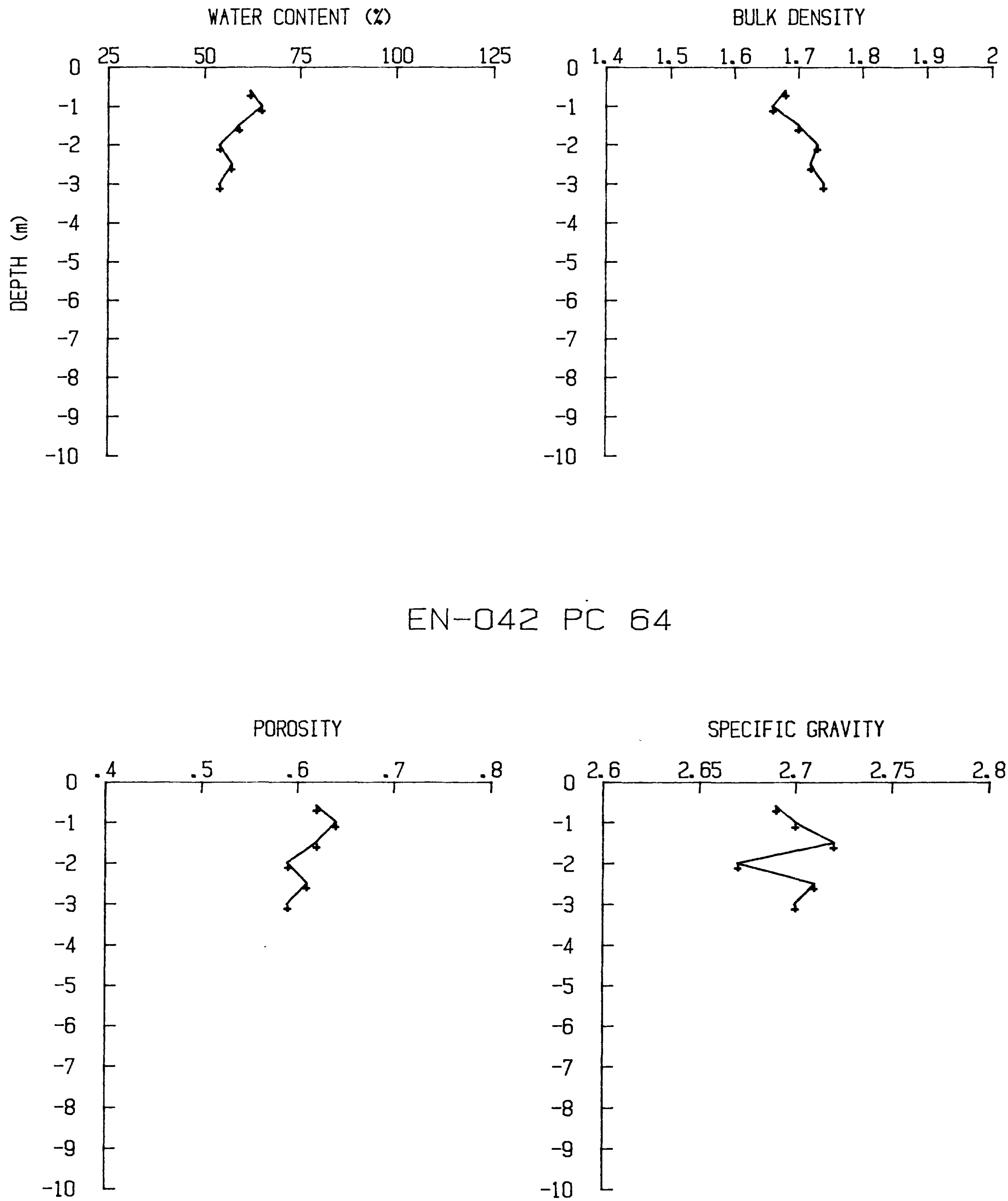


Figure 3e. Water content, bulk density, porosity, and grain-specific gravity vs. depth in core PC64.

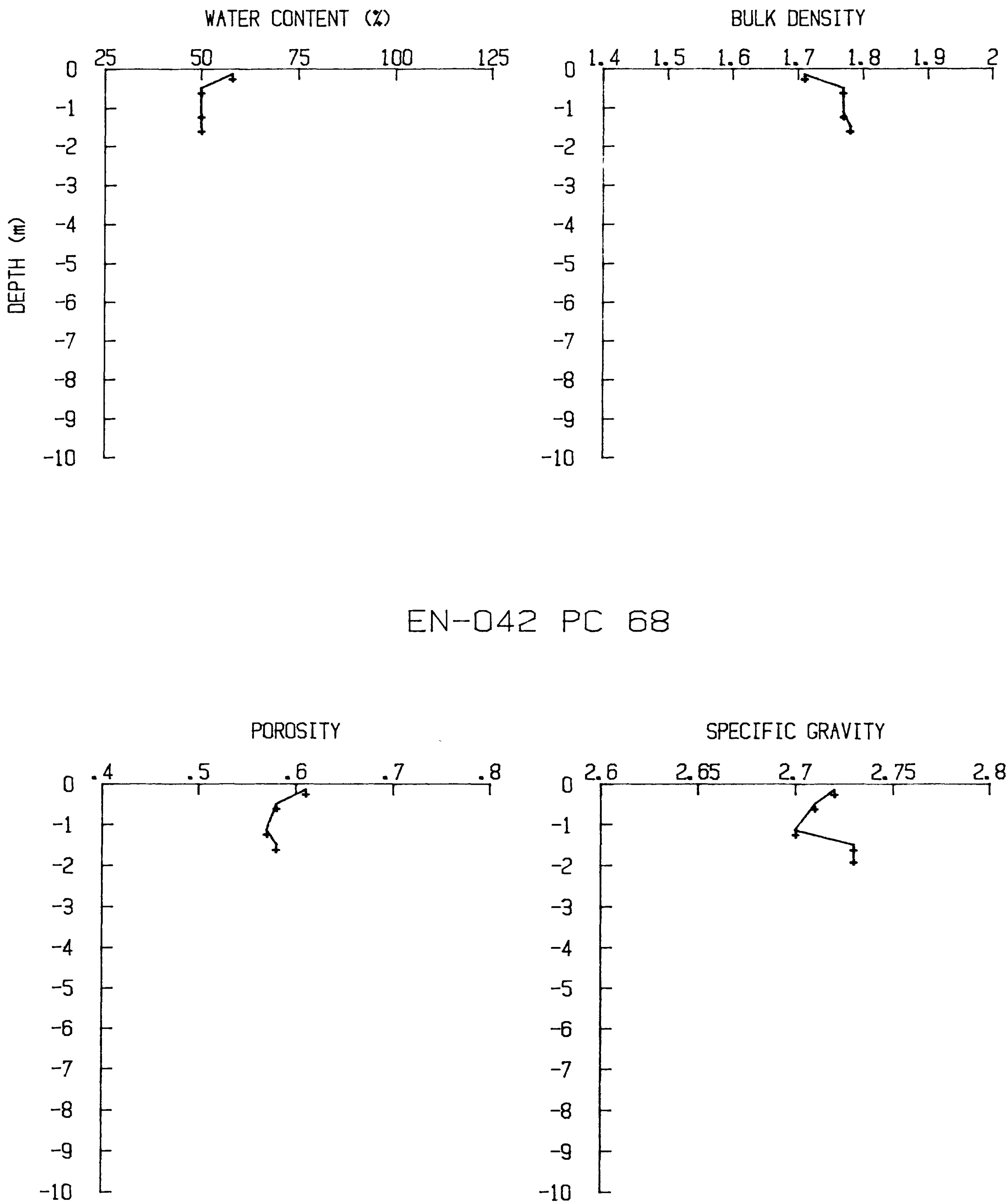


Figure 3f. Water content, bulk density, porosity, and grain-specific gravity vs. depth in core PC68.

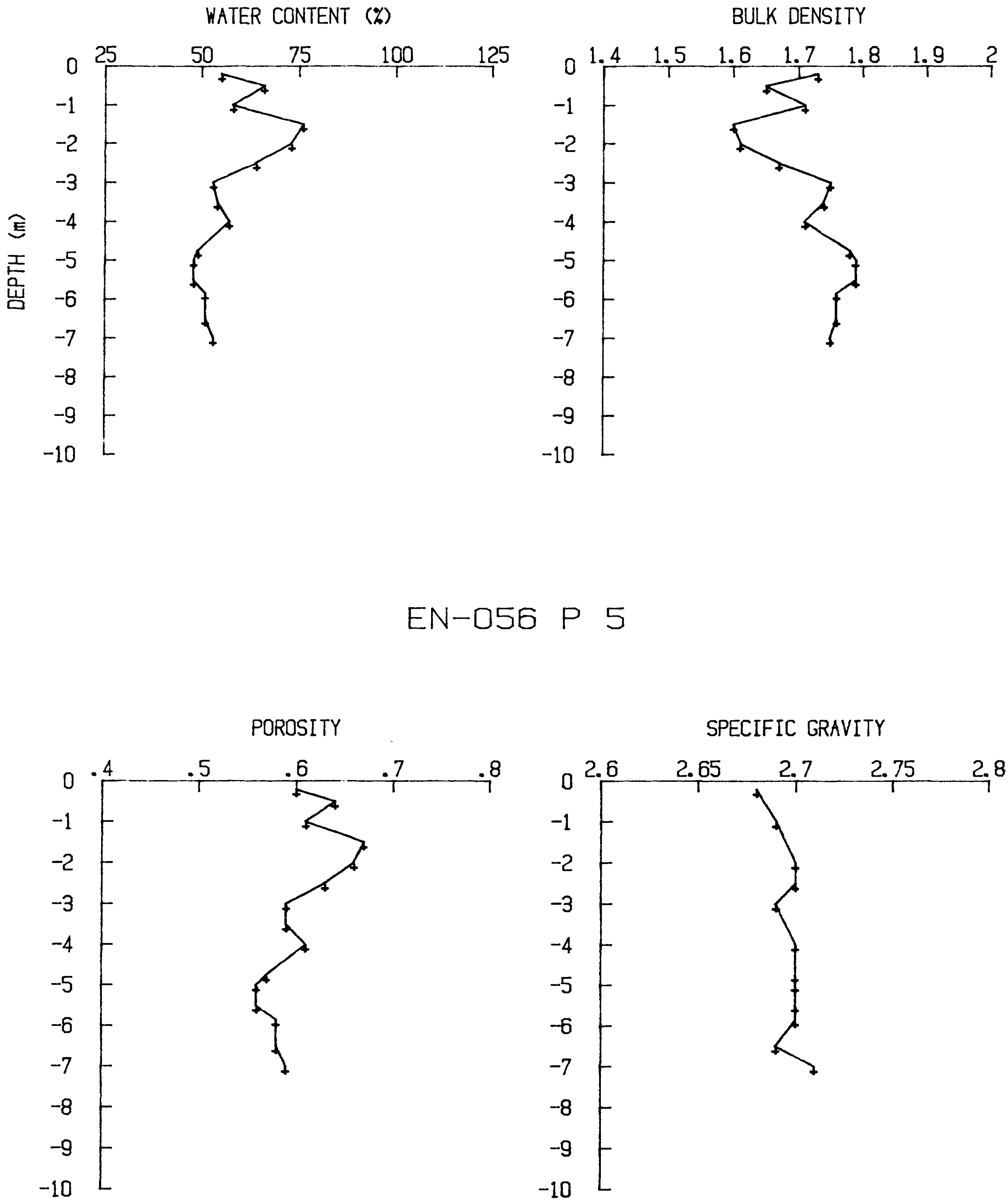


Figure 3g. Water content, bulk density, porosity, and grain-specific gravity vs. depth in core P5.

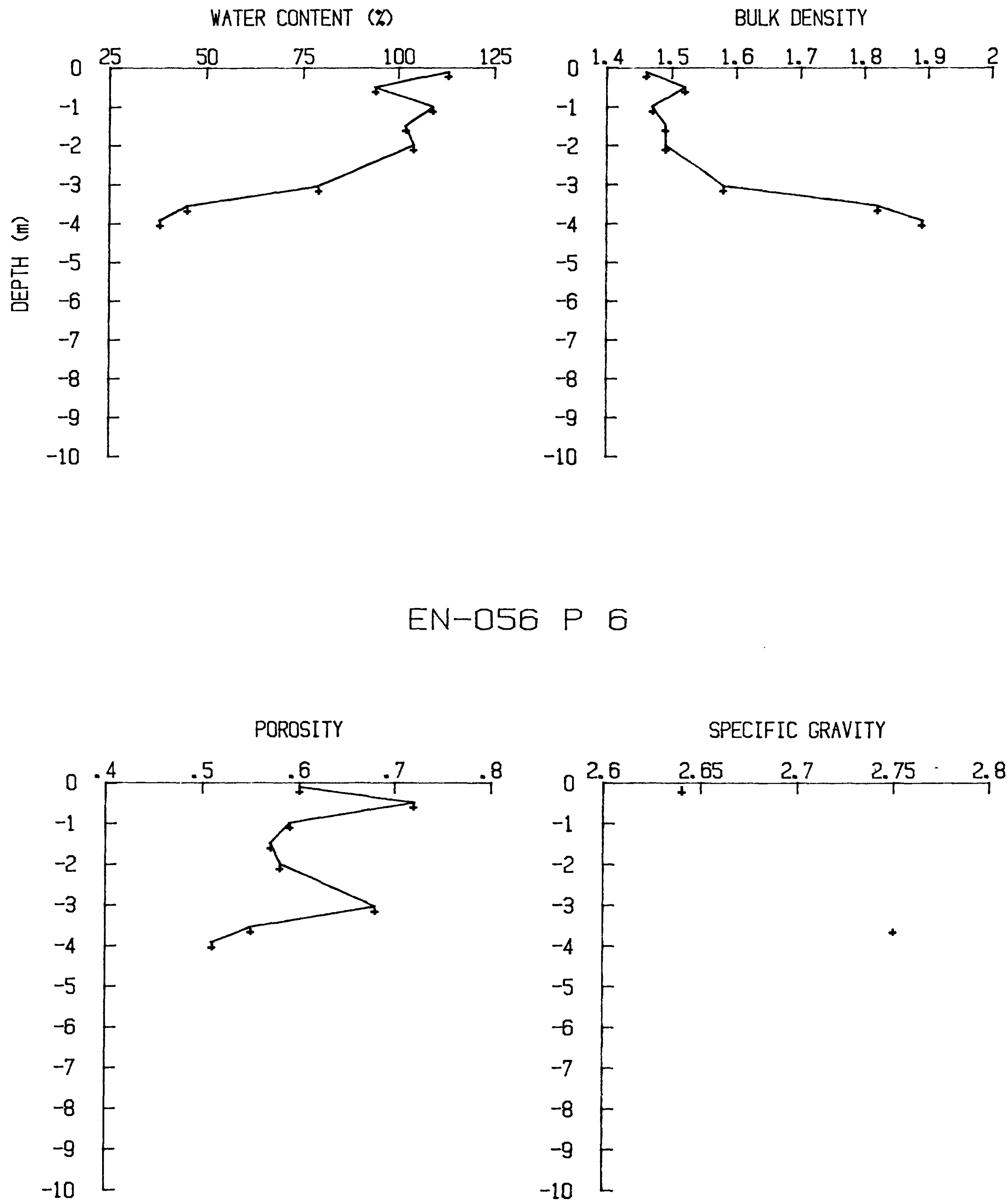


Figure 3h. Water content, bulk density, porosity, and grain-specific gravity vs. depth in core P6.

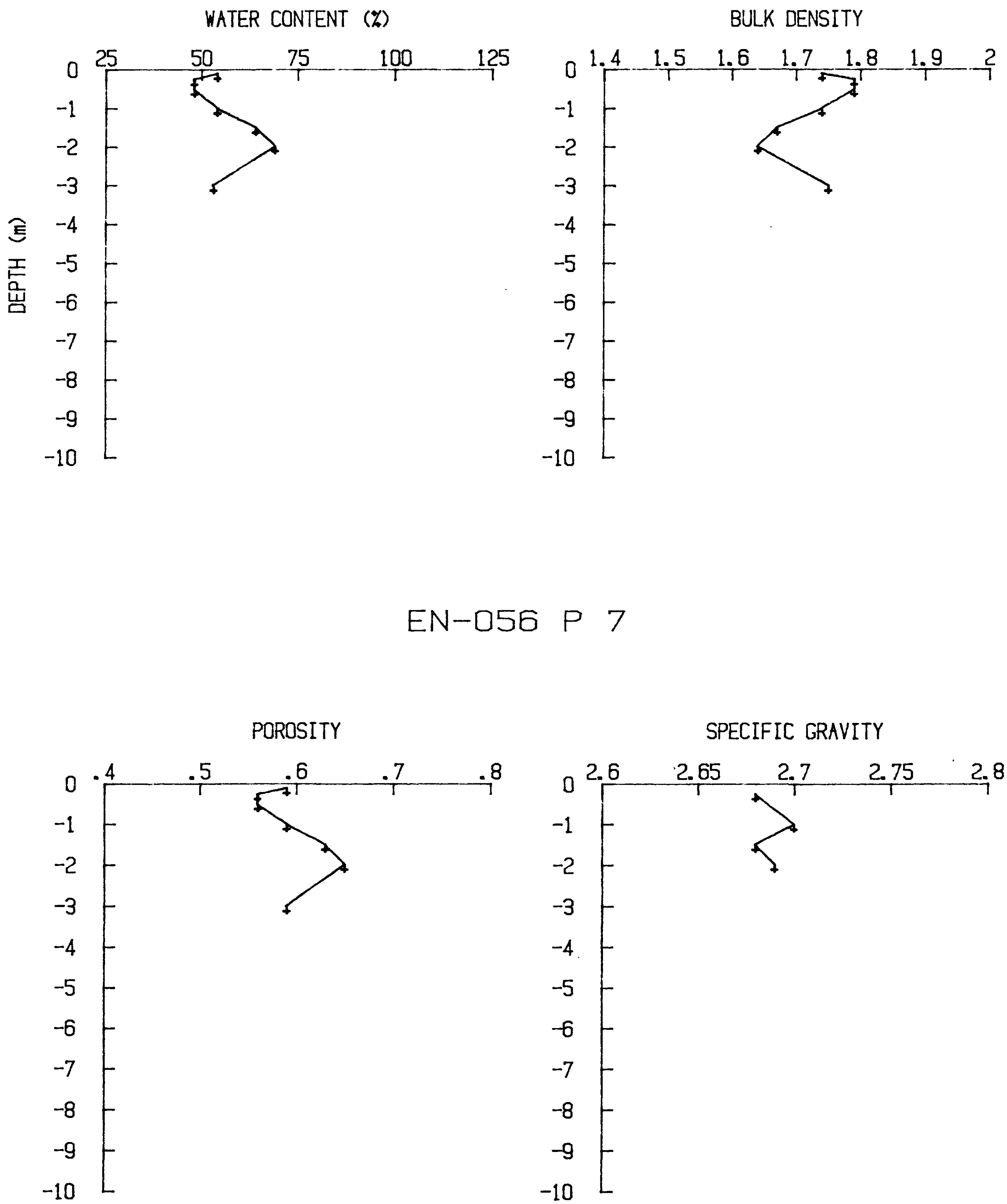


Figure 3i. Water content, bulk density, porosity, and grain-specific gravity vs. depth in core P7.

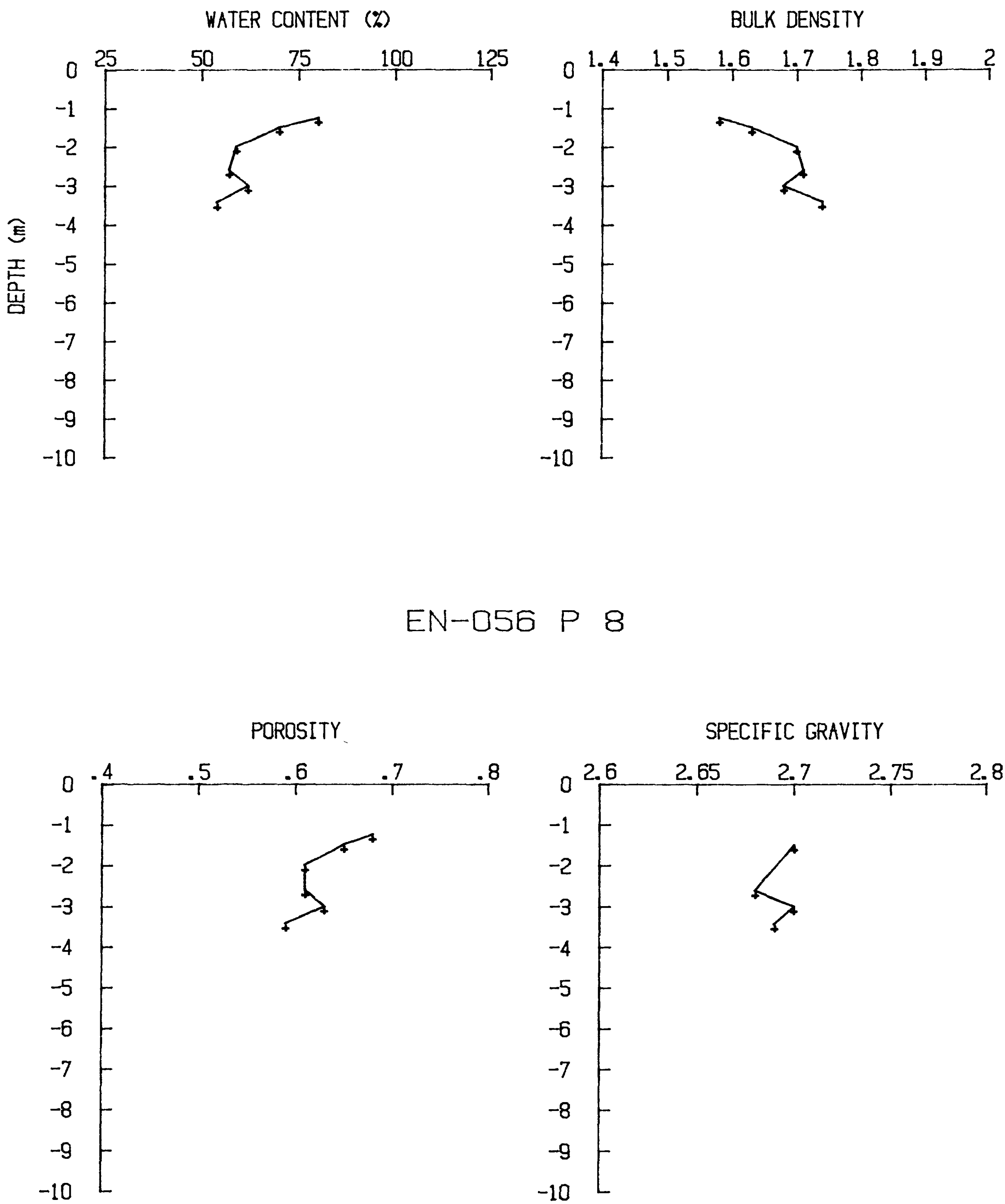


Figure 3j. Water content, bulk density, porosity, and grain-specific gravity vs. depth in core P8.

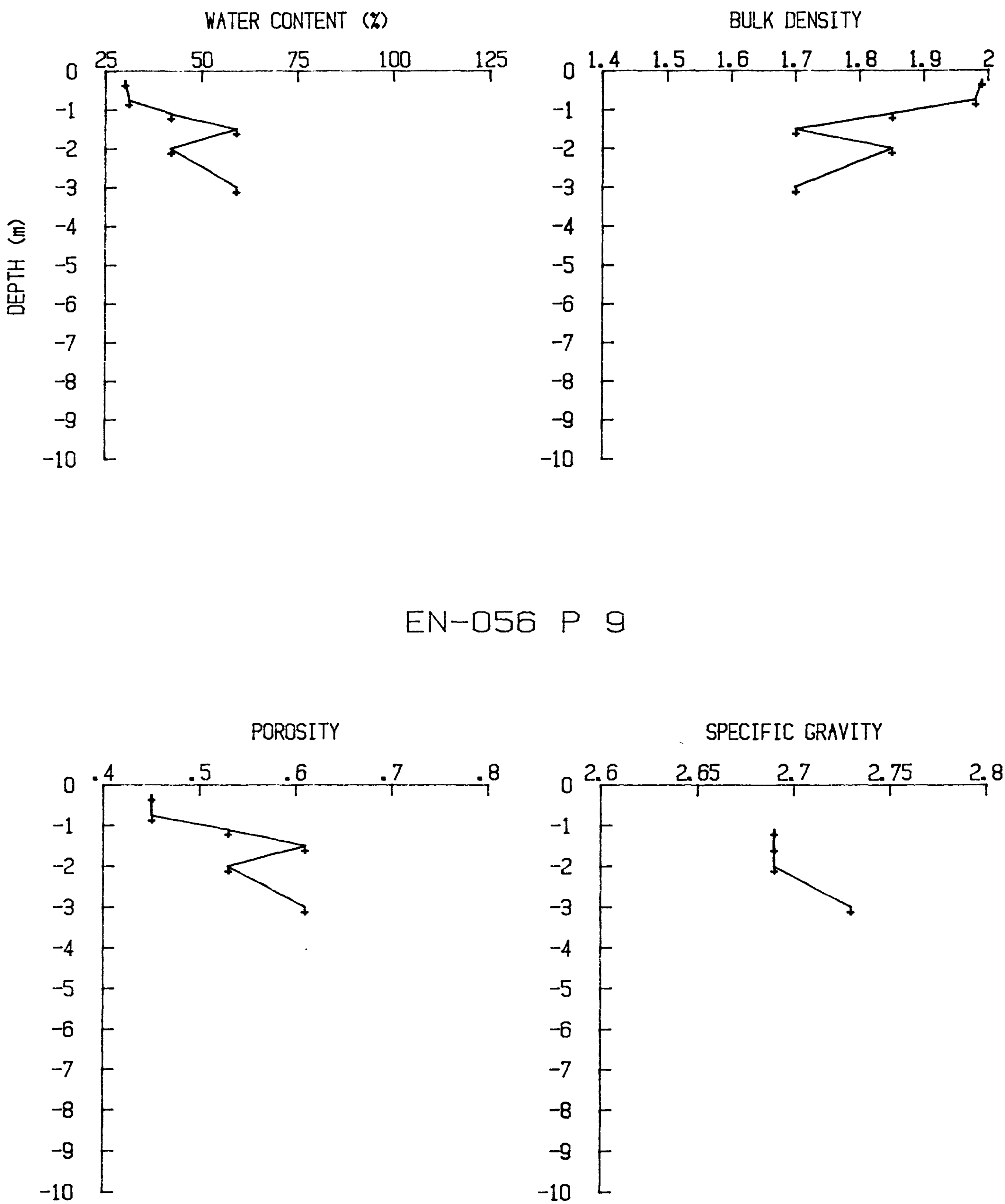


Figure 3k. Water content, bulk density, porosity, and grain-specific gravity vs. depth in core P9.

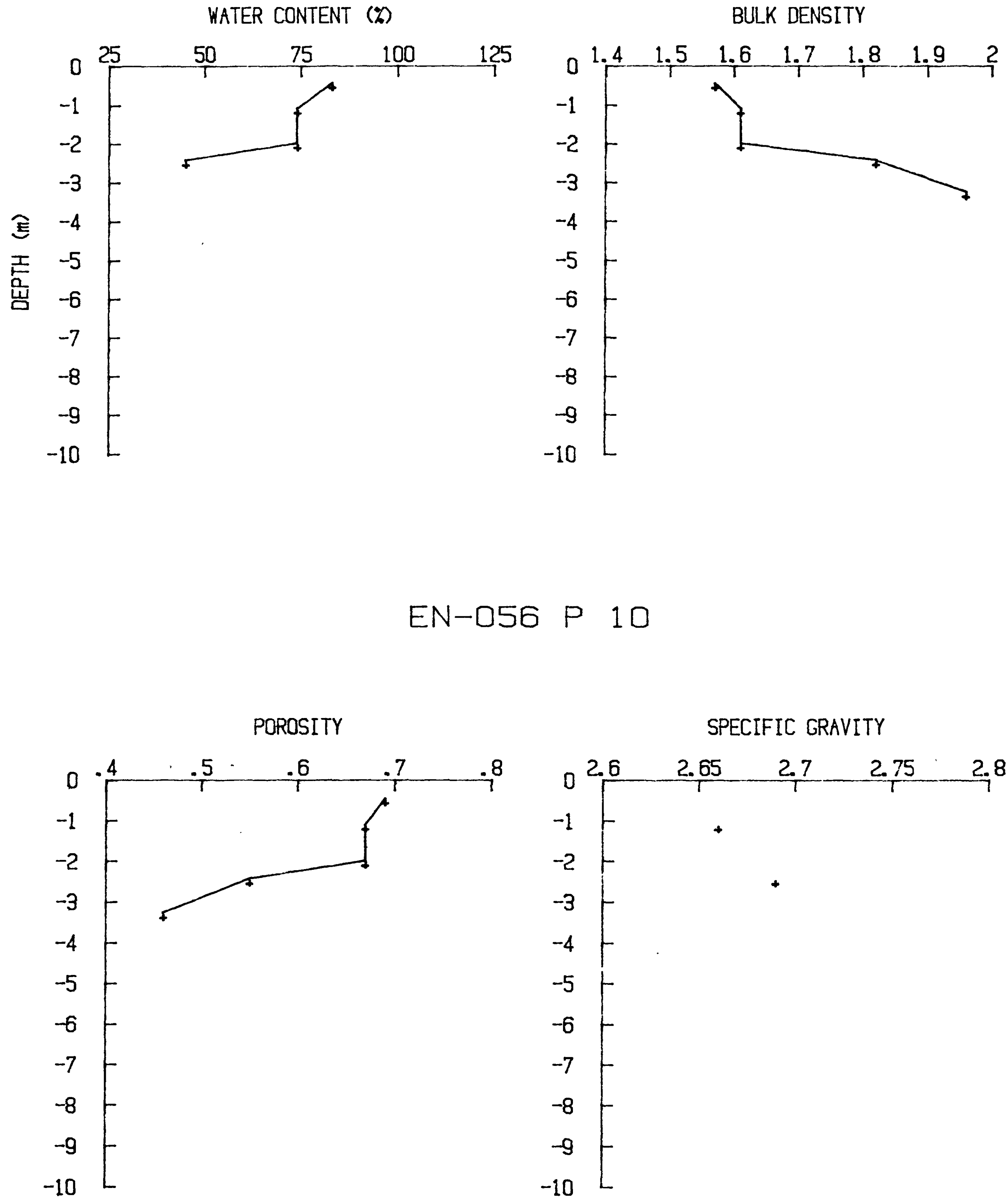


Figure 31. Water content, bulk density, porosity, and grain-specific gravity vs. depth in core P10.

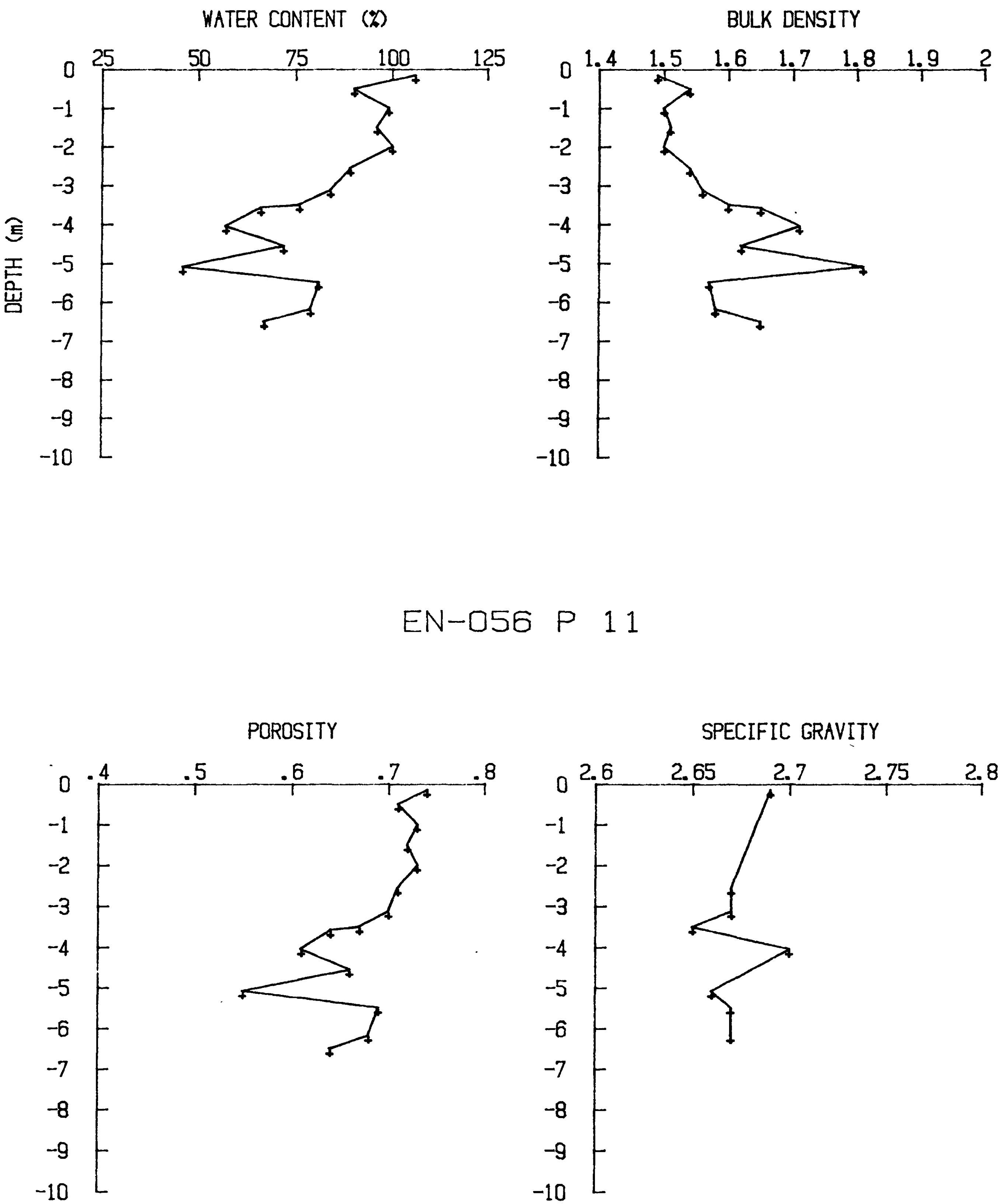


Figure 3m. Water content, bulk density, porosity, and grain-specific gravity vs. depth in core P11.

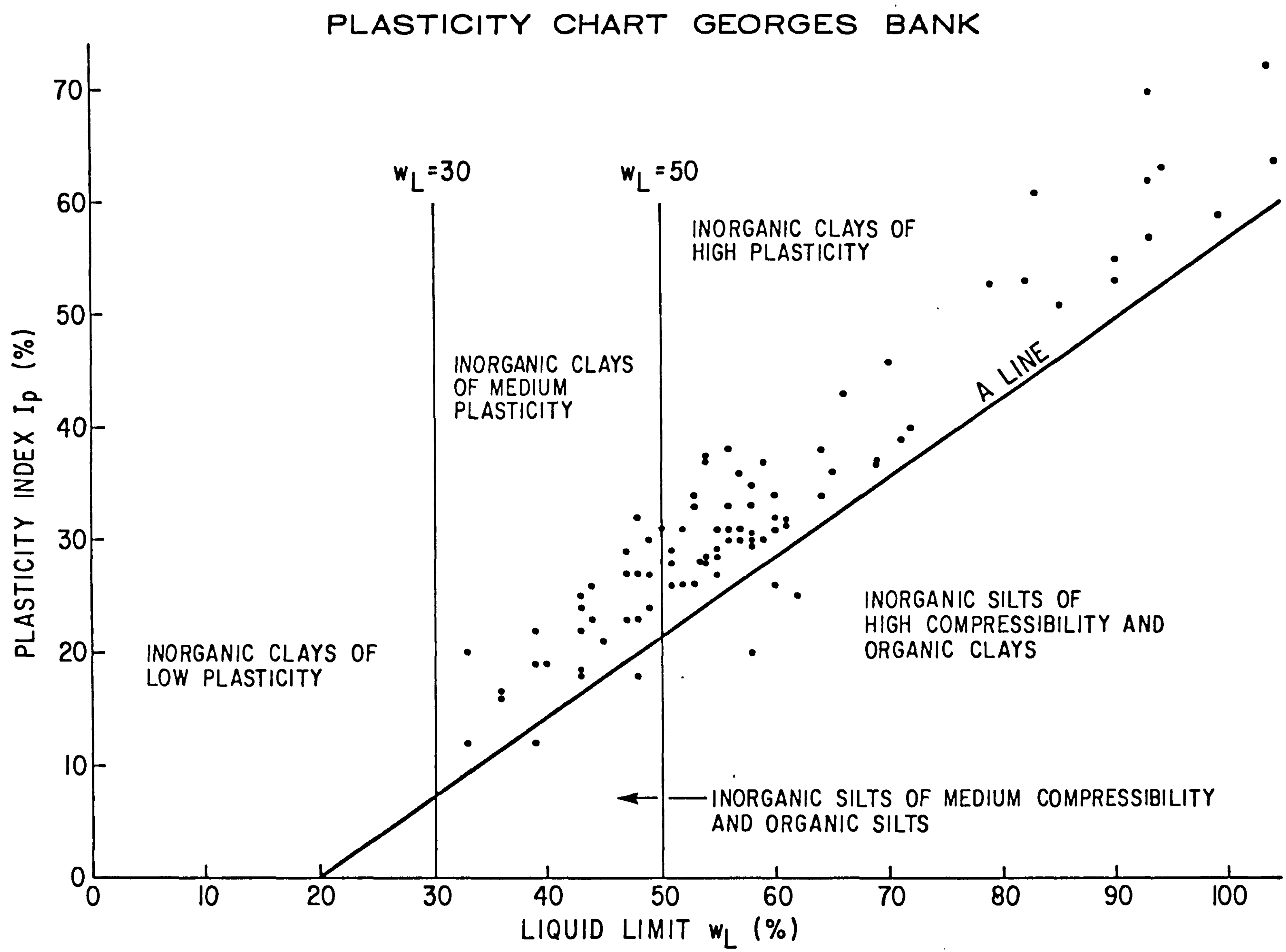


Figure 4. Classification of sediments based on plasticity characteristics.

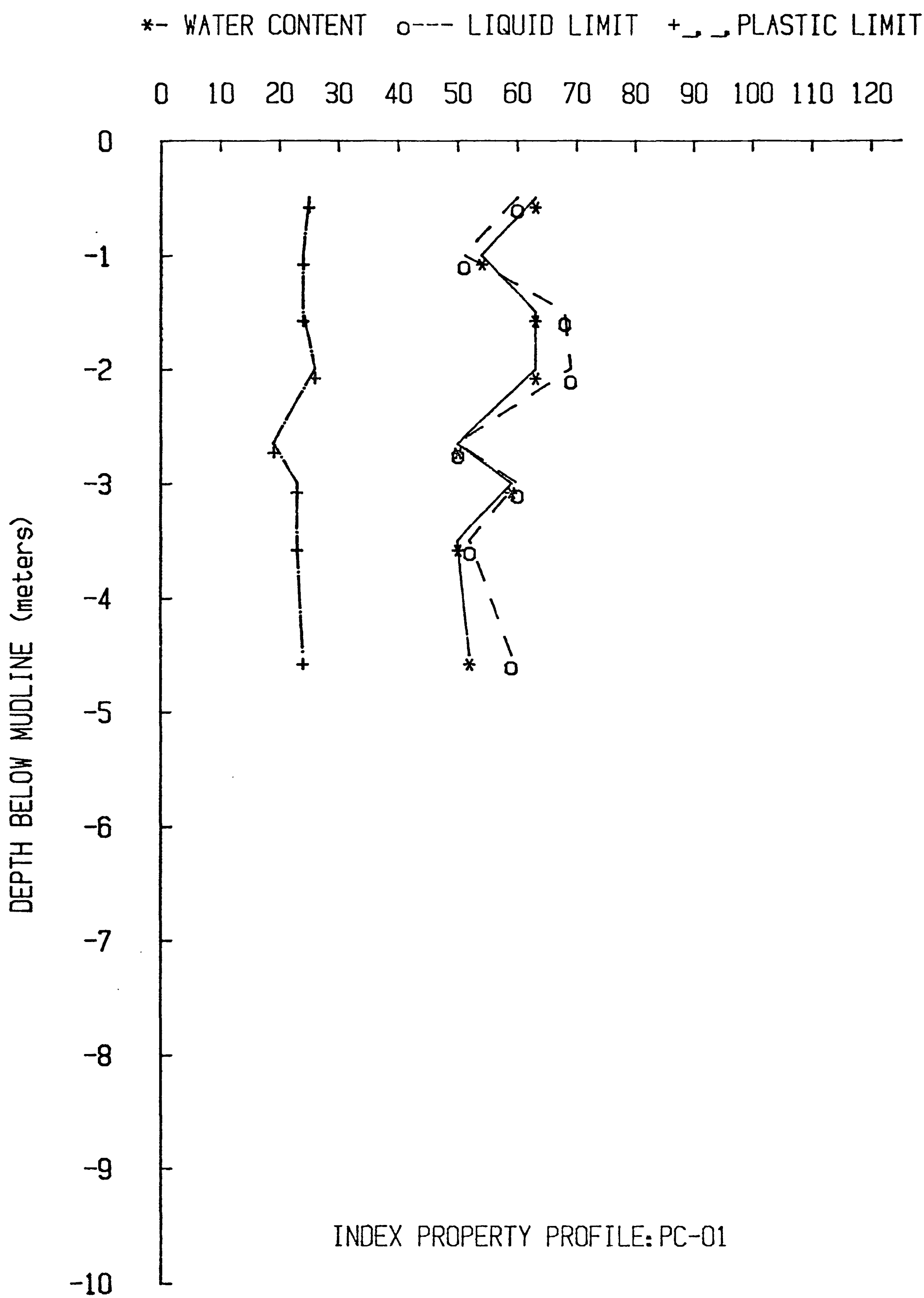


Figure 5a. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core PC01.

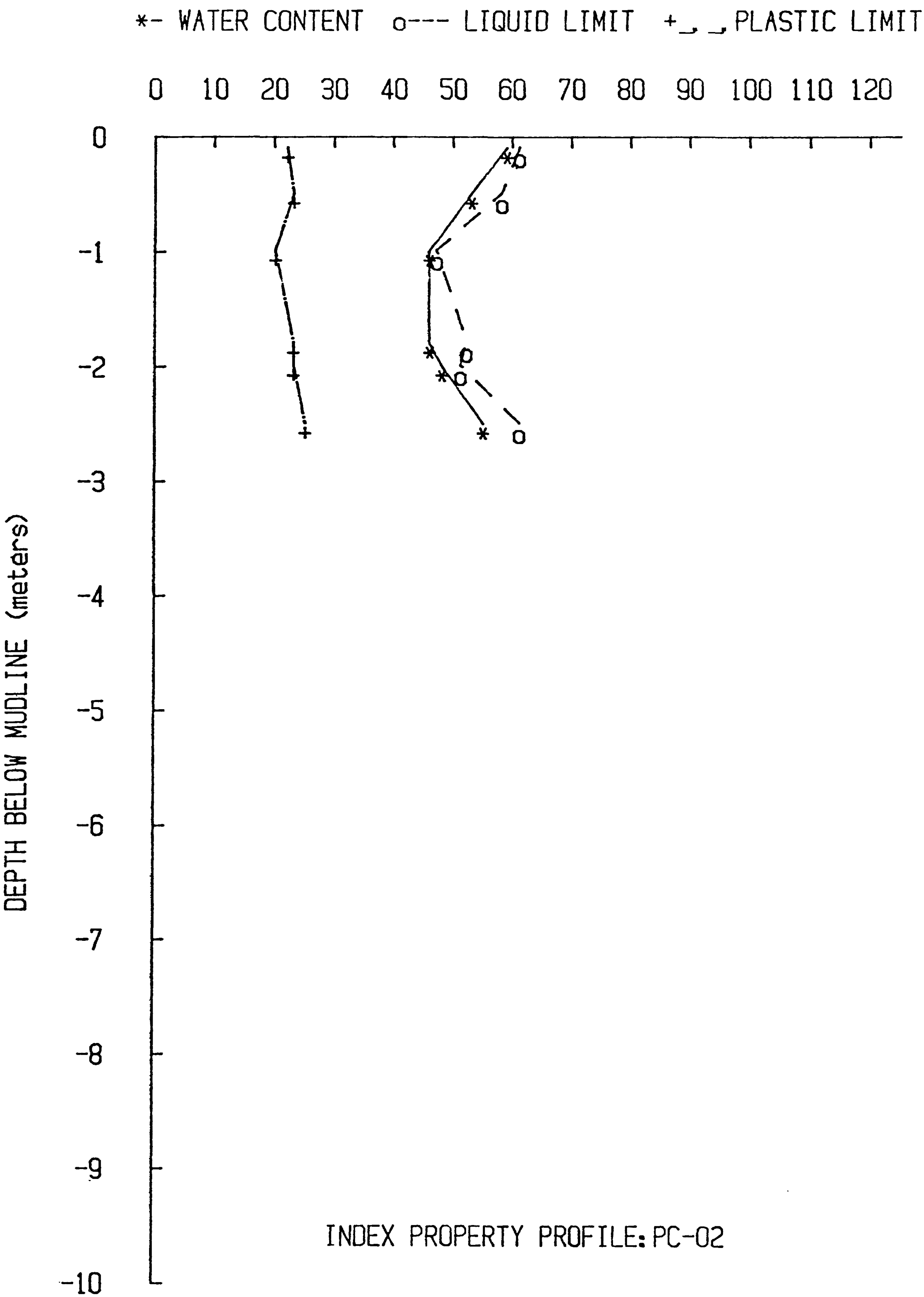


Figure 5b. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core PC02.

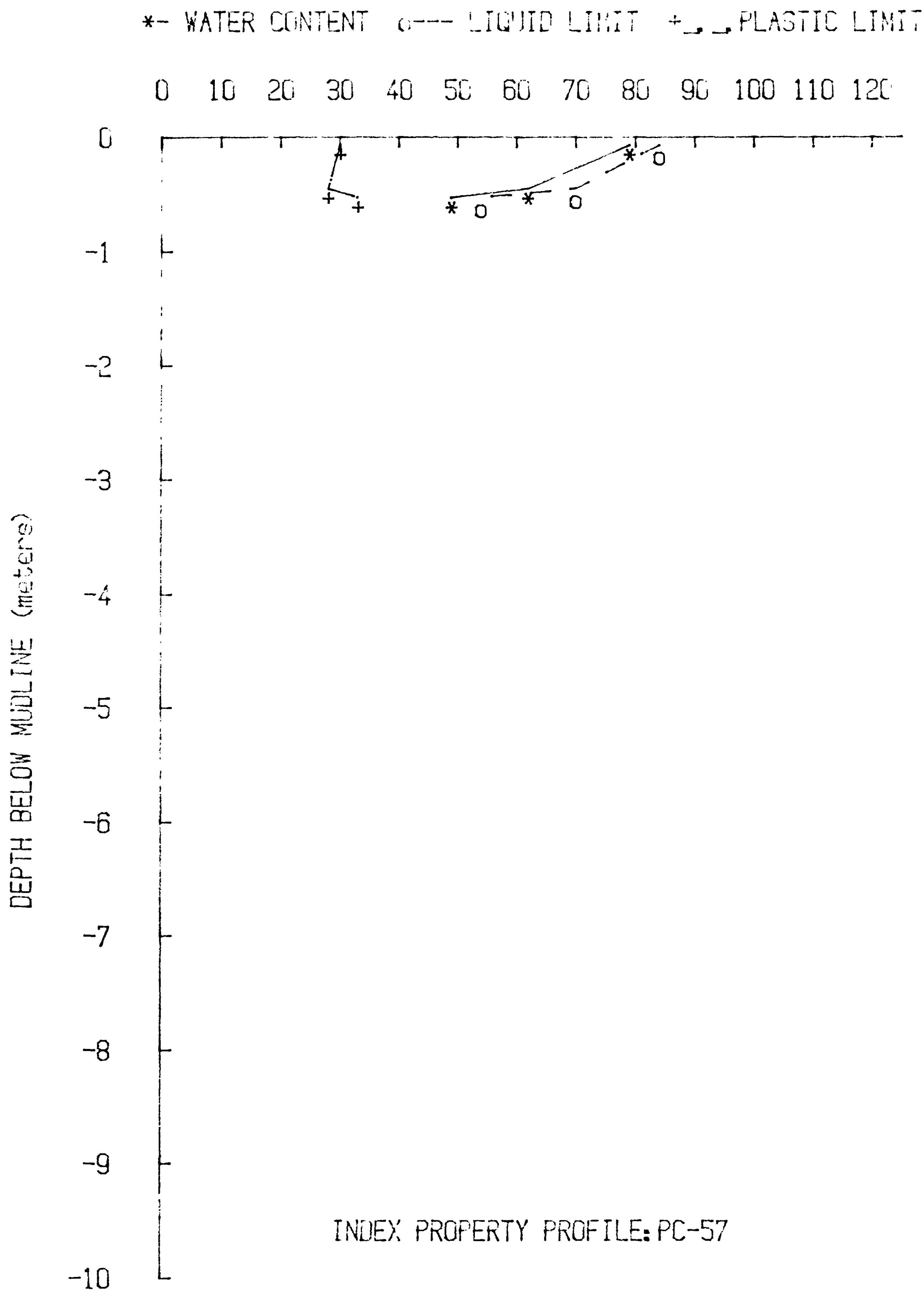


Figure 5c. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core PC57.

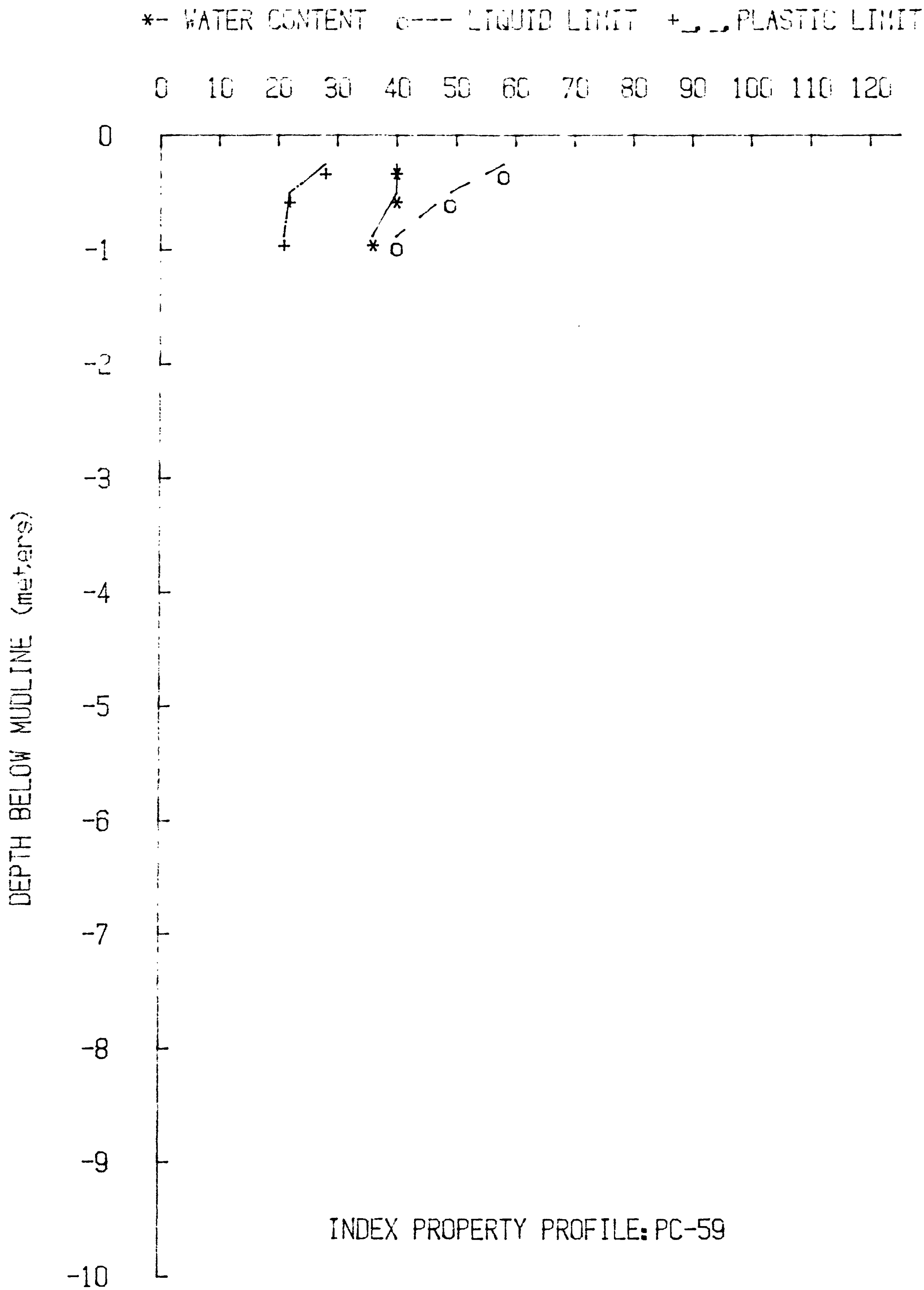


Figure 5d. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core PC59.

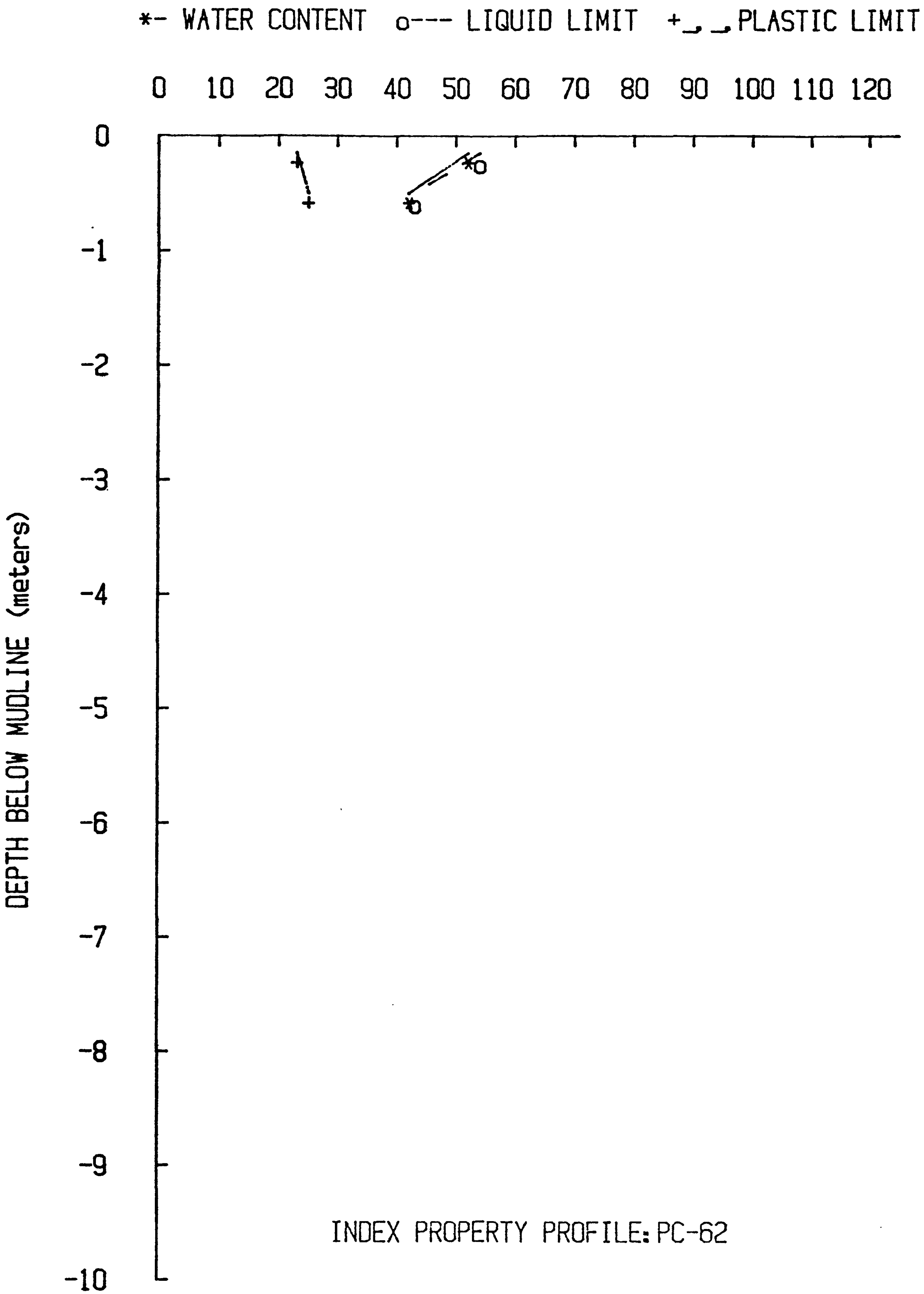


Figure 5e. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core PC62.

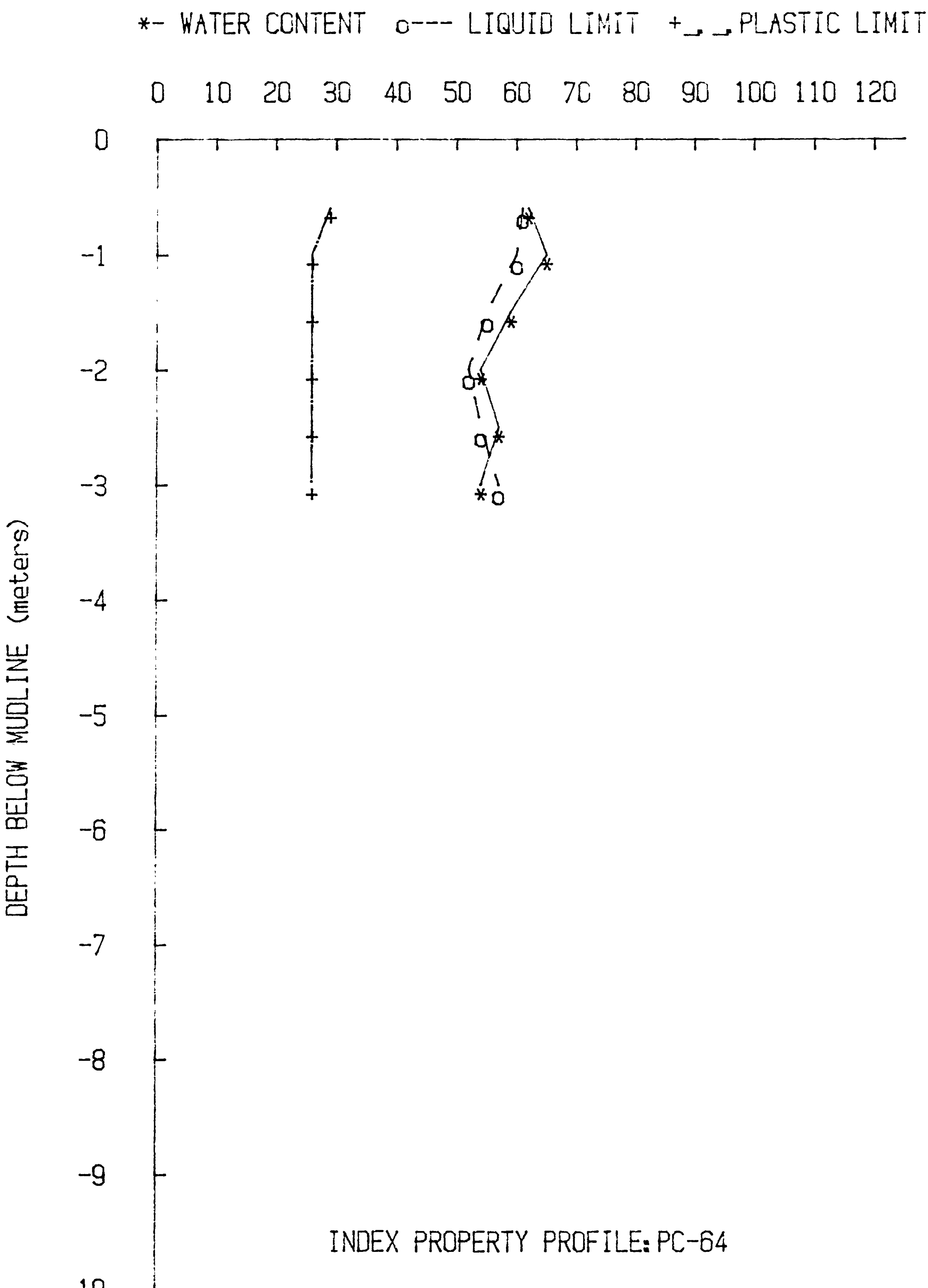


Figure 5f. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core PC64.

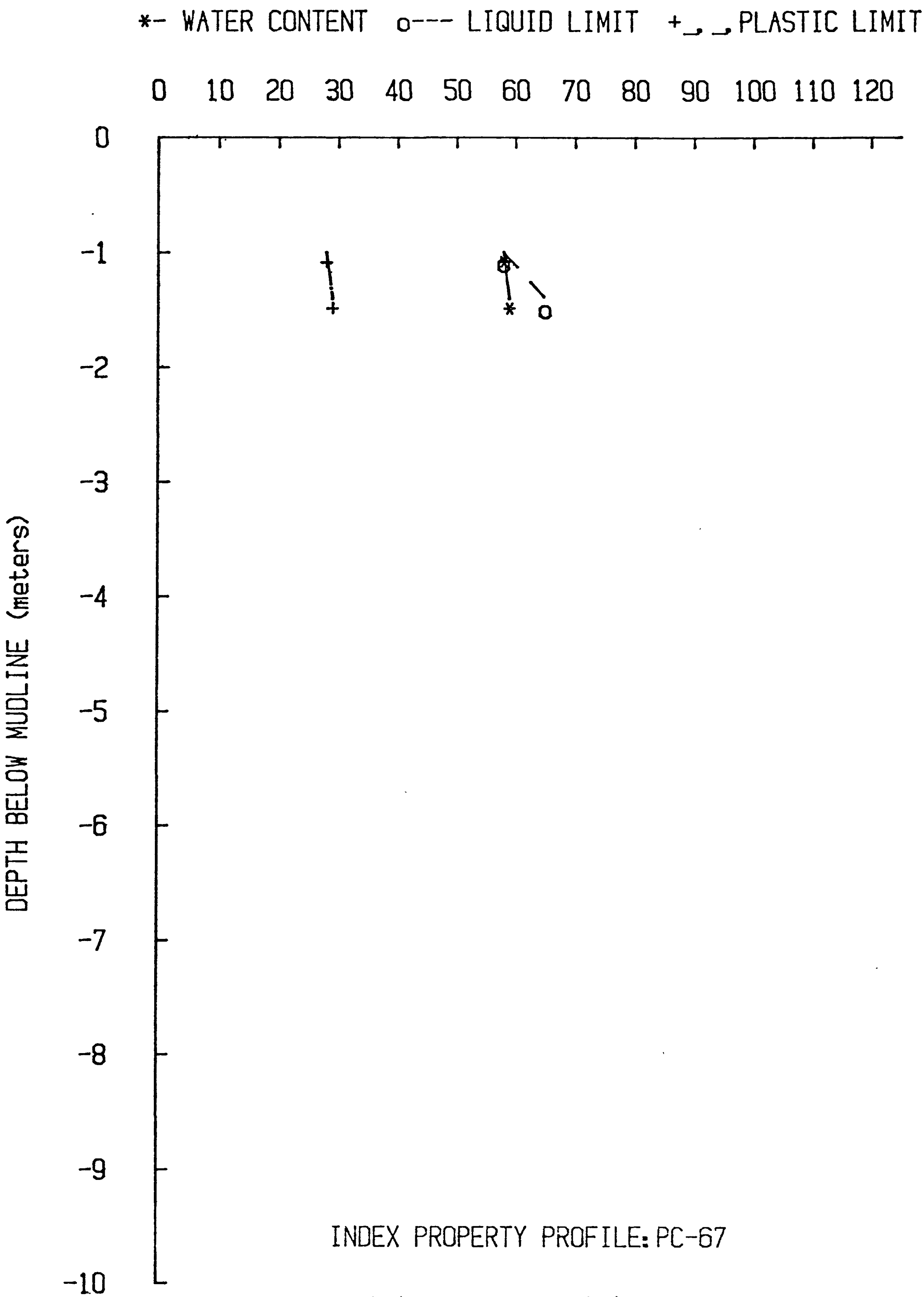


Figure 5g. Plastic limit (w_P), liquid limit (w_L), and natural water content (w) vs. depth in core PC67.

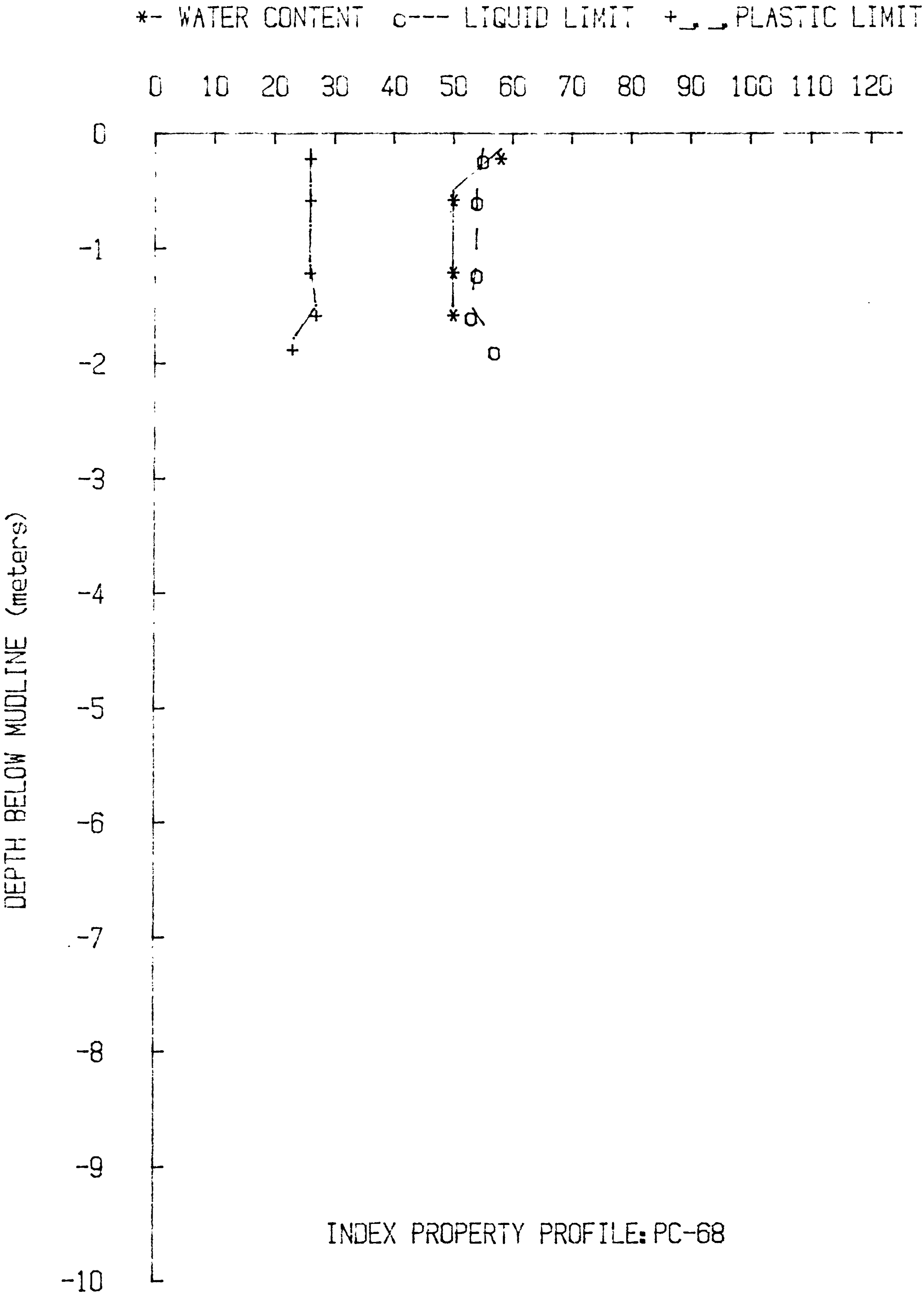


Figure 5h. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core PC68.

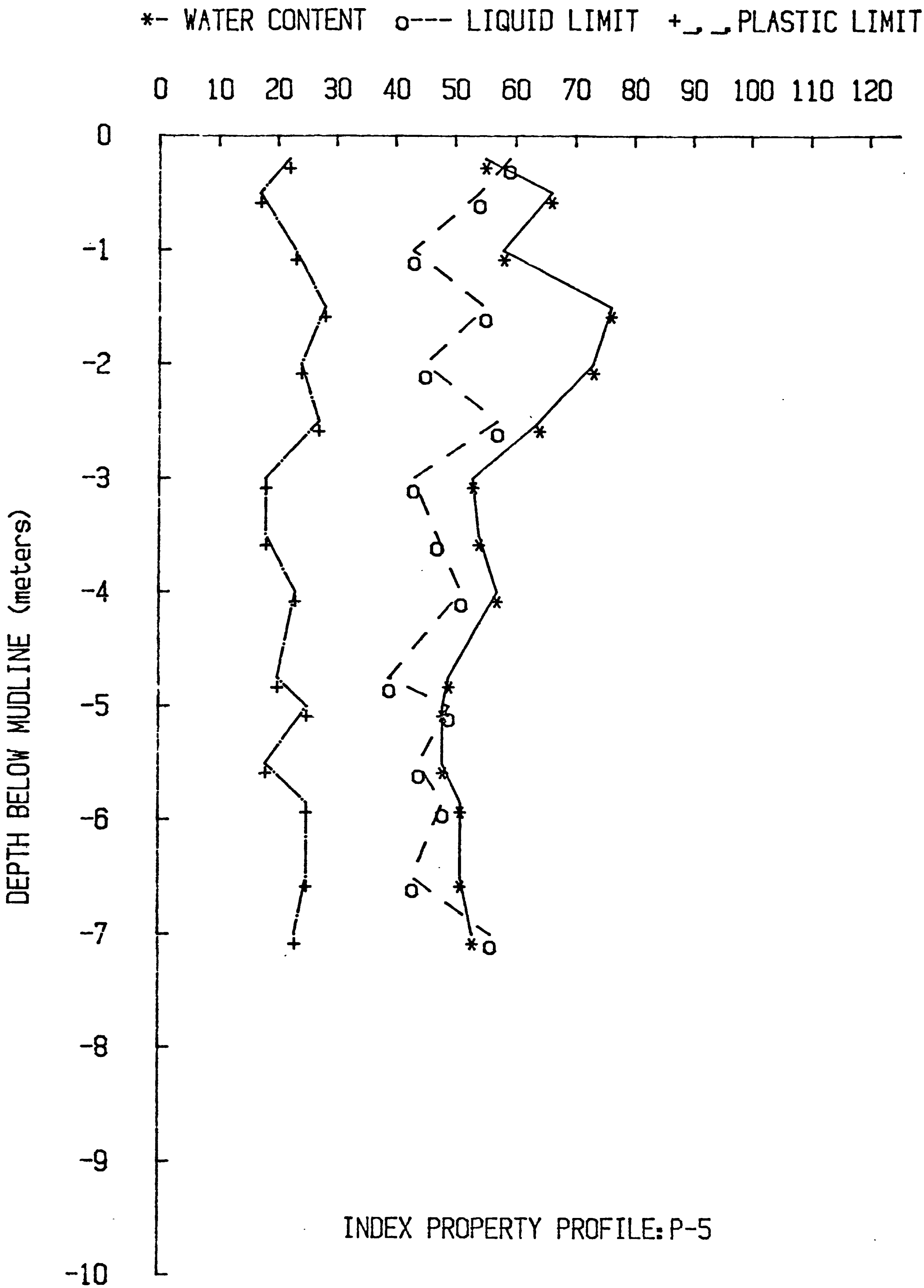


Figure 5i. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core P05.

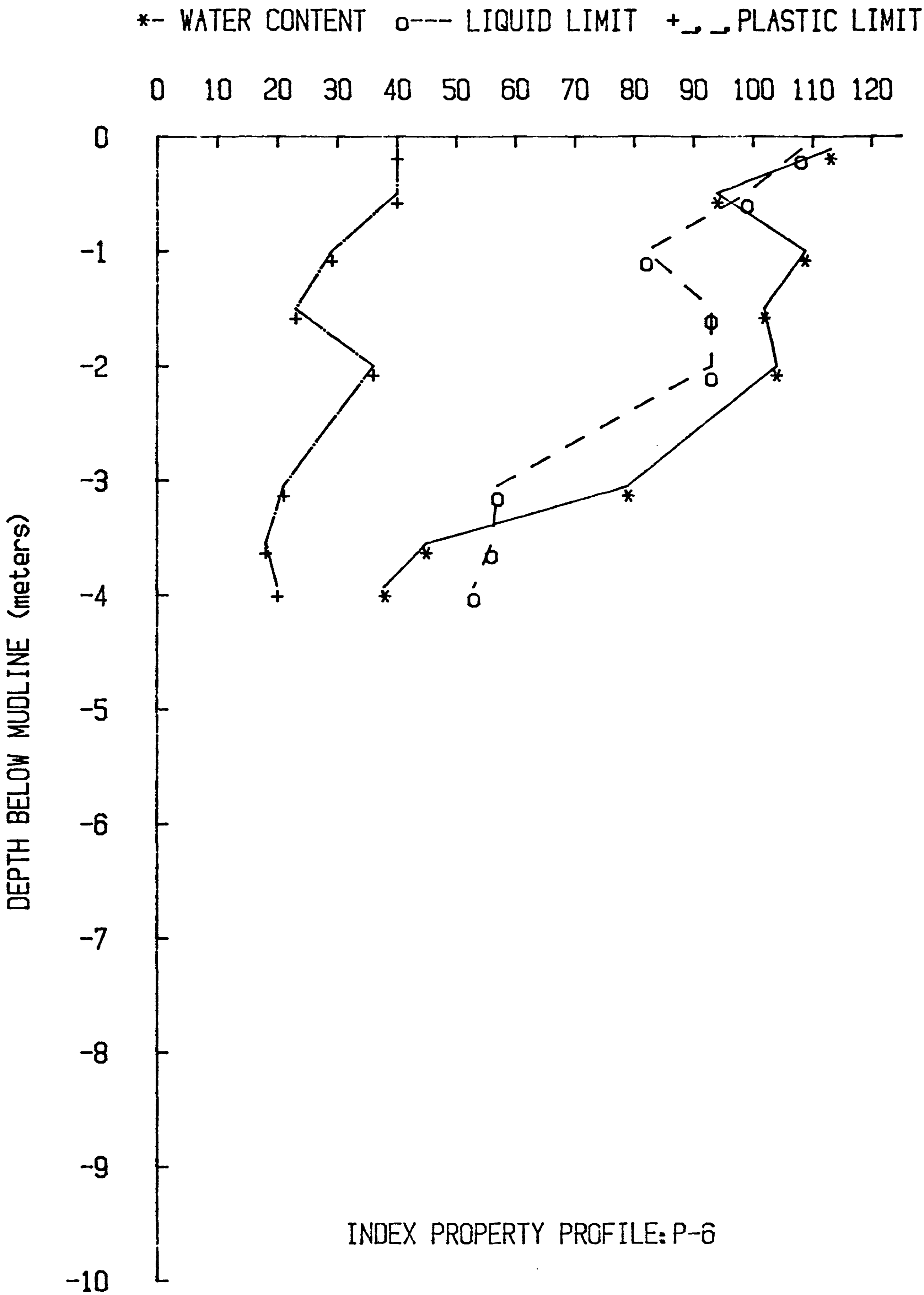


Figure 5j. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core P06.

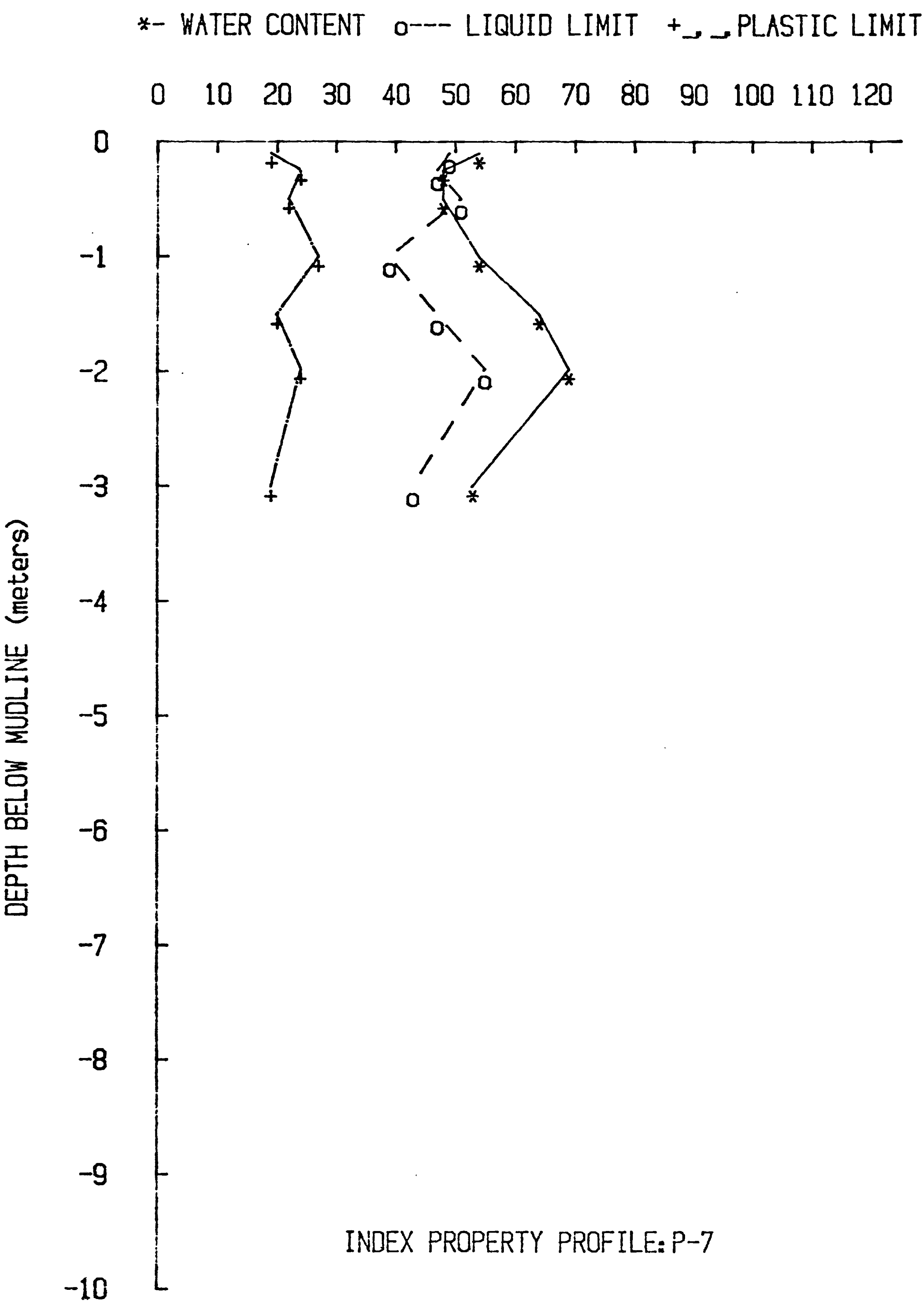


Figure 5k. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core P07.

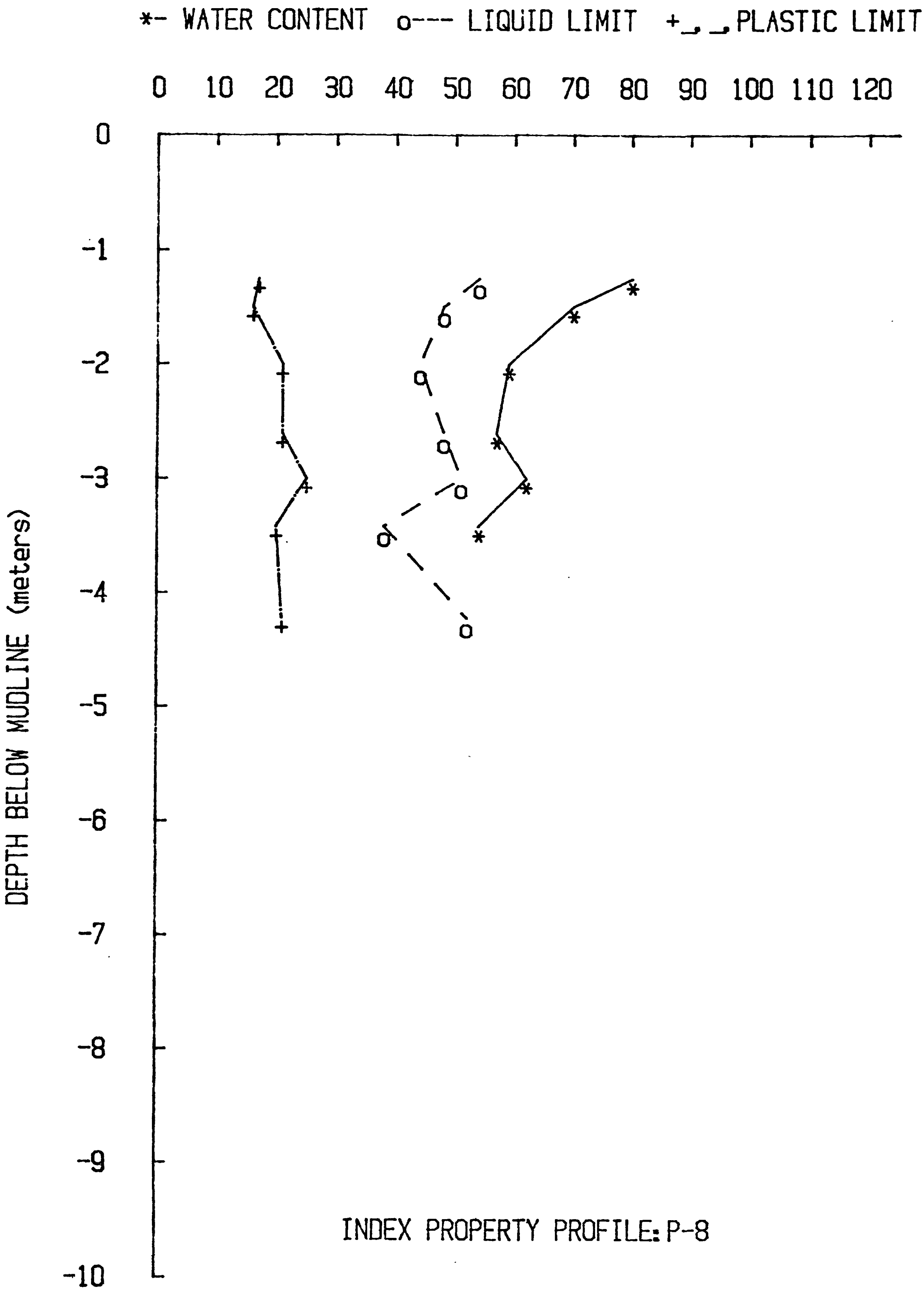


Figure 51. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core P08.

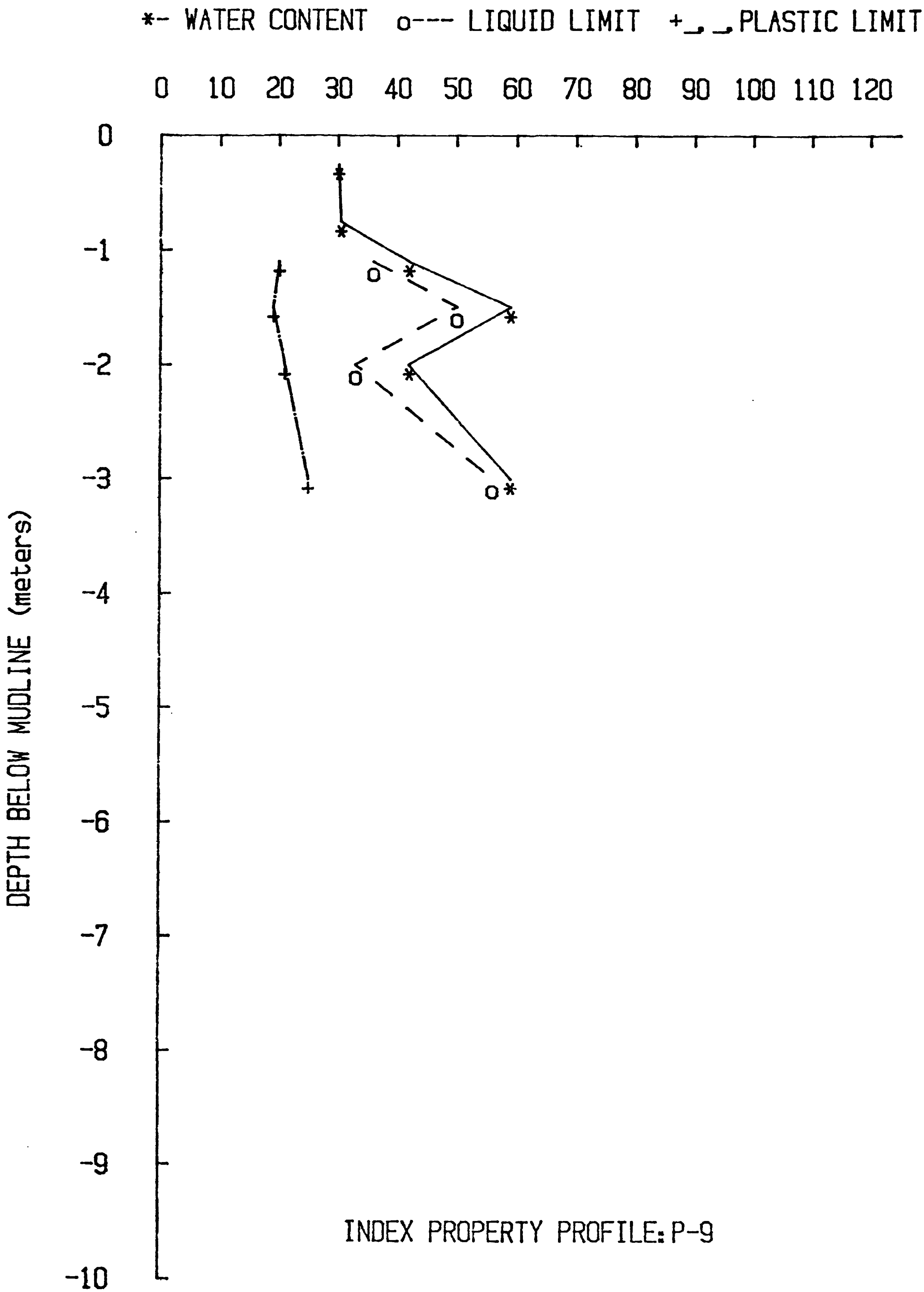


Figure 5m. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core P09.

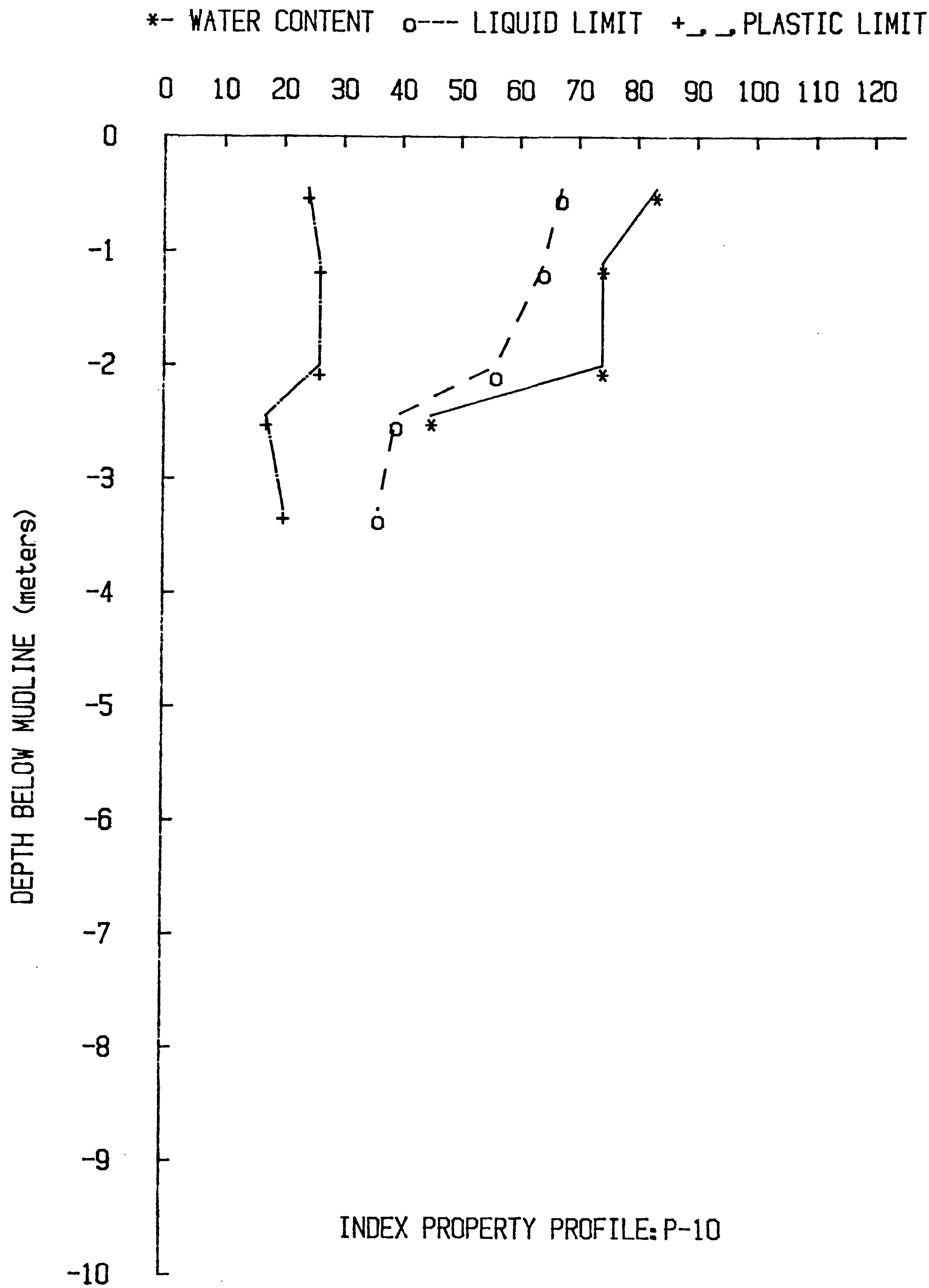


Figure 5n. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core P10.

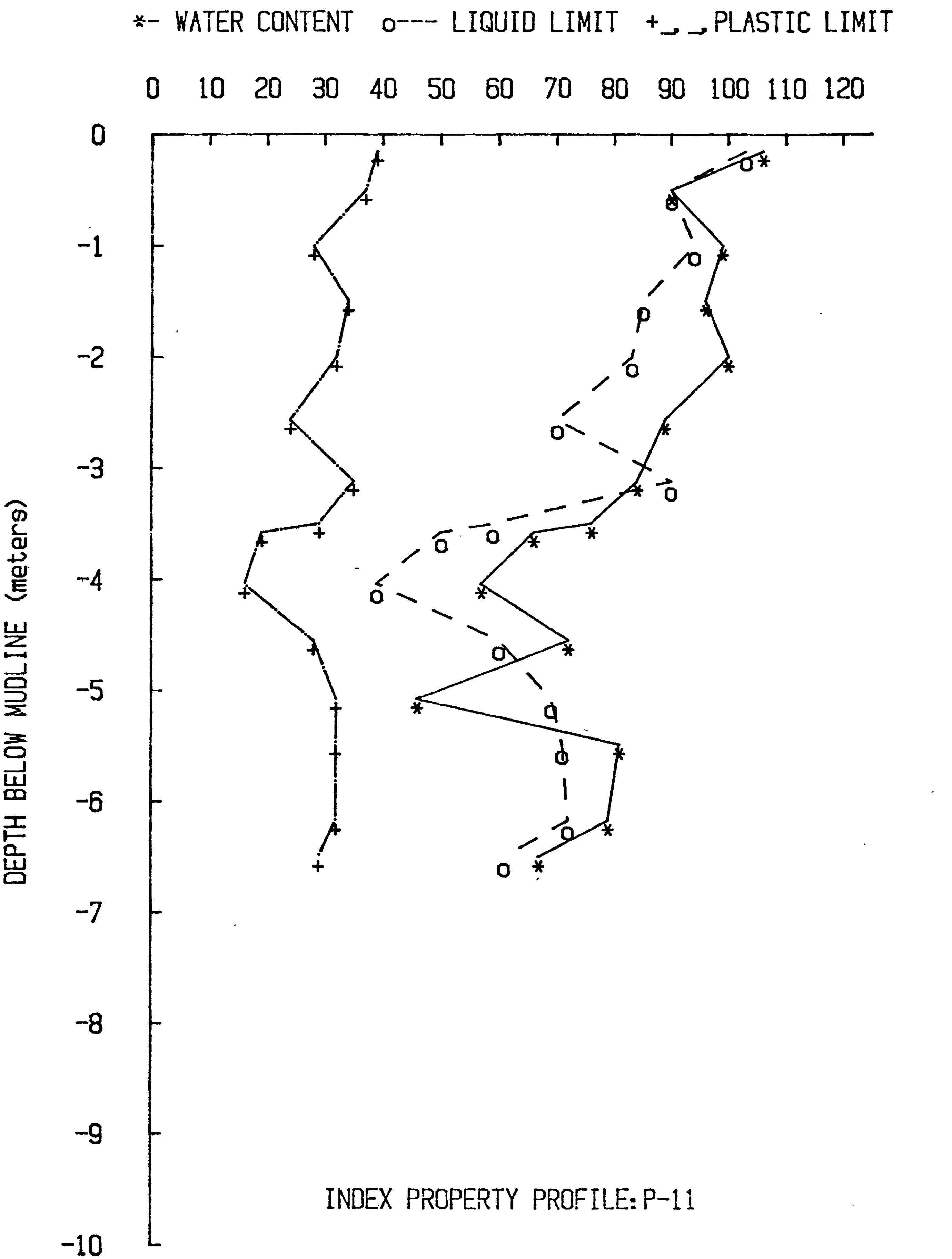


Figure 5o. Plastic limit (w_p), liquid limit (w_L), and natural water content (w) vs. depth in core P11.

short cores supports the contention that these cores were recovered from erosional areas; that is, continued increase in vertical stress from the addition of overburden during deposition eventually reduces water content to below the liquid limit. Exposing this once-buried, compacted material at the surface would thus be exposing a sediment which had a liquidity index of less than one. Ranges and mean values for each of the plasticity variables are given in table II.

Consolidation states and properties

Apparent consolidation states of the slope-rise surface were determined by calculating the overconsolidation ratio (OCR) for several core sites. OCR values are based on the ratio of the preconsolidation stress (the maximum past stress experienced by the sample) to the assumed present overburden stress. Thus, if there is excess overburden relative to the degree of consolidation, the OCR value is <1 and the sediment is underconsolidated; if the present overburden is the greatest stress experienced, OCR is 1 and the sediment is normally consolidated; and if there is insufficient overburden to cause the determined level of compaction, OCR is >1 and the sediment is overconsolidated. Table III shows that no cases of underconsolidation were found in this study. In fact, only one test yielded a result that could be interpreted as an indication of normally consolidated sediment (P08). At all other sites the sediment is apparently lightly to heavily overconsolidated. This widespread apparent overconsolidation in the upper few meters has been observed in many other marine sediments. In this case, overconsolidation resulting from normal compaction under the influence of gravity followed by erosion or mass wasting (i.e., sediment which represents true overconsolidation) may account for some of the anomalous values, but other explanations (e.g., cements, fabric, origin, cohesion, etc.) may also apply.

The compression index (C_c) is a measure of the decrease in porosity or void ratio with increase in overburden. It is largely a function of mineralogy and other compositional elements. Table III shows the compression indices for these cores. The range (0.12-0.42) is within the norm suggested by Mitchell (1976) for fine-grained soils. The range is also essentially predictable from the empirical equation suggested by Terzaghi and Peck (1967). The Continental Rise samples (P05, P06, P07, P08, P11) tend to have higher C_c values than the slope samples.

Coefficient of consolidation (C_v), which relates permeability and compression, is also largely a function of composition and, by association, texture. Table III shows C_v values for both the assumed in situ effective overburden (σ'_{vo}) and for the maximum past overburden stress (σ'_{vm}). The values, which are in the 10^{-3} to 10^{-4} cm^2/s range, are typical for fine-grained sediments (see, for example, Morgenstern 1967).

Permeability

The permeability determined for these sediments is quite varied, although generally "very low" to "practically unpermeable" according to the classification presented in Lambe and Whitman (1969). Table III shows the coefficients of permeability (hydraulic conductivity) as measured during the consolidation tests. Two of the cores (PC68, P05) display a "medium" permeability. The low values (10^{-7} to 10^{-8} cm/s) are in accord with those normally reported for soils of this texture.

Table III

Results of Consolidation Tests

Core ID	Depth in core (m)	σ'_{vo} (kPa)	σ'_{vm} (kPa)	$\sigma'_{vm}-\sigma'_{vo}$ (kPa)	OCR	C_c	$c_v(\sigma'_{vo})$ (cm^2/s)	$c_v(\sigma'_{vm})$ (cm^2/s)	$k(\sigma'_{vo})$ (cm/s)	$k(\sigma'_{vm})$ (cm/s)	I_D*	Degree of disturbance
PC02	1.70	13	117	104	9.0	.32	$7.4x10^{-4}$	$4.0x10^{-4}$	$7.0x10^{-8}$	$9.6x10^{-8}$	.44	moderate
PC64	2.20	12	45	33	3.8	.29	$8.0x10^{-4}$	$6.6x10^{-4}$	$1.7x10^{-7}$	$1.4x10^{-7}$	.42	moderate
PC68	1.00	7	266	259	38	.33	$9.6x10^{-4}$	$1.1x10^{-3}$	$1.0x10^{-3}$	$2.6x20^{-7}$	.54	much
P05	7.92	50	112	62	2.2	.36	$9.6x10^{-4}$	$5.9x10^{-4}$			.20	small
P06	4.06	23	355	332	15	.42	$1.0x10^{-3}$	$3.6x10^{-4}$	$1.3x10^{-7}$	$8.1x10^{-8}$	.35	moderate
P07	3.32	24	47	23	2.0	.29	$2.0x10^{-3}$	$1.4x10^{-3}$	$3.6x10^{-7}$	$3.5x10^{-7}$	.28	small
P08	3.61	23	35	12	1.5	.35	$9.0x10^{-4}$	$6.7x10^{-4}$	$2.1x10^{-7}$	$2.0x10^{-7}$	.21	small
P09	3.53	29	89	60	3.1	.36	$1.3x10^{-3}$	$4.4x10^{-4}$	$1.4x10^{-7}$	$1.5x10^{-7}$	.19	small
P10	2.57	16	48	32	3.0	.12	$2.9x10^{-3}$	$1.5x10^{-3}$	$6.1x10^{-7}$	$4.0x10^{-7}$	.22	small
P11	6.72	37	112	75	3.0	.30	$1.2x10^{-3}$	$9.6x10^{-4}$	$2.1x10^{-7}$	$2.5x10^{-7}$	.35	moderate

where: σ'_{vo} = Assumed in situ effective overburden stress
σ'_{vm} = Maximum past overburden stress (determined by Casagrande method)
OCR = Overconsolidation ratio
I_D = Disturbance index
C_c = Compression index
c_v = Coefficient of consolidation
k = Coefficient of permeability
* = I_D computed from methods suggested by Silva (1974)

Strength parameters

The fundamental strength parameters of the sediment, determined from CIU triaxial tests, are shown in table IV. No attempts were made to normalize or apply correction factors to these data.

The strength to overburden ratio (S_u/σ'_{vo}) for these sediments is generally higher than values predicted or assumed for normally consolidated sediment. Specifically, according to the empirical equations of Skempton (1954), Hansbo (1957), and other sources (e.g., Ladd and others, 1977) values for normally consolidated sediment typically fall between 0.2 and 0.3. Only P05, P08, P09, and P11 had values within or near this range. The other cores have considerably higher values. In effect, this corroborates the results of the consolidation tests in that general apparent overconsolidation is implied by these data. The relationship is directly evident when the OCR and S_u/σ'_{vo} values of PC02 are compared (9 and 1.13, respectively) - both are relatively high in their respective data sets (see tables III and IV).

The S_u/σ'_v values may also be used to calculate slope stability in these sediments (infinite slope, undrained case). Such calculations are presented in the next section.

Also a descriptor of sediments is the amount of strain that can be accumulated before failure occurs. Table IV shows a spread of 3-15 percent in the cores tested. Failure at these strains is common for this type of sediment, although the lowest value (3 percent), which occurred in cores P07 and P09, suggests that a less plastic material or possibly cements may occasionally be present. All failures observed in the triaxial tests were plastic in nature; that is, no discrete failure planes were observed at failure.

Cohesion (c' in table IV) is the strength of the material at zero effective stress. It represents the strength of a sediment due to interparticle attraction, cements, or other agents that are independent of overburden or associated frictional effects (see Mitchell, 1976). The range of 0 to 10 kPa shown in table IV for cohesion (c') is characteristic of fine-grained sediments.

Central to slope-stability calculations, as well as being a basic strength parameter, is the angle of internal friction (here, with respect to effective stress, ϕ'). In these sediments the mean ϕ' value is 25^{o}, which is in accord with established relationships between plasticity and ϕ' for normally consolidated clays (Kenney, 1959) or for clays which are lightly overconsolidated and thus may have slightly depressed ϕ' values (Lambe and Whitman, 1969). In addition, the values shown in table IV are within the range published by Olsen and others (1982) for slope sediments in the contiguous mid-Atlantic area. The ϕ' values of 19^{o} and 18^{o} for PC66 and P05, respectively, are abnormally low for "undisturbed" sediments. In fact, they are close to residual values which would be predicted on the basis of the sediments' plasticity (Mitchell, 1976). Deformation of the sediment is certainly possible and would tend to drive the friction angle down. The presence of organic matter and other agents could also cause a reduction in ϕ'.

Table IV

Results of Triaxial Tests

Core	Depth in core (m)	S_u/σ'_{vo}	Strain at failure (%)	c' (kPa)	ϕ' (^{o})
PC02	2.92	1.13	8	7	27
PC18	3.93	.77	6	10	23
PC59	1.37	1.10	15	2	28
PC66	0.30	1.60	12	8	19
P05	7.66	.18	9	3	18
P06	2.59	.43	6	3	27
P07	3.64	.44	3	1	31
P08	3.96	.30	5	3	23
P09	3.96	.26	3	4	22
P10	1.50	1.0	8	3	24
P11	7.23	.18	4	0	33

S_u/σ'_{vo} = Ratio of undrained shear strength to effective overburden pressure
c' = Cohesion (effective stress)
ϕ' = Angle of internal friction (effective stress)

MASS MOVEMENT

Abundant evidence for past mass movements in the Georges Bank area raises questions concerning the likelihood of such activity in the future and the circumstances under which slope failures would occur. The documentation of past events from high-resolution seismic-reflection profiles and sidescan-sonar images, while pertinent for identifying features and establishing geometry and types of failures, does not provide a quantitative basis for addressing these questions. Nor do core descriptions, which have shown evidence of possible mass movement at sites PC57 and P06, for example. Slope-stability analysis does lend itself to such questions on future slope failures: It not only yields information on the potential for mass movement, but permits evaluation of the impact of the several geologic factors which control slope stability. Thus, it helps to establish the level of activity of geologic processes in the area. In addition, slope-stability analysis represents an approach capable of providing quantitative information pertinent to commercial development of the region.

There are numerous methods (covering a variety of conditions) available for analyzing slope stability. Selection of a basic method along with its variations depends largely on the geology, the environment, the stress history, a postulated failure surface, general geometry, and a postulated failure type--as well as on available data and the level of accuracy required. Figure 6 gives an idea of the selection process and some of the decisions which must be made. Inasmuch as we will concentrate on regional slopes (i.e., noncanyon areas) in this report, local slope methods, such as Bishop's Simplified Method of Slices and Wedge Analysis, were not considered. These methods, because of their applicability to canyons and canyon areas, will be used in a companion report. It is noteworthy that over one-half, and as much as two-thirds, of the slope area between Alvin and Powell Canyons (fig. 1) is occupied by canyon systems (K. M. Scanlon, oral commun., 1983). In addition to the regional slope requirement, we assume for simplicity that the potential failure surface would be planar and oriented parallel to the plane of the Continental Slope surface and that a slide derived from this geometry would therefore be translational. Finally, the method chosen should be versatile and uncomplicated. The basic infinite-slope method meets each of these requirements and thus was chosen for use in this study.

The infinite-slope model is shown in figure 7. As implied by the force polygon, it is an expression of balance between resisting forces and shearing forces. The ratio between the two forces is the factor of safety (F) against slope failure: F=resisting forces/shearing forces, where F>1 indicates stability, F<1 indicates instability, and F=1 indicates limit equilibrium. Two equations may be used to analyze static conditions (dynamic conditions will be discussed subsequently):

$$\text{Drained case:} \quad Fd=(1-\mu e/\gamma' z \cos^2\alpha) \tan\phi'/\tan\alpha \qquad (1)$$

$$\text{Undrained case:} \quad F_u=[S_u/\sigma_{vo} 1-\mu e/\gamma' z)]/\sin\alpha\cos\alpha \qquad (2)$$

In the equations μe is excess pore pressure (i.e., pore pressure in excess of hydrostatic pressure), γ' is the buoyant (submerged) unit weight of the

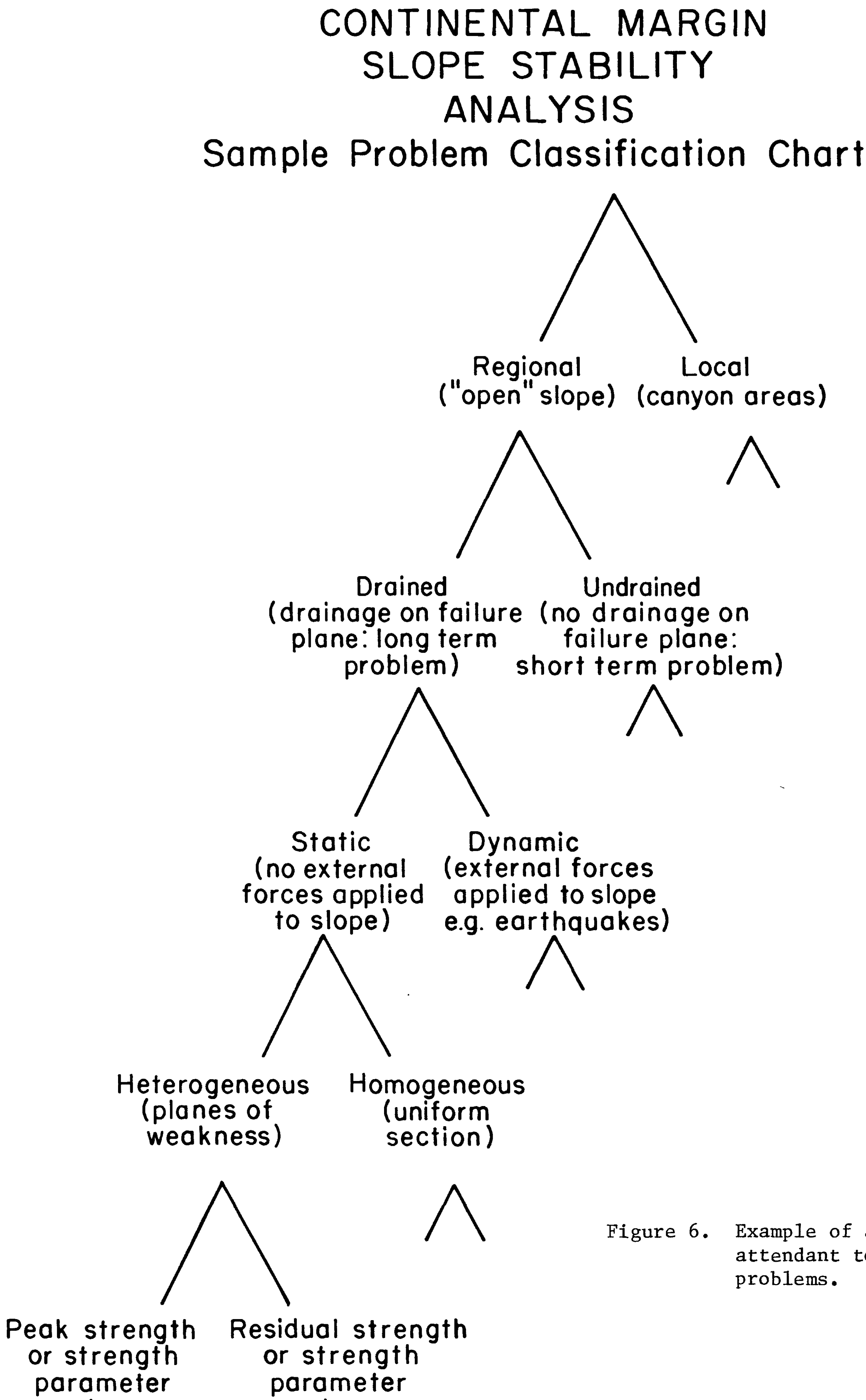
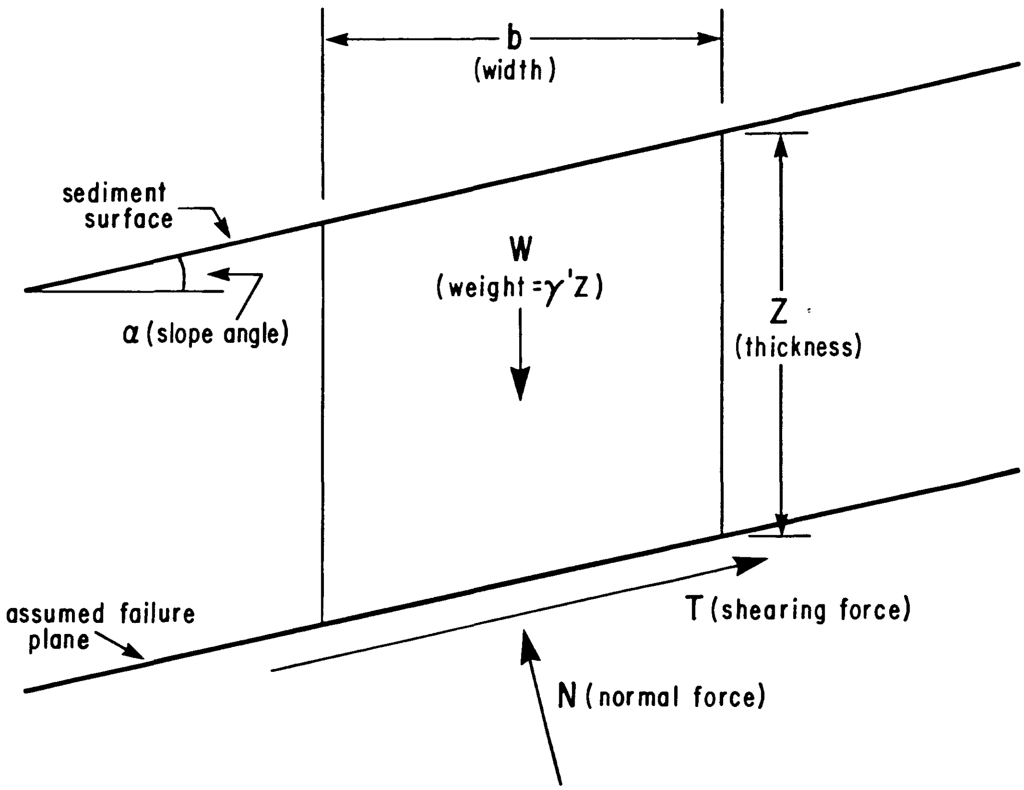


Figure 6. Example of analysis options attendant to slope-stability problems.

stability analysis of a slice of submerged "infinite" slope



Forces:

$W = \gamma' b z$ (where γ' is buoyant unit weight)

$N = W \cos \alpha$

$T = W \sin \alpha$

Stresses:

$$\bar{\sigma} = \text{normal stress} = \frac{N}{b/\cos \alpha} = \gamma' z \cos^2 \alpha$$

$$t = \text{shear stress} = \frac{T}{b/\cos \alpha} = \gamma' z \sin\alpha \cos\alpha$$

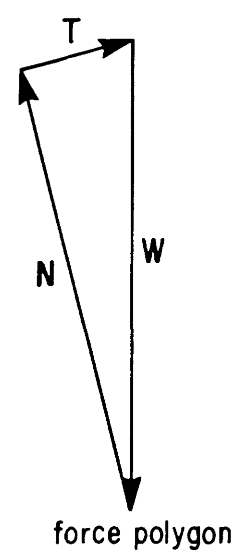


Figure 7. Infinite-slope model for stability analysis.

sediment, z is the sediment thickness under consideration, α is the slope angle, ϕ' is the angle of internal friction with respect to effective stress, S_u is the undrained shear strength, and σ'_{vo} is the effective overburden stress. The derivation of these equations may be found in many soil mechanics publications (e.g., Lambe and Whitman, 1969; Morgenstern and Sangrey, 1978).

The infinite slope has been used widely in marine sediment investigations (see, for example, Almagor and Wiseman, 1977; Hampton and others, 1978; Booth and Sangrey, 1979; Keller and others, 1979; Booth and others, 1981a, 1981b; Sangrey and Marks, 1981). As in this study, its use was largely dictated by the objectives of the study along with practical considerations. Because its use is becoming commonplace, it is worthwhile at this time to review the assumptions and common limitations associated with the method. Infinite slope-stability analysis, as well as other methods of limit equilibrium analysis, only can be used to address a certain class of problem: discrete slope failure. It does not apply to other types of mass wasting nor does it account for or predict slope deformations. In addition, several assumptions are attendant to its application. Among the more salient of these are:

* It assumes that the lateral extent of the slide is infinite in comparison to its thickness: edge effects are insignificant.

* It assumes that the sediment peak strength will be mobilized across the entire failure surface at the time of failure.

* It assumes that the failure surface is a plane, and this plane is parallel to the slope surface (note that in cases where bedding plane failure is under investigation, regional dip must be equivalent to slope declivity).

* It assumes for drained analysis that pore pressures are known at the failure surface or, in the case of undrained analysis, that pore pressures measured in a triaxial test may be validly extrapolated to the field situation.

Certainly for the level of accuracy required by reconnaissance research, these assumptions are reasonable and, in fact, often necessary. However, in practice, an unknown amount of error is introduced into the final result (the factor of safety) because these assumptions have been made. Finally, constraints related to these assumptions are imposed by sampling methods and the area of investigation itself. As mentioned, core site selection in this study was biased and the cores have been mechanically disturbed to some unknown degree. Further, because of the limited penetration, application of the infinite-slope model to a few tens of meters (a typical thickness of observed slide masses) requires that the sediment is homogeneous or predictable with respect to measured strength properties.

Geologically related constraints are also imposed. Variable slope morphology, the presence of gassified sediments, the effect of sea-level change and other aspects of depositional history on the sediment section, as well as numerous other factors must be taken into account in order to apply the model on a regional basis.

In light of the basic objective, which is to conduct a preliminary assessment of regional slope stability with the data available, the infinite slope method is acceptable. However, the assumptions and limitations that go with it must be kept in mind when considering the results.

The factors of safety for static drained and undrained conditions are shown in table V. Drained conditions are traditionally assumed when instability develops over long-time periods; that is, long enough such that there is no change in pore pressure along a potential failure surface. Generally, this applies to conditions of moderate to slow rates of loading from sediment deposition, gradual oversteepening, and other analogous processes. The undrained case is assumed when instability arises over short time periods; that is, shearing will take place too rapidly for drainage (preventing a buildup in pore pressure) to occur. Examples of applicable conditions include rapid undercutting or oversteepening, deltaic deposition rates, and dynamic loading, such as from earthquakes. Probably in no case is drainage complete or nonexistent, with the exception of, perhaps, earthquakes, so that the two types of factors of safety shown in table V represent end members for this general class of problem.

For the static case, and assuming no excess pore pressures, the slope and rise are apparently stable. The values shown in table V indicate that only the site of P09, which is near Welker Canyon, has a drained or undrained factor of safety low enough to warrant further investigation. The widespread presence of apparently overconsolidated sediment, because of its relatively high undrained shear strength, skews the results toward higher values and thus has a significant effect on the undrained factor of safety. It is not known whether the apparent overconsolidation that is not associated with erosional surfaces is representative of consolidation states farther down into the sediment column. If not, and if those sediments were more normally consolidated, the factors of safety would be reduced.

In general, the highest values shown in table V are a product of gentle slope gradients (e.g., P11 at $<1^{\circ}$ and PC02 at $\sim 2^{\circ}$), which underscores the importance of slope angle in these calculations.

Given the results of the static, no-excess-pore-pressure case, it is instructive to consider how these results can be modified if other assumptions or conditions are imposed. This not only shows the sensitivity of the factors of safety, but provides an avenue of evaluating the impact of geologic conditions or processes. Specifically, what would be the effects of dynamic loading? of excess pore pressure? of increased slope angles? of having less than peak strengths available?

Dynamic loading can occur through a variety of processes but, inasmuch as the area of investigation is below wave base for major storms and data are lacking on internal wave forces, we have restricted ourselves to earthquakes in this analysis. Earthquakes affect slope stability in two ways: by increasing shear stress through ground accelerations and by reducing shear resistance because of possible resultant elevated pore pressure. Only the former effect was considered in this analysis. Equations for ground accelerations have, for example, been published by Hampton and others (1977) and Sangrey and Marks (1981). In general, both drained and undrained stability equations may be streamlined by accounting only for horizontal accelerations (for discussion, see Booth and others, in press).

Table V

Static Factors of Safety

(Infinite slope model)

Core	Drained	Undrained
PC01	9.9	18.4
PC02	22.6	54.0
PC59	8.0	16.6
PC66	3.1	14.9
P05	4.9	2.8
P06	12.9	6.2
P07	12.9	9.4
P08	9.2	7.5
P09	2.0	1.4
P10	2.2	5.2
P11	45.8	57.3

The results of the calculations are shown in table VI. The values presented are the horizontal ground accelerations (%g) needed to reduce the static factors of safety to one. A glimpse of the meaning of these values can be garnered from the work of Seed and others (1975), who imply that accelerations from a 6.5 m_b earthquake would probably not exceed 5%g at a distance of 100 km from the energy source and would probably not exceed 10%g at a distance of 50 km from the source. Offshore and other recent earthquake epicenter locations for the northeastern United States have been published by Yang and Aggarwal (1981). Some epicenters on the Outer Continental Shelf appear to be within 100 km of the study area, including sites near Hudson Canyon and in the general vicinity of the Lydonia and Powell Canyons region. Magnitudes determined thus far have been small (m_b ~4), however. Nonetheless, the sites of cores P05, P09, and P11 would seem somewhat vulnerable to the effects of a proximal earthquake and the sites of P66 and P10, perhaps marginally vulnerable.

Dynamic loading, rapid deposition, artesian systems, and the presence of interstitial gas can all elevate pore pressures and, hence, reduce factors of safety. Table VI shows what excess pore pressure values (expressed as percent of overburden pressure) would be required to reduce the static factors of safety to a value of one. Note that with the exception of a few cases, the pore pressures would have to support almost the entire sediment column before the effect would become important. It is doubtful that excess pore pressures due to rapid deposition could approach the necessary values, even at increased rates of deposition during lower sea levels. Deltaic deposition rates would be required to achieve the level needed (Morgenstern, 1967). Excess pore pressure caused by groundwater flow would also seem incapable of instituting such changes. Salinity measurements made during the USGS Atlantic Margin Drilling Project on this section of the continental margin (Hathaway and others, 1979) indicate an absence of fresh, or even brackish, water in the subsurface down to about 300 m below the shelf break, and Manheim and Hall (1976) show data from a deep boring which indicate an absence of low-salinity interstitial water to 1,500 m subsurface (2,500 m below sea level). Gas has been reported from direct measurements in the area (Hathaway and others, 1976) and has been tentatively indentified in specific areas on high-resolution seismic-reflection profiles (D. W. O'Leary, oral commun., 1983). It might increase pore pressure levels to those shown in table VI if gas were present in concentrations above solubility levels. The occasional seismic event in the area may also cause an increase in pore pressure, although application of a model developed by Egan and Sangrey (1978) to this case suggest they probably would not reach the general level required. However, the increased shear stress (ground acceleration) and excess pore pressure which result from earthquakes may be a potent combination for decreasing slope stability if magnitude, distance, and duration criteria are met.

Increased slope angles would also reduce factors of safety. In general, angles must be increased 20° or so, although some sites (e.g., P09) would only require 4° of additional declivity. Depositional oversteepening, undercutting (which may be viewed as increasing slope angle), and tilting (by general tectonic or diapiric activity) may cause such a change. Keller and Shepard (1978) have measured currents in excess of 70 cm/s near Alvin Canyon--a velocity great enough to erode (undercut?) a slope, and Southard and Cacchione (1972) have implied that internal waves, if present, may be capable of eroding (undercutting?) at a fairly well defined depth level over a broad region.

Table VI

Geologic conditions or changes required to bring core site to limit equilibrium

(i.e., to reduce factor of safety to 1)

Core	Basic factors of safety		Ground acceleration required (%g)		Excess pore pressure required ($\%\sigma_{vo}$)		Change in slope angle	
	F_d	F_u	d	u	d	u	d	u
PC01	9.9	18.4	19	29	90	95	20°	*
PC02	22.6	54	30	44	96	98	26°	*
PC59	8.0	16.6	20	43	87	94	24°	*
PC66	3.1	14.9	10	60	68	93	12°	*
P05	4.9	2.8	10	5	79	64	14°	7°
P06	12.9	6.2	17	14	92	91	25°	28°
P07	12.9	9.4	23	15	92	89	29°	4°
P08	9.2	7.5	13	10	91	87	21°	16°
P09	2.0	1.4	9	3	48	26	10°	4°
P10	2.2	5.2	10	33	54	81	13°	*
P11	45.8	57.3	22	6	98	92	32°	10°

F_d = Static factor of safety - drained case
F_u = Static factor of safety - undrained case
g = Gravity
σ_{vo} = Overburden stress
* = Factor of safety >1 at 45° slope angle

However, MacIlvaine and Ross (1979) found little evidence of erosion in the general vicinity of the Keller and Shepard study area and data are lacking with regard to internal waves. Depositional oversteepening (particularly during lowstands of the sea) and undercutting may be more important as local processes. Tilting due to regional tectonic activity or diapirism, even if occurring at a very slow rate in this tectonically "quiet" area, has not yet produced the slope angles necessary for regional instability.

The effect of reduced strength on the factors of safety is a final point in this discussion. Static factors of safety (table VI) were computed assuming that peak strengths were available. However, the strength or strength parameters may be altered through strain-softening or as a result of past slope failures. Elastic rebound, dynamic loading, or creep, for example, can reduce the appropriate strength properties considerably, as implied by the sensitivities; that is, the remolded strength averages 1/5 the peak stength in these sediments. Further, jointing, and hence the development of weak planes, can develop during elastic rebound of overconsolidated sediments. And residual friction angles, which are often manifest along failure planes, are frequently 10-20^{o} less than the initial friction angle for sediments of the plasticity reported here (Mitchell, 1976). What percentage of the region, if any, is appropriate for application of this type of stability analysis is unknown, although slab-type failures have been observed (D. W. O'Leary, oral commun., 1983). Intuitively, however, we believe that local areas (such as those near or within canyons) may be more appropriate areas in which to apply this type of analysis because of the complex morphologies, the often steeper slope angles, and the more complicated stress histories they probably represent in comparison with regional conditions.

Because of the overall complexity of this continental margin surface, because the "local" conditions are superimposed on the "regional" conditions, and because of the presence of numerous, yet unquantified, geologic processes, the slope stability assessment presented herein must be considered reconnaissance in nature.

It is obvious, however, from the preceding analysis that the area does not lend itself to generalizations. The simple question "Is the slope stable?" cannot be answered simply. Conditions, assumptions, and specific areas must be defined and pertinent data must be available before reasonably precise answers can be given.

SUMMARY

The U.S. North Atlantic Continental Slope has experienced widespread mass movement. High-resolution seismic-reflection profiles and sidescan-sonar images have shown evidence of rotational and translational slide scarps, toreva blocks, debris flows, rubble fields, allogenic blocks, and other manifestations of slope failure. Despite this evidence, the cause or causes of these events have not been determined. Geologic mechanisms that could be responsible are present within the area, however, including external (e.g., earthquakes) and internal (e.g., excess pore pressures) processes and agents. It was our purpose to conduct a preliminary analysis of the causes of the past slope failures and to determine if these causes are likely to promote further slope failures. Because sediment geotechnical properties are a necessary part of such an analysis, they formed the data base for the study.

In addition, because these data are useful for establishing a framework for stability analysis, a geotechnical characterization of the area was also a part of the investigation.

Reflecting the interplay between numerous geologic processes, the sediments of the North Atlantic Continental Slope and Rise display a marked geotechnical variability. Underscoring this, undrained shear strength ranges over two orders of magnitude, plasticity ranges from low to high, sensitivities range from insensitive to slightly quick, and the sediments may be normally consolidated to heavily overconsolidated. Spatial and temporal changes in texture along with the presence of both depositional and erosional areas are the direct cause of the variability.

Despite the noted ranges in properties, the sediment can be characterized. Typically, they belong in the category designated "CH" by the Unified Soil Classification System and have a soft consistency. Further, they are very sensitive and have very low permeabilities. In general, they have most of the geotechnical properties associated with a fine-grained sediment dominated by illite and less active minerals.

Excluding areas of canyons and canyon systems, results of stability analyses show that for static drained and undrained conditions, the slope is generally stable, despite having declivities greater than 10° in some locales. However, some sites would be vulnerable if subjected to relatively minor earthquake-induced ground accelerations, to excess pore pressures from gas or other sources, or to slight increases in gradient from oversteepening or undercutting. Thus, any of these possibilities may have caused mass movements in the past at specific sites. Finally, it appears that most processes or agents which may cause instability in this geologic setting have the potential to be more important locally than regionally, with the possible exception of earthquakes. More focused and detailed research will be needed before the relative efficiency or future effects of the different possible causes may be evaluated because relative magnitudes, frequencies, and extents have not yet been fully documented or quantified.

ACKNOWLEDGMENTS

This research was funded in part from Minerals Management Service under Memorandum of Understanding AA851-MU0-18 and Interagency Agreements AA851-IA1-17 and AA851-IA2-26. We are also grateful to the captains and crews of the R/V ENDEAVOR for their assistance during sampling operations and to Richard Sterling (Bates College) and Kathy Andrews (Oberlin College) for their voluntary laboratory assistance. Dennis O'Leary, Homa Lee, Hal Olsen, and Harley Knebel contributed many useful suggestions and comments.

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