72-22

See Rpt.

Private Fuel Storage, L.L.C.

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Phone 303-741-7009 Fax: 303-741-7806
John L. Donnell, P.E., Project Director

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D.C. 20555-0001

June 30, 1999

SUBMITTAL OF COMMITMENT RESOLUTION LETTER #7 INFORMATION
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

Reference: 1. PFS Letter, Donnell to Delligatti, Submittal of Commitment Resolution # 4 Information, dated May 28, 1999.
2. PFS Letter, Donnell to U.S. Nuclear Regulatory Commission, Commitment Resolution Letter # 7, dated June 24, 1999.

In reference No. 2 Private Fuel Storage (PFS) committed to provide additional information and clarification to NRC/CNWRA comments regarding the Private Fuel Storage Facility (PFSF) geotechnical program and the potential for impact of aircraft and air-delivered ordnance at the PFSF.

The requested information is provided in two enclosures. The first enclosure provides the requested information on the geotechnical program while the second enclosure addresses the potential for impact of aircraft and air-delivered ordnance at the PFSF.

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely,

John L Donnell

John L. Donnell
Project Director
Private Fuel Storage L.L.C.

d90023

Copies to:
Mark Delligatti
John Parkyn
Jay Silberg
Sherwin Turk
Asadul Chowdhury
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Scott Northard
Denise Chancellor
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John Paul Kennedy
Joro Walker

ENCLOSURE NO. 1 – GEOTECHNICAL PROGRAM
COMMITMENT RESOLUTION LETTER NO. 7 – JUNE 30, 1999

NRC Comment –

Item 1: The cone penetrometer testing (CPT) data that PFS submitted on May 28, 1999 (ConeTec, 1999) indicates that the soil classifications based on the CPT data include sandy silts, silty sands, and sands in the upper 25 to 30-ft layer, where PFS previously has reported that this layer consists of uniform silt, silty clay, and clayey silt. PFS should develop a map showing the areas where the sandier soils exist.

Item 2: PFS should demonstrate that the revised profiles in the areas identified in Item 1 have an adequate factor of safety against a bearing capacity failure.

Item 3: PFS should use the CPT data to calculate settlements based on Equation 6-16 or 6-17 of Lunne, Robertson, and Powell (1997), which were developed by Meyerhof and Schmertmann, respectively. These analyses need to include discussion of differential settlements and the implications thereof on the casks.

Item 4: PFS should explain why the shear wave velocities reported in Appendix C of the CPT report (ConeTec, 1999) increase with depth to about 10 to 15 ft and then level off for the remainder of the upper 25 to 30 ft in the profile.

Item 5: PFS should explain why the plots of "PHI" in Appendix G of the CPT (ConeTec, 1999) report do not agree with the values of "PHI" shown in the tables of dilatometer data in Appendix H of that report.

PFS Response –

Item 1: The cone penetrometer testing (CPT) data that PFS submitted on May 28, 1999 (ConeTec, 1999) indicates that the soil classifications based on the CPT data include sandy silts, silty sands, and sands in the upper 25 to 30-ft layer, where PFS previously has reported that this layer consists of uniform silt, silty clay, and clayey silt. PFS should develop a map showing the areas where the sandier soils exist.

The purpose of the request that PFS provide a map showing the areas where the sandier soils exist was to readily identify those areas where the subsurface profile differs from the assumption that the soils in the upper layer (~25 to 30 ft) are predominantly cohesive. A review of the CPT data (ConeTec, 1999) indicates that most of the soil behavior type (SBT) values represent soils whose behavior is similar to that of "sandy" soils. As indicated in Figure 5 of ConeTec (1999), these include SBT values that are greater than 5. Therefore, to respond to Item 1, PFS generated a map showing the thickness of those soils for which the behavior type values are greater than 5.

This map, titled "Contour Map Showing Thickness of Soils with CPT Soil Behavior Type > 5 (Sandy)" is included as Attachment 1. The thickness of the soils beneath the cask storage pads that behave as "sandy" soils based on the CPT data are posted under the CPT identifiers shown on this plan view of the site. These values were calculated by subtracting the top three feet, to account for the proposed depth of the pads, as well as the total thickness of all zones where the SBT value was found to be less than 6, from the total depth of the CPT. The thicknesses were contoured to facilitate interpretation of the SBT > 5 data obtained in the CPT program. As indicated in the figure, the thickness of the soils that behave as sandy soils (SBT>5) based on the CPT data ranges from 13.8 feet at CPT-15, near the center of the pad emplacement area, to a high of 26.4 feet at CPT-33 near the center of the western edge of the pad emplacement area. The thicknesses are generally about 20 to 25 feet.

The variability of these different soil behavior types across the site are illustrated in the subsurface profiles included in Attachment 2. The figure titled "Cone Penetration Tests Profile Location Plan," also included in Attachment 2, identifies the locations of these profiles. The graphical logs presented on these profiles are those presented in Appendix E of ConeTec (1999), plotted vs elevation. The soil behavior types are plotted in different colors, as indicated in the legends included on these plots. These soil behavior types also are plotted to scale along the horizontal axes in these profiles, in accordance with the "zones" identified in Figure 5 of ConeTec (1999), to further identify the different soil behavior types.

Item 2: PFS should demonstrate that the revised profiles in the areas identified in Item 1 have an adequate factor of safety against a bearing capacity failure.

As discussed in Item 1 above, based on the CPT program, most of the soils underlying the pad emplacement area are characterized as soils that behave as "sandy" soils, rather than as cohesive soils. These soils were found to be mostly cohesive soils in the borings that were drilled in 1996, as indicated in SAR Appendix 2A. Whereas the bearing capacity of cohesive soils is a function of the strength of the soil, that of cohesionless soils is also a function of the width of the foundation. The foundations in question for this project have widths that are greater than 30 ft. Such large foundations, supported by soils having Standard Penetration Test blow counts that were measured for these soils, have much greater bearing capacities if they are founded on cohesionless soils than if supported by cohesive soils. Therefore, characterizing the soils in the upper layer as cohesive even though some of these were cohesionless provides a conservative estimate of the bearing capacity.

Analyses of bearing capacity were made in Calculation 05996.01-G(B)-4, Rev 3, *Stability Analyses of Storage Pads*, assuming the upper layer was cohesive. In these analyses, the strength was assumed to be greater than that measured in the UU tests ($s_u > 2.2$ ksf) that were performed at depths of approximately 10 to 12 feet. The factor of safety against a bearing capacity must exceed 3 for static loads. As indicated on page 9 of that calculation, the factor of safety of the cask storage pad foundation is 6.3 using the undrained strength of the cohesive soils . Page 10 of that calculation illustrates that the factor of safety against a bearing capacity failure increases to greater than 13 when a drained strength of $\phi = 30°$ is used. This friction angle is less than the friction angle shown for the soils that behave as sandy soils (SBT>5) based on the CPT data presented in Appendix D of ConeTec (1999). These plots illustrate that most of the "Phi" values are between 35° and 40° for these soils, with very few values slightly less than 35°. Therefore, assuming that all of the soils underlying the cask storage pads are cohesionless, as represented by the preponderance of soils that behave as "sandy" soils based on the CPT data, the factor of safety against a bearing capacity failure will be much greater than 13.

Item 3: PFS should use the CPT data to calculate settlements based on Equation 6-16 or 6-17 of Lunne, Robertson, and Powell (1997), which were developed by Meyerhof and Schmertmann, respectively. These analyses need to include discussion of differential settlements and the implications thereof on the casks.

Calculation 05996.02-G(B)-03, *Estimate Static Settlement of Storage Pads*, was revised (Rev. 3) to incorporate the calculation of settlements for the soils whose behavior is similar to that of "sandy" soils based on the CPT data. A copy of this calculation is included as Attachment 3. In this analysis, settlements are calculated based on Equation 6-17 of Lunne, Robertson, and Powell (1997), which was developed by Schmertmann (1970, 1978). This method is applicable for estimating settlements of foundations over sand using CPT data. The Schmertmann method takes into account the depth of footing, time of loading (40 years was used in the analysis), shape of the footing, and strain influence factor which varies with depth. The equivalent Young's modulus, which appears in the equation, is related to the cone penetration resistance by a factor, α, which is related to the degree of loading, soil density, stress history, cementation, age, grain shape, and mineralogy of the deposit. In this analysis, α was assumed to be 5, based on the discussion presented on page 26 of ConeTec (1998) (copy included as page C-2 of Calculation 05996.02-G(B)-03, Rev 3), which indicates that α varies between 4 and 6 for aged (>1,000 years) normally consolidated sands.

The Meyerhof method referred to above was not considered valid in this case due to its simplified approach to settlement prediction and due to the large footing widths applicable for the PFSF facility. The Meyerhof method estimates settlement as the net load, multiplied by the footing width, divided by 2 times the average cone penetration resistance. It is applicable for much smaller footings than those in question for the PFSF facility. Further, it does not take into account the various correction factors that the Schmertmann method incorporates, including the influence of decreasing stresses with respect to depth, the shape of the footing, degree of loading, soil density, stress history, cementation, age, grain shape, and mineralogy. It is more appropriate to utilize the Schmertmann method, which permits modeling of the various sublayers within the profile in a more reasonable manner.

Attachment C of Calculation 05996.02-G(B)-03, Rev. 3, *Estimate Static Settlement of Storage Pads* (copy included as Attachment 3), presents the calculation of settlements of each layer presented in the cone penetrometer data (Appendix E of ConeTec, 1999), based on the value of Q_t measured for each depth. This analysis uses all of the CPT data in the entire soil column between a depth of 3 ft (the bottom of the cask storage pads) and the bottom of the CPT data.

Two sets of estimated settlements were calculated and are summarized in the table presented on Page 44 of the calculation. Because of the preponderance of soils whose behavior is similar to that of "sandy" soils, settlements were calculated assuming that the Schmertmann method is applicable to the entire upper layer. As indicated by the left-hand column of settlements reported on Page 44 of the calculation, the estimated settlements for this case varied from 0.34 inches at CPT-26 to 0.56 inches at CPT-38.

The analyses were repeated, excluding those soils whose behavior is not similar to "sandy" soils, since the Schmertmann method is applicable only for cohesionless soils. In this analysis, cohesionless soils were defined as those with SBT values greater than 5, which includes silts, sandy silts, silty sands, and sands. The variability of these soils across the site is illustrated on the subsurface profiles that are included in Attachment 2. The estimated settlements for this case are presented in the right-hand column on Page 44 of the calculation and range from 0.24 inches at CPT-31 to 0.50 inches at CPT-10.

These results are posted on the map showing the locations of the CPTs on Page 46 of the calculation. As indicated, the differential settlements between CPT locations average less than 0.1 inches. The maximum difference between two adjacent (diagonally) CPTs is 0.19 inches, CPT-34 to CPT-29. Total and differential settlements of this magnitude are not significant in the design of the cask storage pads.

The cask storage pads have been designed using a finite element program that includes the soil and pad stiffness in the model. Uniform and/or differential settlements of the mat foundation are included in the analysis as a result of the load conditions considered. For instance, the sequential placement of casks is considered in varying load cases that will result in different cases of differential settlements. The computer program calculates the maximum moments and shears from the different load cases and the design is then based on the worst case forces in the pad. In this way, the structural design of the pad has considered differential settlements.

Item 4: PFS should explain why the shear wave velocities reported in Appendix C of the CPT report (Reference 1) increase with depth to about 10 to 15 ft and then level off for the remainder of the upper 25 to 30 ft in the profile.

Shear wave velocities of soils are dependent on the effective stress, void ratio, and for clays, the plasticity index and overconsolidation ratio of the soils. If all of these parameters were the same, it would be expected that the shear wave velocities would increase with increasing depth in the profile. The apparent leveling off of the shear wave velocities at a depth of about 10 to 15 ft in the results of the seismic CPTs that were performed at the site (Appendix C of ConeTec, 1999) is an indication that one or more of these parameters have changed. A review of the Qt plots, which are included on the left-hand side of the same pages that present the shear wave velocities vs depth, indicates that the tip resistance increases greatly in this zone. This increase in tip resistance is most likely associated with a change in soil type, as indicated by the SBT plots on the right-hand side of these same pages, as well as by a decrease in the void ratio of these soils. Therefore, it is not unexpected that the shear wave velocities would change within this zone.

A review of the shear wave velocities vs depth presented in Appendix C of ConeTec (1999) indicates that they do not level off with depth. The general trend in the data is to increase with respect to depth; however this trend is masked by the presence of the marked increase in the shear wave velocities in the "harder" zone that exists generally within the depth range of about 13 feet to about 20 feet. If the shear wave velocities associated with this harder zone are excluded, all of the plots of shear wave velocities show a general increase with respect to depth.

Item 5: PFS should explain why the plots of "PHI" in Appendix G of the CPT (Reference 1) report do not agree with the values of "PHI" shown in the tables of dilatometer data in Appendix H of that report.

The tables of values presented in Appendix H are correct, but the plots in Appendix G were in error. Attachment 4 provides replacement pages with corrected dilatometer interpretation plots for Appendix G of the ConeTec report. These are marked Report 05996.02-G(PO30), Rev 2 in the lower-left corner to distinguish them from those included the original report.

REFERENCES:

ConeTec, 1998, "Cone Penetration Testing - Geotechnical Applications Guide", October, 1998.

ConeTec, 1999, "Presentation of Cone Penetration Testing Results of Soils at the Private Fuel Storage Facility, Skull Valley, UT, Report No. 05996.02-G(PO30), Rev 1.

Lunne, Robertson, and Powell, 1997, *Cone Penetration Testing in Geotechnical Practice*, Blackie Pubs., London.

ATTACHMENTS TO

ENCLOSURE NO. 1 – GEOTECHNICAL PROGRAM

COMMITMENT RESOLUTION LETTER NO. 7 – JUNE 30, 1999

1. Contour Map Showing Thickness of Soils with CPT Soil Behavior Type > 5 (Sandy)
2. Cone Penetration Tests Profile Location Plan

   Profile 1-1' Showing Soil Behavior Type Per Cone Penetration Testing

   Profile 2-2' Showing Soil Behavior Type Per Cone Penetration Testing

   Profile 3-3' Showing Soil Behavior Type Per Cone Penetration Testing

   Profile 4-4' Showing Soil Behavior Type Per Cone Penetration Testing

   Profile 5-5' Showing Soil Behavior Type Per Cone Penetration Testing

   Profile 6-6' Showing Soil Behavior Type Per Cone Penetration Testing

   Profile 7-7' Showing Soil Behavior Type Per Cone Penetration Testing

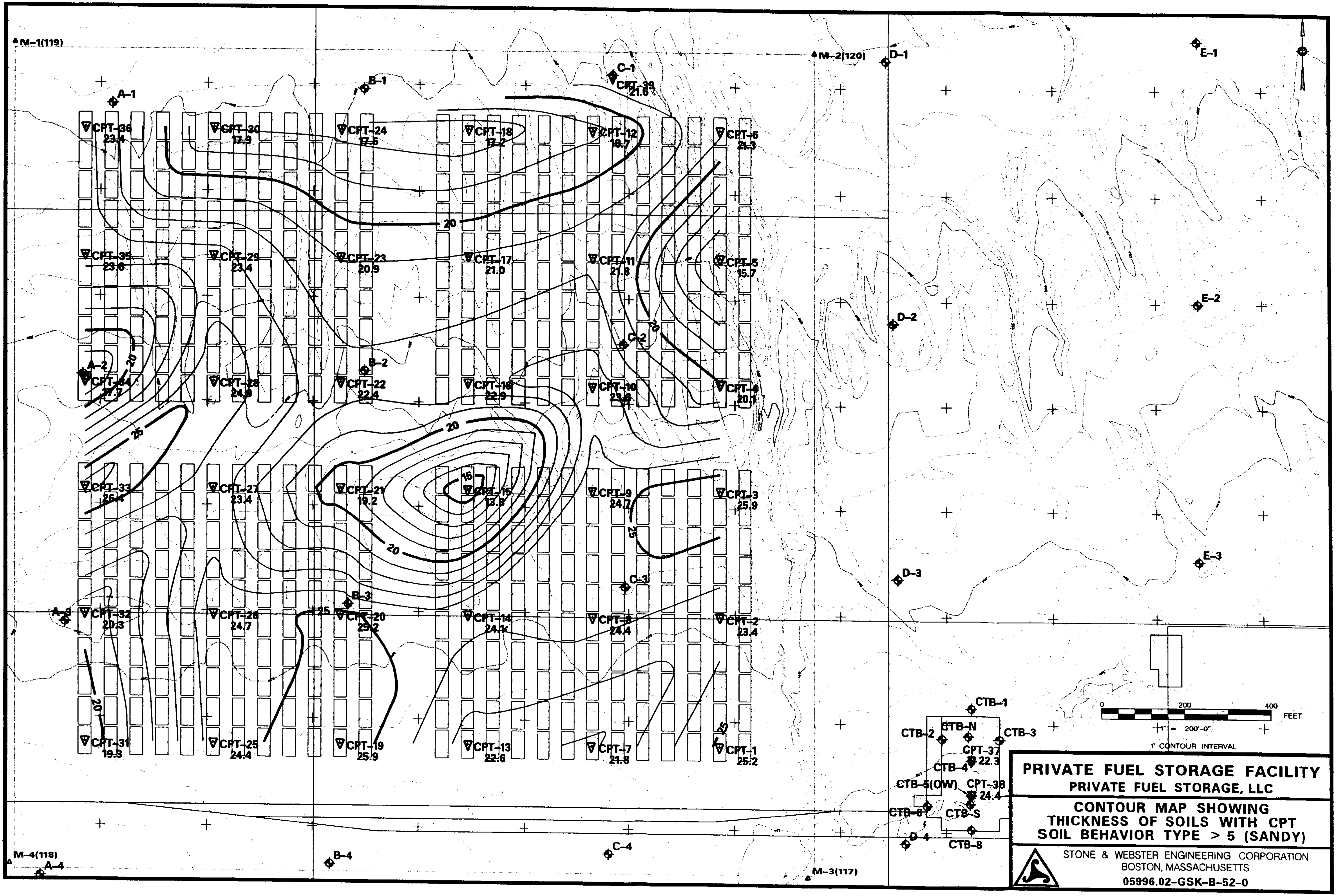
   Profile 8-8' Showing Soil Behavior Type Per Cone Penetration Testing

   Profile 9-9' Showing Soil Behavior Type Per Cone Penetration Testing

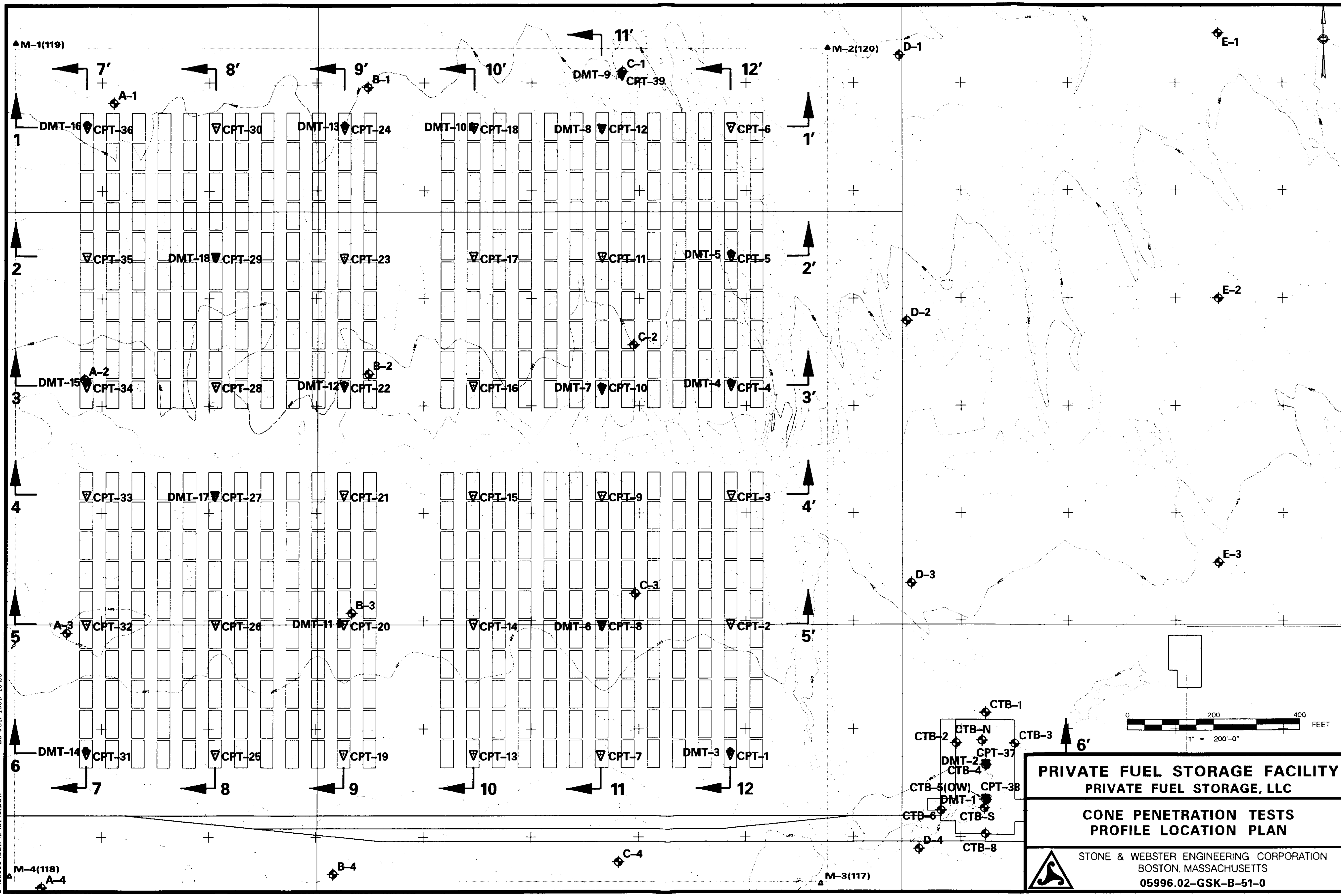
   Profile 10-10' Showing Soil Behavior Type Per Cone Penetration Testing

   Profile 11-11' Showing Soil Behavior Type Per Cone Penetration Testing

   Profile 12-12' Showing Soil Behavior Type Per Cone Penetration Testing
3. Calculation 05996.02-G(B)-03, Rev. 3, *Estimate Static Settlement of Storage Pads*, June 29, 1999, 46 pages, Attachment A 1 page, Attachment B 43 pages, Attachment C 244 pages.
4. Revised Appendix G for ConeTec (1999) Report 05996.02-G(PO30) Rev 2, Dilatometer plots – DMT-1 through DMT-18.



M-1(119)
M-2(120)
M-3(117)
M-4(118)
A-1
A-2
A-3
A-4
B-1
B-2
B-3
B-4
C-1
C-2
C-3
C-4
D-1
D-2
D-3
D-4
E-1
E-2
E-3
CPT-36 23.4
CPT-30 17.9
CPT-24 17.6
CPT-18 17.2
CPT-12 18.7
CPT-6 21.3
CPT-39 21.6
CPT-35 23.6
CPT-29 23.4
CPT-23 20.9
CPT-17 21.0
CPT-11 21.8
CPT-5 15.7
CPT-34 17.7
CPT-28 24.9
CPT-22 22.4
CPT-16 22.9
CPT-10 23.8
CPT-4 20.1
CPT-33 26.4
CPT-27 23.4
CPT-21 19.2
CPT-15 13.8
CPT-9 24.7
CPT-3 25.9
CPT-32 20.3
CPT-26 24.7
CPT-20 25.2
CPT-14 24.1
CPT-8 24.4
CPT-2 23.4
CPT-31 19.3
CPT-25 24.4
CPT-19 25.9
CPT-13 22.6
CPT-7 21.8
CPT-1 25.2
CTB-1
CTB-2
CTB-N
CTB-3
CPT-37 22.3
CTB-4
CTB-5(OW)
CPT-38 24.4
CTB-6
CTB-S
CTB-8
15
20
25
0 200 400 FEET
1" = 200'-0"
1' CONTOUR INTERVAL
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE, LLC
CONTOUR MAP SHOWING THICKNESS OF SOILS WITH CPT SOIL BEHAVIOR TYPE > 5 (SANDY)
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
05996.02-GSK-B-52-0



PRIVATE FUEL STORAGE FACILITY

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CONE PENETRATION TESTS PROFILE LOCATION PLAN

STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

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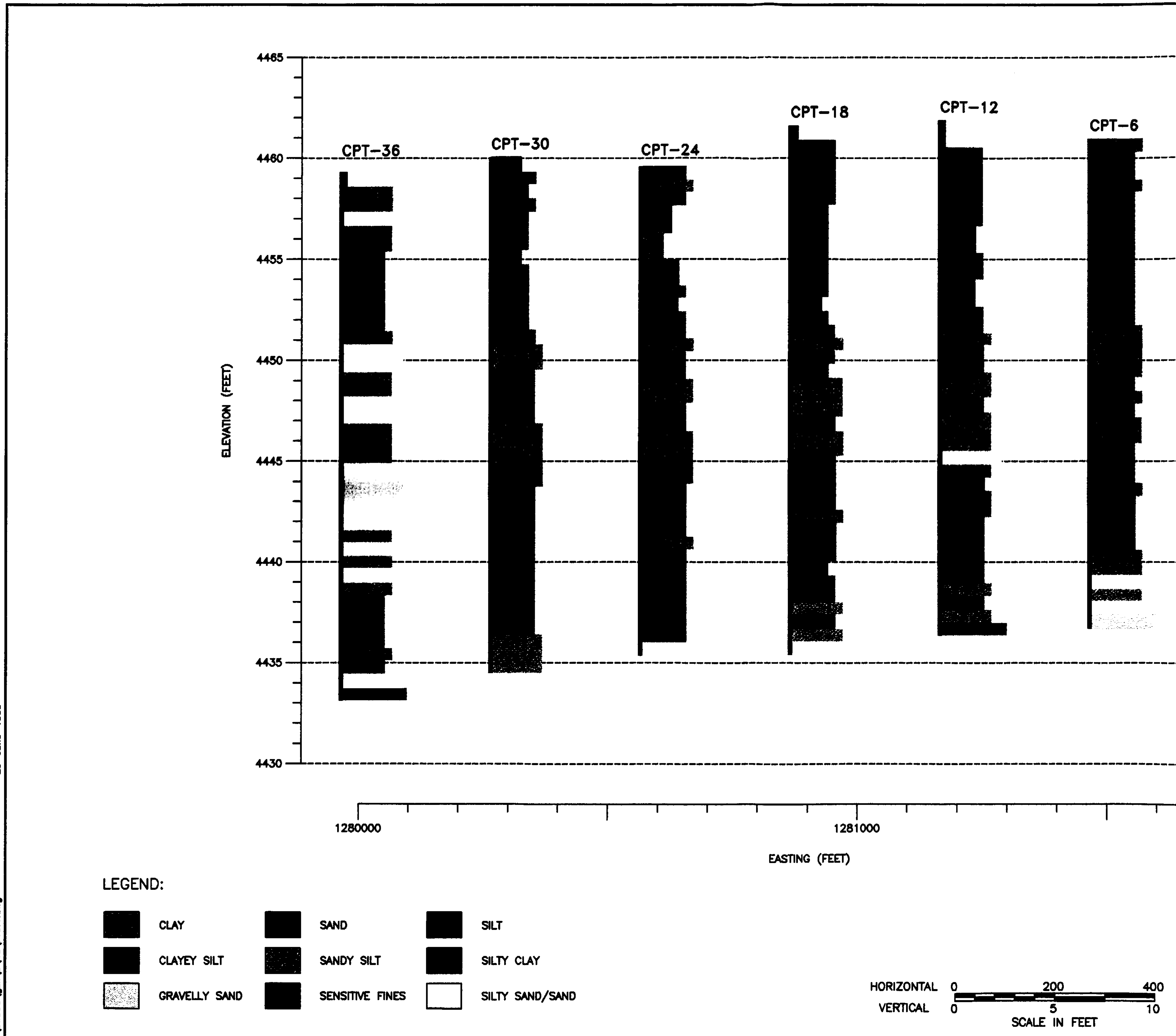
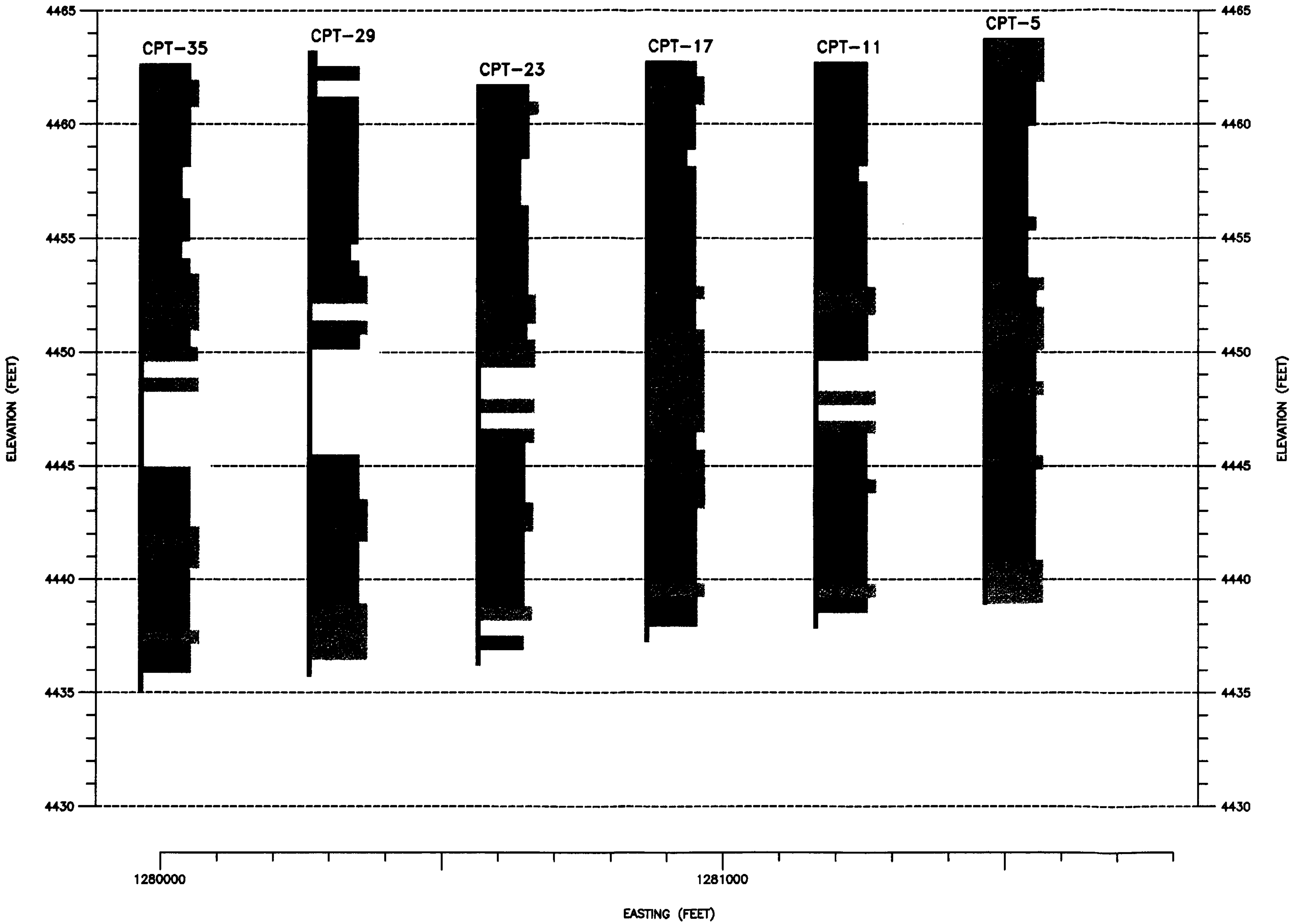


PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE, LLC

PROFILE 1 – 1'
SHOWING SOIL BEHAVIOR TYPE PER CONE PENETRATION TESTING

STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
05996.02-GSK-B-40-0



LEGEND:

- CLAYEY SILT
- SILT
- SANDY SILT
- SILTY SAND/SAND
- SENSITIVE FINES

HORIZONTAL 0 200 400
VERTICAL 0 5 10
SCALE IN FEET

PRIVATE FUEL STORAGE FACILITY

PRIVATE FUEL STORAGE, LLC

PROFILE 2 - 2'
SHOWING SOIL BEHAVIOR TYPE PER CONE PENETRATION TESTING

STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
05996.02-GSK-B-41-0

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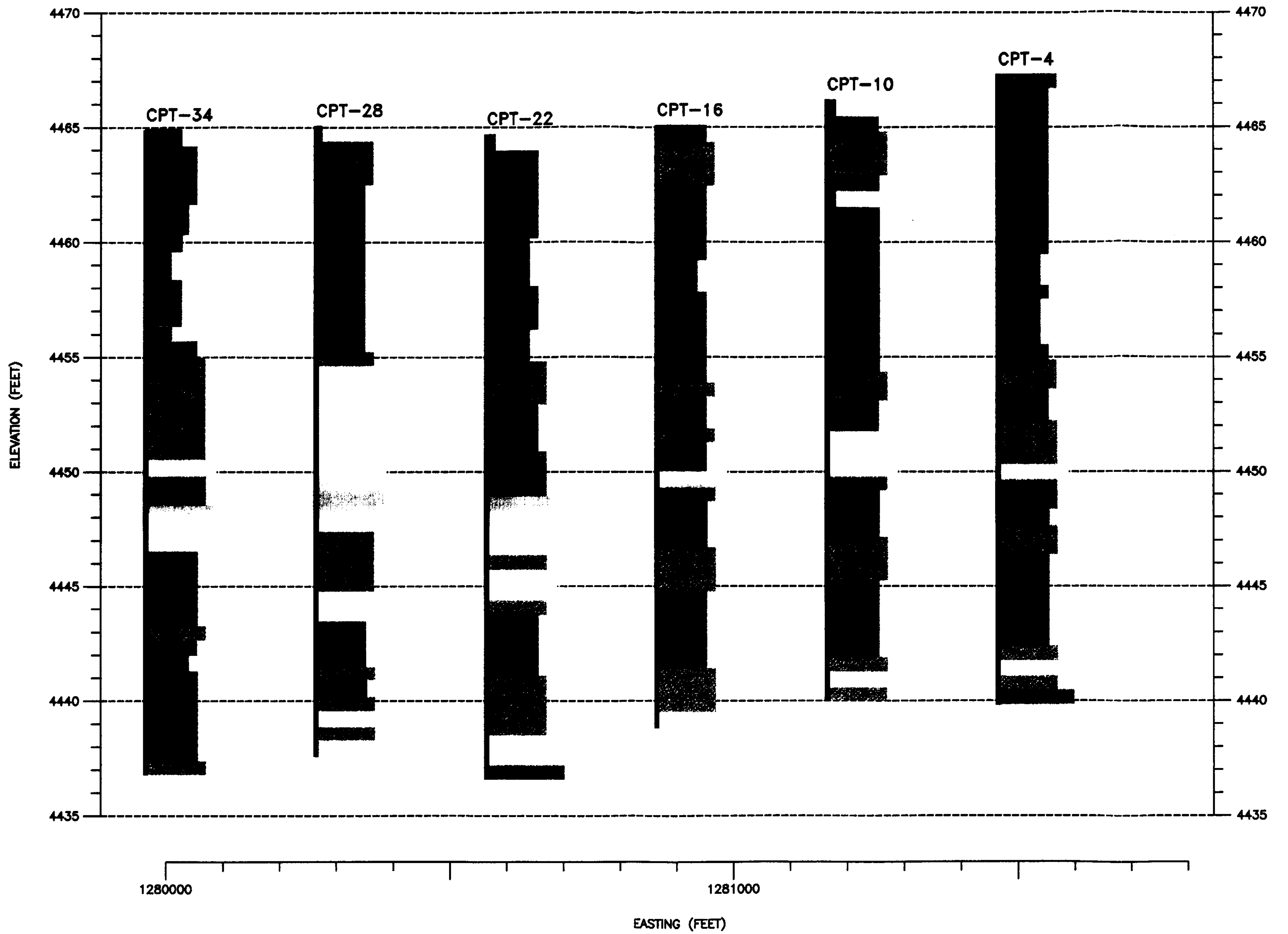

LEGEND:

CLAY	SANDY SILT	SILTY CLAY
CLAYEY SILT	SENSITIVE FINES	SILTY SAND/SAND
SAND	SILT	

HORIZONTAL 0 200 400

VERTICAL 0 5 10

SCALE IN FEET

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PRIVATE FUEL STORAGE, LLC

PROFILE 3 – 3'
SHOWING SOIL BEHAVIOR TYPE PER
CONE PENETRATION TESTING

STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
05996.02-GSK-B-42-0

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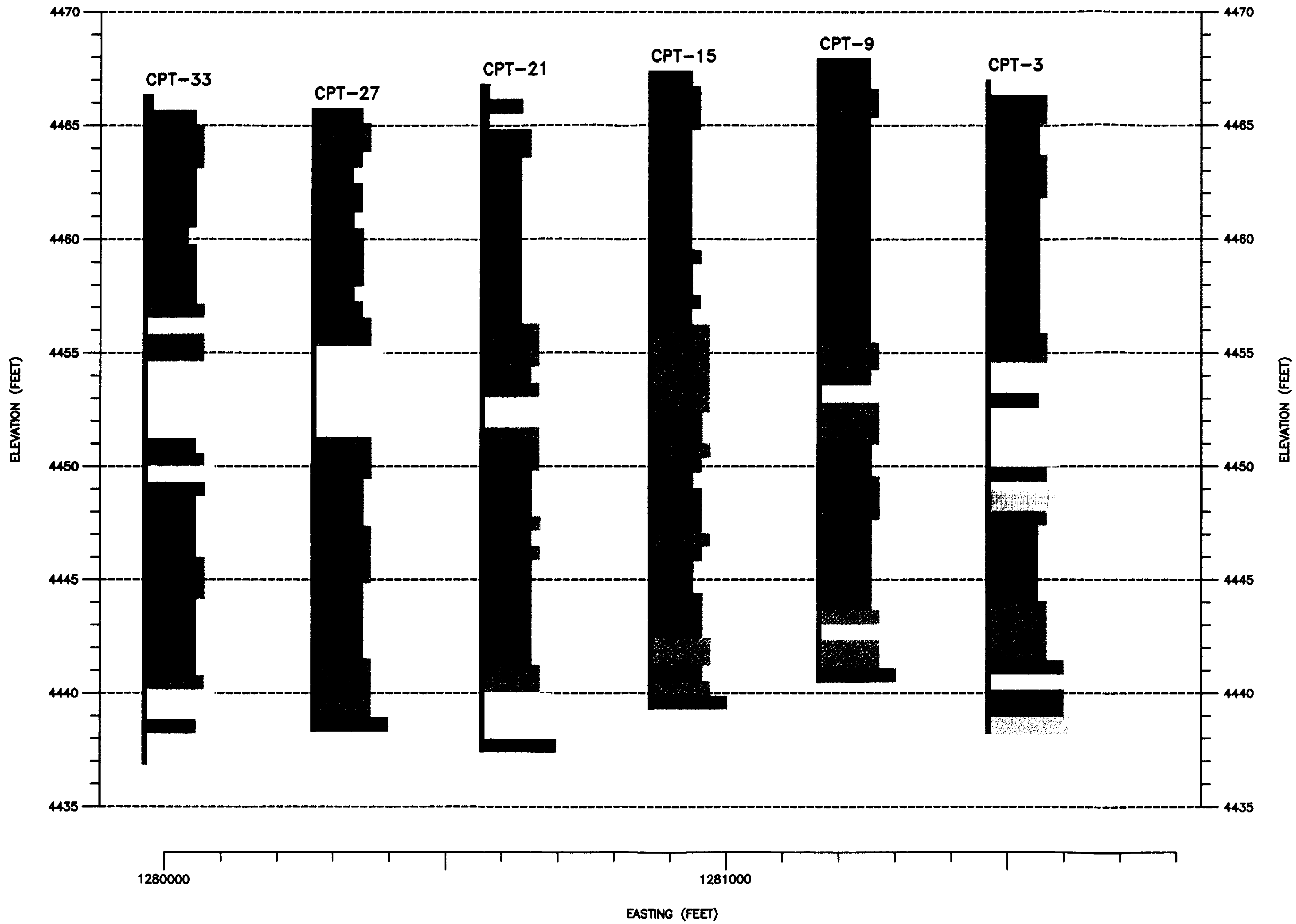

LEGEND:

CLAYEY SILT	SANDY SILT	SILTY SAND/SAND
GRAVELLY SAND	SENSITIVE FINES	
SAND	SILT	

HORIZONTAL 0 200 400
VERTICAL 0 5 10
SCALE IN FEET

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PRIVATE FUEL STORAGE, LLC

PROFILE 4 – 4'
SHOWING SOIL BEHAVIOR TYPE PER CONE PENETRATION TESTING

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BOSTON, MASSACHUSETTS
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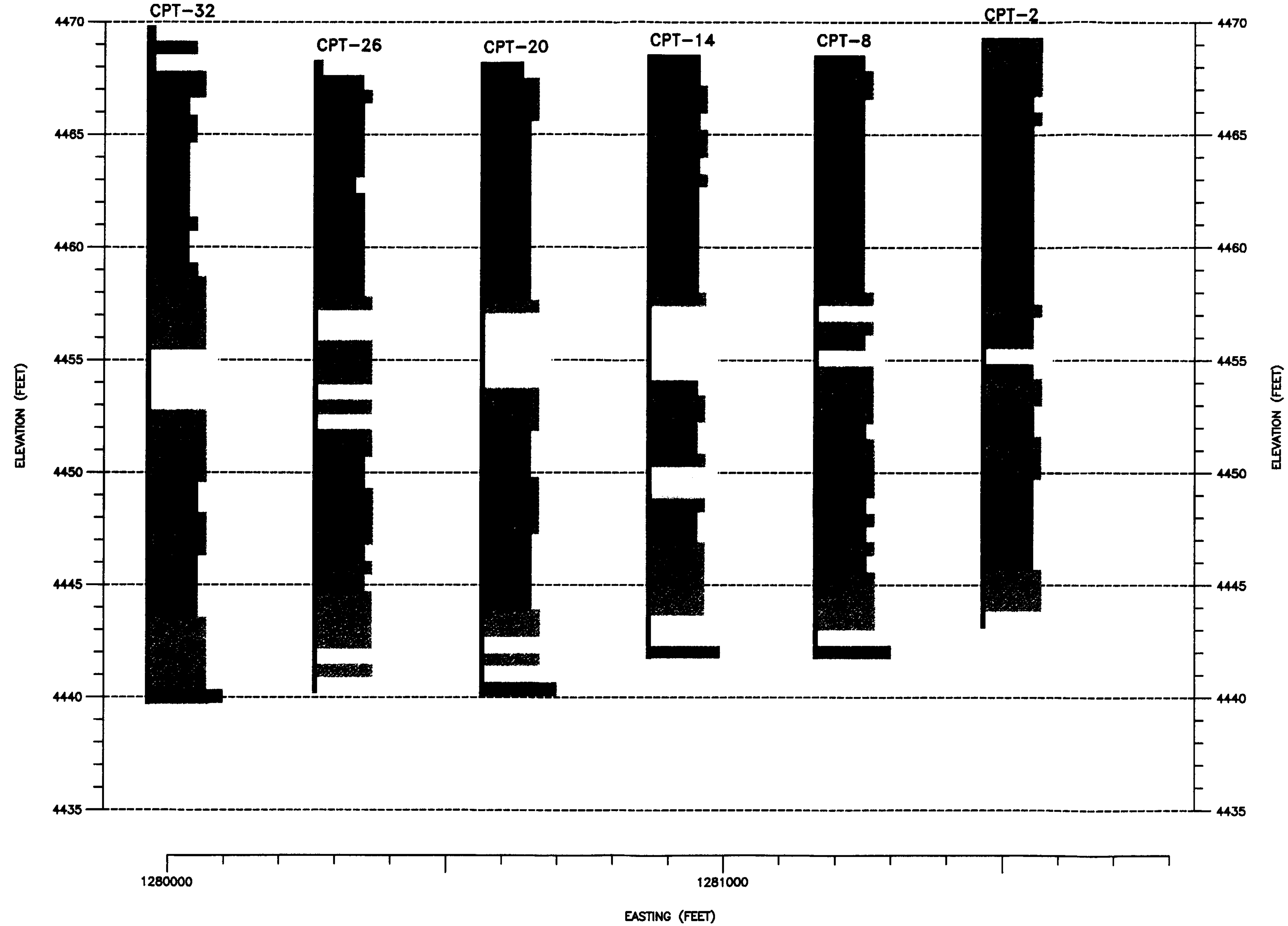

LEGEND:

- CLAYEY SILT
- SENSITIVE FINES
- SAND
- SILT
- SANDY SILT
- SILTY SAND/SAND

HORIZONTAL 0 200 400
VERTICAL 0 5 10
SCALE IN FEET

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PROFILE 5 – 5'
SHOWING SOIL BEHAVIOR TYPE PER
CONE PENETRATION TESTING

STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
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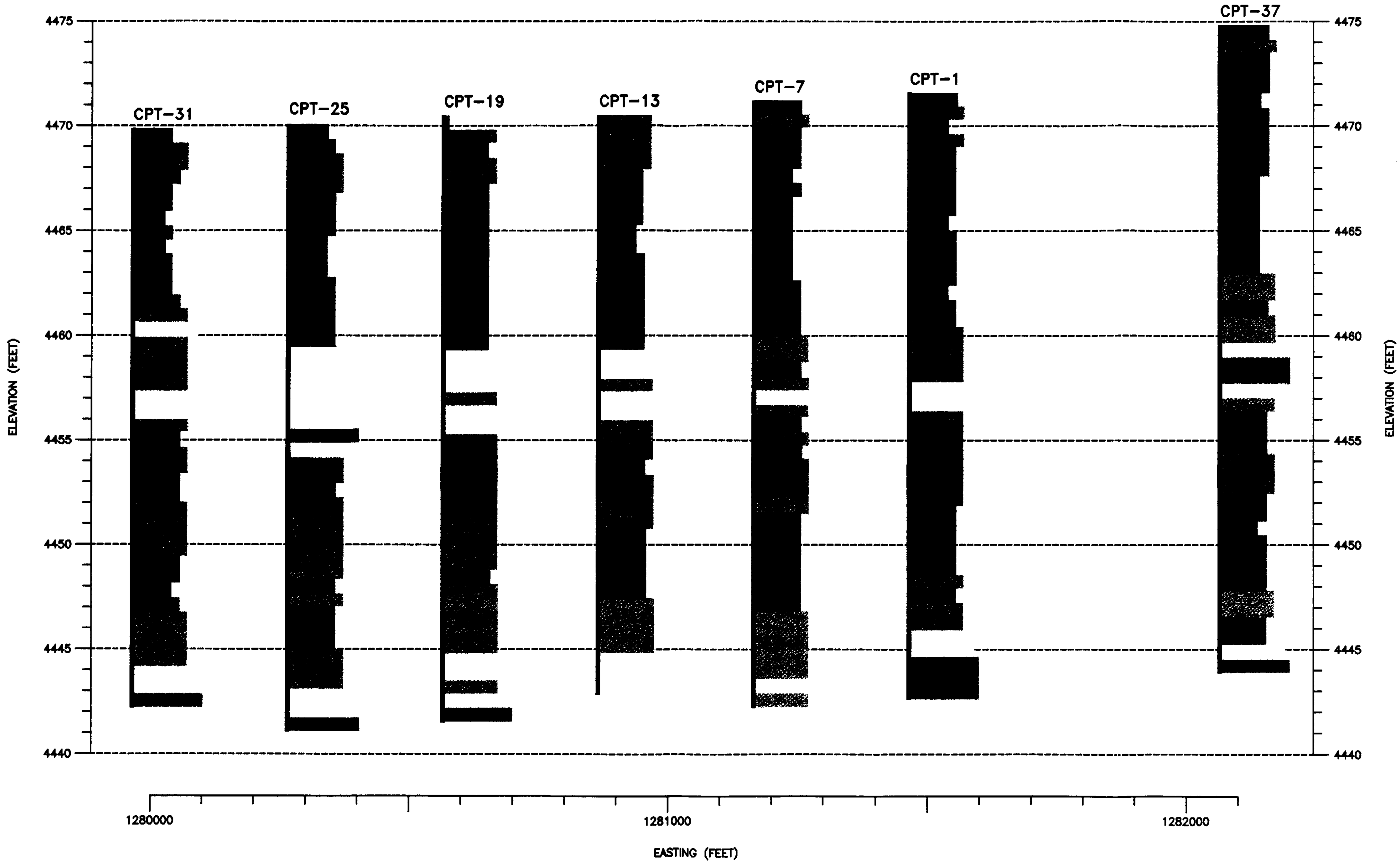

LEGEND:

CLAYEY SILT	SILT
SAND	SILTY CLAY
SANDY SILT	SILTY SAND/SAND

HORIZONTAL 0 200 400
VERTICAL 0 5 10
SCALE IN FEET

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PRIVATE FUEL STORAGE, LLC

PROFILE 6 – 6'
SHOWING SOIL BEHAVIOR TYPE PER CONE PENETRATION TESTING

STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
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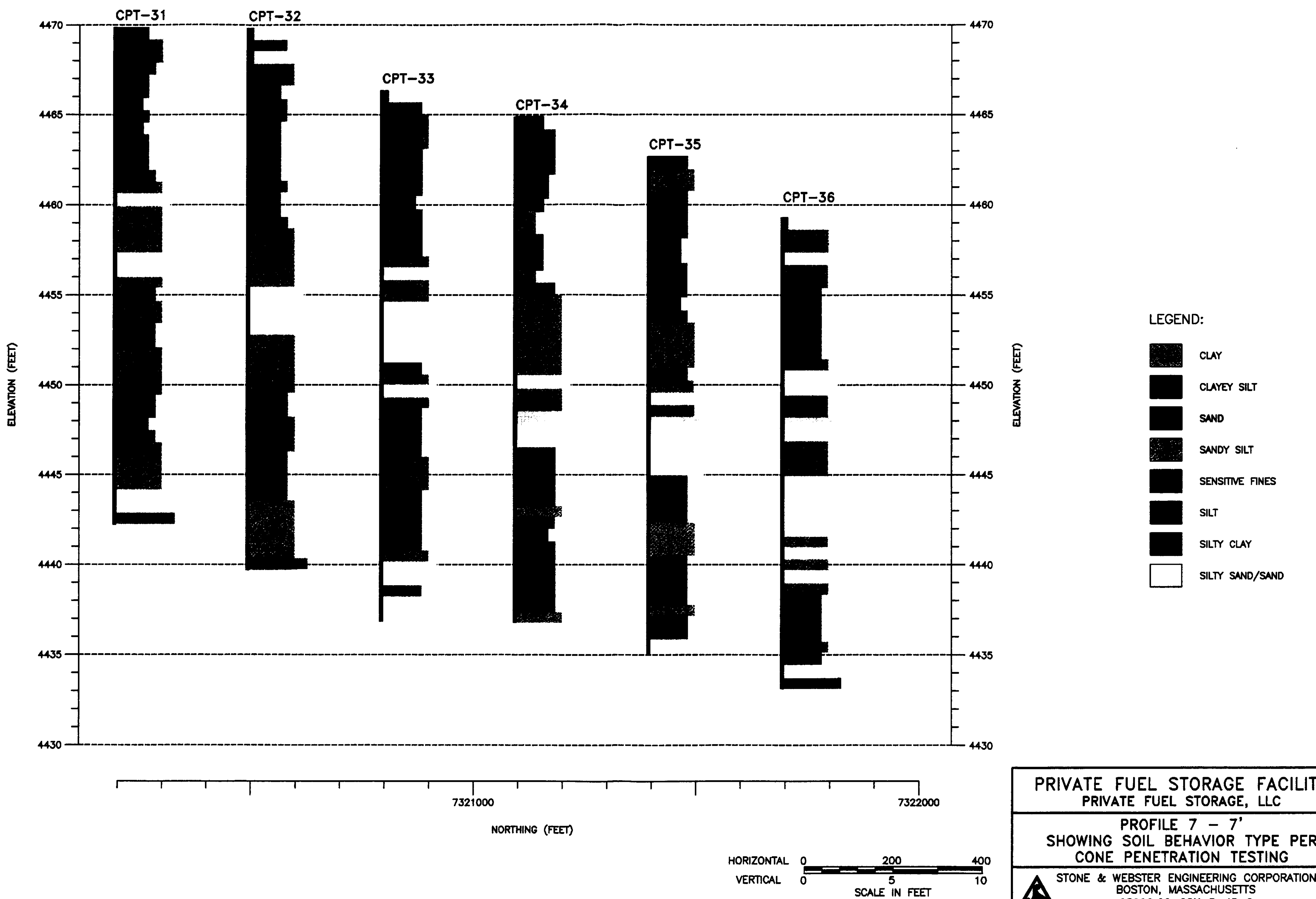

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PRIVATE FUEL STORAGE, LLC

PROFILE 7 – 7'
SHOWING SOIL BEHAVIOR TYPE PER
CONE PENETRATION TESTING

STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
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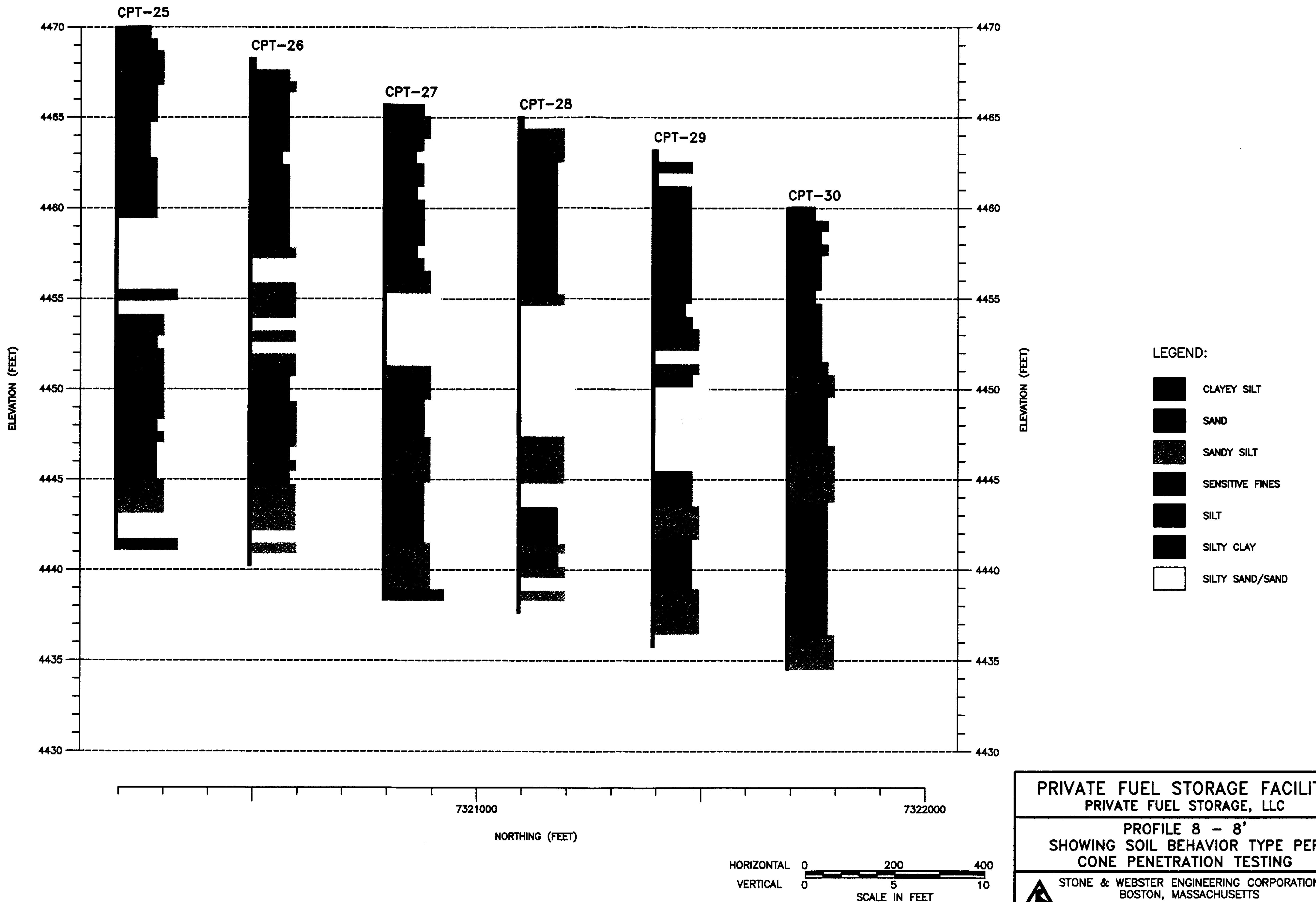

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PRIVATE FUEL STORAGE, LLC

PROFILE 8 – 8'
SHOWING SOIL BEHAVIOR TYPE PER
CONE PENETRATION TESTING

STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
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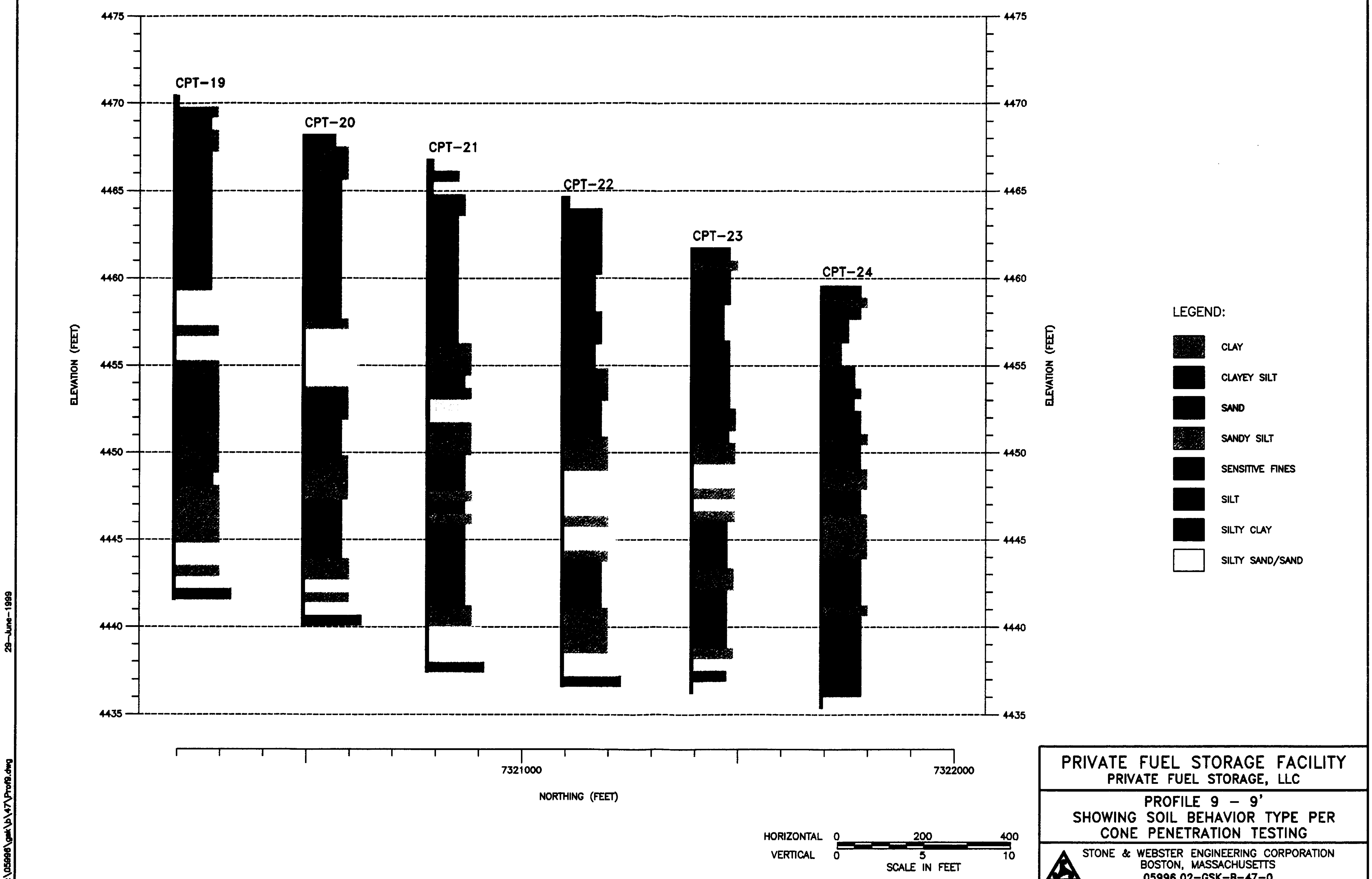

PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE, LLC

PROFILE 9 – 9'
SHOWING SOIL BEHAVIOR TYPE PER
CONE PENETRATION TESTING

STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
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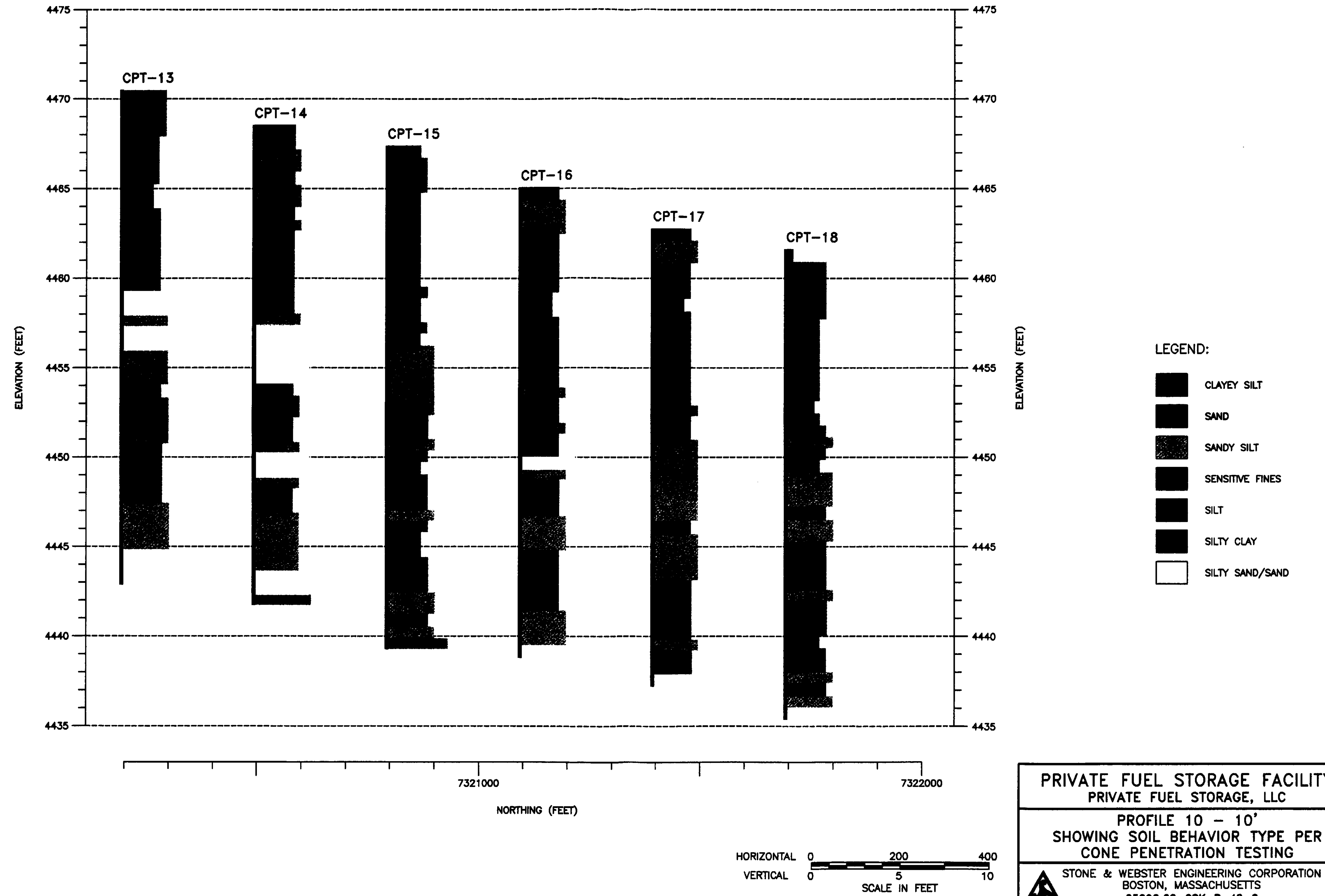

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PRIVATE FUEL STORAGE, LLC

PROFILE 10 – 10'
SHOWING SOIL BEHAVIOR TYPE PER
CONE PENETRATION TESTING

STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
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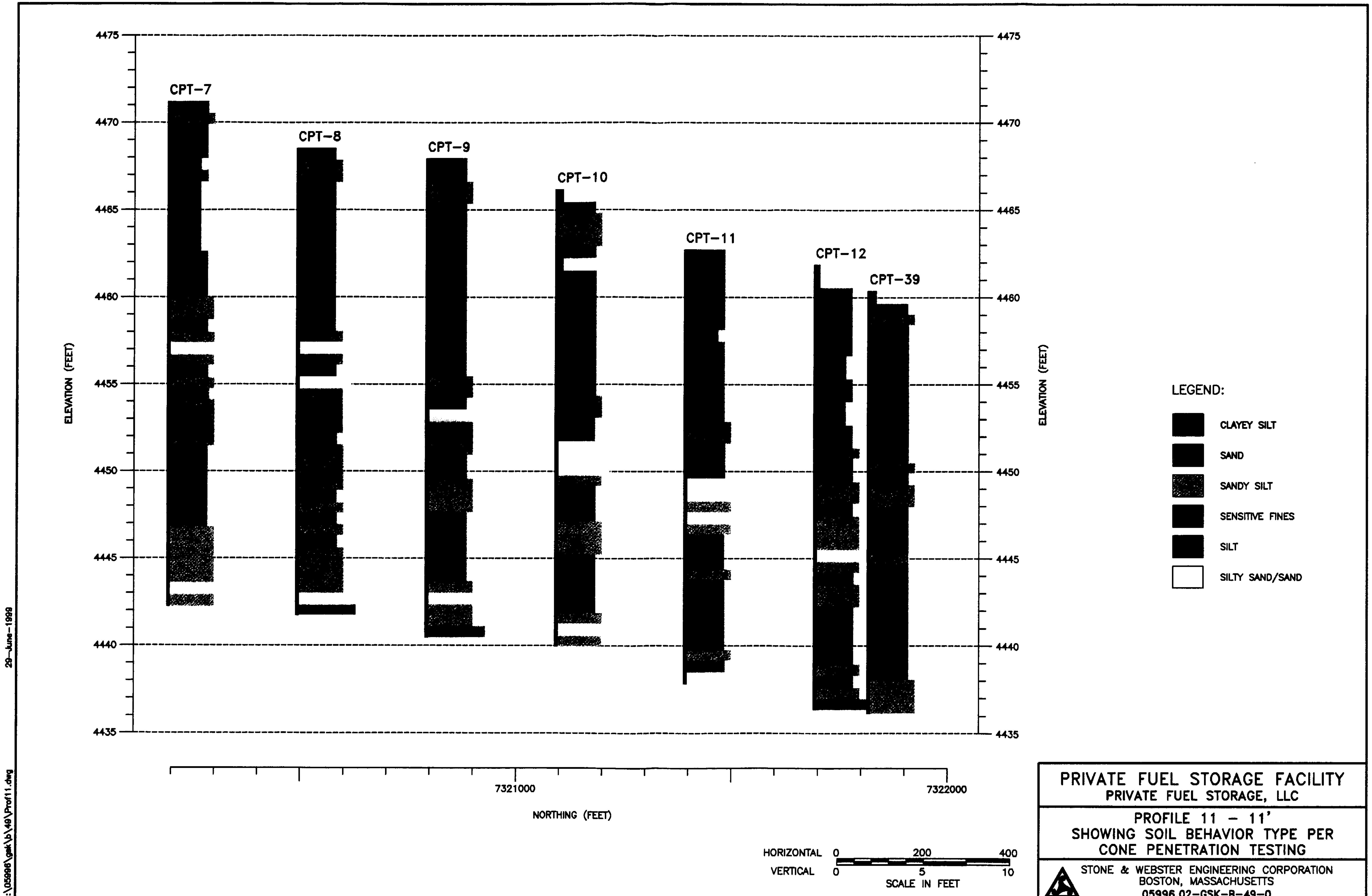

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PRIVATE FUEL STORAGE, LLC

PROFILE 11 – 11'
SHOWING SOIL BEHAVIOR TYPE PER
CONE PENETRATION TESTING

STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
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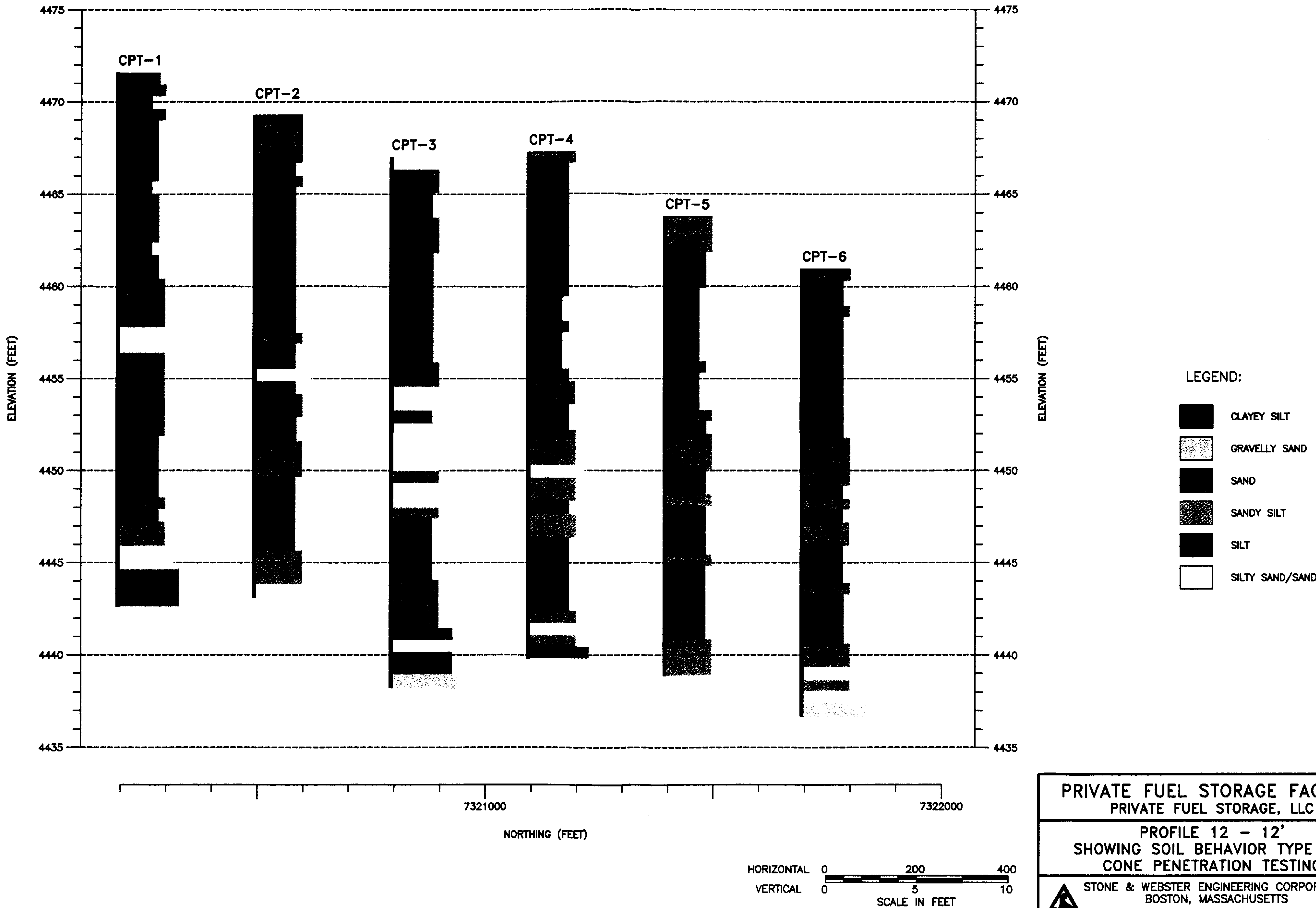

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PROFILE 12 – 12'
SHOWING SOIL BEHAVIOR TYPE PER CONE PENETRATION TESTING

STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS
05996.02-GSK-B-50-0

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