APPENDIX A

SUBSURFACE CONDITIONS

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Available Data

The subsurface conditions near the outlet of Pretty Lake have been interpreted from one historical exploration, whose location is shown in Figure 4-2. The data at this location is limited to geotechnical borings drilled in 1999 by Virginia Department of Transportation (VDOT) for the Shore Drive Bridge with the objective of identifying the sand bearing stratum to estimate pile tip elevations. The data only includes Standard Penetration Test (SPT) blow counts. Note that the superimposed pile tip elevations in Figure 4-2 are based on pile design length of 70 feet. The as-built pile lengths may vary according to encountered subsurface conditions. Even though the data is limited, different soil parameters were still derived using published correlations for preliminary site characterization, and conceptual design of alternative flood mitigation systems.

Regional Geology

Regionally, the Pretty Lake area is located in the Coastal Plain physiographic province. Flat-lying plains and terraces dominate the landscape. The Coastal Plain is underlain by a wedge of Cretaceous to Holocene age sediments that thicken to the east and pinch out at the Fall Line approximately 70 miles west of the project area. Jurassic-Triassic age basement rocks lie approximately 1,800 feet beneath the site. The wedge of Cretaceous and younger sediments were deposited as a result of multiple marine transgressions and regressions. Sediments within the upper 150 feet beneath the site are Pliocene to Recent in age. The Pliocene and younger sediments have been deposited and subsequently eroded in places during the rising and falling sea levels that resulted from glacial and interglacial periods.

Historical Development

The project area has been modified by man’s activities for several years. The historical development of the Pretty Lake outlet and change in the Shore Drive bridge alignment are evident. The potential for encountering remnants of historical construction should be recognized when planning flood mitigation projects in the project area.

Subsurface Stratigraphy

The stratigraphic units encountered beneath the outlet of Pretty Lake are described in descending sequence in the following discussion.

The 1999 historical investigation by VDOT for the Shoreline Drive Bridge at the entrance of Pretty Lake encountered subsurface conditions primarily consisting of very loose to loose sand fill, with boring B-5 encountering a sand with gravel interval within this depth. Beneath the fill, explorations encountered Alluvium consisting of loose to dense sands and a soft fine grained/very loose sand layer of variable thickness. Below the Alluvium are dense silty sands of the Norfolk Formation. The subsurface stratigraphy within this section of Pretty Lake is shown in Figure 4-2 in the main text.

Artificial Fill

The 1999 historical borings encountered a layer of artificial fill assumed to be associated with development and construction of the Shore Drive bridge. This layer consists of very loose to loose fine to medium sand that varies in thickness. Underneath Pretty Lake, the fill layer is less than 10 feet thick but is thicker to the north and south.

Alluvium

Underlying the artificial fill is an alluvium, which has two distinct layers: a loose to dense sand layer over a fine-grained/very loose sand layer. The loose to dense sand layer is primarily medium to coarse dense sand and is considered as fluvial-estuarine sediment. This layer thickens north to south from 15 feet to 30 feet. On the north side of the section, the lower fine grained (clay and silt) layer is encountered at a depth of 25 to 30 feet. This layer initially thins towards to south, then transitions to very loose sand and thickens on the south side of the section. The thickness of this layer ranges from approximately 5 to 25 feet.

The Yorktown Formation

The sand alluvium is underlain by a zone of loose to dense, fine sand with trace shell fragments. It is inferred that this layer is the Yorktown Formation, which is described as a granular Pleistocene age fluvial-estuarine and brackish marine deposits.

Design Subsurface Profiles for Concept Evaluation

To evaluate possible flood mitigation systems at Pretty Lake, it was necessary to idealize the subsurface conditions, and determine soil properties that will govern the selection of an appropriate flood mitigation system. Based on the available data and published correlations between different soil parameters, the following were interpreted:

- Two idealized soil profiles representing an upper and lower bound of expected stratigraphy;
- Design strength parameters including undrained shear strength and friction angles;
- Ultimate bearing capacity values for the upper and lower boundary profiles based on a continuous strip footing with a unit width;
- Active and passive earth pressure coefficients. A drained condition was assumed for the clay and silt layer.

Idealized Stratigraphy

The subsurface condition was idealized into two profiles. The first profile is located inside the channel where the silt and clay layer is thinnest. This profile represents an upper bound of expected design strength parameters. This profile comprises of about 30 feet of loose sand overlying 20 feet of medium dense to dense sand. Below this layer is a 5 feet layer of soft silt and clay layer. The bottom layer comprises of a 65 feet medium dense to dense silty sand layer.

The second profile is located below the southwest abutment of Shore Drive bridge. This profile represents a lower bound of expected design strength parameters. The profile comprises of a 30 feet layer of loose sand overlying a 25 feet layer of medium dense to dense sand. The clay and silt layer at this location is about 20 feet. The bottom layer comprises of medium dense to dense silty sand.

Based on the soil description and the blow counts, each layer was assigned a total unit weight (Table A-1). The total unit weights were used to estimate an effective stress profile as shown in Figures A-1 and A-2.

Table A-1. Total Unit Weights

Soil layer	Total Unit Weight (pcf)	
	Profile 1	**Profile 2**
Loose sand	105	105
Medium dense to dense sand	115	115
Silt and Clay	105	110
Medium dense to dense silty sand	120	120

Design Strength Parameters

The strength properties for the sand layers were obtained by estimating friction angle (ϕ) profiles based on SPT blow counts (N-values). The N-values were corrected for rod length, fines content, and overburden pressure using the correlation provided by Liao and Whitman (1986). The correlations provided by Peck et al. (1974) and the American Petroleum Institute (API) (2000) were then used to estimate ϕ from corrected N-values. The API method resulted in considerably higher ϕ values as shown in Figures A-3 and A-4. Therefore, the mean ϕ values for each layer was calculated based on Peck et al. (1974) and used as an upper bound profile. Two standard deviations were subtracted from the mean value to calculate the lower bound ϕ profile. The upper and lower design ϕ values for each profile are shown in Table A-2.

Since no strength data is available for the clay and silt layer, the undrained shear strength of this layer was conservatively assumed to be 500 psf and 250 psf for an upper and lower bound values, respectively.

Table A-2. Idealized Friction Angles

Soil Layer	Effective Stress Friction Angle (degrees)					
	Profile 1			Profile 2		
	Upper Bound	Lower Bound	COV (%)	Upper Bound	Lower Bound	COV (%)
Loose sand	30	27	5	29	26	5
Medium dense to dense sand	36	33	4	37	30	9
Medium dense to dense silty sand	35	32	4	35	32	5

Active and Passive Earth Pressure Coefficients

Active (k_a) and passive (k_p) earth pressure coefficients were calculated for the two idealized profiles. This can be used to conceptually design flood mitigation alternatives where lateral earth pressure loadings are expected behind the structure such as a retaining or sheet pile wall. The pressure at which the soil fails as the wall moves away from the retained soil is called active earth pressure, whereas the pressure at which the soil fails as the wall moves into the retained soil is called passive pressure. Active and passive earth pressure coefficients were calculated according to Rankine's and Coulomb's theories (Figures A-5 to A-8). Rankine's k_a and k_p were determined based on a frictionless wall where the interface friction (δ) between the retaining structure and the soil is neglected. Coulomb's k_a and k_p where calculated for a steel and concrete wall by varying the value of δ. For a steel wall, δ was equal to ϕ- 5^o, whereas for a concrete wall, δ was equal to $0.58*\phi$.

Ultimate Bearing Capacity

Since some flood mitigation alternatives may be supported on shallow foundations, ultimate bearing capacity values, based on a continuous strip footing with unit width, were calculated. For each profile, lower and upper bound bearing capacity values were determined from the mean and lower bound friction angles, respectively (Figures A-9 and A-10). Further, ultimate bearing capacity values for the clay and silt layer were estimated based on both drained and undrained conditions. For a drained condition, the upper bound effective stress friction angle was assumed to be 33^o degrees. The lower bound effective stress friction angle was obtained by applying a 7% Coefficient of Variation (COV) based on Duncan (2000) recommendations. Tables A-3 and A-4 summarize the bearing capacity factors, which were used to calculate the ultimate bearing capacity. The bearing capacity factors were based on correlations provided by Meyerhof (1963).

Table A-3. Bearing Capacity Factors – Profile 1

Bearing Capacity Factor	Loose Sand		Medium Dense to Dense Sand		Silt and Clay			Medium Dense to Dense Silty Sand	
	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Undrained	Upper Bound	Lower Bound
N_c	30	24	51	39	39	26	5	46	35
N_q	18	13	38	26	26	15	1	33	23
N_γ	16	10	44	26	26	11	0	37	22
COV (%)	5	-	4	-	7	-	-	4	-

Table A-4. Bearing Capacity Factors – Profile 2

Bearing Capacity Factor	Loose Sand		Medium Dense to Dense Sand		Silt and Clay			Medium Dense to Dense Silty Sand	
	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound	Undrained	Upper Bound	Lower Bound
N_c	28	22	56	30	39	26	5	46	35
N_q	16	12	43	18	26	15	1	33	23
N_γ	13	8	53	16	26	11	0	37	22
COV (%)	5	-	9	-	7	-	-	5	-

REFERENCES

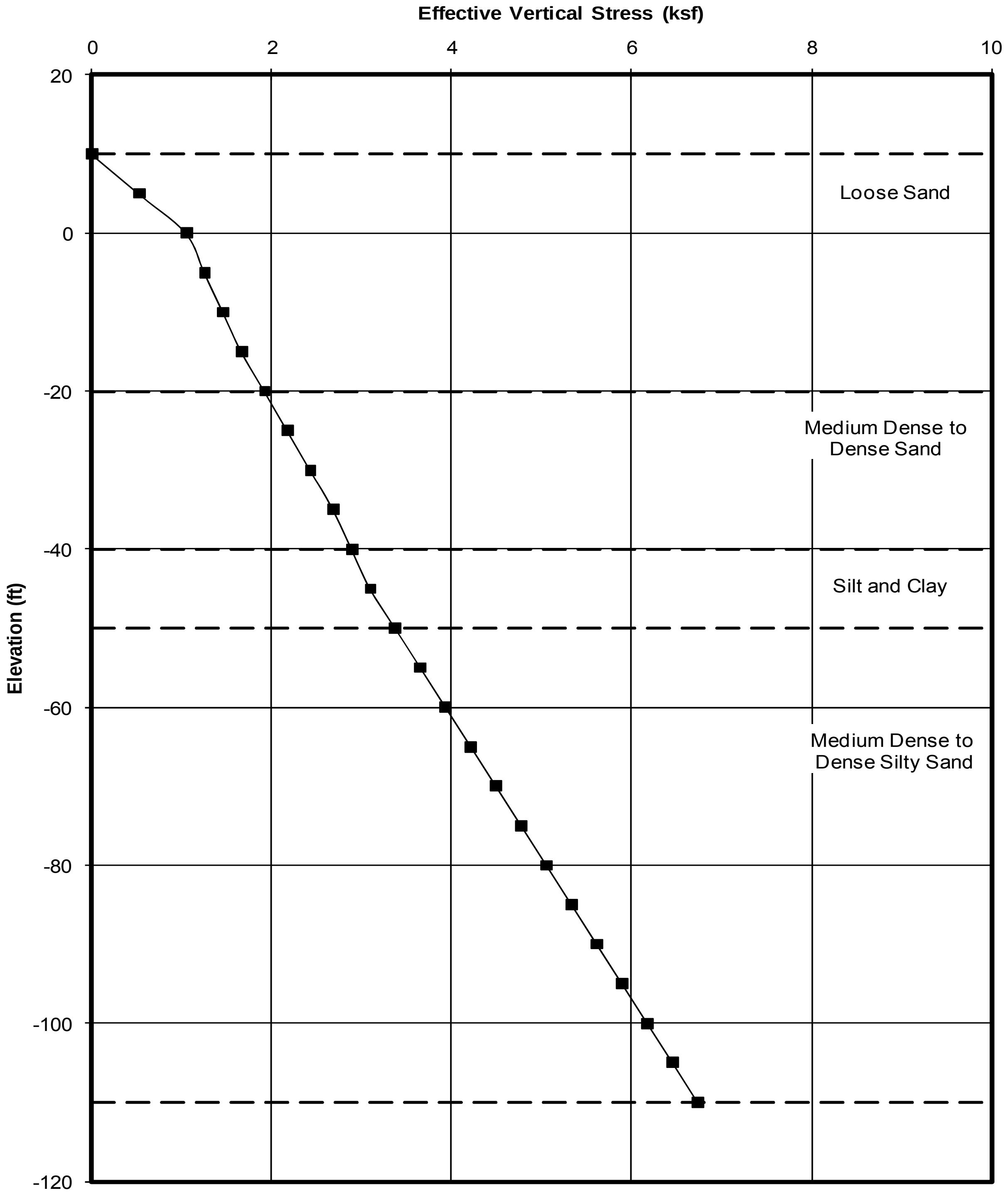
American Petroleum Institute (2000), "*Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms – Working Stress Design*," API Recommended Practice 2A-WSD (RP 2A-WSD), 21st Ed., December 2000, API, Washington, D.C.

Duncan, J. M. (2000), "Factors of Safety and Reliability in Geotechnical Engineering," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 126, No. 4, pp. 307-316.

Liao, S. C., and Whitman, R. V. (1986), "Overburden Correction Factors for SPT," *Journal of Geotechnical Engineering*, ASCE, Vol. 112, No. 3, pp. 373-377.

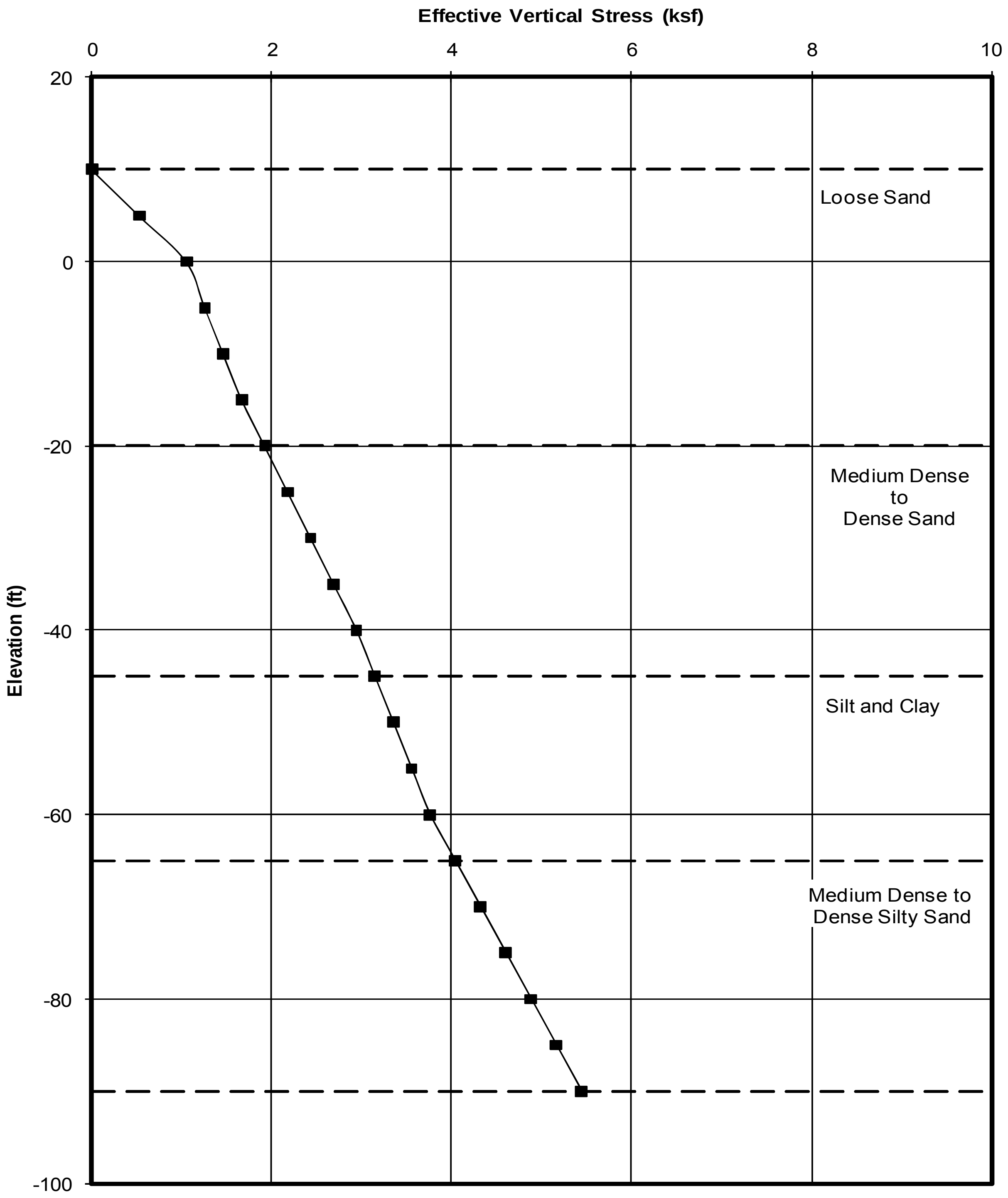
Meyerhof, G. G. (1963), "Some Recent Research on the Bearing Capacity of Foundations," *Canadian Geotechnical Journal*, Vol. 1, No. 4, pp. 57-64.

Peck, R. B., Hanson, W. E., and Thornburn, T. H. (1974), "*Foundation Engineering*," 2nd Edition, John Wiley, New York.

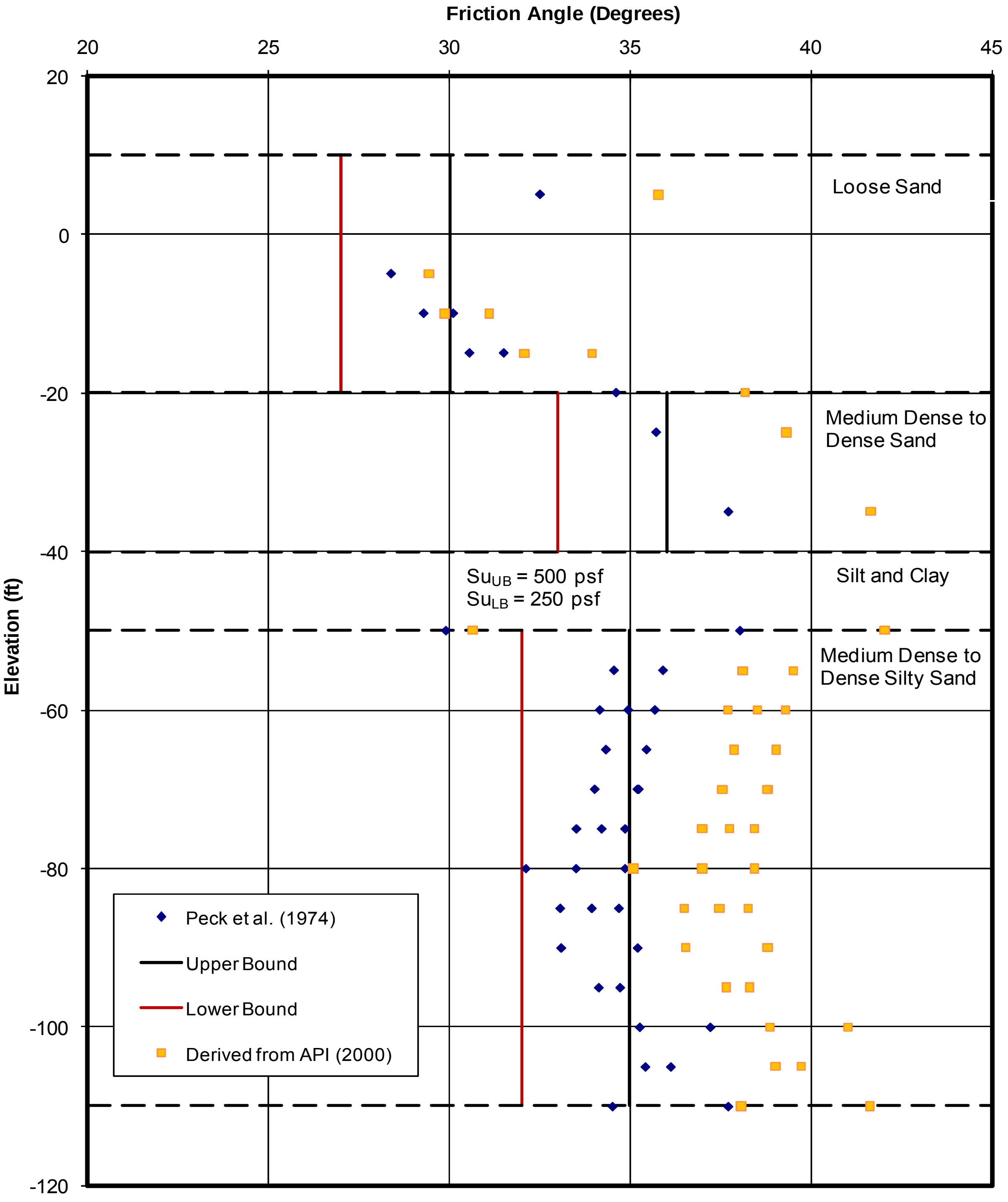


EFFECTIVE STRESS - PROFILE 1
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE A-1



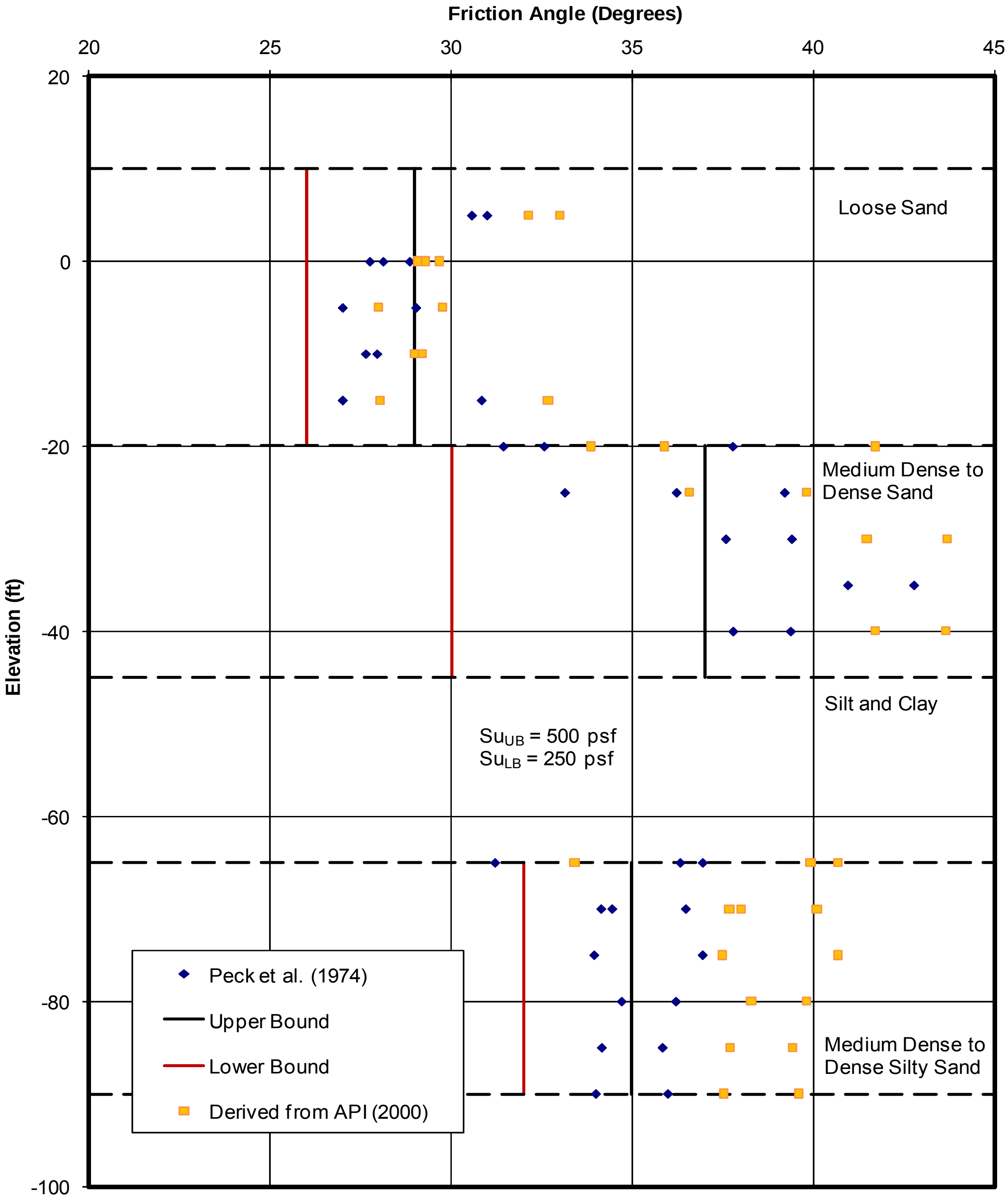
EFFECTIVE STRESS - PROFILE 2
City-wide Coastal Flooding Study
Norfolk, Virginia



DESIGN STRENGTH PARAMETERS - PROFILE 1

City-wide Coastal Flooding Study
Norfolk, Virginia

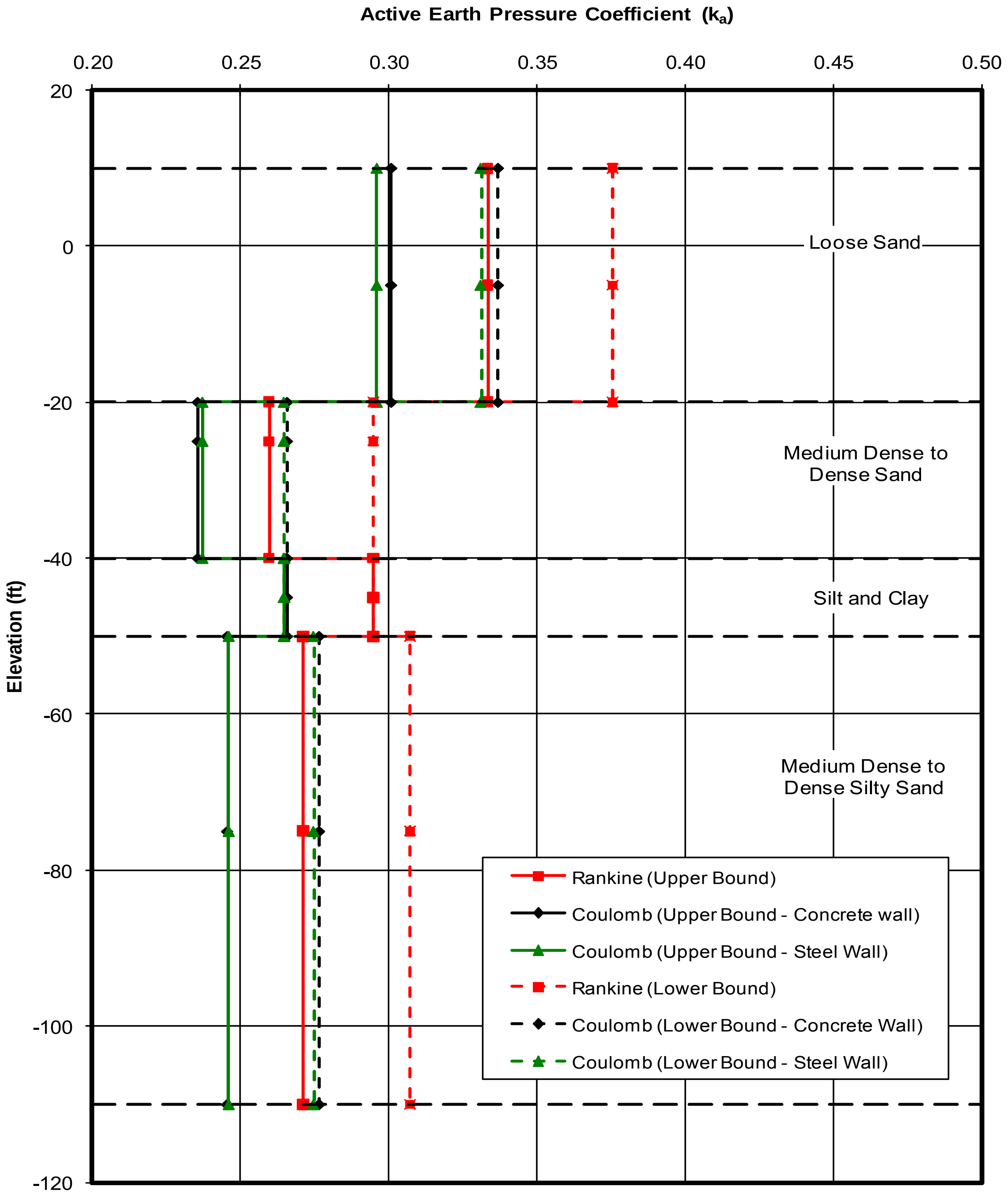
FIGURE A-3



DESIGN STRENGTH PARAMETERS - PROFILE 2

City-wide Coastal Flooding Study
Norfolk, Virginia

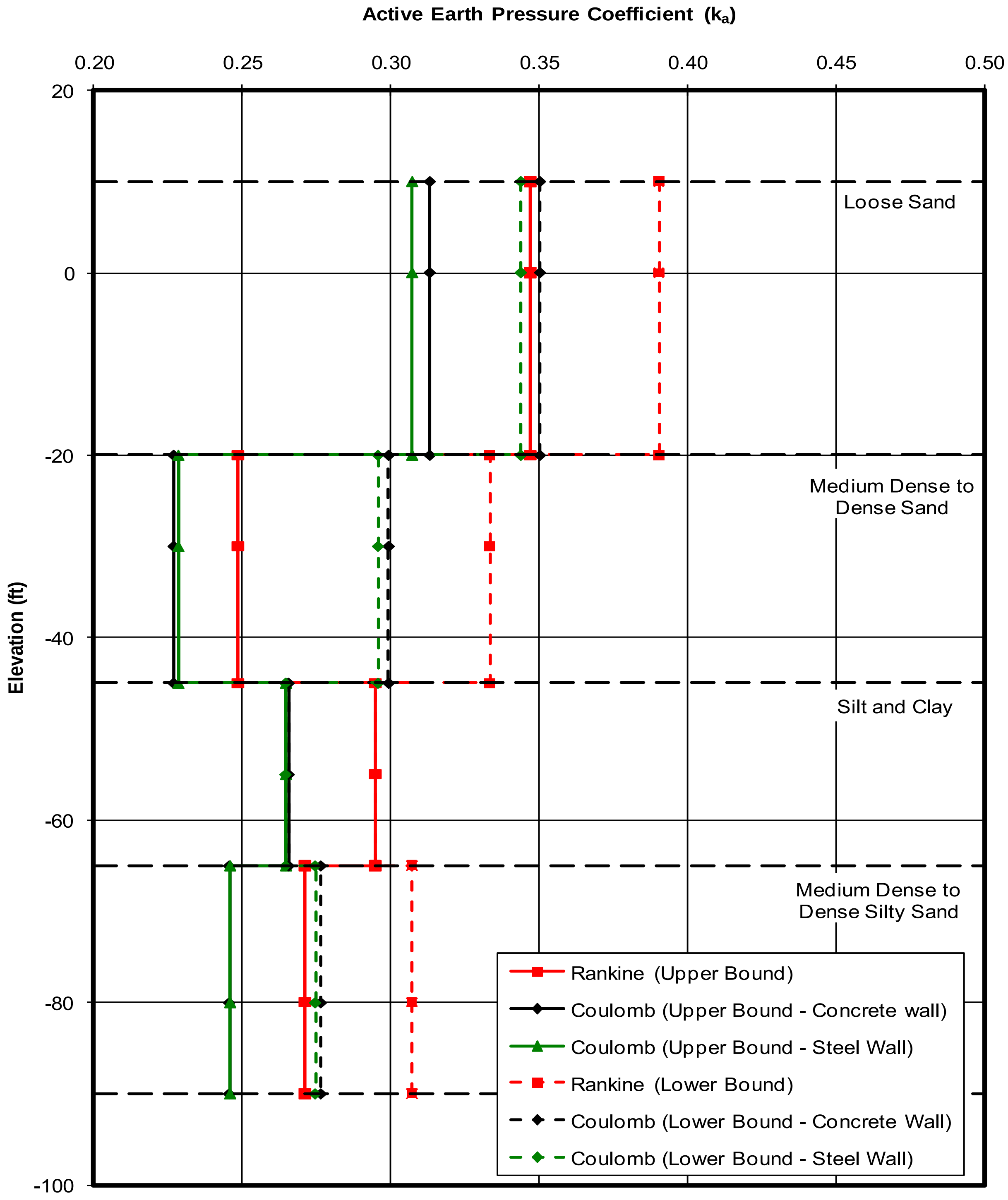
FIGURE A-4



ACTIVE EARTH PRESSURE COEFFICIENT - PROFILE 1

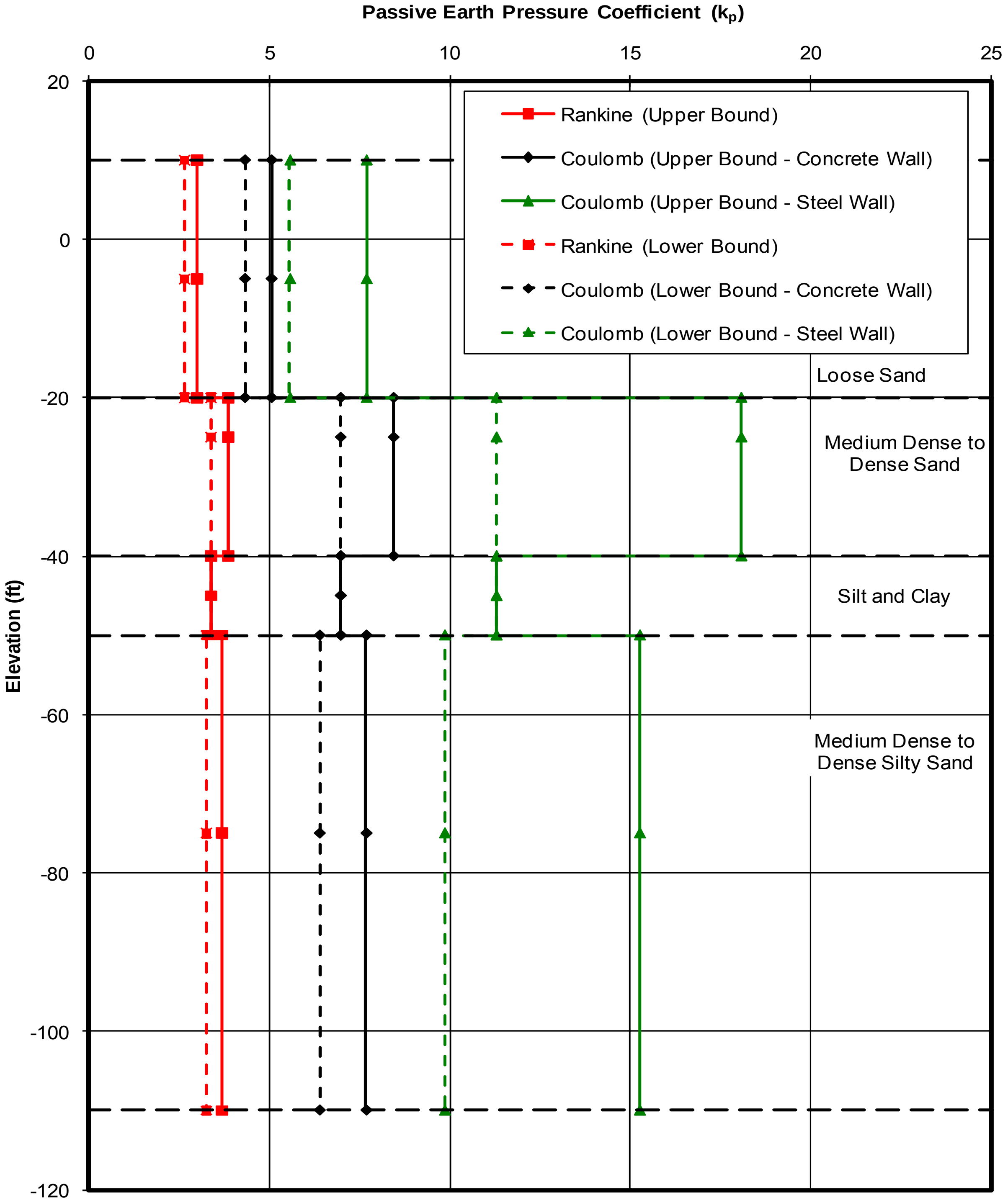
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE A-5



ACTIVE EARTH PRESSURE COEFFICIENT - PROFILE 2

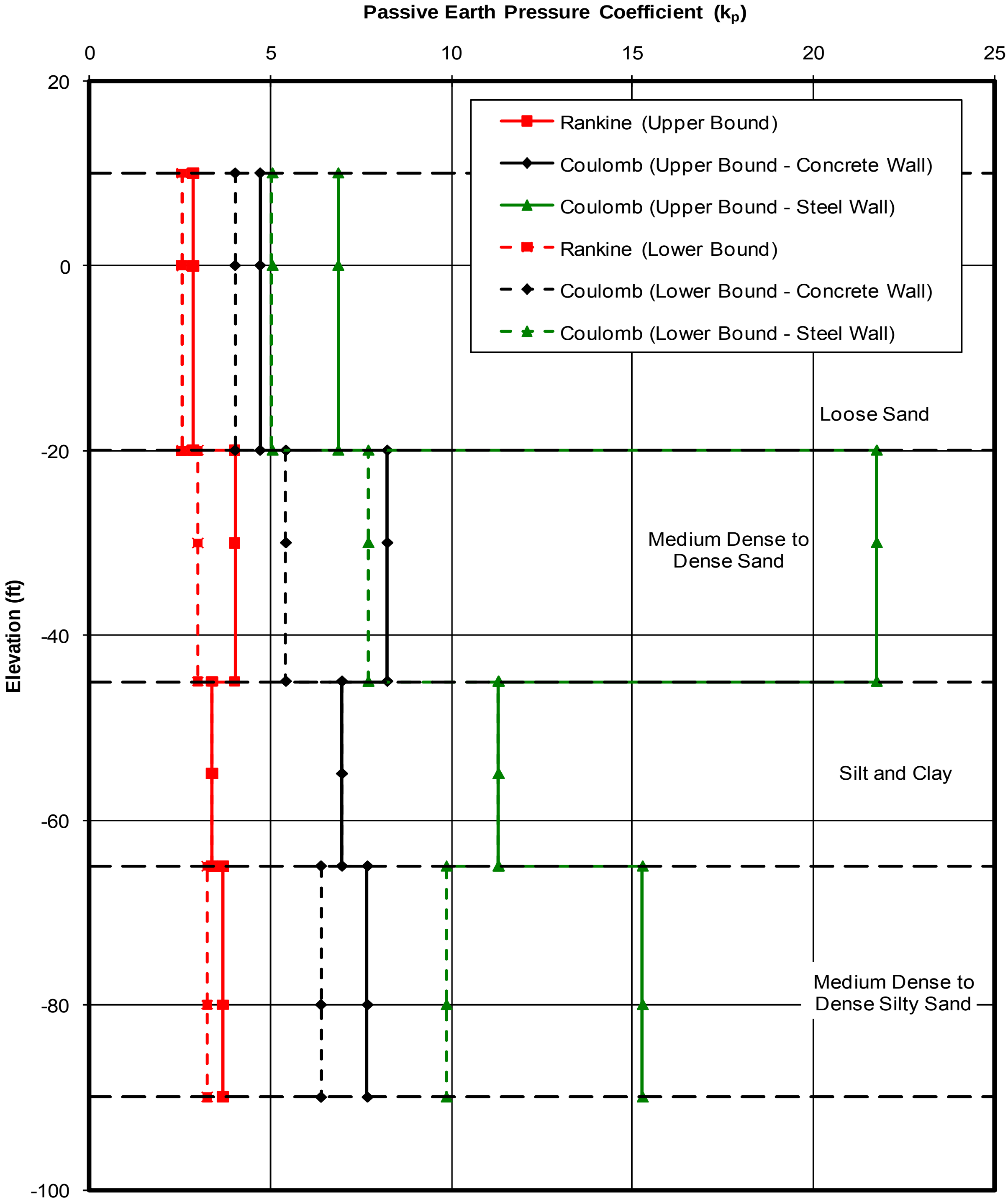
City-wide Coastal Flooding Study
Norfolk, Virginia



PASSIVE EARTH PRESSURE COEFFICIENT - PROFILE 1

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE A-7

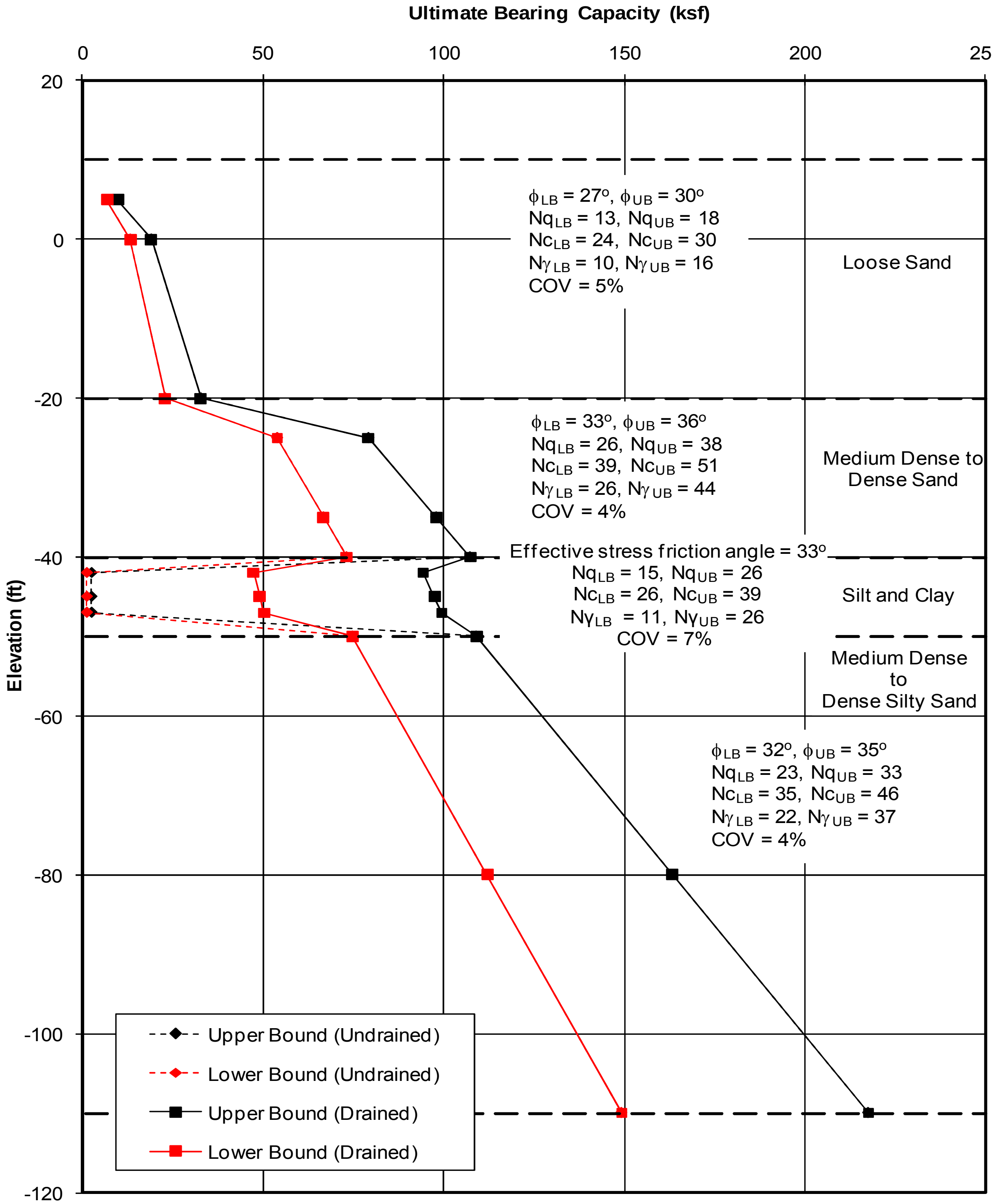


PASSIVE EARTH PRESSURE COEFFICIENT - PROFILE 2

City-wide Coastal Flooding Study

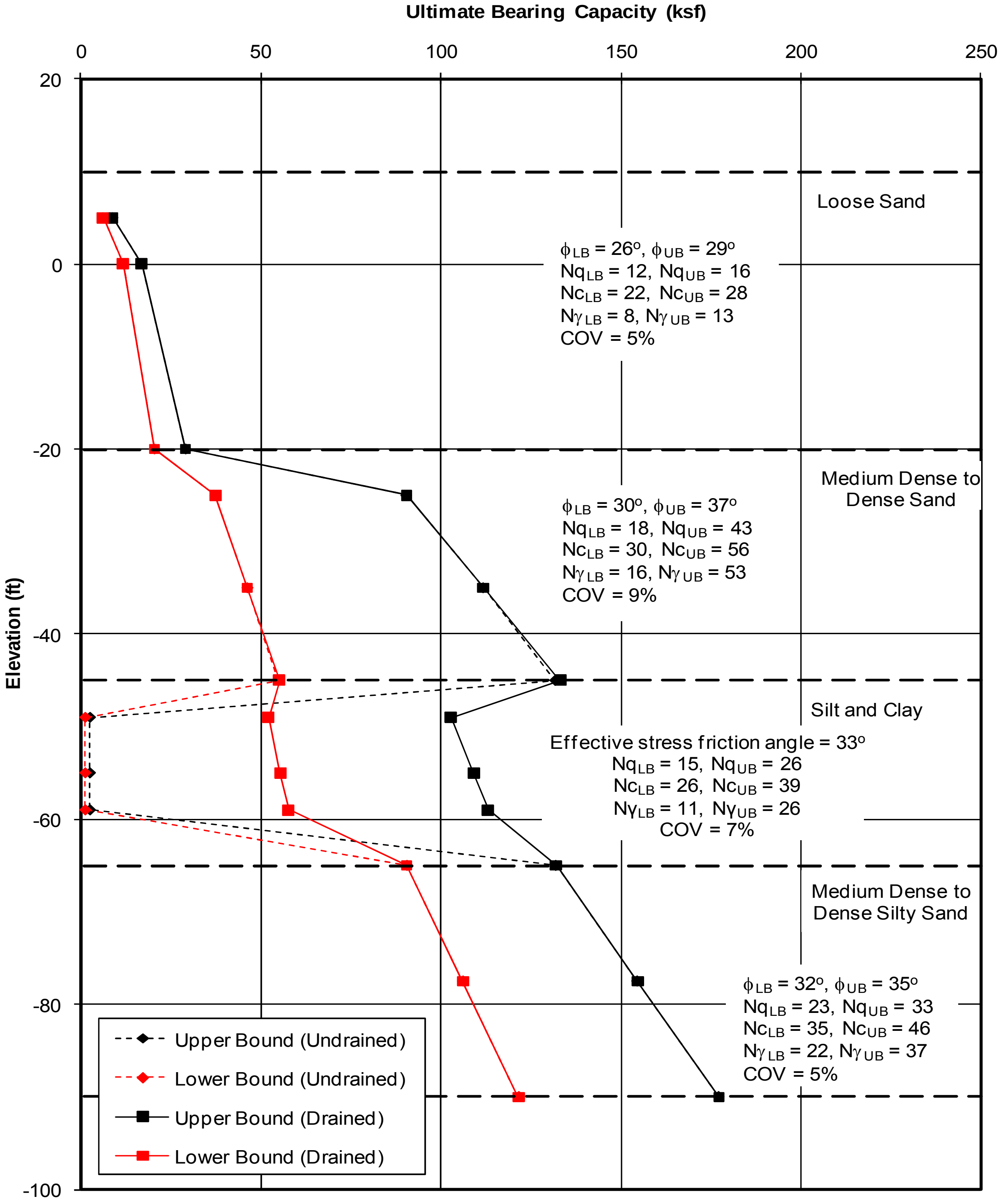
Norfolk, Virginia

FIGURE A-8



ULTIMATE BEARING CAPACITY - PROFILE 1
City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE A-9



ULTIMATE BEARING CAPACITY - PROFILE 2

City-wide Coastal Flooding Study
Norfolk, Virginia

FIGURE A-10