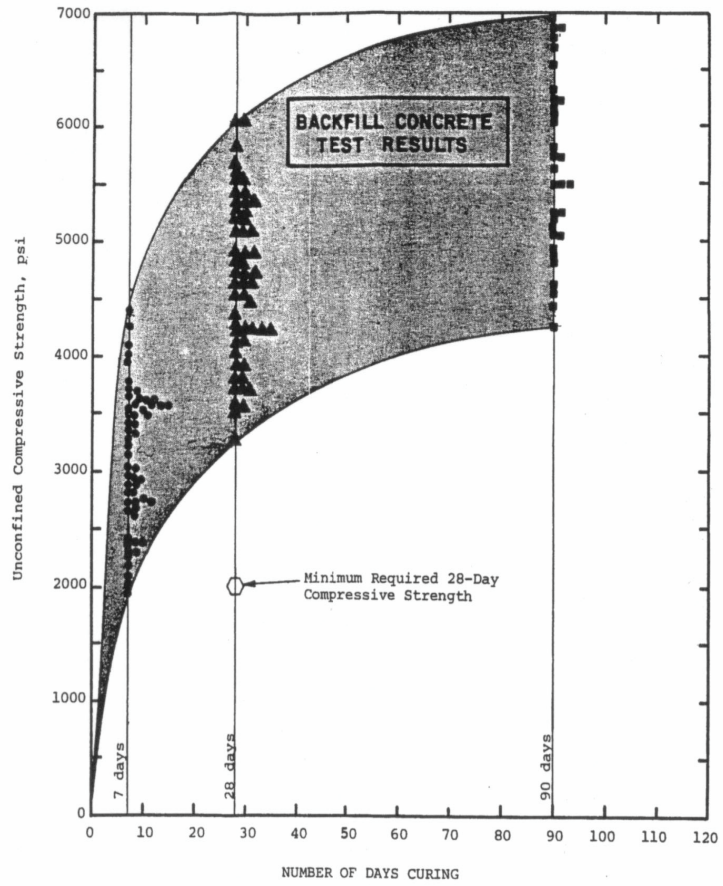


LEGEND

- ● Unconfined Compressive Strength after 7 days of curing
- ▲ Unconfined Compressive Strength after 28 days of curing
- ■ Unconfined Compressive Strength after 90 days of curing

NOTE:

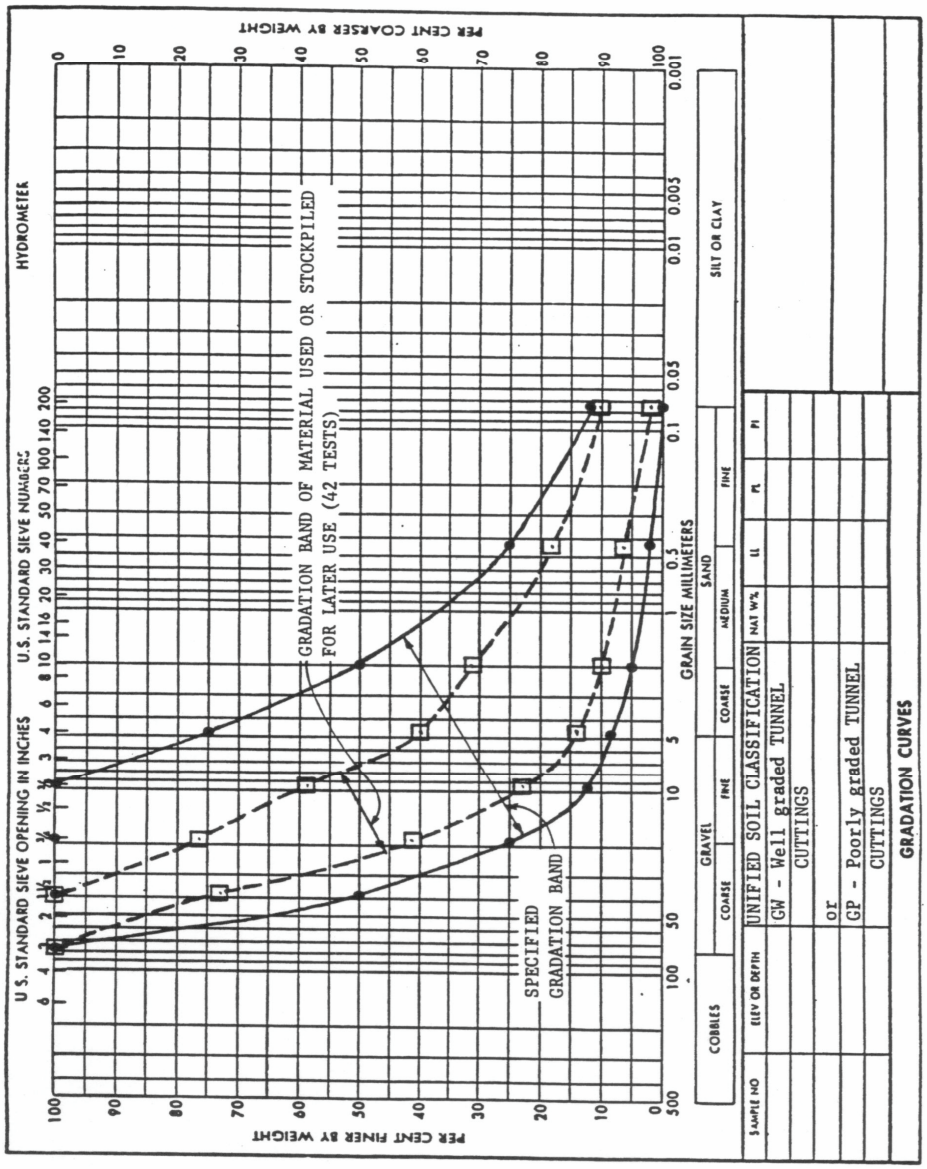
Test results shown are typical for BACKFILL CONCRETE placed in various safety-related areas of the plant site during the period May 30, 1978 to October 30, 1980. Tests performed according to ASTM C39-71.

NOTE:

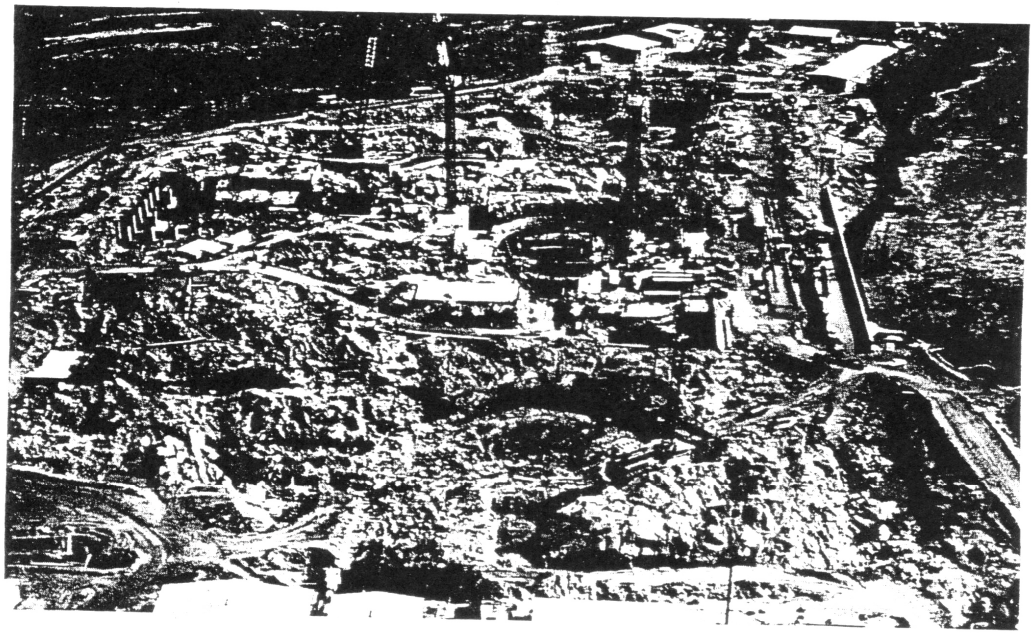
These test results pertain to BACKFILL CONCRETE placed around the outer walls of the following safety-related structures on the dates indicated:

Structure	Date of Pour
Waste Processing Bldg.	May 30,31, 1978 December 19, 1979 October 30, 1980
Diesel Gen. Bldg., Unit 1	March 23,27, 1979 May 4,8,22,23,24,29,30, 1979
Fuel Storage Bldg., Unit 1	September 21,25, 1979 March 12, 1980
Primary Aux. Bldg., Unit 1	May 16,18, 1979
Service Water Pumphouse	January 4, 1980 July 14, 1980
Containment Bldg., Unit 1	June 4,28, 1979 July 3,6, 1979
Control Bldg., Unit 1	March 8, 1979 June 1, 1979

Backfill Concrete Test Results

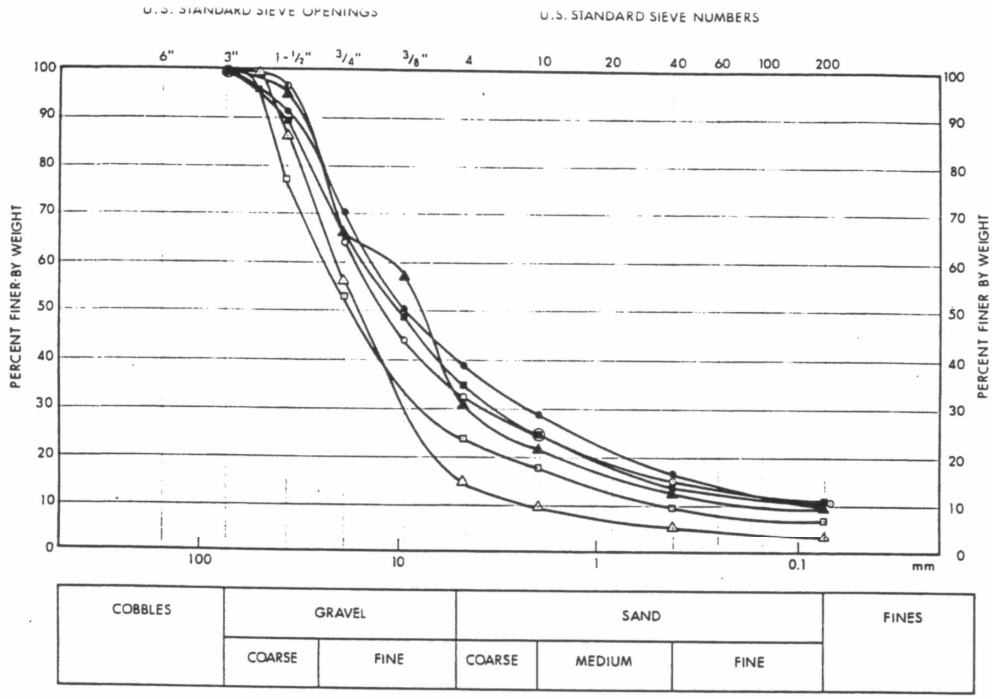


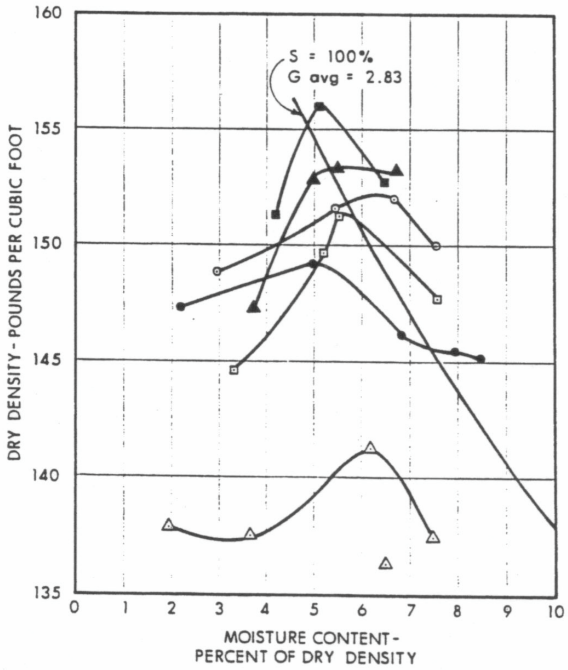
SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT	Tunnel Cuttings	
		Figure 2.5-61



SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT	Overall View of Foundation Excavations, Looking East from Unit 2 toward Unit 1	
		Figure 2.5-62

SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT	Foundation Excavation for Service and Circulating Water Pumphouse Looking North	
		Figure 2.5-63

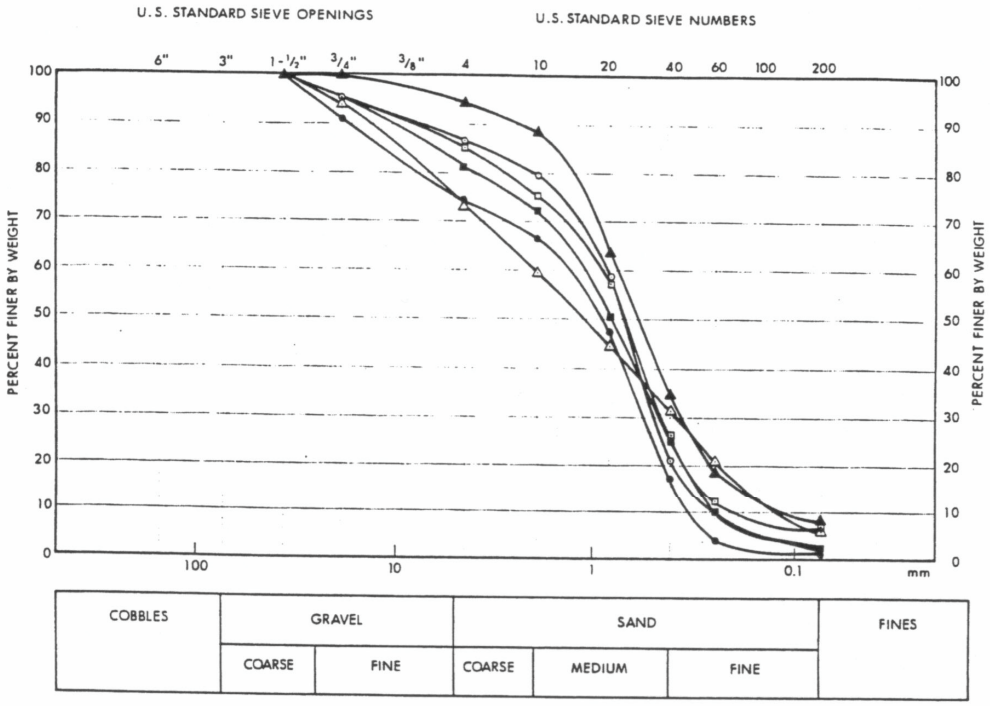


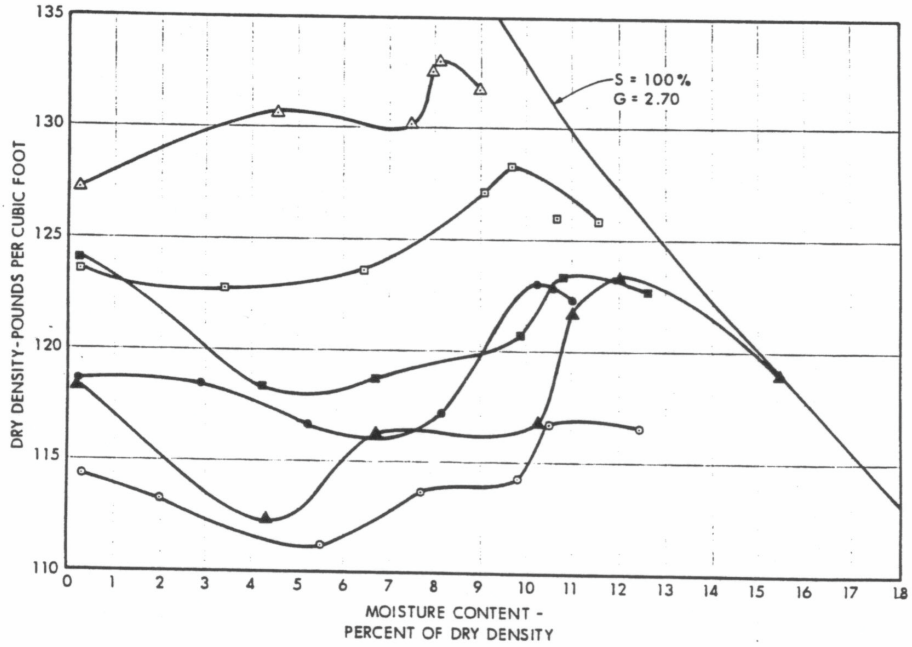


NOTE :

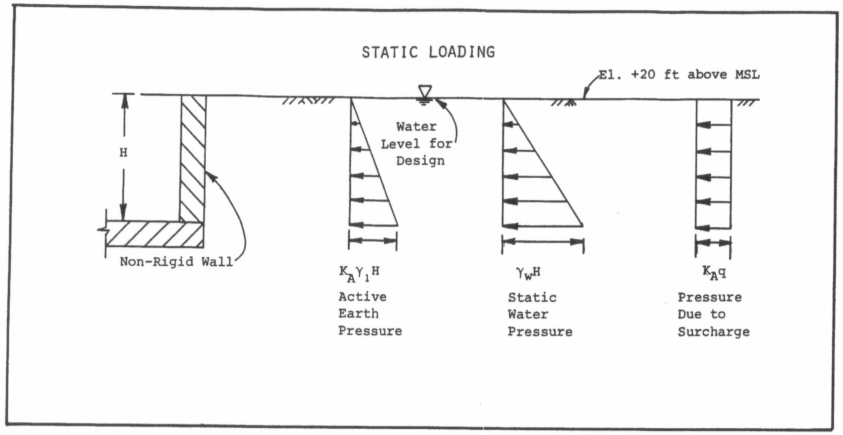
PEAKS OF SOME OF THE PROCTORS OVER THE 100% SATURATION LINE ARE DUE TO MATERIAL HAVING HIGHER SPECIFIC GRAVITY.

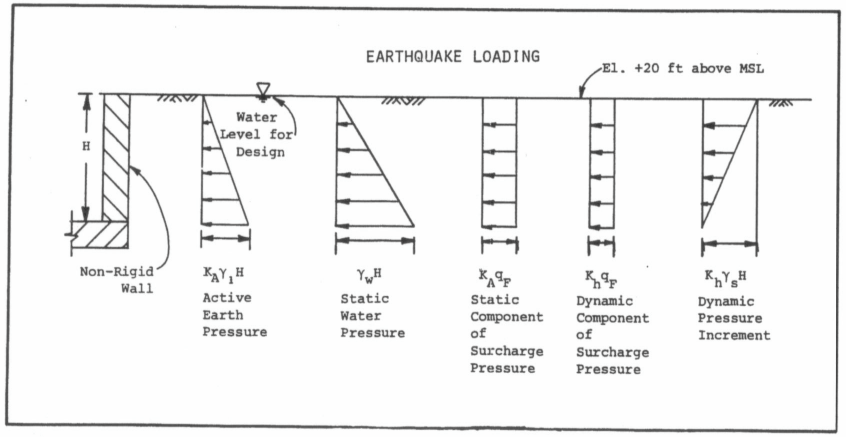
SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT	Summary Plot of Compaction Curves for Tunnel Cuttings (March to September 1979)	
		Figure 2.5-64





SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT	Summary Plot of Compaction Curves for Offsite Borrow (June to December 1979)	
		Figure 2.5-65





NOTATION

H = Depth of wall below grade, ft.

γ_1 = Buoyant Unit Weight, use 62.5 pcf for offsite borrow

γ_s = Saturated Unit Weight, use 125 pcf for offsite borrow

γ_w = Unit Weight of water, use 62.5 pcf

q = Live Load Surcharge = 500 psf minimum

q_F = Fixed or Permanent Surcharge, psf (where applicable)

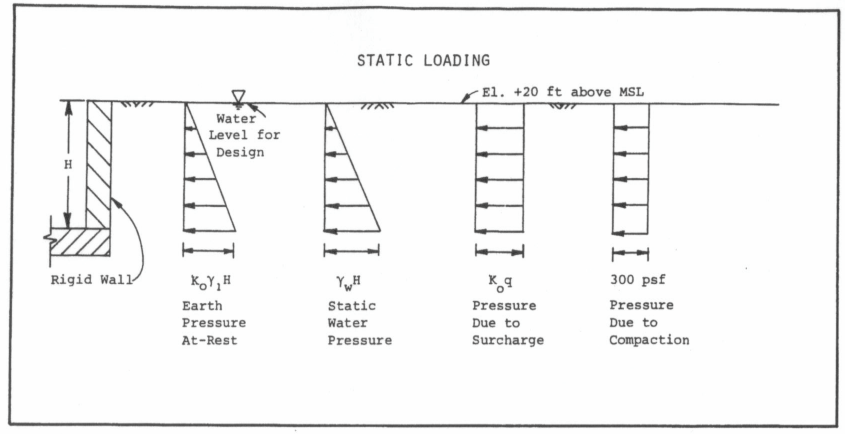
K_A = Coefficient of Active Earth Pressure, use K_A =0.30

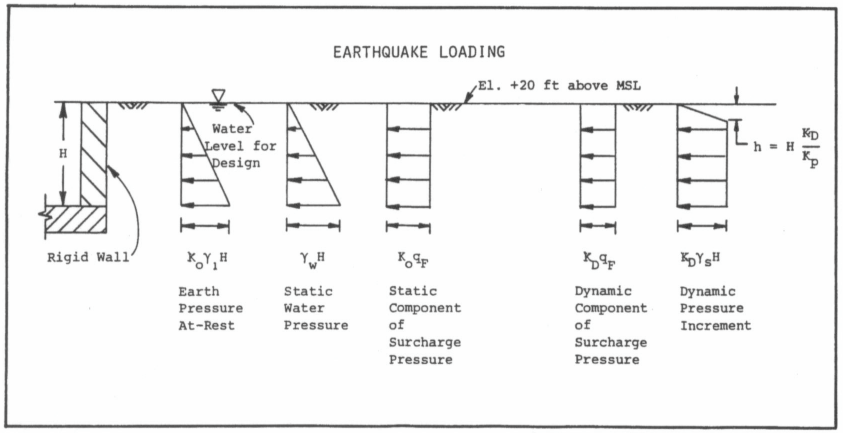
K_h = Coefficient of Dynamic Earth Pressure, use K_h = 0.19 for SSE
K_h = 0.10 for OBE

NOTES

1. A non-rigid wall is defined as a retaining wall which is not supported at the top by floors, etc., and can deflect under earth pressure.
2. Finished plant grade is +20 ft MSL. Design groundwater level is El. +20 ft MSL (refer to Section 2.5.4.6).
3. See Fig. 2.5-53 for lateral loads on rigid walls.

SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT	Lateral Loading Diagrams for Nonrigid Walls
	Figure 2.5-66





NOTATION

H = Depth of wall below grade, ft.

γ_1 = Buoyant Unit Weight, use 62.5 pcf for offsite borrow

γ_s = Saturated Unit Weight, use 125 pcf for offsite borrow

γ_w = Unit weight of water, use 62.5 pcf

q = Live Load Surcharge = 500 psf minimum

q_F = Fixed or Permanent Surcharge, psf (where applicable)

K_o = Coefficient of At-Rest Earth Pressure, use $K_o = 0.5$

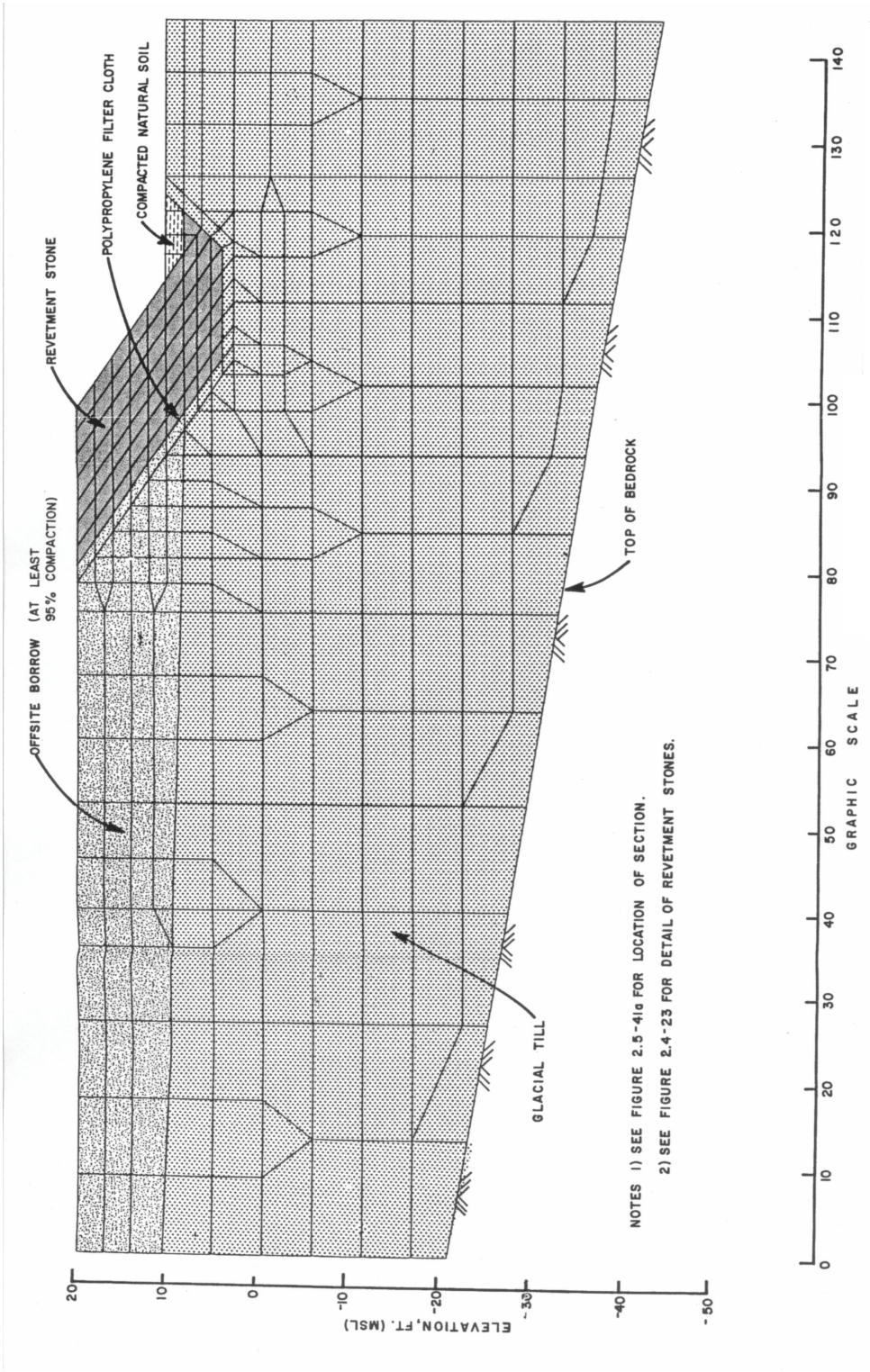
K_p = Coefficient of Passive Earth Pressure, use $K_p = 3.3$

K_D = Coefficient of Dynamic Earth Pressure, use $K_D = 0.28$ for SSE
$K_D = 0.15$ for OBE

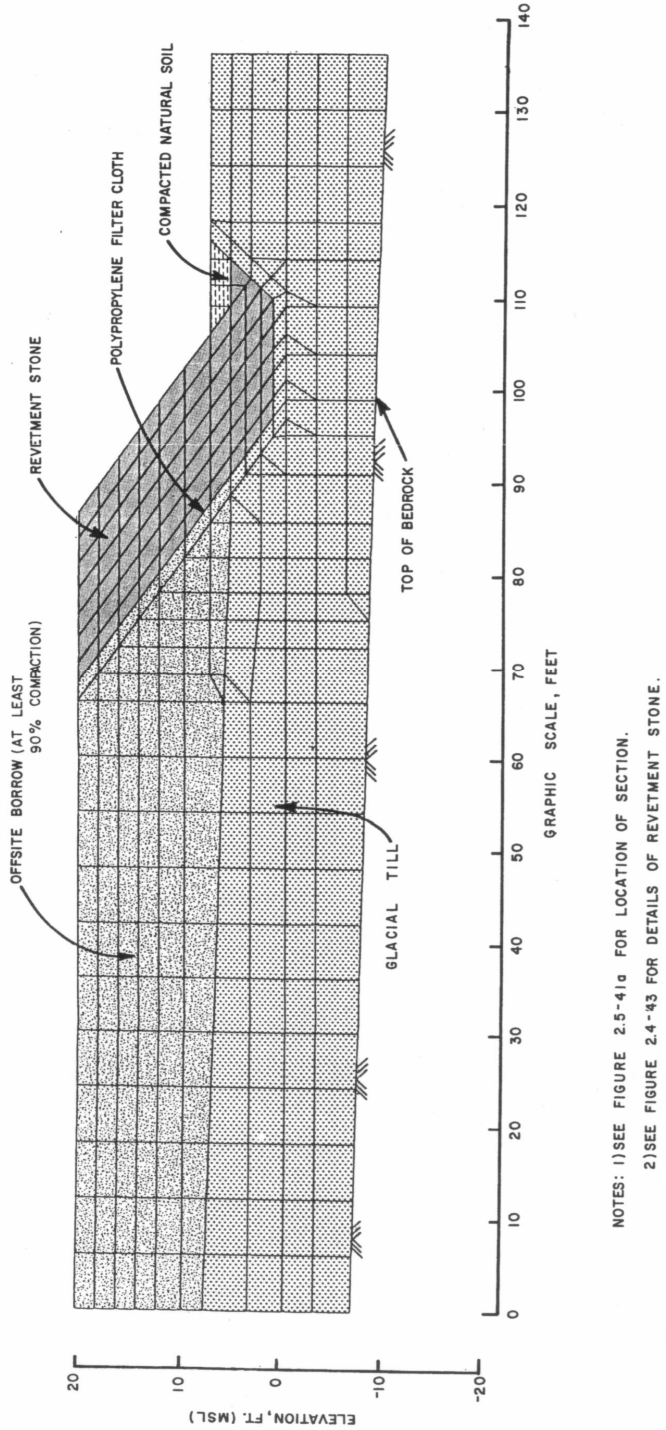
NOTES

1. A rigid wall is defined as a foundation wall supported and effectively restrained by the floors, walls, etc., which cannot deflect under earth pressure.
2. Finished plant grade is +20 ft MSL. Design groundwater level is El. +20 ft MSL (refer to Section 2.5.4.6).
3. See Fig. 2.5-52 for lateral loads on non-rigid walls.

SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT	Lateral Loading Diagrams for Rigid Walls
	Figure 2.5-67



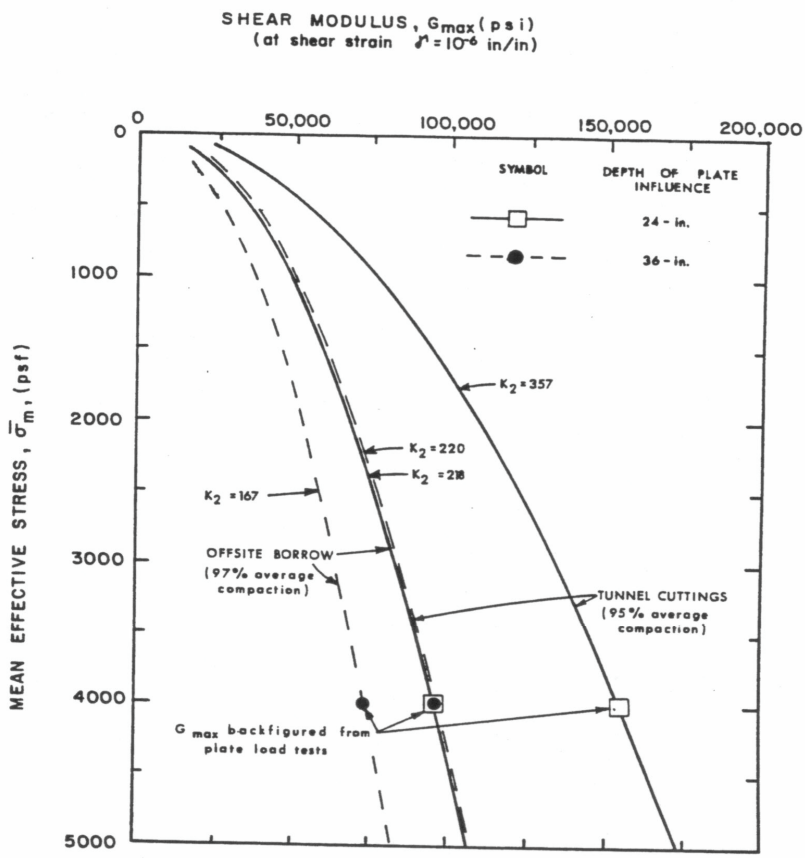
SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT	Soil Profile and Finite Mesh Revetment A – Deepest Soil Deposit Cross Section Q-Q	
		Figure 2.5-68



SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT	Soil Profile and Finite Mesh Revetment A – Highest Section Cross Section R-R	
		Figure 2.5-69

NOTES
1) See Figs. 2.5-54 and 2.5-55 for exact geometry of sections analyzed.
2) See Fig. 2.4-23 for details of revetment stone.
3) Displacements for Wedges 2 and 5 were significantly lower than for the other wedges. Therefore, settlements were not analyzed for these wedges.

SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT	Trial Wedges and Method of Settlement Analysis Revetment	
		Figure 2.5-70



NOTES: 1. See FSAR text, Subsection 2.5.4.7 for description of method used to backfigure G_{max} from plate load tests.

2. Curves for G_{max} vs $\bar{\sigma}_m$ were generated from the plate load test data using the relationship $G_{2max} = G_{1max}\sqrt{\bar{\sigma}_{2m}/\bar{\sigma}_{1m}}$ with G_1 and $\bar{\sigma}_1$ being the plate load test values.

3. Values of G for shear strain levels greater than 10^{-6} in./in. can be obtained using the average modulus reduction curve for sands presented in Seed and Idriss (1970).

4. Values of K_2 for use in the equation $G_{max}=1000\ K_2(\bar{\sigma}_m)^{1/2}$ are shown next to each curve.

SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT	Shear Modules at Low Strain Levels for Offsite Borrow and Tunnel Cuttings	
		Figure 2.5-71