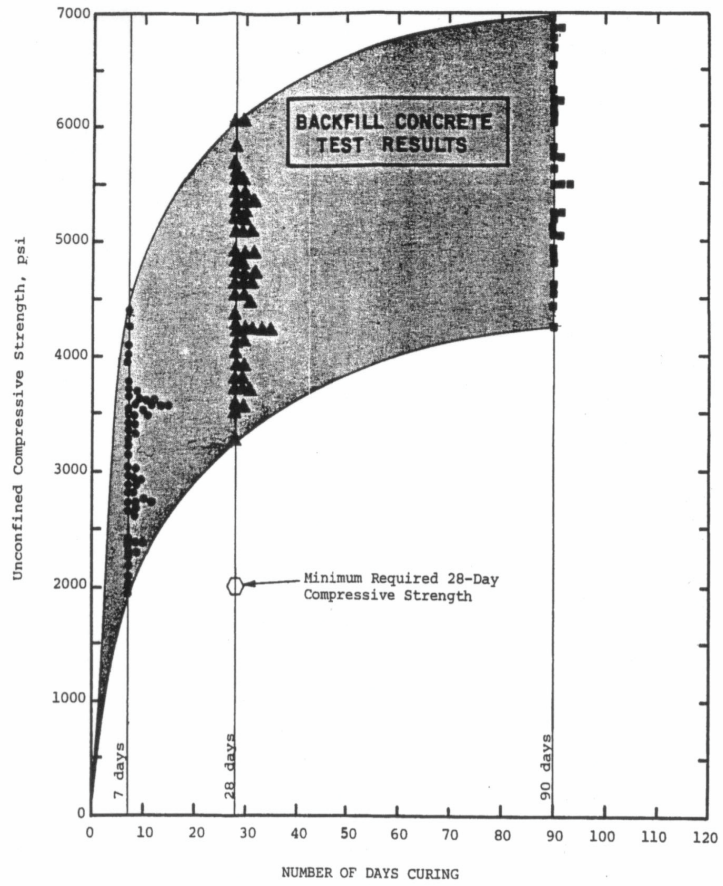



LEGEND

- ● 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.

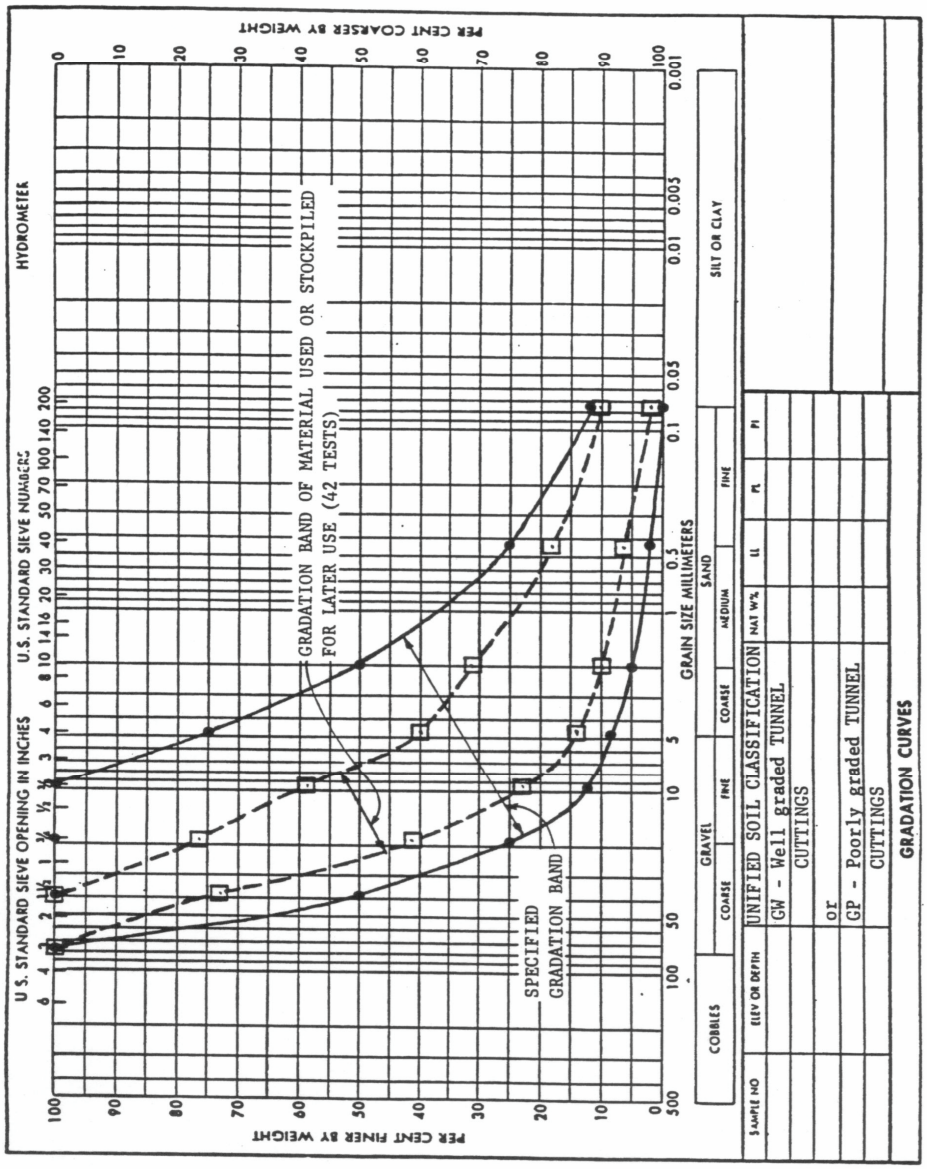
NOTE:

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

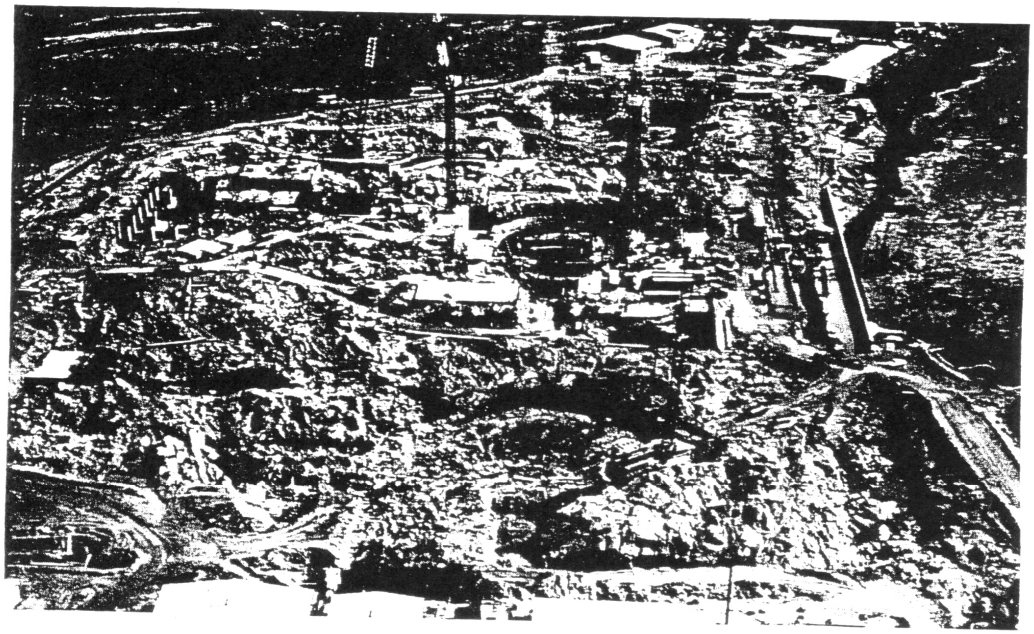
| Structure | Date of Pour |
|---|---|
| Waste Processing Bldg. | May 30,31, 1978<br>December 19, 1979<br>October 30, 1980 |
| Diesel Gen. Bldg., Unit 1 | March 23,27, 1979<br>May 4,8,22,23,24,29,30, 1979 |
| Fuel Storage Bldg., Unit 1 | September 21,25, 1979<br>March 12, 1980 |
| Primary Aux. Bldg.,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



Backfill Concrete Test Results

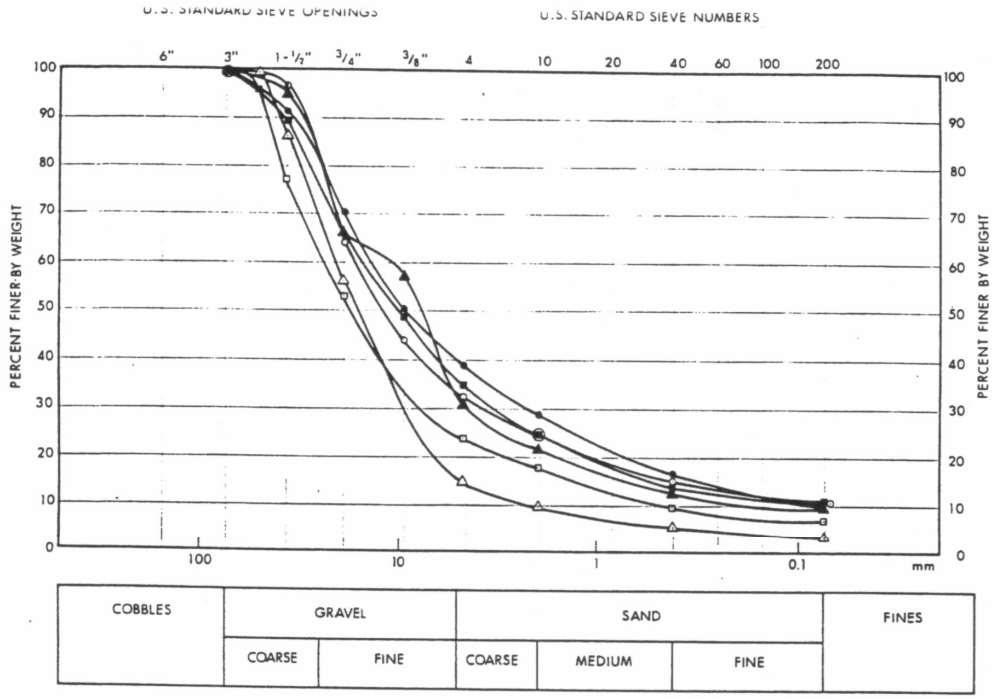


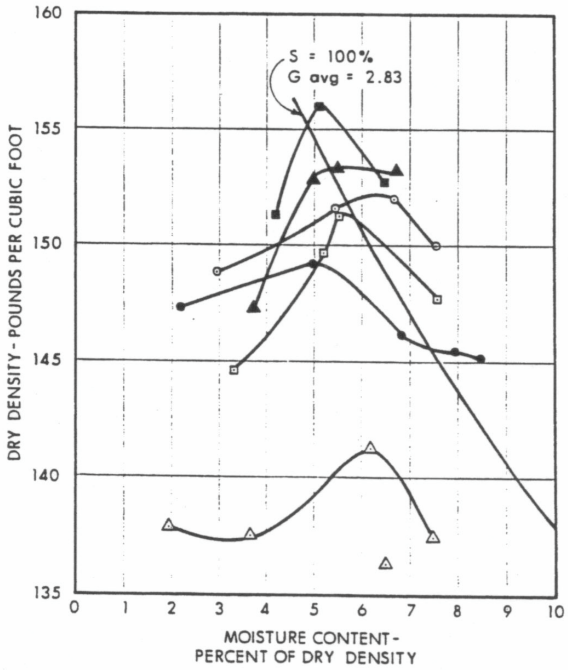

| | | |
|---|---|---|
| SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT | Tunnel Cuttings | |
| | | Figure 2.5-61 |



| | | |
|---|---|---|
| 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 |

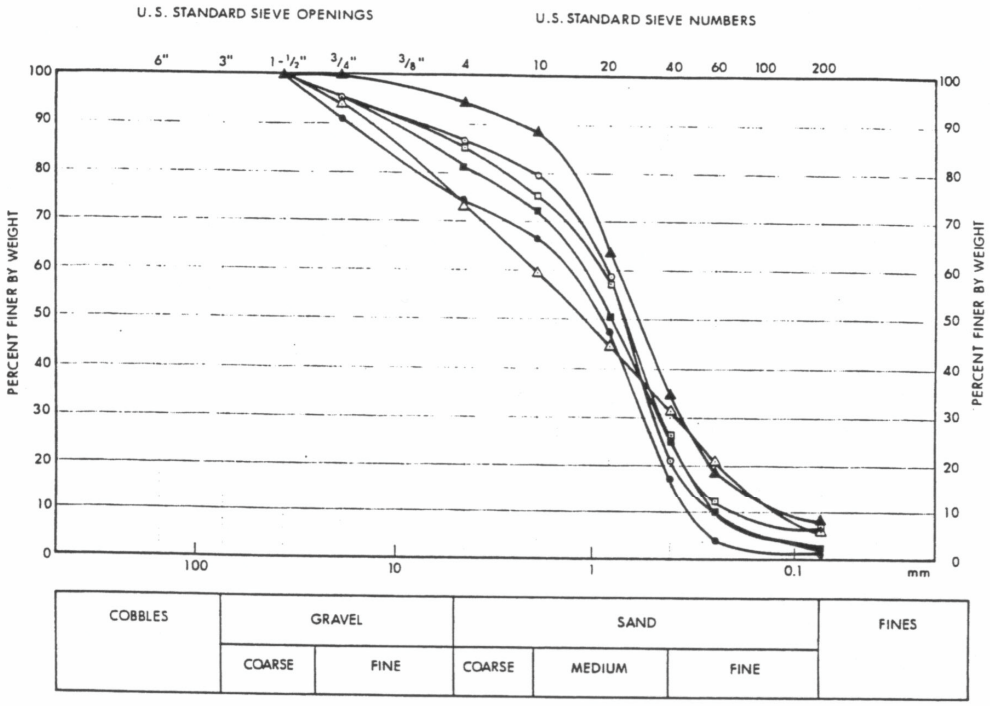


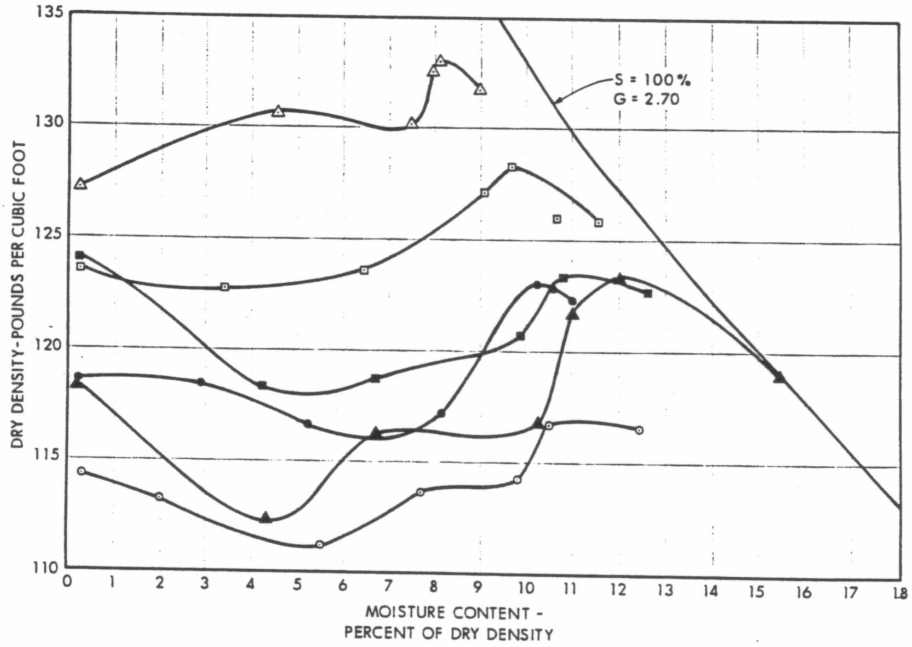




NOTE :

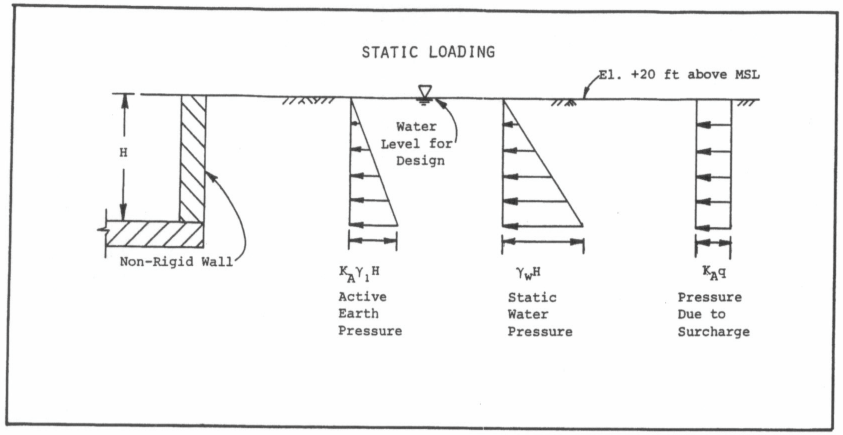
PEAKS OF SOME OF THE PROCTORS OVER THE 100% SATURATION LINE ARE DUE TO MATERIAL HAVING HIGHER SPECIFIC GRAVITY.

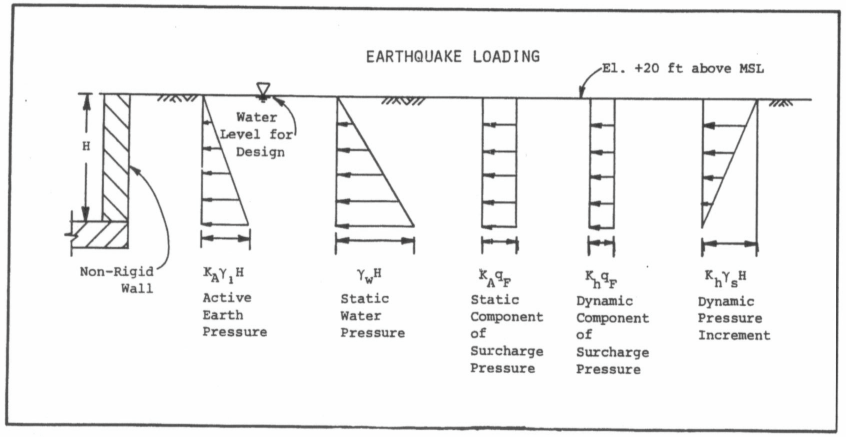
| | | |
|---|---|---|
| SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT | Summary Plot of Compaction Curves for Tunnel Cuttings (March to September 1979) | |
| | | Figure 2.5-64 |




| | | |
|---|---|---|
| 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.

$\gamma_1$ = Buoyant Unit Weight, use 62.5 pcf for offsite borrow

$\gamma_s$ = Saturated Unit Weight, use 125 pcf for offsite borrow

$\gamma_w$ = Unit Weight of water, use 62.5 pcf

$q$ = Live Load Surcharge = 500 psf minimum

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

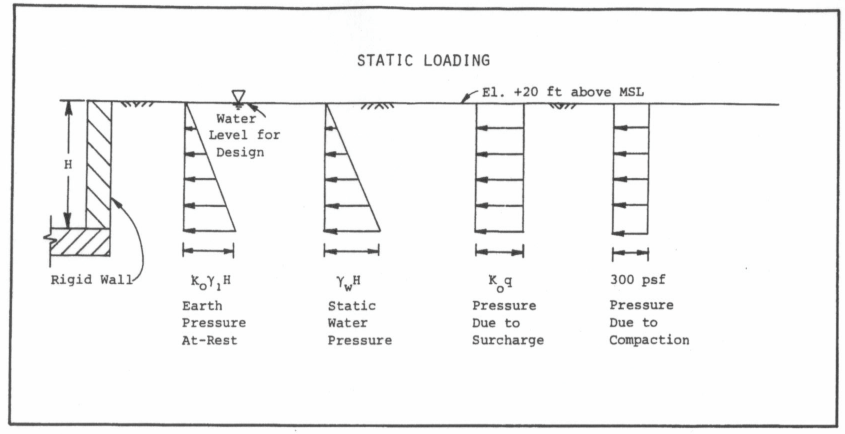
$K_A$ = Coefficient of Active Earth Pressure, use $K_A$ =0.30

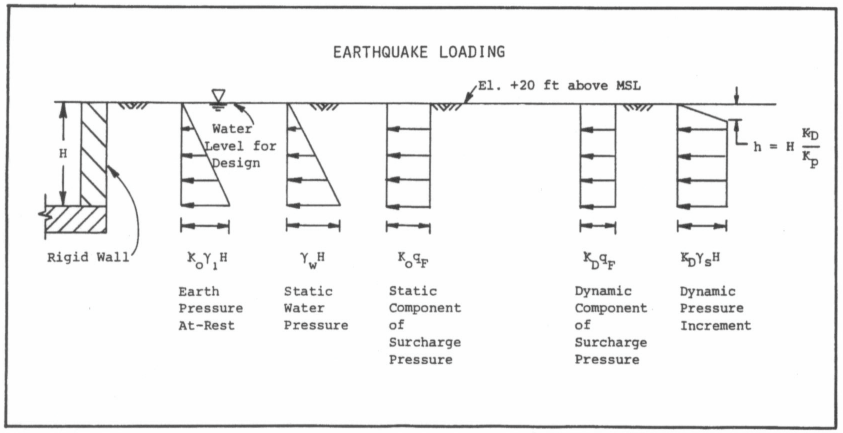
$K_h$ = Coefficient of Dynamic Earth Pressure, use $K_h$ = 0.19 for SSE
$K_h$ = 0.10 for OBE

## 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<br>UPDATED FINAL SAFETY<br>ANALYSIS REPORT | Lateral Loading Diagrams for Nonrigid Walls |
| | Figure 2.5-66 |




## NOTATION

$H$ = Depth of wall below grade, ft.

$\gamma_1$ = Buoyant Unit Weight, use 62.5 pcf for offsite borrow

$\gamma_s$ = Saturated Unit Weight, use 125 pcf for offsite borrow

$\gamma_w$ = 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 $K_o = 0.5$

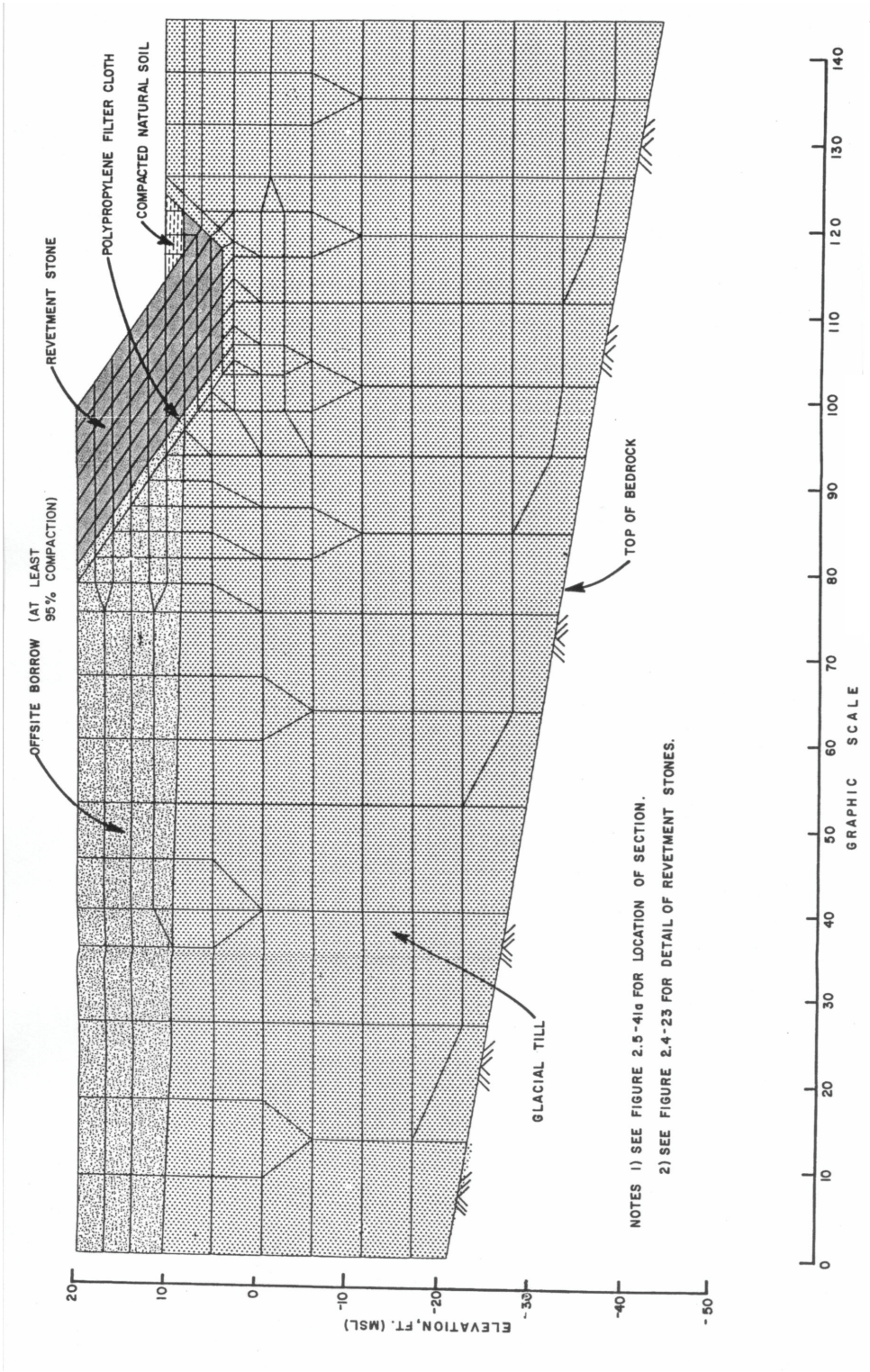
$K_p$ = Coefficient of Passive Earth Pressure, use $K_p = 3.3$

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

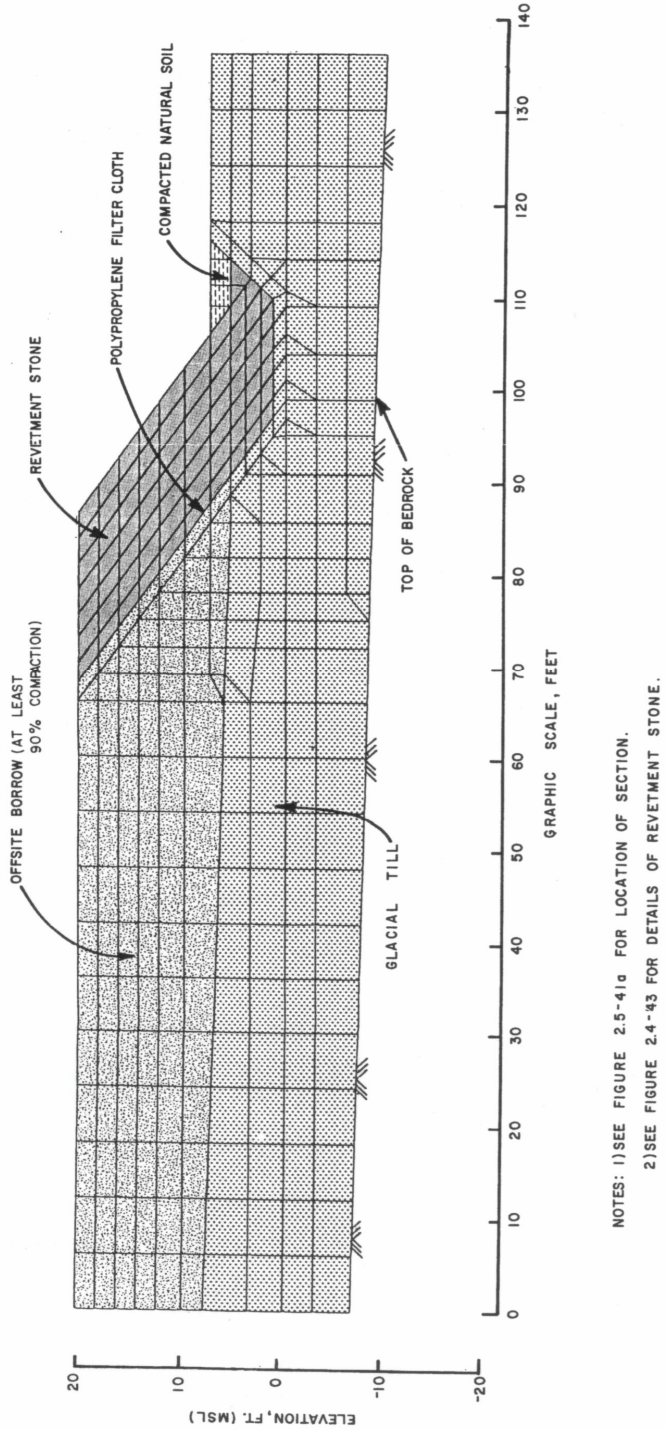
## NOTES

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<br>UPDATED FINAL SAFETY<br>ANALYSIS REPORT | Lateral Loading Diagrams for Rigid Walls |
| | Figure 2.5-67 |



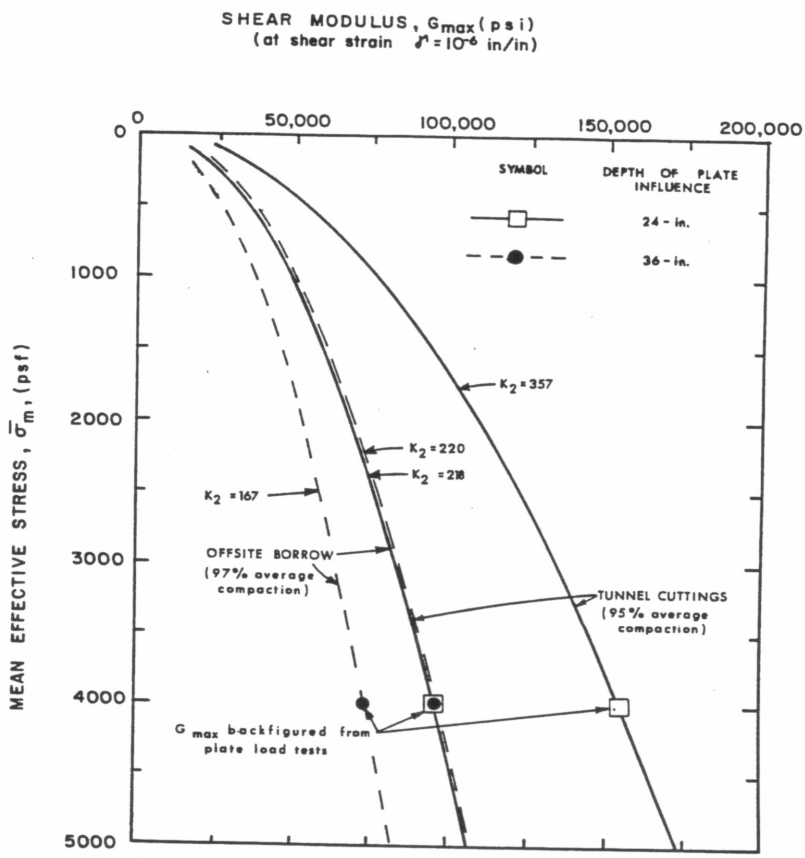

| | | |
|---|---|---|
| SEABROOK STATION UPDATED FINAL SAFETY ANALYSIS REPORT | Soil Profile and Finite Mesh Revetment A – Deepest Soil Deposit Cross Section Q-Q | |
| | | Figure 2.5-68 |




| | | |
|---|---|---|
| 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 |