Private Fuel Storage, L.L.C.

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July 27, 2000

**CALCULATION PACKAGE SUBMITTAL**
**DOCKET NO. 72-22 / TAC NO. L22462**
**PRIVATE FUEL STORAGE FACILITY**
**PRIVATE FUEL STORAGE L.L.C.**

Enclosed for your use please find the following calculations for the Private Fuel Storage Facility (PFSF):

- Calculation No. 05996.01 – G(B) – 11, Revision 1, entitled "Dynamic Settlements of the Soils Underlying the Site"
- Calculation No. 05996.02 – G(C) – 14, Revision 1, entitled "Static Settlement of the Canister Transfer Building Supported on a Mat Foundation"

These calculations were revised based on comments received from the NRC during the July 21, 2000 teleconference. The discussion of the effects of estimated settlements has been revised to more clearly indicate that the estimated settlements will not adversely affect the function or integrity of the Canister Transfer Building.

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

Sincerely

J L Cooper for

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

Enclosure

NMSS01 Public

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# CALCULATION TITLE PAGE

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|---|---|
| CLIENT & PROJECT<br>PRIVATE FUEL STORAGE, LLC - PRIVATE FUEL STORAGE FACILITY | PAGE 1 OF ~~6~~ 15 + 5 PP OF ATTACH'S |
| CALCULATION TITLE (Indicative of the Objective):<br>DYNAMIC SETTLEMENTS OF THE SOILS UNDERLYING THE SITE | QA CATEGORY (√)<br>☒ I - NUCLEAR SAFETY RELATED<br>☐ II ☐ III ☐ ____ OTHER |

| CALCULATION IDENTIFICATION NUMBER | | | | |
|---|---|---|---|---|
| J. O. OR W.O. NO. | DIVISION & GROUP | CURRENT CALC. NO. | OPTIONAL TASK CODE | OPTIONAL WORK PACKAGE NO. |
| 05996.01 | G(B) | 11 | | |

| * APPROVALS - SIGNATURE & DATE | | | | | | |
|---|---|---|---|---|---|---|
| PREPARER(S)/DATE(S) | REVIEWER(S)/DATE(S) | INDEPENDENT REVIEWER(S)/DATE(S) | REV. NO. OR NEW CALC NO. | SUPERSEDES * CALC. NO. OR REV. NO. | CONFIRMATION *REQUIRED (√) YES | NO |
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## RECORD OF CHANGES

| Rev No. | Description of Changes | Pages Revised | Pages Added | Pages Replaced |
|---|---|---|---|---|
| 0 | Original Issue | N/A | N/A | N/A |
| 1 | Respond to NRC RAI No. 1 – Question 2-8 | | | ALL |

**REASON FOR REV 1**

Rev. 1 was prepared to document the estimated dynamic settlements of the nonplastic silts in the upper layer of soils at the site that were reported in response to Question 2-8 of NRC Request for Additional Information No. 1.

## OBJECTIVE

Estimate the dynamic settlement of the soils underlying the proposed Private Fuel Storage Facility (PFSF) at the Skull Valley, UT site due to shaking caused by the design earthquake.

## DATA/ASSUMPTIONS

Figure 1 presents a generalized subsurface profile, which was developed based on the borings that were drilled in late 1996. The groundwater table was not encountered in these borings, the deepest of which were drilled to depths of 100 ft. In addition, seismic refraction surveys indicate that the groundwater table is greater than ~100 ft below grade at the site (see p11 of Geosphere Midwest, 1997).

The top ~30 ft of the profile consists of silt, silty clay, and clayey silt. As documented in Calculation 05996.01-G(B)-05-0, the median blow count for this material is ~14 blows per ft, indicating that it is "stiff", it appears to be weakly cemented, and undrained triaxial tests on the clayey silt indicates that it has a cohesion of greater than 2000 psf.

As indicated in PFSF Report No. 0559602-G(B)-2, Rev 2, Addendum to Geotechnical Data Report Attachment 2 - Geotechnical Laboratory Testing SWEC (1998), additional Atterberg limits tests were performed on split-spoon samples obtained in Borings A-2, B-3, C-4, and D-4. These results, shown in Table 3 of that report, confirmed that Samples S3 in Borings A-2 and C-4, and Sample S3A in Boring D-4 were essentially nonplastic. However, these Atterberg limits indicate that Samples S1 in Borings A-2 and B-3 and Sample S2 in Boring D-4, which were described as nonplastic in the borings presented in the original submittal of the SAR, are actually slightly or moderately plastic.

A review of the sample descriptions included in the boring logs indicates that only two samples of nonplastic silt are characterized as "loose". These two samples, Samples S-1 in Borings AR-2 and AR-3, were both obtained at the ground surface along the access road. All other nonplastic silt samples for which density is included in the description are characterized as being dense, very dense, or compact.

The following discussion applies to the SPT samples obtained in the upper layer of silt, silty clay, and clayey silt in the areas of the site proposed for the cask storage pads, the canister transfer building, and the security and health physics building. It excludes the samples obtained at the ground surface, which represent soils that will be excavated for construction of the facilities.

The borings in these areas (Borings A-1 through A-4, B-1 through B-4, C-1 through C-4, D-1 through D-4, E-3, and E-4) indicate that the upper layer (~30 ft) consists mostly of soils with some plasticity, especially in the cask storage pad area. Table 1 identifies the thickness of nonplastic silts in these borings in the upper layer of the profile. As shown, the average thickness of nonplastic silts in these borings is ~10 ft. Borings A-2 through A-4, B-1 through B-3, C-1 through C-3, and D-3 have less than or equal to 10 ft of nonplastic silts. Borings A-1 in the northwest, D-1 and D-2 in the northeast, and B-4, C-4, D-4, and E-4 along the south have ~15 to 20 ft of nonplastic silts. Note that these nonplastic silts often include occasional thin layers of clay or slightly plastic silt, which will minimize the potential for dynamically induced settlement.

A total of 64 SPT samples of silt (ML, MH, and some "CL,ML") were obtained in the upper layer in these borings. These are identified in Table 2 and a summary of the blow count data are presented in Table 3. Of these, 31 were nonplastic and 33 exhibited some plasticity, ranging from slightly plastic to highly plastic. The N-values for the nonplastic silts in this layer ranged from 11 blows/ft to 40 blows/ft. The median N-value was 18 blows/ft, and the average was 20 blows/ft. This median N-value corresponds to a corrected blow count, $N_1$, of ~23 blows/ft, based on the relationship between penetration resistance and

relative density developed by Gibbs and Holtz (1957) for granular soils. See Figure 1 of the RELDEN User's Manual (SWEC, 1979) – copy of this figure is included as Attachment A.

Assuming that these nonplastic silts are cohesionless, they would behave more like fine sands rather than cohesive soils. In this case, Figure 7.5 of Lambe and Whitman (1969) could be used to estimate their relative density based on their penetration test blow counts. A copy of this figure is included as Attachment B. This figure presents the relationship between penetration resistance and relative density developed by Gibbs and Holtz (1957) for granular soils. This is very conservative, since a decrease in mean grain size tends to cause a decrease in SPT N-value for the same relative density, and the nonplastic silts at the site have a much smaller mean grain-size than the sand and fine sand used by Gibbs and Holtz to develop this relationship. Using the 10 psi curve in this figure, or slightly below it, which is the appropriate overburden stress for the mid-depth of this layer, fine sands having the median blow count of the nonplastic silts in this layer would be characterized as "very dense", not "loose".

As indicated on the boring logs in PFSF Report 05996.01-G(B)-2-1 (SWEC, 1997), this material is underlain by very dense, fine sands, which have uncorrected blow counts that commonly exceed 100 blows/ft. The underlying soils, which are below the groundwater table, are greater than 100 ft below grade, and the P-wave velocities (5,100 fps to 5,900 ft/sec), reported by Geosphere Midwest (1997), indicate that these soils too are very dense.

## METHOD

Dynamic settlements, as reported in the geotechnical literature, are based on two different mechanisms, depending on whether the soils are above the groundwater table or below the groundwater table. Silver and Seed (1971) developed a technique for estimating dynamic settlements of dry cohesionless sands above the groundwater table. For such soils, the dynamic settlement mechanism is compaction due to soil grain slip, and it is a function of the magnitude of the cyclic shear strain developed due to the earthquake, the applied number of cycles of this shear strain, and the relative density of the granular soils.

Dynamic settlements of the clayey silts and silty clays in the upper layer are not expected to occur at the PFSF site due to the design earthquake because the cohesion and plasticity of these materials precludes their compaction due to grain slip as a result of shaking due to the design earthquake. Dynamic settlements of the underlying very dense fine sand are not expected to occur because of their high relative density. The underlying silts, which are assumed to be below the groundwater table, also are very dense and, thus, are not expected to experience dynamically induced settlements due to shaking caused by the Design Earthquake. Furthermore, these soils are too far removed from the surface to cause problems if they were to experience dynamic settlement. Therefore, dynamic settlements of the soils underlying the proposed PFSF at Skull Valley, UT will be limited to those associated with dynamic compaction due to grain slip of the nonplastic silts within the upper 30 ft of the profile.

The method used to estimate dynamic settlements of the nonplastic silts are based on those presented in Tokimatsu and Seed (1987). As they indicate, for soils above the groundwater table, dynamic settlements are calculated based on procedures originally developed by Silver and Seed (1971), and the effects of multidirectional shaking are estimated based on studies reported by Pyke, Seed, and Chan (1975). For soils below the groundwater table, dynamic settlements are calculated based on procedures developed by Lee and Albaisa (1974); however, all of the nonplastic silts in the upper 30 ft of the profile are well above the groundwater table.

Figure 13 of Tokimatsu and Seed (1987) presents the relationship between volumetric strain due to compaction, cyclic shear strain, and corrected penetration resistance ($N_1$) of dry sands for 15 equivalent uniform strain cycles. A copy of this figure is included as Attachment C. The cyclic shear strain is

estimated based on the average cyclic shear stress due to shaking caused by the Design Earthquake and the shear modulus of the soil. Figure 13 is used to estimate the volumetric strain due to compaction for 15 equivalent uniform strain cycles. Table 4 of Tokimatsu and Seed (1987 – copy included as Attachment D) is then used to adjust for differences in the number of representative cycles of applied shear stress due to the Design Earthquake (~12 for Magnitude 7) and the 15 cycles used in Tokimatsu and Seed's studies. The dynamic settlement is calculated as the volumetric strain multiplied by the thickness of the nonplastic silts in the layer. Multidirectional effects of the earthquake are addressed by multiplying this result by 2, based on studies reported by Pyke, Seed, and Chan (1975).

## ANALYSES

The average cyclic shear stress developed in the field due to earthquake shaking is calculated as:

$$\tau_{avg} = 0.65 \bullet a_{max} \bullet \sigma_v \bullet r_d/g = 496 \text{ psf},$$

where: $a_{max}$ = 0.67 g for the Design Earthquake

$\sigma_v$ = $\gamma_{total} \bullet z$ above the groundwater table, = 15' x 80 pcf.

$\gamma_{total}$ = 80 pcf, based on p 4 of Calc 05596.01-G(B)-5, Rev 0.

$z$ = depth below grade, = 15' for mid-depth of upper 30 ft layer.

$r_d$ = 0.95 for z = 15' (stress reduction factor, which varies from 1.0 at z=0 to 0.9 at z=30')

An iterative technique is used to determine the cyclic strain in the field due to the earthquake, $\gamma_{field}$. For an assumed value of the cyclic strain, G is calculated as $G_{max} \bullet G / G_{max}$, where $G / G_{max}$ for the nonplastic silt is estimated using the curve for PI=0 presented in Figure 6 of Vucetic and Dobry (1991). A copy of this figure is included as Attachment E.

$G_{max}$ equals 1400 ksf, as indicated in SAR Table 2.6-1 for Layer 1, which was developed in Calculation 05996.01-G(B)-1, Rev 3 based on the results of the seismic refraction survey performed by Geosphere (1997), which is included in SAR Appendix 2B.

The following table presents the results of these iterations.

## Determination of Cyclic Shear Strain Due to the Design Earthquake

| Iteration No. | $\gamma_{assumed}$ x$10^{-4}$ in./in. | $G / G_{max}$ | G ksf | $\gamma_{field}$ x$10^{-4}$ in./in. | $\Delta\gamma$ % |
|---|---|---|---|---|---|
| 1 | 10 | 0.250 | 350 | 14.2 | 30 |
| 2 | 15 | 0.200 | 280 | 17.7 | 15 |
| 3 | **20** | **0.166** | **232** | **21.4** | **6** |

The cyclic strain in the field, $\gamma_{field}$, is calculated as $\tau_{avg} / G$. Note, it is approximately equal to the assumed cyclic strain for Iteration No. 3; therefore, additional iterations are not required, and $\gamma_{field}$ is ~21 x $10^{-4}$ in./in., or 0.21%.

The volumetric strain due to compaction from 15 cycles is estimated as a function of this cyclic shear strain and $N_1$ of ~23 blows/ft based on Figure 13 of Tokimatsu & Seed (1987). This results in a volumetric strain, $\varepsilon_{c,N=15}$, of 0.17%.

The Design Earthquake is magnitude 7 (SAR Section 2.6.2.3). Table 4 of Tokimatsu & Seed (1987) indicates this corresponds to ~12 cycles of loading and that volumetric strain ratio, $\varepsilon_{c,N=12} / \varepsilon_{c,N=15}$, should be ~0.9. Therefore, the volumetric strain corresponding to the Design Earthquake is $\varepsilon_{c,N=12}$, which is 0.9 x 0.17%, or 0.15%.

$$\varepsilon_c = \frac{\Delta\rho_{dyn}}{\Delta H}$$

where $\Delta\rho_{dyn}$ is the dynamic settlement of the layer, and $\Delta H$ is the thickness of the layer.

The thickness of the nonplastic silts in the upper layer is conservatively estimated to be 20 ft, based on the discussion presented above. Therefore, for unidirectional shaking,

$$\Delta\rho_{dyn,1} = \Delta H \times \varepsilon_{c,N=12} = 20 \text{ ft} \times 12 \text{ in./ft} \times 0.15\% / 100\% = 0.36 \text{ inches.}$$

The dynamic settlement is multiplied by 2 to account for multidirectional shaking due to the earthquake, as recommended by Pyke, Seed, and Chan (1975). This results in an estimated dynamic settlement of the nonplastic silts in the upper layer of 0.72 inches.

## DISCUSSION OF RESULTS

As indicated in SWEC (1998), a considerable amount of calcium carbonate is present in these soils, as evidenced by a vigorous reaction upon application of hydrochloric acid to these soils. Therefore, these soils are believed to be cemented, the result of carbonate cement bonding of the silt and clay-size particles, imparting cohesion to these soils.

The nonplastic silts in the upper 30-ft thick layer of the subsurface profile, as evidenced by the SPT data, are not loose. The dense nature of these soils, which is most likely the result of carbonate cement bonding of the silt particles, minimizes the potential for dynamically induced settlements due to the Design Earthquake. Ignoring the beneficial effect that cementing of these particles would have toward minimizing dynamically induced settlements, the total dynamic settlement is conservatively estimated to be less than ¾ of an inch.

This estimated dynamic settlement was determined based on the thickness of nonplastic silts in areas where the nonplastic silts are thickest, not on an average or median thickness. In addition, it ignores the fact that these nonplastic silts are stratified with layers of clay and clayey silt, which will minimize the potential for dynamically induced settlements. Thus, this estimated dynamic settlement is very conservative.

Dynamic settlements will be much less than this over most of the cask storage pad area, since most of the soils in this area are not nonplastic. Rather, these soils are sufficiently stiff and cohesive that they will not experience dynamic compaction due to the shaking caused by the Design Earthquake.

## CONCLUSIONS

Dynamic settlements of the clayey silts and silty clays in the upper 30-ft thick layer of the subsurface profile at the PFSF site are not expected to occur due to the Design Earthquake because the cohesion and plasticity of these materials precludes their compaction due to grain slip as a result of shaking due to the design earthquake. Dynamic settlements of the underlying very dense fine sand are not expected to occur because of their high relative density. The underlying silts, which are assumed to be below the groundwater table, also are very dense and, thus, are not expected to experience dynamically induced settlements due to shaking caused by the Design Earthquake. Furthermore, these soils are too far removed from the surface to cause problems if they were to experience dynamic settlement. Therefore, dynamic settlements of the soils underlying the proposed PFSF at Skull Valley, UT will be limited to those associated with dynamic compaction due to grain slip of the nonplastic silts within the upper 30 ft of the profile.

The total dynamic settlement of these nonplastic silts is estimated to be less than ¾ of an inch, even when conservatively assuming that they are not cemented, ignoring the fact that they are stratified with occasional layers of clay and clayey silt, which will minimize the potential for dynamically induced settlements, and that they are calculated using the maximum thickness of nonplastic silts within the upper layer, not an average or median thickness. Dynamic settlements of this magnitude are not expected to adversely affect the performance of the facilities.

## REFERENCES


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- SWEC (1979), "Relative Density from Standard Penetration Tests (RELDEN)", User's Manual, Stone & Webster Engineering Corporation, Boston, MA.
- SWEC (1997), PFSF Report No. 05996.01-G(B)-2 Rev 1, "Geotechnical Data Report," Stone & Webster Engineering Corporation, Boston, MA, 1997 – Included as Attachment 2 of SAR Appendix 2A.
- SWEC (1998), PFSF Report No. 0599602-G(B)-2 Rev 2, "Addendum to Geotechnical Data Report Attachment 2 – Geotechnical Laboratory Testing," Stone & Webster Engineering Corporation, Boston, MA – Included as Attachment 3 of SAR Appendix 2A.
- Tokimatsu, A. M., and H. B. Seed, 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking," *Journal of the Geotechnical Engineering Division*, ASCE, 113(8), 861-878.
- Vucetic, M., and R. Dobry, 1991, "Effect of Soil Plasticity on Cyclic Response," *Journal of the Geotechnical Engineering Division*, ASCE, 117(1), 89-107.

## Table 1

## Approximate Thickness of Nonplastic Silts in PFSF Borings

| **Boring** | **Thickness of NP Silts** |
|---|---|
| A-1 | 17 |
| A-2 | 10 |
| A-3 | 5 |
| A-4 | 2 |
| B-1 | 5 |
| B-2 | 0 |
| B-3 | 5 |
| B-4 | 20 |
| C-1 | 0 |
| C-2 | 5 |
| C-3 | 0 |
| C-4 | 17 |
| D-1 | 15 |
| D-2 | 15 |
| D-3 | 5 |
| D-4 | 17 |
| E-3 | 12 |
| E-4 | 18 |
| ***Average*** | ***9.3*** |

## Table 2

## N-Values of Nonplastic Silts in Upper Layer at PFSF

| Boring & | Sample | Depth (ft) | | Rec (in.) | Blows | N-Value | USC | Layer | NP? |
|---|---|---|---|---|---|---|---|---|---|
| A-1 | S 1 | 0.00 | 1.50 | 4.0 | 6-13-10 | 23 | ML | 1 | NP |
| A-1 | S 2 | 5.00 | 6.50 | 12.0 | 4-5-8 | 13 | ML,CL | 1 | NP |
| A-1 | S 3 | 10.00 | 11.50 | 13.0 | 6-9-13 | 22 | ML | 1 | NP |
| A-1 | S 4 | 15.00 | 16.50 | 15.0 | 7-9-10 | 19 | ML | 1 | NP |
| A-1 | S 5 | 20.00 | 21.50 | 18.0 | 5-6-7 | 13 | ML | 1 | Plastic |
| A-1 | S 6 | 25.00 | 26.50 | 12.0 | 7-15-21 | 36 | ML | 1 | NP |
| A-2 | S 1 | 0.00 | 1.50 | 3.0 | 1/12"-1 | 1 | ML | 1 | Plastic |
| A-2 | S 3 | 10.00 | 11.50 | 14.0 | 5-5-6 | 11 | ML | 1 | NP |
| A-2 | S 4 | 15.00 | 16.50 | 13.0 | 7-7-7 | 14 | ML | 1 | NP |
| A-2 | S 5 | 20.00 | 21.50 | 14.0 | 5-6-11 | 17 | ML | 1 | Plastic |
| A-2 | S 6 | 25.00 | 26.50 | 15.0 | 5-6-10 | 16 | ML | 1 | Plastic |
| A-3 | S 1 | 0.00 | 1.50 | 11.0 | 1-2-2 | 4 | ML | 1 | Plastic |
| A-3 | S 4 | 15.00 | 16.50 | 13.0 | 6-7-8 | 15 | ML | 1 | Plastic |
| A-3 | S 5 | 20.00 | 21.50 | 13.0 | 6-8-12 | 20 | ML | 1 | Plastic |
| A-3 | S 6 | 25.00 | 26.50 | 16.0 | 8-12-18 | 30 | ML | 1 | Plastic |
| A-3 | S 7 | 30.00 | 31.50 | 14.0 | 12-14-20 | 34 | ML | 1 | NP |
| A-4 | S 1 | 0.00 | 1.50 | 6.0 | 1-6-8 | 14 | ML | 1 | Plastic |
| A-4 | S 3 | 10.00 | 11.50 | 12.0 | 6-6-7 | 13 | ML | 1 | NP |
| A-4 | S 4 | 15.00 | 16.50 | 14.0 | 6-8-10 | 18 | ML | 1 | Plastic |
| A-4 | S 5 | 20.00 | 21.50 | 18.0 | 6-6-6 | 12 | ML | 1 | Plastic |
| A-4 | S 6 | 25.00 | 26.50 | 12.0 | 4-8-12 | 20 | ML | 1 | Plastic |
| B-1 | S 1 | 0.00 | 1.50 | 8.0 | 2-4-9 | 13 | ML | 1 | NP |
| B-1 | S 3 | 10.00 | 11.50 | 13.0 | 7-7-8 | 15 | ML | 1 | NP |
| B-1 | S 4 | 15.00 | 16.50 | 16.0 | 6-10-10 | 20 | ML | 1 | Plastic |
| B-1 | S 5 | 20.00 | 21.50 | 16.0 | 4-5-7 | 12 | ML | 1 | Plastic |
| B-2 | S 1 | 0.00 | 1.50 | 0.0 | 1-2-2 | 4 | ML | 1 | Plastic |
| B-2 | S 3 | 10.00 | 11.50 | 12.0 | 4-6-7 | 13 | ML | 1 | Plastic |
| B-2 | S 4 | 15.00 | 16.50 | 14.0 | 3-7-9 | 16 | ML | 1 | Plastic |
| B-2 | S 5 | 20.00 | 21.50 | 18.0 | 5-6-6 | 12 | ML | 1 | Plastic |
| B-2 | S 6 | 25.00 | 26.50 | 13.0 | 6-7-8 | 15 | ML | 1 | Plastic |
| B-3 | S 1 | 0.00 | 1.50 | 4.0 | 1-4-5 | 9 | ML | 1 | Plastic |

## Table 2

## N-Values of Nonplastic Silts in Upper Layer at PFSF

| Boring & | Sample | Depth (ft) | | Rec (in.) | Blows | N-Value | USC | Layer | NP? |
|---|---|---|---|---|---|---|---|---|---|
| B-3 | S 2 | 15.00 | 16.50 | 14.0 | 5-8-10 | 18 | ML | 1 | Plastic |
| B-3 | S 3 | 20.00 | 21.50 | 18.0 | 4-6-6 | 12 | ML | 1 | Plastic |
| B-3 | S 4 | 25.00 | 26.50 | 16.0 | 8-10-14 | 24 | ML | 1 | Plastic |
| B-3 | S 5 | 30.00 | 31.50 | 16.0 | 8-12-16 | 28 | ML | 1 | NP |
| B-4 | S 1 | 0.00 | 1.50 | 6.0 | 2-9-13 | 22 | ML | 1 | NP |
| B-4 | S 2 | 5.00 | 6.50 | 13.0 | 3-4-5 | 9 | ML | 1 | Plastic |
| B-4 | S 4 | 15.00 | 16.50 | 12.0 | 8-7-8 | 15 | ML | 1 | NP |
| B-4 | S 5 | 20.00 | 21.50 | 13.0 | 5-10-11 | 21 | ML | 1 | NP |
| B-4 | S 6 | 25.00 | 26.50 | 14.0 | 6-10-11 | 21 | ML | 1 | NP |
| C-1 | S 1 | 0.00 | 1.50 | 4.0 | 2-1-2 | 3 | ML | 1 | NP |
| C-1 | S 2 | 5.00 | 6.50 | 16.0 | 3-4-4 | 8 | ML | 1 | Plastic |
| C-1 | S 4 | 15.00 | 16.50 | 15.0 | 8-8-8 | 16 | ML | 1 | Plastic |
| C-1 | S 5 | 20.00 | 21.50 | 18.0 | 4-4-4 | 8 | ML | 1 | Plastic |
| C-2 | S 1 | 0.00 | 1.50 | 4.0 | 3-8-10 | 18 | ML | 1 | Plastic |
| C-2 | S 2 | 15.00 | 16.50 | 13.0 | 5-6-7 | 13 | CL,ML | 1 | Plastic |
| C-4 | S 1 | 0.00 | 1.50 | 7.0 | 2-6-9 | 15 | ML | 1 | NP |
| C-4 | S 2 | 5.00 | 6.50 | 12.0 | 4-3-4 | 7 | CL/CH/ ML | 1 | Plastic |
| C-4 | S 3 | 10.00 | 11.50 | 15.0 | 3-4-7 | 11 | ML | 1 | NP |
| C-4 | S 4 | 15.00 | 16.50 | 12.0 | 6-6-8 | 14 | ML | 1 | NP |
| C-4 | S 5 | 20.00 | 21.50 | 13.0 | 5-7-8 | 15 | ML | 1 | Plastic |
| C-4 | S 6 | 25.00 | 26.50 | 14.0 | 10-10-10 | 20 | ML | 1 | NP |
| C-4 | S 7 | 30.00 | 31.50 | 13.0 | 8-10-11 | 21 | ML | 1 | NP |
| D-1 | S 1 | 0.00 | 1.50 | 6.0 | 6-20-20 | 40 | ML | 1 | NP |
| D-1 | S 2 | 5.00 | 6.50 | 14.0 | 3-5-7 | 12 | ML | 1 | NP |
| D-1 | S 3 | 10.00 | 11.50 | 14.0 | 5-7-7 | 14 | ML | 1 | NP |
| D-1 | S 4 | 15.00 | 16.50 | 16.0 | 4-6-7 | 13 | ML | 1 | Plastic |
| D-1 | S 5 | 20.00 | 21.50 | 15.0 | 6-7-9 | 16 | ML | 1 | NP |
| D-2 | S 1 | 0.00 | 1.50 | 6.0 | 2-3-3 | 6 | ML | 1 | NP |
| D-2 | S 2 | 5.00 | 6.50 | 15.0 | 3-3-3 | 6 | ML | 1 | Plastic |
| D-2 | S 3 | 10.00 | 11.50 | 16.0 | 3-4-7 | 11 | ML | 1 | Plastic |
| D-2 | S 4 | 15.00 | 16.50 | 16.0 | 5-7-8 | 15 | ML | 1 | NP |

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# CALCULATION SHEET

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## Table 2

## N-Values of Nonplastic Silts in Upper Layer at PFSF

| Boring & Sample | | Depth (ft) | | Rec (in.) | Blows | N-Value | USC | Layer | NP? |
|---|---|---|---|---|---|---|---|---|---|
| D-2 | S 5 | 20.00 | 21.50 | 15.0 | 6-9-9 | 18 | ML | 1 | NP |
| D-2 | S 6 | 25.00 | 26.50 | 15.0 | 6-8-9 | 17 | ML | 1 | NP |
| D-3 | S 6 | 25.00 | 26.50 | 16.0 | 8-14-25 | 39 | ML | 1 | NP |
| D-4 | S 1 | 0.00 | 1.50 | 3.0 | 4-4-4 | 8 | ML | 1 | NP |
| D-4 | S 2 | 5.00 | 6.50 | 3.0 | 3-2-2 | 4 | ML | 1 | Plastic |
| D-4 | S 3 | 10.00 | 11.50 | 8.0 | 5-10-14 | 24 | ML | 1 | NP |
| D-4 | S 4 | 15.00 | 16.50 | 12.0 | 10-12-10 | 22 | ML | 1 | NP |
| D-4 | S 5 | 20.00 | 21.50 | 14.0 | 4-4-5 | 9 | ML | 1 | Plastic |
| D-4 | S 6 | 25.00 | 26.50 | 14.0 | 6-7-9 | 16 | ML | 1 | NP |
| E-3 | S 1 | 0.00 | 1.50 | 7.0 | 2-4-5 | 9 | ML | 1 | NP |
| E-3 | S 3 | 10.00 | 11.50 | 16.0 | 3-5-7 | 12 | ML | 1 | Plastic |
| E-3 | S 4 | 15.00 | 16.50 | 16.0 | 7-10-12 | 22 | ML | 1 | NP |
| E-3 | S 5 | 20.00 | 21.50 | 16.0 | 4-7-8 | 15 | ML | 1 | Plastic |
| E-3 | S 6 | 25.00 | 26.50 | 15.0 | 5-6-12 | 18 | ML | 1 | NP |
| E-4 | S 1 | 0.00 | 1.50 | 7.0 | 3-7-10 | 17 | ML | 1 | NP |
| E-4 | S 4 | 15.00 | 16.50 | 12.0 | 8-20-20 | 40 | ML | 1 | NP |
| E-4 | S 5 | 20.00 | 21.50 | 17.0 | 6-6-10 | 16 | ML | 1 | Plastic |
| E-4 | S 6 | 25.00 | 26.50 | 16.0 | 7-10-14 | 24 | ML | 1 | NP |

NOTES: This sheet only addresses Layer 1 silts, without AR borings and B-1 & 2. Layer 1 SM samples are excluded. All of their blow counts > 20.

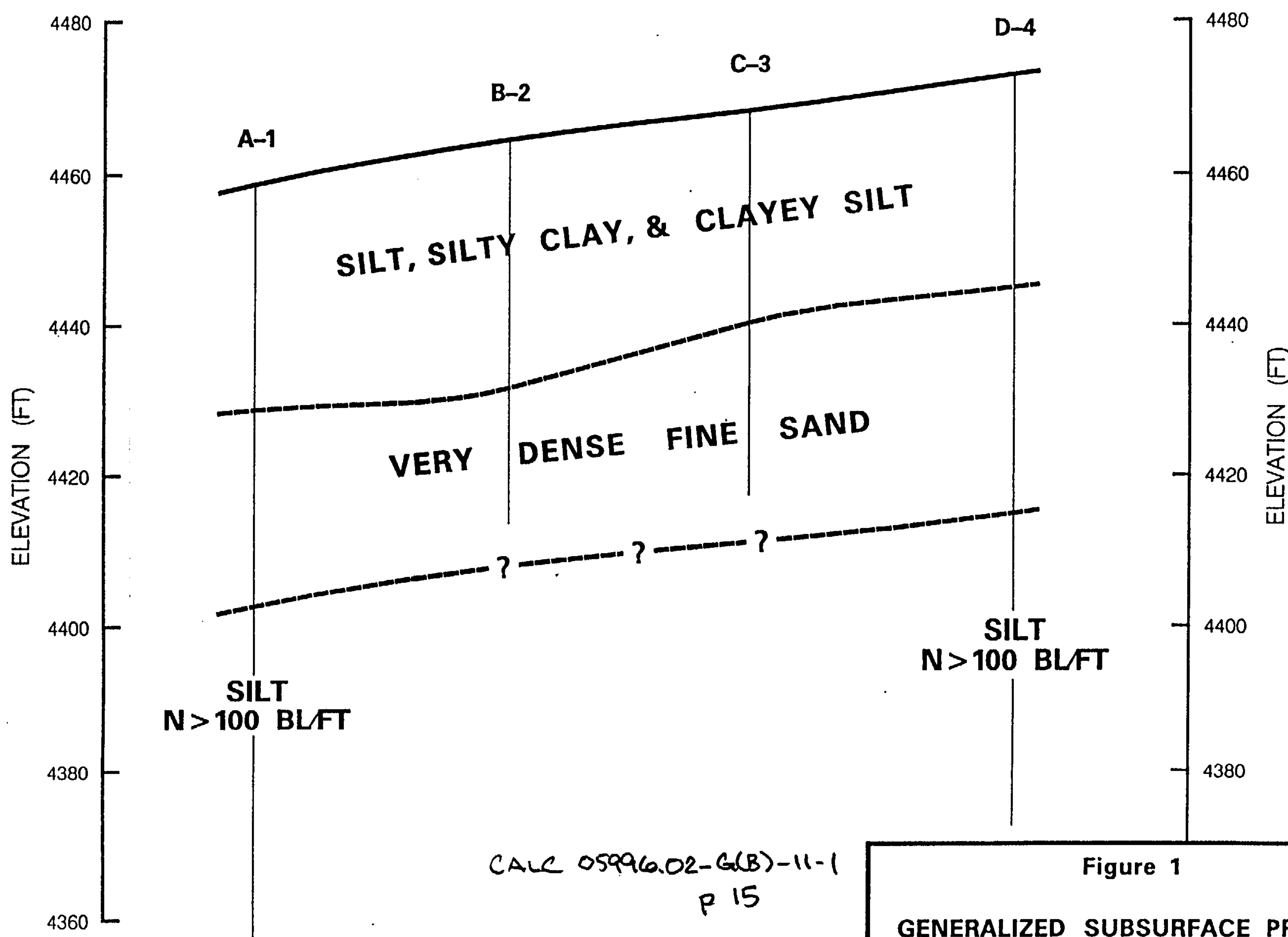
indicates N < 10

**Table 3**

**Summary of N-Values of Nonplastic Silts in Upper Layer at PFSF**

**Excluding Samples from 0 to 1.5 ft**

| | **31** | **NP samples** | **33** | **Plastic samples** | **64** | **NP & Plastic samples** |
|---|---|---|---|---|---|---|
| N-values | Minimum | 11 | Minimum | 4 | Minimum | 4 |
| | Average | 20 | Average | 14 | Average | 17 |
| | **Median** | **18** | **Median** | **13** | **Median** | **16** |
| | Maximum | 40 | Maximum | 30 | Maximum | 40 |
| | N < 10 | 0 | N < 10 | 7 | N < 10 | 7 |




CALC 05996.02-G(B)-11-1
P 15

0 500 1000 FEET
1" = 500'-0" HORIZONTAL

NOTED MAY 7 1997 P. J. Trudeau

**Figure 1**

**GENERALIZED SUBSURFACE PROFILE**

**PRIVATE FUEL STORAGE ISFSI**
**GEOTECHNICAL DESIGN CRITERIA**

Revision 0

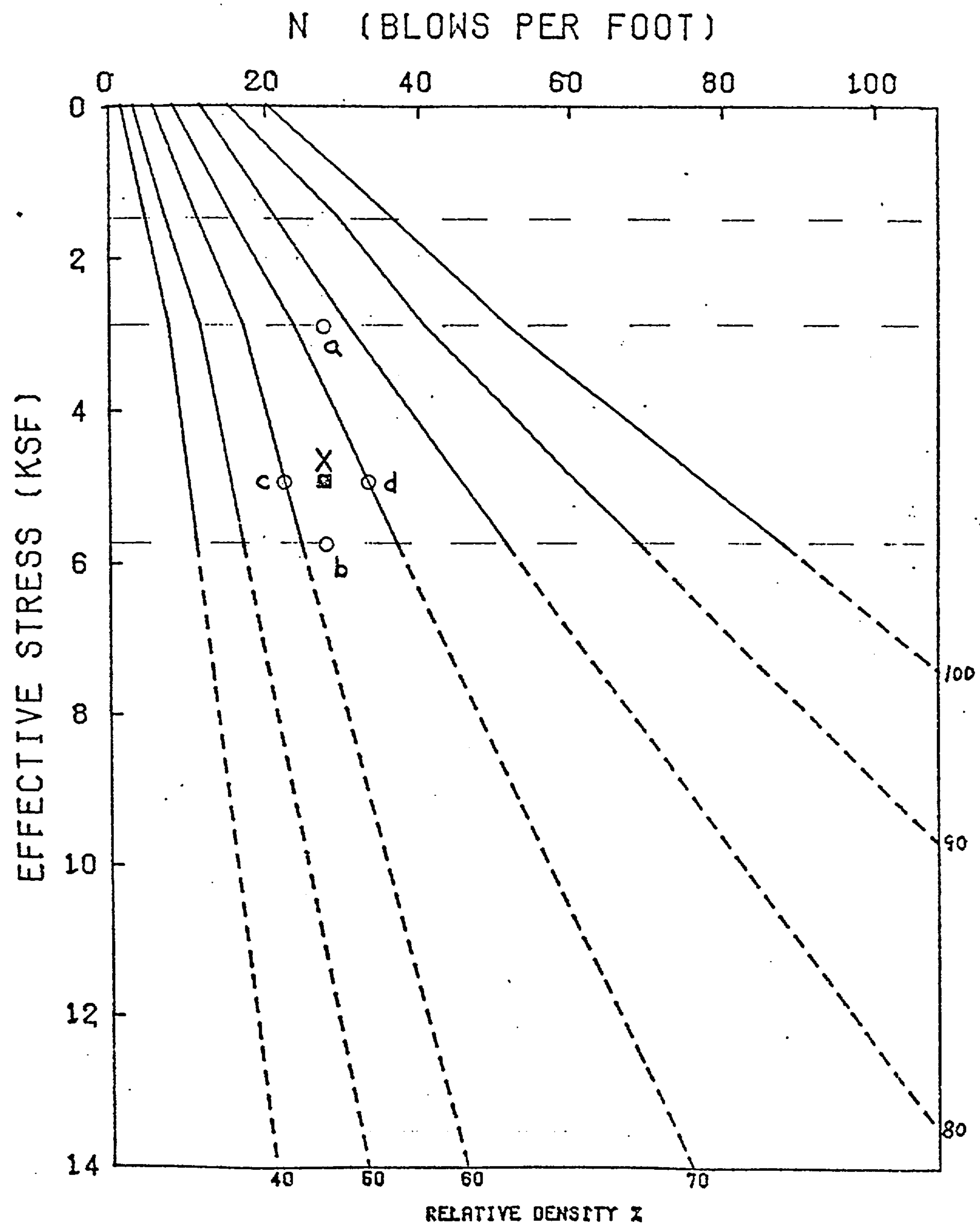



FIGURE 1

ATTACHMENT A TO
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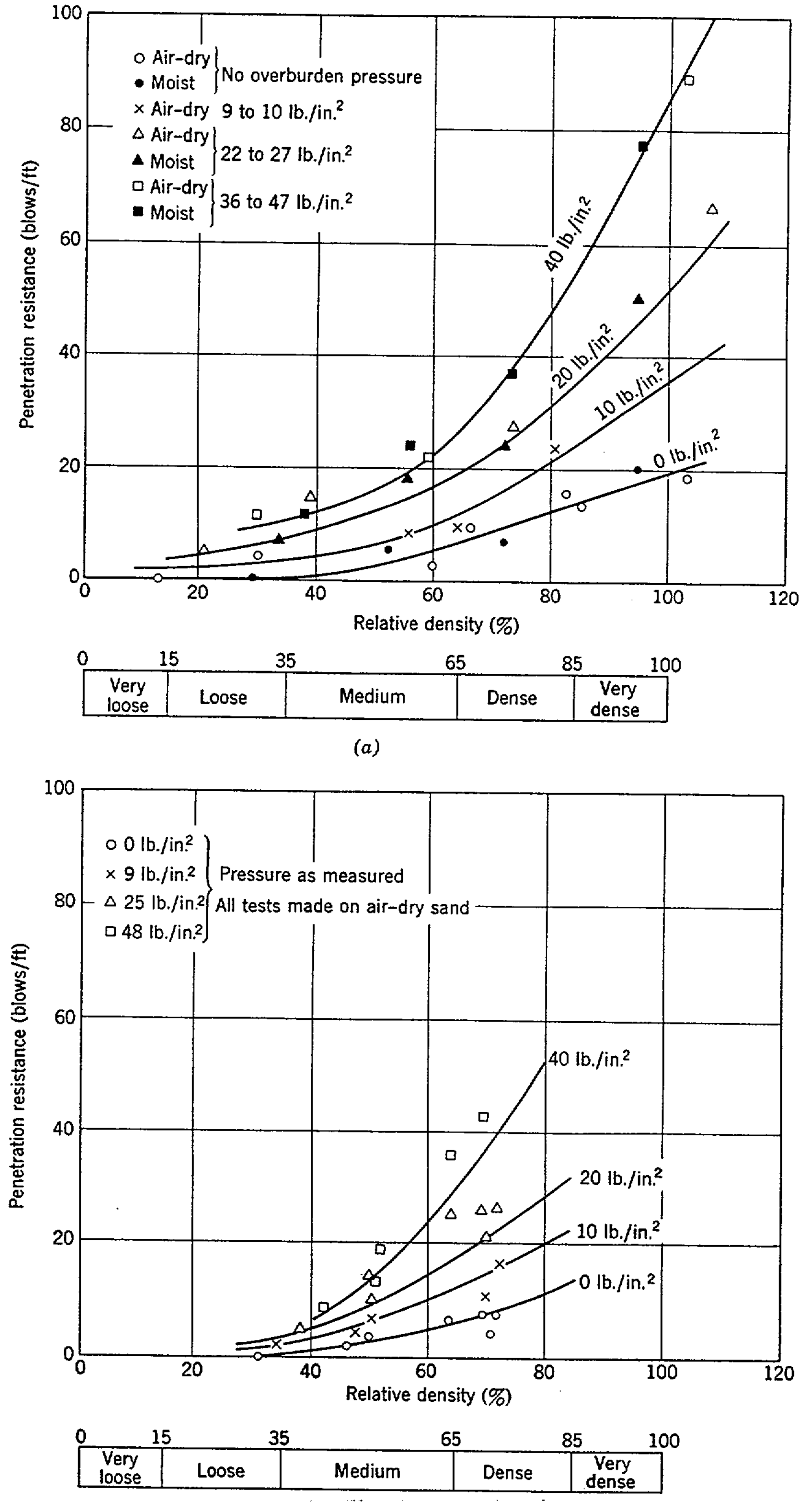


Fig. 7.5 Results of standard penetration tests. (*a*) Coarse sand. (*b*) Fine sand. (From Gibbs and Holtz, 1957.)

ATTACHMENT B
CALC 05996.02-G(B)-11-1
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 FROM LAMBE & WHITMAN (1969)

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**CALCULATION SHEET** ATTACHMENT C

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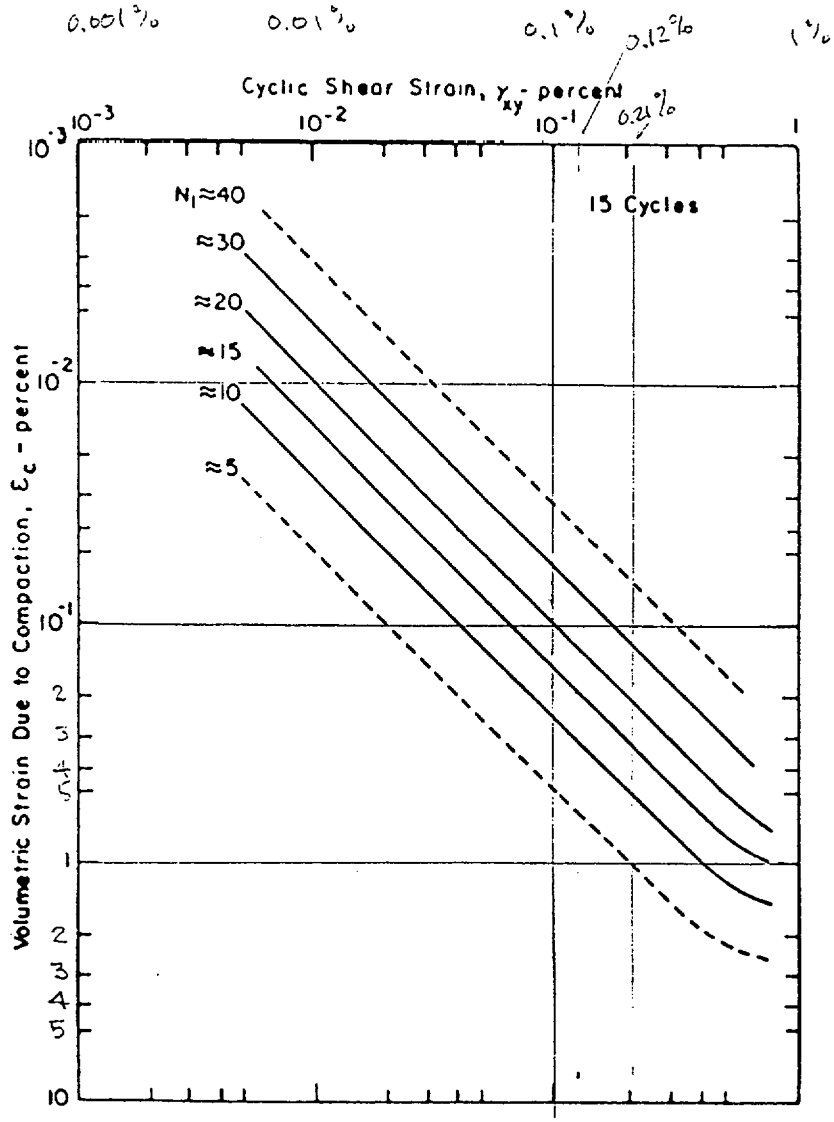


**FIG. 13.—Relationship between Volumetric Strain, Shear Strain, and Penetration Resistance for Dry Sands**

TOKIMATSU & SEED (1987)

**TABLE 4.—Influence of Earthquake Magnitude on Volumetric Strain Ratio for Dry Sands**

| Earthquake magnitude (1) | Number of representative cycles at 0.65 $\tau_{max}$ (2) | Volumetric strain ratio, $\epsilon_{C,N}/\epsilon_{C,N=15}$ (3) |
|---|---|---|
| 8-1/2 | 26 | 1.25 |
| 7-1/2 | 15 | 1.0 |
| 6-3/4 | 10 | 0.85 |
| 6 | 5 | 0.6 |
| 5-1/4 | 2–3 | 0.4 |

TOKIMATSU + SEED (1987)

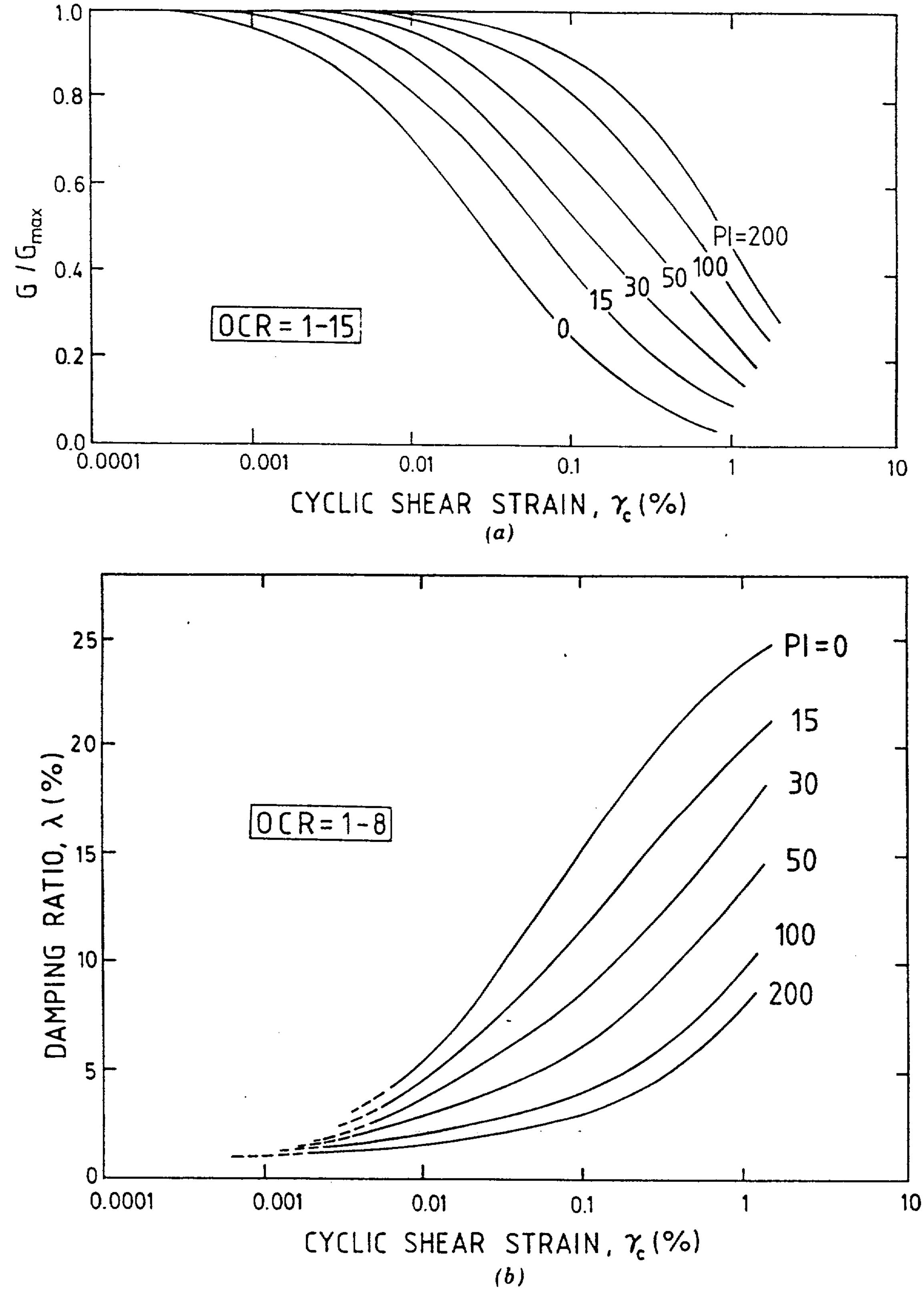


**FIG. 6. Relations between $G/G_{max}$ versus $\gamma_c$ and $\lambda$ versus $\gamma_c$ Curves and Soil Plasticity for Normally and Overconsolidated Soils**

VUCETIC & DOBRY (1991)

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# CALCULATION TITLE PAGE

*SEE INSTRUCTIONS ON REVERSE SIDE

▲ 5010.64 (FRONT)

| CLIENT & PROJECT | PAGE 1 OF 26 |
|---|---|
| PRIVATE FUEL STORAGE LLC - PRIVATE FUEL STORAGE FACILITY | |
| CALCULATION TITLE (Indicative of the Objective): STATIC SETTLEMENT OF THE CANISTER TRANSFER BUILDING SUPPORTED ON A MAT FOUNDATION. | QA CATEGORY (✓) ☒ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ____ OTHER |

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| 05996.02 | G(C) | 14 | | 345H |

*APPROVALS - SIGNATURE & DATE

| PREPARER(S)/DATE(S) | REVIEWER(S)/DATE(S) | INDEPENDENT REVIEWER(S)/DATE(S) | REV. NO. OR NEW CALC NO. | SUPERSEDES * CALC. NO. OR REV. NO. | CONFIRMATION *REQUIRED (✓) YES | NO |
|---|---|---|---|---|---|---|
| L.P. SINGH [signature] 12-9-98 | T.L. Aloysius [signature] 12-10-98 | T.L. Aloysius [signature] 12-10-98 | 0 | | | ✓ |
| P.J. TRUDEAU Paul J. Trudeau 7-26-00 | T.Y. CHANG Thomas Y. Chang 7-26-2000 | T.Y. CHANG Thomas Y. Chang 7-26-2000 | 1 | 0 | | ✓ |

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## RECORD OF CHANGES

| Rev. No. | Description of Changes | Pages Revised | Pages Added | Pages Replaced |
|---|---|---|---|---|
| 0 | Original Issue | N/A | N/A | N/A |
| 1 | Updated Table of Contents | 2 | | None |
| | Added Record of Changes | | 2A | |
| | Added discussion indicating estimated settlements will not adversely affect performance of the structure. | 20 | 20A, & 20B | |
| | Updated References | 22 | | |

**REASON FOR REV. 1**

Rev. 1 was prepared to provide basis for statements that are included in the SAR in Section 2.6.1.12.2 and under Mat Foundation Stability Analyses and Settlement in SAR Section 4.7.1.5.3 indicating that the estimated settlements will not adversely affect the performance of the structure.

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OBJECTIVE

TO ESTIMATE THE STATIC SETTLEMENT OF THE CANISTER TRANSFER BUILDING SUPPORTED ON A MAT FOUNDATION DUE TO STATIC DEAD LOAD AND LIVE LOAD OVER THE 40-YEAR LIFE OF THE FACILITY.

ASSUMPTIONS / DATA

SWEC DWG. NO. 05996.01-EM-1 AND 05996.01-EM-3 CANISTER TRANSFER BUILDING, GENERAL ARRANGEMENT SHEET 1 AND SHEET 3 SHOW THE BUILDING FOOTPRINT AND FOUNDATION CROSS-SECTION.

THIS ANALYSIS IS PERFORMED FOR A BUILDING (AND EQUIPMENT) STATIC LOAD OF 75,000 KIPS (CALC. 05996.01-G(B)-03)

THE BUILDING FOUNDATION MAT IS APPROXIMATED AS A 147 FT BY 275 FT, AND 5 FT IN THICKNESS.

FIGURE 1 SHOWS THE GENERALIZED SOIL PROFILE WHICH WAS DEVELOPED BASED ON BORINGS A-1 TO A-4, B-1 TO B-4, C-1 TO C-4, AND D1-D4.

FIGURE 2 SHOWS THE FOUNDATION PROFILE, SHOWING THE BUILDING FOUNDATION MAT & THE PROFILE USED IN CALCULATING SETTLEMENTS.

CALC. 05996.01-G(B)-01-1 PROVIDES BASES FOR DYNAMIC SOIL PROPERTIES INCLUDING LONG-STRAIN ELASTIC MODULI VALUES SHOWN IN FIG. 2

SKETCH 05996.01-GSK-B-01A-0, "BORING LOCATION PLAN", SHEET 1 OF 2.

FOR SOIL PROPERTIES USED IN DETERMINATION OF SETTLEMENTS, INCLUDING MAXIMUM PAST CONSOLIDATION PRESSURE, REFER TO CALC. 05996.01-G(B)-03

METHOD[1]: TOTAL STATIC ELASTIC SETTLEMENT

= ELASTIC SETTLEMENT + PRIMARY CONSOLIDATION

+ SECONDARY CONSOLIDATION.

THE SOIL PROFILE IS DIVIDED INTO LAYERS. THE CHANGE IN VERTICAL STRESS DUE TO CANISTER BUILDING LOADING IS DETERMINED IN THE MIDDLE OF EACH LAYER. A LOAD DISPERSION OF 2V:1H WITH DEPTH FROM THE BASE OF THE FOUNDATION MAT IS ASSUMED TO ESTIMATE THE STRESS INCREMENT.

ESTIMATE $\Delta \varepsilon_v$ DUE TO $\Delta \sigma_v$

CALCULATE $\Delta \rho_i = \Delta \varepsilon_{v_i} \cdot H_i$

$$\rho_{TOTAL} = \Sigma \Delta \rho_i$$

ELASTIC SETTLEMENT

$$\rho_{ELAS} = \Sigma \varepsilon_{v_i} \cdot H_i$$

WHERE $\varepsilon_{v_i}$ = STRAIN IN LAYER $i$ = $\dfrac{\Delta \sigma_{v_i}}{E_i}$

$H_i$ = THICKNESS OF LAYER $i$

$\Delta \sigma_{v_i}$ = CHANGE IN VERTICAL STRESS AT CENTER OF LAYER $i$ DUE TO FOUNDATION LOADING.

$E_i$ = MODULUS OF ELASTICITY FOR LAYER $i$, (ADJUSTED FOR STRAIN LEVEL)

[1] THIS METHOD WAS ALSO USED IN CALCULATION NO. 05996.01-G(B)-03 TO DETERMINE SETTLEMENT OF THE CANISTER STORAGE PADS.

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METHOD (CONT'D)

## PRIMARY CONSOLIDATION SETTLEMENT

CONSOLIDATION TESTS PERFORMED ON SAMPLES OF CLAYEY SILT FROM A DEPTH OF 11 FT IN LAYER 1 (FIGURE 2), INDICATES THE MAXIMUM PAST PRESSURE,

$\bar{\sigma}_{MPP} \cong 3$ TSF [SEE CALC. 05996.01-G(B)-03-1]

FINAL STRESS AT THE BOTTOM OF LAYER 1 IS 1.99$^{\oplus}$ TSF

HENCE PRIMARY CONSOLIDATION IN THIS LAYER WILL BE BASED ON SOIL RECOMPRESSION.

$$\delta_{PR1} = H \cdot \varepsilon_v = H \cdot RR \cdot \log \frac{\bar{\sigma}_{vf}}{\bar{\sigma}_{vo}}$$

($\oplus$ FROM PAGE 17 OF CALC)

H = LAYER THICKNESS

$\varepsilon_v$ = VERTICAL STRAIN DUE TO INCREASE IN STRESS

RR = RECOMPRESSION RATIO (FROM CONSOLIDATION TESTS)

$\bar{\sigma}_{vf}$ = TOTAL EFFECTIVE VERTICAL STRESS AT CENTER OF LAYER.

$\bar{\sigma}_{vo}$ = INITIAL EFFECTIVE VERTICAL STRESS AT CENTER OF LAYER

METHOD (CONT'D)

SECONDARY SETTLEMENT

$$\rho_{SEC} = H \cdot \varepsilon_v = H \cdot C_\alpha \cdot \Delta \log t$$

WHERE $H$ = LAYER THICKNESS

$\varepsilon_v$ = VERTICAL STRAIN DUE TO ADDITIONAL LOAD

$C_\alpha$ = COEFFICIENT OF SECONDARY COMPRESSION (FROM CONSOLIDATION TEST)

$t$ = LIFE OF FACILITY, ASSUME $t_f = 40$ YRS FOR CALCULATING $\rho_{SEC}$.

USE $C_\alpha$ DETERMINED BASED ON RELOADING FOLLOWING AN UNLOADING CYCLE TO CORRECT FOR SAMPLE DISTURBANCE.

METHOD (CONT'D)

ELASTIC SETTLEMENTS

TABLE 1 SHOWS THE CALCULATION OF ELASTIC SETTLEMENTS IN LAYER 1 TO 3 SHOWN IN FIGURE 2 USING LOW STRAIN MODULI ($E_{MAX}$). IN REALITY, THE ELASTIC MODULI SHOULD BE REDUCED TO CORRESPOND TO THE LEVEL OF STRAIN ASSOCIATED WITH THE SETTLEMENT IN EACH LAYER. THE ELASTIC MODULI IS THEREFORE REDUCED AND A NEW DETERMINATION OF SETTLEMENT VALUES IN EACH LAYER IS MADE. THIS IS AN ITERATIVE PROCESS UNTIL NO FURTHER REDUCTION IN ELASTIC MODULI IS REQUIRED. FROM TABLE 1, ELASTIC SETTLEMENTS OF LAYERS 2 & 3 ≈ 20% AND 80% IN LAYER 1. CORRESPONDING STRAIN IN LAYER 1 WILL BE GREATER AS WELL.

IT IS ALSO SEEN THAT THE ELASTIC MODULUS OF LAYER 4 ($Z \geqslant 120$ FT) IS EVEN GREATER THAN THAT OF LAYERS 2 & 3. HENCE CONTRIBUTION OF LAYER 4 TO TO THE SETTLEMENT OF THE BUILDING MAT WILL BE EVEN LESS, OR NEGLIGIBLE, AS SHOWN LATER IN THIS CALCULATION.

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ESTIMATE MODULUS OF ELASTICITY OF LAYER 4 $(Z > 120')$.

$V_p = 5525$ FT/SEC.

ASSUME $\gamma_m = 125$ PCF, $\mu = 0.5$

$$D = \rho . V_p^2 = \frac{0.125 \; K/FT^3}{32.2 \; FT/SEC^2} \left(5525 \frac{FT}{SEC}\right)^2$$

$= 118,500$ KSF , WHERE D = CONSTRAINED MODULUS.

$\rho$ = MASS DENSITY

$$D = \frac{E(1-\mu)}{(1+\mu)(1-2\mu)}$$ -- (EQ. 12.8 LAMBE & WHITMAN, 1969)

'D' IS INDETERMINATE AT $\mu = 0.5$; HENCE USE $\mu = 0.4$

$$D = \frac{E(1-0.4)}{(1+0.4)(1-2\times0.4)}$$

$$E = \frac{118,500 \times 1.4 \times 0.2}{0.6} = 55,300^{(1)} \text{ KSF.}$$

(1) THIS VALUE OF E FOR LAYER 4 NOT USED IN THIS CALC. VALUE OF E USED IS 44940 KSF WHICH IS CONSERVATIVE.

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ELASTIC SETTLEMENT OF FOUNDATION MAT

STRAIN IN LAYER ① $= \frac{\Delta \rho}{\Delta z} = \frac{0.136 \text{IN}}{(30-5)^{FT} \times 12 \text{ IN/FT}}$ --FROM TABLE 1

$= 4.53 \times 10^{-4}$ IN/IN OR $4.53 \times 10^{-2}$ %

THE E VALUE USED TO CALCULATE THIS SETTLEMENT WAS BASED ON $E_{MAX}$, WHICH IS APPLICABLE FOR LOW SHEAR STRAINS ON THE ORDER OF $\gamma \simeq 10^{-4}$ %

USE AN ASSUMED VALUE OF $\gamma$ AND AN ITERATIVE PROCESS TO ESTABLISH THE VALUES FOR $\gamma$, E, $\varepsilon$, $\rho$

ASSUME $\gamma = 10^{-1}$ %, $E/E_{MAX} = 0.43$ (FROM FIG. 3)

$\Delta\rho_1$ (LAYER 1) $= \frac{\Delta\bar{\sigma}_1}{E_1} \times \Delta z_1 = \frac{1.71}{0.43 \times 3780} \times 25 \times 12 = 0.316$

$\varepsilon_v = \frac{0.316 \text{ IN.}}{25 \text{ FT} \times 12 \text{ IN/FT}} = 1.05 \times 10^{-3}$ IN/IN or 0.1 % (SAME AS INITIALLY ASSUMED)

HENCE $\Delta\rho_1$ (LAYER 1) $= 0.32$ IN.

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ELASTIC SETTLEMENTS IN LAYERS ② THRU' ④

LAYER ② & ③ MATERIALS ARE VERY DENSE (N > 100 BL/FT AND N > 100 BL/6 IN). CORRESPONDING STRAINS WILL BE LOW.

ASSUME $\varepsilon_v \simeq 10^{-2}\%$ , $\frac{E}{E_{MAX}} = \frac{G}{G_{MAX}} = 0.87$ (FIG. 3)

USE $q$ (AT BASE OF MAT) $= \frac{F_V}{AREA} = \frac{75{,}000}{147 \times 275} = 1.86$ KSF

DO NOT REDUCE $q$ FOR DEPTH BECAUSE OF WIDE EXPANSE OF LOADED AREA (CONSERVATIVE ASSUMPTION)

FOR COMBINED LAYERS 2 & 3, $E = 0.87 \times E_{MAX}$

$= 0.87 \times 44{,}940 = 39098$ KSF

$\Delta Z = 90$ FT

$\rho_{2/3} = \frac{\Delta \bar{\sigma}_1}{E_{2/3}} \cdot \Delta Z = \frac{1.86}{39098} \times 90 \times 12 = 0.05$ IN.

$\varepsilon_v = \frac{0.05 \text{ IN}}{90 \text{ FT} \times 12 \text{ IN/FT}} = 4.63 \times 10^{-5}$ IN/IN OR $4.63 \times 10^{-3}\%$

$E/E_{MAX}$ (FROM FIG 3) $= 0.92$

$E = 0.92 \times 44{,}940 = 41{,}345$ KSF

$\rho_{2/3} = \frac{1.86 \times 90 \times 12}{41{,}345} \simeq 0.05$ IN (HENCE OK.)

ALTERNATIVELY,
TAKE STRESS DISPERSION OVER DEPTH INTO ACCOUNT

$\Delta \bar{\sigma}_z$ AT MID-DEPTH OF LAYERS 2 & 3 $= \frac{1.86 \times 147}{[147 + 25 + \frac{1}{2}(30+60)]}$

$= \frac{1.86 \times 147}{217} = 1.26$ KSF

$\rho_{2/3} = \frac{0.05}{1.86} \times 1.26 = 0.03$ IN

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FOR LAYER 4 :

$\Delta Z = 600^{*} - 90 = 510$ FT

[* ROCK IS ESTIMATED TO BE 520 FT TO 880 FT BELOW GRADE BASED ON SEISMIC REFLECTION SURVEY LINE 2 ( GEOSPHERE MIDWEST, 1997 ].

USE $E = 0.9\,E_{MAX} = 0.9 \times 44940 = 40,446$ KSF (STRAIN REDUCED)

$$\Delta \rho_4 = \frac{\Delta \bar{\sigma}_1}{E_4} \cdot \Delta z = 1.86 \frac{K}{FT^2} \times 510\ FT \times 12 \frac{IN}{FT} \times \frac{1}{40446\ K/FT^2}$$

$$= 0.28\ IN.$$

TAKE STRESS DISPERSION OVER DEPTH INTO ACCOUNT

$\Delta \bar{\sigma}_z$ AT MID-DEPTH OF LAYER 4

$$= \frac{1.86 \times 147}{(147 + 25 + 30 + 60 + 510/2)} = \frac{1.86 \times 147}{517} = 0.53\ KSF$$

HENCE $\Delta \rho_4 = \frac{0.28\ IN}{1.86\ KSF} \times 0.53\ KSF = \underline{0.08\ IN.}$ ← USE THIS VALUE FOR SETTLEMENT IN LAYER 4

TOTAL ELASTIC SETTLEMENT $= \rho_{2/3} + \rho_4$

$= 0.034 + 0.08$

$= 0.114$ IN.

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## PRIMARY CONSOLIDATION SETTLEMENT

ESTIMATE SETTLEMENT OF LAYER ① BASED ON RESULTS OF CONSOLIDATION TESTS. (NOTE THAT CONSOLIDATION TEST DATA PLOTTED AS $\varepsilon_v$ vs. $\log \bar{\sigma}_v$ WHERE $\sigma_v$ IS IN TSF).

AT CENTER OF LAYER 1, Z BELOW SLAB $= \frac{30-5}{2} = 12.5$ FT. BELOW THE BASE OF FOUNDATION MAT, WHICH IS 5 FT THICK.

DEPTH FROM GRADE TO CENTER OF LAYER $= 12.5 + 5 = 17.5$ FT.

$\bar{\sigma}_{vo} = 17.5 \text{ FT} \times 0.080 \text{ KCF} = 1.4 \text{ KSF} = 0.7 \text{ TSF}$

$\Delta\bar{\sigma}_v = \dfrac{1.86 \text{ KSF} \times 147}{(147+12.5)} = 1.71 \text{ KSF} = 0.86 \text{ TSF}$

HENCE, $\bar{\sigma}_{vf} = \bar{\sigma}_{vo} + \Delta\bar{\sigma}_v = 0.7 + 0.86 = 1.56$ TSF.

$\bar{\sigma}_{mpp}$ (MAXIMUM PAST PRESSURE) $= 3.0$ TSF (1) [(1) REFER TO CALC 05996.01-G(B)-03]

AVG. RR (RECOMPRESSION COEFFICIENT) $= 0.014$ (1)

$\bar{\sigma}_{vf}$ ($= 1.56$ TSF) $< \bar{\sigma}_{vmm}$

HENCE $\Delta\delta_{cp} = \Sigma\, H \cdot RR \log \dfrac{\bar{\sigma}_{vf}}{\bar{\sigma}_{vo}} = 25' \times 12''/' \times 0.014 \log \dfrac{1.56}{0.7}$

$= 1.46$ IN.

CONSOLIDATION SETTLEMENT OF LAYERS ②③ & ④ WILL BE NEGLIGIBLE SINCE LAYER ② CONSISTS OF VERY DENSE FINE SAND AND UNDERLYING LAYERS ③ & ④ CONSIST OF VERY DENSE SILT (N >> 100 BL/FT). THESE LAYERS COMPRISE OF GRANULAR MATERIALS.

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SECONDARY SETTLEMENT

ESTIMATE SECONDARY SETTLEMENT OF LAYER 1 BASED ON THE RESULTS OF CONSOLIDATION TESTS. USE UNLOAD - RELOAD DATA TO CORRECT FOR EFFECTS OF SAMPLE DISTURBANCE.

FOR RESULTS OF CONSOLIDATION TESTS THAT HAD UNLOAD - RELOAD CYCLES, C1-U3C, C1-U3D, & C2-U2C, REFER TO CALC. 05996.01-G(B)-03, ATTACHMENT B. (REF. 3)

TABLES(1) 2A-2C PRESENT COEFFICIENTS OF SECONDARY COMPRESSION, $C_\alpha$, FOR THESE CONSOLIDATION TESTS.

FIGURES(1) 5 & 6A THROUGH 6C PLOT $C_\alpha$ AS A FUNCTION OF STRESS RATIO, $\bar{\sigma}_c / \bar{\sigma}_{MPP}$.

FIG. 4-2 OF LADD (1971) SHOWS A TYPICAL PLOT OF $C_\alpha$ VS. STRESS RATIO FOR SEVERAL SOIL TYPES. IT IS SEEN THAT FOR OVERCONSOLIDATED SOILS, $C_\alpha$ IS LOW, IN THE RANGE OF 0.2 TO 0.4 % PER Δ LOG TIME.

IN THE VIRGIN COMPRESSION RANGE ($\bar{\sigma}_c / \bar{\sigma}_{MPP} \geq 1$), $C_\alpha$ RISES RAPIDLY.

FIG. 5(1) PLOTS $C_\alpha$ VS $\bar{\sigma}_c / \bar{\sigma}_{MPP}$ FOR CONSOLIDATION TEST C1-U3D. THE SHAPE OF THE CURVE IS SIMILAR TO LADD'S (1971), BUT $C_\alpha$ FOR THE INITIAL LOADING RISES AT A STRESS RATIO ≈ 0.5, RATHER THAN AT 1, AS EXPECTED. THIS DIFFERENCE IS ATTRIBUTED TO EFFECTS OF SAMPLE DISTURBANCE.

IT IS NOTED THAT $C_\alpha$ REMAINS LOW FOR THE RELOAD CYCLE DATA PRESENTED IN FIG. 5.(1)

HENCE $C_\alpha$ VALUES FOR THE RE-LOAD CYCLE ARE USED FOR ESTIMATING SECONDARY SETTLEMENT.

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(1) REFERS TO FIGURES AND TABLES CONTAINED IN REFERENCE 3, CALC. 05996.01-G(B)-03.

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SECONDARY SETTLEMENT

STRESS RATIOS APPLICABLE FOR THE DEPTHS OF THE CONSOLIDATION TEST SAMPLES, AS WELL AS THOSE FOR THE BOTTOM OF LAYER 1 BEFORE AND AFTER CONSTRUCTION OF THE TRANSFER BUILDING ARE CALCULATED. IT IS SEEN FROM THE CALCULATIONS THAT THE STRESS RATIOS OF INTEREST ARE IN THE RANGE OF 0.2 TO 0.5 [ REFER TO PLOTS OF $C_\alpha$ VS $\bar{\sigma}_c / \bar{\sigma}_{mp}$ IN FIGURES 6A TO 6C OF REF. 3 (CALC. 05996.01-G(B)-03).

| $C_\alpha$ | TEST | LOADING IN TSF LOADED | UNLOADED | RELOADED |
|---|---|---|---|---|
| 0.05 | C1-U3C | 2.0 | 0.5 | 8.0 |
| 0.05 | C1-U3D | 8.0 | 0.5 | 8.0 |
| 0.025 | C2-U2C | 2.0 | 0.5 | 8.0 |

$C_\alpha$ = 0.05 % / $\Delta$LOG TIME SHOULD BE USED TO ESTIMATE SECONDARY SETTLEMENTS OF LAYER 1 (SEE CALC. 05996.01-G(B)-03 FOR A DISCUSSION AND DEVELOPMENT OF THIS RECOMMENDED VALUE).

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SECONDARY SETTLEMENT (CONT'D)

CALCULATE $\dfrac{\bar{\sigma}_{vf}}{\bar{\sigma}_{MPP}}$ AT DEPTH OF CONSOLIDATION TEST C1-U3C

NOTE: DEPTH OF SAMPLE TESTED = 11.2 FT, $\bar{\sigma}_{MPP} \geq 2.8$ TSF

$\bar{\sigma}_{vo} \simeq 11.2 \text{ FT} \times 0.080 \dfrac{K}{FT^3} = 0.90 \text{ KSF} = 0.45 \text{ TSF}$

$\bar{\sigma}_{vo} / \bar{\sigma}_{MPP} \leq \dfrac{0.45}{2.8} = 0.16$

$\Delta\bar{\sigma}_v = \dfrac{1.86 \text{ KSF} \times 147 \text{ FT}}{147 + 11.2 - 5} = \dfrac{1.86 \times 147}{153.2} = 1.78 \text{ KSF} = 0.89 \text{ TSF}$

HENCE, $\bar{\sigma}_{vf} = \bar{\sigma}_{vo} + \Delta\bar{\sigma}_v = 0.45 + 0.89 = 1.34$ TSF.

$\bar{\sigma}_{vf} / \bar{\sigma}_{MPP} = \dfrac{1.34}{2.8} = 0.48$

AT DEPTH OF SAMPLE C1-U3D = 11.4 FT,

$\bar{\sigma}_{vo} \simeq 11.4^{FT} \times 0.080 \text{ K/FT}^3 = 0.91 \text{ KSF} = 0.46 \text{ TSF}$

$\bar{\sigma}_{MPP} \simeq 3$ TSF

$\Delta\bar{\sigma}_v$ (AT 11.4 FT DEPTH) $= \dfrac{1.86 \times 147}{147 + 11.4 - 5} = 1.78$ KSF

$= 0.89$ TSF

$\bar{\sigma}_{vf} = \bar{\sigma}_{vo} + \Delta\bar{\sigma}_v = 0.46 + 0.89 = 1.35$ TSF

$\bar{\sigma}_{vo} / \bar{\sigma}_{MPP} = 0.46 / 3.0 = 0.15$

$\bar{\sigma}_{vf} / \bar{\sigma}_{MPP} = 1.35 / 3.0 = 1.35/3.0 = 0.45$

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CALCULATE $\bar{\sigma}_{vf}/\bar{\sigma}_{mpp}$ AT DEPTH OF CONSOLIDATION TEST C2-U2C

NOTE: DEPTH OF SAMPLE TESTED = 10.9 FT

$\bar{\sigma}_{mpp} \geq 3$ TSF.

$\bar{\sigma}_{vo} \simeq 10.9 \text{ FT} \times 0.080 \text{ KCF} = 0.87 \text{ KSF} = 0.44 \text{ TSF}$

$\bar{\sigma}_{vo}/\bar{\sigma}_{mpp} \leq 0.44/3 = 0.15$

$$\Delta\bar{\sigma}_v = \frac{1.86 \times 147}{147 + 10.9 - 5} = 1.79 \text{ KSF} = 0.89 \text{ TSF}$$

$\bar{\sigma}_{vf} = \bar{\sigma}_{vo} + \Delta\bar{\sigma}_v = 0.44 + 0.89 = 1.33 \text{ TSF}$

$\bar{\sigma}_{vf}/\bar{\sigma}_{mpp} \leq 1.33/3 = 0.44$

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AT BOTTOM OF LAYER 1 ($z \simeq 30$ FT)

$\bar{\sigma}_{vo} = 30 \times 0.080 = 2.4$ KSF $= 1.2$ TSF

$\Delta \bar{\sigma}_v = \dfrac{1.86 \times 147}{147 + 30 - 5} = 1.59$ KSF $= 0.79$ TSF

$\bar{\sigma}_{vf} = \bar{\sigma}_{vo} + \Delta \bar{\sigma}_v = 1.2 + 0.79 = 1.99$ TSF

$\bar{\sigma}_{vo} / \bar{\sigma}_{mpp} = 1.2 / 3.0 = 0.40$

$\bar{\sigma}_{vf} / \bar{\sigma}_{mpp} = 1.99 / 3.0 = 0.66$

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SECONDARY SETTLEMENT

ASSUME THAT SECONDARY SETTLEMENT OCCURS AT A RATE 0.05% / LOG CYCLE TIME (MIN) FOR RANGE OF LOAD INCREMENT APPLICABLE TO TRANSFER BUILDING AND THICKNESS OF LAYER ① IS 30'-5' = 25 FT.

$$\varepsilon_{v_{SEC}} = 0.05\% \times \log_{10}\left[\text{ELAPSED TIME (MIN)}\right]$$

$$\rho_{SEC} = (30-5)' \times 12''/' \times \varepsilon_{v_{SEC}} \times \frac{1\ \text{IN/IN}}{100\%} = 3.0\ \varepsilon_v$$

| TIME | TIME (MIN) | $\log_{10}$ (ELAPSED TIME, MIN) | $\varepsilon_{v_{SEC}}$ | $\rho_{SEC}$, IN. |
|---|---|---|---|---|
| 1 DAY | 1440 | 3.16 | 0.16 | 0.48 |
| 1 WEEK | 10,080 | 4.00 | 0.20 | 0.60 |
| 1 MONTH (1) | 43,646 | 4.64 | 0.23 | 0.69 |
| 1 YEAR (2) | 525,600 | 5.72 | 0.29 | 0.87 |
| 10 YEARS | $5.26 \times 10^6$ | 6.72 | 0.34 | 1.02 |
| 20 YEARS | $10.51 \times 10^6$ | 7.02 | 0.35 | 1.05 |
| 40 YEARS (3) | $21.02 \times 10^6$ | 7.32 | 0.37 | 1.11 |

(1) 1 MONTH = 52/12 = 4.33 WEEKS

(2) 1 YEAR = 365 DAYS

(3) THE FACILITY HAS AN ESTIMATED LIFE OF 40 YRS.

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<u>SUMMARY OF SETTLEMENT IN ENTIRE SOIL PROFILE.</u>

THE ESTIMATED TOTAL STATIC SETTLEMENT OF THE TRANSFER BUILDING WILL BE AS FOLLOWS:

| | | |
|---|---|---|
| ELASTIC SETTLEMENT OF LAYER ① | | 0.32 IN. |
| ELASTIC SETTLEMENT OF LAYERS ②-④ | | 0.11 IN. |
| PRIMARY CONSOLIDATION OF LAYER ① | | 1.46 IN |
| (1) PRIMARY CONSOLIDATION OF LAYERS ②-④ | ≃ | 0 |
| SECONDARY CONSOLIDATION OF LAYER ① OVER 40 YRS | | 1.11 IN |

TOTAL STATIC SETTLEMENT: 3.00

---

(1) NO PRIMARY OR SECONDARY CONSOLIDATION IN LAYERS ② THROUGH ④ SINCE THESE LAYERS CONSIST OF GRANULAR MATERIAL.

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DISCUSSION

SETTLEMENT IN LAYERS ② THROUGH ④ (0.11 IN) COMPRISES APPROXIMATELY 3% OF THE TOTAL SETTLEMENT (3.0 IN). THIS PORTION OF THE SETTLEMENT WOULD OCCUR OVER THE ENTIRE SITE AREA AND WOULD NOT CONTRIBUTE TO ANY DIFFERENTIAL SETTLEMENTS.

IT IS SEEN THAT 2.89 IN (0.32+1.46+1.11) WHICH CONSTITUTES APPROX. 96% OF THE TOTAL ESTIMATED SETTLEMENT OF 3.0 IN IS DUE TO SETTLEMENT IN LAYER ①.

1.46 IN, OR 50% OF THE TOTAL SETTLEMENT IS PRIMARY CONSOLIDATION SETTLEMENT AND 0.32 IN. ELASTIC SETTLEMENT (FOR A TOTAL OF 1.78 IN), BOTH IN LAYER 1. BOTH THESE SETTLEMENTS ARE EXPECTED TO OCCUR WITHIN A SHORT PERIOD AFTER CONSTRUCTION.

AN ADDITIONAL SECONDARY CONSOLIDATION SETTLEMENT OF 1.11 IN. IS TO BE EXPECTED IN LAYER 1 OVER A 40 YR ANTICIPATED LIFE OF THE FACILITY. | $\boxed{1}$

***DISCUSSION OF SETTLEMENTS***

The estimated settlements of the Canister Transfer Building will not adversely affect the performance or integrity of the structure. The building settlements are minimal, considering the mat foundation system utilized. The subsurface profile under the building is essentially uniform, as shown on SAR Figures 2.6-21 through 23. The 5-ft thick mat will distribute the building loads over a wide area, resulting in relatively uniform distribution of settlements under the mat. As shown in S&W Drawing 0599601-EA-8-D, the reinforced-concrete walls are arranged more or less symmetrically on the mat, tending to load the mat uniformly, which minimizes potential tilting of the structure. These walls also serve to stiffen the massive, 5-ft thick reinforced-concrete mat foundation, minimizing dishing and resultant differential settlement.

Most of the static settlements will occur as the building is constructed or within a short time after construction. This is true even for the secondary settlements. Page 18 indicates more than half of the secondary settlement (0.69" of 1.11", as indicated) is expected to occur within the first month following construction. In addition, the building loads spread out with respect to depth. The soils adjacent to the mat foundation will also settle somewhat as the building is loaded; thus, there will not be an abrupt differential settlement at the face of the mat foundation. Further, there are no important-to-safety piping systems or electrical ducts connected between the yard area and the structure. Those systems that do run between the yard area and the building will be designed to accommodate appropriate levels of differential settlement, but since these connections will be made late in the construction schedule, most of the settlement will have already occurred, and the potential differential settlements for these connections will be much smaller than the estimated total settlement of 3 inches.

Table 14.1, "Allowable Settlement," in Lambe and Whitman (1969) indicates that the maximum tilting of crane rails and differential settlement of reinforced-concrete building curtain walls should be limited to $0.003\,\ell$, $(\delta/\ell \sim 1/300)$. The walls supporting the crane in the Canister Transfer Building are spaced 65 ft apart (S&W Drawing 0599601-EA-8-D). For spread footings, the differential settlement is generally estimated to be ~3/4 of the total settlement. Because mat foundations spread the loads out over wide areas, they generally are more tolerant of differential settlements. Terzaghi & Peck (1967) indicate:

*"... because of the random distribution of compressible zones in the subsoil (Fig 55.3) combined with the stiffening effect of the raft and building frame, it can safely be assumed that the differential settlement of*

*a raft foundation per inch of settlement is not more than one half the corresponding value for buildings on footings."*

Therefore, for mat foundations, assume the ratio of differential settlement to total settlement is ~3/8. Even if we assume that the total estimated settlement was applicable in contributing to the differential settlement between the two walls supporting the crane, the expected differential settlement would be only 3/8 x 3 in., which equals 1.12 in. This represents a tilt of only 1.12 in. over 65 ft, or

$$\frac{\delta}{\ell} = \frac{1.12 \text{ in.}}{65 \text{ ft} \times 12 \text{ in./ft}} = \frac{1}{700}$$

which is much smaller than the typical allowable of 1/300. Therefore, even using very conservative upper-bound estimates of the differential settlement applicable for the walls supporting the Canister Transfer Building crane, the performance of this crane will not be adversely affected by the estimated settlements.

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CONCLUSIONS:

A TOTAL SETTLEMENT OF 3.0 IN IS TO BE EXPECTED WITH THE FOLLOWING BREAKDOWN:

0.32 IN. OF IMMEDIATE SETTLEMENT IN LAYER 1

0.11 IN. OF IMMEDIATE SETTLEMENT IN LAYERS 2 – 4

1.46 IN. OF PRIMARY CONSOLIDATION SETTLEMENT IN LAYER 1

1.11 IN OF SECONDARY CONSOLIDATION (OVER 40 YRS) IN LAYER 1

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REFERECES


1. SWEC DRAWING 05996.01-EM-1, "CANISTER TRANSFER BUILDING, GENERAL ARRANGEMENT, SHEET 1", STONE & WEBSTER, DENVER, CO.

2. SWEC DRAWING 05996.01-EM-2, "CANISTER TRANSFER BUILDING, GENERAL ARRANGEMENT, SHEET 3", STONE & WEBSTER, DENVER, CO.

3. SWEC CALC. NO. 05996.01-G(B)-03-2, "ESTIMATE STATIC SETTLEMENT OF STORAGE PADS," STONE & WEBSTER, BOSTON, MA 7-7-97

4. SWEC CALC NO. 05996.02-SC-5, "SEISMIC ANALYSIS OF CANISTER TRANSFER BUILDING," STONE & WEBSTER, CHERRY HILL, NJ

5. SWEC CALC NO. 05996.01-G(B)-01-1, "DOCUMENT BASES FOR RECOMMENDED VALUES OF DYNAMIC SOIL PROPERTIES AND COEFFICIENTS OF SUBGRADE REACTION," STONE & WEBSTER, BOSTON, MA 05-08-97.

6. SWEC. CALC NO. 05996.01-G(B)-05-0, "DOCUMENT BASES FOR GEOTECHNICAL PARAMETERS PROVIDED IN GEOTECHNICAL DESIGN CRITERIA," STONE & WEBSTER, BOSTON, MA 05-08-97.

7. TERZAGHI, K, AND PECK, R.B., SOIL MECHANICS IN ENGINEERING PRACTICE, JOHN WILEY & SONS, NEW YORK, NY, 1967. III

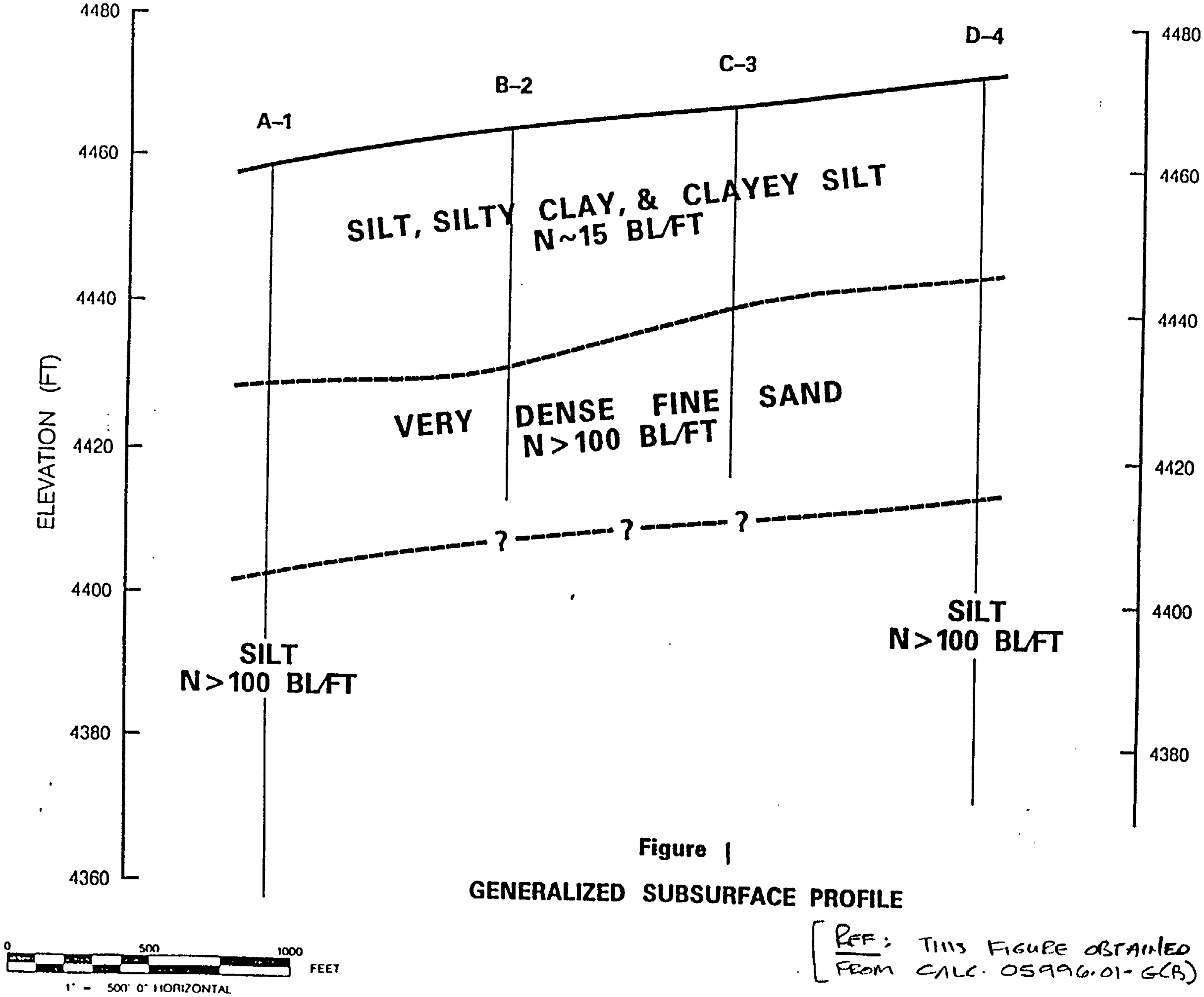


Figure 1

GENERALIZED SUBSURFACE PROFILE

[REF: THIS FIGURE OBTAINED FROM CALC. 05996.01-G(B)-03]

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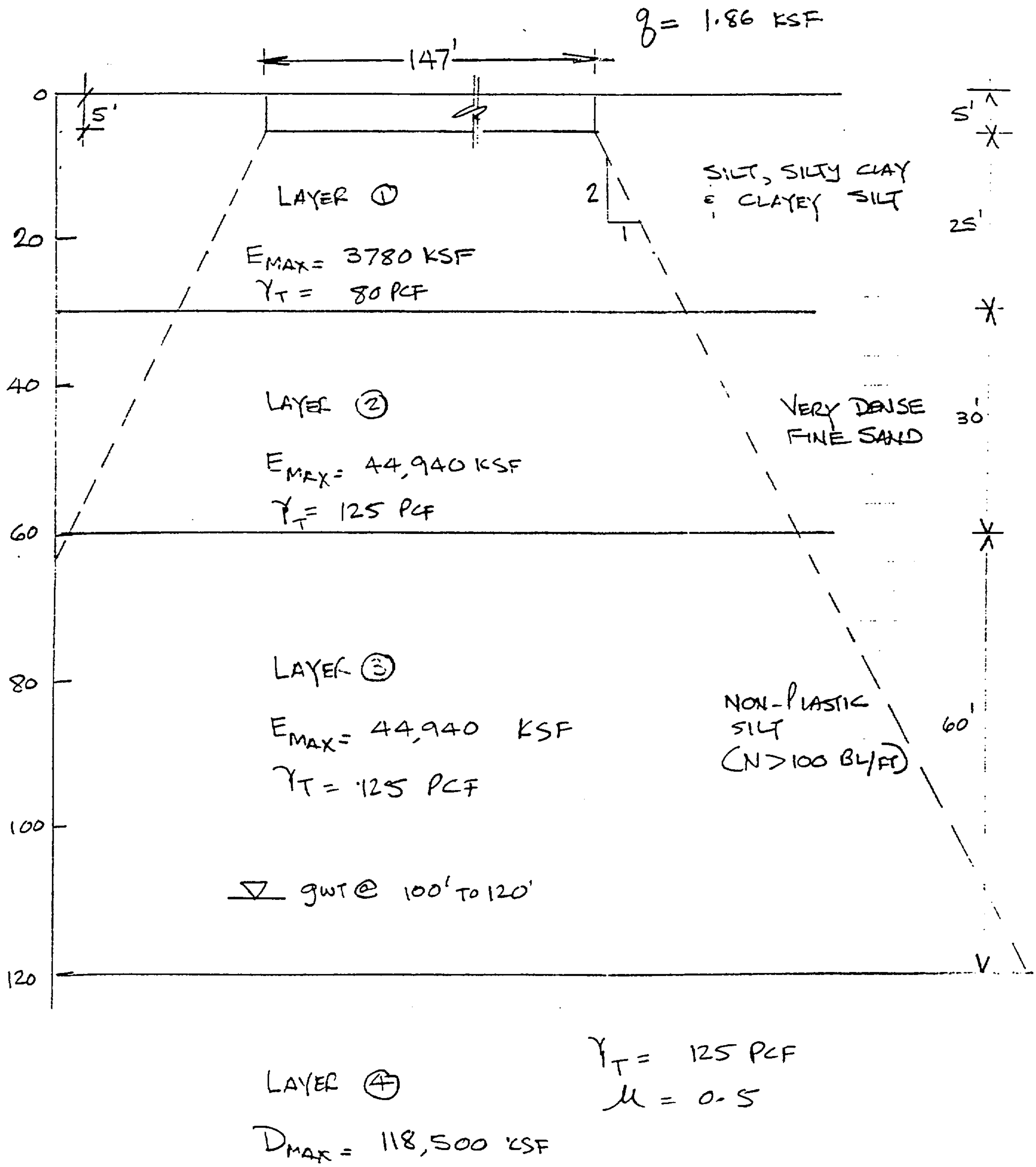
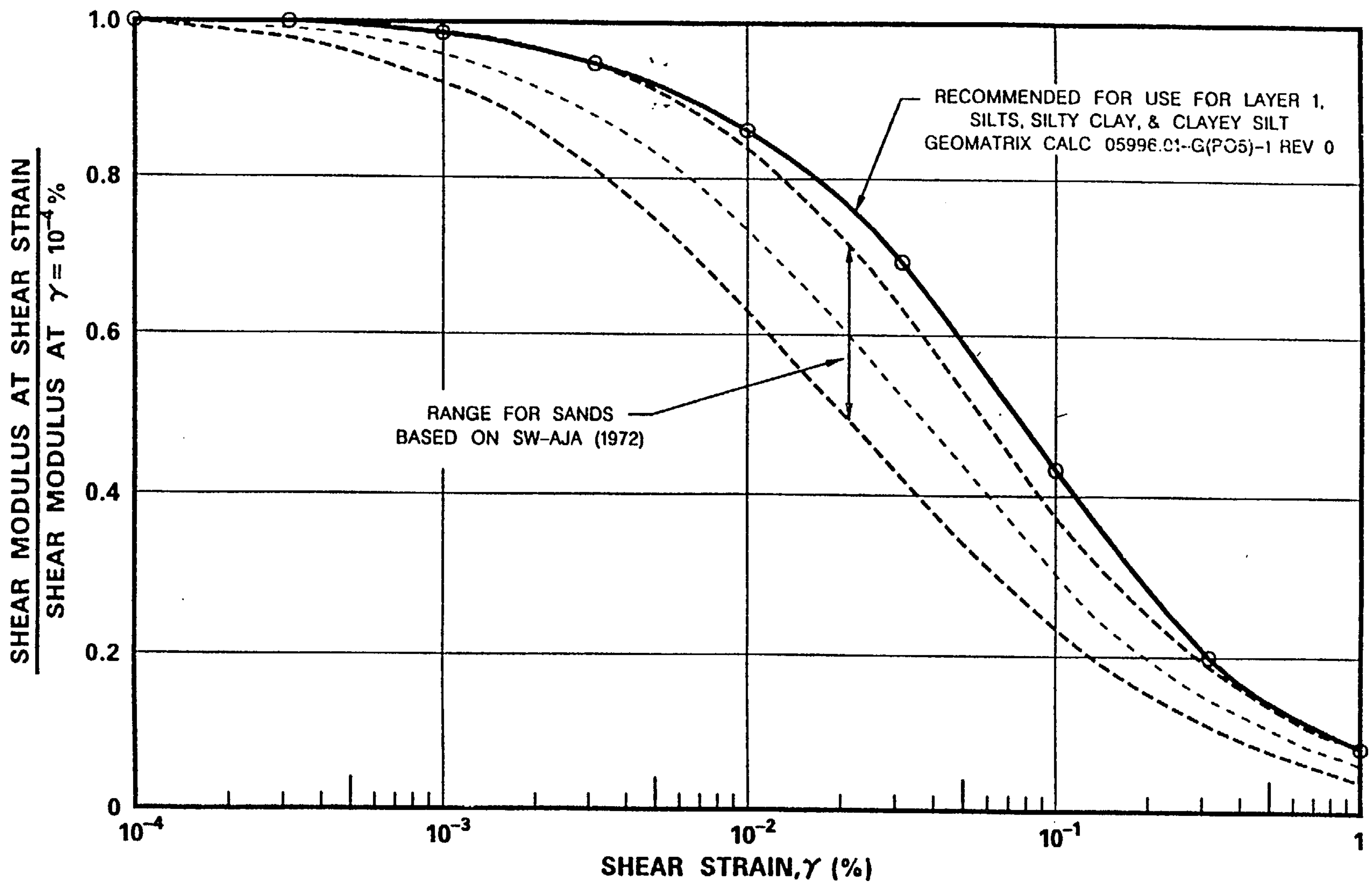
FIGURE 2: FOUNDATION PROFILE - CANISTER TRANSFER BUILDING




[NOTE: GEOTECHNICAL PROPERTIES FROM CALC 05996.01-G(B)-03]




(REF: THIS FIGURE OBTAINED FROM CALC. 05996.01-G(B)-03)

**Figure 3**

**VARIATION OF SHEAR MODULUS WITH SHEAR STRAIN**

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TABLE 1: ELASTIC SETTLEMENTS OF LAYERS 1 TO 3, USING LOW STRAIN MODULI.

| LAYER | $Z^{(1)}$ FT | $Z_{CL}^{(2)}$ FT | $\overline{\Delta\sigma_V}^{(3)}$ KSF | $E_{MAX}^{*}$ KSF | $\Delta\rho_{MIN}$ |
|---|---|---|---|---|---|
| | 5 | | | | |
| 1 | | 12.5 | 1.71 | 3780 | $(30-5)\frac{\times 12 \times 1.71}{3780} = 0.136$ IN. ← 80% OF TOTAL FOR LAYERS 1 TO 3. |
| | 30 | | | | |
| 2 | | 40.0 | 1.46 | 44,936 | $(60-30)\frac{\times 12 \times 1.46}{44936} = 0.012$ IN. |
| | 60 | | | | |
| 3 | | 85.0 | 1.18 | 44,936 | $(120-60)\frac{\times 12 \times 1.18}{44936} = 0.019$ IN. |
| | 120 | | | | TOTAL = 0.167 IN. |

(1) DEPTH BELOW GROUND TO THE TOP/BOTTOM OF STRATUM.

(2) DEPTH FROM BOTTOM OF MAT TO MIDDLE OF STRATUM.

(3) STRESS INCREASE AT THE MIDDLE OF LAYER.

$\overline{\Delta\sigma_V}$ = INCREASE IN STRESS AT CENTER OF LAYER ASSUMING $q$ AT BOTTOM OF MAT = 1.86 KSF, WHICH IS DISTRIBUTED ON 2V:1H WITH RESPECT TO DEPTH BELOW BOTTOM OF MAT. LOAD DISPERSION IS CONSIDERED ONLY ALONG THE WIDTH OF MAT (B = 147'), BUT IGNORED ALONG THE LENGTH OF MAT (L = 275'); A CONSERVATIVE ASSUMPTION.

$$\overline{\Delta\sigma_V} = \frac{1.86 \times 147}{(147 + Z_{CL})} \quad ; \quad E = \frac{\overline{\Delta\sigma_V}}{\varepsilon_V}, \text{ HENCE } \varepsilon_V = \frac{\overline{\Delta\sigma_V}}{E}$$

$$\Delta\rho = \Delta Z \cdot \varepsilon_V = (\text{STRATUM THICKNESS}) \times 12 \tfrac{\text{IN}}{\text{FT}^2} \times \frac{\overline{\Delta\sigma_V}}{E}$$