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AX Tank Farm Closure Settlement Estimates and Soil Testing

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Abstract: This study provides a conservative three-dimensional settlement study of the AX Tank Farm closure will fill materials and a surface barrier. The finite element settlement model constructed included the interaction of four tanks and the surface barrier with the site soil and bedrock. Also addressed are current soil testing techniques suitable for the site soil with recommendations applicable to the AX Tank Farm and the planned cone penetration testing.

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**AX TANK FARM
CLOSURE SETTLEMENT
ESTIMATES AND SOIL TESTING**

March 1999

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LIST OF TERMS

ASTM	American Society of Testing and Materials
CPT	Cone penetration test
CPTu	Cone penetrometers, piezocone
CU	Consolidated-Undrained
DMT	Dilatometer test
DSC	Directional shear cell
GWT	Groundwater table
HTI	Hanford Tanks Initiative
PLT	Plate load test
PMT	Pressuremeter test
SBPM	Self-boring pressuremeter tests
SBPMT	Self-boring pressuremeter test
SCPT	Seismic cone penetration test
SPC	Screw-plate compressometer test
SPT	Standard penetration test
SST	Single-shell tanks
TWRS	Tank Waste Remediation System
VisCPT	Vision cone penetrometer
UU	Unconsolidated - Undrained

1.0 INTRODUCTION

The objective of the AX Tank Farm settlement evaluations and review of current soil testing techniques is to develop engineering data that will support development of the final filling and surface barrier single-shell tank end state configuration. The evaluations included structural analyses of the interaction of the four AX Tank Farm tanks during filling and placement of the closure surface barrier. The soil testing technique review and evaluation include the testing techniques applicable to the granular sandy soil of the AX Tank Farm.

This work is supported as part of the Hanford Tanks Initiative (HTI) as Task D25L2, Structural Analysis, identified in the Tank Waste Remediation System (TWRS) Fiscal Year 1998 Multi-Year Work Plan. The single-shell tank evaluations initiated in FY 1998 are an extension of evaluations initiated in FY 1997. The results of evaluations conducted in FY 1997 are reported in HNF-SD-HTI-ES-003, Revision A, "Structural Performance of Candidate Fill Materials." The work documented in this report supplements the evaluations that were performed in FY 1997.

The focus of the FY 1998 structural evaluations are to provide bounding analytical predictions of settlements associated with tank farm backfill and closure activities. Additionally, soil testing techniques are reviewed and evaluated for application to the tank closure activities to refine or verify the structural settlement calculations.

The primary task in FY 1998 was to complete and document three-dimensional finite element modeling and results of settlement analyses including interaction of the four single-shell tanks (SST) in the AX Farm with the surface barrier, site soil, and the bedrock; also, to complete a variation of soil properties study.

A secondary task was to review and recommend tank farm soil testing techniques. Evaluations of limitations and potentials of soil testing methods, including cone penetrometer, lateral pressure testing, and triaxial testing of samples from bore holes were included. This task provided an assessment of the application of the cone penetrometer testing techniques for obtaining additional soils properties data for confirmation of the existing data and verification and refinement of analyses predictions.

2.0 EXECUTIVE SUMMARY

2.1 SETTLEMENTS BECAUSE OF SURFACE BARRIER CONSTRUCTION

The initial step in the determination of the soil settlements was to obtain a valid relation for soil modulus versus depth. The construction of the tank farm began with the removal of a sand and gravel overburden. Then the tanks were constructed and site sandy soil was compacted into place as fill. Sand and gravel soil that has been loaded with an overburden and/or compacted

becomes very linear and can be characterized by the unload-reload modulus obtained from triaxial testing. The Hanford Site test data (Dames and Moore 1988) were used to derive a simple relation for the variation of unload-reload modulus with confining pressure or depth. The unload-reload modulus was modeled by a simple overburden/confining pressure-dependent power equation.

The soil settlements, because of the construction of the surface barrier, was estimated by an ABAQUS/Standard Version 5.5 quarter symmetry finite element model of the AX Tank Farm. An equivalent pressure force was applied to the ABAQUS quarter tank farm model, to approximate a square 270-by 270-ft barrier with an assumed barrier design as documented by Baxter, Skelly, and Fredenburg (1996). The barrier weight was estimated to be 1,813 lbf/ft².

The finite element model was used with the unload-reload modulus to estimate AX Tank Farm settlements with interaction effects of four tanks, soil, bedrock and the closure surface barrier. The finite element predicted settlements of the surface barrier varied between 0.36 to 1.14 in. A summary tabulation of the calculated surface barrier settlements are provided in Table 2-1.

The finite element predicted settlements of the tank floor and foundation settlements because of the surface barrier load varied between 0.33 to 0.53 in. A summary tabulation of the calculated tank foundation and floor settlements are provided in Table 2-2.

The finite element predicted settlements of the tank floor and foundation because of an assumed tank fill of material with a density of 150 lb/ft³ varied between 0.29 to 0.84. A summary tabulation of the calculated tank foundation and floor settlements are provided in Table 2-3.

For comparison purposes, corresponding calculations performed in FY 1997 by use of the Schmertmann Method (Holtz 1991) are tabulated in Tables 2-1, 2-2, and 2-3. Plots illustrating the ABAQUS finite element model are provided in Figures 3-3 through 3-11.

The ABAQUS finite element modeling of the settlements because of the closure surface barrier were reviewed and checked by a University of Washington soils expert and found to be conservative with actual settlements expected to be smaller than predicted. The reviewer's assessment of the finite element model and analytical approach is provided in Appendix A.

2.2 RECOMMENDATIONS FOR ADDITIONAL AX TANK FARM SOIL TESTING

The conventional approach for obtaining the soil modulus for settlement calculations is to obtain undisturbed samples of soils at the site and perform appropriate laboratory tests on those samples. Unfortunately, for a number of reasons, this conventional procedure is not very reliable for the granular deposits at the AX Tank Farm site. Reconstituting test specimens of granular

soils from disturbed samples is also not very reliable. Thus the only viable alternative is to test the soils in situ.

In situ testing means that a probe of some type is pushed or drilled into the ground, and either the energy or force required to insert the probe is measured, or in some tests, a membrane on the probe is inflated and certain measurements are made during the inflation. In most in situ devices, determination of soil properties such as the soil modulus is not directly possible and must be obtained by empirical correlations.

The in situ devices that appear to be the most suitable for the AX Tank Farm site are the cone penetration test (CPT), the self-boring pressuremeter test (SBPMT), the screw-plate compressometer test (SPC), and the dilatometer test (DMT). These tests are described in some detail in the report including, advantages and disadvantages and the soil properties that may be obtained—either directly or through correlations—from the devices. The CPT and DMT are the most empirical, but the CPT has a long history and considerable research behind the published correlations. Further, it is a relatively inexpensive test to perform in the field, is suitable for both compacted backfill and natural deposits of granular materials, and has the added advantage that possibly contaminated materials are not obtained from below the subsurface. The DMT also has these same advantages and applicability, but the research basis for correlations with soil properties for this test is less developed.

The SBPMT, properly performed, permits possibly the best measure in situ of the compression modulus, although this modulus is determined in the horizontal direction. If significant anisotropy of properties exists at the site, proper interpretation may be a factor. The primary disadvantage of this test is that soil cuttings are obtained, and if contaminated, they must be properly disposed of.

The SPC deserves some serious consideration. With the SPC, the compression modulus is obtained in the vertical direction, and no cuttings are brought to the surface during or after the test is conducted. While not as simple or rapid as the CPT and DMT, the SPC is still inexpensive and rapid enough so that statistical evaluation and the spatial variability both with area and depth can be readily determined.

2.3 RECOMMENDATIONS FOR ADDITIONAL TANK FILL AND SURFACE BARRIER EVALUATIONS

Recommendations for consideration in planning future finite element and/or soil testing activities include the following.

- Provide engineering support to the cone penetrometer testing of the AX Tank Farm. Multisensor CPT probes proposed for use in vicinity of the AX Tank Farm site should be

appropriately evaluated so that they might also provide for the site characterization and profiling, and soil properties such as the compression modulus through empirical correlations. The additional layered soil modulus data of the fill and foundation soils would allow more accurate predictions of the settlements because of the backfilling of the tanks and the final placement of the surface barrier.

- Evaluate and correlate the cone penetrometer data to the soil data used for the settlement calculations. If significant differences are found, revise the soil modeling and redo the settlement calculations.
- In addition to the CPT and related probes, serious consideration should be given to performing seismic cone penetration, self-boring pressuremeter, screw plate compressometer, and dilatometer tests in the backfill and natural soils. We recommend preparation of a detailed work and integration plan for use of the seismic CPT, SBPMT, SPC, and DMT.
- Investigate the interaction of adjacent SST farms on the settlement of tanks in the AX Farms.
- Evaluate/analyze the effects of SST backfilling with various combinations/layers of materials such as gravel, grout, concrete, and asphalt. Investigate the interaction of the backfill materials with tank structure and filling settlements.
- Evaluate potential cracking of the existing tank structure because of backfill material placement and degradation as a function of time.
- Complete and document seismic analyses of the Hanford surface barrier design for performance to current Hanford Site seismic design criteria. The current Hanford specified surface motion criteria will be used for shear and vertical motion. This analyses will include viscoelastic modeling of the asphalt barrier.

Based on the results of seismic analysis above, review and make recommendations on the Hanford surface barrier design. Past geologic records imply that one or more large seismic event can be expected during a time period of 1,000 years. The surface barrier must maintain moisture control after being subjected to seismic surface shear. Also consider bentonite layers or composite membranes combined with bentonite to improve moisture control as the surface barrier degrades with time and geologic events. Consider soil slope stability requirements for barrier design, including the current higher seismic specifications. Investigate composite membranes to maintain sealing from water for this time period. The surface barrier recommendations must also investigate the lapping and splicing of membranes to large sizes (approximately 270-by 270-ft) and the thermal expansion and contraction and cracking of asphalt layers.

Table 2-1. Calculated Settlements at the Top of the Existing Soil Surface (the Bottom of the Barrier), Because of Surface Barrier Loading

Location	Model	Settlement (inch)
Center of Barrier	ABAQUS	1.14
	Schmertmann Perloff	0.41 to 0.71
Corner of Barrier	ABAQUS	0.36
	Schmertmann Perloff	0.20 to 0.36
Side Midpoint of Barrier	ABAQUS	0.62
	Schmertmann Perloff	0.28 to 0.48
Tank Shoulder on Inside East-West Line	ABAQUS	1.02
Tank Shoulder on Outside East-West Line	ABAQUS	1.00
Tank Shoulder on Inside North-South Line	ABAQUS	1.05
Tank Shoulder Outside North-South Line	ABAQUS	1.00
Center of Tank Dome	ABAQUS	1.18

Table 2-2. Calculated Tank Foundation and Floor Settlements Because of Surface Barrier Loading

Location				Settlement (in ch)
ABAQUS Model of AX Tank Farm	Tank Foundation under Wall	East-West Line	Inside Farm	0.52
			Outside Farm	0.47
		North-South Line	Inside Farm	0.53
			Outside Farm	0.46
	Center of Tank Floor			0.33
	Center of Tank Dome			0.86
Tank Foundation Settlement, Schmertmann Method with Surface Barrier Load on one tank only				0.062 to 0.11

Table 2-3. Calculated Tank Foundation and Dome Settlement Because of Tank Backfill With Material of 150 lb/ft³ Density

ABAQUS Model of AX Tank Farm			Settlement (inch)
Tank Foundation under Wall	East-West Line	Inside Farm	0.39
		Outside Farm	0.29
	North-South Line	Inside Farm	0.38
		Outside Farm	0.29
Center of Tank Floor			0.84
Center of Tank Dome			0.35
Tank foundation Schmertmann Method backfill of one tank only			0.15 to 0.27 ¹

¹Schmertmann Estimates

3.0 SETTLEMENT ESTIMATES FOR CONSTRUCTION OF A SURFACE BARRIER

3.1 SOIL MODELING USED FOR SETTLEMENT CALCULATIONS

The most important factor influencing the modulus of granular soils is the stress history, or more precisely, the strain history of the deposit. The construction sequence of the Hanford Site waste tanks began with removal of a sand and gravel overburden down to the level of the present tank foundation. Then the tanks were constructed and the tank farm buried under soil fill.

The tank foundation soil at the Hanford Site has already been subjected to vertical pressure loads greater than a tank filled with waste. Hanford Site history also included an additional overburden of water. Consideration of the history of the tank foundation soil led to an engineering judgment to employ soil unload-reload modulus derived from triaxial test data, Dames and Moore (1988).

Cylindrical samples of the Hanford Site soil have been taken from bore holes at various depths (Dames and Moore 1988, 1989). The soil is typically a cohesionless sand and gravel mix. The cylindrical soil samples were wrapped in a rubber membrane for testing. These samples were subjected to radial confining pressures representative of the bore hole depth as they were also loaded in compression. The stress strain relation of the soil cylinders to failure was very nonlinear. Unloading and reloading of the samples before failure resulted in a nearly linear unload-reload modulus from Dames and Moore (1988). The Hanford Site test data were used to derive a simple relation for the variation of unload-reload modulus with confining pressure or depth.

The University of California at Berkeley has done considerable research on soil constitutive modeling including variation of the unload-reload modulus with confining pressure, Wong and Duncan (1974); Duncan et al. (1980); Seed and Duncan (1983). The results show that the unload-reload soil modulus can be defined as in the following Equation 3-1.

$$E_{ur} = K_{ur} P_a \left(\frac{P_c}{P_a} \right)^n \quad (3-1)$$

where

- E_{ur} = unload-reload elastic modulus in consistent units
- K_{ur} = nondimensional constant = 726.2
- P_a = atmospheric pressure in consistent units
- P_c = confining pressure in consistent units
- n = power constant = .730.

The above derived nondimensional constant and power documented from Dames and Moore (1988) test data was checked against testing documented in Dames and Moore (1989). Equation 3-1 was fit to the unload-reload modulus test cases for confining pressures of 27.8 and 55.6 lbf/in² of Dames and Moore (1988). When the equation was used to predict the 1989 test results at a confining pressure of 13.9 lbf/in², the result was the exact average of the two test points. Since the 1989 testing showed a plus and minus bounding variation from the Equation 3-1 of 27%, it is recommended that the modulus and settlements include this range of variation to establish upper and lower bounds based on the limited Hanford soil testing conducted to date. The documented finite element calculated settlements used Equation 3-1 for soil modulus, thus the 27% variation is easily addressed by a linear scaling of the documented settlement results.

An idealized schematic diagram of our hyperbolic soil model is shown in Figure 3-1 which identifies the elastic and plastic strain components in a typical cohesionless soil triaxial test. A plot of the triaxial test data for a soil sample reported by Dames and Moore (1988) for the grout vault site is shown in Figure 3-2 for comparison.

3.2 SETTLEMENTS BECAUSE OF SURFACE BARRIER CONSTRUCTION

The finite element modeling of the settlement of the tank surface barrier was based on an approximately square 270-by 270-ft barrier with an assumed barrier design as documented by Baxter, Skelly, and Fredenburg (1996). The barrier weight is estimated to be 1,813 lbf/in².

The barrier size was estimated using the ground topography drawing H-2-44555. The centers of AX Tanks 101, 102, 103, and 104 were located. The inside tank radius of the steel liner was 37.5 ft. The barrier width estimate assumes that the barrier extends beyond the edge of the tanks a distance equal to 20 ft to account for known waste plumes below the tanks plus an additional 25-ft overlap to account for expected vadose zone groundwater movement patterns (an additional 45 ft beyond the tank wall). The result was only 4% from being square, thus the barrier construction was assumed square and the numbers rounded off to two significant figures. The result was a square 270-by 270-ft barrier.

A detailed finite element model of the AX Tank Farm was completed to study the settlement interactions of the proposed surface barrier with the four single-shell tanks. The AX Tank Farm finite element model used quarter symmetry to reduce the required numerical effort. Thus, by applying boundary conditions to establish a symmetrical relationship, it was necessary to model only one SST in three dimensions with foundation soil down to the 300-ft elevation of bedrock and fill soil.

Figure 3-3 shows an isometric view of the simplified shell modeling of the SST. Figure 3-4 shows a rotated isometric view of the floor and foundation of the SST. Figure 3-5 shows a cut-away isometric view of the shell modeling of the SST. The plotted display of the shell elements

show the mid-plane location of the tank floor, foundation, wall, and dome. The bottom of the tank foundation was assumed to be at the 621.42-ft elevation.

Figure 3-6 shows the SST with the solid modeling of the foundation soil to the assumed rigid boundary at bedrock level (elevation of 300 ft above sea level). Figure 3-7 shows a close zoomed isometric view of the tank on the foundation soil. Figure 3-8 shows the final quarter model of the AX Tank Farm with the addition of the solid modeling of the fill soil to an elevation of 681.7 ft. Figure 3-9 shows a top view of the quarter model of the AX Tank Farm. In Figure 3-9 arrows point to the boundaries of the proposed surface barrier. Figure 3-10 shows a cut-away view of the quarter model of the AX Tank Farm showing the enclosed tank. Figure 3-11 shows a close zoomed view of the cut-away view of the tank.

The AX Tank Farm quarter symmetry finite element model shown in Figure 3-8 was subjected to a 1,813 lb/ft² pressure load applied to the indicated barrier foot print (with an assumed empty tank). Figure 3-12 shows a deflected shape isometric plot of the tank farm quarter model with a magnification factor of 100. Figure 3-13 shows the actual deflected shape isometric plot with a more realistic magnification factor of 1.0. Figure 3-14 shows a cut-away view of the tank deflected shape with a magnification factor of 100. Figure 3-15 shows a cut-away view of the tank deflected shape with a more realistic magnification factor of 1.0.

The actual deflections at various tank farm surface points were obtained by postprocessing the finite element results. Figure 3-16 shows a zoomed isometric view of the surface of the tank farm quarter model soil at the location of the surface barrier. The surface barrier deflections were tabulated for the four locations indicated on Figure 3-16, the center of the barrier (1.14 in.), the two edge middles (both 0.62 in.), and the one corner (0.34 in.). Figure 3-17 shows the close zoomed view of the soil surface modeling over the tank. The surface barrier settlements were tabulated for the four locations over the tank shoulder indicated on Figure 3-17, on the East-West line inside (1.02 in.) and outside the farm (1.00 in.), on the North-South line inside (1.05 in.) and outside the farm (1.00 in.). Additionally the surface barrier settlement over the tank dome at the apex was 1.18 in. with part of the deflection being the tank dome deflection, with filling of the tank deflections will decrease. A filled tank would have dome deflections approximately equal to the linear average of the tank shoulder deflections (1.02 in.).

The barrier settlements were previously estimated using the Schmertmann method and the same soil modeling using the pressure dependent power relation of unload-reload modulus (Equation 3-1). For the Schmertmann calculation the base of the barrier was assumed as a circular axisymmetric foundation with a diameter of 270 ft. The settlements were calculated down to the 300-ft elevation of the basalt bedrock as compared to the surface barrier base at 681.7-ft elevation. The bedrock was assumed to be rigid and make no contribution to the settlements. The circular flexible foundation settlements were related to settlements of a square flexible foundation by settlement factors from Perloff (1975).

The center of the barrier free soil Schmertmann method settlement assuming a zero foundation depth was estimated to be in the range of 0.41 to 0.71 in. The barrier corner Schmertmann method settlement was estimated to be in the range of 0.20 to 0.36 in. The barrier side midpoint Schmertmann method settlement was estimated to be in the range of 0.28 to 0.48 in.

The tank foundation settlements because of the surface barrier loading were obtained from the tank farm quarter model by detailing the foundation and floor of the single-shell tank and post-processing of the analysis data. Figure 3-18 shows the predicted deflections for various positions on the tank foundation under the wall, along the East-West line inside (0.52 in.) and outside the farm (0.47 in.), along the North-South line inside (0.53 in.) and outside the farm (0.46 in.). The settlement of the center of the tank floor was 0.33 in. These tank foundation settlements assume an empty tank. If the tanks are filled with progressive pours of grout or concrete, the foundation settlements (because of the closure surface barrier load) would result in a slight decrease in the settlements of the tank ring foundation under the wall, but the tank floor settlements would become approximately a linear average of the ring foundation settlements.

The tank foundation settlements because of the surface barrier loading had also been previously estimated using the same soil modeling and the Schmertmann method. In the application of the Schmertmann calculation, the tank foundation was assumed to be rigid with the tank floor included. The Schmertmann method settlements because of the large barrier pressure load transmitted to the foundation were estimated to be in a range of 0.062 and 0.11 in. This local tank foundation Schmertmann settlement did not include a superposition of the large field vertical soil strains imposed by the full surface barrier. Additional settlement variations of the same order as the tank foundation settlement may be caused by an unknown compaction in the backfill around the 241-AX SSTs. This fill soil variation has not been considered in this analysis because of a lack of field data.

Figure 3-1. Hyperbolic Soil Model for Hanford Site Soils (Lade and Duncan 1975).

Figure 3-2. Triaxial Test Data from Dames and Moore (1988).

Figure 3-3. Isometric View of Simplified Shell Modeling of Single Shell Tank (input file axtank15.inp 1/9/98).

Figure 3-4. Rotated Isometric View of Single Shell Tank, Detailing Floor and Foundation Modeling (input file axtank15.inp 1/9/98).

Figure 3-5. Cut-away Isometric View of Shell Modeling of Single Shell Tank (input file axtank15.inp 1/9/98).

Figure 3-6. Isometric View of AX Tank Farm Quarter Model, With One Tank and Foundation
Soil to Bedrock Boundary (input file axtank15.inp 1/9/98).

Figure 3-7. Close Zoomed Isometric View of Tank on Foundation Soil (input file axtank15.inp
1/9/98).

Figure 3-8. Final Quarter Model of AX Tank Farm With the Addition of Fill Soil to an Elevation of 681.7 ft (input file axtank15.inp 1/9/98).

Figure 3-9. Top View of Final Quarter Model of AX Tank Farm (input file axtank15.inp 1/9/98).

Figure 3-10. Cut-away Isometric View of the Quarter Model of the AX Tank Farm Showing the Enclosed Tank (input file axtank15.inp 1/9/98).

Figure 3-11. Close Zoomed View of the Cut-away View of the Tank (input file axtank15.inp
1/9/98).

Figure 3-12. Deflected Shape of the AX Tank Farm Quarter Model With a Displacement Magnification Factor of 100 (input file axtank15.inp 1/9/98).

Figure 3-13. Deflected Shape of the AX Tank Farm Quarter Model With a Realistic Displacement Magnification Factor of 1.0 (input file axtank15.inp 1/9/98).

Figure 3-14. Cut-away View of Tank Deflected Shape With a Displacement Magnification Factor of 100 (input file axtank15.inp 1/9/98).

Figure 3-15. Cut-away View of the Tank Deflected Shape With a Realistic Displacement Magnification Factor of 1.0 (input file axtank15.inp 1/9/98).

Figure 3-16. Zoomed View of the Deflected Shape Plot of the Surface Barrier Footprint, With Displacement Magnification Factor of 1.0 (input file axtank15.inp 1/9/98).

Surface Barrier Load 1,813 lb/ft²

Barrier Corner 0.34 in. Settlement

Barrier
Edge Middle
0.62 in.
Settlement

Barrier
Edge Middle
0.62 in.
Settlement

Barrier Middle 1.14 in. Settlement

Figure 3-17. Close Zoomed View of the Deflected Shape Plot Above the Tank, With Displacement Factor of 1.0 (input file axtank15.inp 1/9/98).

Figure 3-18. Detail of the Tank Foundation and Floor Deflected Shape With a Displacement Magnification Factor of 1.0
(input file axtank16.inp8/5/98).

3.3 EXPECTED SETTLEMENT DURING TANK BACKFILL

The total volume of the 1,000,000-gallon tank, including the dome, was calculated to be 178,120 ft³. If the tank fill is assumed to have a weight density between 110 to 150 lbf/ft³, the total foundation force on each tank because of the fill is approximately 19,600,000 to 26,700,000 lb.

Figure 3-19 shows the deflected shape (magnified by a factor of 100) of the foundation soil and tank at a section cut of the quarter model of the AX Tank Farm assuming the tank backfilled with material having a weight density of 150 lbf/ft³ (Note: The analysis in Section 3.3 did not include surface barrier loads). Figure 3-20 shows the deflected shape section cut with a more realistic magnification factor of 1.0.

The tank foundation settlements because of backfilling the tank were obtained from the tank farm quarter model by detailing the foundation and floor of the single shell tank and postprocessing of the analysis data. Figure 3-21 shows the predicted deflections for midplane positions on the tank foundation under the wall along the East-West line inside (0.39 in.), and outside the farm (0.29 in.); along the North-South line inside (0.38 in.) and outside the farm (0.29 in.). Also, the settlement of the center of the tank floor was 0.84 in. (for fill of 150 lbf/ft³ density). The relative large deflection of the center of the floor would only be valid for a tank filled with loose material. If a series of grout or concrete pours are made, the settlements of the ring foundation under the wall would increase slightly with the center of the floor deflections being approximately a linear average of the ring foundation settlements. The settlement deflections of the walls translate into motion of 0.35 in. at the center top apex of the tank dome.

Emptying the tanks of waste material causes an uplift of the tank foundation and floor. Filling the tanks would result in settlements negating the emptying uplift with some additional settlement because of filling of the normally empty head space. If the empty tank condition is assumed as a reference, the total relative deflections because of tank filling and construction of the surface barrier can be obtained by linear addition of the settlement results of Sections 3.2 and 3.3. The review of the finite element model and the resulting soil settlement predictions were found to be conservative with actual settlements expected to be smaller (Appendix A).

Fill of lighter density would result in linear scaling of the tank foundation settlements. For example, the settlement of the center of the tank floor for 110 lbf/ft³ density fill would be $(110/150) \times 0.84 = 0.62$ in. The settlement of the tank foundation under the wall, along the East-West line inside the farm for 110 lbf/ft³ density fill would be $(110/150) \times 0.39 = 0.29$ in.

With an assumed fill density of 150 lbf/ft³, the Schmertmann method settlement during tank backfilling was estimated to have a range of variation between 0.15 to 0.27 in. (accounting for the 27% soil test data variation). When tank backfilling is done, a "before" and "after" survey is

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recommended. Although the past survey techniques are accurate enough to observe this settlement, improved survey accuracy is recommended for this benchmark verification.

Figure 3-19. Deflected Shape With a Magnification Factor of 100 of the Foundation Soil and Tank at a Section Cut of the Quarter Model of the AX Tank Farm (input file axtank16.inp 8/5/98).

Figure 3-20. Deflected Shape With a Magnification Factor of 1.0 of the Foundation Soil and Tank at a Section Cut of the Quarter Model of the AX Tank Farm (input file axtank16.inp 8/5/98).

Figure 3-21. Detail of the Tank Foundation and Floor Deflected Shape With a Displacement Magnification Factor of 1.0 (input file axtank16.inp 8/5/98).

3.4 SETTLEMENT CONCLUSIONS AND RECOMMENDATIONS

The AX Tank Farm was modeled as a finite element model using ABAQUS/Standard Version 5.5. The finite element settlement calculations include the full three-dimensional effects of the AX Tank Farm. The engineering work done by the Hanford Site engineering staff and reviewed by Professor Holtz of the Civil Engineering Department at the University of Washington (Appendix A) indicated the finite element settlement results conservative with settlement estimates too large.

It is recommended that during the cone penetrometer testing of the AX Tank Farm, the penetrometer forces be recorded and reduced to soil modulus data at different depths. The additional layered soil modulus data of the fill and foundation soils would allow more accurate predictions of the settlements because of the backfilling of the tanks and the final placement of the surface barrier.

Additional finite element studies for consideration include activities to:

- Evaluate/analyze the effects of SST backfilling with various combinations/layers of materials such as gravel, grout, concrete, and asphalt. Investigate the interaction of the backfill materials with the tank structure and filling settlements.
- Investigate the interaction of adjacent SST farms on the settlement of tanks in the AX Farm.
- Evaluate potential cracking of backfill materials and the existing tank structure because of backfill material placement and degradation as a function of time.

During January 1999, a cone penetration test is scheduled for the East AX Tank Farm adjacent to Tank AX-104. Correlate the cone penetrometer data to the soils data used for the settlement calculations. If significant differences are found, revise the soil modeling and redo the settlement calculations.

The Hanford surface barrier concepts/designs should also be evaluated for seismic performance. The current Hanford specified surface motion criteria should be used for shear and vertical motion.

4.0 SOIL TESTING TECHNIQUES

4.1 INTRODUCTION

Because of their nonlinear, nonconservative, and anisotropic mechanical behavior, as well as the variability and heterogeneity of natural deposits because of the capriciousness of nature, soils are very complex engineering and construction materials. Common linear theories applied to other engineering materials have only limited applicability to soils, and thus much of geotechnical practice is highly empirical. For example, in order to estimate the compressibility and strength of soils when subjected to engineering loads, rather simple laboratory tests are performed on specimens of soils taken from the site, or in some cases, tests are performed in the field (in situ). There are advantages and disadvantages to both types of tests, and in many projects, both approaches are used to determine the engineering properties of soils. As noted by Kramer (1996), laboratory tests are performed on relatively small specimens assumed to be representative of the soil deposit under consideration. They are either element-type or model-type tests. In element tests, soil specimens are subjected to reasonably uniform initial stress conditions and stress changes during the test. In model tests, the soils are subjected to nonuniform boundary stresses and strains that must be properly interpreted to obtain meaningful results.

Detailed test procedures are given by, e.g., U.S. Army Corps of Engineers (1970) and ASTM (1998), while interpretation of test results is discussed in textbooks such as Leonards (1962); Holtz and Kovacs (1981); Mitchell (1993); and Terzaghi, Peck, and Mesri (1996) to name a few.

In the following sections, discussion is limited to the tests that are most appropriate for granular soils. Both conventional as well as some of the less common tests primarily used for research on the engineering properties of soils are included.

4.2 LABORATORY TESTS FOR STRENGTH AND COMPRESSIBILITY

4.2.1 Conventional Tests

The conventional laboratory tests for compressibility and strength of granular soils are the direct shear test and the cylindrical compression or "triaxial" test. Triaxial tests may also be conducted in extension, although this mode is much less common than compression tests. (The one-dimensional consolidation test is appropriate only for determination of the compressibility properties of fine-grained, cohesive soils.)

The direct shear test, the oldest of the common tests for determining the strength properties of soils, is in principle a very simple test. The soil is confined in a shear box that is separated horizontally into two halves. One half is fixed, and with respect to that half, the other half is either pushed or pulled horizontally. Normal load is applied to the soil specimen in the shear box

by a stiff loading block. The shear load, horizontal deformation and vertical deformation are measured during the test. When the shear and normal forces are divided by the nominal area of the specimen, the shear stress as well as the normal stress on the horizontal failure plane are obtained. Essential features, typical test results, and the Mohr-Coulomb failure diagram are shown in Figure 4-1.

The main advantages of the direct shear test are that it is inexpensive, fast, and simple, therefore, it is often used to test reconstituted specimens of granular soils. The disadvantages are that failure is forced to occur along a narrow, horizontal plane, and this may not be the weakest plane in the specimen. It is difficult to almost impossible to control drainage, and therefore, the test must be conducted in drained shear which is not a serious disadvantage for granular soils. There are large stress concentrations at the specimen boundaries, that cause highly nonuniform stress conditions within the test specimen itself. And finally, uncontrolled rotation of principal stresses occurs between the start of the test and failure. To accurately model the in situ loading conditions, the amount of rotation should be known and accounted for, but this is not done in ordinary practice. Although the initial part of the τ - σ diagram in Figure 4-1b appears to be similar to a stress-strain diagram, a tangent modulus derived from this diagram has never been considered to be valid, mainly because of all the disadvantages just described.

Detailed test procedures for the direct shear test are given by U. S. Army Corps of Engineers (1970) and ASTM (1997-D 3080).

The *cylindrical compression* test, commonly called the *triaxial* or *triaxial compression* test, was developed about 1930 in an attempt to overcome some of the disadvantages of the direct shear test. Although the triaxial compression test is more complicated in terms of apparatus and procedure, it is by far the more common of the two tests, primarily because of its versatility. In contrast to the direct shear test, both drainage and stress paths can be controlled reasonably well during shear. This means that rather complex stress and drainage paths that occur in the field in specific design situations can be modeled in the laboratory. In addition, principal stresses do not rotate during shear, failure can occur on any preferred weak plane, and stress concentrations, though present, are less influential than in the direct shear box test.

The principle of the triaxial compression test is shown in Figure 4-2a. The soil specimen is encased in a rubber membrane to prevent the pressurized cell fluid—usually water—from penetrating the pores of the soil. Axial load is applied by a piston, and in drained tests, the volume changes of the specimen during shear are measured. As mentioned, drainage can be controlled in this test, and when undrained tests are performed, sometimes the pressures induced in the pore water are measured. As shown in Figure 4-2b, the stresses on the boundary of the specimen are assumed to be principal stresses. This is not strictly correct, as some shear stress acts on these planes, particularly on the end planes of the specimen. However, the assumption permits Mohr's circles of the stress conditions in the specimen at failure to be constructed and

Mohr-Coulomb failure envelopes to be drawn when two or more specimens of the same materials are tested at more than one confining pressure.

Detailed test procedures for triaxial tests are given by U. S. Army Corps of Engineers (1970) and ASTM (1997). Standard test procedure ASTM D 4767 is recommended for the consolidated-undrained (CU) triaxial test and ASTM D 2850 is the procedure for the unconsolidated-undrained (UU) test. As explained by Holtz and Kovacs (1981), the unconfined compression test (ASTM D 2166) is a special case of the UU test.

Figure 4-1. (a) Schematic Diagram of Direct Shear Apparatus; (b) Typical Test Results (dense sand); and (c) Mohr Diagram for Specimens at the Same Relative Density (Holtz and Kovacs, 1981)

Figure 4-2. (a) Schematic Diagram of the Triaxial Apparatus; (b) Assumed Stress Conditions on the Triaxial Specimen; (c) Mohr's Circle Construction of Failure Envelope (Holtz and Kovacs, 1981).

4.2.2 Specialized or Research-Oriented Tests

During the past 60+ years, researchers have developed several specialized tests including the plane strain, cuboidal or “true triaxial,” directional shear cell, hollow cylinder, torsional or ring shear, and direct simple shear tests to model more complex loading situations than are permitted in the direct shear or triaxial compression tests. For a detailed discussion of these specialized tests, see Saada and Townsend (1981).

In *plane strain* tests, the ends of the specimen are fixed in such a way that the intermediate principal strain is zero. This is shown schematically in Figure 4-3a. This test was developed to more closely model certain field loading conditions such as many slopes and embankments, long narrow excavations, and strip footings. These loading conditions are closer to plane strain than they are to axisymmetric, the condition modeled by the cylindrical compression tests for example. Although its apparatus and procedure is not significantly more complex than conventional triaxial compression equipment, thus far plane strain tests have been primarily for research purposes.

Cuboidal or “true triaxial” tests were developed so that stresses and deformations in all three principal directions could be independently controlled or measured. As can be imagined from the schematic in Figure 4-3b, this test is much more complex than conventional triaxial tests, although today computers make control of stress changes and measurement of stresses and deformations much easier than in the past. Stresses can be applied by means of very stiff flat platens, by flexible membranes, or a combination of both; there are advantages and disadvantages of these boundary conditions, as described by Sture and Desai (1979) and Saada and Townsend (1981). If the specimen dimensions are more than about 100 mm on a side, test specimens are difficult to prepare and handle. These factors mean that the true triaxial test is used exclusively for research.

Figure 4-3c shows the three Mohr's circles that can be constructed from both plane strain and true triaxial test results, provided that all three principal stresses are measured.

A device related to both plane strain and true triaxial tests is the directional shear cell (DSC) test. In this test, a cubical specimen is subjected to both normal and shear stresses simultaneously on four faces, as shown in Figure 4-4a, while the other two are maintained in plane strain. Thus, continuous rotation of the principal stresses is possible, and anisotropic soil behavior can be investigated in the DSC. Because of the complexity of the DSC apparatus, it has been exclusively used for research, so far only on sand.

Another research type test is the hollow cylinder test in which a thin-walled tube of soil is subjected to an internal and external cell pressure, axial stress, and torsion on the specimen ends as shown in Figure 4-4b. By varying all three applied stresses, the influence of the intermediate principal stress and the rotation of principal stresses can be studied. Hollow cylinder tests are

really triaxial tests conducted on hollow circular specimens, and with few exceptions, most hollow cylinder tests have been conducted on cohesive soil specimens. In any case, this test has not been used in practice.

The effect or influence of the intermediate principal stress on the shearing resistance and compression modulus of granular soils was discussed in some detail by Moore, Holtz, and Wagenblast (1995). In general, the soil friction angle is several degrees higher in plane strain, and the compression modulus is higher by a factor of between two and four in comparison with the modulus obtained in conventional triaxial tests. The influence of the rotation of principal stresses on the strength and deformation characteristics of sand is not so well established.

There are two other direct shear type tests that should be mentioned. In the *torsional or ring shear* test, a relatively thin ring of soil is gripped top and bottom and the top rotated relative to the bottom so that the specimen is sheared to very large deformations. The ring shear test is used primarily for cohesive soil specimens. On the other hand, the *direct simple shear* test has been used for both soil types, and Figure 4-4c is a schematic diagram of this test. In recent years, the direct simple shear test apparatus has been adapted for testing sand under cyclical or dynamic loading conditions. Because the uniformity of stress conditions developed in the test are especially important for the correct interpretation of dynamic soil behavior, especially liquefaction, the simple shear test has been subjected to considerable investigation but with controversial results and interpretation (Saada and Townsend (1981); and La Rochelle (1981)).

Ladd (1981) presented a general assessment of laboratory shear devices for static loading conditions; Table 4-1 has been adapted from Ladd's paper. By "specialized design practice," Ladd (1981) gives examples of staged construction on soft foundations, underground construction, and offshore structures. It would seem that estimating the settlements because of the construction of a surface barrier at the AX Tank Farm site would also constitute "specialized design practice" because of the nature of the soils at that site, the rather complex stress paths they have or will experience, and the size of the structures.

For any of the tests listed in Table 4-1 to yield meaningful results, the tests must be conducted on undisturbed specimens of the highest quality. Unfortunately, it is virtually impossible to obtain high quality undisturbed samples of granular materials such as exist at the Hanford Site.

TABLE 4-1. Assessment of Laboratory Shear Devices for Static Loading (after Ladd, 1981).

Apparatus	Reliable σ-ϵ ?	Reliable ϕ ?	Continuously vary σ_1?	Use for basic research?	Use in specialized design practice?
Direct shear box	No	Yes	No	No	No
Conventional triaxial	Possibly	Yes	No	Yes	Possibly
Special Triaxial*	Yes	Yes	No	Yes	Yes
Plane strain	Yes	Yes	No	Yes	Yes
Cuboidal or true triaxial	Yes	Yes	Yes	Yes	Possibly
Directional shear cell	Yes	Yes	No	Yes	No
Hollow cylinder	Yes	Yes	Yes	Yes	Possibly

* K_0 ; low friction end platens; extension tests, etc.

Figure 4-3. Schematic Diagrams for the: (a) Plane Strain Test, (b) the True Triaxial or Cuboidal Test, and (c) Mohr Circles for a 3-D State of Stress (Holtz and Kovacs, 1981).

Figure 4-4. Schematic Diagrams for the: (a) Method Used to Apply Normal and Shear Stresses in the DSC, (b) Hollow Cylinder Test, and (c) Direct Simple Shear Apparatus (Holtz and Kovacs, 1981).

4.2.3 Problems peculiar to sampling and laboratory testing of granular soils

Obtaining high quality undisturbed samples of granular materials, especially from some depth below the ground surface, is not a trivial exercise. This is especially true if the materials have been deposited or are otherwise found in a loose state because it is impossible to advance a borehole and insert a sampling tube into such materials without inducing some vibrations and, therefore, causing some densification and alteration of the soil fabric or structure. Even apparently small changes in the fabric and density cause inaccurate and misleading results in laboratory tests performed on specimens taken from granular deposits. Furthermore, as noted by Clayton, et al. (1982), making the borehole results in stress relief and concurrent large reductions in effective stresses on the soil. As the properties of granular soils are highly dependent on their stress history and current effective stress level, the process of sampling has a significant effect on the engineering properties of granular soils.

Near surface deposits of granular materials above the groundwater table (GWT) may be sampled by hand from shallow test pits and trenches. Blocks or cylinders of the soil are carefully excavated using hand tools, waterproofed, and boxed for shipment back to the laboratory for testing. Sometimes large diameter shafts are drilled and cased so that a person can descend into the shaft in a safety cage and excavate specimens from the shaft walls. In any event, for successful block sampling to be made, the soils must have sufficient cohesion—provided by traces of silt or clay or by apparent cohesion because of some capillary moisture present in the deposit—to enable the excavation, trimming, preservation, and transport of the samples to be carried out. USBR (1974), Lowe and Zaccheo (1991), and Terzaghi, Peck, and Mesri (1996) provide some details on this type of sampling.

Conventional geotechnical subsurface exploration and sampling procedures may be attempted when sampling granular materials at some depth below the ground surface (or when access to the site from shallow test pits or shafts is not feasible). Boreholes are advanced with augers and stabilized with either steel casing or drilling mud. Samples are taken with thin wall tubes or other devices pressed or turned into the hopefully undisturbed soils at the bottom of the borehole. Again, the materials have to possess some cohesion for the samples to remain in the tube when it is extracted from the borehole. Fixed piston samplers such as the Osterberg sampler and Denison core barrel samplers have been successfully utilized for obtaining reasonably undisturbed samples of granular soils from above the GWT.

To obtain samples of sand from below the GWT, special techniques are required and Clayton, et al. (1982) described several in some detail. Although very expensive, freezing the ground surrounding the sampling site has been successful in a few notable cases. Large diameter core barrels are then used to extract samples of frozen soils for transport back to the laboratory and subsequent trimming and testing. There have been a few experiments with soluble “grouts” that impart temporary cohesion to the sand so that it can be sampled with conventional equipment and procedures. Bitumen emulsions or organic gels are injected as

liquids into the deposit, then solidify at ground temperatures. The samples are then taken to the laboratory, test specimens are trimmed and set up as usual in the test apparatus, and the “grout” is melted or dissolved from the pores of the specimen. If done carefully and all goes well, the specimen is at its in situ conditions (same void ratio, density, and fabric) and can be conventionally tested. The process works well in the laboratory and in clean homogeneous sand, but the presence of silt seams or siltier portions of real deposits tends to restrict the flow of the “grout” into the pores. Unfortunately, most natural deposits of granular materials are not homogeneous or free of silt size particles.

According to Becker (1999), a technique called vibracoring has been used with some success for obtaining continuous samples of silts and sand from below the GWT. The drilling equipment induces high-frequency vibrations to facilitate penetration of the sampling tube into the deposit.

The following lists the sources and causes of disturbance during the various phases of tube sampling and a few suggestions for mitigating disturbances:

1. Make the Hole

- Cause unavoidable stress release.
- Driving or angering casing causes vibrations.
- Use drilling mud to stabilize the borehole.

2. Inserting the Tube

- Driving or hammering the tube cause vibrations.
- Push with a steady, rapid motion.
- Avoid vibrations, especially with loose sand.

3. Displacement of Soil by Sampler

- Use thin wall tubes with sharp cutting edges.

4. Friction on Inside of Tube

- Almost unavoidable, although lubrication, drilling mud, and an inside clearance ratio or “choke” may help.

5. Extraction of Sample Tube from Borehole

- Without some cohesion, it is almost impossible.
- Spring core catchers or retainers may be used, but they do disturb samples.
- Fixed piston samplers have a chance, with proper procedures.
- OYO membrane sampler + vacuum (used in Japan).

6. Transportation and Storage

- Avoid vibration during shipping.
- Store and ship tubes vertically (in direction of sampling).

- Keep as near to ground temperature as possible.
- Keep tubes tightly sealed to maintain in situ water content.
- Test ASAP after sampling.

7. Extrusion and Test Specimen Preparation

- Extrude in same direction as sampled.
- Extrude vertically.
- Use steady, constant motion, and avoid vibrations
- Avoid changes in horizontal stresses if possible.

Another approach that has been used for many years is to simply recognize that undisturbed sampling of granular deposits is virtually impossible. Thus, disturbed samples are obtained and taken to the laboratory where test specimens of the materials are reconstituted at their in situ density and stress conditions. As Kramer (1996) has noted, while the void ratio and in situ stress conditions can be recreated in reconstituted specimens, other factors that markedly influence soil behavior such as soil fabric, age, stress and strain history and cementation are destroyed by sample disturbance. When test specimens were prepared by several different techniques, e.g., loose dumping then densification by vibration, pluviation through air, pluviation through water, and dry or moist tamping, it was found that their behavior especially during cyclic loading depended greatly on the method of specimen preparation and densification. Specimens prepared to exactly the same void ratios but by different procedures exhibited different stress strain and strength responses. Even materials consisting of uniform rounded sand particles had different behavior depending on how the specimens were prepared. A further complication is that it is unclear which method of specimen preparation reproduces accurately in situ conditions. Consequently, the reconstitution of granular materials into laboratory specimens will not necessarily provide the designer with an accurate measure of the in situ behavior of the deposit.

All these difficulties with sampling of granular materials and use of reconstituted samples for laboratory testing have led geotechnical engineers to increasingly rely on subsurface soundings, commonly called in situ testing, to obtain soil properties.

4.3 IN SITU (FIELD) TESTS

4.3.1 Introduction

In this section, the purposes, types, and pertinent references for the common in situ tests are given. The advantages and disadvantages of each test are discussed, and the soil parameters and properties obtained from the test are described. Only those tests considered to be suitable for the predominantly granular soils found at the AX Tank Farm site are mentioned, and discussion is limited to those characteristics and properties pertinent to determining the response of the tank foundation soils to the placement of a surface barrier.

The tests described include the standard penetration test (SPT), various types of cone penetration test (CPT, piezocone or CPTu, seismic CPT, and dynamic CPT), the pressuremeter test (conventional PMT and the self-boring PMT), plate load test (PLT) including the screw plate compressometer (SPC), and the dilatometer (DMT). Table 4-2 from Becker (1999) lists the advantages and disadvantages of in situ testing.

Important soil characteristics and properties that may be obtained from in situ testing include soil profiling and stratigraphy, soil identification, relative density and the state parameter ψ , stress history and overconsolidation ratio (OCR), in situ horizontal stress and K_0 , deformation parameters or moduli, and possibly some dynamic properties. Table 4-3 is an evaluation of the reliability of various in situ tests for determining several specific granular soil properties. It was adapted from similar tables in Becker (1999) and Lunne, et al. (1997).

4.3.2 Standard Penetration Test

The SPT is one of the oldest and most common in situ soils tests in use today. It was developed in the early decades of this century to take soil samples for the design of deep foundations, and since then it has been used for site investigations in virtually all types of soils for all types of foundation problems. In the test, a standard split-barrel or “split-spoon” sampler tube, 50 mm outside diameter and 35 mm inside diameter (Figure 4-5) is driven by a 63.5 kg hammer falling 0.76 m. The number of blows required to drive the sampler in three 150 mm increments are counted, as the first 150 mm is affected by borehole disturbance, only the blow counts for the second and third increments (300 mm) are summed; this number is called the standard penetration resistance or “blow count” and is commonly given the symbol N or N_{spt} . The soil samples obtained in the split barrel are disturbed, but are nonetheless useful for water content determination and soil classification.

Table 4-2. Summary of Advantages and Disadvantages of In-Situ Testing, Becker (1999).

<u>Advantages</u>	<u>Disadvantages</u>
<ul style="list-style-type: none"> Tests are carried out in place in the natural environment without sampling disturbance which can cause detrimental effects and modifications to stresses, strains, drainage, fabric and particle management. 	<ul style="list-style-type: none"> Samples are not obtained; the soil tested cannot be positively identified. The exception to this is the SPT in which a sample, albeit disturbed, is obtained.
<ul style="list-style-type: none"> Continuous profiles of stratigraphy and engineering properties/characteristics can be obtained. 	<ul style="list-style-type: none"> Fundamental behavior of soils during testing is not well understood.
<ul style="list-style-type: none"> Detection of planes of weakness and defects are more likely and practical. 	<ul style="list-style-type: none"> Drainage conditions during testing are not known.
<ul style="list-style-type: none"> Methods are usually fast, repeatable, produces large amounts of information and are cost effective. 	<ul style="list-style-type: none"> Consistent, rational interpretation is often difficult and uncertain.
<ul style="list-style-type: none"> Tests can be carried out in soils that are either impossible or difficult to sample without the use of expensive specialized methods. 	<ul style="list-style-type: none"> The stress path imposed during testing may bear no resemblance to the stress path induced by the full scale engineering structure.
<ul style="list-style-type: none"> A larger volume of soil may be tested than is normally practicable for laboratory testing. This may be more representative of the soil mass. 	<ul style="list-style-type: none"> Most push-in devices are not suitable for a wide range of ground conditions.
	<ul style="list-style-type: none"> Some disturbance is imparted to the ground by the insertion or installation of the instrument.
	<ul style="list-style-type: none"> There is usually no direct measurement of engineering properties. Empirical correlations usually have to be applied to interpret and obtain engineering properties and designs.

Table 4-3. Evaluation of In Situ Devices for the Determination of Granular Soil Parameters

Device	Profile	Soil type	D_R, I_D	ψ	OCR	σ_h, K_o	E, M	G_o	$\sigma-\epsilon$
SPT	B	A	C	--	C	--	C?	C	--
CPT	A	B	A-B	A-B	B	B-C	B-C	B	--
CPTU	A	A-B	A-B	A-B	B	B-C	B	B	C?
SCPT	A	A-B	A-B	A-B	B	B	B	A	B?
DCPT	B	C	C	--	C	--	C	C	--
DMT	A-B	B	C	A-B	B	A-B	B	B	C
PMT	B	B	C	--	C	C	B	B	C
SBPMT	B	B	B	--	B	A-B	A-B	A-B	A-B
PLT	--	--	B	C	B	C	B	A	--
SPC	C	C	B	C	B	C	B	A	--
Geophysical methods	B-C	C	C	--	--	--	--	A	--

Notes: A = very reliable;
 B = moderate reliable;
 C = low, poor reliable;
 -- = N/A

The SPT is probably the most common in situ test in use today for subsurface investigations, largely because it is relatively quick and inexpensive to perform, and because many useful correlations with soil properties have been developed during the preceding decades.

ASTM D 1586 gives the standard U. S. procedure for performing the SPT and split-barrel sampling of soils. Clayton, et al. (1982) discuss the international standards aspects of the SPT and how improper attention to its many variables can lead to incorrect determination of the blow count.

The SPT is most appropriate for use in granular soils, although the presence of large particles such as cobbles and even gravel can adversely affect the measured blow count. Because the test is dynamic, N_{spt} values obtained in loose sand deposits may be too high because of densification caused by vibrations of the sampling spoon. When loose sand are below the GWT, their testing and sampling is very problematic. Very careful drilling and testing procedures are utilized, and the borehole must be adequately stabilized by drilling fluid to reduce the effects of stress release and piping because of rapid inflow of ground water into the bottom of the borehole. The test is not appropriate for soft or medium clays, other than for obtaining a soil sample for water content and classification testing. Correlations of blow counts with soil properties of cohesive soils, even for stiff and hard clays, are very unreliable.

Advantages and disadvantages of the SPT are as follows:

- Standardized
- Robust equipment
- Relatively inexpensive to perform; quick
- Provides a sample (in contrast to all other in situ tests)
- Widely used for many years; large database of correlations for many engineering properties.
- Used for direct design of foundations and liquefaction assessment
- Results influenced by how borehole is stabilized or cased, borehole disturbance, stress relief, and piping at bottom of borehole.
- Results influenced by operator, equipment (hammer type and shape, drill rods, anvil, whether liners are used inside the split barrel), and test procedures.
- Results influenced by grain size, soil structure, and stress history of the deposit
- Many correlations often required for interpretation and design.

The blow count or N_{spt} value has been correlated with the relative density, friction angle, and compression modulus. It also has been correlated with other in situ tests such as the CPT and DMT. The test values are useful for the direct design of shallow foundations (footings, rafts) on sand, deep foundations of all types, and assessment of a site for liquefaction potential (Kramer 1996).

It is this latter application in geotechnical earthquake engineering that has sparked increased interest in the SPT during the past 25 years. Even though the test is “standardized,” operator differences and permissible variations in procedures and equipment led to large differences in energy input to the sampling spoon and, therefore, the measured blow counts. Attempts to modify the test to make it less variable and more useful for liquefaction assessment have not been successful, largely because of its apparently successful and widespread use for static problems. One approach was to measure the energy input to the spoon, and make appropriate adjustments to the measured N_{spt} values, and ASTM standard D 4633 for stress wave energy measurements for SPT hammers was developed in the late 1970s. However, the procedure

required rather expensive equipment and generated great controversy. The standard was never updated and, subsequently, has been withdrawn by ASTM.

According to Kovacs (1992), it appears that a standard energy input of about 60% of theoretical maximum is easily achievable with modern safety and automatic hammers. The old “donut” type and pin-guided hammers should be avoided, AW drill rods should be used, the sampling spoon should have an ID of 38 mm, and testing depths should be limited to about 20 m. A new standard ASTM D 6066 for the SPT for specifically liquefaction testing has been developed and it is basically good practice for conventional SPT testing.

Figure 4-5. Standard Penetration Test (SPT) "Split-spoon" Sampler.

4.3.3 Cone Penetration Test

The cone penetration test (CPT) is likely to be one of the more effective in situ devices for site and property characterization at the AX Tank Farm Site. The CPT, along with its several variations and modifications, is useful for determining the subsurface soil profile and stratigraphy, identifying soil types at a site, and through empirical correlations, providing good estimates of several soil parameters useful for making settlement analyses.

The CPT apparatus is a steel cone with a 60-degree apex angle and a diameter of 35.7 mm (projected area 10 cm²). The cone is pushed at a constant rate of 20 mm/s into the subsurface; thus the test is quasi-static in contrast to dynamic penetration tests such as the SPT. By the use of built-in load cells, the tip or point resistance q_c and the friction on friction sleeve f_s are measured during pushing. Sometimes the friction ratio R_f , defined as the ratio of the sleeve friction to the tip resistance, is calculated and reported. Typical test results for a layered soil is shown in Figure 4-6.

CPT has several advantages over traditional borings, sampling, and testing (Becker, 1999). It is a robust and easy-to-use standardized test, and continuous or nearly continuous profiles and data are obtained. Tests can be carried out quickly, and a large number of tests can be done in one day. Results are accurate and repeatable. Different parameters can be measured which enhances interpretation, largely because of the many correlations now available for engineering properties and design. Finally, the CPT avoids some of the disturbance of the ground associated with boreholes or the SPT.

CPT does have some disadvantages. It is not suitable for deposits with a significant amount of gravel and other large particles. It may be difficult to penetrate dense or hard materials such as dense glacial tills. As no sample is obtained, identification of soil types is only inferential or requires nearby conventional borings. Different methods of interpretation often give different results, and the CPT is not able to distinguish between normally consolidated and preloaded sand deposits.

The detailed history of penetration testing is given by Sanglerat (1972) and Broms and Flodin (1988); summaries are provided by Meigh (1987) and Lunne, Robertson, and Powell (1997). Table 4-4 from Jamiolkowski (1995) shows the important developments in the CPT and its many derivatives.

The original dutch cone penetrometer was purely mechanical; load was applied by hydraulic jacks that were calibrated for measuring the tip resistance, and later the sleeve friction. Today, virtually all CPTs are done with electrical cones, in which the tip resistance and sleeve friction are measured with strain-gage-type load cells built into the body of the probe. In addition, electrical cone probes may also include piezometers in the tip for concurrent measurement of induced pore water pressure, a particularly valuable characteristic of cohesive soils and

interbedded deposits. This test is denoted as the CPTU. Another important specialized cone is the seismic CPT or SCPT, in which a geophone in the probe measures the velocity of shear waves induced at the surface of the deposit. In recent years, other specialized sensors have been placed in cone probes to measure inclination, lateral stress, soil resistivity, heat flow, groundwater and environmental characteristics, and even a computer vision system. A description of many of these specialized cones is given later in this section.

Both Meigh (1987) and Lunne, et al. (1997), provide detailed descriptions of the cones, pushing equipment, calibration procedures, corrections, test procedures, and presentation of test results from CPT tests. Standardized procedures for the CPT test are in ASTM D 3441 and in ASTM D 5778 for the CPTU and piezocone. The ISSMFE international reference test procedure for the CPT is found in Appendix A of Lunne, et al. (1997), and their Appendix B contains a reprint of the Swedish Geotechnical Society "Recommended Standard for the CPT."

Figure 4-6. Typical CPT Results Correlated With the Soil Profile (obtained by other means)
(Holtz and Kovacs, 1981).

Table 4-4. Historical Development of the CPT and Derivative Devices (Jamiolkowski, 1995; see that reference for the papers listed in the table).

'30	MECHANICAL CPT Barentsen (1934)			
'40	CPT Simple Cone De Beer (1942) MANTLE CPT Vermelden (1948)			
'50	FRICTION SLEEVE CPT Begemann (1953)			
'60				
'70	RESISTIVITY CPT ⁽²⁾ Delft SML (1977)	PIEZOPROBE Wissa (1975) Torstensson (1975)	ELECTRICAL CPT ⁽¹⁾ De Ruiter (1971) FLAT DILATOMETER Marchetti (1975, 1980)	
'80	PRESSIO-CONE Amar et al (1982) Whithers et al (1986) Campanella and Robertson (1986)	SEISMIC CPT Campanella et al (1986) LS-CPT Huntsman (1985), Tseng (1989), Campanella et al (1990)	CPTU Baligh (1980) De Ruiter (1981) ENVIROCON O'Neill et al (1995)	ACOUSTIC CPT Muromachi (1981) Tringali and Mitchell (1982) NUCLEAR CPT VIBRATORY CPT , PWRI (1985)
'90				SCAP'S CPT , Robataille (1994)

- Notes: 1. First attempt because of Perez et al. (1976);
 2. For further development see Kokan (1990), Campanella and Weemes (1990).

With regard to the AX Tank Farm site, there are two primary areas of concern: (1) site characterization and profiling (including spatial and areal variability, as well as variability with depth); and (2) determination of physical and mechanical properties at the site. For settlement analyses, correlations with the deformation properties (Young's modulus E or constrained modulus M —and perhaps the small-strain shear modulus G_0) are the most important. Because the stress and strain history of a granular deposit markedly influences the moduli and, therefore, the settlement estimates, it is also important to obtain if possible some idea of the stress history of the site and its OCR. Has the site been preloaded or prestrained, either geologically or by construction operations? Can we obtain this information from the CPT results? Are any special sensors required? Is I_d or D_r relevant in this connection? What about K_0 or the state parameter ψ ?

Because the available correlations were developed on different soils than exist at the Hanford Site, more than one approach, if possible, should be used for the evaluation of the

compressibility parameters. It is also advisable to check specific design calculations by an alternative approach (e.g., large-scale field loading tests, other in situ tests, or independent laboratory tests).

4.3.3.1 Soil Profile and Classification

The CPT is probably the most useful in situ device for delineating the soil profile. It is able to sense even modest changes in density and stiffness over rather small distances. Depending on the changes in stiffness and the nature of the materials being penetrated, layers as small as 10 to 20 cone diameters (i.e., 350 to 700 mm) can be reliably detected. (In cohesive soil profiles, greater accuracy can be achieved by the use of the piezocone or CPTU.) The cone is useful also for determining the lateral or areal variability over a site, information that is especially valuable for investigations of large foundations such as the AX Tank Farm.

Both Meigh (1987) and Lunne, et al. (1997) summarize the relevant soil identification and classification information. These procedures are usually given in the form of charts; Figure 4-7 shows two examples. It can be seen that sandy soils tend to produce high tip resistances and low sleeve friction ratios. Also it has been found that soils with high horizontal stresses, i.e., with high OCR, tend to have higher cone resistances and friction ratios. As noted by Lunne, et al. (1997), one of the important conclusions of the Douglas and Olson (1981) classification chart is that "CPT classification cannot be expected to provide accurate predictions of soil type based on grain size distribution but provides a guide to soil behavior type." Meigh (1987) notes that it is advisable to check CPT soil identification by some boreholes or other means. For the Hanford AX Tank Farm site, the general soil classification is well known from previous conventional borings.

4.3.3.2 State Characteristics and Stress History

The relative density D_r or density index I_D (see Holtz and Kovacs [1981] for definitions) is often used as an intermediate soil parameter. If the soils at a site are relatively dense, then they will in general be stiff and experience relatively small settlements when loaded. Because the relative density is so important in practice, considerable research has been done with the CPT and sand in large calibration chambers at different densities and stress histories. This research is well summarized by Mayne and Kulhawy (1990) and Lunne, et al. (1997). One example of the correlations obtained between relative density and cone resistance is shown in Figure 4-8 for five sands. Kulhawy and Mayne (1990), in a study of 24 sets of calibration chamber tests on unaged, uncemented sand, found that there was a reasonably linear relationship between the $(D_r)^2$ and a dimensionless normalized cone resistance factor. This is shown in Figure 4-9; or in Equation 4-1 where Q_c is a compressibility factor, OCR is the overconsolidation ratio, and p_a is a reference stress, usually taken as 1 atm. or 100 kPa; σ'_{vo} is of course the vertical effective overburden stress.

$$D_r^2 = \frac{1}{305 Q_c OCR^{0.18}} \frac{\left(\frac{q_c}{P_a} \right)}{\left(\frac{\sigma'_{vo}}{P_a} \right)^{0.5}} \quad (4-1)$$

Kulhany and Mayne (1990) suggest values of Q_c as follows: 0.91 for low compressibility predominantly quartz sand with round grains and little or no fines; 1.0 for medium compressibility, quartz sand with some feldspar and/or several percent fines; and 1.09 for high compressibility sand with mica or other compressible minerals present or with high fines content. Lunne, et al. (1997) suggest ways aging in sand, a phenomenon that seems to be present in almost all natural sand deposits, might be taken into account.

Interpretation of the CPT in terms of a state parameter ψ that combines the influence of relative density and stress level with reference to an ultimate or critical state in the sand deposit is becoming more common. In some respects, the state parameter ψ is analogous to the OCR in clays. See Lunne, et al. (1997) for details about ψ and q_c .

Whether a deposit has been previously loaded or pretrained has a significant influence on its compressibility (see, e.g., Holtz, 1991) and, therefore, its settlement under load. Unfortunately, there is no reliable way to estimate stress history from the CPT. Geologic evidence, if it exists, is probably the best approach. As Meigh (1987) notes, if K_o is known, Figure 4-8 can be used for OC sand by substituting the σ'_{ho} in the equation for D_r . Alternatively, other in situ tests such as the dilatometer (DMT) appear to give more reliable results for OC deposits.

Lunne, et al. (1997) discuss in some detail the determination of the in situ horizontal stress σ'_{ho} from (1) geologic evidence, (2) relative density estimates, and (3) direct measurements of lateral stress. Probably the simplest, though not necessarily the most reliable is the following empirical equation (4-2) determined from calibration chamber results by Kulhawy and Mayne (1990). As before, p_a is a reference stress, usually taken as 1 atm. or 100 kPa.

$$\frac{\sigma'_{ho}}{P_a} = \frac{\left(\frac{q_c}{P_a} \right)^{1.25}}{35 \exp\left(\frac{D_r}{20} \right)} \quad (4-2)$$

As Jamiolkowski (1995) had noted, there is an urgent need to develop nondestructive methods for the evaluation of K_o or σ'_{ho} in situ.

4.3.3.3 Deformation Characteristics and Modulus

Because of all the difficulties in sampling cohesionless deposits and the problems associated with reconstituted laboratory specimens, obtaining a reliable estimate in situ of the sand stiffness is of great practical value. In the case of the AX Tank Farm site, this is the key soil parameter influencing the estimates of settlements because of the construction of the surface barrier. However, as noted by Lunne, et al. (1997), using in situ test penetration tests to determine modulus is also problematic because of the following factors: (1) modulus depends on the effective stresses and on stress history; (2) the in situ test conditions, stress level, drainage, and direction of loading cannot be controlled; (3) rarely are modulus values documented as to the drainage conditions and stress and strain levels over which the modulus is applicable. Even so, correlations with CPT for drained modulus values—the elastic (Young's) modulus E , the one-dimensional or constrained modulus M , and the small strain shear modulus G_o —have been determined for a number of useful conditions.

Figure 4-7. Identification of Soils Using the CPT: (a) Full Scheme by Douglas and Olson (1981), and (b) Working Version by Robertson and Campanella (1983) (from Meigh (1987)).

(a)

(b)

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Figure 4-8. Correlation Between D_r and Dimensionless q_c or q_{c1} ; Uncorrected for Boundary Effects (data from Jamiolkowski, et al., 1985; prepared by Kulhawy and Mayne, 1990).

Figure 4-9. Summary of Calibration Chamber Studies (Kulhawy and Mayne, 1990).

Figure 4-10 shows results from large calibration chamber tests which may be used to estimate the secant Young's modulus E_s' for an average strain of 0.1% for a range of stress histories and aging. This strain level is probably quite reasonable for well designed foundations. The stiffness of aged (>1000 yr old) lies in between that of OC sand and recently deposited NC sand.

Figure 4-11 provides an estimate of the constrained modulus M_o , which is a reference value of M based on the effective vertical stress σ'_{vo} before the start of the CPT test. Using calibration chamber data on quartz sand and other information, the following Equation (4-3) is proposed where n is a stress exponent (equal to 0.2 for NC sand and 0.128 for OC sand); k_M is a dimensionless modulus number which can be determined using Figure 4-11 (based on the normalized cone resistance (q_c/p_a) and estimated OCR); and p_a is the atmospheric pressure in the same units as the other stress values (M_o , σ'_{vo} , and q_c).

$$M_o = K_M P_a \left(\frac{\sigma'_{vo}}{P_a} \right)^n \quad (4-3)$$

The method predicts M_o values in calibration chamber tests within $\pm 25\%$. As noted by Jamiolkowski (1995), correlations between q_c and E or M require an assessment *a priori* of the strain level at which the operational stiffness is needed and the very difficult decision as to whether one is dealing with a NC or an OC deposit. Thus all correlations between q_c and E or M should be considered purely empirical, site dependent, and not very reliable. This topic is a major research need.

Robertson (1991) proposed an alternate approach for estimating the settlement of shallow foundations on sand based on the degree of loading, which is the ratio of the applied foundation stress q to the ultimate bearing capacity q_{ult} of the deposit. The ratio E/q_c for OC sand is taken directly from Figure 4-12 for both loose and dense sand. The variation in estimated E as a function of the degree of loading is because of the amount of mobilized shear strain beneath the footing. As shown in Figure 4-12, E should be reduced appropriately for recent and aged deposits of NC sand.

Small strain shear modulus G_o has been found to be nearly constant for strains less than $10^{-3}\%$ or 0.00001, and thus this parameter can be reliably estimated using the seismic CPT (SCPT), discussed below. A correlation between q_c and G_o has been found for unaged and uncemented sand and is shown in Figure 4-13. The rather wide range of the G_o/q_c ratio at low values of q_c is most likely because of differences in sand compressibility. See Jamiolkowski (1995) for additional discussion on G_o . Fahey (1998) provides another set of correlations between G_o/q_c .

and q_{c1} , the normalized cone resistance (also defined in Figure 4-10); these correlations are shown in Figure 4-14 and show some effect of aging and cementation.

Figure 4-10. Evaluation of Drained Young's Modulus From CPT in Silica Sand (data from Bellotti, et al. 1989; figure from Lunne, et al., 1997).

Figure 4-11. Constrained Modulus Number of Sand as a Function of Cone Resistance and OCR
(after Eslaamizaad and Robertson, 1996; figure from Lunne, et al., 1997).

Figure 4-12. Estimation of Equivalent Young's Modulus for Sand Based on Degree of Loading (Robertson, 1991; figure from Lunne, et al., 1997).

Figure 4-13. G_o/q_c vs. Normalized q_c (after Rix and Stokoe, 1992; figure from Lunne, et al., 1997).

Figure 4-14. G_o/q_c vs. q_{c1} For Various Sand (data from Robertson; quoted by Fahey, 1998).

4.3.3.4 Additional Information and References

Two recent conferences are valuable sources of new information on the CPT and related devices. They are the International Symposium on Cone Penetration Testing (CPT '95), held in Linköping, Sweden, in 1995 and the First International Conference on Site Characterization (ISC '98) held in Atlanta in 1998. Both conference proceedings contain a wealth of new information, some of which has been mentioned in the above discussion. Particularly noteworthy in CPT '95 are Baldi and O'Neill (1995) and Denver (1995). The ISC '98 theme lectures by Robertson, Lunne, and Powell (1998), Fahey (1998), Mitchell and Brandon (1998), and Houlby (1998) are valuable, as is the paper by Fioravante, et al. (1998) on carbonitic sand. Other papers in both conferences will be briefly mentioned in the following discussion.

4.3.3.5 Seismic Cone Penetration Test

In the seismic cone penetration test (SCPT), a geophone in the probe measures the velocity of shear waves induced at the surface of the deposit. This development, described in detail by Baldi and O'Neill (1995) and Lunne, et al. (1997), is attractive because it permits what are essentially down hole or cross hole geophysical measurements to be made more efficiently and economically. Specifically, the small strain shear modulus G_0 can be obtained directly. Jamiolkowski (1995) discusses the SCPT and notes that if the seismic CPT results, namely measurement of the shear velocity V_s , are to be valid, some additional measure of the directional dependency or anisotropy of the soil fabric are required. In this regard, developments in electrical resistivity cones may assist in this endeavor. This, however, is a current research need. Baldi and O'Neill (1995) also discuss developments in resistivity cones.

4.3.3.6 Dynamic Cone Penetrometers

Dynamic cone penetrometers (DCPT) of various types and sizes have been commonly used for decades (Sanglerat 1972), but because of the popularity of the SPT, no DCPT has ever been widely used in the U.S. One of the major difficulties with the DCPT is that the drill rods are uncased and increased friction on the drill rods makes the test increasing unreliable at depths greater than 20 or 30 m. Recently, a drilling company in Vancouver, B.C., has added the innovation of spinning the rods as they are hammered into the subsurface, and this process effectively eliminates the rod friction. This test is called the spin dynamic penetrometer test (SDPT). Similar to the SPT, the number of blows required to drive a cone a distance of 305 mm is recorded; hammer mass, drop height, and drill rods are identical to the conventional SPT. The SDPT cone is disposable, 73 mm in diameter and 200 mm long; its tip has a 60° apex angle with a 19 mm blunt end. The sleeve is 154 mm long.

Experience in the Fraser River Deltaic deposits (interbedded alluvial sand, silts, and gravels) indicates good correlations with conventional SPT and conventional CPT results. Its primary

advantage is time and cost; a continuous profile with depth is obtained more quickly than either the SPT or CPT.

The SDPT properly calibrated may be a suitable alternative to the CPT and SPT for profiling and site characterizing at the AX Tank Farm site.

4.3.3.7 Environmental Cones

In recent years, a number of different types of probes and sensors have been attached to or included in conventional CPTU probes for various geo-environmental purposes. A detailed discussion of these developments is beyond the scope of this review, but Baldi and O'Neill (1995), Lunne, et al. (1997), and the ISC'98 Theme Lectures by Robertson, et al. (1998), and Mitchell and Brandon (1998) give complete descriptions of the most important developments. Also, the proceedings of the ISC'98 contain several papers on geo-environmental applications of direct push technology and sampling and testing for several specific environmental problems.

4.3.3.8 Vision Cone Penetrometer

One of the most exciting recent developments is the vision cone penetrometer (VisCPT). This device yields computer-enhanced images of the soils being penetrated, and details can be found in Hryciw and Raschke (1996) and Hryciw, et al. (1998). Provided the process can be shown to be economical and feasible, the VisCPT has the potential to overcome one of the primary disadvantages of the conventional CPT—that is, no samples are obtained for positive soil identification.

4.3.4 Pressuremeter Test and Self-Boring Pressuremeter Test

In the pressuremeter test (PMT), a cylindrical probe with an inflatable membrane is inserted into a borehole. As the pressure is increased, incrementally and lateral pressure is applied to the sides of the borehole, changes in volume of the probe are recorded. In conventional pressuremeter testing, boreholes are drilled and stabilized prior to insertion of the pressuremeter. Self-boring pressuremeters, on the other hand, can drill themselves down to the desired testing depth. Two fairly recent books, Mair and Wood (1987) and Briaud (1992), provide excellent descriptions of the equipment, test procedures, estimation of soil properties, and procedures for using both tests directly in geotechnical design.

4.3.4.1 Conventional Pressuremeter

A schematic diagram of the conventional or Ménard pressuremeter is shown in Figure 4-15. Typical probes are 45 to 85 mm in diameter and 600 to 750 mm long. The maximum applied pressure is typically 2.5 to 4 MPa, although devices capable of applying up to 20 MPa for testing

rock are available. Ménard, the inventor of the pressuremeter, used two guard cells to induce a uniform distribution of lateral stress at the sides of the borehole. However, the single cell OYO lateral load tester manufactured in Japan, appears to perform just as well.

In both the Ménard and OYO pressuremeters, after a stable borehole is achieved, the probe is lowered to the desired testing depth. Then the pressure is increased incrementally and the resulting volumetric expansion of the membrane is recorded, as shown in Figure 4-16a; the volumetric expansion vs. pressure curve is derived from the basic test data, as shown in Figure 4-16b. The pressuremeter modulus E_{pmt} is the slope of the quasi-linear portion of the volumetric expansion-pressure curve; the limit pressure p_l is related to the shear strength of the soil. ASTM test method D 4719 is appropriate for the conventional PMT.

Ménard also developed methods for using the E_{pmt} and p_l to predict bearing capacity and settlement of both shallow and pile foundations (Baguelin, et al., 1978; Mair and Wood, 1987; and Briaud, 1992). Correlations with several soil properties also have been developed.

Advantages and disadvantages of the conventional PMT are as follows:

- Useful in sand, provided the borehole remains open and is stable
- Large volume of soil is influenced by the loading
- Lateral loading imposed; interpretation or independent measure of anisotropy is required to obtain vertical properties.
- No samples are obtained
- Results very dependent on the quality and stability of the borehole
- Stones in sides of borehole can damage membrane
- Probe requires calibration for membrane stiffness, expansion of lines and volumeter, and hydrostatic effect of lines filled with water.
- Discontinuous results; modulus (lateral) is , but p_l over predicts shear strength, especially in soft clays; correlations with friction angle are OK.
- Test results dependent on interpretation method—different methods yield different results.

4.3.4.2 Self-Boring Pressuremeter Test

The influence of borehole disturbance and stability on PMT results led to the independent development of the self-boring pressuremeter in the early 1970s in both France and the U. K. Figure 4-17 is a schematic of the Camkometer, the English version of the SBPMT. By the use of water jets and a rotating cutter bit, plus gentle vertical pressure, the probe bores itself into the ground with a minimum amount of stress relief and disturbance of the walls of the borehole. Once the probe is inserted to the desired testing depth, the SBPMT is conducted in a very similar manner to the PMT. Results are also similar, especially at large strains, but the initial and pseudo-elastic portions of the volumetric expansion-pressure curve may be significantly different

with the SBPMT, aided largely by accurate electro-mechanical feeler devices for measuring the membrane expansion.

The SBPMT is also much more suitable than the conventional PMT for use in deposits that are difficult to sample and test conventionally, provided that any gravel particles present are small enough to pass through the device during insertion. However, the quality of the modulus data obtained is significantly better, especially if an unload-reload cycle is performed (Figure 4-16 b). Examples of this are the SBPMTs conducted by Washington State Department Of Transportation for a bridge crossing the Nooksack River (Hughes, et al., 1998) and the SBPMTs conducted on Po River sand to obtain modulus and settlement data for power plant foundations in Italy (Jamiolkowski, et al., 1985). See also Fahey (1998). Roy, et al. (1998) report on the use of SBPMT data to estimate the dilation angle and cyclic strength of sand. As noted above, Mair and Wood (1987) and Briaud (1992) provide good descriptions of the use of the SBPMT results for estimation of soil properties and for geotechnical design.

The self-boring pressuremeter is much more complex in terms of equipment and operation than the conventional pressuremeter, and it is also much slower. Some self-boring probes are equipped with piezometers for measuring in situ pore pressure and total stress cells for direct measurement of the in situ horizontal earth pressure. These latter developments appear to be more useful in clay deposits than in sand.

Advantages and disadvantages of the SBPMT are as follows:

- Useful in sand, provided that any gravel particles present can pass through the probe
- Testing can be carried out at both large and small strains
- Avoids disturbance because of stress relief
- Correlations available for many engineering properties
- Use of loading and unloading cycles can help mitigate installation disturbance effects and enhance interpretation, especially at small strains.
- Apparently good measures of G_o and E_{pmt} in sand deposits, especially if unload-reload cycles are performed.
- Expensive, complex equipment; requires experienced operator to run the test
- Slow, relatively expensive test.

Figure 4-15. Diagrammatic Sketch of Menard Pressuremeter (Clayton, et al.,1982)

Figure 4-16a. Incremental Pressure With Resulting Volumetric Expansion of Membrane, (Holtz and Kovacs, 1981).

Figure 4-16b. Pressure Versus Volumetric Expansion, (Mair and Wood, 1987).

4.3.5 Plate Load Test and Screw Plate Compressometer

4.3.5.1 Plate Load Test

One of the oldest in situ tests, the plate load test (PLT), was developed to provide estimates of the bearing capacity and settlement of shallow foundations. Although the test is rarely performed today, at least for foundation design, its procedure is still listed as an ASTM standard test, D 1194. The test setup is shown in Figure 4-18. Plates are round or square, typically 300 to 750 mm in diameter or on a side. Some tests have been conducted with plates as large as 2 m in diameter; of course, with large plates, greater reaction and jacking loads are required and this of course increases the cost significantly. Load is usually applied incrementally and the resulting settlement of the plate observed. Loading is continued until either the bearing capacity of the subgrade soil is reached or the settlement reaches about 0.1 of the plate diameter or width.

The PLT is most suited for tests on granular soils although the depth of influence is limited to approximately one to 1.5 plate widths or diameters. As most shallow foundations are significantly larger in dimension than the common plates, a scaling factor must be applied, and this is always a problem. Clayton, et al. (1982), after a thorough discussion of scaling factors concluded that extrapolation of settlements determined on small plates to large loaded areas on granular soils is rather unreliable. At best, the PLT gives only a compressibility modulus for the soils immediately beneath the plates and only at the location of the test.

Advantages and disadvantages of PLT are as follows:

- Applicable to sand and gravels
- Direct determination of deformation modulus only in the near vicinity of the test
- Application to full scale foundations requires scaling factors
- Near surface; must be accessible in a test pit or trench
- Simple equipment, but test is time-consuming and relatively expensive to perform.

4.3.5.2 Screw Plate Compressometer

The screw plate compressometer (SPC) overcomes the shallow depth limitation of the conventional PLT. In this test, a plate is screwed down to the desired testing depth, and by a hydraulic jack, the plate is loaded incrementally and the resulting settlement observed. If loading continues until the plate plunges, the bearing capacity of the soil can be determined. The SPC has been used in both granular and cohesive soil deposits. While it may be difficult or even impossible to obtain the ultimate bearing capacity in dense sand and very stiff or hard clays, the SPC is still very useful for determining the compressibility parameters of these deposits. The test is not appropriate for gravel deposits.

The SPC was developed in Norway in the 1950s (Janbu and Senneset [1973]; Strout [1998]). Figure 4-19 shows the basic Norwegian device and setup for use in the field. The SPC has been used in the U.S. for sand, Sweden and S.E. Asia for both sand and soft clays, and in Australia and the UK for stiff clays. Some theoretical work on the test has also been done in Canada and the United States (including the University of Washington). Recent research by Strout (1998) is a very comprehensive look at the influence of plate geometry and installation effects, interpretation, and equipment and procedural improvements of the SPC test in sand.

Figure 4-17. Self-boring Pressuremeter and Camkometer (Clayton, et al., 1982).

Figure 4-18. Typical Setup for Conducting Static Load Tests (ASTM).

Advantages and disadvantages of the SPC are as follows:

- Loading is in same direction as most foundation loadings
- Equipment is relatively inexpensive and easy to use, although the test is not rapid
- Interpretation relatively straightforward, at least for soil modulus
- Applicable to loose and dense sand deposits
- Disturbance because of installation of screw plate
- Stress history of sand is not readily determined by the SPC.

4.3.6 Dilatometer Test

The dilatometer test (DMT) is a relatively recent development in in situ testing and site investigations. The device and testing procedures were developed in Italy by S. Marchetti in the 1970s; it thus is often called the Marchetti dilatometer or sometimes the flat dilatometer to distinguish it from the PMT, a cylindrical dilation device.

Figure 4-20 shows the dilatometer blade and pressurizing equipment. The sharpened blade, 95 mm in width and about 14 mm thick, contains an inflatable thin metal diaphragm 60 mm in diameter which is built into the flat part of the probe, as shown in the figure. The blade is attached to drillrods and pushed or driven into the subsurface at a rate comparable to the CPT test (about 20 mm/s). Every 200 to 300 mm, penetration is stopped and the membrane is gradually inflated. (taking about 15 s). Usually two pressure readings are taken; the first is when the membrane just lifts off its sensing device, and the second is when the membrane deflection is exactly 1 mm. These readings, corrected for membrane stiffness, are used to determine three indices that Marchetti correlated with the classification of the soil and parameters such as in situ horizontal stress, stress history, deformation characteristics, and through correlations, even the undrained shear strength of soft clays and the friction angle of sand.

Good references on the DMT are Marchetti (1980), the proceedings of the First International Conference on the Flat Dilatometer (MARL 1983), Schmertmann (1986), Lutenegeger (1988), and Marchetti (1998).

Advantages and disadvantages of the DMT are as follows:

- Simple, robust tool; standardized test equipment and procedures
- Quick and simple to use
- Repeatable results
- Applicable to a wide variety of soil and site conditions
- Correlations available for many properties
- Provides reasonable estimates of in situ horizontal stress, OCR, and small strain compressibility modulus (in horizontal direction).
- Improved estimates of OCR and strength are obtained of granular deposits if thrust during penetration is also measured.

- Not suitable for deposits with significant amount of gravel particles; coarse, angular particles may rupture or tear membrane.
- No sample is obtained
- Difficult in dense and stiff deposits to keep the blade inserted vertically
- Test not as simple nor as rapid as the CPT; results are not as continuous as the CPT
- Properties correlations may not be appropriate for all sites; some local calibration advisable.

Figure 4-19. Screw Plate Compressometer (SPC): (a) Principle; (b) Field Setup (after Janbu and Senneset, 1973).

Figure 4-20. Dilatometer Test Equipment, Schmertmann (1986).

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HNF-3438, Rev. 0

APPENDIX A

**REVIEW OF AX TANK FARM
SURFACE BARRIER SETTLEMENT CALCULATIONS**

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May 25, 1998

Dr. Carleton J. Moore /MSIN H5-57
Fluor Daniel Northwest, Inc.
PO Box 1050
Richland, WA 99352-1050

Dear Carleton:

RE: Report on Task 1:
Soil properties for 3-D FEM modeling and settlement estimates
HTI--AX Tank Farm Surface Barrier
Contract No. 100501-9-K00015

As part of Task 1, the draft report entitled "Settlement Estimates for Construction of a Surface Barrier" including documentation on the soil moduli used for the analyses described in the report was reviewed. The report summarized the results of a series of three-dimensional finite element method (3-D FEM) analyses using the well known structural analyses program ABAQUS. Included were a description of the soil modeling used for settlement calculations and the estimated settlements due to the construction of the proposed surface barrier. In addition, a review was made of some earlier laboratory test results that were made available to me.

A 3-D FEM analysis with variable soil properties, as described in the draft report, is a very sophisticated and modern approach to a very difficult problem. The choice of mesh size and arrangement seems very appropriate for the size of the structures and the soil conditions. As the report correctly points out, the stress-strain behavior of granular soils is highly nonlinear, especially when initially loaded. Furthermore, this behavior is strongly dependent on the confining pressure operative at the time of loading. Probably the most common way (and in my opinion the most appropriate way) to model granular soil behavior is to use the Berkeley constitutive model, and this approach was utilized in the 3-D FEM analyses for settlements due to construction of a surface barrier. In terms of its consideration of the variation in soil properties at the site, this work was carried out with the appropriate degree of care and attention to detail.

In order to verify that the 3-D FEM work was giving reasonable estimates of settlements to be

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expected due to the construction of the surface barrier, some very simple "back-of-the-envelope" type calculations were conducted using reasonable average values of the soil properties and moduli used in the FEM work. The influence of the construction sequence was considered as closely as it could be, and the effect of unloading and reloading was also taken into account. Two simplified approaches were used: Theory of Elasticity and the Schmertmann procedure. In the first case, various procedures developed from the Theory of Elasticity for calculating the settlement of loaded areas on the surface of a half-space were used. These procedures are variations of the Boussinesq stress distributions for different boundary conditions (flexible surface load, rigid loaded areas, infinite and finite layer depths, etc.)

Although the Schmertmann procedure for estimating the settlement of shallow foundations on sands has a theoretical basis, it utilizes soil moduli derived from empirical relationships together with an estimate of the in situ soil modulus from the results of cone penetration tests (CPT). In my analyses, not having CPT results, I used the same average moduli I used for the Theory of Elasticity estimates. Both the settlement due to the barrier over a single tank and that due to a barrier covering the entire four-tank area were determined using the Schmertmann procedure..

From the Theory of Elasticity calculations, I obtained estimates of settlements that ranged from approximately 0.2 to 0.4 in. at the centerline of the loaded area. Edge settlements would be about 70% of the centerline settlements for flexible loading, but for a rigid loading assumption, edge settlements would of course be equivalent to the centerline settlement. The Schmertmann estimates yielded about 0.2 in. at the centerline for the single tank case and 0.6 in. for the four-tank case. These estimates compare very favorably with the 3-D FEM results given in the subject draft report.

Appended to this letter is a copy of my hand calculations for the cases described above.

Subsequent to my review and simple settlement estimates, I received a copy of a report entitled "Slope Retaining Investigation for Additional Tanks at Purex 241 Tank Farm, U. S. Atomic Energy Commission, Richland, Washington", prepared by Shannon & Wilson, Inc. in 1968. This report included the results of two boreholes in which the Standard Penetration Test (SPT) had been conducted, and the results were very encouraging--assuming of course that these test results are representative of the present AX site under consideration. Below the loose near surface deposits (10 ft in one borehole and 25 ft in the other borehole), the soils are dense to very dense, with blow counts between 30 and 60 and occasionally greater required to drive the standard sampling spoon 12 in. When the in situ density is estimated by the SPT blow counts/foot of the standard sampling spoon, the following ratings are commonly used:

loose--0 to 15
medium dense--15 to 30
dense--30 to 50
very dense-->50

Dr. Carleton J. Moore

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Dense to very dense soils are relatively incompressible and thus will experience only small surface settlements when loaded for the first time. At the site under consideration, the settlements would be even smaller due to the unloading the soils experienced during original construction of the tanks. This is because granular soils, once preloaded and unloaded, will exhibit only minimal settlements upon subsequent reloading.

In summary, I feel very confident that the rather small settlements estimated by the 3-D FEM approach are reasonable and conservative for the given soil, site, and loading conditions.

Sincerely yours,

Robert D. Holtz, Ph.D., P.E.

Enclosure

(hand calculations)

wt of tank

31 Jan - 1 Feb 9

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2' avg th

270

46

40'

Orig ground surf. 662 - 668

Excav to 621.4 - 46' - 4830 pcf

Build tank $(32' + 12' + \text{th of shell} = 1.3'') = 45.75' \text{ hi} \rightarrow$
 top of dome @ 667, $\text{dwn} \approx \text{close}$
 - wt of tank = 1184 pcf orig gr. &
 - soil 667.11 = 1575 pcf
 $682 - 667 = 15' \times 105 = 1575$
 Net unbrd
 $6 - 2071 \text{ pcf}$

• waste $\rightarrow$ tanks - 40 gr - then pumped out
 • tanks bluffed w/ gravel, grout or concrete

+ place "Hofstad barrier" 17' $\rightarrow$ 1813 pcf over everything -

682

620

Diams $\approx 30' = B$

$$2B = \frac{320}{460}$$

~320

1) th. of elast.
 (General models)
 $S =$

2) JHS

a. Single tank
 b. large area $\sim$ CJM

Prop $E = 5000 \text{ psi}$ $\rightarrow 30000 \frac{\text{lb}}{\text{in}^2} \times 1.44 \frac{\text{in}^2}{\text{ft}^2}$
 E_i E_{ur} HNF-3438, Rev. 0
 $\nu = 0.25 - 0.33$

Th of elast

Geom $r = a = 40'$

① Semi-infinite space flex g

P+D (1974)
ch 3

$$s = \frac{q(r)}{E} \times DF$$

$$= \frac{1800(40)(1.5)}{30000 \text{ psi}(144)}$$

1.5 @ surf

$$= 0.025 \text{ ft} = 0.3 \text{ in.}$$

$$\xi = r/a = 0$$

Arkhin & Ulagy (1962)

$$P_z = s_z = \frac{P(1+\nu)(a)}{E} \left[\frac{3}{2} A + (1-\nu) \frac{E}{a} \right]$$

$$= \frac{1800(1.33)(40)(12)}{30000(144)} [0.67(2.0)]$$

$$= 0.36$$

$$\nu = ?$$

② flex on finite layer

ch 5

$$h = 30'$$

$$\frac{h}{a} = \frac{30}{40} = 0.75$$

$$\frac{r}{a} = 0 \quad \xi$$

$$\nu = 0.3$$

$$Harr (1966) 0.91 \quad 0.58/0.1$$

$$P_z = \frac{2pa(1-\nu^2)}{E} I_z$$

$$P_z = s = \frac{2pa}{E} I_z$$

$$= \frac{2(1800)(40)(12)(1.7)}{30000(144) \text{ psf}} = 0.68 \text{ ft}$$

③ rigid areas

p166 ∞
p177

$$P_z = \frac{\pi}{2} (1-\nu^2) \frac{P_{av}(a)}{E} = \frac{\pi}{2} (0.67) \frac{1800(40)(12)}{30000(144)} = 0.21 \text{ in}$$

on finite layer

$$\frac{h}{a} = 0.75 \quad \frac{r}{a} = 0 \quad \frac{r}{a} = 0$$

$$P_z = \frac{P_{av} \cdot a}{E} I_z$$

$$= \frac{1800(40)(12)}{30000(144)} (1.33) = 0.27 \text{ in}$$

Range: $0.21 \text{ in} \rightarrow 0.36 \text{ in}$ ξ

$$\Delta q = \frac{1800}{8} - \frac{6510}{8}$$

$$I_{zp} = 0.5 + 0.1 \sqrt{\frac{\Delta q}{\sigma'_{vp}}} \leftarrow \text{negative} \therefore \approx 0$$

$$C \cdot B/2 \text{ below } fnd = 40' \approx \text{elev. } 580'$$

$$s = \sum_{i=1}^n A_i \sum_{j=1}^n \left(\frac{I_z}{E} \right) A_j$$

$$C_1 = 1 - 0.5 \left(\frac{6510}{1800} \right) \geq 0.5 = 1$$

-1.8

$$\text{Layer } I_z \quad A_i \quad x$$

$$1 \quad 0.32 \quad 40' \quad 12.8$$

$$2 \quad 0.25 \quad 120' \quad 30$$

$$\Sigma = 42.8$$

$$S = \frac{1800^{psf} (42.8') (12")}{30000 (144) psf} = 0.21"$$

Carleton's

$$\phi = 270' = B$$

$$2B = 540$$

$$0.5B = 135$$

700

680

 I_2

600

$$I = 0.30$$

$$\begin{array}{rcl} 1 & \Delta Z & I_2 \quad \times \\ 1 & 135 & 0.30 = 40.5 \end{array}$$

$$2 \quad 245 \quad 0.35 = 85.75$$

$$\Sigma = 126.25$$

0.5B = 545

800

$$S = \frac{1500(126.25)(12")}{30000(144)} = 0.63"$$

400

$$I_2 = 0.35$$

300

VIRIK

DISTRIBUTION SHEET

To DISTRIBUTION	From D. L. Becker	Page 1 of 1
		Date 3-17-99
Project Title/Work Order AX Tank Farm Closure Settlement Estimates and Soil Testing, HNF-3438, Rev. 0		EDT No. 624598
		ECN No.

Name	MSIN	Text With All Attach.	Text Only	Attach./ Appendix Only	EDT/ECN Only
CENTRAL FILES	B1-07	X			
PROJECT FILES - HTI	R1-41	X			
DOE READING ROOM	H2-53	X			
D.L. BECKER	H0-22	X			
T.J. CONRADS	R3-73				X
E.A. FREDENBURG	R1-04	X			
J.C. HENDERSON	B2-05				X
D.L. NICHOLS	B2-05				X
W.R. ROOT	R2-53				X