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REPORT OF GEOTECHNICAL INVESTIGATION
PROPOSED HANFORD WASTE VITRIFICATION PLANT
HANFORD, WASHINGTON

For

Kaiser Engineers
Job NO. 10805-383-016

November 15, 1989

DAMES & MOORE

DAMES & MOORE A PROFESSIONAL LIMITED PARTNERSHIP
500 MARKET PLACE TOWER, 2025 FIRST AVENUE, SEATTLE, WASHINGTON 98121
(206) 728-0744

November 15, 1989

Kaiser Engineers Hanford Company
1200 Jadwin Avenue
P.O. Box 888
Richland, Washington 99352

Attention: Mr. Mick Sasala

Revised Final Report
Geotechnical Investigation
Proposed Hanford Waste Vitrification Plant
Hanford, Washington

We are submitting herewith 10 copies of our revised final report of the geotechnical investigation for the proposed Hanford Waste Vitrification Plant near Richland, Washington. Work on this project was performed under the Kaiser Engineers Hanford Company Work Order No. KOA-NZ3-46562 dated September 21, 1989.

Our draft version of this final report was submitted on September 1, 1989. The report has been revised in order to respond to review comments submitted in the September 28, 1989 Fluor Daniel letter. Our direct item-by-item response to the comments are being forwarded in a separate submittal.

We thank you for the opportunity to be of service on this project. Please contact us if further questions should arise regarding the contents of the report.

Very Truly Yours

DAMES & MOORE

Harbans L. Chabra, P.E.
Principal Engineer/Associate

W. Martin McCabe, P.E.
Project Engineer

W. MARTIN MCCABE
STATE OF WASHINGTON
23495
REGISTERED
PROFESSIONAL ENGINEER

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REPORT OF GEOTECHNICAL INVESTIGATION
PROPOSED HANFORD WASTE VITRIFICATION PLANT
HANFORD, WASHINGTON

INTRODUCTION

This report presents the results of the geotechnical investigation for the proposed Hanford Waste Vitrification Plant (HWVP) to be located along the western border of the 200 East area of the Hanford Reservation near Richland, Washington. The location of the proposed plant with respect to the Hanford Reservation is shown on the Area Map, Plate 1. The purpose of the investigation was to evaluate the geologic conditions at the site and provide recommendations regarding the design and construction of foundations for the various facilities that will be included in the HWVP. Recommendations on site preparation and general earthwork activities are also included in this report.

DESCRIPTION OF PROJECT

The proposed project will consist of a variety of steel and/or reinforced concrete buildings, surface and underground tanks, an exhaust stack, equipment and machinery pads, underground utilities, parking areas, a water retention pond and a septic tank effluent leach field. These facilities will occupying a 40 acre site near the intersection of 7th Street and Akron Avenue in the 200 East area. The buildings range in height from 1 to 5 stories and extend as much as 32 feet below grade. The WHT Tank is the deepest structure, extending to 46 feet below grade. The layout of the plant is depicted in the Site Plan, Plate 2. We understand that locations of facilities shown on Plate 2 may be change during the continuing design development process.

Key structural information concerning each facility has been summarized in Table 1. The information, which includes column, wall, floor slab and mat loads as well as proposed finished grades, was used to evaluate the design needs and the expected performance of each of the facilities.

We understand that the location of the retention pond has not yet been finalized.

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TABLE 1. DESIGN DATA FOR PROPOSED FACILITIES

Facility No.	Name	Total Column Loads (Kips)	Total Mat Pressure	Total Floor Load (psf)	Finished Floor Elevation (Feet)	Bottom of Footing/ Mat Elevation (Feet)
1	Vitrification Building		6000		708	695
2	Operations Control Bldg.	75 (Int.) 21 (Ext.)		250	708	704
3	Regulated Entrance Facility	35 (Int.) 21 (Ext.)		250	708	704
4	Service Bldg.	41 (Int.) 25 (Ext.)		400	708	704
5	Canister Storage Bldg.					
	(a) Building	60		350	708	704
	(b) Vault		2500			676
6	Manipulator Repairing Building	180 (Int.) 65 (Ext.)		350	708	704
7	Sand Filter Building		2100			686
8	Fan House	300 (Int.) 15-22/ft. on walls		425	712/714	711
9	Switchgear & Generator Building	15-22/ft. on walls		550	708	704
10	WHT Vault and Building					
	(a) Vault		2500 to 3500			665
	(b) Bldg.	30 (Int.) 8-10/ft. on walls		350	708	704
11	Exhaust Stack		4000			705.5
12	Miscellaneous Structures					
	(a) Cooling Tower			550	702	701
	(b) Solar Evap. Tank			400	708	707
13	Switchyard					
	(a) Building	15/ft. on walls		NA	NA	NA
	(b) Transformer	2150-200		NA	NA	NA

Notes: (Int.) indicates "internal"
(Ext.) indicates "external"
NA indicates "not available"

SCOPE OF SERVICES

The scope of services for this project was presented in the July 14, 1989 Dames & Moore proposal, which was prepared in response to the Kaiser Engineers Hanford "General Requirements, Division 1, For Specification B-595-C-02010", including the Fluor Daniel Inc. "Geotechnical Investigation" section, Revision 0, dated April 1989. The services may be summarized as follows:

TASK 1: FIELD EXPLORATION PROGRAM

Conduct a field exploration program, including drilling 17 borings to depths ranging from 20 to 100 feet at locations designated by Fluor Daniel Inc. in their Drawing H-2-117070, Geotechnical Investigation Site Location Plan, approved June 29, 1989. The program will also include a series of downhole seismic tests in one of the borings and a series of percolation tests in the future Septic Tank and Leach Field area.

TASK 2: PRELIMINARY REPORT

Prepare a preliminary report, in advance of the completion of the borings, that presents the draft logs of any borings completed at that time and a preliminary evaluation of the subsurface conditions and their implication on the foundations of the HWVP.

TASK 3: LABORATORY TEST PROGRAM

Conduct a laboratory testing program to measure physical, engineering, corrosion potential and thermal parameters of the soils at the site. Physical parameters will include the density, moisture content, and grain size distribution of soils. Engineering parameters will include the strength, deformability and permeability of the soils. Corrosion potential parameters include the sulfate, sulfide and chloride content of the soils, as well as their pH, redox potential and electrical resistivity. The thermal parameter of interest to this program is the thermal resistivity.

TASK 4: FOUNDATION SUPPORT EVALUATION

Evaluate the viable methods of foundation support and provide recommendations on allowable bearing capacity of shallow and/or deep foundations, settlements, earth pressures, shear strength and frictional characteristics for resisting lateral loads, parameters for use in designing vibrating foundations and other matters pertinent to foundation performance as identified in the Geotechnical Investigation Specification.

TASK 5: EARTHQUAKE EVALUATION

Evaluate seismic conditions and provide recommendations on static and dynamic (as appropriate) values of density, Poisson's Ratio, modulus of elasticity, shear modulus, coefficient of subgrade reaction and permeability. Also evaluate the potential for liquefaction and recommend an appropriate value of "site coefficient" as identified in the 1988 Uniform Building Code.

TASK 6: CORROSION POTENTIAL EVALUATION

Evaluate the corrosion potential of the soils, including their possible impact on subsurface metallic and concrete structures, and recommend appropriate methods of mitigating any adverse impacts from corrosive conditions.

TASK 7: EARTHWORK, SITE PREPARATION, CONSTRUCTION

Evaluate measures required to prepare the site and construct the facilities, and accordingly provide recommendations on cut and fill slope stability, excavation support, compacted fill placement, pond construction and lining, subgrade support and preparation for pavements and railways, surface and subsurface drainage and other considerations as addressed in the Geotechnical Investigation Specifications. This task includes providing recommended values of percolation rates to be used in design of leach fields and seepage pits for sewage disposal.

TASK 8: UNDERGROUND UTILITIES

Evaluate the potential for corrosion of underground utilities and provide recommendation for their design, installation and corrosion protection.

This task includes the use of laboratory test data and the conclusions developed for Task 6.

TASK 9: FINAL REPORT

Prepare a final report that describes the field exploration and laboratory testing programs, and presents our recommendations regarding foundation support, seismic design, corrosion potential, site preparation, earthwork, foundation construction and all other pertinent considerations as identified in the Geotechnical Investigation Specification.

SITE CHARACTERIZATION

GEOLOGIC SETTING

The site of the proposed structure lies in the Central Plains Section of the Columbia Intermontane physiographic province, and is more specifically situated within Pasco Basin topographic depression created by the Columbia River. The terrain is relatively flat with features created by glacial-related floods and subsequently by alluvial and aeolain (wind blown) deposition.

The bedrock at the site is a very thick sequence of Columbia River basalts which occur at a depth of approximately 400 feet below the ground surface, or roughly Elevation 300. Overlying the basalts are Pliocene age lacustrine and fluvial deposits of the Ringold Formation. This formation consists of cemented soils ranging in gradation from coarse sands and gravels to some interlaminations of fine sand, silt, and clays. Overlying the Ringold Formation is a Pleistocene age glaciofluvial deposit called the Hanford Formation. It may consist of coarse sand and gravels (Pasco Gravel) in areas where flows were highest or a finer gradation (Touchet Beds) in slackwater areas. More recent deposits of loess, dune sand, alluvium and colluvium are found near the surface throughout the Hanford area.

The site of the HWVP is currently unoccupied except by two asphalt-paved roadways, a rail spur and several underground utilities. The ground surface throughout the site is slightly undulating with a moderately thick cover of sage brush, cheatgrass and other deciduous plants of the area. Ground surface elevations range from approximately Elevation 705 to 720, decreasing generally to the north and east.

SITE SPECIFIC SUBSURFACE CONDITIONS

Subsurface conditions at the site were investigated by drilling seventeen borings ranging in depth from 20 to 100 feet at locations shown on Plate 2. Geophysical tests were conducted in one of these borings (Boring VP-8), and percolation tests were conducted in shallow hand-augered holes near Boring VP-17. Details of the field investigation are presented in Appendix A, which also explains apparent anomalies in the values of sampler penetration resistance during drilling (i.e. N values). A further description and separate report of the geophysical tests are contained in Appendix C. Laboratory tests were conducted on selected samples to assist with the identification and classification of soils at the site, as well as to assist with estimation of their engineering characteristics. A summary of the laboratory test program may be found in Appendix B.

The soil profile in the upper 100 feet consists essentially of three strata described in the following paragraphs. Three subsurface profiles have been prepared to provide a better understanding of foundation conditions (see Plates 3, 4 and 5).

STRATUM 1 - Light Brown Silt and Very Fine to Fine Sand

This layer occurs at the surface throughout the site, and appears to have the character of a dune sand deposit. It is in a dry, loose to medium dense condition and contains some roots and organic materials in the upper three feet. The layer ranges in thickness from 2.5 feet in Boring VP-1 to 15.5 feet in Boring VP-13. The thickness is typically 5 to 9 feet except at two locations (VP-6 and VP-13) where the thickness increases to the 10 to 15 feet range.

Laboratory tests indicate that the layer contains 20 to 40 percent non-plastic fines (i.e. passing the No. 200 sieve) and that dry unit weights are in the range from 94 to 105 pounds per cubic foot (pcf). A collapse test performed on one slightly to moderately disturbed sample showed a modest 2.6 percent decrease in height during inundation. The collapse test result is believed to reflect the loose and disturbed nature of the soil, rather than an inherent tendency to collapse. Due to the dry condition of the soil, obtaining truly undisturbed samples was not possible.

The loose, dry nature of the sand also tended to result in misrepresented strength characteristics of the soil as measured in direct shear and triaxial compression tests. In order to transfer the samples to the testing devices, the addition of moisture to the sample was required. This procedure appears to have somewhat consolidated the sample and produced higher shear strengths and deformation modulus values that would be expected of the soil in its natural condition. The results of such tests conducted on samples from this stratum are presented on Plate B-1.

In its current condition, this soil is considered to be marginal from the standpoint of foundation support. It possesses moderate to low strength, high compressibility, and would be subject to continuing consolidation during vibratory loading. We estimate an internal friction angle (phi) of approximately 30 to 32 degrees and a deformation modulus of 200 kips per square foot (ksf) for this soil in its present condition. Significant improvement in the strength, deformability and collapse potential could be obtained using removal-and-recompaction methods or vibratory rolling.

Collapse tests conducted on samples recompacted to 95 percent of the maximum dry density for this soil have resulted in collapse values of less than 0.3 percent. Collapse would therefore not be a concern after recompaction.

STRATUM 2 - Light Brown Silty Fine Sand With Some Gravel and Cobbles

This layer was encountered beneath Stratum 1 in all borings except VP-9, where it was not in evidence. It was typically 3 to 7 feet thick throughout the site, but was found to be 16.5 feet thick and closer to the ground surface at Boring VP-1. The soil is relatively well graded and is in a dense condition.

The interface between this stratum and adjoining strata above and below is usually transitional rather than abrupt, and the amount of gravel and cobbles can vary considerably.

Due to the large particle sizes for this soil, samples suitable for strength testing could not be obtained. However, we consider the soil to possess high strength and low compressibility. We estimate an internal friction angle (phi) of at least 40 degrees and a deformation modulus of at least 1000 ksf for this layer.

STRATUM 3 - Brown or brownish Gray Fine to Medium Sand, Trace of Coarse Sand, Trace of Silt and Occasional Gravel

This layer underlies Strata 1 and 2 throughout the site, extending to a considerable depth. The layer usually exhibits a light cementation, may contain discontinuous silt or sandy silt lenses and is typically in a dense to very dense condition. This deposit has been encountered by Dames & Moore during investigations for the Process Condensate Treatment Facility near B-Plant (Dames & Moore, 1989) and for the Grout Vaults project along the eastern border of the 2000 East Area (Dames & Moore, 1988).

This soil is characterized by high strength and low compressibility. Based on laboratory tests conducted in this and previous investigations, we estimate an internal friction angle of approximately 38 degrees. We feel that a deformation modulus value of 1500 ksf is appropriate for the upper 20 feet of this deposit, while a value of 2000 ksf can be used for the lower portion of the deposit. These estimates have been made on the basis of laboratory and field data, as well as the measured performance of structures at the Hanford reservation.

We have examined the information from borings drilled by others (e.g. Tallman et al, 1979) in order to estimate the type and character of deeper soils at this location. The information indicates that the natural Stratum 3 fine to medium sand continues to a depth of approximately 250 feet, then becomes more gravelly until the top of basalt rock is encountered at approximately 410 foot depth (i.e. Elevation +300).

Ground water

Ground water was not encountered during drilling in this investigation. Investigations by others have indicated that ground water in the vicinity of the proposed HWVP is at a depth of approximately 310 feet, i.e. approximate Elevation 400.

CONCLUSIONS AND RECOMMENDATIONS

FOUNDATION TYPES AND CAPACITIES

We recommend that foundations for the HWVP facilities be supported on mats or on wall or individual spread footings in Stratum 2 or Stratum 3. Stratum 1 may also be used to support mat or spread footing foundations provided measures are taken to densify this layer prior to foundation construction.

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The allowable bearing capacity of the foundation soils depends on the size of the footing/mat and its depth of embedment as well as on the strength characteristics of the soil. Because of the wide variety of footing/mat sizes and embedment depths anticipated throughout the HWVP, a general analysis of bearing capacity as a function of footing/mat width and embedment depth was performed using the method of Terzaghi & Peck (1967). This analysis was based upon the assumption that Stratum 1 will be densified in some manner, and that the resulting angle of internal friction (phi) will be approximately 35 degrees at the time of foundation construction. All three strata were assumed to be cohesionless, in keeping with the results of laboratory tests.

The results of this analysis showed that allowable bearing capacities after applying a factor of safety of 3.0 were high, exceeding 4 ksf for a 3 foot wide footing and 20 ksf for a 10 foot wide footing/mat, even for a shallow 3 foot embedment. In practice, the allowable bearing capacity of footings/mats in these conditions is usually controlled by the magnitude of settlement that occurs following the application of loads. Therefore a general analysis of settlements under simplified geometries and soil conditions was conducted, and a chart of allowable bearing capacities was prepared based on limiting the total settlement of the footing or mat. The chart of allowable bearing capacities is presented on Plate 6. All values of allowable bearing capacities in the chart will result in total settlements ranging from less than about 1 inch for footings to less than about 2.5 inches for large mats. The larger allowable settlements for mats recognizes that differential settlements across a large, flexible mat will still be within tolerable limits for most steel or concrete structures even if total settlements are as much as 2.5 inches or more.

It should be recognized that these allowable bearing capacities do not take into consideration footing settlements that may result due to superimposed loads from overlying floor slabs and from adjacent footings and mats. This topic is discussed in more detail in the following section.

SETTLEMENTS

Settlements of footings, mats and slabs for each individual facility within the HWVP were estimated using the method of Schmertmann (1970), which allows for layering of foundation materials. The analysis was based upon the assumption that some soil improvement technique will be applied to the Stratum 1 Silt and Fine Sand, and that the resulting improved deformation modulus (E) will be approximately 500 ksf. Deformation modulus values for the other two strata are as reported in the Site Characterization section above. Settlements of the footings and floor slab are estimated separately. Total settlement of the footings will be the sum of the footing settlement plus the floor slab settlement values.

A summary of estimated settlements for the HWVP facilities is presented in Table 2. The results indicate that settlements of facilities founded at greater depths will likely be negligible because the applied stresses are nearly equal to the weight of soil removed during the excavation. Moderate settlement magnitudes will be experienced for footings founded within Stratum 1, depending on the magnitude of the floor and column loads. The greatest settlement has been estimated for the very heavily loaded Vitrification Building, primarily due to the large areal extent of the load. The settlements for this structure have been estimated assuming that the mat is moderately flexible. If the mat is designed to be very stiff, e.g. heavily reinforced and greater

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than approximately 5 to 6 feet in thickness, settlement across the mat will be nearly uniform and equal to the total settlement value listed in Table 2.

The presence of the heavily loaded Vitrification Building will induce settlements in the three buildings that directly adjoin it. These facilities include the Regulated Entrance Facility, the Service Building, and the Manipulator Repair Building. Settlements induced by the Vitrification Building will decrease with distance from the exterior wall approximately as indicated on Plate 7. The settlement magnitudes shown on Plate 7 should be added to those in Table 2 to obtain the total estimated settlement for each of the affected structures. We recommend that the potential for damaging settlements be reduced by adopting a construction schedule that will provide for construction of a major portion of the Vitrification Building before superstructures for the adjoining buildings are started. Pouring of floor slabs for the adjoining buildings should be delayed as much as possible.

Settlement estimates presented herein are approximate in nature, and actual settlements may differ from those estimated by as much as 15 to 20 percent. Some sources of uncertainty in the estimates include the inability to directly measure in situ deformation modulus values for the soils encountered, variability in the character of the soils throughout the site and innaccuracies in the analytical methods for estimating settlements. The settlements will be elastic in nature, and will occur essentially simultaneous with the application of loads. We do not anticipate any long term settlements.

We have attempted to utilize to the maximum possible extent case history information from the Hanford site to provide guidance on the selection of deformation modulus values. For example, we have reviewed the settlement history of Washington Public Power Supply System power plants WNP-1 and WNP-4. We have also considered empirical deformation modulus estimates based on penetration resistance values during drilling and on geophysical measurements made in Boring VP-8 and elsewhere on the site.

TABLE 2 - SUMMARY OF ESTIMATED SETTLEMENTS

Facility No.	Name	Footing/mat width (feet)	Footing/mat Approximate Net Applied Pressure (psf)	Footing/mat Settlement (inches)	Slab (1) Settlement (inches)	Total Footing/mat Settlement (inches)	Differential (2) Settlement (inches)
1	Vitrification Bldg.	120.0	4500	2.4		2.4	1.2
2	Operations Control Bldg.	3.5	5500	0.5	0.2	0.7	0.3
3	Regulated Entrance Bldg.(3)	2.5	4900	0.3	0.1	0.4	0.2
4	Service Bldg.(3)	3.0	4100	0.3	0.4	0.7	0.3
5	Canister Storage						
	-Bldg.	3.5	4500	0.2	0.1	0.3	0.2
	-Vault	74.0	0	Negligible			Negligible
6	Manipulator Repair Bldg. (3)	6.0	4800	0.4	0.2	0.6	0.2
7	Sand Filter Bldg.	125.0	0	Negligible			Negligible
8	Fan House	8.5	4200	0.8	0.3	1.1	0.5
		4.5(wall)	4700	0.4	0.3	0.6	0.2
9	Switchgear	2.5	3300	0.2	0.2	0.4	0.2
	Generator Bldg.	4.5(wall)	4700	0.4	0.2	0.6	0.2
10	WHT						
	-Vault	42.0	0	Negligible			Negligible
	-Bldg.	4.0	2000	0.1	0.2	0.3	0.1
		2.5(wall)	3700	0.2	0.2	0.4	0.2
11	Exhaust Stack	36.0	3500	0.9		0.9	0.4
12	Miscellaneous						
	-Cooling Tower	40.0	550	0.1		0.1	Negligible
	-Solar Evap. Tank	50.00	400	0.1		0.1	Negligible

(1) At center of slab
(2) Between adjacent columns for case of footings;
between center and corner of mats, assuming moderately flexible mat
(3) Settlement of portions of these facilities will be influenced by the presence of the vitrification bldg. See text for detailed discussion

SUBGRADE MODULUS

A commonly employed method of representing a soil subgrade for purposes of soil structure interaction analysis is the "Winkler" foundation, in which the subgrade is modeled as a series of vertical or horizontal springs with a spring constant termed the "modulus of subgrade reaction" K_s. For sand subgrades loaded vertically, K_s should be determined for the actual footing or mat size using the expressions by Terzaghi (1955):

SQUARE FOOTINGS:

$$K_s = K_1 \left(\frac{B+1}{2B}\right)^2$$

RECTANGULAR FOOTINGS:

Same as K_s for square footing, except K_1 should be modified as follows:

$$K_1 = K_1 \frac{(m + 0.5)}{1.5\ m}$$

Where K_s = the value of vertical modulus of subgrade reaction for a footing or mat of width B.

K_1 = the value of modulus of subgrade reaction for a 1-foot wide square plate.

m = length of rectangular footing divided by width (L/B).

We recommend the following values of K_1 for vertical loading:

	Static K_1 (pci)	Dynamic K_1 (pci)
Stratum 1	150	1500
Stratum 2	275	2750
Stratum 3	300	3000

These values were selected based upon the results of laboratory triaxial and grain size distribution tests, penetration resistance values during drilling, and experience obtained by others as reported in the literature. The values apply to a rigid, square plate of one foot width with loading applied at the ground surface.

The case of vertical walls in granular soils was also examined by Terzaghi (1955), who proposed the following equation for the horizontal coefficient of subgrade reaction:

$$K_h = \frac{1_h Z}{D}$$

K_h = horizontal coefficient of subgrade reaction
Z = depth below ground surface
D = depth to bottom of wall from ground surface
1_h = constant, depending on moisture and density of soil

As discussed by Scott (1981), the values of 1_h suggested by Terzaghi are likely to be substantially conservative. We accordingly recommend the following 1_h values:

	Static 1_h (pci)	Dynamic 1_h (pci)
Stratum 1	25	250
Stratum 2	35	350
Stratum 3	40	400

LATERAL EARTH PRESSURES

Information about lateral earth pressures will be required to design temporary excavation shoring, permanent subsurface walls, and permanent retaining walls, and to estimate the resistance of structures to lateral movement induced by seismic or wind loads.

Several of the facilities will require excavations to depths in the range from 25 to 46 feet (e.g. the WHT Tank and the Sand Filter Building). It will likely be most expedient to support these excavations with temporary shoring. The shoring may consist of conventional soldier beams and timber lagging with steel rod or tendon tie-backs. Alternatively, a system of soil nails may be employed for excavation support. The earth pressures for the tie-back system may be estimated using information on Plate 7. Other considerations for temporary shoring walls are discussed in the "Shoring of Temporary Excavations" section.

Permanent retaining walls and subsurface walls that are free to rotate or translate away from the soil by an amount equal to approximately 0.1 percent of the height of the wall should be designed for active earth pressures. If the walls are rigid and unable to rotate or translate, at-rest earth pressures should be used. Our recommendation for active and at-rest earth pressure coefficients for the soils present at this site are given in Table 3. Note that no distinction has

been made between Stratum 2 and Stratum 3 as regards earth pressure coefficients because of the similarity in strength characteristics of these two soils. Passive earth pressures may be used to resist the tendency for lateral movement of structural members during seismic events or other appropriate circumstances.

TABLE 3. EARTH PRESSURE COEFFICIENTS

	AT-REST		ACTIVE		PASSIVE*	
	Static	Dynamic	Static	Dynamic	Static	Dynamic
STRATUM 1 (IMPROVED)	0.43	0.52	0.27	0.44	2.5	2.0
STRATUM 2 OR 3	0.38	0.46	0.24	0.40	2.8	2.3

*Factor of safety of 1.5 applied.

The dynamic lateral earth pressure coefficients for the active and passive cases were estimated according to Seed and Whitman (1970) as corrected by Davies et al (1986). The dynamic lateral earth pressure for the at-rest case was estimated in accordance with the findings of Sherif et al (1982). A horizontal ground acceleration of 0.25g was used in the dynamic earth pressure coefficient analysis. This acceleration value has been used in the design of the Washington Public Power Supply system WNP-2 nuclear power plant (NRC, 1982) and the Fast Flux Test Facility. It is based on a postulated Richter Magnitude 6.5 earthquake with a recurrence interval of at least 10,000 years.

Lateral forces may also be resisted by friction between the soil and the bottom of footings on other foundation elements. We recommend a friction coefficiant of 0.42 for the recompacted Stratum 1 soil and 0.47 for the soils of Stratum 2 and 3.

SEISMIC DESIGN CONSIDERATIONS

To facilitate an analysis of the performance of the proposed structures during earthquake motion, a variety of static and dynamic soil properties have been requested in the "General Requirements" document for this project. This section addresses the requested properties and additionally provides data on damping values appropriate for the soils under consideration.

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During this project, direct measurements of unit weights, compressional wave velocities and shear wave velocities were made in the elevation range from 710 to 610. All other seismic design parameters for soils from the ground surface to the top of rock were calculated from these measured parameters, or were estimated from in-situ tests performed by others and from laboratory and in-situ values published in the literature for similar soil conditions. A graphical summary of some of the key parameters as a function of depth is shown on Plate 9.

The shear modulus (G) values presented in Plate 9 are applicable only to the low amplitudes of shear strains which are created by the seismic shear waves during the downhole or crosshole test, i.e., typically in the range from 10^{-3} to 10^{-5} percent. The modulus values are therefore considered "maximum" values, and must be modified to be applicable to the case of earthquake waves that produce much higher shear strains, i.e., on the order of 10^{-1} to 10^{-3} percent. The modifications for sands can be made using the relationship plotted in Plate 10 and for gravels using the relationship plotted in Plate 11. Also shown in the plates are relationships by which the damping values for sands or gravels may be estimated for the expected level of shear strain. The "average" curves in Plate 10 should be used for soils at depths up to 250 feet, while the curves in Plate 11 should be used for deeper soils.

In order to estimate the wave velocities and seismic design parameters shown on Plate 9, we examined reports of seismic testing done by Dames & Moore and others at the Hanford site. Results of deep crosshole and downhole testing performed at the N-Reactor site (United Nuclear Industries, Inc. 1978), crosshole tests performed at the Fast Flux Test Facility (Hanford Engineering Development Laboratory 1975) and downhole tests performed at the future grout vaults site (Dames & Moore 1988) were reviewed. A review was also made of estimates of the dynamic shear modulus reported by URS/John A. Blume & Associates (1977) for the 241-AN and 241-AP tank sites in the 200 East area. The data were not entirely consistent for the lower 150 to 200 feet of soil immediately overlying bedrock. Shear wave velocities varied from approximately 2,200 feet per second to 4,300 feet per second. Some information indicated increasing then decreasing velocity values, while other information indicated a constant or uniformly increasing velocity profile with depth. We believe that the data presented in Plate 9 represent a reasonable interpretation of the collected data, and are sufficiently accurate for design purposes.

Some methods of soil-structure interaction employ elastic modulus values to estimate the performance of structures during seismic shaking. For such an analysis, we recommend the following values:

Elastic Modulus (E) in ksf

	Static	Dynamic	
Stratum 1	500	5,000	
Stratum 2	1,000	10,000	
Stratum 3	1,500	15,000	(upper 20 feet)
	2,000	20,000	(elsewhere)

Static and dynamic values for coefficient of subgrade reaction have been recommended in the "Subgrade Modulus" section.

The substantial depth to the water table at this site means that liquefaction is not a consideration.

We recommend the use of a Site Coefficient (S-Factor) of 1.2 for seismic analysis of HWVP facilities. The Site Coefficient is as defined and described in Section 2312 of the 1988 Uniform Building Code (UBC). If a simple representation of ground motion is required, we recommend that the normalized response spectra shape corresponding to a "Soil Type 2" be selected from Figure No. 3 of the UBC, page 179.

VIBRATORY FOUNDATION LOADS

Foundations for vibratory equipment such as compressors should be designed to prevent excessive deformation in the primary directions of motion of the machine, and to prevent settlement of the foundation soils due to long term vibration-induced consolidation.

Because of the naturally loose condition of portions of Stratum 1 and the attendant possibility of vibration-induced consolidation, we recommend that special preparation measure be adopted for vibrating foundations in this soil. The measures should include the removal and replacement of Stratum 1 soils within a distance of at least 4 feet below the bottom of the footing/pad for the vibrating machine. We further recommend that prior to recompaction, the exposed subgrade should be thoroughly rolled using heavy vibratory compaction equipment to densify the remaining Stratum 1 soils below the 4 foot depth level. More than 4 feet of soil should be removed and recompacted at locations where stratum 1 is thickest, e.g. at Boring VP-13 where as much as 8 feet of soil may need recompaction. The recompaction should be in accordance with recommendations presented in the "Earthwork and Site Preparation" section.

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If the above preparatory measures are adopted, we recommend the following parameters for use in design of vibrating foundations:

SOIL PARAMETERS FOR USE IN VIBRATING FOUNDATION DESIGN

	Total Unit Weight (pcf)	Shear Modulus (ksf)	Poisson's Ratio
Stratum 1 (improved)	106	1,000	0.30
Stratum 2	124	3,000	0.25
Stratum 3	120	5,000	0.25

The shear modulus values shown in the table above have been reduced from those values measured in the geophysical testing program in order to correspond to the 10^{-2} shear strain amplitude that probably better approximates the maximum shear strains that will be induced by vibrating machinery during operation.

SHORING OF TEMPORARY EXCAVATIONS

Earth pressures for tie-back shoring systems have been presented in the "Earth Pressures" section. Soldier piles in this system must have sufficient penetration below the excavation level to counteract the vertical components of anchor loads. The vertical capacity of the soldier piles will be developed by a combination of end bearing and frictional shaft resistance along the embedded length. We recommend a friction (adhesion) value of 750 psf in the looser Stratum 1 soils and 2000 psf in the denser soils of Stratum 2 and 3. An end bearing value of 15 ksf may be used for either Stratum 2 or Stratum 3.

Tieback anchors will derive their capacity through friction (adhesion) between the grout and the soil behind the "no-load" zone. The boundary of the no-load zone should be defined as a horizontal line beginning at the bottom of the excavation and extending back into the soil a distance of one-fourth the height "H" of the excavation. The zone should then extend upward to the surface at an angle of 60 degrees to the extension of the horizontal line. This zone is illustrated on Plate 8. A friction (adhesion) value of 750 psf is recommended for the Stratum 1 soils, but the entire length of the anchor should be at least 10 feet below the ground surface. A friction (adhesion) value of 2000 psf may be used for the denser Stratum 2 and 3 soils.

The actual capacity of each anchor should be verified by testing during construction. All anchors should be prestressed to 1.3 times the assigned design load at the time of installation, then reduced to the design load and locked off.

Installation in stratum 1 and possibly Stratum 2 and 3 may be hampered by caving of the dry soils into the hole during drilling. It therefore may be necessary to case the holes where this situation is encountered.

EARTHWORK AND SITE PREPARATION

Recommendations in this section will encompass excavation stability, compacted fills, footing and slab construction, temporary shoring placement, drainage and erosion, dike construction and other topics associated with development of the site.

Cuts and Fills

Prior to any fill placement, the ground surface should be cleared and grubbed, with all deleterious natural and man-made materials removed from the site. All three strata identified in the "Site Characterization" section are suitable for use as compacted fill. Stratum 2 and Stratum 3 are considered somewhat more desirable because of the lower fines content and a more well graded distribution of particle sizes. Compaction characteristic of Stratum 1 and 3 have been measured in the laboratory. The results are shown on Plates B-14 and B-15 in Appendix B. Fills for general grading may be placed in lifts 10 to 12 inches thick and compacted to 90 percent of the maximum dry density as determined by ASTM D-1557. Compaction recommendations for footings, mats, slabs, water retention dikes and utilities are discussed elsewhere.

Temporary excavations in the loose, dry Stratum 1 soil will likely experience some sloughing if slopes are steeper than about 1.8 to 1.0 (horizontal to vertical). Furthermore, this soil is considered highly erodible, and will be adversely affected if left unprotected during heavy rainfalls. Protection from erosion can be provided by covering the cut slopes with visqueen or similar plastic membranes during construction.

Temporary excavations in the dry Stratum 2 soil may experience some local sloughing if slopes are steeper than about 1.5 on 1.0 (horizontal to vertical). Temporary excavations in the underlying, lightly cemented Stratum 3 fine to medium sand have been observed to stand at 1.25 to 1.0 slopes with only minor sloughing over time.

Construction of Footings, Mats and Slabs

Excavations for footings, mats and slab subgrades may encounter sloughing in the dry Stratum 1 and 2 soils. For structures with shallow footings located 3 to 4 feet below finished grade, it may be more expedient to excavate all soils within the building area down to the footing level so that footings may be formed on the exposed subgrade. The subgrade should first be thoroughly

proofrolled using heavy vibratory equipment. If more than 4 feet of Stratum 1 remains beneath the footing or mat, additional soil should be overexcavated and recompacted prior to pouring the footing/mat.

Compacted fill beneath footings, mats and slabs should be placed in lifts not exceeding 6 inches in thickness and compacted to 95 percent of the maximum dry density as determined by ASTM D-1557. Close attention should be paid to moisture control during hot weather construction.

All footings and mats should be founded at least 2.5 feet below the lowest adjacent finished grade to provide protection against frost action. We recommend that all footings be at least 2.5 feet in width.

Water Retention Dikes

We recommend that dikes for the future water retention basin be constructed at slopes of approximately 1.75 to 1.0 (horizontal to vertical) using any of the three soil strata identified in the Geologic Characterization section. Steeper slopes can be constructed if geogrid reinforcement, such as the Tensar Corporation UX1200 to UX1600 series products, is used. However, a resulting increase in difficulty in lining the pond with natural or synthetic materials may result.

Since the natural soils at the site have permeabilities ranging typically between 10^{-1} to 10^{-3} centimeters per second, natural or synthetic materials must be used to line the pond to prevent loss of retained fluid. Natural silts, clayey silts and sandy silts with sufficiently low permeability values are available at the McGee Ranch site (see Plate 1) near the west border of the Hanford Reservation. Alternatively, a variety of synthetic materials are available to serve as durable, impermeable liners for the pond.

Dikes should be constructed by placing the fill soil in 8 to 10 inch thick lifts and compacting to 92 percent of the maximum dry density as determined by ASTM D-1557.

UNDERGROUND UTILITIES

Excavations for underground utilities will be primarily in the Stratum 1 Silt and Fine Sand that mantles the site. This loose, dry soil will readily cave into any excavation that is sloped steeper than about 1.8 on 1.0 (horizontal to vertical). Some sloughing difficulty may also be experienced during the placement of temporary shoring under these conditions. The addition of moisture before and during the excavation process may assist in reducing the amount of caving.

The excavated soil will be suitable for both bedding and backfill. Backfill soil should be placed in lifts not exceeding 6 inches in thickness and compacted to 90 percent maximum dry density as determined by ASTM D-1557.

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Two bulk samples of the Stratum 1 soil were tested for thermal resistivity to assist in determining ampacity derating factors for power cables. Test results indicated thermal resistivity values in the range from 400 to 530 degrees centigrade-centimeter per watt (°C-cm/watt).

Laboratory electrical resistivity tests were conducted on samples from Stratum 1, 2 and 3 to assist with calculation of resistance-to-soil for equipment grounding and for assessing the potential for corrosion. The results, presented in Plate B-2, indicate that all resistance values are high, typically exceeding 100,000 ohm-centimeters. The potential for corrosion is more fully discussed in the section entitled "Corrosion and Chemical Deterioration".

PAVING

All areas to receive asphalt pavement should be cleared of vegetation and any other deleterious materials. The initial 1 foot of Stratum 1 Silt and Fine Sand should be removed and recompacted in lifts not exceeding approximately 6 inches to at least 92 percent maximum dry density as determined by ASTM D-1557. Prior to recompaction, the exposed subgrade should be thoroughly proofrolled using heavy vibratory compaction equipment. A CBR value of approximately 12 may then be used for design of the pavement section. The pavement should consist of crushed stone aggregate base course overlain by asphalt concrete. The thickness of the base and surface courses should be established following an evaluation of the expected traffic to be supported by the paved area.

SEWAGE LEACH FIELD

The leach field for disposal of septic tank effluents will be located in the vicinity of Boring VP-17, where future ground surface elevations of approximately 712 are anticipated. Percolation tests were accordingly conducted at 4 separate locations in that area. The tests were conducted at elevations ranging from 710.5 to 707, i.e. from 1.5 to 5 feet below the future ground surface. The tests were conducted in accordance with recommendations in the Manual of Septic Tank Practice by the U.S. Department of Health, Education and Welfare. The test location and results are shown on Plate A-16 in Appendix A. The percolation rates varied from 2.0 to 3.3 minutes per inch, with an average of 2.5 minutes per inch. We recommend that the upper limit rate of 3.3 minutes per inch be used in design to account for possible densification of the soils that may occur during site preparation activities.

CORROSION AND CHEMICAL DETERIORATION

Deterioration of metallic and/or concrete structures below the ground surface may occur where chemical and electrochemical conditions within the soil and ground water are unfavorable. A series of field and laboratory tests were conducted during this investigation to examine the potential for deterioration of proposed subsurface tanks, walls, and utilities within the HWVP site. Tests for pH of five soil samples were conducted in the field because the recommended sample holding time for this type of test is typically 24 hours. Additional soil taken from the same five locations was later tested in the laboratory for corrosion accelerating ions (i.e., sulfates, chlorides and sulfides), resistivity, oxidation-reduction (redox) potential and moisture content.

Two of the samples were also tested again for pH. A summary of the test results is presented on Plate B-2 of Appendix B.

The test results do not identify any property of the soils that gives rise to serious corrosion or other deterioration concerns at this site.

All soil resistivities are high. The resistivities ranges from a low of 55,900 ohm-cm to a high of 748,000 ohm-cm. Further there is a minimum amount of moisture present in the soil, and ground water was not encountered in any of the borings. Moreover the pH of the soil is neutral to slightly basic.

The redox potentials were all in excess of 100 millivolts indicating well aerated soil and the absence of sulfate reducing bacteria. The absence of sulfate reducing bacteria is confirmed by a negative finding of any sulfides in the soil.

Similarly the threshold for chlorides in soils to be of concern to reinforced concretes or to metals is between 400 and 600 parts per million. All samples tested were well below these levels.

The test data indicate: (1) The pH of the samples is neutral to slightly basic; (2) The redox potentials indicate non aggressive soil conditions; (3) Significant quantities of highly aggressive ions, i.e., sulfides, sulfates, chlorides have not been identified. Therefore, soil resistivity is the primary analysis tool used in evaluating the relative rate of corrosion of structures to be buried at this site.

The soil resistivity data at this site is classified as high. It averages nearly an order of magnitude higher than levels where minimal corrosion activity will occur. Thus this environment should be compatible with nearly all common metals and concretes used in the construction of underground utilities.

This analysis does not consider stray currents which may occur at the site nor does it evaluate the possibility of the interconnection of dissimilar metals. The above analysis is based only on the corrosion behavior of a single material by itself without outside influences.

From this analysis it may be concluded that special corrosion mitigating features need not be included int eh design of underground utilities. standard quality controlled designs including proper bedding materials should provide structures that will experience little or no corrosion over their design life. However any buried structure viewed as "critical" should be reviewed in detail in the design phase for possible outside influences such as the interconnection of dissimilar metals or the possibility of being under the influence of stray currents.

Based on the above analysis, we recommend the following:

1. Standard construction materials and practices may be employed for the installation of sanitary sewer and water utilities.

2. Standard construction materials and practices may be employed for the installation of potable and fire water utilities.

3. Standard construction materials and practices may be employed for the installation of tanks bottoms, underground tanks or buried concrete structures.

4. Standard construction materials and practices may be employed for the installation of the site groundgrid system.

5. The interface connections between existing utilities and utilities to be constructed should be completed in a manner to minimize or eliminate any galvanic couples and also to avoid the introduction of stray current onto the piping.

6. The sulfate content is such that specialty sulfate resistant portland cements are <u>not</u> required for concrete tanks and utilities for corrosion control. This does not exclude selection of those materials for other than corrosion control reasons.

7. The soil conditions in general (based on the available data) would <u>not</u> require the use of specialty coatings, inert membranes, nor cathodic protection. This does not exclude selection of those materials for other than corrosion control reasons.

8. The groundgrid design should take into consideration the high soil resistivities at this site.

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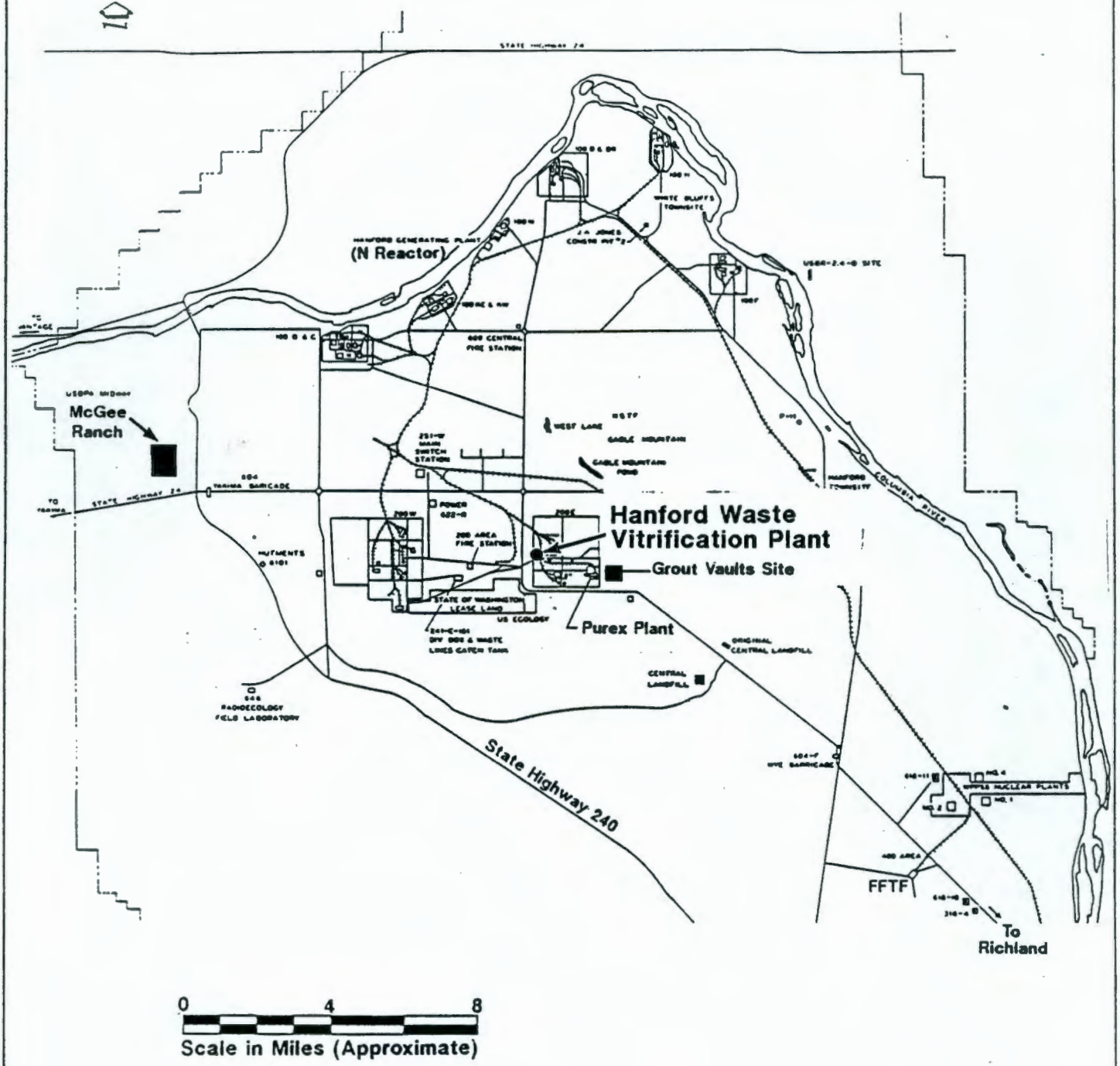

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November 15, 1989

Area Map
Hanford Waste Vitrification Plant
Dames & Moore

Plate 1

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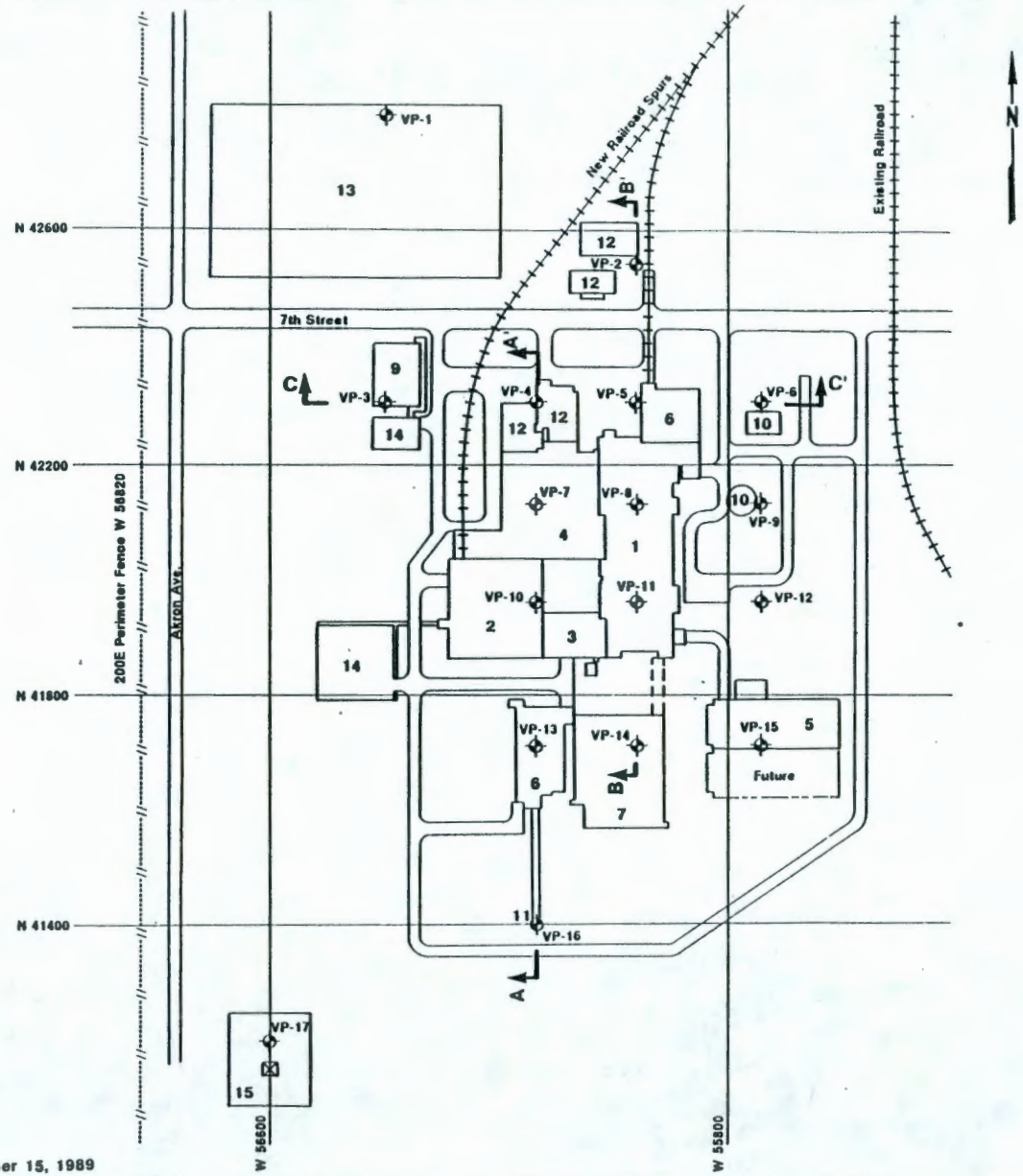

Reference: Flour Daniel Inc. Dwg. H 2-117070 Entitled "Geotechnical Investigation Site Location Plan", Rev. 1, Dated June 29, 1989

Key to Facilities:

1 Vitrification Building
2 Operations Control Building
3 Regulated Entrance Facility
4 Service Building
5 Canister Storage Building
6 Manipulator Repair Building
7 Sand Filter Building
8 Fan House
9 Switchgear & Generator Building
10 Wht Vault and Building
11 Exhaust Stack
12 Miscellaneous Structures
13 Switchyard
14 Parking Areas
15 Septic Tank and Leach Field

Legend:

VP-1 Boring Location and Number

Percolation Test Location

A Cross-Section Location

0 200 400
Approximate Scale in Feet

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Plate 2
Site Plan
Hanford Waste Vitrification Plant

Dames & Moore

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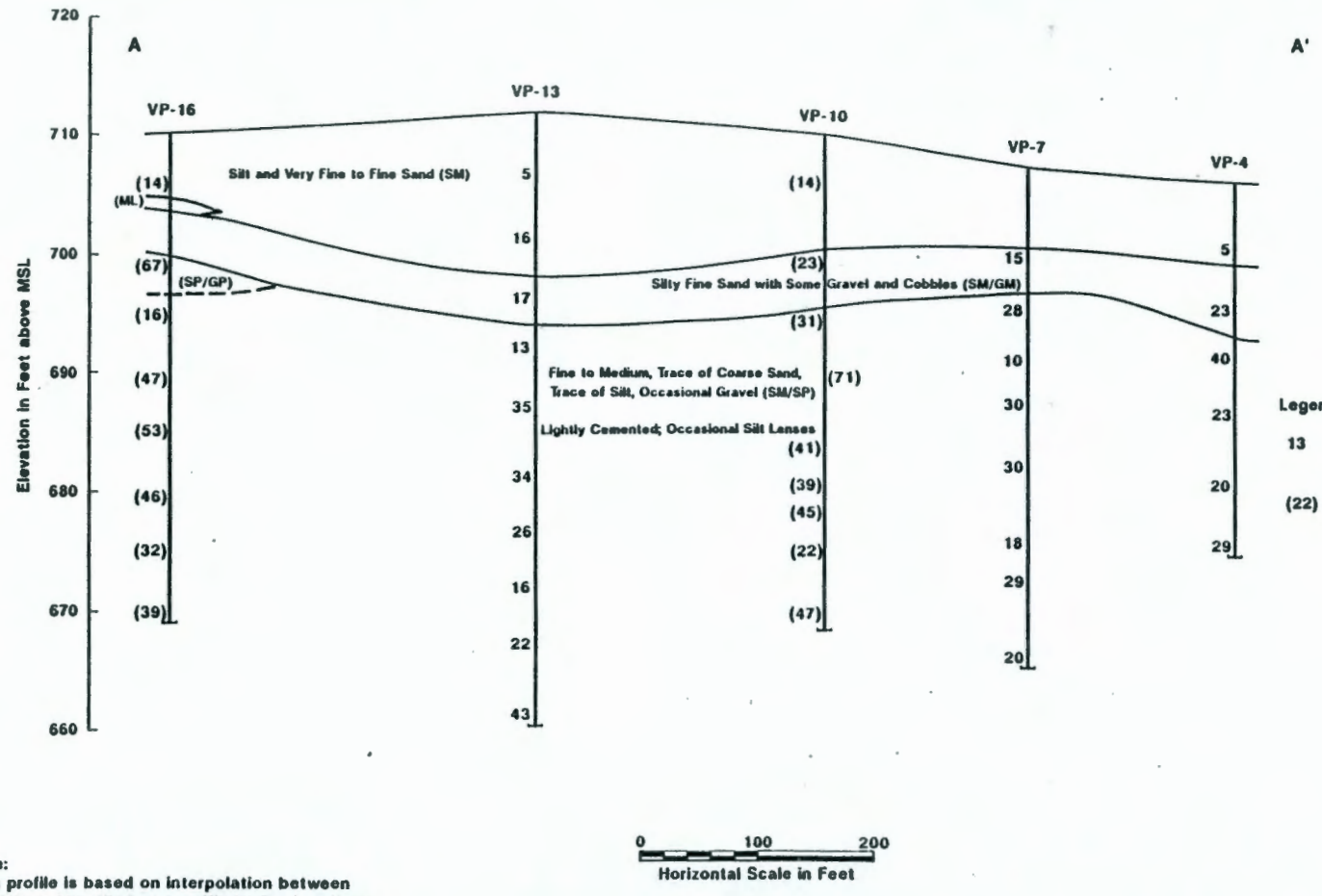

Legend

13 Penetration resistance in blows per foot using 750 lb. hammer with 18 inch drop.

(22) Penetration resistance in blows per foot using 825 lb. hammer with 28 inch drop.

0 100 200
Horizontal Scale in Feet

Note:
This profile is based on interpolation between conditions identified at specific boring locations, and should be considered approximate. Actual conditions may differ considerably from those shown.

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Plate 3
Cross-Section A-A'
Hanford Waste Vitrification Plant
Dames & Moore

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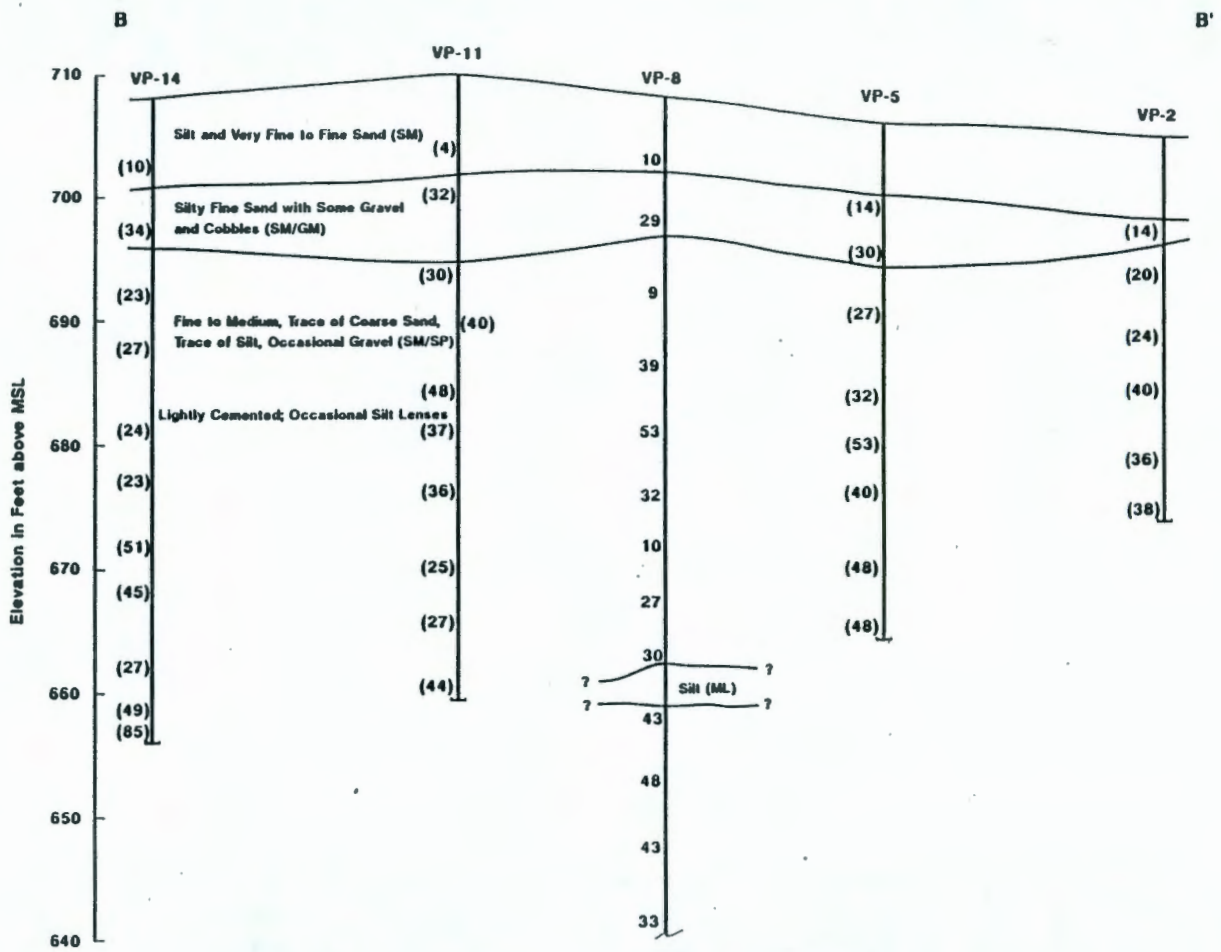

Legend

13 Penetration resistance in blows per foot using 750 lb. hammer with 18 inch drop.

(22) Penetration resistance in blows per foot using 825 lb. hammer with 28 inch drop.

Note:
This profile is based on interpolation between conditions identified at specific boring locations, and should be considered approximate. Actual conditions may differ considerably from those shown.

0 100 200
Horizontal Scale in Feet

Job No. 10805-398-016

November 15, 1989

Plate 4

Cross-Section B-B'
Hanford Waste Vitrification Plant
Dames & Moore

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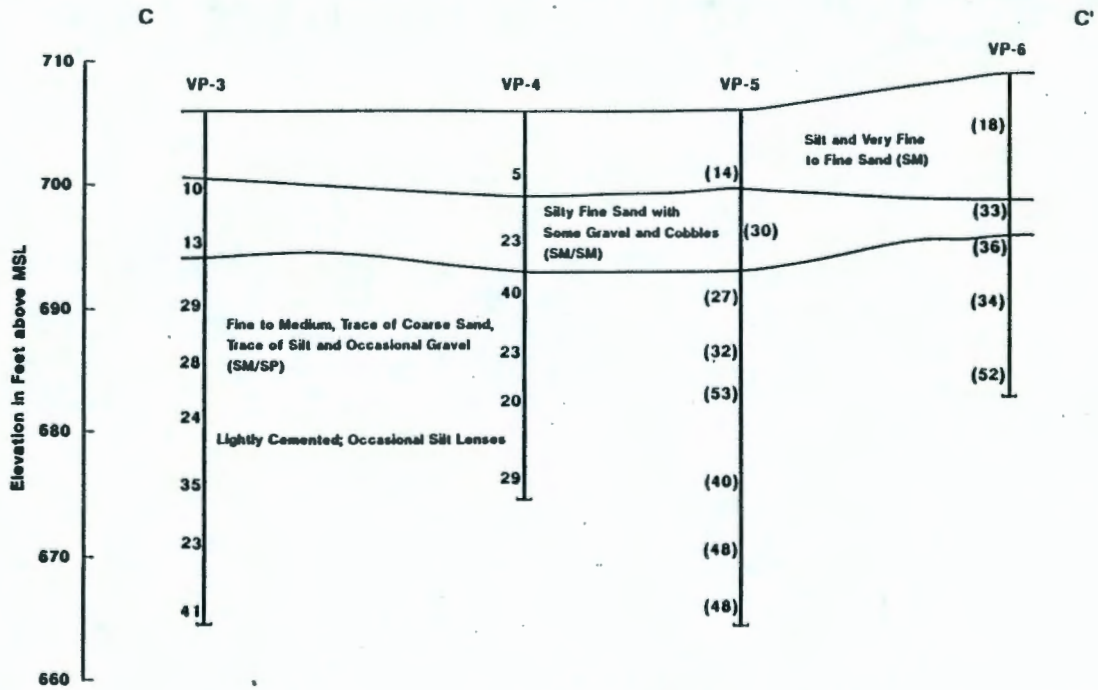


Legend

13 Penetration resistance in blows per foot using 750 lb. hammer with 18 inch drop.

(22) Penetration resistance in blows per foot using 825 lb. hammer with 28 inch drop.

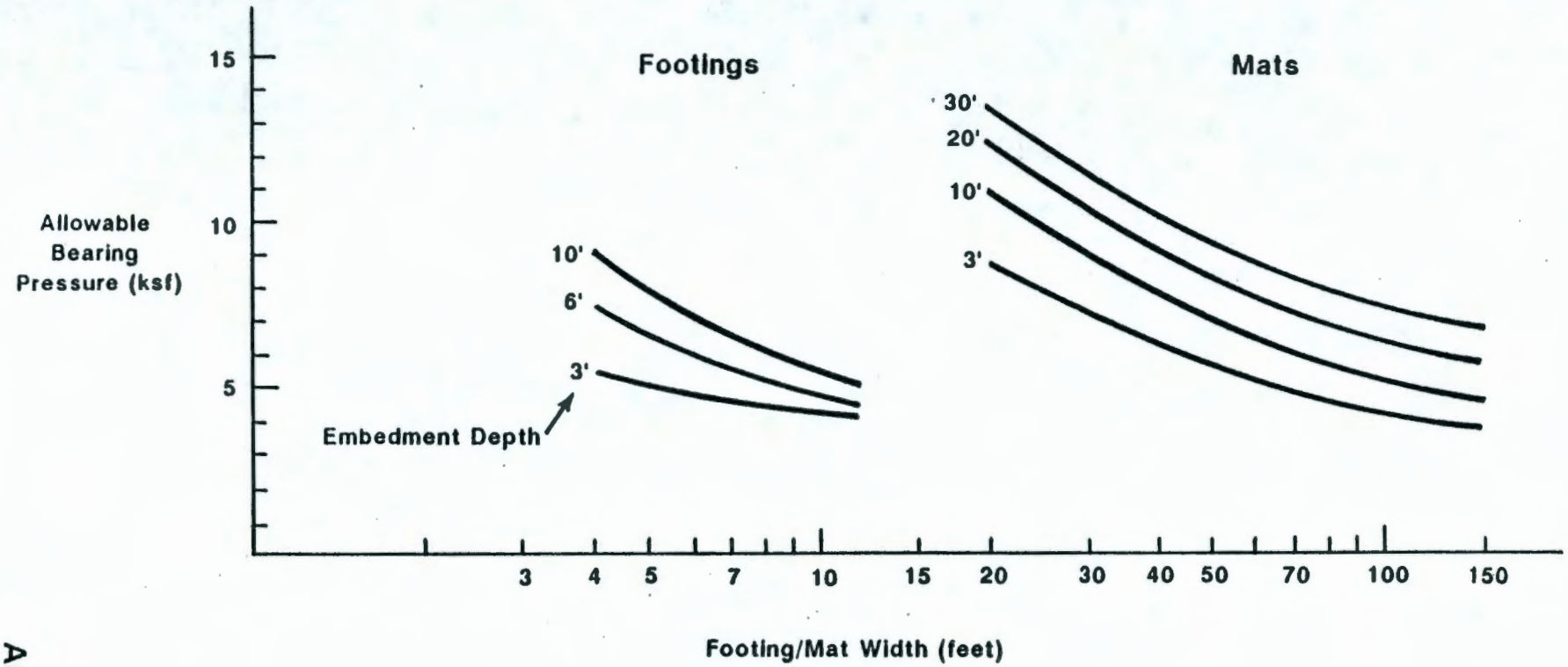
Note:
This profile is based on interpolation between conditions identified at specific boring locations, and should be considered approximate. Actual conditions may differ considerably from those shown.

0 100 200
Horizontal Scale in Feet

Job No. 10805-398-016

November 15, 1989

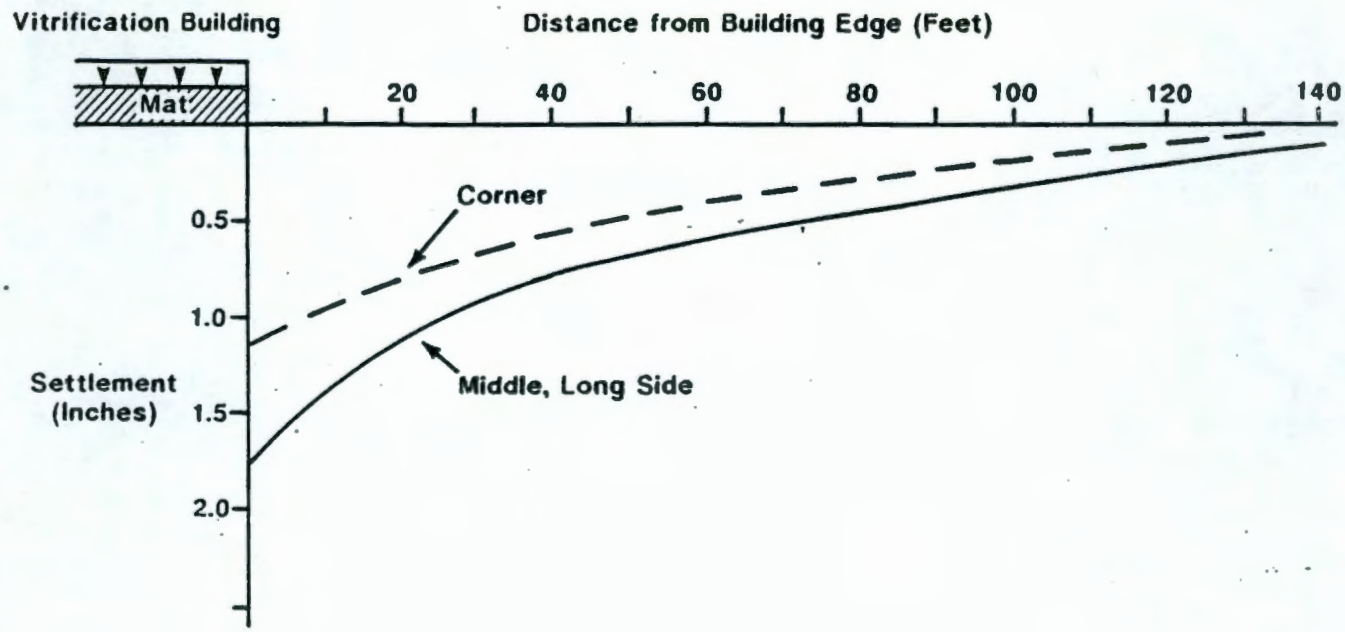
Plate 5
Cross-Section C-C'
Hanford Waste Vitrification Plant
Dames & Moore



Note

Allowable bearing pressures selected from chart above are those which will result in total settlements of less than 1 inch for footings and less than 2.5 inches for mats. Settlement estimates are based on typical soil profile consisting of 12 feet of Stratum 1 followed by 5 feet of Stratum 2 over the deep deposit of Stratum 3.

Allowable Bearing Pressures
Hanford Waste Vitrification Plant
Dames & Moore



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Settlements Induced by Vitrification Building
Hanford Waste Vitrification Plant
Dames & Moore

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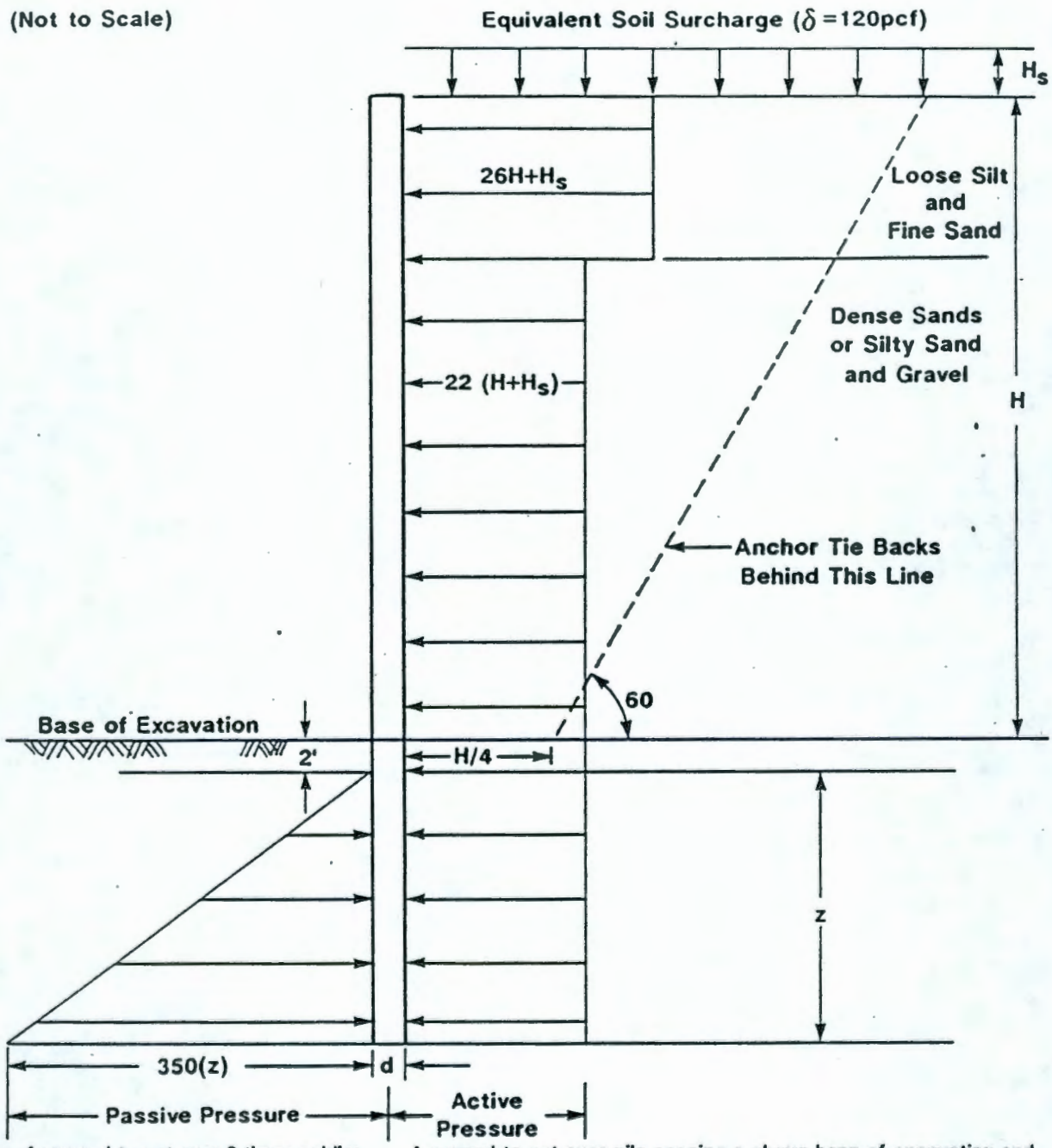

Assumed to act over 2 times soldier pile diameter d or pile spacing s whichever is less

Assumed to act over pile spacing s above base of excavation and over soldier pile diameter d below excavation base

Notes:

1. Above diagrams do not include any allowance for water pressure buildup behind walls.
2. Diagrams are illustrative only. No relation to a specific portion of the proposed excavation is intended. Refer to text for additional discussion.
3. The factor of safety for passive pressure is 1.5.
4. Soldier pile design parameters: Allowable end bearing = 15 KSF Natural Soils
   Allowable side friction = 2 KSF Natural Soils
   Minimum embedment = 10 feet

Design Lateral Earth Pressures
Temporary Excavation Shoring
Hanford Waste Vitrification Plant

November 15, 1989

Dames & Moore

Plate 8

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Depth (feet)	Natural Density (pcf)	Compressional Wave Velocity (fps)	Shear Wave Velocity (fps)	Dynamic Shear Modulus (psf)	Dynamic Poisson's Ratio	Elevation (feet)
0	102	2490	520	0.9×10^6	0.47	708
10	124	3830	1060	4.4×10^6	0.46	700
20						690
30	118	5330	1470	8.0×10^6	0.46	680
40						670
50	122	6160	1630	10.1×10^6	0.46	660
60						650
70						640
80	122	6600	2000	15.3×10^6	0.45	630
						620
100						600
150						550
200						500
250						450
300						400
350	125*	7000*	2500*	24×10^6*	0.43*	350
400	Basalt Bedrock					300

* Estimated using data from previous investigations.

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Seismic Design Data
Hanford Waste Vitrification Plant
Dames & Moore

Plate 9

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Job No. 10800-098-00

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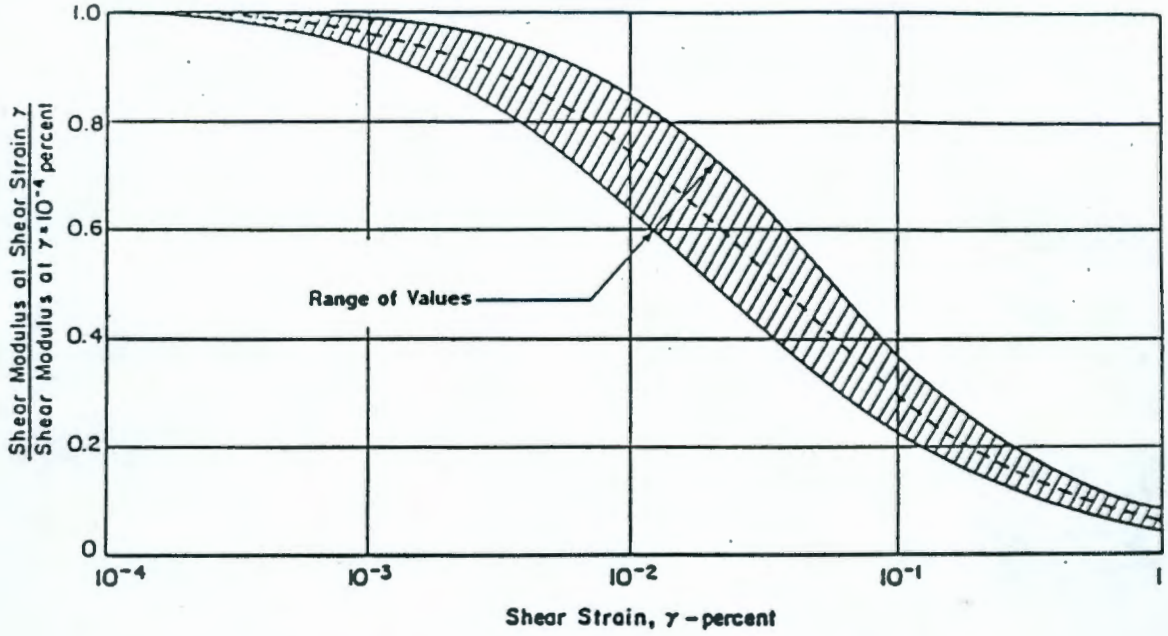
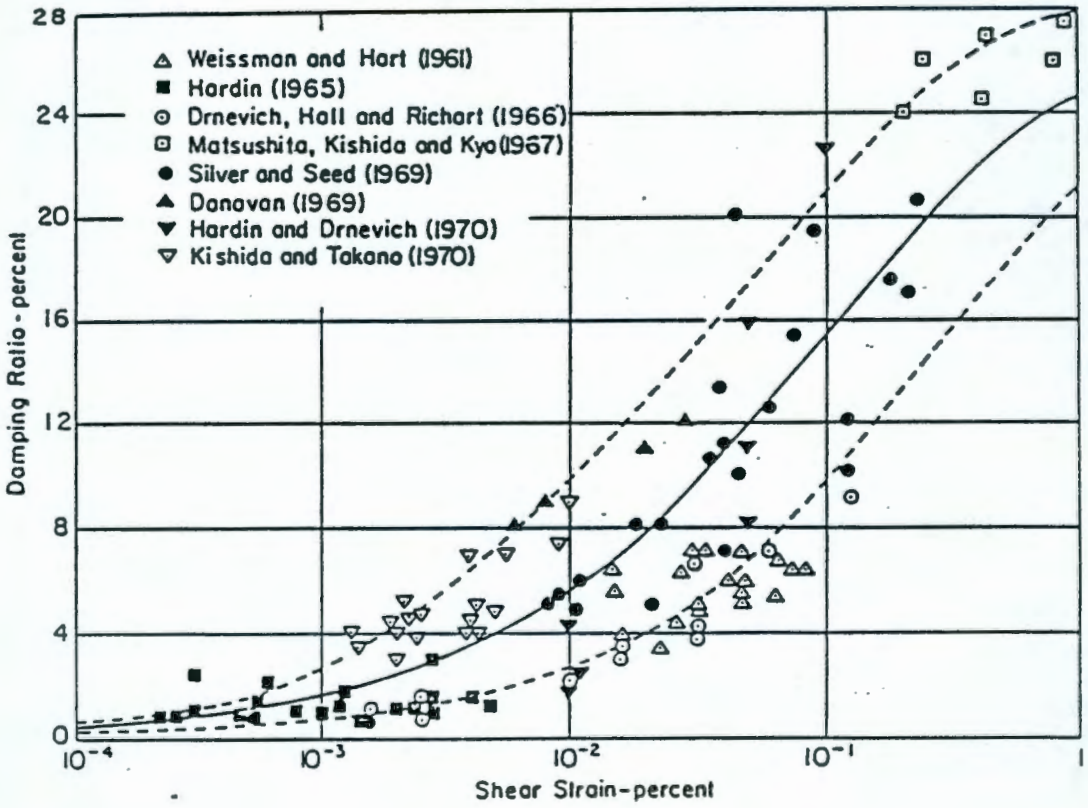


VARIATION OF SHEAR MODULUS WITH SHEAR STRAIN FOR SANDS
(After Seed & Idriss, 1970)



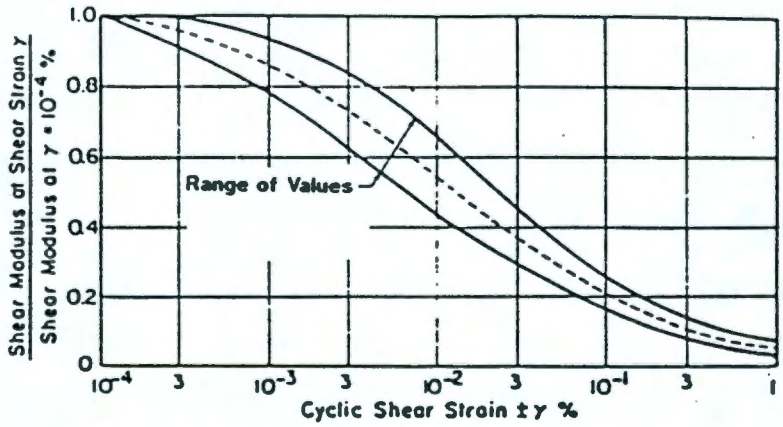
DAMPING RATIOS FOR SANDS
(After Seed & Idriss, 1970)

Job No. 10805-398-016

November 15, 1989

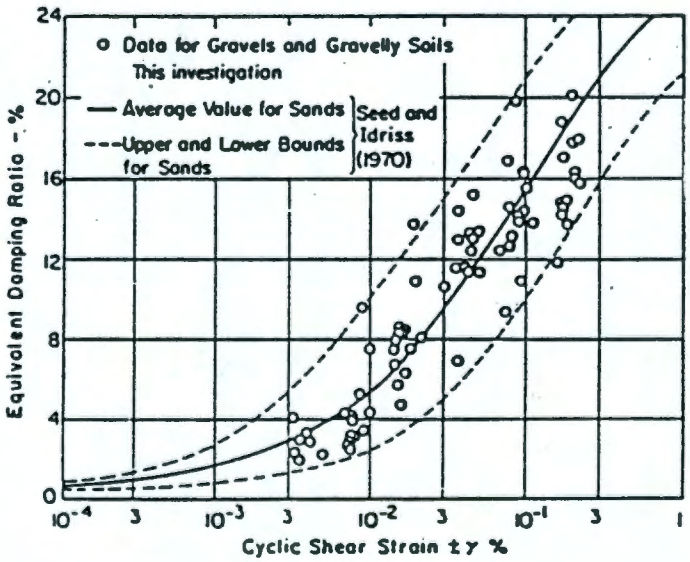
Shear Modulus and Damping Values for Sand
Hanford Waste Vitrification Plant
Dames & Moore

Plate 10



Variation of Shear Modulus with Shear Strain for Gravelly Soils

(After Seed et al, 1986)



Comparison of Damping Ratios for Gravelly Soils and Sands

(After Seed et al, 1986)

Job No. 10805-398-016

November 15, 1989

Shear Modulus and Damping Values for Gravel
Hanford Waste Vitrification Plant
Dames & Moore

Plate 11

APPENDIX A

FIELD INVESTIGATION PROGRAM

Subsurface and ground-water conditions at the site were investigated by drilling 17 boreholes (numbered VP-1 through VP-17) and performing 4 percolation tests in the vicinity of VP-17. The borings were drilled by Kaiser Engineering Hanford using a Bucyrus Erie Model 22-W cable tool rig. The borings consisted of one boring (VP-8) to 102-foot depth and sixteen others ranging from 21.5-foot to 80.5-foot depths. The borings were advanced without drilling fluid by driving a 6-inch diameter steel "core barrel" with a "split jar" downhole hammer. As the borings were advanced, an 8-inch diameter steel casing was driven to prevent caving of soils above the sampling depth. Water was occasionally poured into the hole to prevent caving of the loose, dry soils, and to aid in recovering samples. Slightly disturbed soil samples were taken at 5-foot intervals by driving a Dames & Moore U-type sampler (see Page A-1) using either a 750-lb hammer falling through a distance of 18 inches, or a 825-lb hammer falling through a distance of 28 inches. During the sampling process, resistance to penetration of the sampler, in blows per foot, were recorded and are presented in the Log of Borings in this appendix. The larger hammers employed in this investigation have produced penetration resistance values that are lower than those which would result from the use of Standard Penetration Test (SPT) equipment. The SPT equipment includes a 140 lb hammer falling through a distance of 30 inches. The penetration resistance values must therefore be multiplied be a correction factor to obtain equivalent SPT penetration resistance values. The correction factor is approximately 1.4 for the 750 lb hammer and approximately 2.3 for the 825 lb hammer. All borings were backfilled with dry, powdered bentonite.

We have noted that equivalent penetration resistance (N) values recorded in those borings drilled using the 750 pound hammer are consistently lower than those obtained using the 825 pound hammer. Further more, some N values recorded in Stratum 3 were, in our opinion, anomalously low and not consistent with our knowledge of the character of this stratum. Our review of sampling procedures indicated that N values using the 825 pound hammer were for depth intervals approximately 6 inches further below the bottom of the hole than those for the 750 pound hammer. This situation was the result of using a longer sample barrel with the heavier hammer. We accordingly feel that N values obtained using the 750 pound hammer were more

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likely to reflect disturbed conditions at the bottom of the hole, and are less representative of the undisturbed natural soil.

The drilling program was coordinated by a field representative of Dames & Moore who monitored the drilling activities, obtained samples of the soils encountered, classified the soils by visual and textural examination and maintained continuous logs of the subsurface conditions. Logs of the borings are presented in Plates A-1 through A-16.

The coordinates and depths of the borings are listed below:

Boring No.	Total Depth (feet)	Approx. Ground Surface Elev. (feet)	Coordinates
VP-1	21.5	711	N 42800, W 56400
VP-2	31.0	705	N 42540, W 55960
VP-3	41.5	706	N 42310, W 56400
VP-4	31.5	706	N 42310, W 56140
VP-5	41.5	706	N 42310, W 55960
VP-6	26.0	709	N 42310, W 55740
VP-7	41.5	707	N 42130, W 56140
VP-8	102.0	708	N 42130, W 55960
VP-9	80.5	710	N 42130, W 55740
VP-10	41.5	710	N 41960, W 56140
VP-11	50.5	710	N 41960, W 55960
VP-12	30.0	711	N 41960, W 55740
VP-13	51.5	712	N 41710, W 56140
VP-14	52.0	708	N 41710, W 55960
VP-15	62.5	709	N 41710, W 55740
VP-16	41.0	710	N 41400, W 56140
VP-17	26.0	715	N 41200, W 56600

Percolation Tests: Percolation tests were conducted at four different locations near Boring VP-17 (see Plate A-18). These tests were conducted in general accordance with procedures outlined in the Manual of Septic Tank Practice, U. S. Department of Health, Education and Welfare. The boring was advanced to the bottom of the desired test interval using a hand auger, about 2 inches of washed gravel was poured into the bottom of the hole, and a 4-inch diameter wire mesh screen was placed in the hole and surrounded by gravel to prevent caving during the percolation test. In each test, the water level was brought up to the top of the casing, then permitted to fall to the bottom as the time for the level to drop specified distances was recorded by the Dames & Moore representative. A summary of all the percolation values thus measured is presented in Plate A-18.

90118811021

Geophysical Test: Upon completion of VP-8, a 4-inch inside diameter schedule 80 flush-joint PVC casing was installed in the hole to permit geophysical testing by Geo Recon International. During the installation process, the PVC pipe was assembled and lowered to the bottom of the hole, then filled with water to prevent "floating" of the casing during subsequent grouting operations. The casing was grouted into the hole by pumping a cement-bentonite mixture into the top of the hole around the PVC pipe. The grout was permitted to harden for a period of 3 days prior to the geophysical testing. A description of the testing is presented in Appendix C, which also includes copies of the original field data records.

9 0 1 1 8 8 1 1 0 2 2

Boring VP-1

Elevation 711 ±

Depth in Feet	Sample Symbol		
0		SM	Light brown silt and very fine to fine sand (loose)
	16	SM GM	Light brown silt and fine to coarse sand with some fine to coarse gravel and occasional cobbles (medium dense)
5			
	14		
10			
			Refusal encountered at 13 feet.
			(Moved hole 20 feet south, started sampling at 15' depth)
15	45		
20	40	SM	Light brown fine sand with some silt and occasional gravel (dense)
25			

Boring completed at depth of 21.5 feet on 8-7-89.

No groundwater encountered during drilling.

Water added occasionally to prevent caving.

Hammer weight = 750 lbs.
Hammer drop = 18 inches

Boring VP-2

Elevation 705 ±

Depth in Feet	Sample Symbol		
0		SM	Light brown silt and very fine to fine sand (loose to medium dense)
			grades with trace of gravel
	pH=8.5		
5	14		
		SM GM	Light brown silty fine sand with some gravel and cobbles (dense)
		SM	Light brown or grayish brown fine to medium sand with trace coarse sand, trace silt, occasional gravel (dense)
10	20		
15	24		
20	40		
25	36		
30	38		
35			

Boring completed at depth of 31.5 feet on 8-5-89.

No groundwater encountered during drilling.

Water added occasionally to prevent caving.

Hammer weight = 825 lbs.
Hammer drop = 28 inches

November 15, 1989

Vertical Scale: 1 inch = 5 feet

Log of Borings

Dames & Moore

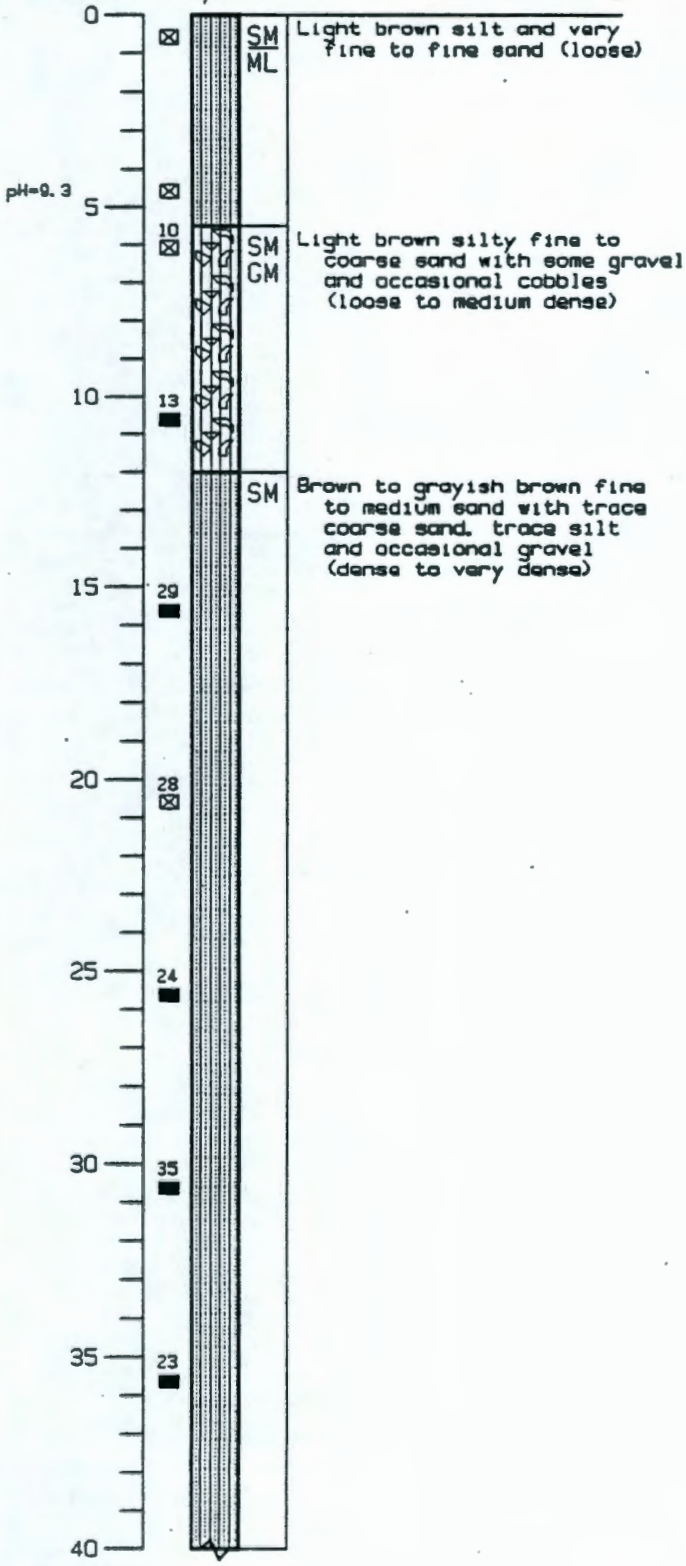
Job No. 10805-398-016

Plate A-1

9011188102 3

Boring VP-3

Elevation 706 ±

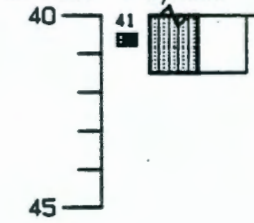


Light brown silt and very fine to fine sand (loose)

Light brown silty fine to coarse sand with some gravel and occasional cobbles (loose to medium dense)

Brown to grayish brown fine to medium sand with trace coarse sand, trace silt and occasional gravel (dense to very dense)

Boring VP-3, Cont.



Boring completed at depth of 41 feet on 8-5-89.

No groundwater encountered during drilling.

Hammer weight = 750 lbs.
Hammer drop = 18 inches

November 15, 1989

Vertical Scale: 1 inch = 5 feet

Log of Borings

Dames & Moore

Job No. 10805-398-016

Plate A-2

90118810246

Boring VP-4

Depth in Feet | Sample Symbol | Elevation 706 ±

Depth in Feet	Sample Symbol	Elevation 706 ±
0		SM, ML — Light brown silt and very fine to fine sand (loose)
5	5	
10	23	SM, GM — Light brown silty fine sand, some gravel and cobbles (medium dense)
15	40	SM — Brown to grayish brown fine to medium sand with trace coarse sand, trace silt and occasional gravel (dense)
20	23	
25	20	
30	29	
35		

Boring completed at depth of 31.5 feet on 8-4-89.

No groundwater encountered during drilling.

Hammer weight = 750 lbs.
Hammer drop = 18 inches

Boring VP-5

Depth in Feet | Sample Symbol | Elevation 706 ±

Depth in Feet	Sample Symbol	Elevation 706 ±
0		SM — Light brown silt and very fine to fine sand (loose)
5	14	
10	30	SM, GM — Light brown silty fine sand, some gravel and cobbles (medium dense)
15	27	SM — Brown or grayish brown fine to medium sand, trace coarse sand, trace silt and occasional gravel (dense to very dense)
20	32	lightly cemented
25	53	
30	40	
35	48	
40		

November 15, 1989

Log of Borings

Vertical Scale: 1 inch = 5 feet

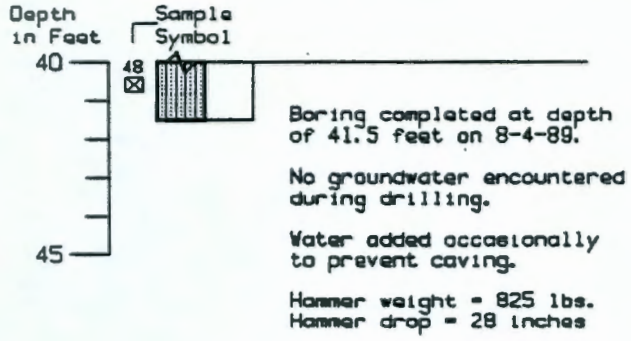
Dames & Moore

Job No. 10805-398-016

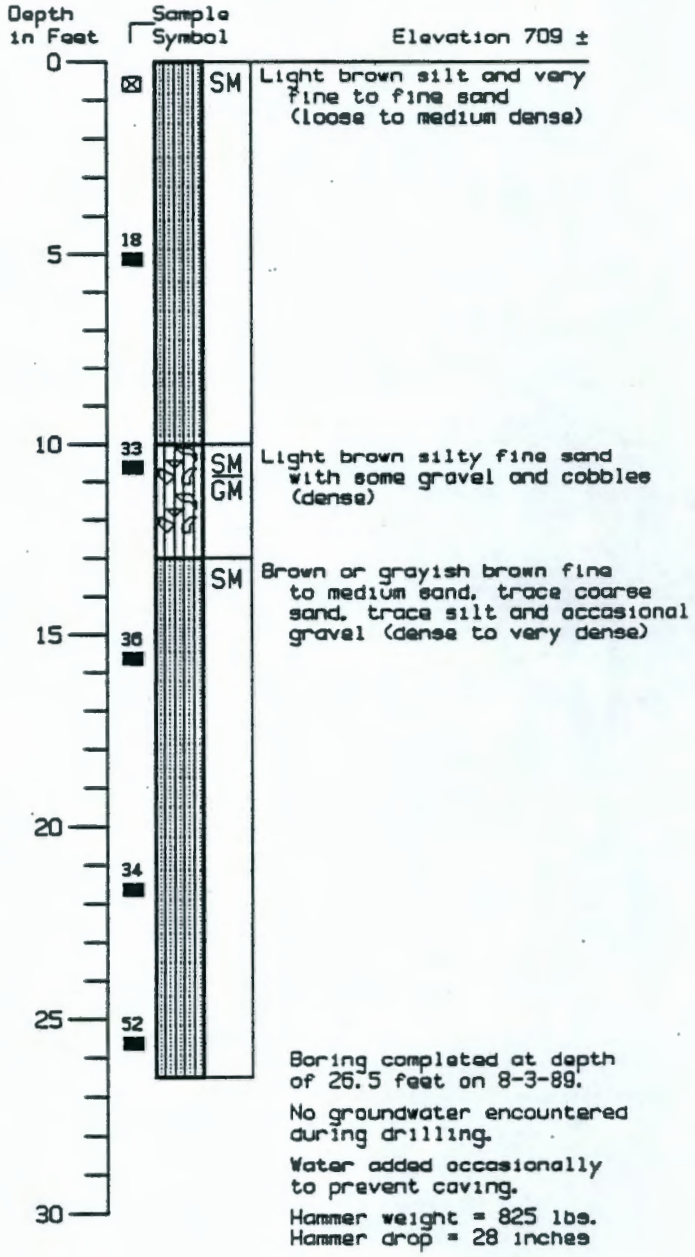
Plate A-3

9011188102 5

Boring VP-5, Cont.



Boring VP-6



November 15, 1989

Vertical Scale: 1 inch = 5 feet

Log of Borings

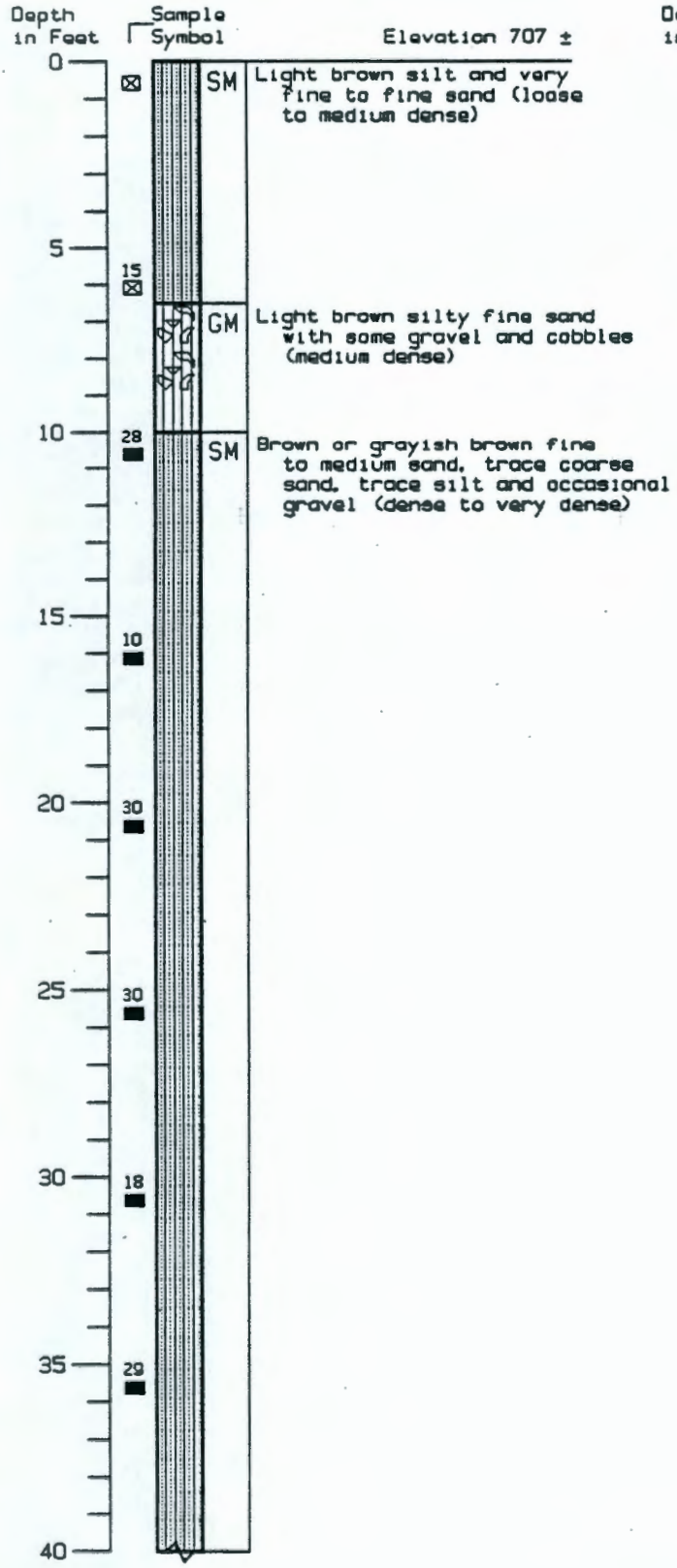
Dames & Moore

Job No. 10805-398-016

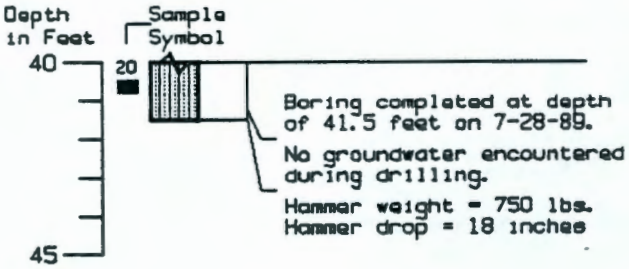
Plate A-4

9011183102 6

Boring VP-7



Boring VP-7, Cont.



November 15, 1989

Vertical Scale: 1 inch = 5 feet

Log of Borings

Dames & Moore

Job No. 10805-398-016

Plate A-5

90118810270

Boring VP-8

Elevation 708 ±

Depth in Feet	Sample Symbol	Unit	Description
0	⊠	SM	Light brown silt and very fine to fine sand (loose)
5	10	SM GM	Light brown silty fine sand with some gravel and occasional cobbles (medium dense)
10	29	SM	Brown or grayish brown fine to medium sand, trace coarse sand, trace silt and occasional gravel (dense to very dense)
15	9 ⊠		
20	39		
25	53		
30	32		
35	10		
40			

Boring VP-8, Cont.

Depth in Feet	Sample Symbol	Unit	Description
40	27		
45	30	ML	Brown silt (stiff)
50	43	SM	Brown or grayish brown fine to medium sand, occasional lenses of very fine sandy silt (dense to very dense)
55	48		
60	43		
65	33		
70	20		
75	30		
80			

November 15, 1989

Vertical Scale: 1 inch = 5 feet

Log of Borings

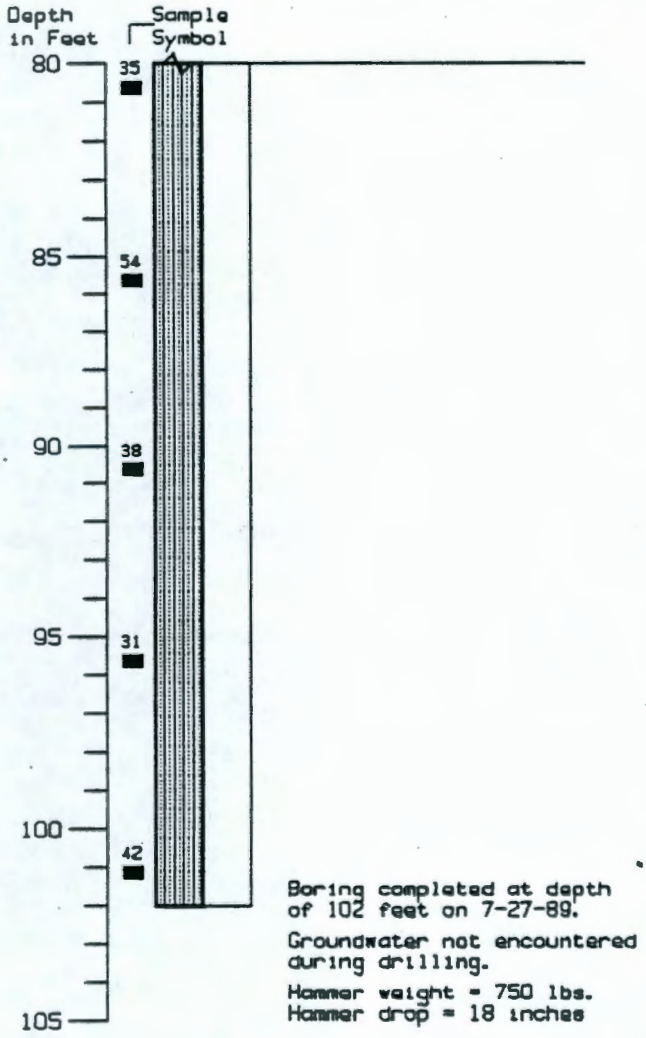
Dames & Moore

Job No. 10805-398-016

Plate A-6

9011188102 8

Boring VP-8, Cont.

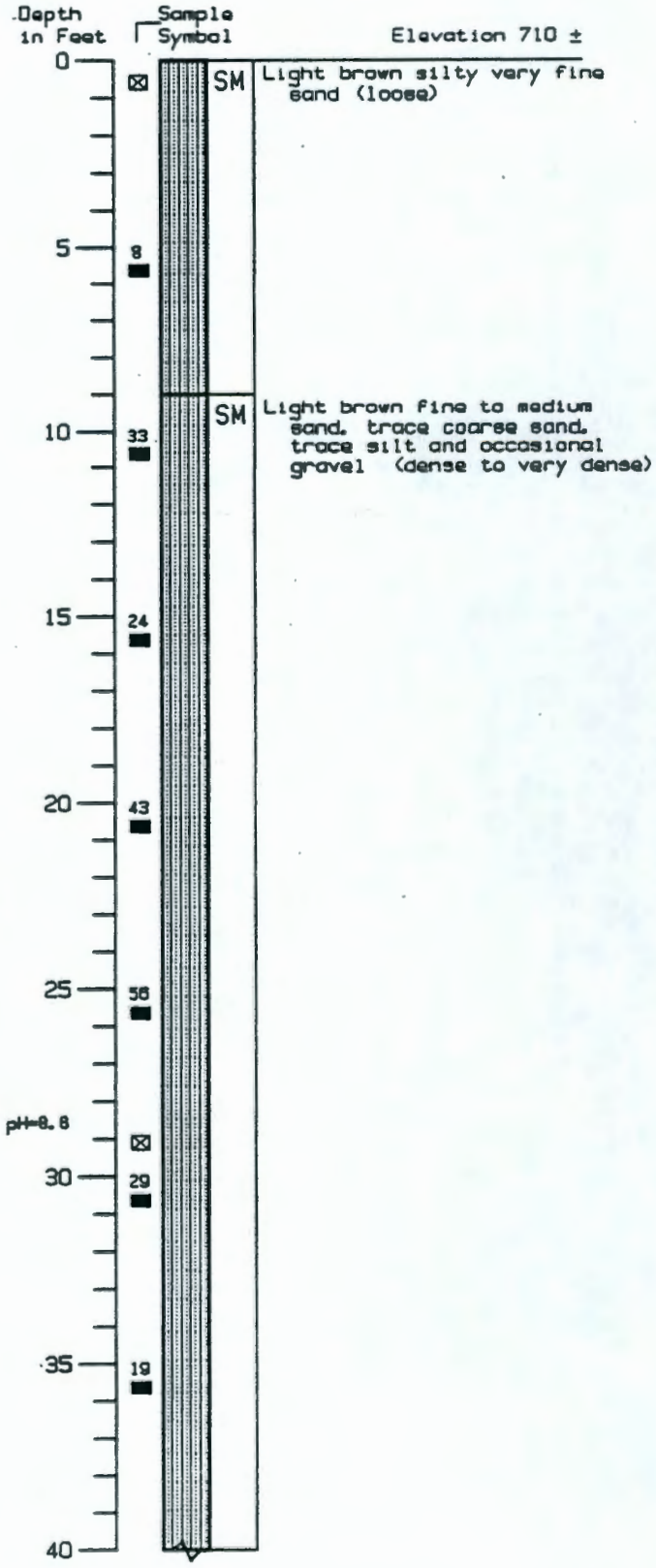


Boring completed at depth of 102 feet on 7-27-89.

Groundwater not encountered during drilling.

Hammer weight = 750 lbs.
Hammer drop = 18 inches

Boring VP-9



November 15, 1989

Vertical Scale: 1 inch = 5 feet

Log of Borings

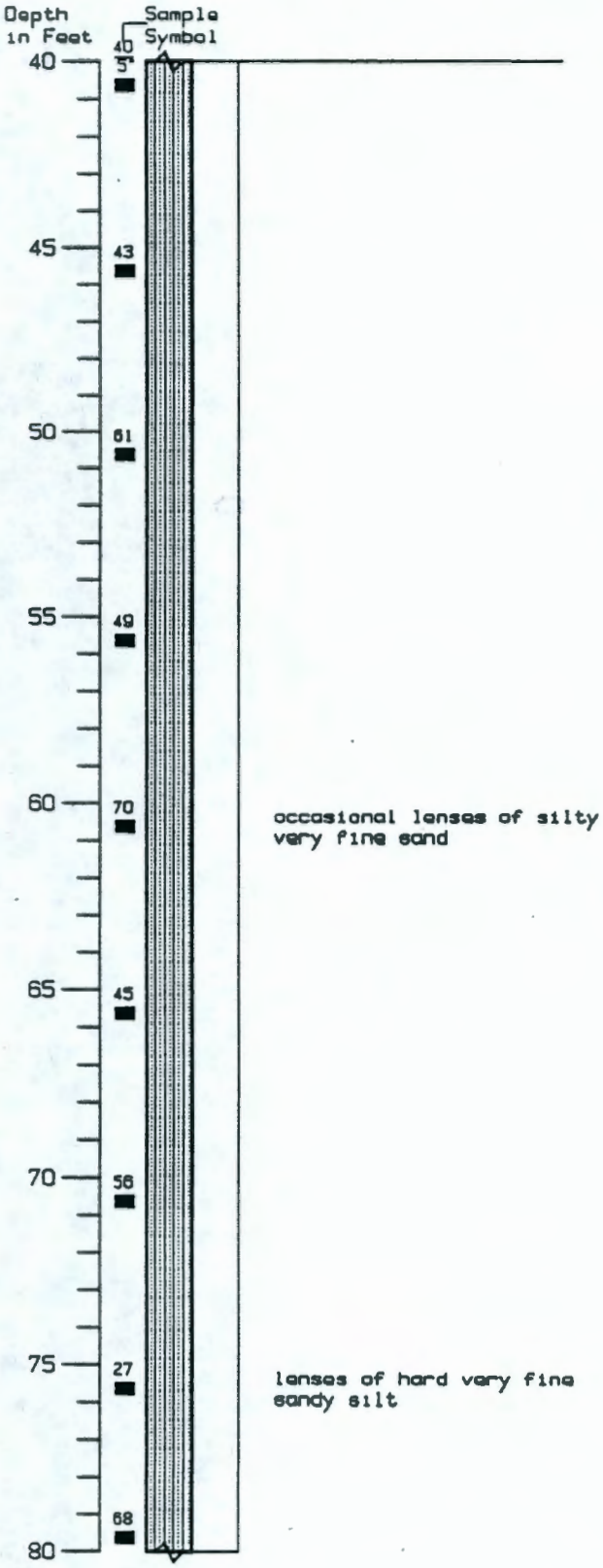
Dames & Moore

Job No. 10805-398-016

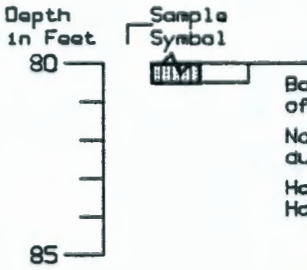
Plate A-7

9011181029

Boring VP-9, Cont.



Boring VP-9, Cont.



Boring completed at depth of 80.5 feet on 8-3-89.

No groundwater encountered during drilling.

Hammer weight = 750 lbs.
Hammer drop = 18 inches

November 15, 1989
Vertical Scale: 1 inch = 5 feet

Log of Borings

Dames & Moore

90118110030

Boring VP-10

Elevation 710 ±

Depth in Feet	Sample Symbol		
0	⊠	SM	Light brown silt and very fine to fine sand (loose to medium dense)
5	14 ■		
10	23 ■	SM GM	Light brown silty fine sand with some gravel and cobbles (medium dense)
15	31 ■	SM	Light brown to brownish gray fine to medium sand, trace coarse sand, trace silt and occ. gravel (dense to very dense)
20	71 ■		Increasing coarse sand 20 to 23 feet
25	41 ■		
	39 □		
30	45 ■		
35	22 ■		
40			

Boring VP-10, Cont.

Depth in Feet	Sample Symbol	
40	47 ■	
45		

Boring completed at depth 41.5 feet on 7-28-89.

No groundwater encountered during drilling.

Hammer weight = 825 lbs.
Hammer drop = 28 inches

November 15, 1989

Vertical Scale: 1 inch = 5 feet

Log of Borings

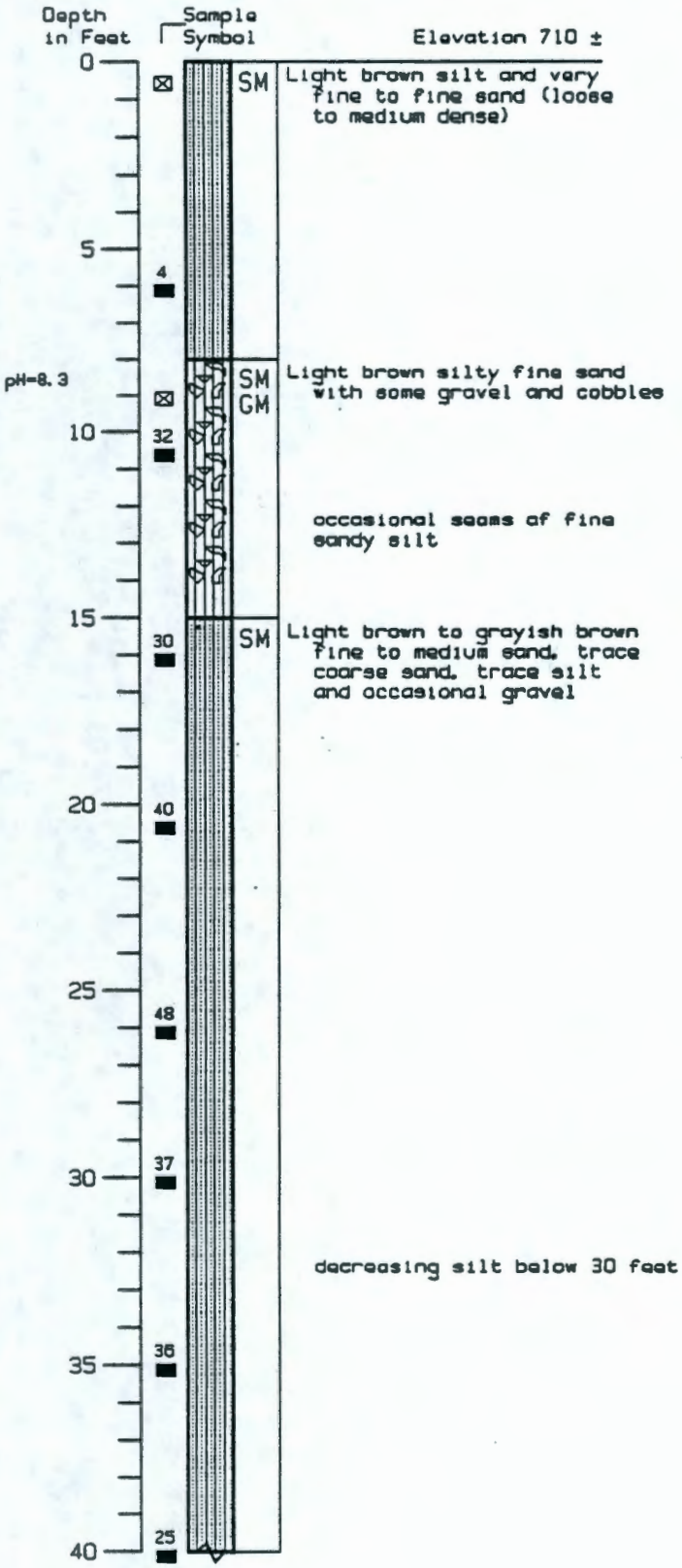
Dames & Moore

Job No. 10805-398-016

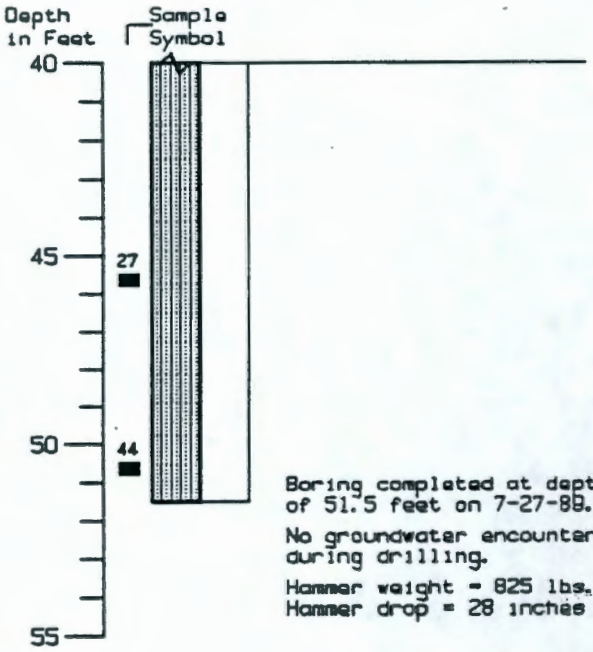
Plate A-9

90118810 31

Boring VP-11



Boring VP-11, Cont.



Boring completed at depth of 51.5 feet on 7-27-89.

No groundwater encountered during drilling.

Hammer weight = 825 lbs.
Hammer drop = 28 inches

November 15, 1989

Vertical Scale: 1 inch = 5 feet

Log of Borings

Dames & Moore

Job No. 10805-398-016

Plate A-10

9011188 1032

Boring VP-12

Elevation 711 ±

Depth in Feet	Sample Symbol	Unified Symbol	Description
0	⊠	SM	Light brown silt and very fine to fine sand (loose)
5	27 ■	SM/GM	Light brown silt fine to medium, trace coarse sand with some gravel (medium dense)
10	63 ■	SM/SP	Light brown or grayish brown fine to medium sand, trace coarse sand, trace of silt and occasional gravel (dense to very dense)
15	29 ■		
			lightly cemented
20	53 ■		
25	33 ■		
30	47 ■		
35			

Boring completed at depth of 31 feet on 8-2-89.

No groundwater encountered during drilling.

Water added occasionally to prevent caving.

Hammer weight = 825 lbs.
Hammer drop = 28 inches

Boring VP-13

Elevation 712 ±

Depth in Feet	Sample Symbol	Unified Symbol	Description
0	⊠	SM	Light brown silt and silty very fine to fine sand with fine roots (loose to medium dense)
5	5 ⊠		
10	18 ⊠		
15	17 ■	SM/GM	Light brown silty fine to medium, trace coarse sand with occasional gravel and cobbles
20	13 ■	SM/SP	Brown or grayish brown fine to medium sand, trace coarse sand, trace silt and occasional gravel (dense to very dense)
25	35 ■		
30	34 ■		
35	26 ■		
40			

November 15, 1989

Vertical Scale: 1 inch = 5 feet

Log of Borings

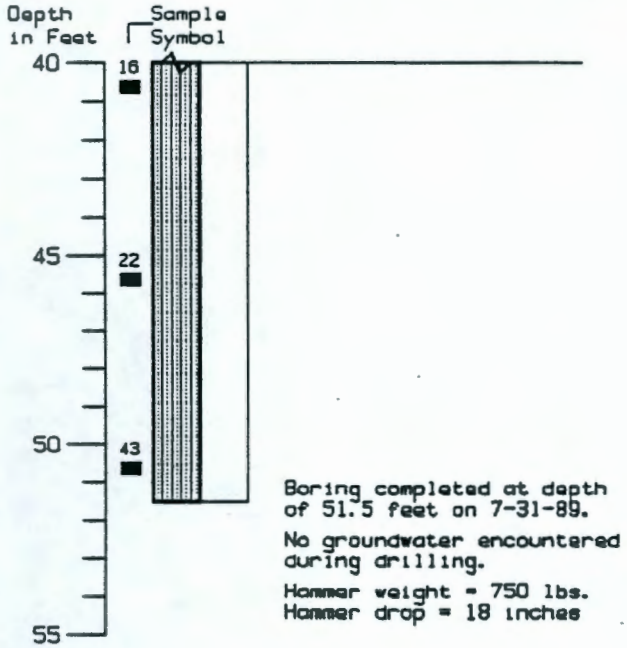
Dames & Moore

Job No. 10805-398-016

Plate A-11

90118810 33

Boring VP-13, Cont.



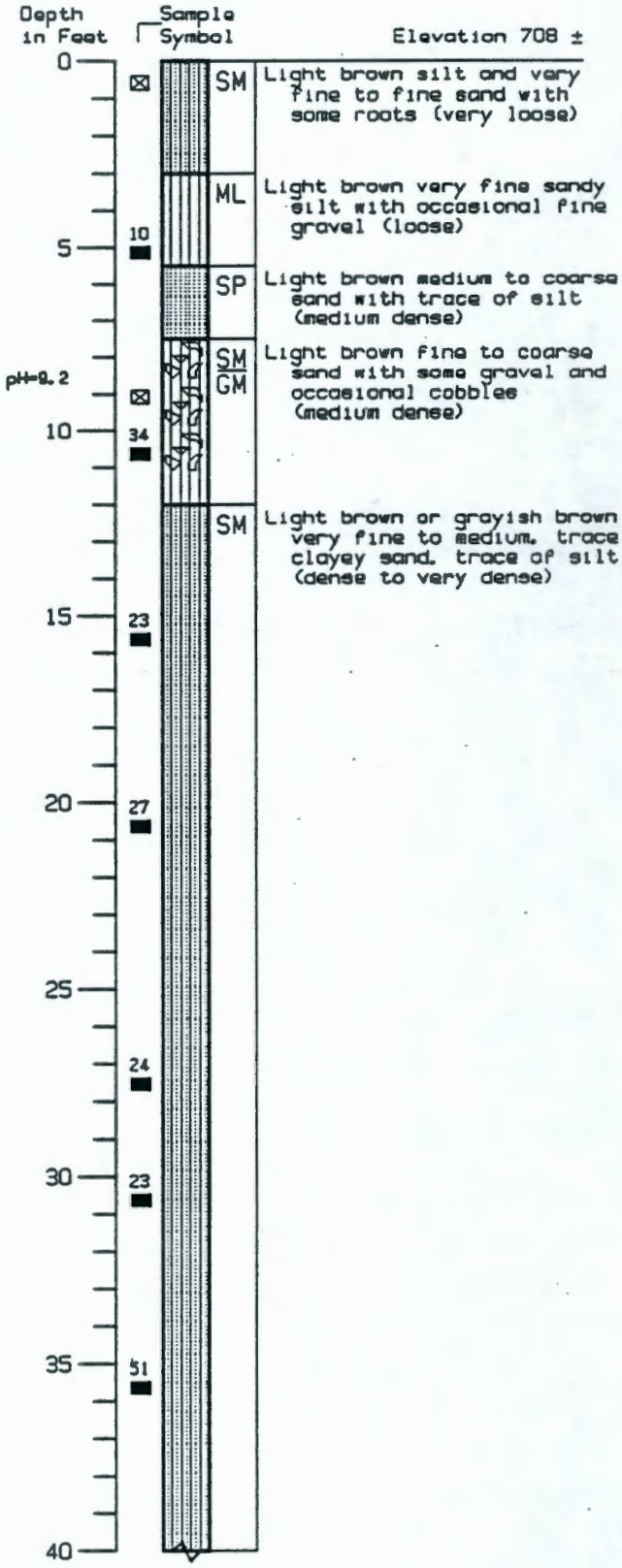
Boring completed at depth of 51.5 feet on 7-31-89.

No groundwater encountered during drilling.

Hammer weight = 750 lbs.
Hammer drop = 18 inches

Boring VP-14

Elevation 708 ±



November 15, 1989

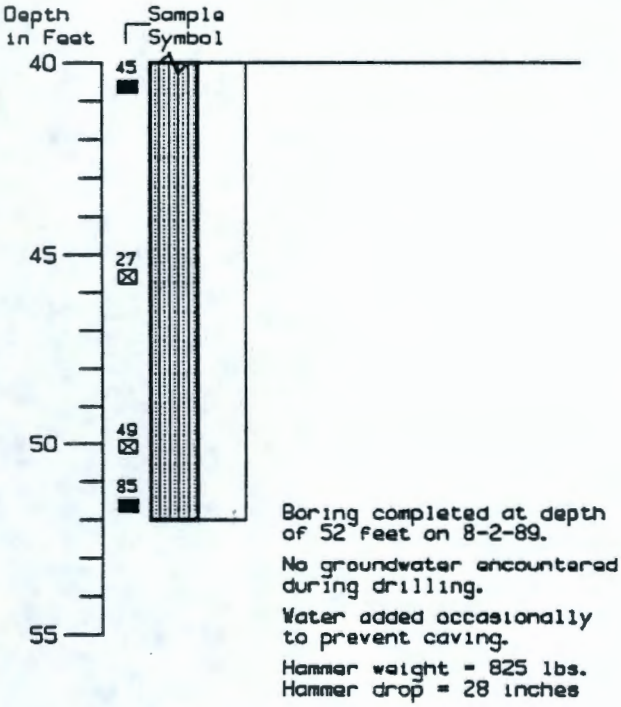
Vertical Scale: 1 inch = 5 feet

Log of Borings

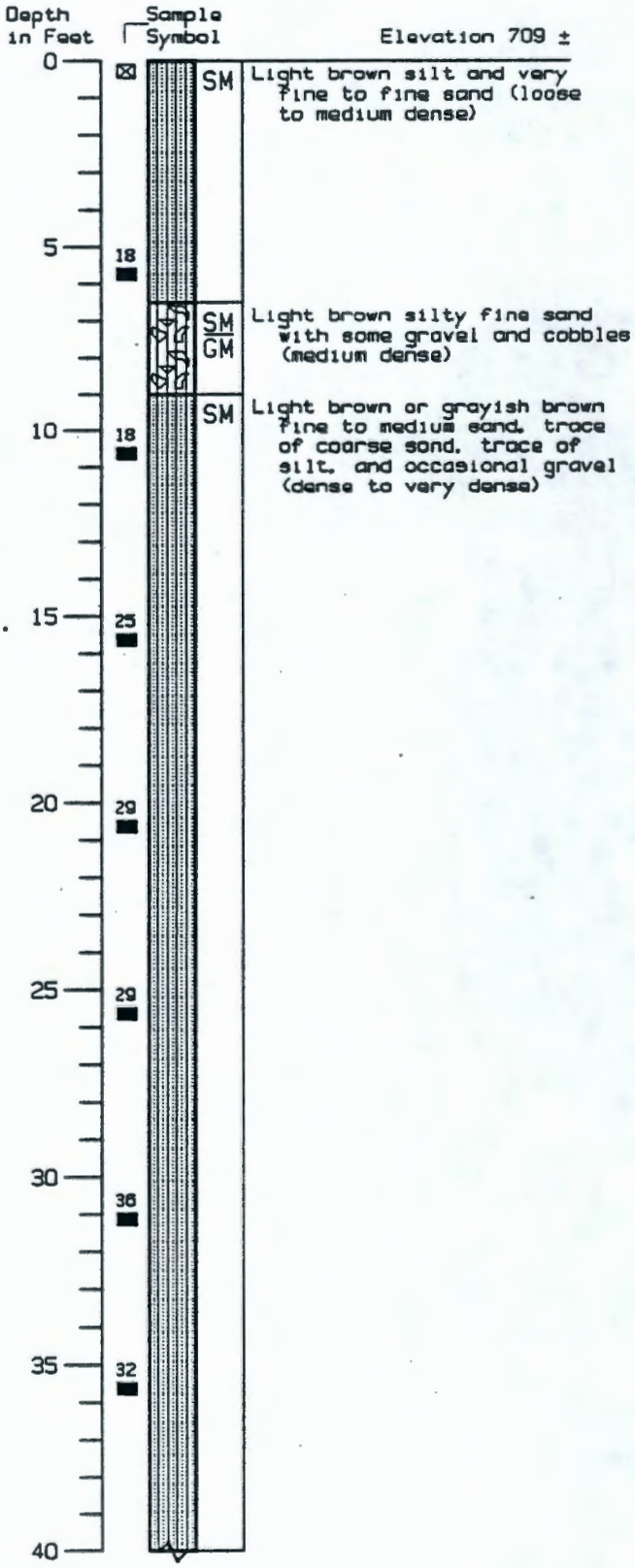
Dames & Moore

9011188110034

Boring VP-14, Cont.



Boring VP-15



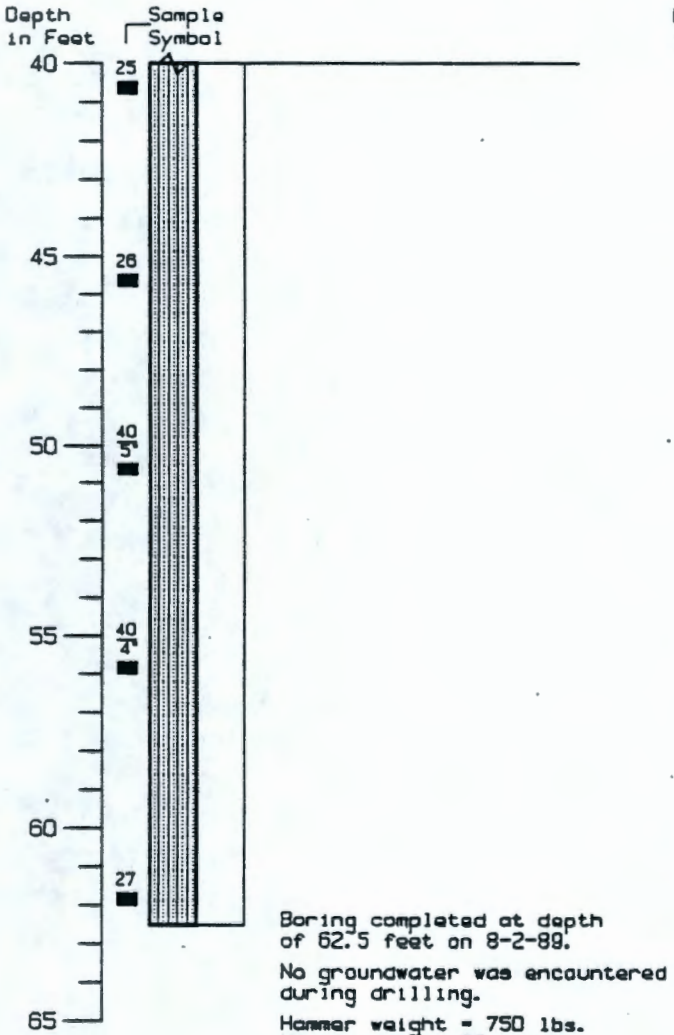
November 15, 1989

Vertical Scale: 1 inch = 5 feet

Log of Borings

Dames & Moore

Boring VP-15, Cont.



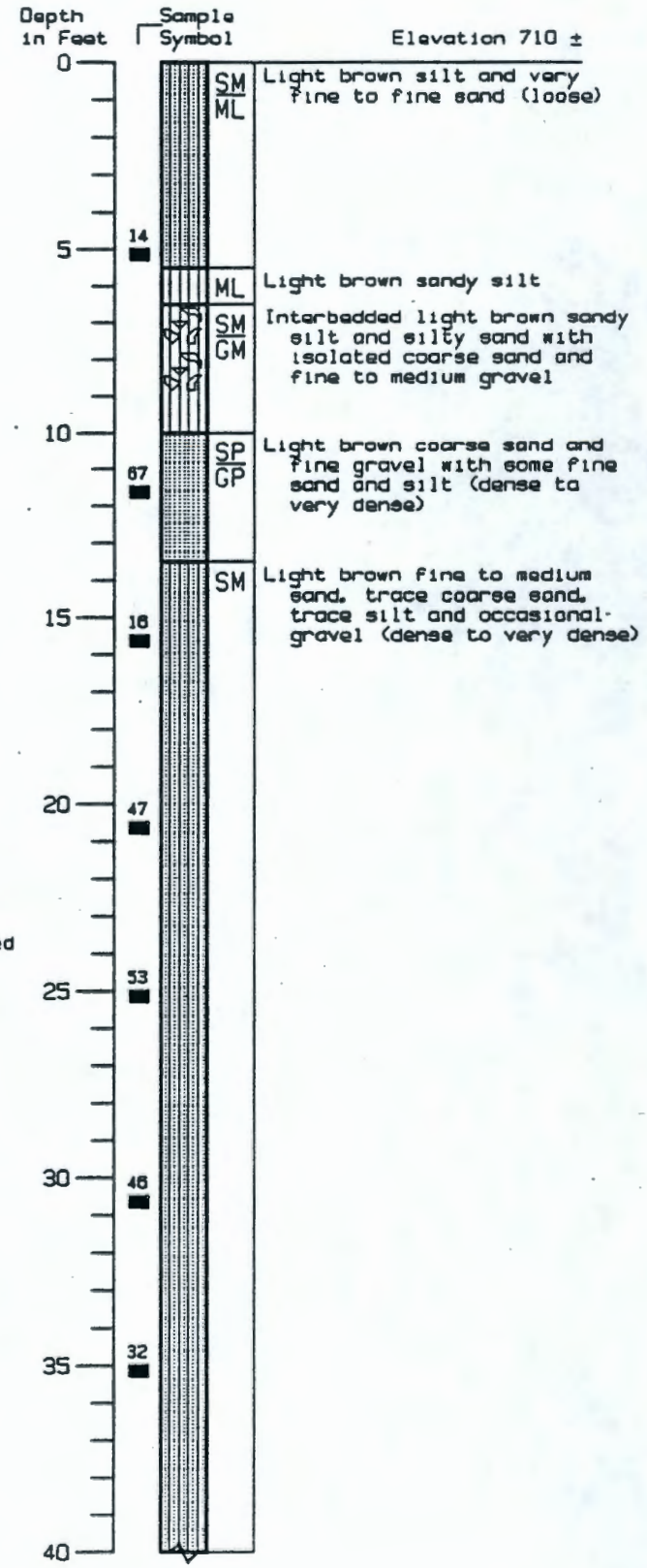
Boring completed at depth of 62.5 feet on 8-2-89.

No groundwater was encountered during drilling.

Hammer weight = 750 lbs.
Hammer drop = 18 inches

Boring VP-16

Elevation 710 ±



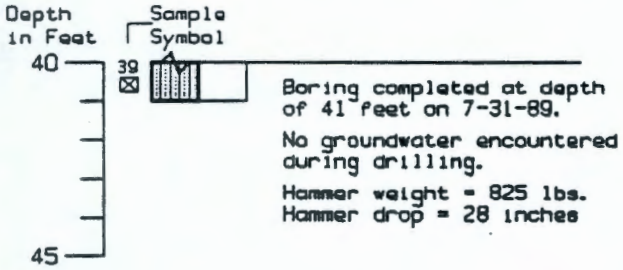
November 15, 1989

Vertical Scale: 1 inch = 5 feet

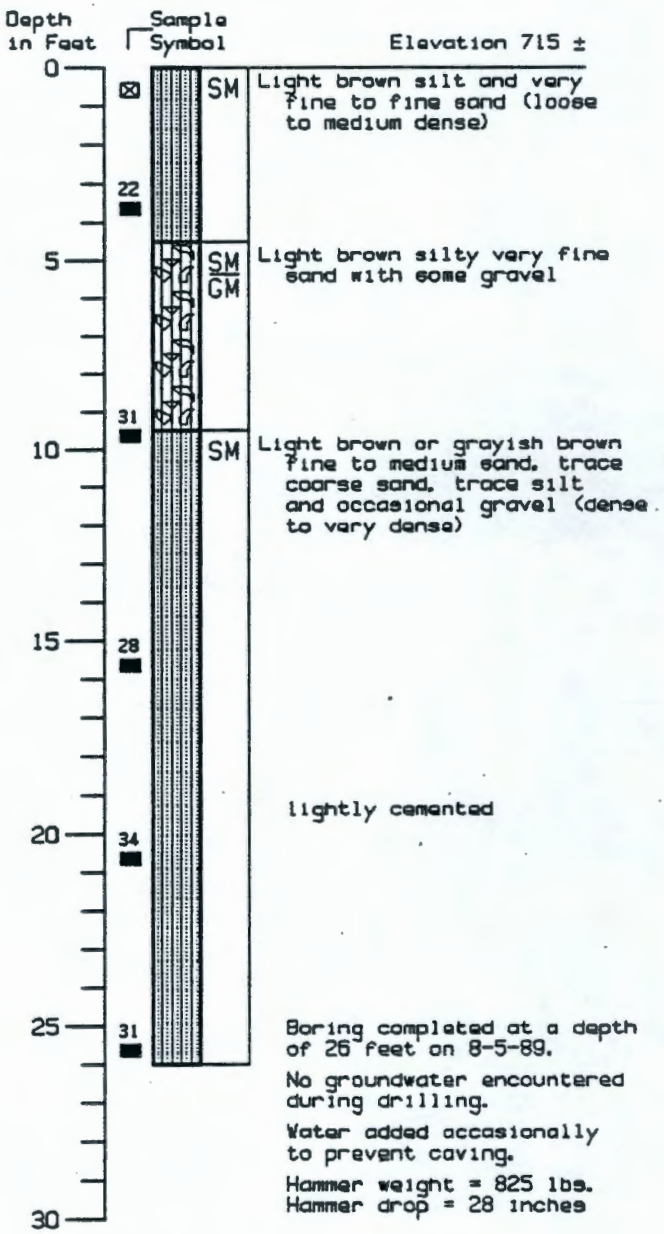
Log of Borings

Dames & Moore

Boring VP-16, Cont.



Boring VP-17



November 15, 1989

Vertical Scale: 1 inch = 5 feet

Log of Borings

Dames & Moore

Job No. 10805-398-016

Plate A-15

9011138 1103 7

KEY:

46.5%-72	Indicates moisture content and dry density in pcf
17	Blows required to drive Dames & Moore sampler one foot with hammer weight and a drop distance as indicated on each log.
■	Indicates depth at which undisturbed Dames & Moore sample was extracted
⊠	Indicates depth at which disturbed Dames & Moore sample was extracted
□	Indicates sampling attempt with no recovery
pH=8.5	NOTE: pHvalue from field test by electrometric method

Elevations obtained from aerial topographic map by DeGrass Aerial Mapping. Job No. 89-357, photographed 4-12-89.

9011181103 8

MAJOR DIVISIONS			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

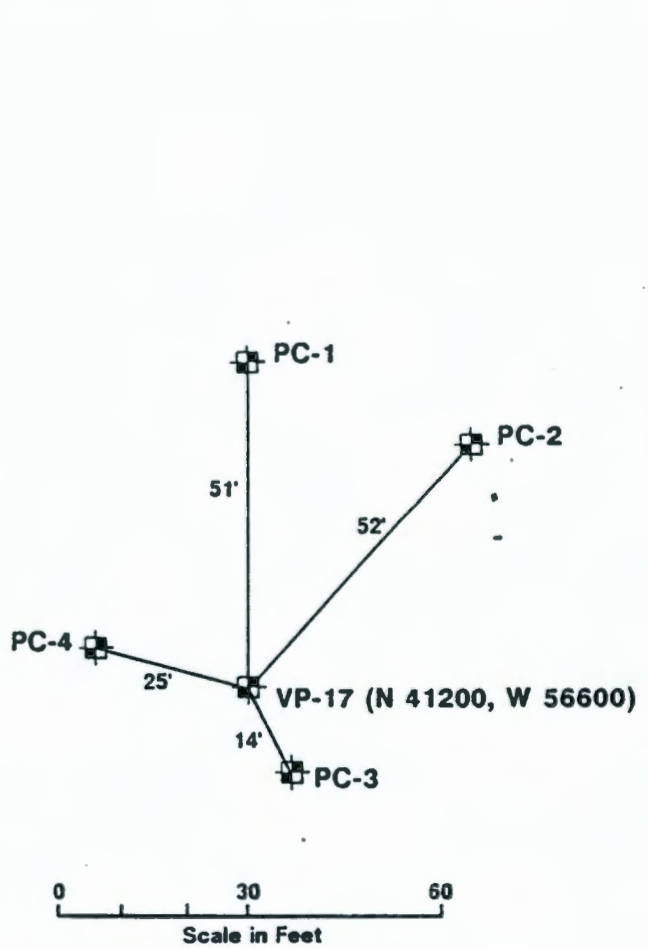
NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

UNIFIED SOIL CLASSIFICATION SYSTEM

November 15, 1989

Dames & Moore

FORM NO. 467.3 (4-78)



Key:

PC-1 Percolation Test Location and Number

Percolation Test No.	Ground Surface Elevation	Bottom of Hole Elevation	Percolation Rate (minutes per inch)
PC-1	714	710.5	2.4
PC-2	714	707.0	2.1
PC-3	714	710.0	2.0
PC-4	714	707.0	3.3

November 15, 1989

Percolation Test Locations and Results
Hanford Waste Vitrification Plant
Dames & Moore

Plate A-18

9011188110040

APPENDIX B

LABORATORY TESTING PROGRAM

A laboratory testing program was conducted to establish pertinent physical and engineering characteristics of soils encountered in the exploration phase of the project. The program consisted of index property tests such as moisture content, density, particle size analysis tests. Specific tests to determine strength, deformability, compaction and collapse characteristics were also conducted and are described below. Moisture content and natural density test results are included in the boring logs adjacent to the appropriate sample location. All other test results are presented in the summary tables in Plates B-1 through B-3.

Particle size (sieve) analysis tests were conducted on samples in order to assist in classifying the soils and to provide a basis for estimating engineering performance such as strength, deformability, collapse potential and erodibility. This data is also useful for estimating permeability. Curves describing the particle size gradation of tested samples are presented in Plates B-4 through B-13.

Strength parameters for foundation soils encountered were obtained using both direct shear and triaxial compression tests performed on slightly disturbed samples. Direct shear tests were conducted on samples from shallow depths to provide data for the analysis of the stability of natural slopes as well as for the analysis of foundation bearing capacity. Samples in the range from Elevation 700 to 650 were tested in triaxial compression primarily to provide deformability estimates for the analysis of foundation settlements. The tests were conducted under consolidated drained conditions at the natural moisture content of the sample, and included several unload-reload cycles to better define the true elastic modulus. Stress versus strain curves for these tests are presented on Plates B-16 to B-20.

Compaction characteristics of soil from Stratum 1 and Stratum 3 were examined by conducting modified Proctor compaction tests in accordance with ASTM D-1557. This information was obtained to provide a basis for compacted fill recommendations, estimates of loads applied by compacted backfill, and estimates of earth pressures. The tests results are plotted on Plates B-14 and B-15.

Collapse tests were performed on one slightly to moderately disturbed sample and 2 recompacted samples of silt and fine sand from Stratum 1. Two of the tests were conducted by standard methods using a consolidometer, and one test on a recompacted sample was conduced in the compaction mold. All samples were loaded to overburden stress levels prior to inundation. The test results are summarized on Plate B-21.

The final section of this appendix includes raw data sheets from selected laboratory tests.

9 0 1 1 8 3 1 1 0 4 1

Boring	Depth (ft)	Soil Type	Dry Density (pcf)	Moisture Content (%)	% Passing #200 Sieve	DIRECT SHEAR TEST (1) Normal Stress (psf)	DIRECT SHEAR TEST (1) Peak Shear Stress (psf)	TRIAXIAL COMPRESSION TEST (2) Confining Stress (psf)	TRIAXIAL COMPRESSION TEST (2) Maximum Deviator Stress (psf)	TRIAXIAL COMPRESSION TEST (2) Modulus of Elasticity (ksf)	Other
VP-1	0.5	SM									Compaction (Plate B-14)
VP-2	16-18	SM/SP									" "
VP-6	0-3	SM									Collapse (Plate B-21)
VP-8	0.2	SM			40.6						
	25.5	SM/SP	108	2.3	7.9	1000 2000	1110 1745				
	30.5	SM/SP	98	1.8	5.6						
	35.5	SM/SP	107	9	5.8						
	40.5	SM/SP	102	2.8	6.4			2000	8350	1870 (3)	
	45.5	SM/SP	116	5	10.1						
	45.6	ML	102	20.1	72.2						
	50.5	SM/SP	107	1.8	7.1			4000	16850	1200	
	75.5	SM/SP	106	4.1	8.1						

Summary of Laboratory Tests

Hanford Waste Vitrification Plant

DAMES & MOORE

11/15/89
10805-398-016

Boring	Depth (ft)	Soil Type	Dry Density (pcf)	Moisture Content (%)	% Passing #200 Sieve	DIRECT SHEAR TEST (1) Normal Stress (psf)	DIRECT SHEAR TEST (1) Peak Shear Stress (psf)	TRIAXIAL COMPRESSION TEST (2) Confining Stress (psf)	TRIAXIAL COMPRESSION TEST (2) Maximum Deviator Stress (psf)	TRIAXIAL COMPRESSION TEST (2) Modulus of Elasticity (ksf)	Other
VP-9	5.5	SM	95	5.2							Collapse (Plate B-21)
VP-10	5 10	SM SM/GM	94	3	18.6 6.2			1000	4350	360	
VP-11	6	SM	101	2.2	23.5			2000	7490	1080 (3)	
VP-14	15.5 30.5	SM/SP SM/SP	115 117	10 10.5		500 1000 2000 1000 2000	610 1220 2290 1160 2280				
VP-17	3.5	SM						1500	7200	430	

Summary of Laboratory Tests

Hanford Waste Vitrification Plant

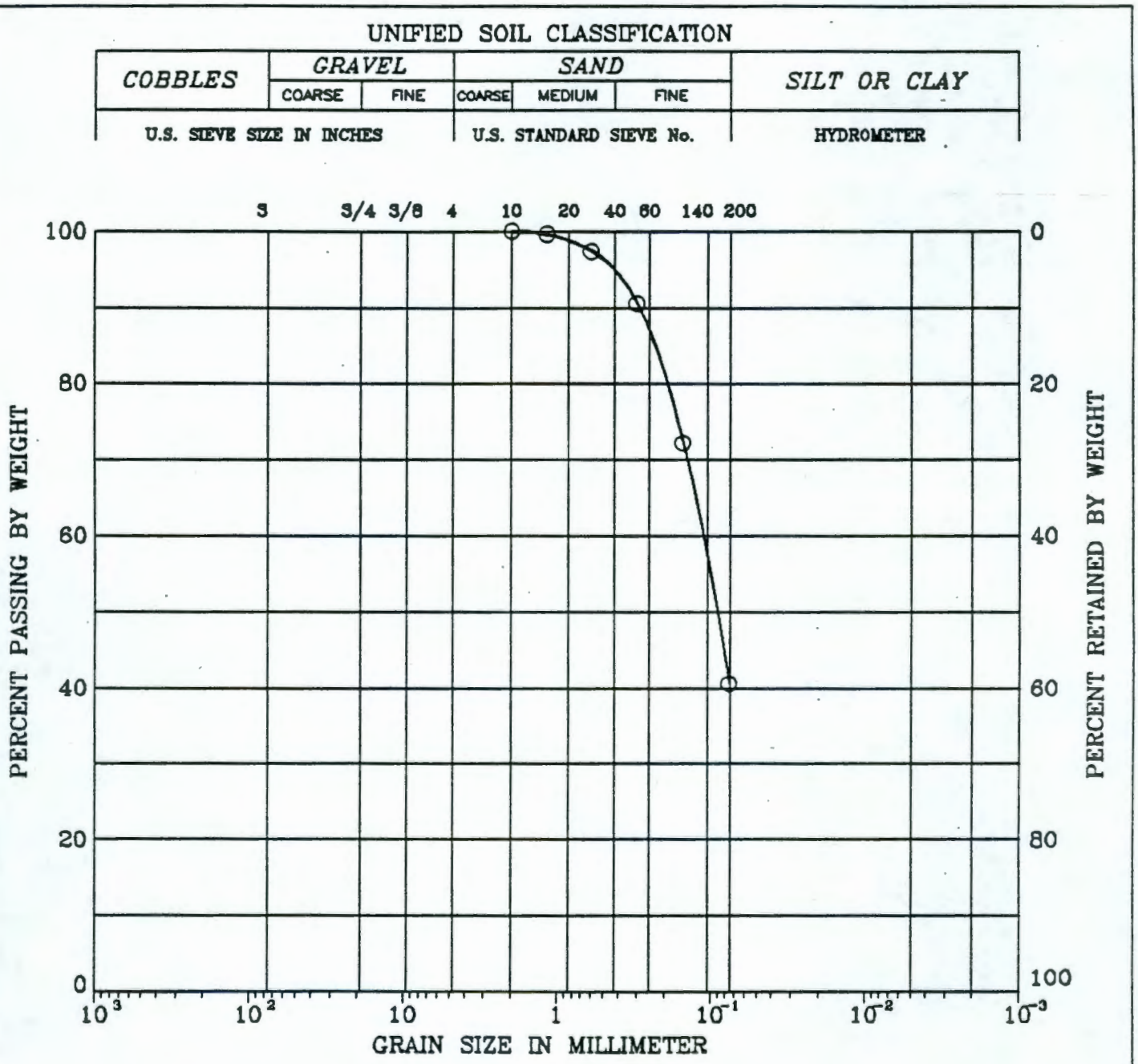
DAMES & MOORE

Notes: 1) Moisture was added to samples tested in direct shear prior to transfering sample to device, in order to maintain intact condition
2) consolidated drained conditions
3) measured using reload cycle (all others measured as initial tangent modulus)

11/15/89
10805-398-016

Boring No.	Sample Depth (ft)	Soil Type	Resistivity (ohm-cm)	Moisture (%)	Sulfides (ppm)	Sulfates (ppm)	Chlorides (ppm)	pH		Redox Potential (mv)
								field	lab	
VP-2	4.5	SM	129 000	3.4	negative	8.2	1.4	8.5	7.9	247.2
VP-3	5.0	SM	55 900	2.6	negative	39.1	66.5	9.3		213.6
VP-9	30.0	SM/SP	132 000	2.3	negative	196	12.1	8.6	8.3	221.8
VP-11	10.0	SM/GM	2.6	2.6	negative	280	44.5	8.3		213.1
VP-14	10.0	SM/GM	748 000	1.4	negative	12.4	9.2	9.2		205.3

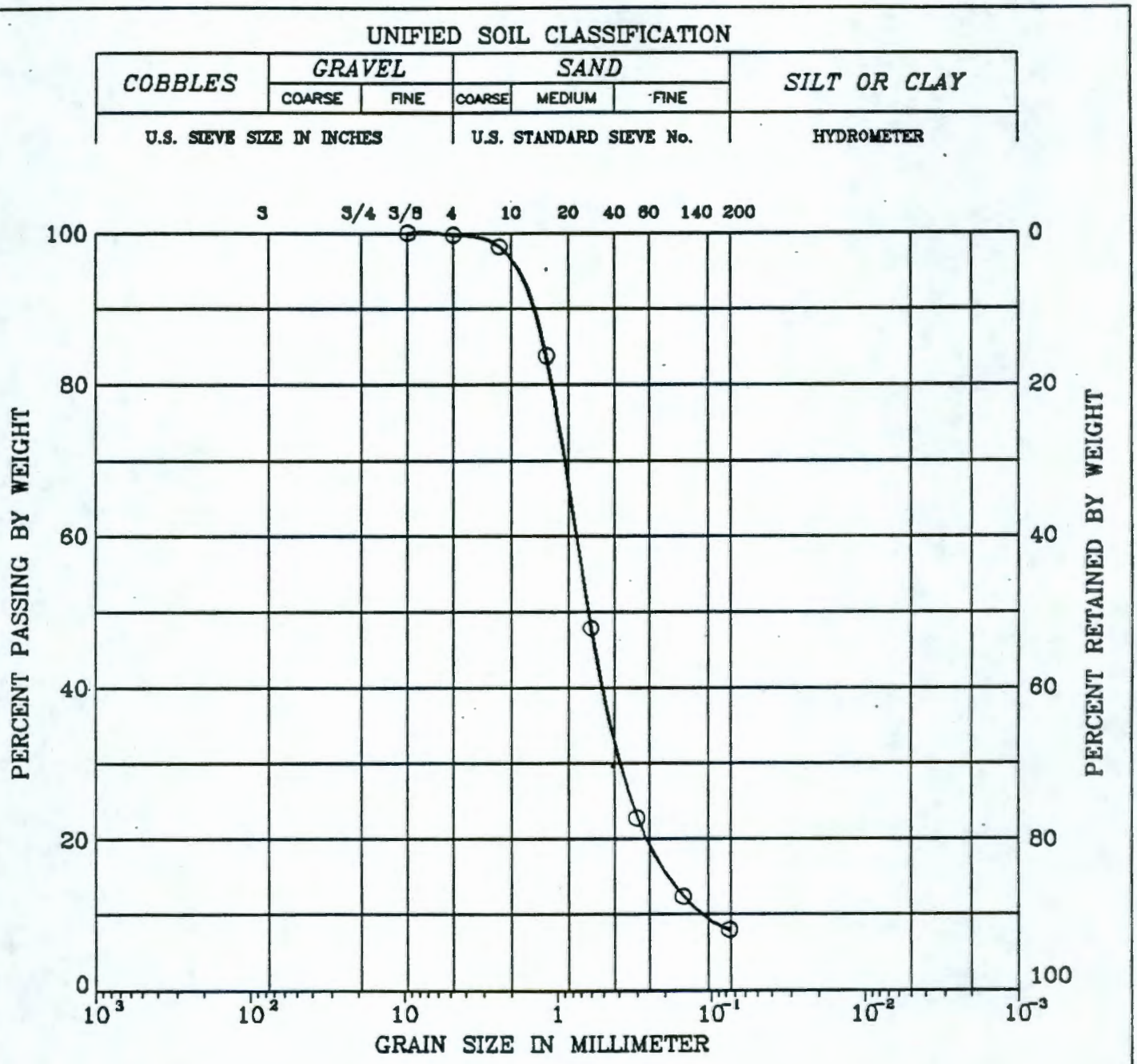
Soil Corrosivity Test Results
Hanford Waste Vitrification Plant



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	VP-8	0.2			SILTY SAND (SM)

Remark :

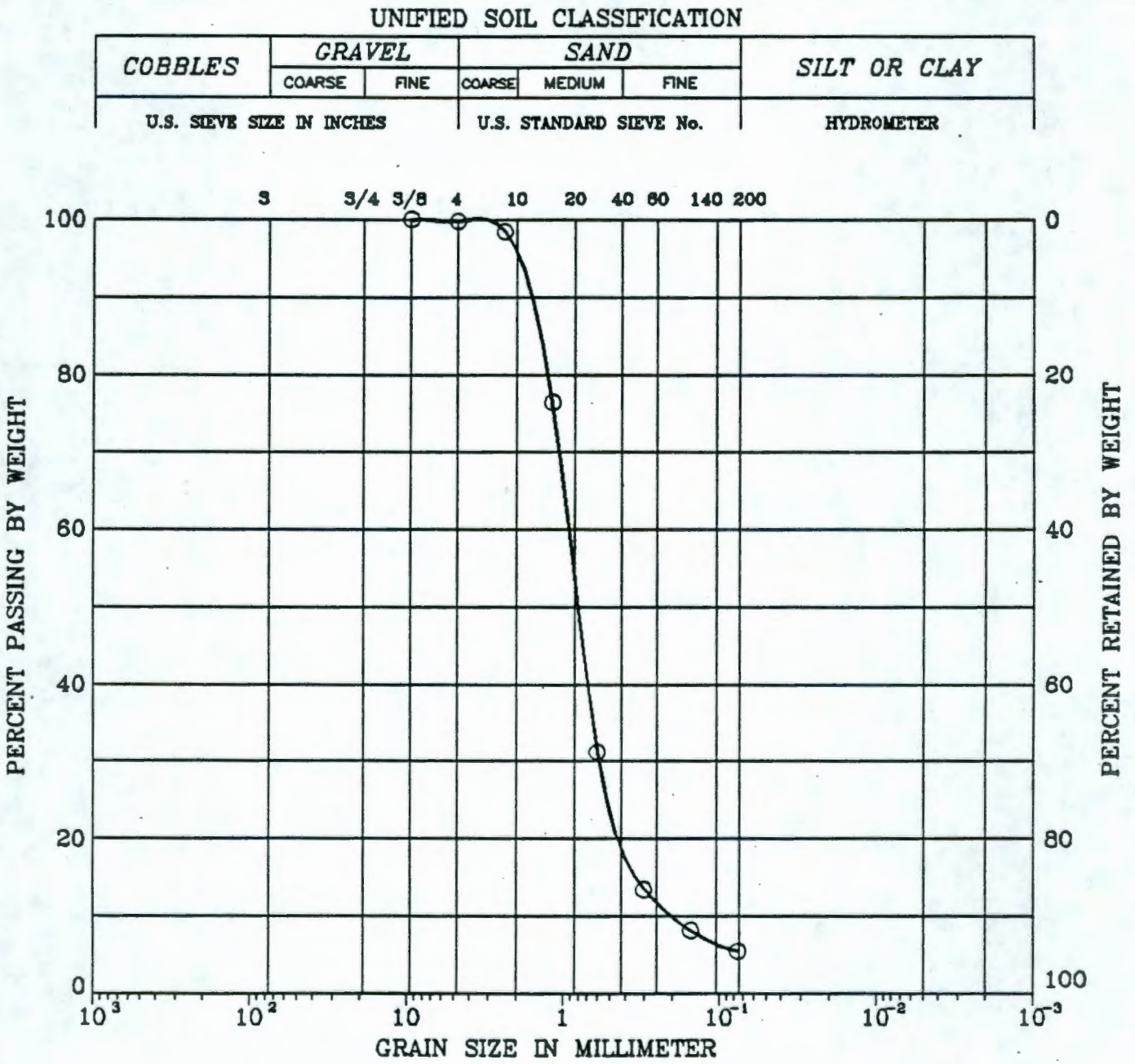
10805-398-016	Hanford Waste Vitrification Plant
Dames & Moore November 15, 1989	GRAIN SIZE DISTRIBUTION Plate No.B- 4



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	VP-8	25.5			SAND (SW)

Remark :

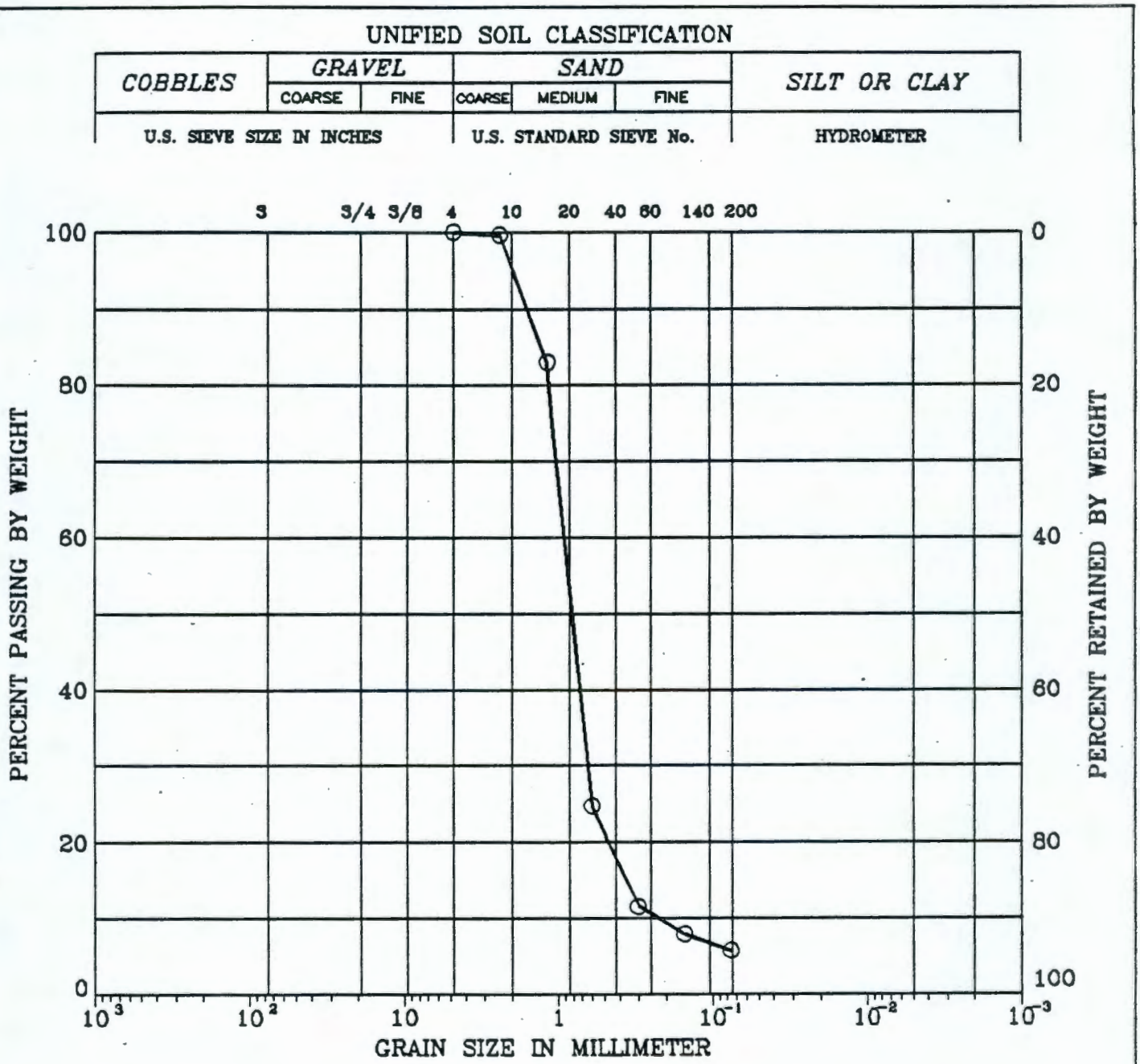
10805-398-016	Hanford Waste Vitrification Plant
Dames & Moore November 15, 1989	GRAIN SIZE DISTRIBUTION Plate No.B- 5



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	VP-8	30.5			SAND (SP)

Remark :

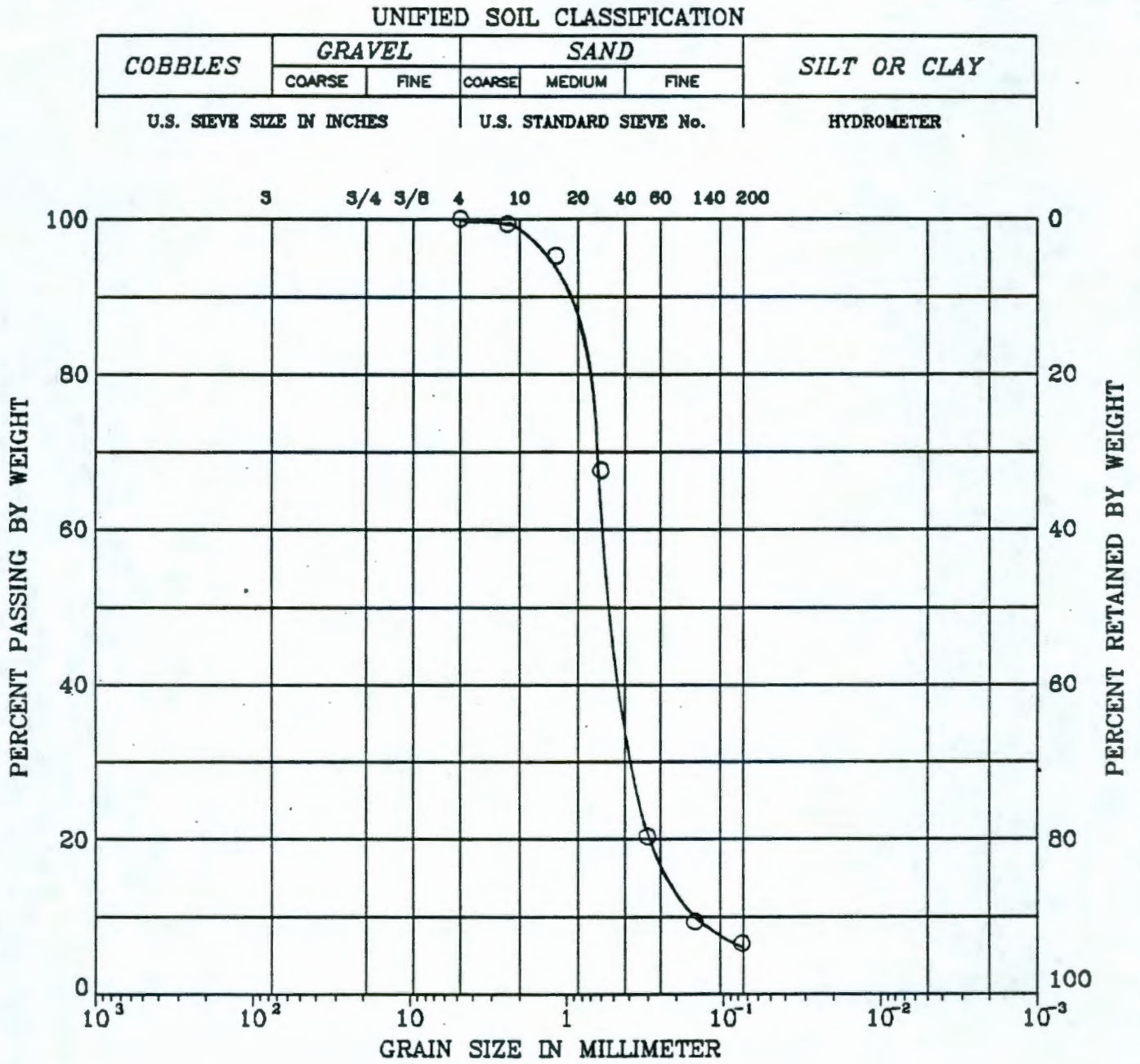
10805-398-016	Hanford Waste Vitrification Plant
Dames & Moore November 15, 1989	GRAIN SIZE DISTRIBUTION Plate No.B- 6



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	VP-8	35.5			SAND (SP)

Remark :

10805-398-016	Hanford Waste Vitrification Plant
Dames & Moore November 15, 1989	GRAIN SIZE DISTRIBUTION Plate No.B- 7

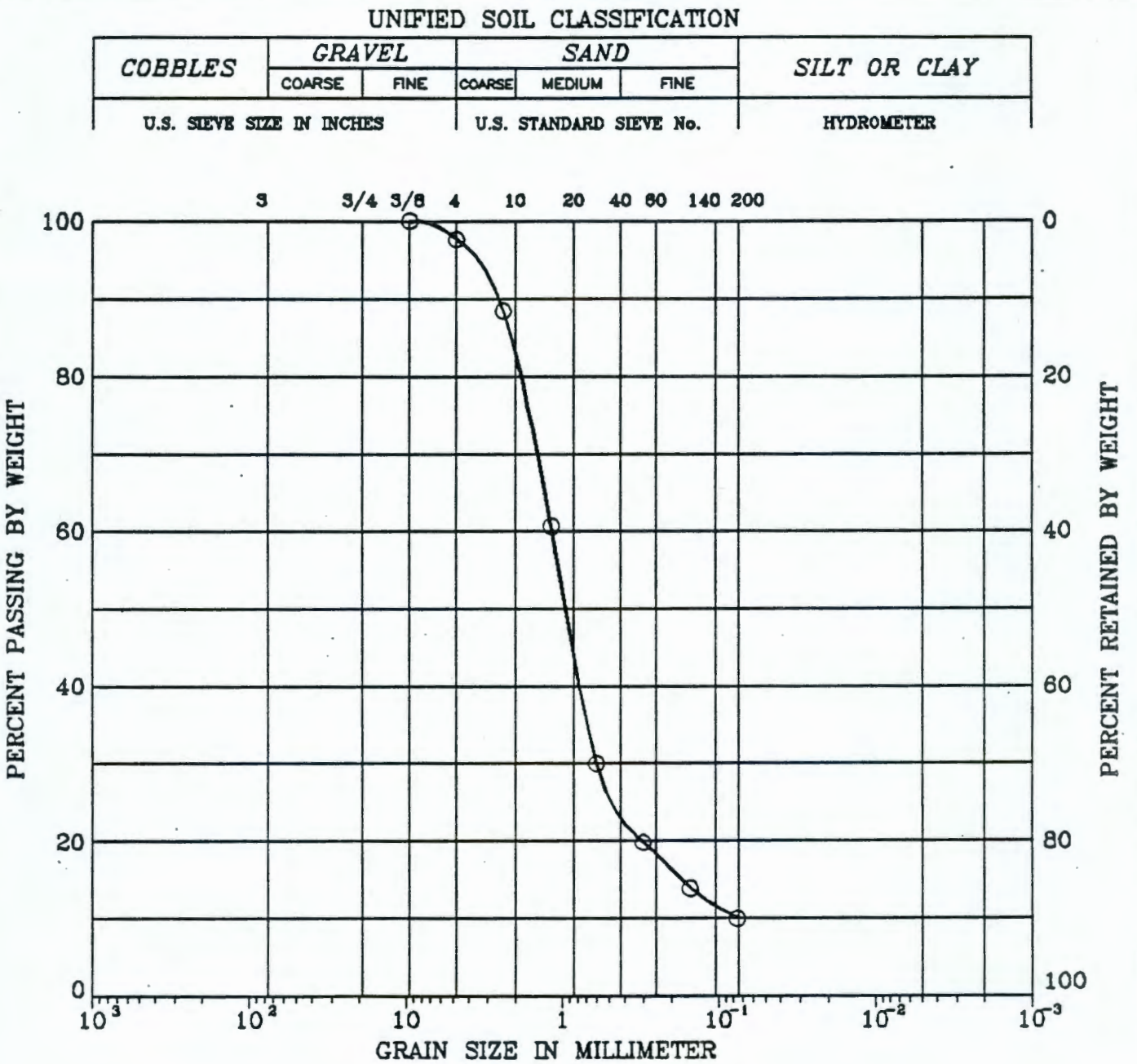


SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	VP-8	40.5			SAND (SP)

Remark :

10805-398-016	Hanford Waste Vitrification Plant
Dames & Moore November 15, 1989	GRAIN SIZE DISTRIBUTION Plate No.B- 8

9011138110049

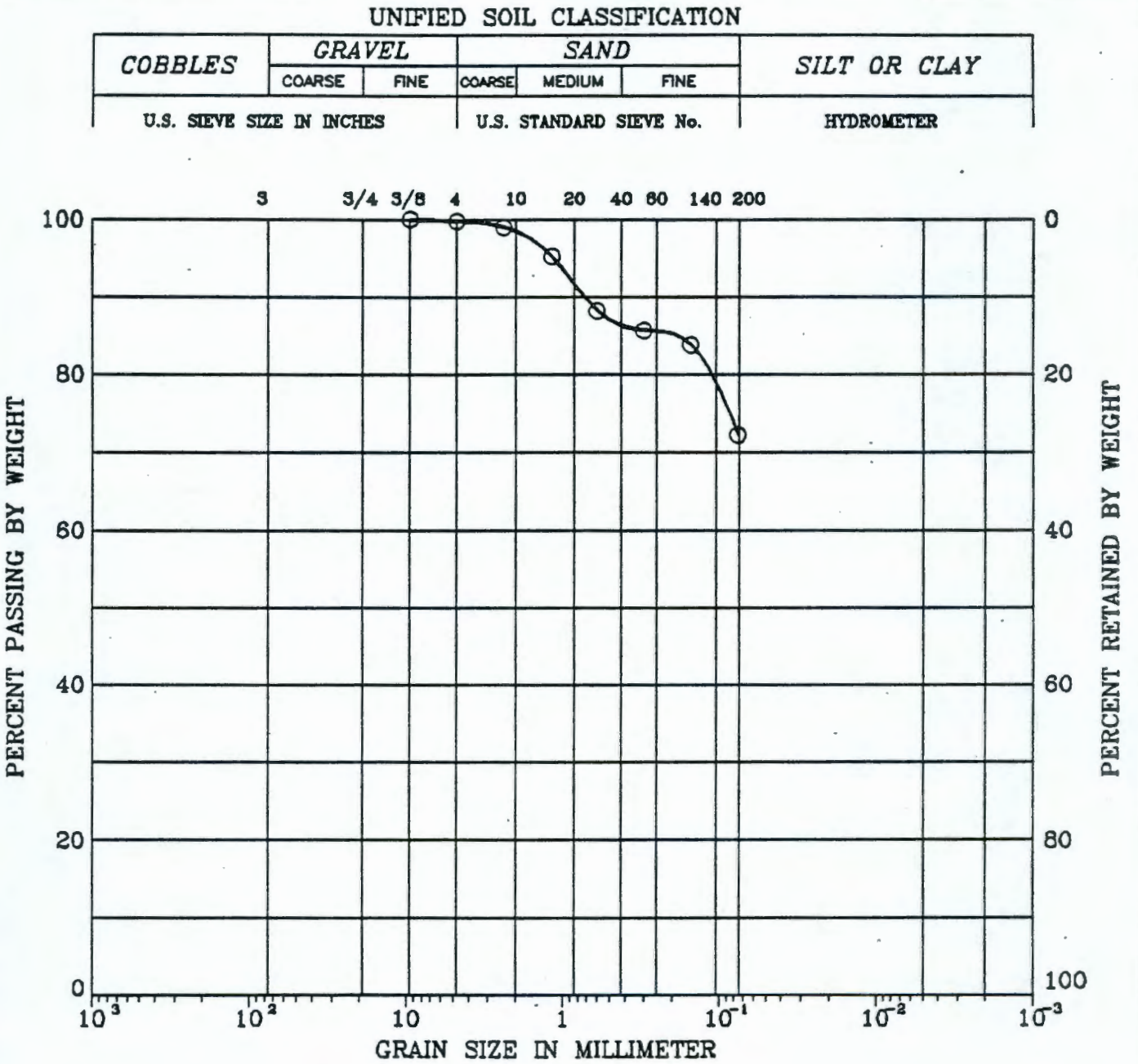


SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	VP-8	45.5			SAND (SM)

Remark : Test result of top 2 rings of sample No.10

10805-398-016	Hanford Waste Vitrification Plant
Dames & Moore November 15, 1989	GRAIN SIZE DISTRIBUTION Plate No.B- 9

9011181105 0

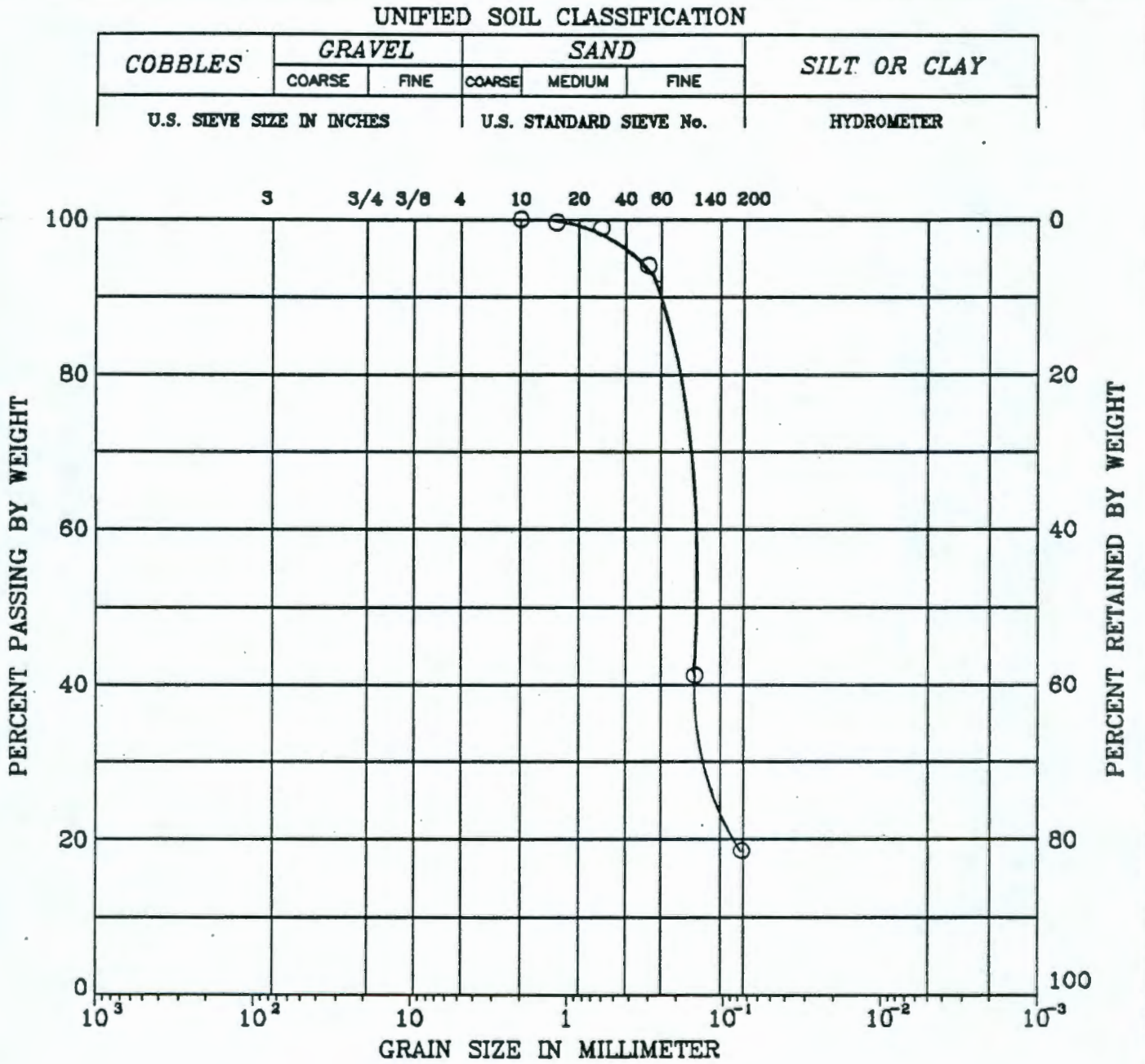


SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	VP-8	45.5			SILT WITH SOME SAND (ML)

Remark : Test result of bottom 2 rings of sample No.10

10805-398-016	Hanford Waste Vitrification Plant
Dames & Moore November 15, 1989	GRAIN SIZE DISTRIBUTION Plate No.B-10

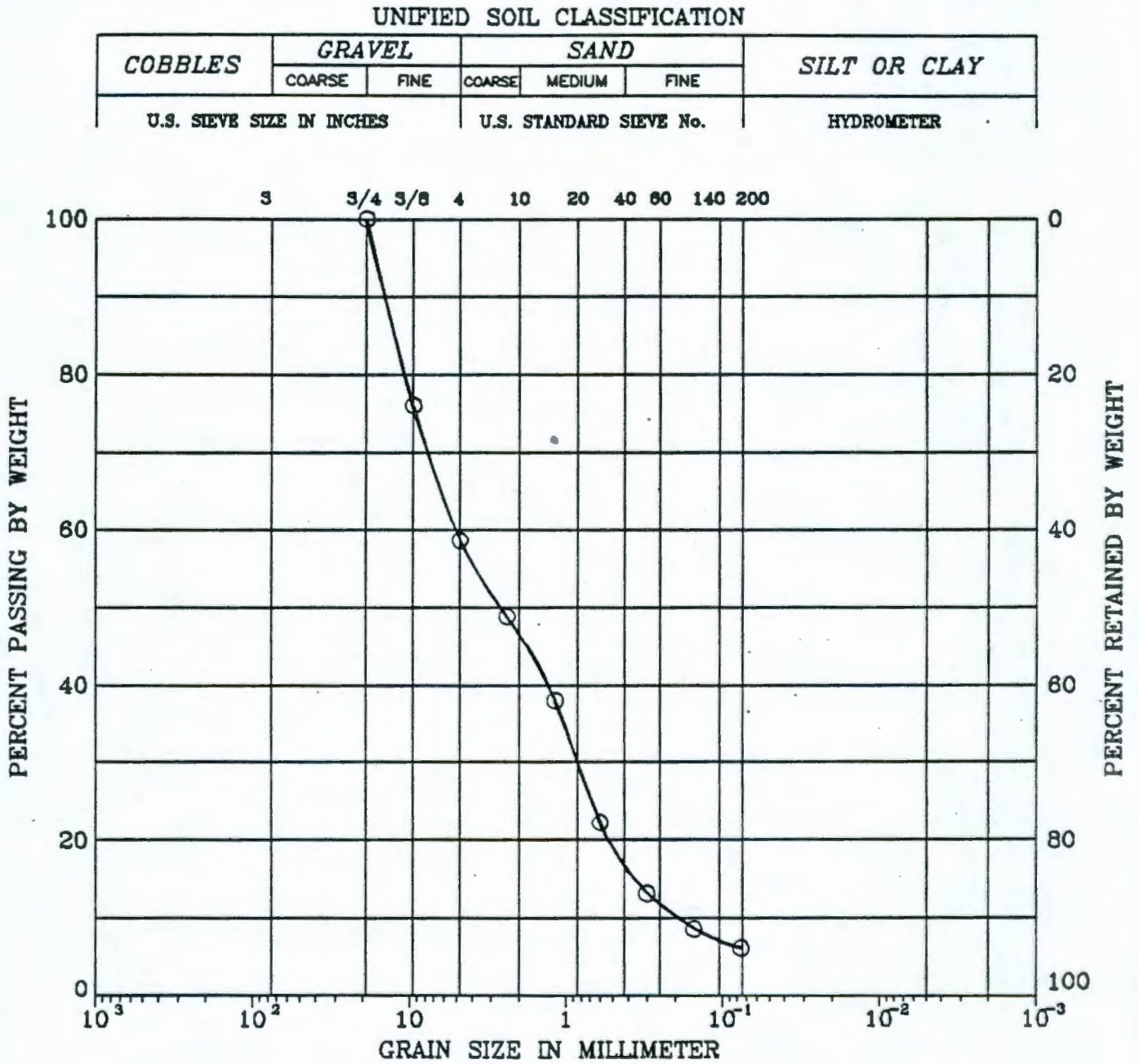
9011838110051



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	VP-10	5			SILTY SAND (SM)

Remark :

10805-398-016	Hanford Waste Vitrification Plant
Dames & Moore November 15, 1989	GRAIN SIZE DISTRIBUTION Plate No.B- 11

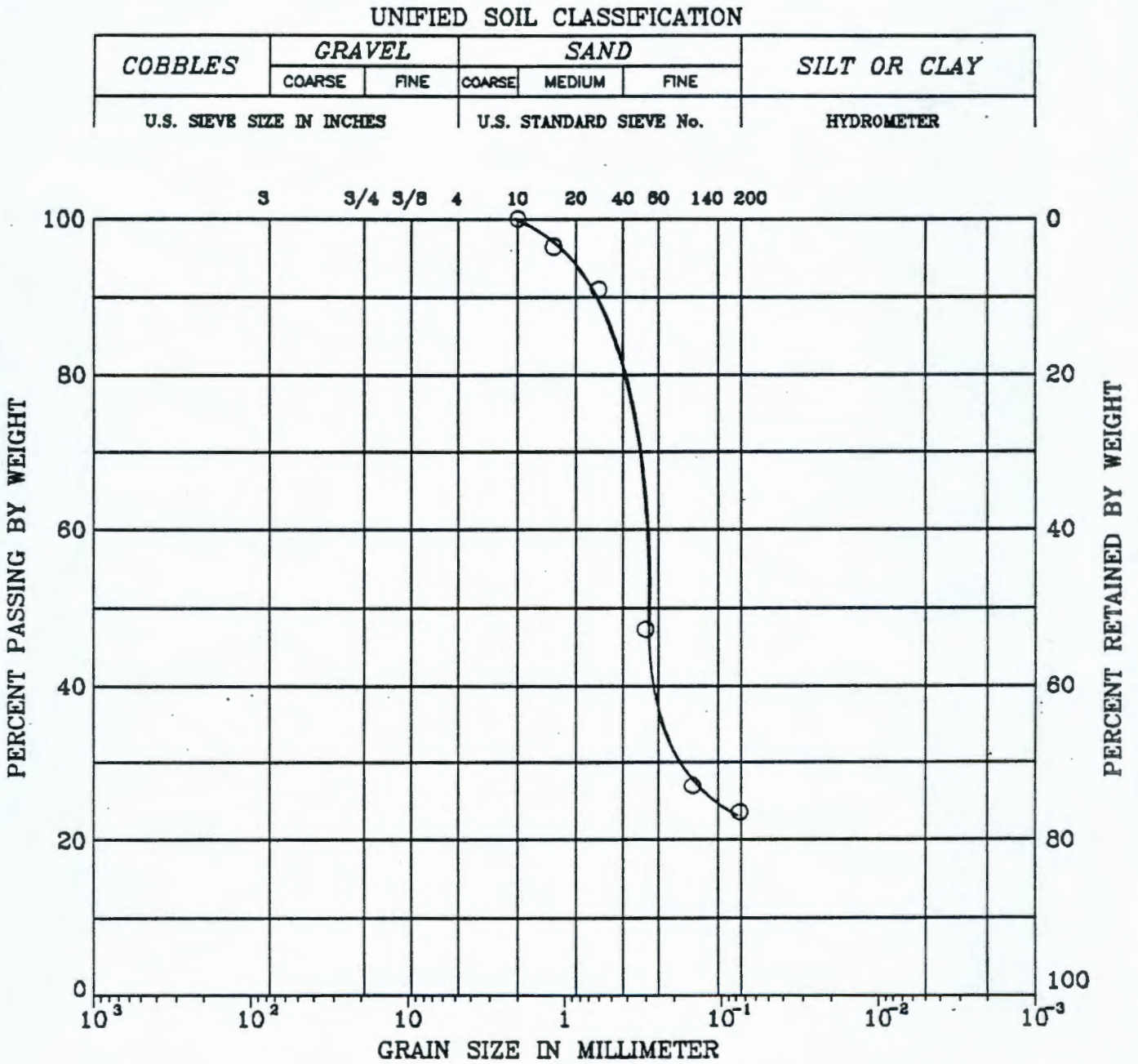


SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
O	VP-10	10			GRAVELLY SAND (SM/GM)

Remark : Test result only on portion finer than 3/4"

10805-398-016	Hanford Waste Vitrification Plant
Dames & Moore November 15, 1989	GRAIN SIZE DISTRIBUTION Plate No.B-12

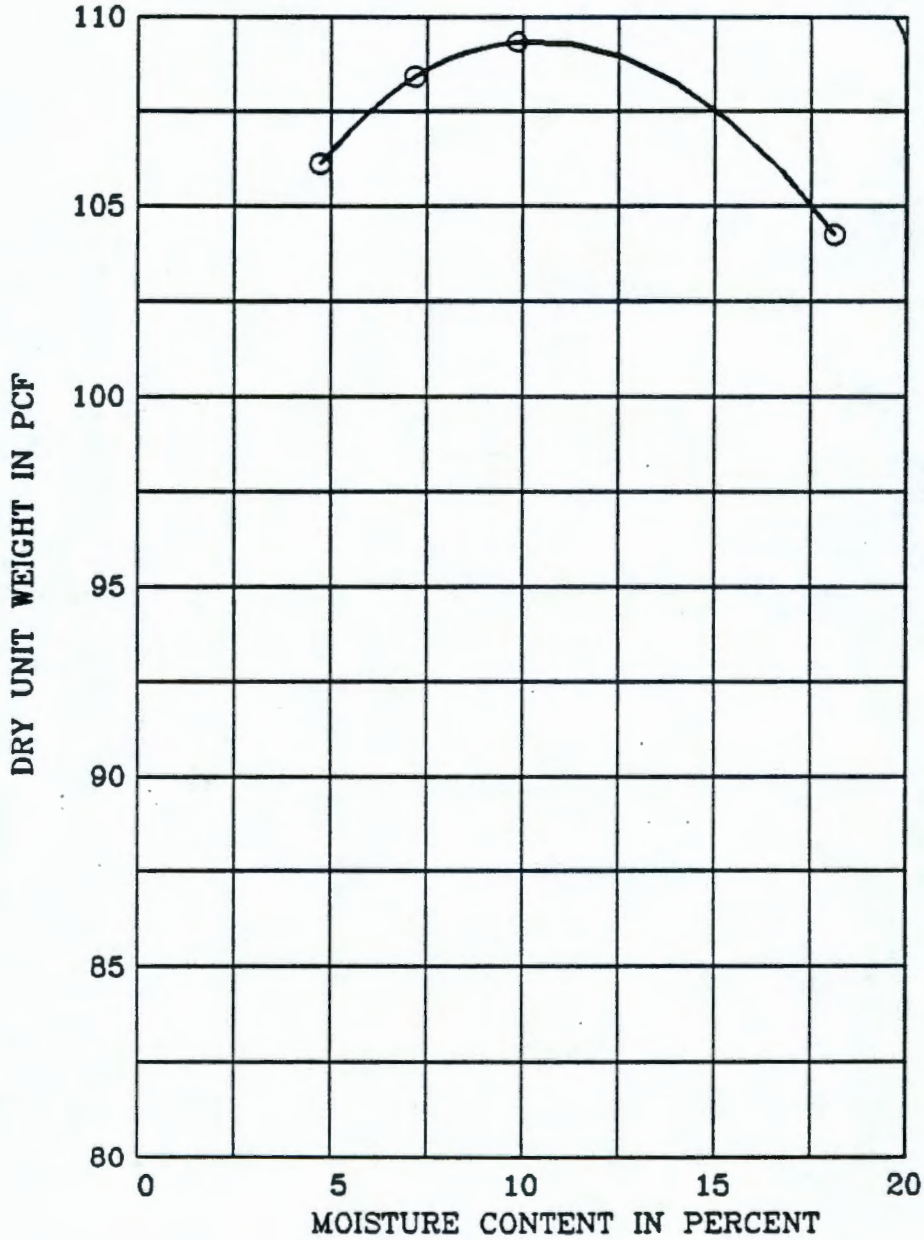
9011381105 3



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	VP-11	6			SILTY SAND (SM)

Remark :

10805-398-016	Hanford Waste Vitrification Plant
Dames & Moore November 15, 1989	GRAIN SIZE DISTRIBUTION Plate No.B- 13

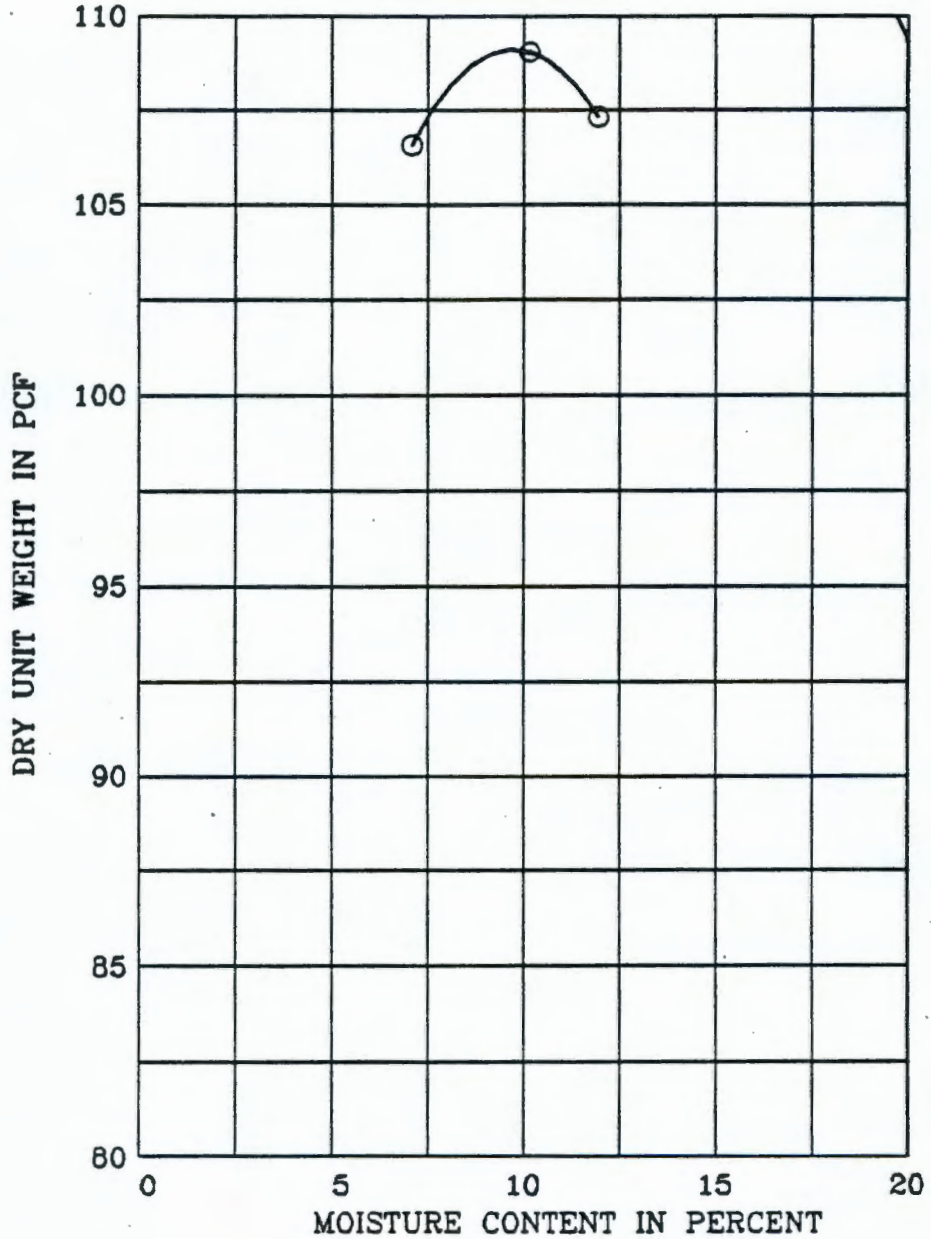


SYMBOL	SAMPLE LOCATION	DEPTH (ft)	DESCRIPTION	TEST METHOD	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
O	VP-1	0-5	SM	Modified	9.9	109.3

Remark :

10805-398-016	Hanford Waste Vitrification Plant
Dames & Moore November 15, 1989	COMPACTION TEST Plate No.B- 14

9011810055



SYMBOL	SAMPLE LOCATION	DEPTH (ft)	DESCRIPTION	TEST METHOD	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
O	VP-2	16-18	SM	Modified	9.7	109.1

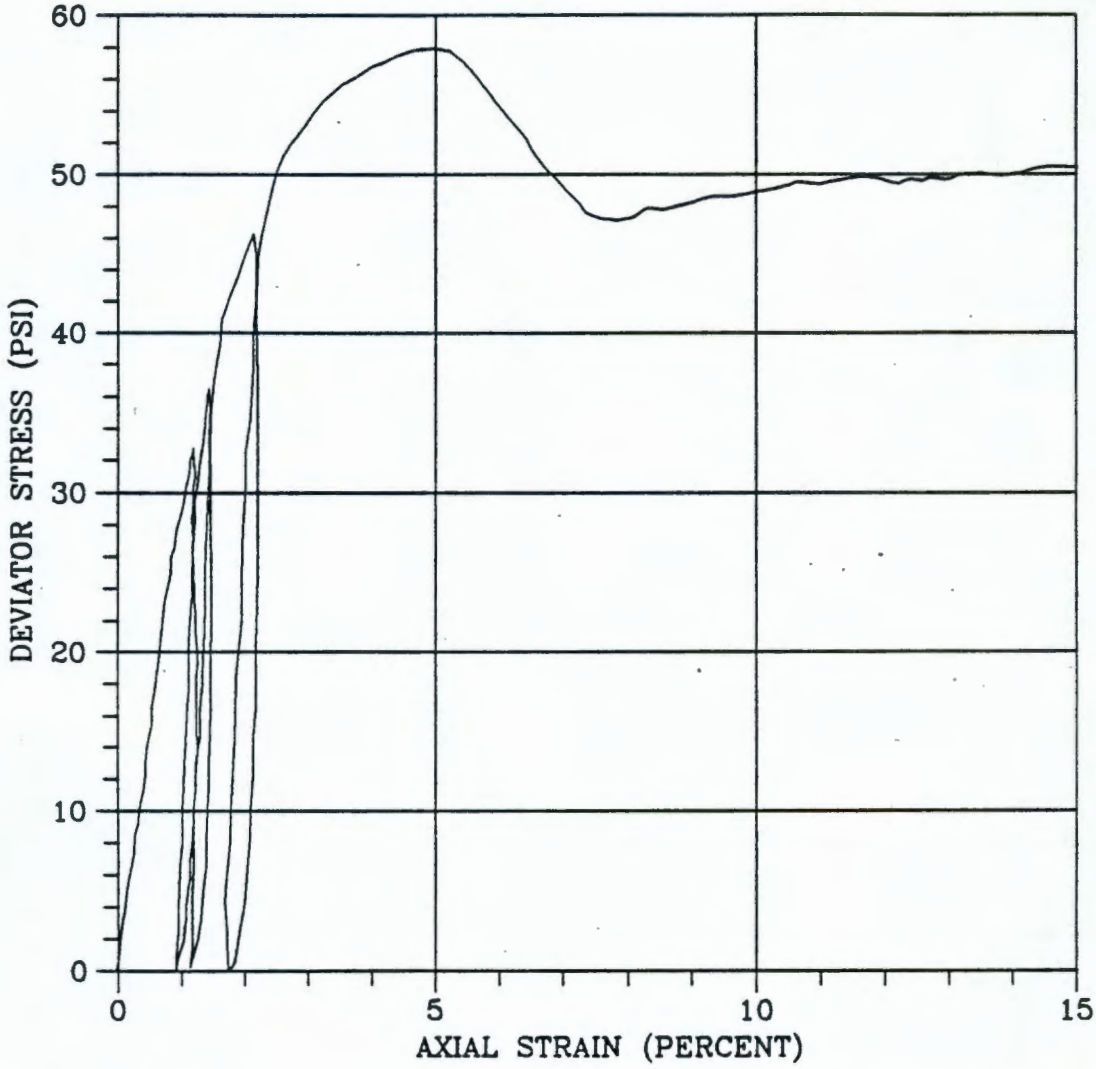
Remark :

10805-398-016	Hanford Waste Vitrification Plant
Dames & Moore November 15, 1989	COMPACTION TEST Plate No.B- 15

9011188110056

CONSOLIDATED–DRAINED TRIAXIAL TEST DATA

CLIENT: KAISER ENGINEERS HANFORD



Boring No. VP-8	Depth(ft)	40.5	Sample No.	9	Initial Hight (in.) 6
SUMMARY OF SAMPLE DATA:					
Moisture Content (%)		2.8			
Wet Density (pcf)		105			
Dry Density (pcf)		102			
SUMMARY OF TEST DATA:					
Confining Pressure (psf)		2000	(13.9 PSI)		
Peak Deviator Stress (psf)		8350	(58 PSI)		
Elastic Modulus (ksf)		1870	(Reload)		
DESCRIPTION:	Fine to Medium Sand, Trace of Silt (Stratum 3)				

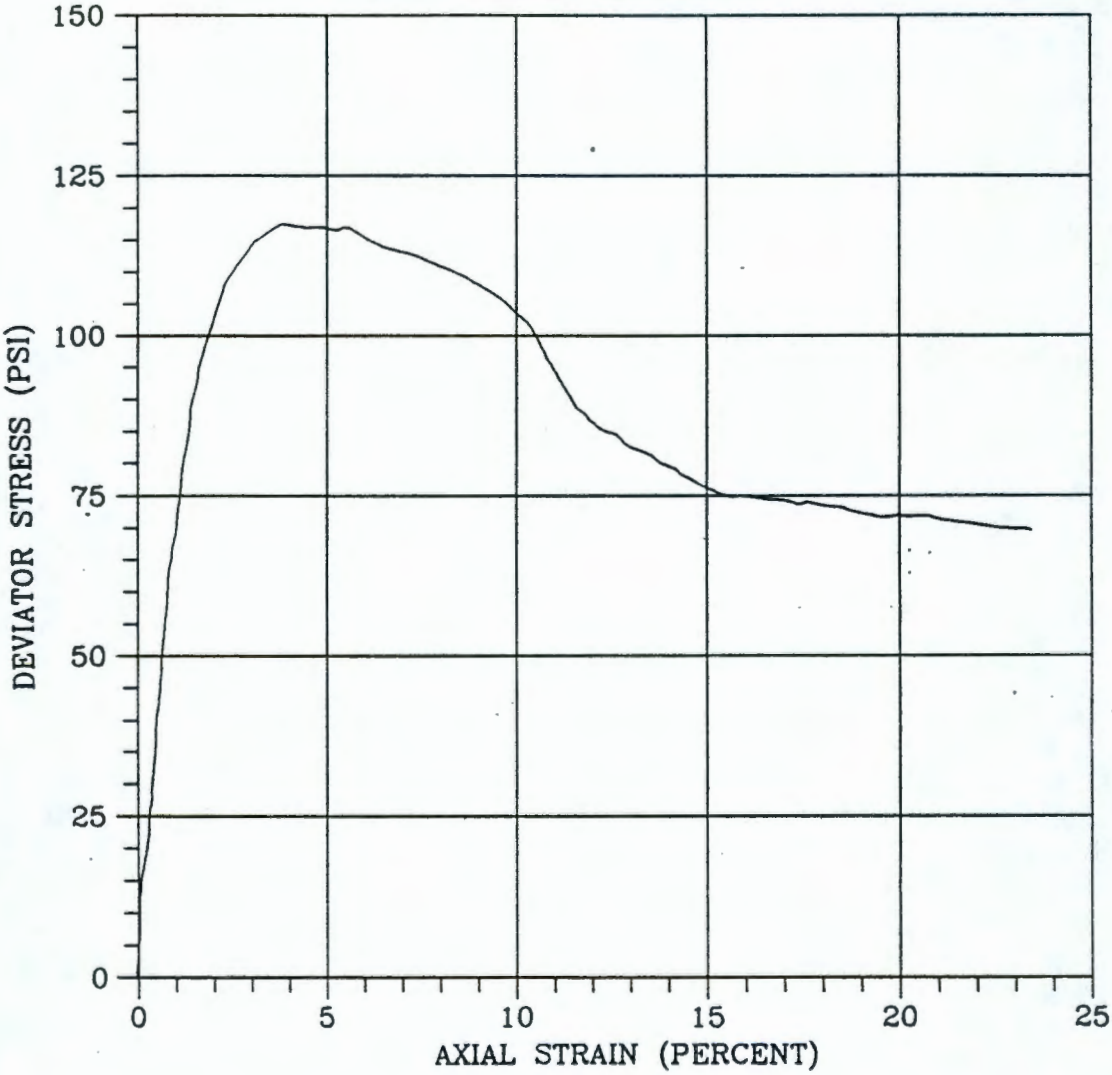
November 15, 1989 DAMES & MOORE

Job No. 10805-398-016 Plate No. B-16

9011831105 7

CONSOLIDATED-DRAINED TRIAXIAL TEST DATA

CLIENT: KAISER ENGINEERS HANFORD



Boring No. VP-8	Depth(ft)	50.5	Sample No.	11	Initial Hight (in.) 6
SUMMARY OF SAMPLE DATA:					
Moisture Content (%)		1.8			
Wet Density (pcf)		109			
Dry Density (pcf)		107			
SUMMARY OF TEST DATA:					
Confining Pressure (psf)		4000	(27.8 PSI)		
Peak Deviator Stress (psf)		16850	(117 PSI)		
Elastic Modulus (ksf)		1200	(8.3 KSI)		
DESCRIPTION:	Fine to Medium Sand, Trace of Silt (Stratum 3)				

November 15, 1989

DAMES & MOORE

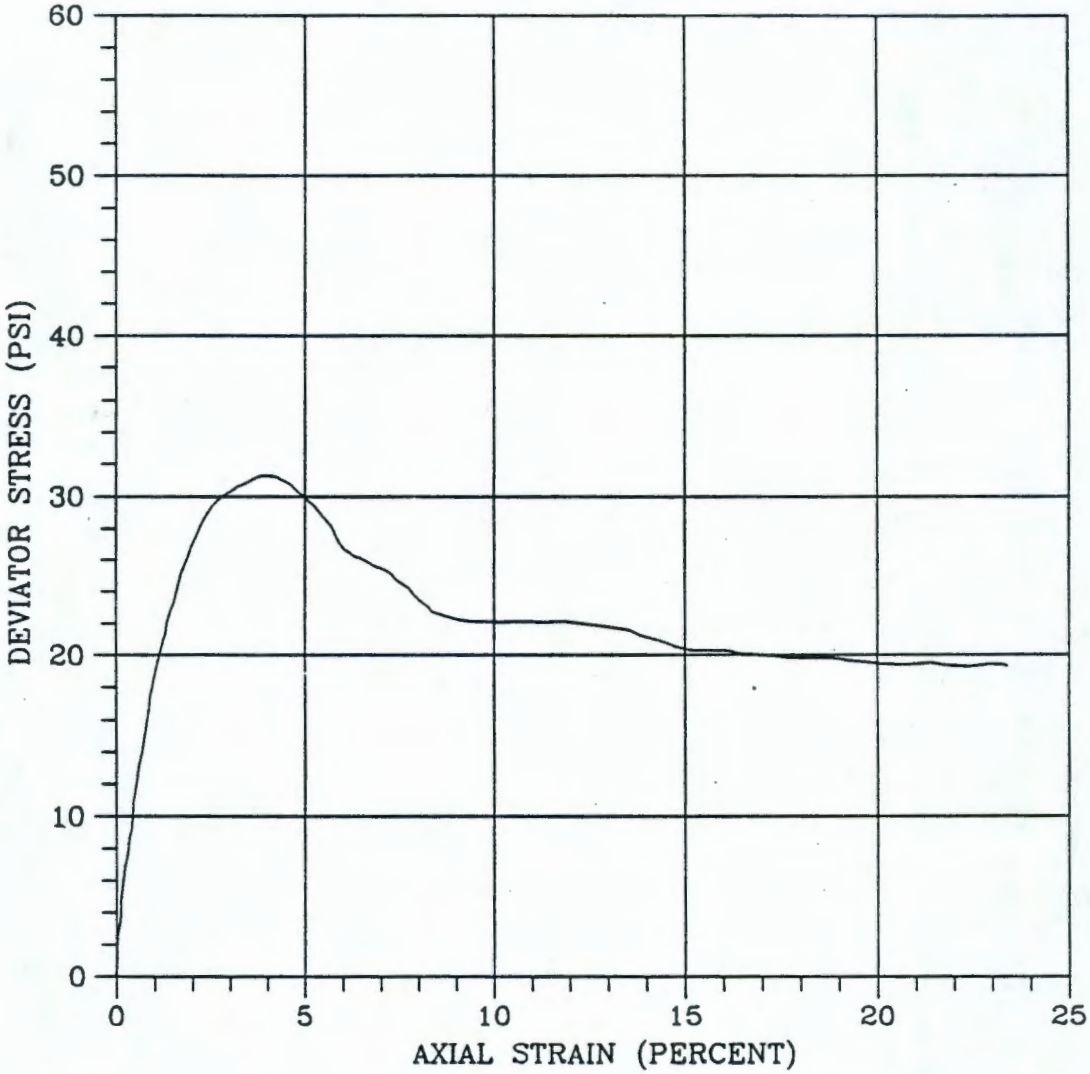
Job No. 10805-398-016

Plate No. B-17

9011381105 8

CONSOLIDATED-DRAINED TRIAXIAL TEST DATA

CLIENT: KAISER ENGINEERS HANFORD



Boring No.	VP-10	Depth(ft)	5	Sample No.	2	Initial Hight (in.)	6

SUMMARY OF SAMPLE DATA:

Moisture Content (%)	3	
Wet Density (pcf)	97	
Dry Density (pcf)	94	

SUMMARY OF TEST DATA:

Confining Pressure (psf)	1000	(6.9 PSI)
Peak Deviator Stress (psf)	4350	(30.2 PSI)
Elastic Modulus (ksf)	360	(2.5 KSI)

DESCRIPTION: Silt and Fine Sand (Stratum 1)

November 15, 1989

DAMES & MOORE

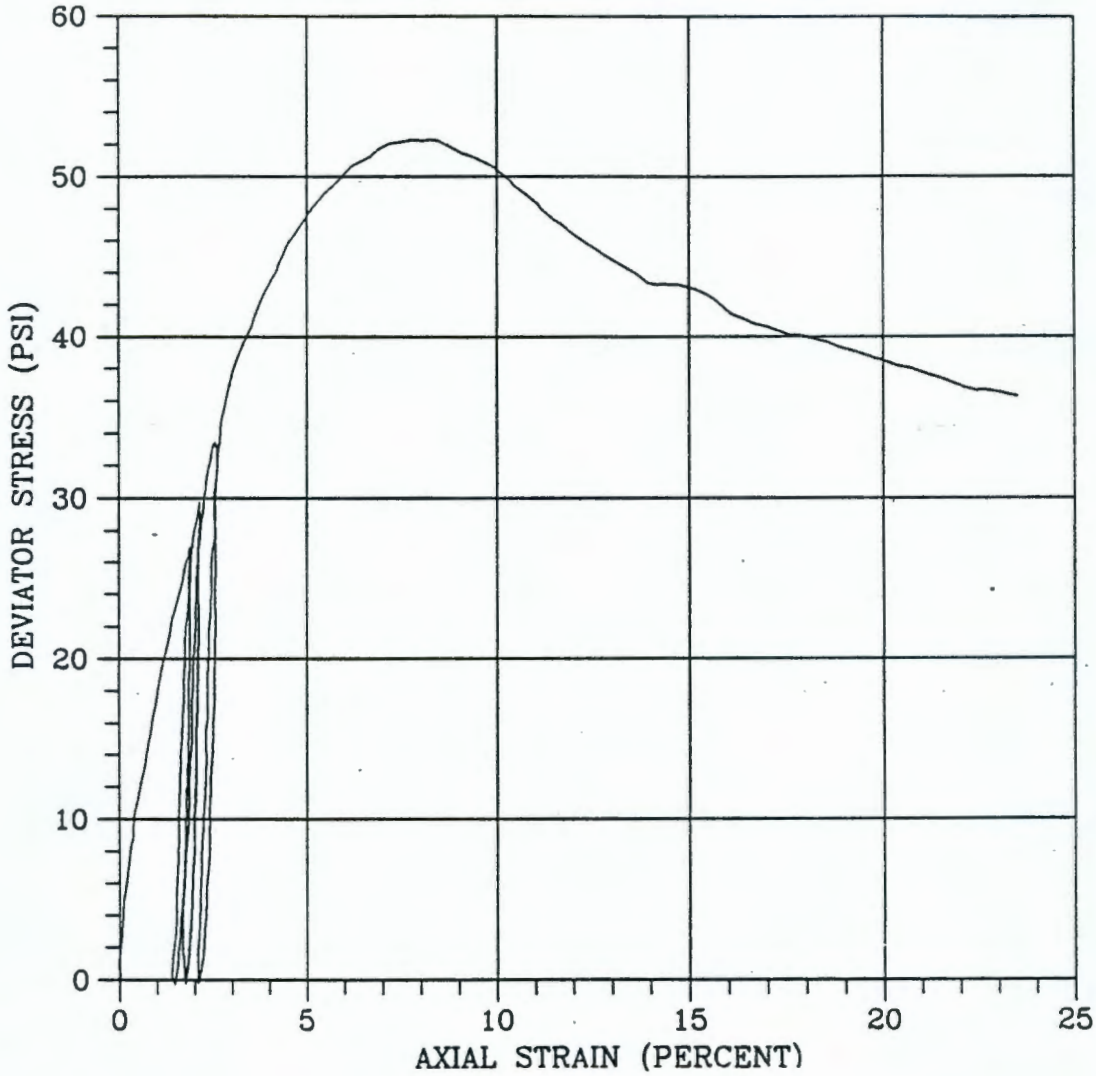
Job No. 10805-398-016

Plate No. B-18

9011198110059

CONSOLIDATED–DRAINED TRIAXIAL TEST DATA

CLIENT: KAISER ENGINEERS HANFORD



Boring No. VP-11	Depth(ft) 6	Sample No. 2	Initial Hight (in.) 6
SUMMARY OF SAMPLE DATA:			
Moisture Content (%)	2.2		
Wet Density (pcf)	103		
Dry Density (pcf)	101		
SUMMARY OF TEST DATA:			
Confining Pressure (psf)	2000	(13.9 PSI)	
Peak Deviator Stress (psf)	7490	(52 PSI)	
Elastic Modulus (ksf)	1080	(Reload)	
DESCRIPTION:	Silt and Fine Sand (Stratum 1)		

November 15, 1989

DAMES & MOORE

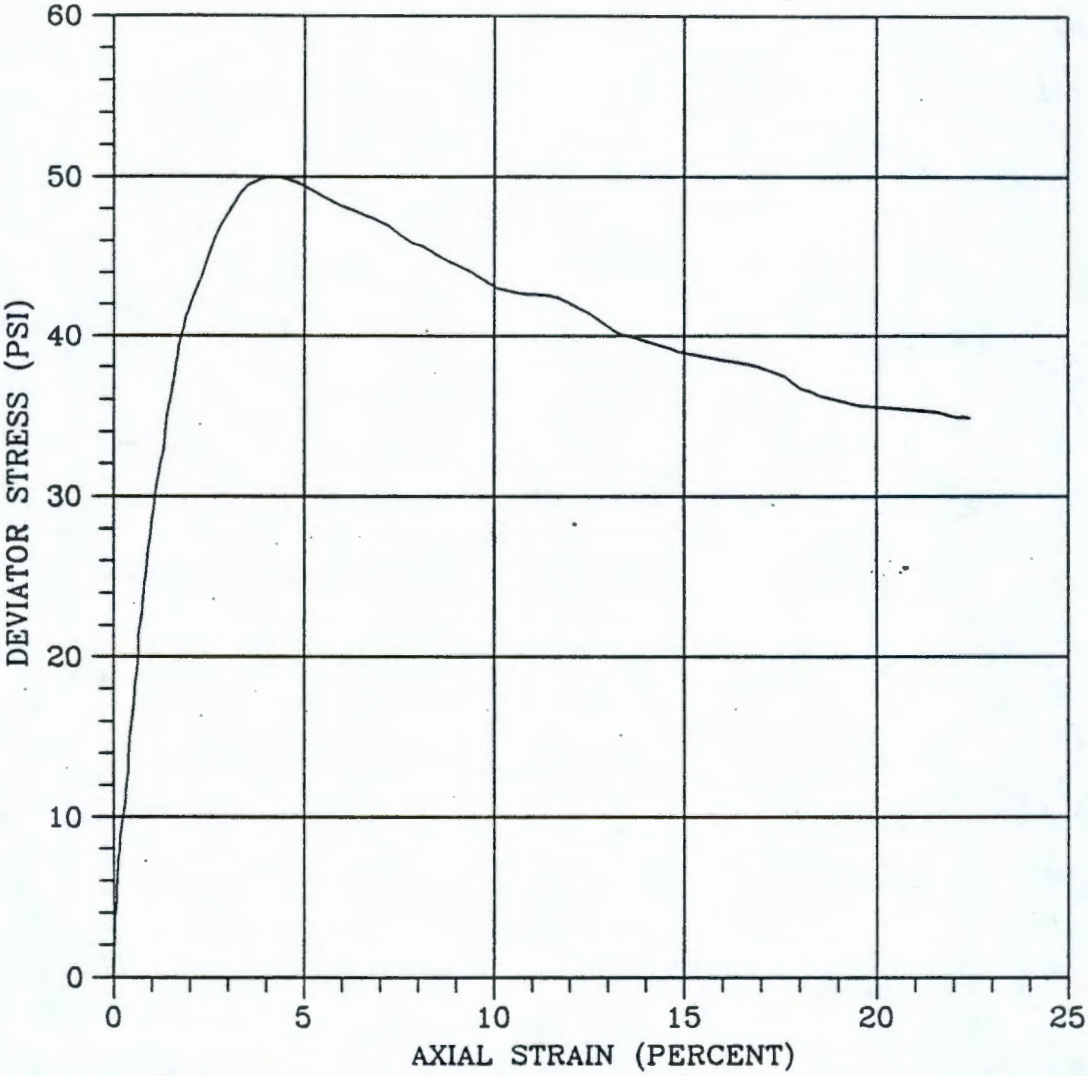
Job No. 10805-398-016

Plate No. B-19

9011188110060

CONSOLIDATED-DRAINED TRIAXIAL TEST DATA

CLIENT: KAISER ENGINEERS HANFORD



Boring No. VP-17	Depth(ft)	3.5	Sample No.	2	Initial Hight (in.) 6

SUMMARY OF SAMPLE DATA:

Moisture Content (%)	2.5
Wet Density (pcf)	106
Dry Density (pcf)	103

SUMMARY OF TEST DATA:

Confining Pressure (psf)	1500	(10.4 PSI)
Peak Deviator Stress (psf)	7200	(50 PSI)
Elastic Modulus (ksf)	430	(3 KSI)

DESCRIPTION: Silt and Fine Sand (Stratum 1)

November 15, 1989 DAMES & MOORE

Job No. 10805-398-016 Plate No. B-20

9011831061

Boring	Sample Depth (feet)	Initial Diameter (Inches)	Initial Height (inches)	Initial Dry Density (pcf)	Initial Moisture Content (%)	% Collapse	Remarks
VP-9	5.5	2.0	1.0	95	5.2	2.58	natural conditions
VP-6	0-3	2.0	1.0	101	10.0	0.26	Recompacted in consolidometer
	0-3	4.0	3.5	103	10.0	0.32	Recompacted in compaction mold

COLLAPSE TEST RESULTS
Hanford Waste Vitrification Plant

Dames & Moore

11/15/89
10805-398-016

Plate: B-21

9011180106 2

ORIGINAL LAB DATA

9 0 1 1 8 8 1 1 0 6 3

Collapse Test (consolidometer)

[redacted] TEST DATA — Collapse Test — Natural Conditions — 8/18/89 – 8/21/89 by JWS

Client:	Kaiser Engr.
Job No:	10805-398
Location:	Hanford
Boring No:	VP-9
Sample No:	2
Depth (ft):	5.5'
Soil Type:	SM
Load Sequence:	Normal load 6' × 110 pcf = 600 psf

Note: Use 1 D&M Ring for test	INITIAL	FINAL
Wet Wt. of Soil + Tare (g)	165.3	187.68
Dry Wt. of Soil + Tare (g)	159.4	159.4
Weight of Tare (g)	45.3	45.3
Length (in)	1	0.93
Diameter (in)*	2.42	
Specific Gravity (est)	—	
Wet Density (pcf)	100.0	127.6
Dry Density (pcf)	95.1	102.2
Moisture Content (%)	5.2	24.8

*D&M Sample = 2.416 in

Time/Comment	Load (psf)	Reading (×0.0001")	Time/Comment	Load (psf)	Reading
8/18 16:10	0				
16:10		1923			
16:15	600				
16:20		1525			
16:30		1511			
16:45		1500			
17:15		1498			
18:21		1497			
8/19 9:05		1496	→ 0.0427		
9:15	Start Saturation				
9:15		1330			
9:16		1320			
10:05		1302			
8/21 8:10		1249			
9:10		1249	→ 0.0674		

$$\% \text{ Collapse} = \frac{0.0674 - 0.0427}{1.0 - 0.0427} \times 100\% = 2.58\%$$

Dames & Moore

9011838110 64

Collapse Test (compaction mold)

CONSOLIDATION TEST DATA Collapse Test Recompacted 10/23 — 10/31/89 by JWS

Client: Kaiser Engr	
Job No: 10805 - 398	
Location: Hanford	
Boring No: VP-6	
Sample No:	
Depth (ft): 0 – 3'	
Soil Type: SM	
Load Sequence: Normal load 6' x 100 pcf = 600 psf	

Note: Use 1 D&M Ring for test	INITIAL	FINAL
Wet Wt. of Soil + ~~Tare~~ (g)	3.795# = 1720.5(g)	
Dry Wt. of Soil + ~~Tare~~ (g)	3.45# = 1564.9(g)	
Weight of ~~Tare~~ water (g)	0.345# = 156.5(g)	
Length (in)	4.58"	
Diameter (in)*	4"	
Specific Gravity (est)	—	
Wet Density (pcf)	113.9	
Dry Density (pcf)	103.6	
Moisture Content (%)	10%	

*D&M Sample = 2.416 in

Time/Comment		~~Load (psf)~~	Reading	Time/Comment		Load (psf)	Reading
Time	load	Reading left	Reading right				
	(psf)	(0.001")	(0.001")				
10/23 14:00 PM	0	950	950	= 950.0			
10/23 14:00	600	876	887				
14:10		881	866				
10/24 9:45		877	854				
AM 10:45		877.0	854.0	= 865.5 → 0.0845			
10:45	Start Saturation						
11:05		869	849				
[illegible]		862	849				
10/25 12:12 PM		862	848				
4:13		862	848				
10/26 8:25		[illegible]	847.3				
4:30		[illegible]	847				
10/27 [illegible]		859.5	847				
10/30 15:30	PM	857.1	845.1				
10/31 9:00	AM	857.1	845.1	= 851.1 → 0.0989			

$$\% \text{ collapse} = \frac{0.0989 - 0.0845}{4.58 - 0.0845} = 0.32\%$$

Dames & Moore

9011188110065

Collapse Test (consolidometer)

CONSOLIDATION TEST DATA Collapse Test Recompacted 10/23/89 — 10/31/89 by JWS

Client:	Kaiser Engr
Job No:	10805-398
Location:	Hanford
Boring No:	VP-6
Sample No:	
Depth (ft):	0-3'
Soil Type:	SM
Load Sequence:	Normal load 6'x100 pcf = 600 psf
	Tested by JWS

Note: Use 1 D&M Ring for test	Designed INITIAL	Actual initial ~~FINAL~~
Wet Wt. of Soil + Tare (g)	154.36	226.4
Dry Wt. of Soil + Tare (g)	141.96	198.38
Weight of Tare (g)	17.66	76.88
Length (in)	1	1
Diameter (in)*	2.42	2.42
Specific Gravity (est)	—	—
Wet Density (pcf)	113.9	124.6
Dry Density (pcf)	103.6	101.3
Moisture Content (%)	10%	10%

*D&M Sample = 2.416 in

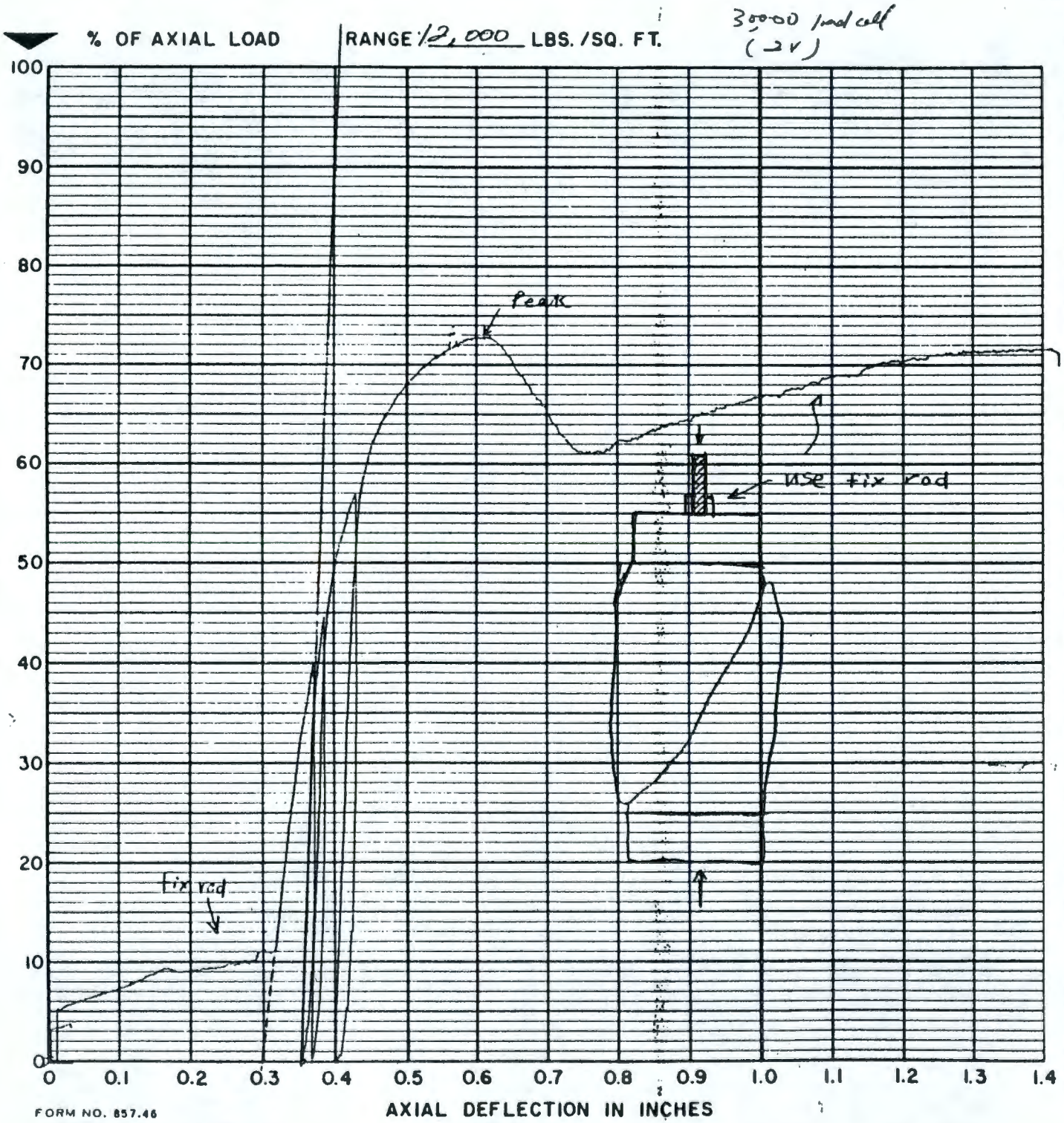
Time/Comment	Load (psf)	0.0001" Reading	Time/Comment	Load (psf)	Reading
10/23/89 15:00 PM	0	3100			
15:00 PM	600				
		2995			
10/24 9:45 AM		2970			
10:45 AM		2969 → 0.0131			
10:45 AM	Start Saturation				
11:10 AM		2952			
3:46 PM		2949			
10/25 12:12		2947			
4:14		2947			
10/26 8:25		2946.5			
4:30		2946.5			
10/27 11:45		2946			
10/30 15:30 PM		2944			
10/31 9:00 AM		2943 → 0.0157			

$$\% \text{ collapse} = \frac{0.0157 - 0.0131}{1.0 - 0.0131} \times 100\% = 0.26\%$$

Dames & Moore

9011888 11067

TEST DATA SHEET



Weight soil & dish no.	1029.3 '6
Dry weight soil & dish	869.4
Net loss of moisture	21
Weight of dish only #3	134.0
Net weight of dry soil	735.4
Moisture, % of dry weight	2.8%

Wt. solids + moisture	W_o	756.4 gms
W_o ÷ 454	W_o'	– lbs.
Weight solids	W_s	– gms
Wet density $W_o' \div V_o'$		105.0 pcf
Dry density		102.0 pcf

Net diameter	D_o	2.42 in.
Area ($0.785\ D_o^2$)	A_o	4.6 sq.
Height	H_o	6 in.
Volume (A_oH_o) ÷ 1728	V_o'	cu. f
Volume (A_oH_o) x 16.4	V_o	cc
Specific gravity of solids	G_s	
Volume of solids $W_s \div G_s$	V_s	cc
$(V_o - V_s) \div V_s$	e_i	
Initial burette reading		cc
Burette reading under pressure		cc
$(V_p - V_s) \div V_s$	e_p	

Deflecting Speed 0.02 in. Hr / Min

Lateral Pressure 2000 PSF; 13.89 PSI

Saturated ☐ Field Moisture ☒ CD

Set-up TWS Tested TWS (SA Office)

Soil Type SM Fine to Med. Sand, trace of silt (stratum 3)

Location Hanford

Owner Kaiser Engr

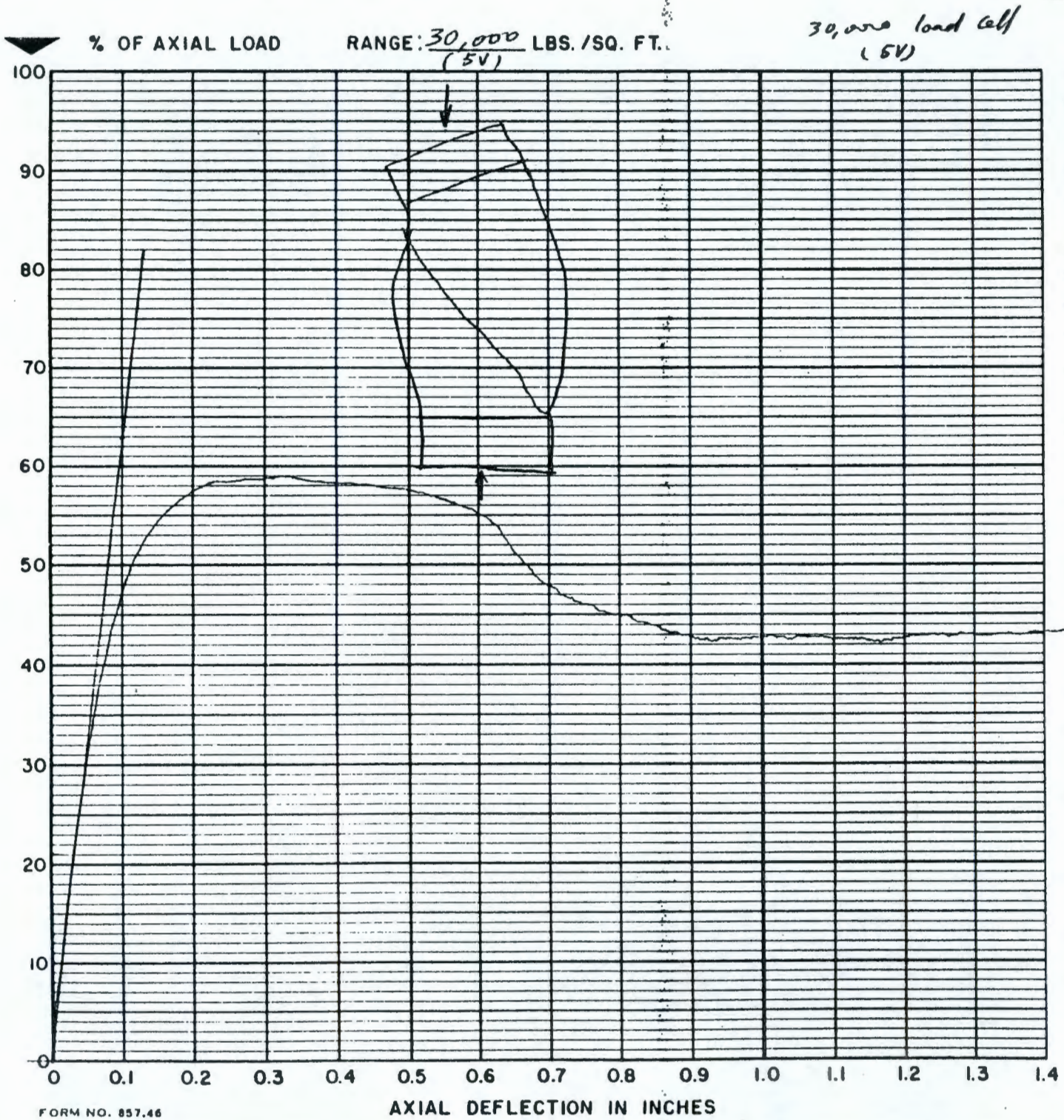
Depth 40.5 Sample # 9

Job # 10805-398 Boring # VP-8

FORM NO. 857.46

APPLIED EARTH SCIENCES

TEST DATA SHEET



Weight soil & dish no.		10548 (6")
Dry weight soil & dish		952.9
Net loss of moisture		13.8
Weight of dish only	#003	184.84
Net weight of dry soil		768.1
Moisture, % of dry weight		1.8%
Wt. solids + moisture	W_o	781.9 gms.
$W_o \div 454$	W_o'	– lbs.
Weight solids	W_s	– gms.
Wet density $W_o' \div V_o'$		108.6 pcf
Dry density		106.7 pcf
Net diameter	D_o	2.42 in.
Area ($0.785\ D_o^2$)	A_o	4.6 sq. i
Height	H_o	6 in.
Volume (A_oH_o) ÷ 1728	V_o'	cu. ft
Volume (A_oH_o) x 16.4	V_o	cc
Specific gravity of solids	G_s	
Volume of solids $W_s \div G_s$	V_s	cc
$(V_o - V_s) \div V_s$	e_i	
Initial burette reading		cc
Burette reading under pressure		cc
$(V_p - V_s) \div V_s$	e_p	

Deflecting Speed 0.02 in. Hr / Min

Lateral Pressure 4000 PSF / 27.8 PSI

Saturated ☐ Field Moisture ☒ CD

Set-up JWS Tested JWS (EXOffice)

Soil Type SM - Fine to Med. Sand, trace of silt (Stratum 3)

Location Hanford

Owner Kaiser Engr

Depth 50.5' Sample # "

Job # 10805-398 Boring # VP-8

FORM NO. 857.46

9 0 1 1 8 8 1 1 0 6 9

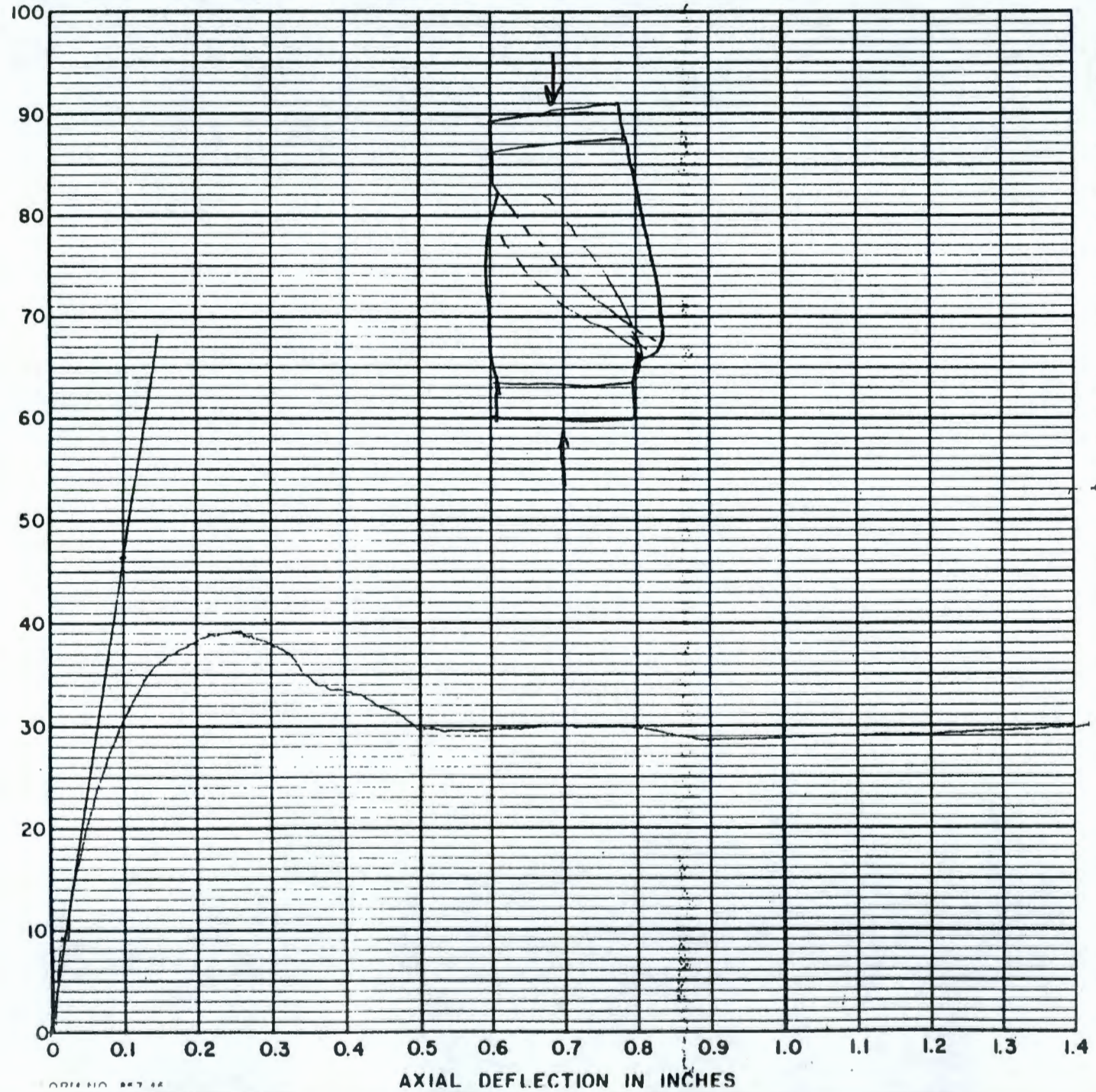
(3)

DAMES & MOORE
APPLIED EARTH SCIENCES

TRIAXIAL COMPRESSION
TEST DATA SHEET

30,000 load cell
2 V

% OF AXIAL LOAD RANGE: 12,000 LBS./SQ. FT.



Weight soil & dish no.	972.1	6"
Dry weight soil & dish	863.5	
Net loss of moisture	20.54	
Weight of dish only #1003	184.84	
Net weight of dry soil	678.7	
Moisture, % of dry weight	3%	

Wt. solids + moisture	W_o 699.2	gms.
$W_o \div 454$	W_o' –	lbs.
Weight solids	W_s –	gms.
Wet density $W_o' \div V_o'$	97.1	pcf
Dry density	94.2	pcf

Net diameter	D_o 2.42	in.
Area ($0.785\ D_o^2$)	A_o 4.6	sq. i
Height	H_o 6	in.
Volume (A_oH_o) ÷ 1728	V_o'	cu. f
Volume (A_oH_o) x 16.4	V_o	cc
Specific gravity of solids	G_s	
Volume of solids $W_s \div G_s$	V_s	cc
$(V_o - V_s) \div V_s$	e_i	
Initial burette reading		cc
Burette reading under pressure		cc
$(V_p - V_s) \div V_s$	e_p	

Deflecting Speed 0.02 in./Hr (Min)

Lateral Pressure 1000 (PSF)
6.9 (PSI)

Saturated ☐ Field Moisture ☒ CD

Set-up JWS Tested JWS (SEA Office)

Soil Type SM - Silt and Fine Sand (Stratum 1)

Location Hanford, WA

Owner Kaiser Engr.

Depth 5' Sample # 2

Job # 10805-398 Boring # VP-10

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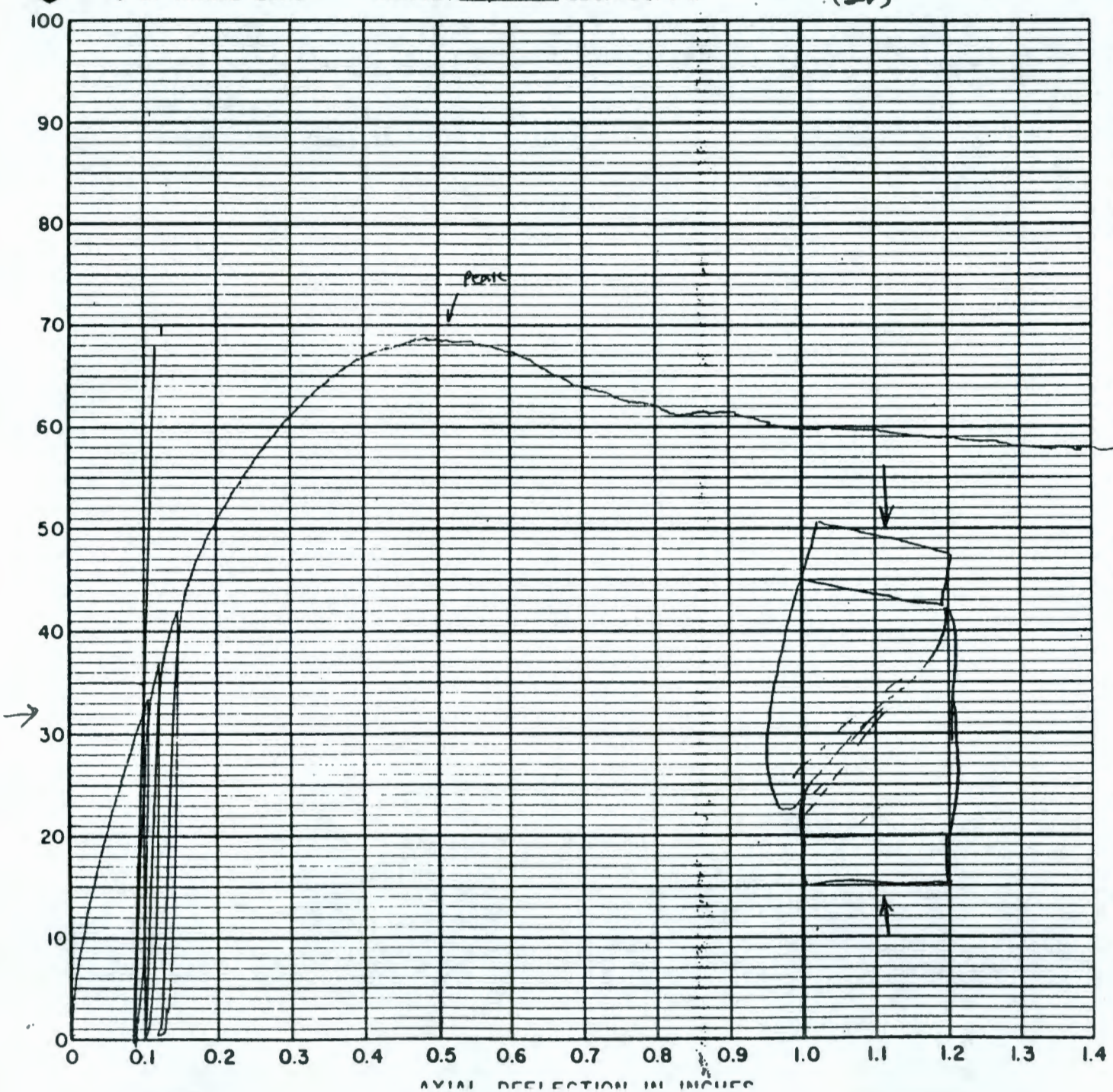
DAMES & MOORE
APPLIED EARTH SCIENCES

MAX COMPRESSI
TEST DATA SHEET

30,000 load cell (2V)



Weight soil & dish no.	1014.8 (6	
Dry weight soil & dish	859.9	
Net loss of moisture	16.0	
Weight of dish only # 3	134.0	
Net weight of dry soil	725.0	
Moisture, % of dry weight	2.2%	
Wt. solids + moisture	W_o 741.9	gms.
W_o ÷ 454	W_o' –	lbs.
Weight solids	W_s –	gms.
Wet density $W_o' \div V_o'$	103.0	pcf
Dry density	100.8	pcf
Net diameter	D_o 2.42	in.
Area ($0.785\ D_o^2$)	A_o 4.6	sq. i
Height	H_o 6	in.
Volume (A_oH_o) ÷ 1728	V_o'	cu. f
Volume (A_oH_o) x 16.4	V_o	cc
Specific gravity of solids	G_s	
Volume of solids $W_s \div G_s$	V_s	cc
$(V_o - V_s) \div V_s$	e_1	
Initial burette reading		cc
Burette reading under pressure		cc
$(V_p - V_s) \div V_s$	e_p	

Deflecting Speed 0.02 in./Hr (Min)

Lateral Pressure 2000 PSF
13.89 PSI

Saturated ☐ Field Moisture ☒

Set-up JWS Tested JWS (SEA Office)

Soil Type SM - Silt and Fine Sand (Stratum 1)

Location Hanford, WA

Owner Kaiser Engr.

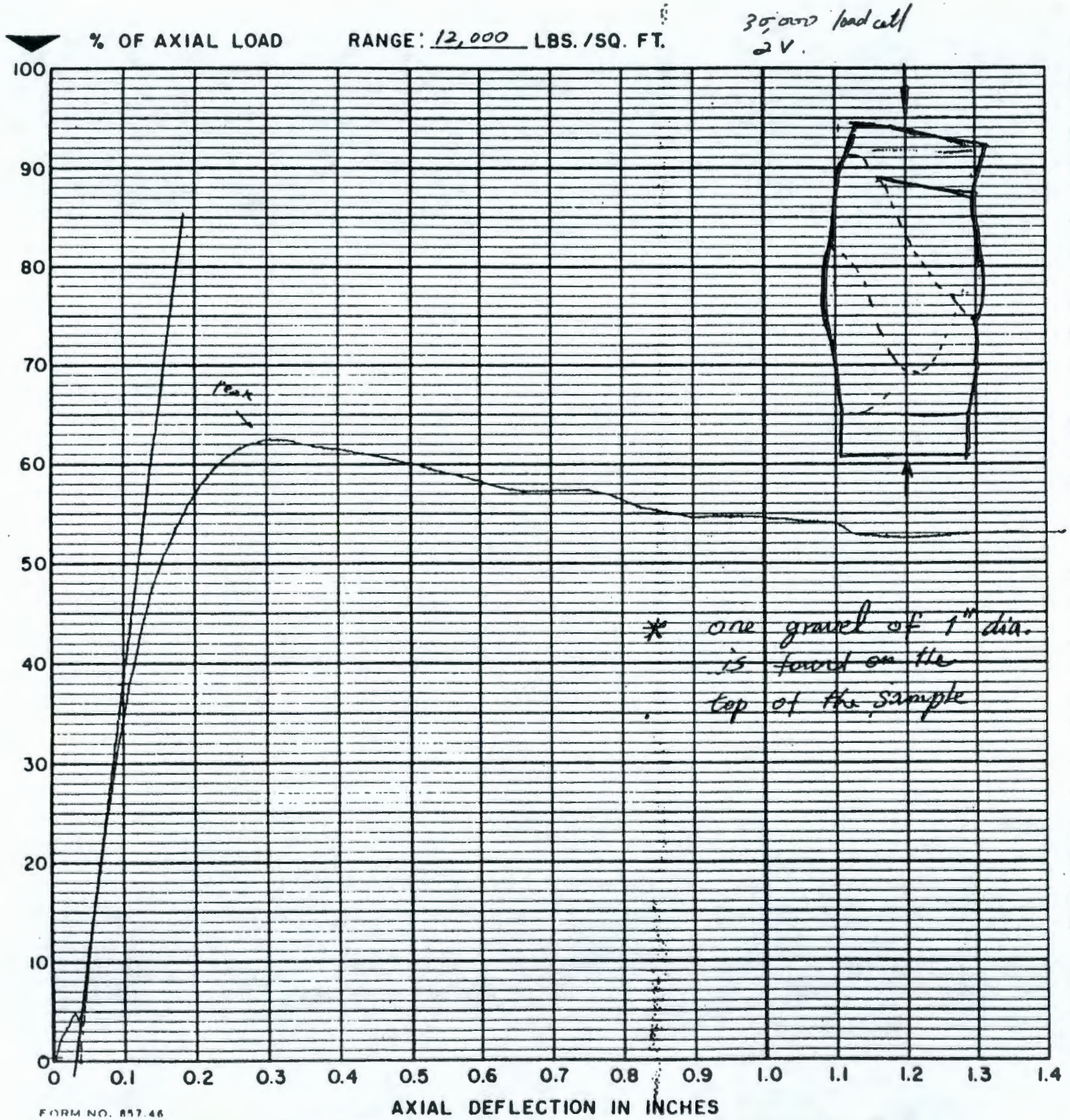
Depth 6' Sample # 2

Job # 10805-398 Boring # VP-11

9 0 1 1 8 8 1 1 0 7 1

DAMES & MOORE
APPLIED EARTH SCIENCES

TEST DATA SHEET



Weight soil & dish no.	1034.5 16"
Dry weight soil & dish	877.1
Net loss of moisture	18.6
Weight of dish only	#31 134.1
Net weight of dry soil	743.0
Moisture, % of dry weight	2.5%
Wt. solids + moisture	W_o 761.6 gm
W_o ÷ 454	W_o' – lbs
Weight solids	W_s – gm
Wet density $W_o' \div V_o'$	105.8 pcf
Dry density	103.2 pcf
Net diameter	D_o 2.42 in.
Area ($0.785\ D_o^2$)	A_o 4.6 sq.
Height	H_o 6 in.
Volume (A_oH_o) ÷ 1728	V_o' ____ cu.
Volume (A_oH_o) × 16.4	V_o ____ cc
Specific gravity of solids	G_s ____
Volume of solids $W_s \div G_s$	V_s ____ cc
$(V_o - V_s) \div V_s$	e_i ____
Initial burette reading	____ cc
Burette reading under pressure	____ cc
$(V_p - V_s) \div V_s$	e_p ____

Deflecting Speed 0.02 in/Hr (Min)

Lateral Pressure 1500 (PSF) (PSI)

Saturated ☐ Field Moisture ☒ 10.4

Set-up JWS Tested JWS (SEA Office)

Soil Type SM/ML - Silt and Fine Sand (Stratum 1)

Location Hanford

Owner Kaiser Engr

Depth 3.5 Sample # 2

Job # 10805-398 Boring # VP-17

FORM NO. 857.46

GEO RECON INTERNATIONAL
applied geophysics

August 8, 1989
J89-451

Martin McCabe
Dames and Moore
2025 1st Avenue
Suite 500
Seattle, Wn 98121

RE: Compressional and Shear Wave Velocity Measurements
Hanford Works Vitrification Plant, 200 E Area,
Hanford, Washington, For Kaiser Engineers, Inc.

This report presents the results of the Compressional and Shear Wave velocity measurements completed in Borehole VP-8, at the proposed site of the Hanford Works Vitrification Plant located on the western side of the 200 East Area. The field measurements were completed on August 8, 1989.

Boring VP-8 was drilled and cased to a depth of 101 feet. The casing was 4 inch Schedule 40, ABS pipe; the 4 inch casing was grouted in the borehole. The ground elevation at the top of the boring was 708 feet. The casing had a 37 inch (3.08 feet) stickup above the ground surface.

The surface soil material was largely dry, loose silty sand underlain by more compacted silty sands undisturbed by the vehicle traffic during the drilling operations. The top surface layer was generally from 1 to 2 feet in depth depending on location.

The measured compressional and shear wave velocities are presented on Tables 1 and 2 respectively, which show the measured time arrivals, interval velocity computations and the averaged layer velocity calculated from the interval layer velocities. Figure 1 is the time-depth plot of the corrected downhole times, from which the layer depths were measured.

The compressional (P) wave energy was a vertical hammer blow on an aluminum plate, offset 7 feet from the boring. The zero time of the hammer blow was determined from the physical hammer contact on the aluminum plate, by circuit closure to the trigger device in the seismograph. The receiver was a piezo-electric crystal transducer placed in the borehole casing,

P.O. Box 55189 Seattle, Wa. 98155 USA [206] 362-9484

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which was filled with water. The measurements were made at 5 foot intervals, measured from the top of the casing.

The shear (S) wave energy source was a horizontal hammer blow on the end of a 6x6 inch plank, offset 10 feet from the borehole. This plank was buried approximately 1 foot to avoid the loose surficial soils found on the compressional wave recordings. This shallow layer is the disturbed ground from drilling activity. The plank was placed tangent to a 10 foot circle through the borehole and weighted by placing the front wheels of a vehicle on it. The zero time of the hammer blow was determined by impact to a piezo-electric crystal taped to the hammer. The generated S wave energy was detected by two transducers, separated by 10 feet, placed in the borehole. Each transducer contained 4 horizontal geophones placed on axis of 45 degrees. The measurements were made at 5 foot intervals, measured from the top of the casing.

Two recordings were made for each 5 foot shear wave measurement interval. The two separate recordings were made with reversed (polarized) energy inputs utilizing the two ends of the 6x6 inch plank (blow L(eft) and blow R(ight). The arrival of the shear wave energy was determined by comparing the times and particle motion directions of the energy arrivals from the two recordings. The particle motion of the shear wave energy is polarized, and is dependent on the direction of the energy input. Thus on blow 1, the particle direction of the motion should be reversed from that produced by blow 2. The polarization of the energy, the corresponding wave form and time arrivals recorded, allows the interpreter to separate P wave arrivals, P to S conversions, tube wave arrivals and other energy arrivals (eg. those of other boundary waves) from the generated shear wave arrival.

The compressional and shear wave velocities with the corresponding, calculated Poisson's ratios are shown below:

Depth	P velocity ft/sec	S velocity ft/sec	Poisson's Ratio
0 to 8 feet	2720	519	0.481
8 to 28.5	3834	1057	0.459
28.5 to 50	5331	1471	0.459
50 to 79	6162	1627	0.463
79 to 100.9	6626	1974	0.451

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The formula for Poisson's ratio is:

$$\nu = \frac{(Vp/Vs)^2 - 2}{2\,((Vp/Vs)^2 - 1)}$$

Where : ν = Poisson's Ratio

Vp = Compressional Wave Velocity

Vs = Shear Wave Velocity

The arrival times were picked initially from the first peak or trough of the energy arrivals. This was done to avoid errors that may be generated by the low amplitude rise time of the particle motion. The time of the first arrival was then determined using the corresponding frequency of the energy wave train to step back to the time of the first energy arrival.

The borehole casing stickup was subtracted from the measured downhole depths to arrive at depths below the ground surface. The arrival times were then converted to downhole times, rather than plotting the arrival time vs the slant distance (a result of the source-borehole offset).

The formula for conversion of the record time to the downhole time is:

DH Time = Record Time x (Cos(Arctan (Offset/detector depth)))

where the DH Time is the vertical travel time down the borehole.

The DH times were then summarized and 5 foot interval velocities computed. The DH times were also plotted on a time-depth plot to determine layer depths, as shown on Figure 1. An average velocity was computed through the interval velocities, generally not utilizing the top velocity.

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The recording equipment used in the survey was an ES-1225, a signal enhancement recording seismograph. The measured record lengths may be adjusted from 25, 50, 100, 200, 500 and 1000 milliseconds sweep length with 1024 samples per record length. P wave recordings were done with a 25 millisecond sweep, and the S wave recordings were done with a 100 millisecond sweep. A sweep delay was applied with increasing depth to maintain the energy arrivals in the approximate center of the record.

Copies of the Compressional and Shear wave recordings and QA records (showing sweep calibration, repeatability of switch closures, etc) will be sent under separate cover.

FOR GEO-RECON INTERNATIONAL

John M. Musser Jr.
Principal Geophysicist

Reviewed by:

Clyde Ringstad
Principal Geophysicist

9011107 5

Downhole Compressional and Shear Wave Velocity Measurements
Borehole VP-8, HWVP, 200 East - Hanford, Washington - August, 1989

Compressional Wave Data - Interval Velocity Computations

Depth below Top of Casing	Depth of Data	P Wave Arrival Time	Corrected DH Time	Interval Time	Interval Velocity	Average Velocity
5	1.9	6.350	2.725	2.725	697	Direct Wave
10	6.9	6.500	4.563	1.838	2720	2720
	Velocity Break at 8 feet					
15	11.9	7.250	6.249	1.686	2966	
20	16.9	8.175	7.553	1.304	3835	3834
25	21.9	9.300	8.858	1.306	3829	
30	26.9	10.500	10.162	1.303	3837	
	Velocity Break at 28.5 feet					
35	31.9	11.700	11.428	1.267	3948	
40	36.9	12.600	12.379	0.951	5257	5331
45	41.9	13.500	13.315	0.936	5341	
50	46.9	14.400	14.242	0.927	5395	
	Velocity Break at 50 feet					
55	51.9	15.200	15.064	0.821	6087	
60	56.9	16.000	15.880	0.817	6122	
65	61.9	16.800	16.694	0.813	6148	6162
70	66.9	17.600	17.504	0.811	6166	
75	71.9	18.400	18.313	0.809	6181	
80	76.9	19.200	19.121	0.808	6192	
	Velocity Break at 79 feet					
85	81.9	19.950	19.878	0.757	6609	
90	86.9	20.700	20.633	0.756	6617	
95	91.9	21.450	21.388	0.755	6624	6626
100	96.9	22.200	22.142	0.754	6629	
104	100.9	22.800	22.745	0.603	6633	

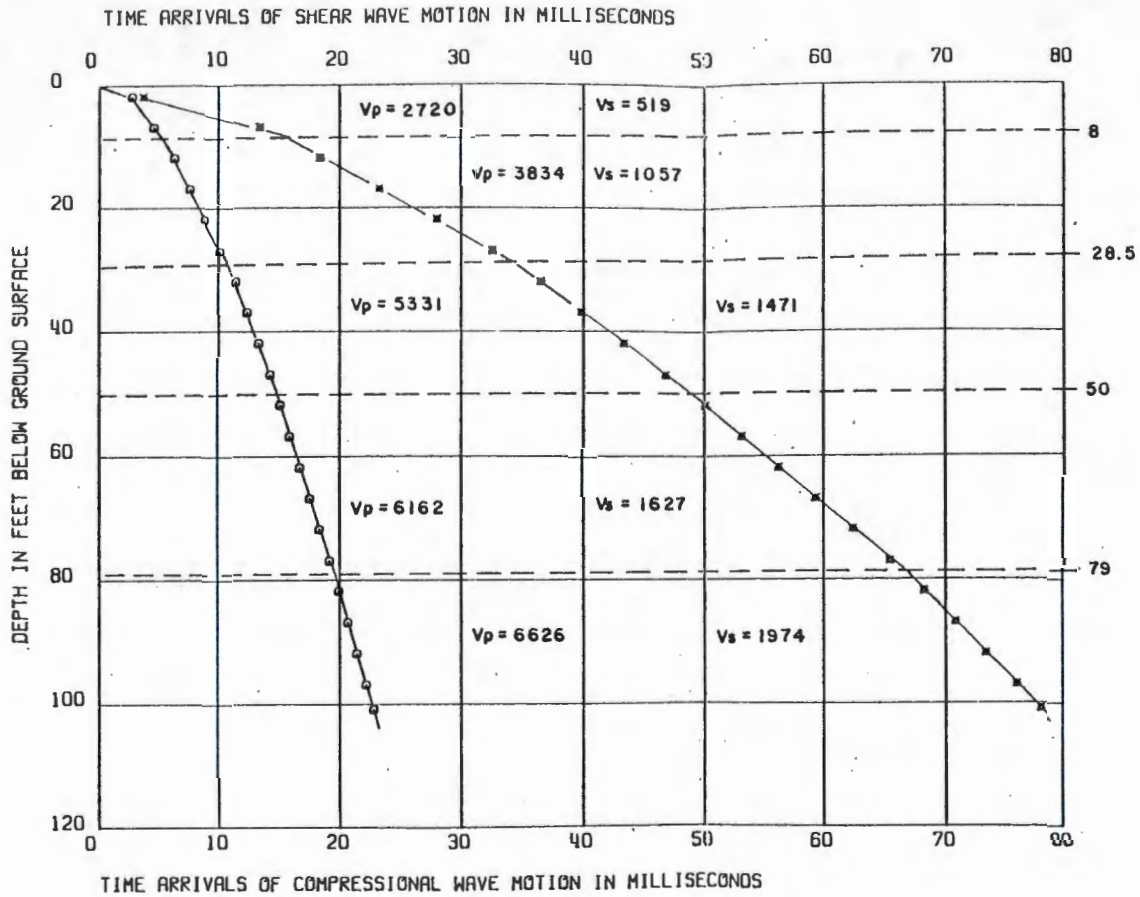
Casing stickup : 3.08 feet (37 Inches) Ground Elevation: 708 feet
Coordinate Location: N 42,130 E 55,960 Plant Datum
Depths in feet - Time in milliseconds - Velocity in ft/sec
Offset at 5 feet depth is 4 feet - remainder at 7 feet.

Downhole Compressional and Shear Wave Velocity Measurements
Borehole VP-8, HWVP, 200 East - Hanford, Washington - August, 1989

Shear Wave Data - Interval Velocity Computations

Depth below Top of Casing	Depth of Data	S Wave Arrival Time	Corrected DH Time	Interval Time	Interval Velocity	Average Velocity
5	1.9	19.50	3.640	3.640	522	519
10	6.9	23.50	13.346	9.706	515	
	Velocity Break at 8 feet					
15	11.9	24.00	18.374	5.028	995	
20	16.9	27.00	23.237	4.863	1028	1057
25	21.9	30.75	27.972	4.735	1056	
30	26.9	34.75	32.572	4.600	1087	
	Velocity Break at 28.5 feet					
35	31.9	38.25	36.499	3.927	1273	
40	36.9	41.25	39.814	3.315	1508	1471
45	41.9	44.50	43.284	3.470	1441	
50	46.9	47.75	46.700	3.416	1464	
	Velocity Break at 50 feet					
55	51.9	51.00	50.079	3.379	1480	
60	56.9	54.00	53.185	3.106	1610	
65	61.9	57.00	56.270	3.086	1620	1627
70	66.9	60.00	59.341	3.070	1629	
75	71.9	63.00	62.399	3.059	1635	
80	76.9	66.00	65.449	3.050	1640	
	Velocity Break at 79 feet					
85	81.9	68.75	68.243	2.794	1789	
90	86.9	71.25	70.783	2.540	1969	
95	91.9	73.75	73.317	2.534	1973	1974
100	96.9	76.25	75.847	2.530	1976	
104	100.9	78.25	77.869	2.021	1979	

Casing stickup : 3.08 feet (37 inches). Ground Elevation: 708 feet
Coordinate Location: N 42,130 E 55,960 Plant Datum



LEGEND

- ⊙ COMPRESSIONAL WAVE ARRIVAL
- ✖ SHEAR WAVE ARRIVAL

BOREHOLE ELEVATION: 708

SCALE:

SCALE AS SHOWN ON GRAPH

DAMES AND MOORE
HWVP - 200 E
HANFORD WASHINGTON

DOWNHOLE GEOPHYSICAL SURVEY

COMPRESSIONAL AND SHEAR WAVE PLOTS

BORING VP-8

AUGUST 1989 J89-451

GEO-RECON INTERNATIONAL FIG. 1

ORIGINAL FIELD DATA

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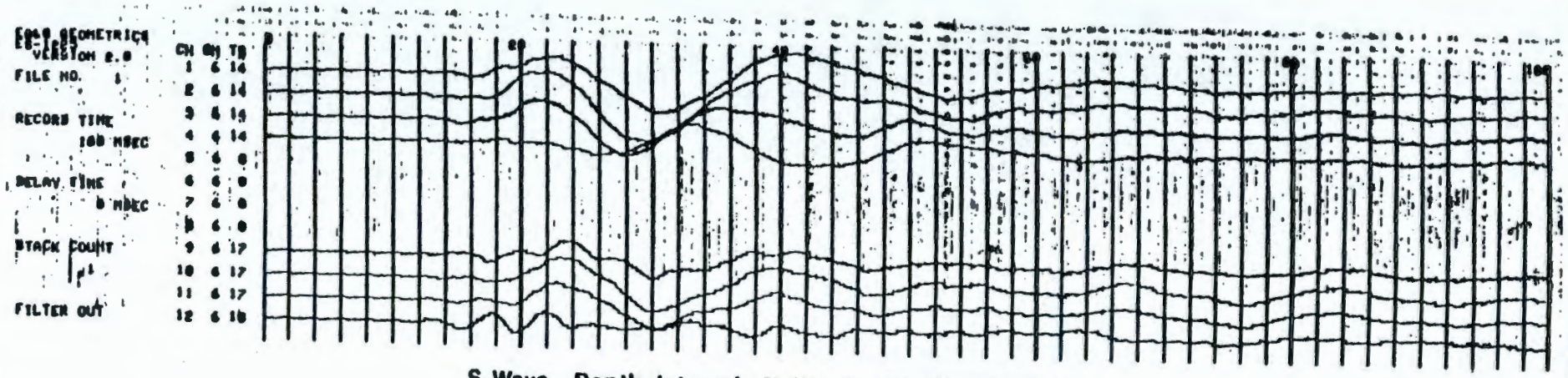
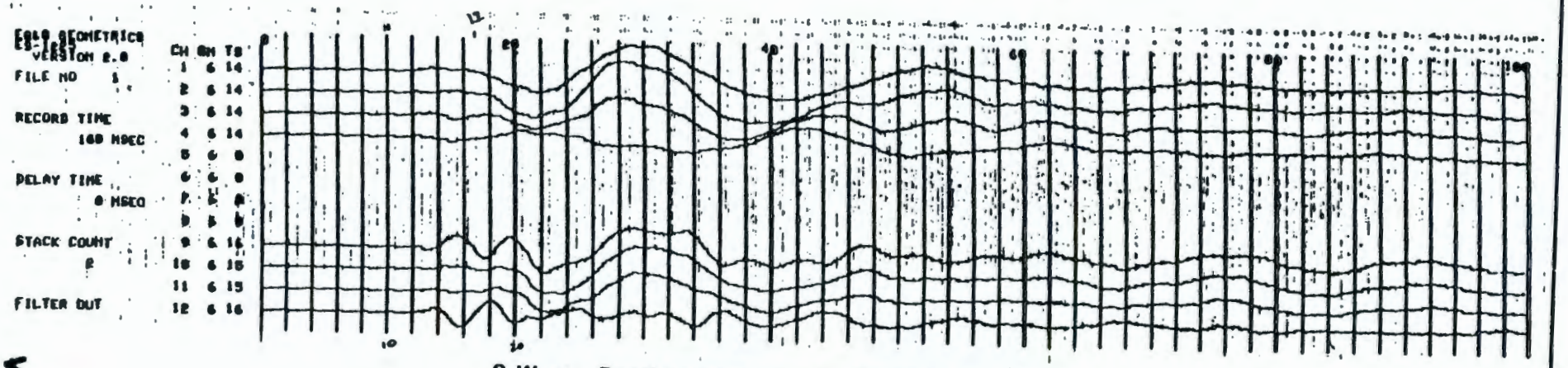
Job No. 10805-398-016

Downhole Seismic Test Data



S-Wave Depth Interval: 5'-15' Impact Direction: Right



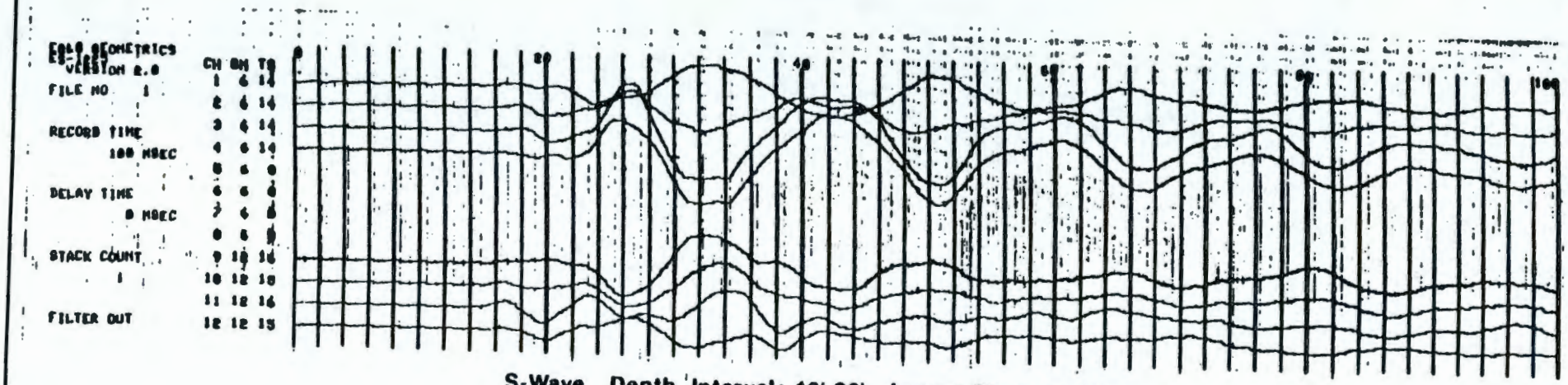
S-Wave Depth Interval: 5'-15' Impact Direction:Left

Hanford Waste
Vitrification Plant
Dames & Moore

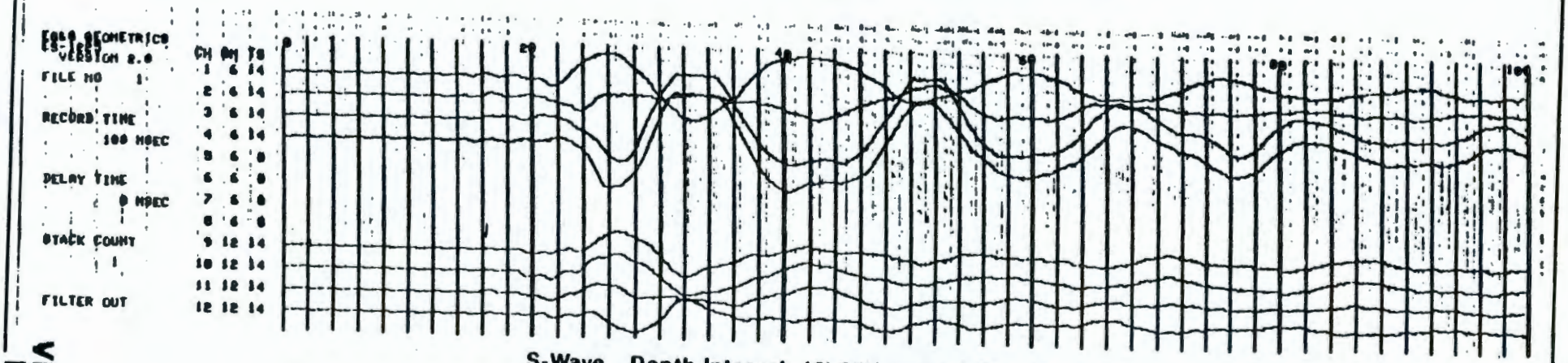
122954

Job No. 10805-398-016

Downhole Seismic Test Data



S-Wave Depth Interval: 10'-20' Impact Direction: Right



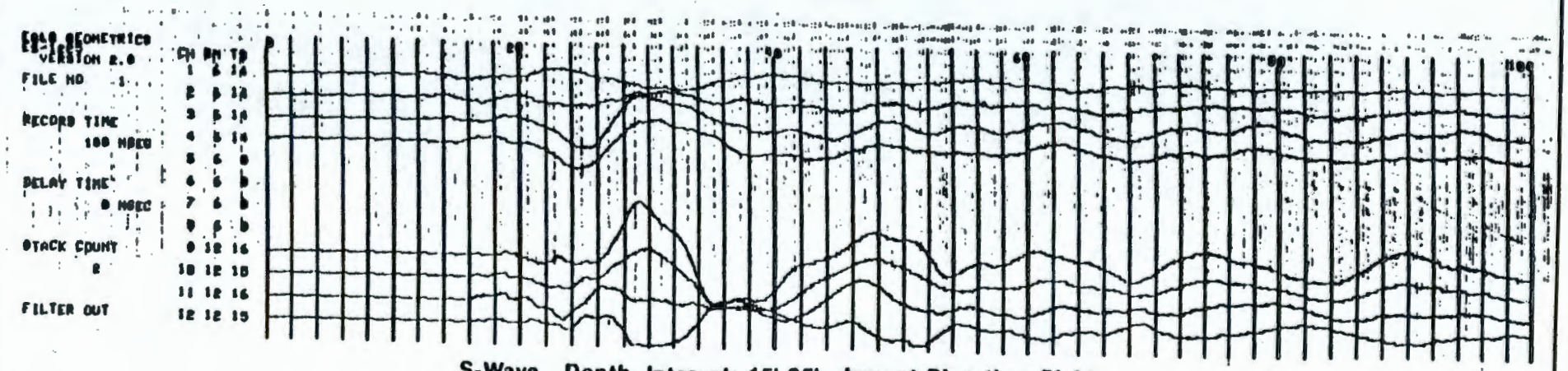
S-Wave Depth Interval: 10'-20' Impact Direction:Left

Hanford Waste
Vitrification Plant
Dames & Moore

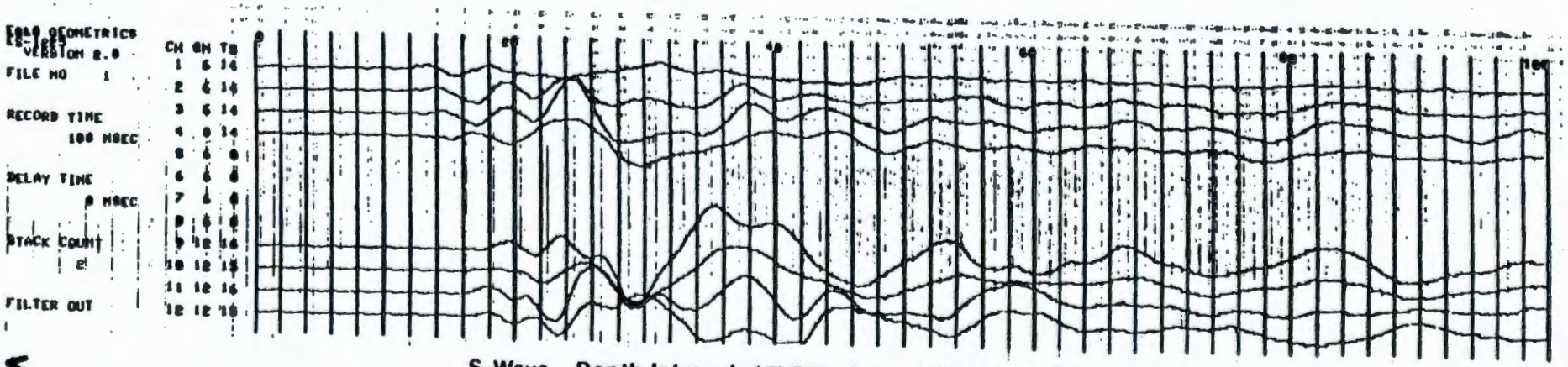
122954

Job No. 10805-398-016

Downhole Seismic Test Data



S-Wave Depth Interval: 15'-25' Impact Direction: Right



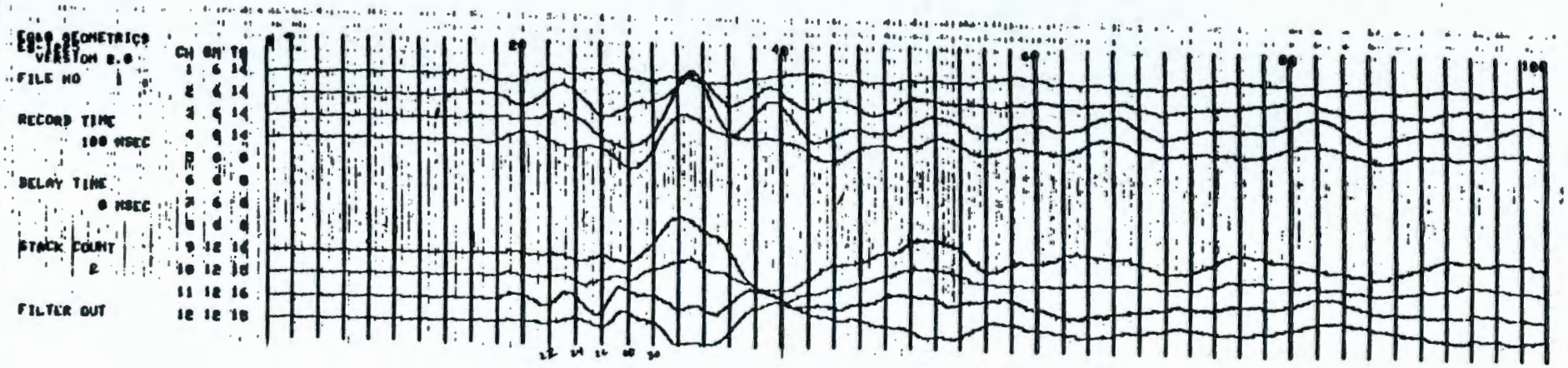
S-Wave Depth Interval: 15'-25' Impact Direction:Left

Hanford Waste
Vitrification Plant
Dames & Moore

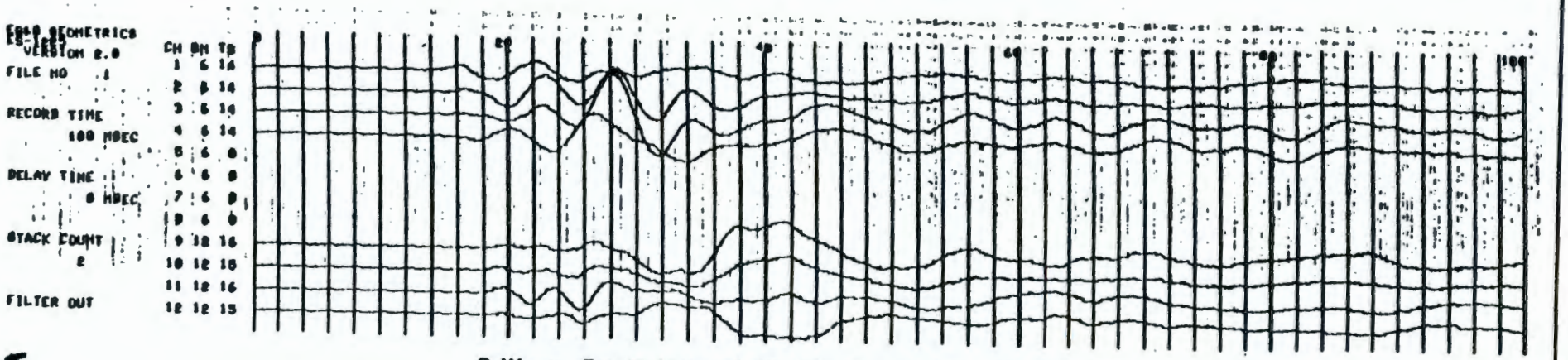
122054

Job No. 10805-398-016

Downhole Seismic Test Data



S-Wave Depth Interval : 20'-30' Impact Direction: Right



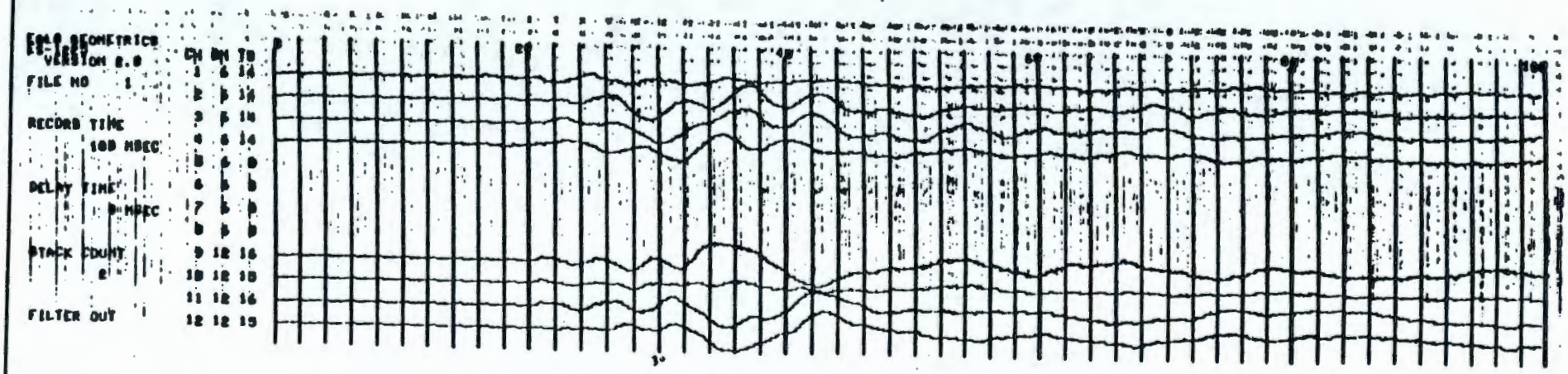
S-Wave Depth Interval: 20'-30' Impact Direction:Left

Hanford Waste
Vitrification Plant
Dames & Moore

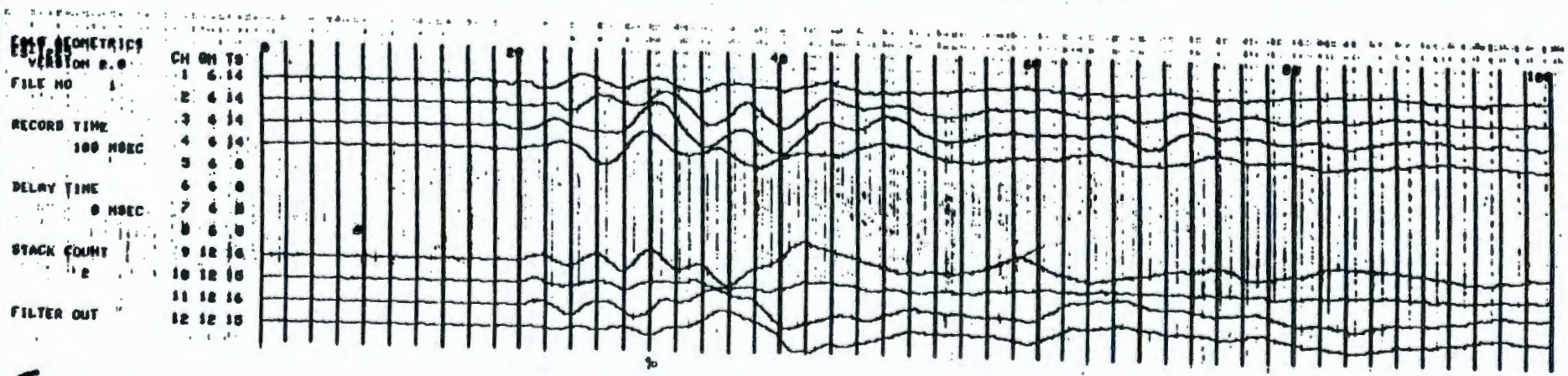
122954

Job No. 10805-398-016

Downhole Seismic Test Data



S-Wave Depth Interval: 25'-35' Impact Direction: Right



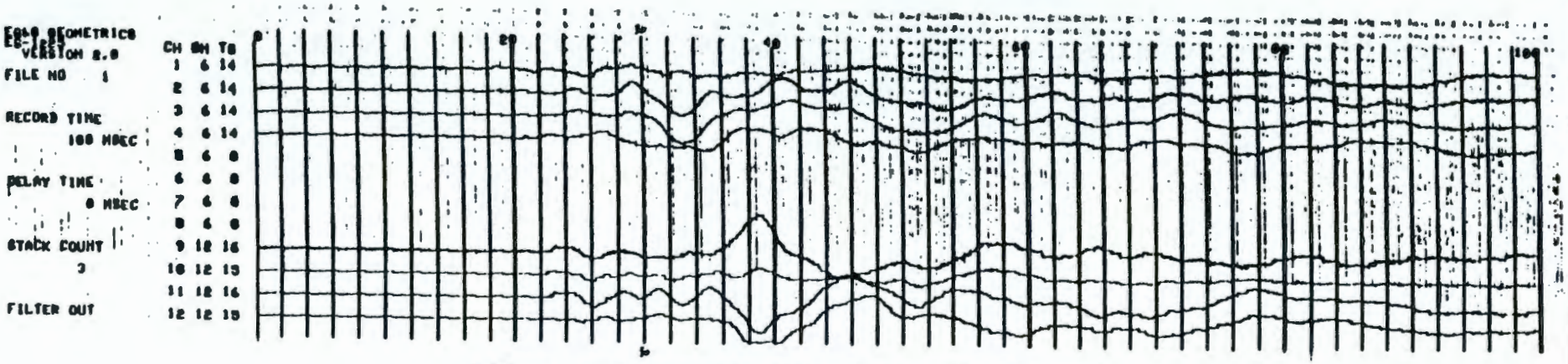
S-Wave Depth Interval: 25'-35' Impact Direction:Left

Hanford Waste
Vitrification Plant
Dames & Moore

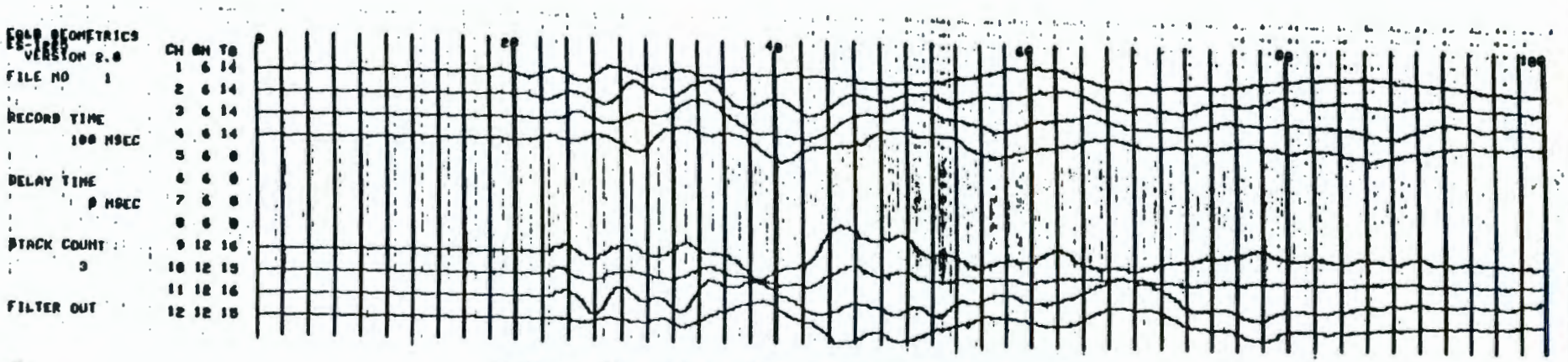
122954

Job No. 10805-398-016

Downhole Seismic Test Data



S-Wave Depth Interval: 30'-40' Impact Direction: Right



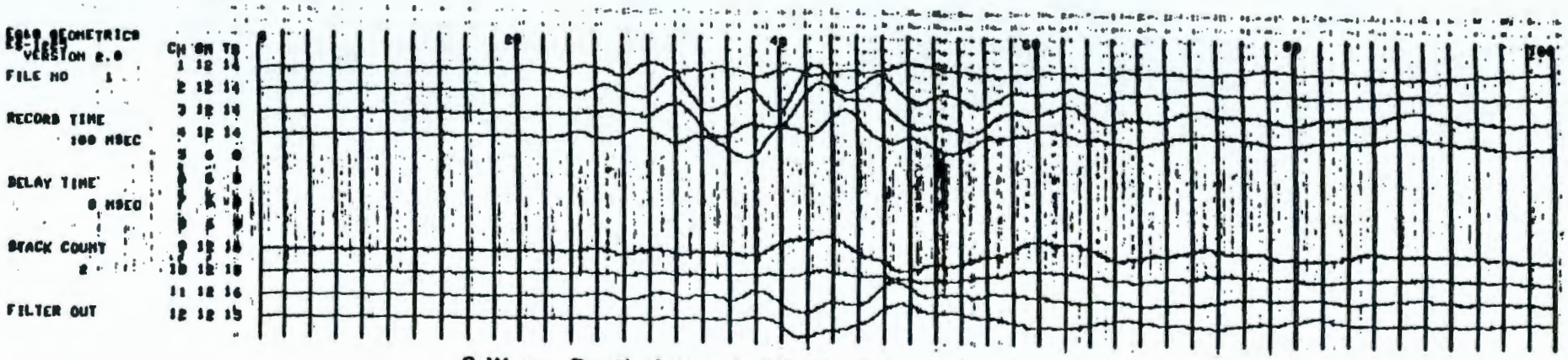
S-Wave Depth Interval: 30'-40' Impact Direction:Left

Hanford Waste
Vitrification Plant
Dames & Moore

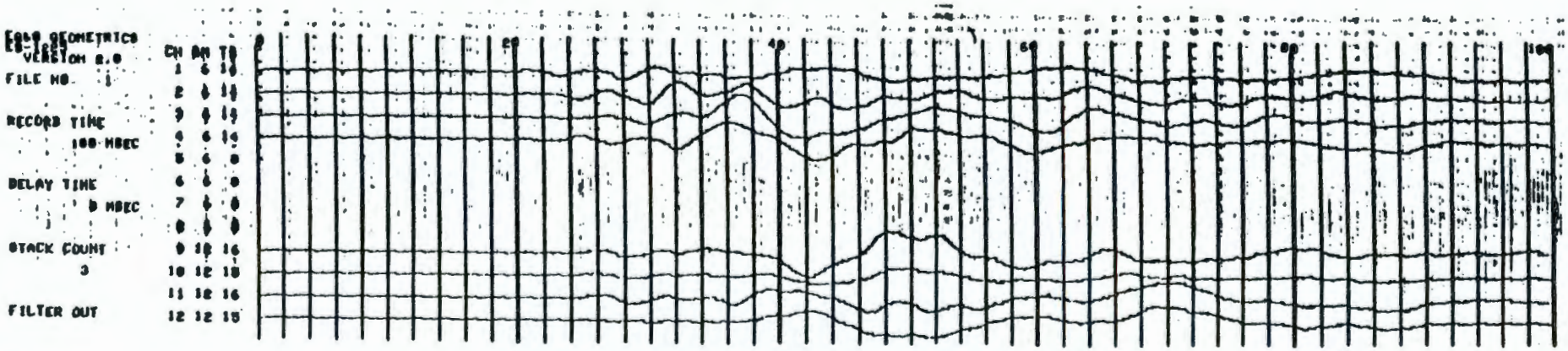
122954

Job No. 10805-398-016

Downhole Seismic Test Data



S-Wave Depth Interval: 35'-45' Impact Direction: Right



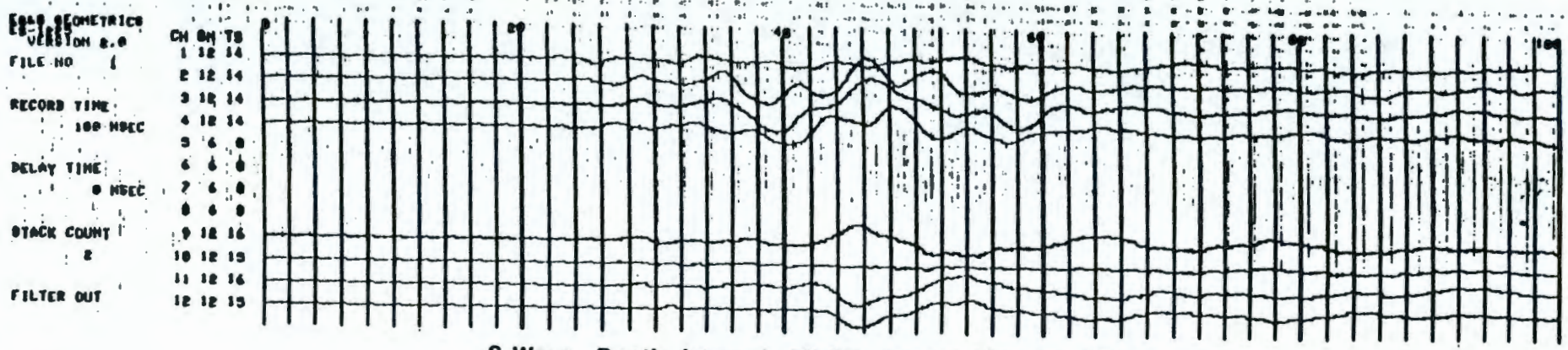
S-Wave Depth Interval: 35'-45' Impact Direction:Left

Hanford Waste
Vitrification Plant
Dames & Moore

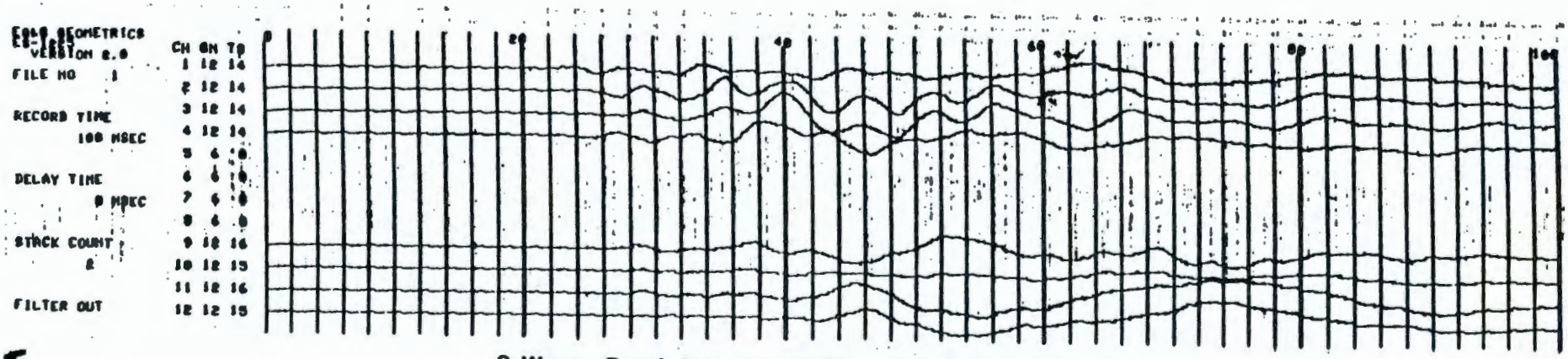
123954

Job No. 10805-398-016

Downhole Seismic Test Data



S-Wave Depth Interval: 40'-50' Impact Direction: Right



S-Wave Depth Interval: 40'-50' Impact Direction:Left

Hanford Waste
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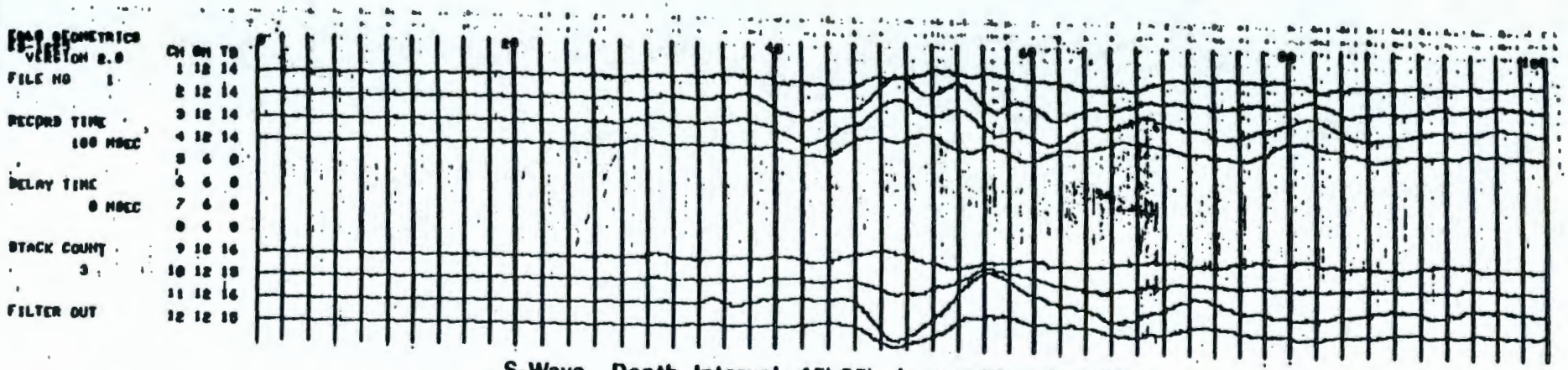
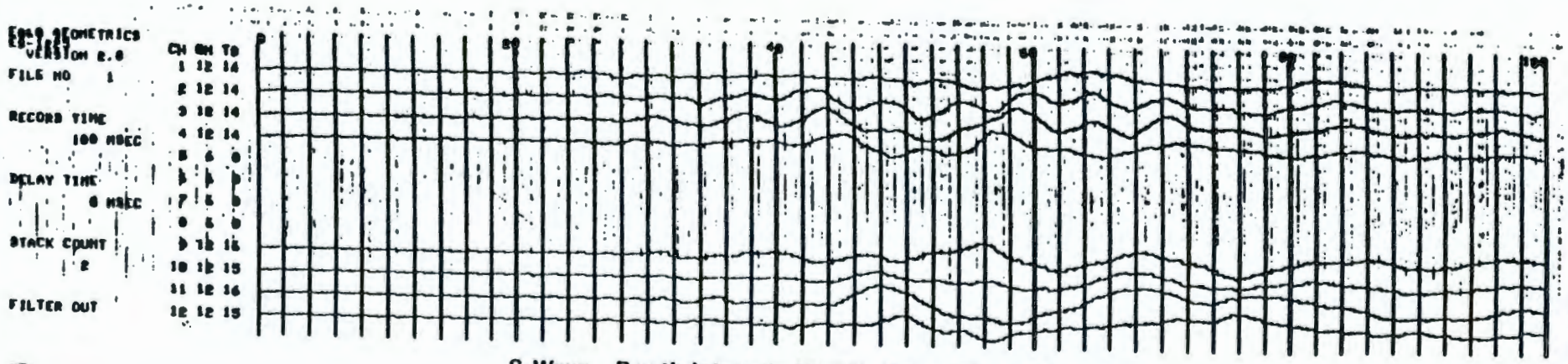
Job No. 10805-398-016

Downhole Seismic Test Data



S-Wave Depth Interval: 45'-55' Impact Direction: Right



S-Wave Depth Interval: 45'-55' Impact Direction:Left

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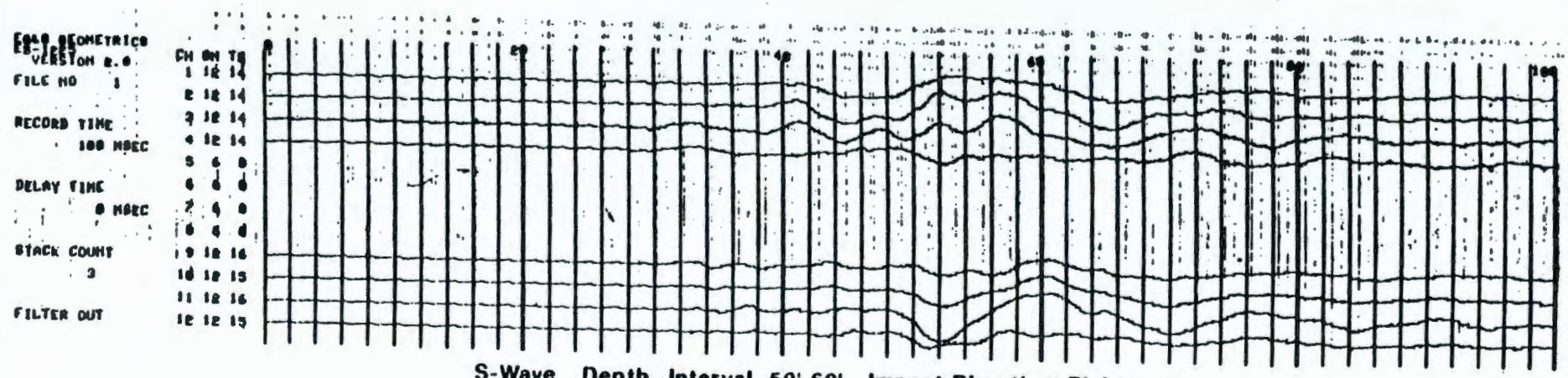
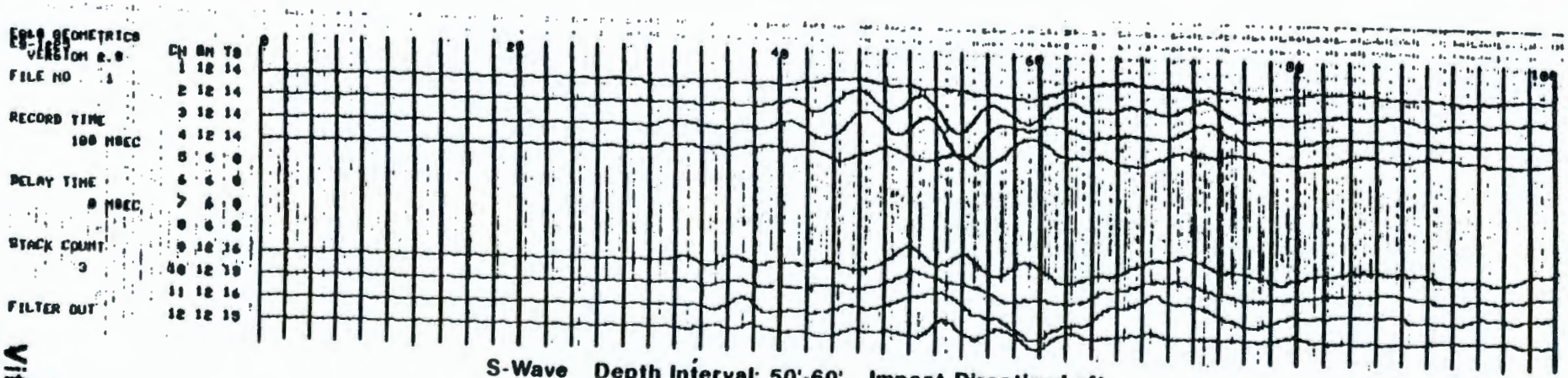
Downhole Seismic Test Data



S-Wave Depth Interval 50'-60' Impact Direction: Right



S-Wave Depth Interval: 50'-60' Impact Direction:Left

Hanford Waste
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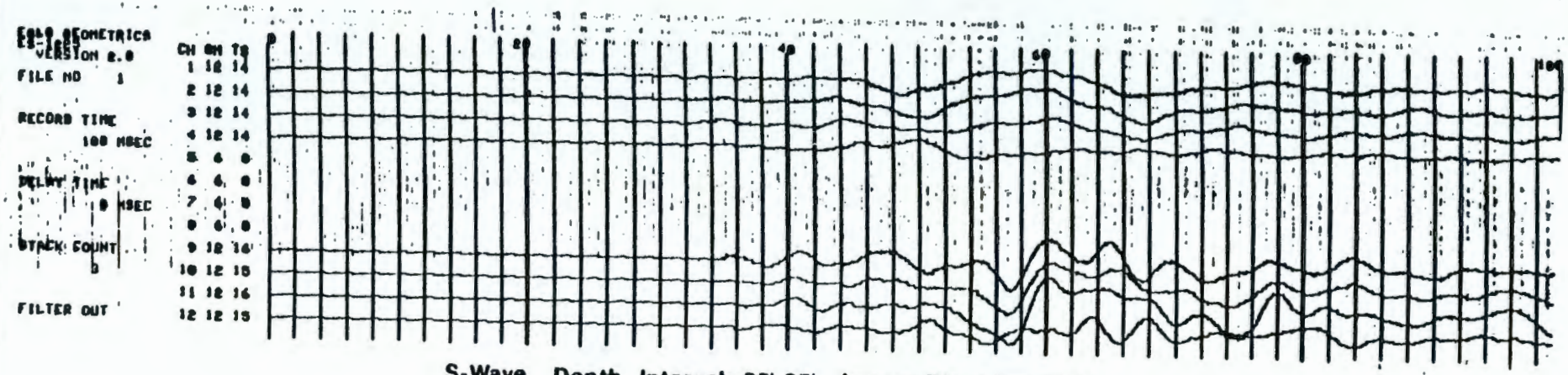
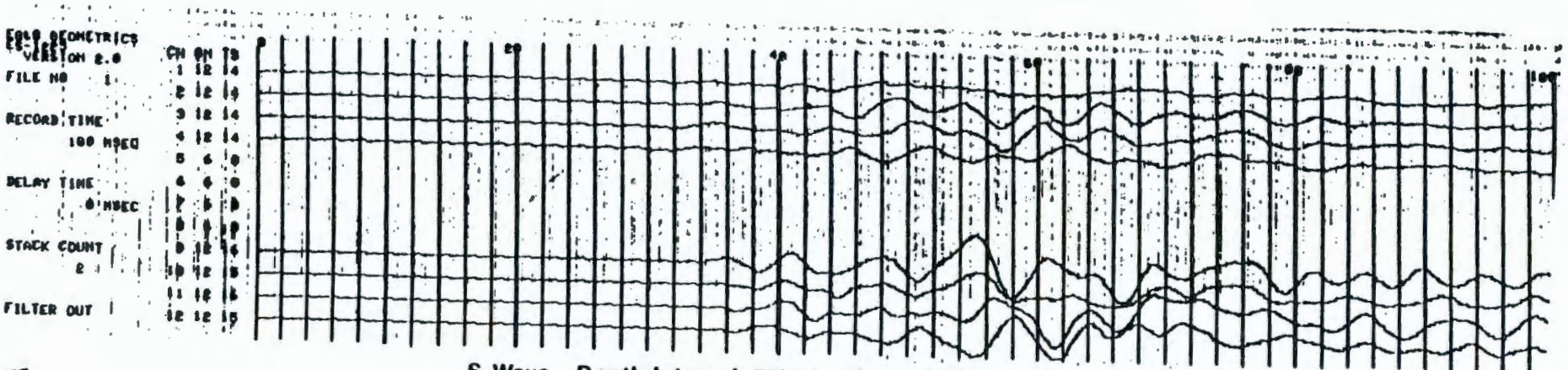
Downhole Seismic Test Data



S-Wave Depth Interval: 55'-65' Impact Direction: Right



S-Wave Depth Interval: 55'-65 Impact Direction:Left

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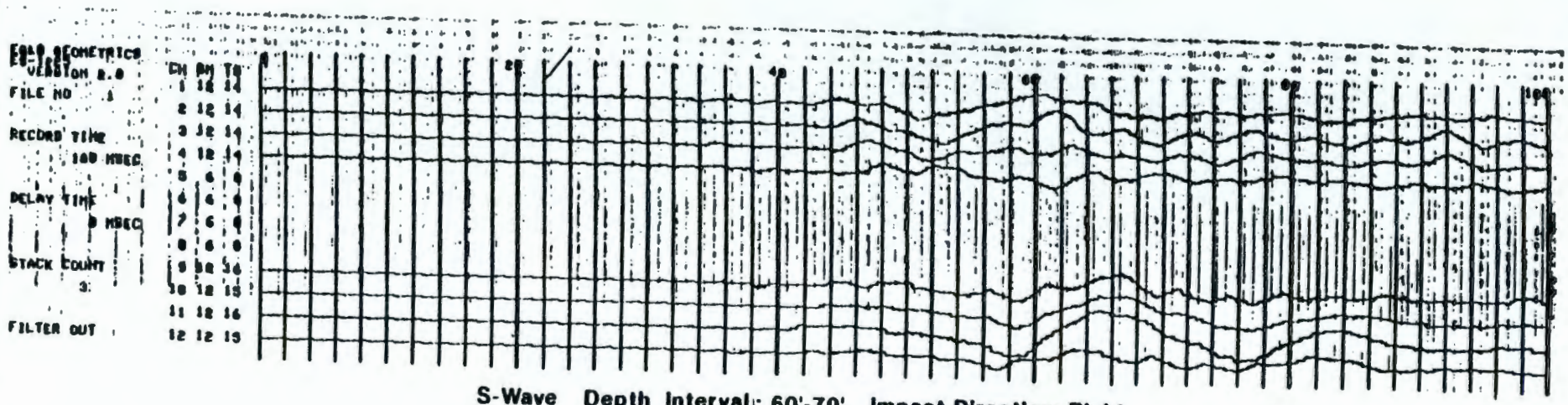
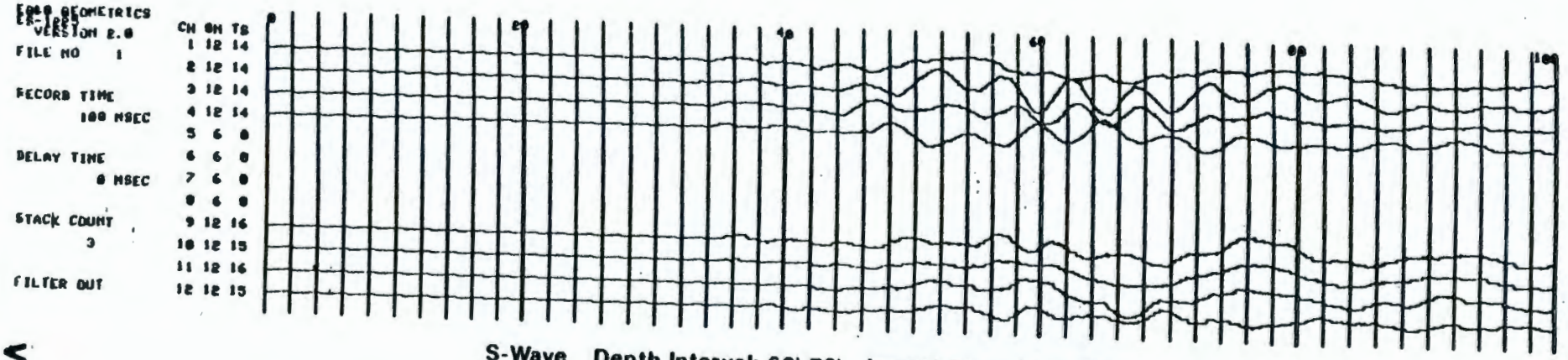
Job No. 10805-398-016

Downhole Seismic Test Data



S-Wave Depth Interval: 60'-70' Impact Direction: Right



S-Wave Depth Interval: 60'-70' Impact Direction:Left

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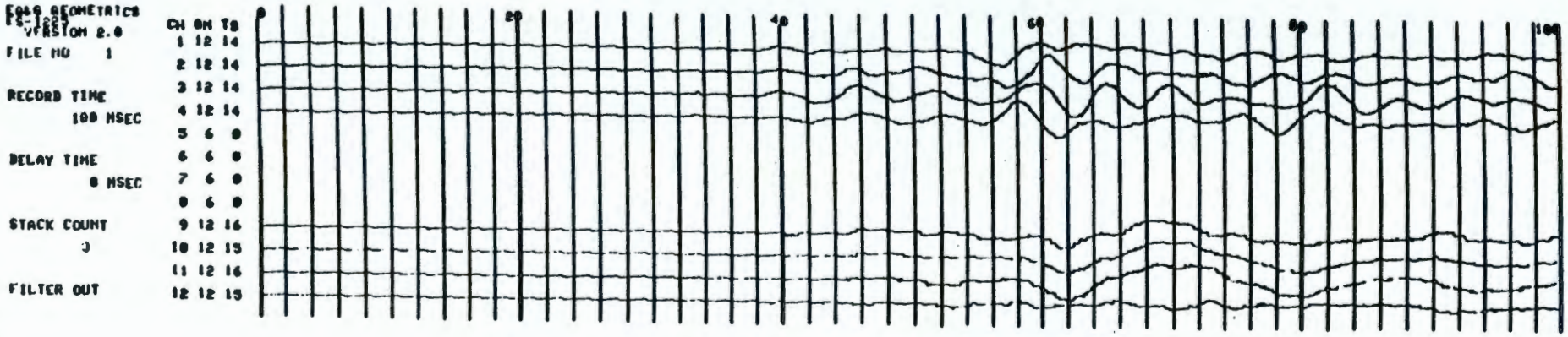
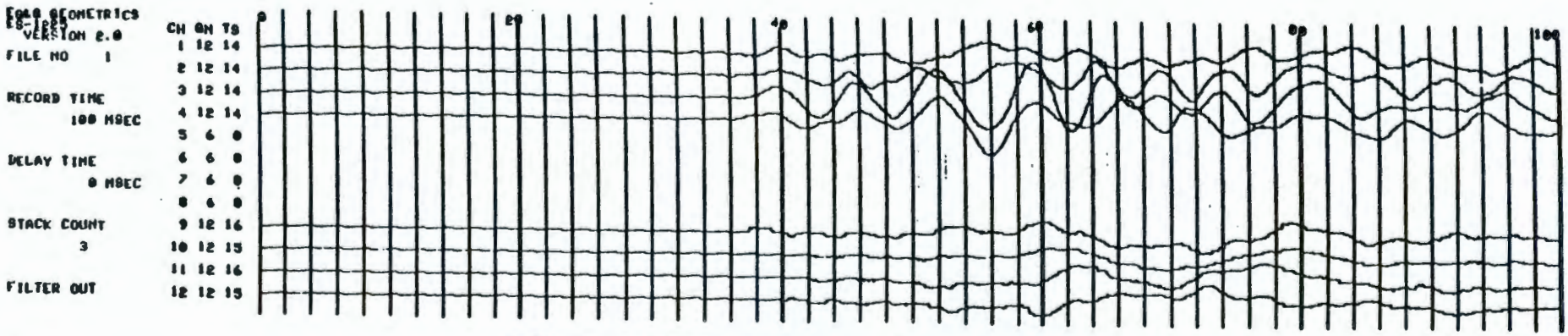
Job No. 10805-398-016

Downhole Seismic Test Data



S-Wave Depth Interval: 65'-75' Impact Direction: Right



S-Wave Depth Interval: 65'-75' Impact Direction:Left

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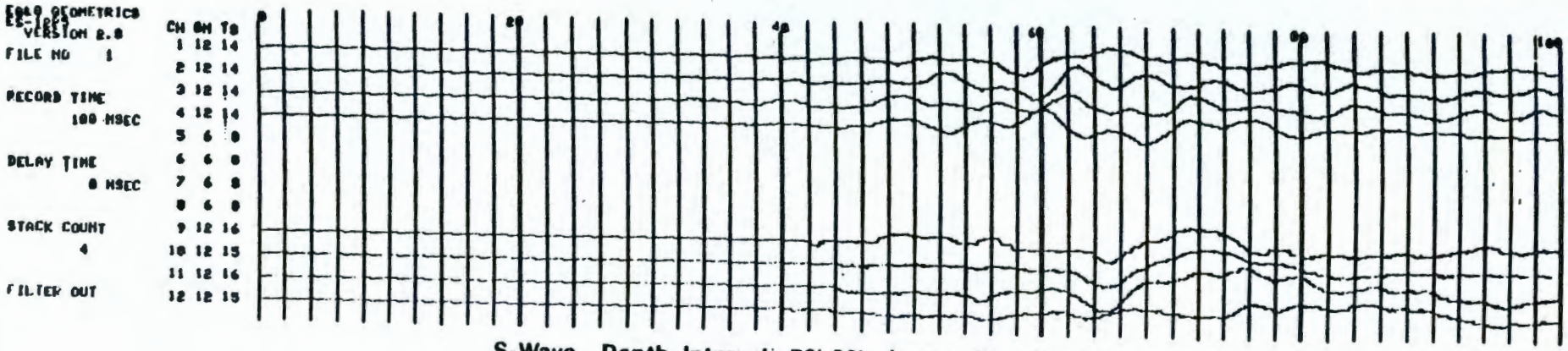
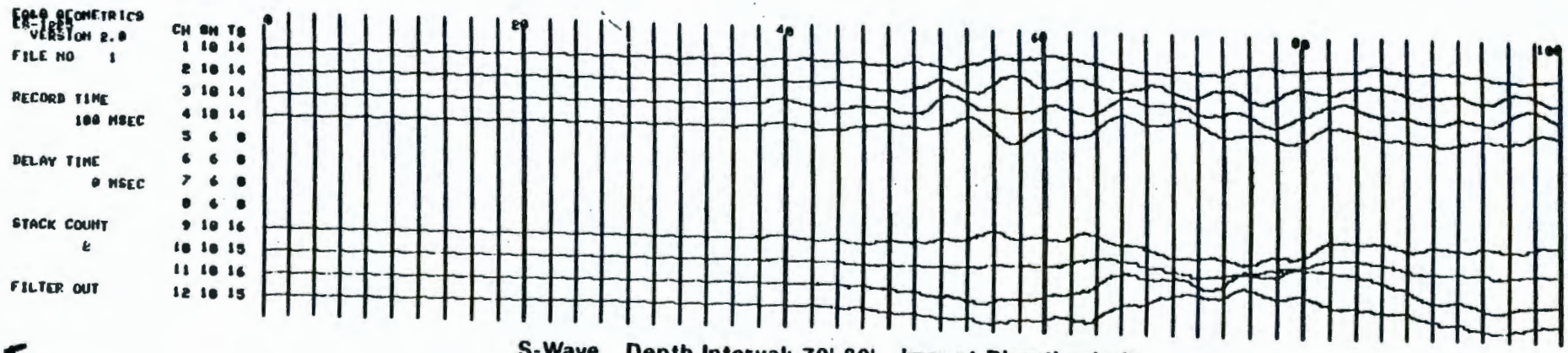
Downhole Seismic Test Data



S-Wave Depth Interval: 70'-80' Impact Direction: Right



S-Wave Depth Interval: 70'-80' Impact Direction:Left

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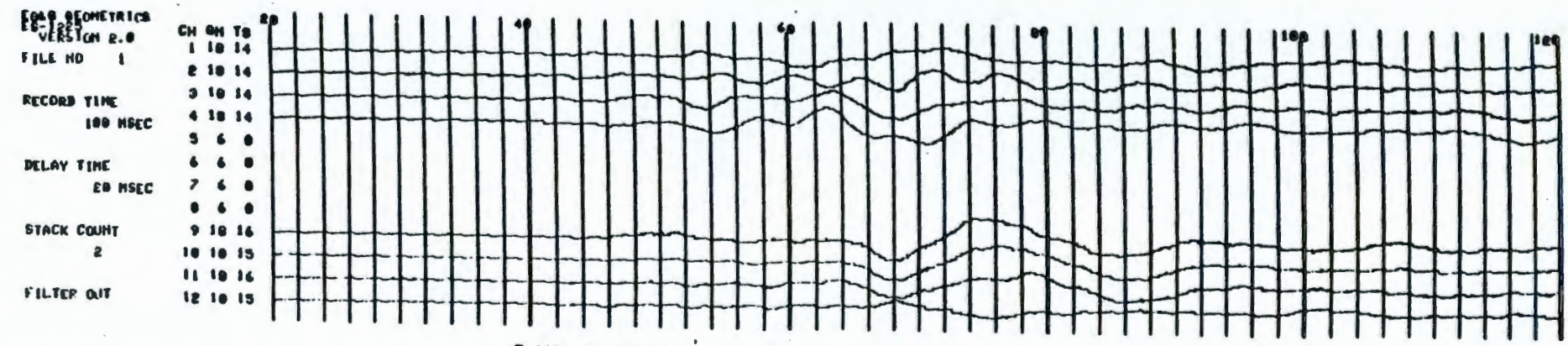
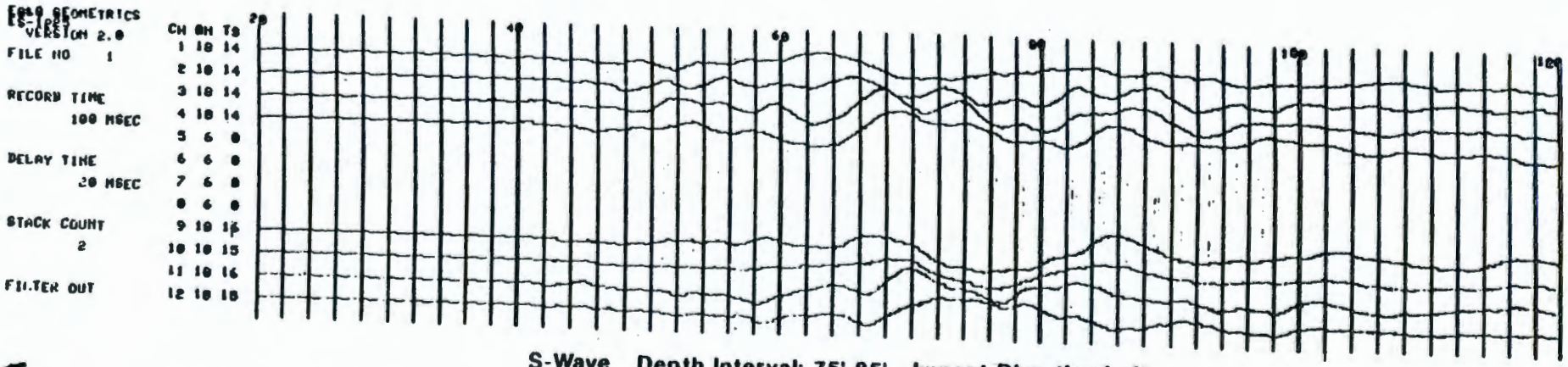
Downhole Seismic Test Data



S-Wave Depth Interval: 75'-85' Impact Direction: Right



S-Wave Depth Interval: 75'-85' Impact Direction:Left

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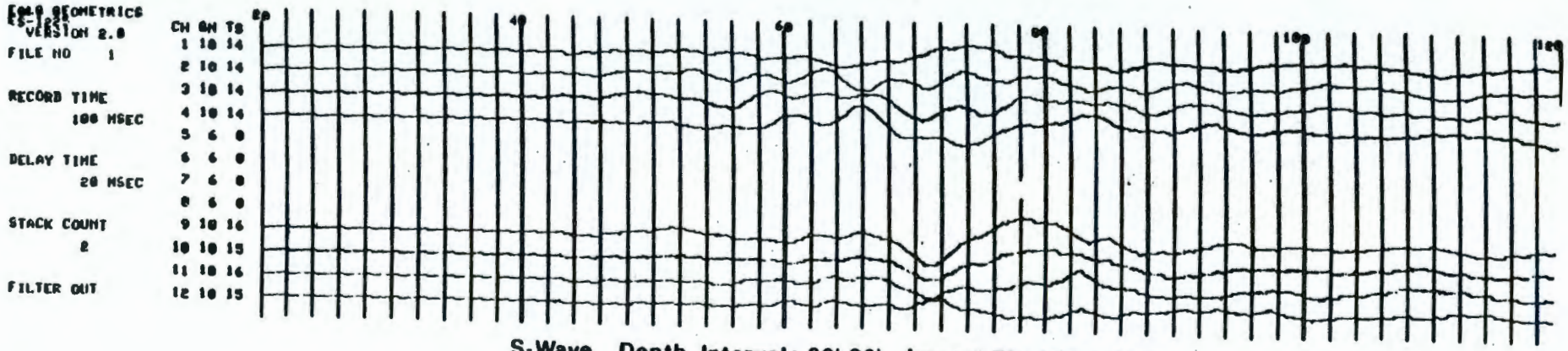
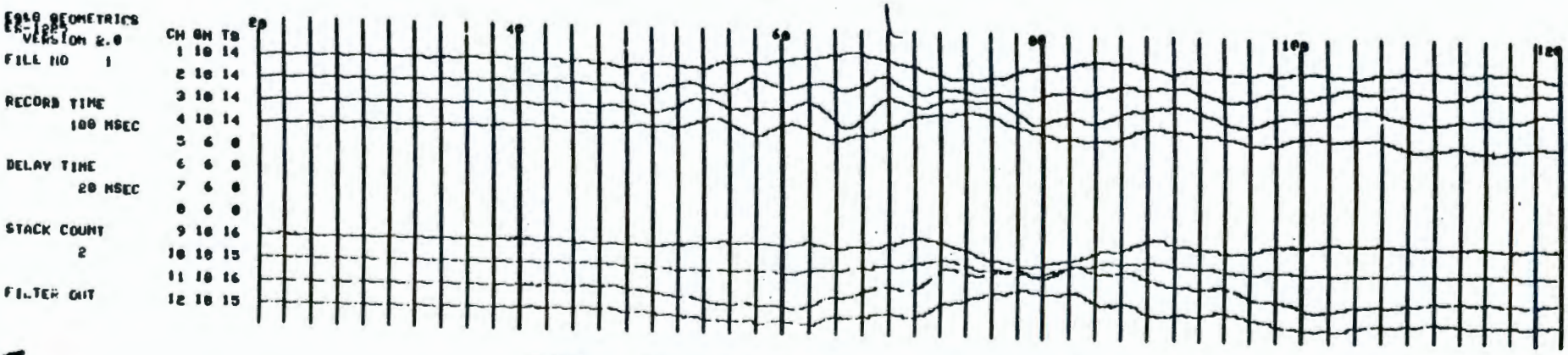
Downhole Seismic Test Data



S-Wave Depth Interval: 80'-90' Impact Direction: Right



S-Wave Depth Interval: 80'-90' Impact Direction:Left

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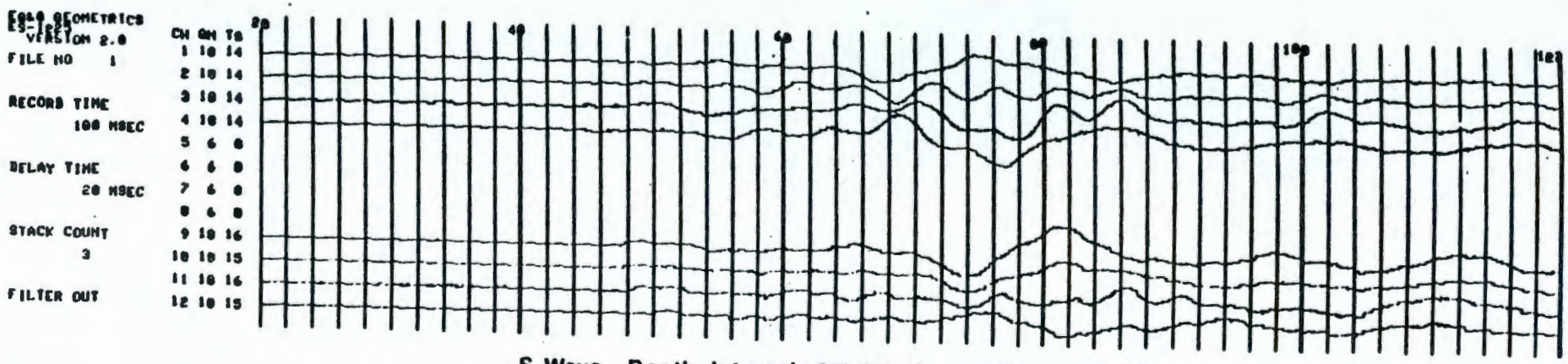
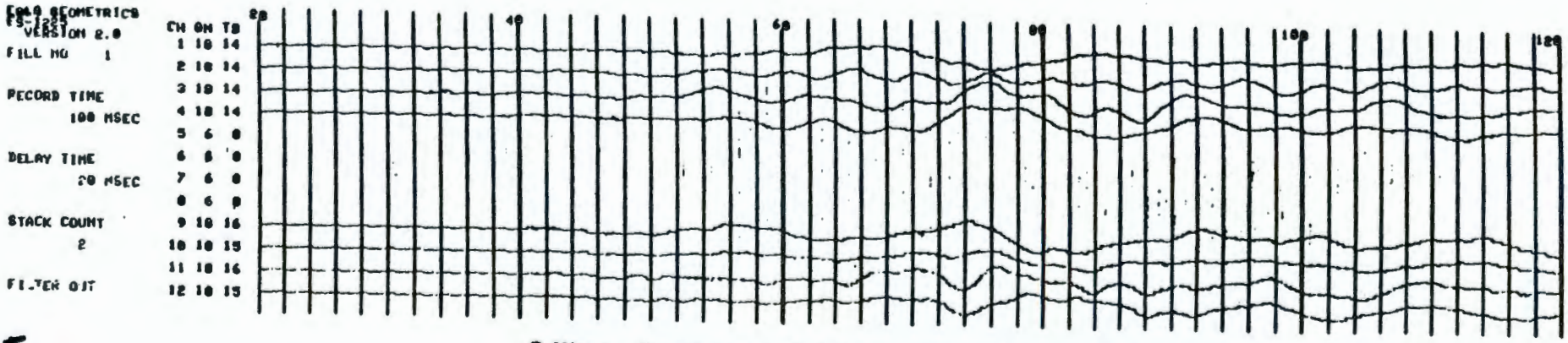
Downhole Seismic Test Data



S-Wave Depth Interval: 85'-95' Impact Direction: Right



S-Wave Depth Interval: 85'-95' Impact Direction:Left

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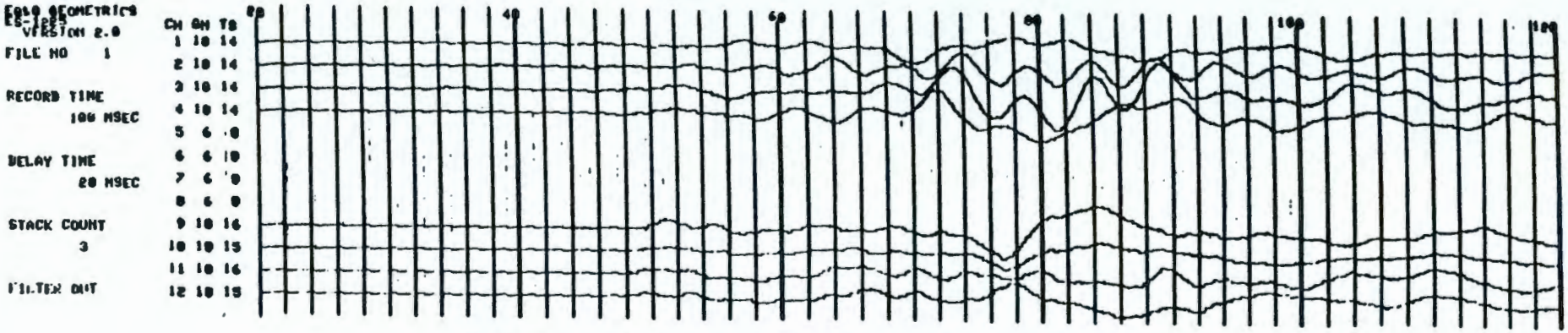
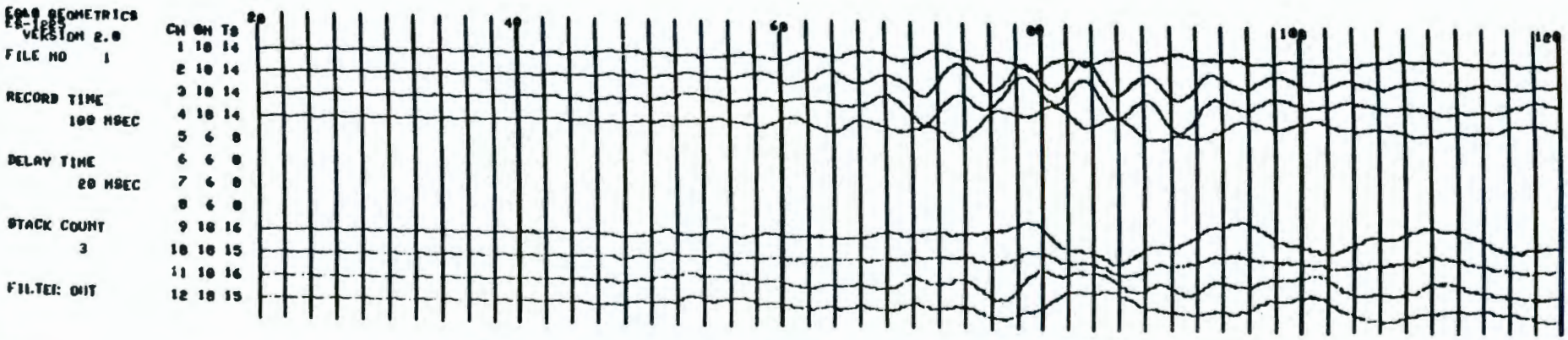
Downhole Seismic Test Data



S-Wave Depth Interval: 90'-100' Impact Direction: Right



S-Wave Depth Interval: 90'-100' Impact Direction:Left

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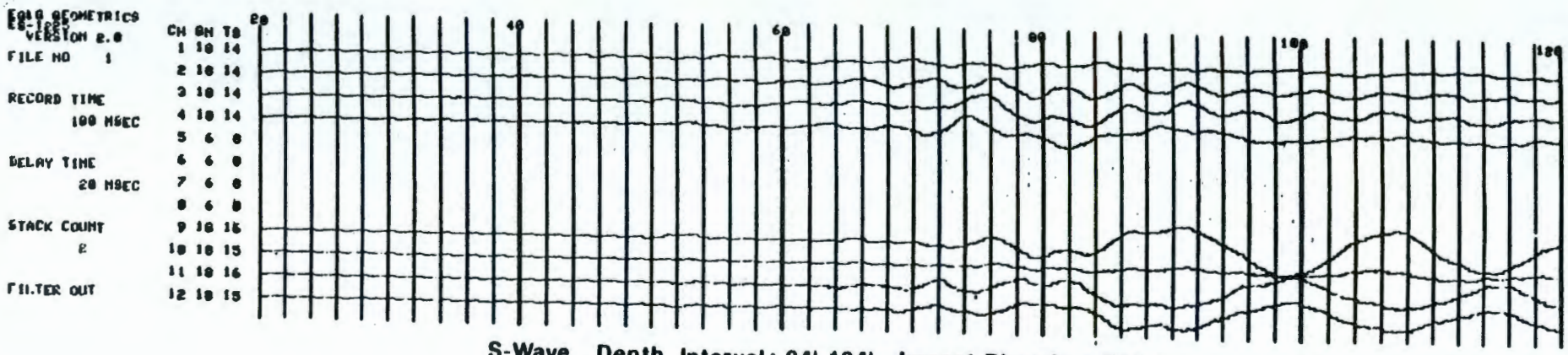
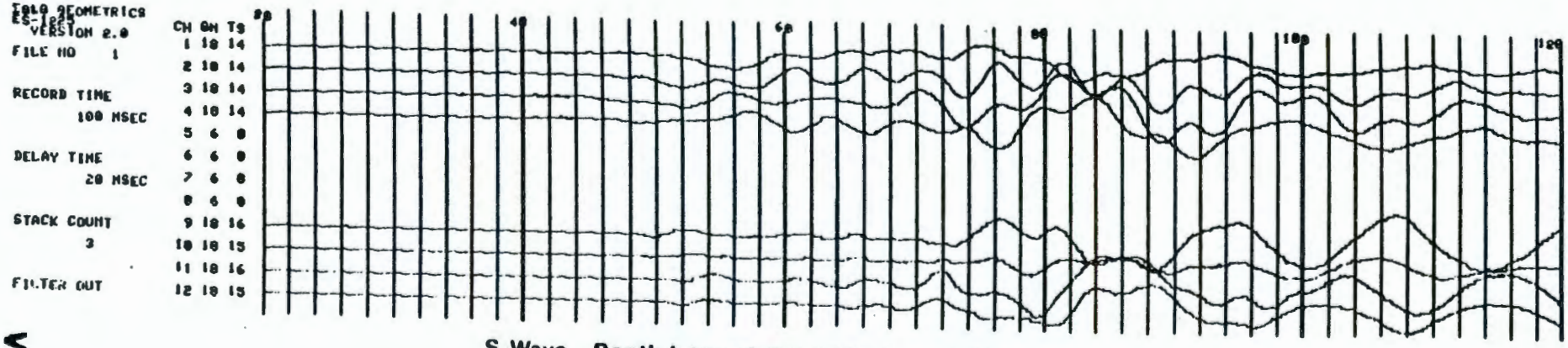
Downhole Seismic Test Data



S-Wave Depth Interval: 94'-104' Impact Direction: Right



S-Wave Depth Interval: 94'-104' Impact Direction:Left

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