New Folder Name Requested Info for UGO

Westinghouse
Hanford Company

P.O. Box 1970 Richland, WA 99352

HGO-T900051-00-D

August 28, 1990

Mr. F. B. Asiri
California Institute of Technology
102/33 E. Bridge Laboratory
Pasadena, California 91125

Dear Fred:

REQUESTED INFORMATION FOR LIGO

During our meeting in July you requested additional information in several areas. This letter is in response to that request.

It was noted that a new laboratory planned for Hanford also has some rigorous vibration isolation requirements, and information about those requirements was requested. I have found that the specifications are not as rigid as expected; the laboratory requires that motion be limited to less than 1/10th micrometer peak to peak for frequencies of 1 hz or greater. A recent meeting with interested A.E.'s revealed that integrated circuit development laboratories routinely require significantly better isolation, and 1/10th micrometer is well within the state-of-the-art. If needed, I can get the names of two or three contractors qualified for this type of work.

Frost penetration criteria was requested. Attachment A is the standard which defines minimum depth of underground water lines at Hanford.

It was noted that the plots of temperatures on pages 3-67 of report DOE-RL-89-15 are difficult to interpret. Attachment B is an excerpt from a report that describes the same data and helps clarify the confusion. As I recall, Bert Sweetser wanted to know what the two plots represented, i.e., different geographic points, different times, etc. I was confused myself until I noted the fine print which describes one plot for daily maximum temperatures and one for daily minimum temperatures.

In response to Steve Reidel's presentation on horizontal motion at the site, information about the deformation rate was requested. Total motion was presented, but the rate, e.g., micro-inches/mile, was not included. This information is provided in Attachment C. Also, a seismic reflection line was requested for Hanford (Skagit/Hanford), and this information is provided in Attachment D.

Finally, Steve Reidel has provided more soil property information which is specifically relevant to the proposed LIGO site and is included as Attachment E.

If there is any additional information we can provide, please do not hesitate to call. We enjoyed your visit to Hanford very much and look forward to seeing you again in the future.

Sincerely yours,

Jeff

J. M. Grover, Manager
Reactor Programs

Attachments

ATTACHMENT A

Grover L5-55

SDC-3.2

STANDARD DESIGN CRITERIA

FOR

MINIMUM DEPTH OF UNDERGROUND WATER LINES

RECEIVED

AUG - 2 1990

J. M. GROVER

This Section consists of one page only.

DESCRIPTION OF REVISION		HANFORD PLANT STANDARDS AEC - RICHLAND, WASHINGTON	NUMBER
New Title Block		ARCH-CIVIL DESIGN CRITERIA	SDC-3.2
PREP OR REV BY HES/WD Byrd	ORIG ISSUE DATE 4-10-64	MINIMUM DEPTH OF UNDERGROUND WATER LINES	REVISION NO.
APPROVED CS Bucholz	DATE 8-20-73		2

SDC-3.2

STANDARD DESIGN CRITERIA

MINIMUM DEPTH OF UNDERGROUND WATER LINES

A. SCOPE

This Standard Design Criteria specifies the minimum depth of earth cover for underground water lines installed within the area of the Hanford Plant.

B. MINIMUM DEPTH OF EARTH COVER

1. Frequent Flows

Where water in pipes and mains is frequently used, causing a regular flow in the line, the minimum earth cover above the top of pipe shall be 30 inches. Pipes and mains located under roads, streets, parking areas, or other locations where the earth cover will be well compacted, the minimum earth cover above top of pipe shall be 36 inches.

2. Infrequent Flows

Where water in pipes and mains does not flow normally, or where water is infrequently used (dead end installations, pipe lines servicing safety showers, fire hydrants, etc.), the minimum earth cover above top of pipe shall be 42 inches.

[2] From: Stephen P Reidel at ~WHC37 8/7/90 11:18AM (676 bytes: 9 ln)
To: Jeffrey M Grover at ~WHC76
Subject: LIGO DATA

------------------------------ Message Contents ------------------------------

I am getting together the information for items 9 and 10 but Tony Riewe never called with his concern about the unconsolidated top sands in the report that he has. I will put together an explanation of the differences between the HWVP site and the one that they are interested in using. I believe that that will solve his problem.

I will be out of town until next Monday. I hope to have everything by the end of that week.

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ATTACHMENT B

MONTHLY EXTREMES OF DAILY MAXIMUM AND MINIMUM TEMPERATURES

These data are presented in Table 5 and Figure 5. Note that there is a greater temperature range during the winter months than in summer. December temperatures have ranged from 69°F to -27°F, a range of 96°, while July temperatures have ranged from 115°F to 39°F, a range of 76°. The December minimum temperature has ranged from 56°F to -27°F, an 83° range, while July minimum temperatures have varied only from 82°F to 39°F, a 43° range.

It is possible for high winter maxima to exceed low summer maxima. For example, on January 31, 1972, the maximum temperature reached 72°F while the high temperature for July 2, 1966, was 59°F.

High winter minimum temperatures may also exceed low summer minima. The minimum temperature on December 2, 1975, was 56°F compared to July 2, 1979, which was 39°F.

NORMALS AND EXTREMES OF DAILY TEMPERATURES

Figure 5 is a graph of daily extremes of maximum and minimum temperatures and the daily normals. Note that the large ranges between the extremes of maxima and minima temperatures occurred during the winter months. During winter also the minimum extremes depart more from normal than do the maximum extremes.

RECORD COLD TEMPERATURES

The periods of December 1 through 17, 1919 (17 days duration), January 24 through February 5, 1950 (13 days duration), and January 12 through February 3, 1957 (23 days duration) were record-breaking cold periods. These three periods accounted for 7 of 10 days of record with a daily maximum temperature of $\leq$0°F and for 9 of 16 days of record with a minimum temperature of $\leq$-20°F. Some comparative statistics follow:

TABLE 5. Monthly Extremes of Daily Maximum and Minimum Temperatures and Daily Temperature Ranges for Hanford Townsite (Near) and HMS Based on Period of Record 1912 Through 1980

	DAILY MAXIMUM TEMPERATURE				DAILY MINIMUM TEMPERATURE				DAILY TEMPERATURE RANGE*			
	RECORD HIGHEST		RECORD LOWEST		RECORD HIGHEST		RECORD LOWEST		GREATEST		LEAST	
MONTH	°F	DATE	°F	DATE	°F	DATE	°F	DATE	°F	DATE	°F	DATE
JAN	72	31, 1971	-2	31, 1950	53	30, 1971	-23	17, 1916#	43	13, 1974	1	22, 1956#
FEB	71	12, 1924	-3	1, 1950	55	26, 1932	-23	3, 1950#	42	1, 1947	2	16, 1959#
MAR	83	25, 1960	24	3, 1960	54	9, 1942	6	5, 1955	45	24, 1979	2	6, 1957
APR	95	22, 1934#	41	7, 1945	60	21, 1956	12	1, 1935	47	28, 1968#	7	20, 1980
MAY	103	26, 1936#	49	8, 1918	70	19, 1956	28	1, 1954	48	11, 1946	6	26, 1980#
JUNE	110	25, 1912	55	3, 1966	81	30, 1924	33	9, 1933	46	28, 1979	8	28, 1946#
JULY	115	27, 1939	59	2, 1966	82	30, 1925	39	2, 1979	44	16, 1976#	7	2, 1966
AUG	113	4, 1961	63	25, 1920	81	4, 1961	40	18, 1918	45	10, 1979	10	7, 1962#
SEPT	102	1, 1976#	52	23, 1934#	72	7, 1955	25	23, 1926	45	25, 1974#	5	13, 1980
OCT	90	6, 1933	31	30, 1935	60	26, 1945	6	31, 1935	45	4, 1980	2	30, 1971
NOV	75	3, 1975	14	13, 1955#	52	23, 1959#	-1	14, 1955	36	24, 1970	1	26, 1952
DEC	69	26, 1980	-3	13, 1919#	56	2, 1975	-27	12, 1919	46	1, 1975	1	24, 1975#
ANN	115	JULY 27, 1939	-3	FEB. 1, 1950#	82	JULY 30, 1925	-27	DEC. 12, 1919	48	MAY 11, 1946	1	NUMEROUS DATES

* STATISTICS TABULATED FOR THE DAILY TEMPERATURE RANGE ARE BASED ONLY ON THE PERIOD OF RECORD 1945-1980

\# DENOTES ALSO ON EARLIER DATES

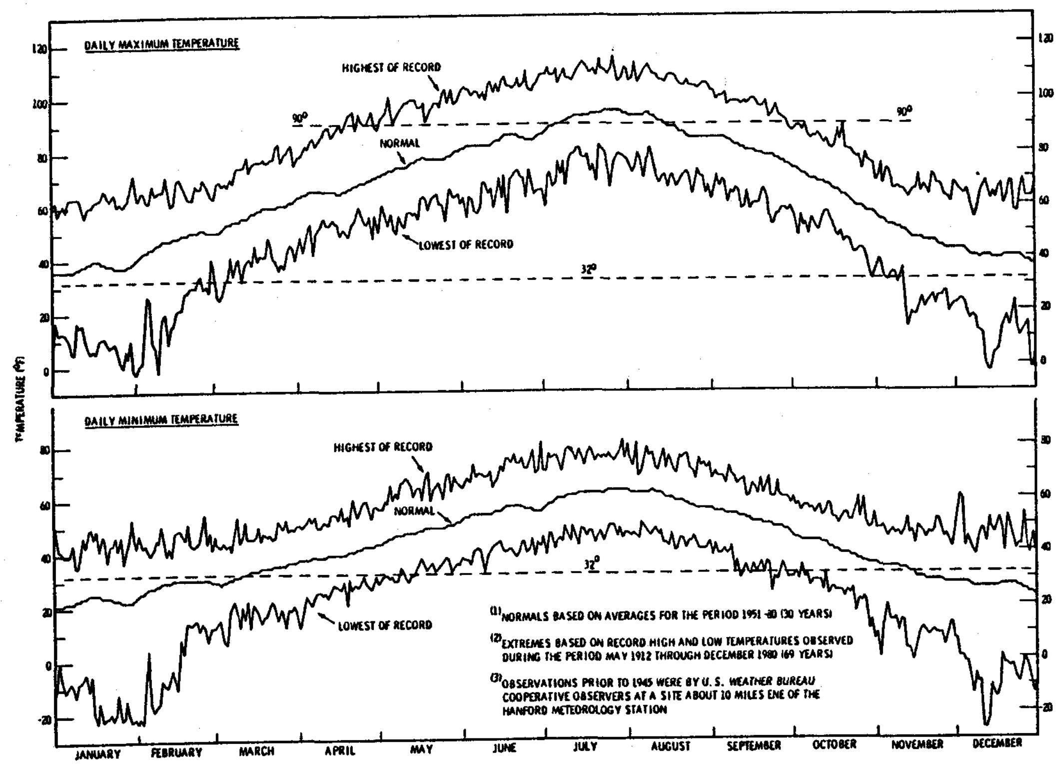

FIGURE 5. Normal[1] and Extreme[2] Daily Temperatures at Hanford[3]

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ATTACHMENT C

north-south cross sections across the western part of the Columbia Plateau and, from them, calculated minimum amounts of crustal shortening. For the areas west of 120° W. longitude, along the 120° W. longitude, and east of 120° W. longitude, the total minimum amounts of shortening are 15, 9, and 4 km (9, 5.5, and 2.5 mi), respectively. Reidel (1984, pp. 972-973) estimated a minimum amount (3 km (1.8 mi)) of shortening at Sentinel Gap, due to folding and faulting of the Saddle Mountains anticline, with at least 2.5 km (1.5 mi) of shortening due to fault displacement alone.

Beck (1976, 1980) and Reidel et al. (1984) noted that varying amounts of clockwise rotation are consistently identified in paleomagnetic studies of rocks from eastern and western Washington and Oregon. Within the Pasco Basin, clockwise rotation of up to 30° in the Pomona flow (12 m.y.B.P.) has predominated, although counterclockwise rotation of as much as 6° has occurred locally (Reidel et al., 1984, Fig. 7). The clockwise rotation reaches its maximum in the crest and hinge areas of the anticlines of the Pasco Basin and decreases toward the synclinal troughs. The amount of rotation appears to be controlled by segments of the anticlines. It has probably occurred along a closely spaced, northwest-trending, right-lateral shear system that developed in the anticlines as they evolved structurally under the presence of north-south compression. Because of the apparently close relationship between the anticline and the rotation, rotation probably began with folding and continued as the folds evolved structurally (Reidel et al., 1984).

1.3.2.4.2 Contemporary crustal movement

Indications of contemporary crustal movement come from geodetic surveys and earthquake seismicity. Studies of contemporary vertical and lateral crustal movement in the Columbia Plateau have focused on the site and have relied primarily on the analysis of geodetic data. The data from these surveys provide information on contemporary crustal deformation through repeated measurements of changes in relative elevation and distance that occur with time. Vertical control data are obtained through leveling surveys, and horizontal control data through trilateration surveys.

The analysis of leveling data for the Columbia Plateau is based on the review and evaluation of survey data and results reported by Tillson (1970). Tillson (1970) used first-order leveling data obtained during the 1904 through 1959 period from surveys along four regional survey lines: (1) Seattle, Washington, to Pasco, Washington; (2) Pasco, Washington, to Spokane, Washington; (3) Pasco, Washington, to Ontario, Oregon; and (4) Pasco, Washington, to Portland, Oregon. Because elevation data for the four survey lines were collected at different times and along numerous line segments, no single point was considered constant for the purpose of calculating absolute uplift or subsidence. Tillson's results are, therefore, relative vertical velocities calculated for the remaining survey lines relative to the Seattle, Washington, to Pasco, Washington, line. From these relative vertical velocities, Tillson constructed a velocity contour map of the Columbia Plateau that was used in the analysis of crustal changes.

Tillson's (1970) analysis of the regional leveling data suggests that the Pasco Basin is gradually subsiding at an average rate of approximately 1 mm/yr (0.04 in/yr). Only data from first-order leveling surveys were used to calculate the rates. The most consistent and accurate data come from the Pasco, Washington, to Portland, Oregon, line and show a maximum of 33 mm (1.3 in.) of subsidence at Umatilla, Oregon, in 22 yr. The average rate of subsidence is less than 1 mm/yr (0.04 in/yr) along The Dalles, Oregon, to Pasco, Washington, line. The Seattle, Washington, to Pasco, Washington, and Pasco, Washington, to Spokane, Washington, lines also show an average rate of subsidence of 1 to 2 mm/yr (0.04 to 0.08 in/yr), with a maximum rate of approximately 3 mm/yr (0.12 in/yr) at Selah, Washington. The leveling surveys do not indicate any significant movements where the lines cross known tectonic structures. In particular, the leveling data do not indicate vertical displacement at Wallula Gap, where the Pasco, Washington, to Ontario, Oregon, level line crosses the Olympic-Wallula lineament. This line showed 1 mm/yr (0.04 in/yr) of subsidence as far south as Pendleton, Oregon. Tillson (1970) considered the 1-mm/yr (0.04-in/yr) rate of subsidence observed in the Pasco Basin to be comparable with other surveyed areas where crustal movement is minor or negligible.

Tillson's (1970) analysis of triangulation data for the Columbia Plateau provides no evidence for horizontal crustal movement. All position changes were less than the magnitude of the errors associated with the measurement techniques or were attributable to the instability of surveying monuments.

The analysis of leveling data for the site is based on data from a 20-km (12-mi) first-order level line that traverses the reference repository location in a north-south direction from Gable Mountain to Rattlesnake Mountain (Prescott and Savage, 1984, pp. 22-28; Fig. 1.3-67). This survey line was established to investigate possible contemporary vertical crustal movements in proximity to the reference repository location. The line consists of 15 monuments that were emplaced in 1982 and 5 preexisting benchmarks. The line was surveyed initially in 1982 for baseline elevation data and again in 1983 to assess monument stability. It is not possible to compare directly the elevations observed for this 1-yr interval for several reasons: not all first-order surveying specifications were adhered to in the two surveys, different sets of intermediate monuments were used during the two surveys, and some monuments were reset without being tied to the original location (Prescott and Savage, 1984, pp. 23-24). However, Prescott and Savage believe that a baseline now exists to which the data from future surveys can be compared.

Horizontal control data come from a trilateration network established by the U.S. Geological Survey in 1972 to measure horizontal strain accumulation in the Pasco Basin (Savage et al., 1981). This network consists of 19 monuments and their corresponding trilateration observation lines (see Fig. 1.3-67). Nearly all these lines have been surveyed eight times: twice in 1972 and once each in 1973, 1975, 1978, 1979, 1981, and 1983. During 1981, nine additional monuments were established for the BWIP to expand the previously established network in the reference repository location. The purpose of this expansion was to measure any horizontal strain that might be accumulating along the Rattlesnake-Wallula alignment.

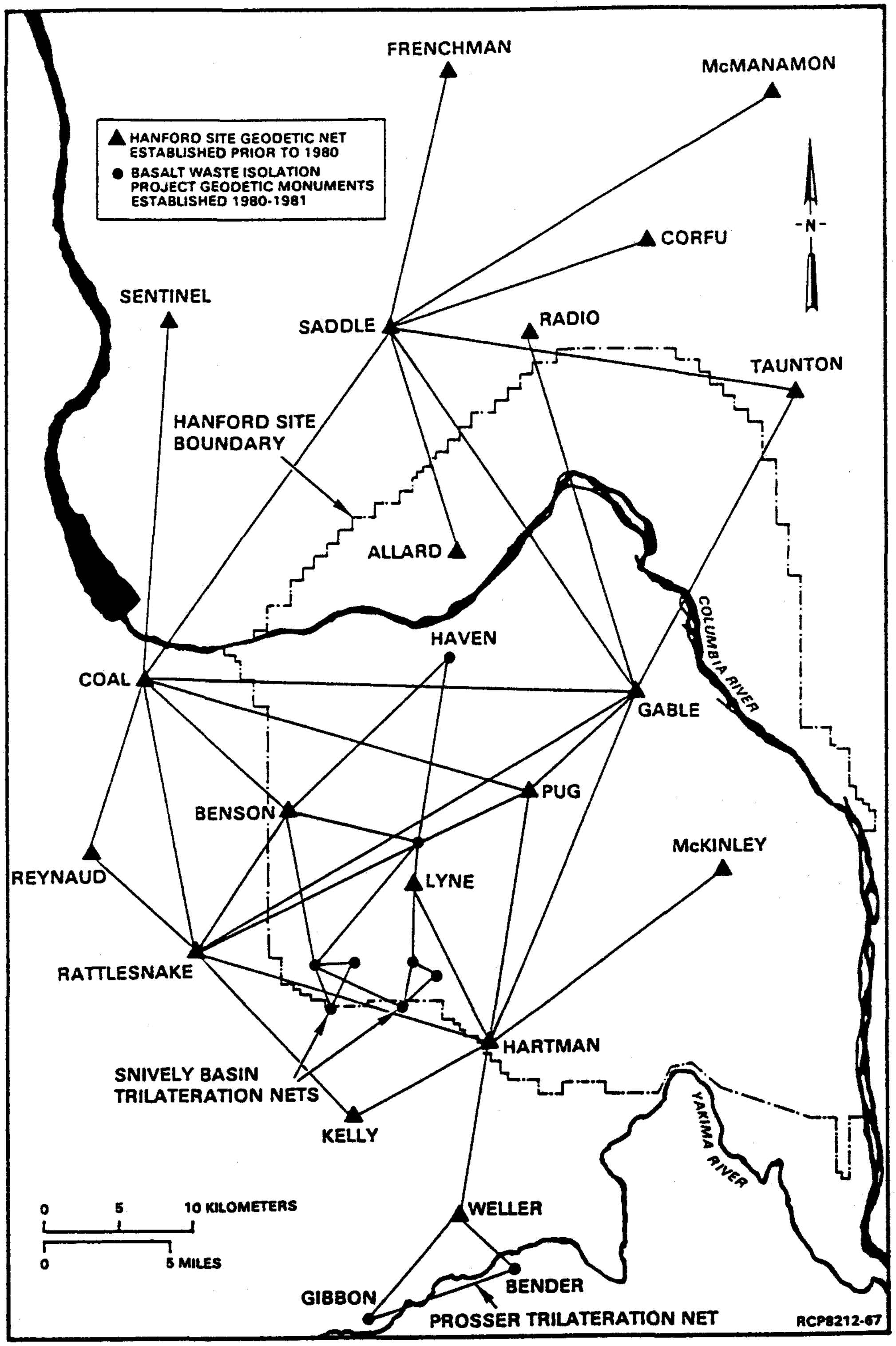

Figure 1.3-67. Locations of geodetic trilateration survey lines within and immediately adjacent to the Hanford Site, Washington (Prescott and Savage, 1984).

A survey of the 29 lines of the Hanford trilateration network was last conducted in 1983 (Prescott and Savage, 1984, pp. 4-20). The results of this survey indicate only minor horizontal changes since the 1981 survey. The horizontal changes for the 1982 to 1983 period could represent uniform strain accumulation, with principal strain rates of -0.016 ±0.013 microstrain per year oriented N. 3° W. ±34° and -0.024 ±0.013 microstrain per year oriented N. 87° E. ±34° (Prescott and Savage, 1984). The horizontal changes from 1981 to 1983 observed across the Cold Creek syncline suggest north-south shortening at a rate of -0.27 ±0.22 microstrain per year (Prescott and Savage, 1984, p. 6).

However, these measured changes are not statistically significant at the 95% confidence level and are within the error limits for the recording equipment. Thus, it is not evident that deformation is being measured. To verify that shortening is being recorded, surveying of the network will be necessary for several more years (see Section 8.3.1.2.4.3.1 for geodetic survey plans). However, the present data, which show very low rates of horizontal changes in the surveys, are consistent with the long-term, low-average rates derived from geologic information.

1.3.2.5 Geothermal regime

1.3.2.5.1 Geothermal characteristics

1.3.2.5.1.1 Temperature

Temperature data provide a direct measure of subsurface heat energy, which is the primary target of geothermal assessment. Numerous temperature surveys and geothermal resource assessments have been made through the Washington State Department of Natural Resources and through DOE-funded contracts (Schuster, 1974; Blackwell, 1974; Korosec and Schuster, 1980; Biggane, 1982; Korosec et al., 1983; Widness, 1983; Stoffell and Widness, 1983; Blackwell et al., 1985; and Barnett, 1986).

A temperature gradient contour map of Washington (Fig. 1.3-68) was published by the Washington State Department of Natural Resources (Korosek et al., 1983). This map demonstrates the general configuration of temperatures in southeastern Washington and includes the Hanford Site. Based on the temperature data selected and analyzed in the above-mentioned report, the geothermal gradient in the Hanford Site ranges from 35 to 55 °C/km (152 to 210 °F/mi).

The ambient temperature for the Cohassett flow at the approximate proposed repository depth of 1 km (3,280 ft) can be determined from borehole temperature surveys. The temperature data available that are closest to the proposed repository location are from borehole RRL-2A (Wintczak, 1984, pp. 30-37). Two BWIP contractors ran temperature surveys in the borehole. Birdwell ran an open-hole temperature log that indicates 36 °C (97 °F) at 826 m (2,710 ft). Extrapolating the temperature gradient to the center of the

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ATTACHMENT D

Excerpt - Site Investigation Report for FFTF, Richland, WA for USAEC
December, 1970
Bechtel Corp.

3. SEISMIC VELOCITY AND ELASTIC MODULI MEASUREMENTS

A seismic survey was made at the site as part of the evaluation of foundation conditions including the potential for liquefaction and also to analyze the dynamic response of foundations and structures during earthquakes.

Seismic measurements were made to determine in-situ seismic wave velocities of overburden and rock materials. The cross-hole technique was used for the detection and measurement of both "P" (longitudinal compressional) and "S" (shear, transverse) wave velocities to a depth of 200 feet ±.

Thicknesses of layers with different velocities were determined by seismic refraction measurements. The lengths of the refraction lines were extended for penetration to the basalt layer shown to exist at depths of approximately 500 feet.

All of these measurements indicate relatively high velocity and moduli values for materials underlying the thin surface layer which is about 15 feet thick. The velocity data for the deeper Ringold Formation indicate that this material has properties comparable to many rock strata, namely high velocity values for both "P" and "S" waves.

Average velocity and shear moduli data are shown in the following table:

AVERAGE VELOCITY AND MODULI DATA

Depth (feet)	Vp (ft./sec.)	Vs (ft./sec.)	G (psi)	Poisson's Ratio (ν)
0 - 15	1200	550	7.2×10^3	0.36
15 - 110	2800	1600	6.1×10^4	0.26
110 - 195	3600	1800	7.7×10^4	0.33
195+	8900	4200	4.2×10^5	0.36
500±	15000	----	--------	----

This work was performed for Bechtel by Weston Geophysical Engineers Inc; their report is given in Appendix VI.

4.0 FOUNDATION INVESTIGATION

4.1 SUMMARY

A foundation investigation program was carried out to select a site location and develop preliminary criteria for design of the foundations for the Fast Flux Test Facility Structures at Hanford Reservation, Washington.

The investigation consisted of drilling five holes, excavating eleven test pits, obtaining soil samples in the drill holes and test pits, determining the in-situ density of overburden soils, testing samples in the laboratory, and analyzing the data obtained.

The overbruden consists of a shallow mantle of windlaid sand underlain by glaciofluvial sands to a depth of about 125 feet, where gravelly sands are encountered. Beneath the glaciofluvial sands and gravelly sands at a depth of 150 feet there is a dense gravel stratum of the Ringold Formation. This is underlain at 200 feet by a succession of strata of cemented or partly cemented siltstone, claystone and conglomerate down to the Yakima Bassalt Formation at 595 feet. The water table was encountered between the depths of 170 to 175 feet in the borings.

The investigation showed that the proposed plant can be constructed at the site. The loose surface soils should be removed down to dense material and replaced with compacted sand or sand and gravel at the locations of all permanent structures. Structures can be supported on spread footings or mat foundations and criteria for design and construction are given. The net allowable bearing pressure recommended for one inch total settlement is 3 tsf for spread footings and mats 20 feet wide or larger, and 4 tsf for spread footings 5 feet wide. Recommended values are also given for intermediate footing sizes. Spread footings smaller than 5 feet are not recommended when using these recommended values. Continuous wall footings may be narrower provided that the minimum width

is limited to 2 feet. The net allowable bearing pressure for such continuous wall footings may be determined by multiplying the net allowable bearing pressure for a 5-foot spread footing by the ratio of the wall foundation width to 5 feet.

All granular material for backfill beneath and adjacent to structures can be obtained from the required foundation excavation or from nearby gravel pits. The material should be placed in layers and compacted with a heavy vibratory roller.

Active pressures are presented for the design of retaining walls that are not restrained at their tops. Where structures are rigid or where they cannot deflect as backfill is placed, the at-rest pressures are recommended for design and values are presented in the report which take into account the degree of compaction of the backfill material. The great depth to the ground water table combined with the high relative density of the site soil indicates a high margin of safety against liquefaction of the site soil due to seismic effects.

Temporary slopes in the sand should be excavated to 1.5 horizontal to 1 vertical. Permanent cut slopes in the dense natural overburden soil should be constructed to 2.5 horizontal to 1 vertical to be stable under all conditions including the Design Basis Earthquake condition. However, under the same conditions, permanent slopes in compacted fill should be 2.75 horizontal to 1 vertical.

Spring constants and damping factors are presented for use in dynamic analyses.

Further soil exploration requirements for final design are outlined briefly.

4.2 DESCRIPTION OF THE SITE

The proposed site is located on a wide bench of glaciofluvial materials in the south-central part of the Hanford Reservation about 12 miles north northwest of Richland, Washington, as shown on Fig. 4.1. The Columbia River is about four miles to the east of the site and the Rattlesnake Hills are to the south-west and about 8 to 10 miles away. The ground surface is about elevation 550 at the site.

The overburden consists mainly of glaciofluvial sands grading to gravelly sands at a depth of 120 to 125 feet and these become gravels and sandy gravels at a depth of about 150 feet. This is overlain by a shallow mantle of windlaid sand. Beneath the glaciofluvial sands and gravelly sands lies the Ringold Formation consisting of silty sands, gravels, silt-stone, claystone, sandy claystone and conglomerate. The upper member encountered at the site is gravel which at a depth of about 200 feet becomes compact enough to be called conglomerate although it has little or no cementation. The Yakima basalt formation lies beneath the Ringold formation at a depth of about 595 feet. The present ground water table is in the Ringold Formation at a depth of about 170 to 175 feet.

4.3 SOIL EXPLORATIONS

4.3.1 <u>Drill Holes</u>

Five exploratory borings were drilled at the locations shown on Fig. 4.2. The work was inspected by Bechtel. All borings were logged and sampled to determine the characteristics of the material encountered. The holes were drilled with a Model 22W and a Model 60L Bucyrus Erie cable tool drilling rig. Samples were taken at 5 feet or closer intervals. Standard penetration tests were made at intervals of from about 5 to 10 feet and Shelby tube samples were taken at wider intervals in the overburden sands. A special drive sampler was used to obtain samples in gravel.

The following sample types were taken and the sample locations are shown on the drill logs in Appendix I.

a. Split spoon - standard penetration tests were made in accordance with ASTM Designation D1586 by advancing the sampler with a 140-pound weight dropping 30 inches.

b. Shelby - thin walled, 3-inch ID driven with the drill stem and jars.

c. Drive Barrel - this consisted of 6-inch pipe and was also used in dry sands to advance the hole.

d. Special Drive Barrel - a thick walled sampler used to recover gravel samples.

e. Bailer - these were taken to visually classify the sample cuttings during drilling.

The following tabulation gives the drill hole number, ground surface elevation, water table elevation at the time of drilling, and depth of drilling. It should be noted that the cable tool rig drilled only to a depth of 567 feet in hole Number 2. The hole was then cored to a depth of 649 feet. The top of the basalt was encountered at a depth of 595 feet in that hole.

Drill Hole No.	Ground Surface El.	Water Table El.	Depth Drilled (feet)	Coordinates South	Coordinates West
1	554.2	380.6	220.0	1,059.6	7,458.8
2	554.3	385.5	649.0	1,008.3	7,458.4
3	549.5	380.3	206.5	760.1	7,459.7
4	553.5	383.0	206.0	1,109.8	7,459.1
5	556.7	-	209.5	1,058.2	7,310.4

The drill hole logs in Appendix I, in addition to giving the sample locations and a description of the material encountered, show the results of standard penetration tests. The standard penetration test locations and the soil types encountered in the borings are summarized on Fig. 4.4.

4.3.2 Test Pits

Eleven test pits were excavated at the locations shown on Fig. 4.2. The holes were excavated with a track-mounted backhoe to depths up to 18.5 feet to obtain measurements of the field density and relative density of the soils encountered. A description of this part of the work and the results of the field tests are given in the Shannon and Wilson report in Appendix IV entitled, "Report on Sampling of Soils and Laboratory Testing FFTF Site, Richland, Washington," dated January 24, 1970. The locations of all test pits are shown on Fig. 4.2.

A geologist from Shannon and Wilson logged each test pit and their report gives the logs of all eleven test pits together with the location of density test samples. The test pits indicated similar types of soils at relatively similar elevations. The Shannon and Wilson geologist divided the surface soils into five principal zones designated Zones I, II, III, IV, and V; and the boundaries of these zones are shown on the logs of the test pits given in their report. Shannon and Wilson's Zone V was further sub-divided into Zones Va, Vb, and Vc. The subdivision of the surface sands into zones is of interest from a geologic point of view; therefore, a more complete description is given in the geologic section of this report, Section 2. However, from a soil mechanic's point of view, the relative density is more important, so no further discussion of the origin of the surface sands appears warranted here.

All test pits were backfilled after completion of field testing.

4.3.3 Field Testing

Field density tests were made at 33 locations in the 11 test pits with a six-inch sand cone since the materials encountered were sands and silty sands. Ottawa standard sand was used in the sand cone and the U S Bureau of Reclamation Test Designation E-24 procedure was followed. A description of the work is given in the Shannon and Wilson Report in Appendix IV. Material for additional laboratory testing was obtained at the location of each field density test for shipment to the soils laboratory.

4.4 FOUNDATION CONDITIONS

The explorations indicate that the overburden at the site consists of glaciofluvial sands and gravels to a depth of about 150 feet. These are covered with a thin mantle of medium to fine grained surface sands having a depth of 11 to 17 feet. Beneath the glaciofluvial sands at a depth of 150 to 157 feet lies the Ringold Formation consisting of gravels, sandy silts, and gravels variably cemented (siltstone and conglomerate), and beds of sandy clay variably indurated (claystone and sandy claystone). Beneath the Ringold at a depth of about 595 feet is the Yakima Basalt Formation which is the hard bedrock at the site. The ground water table is at a depth of about 170 to 175 feet below the present ground surface.

The materials down to the siltstone and conglomerate of the Ringold Formation may be divided into four distinct soil

s which are loose, uniform, medium
The uppermost 5 to 10 feet (Shannon
es I and II) have been transported
nd their surface is presently
a surface cover of sagebrush. There
material in the uppermost foot of
his stratum varied in thickness from
the boring locations.

oose nature, this material would be
is unsuitable in its present state
of permanent structures.

urface sands is a stratum of <u>dense</u>
feet in thickness which, except
y, are generally similar in appearance
ands. However, there are lenses of
ained or fine grained sands within
t is typically tan to buff colored,
edium to fine grained, arkosic sand.
pan" zone about 5 feet thick at the
tum. Rounded, fine-gravel size
iche are present in the "hardpan."
Wilson's report in Appendix IV shows
in the "hardpan" layer reacts
drochloric acid indicating the presence

se <u>gravelly sands</u> underlies the dense
thickness of 20 to 35 feet. The
to consist of either lenses of gravel
ilty sand or interbedded gravelly
sands.

se <u>silty gravels</u> underlies the gravelly
to 60 feet thick. This has been

obtained from the drill
ined from the test pits
son, Inc. of Seattle,

ped to the laboratory in
addition, bulk samples
sacks to the laboratory.

ed in accordance with ASTM
ations were based on ASTM
of Soils for Engineering
System). The reaction of
oric acid was determined,
abulations of visual
Wilson report, Appendix

types of tests and number

	Number of Tests
	31
	16
	5
	5
	5
49)	36
	4

The results of the laboratory tests are summarized in Table 4.1 at the end of this Section.

The visual classification had disclosed that a large proportion of samples had reacted to some degree to the application of hydrochloric acid, thus indicating the presence of carbonate. The strongest reaction occurred in the hardpan layer beneath the surface sands. The drill logs indicate the carbonate as "caliche" and it is readily apparent as a white film, coating the underside of gravel in gravel pits at the Hanford Reservation. Because of the presence of the carbonate, an effort was made to determine if any samples were cemented and, by making unconfined compression tests, determine the magnitude of the cementation. Because of the dry nature of the sand, it was not found possible to obtain undisturbed samples for this, although a tube sample from Test Pit TP-1 yielded two short sections which were tested. The short length of the samples would result in substantial end effects so that the compressive strength was reduced by using a correction factor of 0.9 to get an approximate evaluation of the compressive strength due to cementation. Two samples were prepared by moistening sand from drill hole DH-2, preparing test specimens, and heating them to 60 degrees C in an oven overnight. It was considered that this might approximate the conditions under which the sand deposit was built up in nature. These specimens were then tested in unconfined compression, but their strengths were very low. The results of the unconfined compression tests are given below:

Sample Location	TP-1	TP-1	DH-2	DH-2
Sample Depth (feet)	16	16	27	80
Unconfined Compressive strength (tsf)	1.25	1.38	0.15	0.51

The samples from TP-1 were taken in the "hardpan" zone, hence the higher strength.

SEISMIC SURVEYS

Seismic measurements were made at the site by Weston Geophysical Engineers, Incorporated. The work by Weston is discussed in the Geology portion of this report. However, because of their importance, data from the seismic explorations which are pertinent to foundation design are tabulated below:

Depth (feet)	"P" Wave (ft/sec)	"S" Wave (ft/sec)	Shear Modulus (G) (psi)	Poisson's Ratio (ν)
0- 15	1,200	550	7.2×10^3	0.36
15-110	2,800	1,600	6.1×10^4	0.26
110-195	3,600	1,800	7.7×10^4	0.33
195 +	8,900	4,200	4.2×10^5	0.36

The high shear wave velocities confirm the dense nature of the overburden sands, gravelly sands and silty gravels as determined by the drilling. The shear moduli values shown above are required for the dynamic analyses made by Dr. H. B. Seed.

4.7 DISCUSSION AND RECOMMENDATIONS

4.7.1 General

The surface mantle of loose sand varied in thickness from 11 to 17 feet in the drill holes. The test pits indicated that the loosest material is encountered in the uppermost 6 to 12 feet. The loose surface sand stratum is too loose to support permanent structures with tolerable settlements, but the underlying dense sand will make a very suitable foundation for supporting heavy foundation loads. This is evident from the excellent performance of structures founded on the glaciofluvial materials elsewhere at Hanford, as discussed in Shannon and Wilson's report dated Sept. 1965 and entitled, "Review of Hanford Reservation Soils and Geology as Related to Proposed 200 BEV Accelerator."

It is recommended, therefore, that the loose surface sands should be removed down to dense material and replaced with compacted sand or sand and gravel at the locations of all structures. All granular fill beneath structures should be placed in layers and compacted with a heavy vibratory type roller. All spread and wall footings should extend at least 3 feet below adjacent plant grades. All mat foundations 20 feet or more in width should be founded at a depth of at least 8 feet below adjacent plant grade or alternatively, they may have edge walls extending to a depth of 8 feet in order to obtain sufficient edge support.

The high relative density combined with the great depth to the ground water table eliminates the possibility of liquefaction of the site soils. This matter is discussed in more detail in Dr. H. Bolton Seed's report in Appendix V.

4.7.2 <u>Allowable Bearing Pressures</u>

The ultimate bearing capacity of dense sand is very high except in the case of narrow foundations; therefore, the allowable bearing pressure is that load which will result in tolerable settlements of foundations. In sands this load is a function of the width of the footing and the depth below the ground surface. The allowable bearing pressure for foundations has been determined using the relative density of the soil as determined in test pits in the field and as obtained by testing undisturbed samples from the drilling. The results of these tests are shown on Fig. 4.5. In addition, the measured relative density of the overburden soils was checked by empirical rules relating the relative density to the standard penetration test blowcounts obtained during the drilling. The procedure given in a discussion by Peck and Bazzara in the Journal of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, Volume 95, No. SM-3, May 1969, was used to relate the relative density to the standard penetration test blowcounts.

a. Shallow foundations are defined as those which are located at a depth no greater than their least dimension. The net allowable bearing pressure of shallow foundations for one inch total settlement will be as follows:

Least width of footing (feet)	5	10	15	20
Net allowable bearing pressure (tsf)	4	3.4	3.1	3

Spread footings smaller than 5 feet are not recommended.

Mat foundations larger than 20 feet may be designed using an allowable bearing pressure of 3 tons per square foot.

Continuous wall foundations may, of course, be narrower than 5 feet, but they should not be narrower than 2 feet. The net allowable bearing pressure for such continuous wall foundations may be determined by multiplying the net allowable bearing pressure for a 5-foot spread footing by the ratio of the wall foundation width to 5 feet. For example, if a continuous wall foundation is 2 feet in width, then the net allowable bearing pressure will be 40 percent of the allowable bearing pressure for a 5-foot wide spread footing. The allowable pressures given above may be increased by 1/3 for transient live loads such as earthquake and wind.

Settlement of structures founded on or in the dense sand will occur almost as quickly as the load is applied; therefore, post-construction settlements of foundations designed on the above basis will result mainly from live loading and will be substantially less than 1 inch.

If the net applied pressure is less than the net allowable given above, the reduced settlement may be estimated on the basis that it will be roughly equal to the ratio of the net applied loading to the net allowable loading times the predicted one inch settlement.

Because settlement under dead load occurs almost as soon as the load is applied, the post-construction differential settlement between two foundations cannot exceed the differential settlement due to the real live load. Post-construction differential settlements between footings is therefore expected to be much less than 1 inch. For this reason differential settlements between adjacent footings are expected to be in the order of 0.5 inches for footings designed with the allowable bearing pressures as tabulated above.

(b) Deep foundations are defined as those which are located at a depth at least 4 or 5 times the least width of the foundation. It is not anticipated that there will be any foundations that conform to this definition; but if there are, it would be conservative to use the net allowable bearing pressures recommended for shallow foundations. Settlements of the deep foundations designed in that manner would then be substantially smaller than those discussed above for shallow footings due to the much greater resistance to deformation at increased depths in sand.

(c) Foundations on compacted fill may be designed using the net allowable bearing pressures given for shallow foundations provided that the fill is placed in layers and compacted by heavy vibratory rollers.

4.7.3 Sources of Borrow Material

Excavation for the main plant complex will provide a sufficient quantity of granular material for backfill beneath and adjacent to structures; therefore, material from required foundation excavation may be stockpiled for use as structural backfill. However, topsoil and any zones rich in "caliche" should be wasted to prevent formation of zones of this material in backfilled areas.

There are gravel pits within the Hanford Reservation from which coarse grained material could be obtained to stabilize exposed areas of sand and prevent erosion and transportation of the fine surface sand by wind action. No soils survey of these pits was made and no tests of gravel material are necessary for its utilization for this purpose, since any reasonably well-graded pit run gravel will be satisfactory.

4.7.4 Compaction Requirements of Backfill

All areas to be backfilled shall be cleaned of all organic material, loose soil and deleterious material such as trash, lumber, etc. Where fill or backfill is required on both sides of structure elements, it shall be brought up evenly on each side to prevent damage due to displacement from unbalanced loading. All structural backfill shall be placed in layers and each layer shall be uniformly spread and compacted with a heavy vibratory roller. Where necessary, the moisture content of the fill shall be adjusted to obtain the required density.

4.7.5 Earth Pressures

Very rigid structures that will not deflect, as backfill is placed, should be designed for earth pressure at rest. Flexible structures or retaining walls that are not restrained at their tops may be designed for active earth pressure.

Earth pressures in overburden or compacted backfill may be computed using the following equivalent fluid unit weights in pounds per cubic foot.

Condition	Equivalent Fluid Unit Weight lbs/cu. ft.
Active Earth Pressure	27
At-rest Earth Pressure, moderate compaction	53
At-rest Earth Pressure, heavy compaction	74
At-rest Earth Pressure, undisturbed in-place soil	42

An earth pressure coefficient of 0.5 has been assumed for cases of moderate compaction where the only compaction is obtained from the equipment placing and spreading the fill. Where the backfill is placed in layers and compacted with a heavy vibratory roller, an earth pressure coefficient of 0.7 was assumed.

4.7.6 Temporary and Permanent Slopes

Temporary excavation side slopes of 1.5 horizontal to 1 vertical will be satisfactory in the sand at the site for all normal static conditions.

Permanent cut slopes of 2.5 horizontal to 1 vertical in the dense natural soil below the surface sand stratum will be satisfactory and stable under all conditions up to and including the design basis earthquake. Permanent cut slopes of 2 horizontal to 1 vertical in the dense natural soil below the surface sand stratum will be satisfactory and stable under all conditions up to and including the operating basis earthquake.

Permanent compacted sand fill slopes constructed to 2.75 horizontal to 1 vertical will be satisfactory and stable for all conditions including the Design Basis Earthquake.

Similarly, permanent compacted sand fill slopes of 2 horizontal to 1 vertical will be satisfactory and stable for all conditions up to and including the Operating Basis Earthquake.

The above stable slopes have been determined by analysis of dry slopes using the infinite slope procedure and assuming an angle of internal friction of 40 degrees for the in-place overburden soils, estimated from the relative densities determined in the exploration. An angle of internal friction of 37 degrees was assumed for the compacted site soils based on a conservative appraisal of angle of internal friction. A factor of safety of 1.1 was considered satisfactory for slopes subjected to earthquake forces.

All exposed cut or fill slopes should be protected from wind erosion by covering them with a layer of pit-run gravel at least 3 inches thick.

4.7.7 Ground Water Control During Construction

The ground water is at a depth of 170 feet and will never be encountered in the foundation construction; therefore, no ground water control procedures are required.

4.7.8 Properties for Dynamic Analyses

Dr. H. B. Seed of the University of California, Berkeley, under a consulting agreement with Bechtel Corporation reviewed and analyzed the field exploration data in order to determine recommended spring coefficients and damping values for dynamic analyses. Dr. Seed's report is given in Appendix V. The following summarizes his conclusions and recommendations concerning the properties of the foundation to be used in the dynamic analyses.

Spring coefficients and damping factors for two configurations of the plant are shown in Table 4.2. Configuration I refers to the Reactor Containment Building alone, while Configuration II refers to the Reactor Containment Building and adjacent Reactor Support Buildings assumed to be so connected that they act as a unit.

4.7.9 Liquefaction Potential of Site Soils

In order to investigate the liquefaction potential of the site soils, Dr. H. B. Seed made a ground response analysis for a magnitude 6.8 earthquake located on a fault 10 miles from the site. The shear modulus variation with depth was based on shear wave velocities and relative densities. The maximum ground acceleration was found to be 0.25 g. Since the earthquake criteria for the site had not been completed at the time Dr. Seed made his analysis, he investigated the relative density required for the soils below the ground water table to provide a factor of safety of 1.5 against liquefaction for maximum ground accelerations ranging from 0.25 g to 0.35 g. The results are tabulated below:

Maximum Ground Surface Acceleration	Relative Density Required to Provide a Factor of Safety of 1.5 Against Liquefaction
0.25 g	45%
0.30 g	54%
0.35 g	63%

The relative densities of the site soils exceed the above values; therefore, there is no danger from soil liquefaction at this site. Dr. Seed's report in Appendix V elaborates this conclusion. The high relative density of the site soils indicates negligible settlements of the site soils due to seismic effects.

4.8 EXPLORATION REQUIREMENTS FOR FINAL DESIGN

The exploration covered by this report indicates that the site conditions are very suitable for the intended structures. However, additional explorations will be required for final design to confirm that similar suitable conditions occur beneath the major structures. These borings should be carried to sufficient depth to predict satisfactory performance of the structures. Fig. 4.6 shows the presently proposed locations and estimated depths of these borings. These locations, number of borings, and depths are subject to change to suit the final locations of major structures.

A test fill will be constructed at the site to determine suitable placement and compaction control procedures for use during construction.

TABLE 4.2

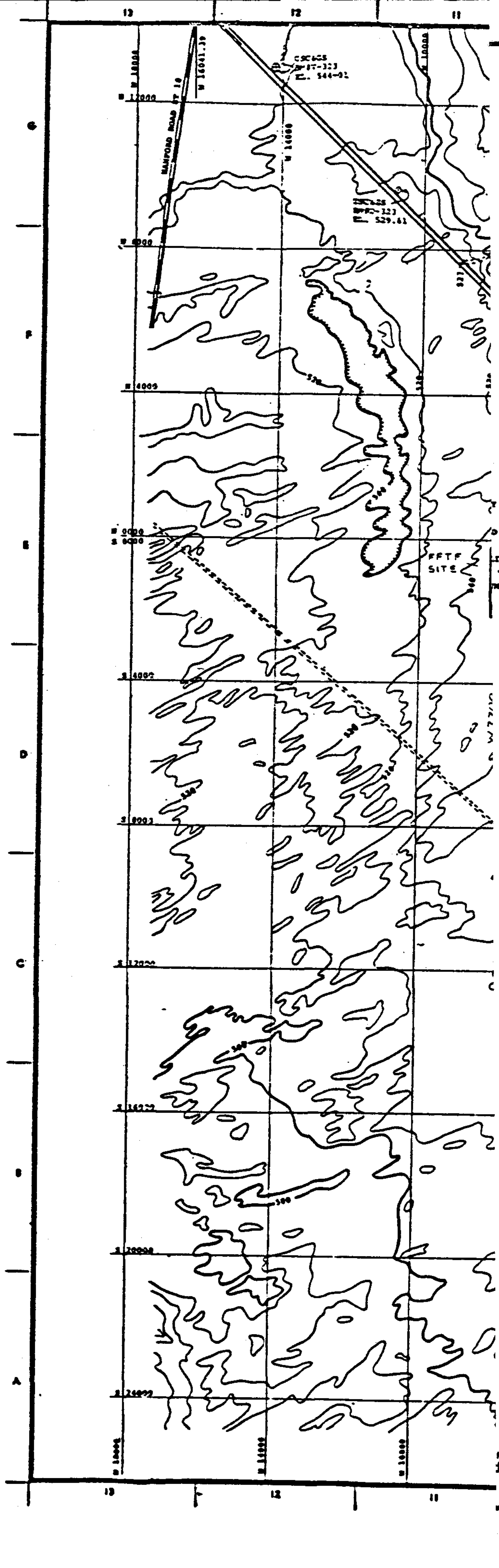
STIFFNESS COEFFICIENTS

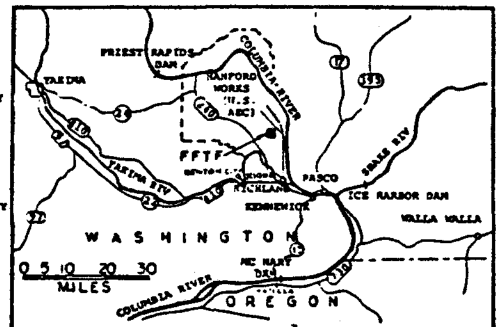
Spring Constant	Operating Basis Earthquake Structural Config. I	Operating Basis Earthquake Structural Config. II	Design Basis Earthquake Structural Config. I	Design Basis Earthquake Structural Config. II
Vertical translation - k_{vv} - kip/ft.	12×10^6	15.5×10^6	11×10^6	14×10^6
Horizontal translation - k_{uu} - kip/ft	10.5×10^6	11.5×10^6	9.5×10^6	10.5×10^6
Rotation or rocking - $k_{\theta\theta}$ - kip/ft/rad	6.5×10^{10}	9×10^{10}	6×10^{10}	8.5×10^{10}
Coupling between rotation and translation- $k_{u\theta}$ - kip/rad	-4×10^8	-4×10^8	-4×10^8	-3.5×10^8

DAMPING FACTORS

	Operating Basis Earthquake	Design Basis Earthquake
Soil damping	1 to 2 %	1½ to 3 %
Radiation damping	2 to 3 %	2 to 3 %
TOTAL DAMPING	3 to 5 %	3½ to 6 %

NEXT USED ON
REFERENCE DRAWINGS



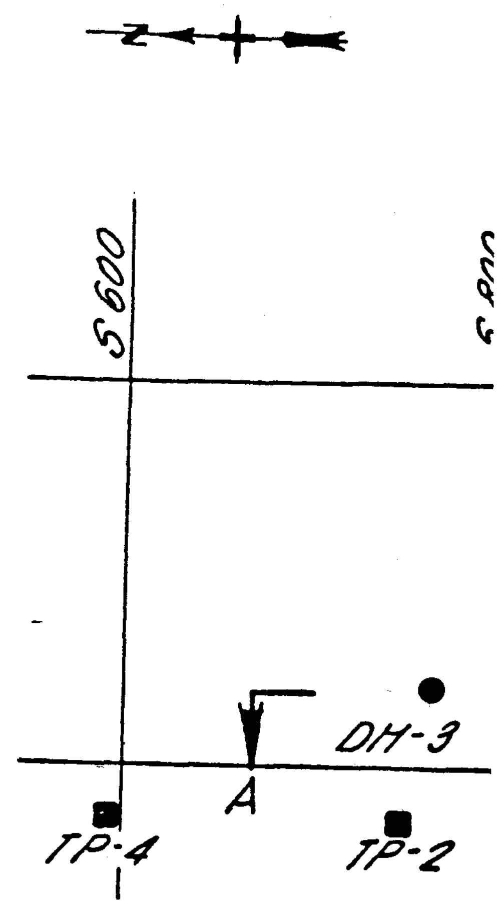


VICINITY MAP

NOTES.

1. THIS DRAWING IS AN ENLARGEMENT OF A PORTION OF A USGS MAP RICHLAND WASHINGTON N 4615 - W 11915/15, 1951.
2. ELEVATIONS SHOWN ARE MEAN SEA LEVEL, NORTH AMERICAN DATUM 1929.
3. THE HANFORD GRID SYSTEM IS SHOWN ON THIS DRAWING. "AID" AND " ALP" ARE TIE-IN MONUMENTS BETWEEN THE WASHINGTON STATE GRID SYSTEM SOUTH ZONE AND THE HANFORD GRID SYSTEM. THE WASHINGTON STATE GRID SYSTEM NORTH IS 0° - 08' - 50" EAST OF THE HANFORD GRID NORTH.

REVISIONS	0
DRAWING STATUS	
U. S. ATOMIC ENERGY COMMISSION RICHLAND OPERATIONS OFFICE	
PACIFIC NORTHWEST LABORATORY OPERATED BY BATTELLE MEMORIAL INSTITUTE	
BECHTEL SAN FRANCISCO	JOB 5653
SITE MAP	
FAST FLUX TEST FACILITY	
SCALE 1" = 1500'	FIG. 4-1



NOTES

1. Cross-sections A-A and B-B are shown on Fig. No. 3.
2. Coordinates shown are Hanford Project Coordinates.
3. Surface elevation varies from 550 feet to 557 feet.

DRILL HOLE NO.	SURFACE ELEVATION Feet
DH-1	554.2
DH-2	554.3
DH-3	549.5
DH-4	553.5
DH-5	556.7

NO.	DATE	REVISIONS	BY	CHKD	DESIGN SUPV	ENGR	PROJ ENGR	APPR
△								
△								
△								

SCALE 1"=100' DESIGNED DRAWN CHIEF ENGR

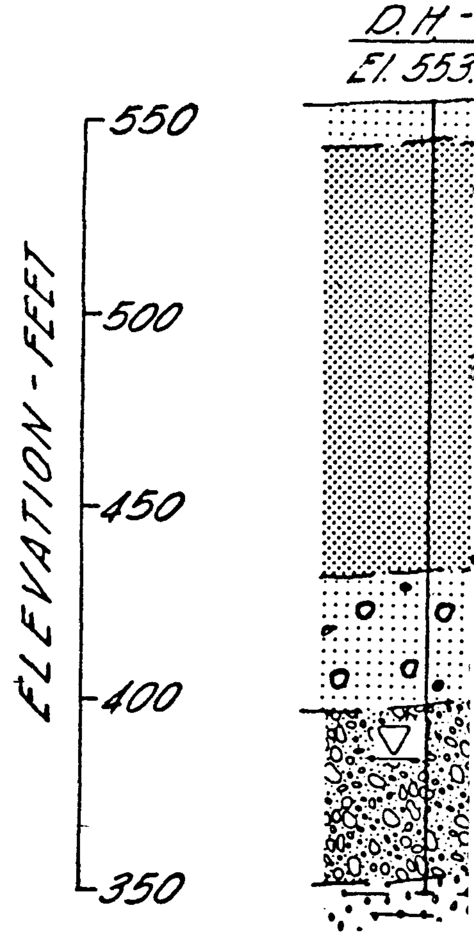
BECHTEL
SAN FRANCISCO

FAST FLUX TEST FACILITY

LOCATION OF DRILL HOLES AND TEST PITS

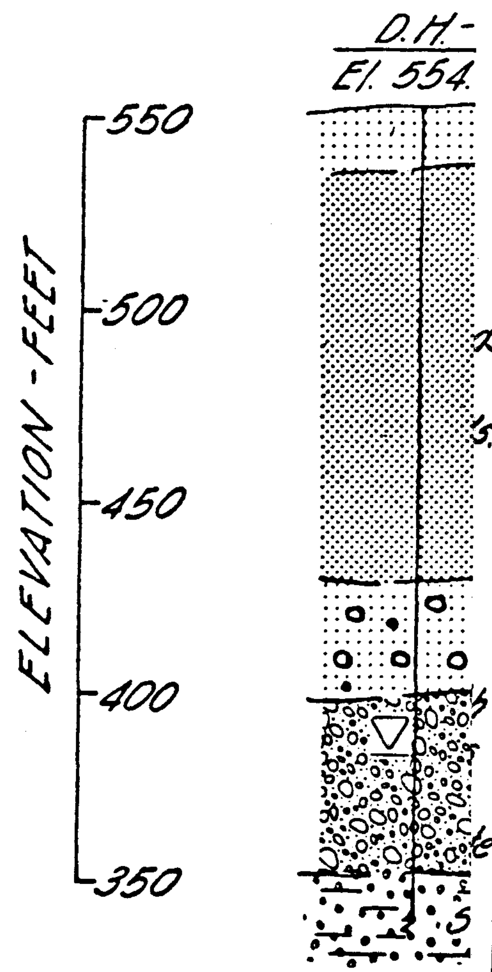
JOB No.	DRAWING No.	REV.
5853	FIG. NO. 4·2	

(3)



NOTES

1. Hole DH-2 was drilled to the Columbia River Basalt encountered at approx. el. -50
2. Plan locations of cross-sections and borings are shown on Fig. No. 2.
3. For a full description of materials in the overburden soil strata see the logs of drill holes and test pits.



No.	DATE	REVISIONS	BY	CHKD	DESIGN SUPV	ENGR	PROJ ENGR	APPR
△								
△								
△								

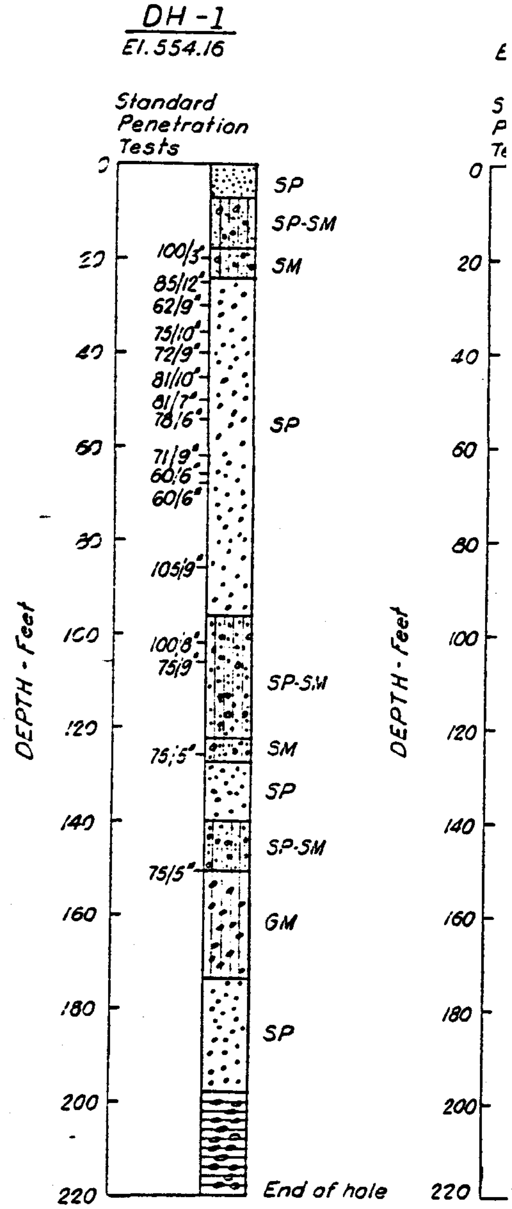
SCALE 1" = 50' DESIGNED DRAWN CHIEF ENGR

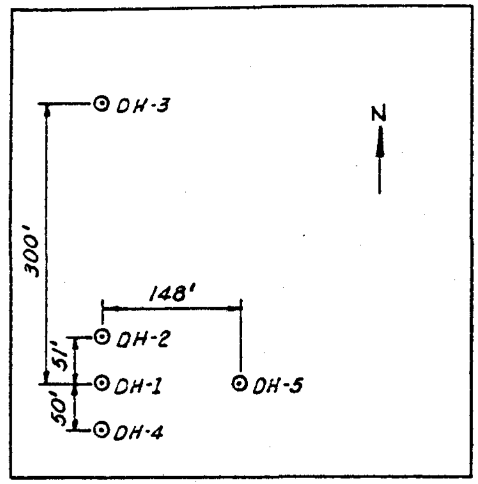
BECHTEL
SAN FRANCISCO

FAST FLUX TEST FACILITY

CROSS - SECTIONS

JOB No.	DRAWING No.	REV.
5853	FIG. NO. 4-3	





LOCATION OF DRILL HOLES
IN PLAN
Scale 1" = 100'

NOTES

1. Coordinates of the drill holes are shown on Fig. No. 4.2
2. The Standard Penetration Tests were carried out in accordance with the procedures in ASTM Designation D 1586. The penetration resistance (N) is defined as number of blows of a 140 lb hammer dropping freely 30 inches required to drive the Standard Split Barrel Sampler one foot. If less then 1 foot was penetrated, the fraction was recorded as shown alongside the drill holes.

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FAST FLUX TEST FACILITY
SUMMARY OF STANDARD PENETRATION TESTS

5853 FIG. NO. 4-4

5

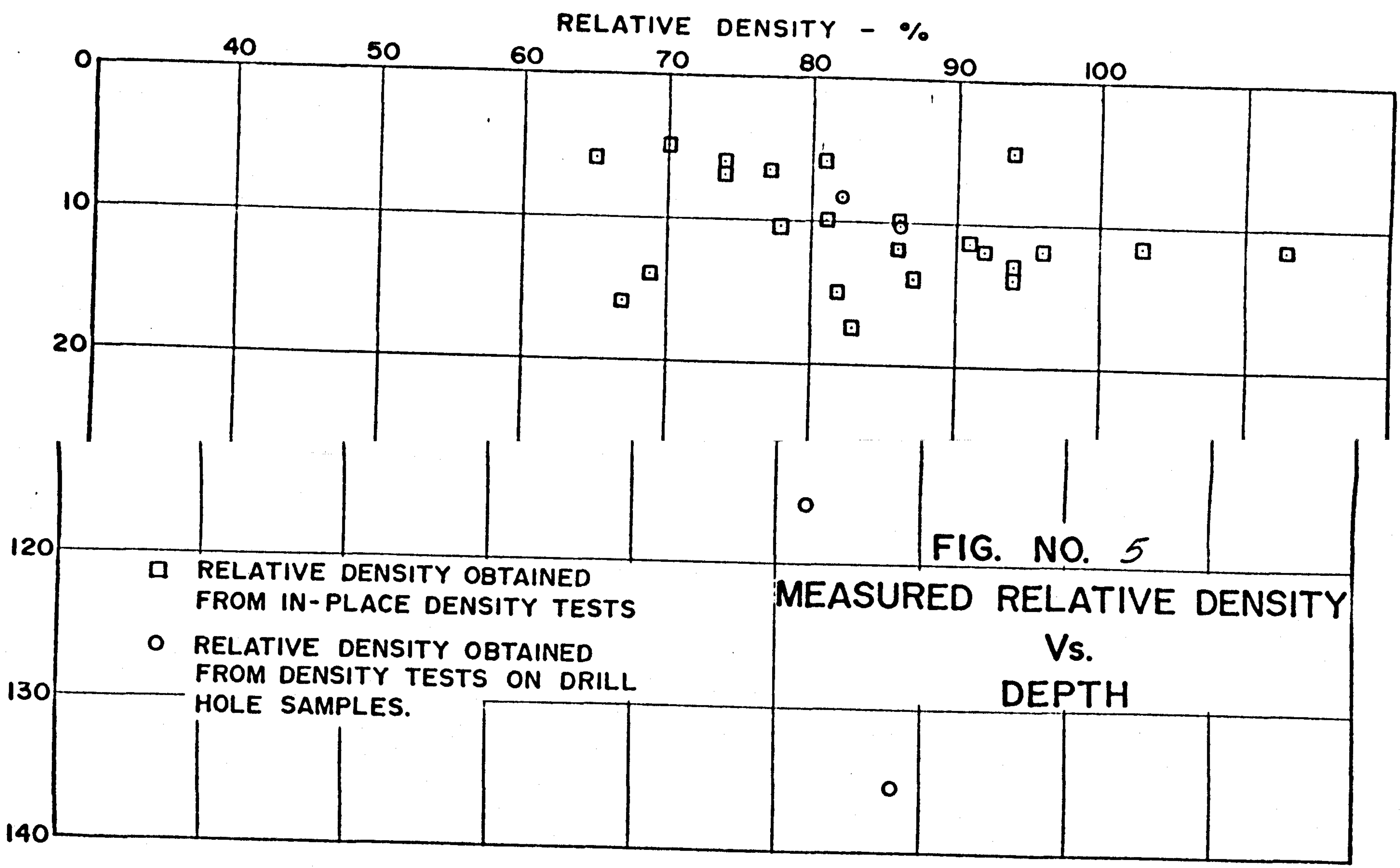


FIG. NO. 5
MEASURED RELATIVE DENSITY
Vs.
DEPTH

FIG. NO. 4.5

BWIP SCP

the northwest and southeast directions. These results do not support the unusually high velocities (8.4 km/s (2.75 x 10^4 ft/s)) shown by Catchings et al. (1984). The 8.4-km/s (2.75 x 10^4-ft/s) velocity could be revised if the crustal model were thinner or had a higher average velocity.

Local refraction surveys

Active seismic surveys on the Hanford Site have been conducted intermittently since 1959. These surveys consisted mainly of short (less than 1 km (0.6 mi)) seismic refraction profiles. They were conducted primarily for site investigations in support of engineering and foundation studies. Other surveys were conducted to test the method and for general exploration.

In 1959, a small test of the method was conducted by Raymond and Ratcliffe (1959) in support of defense radioactive waste management activities. In 1963, a development test of seismic reflection and refraction methods was conducted to test the feasibility of determining the depth to the basalt beneath the Hanford Site. Donaldson (1963) recommended that drilling and seismic refraction be combined to obtain the required data. Additional work was conducted for the Fast Flux Test Facility for foundation studies and to determine depth to bedrock (Blume, 1971).

Washington Public Power Supply System seismic refraction surveys

A significant amount of seismic refraction surveying was conducted for the Washington Public Power Supply System Nuclear Project 2 (WPPSS, 1981) and Nuclear Projects 1 and 4 (WPPSS, 1986), and for the Puget Sound Power and Light Company Skagit/Hanford Nuclear Project (PSPL, 1982). Over 50 km (31 mi) of seismic refraction lines were shot for Nuclear Project 2 (WPPSS 1981, Appendix 2 5D). Some of these lines are shown in Figure 1.3-45. Most of these data were either low resolution for depth-to-bedrock determinations or relatively high resolution for shallow studies to determine overburden material velocities.

The best general purpose data from the Washington Public Power Supply System Nuclear Project 2 survey was obtained along line 1 in the Hanford Road survey area (see Fig. 1.3-45), which was designed to investigate the possible extension of Yakima Ridge. The surface densities of shot and receiver locations were sufficient to give a somewhat detailed picture of not only the top of basalt, but the suprabasalt sediments as well. The results of this survey are shown in Figure 1.3-46. Because of the nature of the sediments and top-of-basalt velocity structures, the contractor-estimated depth errors of approximately 6 m (20 ft) seem to be somewhat optimistic. A more reasonable estimate might be 10% of the depth or velocity, approximately 15 to 20 m (49 to 66 ft) for top of basalt. The ability of the seismic refraction method to detect a fault of the size (6 m (20 ft)) mentioned in the report also should be questioned, because the seismic refraction method averages out variations in the refracting horizon.

Nineteen seismic refraction lines were run for siting the Washington Public Power Supply System Nuclear Projects 1 and 4 (WPPSS, 1986). Four of these lines were low resolution with large shot spacings and determined depth

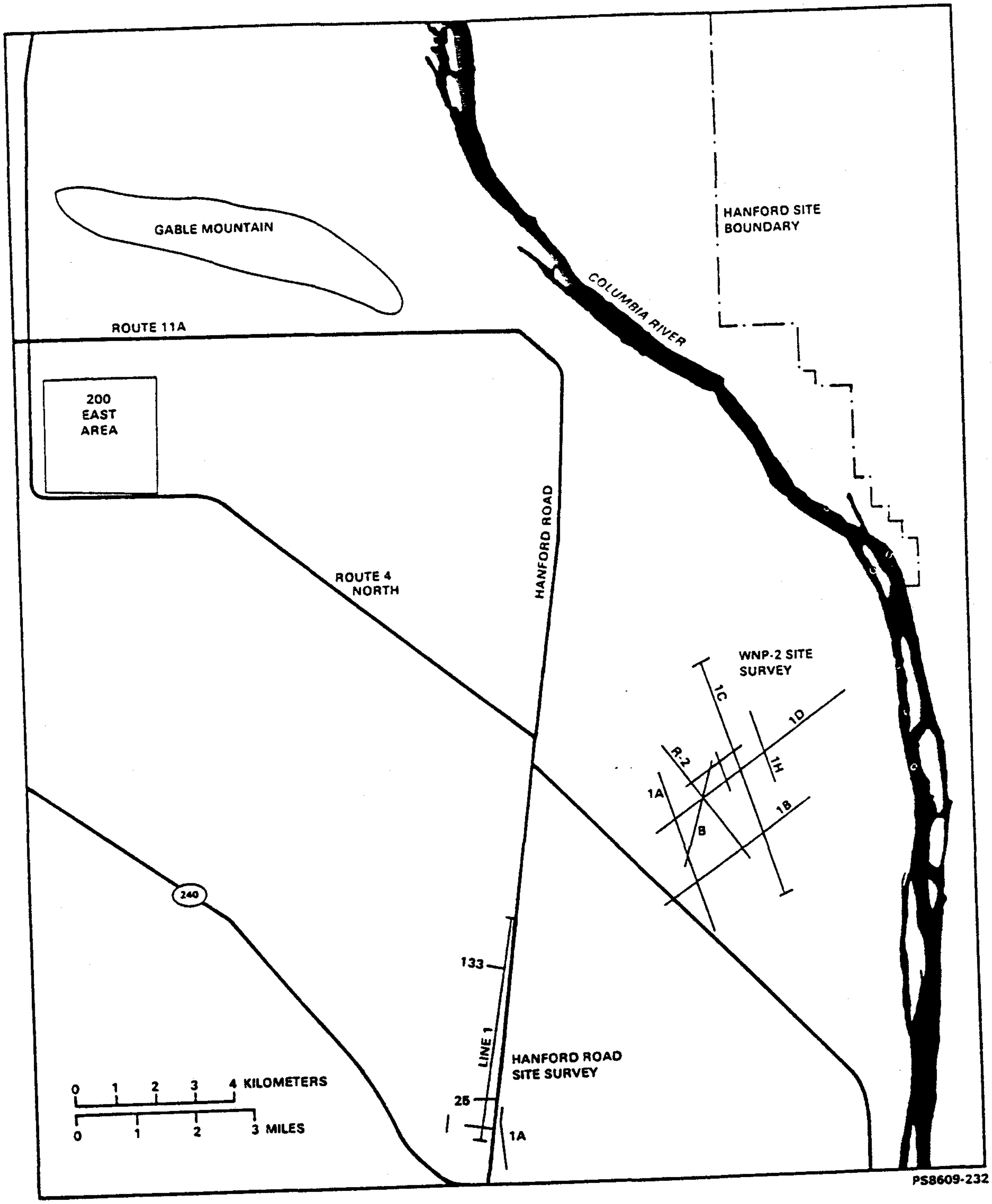


Figure 1.3-45. Locations of seismic refraction lines shot for the Washington Public Power Supply System Nuclear Project 2 (WPPSS, 1981, App. 2.5D).

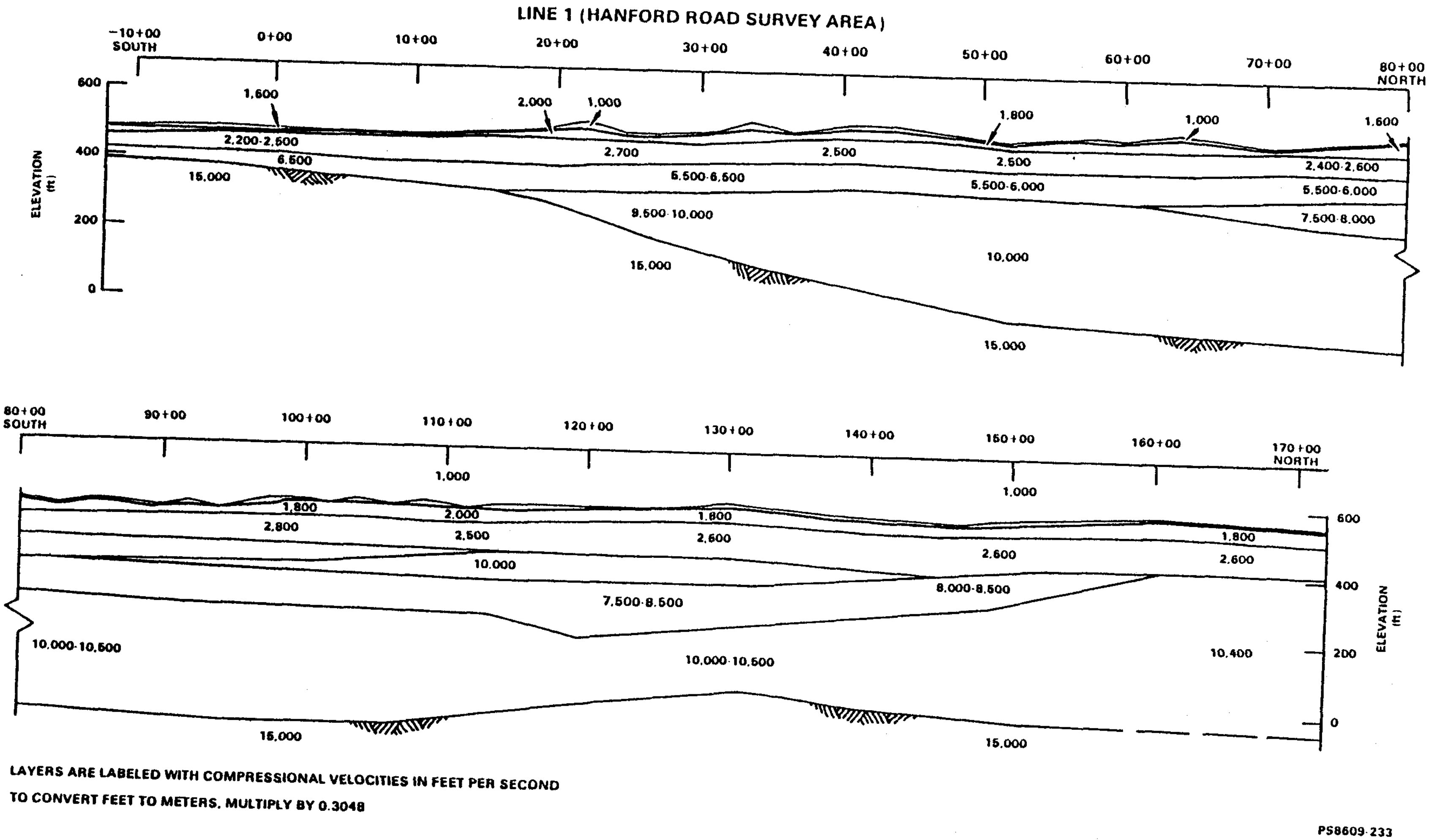


Figure 1.3-46. Results of seismic refraction survey along Line 1 and crosslines of the Hanford Road survey area (from WPPSS, 1981, App. 2.5D). Elevations are above mean sea level. See Figure 1.3-45 for location of Line 1 and crosslines. (sheet 1 of 2)

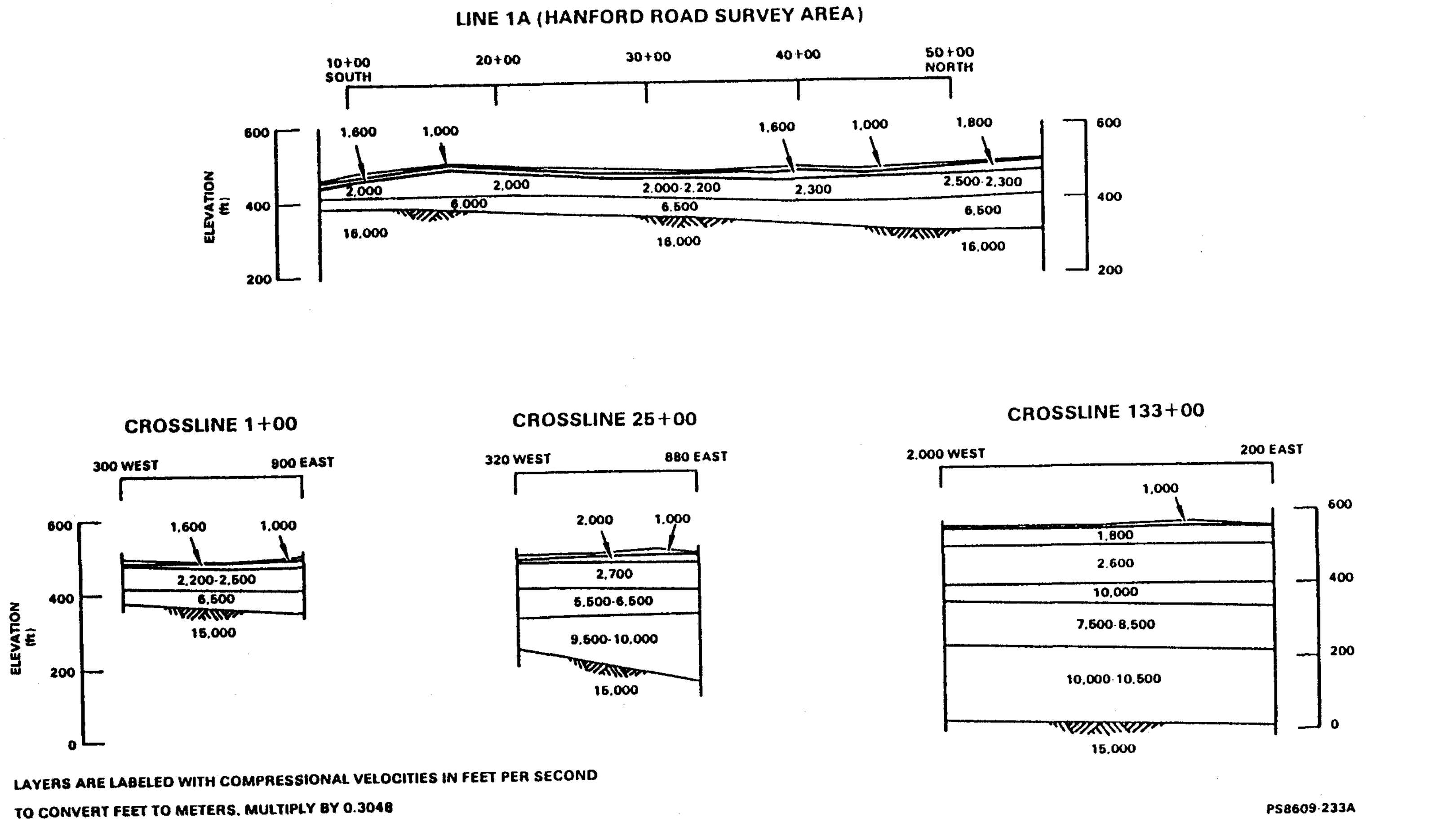


Figure 1.3-46. Results of seismic refraction survey along Line 1 and crosslines of the Hanford Road survey area (from WPPSS, 1981, App. 2.5D). Elevations are above mean sea level. See Figure 1.3-45 for location of Line 1 and crosslines. (sheet 2 of 2)

to bedrock. No sediment or basalt velocity values were given on these sections. No significant features were observed in the bedrock. The 4 low-resolution lines and the other 15 lines were used to determine the velocity structure of the sediments. A high-velocity refractor was observed in the Ringold Formation, but no anomalous features were observed. This high-velocity refractor apparently corresponds to the upper-middle Ringold Formation found in the BWIP survey discussed below.

Skagit/Hanford seismic refraction surveys

The Skagit/Hanford Nuclear Project conducted over 115 km (71 mi) of seismic refraction surveys to determine the velocity structure of the suprabasalt sediments, the depth to top of basalt, and the geologic character of structural features in the area. Except for a few lines in the area of the proposed reference repository location (south of Gable Mountain), most of the lines were used to investigate the Gable Mountain structure and its extensions (PSPL, 1982). A survey, comprising 30 refraction lines, was conducted on a portion of Gable Mountain to investigate a northeast-trending fault reported by Fecht (1978) and a possible pull-apart feature in the same location. The fault could not be traced effectively with the seismic lines, but the seismic lines were subsequently used to plan a trenching survey. The pull-apart feature was hypothesized to be a gravity slide, but was found to be a fractured and weathered zone.

Other lines in the Skagit/Hanford Nuclear Project survey were used to examine the May Junction linear, which was originally defined in BWIP gravity and magnetic data (Myers/Price et al., 1979). This linear is a north-south-trending disturbance in gravity and magnetic features between Gable Mountain and the Southeast anticline described below. It also appears to terminate the DB-10 ridge/anticline described below. The refraction lines, series 7 and 8 in Figure 1.3-47, did not indicate that this feature was fault controlled. As indicated by the central Gable Mountain survey, it is, however, difficult for the seismic refraction method to delineate buried fault features. The seismic reflection data on line FY 79-3 near refraction line 8, described in Section 1.3.2.2.3.5, did show a possible fault feature, which is indicated in Figure 1.3-48, at vibrator point 340. The trend of this fault feature is not known, however, because there are no parallel reflection lines; consequently, its association with the May Junction linear is unknown.

Seismic refraction lines to the northwest of the May Junction linear lines were used to investigate two faults identified by Myers/Price et al. (1979) in borehole DB-10 (see Fig. 1.3-47). These lines, together with other boreholes, showed that the upper fault had a north-south strike with a 32° dip. The lower fault was not encountered in the other boreholes. This would seem to require that it have a northwest-southeast strike. The refraction lines also confirmed a buried anticline that is in line with the west anticline of Gable Mountain. This anticline would be parallel to the lower fault strike. The refraction data, as expected, were unable to confirm the existence of the fault features shown in Figure 1.3-49 (from Myers/Price et al., 1979). The location of these features correspond to the sides of the buried anticline, and the feature at vibrator point 435 could correspond to the lower fault found in borehole DB-10.

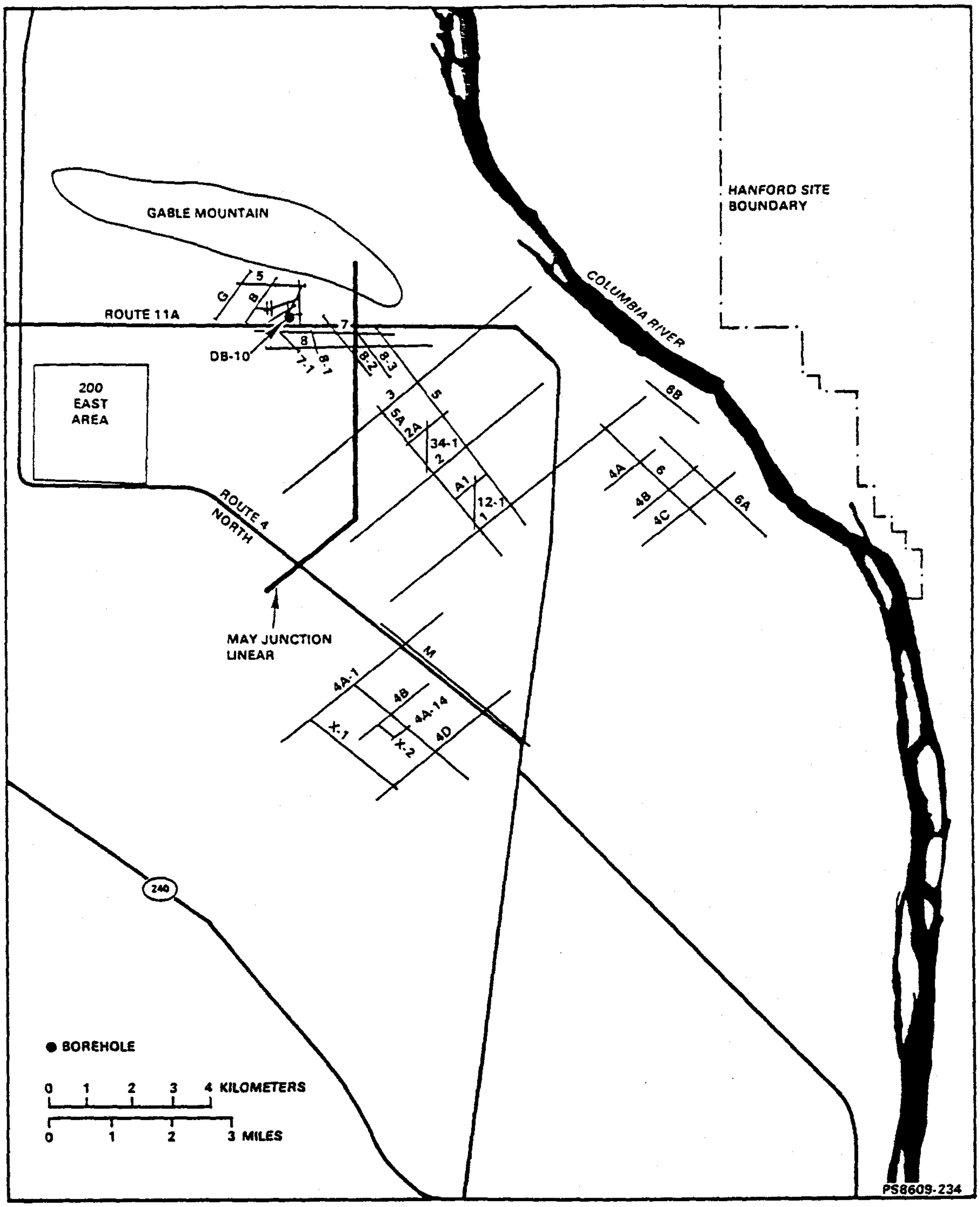


Figure 1.3-47. Location of Skagit/Hanford Nuclear Project seismic refraction survey lines (from PSPL, 1982).

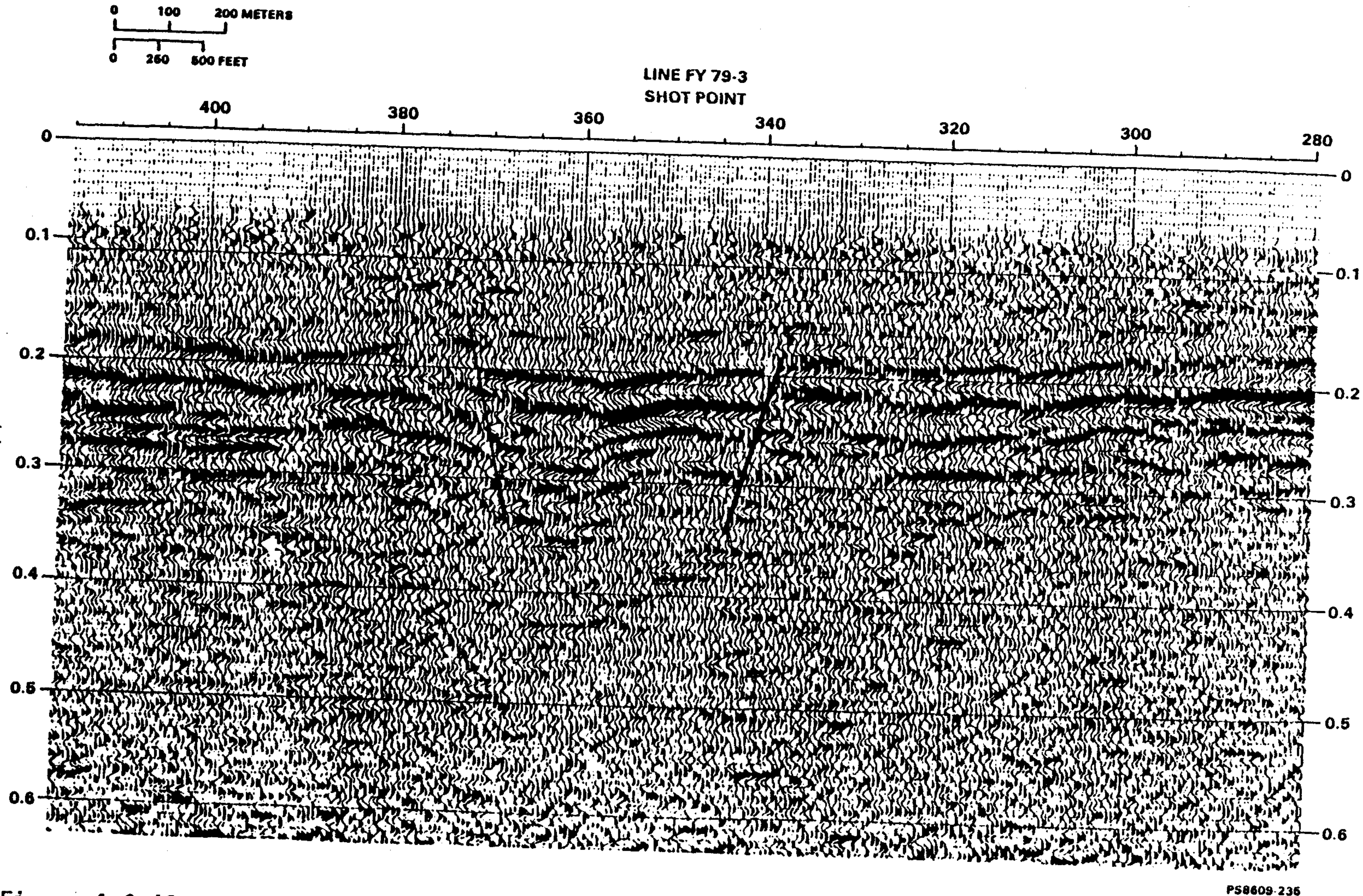


Figure 1.3-48. Seismic reflection line crossing location of May Junction linear from the Basalt Waste Isolation Project seismic reflection survey (SSC, 1979). See FIgure 1.3-54 for location of line FY 79-3. The feature at shot point 340 may be associated with the May Junction linear.

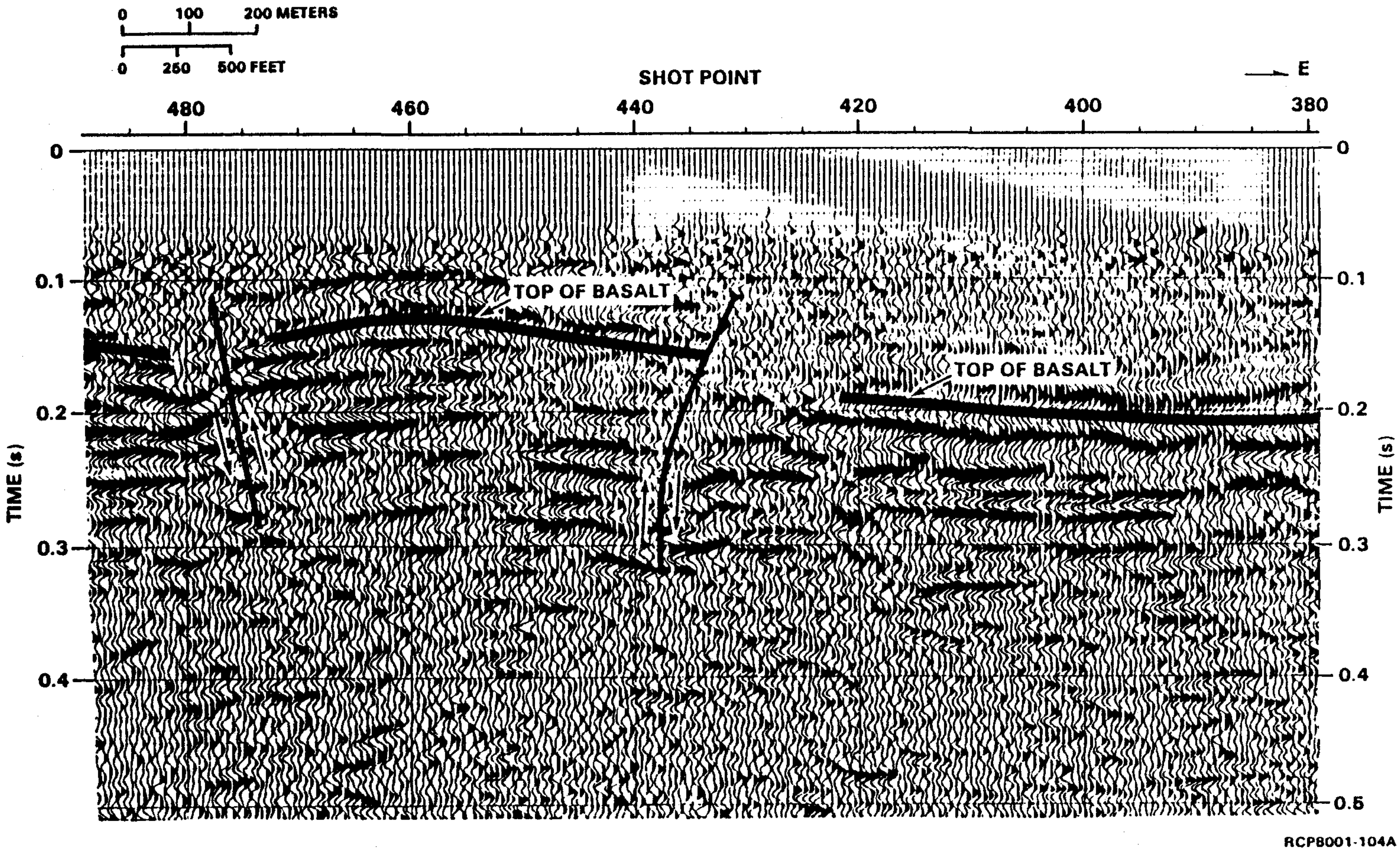


Figure 1.3-49. Seismic relection line crossing location of the borehole DB-10 ridge/anticline from the Basalt Waste Isolation Project reflection survey (SSC, 1979).

Refraction-line series 1 through 6 was used to investigate the Southeast anticline. The top-of-basalt contours from these data show a linear anticlinal feature that appears to be an extension of the Gable Mountain anticline. These anticlines separate areas of sediments with a high-velocity interior layer located in the southwest from areas without this layer located to the northeast. This layer is discussed in detail in Section 1.3.2.2.3.6.

Basalt Waste Isolation Project seismic refraction surveys

Refraction surveys were conducted for the BWIP by Seismograph Service Corporation. Data were collected over eight lines with a total length of 32 km (20 mi) in the vicinity of the reference repository location, as shown in Figure 1.3-50. A low-powered energy source, the Betsy Seisgun (trademark of Mapco, Inc.), was used with 36 shots usually detonated at each shot point. The resulting data do not reveal penetration to the top of the basalt except in certain areas.

The initial interpretation of the data was made by Weston (1982). The interpretation consisted of cross sections of the velocity structure and static correction tables for a 168-m (550-ft) datum above mean sea level. These static data were used in subsequent reflection processing (Berkman, 1984).

A more detailed analysis of the refraction data along line 15 is presented in Mitchell and Odegard (1984) and Odegard and Mitchell (1987). They constructed a velocity-versus-depth section for this line and related it to lithology, using borehole data as shown in Figure 1.3-51. The interpretation of the data also incorporated the vertical seismic profiling data, which are discussed in Section 1.3.2.2.3.6. A comparison was made between the velocity model from the vertical seismic profiling data from borehole RRL-2 and an average of the seismic refraction profiles located around this borehole. The lithologic and stratigraphic interpretation of the borehole core superimposed on the average refraction and vertical seismic profiling velocity data is shown in Figure 1.3-52.

The velocity data have resulted in the middle Ringold unit being subdivided into upper and lower units. (These data also are consistent with the Skagit/Hanford Nuclear Project data (PSPL, 1982, Fig. 2L-2G)). The striking feature of the section shown in Figure 1.3-51 is the relatively large discontinuities in the velocity contours in the middle Ringold. Erosion of the middle Ringold unit, known to exist in the area, appears to be the cause of these discontinuities, although they may be the result of deformation by faulting.

Summary of results

The results to date of the local seismic refraction surveys have been the determination of depths to top of basalt and the delineation of the structure and stratigraphy of the suprabasalt sediments. The results of these surveys have assisted in better interpretations of geologic problems. In addition, they have allowed investigators to extend these solutions into areas

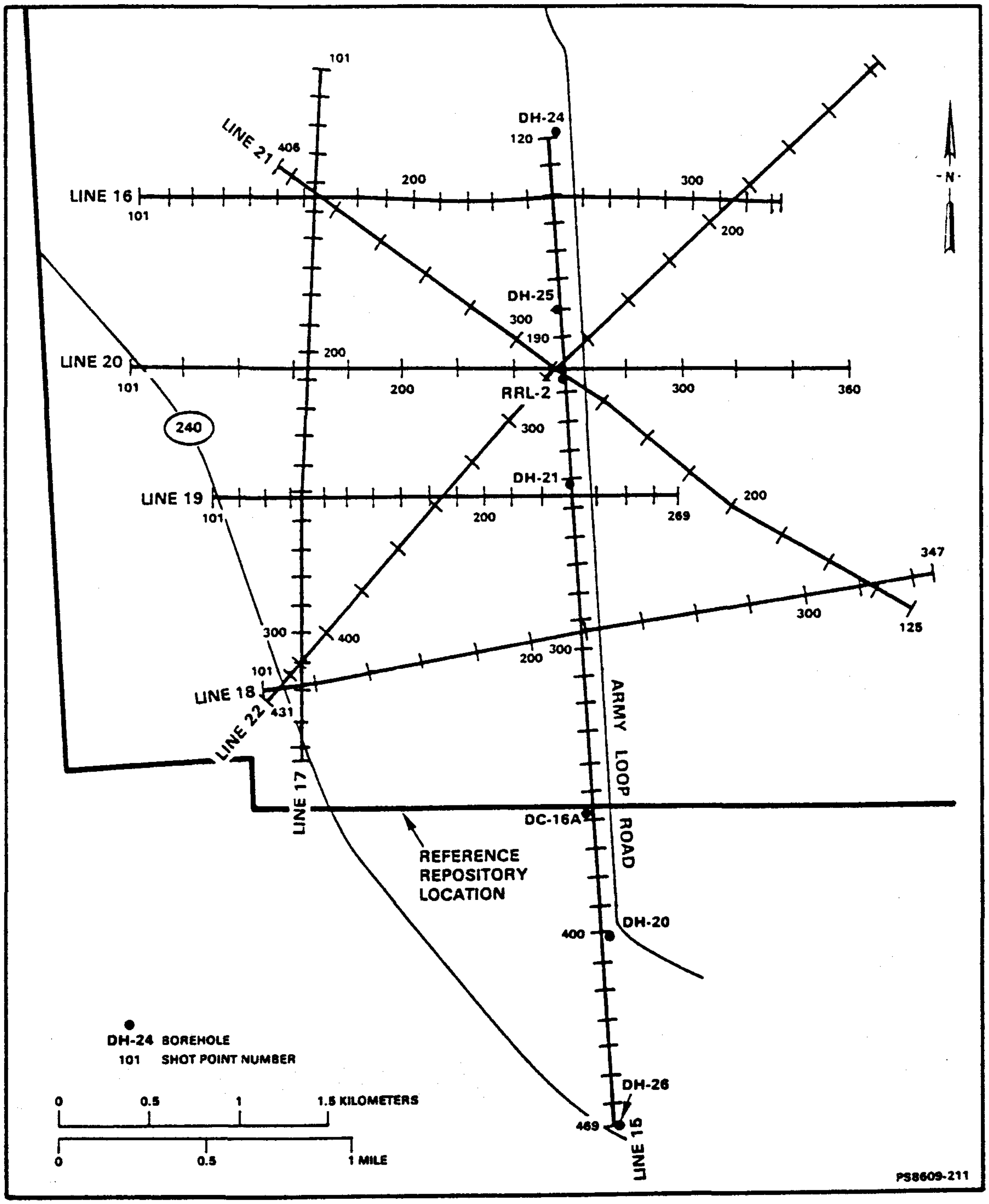


Figure 1.3-50. Location map for refraction lines shot during the Weston geophysical survey (Weston, 1982).

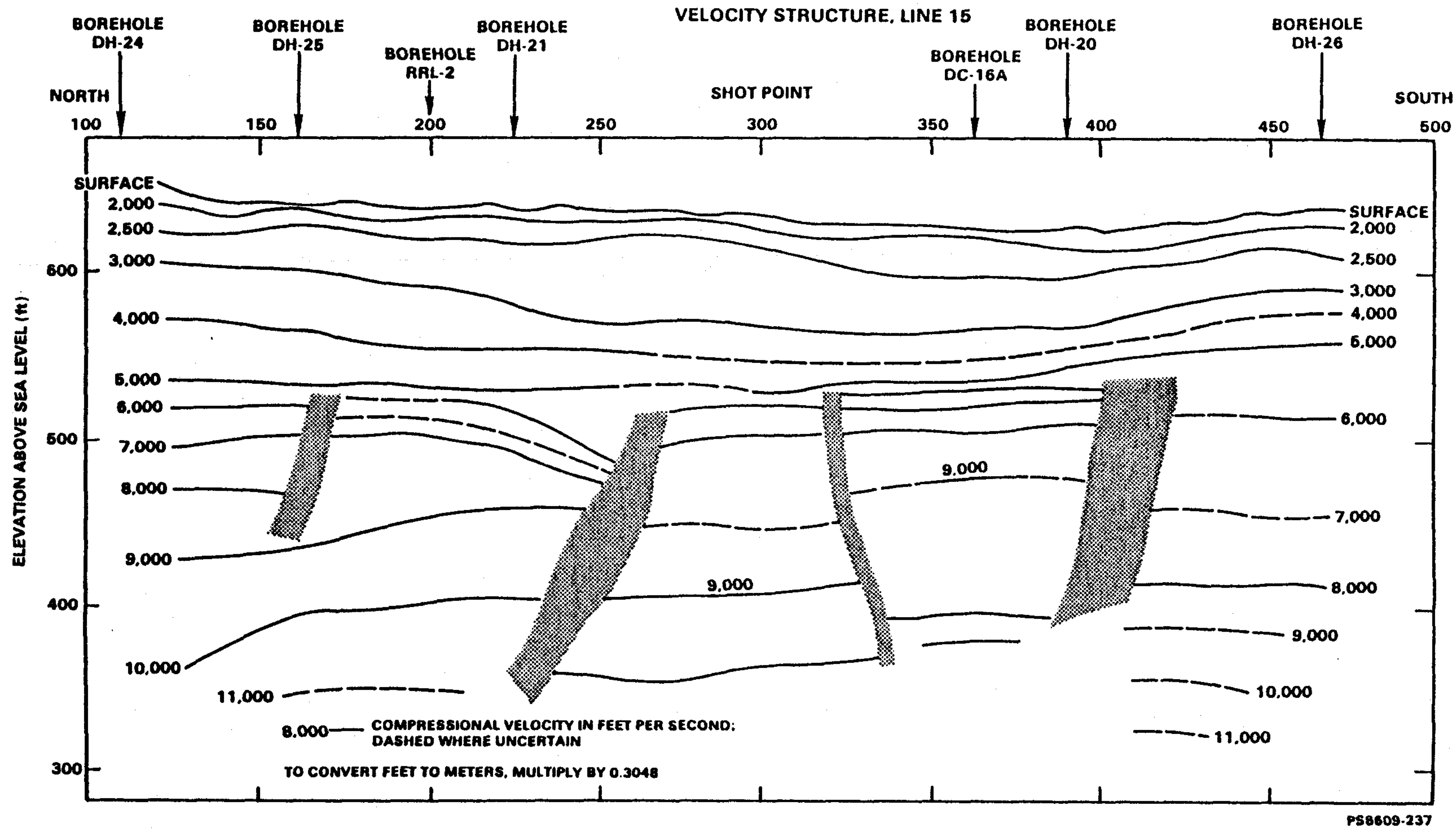


Figure 1.3-51. Interpreted section for Line 15 of the Basalt Waste Isolation Project seismic refraction survey (from Odegard and Mitchell, 1987). See Figure 1.3-50 for location of Line 15.

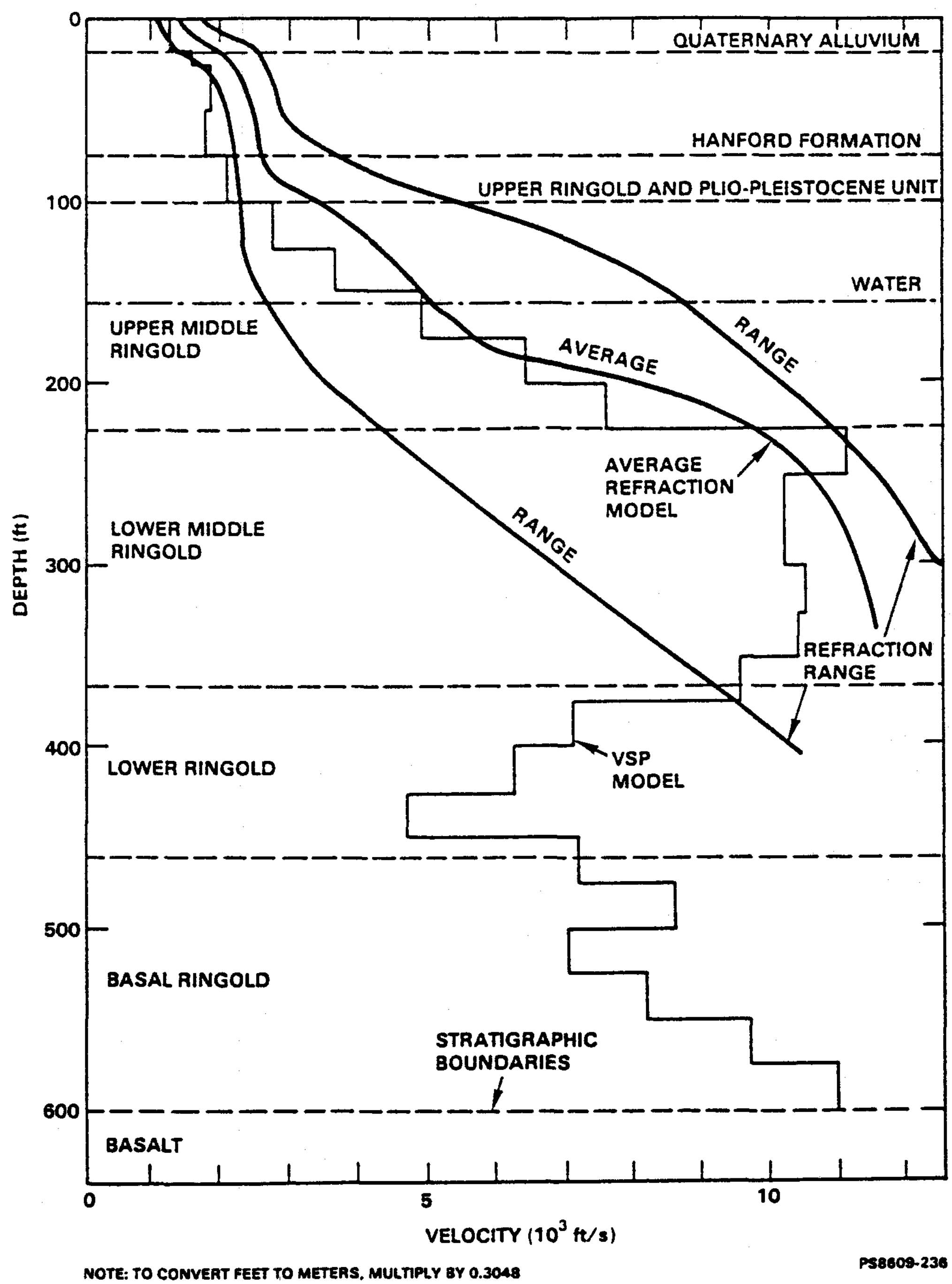


Figure 1.3-52. Average refraction and vertical seismic profiling velocity models associated with the stratigraphic section at borehole RRL-2 (from Odegard and Mitchell, 1987).

unavailable to direct geologic investigation. Several of these investigations involve the use of top-of-basalt determinations for structural problem resolution.

The high-velocity layer associated with the middle Ringold sedimentary unit is a distinctive feature of the sediments in the area of the reference repository location. It appears to be found throughout much of the Hanford Site. Based on refraction and reflection surveys, the high-velocity layer does not appear to exist north of the Umtanum Ridge-Gable Mountain structure and its extension, probably due to erosion in that area. The layer also has been eroded, partially or completely, in certain areas south of the Umtanum Ridge-Gable Mountain structure. These erosional features result in statics problems in seismic processing. These statics shifts may, in some cases, result in the interpretation of the resultant discontinuities in seismic sections as faults or disturbed areas. These problems will be investigated further during site characterization.

Sediments with velocities as high as those in the middle Ringold Formation are usually found only at much greater depths. Because these high-velocity sediments overlay sediments with lower velocities, the sediment structure and stratigraphy in the low-velocity zone cannot be determined with the seismic refraction method. In fact, the exact location of the top of basalt may be difficult to determine. This is because the actual top of basalt may be severely weathered and brecciated, as mentioned in the borehole analysis discussed in the Preliminary Safety Analysis Report for the Skagit/Hanford Nuclear Project (PSPL, 1982). That is why many of the reports mention the depth to high-velocity basalt, and why some of the borehole determinations differ from those of the refraction survey.

1.3.2.2.3.5 Seismic reflection surveys

The purpose of the seismic reflection surveys conducted by the BWIP has been to delineate the stratigraphy of sediment and basalt layers within the Hanford Site and to determine the presence and extent of anomalous features with potential structural significance.

Seismic reflection data are typically used to interpolate or extrapolate lithology and stratigraphy among borehole and outcrop data. This requires careful integration of geology and borehole data with the seismic reflection data. The integration of the seismic reflection data with borehole data, other geophysical survey data, and geologic data will be a primary objective during site characterization (Sections 8.3.1.2.3.3.1 and 8.3.1.2.3.3.4).

Several seismic surveys have been conducted on the Hanford Site since 1963. Until 1979 these surveys consisted of testing the method. The 1963 survey was a developmental test of seismic reflection and refraction methods to determine the depth to basalt beneath the Hanford Site. The tests and results are discussed in Donaldson (1963). Seismic reflection was not successful because of the limited processing capabilities available at that time, coupled with the difficult seismic characteristics of the area.

Surveys conducted from 1978 through 1980

Testing of seismic reflection techniques was conducted by the BWIP in 1978 to determine the ability of the method to penetrate the basalt to the Umtanum flow of the Grande Ronde Basalt. A report of the test results is given in Seismograph Service Corporation (SSC, 1978) and Heineck and Beggs (1978). The data, as shown in Figure 1.3-53, showed good penetration with reasonably coherent energy down to approximately 0.5 s. Below this, there appears to be energy at scattered locations penetrating to at least 1.0 s. Also apparent are problems due to statics and off-line scattering. A section located just to the northeast of Gable Mountain shows the basalt layers ramping up as the line approaches the mountain.

Because the results of the 1978 test were encouraging, 140 line kilometers (87 line miles) of seismic reflection surveys were conducted by the BWIP in 1979 and 55 line kilometers (34 line miles) in 1980. The locations of the lines are shown in Figure 1.3-54. These lines were located to provide general coverage of the Hanford Site, but were concentrated in the Cold Creek syncline. Results of the surveys are reported in Seismograph Service Corporation (SSC, 1979, 1980), Myers/Price et al. (1979), and Holmes and Mitchell (1981). In addition to the record sections, Seismograph Service Corporation provided interpretations of the data to locate the top of basalt and anomalous features in the basalt. These anomalies, which ascribe to a variety of geologic structures or problems in processing, are compiled in Figure 1.3-55.

The top-of-basalt maps produced by Seismograph Service Corporation were based on an assumption of a velocity-depth function from the stacking velocities. As discussed by Odegard and Mitchell (1987), this assumption was in error along line 5 and probably in much of the Hanford Site south of the Umtanum Ridge-Gable Mountain structure. These errors result from a lack of velocity data from the sediments in the form of acoustic velocity logs or vertical seismic profile data. In addition, the interpretation appears to have been done without the constraint of top-of-basalt data from boreholes, although these data would be of limited use without velocity data. A reinterpretation of these data, constrained by available geological and geophysical data, will be a primary task during site characterization (Section 8.3.1.2.3.3.4); to be effective, this reinterpretation will require adequate velocity data from boreholes.

Holmes and Mitchell (1981) evaluated the seismic reflection data from the 1979 and 1980 surveys in conjunction with other geophysical data. They concluded that some of the anomalous seismic features correlated with anomalies identified in magnetic and gravity data.

During 1983, sections of lines 3, 5, and 8 were reprocessed for the BWIP. The results of this reprocessing were reported by Berkman (1984). The reprocessing was aimed at using better statics correction and stacking procedures based on velocity data from seismic reflection surveys. Careful attention also was paid to other parts of the processing sequence. This reprocessing resulted in somewhat better record sections.

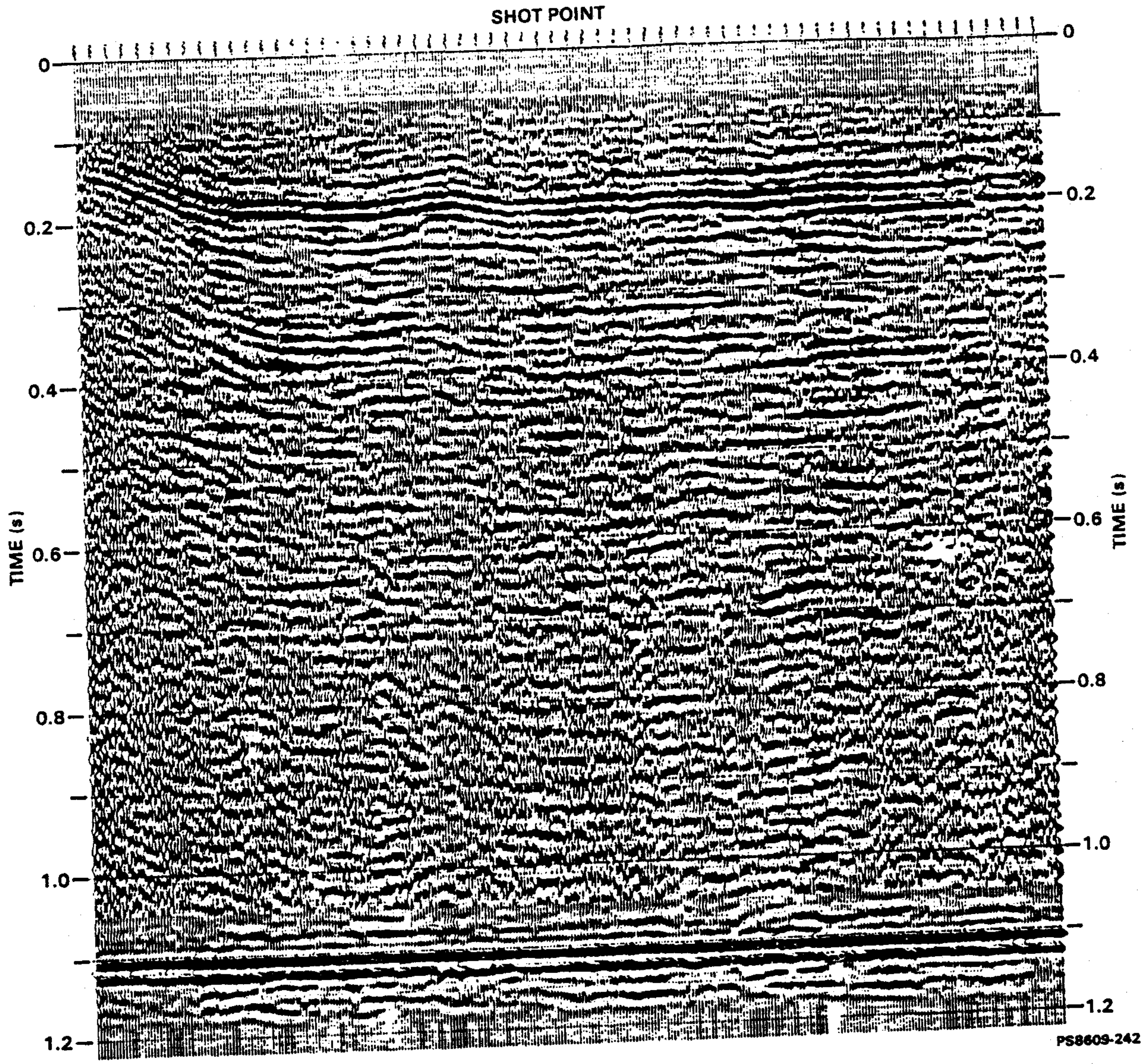


Figure 1.3-53. Wavelet processed seismic reflection line from the Basalt Waste Isolation Project test survey over DC-6 well (Heineck and Beggs, 1978). The wavelet used is shown at the bottom of the section.

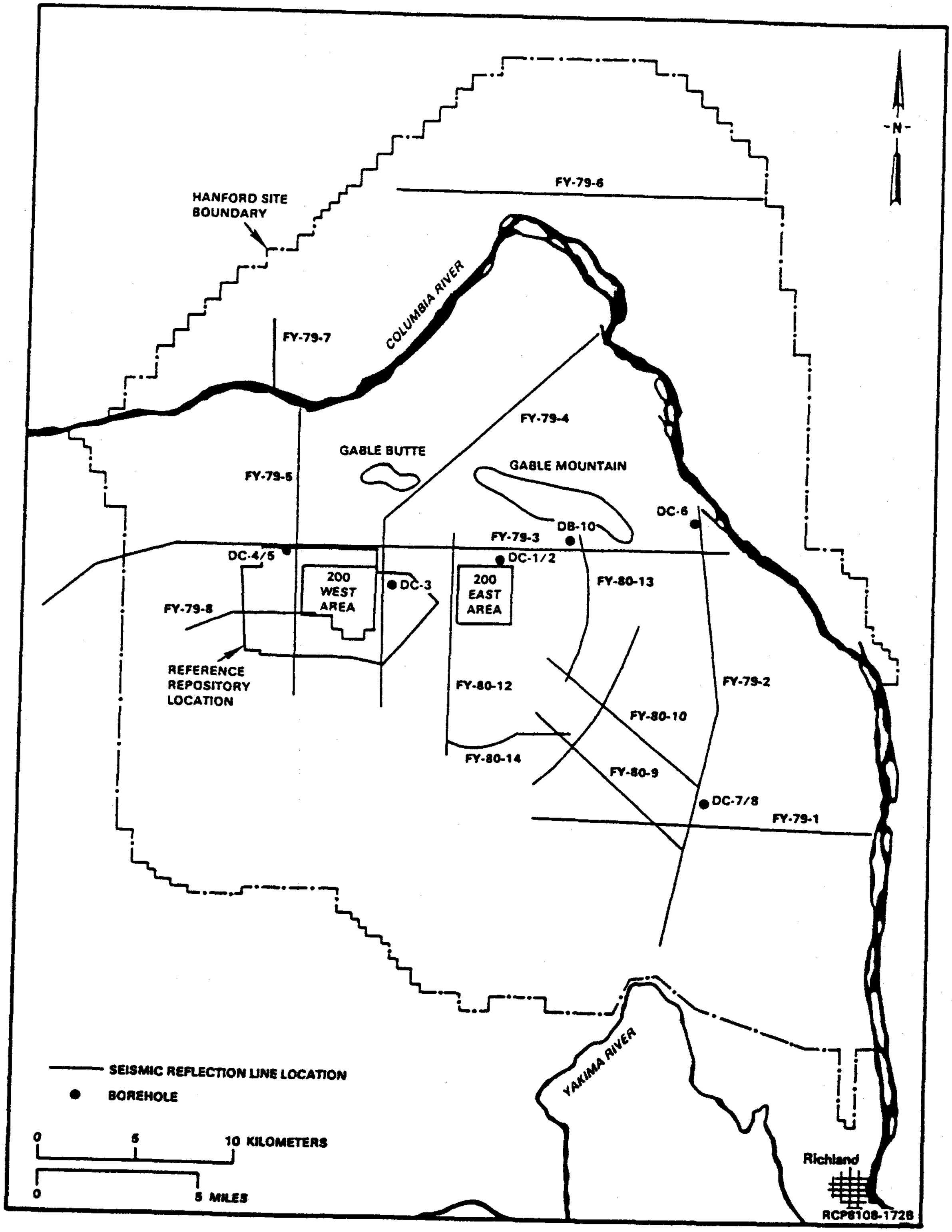


Figure 1.3-54. Seismic reflection line locations from surveys conducted for the Basalt Waste Isolation Project (SSC, 1979; 1980).

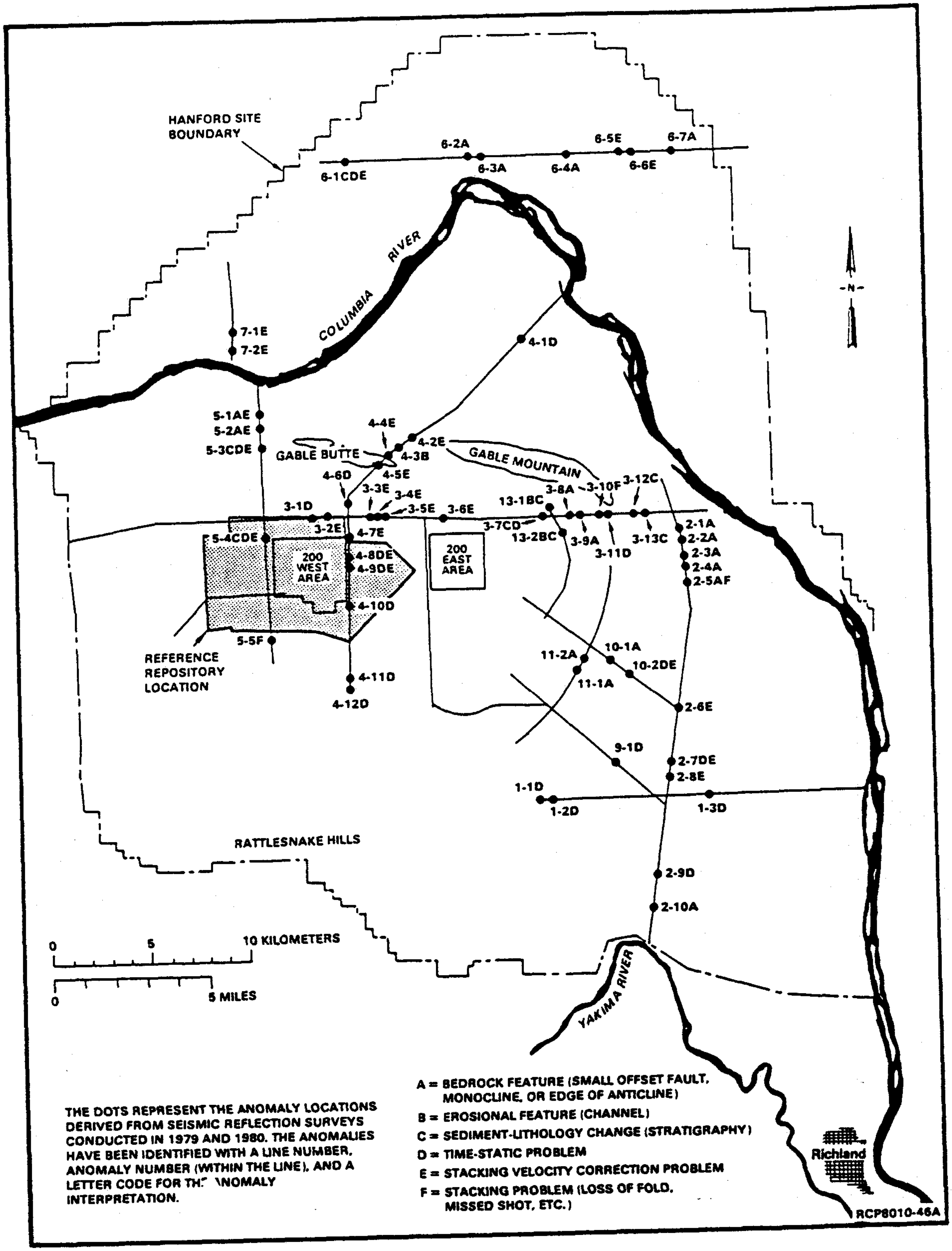


Figure 1.3-55. Seismic reflection anomaly location map (from Holmes and Mitchell, 1981).

The major result of the reprocessing was the realization that there are severe statics problems in the western part of the Cold Creek syncline. It also was recognized that data acquisition should be conducted to give a higher fold coverage at near offsets to better image the top of basalt. The statics problems and the need for higher fold data were not recognized initially, because the 1978 tests were conducted in an area that on the surface appeared to be similar, but in the subsurface was significantly different from the rest of the Hanford Site. Specifically, the thicknesses of the Hanford Formation and middle Ringold layers are significantly different, thus affecting the seismic response.

Surveys conducted during 1985

In 1985, additional seismic reflection was conducted in the area of the reference repository location to determine the best acquisition and processing parameters in the reference repository location. This survey was conducted along a 3.2-km (2-mi) test line shown in Figure 1.3-56; this is just to the west of a segment of line 5 of the Seismograph Service Corporation survey. Borehole RRL-2 is near shot point 120 in the following seismic sections. The survey included a series of tests to determine the best technique to acquire high-resolution seismic data.

The acquisition of these data was aimed at determining the depth to top of basalt. The acquisition used short geophone spreads, 1-ms sample rates, and relatively small energy sources. Three energy sources were tested, including a land air gun. Dynamite and primacord were tested to determine the optimum shot pattern, charge depth, and number of shots or shot size. After the initial tests, a shot point and group spacing of 7.6 m (25 ft) with a 15.44-m (50-ft) group length was chosen for production work. The spread geometry was asymmetric with a maximum offset of 350 m (1,150 ft).

The resultant data were processed by the contractor (Kunk, 1986). It was determined that the best energy sources were the air gun and dynamite. The results from these two sources were quite similar, so detailed processing was conducted only on the air gun section, which is the easiest to acquire. An example of the initial processing results are shown in Figure 1.3-57. This was processed by Walker Geophysical, using standard seismic processing techniques. The principal coherent arrival seen at a reflection time of just over 0.1 s can be correlated with the base of the Plio-Pleistocene layer. The apparent reflection is a result of the large velocity gradient at this point, which acts as an interface. Coherent reflectors can be seen at times down to slightly over 0.3 s, although they do not correlate well across the section. The top of basalt is at a depth of 0.27 s near shot point 120 but does not correlate well across the section.

This same line also was processed through Emerald Exploration Consultants, Inc. (Berkman, 1986). The results of this processing are shown in Figure 1.3-58. These data were processed using a similar processing sequence to that of Kunk (1986), except that an iterative, surface-consistent, statics and velocity-analysis procedure was used. In addition, a

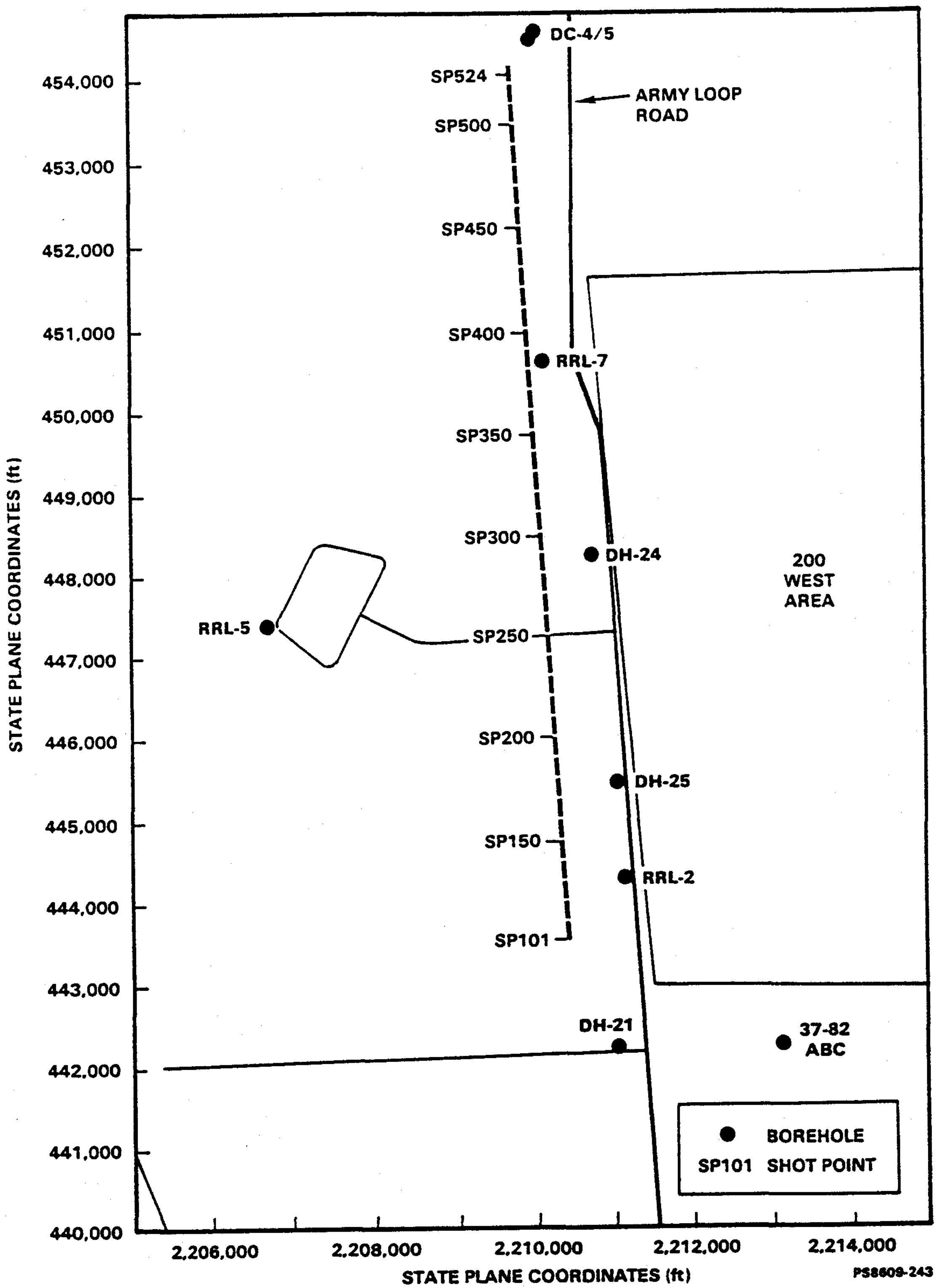


Figure 1.3-56. Location of seismic reflection test line acquired for the Basalt Waste Isolation Project (Kunk, 1985).

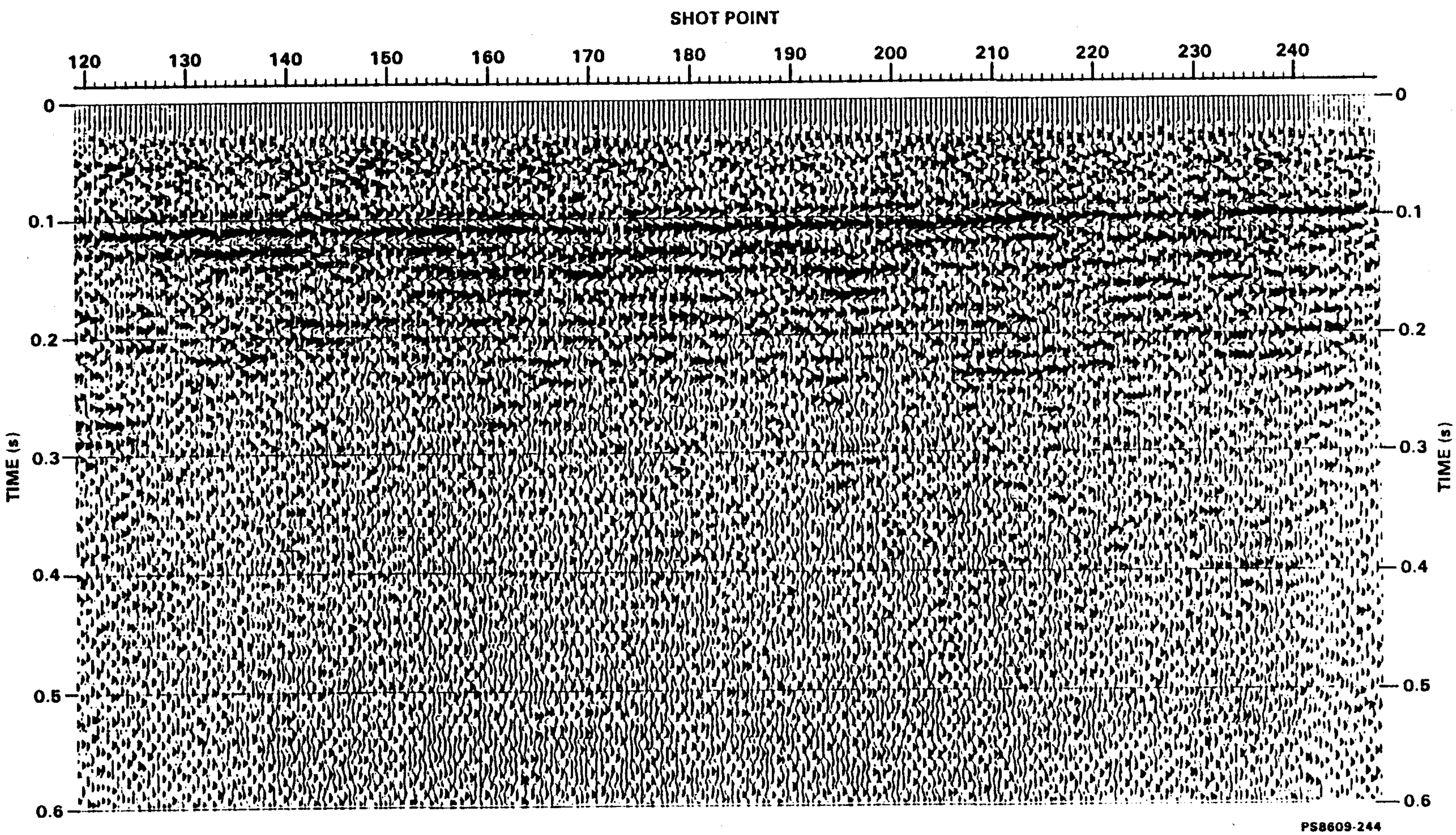


Figure 1.3-57. Initial processing of seismic reflection test line in Figure 1.3-56 (Kunk, 1985).

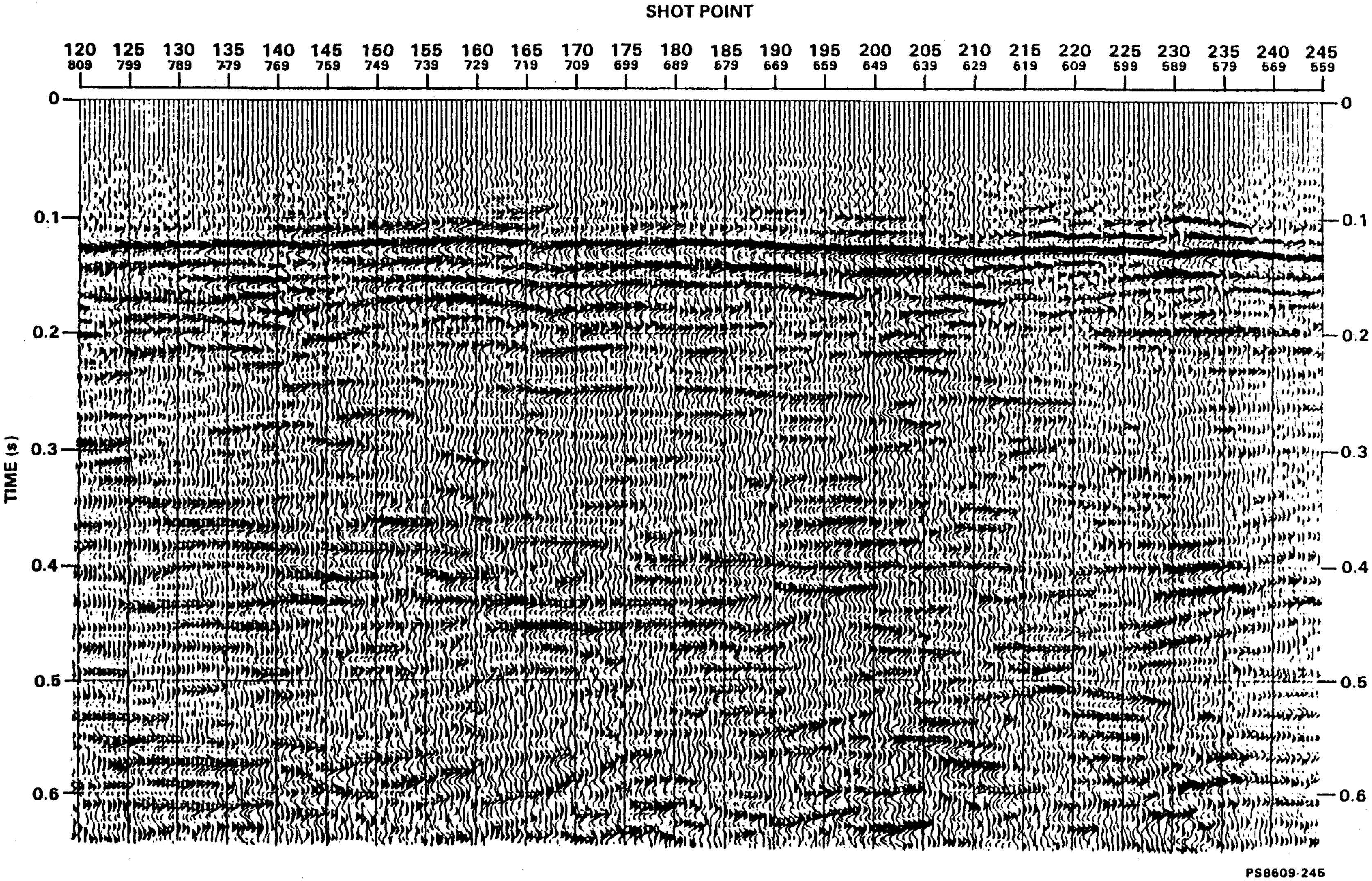


Figure 1.3-58. Processed section of test line in Figure 1.3-56 using repetitive statics corrections and a post stack dip correlation filter (Berkman, 1986).

poststack, dip-correlation filter was used. This section also shows the Plio-Pleistocene boundary quite well, as well as other reflectors at reflection times at least to 0.5 s. This is approximately the depth of the Cohassett flow in this area. The top of basalt is not an obvious arrival in this section. The abundance of the coherent reflectors in this section could, to some extent, be an artifact of the dip-correlation filter, and the interpreter must be careful in interpreting these reflectors.

Kunk (1986) also processed this same section using a muting technique, which surgically removes areas of noise from the prestack data. The resultant common depth point gathers can be stacked in the normal way. The section shown in Figure 1.3-59 was processed in this way. Coherent noise trains due to channel waves produced in the sediments by the upper-middle Ringold unit and top-of-basalt interfaces were removed from these data by muting data in a wedge near zero offset. The resultant reflectors used in the common depth point stack are primarily refracted. Thus, the resultant section is much like a high-resolution seismic refraction section. This section shows a coherent reflector near the expected top-of-basalt reflection time. Other reflectors are clearly seen in the sediments, although the Plio-Pleistocene reflector is absent. No reflectors are seen below a reflection time of approximately 0.3 s. Reflectors below this depth were surgically muted. The extent of the sedimentary reflectors may be related to the existence of the upper-middle Ringold unit.

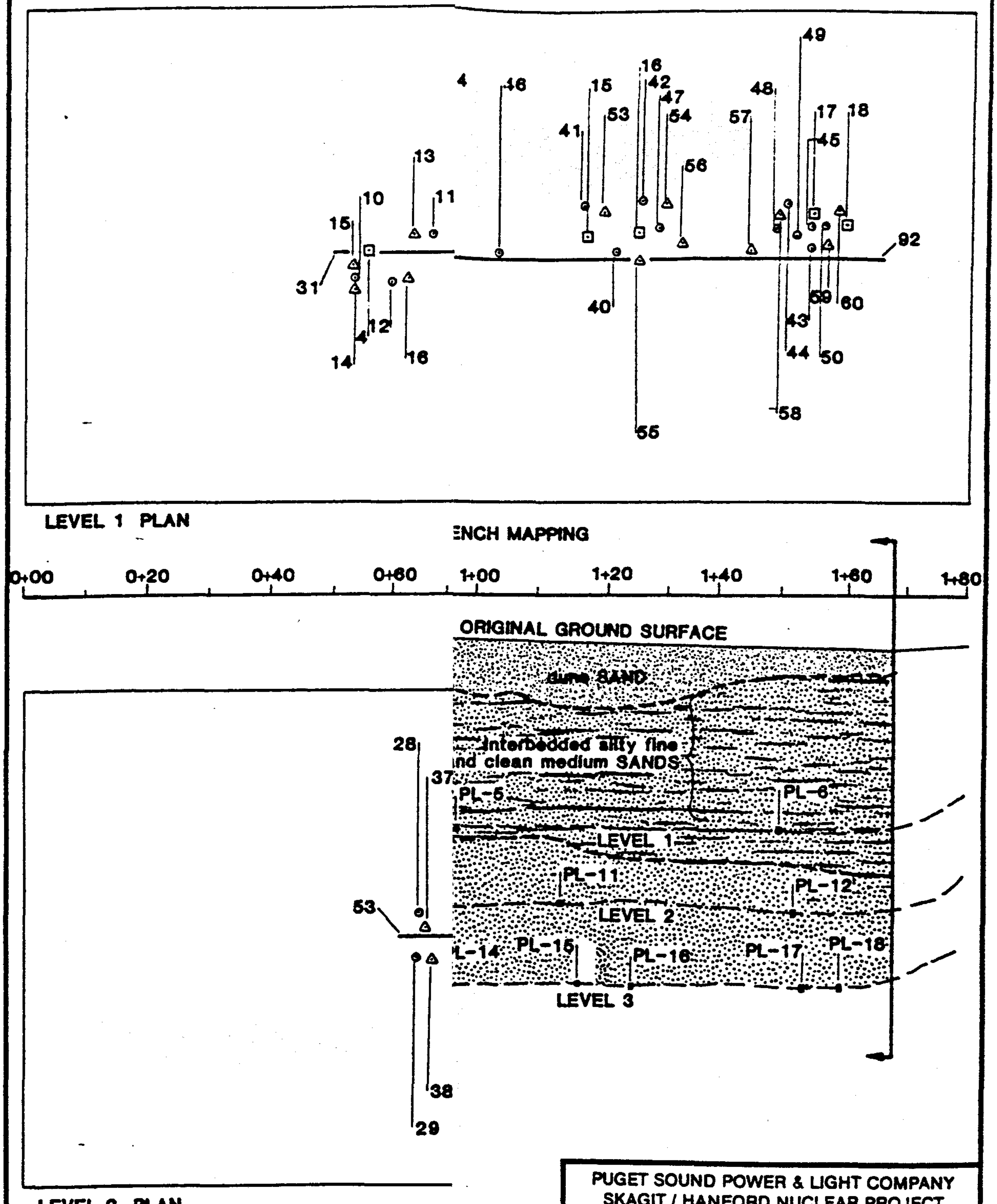
General seismic reflection results

From a comparison of the seismic sections shown in Figures 1.3-57 through 1.3-59, it is obvious that different seismic processing procedures can be applied to seismic reflection data to enhance different areas of the record. From a careful consideration of the data, the BWIP also has concluded that more attention must be paid to the coherent noise trains, statics problems in the sediments, and on- and off-line scattering from sedimentary and basaltic features. These problems can be attacked with available state-of-the-art processing techniques. Attention also will have to be paid to acquisition techniques to image deeper basaltic features in the units in which the repository may be located. Plans to address these problems are included in the stratigraphy and structural geology study plans.

A principal objective of the seismic reflection program is to determine the location, extent, and origin of anomalous features within the controlled area study zone. As discussed in Section 1.3.2.2.3.4, anomalous features may occur in the record sections due to a variety of causes. During site characterization these features will be analyzed in great detail as discussed in Section 8.3.1.2.3.3.4.

1.3.2.2.3.6 Borehole geophysical surveys

Borehole geophysical surveys and logs are used to gather data on rock densities, velocities, porosities, and other parameters. The following discussion outlines several applications of these techniques at the Hanford Site and their potential for generating data useful to site characterization.



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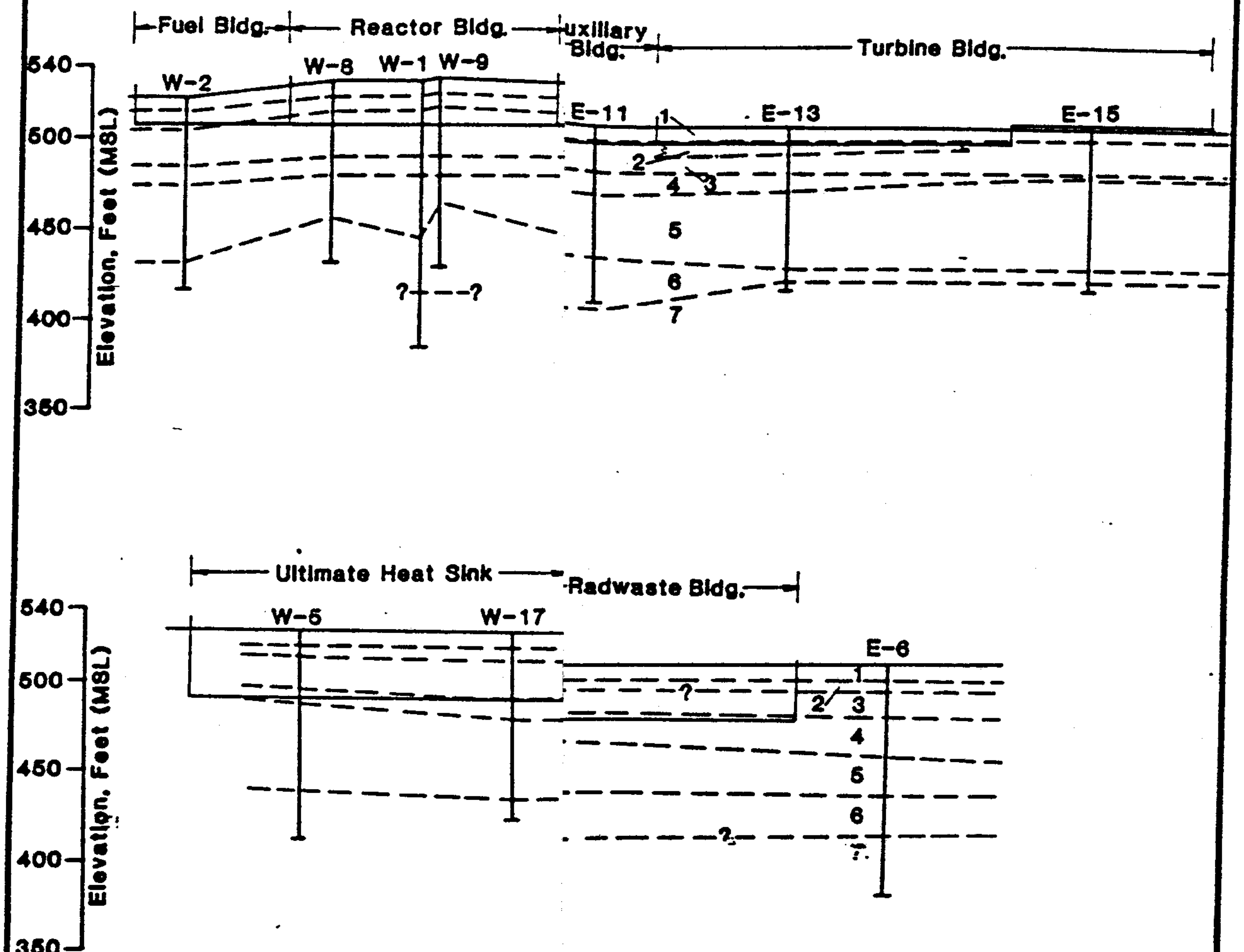
TRENCH #2 LEVELS 1,2,3 PLANS AND PROFILE

FIGURE 2Q-4



NOTE: Groundwater table at e

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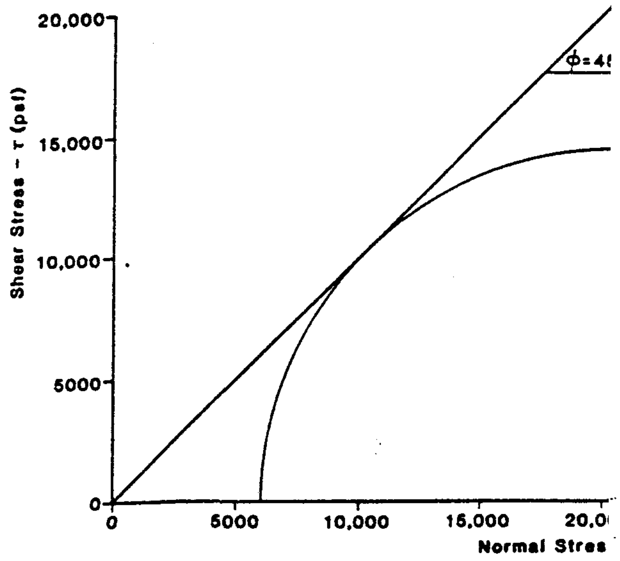
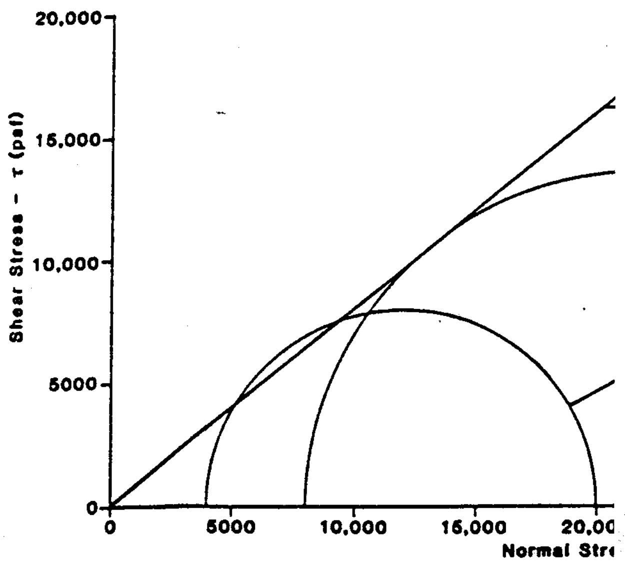
GENERALIZED SUBSURFACE PROFILE

FIGURE 2Q-5

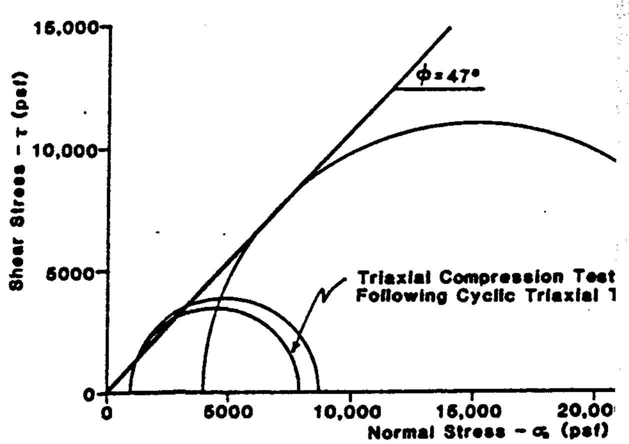
8



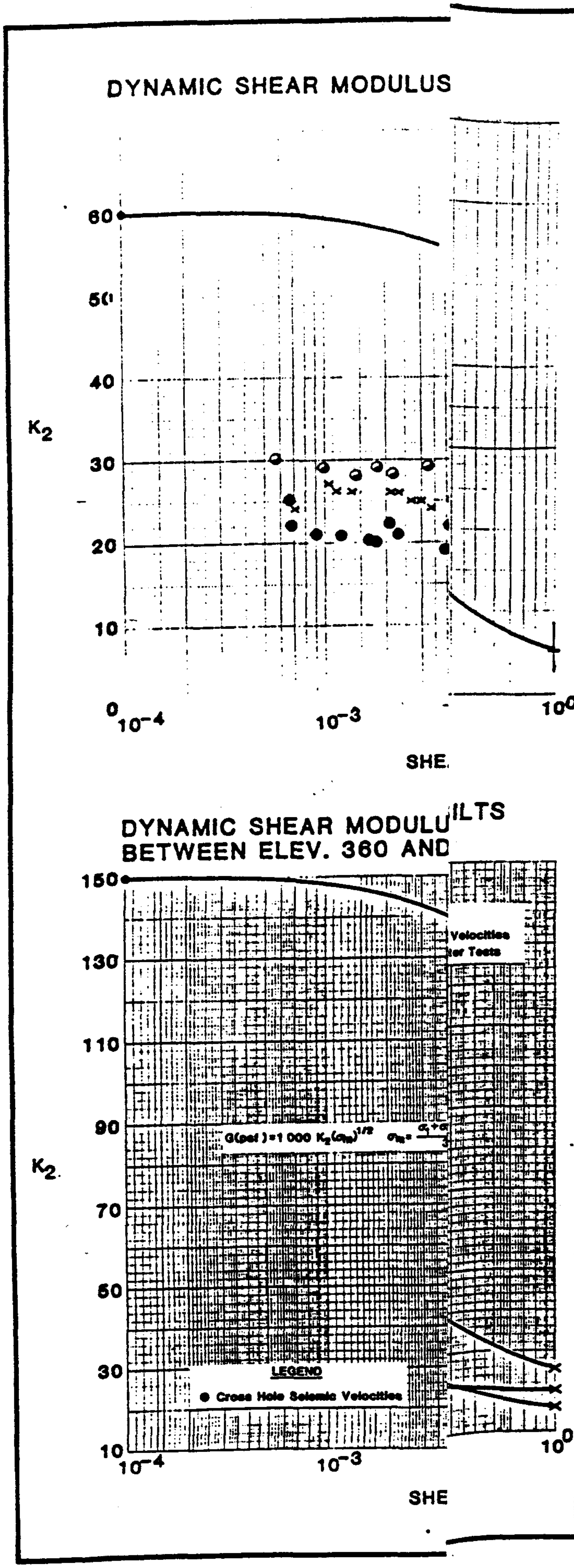
(a) Undisturbed I



(b) Undisturbed Pre-Mis



(c) Missoula Sands Reconstituted to 75% Relati



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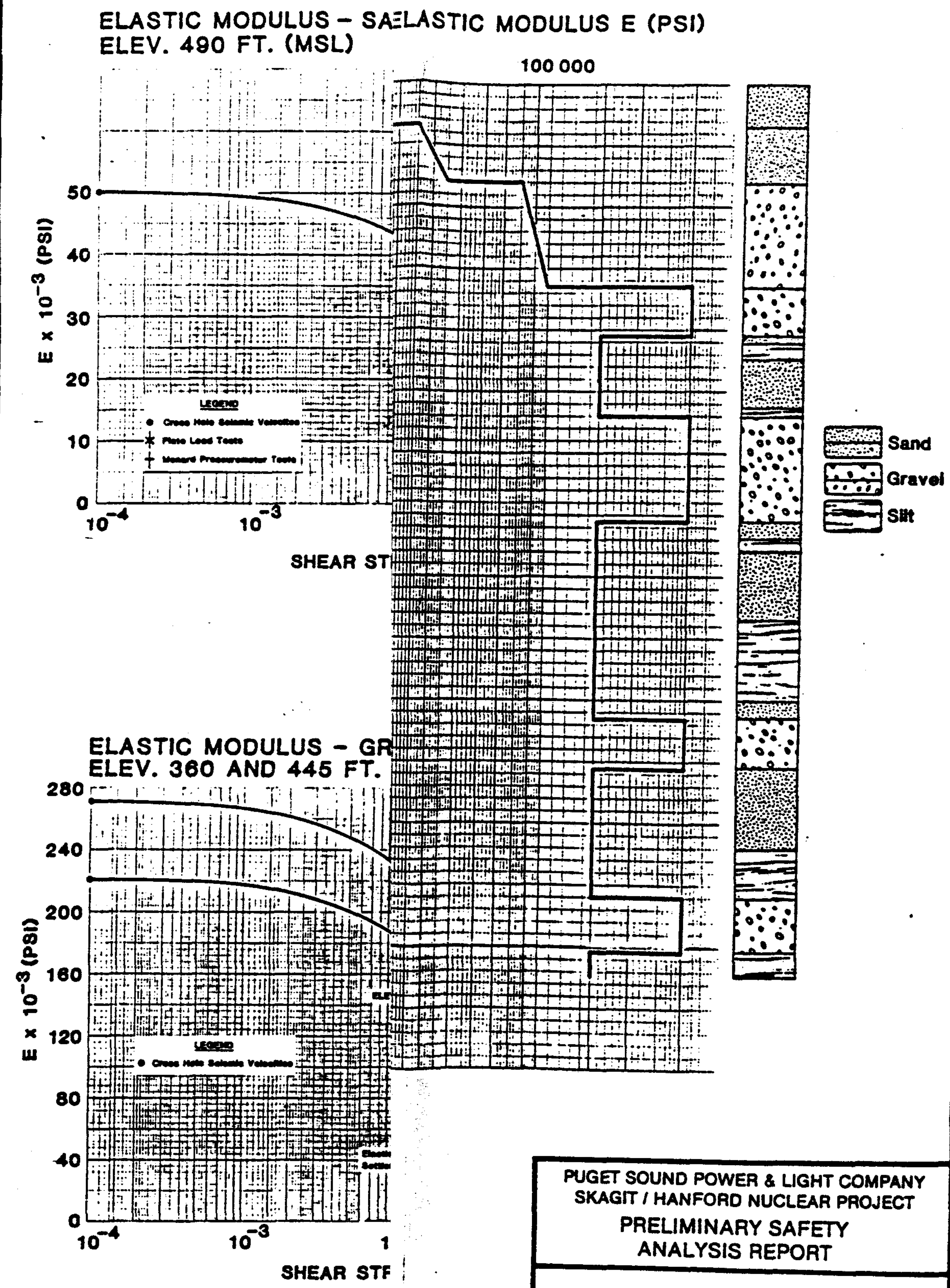
DYNAMIC SHEAR MODULI

FIGURE 2Q-7



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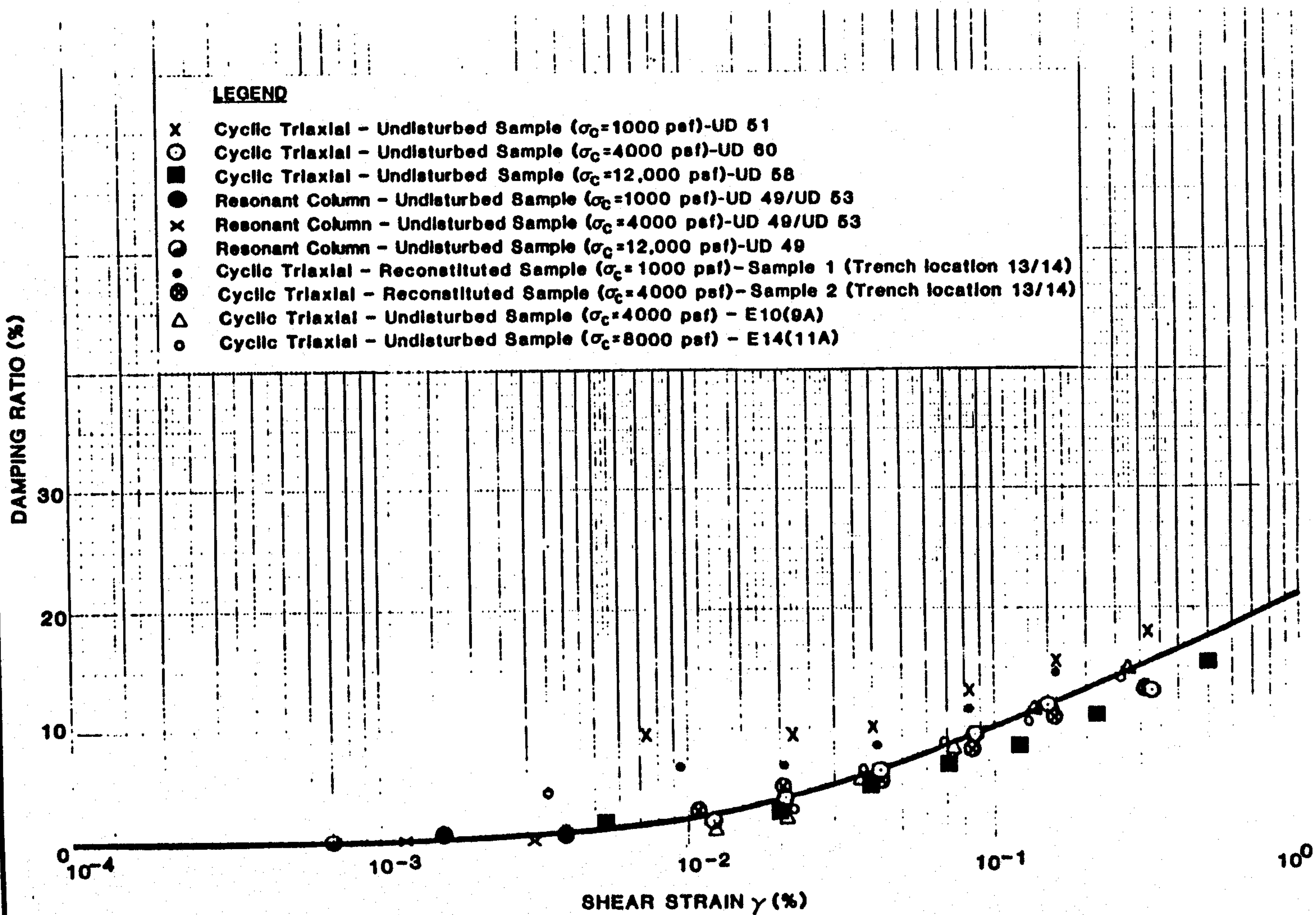
ELASTIC MODULUS PROFILE

FIGURE 2Q-8



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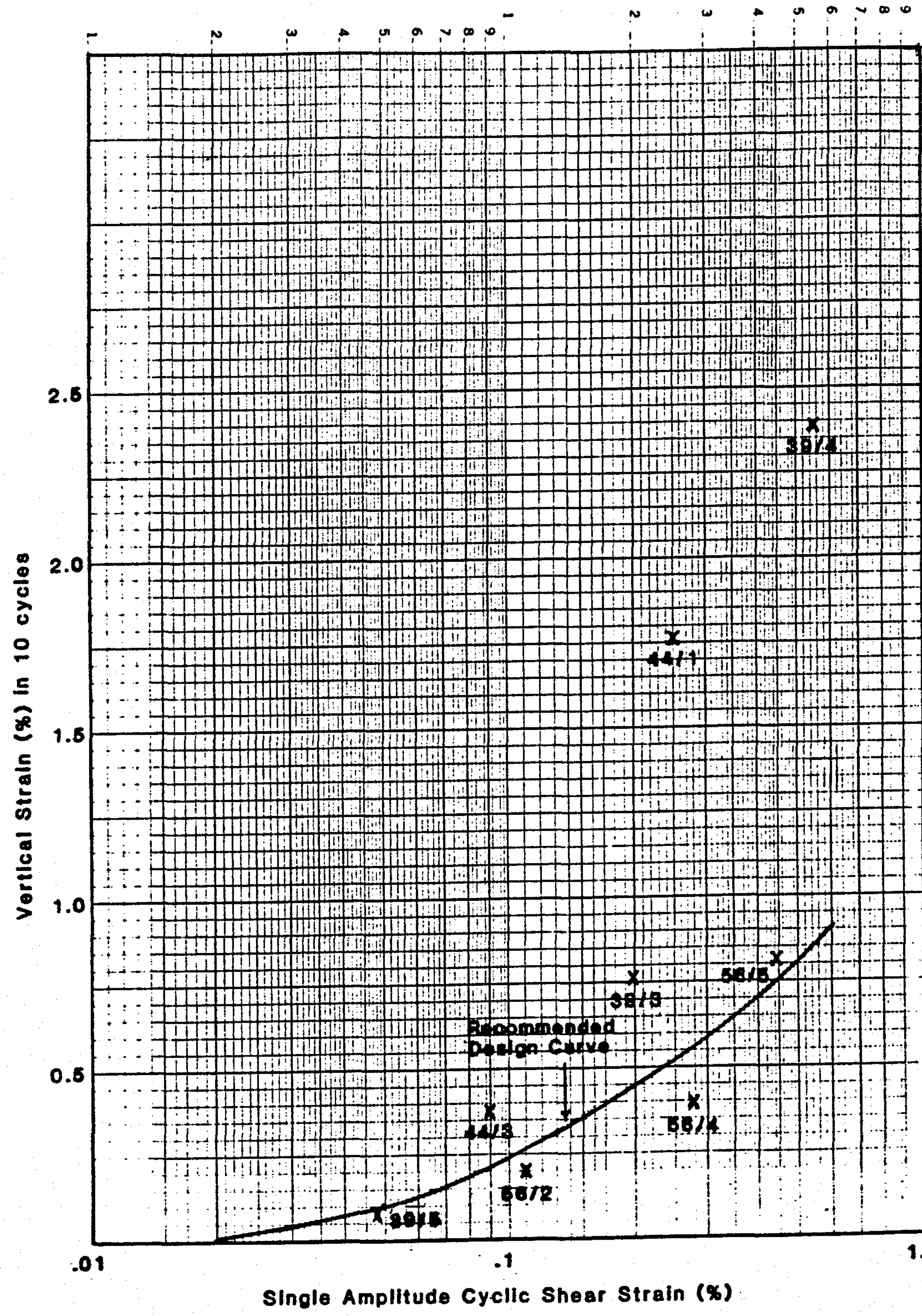
DAMPING RATIOS

FIGURE 2Q-9



Note: Sample numbers refer to Undisturbed Tube Samples (locations shown on Fig.2Q-4)

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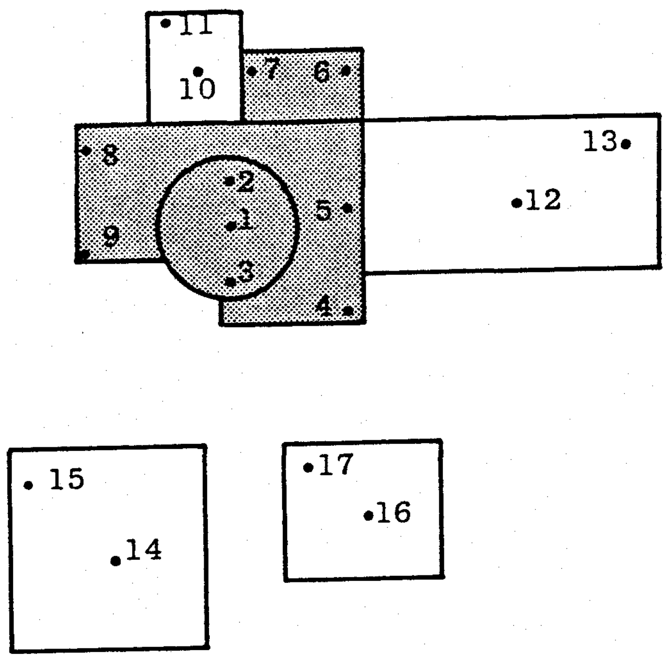
DYNAMIC COMPACTION
TEST RESULTS

FIGURE 2Q-10



Structure	Foundation Pressure kips per sq.ft.	Foundation Elevation feet	Point	(1) Calculated Elastic Settlement	(2) Calculated Elastic Settlement	(3) Secondary Settlement	(4) Maximum Total Settlement	(5) Maximum Post Construction Total Settlement (25% live load)
Reactor			1	2.2	1.6		2.7	1.1
	10.9	507	2	2.0	1.5	0.5	2.5	1.0
			3	2.0	1.4		2.5	1.0
Auxiliary	5.8	507	4	1.0	0.9	0.3	1.3	0.6
			5	1.3	1.1		1.6	0.6
Control	5.3	507	6	0.9	0.8	0.3	1.2	0.5
			7	1.0	0.9		1.3	0.6
Fuel	5.5	507	8	1.0	0.8	0.3	1.3	0.6
			9	1.0	0.8		1.3	0.6
Diesel Generator	2.4	520	10	0.8	0.8	0.3	1.1	0.5
			11	0.6	0.6		0.9	0.5
Turbine	3.8	508/518	12	0.9	0.8	0.3	1.2	0.5
			13	0.7	0.7		1.0	0.5
Ultimate Heat Sink	3.7	477	14	0.6	0.5		0.6	
			15	0.5	0.5		0.5	
Radwaste	3.8	488	16	0.6	0.5		0.6	
			17	0.5	0.5		0.5	

NOTE: Settlements in inches.

(1) Elastic Settlement using modulus data on Fig.2Q-8

(2) Elastic Settlement using modulus data on Fig.2Q-7

(3) Secondary Settlement=Elastic Settlement of Missoula sediments

(4)=(1)+(3)

(5)=[0.25 (1)]+(3)

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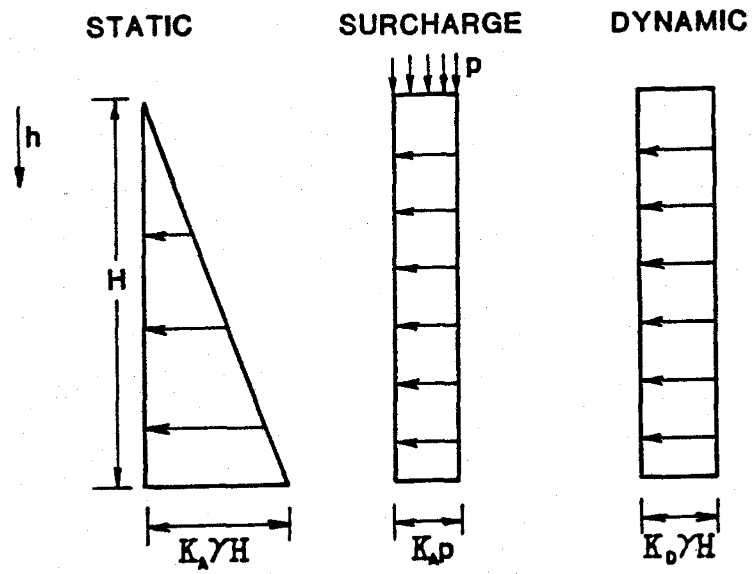
PRELIMINARY SAFETY ANALYSIS REPORT

FOUNDATION SETTLEMENTS

FIGURE 2Q-11

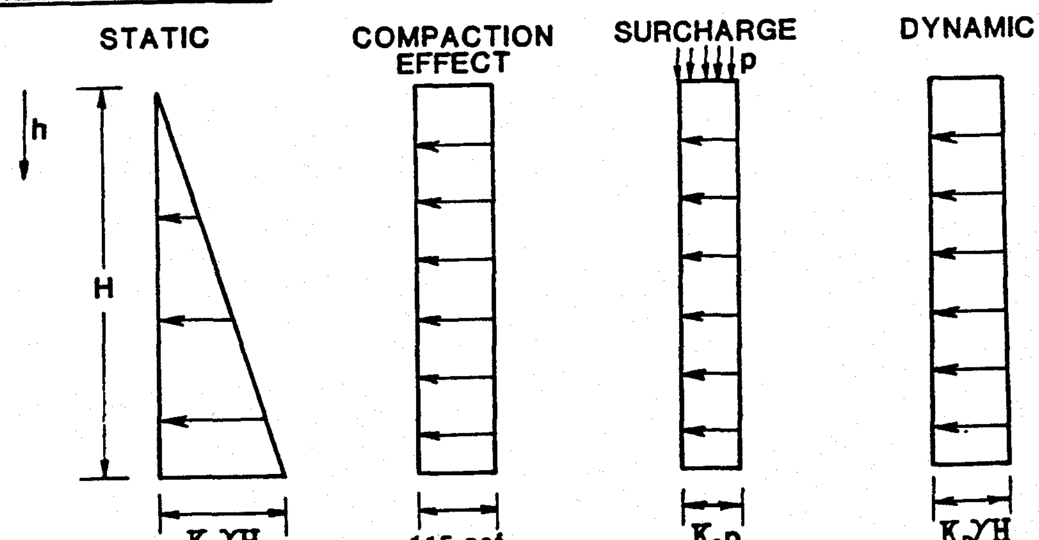
YIELDING WALLS



h = depth below grade
H = height of wall below grade
γ = backfill specific weight (115 pcf)
K_A = active earth pressure coefficient = 0.27
p = surcharge
K_D = dynamic earth pressure coefficient (= 0.13 for 0.35 g)

Static Pressure = $K_A \gamma h + K_A p$
Dynamic Pressure = $K_A \gamma h + K_A p + K_D \gamma H$

UNYIELDING WALLS



h = depth below grade
H = height of wall below grade
K_0 = at rest earth pressure coefficient = 0.5
p = surcharge
γ = backfill specific weight (115 psf)
K_D = dynamic earth pressure coefficient (= 0.13 for 0.35 g)

Static Pressure = $K_0 \gamma h + K_0 p + 600$ psf
Dynamic Pressure = $K_0 \gamma h + K_0 p + K_D \gamma H$

Note: Effect of compaction stresses need not be considered in dynamic analysis

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LATERAL EARTH PRESSURES

FIGURE 2Q-12

APPENDIX 2Q

FOUNDATION INVESTIGATION AND ANALYSIS

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OCTOBER 1981

GOLDER ASSOCIATES

APPENDIX 2Q

CONTENTS

APPENDIX 2Q

TABLES

APPENDIX 2Q

FIGURES

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF STUDY

This report presents the results of foundation investigations and analyses for the central plant facilities of the Skagit/Hanford Nuclear Project Nos. 1 and 2 nuclear power plants. The locations of the proposed facilities are shown on Figures 2Q-1 and 2Q-2.

The purpose of the study was to evaluate foundation conditions at the Site, and to provide engineering data and analyses required for the design of foundations and subsurface walls under both static and dynamic loading conditions.

The scope of the study included field and laboratory investigation programs, together with engineering evaluations for foundation design. The field program comprised subsurface borings, trenching, in situ deformation testing within boreholes and trenches, subsurface soundings, in situ density measurements and undisturbed sampling for laboratory testing. The details of the field program are discussed in Appendix 2Q A. In addition, field seismic surveys were undertaken at the Site (Appendix 2L) and a groundwater monitoring system (water sampling and piezometric head monitoring) was established (Appendix 2P).

The laboratory testing program consisted of index testing (soil descriptions, water contents, grain size analyses, specific gravities, Atterberg limits, in place densities, vibratory and impact compaction testing) to classify the Site soils, and physical property determinations (consolidation tests, cyclic triaxial and triaxial compression tests, resonant column tests, cyclic simple shear tests) to define the static and dynamic responses of the foundation materials under the design loading conditions. The laboratory test methods and results are summarized in Appendix 2Q B.

The engineering evaluation included an assessment of foundation bearing capacities and settlements under static loading, movements during earthquake loading, foundation soil stability under dynamic loading (liquefaction and dynamic compaction), lateral earth pressures on subsurface walls under both static and dynamic loading conditions, together with a discussion of excavation conditions and fill/backfill requirements for the Site.

1.2 CONCLUSIONS

The field exploration has indicated that beneath a surficial layer of loose silty sand of eolian origin, the Site is underlain with medium dense to dense sands of late Pleistocene age to approximate elevation 490 feet (MSL), very dense glaciofluvial sands and gravels of Pleistocene age to approximate elevation 320 feet, and lacustrine and fluvial very dense sands and gravels and hard silts and clayey silts of late Miocene to Pliocene age (Ringold Formation) through to basalt bedrock at approximate elevation -200 feet. The groundwater table is at elevation 400 feet, some 125 feet below the mean surface elevation at the Site of 525 feet.

All the major central plant structures will be founded on mat foundations and allowable bearing capacity based on failure considerations will not, therefore, control design. The ultimate heat sinks and the radwaste buildings will be founded in the very dense sands below approximate elevation 490 feet. The auxiliary/fuel/control/reactor buildings will be founded within the medium dense to dense sands above elevation 490 feet, as will the lower sections of the turbine building stepped-mat foundations. The diesel generator buildings and the upper sections of the turbine building stepped-mat foundations will be founded on structural backfill in those areas where the founding elevations occur above the base of the surficial loose silty sand.

Based on the foundation static loads, founding elevations and the soil properties developed from field and laboratory tests, maximum settlements beneath the auxiliary/fuel/control/reactor building mat are estimated at 2.7 inches. Maximum differential settlements across the mat are estimated at 1.5 inches. The major part of these settlements will be elastic in nature, although it is considered that the upper medium dense to dense sands might contribute secondary (time dependent) settlements of up to 0.5 inch. The proportion of the elastic settlement component which occurs post-construction will depend on the proportion of the total load which occurs as live load. The more lightly loaded diesel generator and turbine buildings are anticipated to show maximum settlements on the order of 1.2 inches with differential settlements of 0.4 inch. The deeply founded ultimate heat sinks and radwaste buildings should settle elastically with total settlements less than 0.6 inch and differential settlements of 0.2 inch.

Dynamic recoverable movements of the structure foundations during the postulated design basis earthquake are summarized in Table 2Q-4. In addition, permanent settlements can be anticipated within the medium dense to dense sands as a result of dynamic compaction. Under the dynamic motions associated with a 0.35 g design basis earthquake, maximum permanent settlements of 1.0 inch are estimated beneath the auxiliary/fuel/control/reactor building mat.

Based on the depths of the current and potential water tables and the very dense/stiff nature of the materials beneath the water table, the foundation materials are not susceptible to stability problems associated with liquefaction or cyclic strength deterioration.

For static design of subsurface structural walls, an active earth pressure coefficient Ka = 0.27 should be used for yielding walls and an at-rest earth pressure coefficient Ko = 0.5 should be employed for relatively rigid or unyielding walls. Earth pressures induced by backfill compaction and those associated with dynamic loadings are also summarized within the report.

No water will be encountered in any of the contruction excavations at the Site. The surficial loose silty fine sand should be excavated and used for non-structural area fill. Where foundation elevations occur above the base of the surficial soils, it will be necessary to overexcavate and replace with compacted structural backfill. The clean sands exposed within the deeper excavations will provide sufficient volumes of suitable structural backfill materials.

2.0 SITE DESCRIPTION

The proposed Skagit/Hanford Site is located within the Department of Energy Hanford Reservation in south central Washington. The main Plant areas are situated within Section 33 (T12N, R27E), about 16 miles north-northwest of the city of Richland, Washington (Figure 2Q-1).

The Site (mean elevation 525 feet above sea level) is located within the Pasco Basin and lies within a relatively flat plain which extends for several miles in all directions. Approximately 5 miles to the southwest, the basin slopes upward at the foot of the Rattlesnake Hills which rise to approximate maximum elevations of 3600 feet. About 7 miles north of the Site, Gable Mountain extends above the floor of the basin to approximate elevation 1100 feet. At its closest point, the Columbia River lies some 7 miles northeast of the Site, and has cut steep bluffs which form the east bank of the river. The basin continues east of the river but at a higher elevation of 800 to 900 feet above mean sea level.

Within the area defined by the topographic highs mentioned above, the surface elevation varies by about 300 feet. At the location of the central Plant facilities, maximum relief is about 20 feet (Figure 2Q-2).

The major geologic units underlying the Site are:

- Basaltic lavas of the Columbia River Basalt Group with an upper surface elevation of about -200 feet at the Site.

- Lacustrine and fluvial sediments of the Ringold Formation, the Upper Member of which is exposed in the "White Bluffs" along the Columbia River. The Ringold Formation consists of slightly lithified gravels and sands with indurated silts and clays, and is considered to be late Miocene to Pliocene in age. At the Site, the upper surface elevation of the Ringold sediments is about 315-320 feet.

- Glaciofluvial sands and gravels of Pleistocene (Pre-Missoula sands and gravels) and late Pleistocene (Missoula sands) age. At the Site, the Pre-Missoula sediments have an upper surface elevation of approximately 490 feet, and the Missoula sediments extend almost to surface. The whole area is covered with several feet of eolian silty fine sands.

3.0 PLANT STRUCTURES DESCRIPTION

Within the central Plant area for each of the two units, the major structures are: reactor building, auxiliary building, fuel building, control building, turbine building, diesel generator building, ultimate heat sink and radwaste building. The positioning of these central Plant facilities is shown on Figure 2Q-2. The following describes the major details of each of the above structures:

- Auxiliary/Fuel/Control/Reactor Buildings

  The auxiliary/fuel/control/reactor buildings will be located on a common base mat, 20 feet thick, and founded at elevation 507 feet which is 19 feet below mean surface grade.

  - Reactor Building

    The reactor building is approximately 140 feet in diameter with a total weight of structure and equipment (dead plus live load) of about 170,000 kips. This load will produce an average unit pressure of 10.9 kips per square foot at the foundation level.

  - Auxiliary Building

    The auxiliary building has a total floor area of 23,500 square feet. The total foundation load of this structure will be approximately 136,000 kips, resulting in an average pressure at the foundation level of 5.8 kips per square foot.

  - Fuel Building

    The fuel building has a total floor area of 14,800 square feet. The total foundation load of 81,000 kips will result in an average foundation pressure of 5.5 kips per square foot.

  - Control Building

    The control building has a total foundation load of 49,000 kips, giving an average foundation pressure of 5.3 kips per square foot over its rectangular dimensions of 125 by 75 feet.

- Turbine Building

  The 305-by-155 ft turbine building will be supported on a stepped base mat which will be founded at elevations 508 and 518 feet. Total loading is approximately 182,000 kips, giving an average foundation pressure of 3.8 kips per square foot.

- Diesel Generator Building

  The diesel generator building will be founded at elevation 520 feet, on a base mat. The total loading is approximately 27,000 kips, corresponding to an average foundation loading of 2.4 kips per square foot over the 100 by 110 ft dimensions of the base mat.

- Ultimate Heat Sink

  The ultimate heat sink has rectangular plan dimensions of 210 by 205 feet and a total loading of 161,000 kips. The base mat will be founded at elevation 477 feet, with an average applied pressure of 3.7 kips per square foot at foundation level.

- Radwaste Building

  The rectangular plan dimensions of this building are 170 by 140 feet. The total load of the radwaste building is 90,000 kips, giving an average applied pressure of 3.8 kips per square foot at the foundation elevation of 488 feet.

4.0 FIELD INVESTIGATION AND TESTING

Field exploration for the 2 Skagit/Hanford units consisted of drilling, sampling and testing at a total of 37 locations within the central Plant facilities areas, together with the excavation of 2 test trenches within the Unit 1 central plant facilities area (Figure 2Q-2). In addition, 10 holes were drilled for cross-hole seismic geophysical testing. Four monitoring wells and 4 multiple completion standpipe piezometers were also installed to provide for water sampling and piezometric head monitoring at the Site (see Appendix 2P).

The test borings were advanced to depths ranging from approximately 90 to 770 feet by a variety of drilling methods. Hole coordinates, collar elevations, depths and drilling methods are summarized in Table 2Q-1. The boring program was designed to obtain representative samples for material identification purposes, to attempt undisturbed sampling, to perform in situ standard penetration testing to evaluate the relative densities of the soils, and to permit Menard Pressuremeter testing down the holes in order to evaluate the stiffness properties of the in situ soils. Drilling and sampling methods are described in Appendix 2Q A. Borehole logs which summarize the standard penetration test values in graphic format are also presented in Appendix 2Q A. The Menard pressuremeter testing procedures are described in Appendix 2Q A, along with the test results and modulus determinations.

As noted on Figure 2Q-2, relocation of the radwaste buildings following the site investigation resulted in the borings lying outside the radwaste footprints. Because of the uniformity of subsurface conditions across the Site, additional borings were not considered necessary.

In order to supplement the standard penetration testing in the upper sandy materials, a program of Dutch cone static penetration testing was undertaken. Locations of the Dutch cone testing are shown on Figure 2Q-2, and further presented in Table 2Q-2. The test procedure is discussed in Appendix 2Q A, where the Dutch cone logs are also given.

Two large test trenches were excavated within the central plant facilities area of Unit 1, beneath the reactor and turbine buildings. The trench locations are shown on Figure 2Q-2. The purpose of the trenches was to examine the nature of the most recent glaciofluvial sediments, within which most structures are to be founded. Both trenches were logged and photographed (Appendix 2Q A). Testing carried out within the trenches consisted of in situ density measurements using both the sand cone apparatus and the nuclear probe, together with plate load testing. The locations of these tests are shown on Figures 2Q-3 and 2Q-4 for trenches 1 and 2 respectively. Both the in situ density test results and the plate load test data are summarized in Appendix 2Q A.

In addition to the testing carried out within the trenches, undisturbed samples for quantitative laboratory testing and bulk samples for compaction and maximum/minimum density testing were taken.

5.0 LABORATORY TESTING

Laboratory tests were carried out on samples obtained from the test borings and trenches in order to determine the engineering properties of the soils and to define a representative soil profile for use in engineering analysis. Testing included a detailed visual classification of each sample, water content determinations, grain size analyses, specific gravity determinations, Atterberg limits tests, maximum and minimum density tests, compaction tests, density determinations from core and undisturbed tube samples, consolidation tests, resonant column tests, cyclic and static triaxial tests and cyclic simple shear tests. Brief descriptions of all laboratory tests performed are given in Appendix 2Q B.

The detailed visual classifications of all borehole samples, which were used to prepare the borehole logs and generalized soil sections, are given in Appendix 2Q B. Grain size analyses were undertaken to support the visual classifications and to characterize the material types which were tested both in the field and in the laboratory. Maximum and minimum density tests and compaction tests were undertaken in order to establish the suitability of the Site material for structural fill, to assist in characterizing the in situ condition of the materials, and to provide controlled conditions for preparing remolded specimens for laboratory testing. The resonant column and cyclic triaxial tests were performed to evaluate the dynamic and static moduli and damping characteristics of the Site soils, and the cyclic simple shear tests were undertaken to evaluate potential dynamic compactions during earthquake shaking. The static triaxial tests provided information on the strength properties of the granular soils, while the consolidation testing provided information on the compressibility and pre-consolidation characteristics of the clayey-silt Site soils.

All of the laboratory test data are presented in Appendix 2Q B, and the information is evaluated in Section 6.0.

6.0 DESCRIPTION AND ENGINEERING PROPERTIES OF SITE FOUNDATION SOILS

6.1 DESCRIPTION OF SITE FOUNDATION SOILS

Based on the results of investigations previously described, the Site subsurface materials above basalt bedrock consist of the following (Figure 2Q-5):

- Loose silty fine SAND
- Medium dense to dense clean medium SAND
- Very dense silty fine SAND
- Very dense clean fine to medium SAND
- Very dense clean gravelly SAND
- Very dense clean sandy GRAVEL
- Interbedded hard SILTS and very dense SANDS and GRAVELS

6.1.1 LOOSE SILTY FINE SAND

Overlying the entire Site is a mantle of loose dark yellowish brown to gray silty fine sand of eolian origin, with an average thickness of 6 to 8 feet. In general, the upper foot or so of material tends to be somewhat cleaner. The natural water content of this horizon ranges from about 2 to 9 percent. Typical grain size distributions are shown on Figures 2Q B-1 and 2Q B-8.

6.1.2 MEDIUM DENSE TO DENSE CLEAN MEDIUM SAND

Underlying the loose surface sand is a layer of medium dense to dense gray/black sand. At the Unit 1 location, this layer extends to an average depth of 40 feet (elevation 490 feet) and at Unit 2 it extends to an average depth of 25 feet (elevation 495 feet). The upper 6 to 8 feet of this unit (depth 8-15 feet) tends to consist of an interbedded sequence of black clean medium sand and gray silty fine sand. This interlayered sequence was well exposed in the two trenches excavated at Unit 1 location, but appears to be less significant towards the east, at Unit 2 location. Although silty horizons occur throughout the remaining thickness of this layer, they tend to be much less frequent with depth and the lower 15 to 25 feet of sand is generally clean.

The material of this unit is basaltic in composition and of late Pleistocene age (Missoula sediments are considered to be 13,000 to 19,000 years in age). The unit is locally lightly cemented and contains scattered gravel-size material throughout.

The range of grain size distributions for the upper interbedded sequence at Unit 1 location is shown on Figure 2Q B-2, and corresponding data for the remainder of this unit are given on Figures 2Q B-3 and 2 B-9. The natural moisture content averages about 3 percent. Based on the standard penetration tests (summarized on Figures 2Q A-38 and 2Q A-39 of Appendix 2Q A) the relative density of this layer ranges from about 60 to more than 90 percent, with an average of approximately 75 percent (using correlation from Gibbs and Holtz, 1957). This assessment of the in situ relative density is generally confirmed by the results of static cone penetration testing (summarized on Figures 2Q A-50 and 2Q A-51 of Appendix 2Q A). Attempts to measure relative density directly, by measuring in situ sand cone and nuclear probe densities and by performing maximum/minimum density tests on representative samples of the material, could not confirm the relative densities inferred from the penetration testing. These data are summarized on Table 2Q B-6 of Appendix 2Q B. Direct procedures were attempted only within the more homogeneous zones of the unit, since any attempt to determine relative densities directly within stratified materials (such as occur within the upper section of the layer) would result in an underestimate of true relative density as a result of mixing of the separate soil fractions. Even within the clean medium sands, however, sorting existed (Figures 2Q A-54 and 2Q A-55 of Appendix 2Q A) and thin gradational layers could be observed. It is concluded that the slight stratification observed within the relatively homogeneous materials is sufficient to render direct measurements of relative density questionable.

6.1.3 VERY DENSE SILTY FINE SAND

At approximate elevation 490 feet at Unit 1 location (rising to approximate 495 feet at Unit 2) there occurs a very distinct contact between the recent Missoula and older Pre-Missoula sediments. At the Site, this contact is delineated by a very dense, dark yellowish brown/olive gray silty fine sand which has an average thickness of about 10 feet. This silty material is noticeably less basaltic in composition than the overlying Missoula sediments. In addition, the standard penetration testing shows a sharp increase in recorded blowcounts (Figures 2Q A-38 and 2Q A-39) with average SPT values on the order of 100 blows/foot. It was found impossible to advance the cone penetration test apparatus beyond the Pre-Missoula/Missoula contact. Typical grain size distributions for the silty fine sand are given on Figures 2Q B-4 and 2Q B-10. In situ moisture contents for this unit range from 5 to 10 percent.

6.1.4 VERY DENSE CLEAN FINE TO MEDIUM SAND

Underlying the very dense silty fine sand which marks the top of the Pre-Missoula sediments at the Site, there is a relatively thick layer (35 to 40 feet) of very dense olive-black clean medium sand containing zones of finer sand (a relatively persistent fine sand horizon occurs across the Site near the base of this unit). There is a gradational contact between this unit and the underlying gravelly horizon at an approximate average elevation of 445 feet.

Typical grain size distributions for this layer are shown on Figures 2Q B-5 and 2Q B-11, of Appendix 2Q B. The average in situ water content is about 2 to 3 percent. The upper 5 to 10 feet of this unit also show very high blowcounts (average SPT of 100 blows/foot), as does the overlying silty fine sand. The remainder of the unit shows interpreted relative densities ranging from 75 to in excess of 100 percent (Figures 2Q A-38 and 2Q A-39), with an average relative density on the order of 85 percent.

6.1.5 VERY DENSE CLEAN GRAVELLY SAND

Between average elevations of 445 feet and 420 feet there is a horizon of dense black clean gravelly sand which forms a gradational transition between the overlying sands and the underlying gravels. Typical grain size distributions are given on Figures 2Q B-6 and 2Q B-12. The average in situ water content is 2 to 3 percent.

6.1.6 VERY DENSE CLEAN SANDY GRAVEL

Below an average elevation of about 420 feet, the Site soils consist of olive black clean sandy gravels. Typical grain size distributions are shown on Figures 2Q B-7 and 2Q B-13. This zone also contains scattered cobbles and boulders, as noted during the drilling and sampling operations. Above the water table (mean elevation approximately 400 feet) the average water content is on the order of 2 to 3 percent.

This gravel unit is relatively thick, extending approximately 100 feet to the first silt unit of the Ringold Formation which is encountered at approximate elevation 315 feet. At the approximate mid-thickness of this gravel zone (elevation 360 to 380 feet) there is a noticeable increase in the seismic shear wave velocity and the gravels become more rock-like in character. In addition, during preparation of the cross holes for seismic testing (employing mud rotary drilling with driven casing), it was found impossible to advance the casing beyond approximate elevation 360 feet in all five holes, thus confirming the rock-like nature of the material.

6.1.7 INTERBEDDED HARD SILTS AND VERY DENSE SANDS AND GRAVELS

Below an average elevation of 315 to 320 feet, there is an interbedded sequence (Ringold Formation) of hard, gray and brown, fine sandy silt; very dense gray, black and brown, fine to coarse sand and silty sand; and varicolored to gray, sandy gravels. One core hole (E-1/E-19) at the Site penetrated this entire unit to basalt, and the core hole log is given in Appendix 2R. Green-black vesicular basalt was encountered at elevation -200 feet, approximately 725 feet below ground surface.

In situ water contents of the hard silts range from 20 to 35 percent. These water contents are lower than the plastic limits, indicating the overconsolidated nature of the materials. The consolidation testing carried out on core samples from these zones indicates that the Ringold materials at the Site have been preloaded to about 8-10 tons per square foot in excess of existing effective overburden pressures.

6.2 ENGINEERING PROPERTIES OF SITE FOUNDATION SOILS

6.2.1 INDEX PROPERTIES

The measured and estimated index properties of the Site foundation soils are given in Table 2Q-3. Because of the relative uniformity of conditions across the Site (Figures 2Q A-1 through 2Q A-37), with interface elevations of major layers differing by only a few feet, the profile suggested in Table 2Q-3 can be considered applicable to both units. The in-place densities of some of the units have been estimated (based on knowledge of grain size distributions and specific gravities) because reliable in situ measurements could not be obtained.

All the index and classification tests carried out on soil samples from the Site are given in Appendix 2Q B. Relative densities for the upper sand units are estimated from in situ penetration test results, given in Appendix 2Q A and summarized on Figures 2Q A-38, 2Q A-39, 2Q A-50 and 2Q A-51. Attempts to measure relative densities directly from in situ density tests within the trenches and laboratory maximum/minimum density tests, were not able to confirm the relative densities inferred from the penetration testing. Within the clean sands at approximate elevation 510 feet, testing at two locations indicated relative densities on the order of 50 percent (Table 2Q B-6). The in-place material demonstrated gradational layering; this condition results in an underestimate of the true relative density. In addition, the results of plate load tests carried out at these locations indicate that the soil properties are not representative of a low relative density material.

6.2.2 SHEAR STRENGTH

Static - Figure 2Q-6 presents a summary of shear strength data determined for the upper sand materials at the Site. The tests were carried out on both thin-walled tube samples and on recompacted samples. The undisturbed samples were obtained primarily for cyclic triaxial modulus and damping ratio determinations, and the triaxial compression tests were performed at the completion of each of the cyclic triaxial tests. The reconstituted samples, which were tested in the cyclic triaxial apparatus, were also subsequently triaxial-compression tested. In addition, reconstituted samples were prepared specifically for static triaxial compression testing.

The shear strength data for the samples recompacted to 75 percent relative density, shown on Figure 2Q-6, indicate peak friction angles on the order of 47 degrees. These high values are attributed to the angular nature of the Missoula sands, which results in an interlocking assemblage of grains. It should be noted that vibration methods were necessary to prepare the samples to 75 percent relative density, within the sample preparation molds.

Testing procedures and laboratory data are presented in Appendix 2Q B.

- Because of the deep water table at the Site
e elevation 400 feet) and the nature of the materials
he water table (very dense gravels, hard silts and
nse sands with shear wave velocities in excess of
s) earthquake-induced instabilities resulting from
ction, cyclic strain softening or strength degra-
were not considered relevant. No cyclic testing,
d specifically to investigate strength degradation
, was therefore undertaken.

MODULUS

ion material modulus values are required for static
ent analyses and for dynamic soil/structure inter-
analysis. Static moduli were determined in situ by
oad testing within the near surface Missoula sands
Menard pressuremeter testing within both the upper
to approximate depth 100 feet) and the more competent
ls below the water table. Descriptions of the test-
cedures, together with the field data, are given in
x 2Q A.

moduli were measured in situ using geophysical
techniques, and in the laboratory by cyclic triaxial
onant column testing. The laboratory procedures and
e presented in Appendix 2Q B.

amic and static moduli are summarized by material
d as a function of level of strain, on Figures 2Q-7
8. Each of the data sets are plotted at the appro-
level of shear strain, and intermediate values are
lated using the empirical curves given by Seed and
1970. Also shown on Figure 2Q-8 is an interpreted
modulus profile for calculation of static settle-
nder the major structures. The modulus profile was
shed by choosing moduli corresponding to representa-
ear strain levels expected to be induced at various
beneath the loaded surface. The shear strain levels
derive the appropriate moduli for each material type
icated on Figure 2Q-8.

oratory data shown on Figure 2Q-7 are seen to lie
the recommended design curves, which are based
ly on the in situ test results with interpolations
ng to data presented by Seed and Idriss, 1970. In
lar, the relative differences between field and
ory data are greatest at the lower levels of shear
These findings are consistent with comparisons of
nd laboratory moduli presented by others (e.g.,

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7.0 GROUNDWATER

ater conditions in the vicinity of the Site are ed in Appendix 2P. At present the groundwater table pproximate elevation 400 feet, and slopes very gently the approximate south. A rise in the groundwater o an approximate elevation of 412 feet (MSL) would be ated with the construction of the Ben Franklin Dam 1979) in the vicinity of the Site.

planned excavation depths at the Site will extend to mate elevation 490 feet. Groundwater will not there- encountered in any of the Site excavations.

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8.0 ENGINEERING ANALYSES AND EVALUATIONS

8.1 GENERAL

The subsurface soils at the Skagit/Hanford Site are competent to provide foundation support for the central plant structures (at their proposed founding elevations) under both static and dynamic loading conditions. Except where the loose surficial soils occur beneath proposed founding elevations, there will be no requirement to overexcavate and replace with structural backfill beneath the central plant foundations. Where adjacent structures are founded at different elevations, portions of the upper structure will be founded partially on structural backfill (e.g., diesel generator buildings).

A rise in the groundwater table due to possible future projects (e.g., Ben Franklin Dam, Hardy, 1979) will not impact foundation stability, while liquefaction, under the dynamic motions imposed by the design basis earthquake, is not a potential problem under existing or potential conditions.

Brief abstracts of all computer programs employed in the engineering analyses are given in Section 8.5.

8.2 FOUNDATION EVALUATIONS

8.2.1 BEARING CAPACITY

The major central plant structures will all be supported on large mat foundations. Based on the nature of the subsurface materials and the use of mat foundations, the allowable bearing capacities based on failure considerations will be very large and will not control foundation design. Instead, structure settlements will control.

8.2.2 STATIC SETTLEMENTS

Settlements beneath the central plant facilities have been estimated on the basis of elasticity theory, using the soil modulus profile presented on Figure 2Q-8. The loads were considered to act uniformly over each foundation area at the respective foundation elevations. The stress distributions were determined by superimposing Boussinesq solutions (computer program SETTLE) for uniformly-loaded rectangular areas on a semi-infinite, homogeneous, isotropic mass (Taylor, 1948). The resulting displacements were determined by integrating strains (evaluated from the stress values and the elastic modulus profile) over a vertical line beneath the point of displacement evaluation. The results are given on Figure 2Q-11 (Column 1).

The use of the modulus profile shown on Figure 2Q-8 represents an idealization for materials such as exist at the Skagit/Hanford Site. In reality, the soil modulus will depend on both the level of strain and on the effective confinement. An attempt has been made to use modulus values at representative strain levels (Figure 2Q-8), but the effect of level of confining stress has been averaged. For comparison purposes, another settlement computation was undertaken using the data presented on Figure 2Q-7. The same strain levels were chosen as for the initial settlement analysis, but the modulus was adjusted to reflect increased stiffness with increased confinement in accordance with the data presented on Figure 2Q-7. The same settlement computation procedure was used, but the modulus values were varied in accordance with the level of confining pressure. The results of this second settlement analysis are also presented on Figure 2Q-11 (Column 2). Beneath all of the comparatively lightly loaded structures, the agreement in settlement predictions between the two methods is very close. Beneath the heavily loaded reactor building, settlements are reduced on the order of 25 to 30 percent as a result of the stiffening of the soil profile associated with the increased level of confinement.

It is conservatively recommended that estimated total elastic settlements be taken as those given in Column 1 (Figure 2Q-11). The settlements will occur essentially at the same time as loading, and post-construction components of the elastic settlements will be determined by the proportion of the total loading which occurs as live loading.

Inspection of some of the borehole logs (e.g., W-1, Figure 2Q A-1) suggests a decrease in standard penetration resistance blowcounts in the lower 10 to 15 feet of the Missoula sediments. Since the plate load tests were carried out in the upper Missoula sands (above elevation 510 feet), it might be inferred that the deduced plate load moduli are not representative of the entire thickness of the Missoula sediment layer. It is not anticipated that the actual settlements will exceed estimated settlements, however, for the following reasons:

- on average, standard penetration and Dutch cone resistances do not show any decrease towards the base of the Missoula sands (Figures 2Q A-38, 2Q A-39, 2Q A-50, 2Q A-51);

- the elastic modulus value used to model the Missoula sands has been conservatively chosen with respect to the plate load test values when an adjustment is made for the appropriate level of shear strain (Figure 2Q-8); and

- the proportion of the total estimated settlements attributed to elastic settlements of the Missoula sands is generally less than 20 to 25 percent.

In addition to elastic-type settlements discussed above, long-term settlements may occur. Consolidation testing of the silt and clay-silt horizons of the Ringold Formation (Appendix 2Q B) demonstrated the very stiff nature of these materials in the lower part of the formation (e.g., below approximate elevation 175 feet). In the upper Ringold Formation there are relatively thin sandy-silt units (total thickness approximately 20 feet) which appear to be somewhat less stiff than the underlying silts (Appendix 2Q B, Figures 2Q B-28 and 2Q B-29). The surface loadings imposed by the central plant facilities may induce some consolidation-type settlements within the upper sandy-silt horizons. It is considered, however, that the total contribution to foundation settlements from materials of the Ringold Formation will not exceed those determined by the elastic analysis as a result of the relatively conservative modulus values used to represent the Ringold materials. In addition, the thin upper silty zones will consolidate relatively rapidly; long-term, post-construction settlement contributions from these materials are not anticipated.

Long-term settlements may also be associated with medium dense to dense granular deposits. Swiger, 1974, reports that long-term or secondary settlements may ultimately equal the primary or instantaneous settlements, with up to 25 percent of the secondary settlements occuring in the first year following construction. Assuming that secondary settlements will occur within the medium dense to dense Missoula sands (above elevation 490 feet), and will equal the magnitude of the initial elastic settlements within this material, the long-term settlements are estimated as shown on Figure 2Q-11 (Column 3). Secondary settlements within the very dense and stiff materials below elevation 490 feet are considered negligible.

Maximum total estimated settlements (primary plus secondary) for various locations on the Plant structures are shown on Figure 2Q-11 (Column 4). Assuming that, on average, live loads constitute 25 percent of the total load and that all the secondary settlements occur following construction (conservative), the maximum post-construction settlements for the structures are as shown on Figure 2Q-11 (Column 5).

Maximum total differential and post-construction differential settlements may be evaluated from the information presented on Figure 2Q-11 (Columns 4 and 5 respectively). Differential settlements for the large mat foundations may be taken as the difference between the estimated total settlements or as 35 percent of the total settlement (Terzaghi and Peck, 1967), whichever is greater.

Vertical elastic subgrade moduli values for use in structural design of the mat foundations may be evaluated from the data given on Figure 2Q-11. Vertical subgrade moduli for the auxiliary/fuel/control/reactor mat foundation are estimated at 30 to 40 pounds per square inch per inch, while the more deeply founded ultimate heat sinks and radwaste buildings have a subgrade modulus of approximately 50 pounds per square inch per inch.

8.2.3 DYNAMIC MOVEMENTS

Dynamic motions under a design earthquake loading, corresponding to a synthesized time history conforming to Reg. Guide 1.60 response spectra and anchored to a zero period acceleration of 0.35 g, were determined using a soil/structure interaction analysis. The soil profile indicated by the data given on Figures 2Q-7 and 2Q-9 was subjected to a one-dimensional wave propagation analysis using program SHAKE. This analysis enabled the evaluation of the effective shear strain levels (and hence the shear modulus values) throughout the soil column corresponding to the 0.35 g design time history applied at ground surface (elevation 525 feet MSL). Using the layered half space representation of the foundation soils derived from the SHAKE analysis, complex impedance functions were derived to represent the foundation soil dynamic response in the vertical and horizontal translational, and rotational modes of motion. The impedance functions were derived for a rigid circular plate of zero thickness located at the ground surface, using program LUCON (based upon information presented in Luco, 1976). The auxiliary/fuel/control/reactor building was represented by a simplified stick model with an equivalent circular rigid basemat of radius 142 feet. The simplified structural model was coupled with the complex impedance functions representing the soil foundation, and the response of the combined system to the 0.35 g design time history evaluated using program FASS. Maximum calculated movements at the center of the rigid basemat (at ground surface) are shown in Table 2Q-4. The horizontal and rotational motions refer to the response to a horizontal ground motion applied in the N-S direction; the vertical response was calculated for vertical ground motions with a peak free field surface acceleration of 0.35 g and a time history satisfying Reg. Guide 1.60 response spectra.

8.2.4 STABILITY OF SUBSURFACE SOILS

8.2.4.1 Liquefaction

The subsurface materials below approximate elevation 360-380 feet (MSL) show shear wave velocities in excess of 2000 feet per second (see Appendix 2L); therefore earthquake-induced instabilities resulting from liquefaction, cyclic strain softening and strength degradation do not require further consideration.

Within the saturated gravels below the water table (elevation 400 feet MSL), a free field dynamic analysis (using the dynamic material properties given on Figures 2Q-7 and 2Q-9) indicated that for a maximum ground acceleration of 0.35 g and a synthetic time history conforming to Reg. Guide 1.60 requirements, the average shear stress to effective overburden stress ($t/\sigma v$) ratio (taken as approximately 65 percent of the maximum $t/\sigma v$ ratio) has a value of about 0.13. This value can also be derived using the approximate approach developed by Seed and Idriss, 1971. Information available within the literature (Seed and Idriss, 1971) indicates that under these conditions there is no potential for the development of significant excess pore water pressures during 0.35 g earthquake shaking. In addition, the relatively free-draining nature of these saturated gravelly materials will further mitigate against the development of high pore water pressures (Wong et al., 1975). Potential increases in the water table elevation at the Site (Hardy, 1979) will not impact the above conclusions. The above discussion refers to the free field condition, but the liquefaction potential will not be increased by the presence of the planned surface structures.

8.2.4.2 Dynamic Compaction Settlements

During earthquake shaking, shear strains will be induced within the foundation soils. Depending on the relative densities of these materials, dynamic compaction permanent settlements may occur. Assuming vertically propagating shear waves, the average effective shear strain levels (equal to 0.65 times the maximum shear strain level) within the free field (i.e., without surface structures) for each of the soil layers are summarized in Table 2Q-5. These strains correspond to a synthesized time history satisfying Reg. Guide 1.60 response spectra with a maximum surface horizontal acceleration of 0.35 g.

Additional strains will occur within the foundation soils as a result of the relative motions between the structure and the soil foundation during earthquake shaking. The soil/structure interaction analysis discussed previously indicates a maximum relative horizontal movement between the structure and free field surface displacements of 1.71 inches during a 0.35 g earthquake. Subsurface shear strains corresponding to a relative surface movement of 1.71 inches were determined approximately using standard static solutions. The pseudo-elastic soil properties (derived from data presented on Figure 2Q-7) employed in the soil/structure interaction analysis were combined with stress distributions for a horizontal surface shear loading on an isotropic homogeneous half space (Boussinesq solution), in order to determine the relationships between surface movements and subsurface shear strains.

The average shear strains within each of the soil layers as a result of motions of the auxiliary/fuel/control/reactor building during 0.35 g shaking, are given in Table 2Q-5. These shear strain levels are effective strain levels, taken as 65 percent of the maximum strain levels corresponding to the maximum horizontal movement of 1.71 inches.

Below elevation 490 feet, the effective shear strain levels are very low. The very dense nature of the soil below elevation 490 feet, combined with the low shear strain levels during earthquake shaking, ensures that dynamic compaction settlements within these materials during a 0.35 g design event would be negligible.

Effective shear strain levels within the Missoula sands (above elevation 490 feet) are seen to be on the order of 0.25 percent. The experimental data presented on Figure 2Q-10 suggest that for 10 cycles (Seed et al., 1975) of 0.25 percent single amplitude shear straining within these materials, vertical compaction strains on the order of 0.5

percent may occur. Beneath the auxiliary/fuel/control/reactor building founded at elevation 507 feet, this would correspond to a permanent vertical settlement due to dynamic compaction of almost 1 inch.

Pyke et al (1975) have shown that permanent dynamic compaction strains under multidirectional shaking, such as occurs with earthquake loadings, will exceed the corresponding strain levels induced by unidirectional shearing generally employed in laboratory testing. Because of the nature of the testing program (inability to test undisturbed samples under zero lateral strain conditions) and the inevitable disturbance to the test samples, the design curve given on Figure 2Q-10 is considered conservative. Data presented by Seed and Silver, 1972, suggest vertical compaction strains (at 0.25 percent applied shear strain level) of about half of those shown on Figure 2Q-10 for an inferred average relative density of 75 percent. It is therefore considered that for a design earthquake with a peak ground acceleration of 0.35 g, permanent settlements beneath the auxiliary/fuel/control/reactor building will not exceed 1 inch.

8.3 LATERAL EARTH PRESSURES

8.3.1 STATIC EARTH PRESSURES

Static earth pressures will develop on subsurface walls as a result of the self-weight of the soil and the existence of and surcharge loading adjacent to the subsurface wall. The minimum lateral earth pressure which can be applied to any subsurface wall is that which corresponds to mobilization of the full shear strength of the soil, i.e., the minimum lateral pressure under which the soil achieves a state of limiting equilibrium. The lateral earth pressure at any depth under these conditions is equal to the active earth pressure coefficient (Ka) times the vertical stress (overburden plus surcharge). For an angle of internal friction (Ø) of backfill materials equal to 35 degrees, the active earth pressure coefficient (Ka) has a value of 0.27. Figure 2Q-6 presents strength data from static triaxial compression tests carried out on specimens of structural backfill material (clean Missoula sands above elevation 490 feet) reconstituted to 75 percent relative density. These data suggest that use of an angle of internal friction of 35 degrees constitutes an acceptable design assumption.

Mobilization of active earth pressures requires that the wall be free to translate or rotate sufficiently to reduce the earth pressures to minimum values. The active earth pressure coefficient applied to backfill self weight and surcharge is therefore applicable only to subsurface walls which are of the yielding type.

For massive relatively rigid subsurface walls (or walls effectively restricted from lateral movement), the static earth pressures will exceed active earth pressures. In this case, it is recommended that an at-rest earth pressure coefficient (Ko) equal to 0.5 should be applied to the backfill self weight and surcharge stresses.

A summary of static earth pressures due to backfill self weight and surface surcharge, for yielding and unyielding walls, is shown on Figure 2Q-12. Since all subsurface walls will be above the water table, hydrostatic loading on subsurface walls does not require consideration.

8.3.2 COMPACTION STRESSES

Additional horizontal stresses will be induced on unyielding subsurface walls as a result of compaction of structural backfill by vibratory compactors. Based on data presented by D'Appolonia et al, 1969, it is recommended that a horizontal pressure of 115 pounds per square foot be uniformly applied to subsurface unyielding walls in order to account for stresses induced by backfill compaction.

During dynamic loading (discussed in 8.3.3) compaction induced stressed will tend to be relieved, and the compaction component of lateral stress need not therefore be considered in conjunction with full dynamic lateral earth pressures.

8.3.3 DYNAMIC EARTH PRESSURES

The response of subsurface walls during earthquake loading is a function of the relative soil-structure displacements, structural rigidity, backfill soil properties, foundation conditions and characteristics of the applied earthquake motions.

For subsurface walls which essentially move in phase with the ground motions during an earthquake, dynamic forces are generally calculated using the Mononobe-Okabe limiting equilibrium procedure as presented by Seed and Whitman,

1970. In accordance with recommendations given in Seed and Whitman, 1970, for backfill with an angle of friction equal to 35 degrees, the dynamic force is approximately equal to the inertia force on a soil wedge which extends behind the wall a distance equal to three-quarters of the height of the wall. The dynamic force resultant is generally considered to act somewhat above the mid-height of the wall, but a uniform pressure distribution (i.e., resultant force at mid-height of wall) may be employed to represent the dynamic pressure increment. The dynamic pressure distribution for the design basis earthquake is summarized on Figure 2Q-12.

For large massive structures (such as the auxiliary/fuel/control/reactor building) which may move out of phase with the ground motions, dynamic pressures on subsurface walls should be established from a soil/structure interaction analysis. There are, however, no subsurface structural walls associated with the auxiliary/fuel/control/reactor building. For structures with significant embedment, such as the Ultimate Heat Sink, dynamic pressures on subsurface walls will be established by soil/structure interaction analysis. The effect of horizontally propagating surface waves (such as Rayleigh waves) will be considered if significant in establishing these pressures. 25

8.4 EARTHWORK

8.4.1 EXCAVATIONS

With the exception of the upper 6 to 8 feet of loose silty fine sand at the Site, the in-place soils will provide suitable foundation materials for all the major structures. Therefore, there is no requirement to overexcavate beyond planned foundation depths and replace with structural backfill, except where the proposed founding elevations lie above the competent soils. In-place materials which will provide suitable founding materials are readily identified as clean black medium sands or, at specific locations, interbedded clean black medium sands and gray silty fine sands. The overlying dark yellowish brown to gray silty fine sands must be removed beneath all structural foundations.

Although exact foundation elevations are not currently available, the approximate foundation grades in relation to the in-place soils for all the major structures are shown on Figure 2Q-5. The deepest foundations are those of the ultimate heat sinks and radwaste buildings, and these structures will be founded on the very dense sands at or below elevation 490 feet. All other structures will be founded within the overlying medium dense to dense sands, or on structural fill or backfill. The diesel generator buildings, being located adjacent to the more deeply founded auxiliary/fuel/control/reactor buildings, will be founded largely on structural backfill. In addition, there will be a requirement at the Unit 2 site, in particular, to remove the surficial material and replace with structural backfill to the diesel generator building foundation grade. It will probably also be necessary to overexcavate and backfill beneath the eastern sections of the turbine buildings (particularly at Unit 2 location), depending upon exact founding elevations.

All temporary excavation slopes should be cut no steeper than 1.0 vertical on 1.5 horizontal. Although the light cementation within the sands permits the excavation of steeper slopes, such slopes will slough within a short period of time. The toes of all slopes should be set back from the structures sufficiently to provide adequate working space for backfilling and compacting with heavy equipment. Chemical or other types of slope protection materials should be applied as necessary to prevent wind erosion, and prepared foundation soils should be protected with mud mats.

Because of the very deep water table, dry conditions will exist in all excavations.

8.4.2 FILL AND BACKFILL

All Category I structures, with the exception of the diesel generator buildings, will be founded on the existing in-place soils. Of the central plant structures, only the diesel generator buildings (Category I structures) and portions of the turbine buildings (non-Category I structures) will be founded on structural backfill. Backfill will also be placed around all of the structures to Site grade (tentatively established as elevation 526 feet).

During excavation, material suitable for structural backfill should be stockpiled and all other material separately stored for use as non-structural area fill. The surficial zone of loose silty fine sand which occurs across the Site will not be suitable as structural fill or backfill. In addition, the upper zone of the underlying material is very silty in parts (interbedded gray silty fine sand and black clean medium sand) and this material should not be used as structural backfill. The black clean medium sands (Missoula sands), which occur beneath the upper silty materials and above the very dense Pre-Missoula sediments, will provide suitable structural backfill material and should be appropriately stockpiled during excavation of the deeper foundations. The maximum particle size for fill and backfill materials should not be larger than 3 inches. The presence of larger particles may interfere with obtaining adequate compaction. It is anticipated that minimal oversize material will be encountered within the zone of soil identified as suitable fill material.

Backfill placement procedures should be established for the Site materials using a test fill. For the granular materials to be employed, a heavy steel drum vibratory roller will provide best results. Water contents, lift thickness and number of passes to achieve an average relative density of 85 percent and minimum relative density of 75 percent will be established by the test fill. Hand-operated compactors should be used close to structural walls.

For compaction control purposes, it will probably be more efficient to specify density requirements in terms of relative compaction rather than relative density. A test program will be necessary to establish the correlation between relative density and relative compaction for the Site soils. In addition, in order to provide a relatively rapid evaluation of the suitability of the placed backfill during construction, it may be necessary to establish correlations between the material grain size characteristics [e.g., effective grain size range (Burmister, 1962)] and the maximum density (as determined by ASTM Designation: D 2049). In this manner, two relatively rapid tests (i.e., in-place density measurement and grain size analysis) can be used to obtain a rapid assessment of the relative compaction (and hence relative density). Alternatively, correlations could be established between relative density and percent Modified Proctor compaction, in order to provide for a rapid compaction control procedure.

To assure that proper control of fill operations is maintained, a qualified soil engineer should be present to systematically check the fill quality and placed density. For quality control purposes, at least one density test should be performed for each one-foot thickness of compacted fill for every 22,500 square feet of material placed.

8.5 COMPUTER PROGRAMS ABSTRACTS

FASS (Bechtel Program CE935) is a computer program for calculating the horizontal and vertical seismic time history response of a soil-structure interaction system. The system consists of a structure with a rigid base mat founded on the surface of a foundation medium. Dynamic characteristics of the foundation medium are represented by a set of frequency-dependent foundation impedances associated with each component of motion of the structure base.

Because the foundation impedances are frequency dependent, standard techniques (such as model superposition or direct integration) cannot be used directly. Instead, FASS uses the frequency domain analysis procedures and the Fourier transform method for computing response.

LUCON (Bechtel Program CE970) is a computer program for evaluating the impedance functions for a rigid circular foundation placed on a layered visco-elastic medium. The program computes for given site characteristics and foundation geometry, the horizontal, vertical and rocking impedance functions and compliance functions for a given set of frequencies. The foundation may be either layered or a uniform elastic half space. Hysteric and Voigt types of soil damping may be considered. Only one type of damping can be used for all layers, but the damping constants may vary from layer to layer.

FLUSH (John Lysmer et al., University of California, Berkeley) is a program for performing 2-D or approximate 3-D analyses of soil-structure interaction problems. The program works in the frequency domain to determine the response of a soil-structure system to vertically propagating shear or compression waves associated with a specified seismic acceleration time history. The modulus and damping ratio of the soil may vary with effective shear strain. FLUSH will also perform a one-dimensional analysis to determine the free field soil response to earthquake shaking.

SETTLE (Golder Associates, Version 780) is a computer program for calculating vertical surface settlement and stresses at depth due to surface loads and excavations. The program can approximate any surface loading pattern as a sum of rectangular loaded areas. SETTLE utilizes Boussinesq or Westergaard stress distribution equations. Deformations are calculated using elastic or consolidation equations, with layered inhomogeneity of deformation properties.

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TABLE 2Q B-1

SOIL SAMPLE DESCRIPTIONS

BORING W-1

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS/FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION
S-1	0.0-1.5	3	3.5	-	Slightly silty SAND, poorly graded. Maximum size about 5mm. About 90% subangular to rounded, fine to medium sand and 10% non-plastic fines. Dark yellowish brown. Dry. Very loose. No reaction to HC1.
S-2	5.0-6.5	8	0.3	M.A.	Silty SAND. Maximum size about 5mm. About 70% subangular to rounded, fine sand and 30% non-plastic fines. Dark yellowish brown. Dry. Loose. No reaction to HC1.
S-3	10.0-11.5	18	4.1	M.A.	Silty SAND. Maximum size about 10mm. About 60% subangular to rounded, fine sand and 40% non-plastic fines. Olive gray. Dry. Medium dense. Weak reaction to HCl.
S-4	15.0-16.5	20	-	-	No recovery
S-5	17.5-19.0	22	-	-	No recovery
S-6	20.0-21.5	33	-	-	No recovery
S-7	22.5-24.0	34	7.2	M.A. S.G.	Clean SAND, poorly graded. Maximum size about 10mm. Subangular to rounded, fine to medium sand. Varicolored to olive black. Dry. Dense. Weak reaction to HCl.
S-8	25.0-26.5	28	0.3	M.A.	Clean SAND, poorly graded. Maximum size about 12mm. Subangular to rounded, fine to coarse sand. Varicolored to olive black. Dry. Medium dense. Weak reaction to HCl.
S-9	30.0-31.5	22	0.9	M.A.	Clean gravelly SAND, poorly graded. Maximum size about 20mm. About 85% subangular to rounded, fine to coarse sand and 15% fine gravel. Varicolored to olive black. Dry. Medium dense. Weak reaction to HC1.
S-10	35.0-36.5	23	3.5	M.A. S.G.	Clean SAND, poorly graded. Maximum size about 10mm. Subangular to rounded, fine to coarse sand. Varicolored to olive black. Dry. Medium dense. Weak reaction to HCl.

TABLE 2Q B-1

SOIL SAMPLE DESCRIPTIONS

BORING W-1

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS/FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION
S-11	40.0-41.5	66	8.1	M.A. S.G.	Silty SAND, poorly graded. Maximum size about 6mm. About 80% subangular to subrounded, fine sand and 20% non-plastic fines. Dark yellowish brown. Dry. Very dense. Weak reaction to HCl.
S-12	45.0-46.5	71	6.3	M.A.	Slightly silty SAND, poorly graded. Maximum size about 6mm. About 90% subangular to rounded, fine to medium sand and 10% non-plastic fines. Dark yellowish brown. Dry. Very dense. Weak reaction to HCl. | 24
S-13	50.0-51.5	92	5.7	M.A.	Slightly silty SAND, poorly graded. Maximum size about 5mm. About 90% subangular to rounded, fine to medium sand and 10% non-plastic fines. Varicolored to dark yellowish brown. Dry. Very dense. Weak reaction to HCl.
S-14	55.0-56.5	66	-	-	No recovery
S-15	57.5-59.0	126	-	S.G.	Clean SAND, poorly graded. Maximum size about 6mm. Subangular to rounded, fine to medium sand. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-16	60.0-61.5	128	-	-	No recovery
S-17	63.0-64.5	116	-	M.A.	Slightly silty SAND, poorly graded. Maximum size about 5mm. About 90% subangular to rounded, fine to medium sand and 10% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-18	65.0-66.5	68	2.2	M.A.	Clean SAND, poorly graded. Maximum size about 10mm. Subangular to rounded, fine to coarse sand. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-19	70.0-71.5	66	2.0	-	Clean SAND, poorly graded. Maximum size about 10mm. Subangular to rounded, fine to coarse sand. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.

TABLE 2Q B-1

SOIL SAMPLE DESCRIPTIONS

BORING W-1

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS/FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION
S-20	75.0-76.5	54	1.8	M.A. S.G.	Clean SAND, poorly graded. Maximum size about 5mm. Subangular to rounded, fine to medium sand. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-21	80.0-81.5	62	3.2	M.A.	Clean SAND, poorly graded. Maximum size about 10mm. Subangular to rounded, fine to coarse sand. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-22	85.0-86.5	79	1.8	S.G.	Clean gravelly SAND, well graded. Maximum size about 10mm. About 70% subangular to rounded, fine to coarse sand and 30% fine gravel. Varicolored. Dry. Very dense. Weak reaction to HC1.
S-23	90.0-91.5	105	2.5	-	Clean gravelly SAND, well graded. Maximum size about 25mm. About 75% subangular to rounded, fine to coarse sand and 25% fine to coarse gravel. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-24	95.0-96.5	126	0.2	-	Clean gravelly SAND, well graded. Maximum size about 25mm. About 70% subangular to rounded, fine to coarse sand and 30% fine to coarse gravel. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-25	100.0-101.5	106	4.2	-	Slightly silty SAND, poorly graded. Maximum size about 25mm. About 90% subrounded, fine to coarse sand and 10% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HC1.
S-26	104.5-106.0	-	-	M.A.	Slightly silty gravelly SAND, poorly graded. Maximum size about 50mm. About 75% subangular to rounded, fine to coarse sand, 15% fine to coarse gravel and 10% non-plastic fines. Varicolored to dark yellowish brown. Dry. Weak reaction to HC1.
S-27	111.5-113.0	-	-	M.A.	Clean gravelly SAND, well graded. Maximum size about 40mm. About 55% subangular to rounded, fine to coarse sand and 45% fine to coarse gravel. Varicolored to olive gray. Dry. Weak reaction to HCl.

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TABLE 1

SOIL SAMPLE DESCRIPTIONS

BORING W-1

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS/FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION
S-28	115.0-116.5	-	-	M.A.	Clean sandy GRAVEL, well graded. Maximum size about 50mm. About 55% subangular to rounded, fine to coarse gravel and 45% fine to coarse sand. Varicolored to olive gray. Dry. Weak reaction to HCl.
S-29	120.2-121.7	-	-	M.A. S.G.	Slightly silty sandy GRAVEL, well graded. Maximum size about 75mm. About 60% angular to rounded, fine to coarse gravel, 35% fine to coarse sand and 5% non-plastic fines. Varicolored to olive black. Dry. Weak reaction to HCl.
S-30	125.0-125.8	-	-	-	Clean sandy GRAVEL, well graded. Maximum size about 50mm. About 60% subangular to rounded, fine to coarse gravel and 40% fine to coarse sand. Varicolored to olive black. Dry. Weak reaction to HCl.
S-31	130.5-132.0	-	3.5	M.A.	Clean sandy GRAVEL, well graded. Maximum size about 50mm. About 65% subangular to to rounded, fine to coarse gravel and 35% fine to coarse sand. Varicolored to olive gray. Dry. Weak reaction to HCl.
S-32	135.5-137.0	-	-	M.A. S.G.	Slightly silty, sandy GRAVEL, well graded. Maximum size about 60mm. About 65% angular to rounded, fine to coarse gravel, 30% fine to coarse sand and 5% non-plastic fines. Varicolored to olive gray. Saturated. Weak reaction to HCl.

TABLE 2Q B-1

SOIL SAMPLE DESCRIPTIONS

BORING W-2

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS/FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION
S-19	73.5-75.0	66	-	-	Clean gravelly SAND, well graded. Maximum size about 10mm. About 90% subangular to rounded, fine to coarse sand and 10% fine gravel. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-20	78.5-80.0	86	2.3	M.A.	Slightly silty SAND, poorly graded. Maximum size about 10mm. About 90% subangular to subrounded, fine to medium sand and 10% non-plastic fines. Light olive gray. Dry. Very dense. Weak reaction to HCl.
S-21	83.5-85.0	79	2.6	-	Clean SAND, poorly graded. Maximum size about 10mm. Subangular to rounded, fine to medium sand. Light olive gray. Dry. Very dense. Weak reaction to HCl.
S-22	88.5-90.0	77	2.9	-	Clean SAND, poorly graded. Maximum size about 10mm. Subangular to rounded, fine to coarse sand. Light olive gray. Dry. Very dense. Weak reaction to HCl.
S-23	93.5-95.0	116	4.6	-	Clean gravelly SAND, well graded. Maximum size about 25mm. About 65% subangular to rounded, fine to coarse sand and 35% fine to coarse gravel. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-24	98.5-100.0	151	1.7	M.A.	Clean gravelly SAND, well graded. Maximum size about 25mm. About 55% subangular to subrounded, fine to coarse sand and 45% fine to coarse gravel. Varicolored to olive black and light olive gray. Dry. Very dense. Weak reaction to HCl.
S-25	103.5-105.0	87	1.6	-	Clean gravelly SAND, well graded. Maximum size about 25mm. About 55% subangular to rounded, fine to coarse sand and 45% fine to coarse gravel. Varicolored to olive black and light olive gray. Dry. Very dense. Weak reaction to HCl.

TABLE 2Q B-1

SOIL SAMPLE DESCRIPTIONS

BORING W-3

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION	
S-1	0.0-1.5	6	4.8	-	Silty SAND, poorly graded. Maximum size about 2mm. About 85% subrounded, fine sand and 15% non-plastic fines. Dark yellowish brown. Dry. Loose. No reaction to HCl.	24
S-2	3.5-5.0	10	9.8	-	Silty SAND. Maximum size about 2mm. About 65% subrounded, fine sand and 35% non-plastic fines. Dark yellowish brown. Dry. Loose. No reaction to HCl.	24
S-3	8.5-10.0	14	4.2	-	Silty SAND, poorly graded. Maximum size about 20mm. About 85% subrounded, fine to medium sand and 15% non-plastic fines. Dark yellowish brown. Dry. Medium dense. No reaction to HCl.	
S-4	13.5-15.0	44	3.9	M.A.	Slightly silty SAND, poorly graded. Maximum size about 10mm. About 95% subangular to subrounded, fine to coarse sand and 5% non plastic fines. Varicolored to olive black. Dry. Dense. Weak reaction to HCl.	
S-5	18.5-20.0	35	2.6	-	Clean SAND, poorly graded. Maximum size about 25mm. Subangular to subrounded, fine to medium sand. Varicolored to olive gray. Dry. Dense. Weak reaction to HCl.	
S-6	23.5-25.0	32	2.8	-	Clean SAND, poorly graded. Maximum size about 30mm. Subangular to subrounded, fine to coarse sand. Varicolored to olive black. Dry. Dense. Weak reaction to HCl.	
S-7	28.5-30.0	37	4.3	-	Clean SAND, poorly graded. Maximum size about 35mm. Subangular to subrounded fine to coarse sand. Varicolored to olive black. Dry. Dense. Weak reaction to HCl.	
S-8	33.5-35.0	35	4.4	-	Slightly silty SAND, poorly graded. Maximum size about 20mm. About 95% subangular to subrounded, fine to coarse sand and 5% non plastic fines. Varicolored to olive black. Dry. Dense. Weak reaction to HCl.	
S-9	38.5-40.0	79	7.1	-	Slightly silty SAND, poorly graded. Maximum size about 1mm. About 90% subangular to rounded, fine to medium sand and 10% non plastic fines. Dark yellowish brown. Dry. Very dense. Weak reaction HCl.	

TABLE 2Q B-1

SOIL SAMPLE DESCRIPTIONS

BORING W-18

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS/FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION
S-1	0.0-1.5	6	1.6	-	Silty SAND, poorly graded. Maximum size about 3mm. About 85% angular to subrounded, fine to medium sand and 15% non-plastic fines. Dark yellowish brown. Dry. Loose. No reaction to HCl.
S-2	3.2-4.7	9	-	-	Silty SAND. Maximum size about 4mm. About 60% angular to subrounded, fine to medium sand and 40% non-plastic fines. Varicolored to olive gray. Dry. Loose. No reaction to HCl.
S-3	8.2-9.7	25	3.5	-	Slightly silty SAND, poorly graded. Maximum size about 8mm. About 95% angular to subrounded, fine to medium sand and 5% non-plastic fines. Varicolored to olive black. Dry. Medium dense. Weak reaction to HCl. (8.2-9.5)
			7.1	-	Silty SAND. Maximum size about 2mm. About 60% angular to subrounded fine sand and 40% non-plastic fines. Olive gray. Dry. Medium dense. No reaction to HCl. (9.5-9.7)
S-4	13.2-14.7	31	2.4	-	Clean SAND, poorly graded. Maximum size about 5mm. Angular to subrounded, fine to coarse sand. Varicolored to olive black. Dry. Dense. Weak reaction to HCl.
			4.3	-	Silty SAND. Maximum size about 4mm. About 75% angular to subrounded, fine to medium sand and 25% non-plastic fines. Varicolored to olive black. Dry. Dense. Weak reaction to HCl. (13.6-14.7)
S-5	18.2-19.7	21	2.7	-	Slightly silty SAND, poorly graded. Maximum size about 10mm. About 95% angular to subrounded, fine to coarse sand and 5% non-plastic fines. Varicolored to olive black. Dry. Medium dense. Weak reaction to HCl.
S-6	23.2-24.7	16	2.2	-	Clean SAND, poorly graded. Maximum size about 10mm. Angular to subrounded, fine to coarse sand. Varicolored to olive black. Dry. Medium dense. Weak reaction to HCl.
S-7	28.2-29.7	35	2.4	-	Clean gravelly SAND, well graded. Maximum size about 30mm. About 80% angular to subrounded, fine to coarse sand and 20% fine to coarse gravel. Varicolored to olive black. Dry. Dense. Weak reaction to HCl.
S-8	33.2-34.7	41	2.7	-	Slightly silty gravelly SAND, poorly graded. Maximum size about 20mm. About 85% angular to subrounded, fine to coarse sand, 10% fine gravel and 5% non-plastic fines. Varicolored to olive black. Dry. Dense. Weak reaction to HCl.
S-9	38.2-39.7	103	7.2	-	Silty SAND, poorly graded. Maximum size about 3mm. About 85% angular to subrounded, fine to medium sand and 15% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-10	43.2-44.7	97	5.9	-	Silty SAND, poorly graded. Maximum size about 5mm. About 85% angular to subrounded, fine to coarse sand and 15% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl. 24
S-11	48.2-49.7	97	2.5	-	Slightly silty SAND, poorly graded. Maximum size about 7mm. About 90% angular to subrounded, fine to medium sand and 10% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.

TABLE 2Q B-1

SOIL SAMPLE DESCRIPTIONS

BORING W-18

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS/FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION
S-12	53.2-54.7	159	2.8	-	Slightly silty gravelly SAND. Maximum size about 40mm. About 75% angular to subrounded, fine to coarse sand, 15% fine to coarse gravel and 10% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-13	57.9-59.4	64	2.2	-	Slightly silty SAND, poorly graded. Maximum size about 20mm. About 95% angular to subrounded, fine to coarse sand and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-14	63.0-64.5	70	2.1	-	Slightly silty SAND, poorly graded. Maximum size about 10mm. About 90% angular to subrounded, fine to coarse sand and 10% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-15	67.8-69.3	50	1.9	-	Slightly silty SAND, poorly graded. Maximum size about 20mm. About 95% angular to subrounded, fine to coarse sand and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-16	72.9-74.4	69	2.0	-	Slightly silty SAND, poorly graded. Maximum size about 10mm. About 95% angular to subrounded, fine to coarse sand and 5% non-plastic fines. Dry. Very dense. Weak reaction to HCl.
S-17	77.9-79.4	78	2.5	-	Slightly silty SAND, poorly graded. Maximum size about 15mm. About 95% angular to subrounded, fine to coarse sand and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-18	82.8-84.3	81	-	-	Slightly silty gravelly SAND, well graded. Maximum size about 30mm. About 70% angular to subrounded, fine to coarse sand, 25% fine to coarse gravel and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-19	87.9-89.4	119	-	-	Slightly silty gravelly SAND, well graded. Maximum size about 30mm. About 75% angular to subrounded, fine to coarse sand, 15% fine to coarse gravel and 10% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl. (87.9-88.9)
			-	-	Silty SAND, poorly graded. Maximum size about 2mm. About 80% angular to subrounded, fine to medium sand and 20% non-plastic fines. Olive gray. Dry. Very dense. No reaction to HCl. (88.9-89.4)
S-20	92.9-94.4	111	-	-	Slightly silty gravelly SAND, poorly graded. Maximum size about 25mm. About 80% angular to subrounded, fine to coarse sand, 10% fine gravel and 10% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-21	97.9-98.6	112/2"	-	-	Slightly silty gravelly SAND, well graded. Maximum size about 25mm. About 60% angular to subrounded, fine to coarse sand, 30% fine gravel and 10% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.

TABLE 2Q B-1

SOIL SAMPLE DESCRIPTIONS

BORING E-5

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS. FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION
S-11	48.1-49.6	56	2.2	-	Slightly silty SAND, poorly graded. Maximum size about 7mm. About 95% angular to rounded, fine to medium sand and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-12	53.2-54.7	93	2.1	-	Slightly silty gravelly SAND, well graded. Maximum size about 25mm. About 85% angular to rounded, fine to coarse sand, 10% fine to coarse gravel and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-13	57.6-59.1	165	3.9	S.C.	Slightly silty gravelly SAND, well graded. Maximum size about 30mm. About 75% angular to rounded, fine to coarse sand, 20% fine to coarse gravel and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-14	62.6-64.1	132	2.2	-	Clean gravelly SAND, well graded. Maximum size about 25mm. About 85% angular to rounded, fine to coarse sand and 15% fine to coarse gravel. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-15	67.6-69.1	131	-	-	Clean SAND, poorly graded. Maximum size about 15mm. Angular to subrounded, fine to medium sand. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-16	72.8-74.3	96	1.2	-	Silty SAND, poorly graded. Maximum size about 1mm. About 85% angular to subrounded, fine sand and 15% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-17	77.8-79.3	139	1.7	-	Slightly silty gravelly SAND, well graded. Maximum size about 35mm. About 50% angular to rounded, fine to coarse sand, 40% fine to coarse gravel and 10% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-18	83.2-84.7	76	2.1	S.C.	Clean gravelly SAND, well graded. Maximum size about 20mm. About 70% angular to rounded, fine to coarse sand and 30% fine gravel. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-19	88.2-89.7	138	1.8	-	Slightly silty gravelly SAND, well graded. Maximum size about 30mm. About 65% angular to rounded, fine to coarse sand, 30% fine to coarse gravel and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-20	93.2-94.7	96	2.5	-	Slightly silty gravelly SAND, well graded. Maximum size about 20mm. About 65% angular to rounded, fine to coarse sand, 30% fine gravel and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.

TABLE 2Q B-1

SOIL SAMPLE DESCRIPTIONS

BORING E-6

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS/FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION
S-1	0.0-1.5	5	4.4	-	Slightly silty SAND, poorly graded. Maximum size about 3mm. About 95% angular to subrounded, fine to medium sand and 5% non-plastic fines. Dark yellowish brown. Dry. Loose. No reaction to HCl.
S-2	3.5-5.0	7	4.6	-	Slightly silty SAND, poorly graded. Maximum size about 4mm. About 90% angular to subrounded, fine to medium sand and 10% non-plastic fines. Dark yellowish brown. Dry. Loose. No reaction to HCl.
S-3	8.5-10.0	14	5.7	-	Silty SAND, poorly graded. Maximum size about 6mm. About 85% angular to subrounded, fine to medium sand and 15% non-plastic fines. Varicolored to olive gray. Dry. Medium dense. Weak reaction to HCl.
S-4	13.5-15.0	26	2.4	-	Clean SAND, poorly graded. Maximum size about 10mm. Angular to subrounded, fine to coarse sand. Varicolored to olive black. Dry. Medium dense. Weak reaction to HCl.
S-5	18.5-20.0	51	2.6	-	Clean gravelly SAND, poorly graded. Maximum size about 20mm. About 85% angular to subrounded, fine to coarse sand and 15% fine gravel. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-6	23.5-25.0	57	3.0	-	Slightly silty gravelly SAND, well graded. Maximum size about 25mm. About 80% angular to subrounded, fine to coarse sand, 15% fine to coarse gravel and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-7	28.5-30.0	88	-	-	Slightly silty SAND, poorly graded. Maximum size about 3mm. About 90% angular to subrounded, fine to medium sand and 10% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-8	33.5-35.0	106	2.6	-	Slightly silty SAND, poorly graded. Maximum size about 20mm. About 95% angular to subrounded, fine to medium sand and 5% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-9	38.5-40.0	96	3.5	-	Slightly silty SAND, poorly graded. Maximum size about 4mm. About 90% angular to subrounded, fine to medium sand and 10% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-10	43.5-45.0	127	4.0	-	Slightly silty SAND, poorly graded. Maximum size about 3mm. About 90% angular to subrounded, fine to medium sand and 10% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-11	48.5-50.0	99	2.3	-	Slightly silty SAND, poorly graded. Maximum size about 3mm. About 95% angular to subrounded, fine to medium sand and 5% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-12	53.5-54.8	163/10"	6.4	-	Slightly silty gravelly SAND, well graded. Maximum size about 25mm. About 85% angular to subrounded, fine to coarse sand, 10% fine to coarse gravel and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.

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TABLE 2Q B-1

SOIL SAMPLE DESCRIPTIONS

BORING E-11

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS/FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION
S-14	58.6-60.1	117	1.4	-	Clean SAND, poorly graded. Maximum size about 20mm. Angular to subrounded, fine to medium sand. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-15	63.6-65.1	93	1.4	-	Clean SAND, poorly graded. Maximum size about 12mm. Angular to subrounded, fine to medium sand. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-16	68.6-70.1	98	1.4	-	Slightly silty SAND, poorly graded. Maximum size about 7mm. About 90% angular to subrounded, fine to medium sand and 10% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-17	73.6-75.1	127	1.7	-	Slightly silty gravelly SAND, well graded. Maximum size about 35mm. About 75% angular to subrounded, fine to coarse sand, 20% fine to coarse gravel and 5% non-plastic fines. Varicolored. Dry. Very dense. Weak reaction to HCl.
S-18	78.6-80.1	95	1.7	-	Clean gravelly SAND, well graded. Maximum size about 30mm. About 65% angular to subrounded, fine to coarse sand and 35% fine to coarse gravel. Varicolored. Dry. Very dense. Weak reaction to HCl.
S-19	83.6-85.1	135	1.4	-	Clean gravelly SAND, well graded. Maximum size about 25mm. About 75% angular to subrounded, fine to coarse sand and 25% fine to coarse gravel. Varicolored. Dry. Very dense. Weak reaction to HCl.
S-20	88.6-90.1	64	1.5	-	Clean sandy GRAVEL, well graded. Maximum size about 35mm. About 60% angular to subrounded, fine to coarse gravel and 40% fine to coarse sand. Varicolored. Dry. Very dense. Weak reaction to HCl.
S-21	93.6-95.1	108	1.7	-	Clean gravelly SAND, well graded. Maximum size about 30mm. About 55% angular to rounded, fine to coarse sand and 45% fine to coarse gravel. Varicolored. Dry. Very dense. Weak reaction to HCl.
S-22	96.8-97.1	86/4"	2.0	-	Slightly silty SAND, well graded. Maximum size about 20mm. About 90% angular to subrounded, fine to coarse sand and 10% non-plastic fines. Varicolored. Dry. Very dense. Weak reaction to HCl. 24

TABLE 2Q B-1

SOIL SAMPLE DESCRIPTIONS

BORING E-12

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS/FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION
S-1	0.0-1.5	5	2.9	-	Silty SAND, poorly graded. Maximum size about 3mm. About 85% angular to subrounded, fine to medium sand and 15% non-plastic fines. Dark yellowish brown. Dry. Loose. No reaction to HCl.
S-2	3.5-5.0	15	-	-	Silty SAND. Maximum size about 2mm. About 60% angular to subrounded, fine to medium sand and 40% non-plastic fines. Varicolored to olive gray. Dry. Medium dense. Weak reaction to HCl. (3.5-4.2)
			5.3	-	Slightly silty SAND, well graded. Maximum size about 15mm. About 90% angular to subrounded, fine to coarse sand and 10% non-plastic fines. Varicolored to olive gray. Dry. Medium dense. Weak reaction to HCl. (4.2-4.7)
S-3	8.5-10.0	31	3.1	-	Clean SAND, poorly graded. Maximum size about 10mm. Angular to subrounded, fine to coarse sand. Varicolored to olive black. Dry. Dense. Weak reaction to HCl. (8.5-9.7)
			6.7	-	Slightly silty SAND, poorly graded. Maximum size about 3mm. About 90% angular to subrounded, fine to medium sand and 10% non-plastic fines. Varicolored to olive gray. Dry. Dense. Weak reaction to HCl. (9.7-9.9)
S-4	13.5-15.0	13	2.4	-	Slightly silty gravelly SAND, poorly graded. Maximum size about 30mm. About 85% angular to subrounded, fine to coarse sand, 10% fine to coarse gravel and 5% non-plastic fines. Varicolored to olive black. Dry. Medium dense. Weak reaction to HCl.
S-5	18.5-20.0	60	2.0	-	Clean gravelly SAND, well graded. Maximum size about 30mm. About 70% angular to subrounded, fine to coarse sand and 30% fine to coarse gravel. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-6	23.5-25.0	140	10.6	-	Silty SAND. Maximum size about 0.5mm. About 80% angular to subrounded, fine to medium sand and 20% non-plastic fines. Olive gray. Dry. Very dense. Weak reaction to HCl.
S-7	28.5-30.0	145	6.0	-	Silty SAND. Maximum size about 1mm. About 85% angular to subrounded, fine to medium sand and 15% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-8	33.5-35.0	83	3.1	-	Slightly silty SAND, poorly graded. Maximum size about 7mm. About 95% angular to subrounded, fine to medium sand and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-9	38.5-40.0	77	3.8	-	Slightly silty SAND, poorly graded. Maximum size about 5mm. About 95% angular to subrounded, fine to medium sand and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-10	43.5-45.0	54	3.9	-	Slightly silty SAND, poorly graded. Maximum size about 4mm. About 95% angular to subrounded, fine to medium sand and 5% non-plastic fines. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.
S-11	48.5-50.0	117	2.4	-	Clean gravelly SAND, well graded. Maximum size about 15mm. About 90% angular to subrounded, fine to coarse sand and 10% fine gravel. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.

TABLE 2Q B-1

SOIL SAMPLE DESCRIPTIONS

BORING E-15

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS/FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION
S-14	63.2-64.7	103	1.4	-	Clean SAND, poorly graded. Maximum size about 10mm. Angular to subrounded, fine to medium sand. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-15	68.2-69.7	99	1.1	-	Clean SAND, poorly graded. Maximum size about 10mm. Angular to subrounded, fine to medium sand. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-16	73.2-74.7	87	1.5	-	Clean SAND, poorly graded. Maximum size about 12mm. Angular to subrounded, fine to medium sand. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-17	78.2-79.7	83	1.5	-	Clean gravelly SAND, poorly graded. Maximum size about 25mm. About 90% angular to subrounded, fine to coarse sand and 10% fine to coarse gravel. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-18	83.2-84.7	169	1.1	-	Clean sandy GRAVEL, well graded. Maximum size about 30mm. About 50% angular to subrounded, fine to coarse gravel and 50% fine to coarse sand. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.

TABLE 2Q B-1

SOIL SAMPLE DESCRIPTIONS

BORING E-16

SAMPLE NUMBER	DEPTH (FEET)	SPT (BLOWS/FT)	NATURAL WATER CONTENT	OTHER TESTS	SAMPLE DESCRIPTION
S-1	0.0-1.5	5	5.1	-	Clean SAND, poorly graded. Maximum size about 3mm. Angular to subrounded, fine sand. Varicolored to dark yellowish brown. Dry. Loose. No reaction to HCl.
S-2	3.5-5.0	5	3.7	-	Clean SAND, poorly graded. Maximum size about 3mm. Angular to subrounded, fine sand. Varicolored to dark yellowish brown. Dry. Loose. No reaction to HCl.
S-3	8.5-10.0	19	3.3	-	Clean SAND, poorly graded. Maximum size about 2mm. Angular to subrounded, fine sand. Varicolored to dark yellowish brown. Dry. Medium dense. No reaction to HCl. (8.5-8.8)
			4.7	-	Silty SAND, poorly graded. Maximum size about 15mm. About 85% angular to subrounded, fine to medium sand and 15% non-plastic fines. Varicolored to olive gray. Dry. Medium dense. Weak reaction to HCl. (8.8-9.6)
			2.0	-	Clean SAND, poorly graded. Maximum size about 10mm. Angular to subrounded, fine to coarse sand. Varicolored to olive black. Dry. Medium dense. Weak reaction to HCl. (9.6-9.8)
S-4	13.5-15.0	13	2.1	-	Clean SAND, poorly graded. Maximum size about 10mm. Angular to subrounded, fine to coarse sand. Varicolored to olive black. Dry. Medium dense. Weak reaction to HCl.
S-5	18.5-20.0	25	3.0	-	Clean SAND, poorly graded. Maximum size about 15mm. Angular to subrounded, fine to coarse sand. Varicolored to olive black. Dry. Medium dense. Weak reaction to HCl.
S-6	23.5-25.0	105	3.4	-	Clean SAND, poorly graded. Maximum size about 10mm. Angular to subrounded, fine to coarse sand. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl. (23.5-23.9)
			7.6	-	Silty SAND, poorly graded. Maximum size about 3mm. About 85% angular to subrounded, fine to medium sand and 15% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl. (23.9-24.7) 24
S-7	28.6-30.1	84	5.2	-	Silty SAND. Maximum size about 2mm. About 70% angular to subrounded, fine sand and 30% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl. 24
S-8	33.6-35.1	118	6.7	-	Silty SAND. Maximum size about 12mm. About 80% angular to subrounded, fine to medium sand and 20% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-9	38.6-40.1	106	-	-	Silty SAND. Maximum size about 1mm. About 80% angular to subrounded, fine to medium sand and 20% non-plastic fines. Varicolored to olive gray. Dry. Very dense. Weak reaction to HCl.
S-10	43.6-45.1	155	3.7	-	Clean SAND, poorly graded. Maximum size about 10mm. Angular to subrounded, fine to medium sand. Varicolored to olive black. Dry. Very dense. Weak reaction to HCl.

TABLE 2Q-1 Sheet 1 of 2

SUMMARY OF TEST BORINGS

Boring Number	Coordinates Northing	Easting	Offset Location	Boring Method	Depth (Feet)	Ground Elevation
W-1	422698+92	2268389+43		1	0 - 100	532.3
				2	100 - 137	
				3	137 - 253	
W-2	422800	2268260		1	0 - 105	530.7
W-3	422870	2268500		1	0 - 90	527.7
W-4	422680	2268780		1	0 - 105	527.2
				2	105 - 137	
W-5	422300	2268220		1	0 - 114.5	527.6
W-6	422340	2268660		1	0 - 90	525.7
				2	90 - 131.5	
W-7	422700	2268260		1	0 - 84.7	
W-8	422900	2268340		1	0 - 100.6	531.3
W-9	422800	2268400		1	0 - 104.7	532.7
				3	104.7 - 131.7	
W-9A			30' NW of W-9	4	0 - 75	532 (approx.)
W-10	422620	2268400		1	0 - 40	530.6
W-10A			4' W of W-10	1	40 - 89.7	530 (approx.)
W-10B			8' E of W-10	4	0 - 85	530 (approx.)
W-11	422740	2268500		1	0 - 100	529.8
W-11A			15' SW of W-11	4	0 - 33	530 (approx.)
W-11B			6' S of W-11A	4	33 - 55	530 (approx.)
W-11C			5' S of W-11B	4	55 - 75	530 (approx.)
W-12	422620	2268500		1	0 - 97	529.5
W-13	422780	2268610		1	0 - 100	528.4
W-14	422680	2268610		1	0 - 103	526.6
W-15	422780	2268780		1	0 - 35	528.0
W-15A			4' W of W-15		35 - 85	528 (approx.)
W-15B			8' W of W-15		85 - 104.3	528 (approx.)
W-16	422440	2268220		1	0 - 99.7	525.4
W-17	422300	2268340		1	0 - 104.5	529.0
W-18	422460	2268660		1	0 - 99.4	527.4
W-19	422540	2268260		1	0 - 92.4	528.7
				3	92.4 - 146	
E-1	422717+72	2269289+23		1	0 - 95	516.6
				2	95 - 130	
				3	130 - 459.6	
E-2	422870	2269160		1	0 - 20.1	524.1
E-2A			4' N of E-2	1	23.8 - 40.1	524 (approx.)
E-2B			7' S of E-2	1	43.6 - 89.9	524 (approx.)
E-3	422870	2269400		1	0 - 100.2	521.4
E-4	422680	2269680		1	0 - 85	516.9
				2	85 - 126.5	517 (approx.)
E-5	422300	2269120		1	0 - 40	520.1
E-5A			10' E of E-5	1	40 - 94.7	520 (approx.)
E-6	422340	2269560		1	0 - 95	519.1
				2	95 - 126.6	
E-7	422700	2269160		4	0 - 76	520.0
E-7A			5' S of E-7	1	0 - 90	520 (approx.)
E-8	422900	2269240		1	0 - 100.1	521.5
E-9	422800	2269300		1	0 - 78	520.6
E-9A				4	0 - 26	520 (approx.)
E-9B			17' N of E-9	4	26 - 65.6'	520 (approx.)
			20' N of E-9			
E-10	422620	2269300		1	0 - 94.6	517.2
E-11	422740	2269400		1	0 - 97.1	518.4
E-12	422620	2269400		4	0 - 60	516.6
E-12A				4	60 - 70	516 (approx.)
E-12B			3' S of E-12	1	0 - 90	516 (approx.)
E-13	422780	2269510	5' W of E-12	1	0 - 89	517.3
E-14	422680	2269510		1	0 - 25	516.4
E-14A				1	25 - 85	516 (approx.)
E-15	422780	2269680	4' NW of E-14	1	0 - 87	517.5
E-16	422440	2269120		1	0 - 100.2	519.8
E-17	422300	2269240		1	0 - 92	516.6
E-18	422460	2269560		1	0 - 97	516.5

TABLE 20-1

Boring Number	Coordinates Northing	Easting	Offset Location	Boring Method	Depth (Feet)	Ground Elevation
16	422645	268305		2	0 - 240	530.30
17	422645	268265		2	0 - 240	529.6
18	442646	268416		2	0 - 240	531.3
19	422645	268475		2	0 - 240	530.8
20	422648	268545		2	0 - 240	529.1
21A	422720	268278		2	0 - 190	530.4
22	422734	268323		2	0 - 190	532.0
23	422809	268340		2	0 - 190	532.0
24	422854	268415		2	0 - 190	532.4
25	422861	268450		2	0 - 179.5	531.8

1 = Hollow Stem Auger
2 = Top Drive Rotary
3 = Diamond Coring
4 = Mud Rotary

TABLE 2Q-2

SUMMARY OF DUTCH CONE SOUNDINGS

Boring Number	Location	Ground Elevation
E-1B	15' SW of E-1	518 (approx.)
E-2C	5' E of E-2	524 (approx.)
E-7B	5' E of E-7	520 (approx.)
E-14B	5' E of E-14	516 (approx.)
E-15A	5' E of E-15	517 (approx.)
W-1A	5' E of W-1	532 (approx.)
W-2A	5' E of W-2	530 (approx.)
W-14A	5' E of W-14	528 (approx.)
W-15C	5' E of W-15	528 (approx.)
W-19A	5' E of W-19	528 (approx.)

TABLE 2Q-3

BASIC INDEX PROPERTIES OF SITE FOUNDATION SOILS

Soil Property	Soil Unit: Medium Dense to Dense Sand (>El. 490)	Very Dense Sand (El. 490 -- El. 445)	Very Dense Gravels (El. 445 -- El. 360)	Materials Below Elevation 360: Gravels	Sands	Silts
Average Wet Unit Weight (pounds per cubic foot)	110	105 (Estimated)	Above water table: 130 (Estimated) Below water table: 140 (Estimated)	150	125	125
Average Natural Water Content (percent)	3	3	Above water table: 3 Below water table: 12 (Estimated)	7	22	25
Atterberg Limits (percent)	Non-Plastic	Non-Plastic	Non-Plastic	Non-Plastic	Non-Plastic	L.L. = 50 P.L. = 45
Poisson's Ratio	0.3 (Estimated)	0.3 (Estimated)	0.3 (Estimated)	0.3 (Est.)	0.3 (Est.)	0.4 (Est.)
Average Relative* Density (percent)	75	85	---	---	---	---

*Estimated from penetration testing.

TABLE 2Q-4

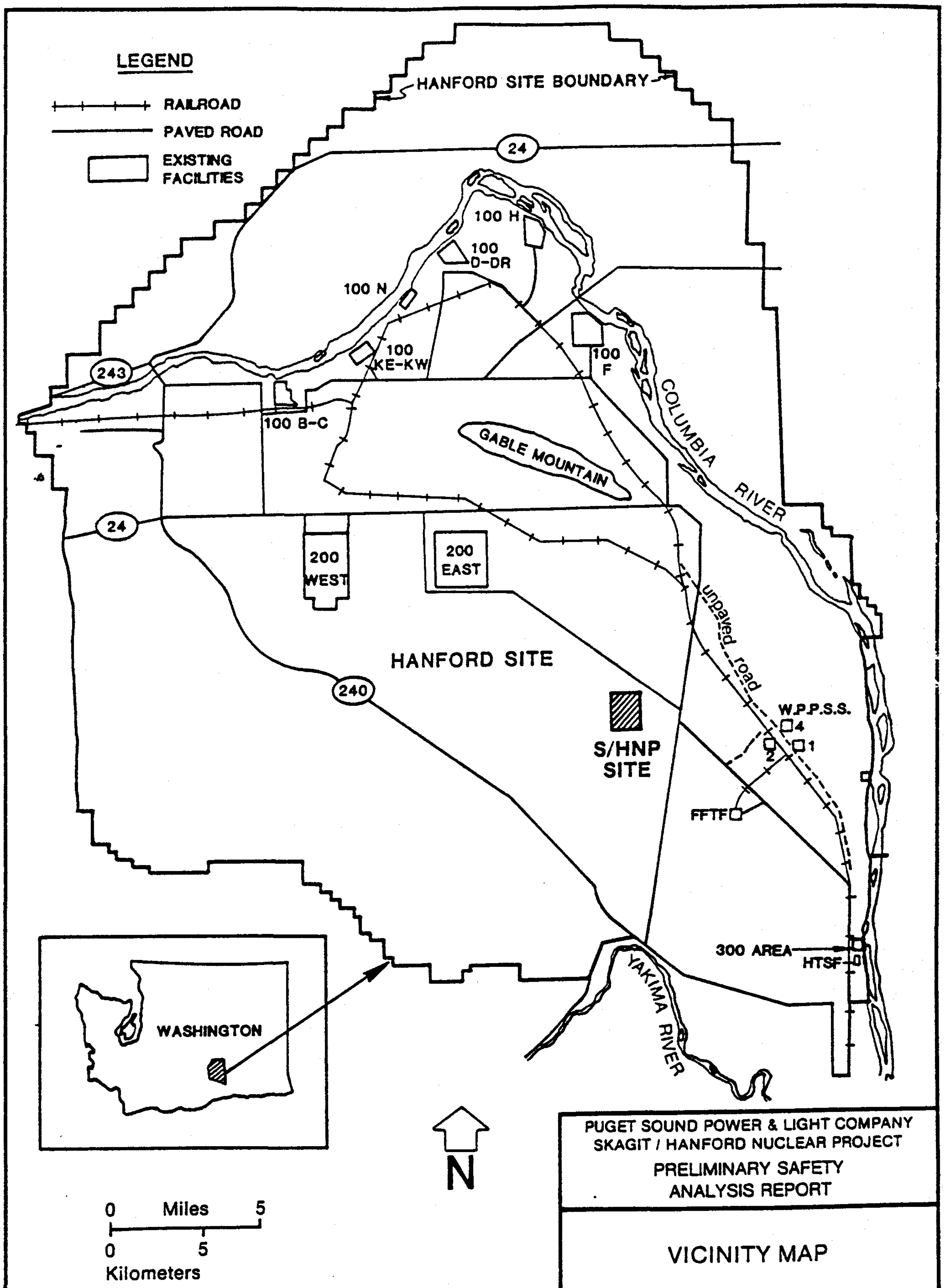
AUXILIARY/FUEL/CONTROL/REACTOR
BASEMAT MOVEMENTS DURING 0.35 g EARTHQUAKE

Mode	Maximum Movement
Horizontal Translation	1.71 inches
Rotation	0.00059 radians
Vertical Translation	0.61 inches

TABLE 2Q-5

EFFECTIVE SHEAR STRAIN LEVELS DURING
0.35 g EARTHQUAKE

Depth (ft)	Elevation (ft)	Free Field Effective Shear Strain (%)	Effective Shear Strain Due to Surface Structure (%)
0-35	525-490	0.04	0.22
35-80	490-445	0.02	0.03
80-165	445-360	0.02	----



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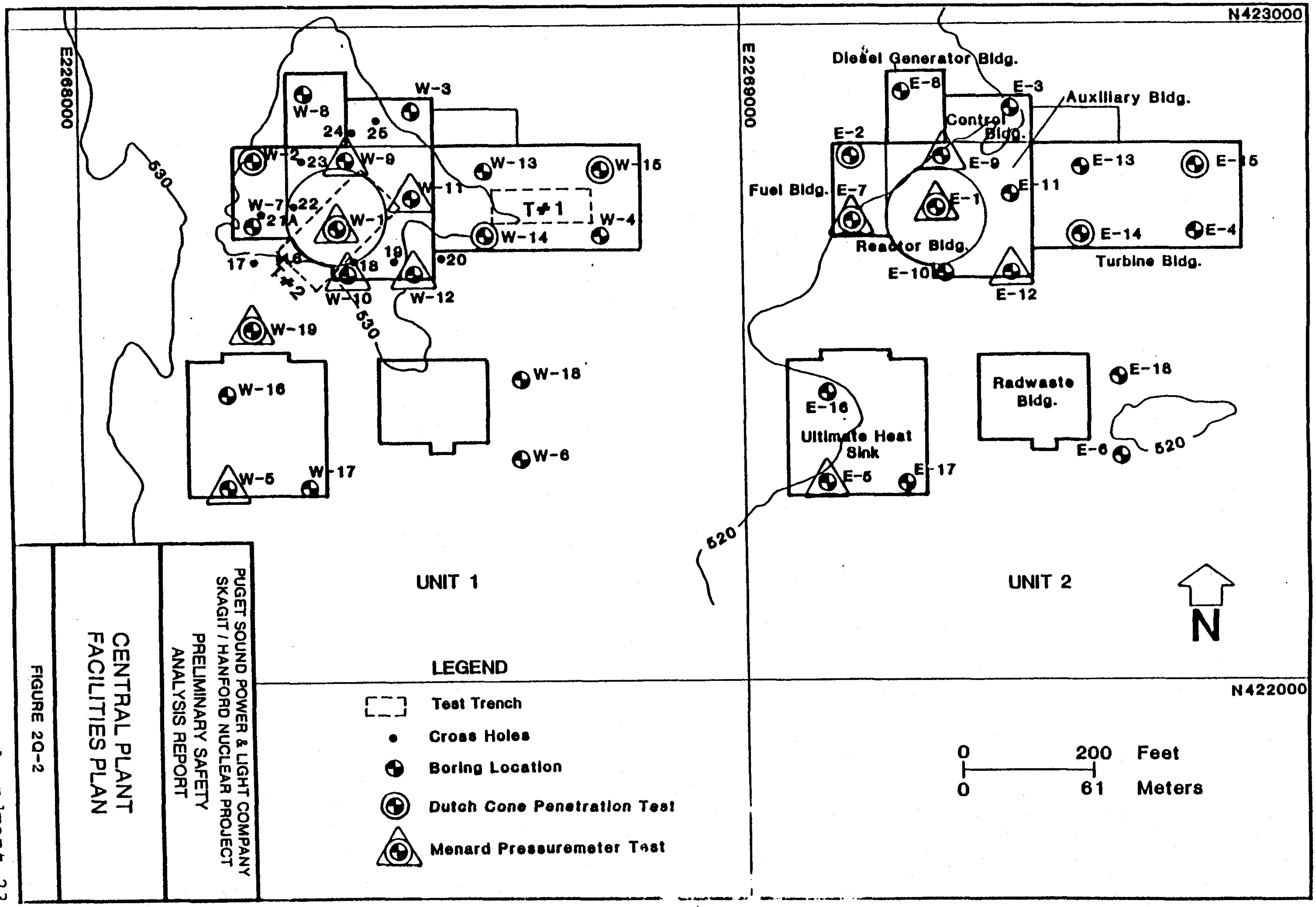
VICINITY MAP

FIGURE 2Q-1



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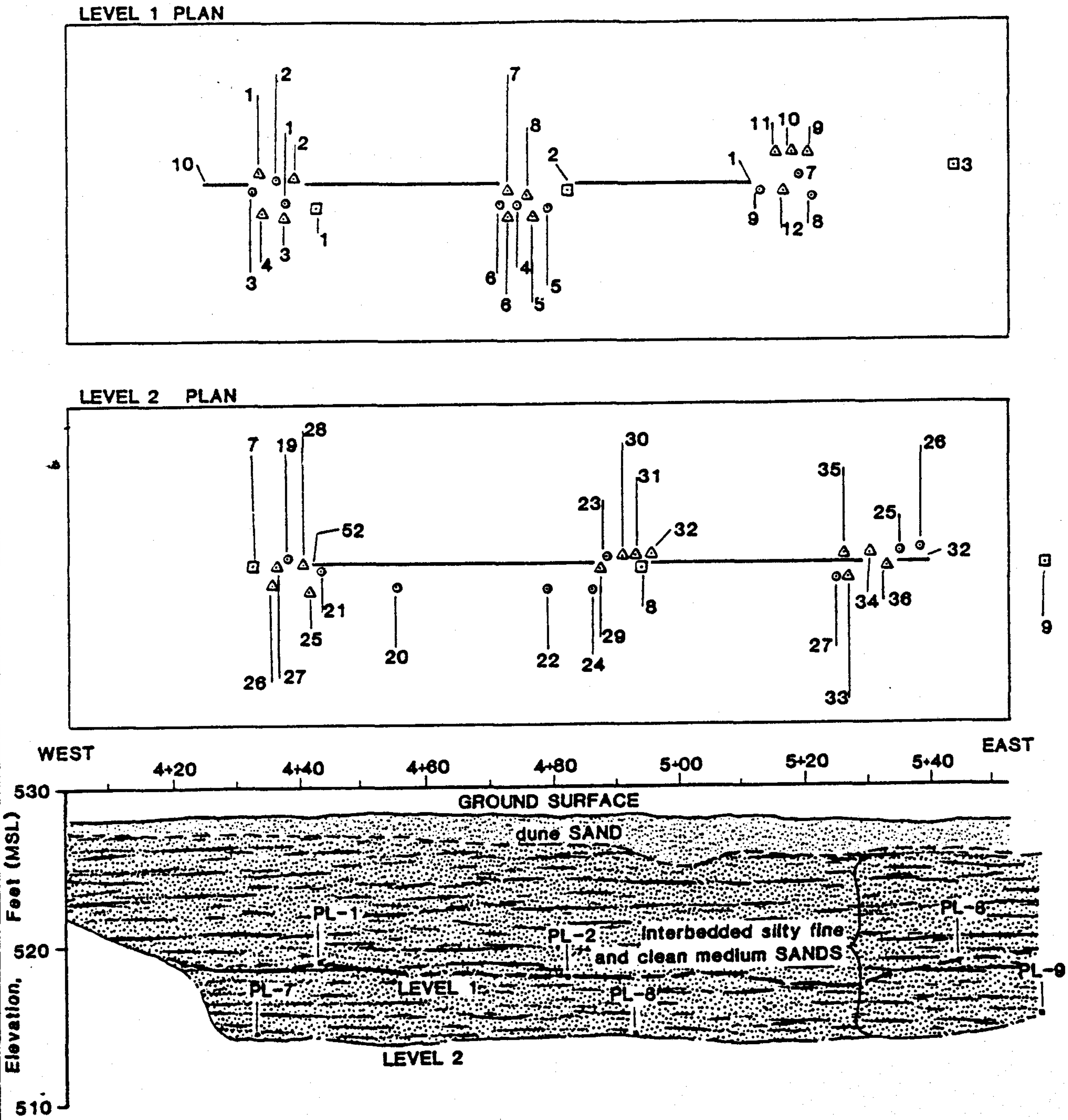
ANALYSIS REPORT

CENTRAL PLANT
FACILITIES PLAN

FIGURE 2Q-2

Amendment 23



SC ⊙ SAND CONE TEST
UD △ UNDISTURBED SAMPLE
PL ⊡ PLATE LOAD TEST
—— NEUTRON DENSITY TESTS

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TRENCH #1
LEVELS 1 AND 2
PLANS AND PROFILE

FIGURE 2Q-3

BATCH
START

STAPLE
OR
DIVIDER

ATTACHMENT E

ENGINEERING PROPERTIES OF SOILS AT LIGO SITE COMPARED TO THE HWVP SITE

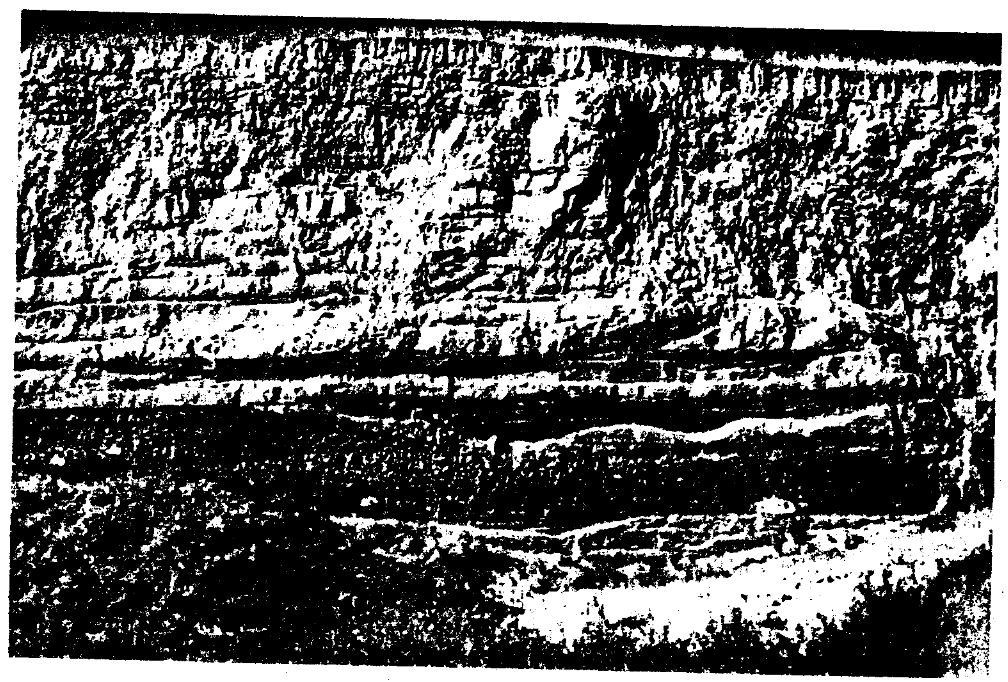
Previous information on geotechnical properties of the soils at the LIGO site was estimated from a study of the HWVP site to the north. More relevant information from the nearby FFTF and Skagit/Hanford sites is enclosed. The HWVP site reports information from the surface dune field on the southern part of the site. This is reported as the "unconsolidated surface sands". The LIGO site is different in that the surface sands are more consolidated. This was evident in the fact that we were able to drive over the LIGO area without and trouble, something that is impossible at the dune field at HWVP. In LIGO Photo 1 several pit walls are shown from the Hanford Site. Photos "b" and "c" are similar to that expected at the LIGO site. Note that the pit walls maintain a steep angle indicating a reasonably consolidated sand. Only where weathering processes have affected the top several feet of surface (as in "c") is there "unconsolidated sand"; yet this is sufficiently consolidated enough to allow a jeep to drive across without any difficulty.

LIGO - Photo 1

Sand

a

b



LiGO Photo 2

LIGO Photo 3 Soil Moisture content with depth