APPENDIX 2.5E

FOUNDATION INVESTIGATION, WASHINGTON PUBLIC POWER SUPPLY SYSTEM, HANFORD NO. 2 NUCLEAR PLANT, CENTRAL PLANT FACILITIES BENTON COUNTY, WASHINGTON, DATED MAY 28, 1971.

FOUNDATION INVESTIGATION

WASHINGTON PUBLIC POWER SUPPLY SYSTEM

HANFORD NO. 2 NUCLEAR POWER PLANT

CENTRAL PLANT FACILITIES

BENTON COUNTY, WASHINGTON

FOR

BURNS and ROE, INC.
ENGINEERS AND CONSTRUCTORS

MAY 28, 1971

Revised as noted 8/5/71

SHANNON & WILSON, INC.
SOIL MECHANICS AND FOUNDATION ENGINEERS
SEATTLE, WASHINGTON

SUMMARY

This report summarizes the results of a comprehensive foundation investigation at the site of the proposed Hanford No. 2 Nuclear Power Plant near Richland, Washington. In general, the recommendations and conclusions presented are based on detailed engineering evaluation of subsurface soils data obtained from eleven borings and two test pits accomplished during the field exploration phase of these studies. It should be noted that this is not a complete site evaluation in that certain aspects (i.e., geology, geophysical exploration, site seismicity evaluation, groundwater hydrology, etc.) are being accomplished and reported on by others.

The field explorations indicate that the Hanford No. 2 site is underlain by dense granular soils to at least a depth of 250 feet, which corresponds to the depth of the deepest boring. The entire site is mantled with a 2 to 3-foot layer of fine, eolian SAND. This thin blanket is immediately underlain by about a 100-foot thick deposit of fine to coarse SAND, which varies in consistency from slightly gravelly to that of a sand and gravel mixture. In the upper 40 feet, these sands increase in density with depth from medium dense to very dense. Below a depth of about 40 feet, all soils were found to be very dense, as indicated by penetration test values which were with a single exception greater than 100 blows per foot. Below an average depth of 107 feet, the borings encountered the extremely dense Ringold conglomerates (sand-gravel mixtures), which are underlain at about 217 feet by the lower unit of the Ringold Formation consisting primarily of very dense or hard, interbedded sand, silt, clay and gravel. The water table was measured to be at an average depth of 62 feet and the estimated top of basalt bedrock is approximately 420 feet below the ground surface as determined by geophysical methods.

Based on our field inspection and evaluation of samples, on data from our laboratory test program and preliminary plant

design information furnished by Burns and Roe, Inc., we have conducted various engineering studies to determine foundation design criteria. From our studies, we conclude that for the dense granular soils and the three buildings with mat foundation configurations described herein, the ultimate soil bearing capacity is in excess of 50 tons per square foot. Thus, the allowable bearing pressure for these structures will be governed by the differential settlements that may be tolerated. Based on the present design loads, the proposed depth of excavation, and average elastic moduli (E) of 25,000 psi, 60,000 psi and 90,000 psi for the major soil zones, the maximum elastic settlement that will occur during construction is computed to be slightly more than 2 inches, while the maximum differential settlement should be in the order of 1 inch. Maximum post-construction differential settlements should not exceed about 0.25 inch. The strip footing foundations for the diesel generator building and the truck lock, and the spread footings for the service building will need to be designed for somewhat lower bearing pressures than the broader mat foundations.

Because of the dense nature of the foundation soils, there is no possibility for the occurrence of soil liquefaction beneath the major structures. Due to the possibility of a future groundwater rise, the reactor building will require waterproofing to resist hydrostatic pressures or the use of a dewatering and drainage system. No additional settlements are anticipated as a result of changes in groundwater level.

Criteria have been developed and presented regarding the placement and compaction of structural fills as well as the lateral pressures such fills, backfills and adjacent foundations will exert on subsurface, exterior walls. In general, we recommend that these walls be designed for coefficients of earth pressure at rest which vary from K_o = 0.5 to 0.8. Where applicable, active and passive pressure coefficients may be used as given.

We have concluded that the soil conditions at the proposed site are suitable for the design and construction of the central facilities presently planned. However, it is recognized that

additional explorations will be required for some of the major central plant structures, other major plant structures (such as cooling towers) and various minor structures associated with the plant. In general, such explorations would be confirmatory in nature to verify that the anticipated conditions at other site locations are consistent with the conclusions reported herein.

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HANFORD NO. 2 NUCLEAR POWER PLANT
CENTRAL PLANT FACILITIES
BENTON COUNTY, WASHINGTON

I. INTRODUCTION

1.1 PURPOSE AND SCOPE

This report presents the results of our foundation investigation for the central plant facilities of the proposed Hanford No. 2 Nuclear Power Plant. The purpose of this investigation was to determine the subsurface conditions at the proposed site and to make detailed recommendations for design and construction of the foundations for the major buildings within this area. Emphasis in this study was placed on establishing foundation requirements imposed by normal, static load conditions as well as determining the static and dynamic engineering soil properties. It is our understanding that the seismic design criteria and response analyses will be evaluated by others. Included in the field explorations at the proposed central plant site were eleven subsurface borings and two test pits.

This report contains a description of the general site and proposed facilities, geologic setting, boring explorations, laboratory testing, interpretation of subsurface conditions, and our detailed recommendations and conclusions. The engineering studies and conclusions based on these data include recommendations for foundation types of the various buildings, bearing pressures, estimated settlements, together with recommended excavation slopes, waterproofing and drainage, backfill and compaction requirements, and earth pressures for design of permanent subsurface exterior walls. In general, we have concluded that the proposed Hanford No. 2 site is suitable with respect to

soil conditions for plant construction and operation.

To further supplement the main text of this report, a list of references and five appendices are included. Appendix A presents a discussion of the various field exploration techniques used to obtain subsurface data during this investigation. Because some aspects of the penetration tests deviated slightly from that designated as "standard", an evaluation of penetration test procedures was prepared and is presented in Appendix B. A detailed summary of laboratory test methods and results is presented in Appendix C. Appendix D is a bibliography of various reports, papers and books, which include information on the Hanford area with respect to the following general subjects: geology, seismology, groundwater, foundation design and general information. Appendix E presents preliminary data obtained from drilling one out of the three planned borings at the site of the proposed pump station along the Columbia River. It is presently anticipated that the foundation studies for this structure will be accomplished once the river level recedes to a point where the other two boring locations are accessible to truck-mounted drilling equipment.

Following completion of our report on May 28, 1971, subsequent changes in building grades and dimensions, and the general re-orientation and positioning of the various facilities, have required that revisions be made in the engineering studies and conclusions contained in the original report. These changes have been incorporated in the following pages of the original text. Where revisions on a specific page were made, that page is appropriately identified and dated in the top right hand corner as noted above.

1.2 AUTHORIZATION

This investigation was authorized verbally on February 24, 1971, by Mr. A. M. Capobianco, General Purchasing Agent for Burns and Roe, Inc. and subsequently verified by Purchase Order No. BR-2808-3 (also dated February 24, 1971). The scope of work as originally detailed in our proposal of January 26, 1971, was revised to exclude both the administration of drilling subcontracts and

the conduct of geophysical explorations. Subcontracting of these latter two items was administered directly by Burns and Roe, Inc. The investigation was subsequently expanded to include a preliminary study for the proposed pumping station to be located on the Columbia River, and the drilling of two extra borings at the plant site for use in the geophysical explorations. All work in conjunction with this investigation was generally accomplished as described in Burns and Roe Purchase Requisition No. 2808-3 dated February 18, 1971.

The revised study contained herein was authorized verbally by Mr. Gus Satir of Burns and Roe, Inc. on July 21, 1971, and confirmed by change order dated July 26, 1971.

1.3 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions as they presently exist and further assume that the exploratory borings and test pits are representative of the subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the investigation. Though the foundation soil conditions underlying this site are considered to be very good, certain additional explorations will be necessary at various facilities to confirm that the soil conditions at those locations are consistent with the conclusions contained herein.

1.4 CONTRIBUTORS

For Shannon & Wilson, Inc., the field inspection of drilling was accomplished primarily by Mr. Rohn Abbott and Mr. Fred Nygren. Mr. Fred Brown and Mr. E. D. Schwantes jointly compiled the data, supervised laboratory testing, performed most of the engineering studies and largely prepared the main text and subsequent revisions of the report. The overall project was under the direction of Dr. R. P. Miller, who supervised all phases of the project.

II. GENERAL INFORMATION

2.1 SITE AND REGIONAL DESCRIPTION

a. Site Description

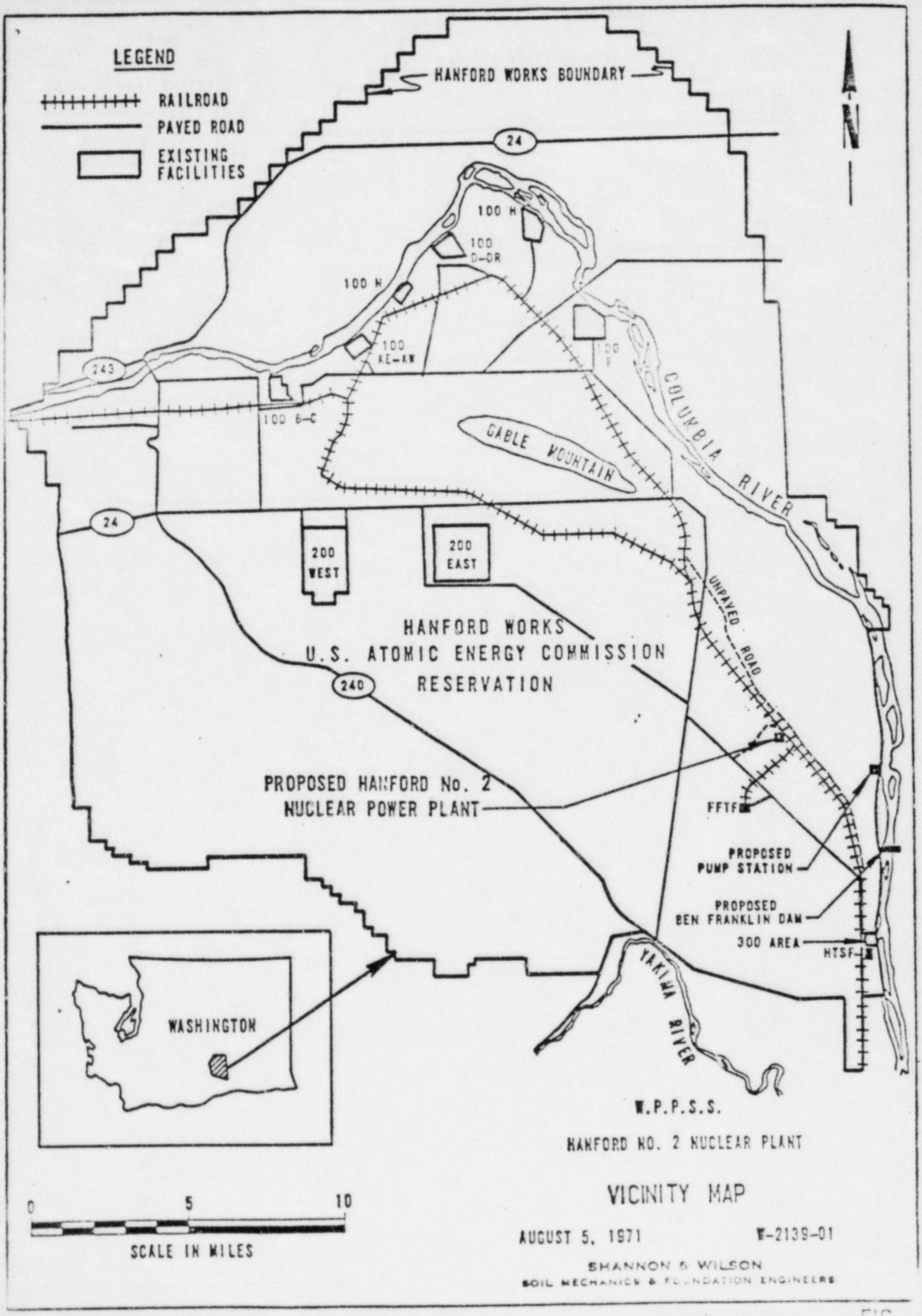
The proposed Hanford No. 2 Nuclear Power Plant site is located in the 575 square mile U. S. Atomic Energy Commission Hanford Reservation, in the south central part of Washington. The main plant area lies in Section 5 (T 11 N, R 28 E), approximately 3-1/2 miles west of the Columbia River and 14 miles north of the city of Richland, Washington. The Vicinity Map, Fig. 1, shows the location of both the proposed plant site and pump station. Fig. 2 presents a more detailed site plan of the main plant, and also identifies the other major plant facilities and their tentative positions.

The central plant location, approximately 400 feet by 400 feet in dimension, is situated near the middle of a relatively flat, essentially featureless plain extending in all directions for several miles. Maximum relief across the central plant site, as shown in Fig. 2, is approximately 10 feet or less. In general, the terrain slopes slightly to the northeast toward the Columbia River. The average ground surface elevation at the site is about 440 feet (MSL Datum).

Access to the plant area is presently provided by gravel and dirt roads which connect with a major four-lane roadway, which cuts across the Reservation in a NW-SE direction as shown in the Vicinity Map (Fig. 1). The flat terrain is such that most vehicles can easily drive onto and around the site. The only recognizable, nearby landmarks visible from the site are: the AEC railroad tracks running in a NW and SE direction about 700 feet east of the site, the telephone lines running in the same general direction about 800 feet west of the site, and the WYE burial ground located approximately 1800 feet northwest of the site.

Vegetation over the plant area is minimal and consists of young plants found typically in arid regions. Larger plants such as sagebrush once covered this entire area; however, a

Revised 8/5/71



W.P.P.S.

HANFORD NO. 2 NUCLEAR PLANT

VICINITY MAP

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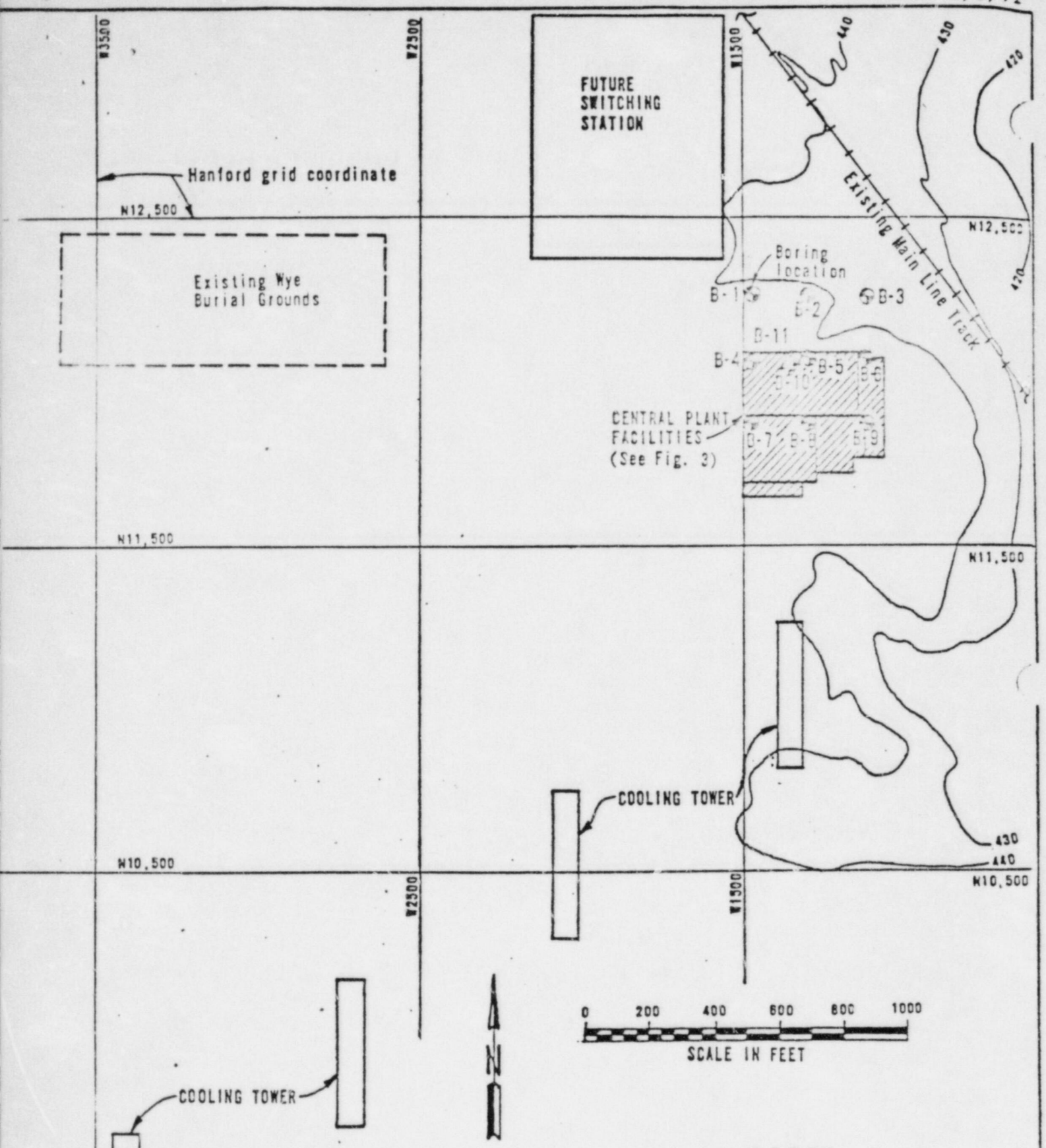
FIG.



W.P.P.S.S.
HANFORD NO. 2 NUCLEAR PLANT

PLAN OF
PROPOSED NUCLEAR PLANT

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NOTE

THE PLANT LAYOUT WAS PROVIDED BY BURNS & ROE, INC. THIS LAYOUT IS TENTATIVE AND SUBJECT TO REVISION.

FIG. 2

1970 brush fire destroyed most of this growth. Beneath the sparse vegetation, the ground surface is mantled with a light brown to tan, dry, fine, eolian sand. These sands were transported and deposited over the site by strong winds which, during the field explorations, generally prevailed from a southwesterly or northwesterly direction. Since the fire, the absence of vegetation has allowed the winds to reach, erode, and transport more readily these unprotected deposits. Severe dust or sand storms are not uncommon.

b. Regional Description

The regional topography is that of a broad, relatively flat basin (Pasco Basin) with the plant site being located in one of the lower areas. The maximum variation in elevation within a 7-mile radius of the site is about 200 feet on the north, south and west sides (between elevations 400 and 600 feet). However, 3-1/2 miles to the east of the site, steep, river-cut bluffs rise to form the east bank of the Columbia River. Beyond the river for at least 10 miles, the flat basin continues; however, it is terraced and elevated approximately 300 to 500 feet above the plant elevation. On the west and southwest horizons, 8 to 10 miles away, the basin slopes upward forming the edge of the Rattlesnake Hills. The peaks of the Rattlesnake Hills rise a maximum of 3200 feet above the plant site. To the north about 8 miles, Gable Mountain rises above the floor of the basin to an elevation some 670 feet above the plant site.

The Columbia River has cut a channel through the center of the basin, about 3-1/2 miles east of the plant site. The river in this vicinity is over a half mile wide, however, some of this width consists of small sand bar islands. Based on published literature, the average flow past this location is on the order of 120,000 cfs with a maximum and minimum range of about 695,000 and 40,000 cfs, respectively. Most of this flow results from runoff of snow melt in areas far to the north, more than half coming from Canada.

The precipitation in the Hanford area is very low, with a mean annual rainfall of approximately 7 inches. The

temperatures are moderate, with a mean of 54 degrees and an average range between 24 and 92 degrees F. Low precipitation and moderate temperatures contribute to the semi-arid desert environment within the Reservation, which allows only desert vegetation to survive. In areas where vegetation is minimal, sand dunes develop and migrate locally with the prevailing winds.

South of the project site are the tri-cities of Richland, Pasco and Kennewick, which constitute the major center of population in the region. These cities are more or less dependent on the Hanford operation as their major industry. However, there is some farming and cattle raising, especially on the east side of the Columbia.

2.2 GEOLOGY

The geology of the Hanford area is the subject of a detailed report for Douglas United Nuclear Inc. by Fred O. Jones and Robert J. Deacon, titled "Geology and Tectonic History of the Hanford Area", June 15, 1966. The following are excerpts from that report to provide the general geologic setting for the subsurface conditions existing at the plant site.

> "The surficial geology of the Hanford area is probably better known than any other area of Washington. R. E. Brown, Battelle Institute geologist, has written extensively about the geology and hydrology of the basin. Others who have written important papers include R. C. Newcomb, U.S.G.S., Donald J. Brown, M. W. McConiga, and a host of other scientists who have been associated with research projects in the Hanford area. The geology of the basin has been compiled using information from over 700 wells that total more than 150,000 feet of exploratory drilling.
>
> The Pasco Basin is one of the principal physiographic and structural features of the Columbia Plateau in south central Washington. The basin is a moderately deformed late Tertiary-early Quaternary structural feature which is bounded on the north by Saddle Mountain Anticline, on the south by Rattlesnake Hills and Horse Heaven Hills Anticline, on the

west by east-plunging Yakima Ridge and Umtanum Ridge Anticlines, and on the east by Jackass Mountain Monocline. The Hanford Reservation comprises the southwestern part of the Pasco Basin. The basin is separated into the Wahluke Syncline, the Cold Creek Syncline, and the Pasco Syncline. It reaches its deepest point 200 feet below sea level in the Wahluke Syncline.

The rocks consist of five major stratigraphic units. In ascending order these are the Yakima Basalt Formation with interbeds of the Ellensburg Formation in its upper part, the Ringold Formation, Palouse Soil, and unconsolidated glacio-fluvial sands and gravels. Touchet beds are a phase of the glacio-fluvial sediments. A major unconformity is present between the Ringold and glacio-fluvial sediments.

a. Yakima Basalt Formation

Basaltic lavas of the Yakima Basalt Formation form the bedrock of the Pasco Basin and are part of a regional sequence of the Columbia River Group of lavas which cover about 200,000 square miles in Washington, Oregon, and Idaho. The basalts are exposed in the nearby anticlinal ridges on the flanks of the basin and in one anticline ridge, Gable Butte - Gable Mountain, near the center of the basin. The basalt sequence consists of dense, relatively unweathered, conformably layered flows, with occasional interbeds of flow breccias, pyroclastics, and sedimentary rocks. The lava sequence is believed to be in excess of 10,000 feet thick (Standard Oil Co., Rattlesnake Hills Oil and Gas Test, T.D. 10,655).

b. Ringold Formation

Lacustrine and fluvial sediments of the Ringold Formation conformably overlie the lava sequence in the basin and are exposed in the "White Bluffs" along the Columbia River near the eastern edge of the Reservation. The Ringold Formation consists of semi-consolidated and locally cemented sand, semi-coherent gravel or conglomerate with very stiff to hard (indurated) silt and clay.

The Ringold Formation locally contains fossilized bones of many types of vertebrate animals and is considered to be early to middle Pleistocene in age. The formation is highly variable both vertically and laterally but generally can be divided into three members described as follows:

Upper Member. This member consists largely of light buff to cream and tannish brown, sandy silt and fine sand. It is exposed in the bluffs along the left bank of the Columbia River.

Middle Member. This member consists of brown and gray, locally consolidated to semi-consolidated silt, sand and gravel and is referred to as the "conglomerate" zone.

Lower Member. This member is characteristically a blue green, clayey silt and is known as the "blue clay" zone. Locally, however, the zone is composed largely of sand and cobble gravel with subordinate amounts of silt beds.

The base of the formation has been moderately deformed but the middle and upper portions are only slightly deformed. The upper surface of the formation beneath the Reservation has been deeply channeled. The aggregate thickness of the Ringold Formation is approximately 1,200 feet, but because of erosion only a maximum of about 600 feet remain within the Pasco Basin.

c. Glacio-Fluvial Sediments

Coarse grained deposits that lie above a channeled surface of the Ringold Formation are referred to as Glacio-Fluvial Sediments. Part of the sediments were deposited by the ancestral Columbia River and part by glacial melt water. The glacio-fluvial deposits cover most of the surface of the Pasco Basin and consist of fine gravel, sand and pebble gravel, and cobble and boulder gravel with occasional interbeds of fine sand and silt. The sediments locally contain highly erratic torrential cross-bedding, slump structures, and clastic dikes. The glacio-fluvial

sequence is considered to be late Pleistocene to early Recent. Radiocarbon dating of Touchet Beds (a fine grained phase) indicates that the sequence is about 13,000 years old.

d. Palouse Soil

Palouse soil locally overlies the Ringold Formation in the Pasco Basin and consists of fine sand and silt with a maximum thickness of about 60 feet. The unit is believed to be of eolian origin and is locally called "Palouse Soil" because of its similarity to the Palouse soil of eastern Washington and western Idaho.

e. Touchet Beds

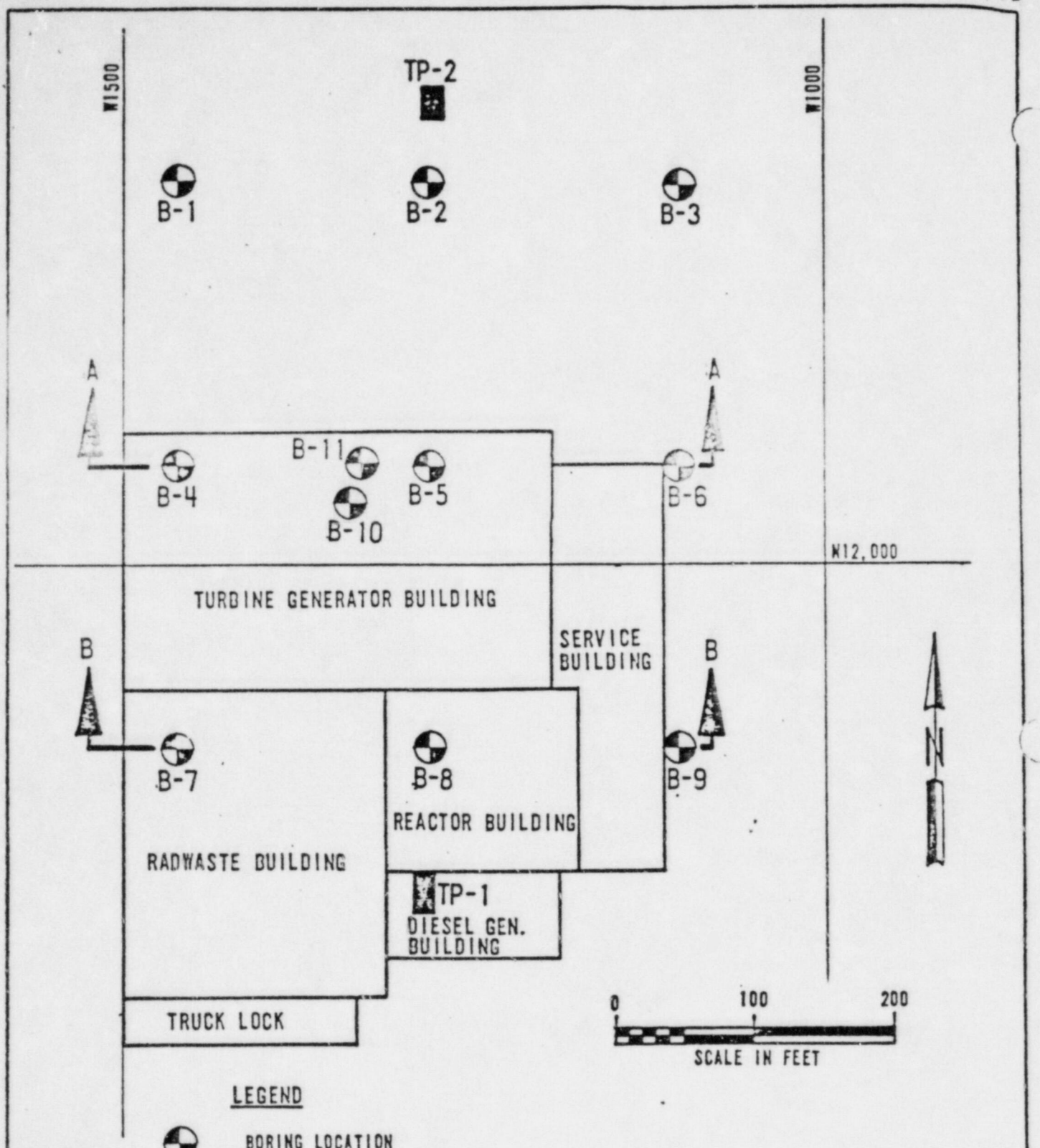
Touchet Beds are fine grained, lacustrine deposits laid down on the bottom of pro-glacial Lake Lewis. They are equivalent to the upper portion of the glacio-fluvial sequence but are lake deposits rather than fluvial deposits. The Touchet Beds lie between altitudes of about 400 and 1,150 feet. They are present at the lower altitudes in the southwest and northern part of the Pasco Basin."

The Touchet Beds apparently are not present at either the plant or pump station sites.

For more detailed information on the geology of the Hanford area, a bibliography of references pertinent to this subject is contained in Appendix D-1.

2.3 DESCRIPTION OF PROPOSED PLANT STRUCTURES

Within the central plant area, preliminary plans show six major structures: reactor building, turbine generator building, radwaste building, service building, diesel generator building and truck lock. The positioning of these central plant facilities, as furnished to us by Burns and Roe, Inc., is shown in Fig. 3. It is our understanding that this layout is tentative and subject to revision, as more detailed design information is developed. The following describes each structure based on presently available information:



LEGEND

BORING LOCATION

TEST PIT LOCATION

W1500 HANFORD GRID COORDINATE

B SUBSURFACE PROFILE (See Fig. 4)

NOTE

BUILDING OUTLINES WERE PROVIDED BY BURNS & ROE, INC. THIS LAYOUT IS TENTATIVE AND SUBJECT TO REVISION.

W.P.P.S.S.
HANFORD NO. 2 NUCLEAR PLANT

PRELIMINARY CENTRAL PLANT LAYOUT

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FIG. 3

a. Reactor Building

The Reactor Building is to be 130 by 140 feet in plan and extend approximately 31 feet below present grade (approximate bearing elevation 409 feet). The total weight of the structure including equipment (dead load plus live load), is roughly estimated to be 82,000 tons. This load will produce a unit pressure at the foundation level of approximately 9,000 pounds per square foot (psf), over the projected plan area of this structure.

b. Turbine Generator Building

This 304-foot by 184-foot building is the largest of the plant facilities in floor area. The turbine, generator and subsidiary equipment along with the dead load of the structure are expected to apply an average unit pressure of about 4,000 psf over the entire building area. The foundation grade is presently designed to be about 7.5 feet below the ground surface (elevation 432.5 feet).

c. Radwaste Building

This structure is the second largest building in the central plant complex, with rectangular plan dimensions of about 218 by 184 feet. The total foundation load of this structure is expected to be about 110,000 tons. This load will produce an average unit pressure of 5,500 psf over the projected plan area of the building. The foundation grade is to be at elevation 429, which is about 11 feet below the existing ground surface.

d. Service Building

This structure is to be located in the eastern part of the central plant area. As designed it will cover a floor area of some 20,000 square feet, as shown in Fig. 3, and will be supported by isolated column footings spaced approximately 20 foot on centers. Unit pressures on these footings are estimated to be in the order of about 6,000 psf. This building, as planned, will extend about 3.5 feet below the present ground surface to bear at elevation 436.5 feet.

e. Diesel Generator Building

The Diesel Generator Building is one of the smaller of the major structures in the central plant area. This building will

be designed to transmit loads (dead and live) of about 5,000 pounds per lineal foot onto continuous wall footings running around the perimeter of the building. The building itself is 60 by 124 feet in plan. Since considerable excavation will be required for the adjacent Reactor Building, it is very likely that the Diesel Generator Building will be founded entirely in compacted, structural backfill after the subsurface work for the Reactor is complete.

f. Truck Lock

This facility will be located adjacent to the radwaste building on the south side. It will be some 32 by 166 feet in plan and will be founded very near the existing ground surface. The foundation, as planned, will consist of a continuous wall footing around the perimeter that will impose an average unit pressure of 5,000 psf on the supporting soil. The footings will be loaded about 3.5 feet below the ground surface at approximately elevation 436.5.

Other major plant structures (shown in Fig. 2) and a number of minor structures, presumably one or two story buildings with relatively light loads, are also anticipated. The actual location, size and description of these buildings, as well as roads, railroads, embankments or other structures within the overall plant complex have only been tentatively established. However, the apparently uniform subsurface conditions in the general area should allow recommendations to be formulated which could apply to a majority of these structures with certain additional subsurface explorations necessary only to verify these conditions.

2.4 FIELD EXPLORATION

For this foundation investigation, field explorations consisted of excavating two test pits and drilling eleven exploratory borings within the general area of the central plant facilities. The two test pits (TP-1 and 2), were dug as a minimal effort at the plant site to compare with information obtained from the borings and from other nearby investigations. The eleven borings,

designated B-1 through B-11, were advanced to depths ranging between 77 and 250 feet. The location of the various borings and test pits are shown on the Preliminary Central Plant Layout (Fig. 3). As noted in this plan, the first nine borings are laid out on a rectangular grid pattern covering an area of approximately 400 by 400 feet. These original nine borings were located in anticipation that they would provide reasonable coverage of the entire central plant area since the exact location and orientation of individual buildings had not been determined at that time. Borings B-10 and B-11 were drilled at locations required for geophysical cross-hole measurements, as requested by Burns and Roe, Inc. Because of its close proximity to B-5 and B-10, no sampling was accomplished in boring B-11. The individual logs of borings and test pits are presented in Appendix A. Also presented in Appendix A is a detailed description of drilling and sampling procedures, and groundwater observation installations.

For the purpose of measuring the depth to groundwater, the 6-inch I.D. steel drill casing was slotted at appropriate depths and left in place in three borings (B-3, 7 and 9) following drilling. In boring B-1, a piezometer tip was installed near the bottom of the boring and subsequently sealed into the middle unit of the Ringold Formation with a bentonite layer. As installed, this piezometer should measure any excess hydrostatic pressures existing within that portion of the Ringold Formation. Results of groundwater readings from the observation wells and piezometer are summarized in Section 3.5.

Additional field explorations consisting of geologic reconnaissance and mapping, and geophysical surveys were conducted by others.

2.5 LABORATORY TESTING

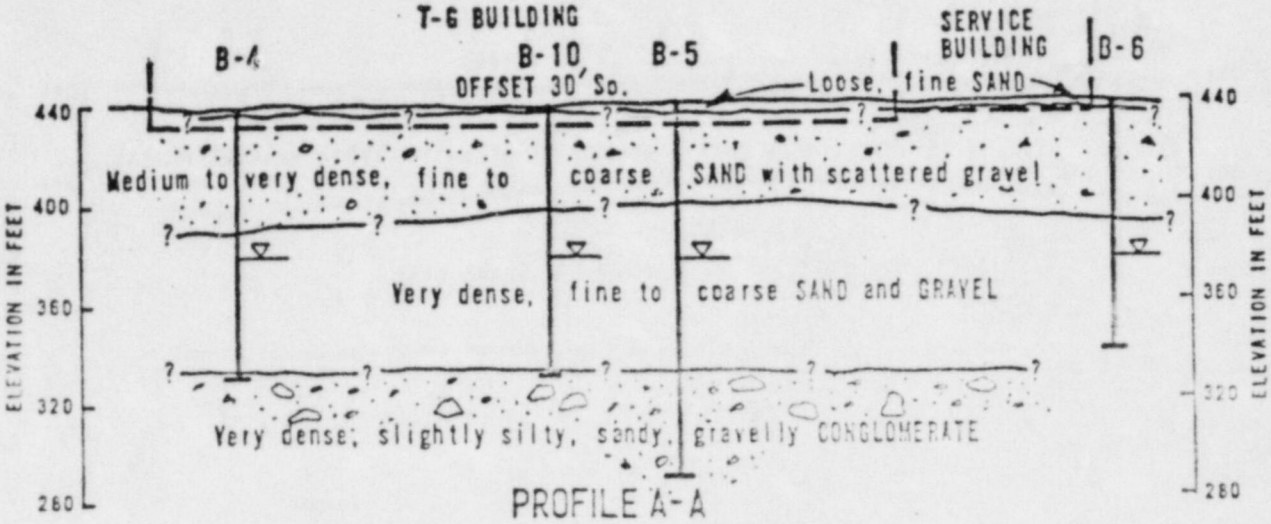
Laboratory tests were performed on representative samples recovered from the borings to determine the applicable physical and engineering properties. These tests, in addition to a detailed visual classification of each sample, included: water content determinations, grain size analyses, maximum and minimum density

determinations, permeability tests, compaction tests, resonant column tests, and repetitive triaxial tests.

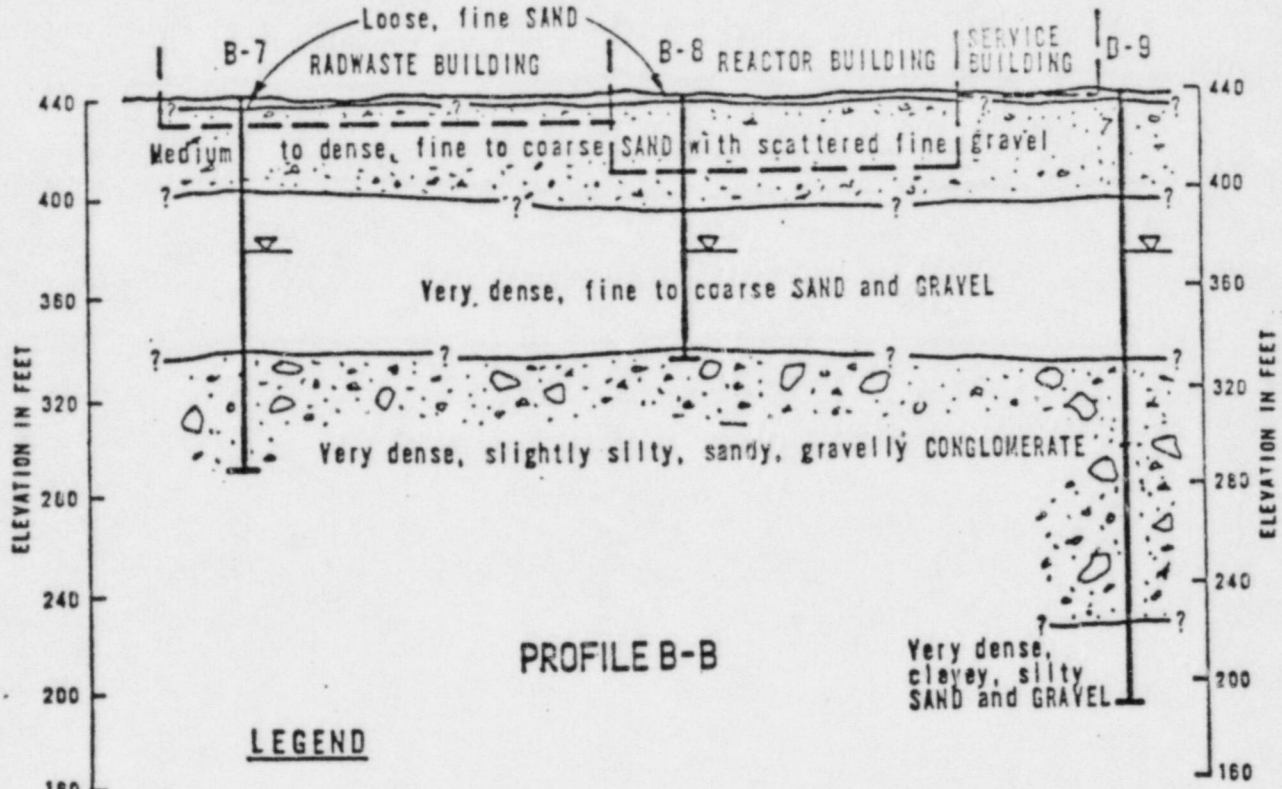
The grain size analyses were conducted primarily to verify visual classifications conducted both in the field and laboratory. Maximum and minimum density determinations were made such that remolded specimens for other tests could be prepared and tested in the laboratory at controlled relative densities. Permeability tests were used to evaluate potential drainage requirements for deep foundations; while compaction tests, together with gradation and moisture content tests were used to establish the suitability of site materials for fill, its placement, and compaction requirements. Resonant column tests and repetitive triaxial tests were conducted to provide laboratory data to be used in conjunction with field data from the geophysical explorations, for establishing design modulus and damping values for static and dynamic response calculations. The results of the laboratory test program are contained in Appendix C, together with brief descriptions of each laboratory test performed.

III. SUBSURFACE CONDITIONS

The subsurface conditions encountered beneath the plant site to a maximum depth of 250 feet are depicted in the Subsurface Profiles, Fig. 4. In general, three basic soil types were encountered in the borings. In descending order, these soils consist of: 1) fine surface SAND, 2) relatively clean, uncemented SAND and GRAVEL, and 3) sandy and gravelly CONGLOMERATE with silty and clayey zones. The following three sections (3.1, 3.2 and 3.3) discuss in detail each of these basic soil types and their physical characteristics based on: visual classification of the samples, field penetration tests, and laboratory tests. The fourth section (3.4) is devoted to a discussion of the average soil conditions at the plant site and presents certain engineering properties for possible use in site response calculations. Included in the final section (3.5) is an evaluation of the groundwater conditions.



PROFILE A-A



PROFILE B-B

LEGEND

BUILDING FOUNDATION

GROUNDWATER LEVEL

BOTTOM OF BORING

0 80 160

SCALE IN FEET

NOTES

1. THE PROFILES ARE GENERALIZED FROM THE MATERIALS ENCOUNTERED IN THE BORINGS AND VARIATIONS BETWEEN THE PROFILES AND ACTUAL CONDITIONS MAY EXIST.
2. DATUM: U.S.G.S. MEAN SEA LEVEL.

W.P.P.S.S.
HANFORD NO. 2 NUCLEAR PLANT

SUBSURFACE PROFILES

AUGUST 5, 1971 W-2139-01

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FIG. 4

3.1 SURFACE SAND

Mantling the entire plant site is a relatively thin layer of fine, slightly silty, eolian SAND (locally referred to as blow sand), which ranges from 1.5 to 3 feet in thickness. Based on gradation curves of select samples, and visual classification of all near-surface samples, these surface sands are classified as SP or SM according to the Unified Soil Classification system. These sands are generally brown to light brown, or tan in color, and have a relative density varying from loose to medium dense. The measured water content of this material averages about 6 percent. The laboratory permeability of the surface sands, recompacted in a loose to medium dense state was measured and computed to be 7.8×10^{-4} cm/sec. Based on our experience with other nearby sites, we anticipate that the average permeability of this material is slightly greater than this single laboratory test indicates.

3.2 SAND AND GRAVEL DEPOSITS

Underlying the thin layer of surface sand is a relatively thick deposit of uncemented granular soil, which extends to an average depth of about 107 feet below the ground surface. These materials, geologically, constitute portions of two formations, namely: glacio-fluvial deposits, underlain by a sandy and gravelly unit of the Ringold Formation. The distinction between the two formations is difficult to determine precisely at this site because the caliche and Palouse soils that mark the surface of the Ringold Formation throughout much of the Hanford area are missing. However, based on the penetration resistance of these materials, a comparison of average gradation curves (Appendix C), and local gradation changes noted in the boring logs, the interface between these two zones is estimated to be an average of 40 feet below the ground surface (approximately elevation 398±).

In the upper 40 feet of this deposit, the soils consist predominantly of SAND which is light brown to dark gray in

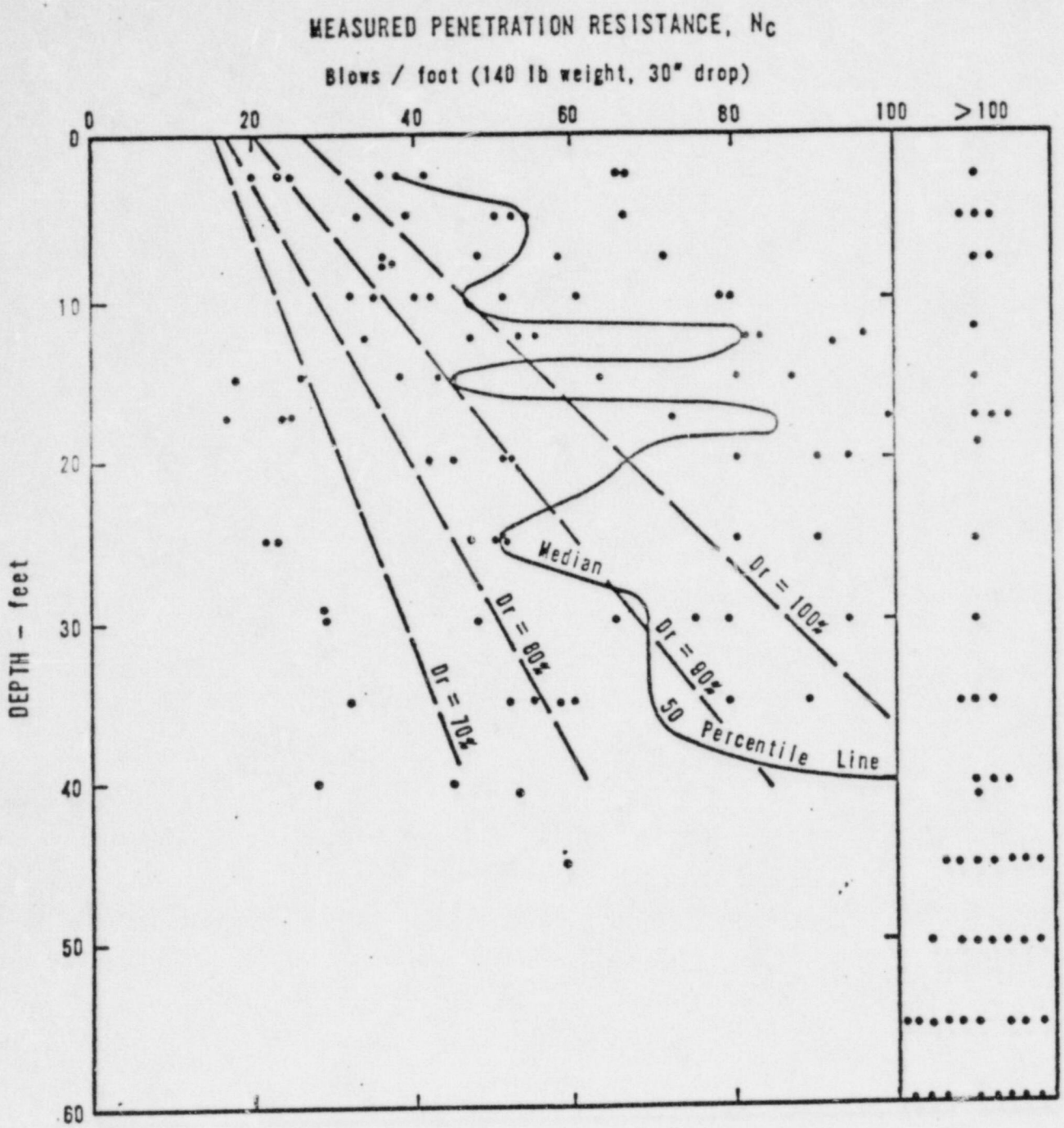
color. This sand varies locally in gradation from fine to coarse, is relatively clean to slightly silty, and generally contains considerable fine gravel throughout. Based on gradation tests, summarized in Appendix C, these soils generally correspond to either an SM or SW classification, or a combination of the two symbols, according to the Unified Soil Classification system.

Below 40 feet, the soils consist of a complex interbedded system of reasonably well graded sands and gravels. Locally, as noted on the boring logs (Appendix A), either sandy GRAVEL zones (GM-GW) or gravelly SAND zones (SM-SW) may exist within any given depth interval. Both the sands and gravels have particles which are generally rounded to subrounded. Occasional cobble zones are also present. Although no boulders were encountered in the borings, they have been encountered in other borings, especially those drilled near the Columbia River.

Most of the existing major facilities on the Hanford Reservation have been founded within and supported by these sands and gravels. Likewise, most of the proposed Hanford No. 2 Plant structures will be founded within the upper 40 feet of this deposit.

The relative density of the sand and gravel deposits, as indicated by the field penetration resistance tests summarized in Figs. 5 and 6, generally increases with depth from medium to dense or very dense in the upper 40 feet. Below this depth, the penetration resistance indicates the presence of very dense soils in essentially all cases. Exposure of similar materials on Gable Mountain, together with other geological considerations indicate that these sediments were once deposited to at least elevation 800 feet, which is about 360 feet above the present ground surface at the Hanford No. 2 site. Thus the soils beneath the plant site probably have been preloaded by loads far in excess of those to be expected from the reactor and other buildings.

Other physical properties and permeability were determined for selected specimens in the laboratory using standard test procedures. The average properties from these tests are



NOTES

1. Penetration Resistances obtained from Borings B-1 through B-10.
2. Relative density, Dr, based on Gibbs and Holtz criteria.
3. N_c values were obtained using cable tool drilling methods.
4. Penetration resistance, N_c, was reduced 25% to obtain an 'equivalent' Standard Penetration Resistance (i.e. $N = 0.75\ N_c$).
5. Net unit weight = 129 pcf.

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HANFORD NO. 2 NUCLEAR PLANT

2-INCH SPLIT-SPOON SAMPLER

PENETRATION RESISTANCE VS DEPTH

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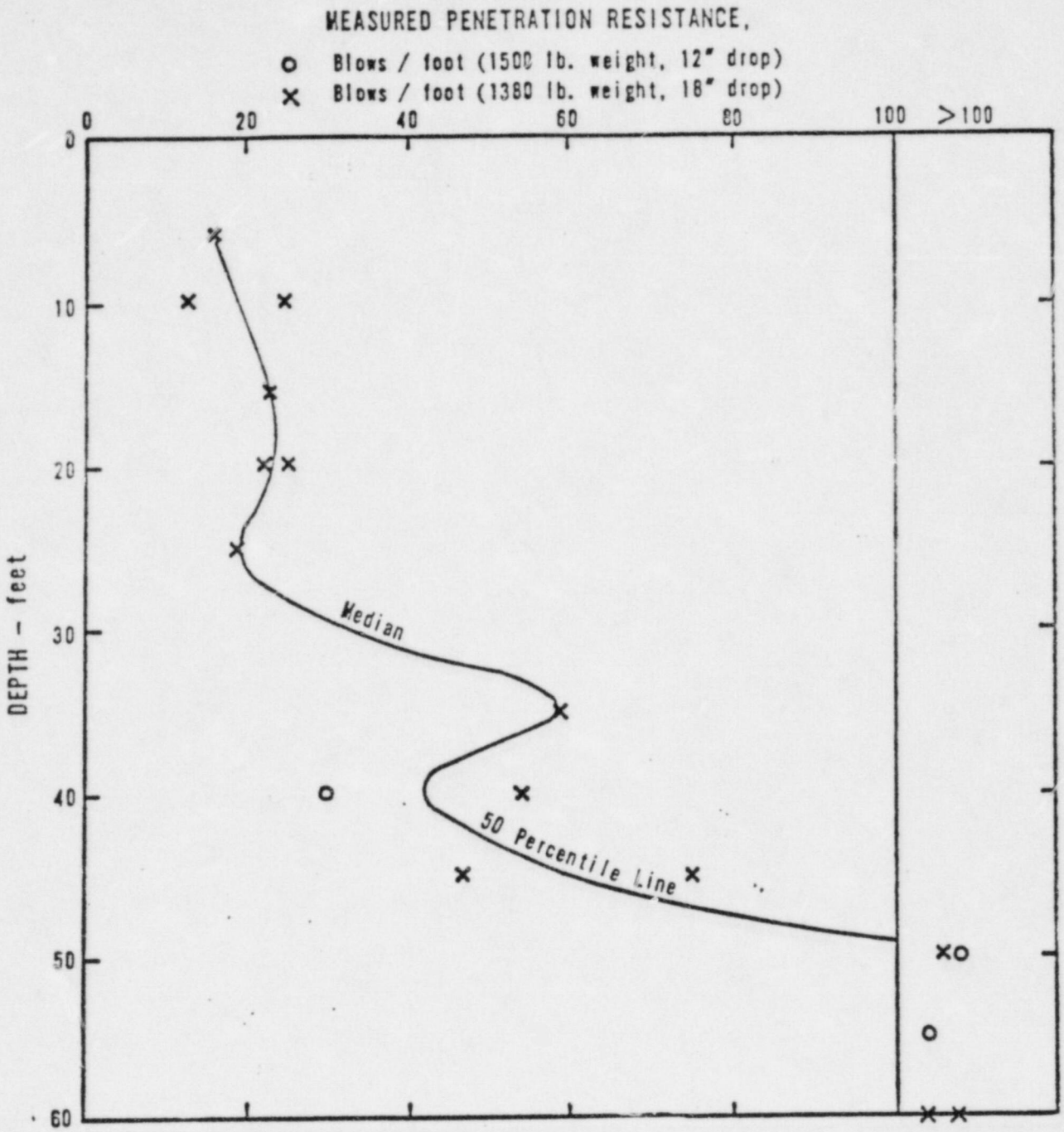
FIG. 5



NOTES

1. Penetration Resistances obtained from Borings B-1 through B-10.
2. All values measured using 4.5" O.D.(3.5" I.D.) thick wall sampler.

W.P.P.S.S.

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4.5 - INCH THICK WALL SAMPLER

PENETRATION RESISTANCE VS DEPTH

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FIG. 6

summarized in Table 1. Also specialized tests, consisting of resonant column and repeated loading triaxial tests, were performed to evaluate the moduli and damping factors. The results from these tests are presented in Section 3.4. Laboratory test procedures and results are discussed and presented in Appendix C.

TABLE 1

AVERAGE SOIL PROPERTIES-SAND AND GRAVEL DEPOSITS

1)	Approx. Average Water Content Above Water Table	6%
	Approx. Average Water Content Below Water Table	10%
2)	Mean Grainsize Above 40'	0.45 mm
	Mean Grainsize Below 40'	1.90 mm

Density Determinations

3) Maximum Dry Density	Minimum Dry Density	Depth	Material
132.3 pcf	107.1 pcf	16 ft.	Sand
118.2 pcf	98.4 pcf	20 ft.	Sand
110.2 pcf	99.6 pcf	25 ft.	Sand
115.1 pcf	104.7 pcf	35 ft.	Sand
136.6 pcf	115.0 pcf	40 ft.	Gravelly Sand

4) Permeability	Depth	Material
4.2×10^{-4} cm/sec	10 ft.	Sand
1.1×10^{-2} cm/sec	35 ft.	Sand
2.1×10^{-4} cm/sec	65 ft.	Sand & Gravel

3.3 CONGLOMERATE DEPOSIT

Beneath the sand and gravel deposits at an average depth of 107 feet, the borings encountered the middle unit of the Ringold Formation. This readily identifiable CONGLOMERATE, consisting of a multicolored gravel in a slightly clayey to silty sand matrix (GM or SM), was partially penetrated by nine of the borings, two of which (B-1 and B-9 extended completely through this unit and into the lower unit of the Ringold. The middle unit has an approximate thickness of 110 feet.

Borings were extended into this formation primarily to verify its competency as well as to establish the general

classification and pertinent engineering properties of these materials for use in site response calculations. The very high penetration resistance, with blow counts per foot far in excess of 100 in 79 tests, establishes its very dense consistency. In general, this formation is relatively uniform with depth, though occasional sandy or cobbly zones were penetrated. In three borings (B-1, B-2 and B-3) a relatively thin (4 to 9 foot) layer of hard, tan, fine sandy SILT was encountered directly overlying the Ringold CONGLOMERATE. These zones are noted in the Boring Logs (Appendix A).

Because of the very high density of this material, only minimal specialized testing was accomplished or considered necessary. The results of these tests are summarized in Section 3.4 and described in more detail in Appendix C.

The lower unit of the Ringold, penetrated in borings B-1 and B-9 at an average depth of 217 feet, was likewise found to be very competent, so much so that identification samples were difficult to obtain since these materials were difficult to penetrate with conventional soil sampling devices. The lower unit is classified much the same as the middle unit. The predominant distinguishing features are its darker color and a slightly greater percentage of fines. The complete penetration of this layer and into underlying rock with borings was not accomplished nor considered necessary as the available information including the results of the present investigation, demonstrate that these deep materials are very competent rock-like materials. Consequently, we believe that the sampling and testing of soils and rock below a 250-foot depth is not required for the prediction of the behavior of a power plant founded at the proposed location. To obtain the general classification of materials below the plant site borings, reference should be made to the brief geology section of this report or directly to the logs of deep borings made at nearby sites. These logs are reported in a nearby site investigation report (referenced in Appendix D-4).

Directly underlying the Ringold Formation is basalt bedrock of the Yakima Formation. Based on a seismic refraction survey recently completed at the site, the depth to rock at the plant is indicated to be about 420 feet.

3.4 AVERAGE SOIL CONDITIONS

In the investigation and evaluation of potential nuclear power plant sites, it is necessary to predict the site response under earthquake loading for structural design. For plants founded in soil, this requires the determination of the average subsurface conditions, including: groundwater, soil classification, average stratification and lateral distribution of the soil materials and the pertinent engineering properties of each layer. Also the depth to bedrock or dense, rock-like soils must be assessed within reasonable limits. Based on the data obtained from our field and laboratory investigations, together with other available data, we have evaluated the soil conditions and properties and present part of this information as a function of depth in the Generalized Soil Profile in Fig. 7. This profile can be considered as representative for all of the central plant structures since the soils throughout the site appear to be relatively uniform, with the interface elevation of successive major layers generally not varying more than about 5 feet between borings. Although the data shown in Fig. 7 have been projected to extend to bedrock, the upper soil materials are sufficiently dense or rock-like that input seismic bedrock motions, in our opinion, may be placed at the top of the sand and gravel conglomerate (107 feet). However, this decision is left to those performing the site seismicity evaluations.

In addition to the above subsurface conditions and parameters, the dynamic soil properties, including moduli and damping factors also must be assessed as a function of depth. These dynamic properties are essentially strain dependent and therefore their magnitudes vary according to the time history of the shear stresses applied by the design earthquake at the

DEPTH (feet)	DESCRIPTION	UNIT WEIGHT (wet)	RELATIVE DENSITY	AVERAGE MOISTURE CONTENT
0				
2	Fine SAND	γ = 115 pcf	Dr = 50%	w = 6 %
	Fine to coarse SAND with gravel	γ = 129 pcf	Dr = 85% min.	w = 6 %
40				
▽ 62	Silty SAND and GRAVEL	γ = 145 pcf	Dr = 95%	w = 8%
107				
	SAND and GRAVEL CONGLOMERATE	γ = 145 pcf	Dr = 100%	w = 12%
217				
	Silty, sandy GRAVEL	γ = 145 pcf	Dr = 100%	w = 13%
	Bottom of deepest boring			
250				
	Interbedded SAND, SILT, CLAY and GRAVEL*	γ = 145 pcf	Dr = 100%	
420 ±	Approximate top of bedrock			

LEGEND

▽ Water level

γ Unit weight

* Based on data from nearby site (S&W, Inc. files)

Dr Relative density

w Moisture content

NOTE

Average surface elevation 440' (MSL)

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GENERALIZED SOIL PROFILE

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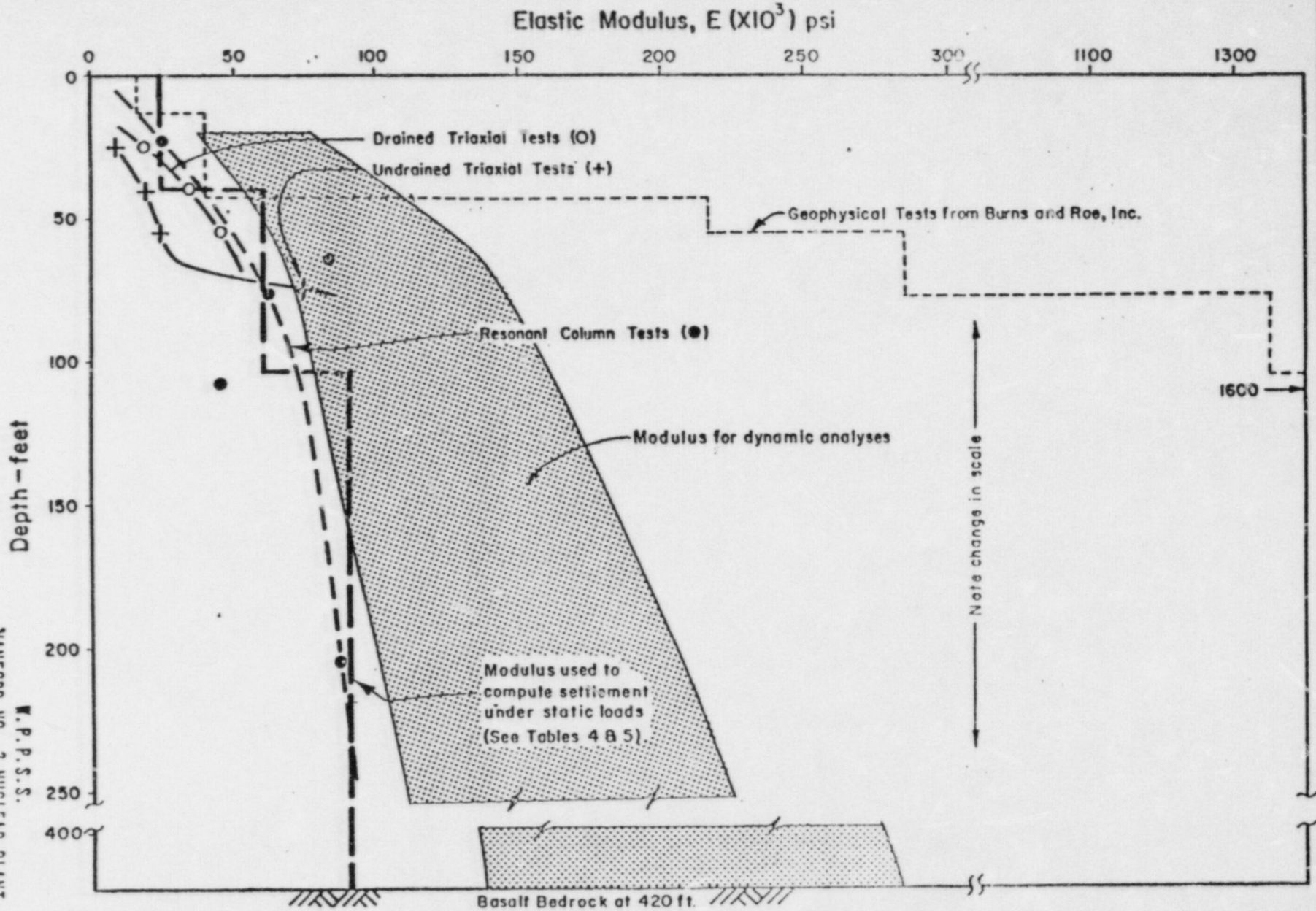
FIG. 7

appropriate depth interval being considered.

Shear and elastic moduli values were obtained in this investigation by resonant column tests and by others using geophysical techniques. The results of the geophysical tests were provided to us by Burns and Roe, Inc. Elastic moduli values were also obtained by repeated loading triaxial compression tests under both drained and undrained conditions. The moduli determined under undrained conditions were obtained for use in dynamic analyses, while those determined under drained conditions were obtained for use in evaluating settlement estimates under static loads. Those moduli values (E and G) obtained from these tests are summarized as a function of depth in Figs. 8 and 9. Since each test was performed at different strain levels, the magnitude of the modulus obtained will depend upon the test procedure used and its corresponding strain level. In general, geophysical and resonant column tests are performed at strain levels smaller than would occur during strong motion earthquakes, while repeated loading triaxial tests are performed at higher strain levels. To obtain realistic moduli values which are consistent with the strain levels of interest for earthquakes, adjustments of the test data are necessary. Since the resonant column test values were performed at reduced relative densities and on predominantly sandy samples (as discussed in Appendix C), a somewhat lower modulus was anticipated and obtained than would be expected under actual field conditions. The shaded areas in Figs. 8 and 9 represent what we believe to be realistic moduli values for design use at the Hanford No. 2 site when considering strain levels corresponding to strong motion earthquakes.

Also for use in seismic analyses, the damping ratio was measured using resonant column tests and repeated loading triaxial compression tests. Damping values of 1 to 2 percent were obtained from four resonant column tests, while higher values of 7.9 and 9.4 percent were obtained using the repeated loading triaxial test. These data are superimposed at their



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SUMMARY OF
ELASTIC MODULUS DATA
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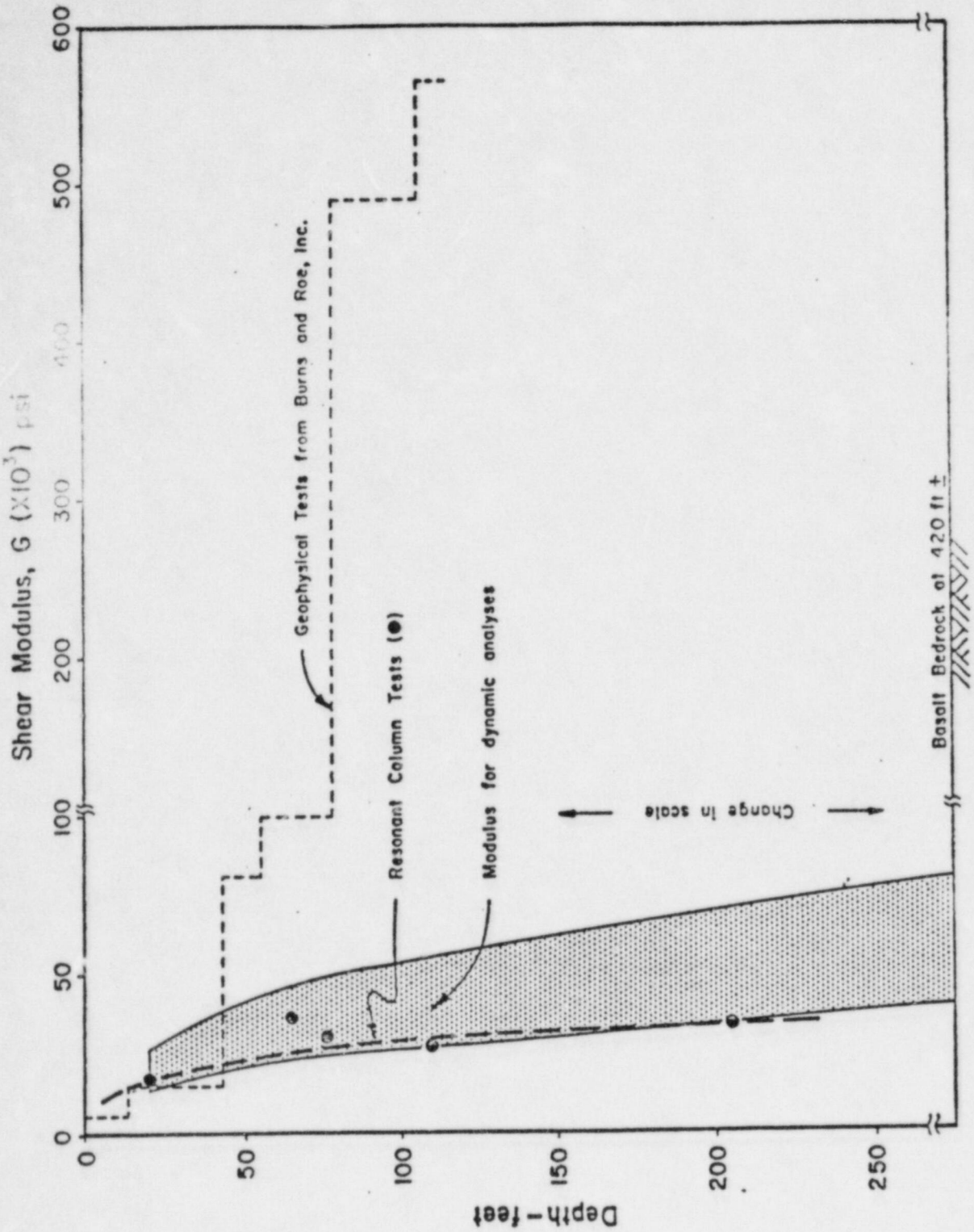
FIG. 8



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SUMMARY OF
SHEAR MODULUS DATA

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FIG. 9

corresponding strain levels on a summary plot of other published data for sands[1] in Fig. 10. From the data shown on this plot, it is evident that the damping ratio is strain dependent, increasing with higher levels of shear strain as suggested by the curves through the data points. These data may be used to guide the selection of soil damping factors appropriate to the shear strains considered for design at the Hanford No. 2 site.

3.5 GROUNDWATER

In order to obtain groundwater information, observation wells were installed in three of the four corner borings at the plant site. At the northwest corner, a sealed piezometer was installed to determine the hydrostatic pressure in the conglomerate zone of the Ringold Formation. The method used to install the piezometer and observation wells is described in Appendix A-2. During the drilling program, groundwater readings were observed in all of the borings following their completion. A summary of these data is contained in Table 2. The latest water level readings were taken on May 3 for temporary wells, and May 17 for permanent wells. These readings indicate that the groundwater at the plant site is an average of 62 feet below the ground surface (elevation 378 feet). This depth to water is consistent with other published groundwater contour maps prepared from wells drilled throughout the Hanford area. The piezometer also indicates a water level reasonably close to this average depth.

Mr. Dave Tillson of Battelle Northwest, states that Well 17-5 located approximately 2 miles north of the site, in his opinion, provides the best indication of the expected seasonal fluctuation in this area. His records on this well show an average annual fluctuation of approximately 1/2 foot, being high in the summer and low in the winter.

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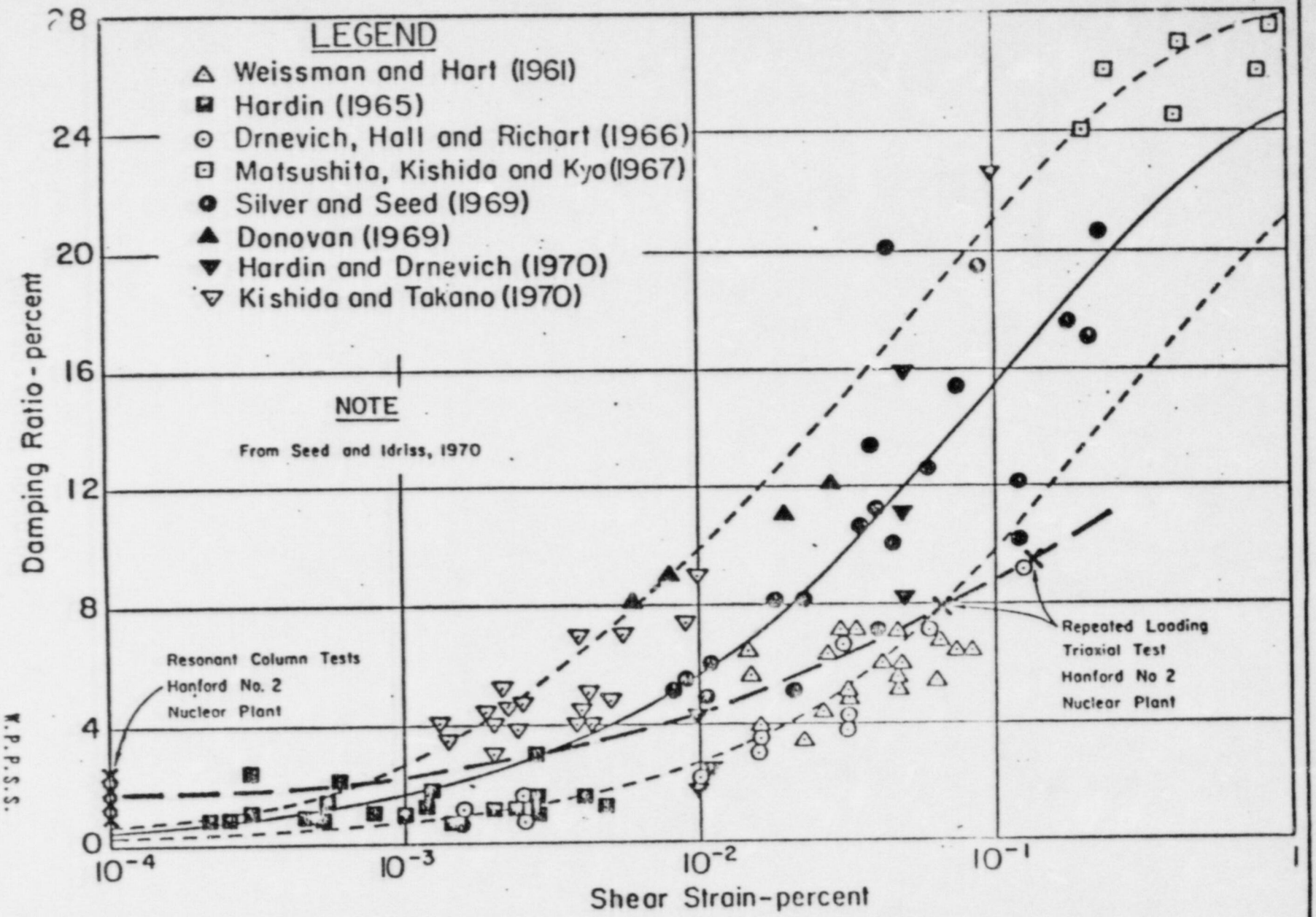
[1] Seed, H.B. and Idriss, I.M., "Soil Moduli and Damping Factors For Dynamic Response Analyses", Earthquake Engineering Research Center Report No. EERC 70-10, December 1970.



W.P.P.S.S.
HANFORD NO. 2 NUCLEAR PLANT
SUMMARY OF
DAMPING RATIOS FOR SANDS
MAY 26, 1971
K-2139-01
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FIG. 10

GROUNDWATER OBSERVATIONS

Depth to Water (feet)

Boring No. (Elevation)	B-1 (438.8)	B-2 (437.6)	B-3 (435.2)	B-4 (439.4)	B-5 (438.9)	B-6 (439.2)	B-7 (441.1)	B-8 (440.6)	B-9 (439.8)	B-10 (439.4)
Date 1971										
3/31						Bailed*				
4/2						67				
4/3						64				
4/4						62				
4/5						62				
4/6						62		Bailed*	Bailed*	
4/7						62		63.5	70	
4/12		Bailed*				62.5		64.5	69	
4/13		60				62.2		62.5	69	
4/14		60	Bailed*			62.2		62.5	69	
4/15		60	71			62.2		62.5	68.5	
4/17		60	58	62*		62.2		63	68.5	
4/18		60	59	62		62.2		63	68.5	69*
4/19		60	57.5	61.7		62.2		63	68.0	61.5
4/20		59.5	58	60.8		61.9		62.8	68	61.1
4/21		59.5	57.5	60.8		61.9		62.8	68	61
4/22		59.5	57.5	61.0		61.9		62.8	67.5	61
4/23		59.5	57.5	61.0		61.9		62.8	67.5	61.0
4/26		59.3	57.5	61.0		61.6		62.8	67.7	61.2

* Date boring completed

TABLE 2 cont.

GROUNDWATER OBSERVATIONS

Depth to Water (feet)

Boring No. (Elevation)	B-1 (438.8)	B-2 (437.6)	B-3 (435.2)	B-4 (439.4)	B-5 (438.9)	B-6 (439.2)	B-7 (441.1)	B-8 (440.6)	B-9 (439.8)	B-10 (439.4)
Date 1971										
4/27		59.5	57.2	61.1		61.5		63.4	67.1	61.2
4/28		59.5	57.5	61.2	Bailed*	61.7		62.3	66.9	61.2
4/29		Pulled	57.2	61.1	63.2	61.5		63.8	66.6	61.0
4/30			57.0	60.9	62.3	61.3		62.3	66.7	61.0
5/1			57.1	61.0	62.2	61.4		62.2	66.7	61.5
5/2			57.2	60.9	62.0	61.5		62.2	66.5	60.9
5/3	Bailed*		57.1	60.9	61.9	61.5		62.3	66.6	60.8
5/4	61.4								66.3	
5/5	60.4'		57.2						66.3	
5/6	60.4		56.9						66.5	
5/7	59.9		57.2						66.5	
5/8	60.2		57.2				Bailed*		66.3	
5/10	60.1		57.1				65.2		65.9	
5/11	60.1		57.0				62.5		65.9	
5/12	60.1		57.0				62.3		65.9	
5/13	60.0		57.2				62.5		66.0	
5/14	60.0		57.2				62.6		66.0	
5/17	60.1		57.1				62.5		65.8	

* Date boring completed

IV. FOUNDATION AND CONSTRUCTION CONSIDERATIONS

The foundation and construction considerations contained in this section are based on the data obtained during the boring program, on laboratory testing and on foundation experience from other structures in the area. This section discusses both the general foundation types and earth pressures that will be required, and also those construction problems relative to foundation and earthwork, that should be considered in preparing the final design. In addition, certain soil properties for use in the prediction of ground response under earthquake loading have been furnished, though it is understood that these analytical studies will be accomplished by others.

The conclusions and recommendations contained herein are based on data from ten borings (B-1 through 10). As shown in Fig. 3, not all of these borings fall within the present limits of the central plant facilities because the entire central complex has been relocated several hundred feet to the south since the completion of the boring program. However, because of the near ideal site conditions, encountered in the borings, we believe this investigation provides the information necessary for final design of the structures discussed herein, recognizing that additional borings will be necessary to verify the anticipated consistency of the subsurface materials at the final locations of the various plant structures. In general, we believe the depositional nature of these soils is such that the consistency in a lateral direction is not likely to change significantly from the results presented in this report.

4.1 FOUNDATION TYPES AND BEARING PRESSURES

Based on the proposed design loads for the various structures (as described in Part II and summarized in Table 3), the reactor, turbine generator and radwaste buildings should be founded on either large spread footings or mat type foundations. Because of their narrow configuration and light loads, the diesel generator

TABLE 3

SUMMARY OF BUILDING

LOADS AND FOUNDATION CRITERIA

Building	Length (ft.)	Width (ft.)	Bearing Elev. (MSL)	Estimated Area Load DL + LL (tsf)	Assumed Foundation Type
Reactor Building	140	130	409	4.5	Mat
Turbine-Generator Bldg.	304	184	432.5	2.0	Mat
Radwaste Building	218	184	429	2.75	Mat
Service Building	280	80	436.5	3.0*	Spread Footing
Diesel Generator Building	124	60	436.5	2.5*	Strip
Truck Lock	166	32	436.5	2.5*	Strip

Note: Specific information on other structures was not available prior to the submittal of this report.

* Footing stress

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building and truck lock are best suited for strip wall footings, while the larger service building can most efficiently utilize spread footings. As presently designed, all buildings in the central plant area, except for the reactor building, will be founded 3.5 to 11.0 feet below the present ground surface, and bear either partially in structural fill or in the medium to dense sands identified in the borings. The reactor building will extend about 20 feet deeper for a total depth to foundation grade of 31 feet. At this depth the sand deposit was determined to be dense to very dense. As summarized in Table 3, we believe the undisturbed soils are sufficiently compact to support the large footing or mat loads imposed by the reactor, turbine generator and radwaste buildings.

Based on the high relative density of the in situ granular soils (Fig. 5) the depth to foundation grade, and the proposed use of mat foundations, it was calculated that for these three structures the allowable bearing pressure, with respect to a possible bearing capacity failure, would exceed 50 tons per square foot. Consequently, we would conclude that bearing capacity for such foundations will not be the controlling factor in the foundation design, but rather, the settlements which the structures may undergo, as discussed subsequently in Section 4.2.

The service building, because of its wide column spacing and relatively light loads, is best suited for support by spread footings placed within the undisturbed sands encountered at the proposed grade (Elev. 436.5 feet). For the proposed 3.5-foot depth of burial, the allowable static design pressure (factor of safety = 3 against bearing capacity failure) should not exceed 3.5 tsf. The minimum width for spread footings should be 2.5 feet. If the depth of burial is increased to 5 feet, the allowable design pressure may be increased to 5 tsf for the same factor of safety.

Unless extensive shoring is used to support the reactor building excavation, it will be necessary to remove much of the natural ground needed to support the diesel generator building and portions of other surrounding buildings. Because of this, it is recommended that the soil beneath the diesel generator area be

removed to about elevation 410 feet and then replaced by compacted structural fill, as described in Section 4.7, up to final grade. Due largely to their shallow depth, and narrow width, the wall footings for this building and the truck lock must be designed for lower loads than permitted for the deeper and larger mat foundations. The minimum width of these footings should not be less than 2 feet. For a 2-foot depth of burial, the allowable static design pressure (F.S. = 3 against bearing capacity failure) should not exceed 3 tsf. For a 5-foot depth of burial the allowable design pressure may be increased to 5 tsf.

To provide safe, temporary slopes around the reactor building, a considerable amount of excavation and subsequent backfilling will be required. As a result, portions of the surrounding buildings will bear in highly compacted structural fill. Compaction requirements for such structural fill are described in Section 4.7.

The bearing capacity, in our opinion, will not be significantly reduced if at some future date the groundwater table should rise to within a few feet of the ground surface, due to the construction of Ben Franklin Dam, or some other cause. Others have calculated that with a maximum reservoir pool level of Elev. 400 feet, the groundwater level at the Hanford No. 2 site would rise to within 20 feet of the ground surface (Elevation 420 feet). This situation should be considered in design as long as there is a possibility of this occurrence within the 40-year design life of the Hanford No. 2 Plant.

One other critical factor regarding foundation support that must be considered in the design of nuclear power plants founded on granular soils, is liquefaction of the bearing soils. Based on the existing site conditions (deep water table and dense, granular soils), in our opinion, there is no potential for liquefaction at the proposed Hanford Plant. Since there is a possibility that the water table could rise some 40 feet or more in the future, we have also considered this potential condition by evaluating the in situ relative densities of the soils underlying the plant. The summary

plot of penetration test values and corresponding relative densities using the modified Gibbs and Holtz correlation (Fig. 5) demonstrates that below a depth of 40 feet, relative densities approaching 100-percent are not unrealistic, while between 25 and 40 feet, minimum relative densities on the order of 85-percent are present. Above 25 feet, the median relative densities exceed 90-percent. Further discussion and evaluation of the penetration resistance values used to obtain an "equivalent" Standard Penetration Resistance and relative density correlations are presented in Appendix B. Based on the high relative densities and assuming that the placement of structural fill is adequately controlled, we believe there will be no problem of liquefaction at this site, even under the hypothetical high water case cited.

4.2 SETTLEMENTS

Potential settlements have been computed for each of the major structures. These settlements were determined using an elastic analysis where all of the structural loads (DL+LL) were considered to act at their respective foundation grades, as uniform area loads for mat and spread footing foundations, and as individual strip loads beneath wall footings. These analyses were based on the estimated building loads as furnished by Burns and Roe, Inc. and the following elastic moduli.

TABLE 4

SUMMARY OF ELASTIC MODULI VALUES

Depth Range*	Avg. Modulus	Soil Type
0 - 40'	25,000 psi	Dense SAND
40' - 107'	60,000 psi	V. Dense SAND & GRAVEL
107' - 217'	90,000 psi	V. Dense Sand & Gravel CONGLOMERATE
217' - 420'	90,000 psi	Interbedded SAND, SILT, CLAY & GRAVEL

*Note: See Fig. 7 for generalized soil profile

These average values for the major soil zones were selected based on the general trend of modulus values obtained from drained triaxial tests as shown in Fig. 8. These values represent at best a conservative evaluation of the moduli and, in fact, are probably on the low side since it is not possible to fully recreate the in situ density conditions using small scale laboratory test methods.

What is described herein (and summarized in Table 5) as settlement, for the most part will be elastic recompression of the soils below the bottom of the excavation. Since the majority of the total and differential settlement is likely to occur elastically during construction (i.e. during the application of dead loads), post-construction settlements represent only that percentage of the maximum total or differential settlements (Table 5, Columns 1 and 2) due largely to the application of live loads. It is our understanding that for other nuclear power facilities, the live loads have generally averaged only 10 to 15-percent of the total load, with a maximum of 25-percent. The post-construction settlements presented in Table 5 (Columns 3 and 4) are based on an assumed 25-percent live load and thus represent the maximum expected values.

Because of the dense nature of the foundation soils at the project site, fluctuation of the water table, in our opinion, is not likely to significantly affect the predicted settlements, although a rise in water table reduces the effective confining pressure of the soil and thus the effective modulus. The magnitude of this change, in our opinion, is negligible when considering the high lateral stresses that must be present between the soil grains due to the previous preload of approximately 360 feet of soil, which once overlaid this site.

4.3 POSITIONING OF FACILITIES

Because the soils in the vicinity of this site were deposited and subsequently eroded under the same geological environment, their density, texture and gradation as a function of depth are generally found to be remarkably similar with only minor local variations. Because of the lateral uniformity of the supporting

TABLE 5

SUMMARY OF BUILDING SETTLEMENTS[1]

Building	Area Loading (DL+LL tsf)	SETTLEMENTS, inches Maximum[2] Total	Maximum Differential	Max. Post Construction[3]	Maximum Diff. Post Construction[3]
Reactor Building	4.5	2.1	0.8	0.5	0.2
Turbine Generator Bldg.	2.0	1.4	0.9	0.35	0.22
Radwaste Building	2.75	1.7	1.0	0.4	0.25
Service Building	3.0	1.8	1.1	0.45	0.28
Diesel Generator Bldg.	2.5[4]	0.8	0.7	0.2	0.17
Truck Lock	2.5[4]	0.7	0.2	0.17	0.05

[1] Settlements based on an elastic analysis using estimated foundation loads furnished by Burns and Roe, Inc. and the moduli values presented in Table 4.

[2] Maximum settlement from time of finished excavation to completion and occupancy of completed structures.

[3] Based on 75% of settlement occurring during construction.

[4] Footing stress.

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soils, we believe there is little or no advantage in shifting or repositioning any of the proposed structures on the site because of foundation considerations.

4.4 EXCAVATION

Excavation for the reactor building will extend 31 feet below the present ground surface, while the other buildings will be founded in the upper 11 feet of soil. All of these excavations will remain well above the water table, therefore, dry conditions should prevail during construction.

Earthwork for all excavations should be possible using conventional earth excavation equipment. Even though the sands and gravels are very dense with depth, they are generally uncemented, therefore, they should be relatively easy to excavate. No ripping should be necessary. Because of the dry climatic conditions and the relatively clean nature of the materials, construction equipment should be able to work the year round without interruption, except for occasional, severe dust storms. To improve working conditions during construction, it would be desirable to place excavated materials downwind of the construction site.

The safety of side slopes into the various excavations generally depends indirectly upon the density of the soils. As a preliminary estimate, temporary slopes no steeper than 1.5 horizontal to 1 vertical (1.5:1) should remain stable during plant construction, however, local ravelling resulting from vibration, drying and wind erosion should be anticipated. Ravelling, if found to be excessive, can be controlled or prevented either by flattening the side slopes, occasionally wetting the slopes, or by spraying the slope with one of several commercially available chemicals which provide a thin protective film or crust over the surface.

4.5 DRAINAGE

Because of the rather dry climatic conditions in the Hanford area, we do not anticipate any significant problems in regard to drainage. Also, since there is very little annual groundwater fluctuation, and because the deepest facility is located some 30 feet above the water table, we expect that there

will be no water problems.

One possibility which should be considered, however, is the effect of a significant water table rise (possibly to an elevation of 420 feet) due to the construction of Ben Franklin Dam. Under this condition, drainage or waterproofing measures must be taken for the reactor building. The other facilities are all well above this level and therefore should be unaffected by this potentially changing condition.

The waterproofing system for the reactor building substructure should be designed to resist a hydrostatic uplift pressure equal to 62.4 pounds per square foot for each foot that the water level is expected to rise above the lowest floor level. Additional lateral earth pressures on exterior, subsurface walls caused by such conditions are described in Section 4.8. Based on presently available information, the reactor building should have sufficient dead load per unit area to overcome buoyancy due to the maximum possible uplift pressure.

As an alternate to waterproofing, an extensive drainage system possibly could be designed and constructed to permanently maintain dry conditions around the reactor building. However, this approach is less practical since drainage facilities generally are more suited to those cases where the quantity of seepage is small enough to permit removal of water at a low expense, usually by gravity flow. The potential conditions at the Hanford No. 2 site (i.e., high water table, soils with high permeabilities and groundwater flow) would require a very extensive drainage system and high capacity pumping equipment to remove the water and discharge it from the plant area. The extra cost of maintaining such a dewatering system, which may never be used, in our opinion, makes a waterproofing system much more practical.

4.6 MATERIAL RESOURCES

The granular glacio-fluvial deposits (consisting primarily of clean sand and gravel mixtures), located beneath the proposed site and in various borrow pits throughout the Reservation, provide an abundant source of suitable fill material. Based on grain

size curves, water content determinations and compaction tests (as summarized in Appendix C), most of the soils to be excavated during the initial stage of site grading and building construction should be, in our opinion, suitable for use as structural fill. To use the excavated site materials, a minimal amount of screening or possibly washing may be required to remove large cobbles or excessive amounts of fines. As an alternate, more uniform deposits of sand and gravel probably exist in other nearby borrow pits. The glacio-fluvial deposits found throughout the Hanford Reservation likewise should provide an abundant source of aggregate for concrete.

The fine surface sands, located in the upper 2 to 3 feet at the proposed site, are unsatisfactory for use in building structural fills or for backfill. However, these sands are probably suitable for use in random fills, though exposed surfaces will be susceptible to wind erosion.

4.7 STRUCTURAL FILL AND COMPACTION

To ensure proper drainage and satisfactory compaction, structural fills should consist of well graded, inorganic, sand and gravel mixtures which are free of boulders and cobbles over 3 inches in diameter and contain no more than 10-percent fine grained, non-plastic materials (passing a 200 mesh sieve). As discussed in Section 4.6, most of the material to be excavated at the plant site satisfies this requirement. In some cases, a slightly higher percentage of fines (more than 10-percent) may be tolerable since those fines observed to be present in the samples from the various borings are essentially non-plastic. Also, the on-site materials are, in most cases, dry of optimum such that they should be easily worked under the dry climatic conditions existing in this area.

Structural fill and/or backfill which will support building loads, should be placed in 6 to 8-inch lifts and systematically compacted using heavy vibratory rollers. Each lift should be compacted to a minimum of 75-percent of its relative density and with an average relative density of no less than 85-percent. To assure that proper field control of filling operations is maintained, a

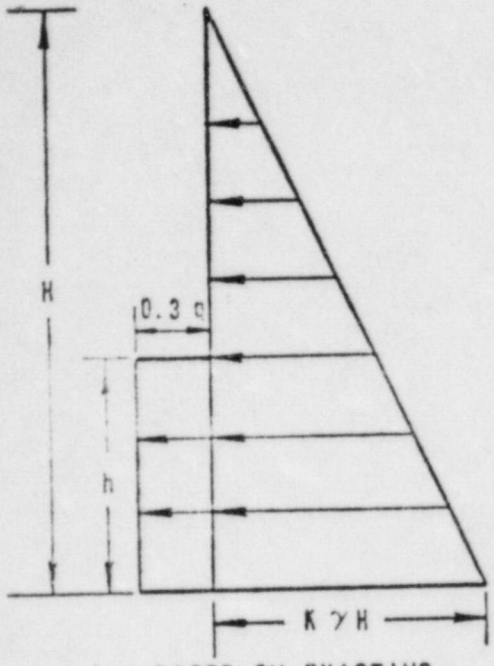
qualified inspector or soil engineer should be present on a full-time basis to check the quality of the fill and its density in a systematic manner. This should also include inspection of fill placement procedures to assure that the structural fill is uniform throughout and contains no loose pockets or zones.

From two compaction tests (AASHO T180-57) performed on samples of on-site materials, we have determined the maximum dry density to be 125 and 128 pounds per cubic foot with an optimum water content of 8 and 10-percent respectively. Since the on-site materials generally are dry of optimum by about 1 to 5-percent, the addition of water will be required to achieve maximum densities during compaction.

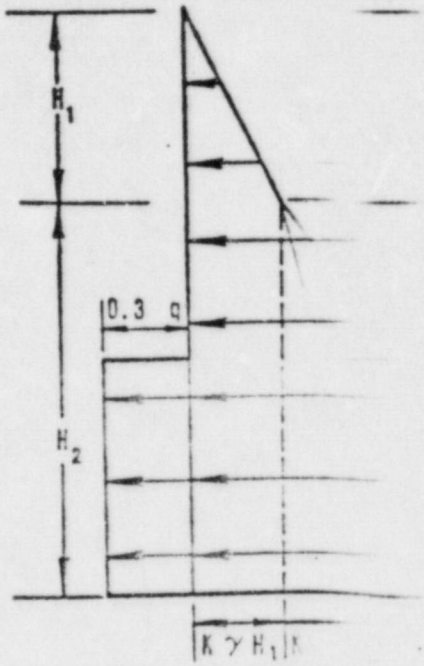
4.8 LATERAL EARTH PRESSURES

All exterior walls extending below the ground surface should be designed to resist lateral earth pressure. The magnitude of this pressure for purposes of design is primarily dependent upon the conditions assumed (i.e., with the water table as presently exists or with the water table at some predicted level on the wall). Earth pressures for these two conditions are summarized in Fig. 11. Generally, flexible walls are designed for active earth pressures (K_a), while rigid walls are designed for at-rest pressures (K_o). For design of exterior, subsurface walls of the reactor building, high at-rest pressures ($K_o = 0.8$) are considered necessary for those walls extending below elevation 432, since backfill in these areas must be densely compacted to support high footing and mat loads of surrounding buildings. Where walls will not be subjected to adjacent footing or mat loads, a lower coefficient of earth pressure ($K_o = 0.5$) may be used, as the degree of compaction need not be as high.

Also noted in Fig. 11 are criteria for determining earth pressures using a coefficient of passive earth pressure ($K_p = 8.5$) and a coefficient of active earth pressure ($K_a = 0.28$). These values were determined assuming the surrounding granular soils will have an internal friction angle (ϕ) of 36 degrees, an angle of wall friction equal to 2/3 ϕ and a log spiral failure surface (in the case of passive pressure).



A. BASED ON EXISTING GROUNDWATER CONDITIONS



B. BASED
GROUNDWATE

WHERE K = Coefficient of Lateral Earth Pressure
K_o (at rest)
K_A (active)
K_P (passive)
γ = 130 pcf (moist unit weight)
γ^1 = 68 pcf (buoyant)
γ^w = 62.4 pcf
H = Height of wall or height above or below the water table
▽ = Maximum water level on wall
h = Height between base of wall and any adjacent footing or mat
q = Actual contact stress of adjacent footing or mat foundation.

FOR DESIGN OF BASEMENT WALLS UNDER STATIC LOAD CONDITIONS

USE $K_o = 0.8$ below elevation 432

$K_o = 0.5$ above elevation 432

FOR DESIGN OF BASEMENT WALLS TO RESIST PASSIVE EARTH PRESSURE

$K_P = 8.5$ for all walls (Used for computing the maximum passive resistance of the soil)

FOR DESIGN OF FLEXIBLE (YIELDING) WALLS USING ACTIVE EARTH PRE

$K = K_A = 0.28$

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4.9 ADDITIONAL EXPLORATIONS AND STUDIES

Since the original nine borings (B-1 through B-9) were laid out on a grid system prior to final building location, the borings do not coincide with the present layout of structures in the central plant area. Though the soil conditions underlying the plant site appear to be uniformly very competent across the site and the presently available data sufficient for final design, additional explorations, in our opinion, are necessary prior to the start of construction to verify the competency of the subsurface materials at certain locations within the central plant area.

In addition, we believe the foundations for the other structures and appurtenant facilities such as: cooling towers, office buildings, parking areas, roads, railroads, rail loading docks, transmission towers, switch yard, etc., at other locations around the site should also be investigated. Many of these lighter weight structures, which presumably will be founded near the ground surface, probably can be adequately investigated with backhoe test pits and a few relatively shallow borings. A minimal laboratory testing program also will be necessary to supplement the tests performed to date.

In addition to liquefaction, discussed in Section 4.1, there are a number of other natural phenomena which should be evaluated during final design but which were not included in the scope of this investigation. In part, these would include such things as: flooding (possibly caused by the failure of an upstream dam), volcanic activity or other geologic events (such as faulting). We understand that these evaluations are being accomplished by others.

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