EVALUATION OF QUALITY CLASS I
UTILITY BACKFILL
WNP-2
HANFORD, WASHINGTON
GA FILE 81-605

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1.0 SCOPE

1.1 Introduction

The Quality Class I fill at the WNP-2 site that was placed prior to May 1976 was installed in accordance with FSAR requirements to approximately elevation 438 (see appendix of FSAR for report by Shannon and Wilson accepting all fill placed prior to May 1976). Subsequent to May 1976, excavations were made in this fill for placement of the remote air intake piping, the remote air intake structure, and the standby service water pipeline with parallel duct banks, (see Figures 1 and 6 for utility locations). It was found that the backfill used in these excavations did not conform to Quality Class I specification requirements for gradation and compaction. These nonconforming items resulted in the writing of 50.55(e) Condition 146. It is significant to note that none of the fill in question is beneath Category I buildings; five years of settlement monitoring of Category I buildings has shown without exception that structural settlements are very small and well within the range previously predicted from elastic analysis.

1.2 Objective

In order to resolve this nonconforming condition discussed in 1.1, a testing program was undertaken to determine the pertinent engineering properties of the insitu backfill. This was accomplished by relating indirect testing method results (Standard Penetration Tests, downhole pressuremeter testing and downhole nuclear density testing) to those engineering properties of the backfill which were used in design. After

these properties were determined they were used to predict the long-term performance of the backfill for both static and dynamic conditions.

2.0 SUMMARY

In order to assess the effects of the nonconforming backfill (subject of 50.55(e) Condition 146) a comprehensive testing and evaluation program was initiated. The testing program consisted of measuring various properties of the backfill using primarily the Standard Penetration Test, the downhole nuclear density test, and the downhole pressuremeter test. The results of this program indicate a good correlation between the various test methods. Furthermore, the correlations between test methods and engineering properties developed during this study agree well with similar correlations previously reported by others.

Relative densities measured in the field near the safety related utilities were found to be lower than those required in the Specification. Nevertheless, both dynamic and static settlement analysis performed to determine the effects of these lower relative density values on safety related utilities have shown that over-stress of these utilities will not occur.

3.0 METHODS OF TESTING

3.1 Indirect

3.1.1 Drilling and Testing, General

The test program utilized the standard penetration tests (SPT), downhole pressuremeter tests (PMT), and downhole nuclear density tests (DNDT) in selected areas beside the standby service water pipeline and the remote air intake structures and piping.

The borings extended to whichever of the following depths was greater:

(1) a minimum of three feet below the Category I utility, or

(2) the bottom of trenches where backfill was placed for circulating water and storm sewer Class II systems that cross under the area of investigation, or

(3) until two consecutive SPT values were each equal to or greater than 15.

Initially, at each boring location an SPT sample was taken beginning from the surface and extending to a depth of 18 inches. The split-barrel sampler was then removed to obtain the sample and the sampler was relowered to the bottom of that hole. A second SPT sample was taken to create a hole extending to a total depth of three feet. Subsequently, an aluminum casing (2" O.D. and 1.9" I.D.) was inserted in the open hole created during the SPT sampling in preparation for the downhole nuclear density testing. The nuclear probe was then lowered down the casing in order to determine the wet density of the soil.

After the nuclear density testing of the upper level soils was completed, the aluminum casing was removed, the hole was augered to the depth of three feet (to the bottom of the zone previously tested), and two consecutive SPT samples were taken below the augers, (creating a hole with a bottom depth of six feet beneath the surface). As before, the aluminum casing was placed in the open hole created beneath the augers so that the nuclear density testing could again be performed. This procedure of continuous SPT sampling and nuclear density testing was followed throughout the borings.

At selected intervals within each borehole, the aluminum casing was removed after the density testing was completed, and BX-Size Steel casing (2-7/8" O.D., 2-3/8" I.D.) was driven to the bottom of the hole and then removed. The BX casing was used to enlarge the hole three feet beneath the augers to allow insertion of the pressuremeter probe and subsequent pressuremeter testing.

The following paragraphs discuss the indirect testing methods in more detail.

3.1.2 Standard Penetration Tests

Standard Penetration Tests were performed using an 18 inch split-barrel sampler in accordance with ASTM D 1586. All borings were advanced by means of a Mobile B-61 drill rig equipped with hollow stem augers. Photograph 2.1 shows the drill rig during the performance of the Standard Penetration Test. Representative portions of each split-

barrel sample were preserved in a glass sample jar clearly labeled with the project title, date, number of boring, sample number, depth between which sample was taken, soil classification (ASTM D 2487) and SPT values. The samples are stored at the WNP-2 site and are available for examination. All field testing was monitored by a Geotechnical Engineer, who maintained detailed boring logs, which are contained in Appendix II.

3.1.3 Pressuremeter Test

A Menard pressuremeter was used to measure the insitu deformation modulus of the soil. Generally, a downhole probe which consisted of inner and outer expanding tubes was lowered to the desired depth; a coaxial cable connected the probe to the volume measuring panel board (see Photograph 2.3). Nitrogen gas was forced under pressure in the outer part of the coaxial cable while water under the same pressure was forced down the inner part of the coaxial cable. The water under pressure caused the probe to enlarge and deform the borehole wall, and the amount of volume change was measured on the panel board. A separate nitrogen system kept the water system from expanding beyond the test limits so that a controlled interval 210 mm long could be tested. Photograph 2.4 shows a pressuremeter test being performed.

The pressuremeter used in the testing was manufactured by Menard, Inc., and procedures generally followed were those described by Louis Menard in the equipment operation manual. Testing was performed in 210 mm segments at locations shown on the Profiles, Figure 5.

3.1.4 Down-Hole Nuclear Density Tests

The wet density of the relatively undisturbed soil in the borehole was determined using the DNDT; the nuclear gauge was calibrated for use in thin-walled aluminum casing. The nuclear gauge and probe used in the density testing is a Campbell Pacific Nuclear Model 501 calibrated and operated as described in the CPN Operator's Manual dated 1980. Generally, wet and dry densities were determined at three foot intervals. The density determined at each three foot interval is that which is contained in the volume of influence of a sphere having a diameter of 10 inches. Figures 2.5 and 2.6 show a DNDT being performed.

In order to convert the wet density determined by nuclear methods to dry density, the moisture contents of SPT samples were determined in accordance with ASTM D 2216. Further, at selected locations, test pits were excavated adjacent to the boring locations and the insitu densities at the bottom of these test pits were determined using a Washington Densometer and/or the sand cone. The corresponding relative densities are included in Figure 4.4 and the insitu densities are included in Appendix IV. These values of inplace density were compared with the densities determined by nuclear methods at adjacent depths as shown in Figure 4.1. In addition, DNDT results were compared to other test results (see Section 7.0).

3.2 Direct Methods

3.2.1 General

In conjunction with the indirect test methods the direct methods discussed below were used to determine insitu densities.

3.2.2 Washington Densometer

The insitu density was determined in accordance with ASTM D 2167, Standard Test Method for Density of Soil in Place by the Rubber-Balloon Method. Density test results obtained using the Washington Densometer are included in Appendix VI.

3.2.3 Sand Cone

In conjunction with the Washington Densometer, the insitu density was also determined at selected locations in accordance with ASTM D 1556, Density of Soil in Place by Sand-Cone Method. Results of these tests are included in Appendix VI.

4.0 FIELD TEST RESULTS

4.1 Subsurface Conditions

Subsurface conditions at the WNP-2 site generally consist of a layer of dense, pre-1976 sand fill overlying the very dense Ringold Formation. As mentioned, the soils that are the subject of this study are the backfill for trenches excavated into the pre-1976 fill. At the locations drilled, the deepest extent of the backfill was found to be elevation 413 feet (MSL). Both the post-1976 backfill and the pre-1976 fill consist of sand containing varying percentages of silt and gravel. This sand is known to be glacial outwash in origin and was found to range in description (Unified Soil Classification System, USCS) from a poorly graded clean sand (SP) to a well graded silty, gravelly sand (SW-SM). The majority of the backfill encountered by this testing program was found to be poorly graded (SP), and was found to contain from four to ten percent fines (i.e. material passing a #200 sieve) and from 10% to 20% gravel. The density of the sand backfill under investigation was found to be erratic and varied from loose to very dense. However, most of the backfill ranged from medium dense to dense, and moisture contents ranged from 3% to 10%. The soils that are the subject of this study are well above the present and expected future groundwater table at Elevation 405; therefore, groundwater will have no effects on the engineering properties of the backfill.

The photograph included as Figure 2.2 shows typical backfill soil in the sides of an excavation.

4.2 Standard Penetration Tests

The results of the Standard Penetration Tests are reported in the form of an N value (i.e. the number of blows required to drive the sampler the final 12 inches); the N values measured during the continuous SPT are shown on the Profiles (Figure 5). Further, the soil recovered from the split-barrel samplers during the SPT was classified in the field by a Geotechnical Engineer and these descriptions are contained in boring logs included in Appendix II, which is included in Volume 2 of this report.

The N values for the sand backfill are erratic and range from extremes of 3 to 100 blows per foot, which indicates that the relative compactness of the sand backfill varies from very loose to very dense. However, most of the N values are in the range of 20 to 40 blows per foot indicating that the relative compactness ranges from medium dense to dense for most of the soil. At borings where loose fill was encountered, additional borings were drilled on approximately 20 foot centers on either side of the initial boring until the extent of the loose zone had been defined in both horizontal and vertical extent. It was found that, at those locations examined, the loose sand fill is contained in discrete and discontinuous zones which are surrounded by denser fill. The predicted effects of these loose zones of fill on the respective utilities are described in detail in Section 8.

4.3 Pressuremeter Tests

Graphs of pressure versus volume change were developed during the pressuremeter testing and these graphs are included in Appendix III. The deformation modulus, which is proportional to the modulus of elasticity (Young's modulus), was caluclated from the pressure-volume change data for each pressuremeter test. The calculations for the deformation moduli are included on the pressuremeter plots; these values are summarized on the Profiles included as Figure 5.

The deformation modulus measured for the WNP-2 backfill ranged from extremes of 8 Kg/cm^2 to approximately 800 Kg/cm^2; however, most values were in the range of 150 Kg/cm^2 to 250 Kg/cm^2. Specifically, in the area of influence, the deformation modulus values were above 50 Kg/cm^2, and conservatively this value was used to calculate the static settlements of the various utilities as discussed in Section 8. Further, the data from the pressuremeter tests were used to evaluate the at-rest pressure coefficient (K_o) of the soil.

4.4 Down-Hole Nuclear Density Tests

Appendix IV contains a summary of the wet (moist) densities determined in the boreholes using nuclear density methods; the corresponding dry densities are also included in Appendix IV. Dry densities were calculated after determining moisture contents in the laboratory according to ASTM D 2216. The relative densities of the soil at these specific locations are summarized on the profiles included in Figure 5. These relative densities were determined by comparing down-hole nuclear density

test results to maximum densities determined in the laboratory, and by using the correlations shown in Figure 4.2.

The dry densities of the soils at the site ranged from approximately 98 pcf to 138 pcf. These dry densities correspond to relative densities from approximately 30% to 100%.

4.5. Direct Method Tests

Near surface (0-10 feet) density test results obtained by using the Washington Densometer and sand cone are included in Appendix VI. Generally, these dry densities ranged from 100 pcf to 135 pcf; these values correspond to relative densities of 30% to 100%.

5.0 LABORATORY TESTS

5.1 Grain Size Analysis

In order to classify the soil according to the Unified Soil Classification System (USCS) the particle size distribution of representative soil samples were determined in accordance with ASTM D 422. Table 2 contains a summary of the USCS classification and Appendix V contains the grain size distribution curves.

5.2 Natural Moisture Content Determinations

In order to convert wet densities into dry densities the natural moisture content of the SPT samples were determined according to ASTM D 2216. Table 2 contains a summary of the moisture contents for the site.

As stated, the moisture contents of the backfill ranged from 3% to 10%, and accordingly these low values of moisture content have no significant effect on the engineering properties of the backfill.

5.3 Triaxial Compression Tests

The shear strength and modulus of elasticity of selected soil samples were determined by unconsolidated undrained triaxial compression tests (similar to ASTM D 2850). The modulus of elasticity and angle of internal friction, determined from these triaxial compression stress-strain curves, are shown in Appendix V and are further summarized in Table 2.

The moduli of elasticity determined in the laboratory ranged from 150 Kg/cm^2 to 250 Kg/cm^2. The angles of internal friction were 31° and 34° for soils remolded to 25% and 40% relative density respectively. These values were used to verify the correlations of field test results to engineering properties as described in Section 7.0.

5.4 Maximum and Minimum Density Determinations

In order to calculate relative density in the test sections, the maximum and minimum densities were determined in accordance with ASTM D 2049. The maximum density varies from 111 pcf to 135 pcf, and the minimum density ranged from 87 pcf to 105 pcf; a summary of the maximum and minimum density results are included in Table 2.

6.0 FACTORS AFFECTING TEST RESULTS

6.1 Gravel Size Material

6.1.1 Effects on Standard Penetration Tests

The coarse gravel and cobble size particles contained in the subject backfill locally affected the results of the Standard Penetration Test. However, because these coarser particles were found to be isolated throughout the backfill, the majority of the SPT results were not affected. For those SPT results which were judged to be affected by coarse gravel particles, appropriate notes were made on the field boring logs and those values were subsequently not included in the development of correlations or in the evaluation of the backfill.

The following list contains the general criteria which were used to define SPT's which were judged to yield erroneously high N values:

(1) Greater than 10% coarse gravel size material was found in the split-barrel sampler,

(2) A loss of split-spoon sample occurred, indicating that a coarse particle may have been lodged in the end of the sampler,

(3) Angular gravel fragments were found in the split-spoon, indicating to the geotechnical engineer that a particle had been broken during driving, and/or

(4) Comparison of SPT values with other borehole test methods, indicating that SPT values were unusually high due to the presence of gravel.

6.1.2 Effect on Pressuremeter Test and Downhole Nuclear Test Results

Coarse gravel size material was judged not to have a significant effect on evaluation of pressuremter testing data or on the downhole nuclear density testing data. This results because the length of the area of influence along the borehole wall for both of these devices was approximately 10 inches (measured vertically). Therefore, in the vast majority of cases, the effect of the gravel particles was small relative to the larger size of the area being tested. In addition, these methods tend to "average" the soil properties in the area being tested, thus permitting the PMT and DNDT to approach a truer value of the insitu properties than the SPT value which only measures the resistance in the area of the spoon tip.

6.1.3 Effects on Comparison of Indirect Tests to Direct Tests

In areas where gravelly soils are present, it is believed that the PMT and DNDT measure soil properties at least as accurately as those obtained from insitu tests such as the sand cone or the Washington densometer. This results because methods measure average properties within the influence zone of the probe without removal and disturbance of the soil in the area being tested.

6.2 Percent Passing No. 200 Sieve

Occasionally, localized zones of appreciable fines (material with greater than 10% passing the No. 200 sieve) were encountered in the

borings. However, the percentage of material passing the U.S. No. 200 sieve was not a factor in evaluating the test results.

7.0 CORRELATIONS OF TEST RESULTS

7.1 General

In order to develop a correlation between the various indirect methods and relative density, three test fills were constructed using soils typical of those used for trench backfill. One fill was constructed by placing the soil in a loose condition, one by placing and compacting the soil to a dense condition, and one by placing and compacting the soil to a very dense condition. As these test fills were being constructed, numerous Washington Densometer and/or sand-cone inplace density tests were performed concurrent with the fill placement. After the test fills were completed, borings were drilled and SPT, PMT, and DNDT tests were performed. Further, after the drilling was completed, test pits were machine excavated into the test fills so that insitu densities and subsequent relative densities could again be determined using Washington Densometer and/or sand-cone devices.

After preliminary test method correlations were developed from the test fill data, several borings were drilled outside the Class I utility areas in Class II piping backfill to furnish additional data for correlations. This testing consisted of continuous SPT, DNDT, and PMT. In addition, during the drilling and the testing of the Class I utility backfill, additional results of SPT, PMT, and DNDT were compiled and compared against each other to further enhance these correlations. Moreover, at selected locations, additional test excavations were made

to again allow correlations between relative density determined by both indirect and direct test methods.

7.2 Indirect Methods Correlated to Relative Density

7.2.1 Standard Penetration Tests

A correlation between Standard Penetration Test N values (corrected for overburden pressure as described in reference 13) and relative density was developed based on the data obtained during this study. The results of this correlation are presented in Figure 4.4, where a well defined, correlation between the N values and relative density is shown (using both the Washington densometer and the DNDT to measure densities).

The results of many studies have been published which correlate Standard Penetration Test results with relative density. Some of the most widely accepted of these are the studies by Gibbs and Holtz (1960), Peck and Bazaraa (1969), and Marcuson and Bieganousky (1977) which are referenced in Section 11.0. The data developed at the WNP-2 site closely approximate the correlations reported by Peck and Bazaraa and primarily for that reason, their work was selected for comparison with this study.

7.2.2 Pressuremeter Tests

As shown on Figure 4.6 a correlation was developed between the deformation modulus and relative density of the soil at the WNP-2 site. However, because this correlation was not as well defined as those shown

in Figure 4.4 and Figure 4.7, we elected not to use this correlation in the analysis.

7.2.3 Down-hole Nuclear Density Tests

The relative density of the backfill was determined using the downhole nuclear density device and at selected locations, test pits were excavated adjacent to the boring locations and the insitu wet and dry densities at the bottom of these pits were determined using a Washington Densometer and/or a sand cone. These values of in-place density and calculated relative density were used to compare with the densities determined by nuclear methods at adjacent depths as shown in Figure 4.1, and as can be seen a good correlation was developed.

7.3 Correlations to Engineering Properties

7.3.1 Standard Penetration Tests

In addition to developing correlations to relative density, the field testing program was developed such that correlations could be developed between N, relative density, and actual engineering properties reported in the literature. For example, Schmertmann (1970) published a correlation between N and Young's modulus. Figure 4.7 shows Schmertmann's correlation between N and Young's modulus as compared to the N and deformation modulus correlation developed at the WNP-2 site.

Further, Peck (1974) developed a correlation between N (corrected for overburden pressure) and the angle of internal friction for cohesionless

soils. This correlation is shown on Figure 4.5 which also shows the relative density and N correlation developed during this study. In order to substantiate this correlation, the angle of internal friction was calculated from data obtained in the triaxial testing of WNP-2 soils remolded to both 25% and 40% relative density. Results of the triaxial tests are plotted on Figure 4.5 and in both cases, the actual angles of internal friction were slightly higher than predicted in the correlation. Thus, based on these two cases, there is a definite correlation between N values, relative density and the angle of internal friction for the WNP-2 soils.

7.3.2 Pressuremeter

The pressuremeter was used to determine the deformation modulus at different locations within the backfill. The deformation modulus was used in conjunction with the SPT N values in developing correlations with Young's modulus, and as described in the previous section, this was compared with Schmertmann's correlation between N and Young's modulus (Figure 4.7). Further, Martin (1977) used the pressuremeter in predicting settlements of structures founded on silty sand and sandy silt in residual soils. Martin reported in his studies that the deformation modulus obtained from the pressuremeter was equal to Young's modulus based on comparisons of predicted and actual settlements. The Schmertmann correlation and the results reported by Martin both substantiate the correlation shown on Figure 4.7 between N and the deformation modulus. Finally, the data developed during this study and the data reported by

others indicate that the deformation modulus is equal to Young's modulus for the WNP-2 soils.

7.3.3 Down-hole Nuclear Density

The nuclear density gauge was used to determine insitu densities from which relative densities could be calculated. The relative densities determined in this manner were used in conjunction with the SPT N values in developing correlations with the angle of internal friction as described in Section 7.3.1.

8.0 TEST RESULTS AS RELATED TO DESIGN FUNCTION

8.1 Liquefaction

It has been found that liquefaction is not possible in the soil placed after May 1976, because the present and future position of the water table is well below all of the backfill. The highest predicted elevation of the water table is elevation 405 and since the lowest extent of the backfill in question is elevation 413, liquefaction cannot occur in the dry to moist soil conditions. (Note: elevation 405 has been predicted conservatively as the maximum future elevation of the water table at the WNP-2 site if the Ben Franklin Dam is constructed).

8.2 General

Determination of the adequacy of insitu conditions relative to the design function of the standby service water pipeline, and the remote air intake structure and piping has been accomplished by considering stress conditions that may result from potential static and dynamic settlement in the lowest relative density zones found.

Zones of low relative density were found in the following areas:

(1) at line WOA 51A of the remote air intake piping, low relative densities ranged from 45% to 50% in Boring CT-43;

(2) at line WOA 51B of the remote air intake piping, low relative densities ranged from 30% to 40% in Borings CT-3 and CT-40. Boring CT-40, however, reflects the condition of the backfill around the manhole;

(3) at the standby service water pipeline, the lowest relative density was 45% in Boring CT-11.

None of these low relative density zones were found to be continuous from one boring to another. Moreover, observation of excavations made in these safety related areas indicate that the loose zones are limited from 5 to 10 feet in extent. However, for design purposes a horizontal extent of 20 feet was conservatively selected for the length of any loose zones (refer to Section 4.2). This distance is consistent with the requirement in the testing procedure to add an additional boring offset 20 feet from any boring where a loose zone (N value less than 15) was found.

8.3 Static Conditions

For the static case, settlements were determined using elastic-half-space theory employing Young's modulus determined from actual field measurements, made during the backfill testing program (after Schmertmann, 1970). Figures 7.1 and 7.2 show the settlement plots resulting from this analysis. Conservatively, these plots were developed using the lowest deformation modulus found at each utility; the respective values used are shown on Figures 7.1 and 7.2.

It has been conservatively estimated that the net contact pressure under the foundation of the remote air intake lines, the remote air intake structures, and the standby service water pipeline is less than 0.2 KSF. For this value (0.2 KSF) of net stress, total settlements of less than 0.1 inches are estimated. For purposes of calculating piping stresses this total static settlement was conservatively selected to

represent the differential settlement that may occur at the center of any 20-foot section of safety related piping.

Using this value of settlement, it has been determined that negligible stress increase in the piping will occur; it is therefore concluded that for the static case, soil conditions near the safety related piping will have no detrimental effect on the integrity of these systems.

8.4 Dynamic Conditions

For the dynamic case (SSE conditions), two determinations were made: first, the potential for less dense backfill near the pipe to cause an overstress in those safety related systems; and second, the adequacy of the safety related piping to accommodate seismic settlements in less dense backfill.

For the first condition, the effect of less dense backfill adjacent to buried piping has been found to not affect the seismic wave passage (particle velocity) for the total plant. On the contrary, less stringent compaction will result in the potential for slippage (between the pipe and the backfill) which is beneficial as seen from the equation by Newmark in the FSAR reference 3.7-12.

For the second condition, seismic settlements of the fill were computed using the cyclic shear strain method. This is generally similar to the cyclic strain approach to liquefaction of saturated sand proposed by Dobry, et al (see Reference No. 2).

Utilizing this method in representative zones of lowest relative density, and using the corresponding lowest values of K_o, calculations were performed to determine "best estimate" settlement for the remote air intake piping line WOA 51A, and WOA 51B, of 1.1 inches and 1.5 inches, respectively. Similarly, for the standby service water pipeline Dr. Dobry has calculated settlements of 0.3 inches (see Table 3).

For purposes of calculating piping stress, these total seismic settlements were increased to three inches and were conservatively assumed to represent the differential settlements that may occur at the contour of any 20 feet section of safety related piping. Imposing these conditions on all buried safety related utilities has shown that pipe stresses are well within the allowable limit. It is therefore concluded, that insitu soil conditions near the piping have no detrimental effect during SSE conditions.

9.0 CONCLUSIONS AND RECOMMENDATIONS

In the previous discussion, it has been shown that stress conditions resulting from potential static and dynamic settlements in loose soil zones will have no detrimental effects on buried safety related piping.

Based on the conclusions made during the backfill testing program it is our recommendation to accept the backfill placed after May 1976 around these safety related utilities.

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10.2 Correlations of data and static settlement analysis by Geologic Associates, Inc.

10.3 Dynamic settlement analysis was performed by R. Dobry.

10.4 Determinations of seismic and settlement general stress effects on piping by Burns & Roe, Inc.

10.5 Conclusions and recommendations by Burns & Roe, Inc.

10.6 Preparation of text jointly by Geologic Associates, Inc., and Burns & Roe, Inc.

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15. Schmertmann, J. H. (1970), "Static Cone to Compute Static Settlement Over Sand," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol 96, No. SM3, Proc. Paper 7924, May, pp 1011-1043.

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WPPSS Hanford No. 2
50.55(e) Condition No. 146

TABLE I - BORING LOCATION AND TESTING TABULATION

Burns and Roe, Inc.
See page ____ for notes

Correlation Tests (CT)	Hanford Area Coordinates North	West	Type, Depth & Number of Tests SPT	PMT Depth of Tests	DNDT Number of Tests	Subject of Testing
1	11,098	488	Continuous	--	--	Class G Light Pole
2	11,086	493	Continuous	--	--	Class G Light Pole
3	11,565	1,020	Continuous	--	--	Air Intake Structure
4	12,281	1,596	Continuous	--	--	Air Intake Structure
5	12,072	1,560	Nearly Continuous	--	--	Air Intake Line WOA 51A
6	11,713	1,560	Continuous	--	--	Radwaste - Control Bldg.
7	11,679	1,191	Nearly Continuous	--	--	Service Water and Class II Storm Sewer
8	12,134	1,128	Nearly Continuous	--	--	Class II Circulating Water (2) - 1
9	11,802	1,077	Nearly Continuous	--	--	Class II Circulating Water (1) - 1 and (2) - 1.
10	11,916	1,043	Continuous	--	--	Class II Storm Sewer and Sanitary Sewer
11	11,485	1,075	Continuous	--	--	Service Water and Class II Circulatory Water (2) - 1
12	11,317	861	Continuous	--	--	Service Water and Class II Circulation Blow-Down.

WPPSS Hanford No. 2
50.55(e) Condition No. 146

TABLE I - BORING LOCATION AND TESTING TABULATION

Burns and Roe, Inc.
See page _____ for notes

Correlation Tests (CT)	Hanford Area Coordinates North	Hanford Area Coordinates West	Type, Depth & Number of Tests SPT	PMT Depth of Tests	DNDT Number of Tests	Subject of Testing
13	11,132	1,012	Nearly Continuous	--	--	Spray Pond 1B
14	10,956	813	Nearly Continuous	--	--	Class G Light Pole
15	11,021	1,012	Continuous	1.4 18.5	17	Spray Pond 1B
16	11,219	1,057	Continuous	6.4 12.4 18.6	7	Class II Circulating Water (2) - 1
17	10,887	1,502	Continuous	6.4 12.6	9	Class II Circulating Water
18	10,884	1,406	Nearly Continuous	9.1	--	Class II Circulating Water
19	10,967	1,406	Continuous	10.6 12.0 17.0	6	Class II Circulating Water Drain from Cooling Tower 1B
20	10,837	1,384	Continuous	10.8 14.0	13	Class II Circulating Water Drain from Cooling Tower 1C
21	10,913	1,270	Continuous	11.0 14.0	9	Class II Circulating Water Drain from Cooling Tower 1A
22	10,996	1,258	Nearly Continuous	9.0 15.6 21.6	10	Class II Circulating Water (2) - 1

WPPSS Hanford No. 2
50.55(e) Condition No. 146

TABLE I - BORING LOCATION AND TESTING TABULATION

Burns and Roe, Inc.
See page ____ for notes

Correlation Tests (CT)	Hanford Area Coordinates North	Hanford Area Coordinates West	Type, Depth & Number of Tests: SPT	PMT Depth of Tests	DNDT Number of Tests	Subject of Testing
23	11,128	1,500	Nearly Continuous	12.4	10	Class II Circulating Water (1) - 1
24	11,081	1,576	Continuous	9.4	7	Class II Circulating Water (1) - 1
25	11,385	661	Continuous	13.2	9	Class II Tower-Make-Up and Circulation Blow-Down
26	11,399	538	Nearly Continuous	11.1	6	Class II Tower-Make-Up and Circulation Blow-Down.
27	12,155	1,060	Continuous	9.6	12	Class II Circulating Water (2) - 1 and Class G Light Pole
28	11,687	1,178	Nearly Continuous	--	9	Service Water and Class II Storm Sewer
29	11,574	1,182	Continuous	6.2	3	Service Water and Class I Duct Banks
30	11,676	1,328	Continuous	--	--	Class II Fire Protection (7) - 1
31	11,645	1,421	Nearly Continuous	--	--	Class II Fire Protection (6) - 1
32	12,017	1,565	Continuous	--	4	Air Intake Line 51A

WPPSS Hanford No. 2
50.55(e) Condition No. 146

TABLE I - BORING LOCATION AND TESTING TABULATION

Burns and Roe, Inc.
See page _____ for notes

Correlation Tests (CT)	Hanford Area Coordinates		Type, Depth & Number of Tests			Subject of Testing
	North	West	SPT,	PMT Depth of Tests	DNDT Number of Tests	
33	12,121	1,565	Continuous	4.7	--	Air Intake Line WOA 51A
34	11,377	1,354	Continuous	6.2 12.5	--	Class II Fire Protection (1) - 1
35	12,286	1,565	Continuous	4.8	--	Air Intake Line WOA 51A
36	12,244	1,025	Continuous	5.0	5	Class II Fire Protection (4) - 1 and Storm Sewer
37	11,292	956	Continuous	7.7 14.0	--	Service Water
38	11,319	769	Continuous	4.9	--	Service Water
39	11,623	1,031	Continuous	4.4	6	Air Intake Line WOA 51B
40	11,698	1,031	Continuous	4.7 11.3 26.0	14	Air Intake Line WOA 51B and Class II Storm Sewer
41	11,771	1,031	Nearly Continuous	6.6	7	Air Intake Line WOA 51B and Class II Storm Sewer.
42	11,681	1,264	Nearly Continuous	9.2	6	Service Water Class I Duct Banks

WPPSS Hanford No. 2
50.55(e) Condition No. 146

TABLE I - BORING LOCATION AND TESTING TABULATION

Burns and Roe, Inc.
See page _____ for notes

Correlation Tests (CT)	Hanford Area Coordinates		Type, Depth & Number of Tests			Subject of Testing
	North	West	SPT	PMT Depth of Tests	DNDT Number of Tests	
43	12,200	1,565	Continuous	7.6 19.4	13	Air Intake Line WOA 51A
44	11,509	1,057	Nearly Continuous	9.1 19.8	11	Service Water, and Class I Duct Banks and Class II Circulating Water (2) - 1
45	11.563	1,120	Continuous	7.8 16.6	14	Service Water, Class I Duct Banks and Class II Circulating Water (1) - 1
46	11,591	1,031	Continuous	7.7	4	Air Intake Line WOA 51B
47	11,678	1,031	Continuous	7.7 9.4	13	Air Intake Line WOA 51B
48	11,717	1,031	Continuous	6.5	5	Air Intake Line WOA 51B and Class II Storm Sewer
49	11,657	1,029	Continuous	6.1	5	Air Intake Line WOA 51B
50	11,694	1,050	Continuous	17.0	13	Air Intake Line WOA 51B and Class II Storm Sewer
51	11,700	1,015	Nearly Continuous	16.8	14	Air Intake Line WOA 51B and Class II Storm Sewer
52	11,744	957	Continuous	8.0 16.7	14	Air Intake Line WOA 51B and Class II Storm Sewer

TABLE 2

SUMMARY OF LABORATORY TEST RESULTS

Project: WNP-2
Project No.: 81-605
Date:

Hole No.	Sample No.	SAMPLE TYPE*	Depth	Natural Moisture (%)	Relative Density Standards: Maximum	Relative Density Standards: Minimum	Modified Proctor: Maximum Dry Density	Modified Proctor: Optimum Moisture	UNIFIED SOIL CLASSIFICATION	TRIAXIAL SHEAR TEST: ANGLE OF INTERNAL FRICTION, φ	TRIAXIAL SHEAR TEST: COHESION C (PSF)	OTHER TESTS**: Lab-Young's Modulus (Kg/cm^2)	Soil Description
CT5		B	3.0-4.0						SP-SM				- See Logs-
CT6		B	3.0-4.0		122.5	87.7	123.4	10.2	SP				
CT7		B	3.0-4.0				132.4	6.7	SP				
CT10		B	3.0-4.0						SP				
CT15		B	3.0-4.0		128.7	106.4	131.9	7.1	SW-SM	31°	350	140	
CT15A		B			130.2	99.0			SP-SM				
CT17		B	3.0-4.0	5.1	120.9	101.5	112.9	-	SP	39°	300	160	
CT17A		B	3.0	5.1	116.4	97.8			SP				
CT19		B	3.0-4.0	4.7					SW				
CT21		B	3.0-4.0	4.5	111.1	94.4			SW				
CT22		B	3.0-4.0	5.1	133.9	92.6			SP-SM				
CT43		B	3.0-4.0	4.0					SM				

* ST—SHELBY TUBE SAMPLE, SS—SPLIT SPOON SAMPLE, B—BAG SAMPLE
** TEST RESULTS REPORTED ON OTHER SHEETS:
C—CONSOLIDATION
S—SIEVE OR GRAIN SIZE ANALYSIS
U—UNCONFINED COMPRESSION TEST
D—DIRECT SHEAR TEST
T—TRIAXIAL TEST

TABLE 2

SUMMARY OF LABORATORY TEST RESULTS

Project: WNP-2
Project No.: 81-605
Date:

Hole No.	Level	Sample Type*	Depth	Natural Moisture (%)	Relative Density Standards: Maximum	Relative Density Standards: Minimum	Modified Proctor: Maximum Dry Density	Modified Proctor: Optimum Moisture	Unified Soil Classification	Triaxial Shear Test: Angle of Internal Friction, ϕ	Triaxial Shear Test: Cohesion C (PSF)	Other Tests**: Lab-Young's Modulus (Kg/cm^2)	Soil Description
TS		B	3.0-4.0		126.8	99.7	130.7	5.8	SM				- See Logs-
TS	6	B	4.0						SP-SM				
TS	5	B	5.0						SP-SM				
TS	4	B	6.0						SP-SM				
TS	3	B	7.0						SP				
TS	2	B	8.0						SM				

* ST—SHELBY TUBE SAMPLE, SS—SPLIT SPOON SAMPLE, B—BAG SAMPLE
** TEST RESULTS REPORTED ON OTHER SHEETS:
C—CONSOLIDATION
S—SIEVE OR GRAIN SIZE ANALYSIS
U—UNCONFINED COMPRESSION TEST
D—DIRECT SHEAR TEST
T—TRIAXIAL TEST

GEOLOGIC ASSOCIATES, INC.

TABLE 3

ESTIMATED SEISMIC SETTLEMENTS OF QUALITY CLASS I UTILITIES

Area	Boring	Z_f(x)	Z_m(x)	Seismic Settlement, S_t		
				M = 6		M = 6-1/2
				Estimated Range	Best Estimate	Best Estimate
Air Intake - Line WOA 51A	CT-43	6'	22'	0.8"-2"	1.1"	1.4"
Air Intake Line WOA 51B	CT-40	6'	30'	1"-3"	1.5"	1.8"
	CT-3	6'	25'	0-0.5"	0.1"	0-0.5"
Standby Service Water Pipeline	CT-11	8'	22'	0.1"-0.8"	0.3"	0.4"

(x) Z_f = Depth of facility

Z_m = Maximum depth of boring

M = Richter Magnitude

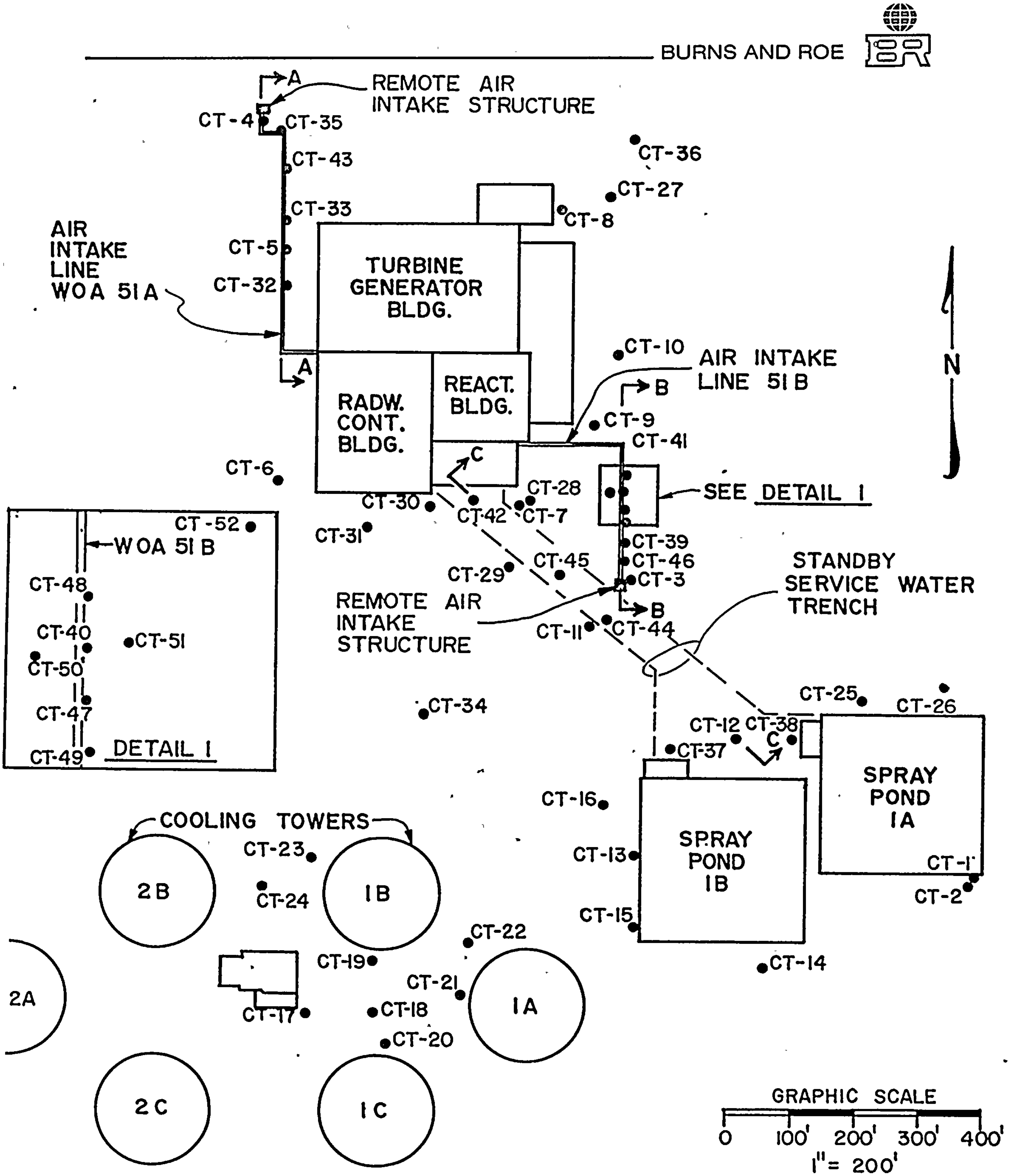
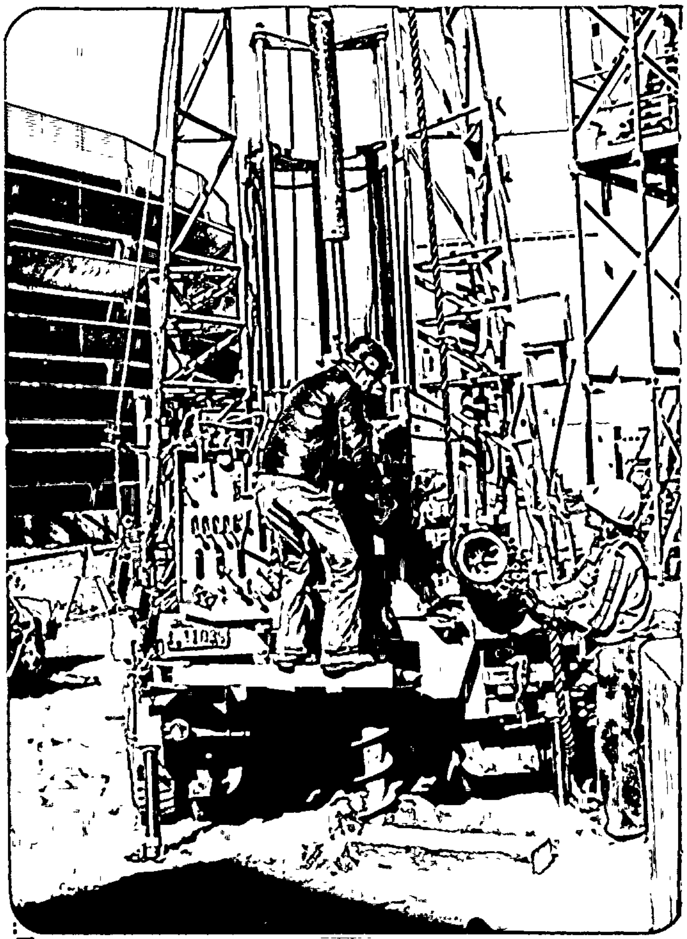


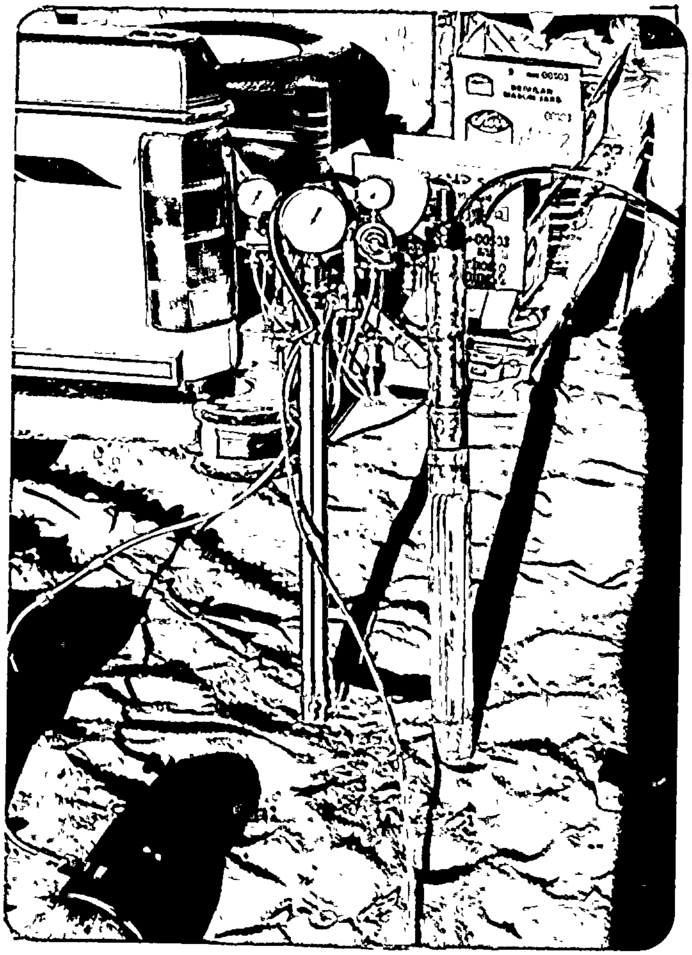
FIGURE I BORING LOCATION PLAN

PHOTOGRAPH 2.1 (at left). Drilling crew performing Standard Penetration Test using Mobile B-61 drill rig.

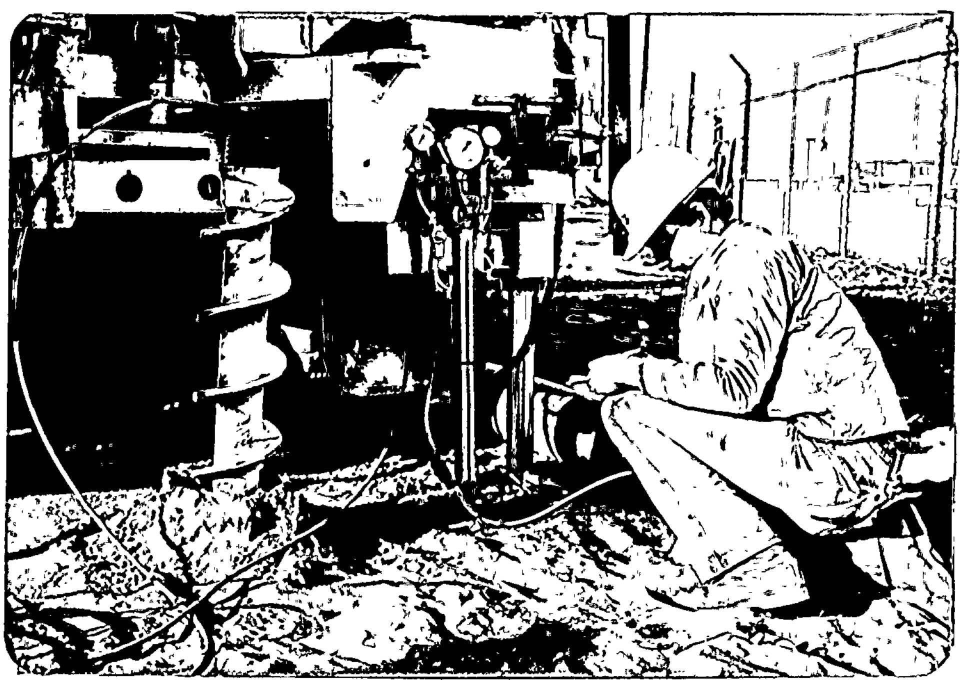


PHOTOGRAPH 2.2 (at right). Typical sand fill at sides of trench.

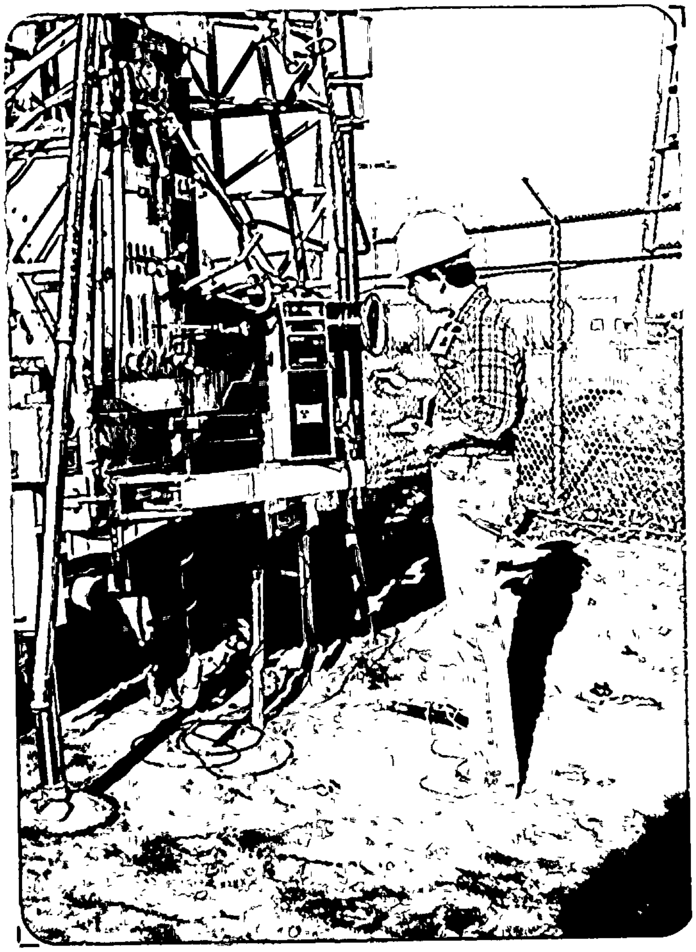
PHOTOGRAPH 2.3 (at right).
Pressuremeter testing equipment.
Expanding probe (on right) and
volume measuring panel (on left)
connected with coaxial cable.



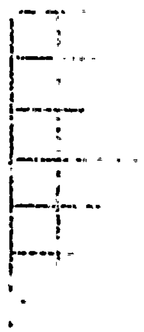
PHOTOGRAPH 2.4 (below).
Pressuremeter test being
performed.

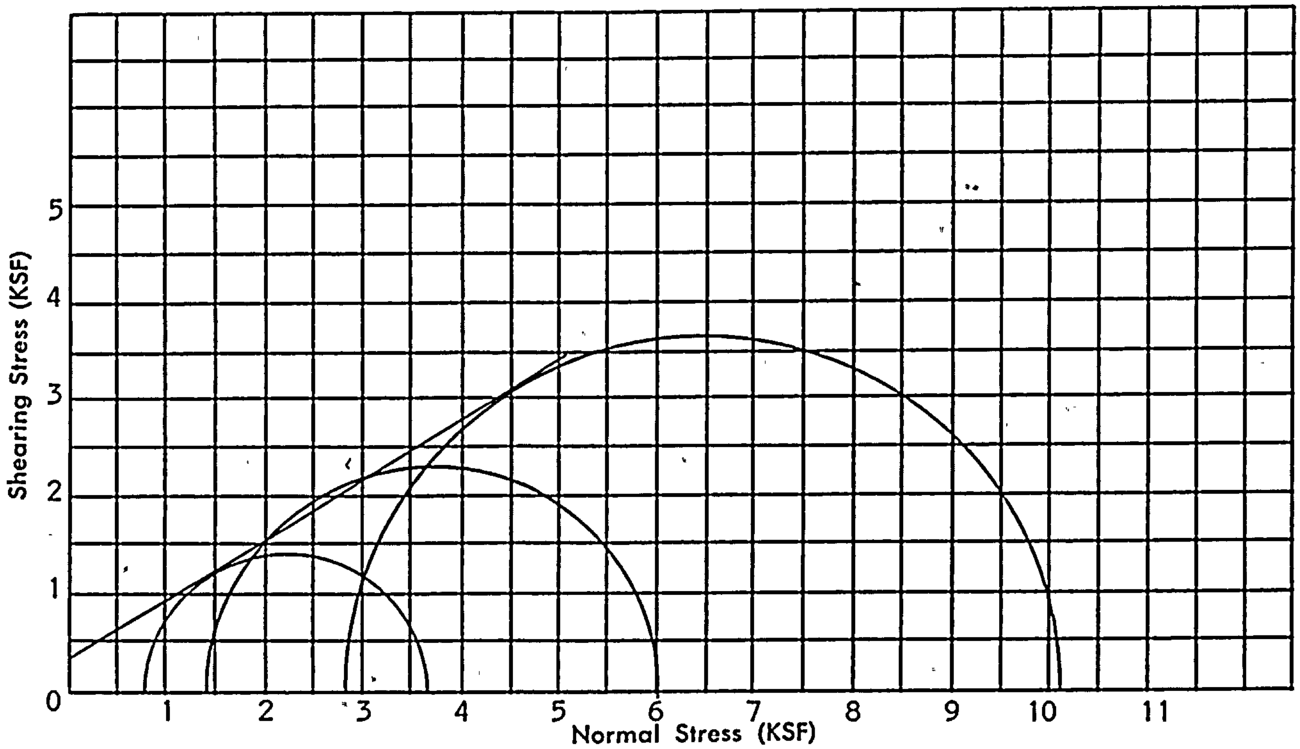


PHOTOGRAPH 2.5 (at left). Downhole nuclear density test probe being lowered down aluminum casing.

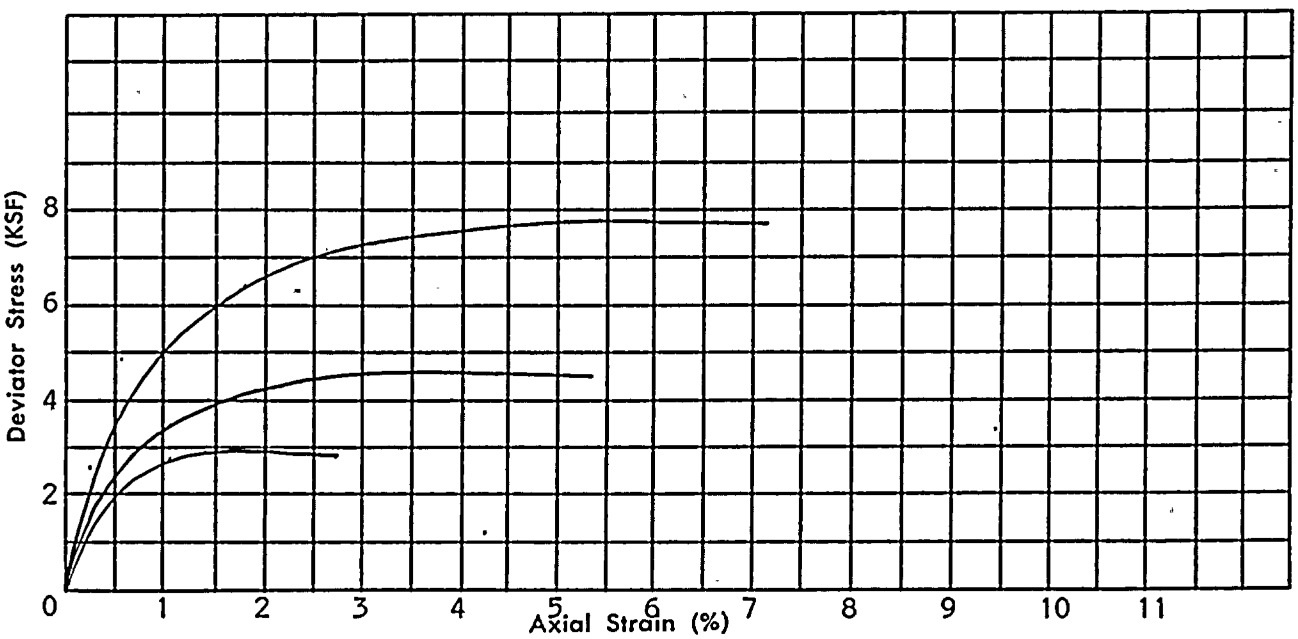


PHOTOGRAPH 2.6 (above right). Downhole nuclear density test being performed.





MOHR DIAGRAMS — ϕ



STRESS — STRAIN CURVES

TRIAXIAL SHEAR TEST

SOIL DESCRIPTION See logs

COHESION (c) 350 PSF

ANGLE OF INTERNAL FRICTION (ϕ) 31°

UNIT WEIGHT, PCF 110.4

WATER CONTENT, % 11.0

SPECIFIC GRAVITY

VOID RATIO

CLIENT

PROJECT WNP-2

PROJECT NO.: 81-605

BORING NO.: CT-15

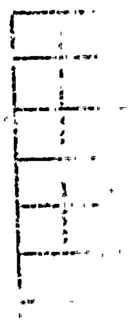
SAMPLE NO.: BAG

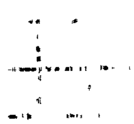
ELEV. OR DEPTH

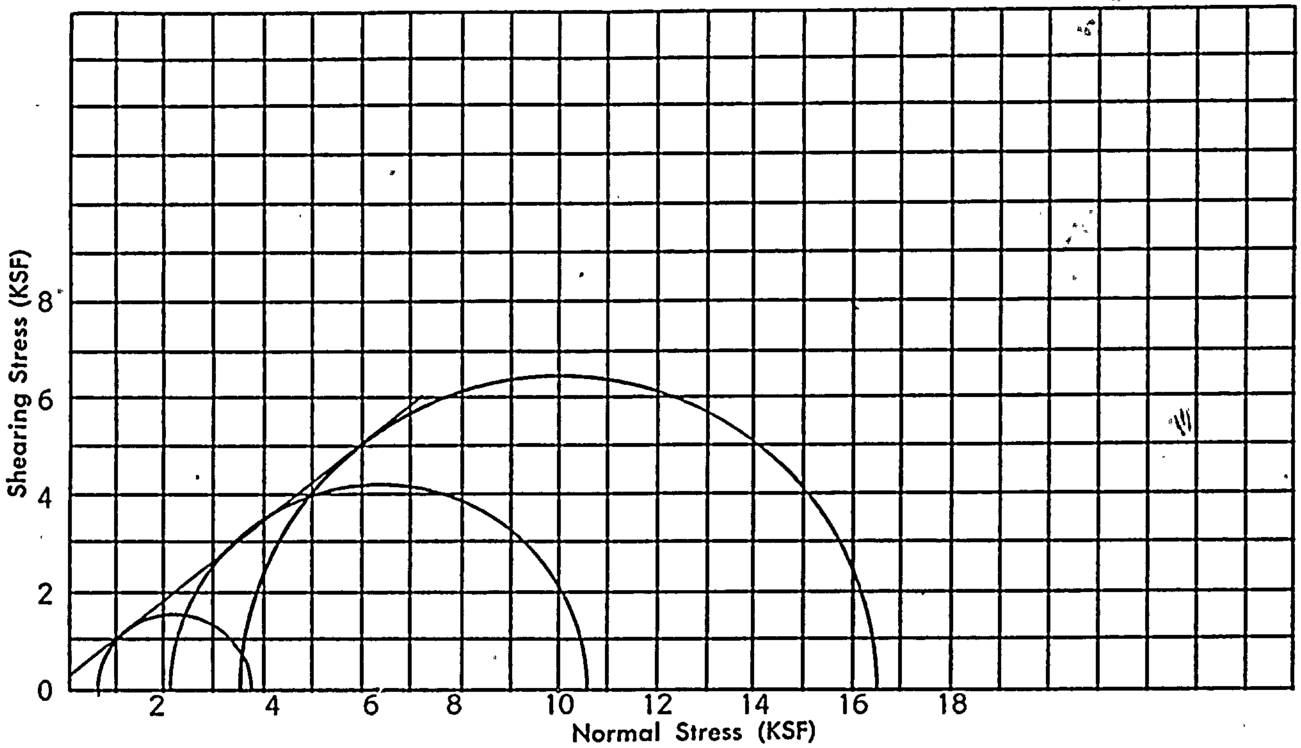
DATE: November 16, 1981

Figure 3.1
Plots of Laboratory Data

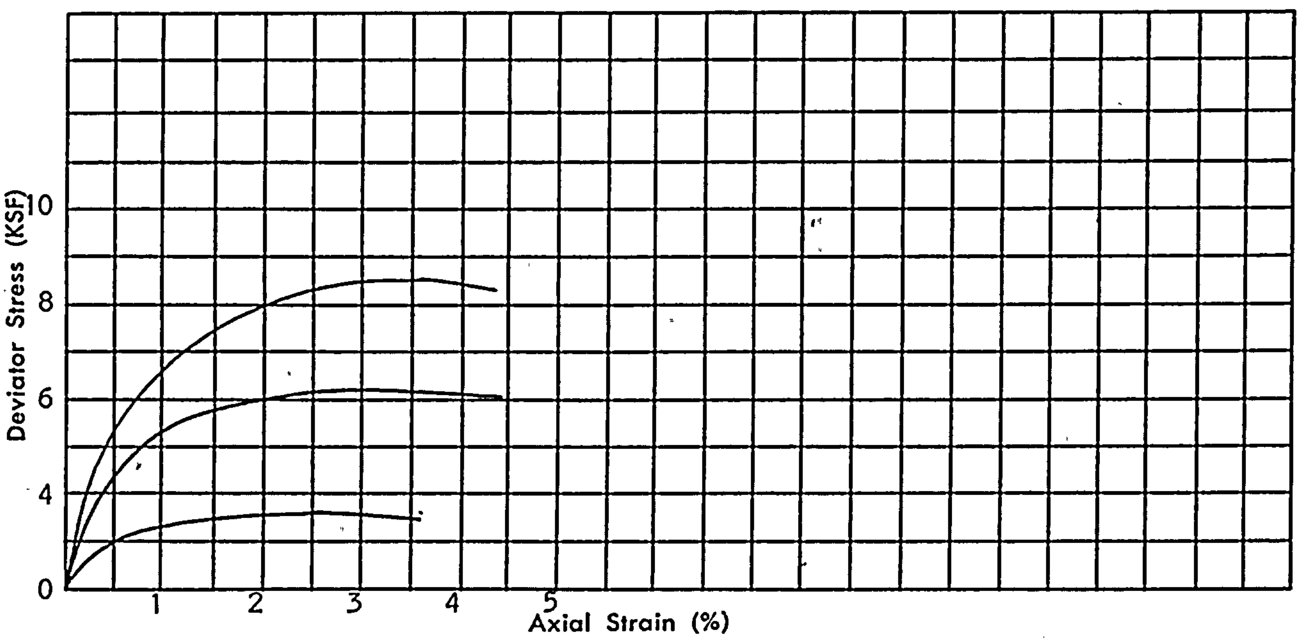
GEOLOGIC ASSOCIATES, INC.







MOHR DIAGRAMS — ϕ



STRESS — STRAIN CURVES

TRIAXIAL SHEAR TEST

SOIL DESCRIPTION	See logs	CLIENT	
		PROJECT	WNP-2
COHESION (c)	300 PSF	PROJECT NO.:	81-605
ANGLE OF INTERNAL FRICTION (ϕ)	39^{o}	BORING NO.:	CT-17
UNIT WEIGHT, PCF	105.2	SAMPLE NO.:	BAG
WATER CONTENT, %	13.3	ELEV. OR DEPTH	
SPECIFIC GRAVITY		DATE:	November 16, 1981
VOID RATIO			

Figure 3.2
Plots of Laboratory Data

GEOLOGIC ASSOCIATES, INC.

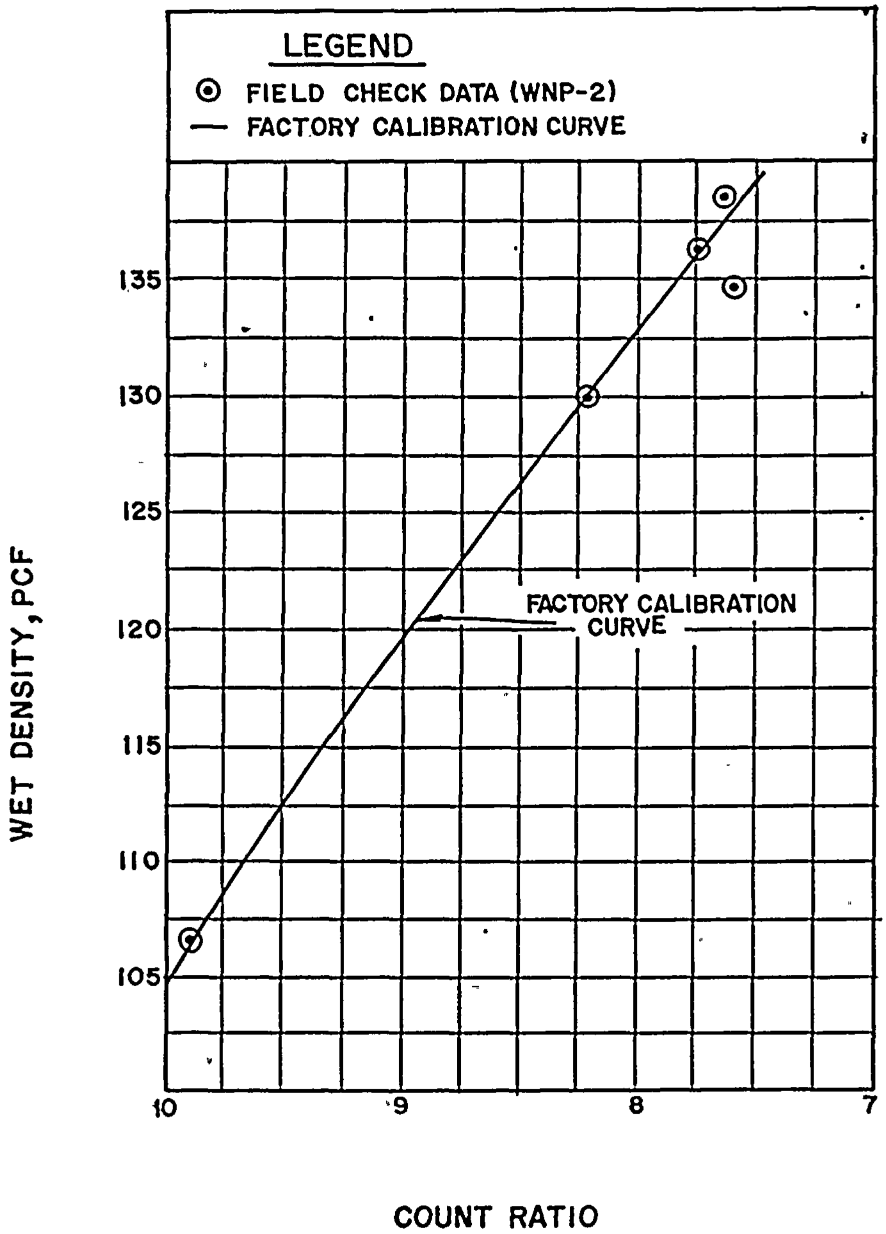


FIGURE 4.1.. — FIELD VERIFICATION OF FACTORY CALIBRATION CURVE FOR DOWNHOLE NUCLEAR DENSITY GAUGE

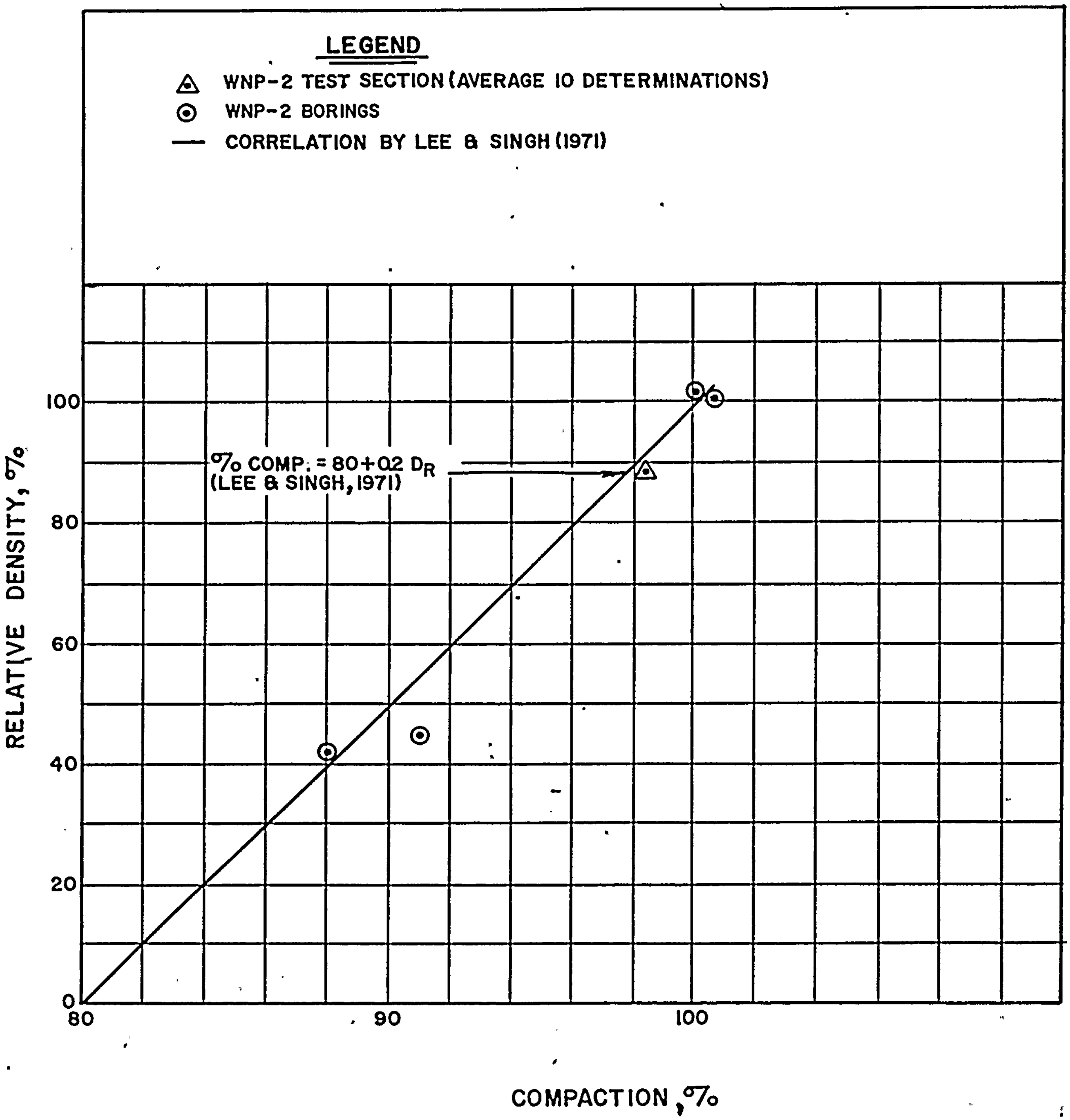


FIGURE 4.2.—RELATIONSHIP BETWEEN RELATIVE DENSITY AND PERCENT COMPACTION

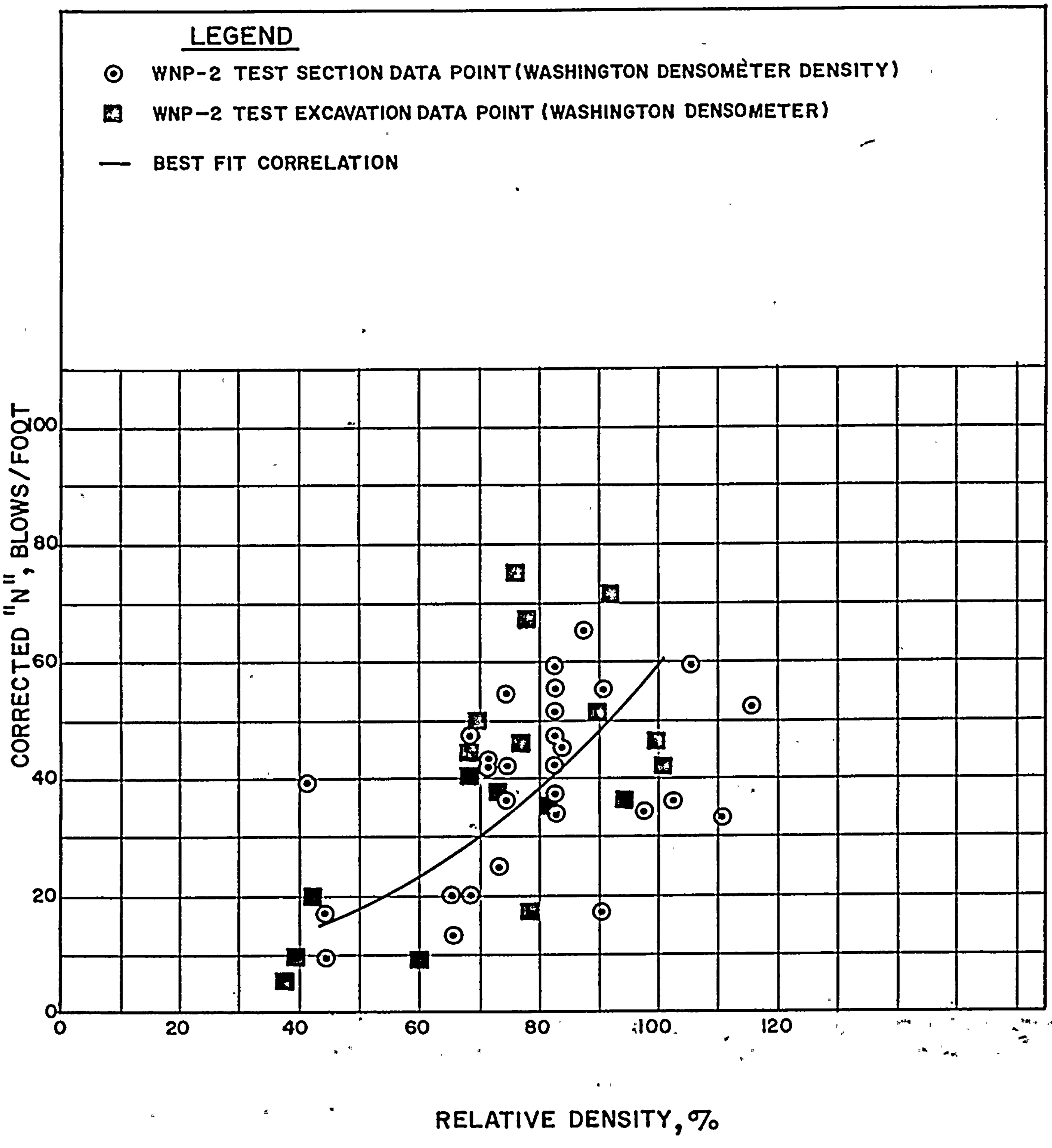


FIGURE 4.3.– RELATIONSHIP BETWEEN RELATIVE DENSITY & CORRECTED "N" (FOR DENSITIES DETERMINED WITH WASHINGTON DENSOMETER)

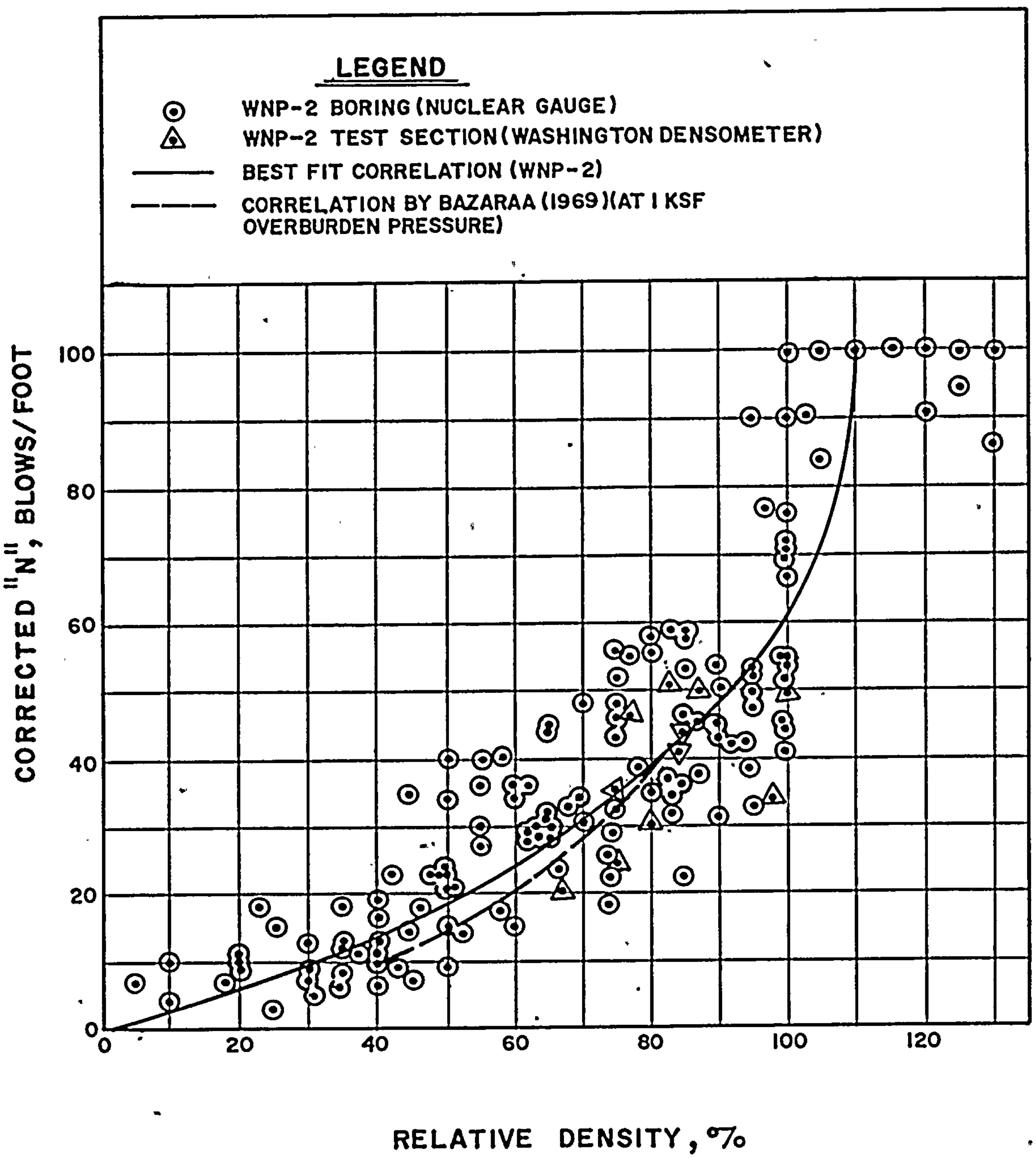


FIGURE 4.4.— RELATIONSHIP BETWEEN RELATIVE DENSITY & CORRECTED "N" (FOR DENSITIES DETERMINED WITH WASHINGTON DENSOMETER & DNDT)

[illegible]

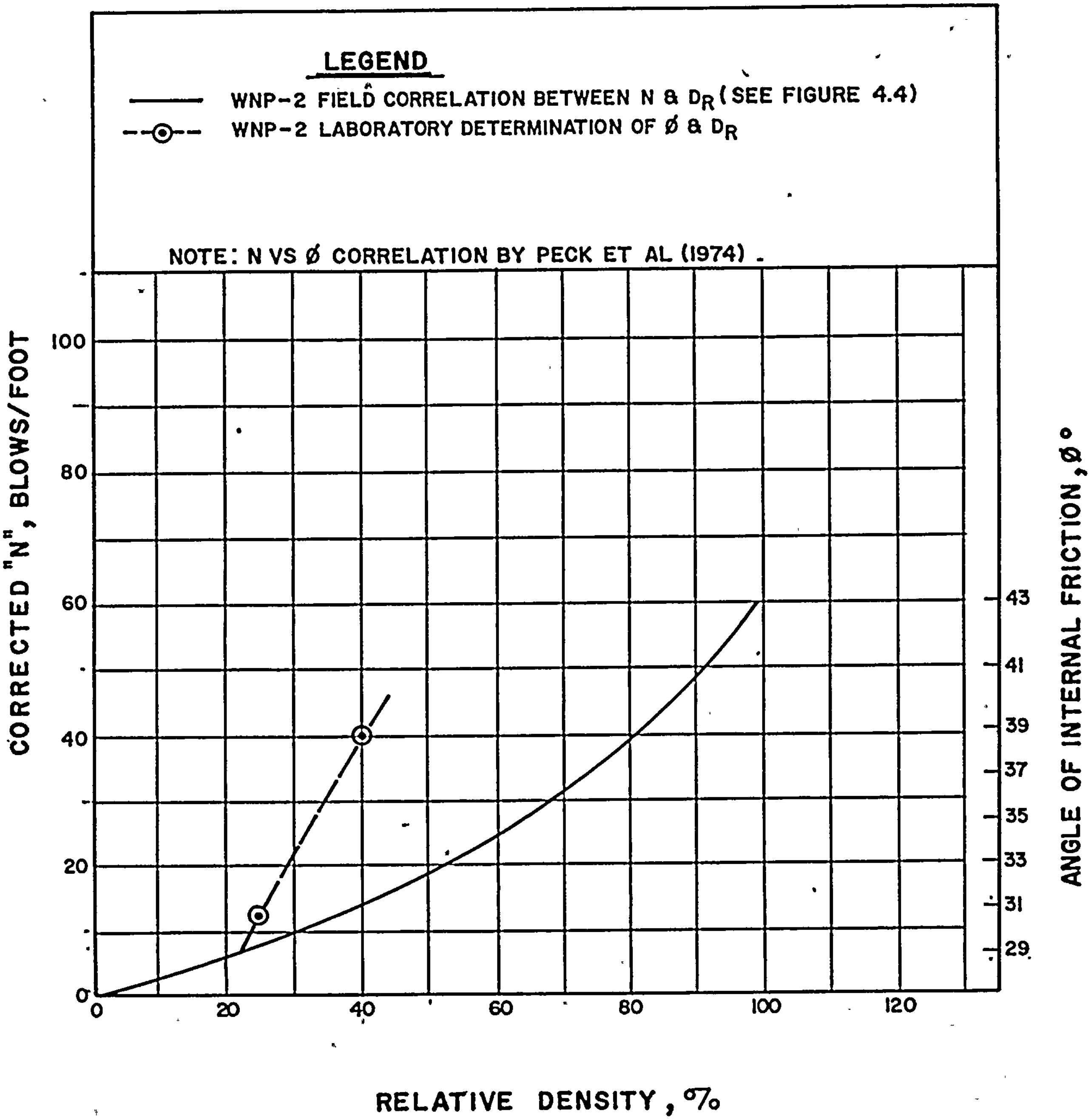


FIGURE 4.5.—CORRELATION BETWEEN N, RELATIVE DENSITY, AND Ø.

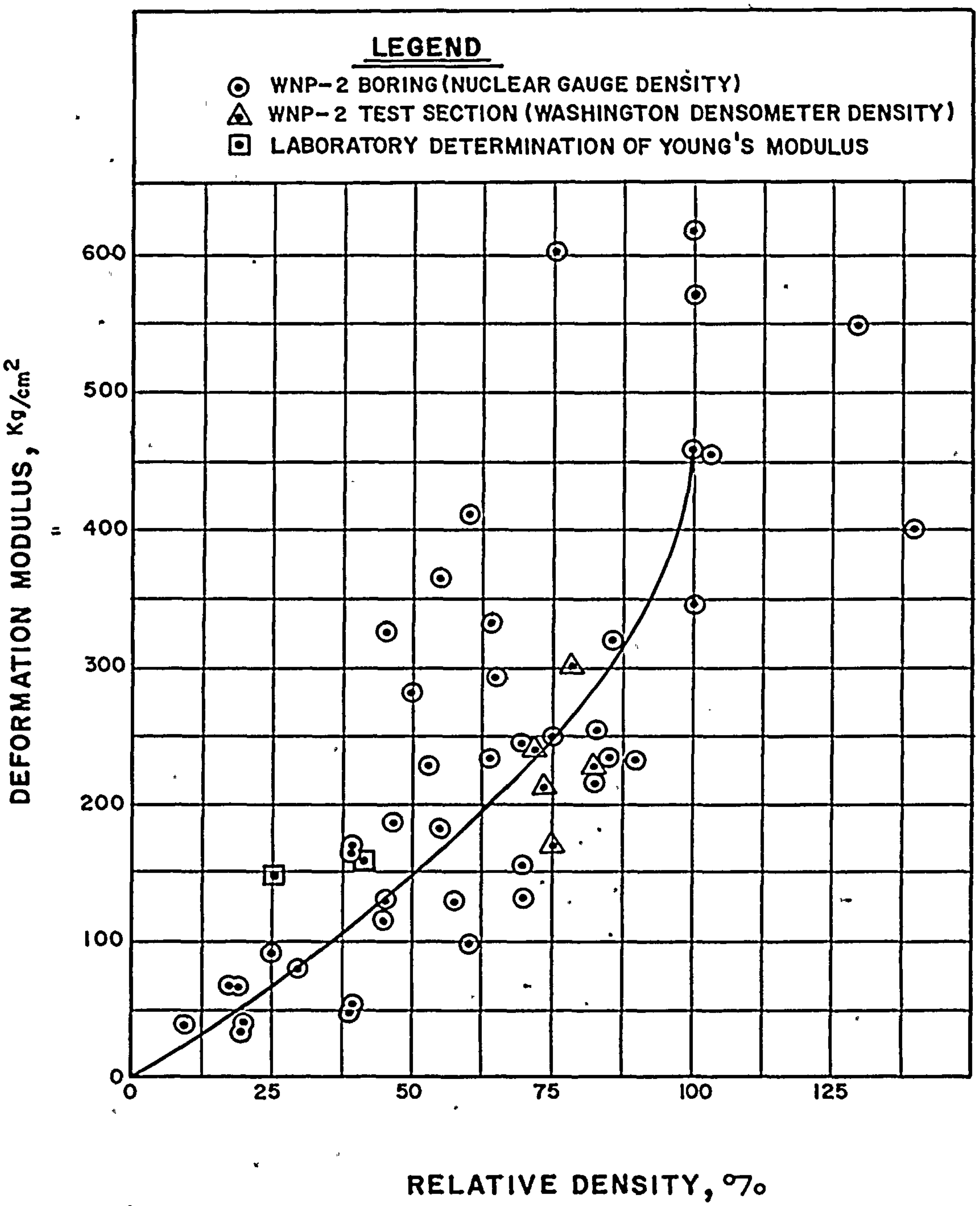
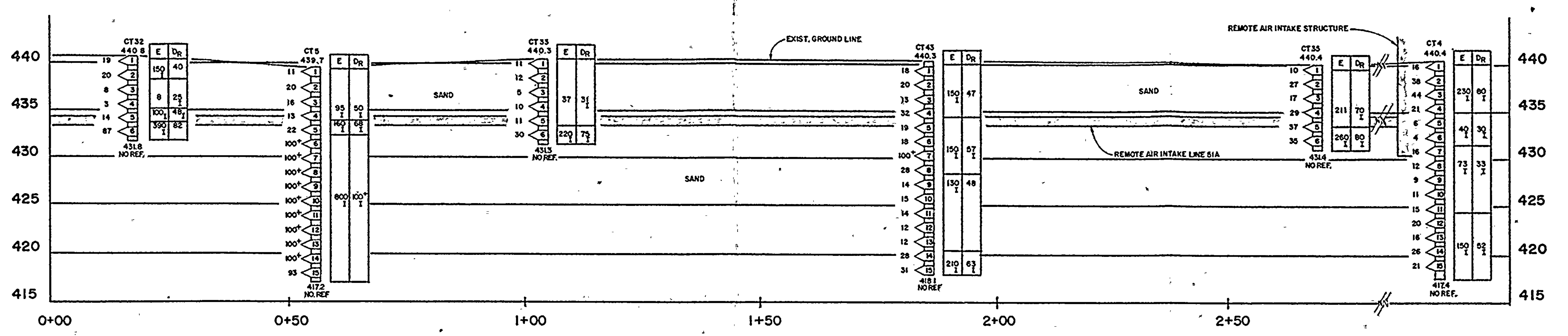
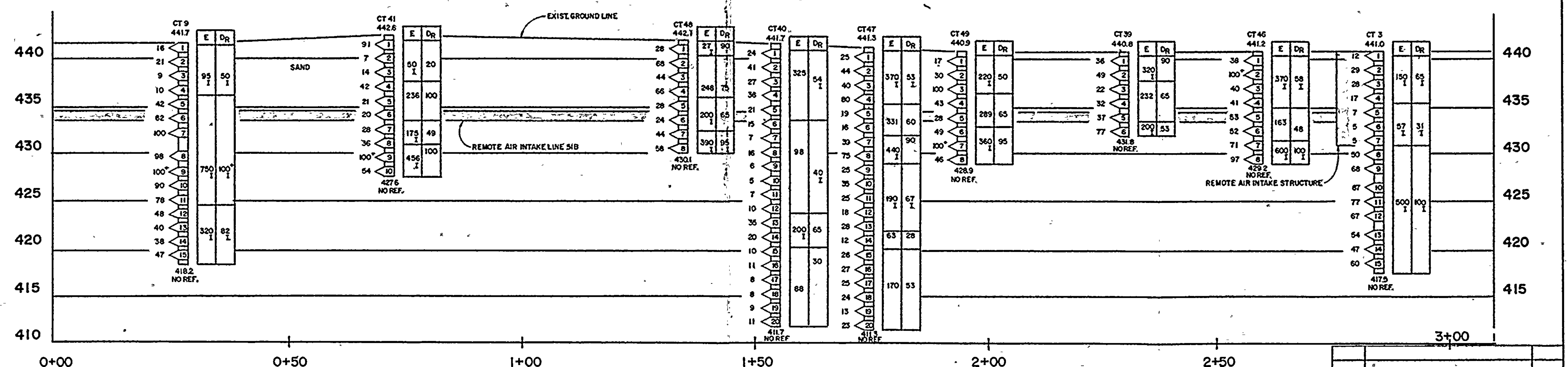


FIGURE 4.6.— RELATIVE DENSITY DERIVED FROM PMT DATA



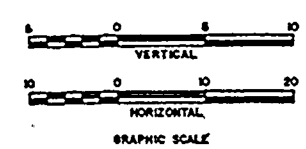
PROFILE A-A
AIR INTAKE LINE 51A
NOTE: VERTICAL EXAGGERATION = 2X

NOTE: ALL HOLES DRY UPON COMPLETION



PROFILE B-B
AIR INTAKE LINE 51B
NOTE: VERTICAL EXAGGERATION = 2X

NOTES:
1. SEE SHEET 5.2 FOR LEGEND
2. SEE APPENDIX II FOR BORING LOGS
3. SEE FIGURE 1 FOR PROFILE LOCATIONS



DATE	REVISIONS	BY

WASHINGTON PUBLIC POWER SUPPLY SYSTEM
WNP-2

PROFILES

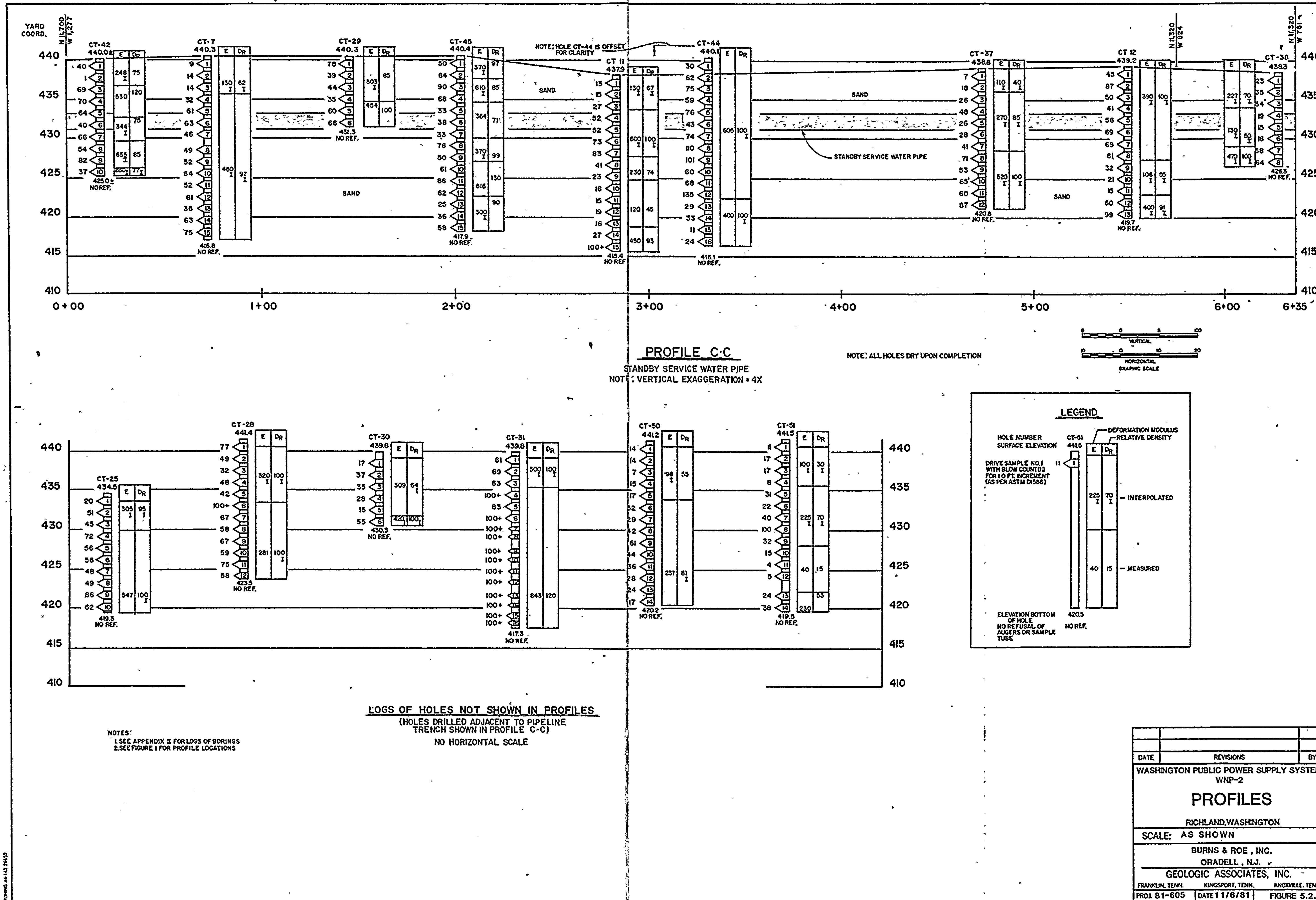
RICHLAND, WASHINGTON

SCALE: AS SHOWN

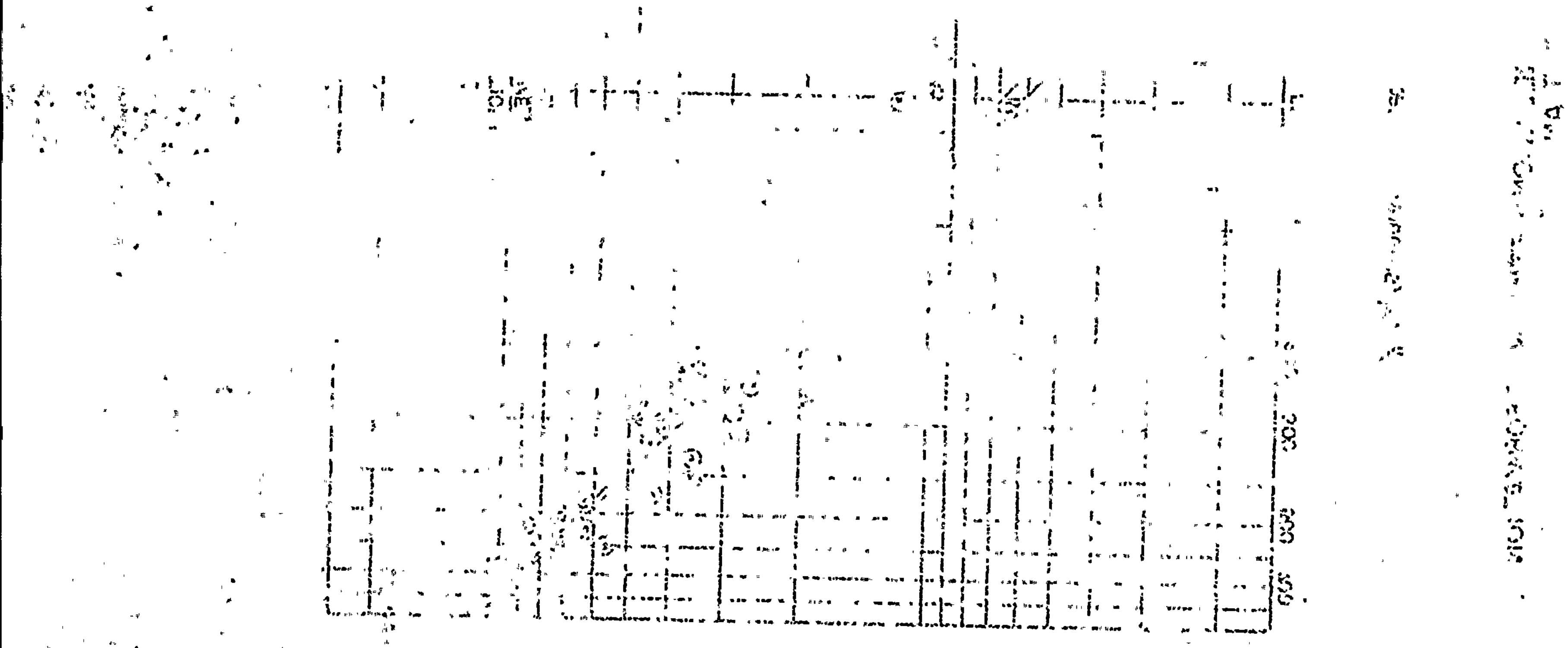
BURNS & ROE, INC.
ORADELL, N.J.

GEOLOGIC ASSOCIATES, INC.
FRANKLIN, TENN KINGSPORT, TENN KNOXVILLE, TENN

PROJ. 81-605 DATE 11/6/81 FIGURE 5.1.



YARD COORD.
N 11,700 W 1,277
N 11,320 W 824
N 11,320 W 761
CT-42 440.0±
CT-7 440.3
CT-29 440.3
CT-45 440.4
NOTE: HOLE CT-44 IS OFFSET FOR CLARITY
CT 11 437.9
CT-44 440.1
CT-37 438.8
CT 12 439.2
CT-38 438.3
SAND
STANDBY SERVICE WATER PIPE
NO REF.
0+00
1+00
2+00
3+00
4+00
5+00
6+00
6+35
PROFILE C-C
STANDBY SERVICE WATER PIPE
NOTE: VERTICAL EXAGGERATION = 4X
NOTE: ALL HOLES DRY UPON COMPLETION
VERTICAL
HORIZONTAL
GRAPHIC SCALE
CT-25 434.5
CT-28 441.4
CT-30 439.8
CT-31 439.8
CT-50 441.2
CT-51 441.5
LEGEND
HOLE NUMBER
SURFACE ELEVATION
DEFORMATION MODULUS
RELATIVE DENSITY
DRIVE SAMPLE NO. 1 WITH BLOW COUNT(N) FOR 1.0 FT. INCREMENT (AS PER ASTM D1586)
INTERPOLATED
MEASURED
ELEVATION BOTTOM OF HOLE
NO REFUSAL OF AUGERS OR SAMPLE TUBE
LOGS OF HOLES NOT SHOWN IN PROFILES
(HOLES DRILLED ADJACENT TO PIPELINE TRENCH SHOWN IN PROFILE C-C)
NO HORIZONTAL SCALE
NOTES:
1. SEE APPENDIX II FOR LOGS OF BORINGS
2. SEE FIGURE 1 FOR PROFILE LOCATIONS
DATE
REVISIONS
BY
WASHINGTON PUBLIC POWER SUPPLY SYSTEM
WNP-2
PROFILES
RICHLAND, WASHINGTON
SCALE: AS SHOWN
BURNS & ROE, INC.
ORADELL, N.J.
GEOLOGIC ASSOCIATES, INC.
FRANKLIN, TENN. KINGSPORT, TENN. KNOXVILLE, TENN.
PROJ. 81-605
DATE 11/6/81
FIGURE 5.2.



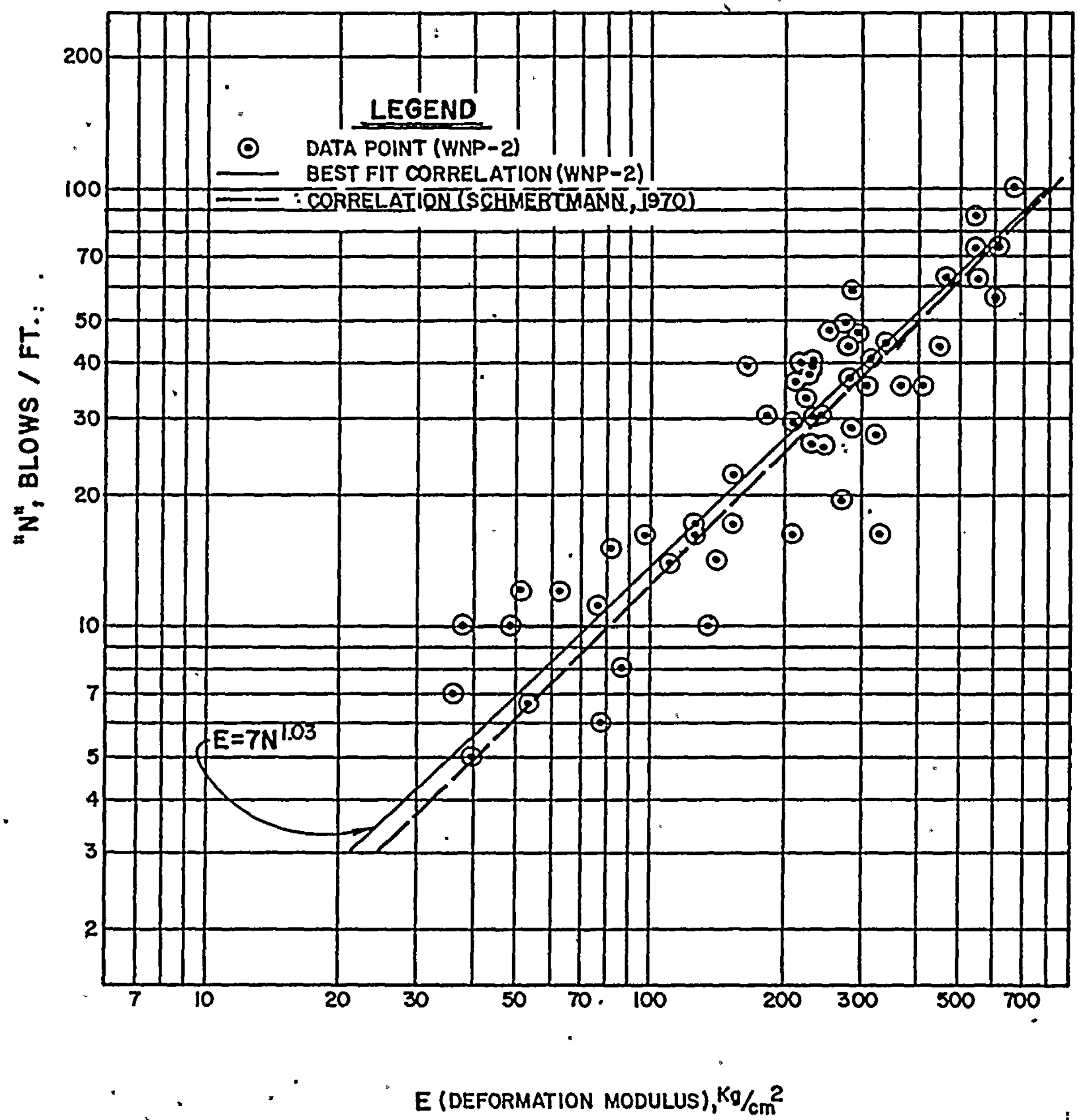
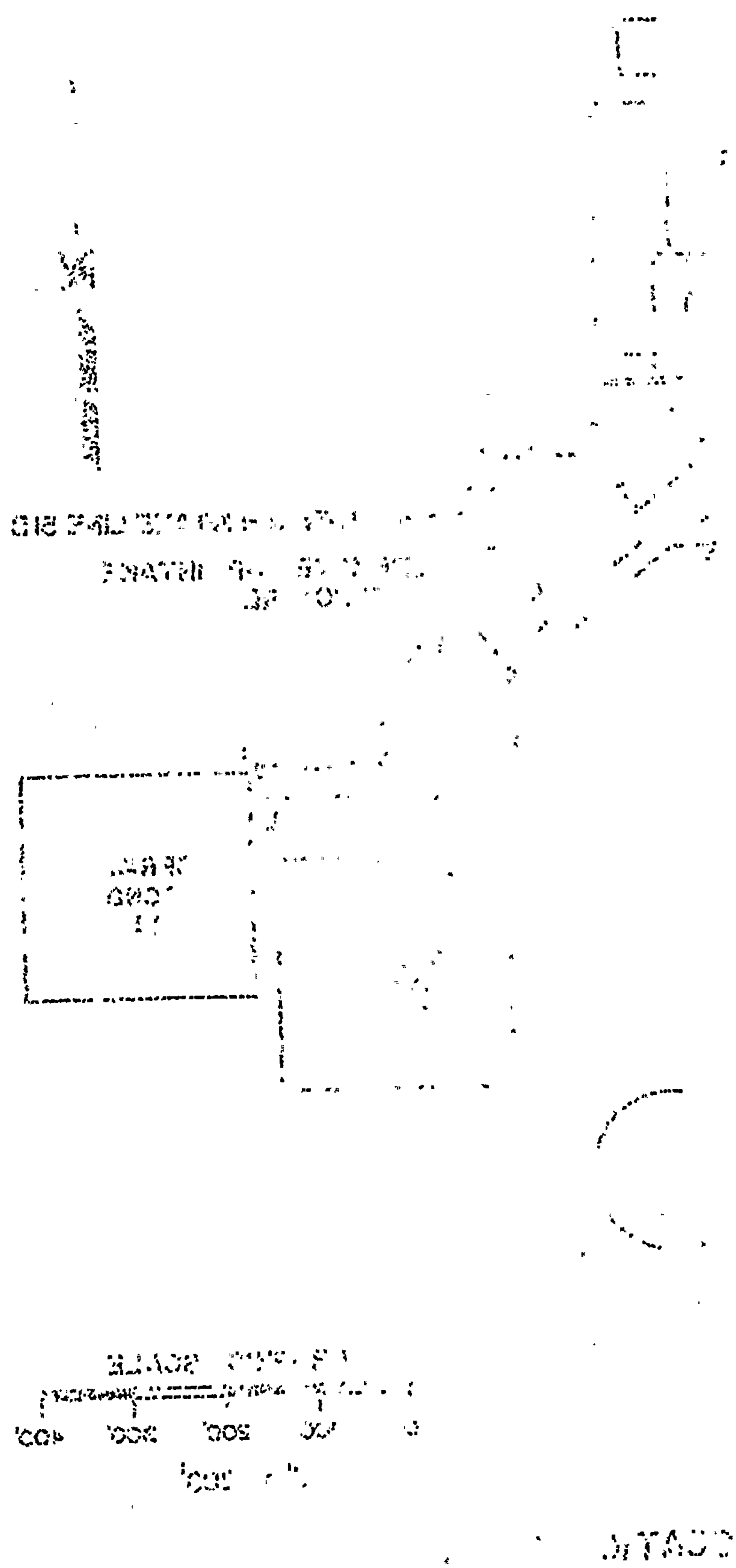


FIGURE 4.7.— CORRELATION BETWEEN DEFORMATION MODULUS FROM PMT AND SPT



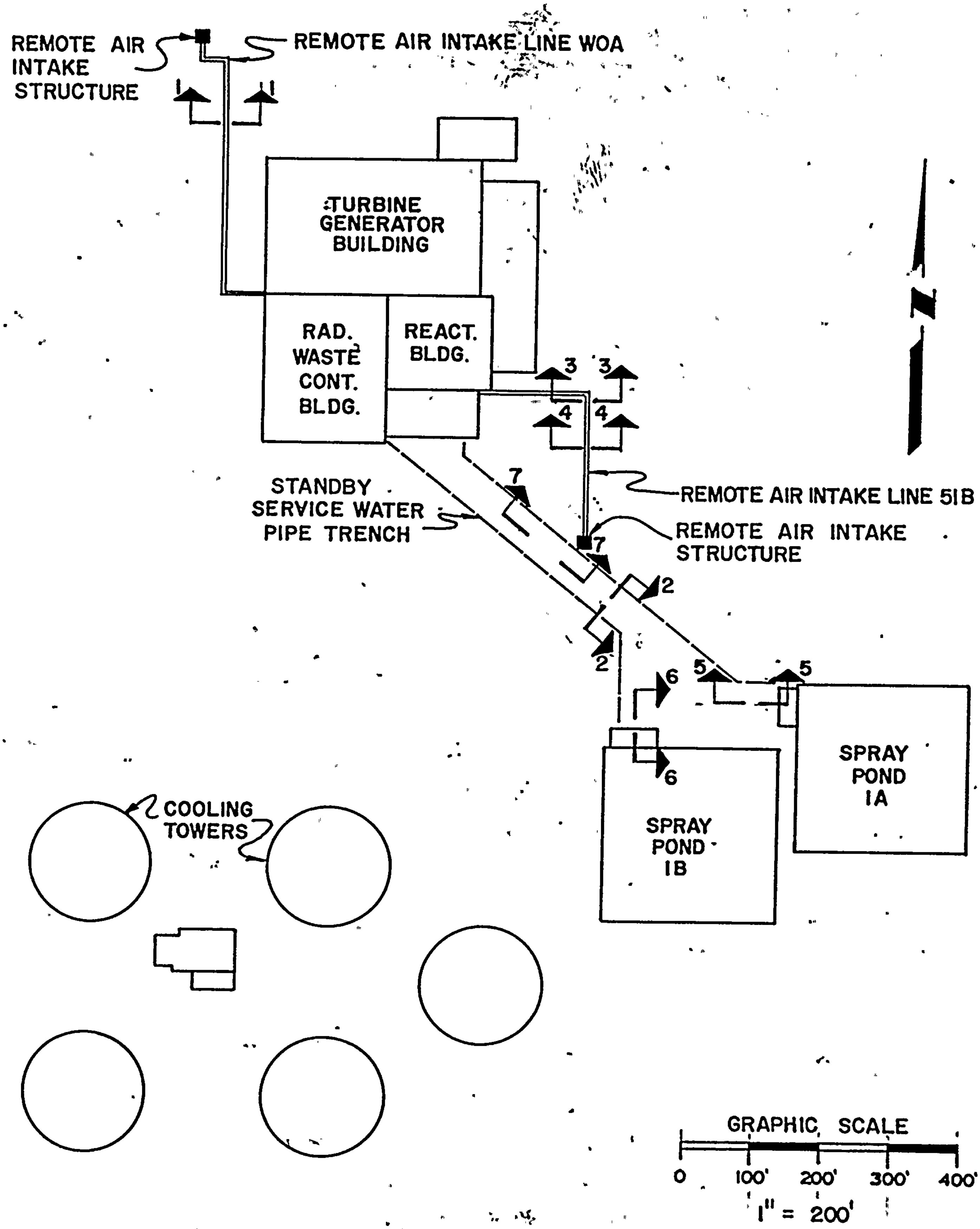
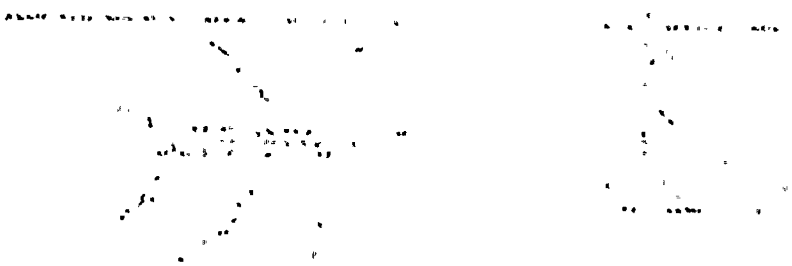
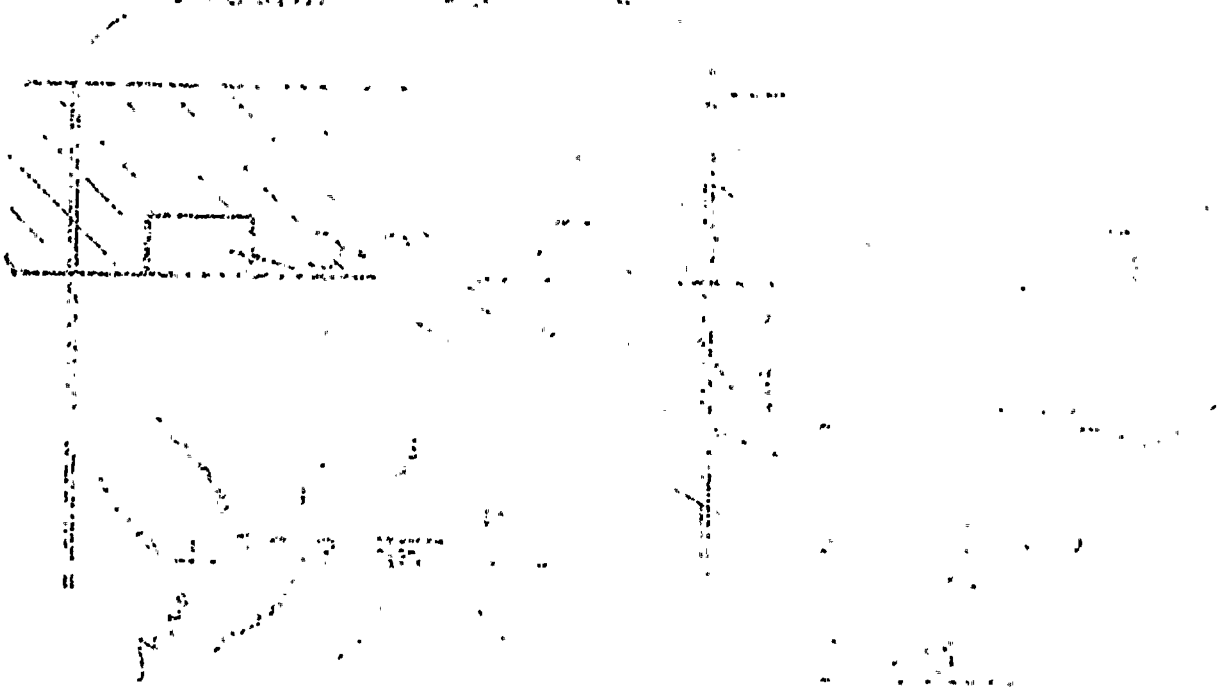


FIGURE 6.1 LOCATION PLAN





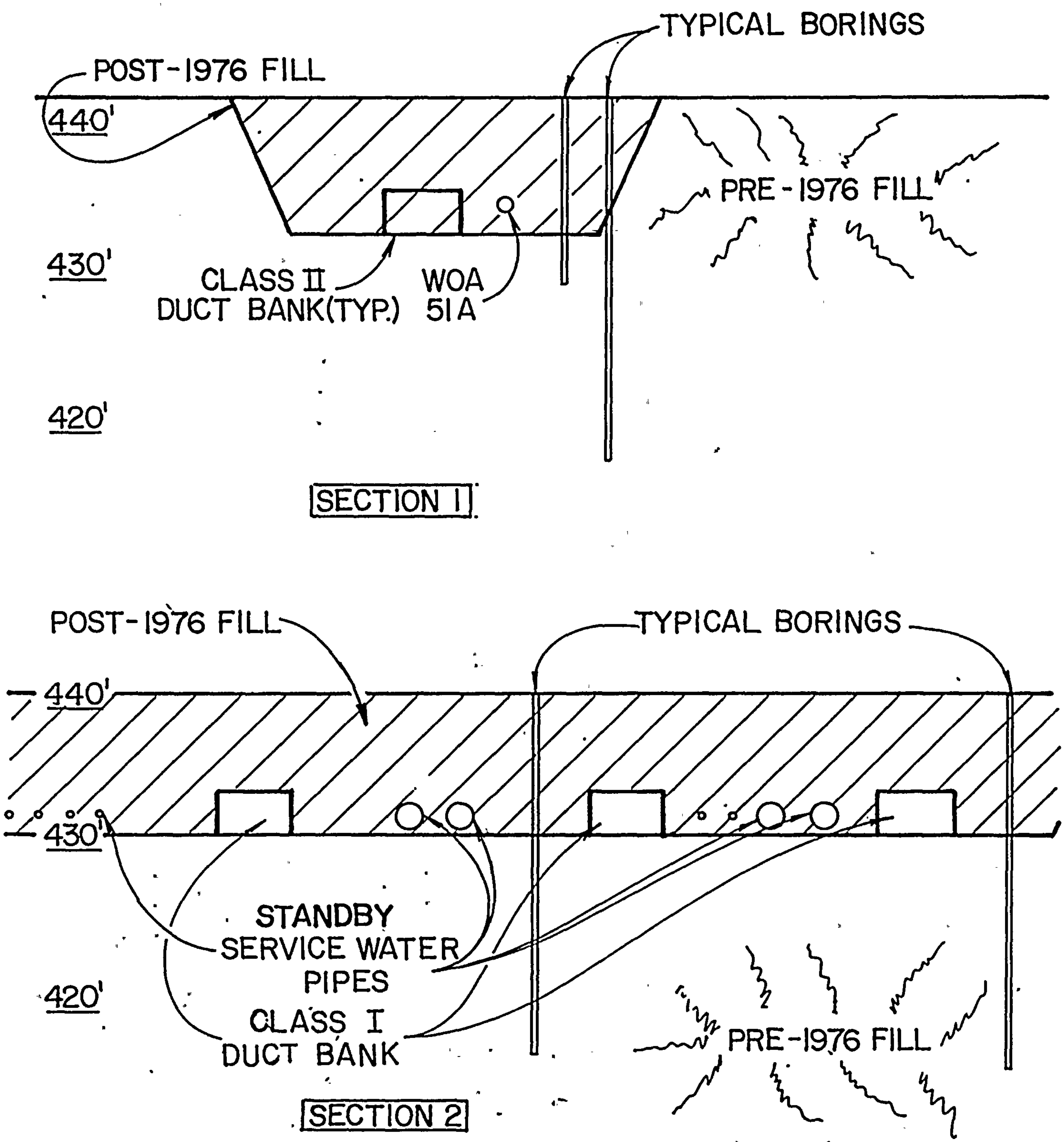
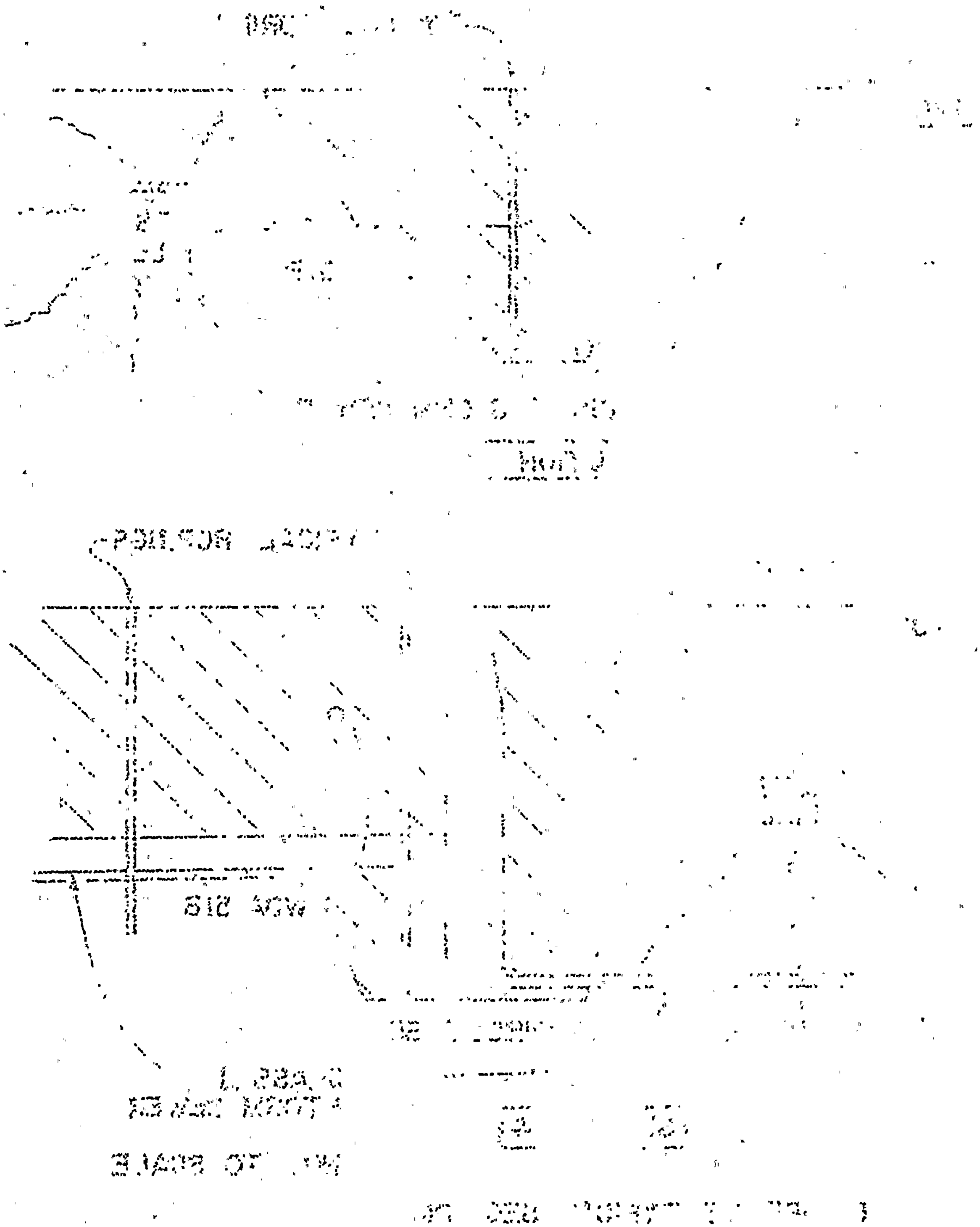
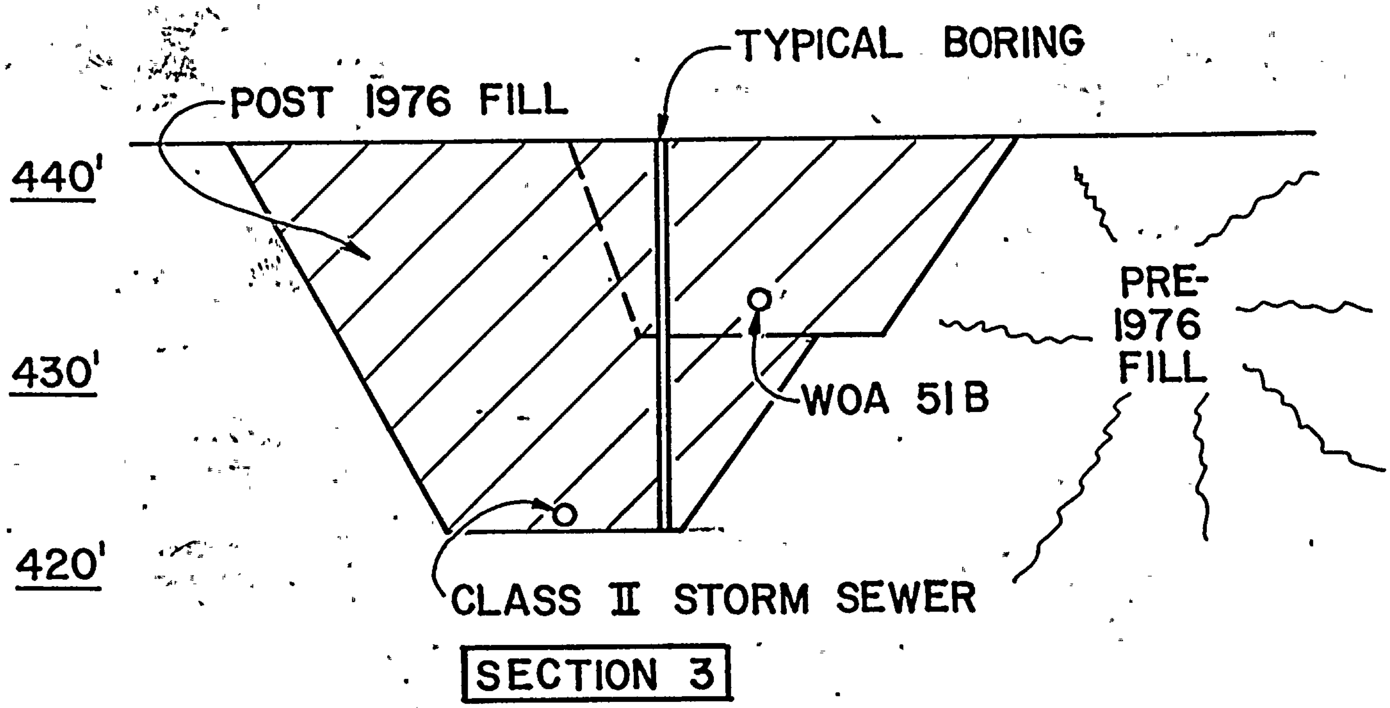
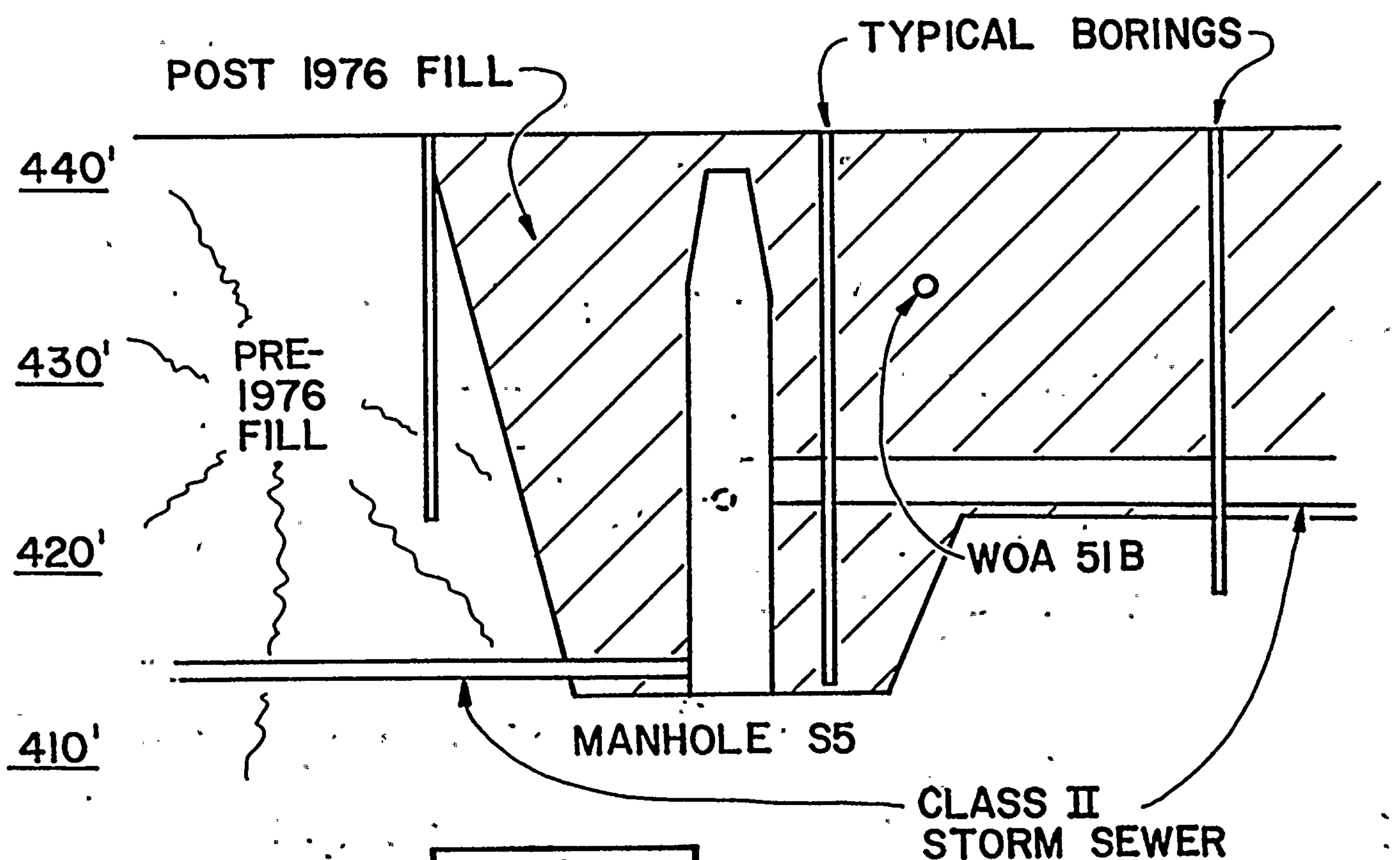


FIGURE 6.2 TYPICAL SECTIONS

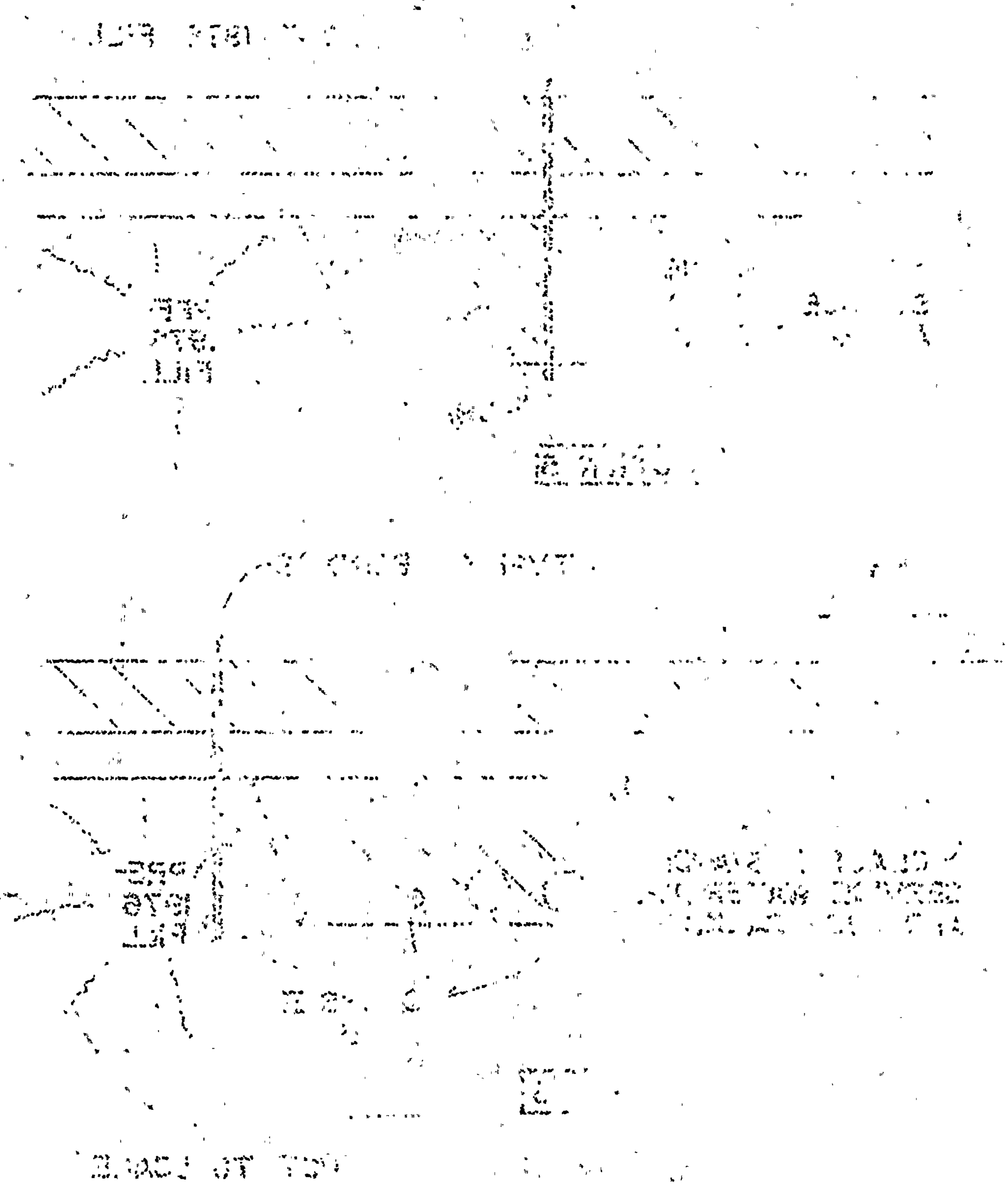






NOT TO SCALE

FIGURE 6.3 TYPICAL SECTIONS



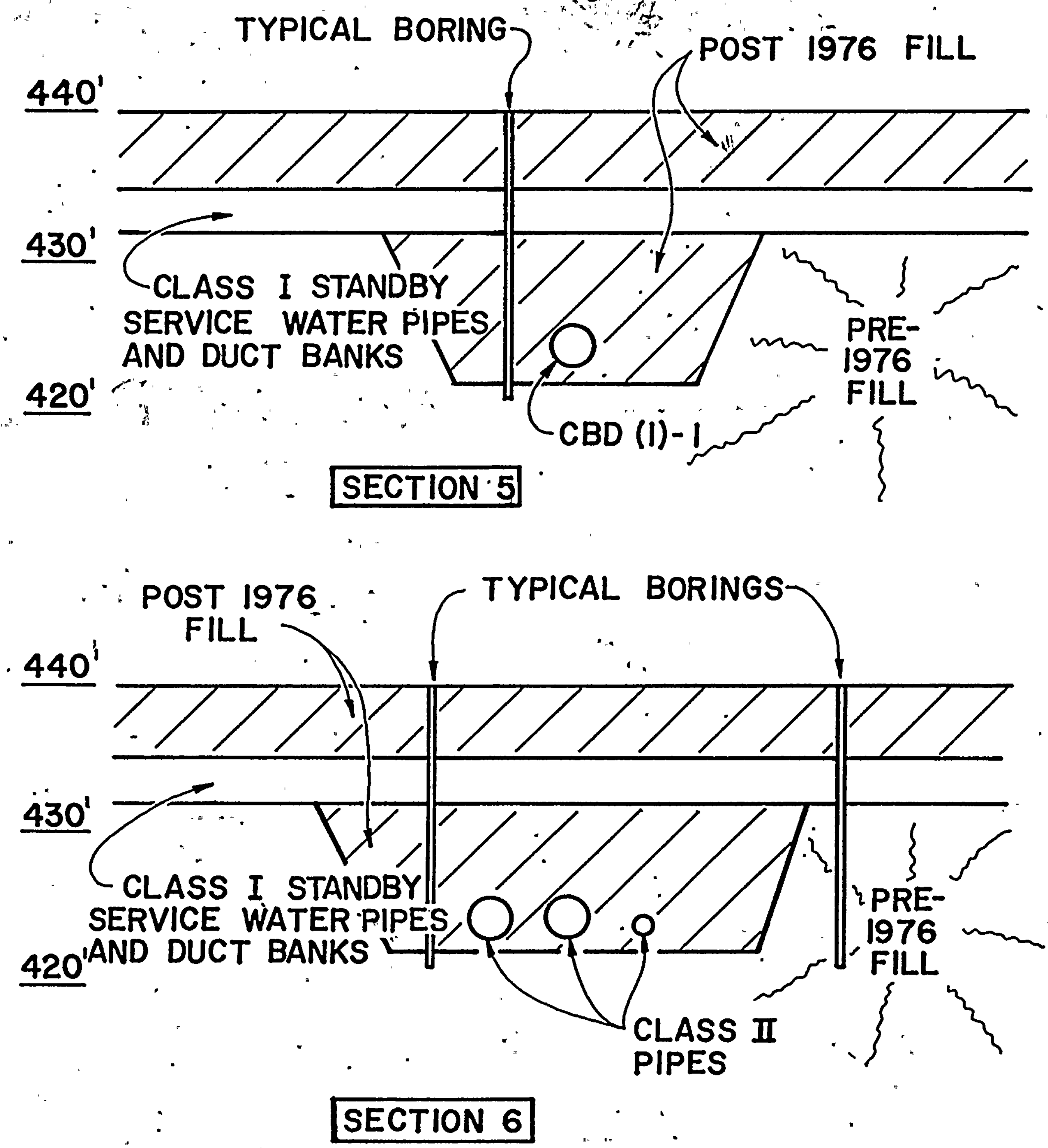


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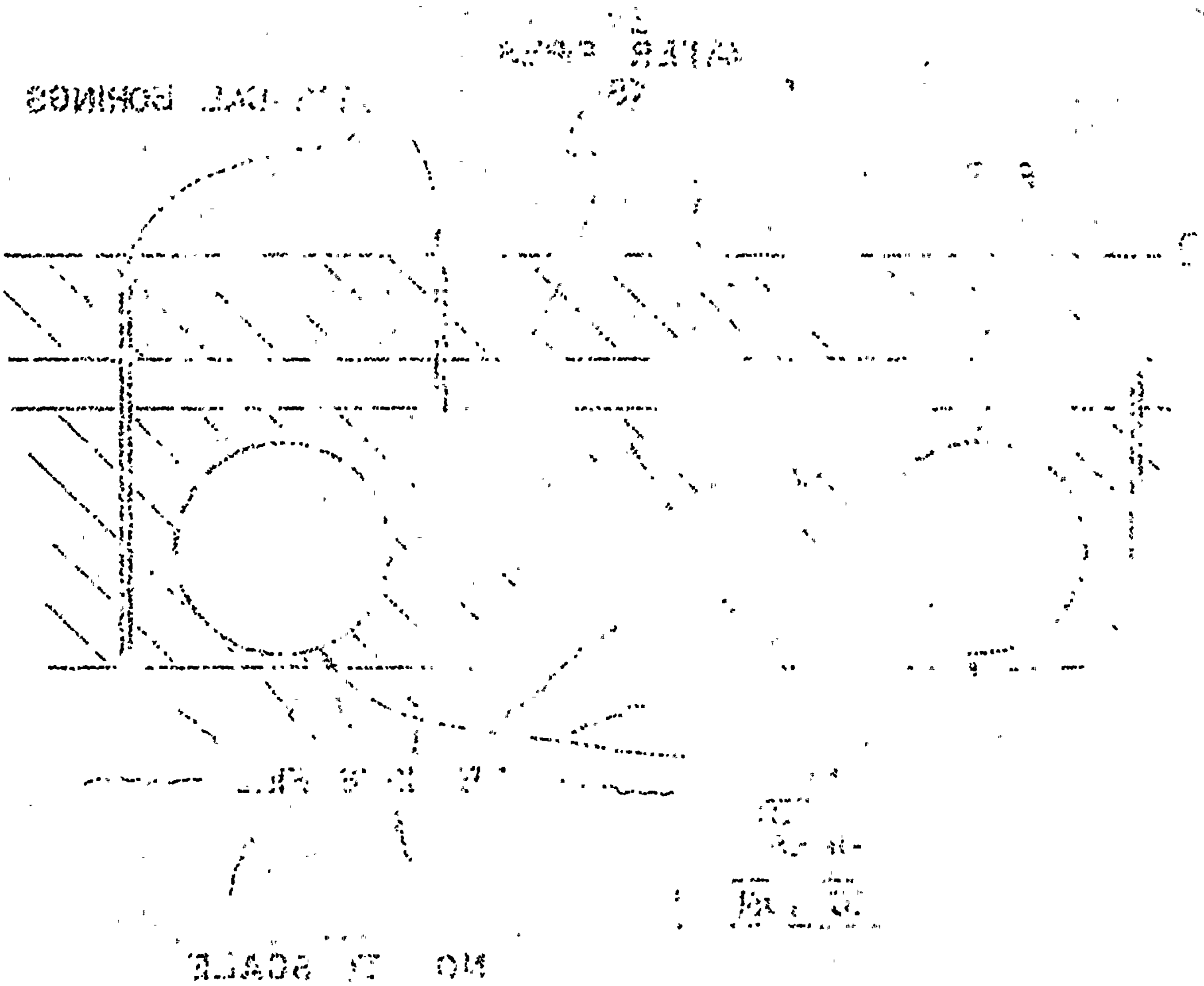
FIGURE 6.4 TYPICAL SECTIONS



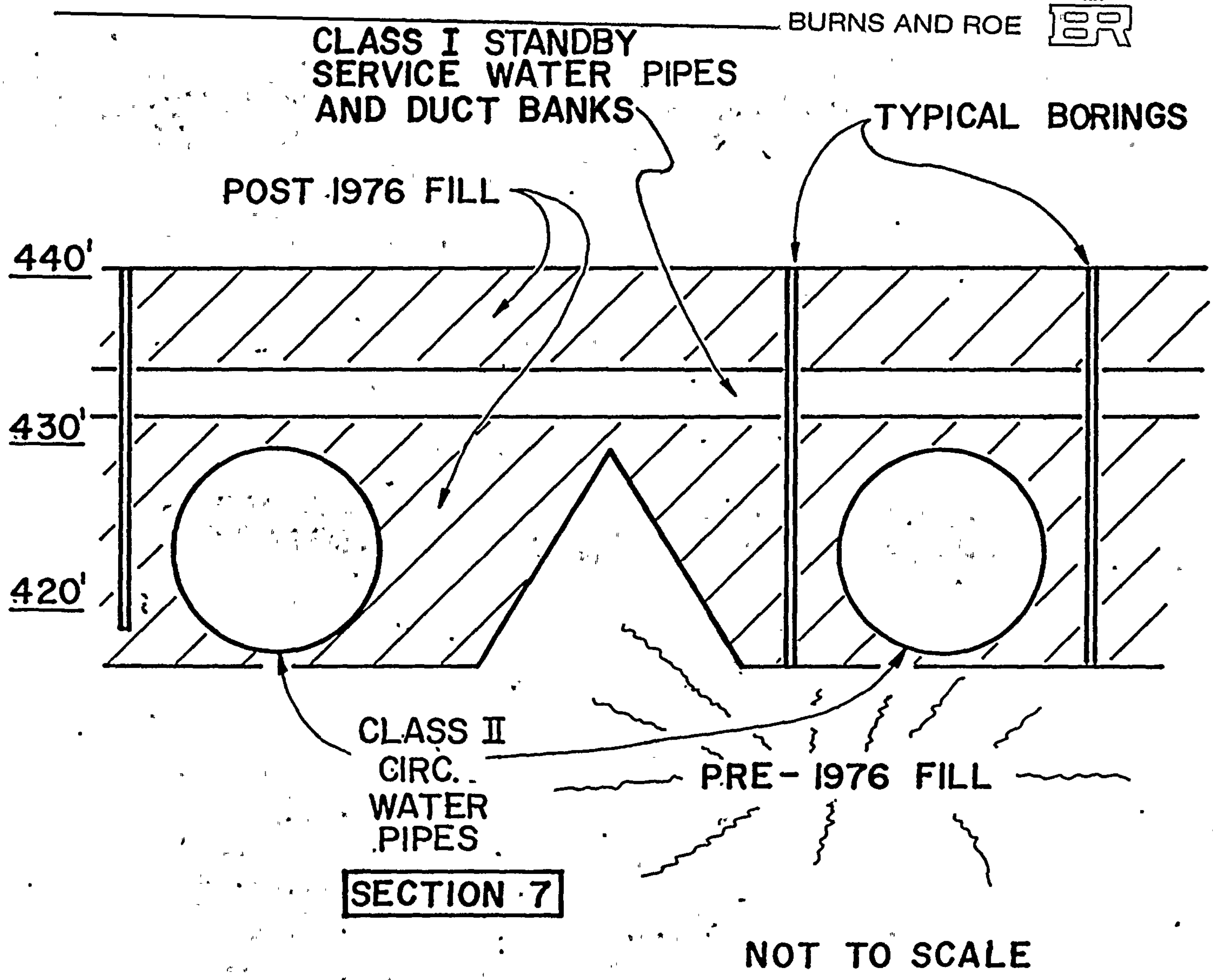
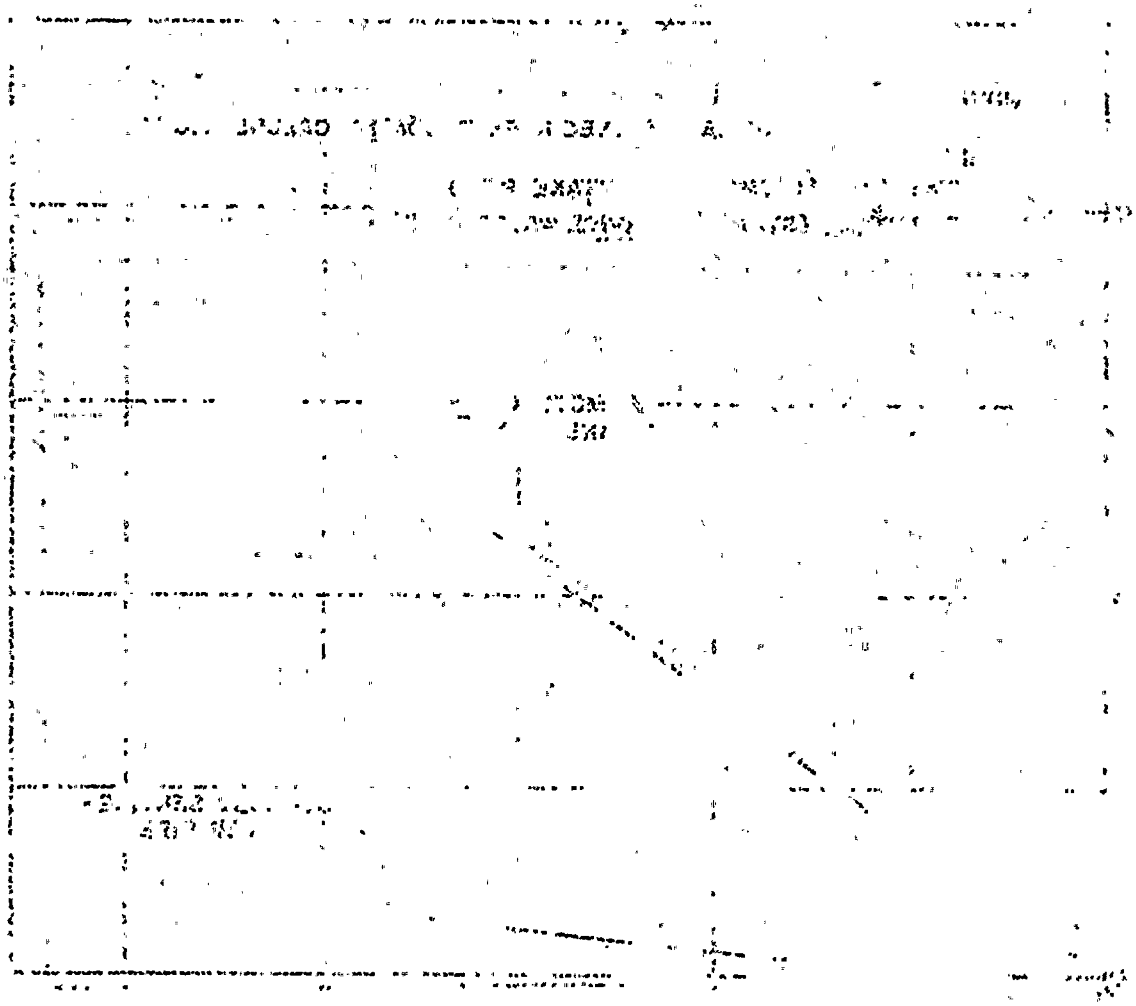


FIGURE 6.5 TYPICAL SECTIONS



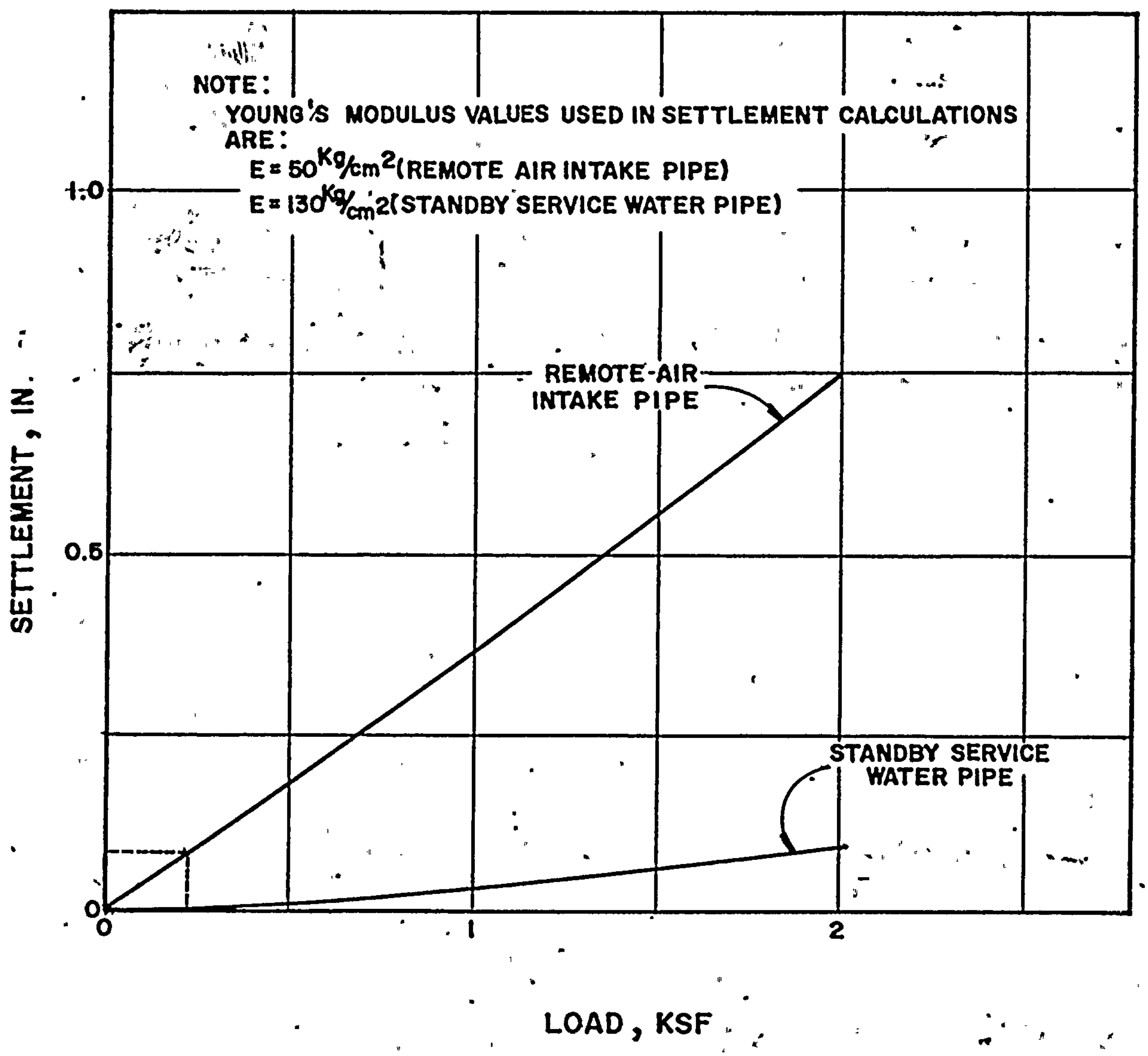
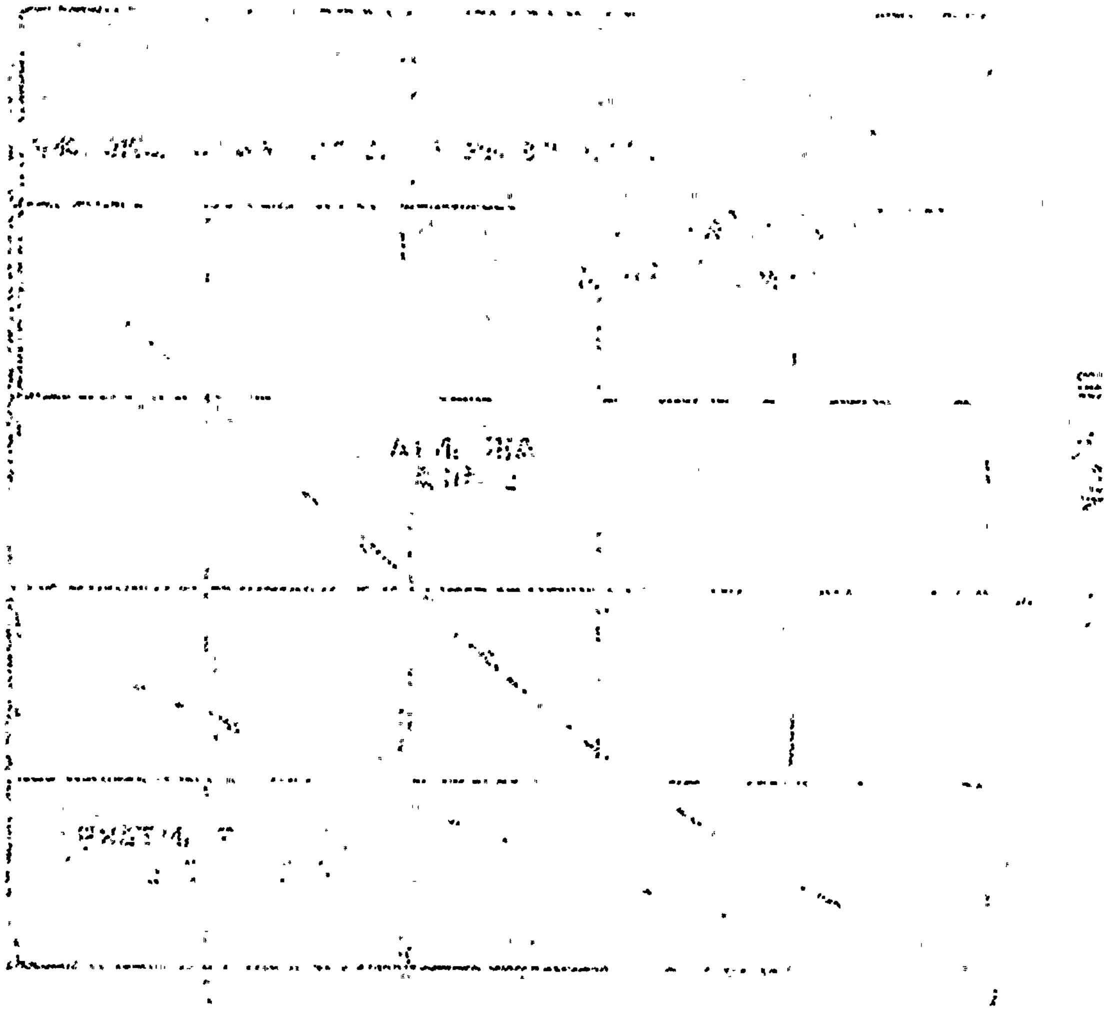


FIGURE 7.1 ESTIMATED STATIC SETTLEMENT-LOAD CURVES FOR RESPECTIVE STRUCTURES



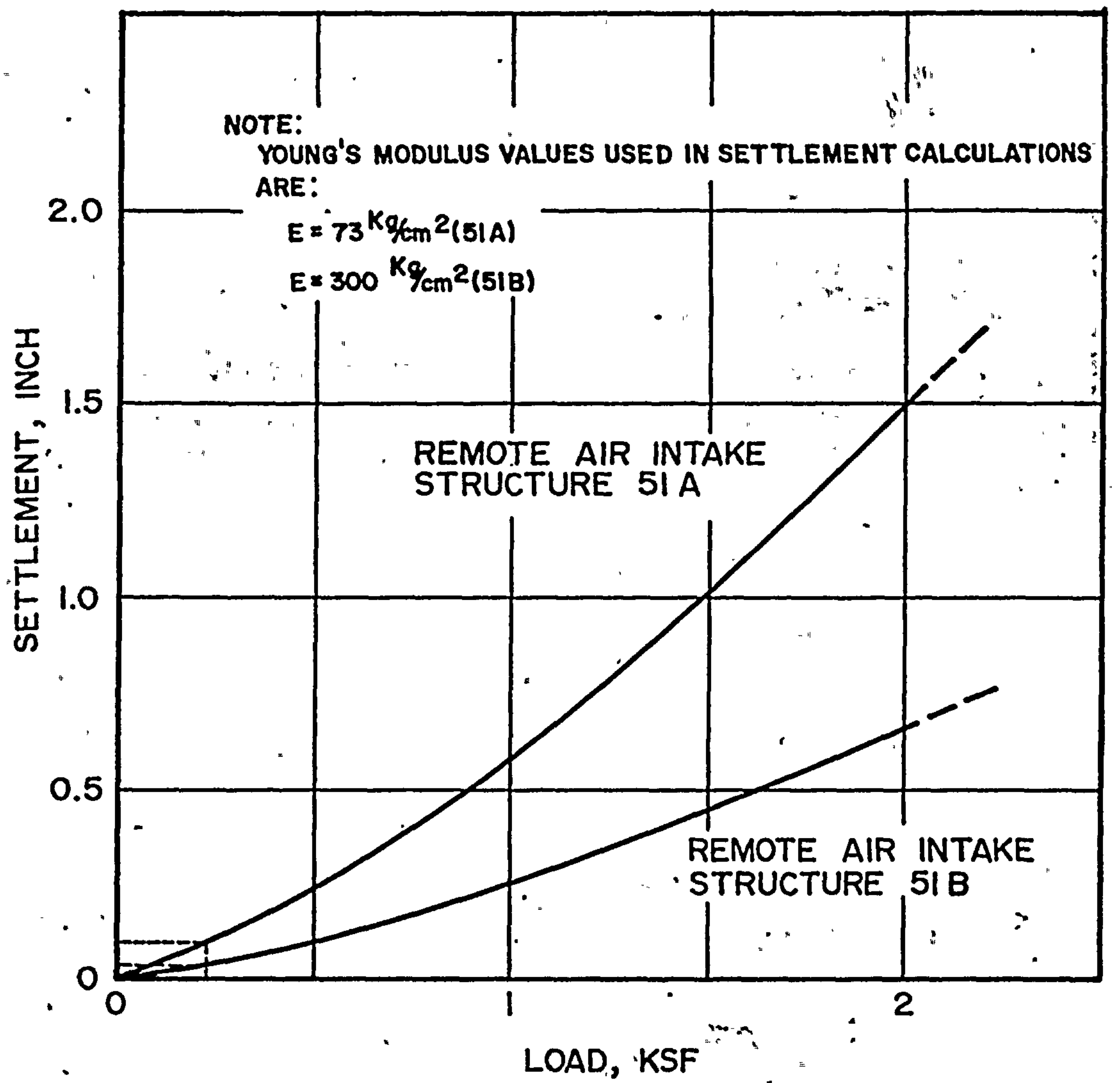


FIGURE 7.2 ESTIMATED STATIC SETTLEMENT-LOAD CURVES FOR RESPECTIVE STRUCTURES.

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