DR-22-V-01

Soils Design Report - Volume I
Plant Site Near Surface

DR-22-V-02

Soils Design Report - Volume II
Access Roads And
Railroad Soils Report

The Title I revisions for these documents
were not available at this time. They will
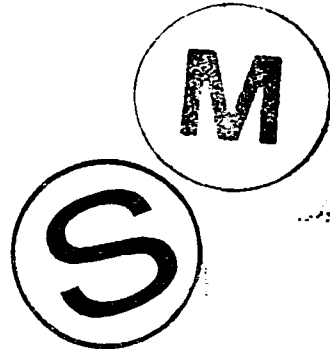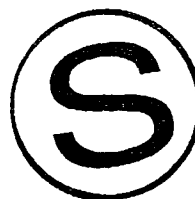be sent later for inclusion in this report.

TABLE OF CONTENTS

(S)

NO.	DATE	DESCRIPTION	BY	CHK'D	DESIGN SUPV	ENG'R	PEM	DOE
⚠	8-31-79	REV. 6.2.1, 6.2.5, 7.3, 10.3, 10.4 & Figs. 4, 5 & 6 REISSUED FOR TITLE I			WRF			
⚠	8-9-79	ISSUED FOR TITLE I						
⚠	7-5-79	ISSUED FOR CLIENT APPROVAL						

(1-79)WIPP ENG1-J

U.S. DEPARTMENT OF ENERGY
WASTE ISOLATION PILOT PLANT
SOILS DESIGN REPORT – VOLUME I
PLANT SITE NEAR SURFACE STRUCTURES

BECHTEL

JOB NO.	12484 – 2	
DOCUMENT NO.		REV.
	DR-22-V-01	2

WIPP PROJECT

SOILS DESIGN REPORT
VOLUME I

PLANT SITE NEAR SURFACE STRUCTURES

10. FOUNDATION CONSTRUCTION

List of Tables

List of Figures

(1-79) WIPP ENG-1-2

List of Figures

1. <u>INTRODUCTION</u>

Volume I of the soils design report presents the results of foundation studies for the surface structures at the Waste Isolation Pilot Plant (WIPP) located in southern New Mexico about 26 miles east of Carlsbad in eastern Eddy County. The results of studies for the access roads and railroad will be presented in Volume II of the soils design report.

To develop information for this report, a field exploration program, laboratory testing and engineering analyses were performed during November 1978 through May 1979.

The locations of the borings and test pits used in the evaluations are shown on Figure 1. The logs of all borings and test pits together with the results of all field and laboratory tests are contained in the series of reports by Sergent, Hauskins & Beckwith which are in Reference 16 to this report. The soil test results are summarized in Table 1.

2. <u>SCOPE OF WORK</u>

This report summarizes the soil and foundation investigations made to evaluate the near surface conditions at the WIPP site. The foundation investigations consisted of drilling, excavating and sampling the near surface soils and rock, conducting field and laboratory tests, and performing engineering analyses to develop foundation recommendations for the near surface structures.

The results of the field exploration and laboratory testing programs, site conditions, foundation evaluation as well as recommendations for earthwork construction are provided in the report.

3. <u>SITE GEOLOGY</u>

The WIPP site is located near the eastern edge of the Pecos Valley section of the Southern Great Plains physiographic province. The site lies on a caliche and sand covered drainage divide separating two major solution-erosional features, Nash Draw four miles to the west and San Simon Swale eight miles to the east. Surface runoff from the site drains west into Nash Draw, eventually reaching the Pecos River, about 1 miles southwest of the site.

Recent windblown sand and partly stabilized sand dunes blanket most of the site area. The sand is believed to have been moved westward from the High Plains, where the inferred source material, the sandy Ogallala Formation, is abundant. A hard, resistant duricrust or caliche (Mescalero Caliche) is typically

present beneath the sand blanket. The caliche formed near the surface through capillary rise of carbonate-laden water. The caliche is an accumulation of calcareous and clastic material cemented with calcite and silica. Its resistance to weathering in the dry climate has protected the more erodible underlying strata from exposure.

The caliche has developed upon the surface of the underlying bedrock called the Gatuna Formation. The Gatuna Formation is the only Pleistocene deposit at the WIPP site assigned a formal stratigraphic name. The Gatuna Formation consists of a fine-grained, reddish-brown sandstone with some conglomerate lenses. The Gatuna Formation is tentatively assigned a Kansan age and the caliche formed upon it a Yarmouthian (interglacial) age; that is, the caliche formed starting about 500,000 years ago.

The WIPP site is within seismic Zone 1, according to the Uniform Building Code, 1976. Within this zone, seismic risk is defined such that minor damage may be expected; distant earthquakes may cause damage to structures with fundamental periods greater than 1.0 second; and earthquake parameters typical of those associated with intensities of V and VI on the Modified Mercalli scale are appropriate. The results of seismic analysis of the site by Sandia (1) have shown that the Design Basis Earthquake acceleration is less than or equal to 0.06g. For additional conservatism, however, a Design Basis Earthquake acceleration of 0.1g is used for foundation evaluation as concluded in the Seismic Evaluation Report (2).

4.   EXPLORATION

   4.1   General

The near surface exploration program has been developed by Bechtel and carried out by Sergent, Hauskins and Beckwith. Bechtel soil engineers and geologists observed drilling, sampling and testing operations. The purpose of the exploration was to establish the near surface conditions at the site and to determine static and dynamic properties of the soil and rock to develop foundation design requirements for the surface structures. Both the field and laboratory work were done under a Quality Assurance Program in conformance with the applicable requirements of ANSI N45.2 as modified by NRC Regulatory Guide 1.28.

The near-surface exploration program for the plant site was conducted in two stages. Stage I included drilling, sampling and testing of 52 shallow borings numbered B-1 through B-24 and B-26 through B-53 and
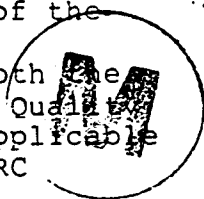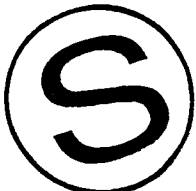
one deep boring numbered B-54. Nine shallow borings were drilled for a seismic cross-hole and downhole survey which was conducted by Harding Lawson and Associates. In addition, a seismic refraction survey was made in shallow holes along grid lines, permeability tests were performed and 11 observation wells were installed at various depths in selected borings for ground water studies at the site.

Stage II included excavation, sampling and testing of 5 test pits numbered TP-1 through TP-5 at the plant site. Plate load tests were made at selected depths in test pits TP-3 and TP-5. In addition, electrical resistivity measurements were conducted at six locations numbered R-1 through R-6 to determine the corrosion potential in the upper materials.

The locations of all the borings, observation wells, seismic refraction survey lines, test pits and electrical resistivity tests are shown on Figure 1. Geologic profiles showing the different strata including some of the significant engineering properties are shown on Figures 5 and 6.

4.2   Shallow Borings

The shallow borings were drilled during the period of November 1978 to January 1979. The borings were advanced with Central Mine Equipment rotary drill rigs, Model 55, using 6-1/2 inch hollow stem augers and NX core barrels. The number of drill rigs in operation varied from one to two. Air pressure was used during drilling and sampling in the shallow borings without use of water or drilling mud. A total of 52 borings were drilled to a maximum depth of 100 feet.

The initial 24 borings numbered B-1 through B-24 were drilled to depths of 24.5 to 100 feet on a grid pattern shown on Figure 1. In addition thin-walled tube samples were taken at selected depths. The borings were advanced at least 15 feet into the Gatuna Formation.

21 additional borings numbered B-26 through B-46 were drilled to depths between 26 and 100 feet at proposed locations for the surface structures. Borings numbered B-26 through B-40 were sampled and cored into the Gatuna Formation to a minimum depth of 15 feet. Six borings numbered B-41 through B-46 were drilled to a depth of 100 feet for the cross-hole and downhole seismic survey. Borings numbered B-8, B-32 and B-34 were also used for the seismic survey. Finally, seven additional borings numbered B-47 through B-53 were drilled to depths of 15 to 30 feet. Five of these borings numbered B-47 through B-51 were drilled at the plant site to obtain

additional tube samples in the caliche and Gatuna Formation for laboratory testing. Borings B-52 and B-53 were drilled at the sewage treatment plant, and were sampled and cored 15 feet into the Gatuna Formation.

Standard penetration tests with split spoon sampling, thin-walled Shelby tube samples, and NX cores were obtained in these borings at selected intervals.

Standard penetration tests with split spoon sampling were made in the sand and caliche, and in some borings in the upper portion of the Gatuna Formation. The standard penetration tests were performed in accordance with ASTM D-1586 with a split barrel sampler 1-3/8 inch I.D. and 2 inch O.D. The results of standard penetration tests are shown on Figure 8. The average percent recovery for the split spoon samples was 95 for the upper sand, 86 for the caliche, and 79 for the Gatuna Formation.

Thin-walled tube samples were obtained in the sand, caliche, and in the top 6 feet of the Gatuna Formation. The tube samples were taken with a Pitcher sampler equipped with a 2-7/8 inch I.D. thin walled Shelby tube.

Difficulties were encountered in the field during Pitcher sampling of the caliche and Gatuna Formation. In addition, some of the samples were unsuitable for testing when extruded in the laboratory.

In the sand stratum, 11 Pitcher samples were attempted; 10 of those were successful. The average percent recovery for the sand samples was 91. All the Pitcher samples of sand that were extruded in the laboratory were suitable for testing.

Pitcher sampling of the caliche and Gatuna Formation was less successful because it was hard for the sampler to advance in these strata. In the caliche, 69 Pitcher samples were attempted but only 50 of those were successful. The average recovery of the caliche samples was 64 percent. When extruded in the laboratory only 83 percent of the caliche samples were found suitable for testing. In the Gatuna Formation, 68 Pitcher samples were attempted but only 61 of those were successful. The average recovery of the Gatuna samples was 68 percent. When extruded in the laboratory only 44 percent of the Gatuna samples were found suitable for testing.

The effects of sample disturbance on the test results and on selection of design properties for foundation evaluation are discussed in detail in Sections 6, 7 and 9.

(1-78) WIPP ENG-1-2
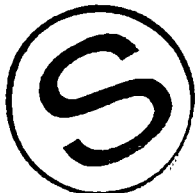
NX size rock cores were obtained in the Gatuna Formation in accordance with ASTM D-2113.

All borings were grouted with cement grout upon completion of sampling and testing.

4.3   Geophysical Surveys .

A seismic refraction survey was made by Sergent, Hauskins & Beckwith in December 1978 as part of the near surface exploration at the site. The locations of the seismic survey lines are shown on Figure 1, and the detailed results of the refraction survey are presented in Reference 3. In addition, a seismic cross-hole and downhole survey was performed by Harding Lawson and Associates in January 1979 in nine shallow borings to determine compressional and shear wave velocities of the near surface materials to a depth of 100 feet. The nine seismic borings are in three arrays as shown on Figure 1. The results of the cross-hole and downhole seismic survey are presented in Reference 4, and are summarized in Table 6 of this report.

The compresssional and shear wave velocities measured in the cross-hole survey were used to determine the dynamic elastic properties of the near surface materials.

4.4   Permeability Tests and Observation Wells

Permeability tests were performed in selected shallow borings in the upper sand, caliche and Gatuna Formation. These tests were done in accordance with the Bureau of Reclamation procedure designation E-18. In addition, well permeameter tests (Bureau of Reclamation designation E-19) were performed in the upper sand at the site. Results of the field permeability tests indicated that the upper sand has a high permeability in the range of 1170 to 6460 ft/yr (1.1×10^{-3} to 6.2×10^{-3} cm/sec). Although the caliche is fractured it was found to be relatively impermeable with permeabilities in the range of 14 to 240 ft/yr (1.4×10^{-6} to 2.3×10^{-5} cm/sec), and therefore acts as an aquiclude. The Gatuna Formation is about one order of magnitude less permeable than the upper sand and has permeabilities in the range of 70 to 1860 ft/yr (6.8×10^{-5} to 1.8×10^{-3} cm/sec.)

Eleven observation wells were installed at the site, ten in shallow borings to depths between 8 and 53 feet and one in the deep boring B-54 at a depth of 195 feet. These observation wells have been monitored since February, 1979 and have showed no ground water within a depth of 195 feet. The detailed results of the permeability tests and

monitoring of the observation wells are presented in Reference 16.

4.5   Test Pits

Five test pits, numbered TP-1 through TP-5, were excavated to depths of 13.5 to 30 ft at the site during March 1979 to determine in-situ properties of the soil and rock and to obtain bulk samples for laboratory testing. Field density tests and bulk samples were obtained at selected depths in the sand, caliche, and Gatuna Formation. Plate load tests were made in the two pits numbered TP-3 and TP-5. Excavation of the test pits was by dozer and backhoe, down to the Gatuna Formation. The locations of the test pits are shown on Figure 1, and the logs of test pits are included in Reference 16.

A total of 27 in-situ density tests were made in the test pits in accordance with ASTM D 1556. At least one in-situ density test was made in each stratum encountered in each of the five test pits. In-situ density tests were also made adjacent to the plate load tests in test pits TP-3 and TP-5. The results of in-situ density and natural water content determinations are shown on Figure 10.

A total of 22 bulk samples were obtained from the test pits. At least one bulk sample was obtained for each stratum encountered in each pit. Bulk samples were obtained adjacent to in-situ density tests.

A total of 14 plate load tests were made in test pits TP-3 and TP-5. In both TP-3 and TP-5, plate load tests were made at depths of 3 ft into the upper sand layer, on top of the caliche and at intermediate depths in the caliche. Also, plate load tests were made at the top of the Gatuna Formation in test pit TP-3. In addition to the 14 plate load tests, one plate load test was made in a hand-dug hole located 116 ft northeast of boring B-25 in the upper sand stratum at a depth of 3 ft. This plate load test was made outside the test pits in order to test the upper sand with a minimum disturbance from the excavation equipment. The results at hand dug hole were significantly lower than those inside the test pits. It was concluded that the plate load tests for the upper sand in the test pits were affected by the dozer operation. Therefore, the elastic moduli obtained in the hand dug hole were used for the in-situ upper sand layer. The plate load tests were made in accordance with ASTM D 1196. The results of all plate load tests are shown on Figure 9 and are summarized in Table 11.

Photographs were taken of each of the five pits which are included in Reference 17.
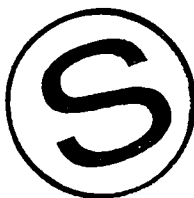
The test pits were backfilled with loose excavated material upon completion of sampling, testing and photographing.

4.6   Electrical Resisitivity

Electrical resistivity measurements were conducted at six locations numbered R-1 through R-6 to determine the corrosion potential in the upper materials.

In-situ resistivity values in ohms-cm were obtained by the fall-of-potential method to depths of 5 and 10 feet at the six locations. The locations of electrical resistivity tests are shown on Figure 1, and the results are included in Reference 16 and are given in Table 14.

5.   SITE CONDITIONS

5.1   Surface Conditions

The ground at the site has local undulations of a few feet and slopes gently to the west and southwest. Elevations in the plant site range from 3385 feet at boring B-52 in the southwest to 3440 feet at boring B-37 in the east. Based on these elevations, the plant site slopes about 1% to the southwest. The surface soils consist of eolian sand plains and sand dunes. The sand dunes are partly stabilized by vegetation, mainly mesquite, scattered grasses and annuals.

5.2   Near Surface Conditions

5.2.1   Strata

The materials above bedrock vary in thickness from 10 to 20 feet and consist of an upper stratum of reddish-brown, fine, poorly graded, very loose to medium dense sand which is underlain by a stratum of "caliche" consisting of white to brown well-cemented, hard, fine silty sand. Bedrock, underlying the caliche, consists of sandtone of the Gatuna Formation. The various strata are shown on the surface geological profiles on Figures 5 and 6, and are described below.

Upper Sand

The upper sand stratum extends from the ground surface to a depth varying from 3 to 16 feet. The sand is wind blown, fine, poorly graded, and ranges from light brown clean sand at the ground surface to dark brown silty sand with some clay near the contact with the caliche surface. Contours of thickness of upper sand are shown on Figure 2.

The standard penetration resistance for the upper sand ranged from 2 to 20 blows per foot. The higher blow counts were generally encountered near the contact with the caliche. Based on the standard penetration resistance, the upper sand varies in density from very loose to medium dense. The shear wave velocities as measured by the seismic cross-hole survey method varied between 450 and 900 feet per second and the compression wave velocity varied between 1100 and 1800 feet per second.

Caliche

Beneath the upper sand is a continuous stratum of hard caliche which is locally called "Mescalero Caliche" and forms a resistant "caprock" over the Gatuna Formation. The caliche appears to be irregular and undulating as shown on Figure 3 and varies in thickness from 3 to 15 feet. The caliche is made up of fine silty sand particles and is moderately to strongly cemented with calcium carbonate. The lower portion of the caliche (about 1 to 3 feet) blends gradually with the underlying Gatuna Formation. The color of caliche is white to brown.

The standard penetration resistance of the caliche ranged from 50 to 100+blows per foot, except in one case where 36 blows per foot was encountered. In general, the caliche is hard to very hard. The top 2 feet closely resembles limestone. The structure of the upper caliche is plated and changes gradually to nodular with depth. The shear wave velocities in caliche varied between 1000 and 1900 feet per second and the compression wave velocities varied between 2000 and 4000 feet per second.

Gatuna Formation

Bedrock underlying the caliche is sandstone of the Gatuna Formation varying in thickness from 16 to 28 feet. This rock is poorly indurated, relatively weak and friable. The top of the Gatuna Formation is encountered at about 10 to 20 feet below the natural ground surface and is irregular as shown on Figure 4. The Gatuna Formation consists of fine-grained cemented sandstone. In most of the site area, the upper 6 feet of the Gatuna is weakly cemented and the degree of cementation increases with depth. The cementing agents based on the chemical analyses, are aluminum and ferric oxides which give the Gatuna sandstone its reddish brown color.
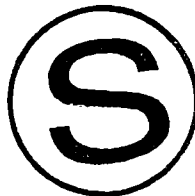
The standard penetration test resistance of the upper
25 feet of the Gatuna Formation ranged from 50 to
100+blows per foot, except in one case where 34 blows
per foot was encountered. The shear wave velocities
varied between 1300 and 2200 feet per second. The
compression wave velocities varied between 2900 and
4700 feet per second.

5.2.2   Ground water

Ground water was not encountered during drilling of
the shallow borings to a maximum depth of 100 feet
and the observation wells to a maximum depth of 200
feet. Eleven observations wells were installed at
the site at depths between 8 and 195 feet. These
observation wells have been monitored since February,
1979 and have showed no ground water within a depth
of 195 feet. Rainfall in this area amounts to
between 11 and 13 inches anually, which is not
sufficient to significantly affect the design of
foundations. Therefore, the foundation evaluation
was generally based on data from samples tested at
natural moisture content with the exception of the
evaluation of the upper sand layer as foundation for
lightly loaded structures. Some of the tests were
made under saturated conditions in order to evaluate
the change in properties due to saturation. The
differences in the test results are discussed in
Section 6.
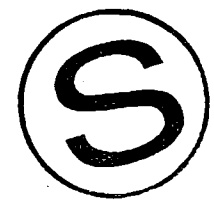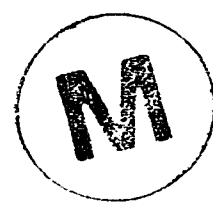
6. LABORATORY SOIL TESTING

6.1 Introduction

The laboratory soil testing program was developed by Bechtel and carried out by Sergent, Hauskins & Beckwith in their laboratories in Albuquerque and Phoenix and by Dames & Moore in their laboratory in San Francisco. The tests were performed on jar, thin-walled Shelby tube and bulk samples obtained from the shallow borings and test pits to a depth of 25 feet below the ground surface which is 6 feet into the Gatuna Formation.

The consolidation tests as well as classification, permeability, and electrical resistivity tests were made by Sergent, Hauskins & Beckwith. The triaxial tests and resonant column tests as well as classification and permeability tests were made by Dames & Moore. In addition, chemical analysis of the foundation materials was carried out by Metallurgical Laboratories in San Francisco. The testing program included the soil tests listed below and described in the following paragraphs.

a. Visual and laboratory classification

b. Sieve and hydrometer analyses

c. Atterberg limits

d. In-situ moisture content and unit weight

e. Specific gravity

f. Moisture-density relationship

g. Relative density

h. Unconsolidated undrained triaxial compression

i. Consolidated undrained tiraxial compression with pore pressure measurements

j. Consolidated drained triaxial compression

k. Strain-controlled triaxial compression

l. Resonant column

m. Permeability

n. Consolidation tests

o. Chemical analysis

p. Electrical resistivity
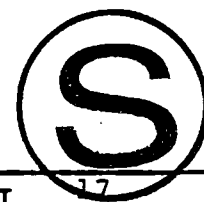
(1-78) WIPP ENG-1-2

All test results are given in the reports by
Sergent, Hauskins & Beckwith and by Dames & Moore
in Reference 16 to this report. In addition,
all test results are summarized in Table 1 on the
soil test results summary sheets.

Based on field observation as well as seismic
cross-hole survey data the Gatuna Formation improved
with depth. Therefore, for the type of foundation
and loads of the near surface structures it was
considered adequate to test the materials down to a
depth of 25 feet which is 6 feet into the Gatuna
Formation.

6.2   Classification Tests

All samples for soil testing were examined and
classified in the laboratory to check the field
classification. The tube samples, in particular,
were examined for disturbance, and only those that
did not indicate apparent disturbance were used for
testing. Visual classification was made in
accordance with ASTM D 2488, and laboratory
classification was in accordance with ASTM D 2487.

6.2.1   Sieve and Hydrometer Analyses

Sieve and hydrometer analysis determinations were
made on selected samples from the sand, caliche and
Gatuna in accordance with ASTM D 422. The results
are summarized in Table 1, and the envelope of the
grain size distribution curves for each material is
plotted on Figures 11 through 13. Individual grain
size plots are provided in Reference 16 of this
report. The D_{10} size and coefficient of uniformity
of tested sand backfill are given in Table 9.
Figure 11 gives the envelope of the results of all
samples of the upper sand stratum that were tested.
It is not intended to represent a typical size range
for the predominant sand encountered in this
stratum.

6.2.2   Atterberg Limits

Atterberg limit tests were made on selected samples
from the sand, caliche and Gatuna in accordance with
ASTM D 423 and D 424. The Atterberg limit tests
showed that most of the soils at the site are
nonplastic. Only four of the tested samples
exhibited some plasticity. The plasticity index was
3 for one sand sample, 3 and 13 for two caliche
samples, and 3 for one Gatuna sample. Atterberg
limits vs depth are shown on Figure 14, and the
results of the tests are given in Table 1.

6.2.3 In-Situ Moisture Content and Unit Weight

Moisture content and dry unit weight were determined for the tube samples from the shallow borings. Determinations of moisture content were made in accordance with ASTM D 2116 and the unit weight was determined by direct measurement of weight and volume. In-situ moisture content and unit weight vs depth are shown on Figure 10, and the results are summarized in Table 1.

6.2.4 Specific Gravity

Specific gravity tests were made in accordance with ASTM D 854 on selected samples of each of the foundation soils. The results are summarized in Table 1.

6.2.5 Moisture-Density Relations

Moisture-density relations (compaction tests) were made in accordance with ASTM D 1557, on selected bulk samples of the upper sand stratum, the caliche and Gatuna Formation. Bulk samples of the upper sand were taken from shallow borings as well as from test pits to determine the properties of the sand as a backfill material. The caliche and Gatuna samples were taken from test pit excavations. The compaction curves are given in Reference 16, and the results of optimum moisture content and maximum dry unit weight are summarized in Table 1.

Where the moisture-density curve is very flat, as in the case of cleaner sands, no well defined optimum moisture content exists. Nevertheless moisture will be required for dust control and to enhance compaction. The amount of moisture required will be investigated during the test fill program.

6.2.6 Relative Density

Attempts were made to determine the in-situ relative density of the upper sand by performing maximum and minimum density tests on selected samples from field density tests in the test pits. Maximum and minimum relative density tests were made in accordance with ASTM D 2049. A total of six in-situ relative density tests were made for the upper sand material and the results are given in Table 8. Two of the six samples tested had more than 12 percent fines are not included in Table 8. Based on the results, the relative density of the upper sand varied between 39 and 81 percent. The high relative densities are not consistent with other data and the high values obtained are considered to overestimate the relative density of the upper sand. The relatively high in-situ dry densities determined in
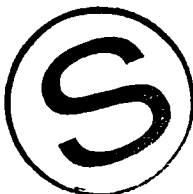
the test pit were probably affected by the dozer during the test pit excavation. Based on the standard penetration and the plate load test data, the density of the upper sand has been shown to vary from very loose to medium dense.

6.3   Engineering Properties Tests

Tests were made to determine the static and dynamic engineering properties of in-situ soil and shallow rock for use in analyses made to develop soil foundation design criteria. These tests are described below. Summaries of the engineering properties of the in-situ soils and the sand backfill are given in Tables 4 and 5 respectively. Design dynamic and static shear and elastic moduli are given in Table 7. The detailed test results are given in Reference 16, and summarized in Table 1.

6.3.1   Unconsolidated Undrained Triaxial Compression Tests

Unconsolidated undrained triaxial compression tests were made on 2 7/8 inch diameter specimens prepared from thin-walled tube samples from the caliche and the Gatuna Formation. The tests were made in accordance with ASTM D 2850.

Specimens from both the caliche and the Gatuna Formation were tested at natural moisture content and under saturated conditions. Each specimen was approximately 6 inches in height, and was encased in a rubber membrane and placed in the tiraxial chamber. A constant confining pressure of 1, 6 or 12 ksf was imposed on the specimen without permitting drainage. The test specimen was then sheared under the confining pressure and without drainage. The deviator stress and axial strain were recorded and also the moisture content and dry unit weight were measured.

The test results for the caliche and Gatuna specimens are given in Table 2 and are shown on Figure 15. The undrained shear strength test results of the caliche varied between 2.4 and 6 ksf at natural moisture content, and between 1.6 and 17.0 ksf under saturated conditions. Saturation did not appear to have a significant effect on the undrained strength of the caliche.

The Gatuna Formation sandstone underlying the caliche had an undrained shear strength of 1.6 to 9.5 ksf at natural moisture content, and 1.4 ksf when saturated.

It is believed that the lower strength values measured in the caliche and Gatuna specimens are due to sample disturbance and the upper values are more

representative of the actual strength of these materials.

6.3.2 Consolidated Undrained Triaxial Compression Tests with Pore Pressure Measurements

Consolidated undrained ($\overline{CU}$) triaxial compression tests with pore pressure measurements were made on specimens 2 7/8 inches in diameter and 6 inch high prepared from thin-walled tube samples from the caliche and the Gatuna Formation. Initially, the test program consisted of three test specimens to be prepared from tube samples. However, because large quantities of the caliche and Gatuna materials were not usable, only two test specimens were prepared from these samples.

In addition, $\overline{CU}$ tests were made on compacted specimens 3 inches in diameter and 6 inch high prepared from bulk samples of the caliche and the Gatuna Formation. These specimens were compacted at optimum moisture content to 95% of the maximum dry density as determined in accordance with ASTM D 1557, Method D.

Each specimen was encased in a rubber membrane, placed in the triaxial chamber and saturated by the back pressure method. After saturation, the test series with the three specimens was consolidated isotropically at confining pressures of 1, 6 and 12 ksf, respectively. The test series with the two specimens was tested at confining pressures of 1 and 6 ksf respectively. After consolidation the specimen was sheared without permitting drainage and pore pressure measurements were made. The deviator stress, axial strain and pore pressure were recorded, and the moisture content and dry unit weight were also measured.

The effective strength Mohr envelopes obtained at peak deviator stress for the in-situ caliche and Gatuna samples are given in Table 2 and are shown on Figures 18 and 19, respectively. The results showed that the in-situ caliche has a cohesion $\overline{c}$ = o and an angle of internal friction $\overline{\phi}$ = 43 degrees. For the in-situ Gatuna Formation, the cohesion $\overline{c}$ = 2.8 ksf and the angle of internal friction $\overline{\phi}$ = 35 degrees.

The test results for the compacted caliche and Gatuna are shown on Figures 16 (b) and 16 (c), respectively. The results showed that the compacted caliche has a cohesion $\overline{c}$ = 2.0 ksf and an angle of internal friction $\overline{\phi}$ = 34 degrees. For the compacted Gatuna, the cohesion $\overline{c}$ = 0.4 ksf and the angle of internal friction $\overline{\phi}$ = 31 degrees.

6.3.3 Consolidated Drained Triaxial Compression Tests

Consolidated drained (CD) triaxial compression tests were made on thin-walled tube samples from the upper sand, caliche and Gatuna Formation. The specimens were 2 7/8 inches in diameter and 6 inches in height. One series of tests was made for the upper sand, three series for the caliche, and three series for the Gatuna Formation.

In addition, CD tests were made on compacted specimens 3 inches in diameter and 6 inch high prepared from bulk samples of the upper sand, caliche and Gatuna materials. These specimens were compacted at optimum moisture content to 95% of the maximum dry density. The maximum dry density was determined in accordance with ASTM D 1557, Method C for the upper sand specimens, and in accordance with ASTM D 1557, Method D for the caliche and Gatuna specimens. Three series of tests were made for the compacted sand, one series for the compacted caliche and one series for the compacted Gatuna.

Each specimen was encased in a rubber membrane, placed in the triaxial chamber and saturated by the back pressure method. After saturation, the test series with three specimens was consolidated isotropically at confining pressures of 1, 6 and 12 ksf, respectively. The test series with two specimens was tested at confining pressures of 1 and 6 ksf, respectively. The test specimen was then sheared under strain-controlled load without permitting any buildup of pore pressure. The deviator stress, axial strain and volumetric strain were recorded, and the moisture content and dry unit weight were also measured.

The effective strength at peak deviator stress for the in-situ sand, caliche and Gatuna samples are given in Table 2 and are shown on Figures 17, 18 and 19, respectively. The results showed that the in-situ sand has a cohesion $\bar{c} = 0$ and an angle of internal friction $\bar{\phi} = 33$ degrees. This angle of friction is considered high for the in-situ sand, and could be due to densification of the tube samples during field sampling and transportation. For the in-situ caliche, the cohesion $\bar{c}$ ranged from 0.14 to 2.6 ksf and the angle of internal friction $\bar{\phi}$ ranged from 31 to 33 degrees. For the in-situ Gatuna Formation, the cohesion $\bar{c}$ was between 0.15 and 4.0 ksf and the angle of internal friction $\bar{\phi}$ was between 28 and 40 degrees. The lower values of the strength parameters of the in-situ caliche and Gatuna could be due to sample disturbance.

The test results for the compacted sand are given in Table 3 and are shown on Figure 16(a). The results
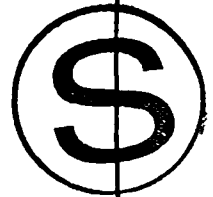
showed that the compacted sand is cohesionless and has an angle of internal friction $\bar{\phi}$ between 33 and 33.5 degrees. The results for the compacted caliche and Gatuna are shown on Figures 16(b) and 16(c), respectively. These results showed that the compacted caliche has a cohesion $\bar{c}$ = 2.0 ksf and an angle of internal friction $\bar{\phi}$ = 35 degrees. The compacted Gatuna has a cohesion c = 1.6 ksf and an angle of internal friction $\bar{\phi}$ = 36 degrees.

6.3.4 Strain-Controlled Cyclic Triaxial Compression Tests

Cyclic triaxial tests for measurements of the dynamic moduli and damping ratios were made on thin-walled tube samples from the caliche and the Gatuna Formation. The specimens prepared from these samples were 2 7/8 inches in diameter and 6 inches in height. Specimens were tested at natural moisture content and under saturated conditions.

In addition, cyclic triaxial tests were made on compacted specimens 3 inches in diameter and 6 inches high prepared from bulk samples of the upper sand material. These specimens were compacted at optimum moisture content to 95% of the maximum dry unit weight as determined in accordance with ASTM D 1557, Method C.

The method of compaction has a major influence on the dynamic properties of compacted specimens. In the field the backfill material will be compacted in relatively thin layers with a vibratory roller providing vertically oscillating vibrations at relatively low frequency (1200-1600 rpm). In order to simulate field conditions and to obtain uniformity the specimens were prepared in six layers, each about 0.9 inch high, compacted to the required density and at the specified moisture content. The specimens were compacted in uniform layers using low frequency vibrations applied vertically to the specimens. Preparation of the specimen in layers was according to the procedure of under compaction recommended by Ladd and Silver (5). The compacted sand specimens were tested at optimum moisture content and at 100% saturation.

Each series consisted of 3 specimens, one was tested at a confining pressure of 1 ksf, one at 6 ksf and one at 12 ksf, respectively. Each test specimen was loaded by 10 cyclic axial loads of such magnitude that it produced axial strains in the range 10^{-3} to 1.0 percent.

The variation of the dynamic moduli and damping ratios with strain are shown on Figures 20 through 25. The variation of shear moduli with strain for the compacted sand samples are shown on Figures
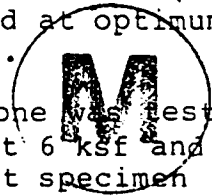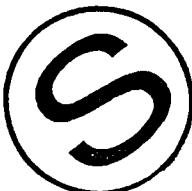
20(a) and 20(b). Although sample B-32, BL-1 has distinctly different grain size distribution with as much as 36% of fines, the variation of shear moduli with strain of this sample was similar to that of sample B-29, BL-1 which has only 8% fines. Therefore, the results of the two samples are included on Figure 20(a). The reduction of shear moduli with strain for the compacted sand is less than indicated by the standard curves proposed by Seed and Idriss[6]. But it is within the range of data shown in this reference. The curves for the compacted sand are flatter than those for the caliche and Gatuna probably due to sample disturbance of the caliche and Gatuna. The curves also show that saturation of the compacted sand and the caliche has no effect on the shear modulus. However, saturation of the Gatuna Formation resulted in a significant reduction of its shear modulus.

6.3.5   Resonant Column Tests

Resonant Column tests for measurements of the dynamic moduli and damping ratios were made on thin-walled tube samples from the caliche and the Gatuna Formation, and on compacted specimens from the upper sand material. The preparation of the test specimens was the same as for cyclic triaxial tests in subsection 6.3.4. However, the resonant column tests were made at smaller strains in the range of 10^{-5} to 10^{-2} percent.

Each specimen was tested at three different confining pressures of 1, 6 and 12 ksf. The dynamic moduli and damping ratios of each specimen were determined for several strain levels. In the resonant column apparatus the specimen base was fixed and the top was excited by torsional oscillations using a Hardin oscillator driven by a variable sine wave frequency. The response of the specimen was measured by an accelerometer mounted in the oscillator and the output was displayed on an oscilloscope. The equivalent linear shear modulus of the specimen was obtained from the resonant frequency of the system according to the procedure given by Drnevich and Hardin (7). The damping ratio was determined from the decay curve of the vibration after shutting-off the torsional oscillator.

The dynamic moduli are given in Table 7 and the variation of dynamic moduli and damping ratios with strain are shown on Figures 20 through 25. As in cyclic tests , saturation did not have a significant effect on the dynamic properties of the compacted sand backfill or the caliche. However, saturation of the Gatuna samples resulted in a significant reduction of the dynamic shear modulus.

(1-79) WIPP ENG-1-2

The results of the resonant column tests are fairly consistent with those of the cyclic triaxial tests for both the compacted sand and the Gatuna Formation. However, for the caliche the results of resonant column tests show a wide scatter in the shear modulus values as well as large discontnuity between the resonant column and cyclic triaxial test data. This scatter is probably due to variation in sample properties such as degree of cementation, grain size distribution and relative density of the caliche material. Sample disturbance coudld also be a major factor in causing the scatter in the shear modulus for the caliche samples.

The discontinuities between the resonant column and cyclic triaxial test data could be due to the different loading conditions of the test procedures. In the cyclic triaxial test, a vertical loading is used but in the resonant column test horizontal vibration is used. In addition, shear moduli for the resonant column test were calculated based on the sample dimensions, density, torsional acceleration and resonant frequency where in the cyclic triaxial test the shear moduli are determined directly from measurement of the elastic moduli and an estimated value of Poisson's ration.

6.3.6 Permeability Tests

Permeability tests were performed in accordance with ASTM D 2434 on thin-walled tube samples from the upper sand, the caliche and the Gatuna Formation. The results of these laboratory permeability tests are given below.

These laboratory permeabilities were consistent with permeabilities measured in the field (Subsection 4.4) as summarized in the following:

Material	Laboratory Permeability(cm/sec)	Field Permeability(cm/sec)
In-situ Sand	$9.0 \times 10^{-5} - 3.6 \times 10^{-3}$	$1.1 \times 10^{-3} - 6.2 \times 10^{-3}$
Caliche	$6.8 \times 10^{-6} - 6.3 \times 10^{-4}$	$1.4 \times 10^{-6} - 2.3 \times 10^{-4}$
Gatuna	$5.7 \times 10^{-4} - 8.4 \times 10^{-4}$	$6.8 \times 10^{-5} - 1.8 \times 10^{-3}$

6.3.7 Consolidation Tests.

One consolidation test was made in accordance with ASTM D 2435 on a thin-walled tube sample from the upper sand. The test was made to determine the potential of collapsing in the uper sand material caused by saturation due to a broken pipeline or irrigation.
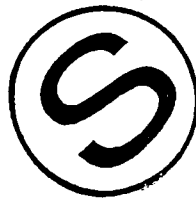
The test specimen was preloaded to 2 ksf and then saturated, and the load maintained for 24 hours. The loading was increased in increments to 32 ksf and then reduced to 0. The consolidation test showed that under saturation the specimen has a vertical deformation of about 3 percent, indicating the upper sand is susceptible for additional settlement under saturation. The results of the consolidation test are given in Reference 16.

6.3.8 Chemical Analysis

Chemical analysis was carried out on 5 selected samples of the foundation materials. The samples analyzed included 3 samples from the upper sand, one from the caliche, and one from the Gatuna Formation.

The samples were analyzed for major elements, and the results of the chemical analysis for the different materials are presented in Table 10. The results show that the main constituent of the upper sand is silica which is typical of sand material. However, the silica content decreased with depth from 93% at the ground surface to 85% at a depth of 10 feet. The upper sand contained some aluminum oxide, ferric oxide and calcium carbonate. The percentage of these materials increased with depth from 4.6% at the surface to 8.4% to a depth of 10 ft.

The caliche sample contined 55% silica and 16% calcium carbonate. This high percentage of calcium carbonate provides the cementation and hardness of the caliche and gives the caliche its whitish color. Additional cementing agents of 6% aluminum and ferric oxides were also found in the caliche sample.

The Gatuna sample contained a higher percentage of silica which was 80%. The cementing agents in the Gatuna sample were mainly aluminum and ferric oxides of approximately 10%. Additional cementing material of 3% calcium carbonate was also found in the Gatuna sample. The variation in the amounts of cementing agents as given above shows the gradational change that could occur between the caliche and the Gatuna Formation.

6.3.9 Electrical Resistivity

Laboratory resistivity measurements were performed on selected samples from test pits and shallow borings in the vicinity of six locations numbered R-1 through R-6. These locations are shown on Figure 1, and the laboratory resistivity results are given in Table 15. In addition, pH values, and sulphate and chloride concentrations were measured for the resistivity samples and are summarized in Table 16.

6.4 Design Engineering Properties
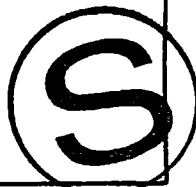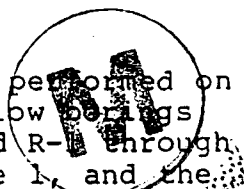
The results of engineering properties tests are summarized in Table 3 for the sand backfill and in Table 2 for the in-situ soils and shallow rock. The design engineering properties were selected on the basis of these results. However, it is believed that some of the tested samples in the caliche or Gatuna materials were disturbed and therefore engineering judgement was applied in selecting design parameters for these materials. The design properties of the in-situ materials and sand backfill are provided in Tables 4 and 5 respectively.

The seismic velocities, Poisson's ratios, and elastic moduli determined from the cross-hole and downhole survey for the in-situ materials are given in Table 6. Dynamic and static design shear and elastic moduli for the in-situ materials as determined from the seismic cross-hole survey and plate load tests, respectively, are provided in Table 7. The design dynamic moduli for the sand backfill were determined from resonant column tests and are included in Table 7. The variation of the shear moduli and damping ratios with strain as determined from resonant column tests and cyclic triaxial tests is shown on Figures 20 and 23 for the sand backfill, Figures 21 and 24 for the caliche, and Figures 22 and 25 for the Gatuna.

(1-79) WIPP ENG-1-2
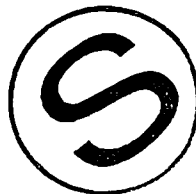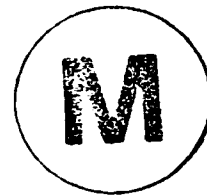
7.   FOUNDATION EVALUATION

7.1   General

The investigations showed that either the caliche or the Gatuna Formation would provide an excellent foundation. The upper loose sand is not suitable for supporting moderately to heavily loaded structures. However, the upper sand when removed and placed in properly compacted layers would also provide suitable foundation for Design Class I and II structures. This is discussed in greater detail in Section 10. Lightly loaded non-settlement sensitive structures, other than Class I and II structures, with bearing pressures less than 1.5 ksf may be founded at a shallow depth in the upper sand.

7.2   Design Criteria

The performance of foundation materials under loading is evaluated based on two criteria:

(1) The ability of the ground to support loads transferred through the structural foundation with an ample factor of safety against soil failure.

(2) The ability of the foundation to support structural loads with tolerable settlements.

The first criterion is related to the strength of the supporting foundation materials. The second criterion is related to the "stress-deformation" characteristics of the foundation material and its influence on the structure.

In the case of a structure foundation on sand, caliche or the Gatuna Formation, the allowable bearing pressure is limited by tolerable settlements rather than the bearing capacity criterion because of the high strength of these material.

Any foundation design must satisfy the following safety requirements:

(1) The factor of safety for bearing capacity must be at least 3 for dead plus normal live loading.

(2) The factor of safety must be at least 2 for dead plus maximum live loading including wind or seismic loading.

(3) Settlements under static plus dynamic conditions should be tolerable in order not to create distress in the superstructure or impair its function.

(4) Although ground water is very deep and the sand back-fill is not likely to get saturated, foundation grade
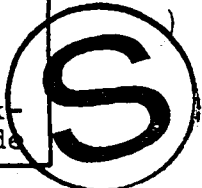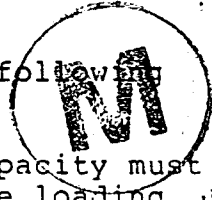
for spread of strip footings should be at
least 2 feet below the ground surface to provide
adequate edge support.

7.3   Foundation Treatment

The upper sand stratum is not suitable for supporting
structures with net static pressures in excess of 1.5
ksf as discussed in Section 7.6.   The sand must be
removed and these structures must be supported on
caliche, Gatuna Formation sandstone or compacted sand
fill.   All select backfill beneath and adjacent to
the structures to the limits discussed in Section 10
must be compacted to at least 95% of the maximum
density determined by ASTM D-1557.   Backfill in
non-load bearing areas of the site must be compacted
to at least 90% of the maximum density determined by
ASTM D-1557.

Preferably, all structures should be founded on
compacted sand or caliche foundations.   However,
non-Class I and II structures, where there is a signif-
icant economic saving in not excavating the sand down
to the caliche layer can be founded at a depth of 2 feet
below the surface on the in-situ sand layer.   These
structures should be flexible enough to withstand dif-
ferential settlements of at least 1 inch and the net
allowable pressures must not exceed 1.5 ksf.

In arid areas, sandy soils may develop a loose,
lightly cemented structure which could collapse under
loading when water is introduced into them.   This can
result in excessive settlements.   In order to reduce the
potential for excessive settlements of structures founded
directly on in-situ sand, it is recommended that the
foundation soil should be inundated and compacted with
vibratory equipment so that the density of the sand in
the top 12 inches under the footings is at least 95% of
the maximum determined by ASTM D-1557.   Careful control
should be exercised during compaction to prevent local
instantaneous liquefaction under vibrations of the roller.

7.4   Net Ultimate Bearing Capacity

The net ultimate bearing pressure is the pressure over
and above that due to the weight of soil and water at
foundation level that will result in overstressing the
foundation soil.

The net ultimate bearing pressure of the foundation
materials was determined to evaluate the factor of
safety of the foundation elements.   The effective shear
strength was used in determining the ultimate bearing
pressure for caliche, Gatuna Formation sandstone, and
compacted sand.   The design strength parameters of the
different foundation materials are given in Tables 4
and 5.   In the bearing capacity analysis for foundations
resting directly on compacted sand, an angle of internal
friction of 33 degrees and a cohesion of 0 was used,
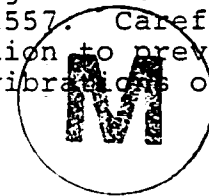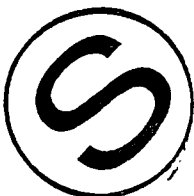
and for foundations placed on caliche or Gatuna Formation, an angle of internal friction of 33 degrees and a cohesion of 0.9 ksf was used. These strength parameters are the lowest values for the strata under the foundations and therefore the bearing capacity values are conservative.

The ratio of the net ultimate bearing pressure to the net applied pressure is defined as the factor of safety. The net ultimate bearing capacity supporting a circular, square or rectangular footing is defined by the following expressions (Vesić[8]):

Circular and Square Footing

$$q_{ult} = 1.2cN_c + \gamma_e D_f N_q + 1/2 \times 0.6 \, \gamma_e \, B \, N_\gamma$$

Rectangular Footing

$$q_{ult} = cN_c \times (1 + 0.2 \frac{B}{L}) + \gamma_e D_f N_q + 1/2 (1-0.4 \frac{B}{L}) \gamma_e B N_\gamma$$

where

q_{ult} = net ultimate bearing capacity

c = cohesion

γ_e = effective unit weight

D_f = depth of footing below lowest adjacent grade

N_c, N_q, N_γ = dimensionless bearing capacity factors which depend on the friction angle of the soil

B = width or diameter of footing

L = length of rectangular footing

These equations were simplified assuming $\tan \emptyset = 0$.

The ultimate bearing capacity of mat foundations was also calculated based on the above formulas. The net ultimate bearing pressures, the net applied static pressures, and the factors of safety for the various Design Class II structures are summarized in Table 12. The factors of safety calculated are large (9 to 29), and significantly exceed the allowable factor of safety of 3.

7.5   Foundation Settlements

7.5.1   Static Settlements

The allowable pressures for structures are controlled by tolerable settlements. The settlement of foundations on compacted sand, caliche, or Gatuna
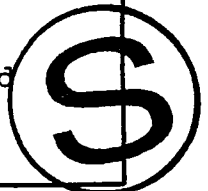
Formation sandstone can be estimated based on elastic theory using elastic moduli determined from plate load and laboratory testing. When a load is applied to sand, caliche or Gatuna Formation sandstone, deformation will occur rapidly and most of the settlement will occur during construction.

The elastic moduli for the caliche and Gatuna Formation sandstone were determined from the plate load test results which are summarized in Table 11. The elastic moduli for compacted sand were determined from consolidated, drained, triaxial tests, and the results are given in Table 5. For foundations resting on the caliche, a modulus of elasticity of 20,000 lb/in^2 was used. The elastic settlement of uniformly loaded circular, square, and rectangular footings on a semi-infinite elastic medium can then be calculated from the following equation (Lambe and Whitman[9]):

$$S = q \cdot B \cdot \frac{(1 - \mu^2) I}{E}$$

where

q = bearing pressure

B = width or diameter of footing

μ = Poisson's ratio

E = modulus of elasticity

I = displacement influence factor.

The Poisson's ratio was determined from the seismic cross-hole survey data presented in Table 6.

For foundations on sand backfill extending down to the caliche, the static settlement was calculated by integrating the vertical strains in the sand, caliche, and Gatuna layers using the following elastic moduli for each of these layers:

Sand backfill = 3,600 lb/in^2

Caliche = 25,000 lb/in^2

Gatuna sandstone = 15,000 lb/in^2

The rock formations below the Gatuna have elastic moduli equal to or greater than 8×10^5 lb/in^2.

The strains were determined from the following equation (Lambe and Whitman[9]):

$$e_v = (\Delta\sigma_v - 2\mu \Delta\sigma_h)/E$$

(1-79) WIPP ENG-1-2

where

$$e_V = \text{vertical strain at a given depth}$$

$$\Delta\sigma_V = \text{increase in vertical stress which was calculated using Boussinesq equations}$$

$$\Delta\sigma_h = \text{increase in horizontal stress}$$

For the purpose of analyses $\Delta\sigma_h$ was considered equal to 0 and the above equation was simplified to:

$$e_V = \Delta\sigma_V/E$$

The net applied static pressure and the settlement for the various Design Class II structures are summarized in Table 13. The calculated settlements for footings of these structures were small and in the range of 0.1 to 0.5 inch for pressures in the range of 2 to 5 ksf. Under the hot cell mat of the waste handling building, the settlement was 1.0 inch for a net applied pressure of 4 ksf. In addition, Figure 26 provides plots of bearings pressures versus settlements for various footing widths resting on compacted sand and on caliche foundations.

7.5.2 Earthquake-Induced Settlements

The earthquake-induced settlements for footings supported by the compacted sand, caliche and Gatuna Formation materials were evaluated using the method proposed by Seed and Silver (10). Following this method of analysis, the distribution of average induced shear strain with depth was obtained using the SHAKE computer program (11). The shear moduli for the caliche and Gatuna Formation were determined from the seismic cross-hole survey data and for the compacted sand from cyclic triaxial and resonant column test data. The variation of the shear moduli and damping ratios with strain used for the analysis were determined from dynamic laboratory tests and are shown on Figures 20 and 23 for the compacted sand, Figures 21 and 24 for the caliche, and in Figures 22 and 25 for the Gatuna Formation sandstone.

Bechtel's synthetic earthquake time history with a maximum ground acceleration of 0.1 g was used in the settlement analysis. The Bechtel synthetic time history is given in the Bechtel Topical Report[12]. It is a compilation of data from several real earthquake records. The response spectra of the time history envelop the design spectra given in the NRC Regulatory guide 1.60. The Bechtel synthetic time history has a total duration of 24 seconds. The maximum integrated velocity of the time history is about 5 feet per second for a peak ground acceleration of 1.0g.
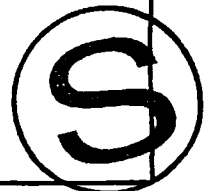
The geologic and the induced shear strain profiles used are shown on Figure 27. The induced vertical strains were then calculated from the shear strains using a correlation that was developed from the dynamic test data for the foundation materials. The seismically induced settlements below the foundation were calcualted by integrating the vertical strain values of each layer. By this approach, the calculated induced settlements under all the Design Class II structures were negligible (less than 0.015 inch).

7.6   Foundations on In-Situ Sand

Non-Class I and II, lightly loaded non-settlement sensitive structures which are founded 2 feet below the ground surface on the in-situ sand, should have a maximum net pressure of not more than 1.5 ksf. The foundation treatment for these structures is discussed in Section 7.3 of this report. The most suitable types of foundation for these structures are strip footings.

The foundation settlement under these structures were estimated from the standard penetration test data in the upper sand using procedures proposed by Peck et al (13). The standard penetration test blowcounts in the upper sand are shown on Figure 8. The estimated settlement for 1.5 ksf load based on the Peck et al (13) settlement charts is about 1 inch. Because of large variation in the densities of the upper sand, these structures should be designed for a differential settlement equal to at least 1 inch. Additional settlement of the upper sand can be caused by saturation due to a broken pipeline or irrigation. Consolidation test data (Section 6.3.7) showed that the upper sand can settle about 3 percent due to saturation. Therefore, the in-situ sand foundation should be inundated and compacted so that the density in the top 12 inches under the footings is at least 95% of the ASTM D-1557.

The minimum width of strip footings should be 2 feet and for spread footings it should be 3 feet. The settlement estimate is based solely on the foundation loading. Therefore, any grading and areal fill which would result in additional settlements must be placed prior to construction of the footings in order to prevent additional settlements.

8.   LATERAL EARTH PRESSURES

8.1   Introduction

The earth pressure that a foundation wall or retaining wall must support depends on the type of soil, the soil strength, the wall friction, the ground water conditions, the degree of compaction in
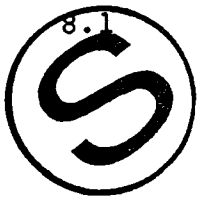
the backfill, the method used in the backfilling
process and the amount of deflection that the wall
undergoes. The principal conditions involved are
"active", "at rest" and "passive" earth pressure.

These conditions are discussed in detail in the
following paragraphs. The recommended lateral earth
pressures for design do not include hydrostatic water
pressures since there is no ground water table within
the foundation depth at the WIPP site.

In addition to the lateral earth pressures discussed
below, there will be pressures due to surchage
loading of construction equipment placed adjacent to
the walls. These pressures should be calculated when
the equipment types and loading become available and
should be added to the earth pressures.

8.2   Active Earth Pressure

If a wall with a horizontal backfill surface behind
it is free to move away from the backfill, then the
soil can expand laterally. The vertical stress in
the soil remains constant but the horizontal stress,
or earth pressure, reduces until the shear strength
of the backfill is fully developed. The horizontal
component of stress in the backfill under this
condition is known as the active earth pressure.
Most unrestrained retaining walls can move
sufficiently to permit development of the active
earth pressure. For sand backfill with an angle of
internal friction of 33 degrees, and neglecting wall
friction, the active earth pressure coefficient, K_a,
is 0.29. The active earth pressure coefficient is
the ratio of horizontal to vertical stress
immediately behind the wall.

The active earth pressure recommended for design is:

$$p_a = 0.29 \, \gamma_m H$$

where

H = height of the backfill above base of wall

γ_m = moist unit weight of the backfill.

The active force is:

$$P_a = 0.29 \, \gamma_m H^2/2$$

This force is acting at a depth of 2/3 H below the back-
fill surface. Lateral earth pressure diagrams are shown
on Figure 28.

8.3   At Rest Earth Pressure

When a rigid wall is restrained from moving, as in
the case of a foundation wall of a building, the

(1-79) WIPP ENG-1-2
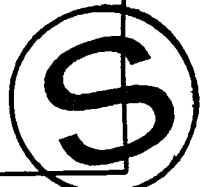
horizontal component of earth pressure is called the "at rest" earth pressure and its magnitude will depend on the degree of compaction of the backfill. For normally consolidated clean sands, the theoretical "at rest" earth pressure coefficient, K_O, varies from about 0.35 for dense sands to about 0.5 for loose sands.

However, around structures the backfilling process will increase the earth pressure considerably. Based on emprical evaluations, the current practice is to use an earth pressure coefficient of 0.5 for moderate compaction and 0.7 for heavy compaction where the backfill is sand or silty sand.

Since the plant backfill will be compacted to 95% of the maximum density determined by ASTM D-1557, an "at rest" earth pressure coefficient of 0.7 should be used.

The "at rest" earth pressure is:

$$p_O = 0.7 \ \gamma_m \ H$$

The "at rest" force is:

$$P_O = 0.7 \ \gamma_m \ H^2/2$$

This force is acting at a depth of 2/3 H below the back-fill surface. Lateral earth pressure diagrams are shown on Figure 28.

8.4   Passive Earth Pressure

When a wall with an adjacent horizontal soil surface is pushed into the backfill, the horizontal stresses in the soil will build up while the vertical stresses remain constant until the shear strength of the soil is fully developed. The horizontal stress developed under this condition is known as the passive earth pressure. For an angle of internal friction of 33 degrees for the sand backfill, and conservatively neglecting wall friction, the passive earth pressure coefficient (ratio of horizontal to vertical stresses) is 3.39.

However, the movement necessary to develop full passive pressure is quite large being on the order of five percent of the height of the wall. Since such movements could not normally be tolerated, a passive pressure in design equal to 50% will be used.

The recommended passive earth pressure is:

$$p_p = 1.7 \ \gamma_m \ H$$

and the passive force is:

$$P_p = 1.7 \ \gamma_m \ H^2/2$$
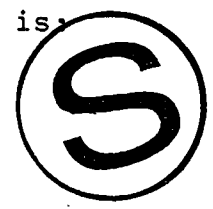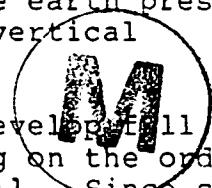
This force is acting at a depth of 2/3 H below the backfill surface. Lateral earth pressure diagrams are shown on Figure 28.

8.5 Dynamic Earth Pressure

8.5.1 General

The previous paragraphs discussed earth pressures under normal static conditions. However, during earthquakes these pressures will change. Design Class I and II structures should be designed for the at-rest static pressure plus a dynamic pressure increment due to seismic loading. This section develops a basis for evaluating the dynamic pressure increment so that a conservative appraisal of the dynamic pressure can be made for earthquake conditions. This analysis applies to vertical retaining walls as well as to embedded foundation walls. For a more detailed evaluation of dynamic earth pressure, a dynamic analysis may be performed using finite element computer programs. The dynamic engineering properties for this type of analysis are furnished in this report. The dynamic passive pressure has been conservatively disregarded since the movement necessary to develop full passive pressure is quite large and could not normally be tolerated.

.The design procedures for dynamic soil loads are based on the Mononobe - Okabe analysis of dynamic pressure in dry cohesionless materials. These procedures were further simplified by Seed and Whitman (14).

The Mononobe - Okabe solution was based on the following assumptions:

a. The wall yields sufficiently to produce active earth pressure.

b. The maximum shear strength is mobilized along the potential sliding surface.

c. The soil wedge behind the wall acts as a rigid body so the accelerations are uniform throughout the mass, and the effect of the earthquake can be represented by inertia forces KW where W is the weight of the sliding wedge and K represents the ratio between the horizontal and vertical components of the earthquake accelerations.

Seed and Whitman (14) present the following procedure for obtaining the Mononobe - Okabe earth pressures:

a. The maximum earth pressure during an earthquake is equal to the sum of the static earth pressure and a dynamic pressure increment.
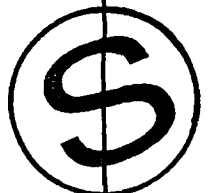
b. For a backfill with an angle of internal friction equal to 33 degrees, the dynamic pressure increment is approximately equal to the inertia force on a soil wedge which extends behind the wall a distance equal to 3/4 the height of the wall.

c. The dynamic pressure increment will act on the wall at a height of 2/3 H above its base.

In the earthquake design of structures where no sophisticated analysis is used to determine soil-structure interaction forces, the following procedures should be followed to develop earth loads for the design of foundation walls.

a. For the particular type of wall restraint and degree of backfill compaction, choose the appropriate static earth pressure from the recommendations given for static earth pressure. This static loading may be considered to act at a height 1/3 H feet above the wall from base to top of soil surface.

b. Apply the dynamic load increment, as developed in the following paragraph, at a point 2/3 H feet above the base of the wall for each linear foot of wall.

The above rules apply to walls where the backfill surface is horizontal.

8.5.2 **Dynamic Lateral Pressure Increment due to Seismic Loading**

The incremental increase of the lateral pressure due to seismic forces for the case of vertical walls and horizontal backfills with no ground water table can be determined using the equation:

$$\Delta_{AE} = 1/2 \ (\gamma_m H^2)(3/4 \ K_h)$$

where:

γ_m = Moist unit weight of backfill

K_h = Ratio of the horizontal ground acceleration to the acceleration due to gravity

H = Height of backfill above base of wall

The dynamic pressure increment Δ_{AE} acts on wall at a height 2/3 H above the base.

For the plant site conditions with design earthquake having a maximum ground acceleration equal to 0.1g the dynamic increments are:
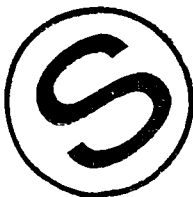
(1-79) WIPP ENG-1-2

a. For Walls Supported on Backfill:
$$\Delta_{AE} = 0.04 \ \gamma_m \ H^2$$

b. For Walls Supported on Caliche or Gatuna:
$$\Delta_{AE} = 0.11 \ \gamma_m \ H^2$$

The increase in the dynamic increment for the caliche and the Gatuna by a factor of 3 is based on results of finite element analysis discussed with Dr. Seed of the University of California in Berkeley.

These dynamic increments should be added to the "at rest" pressure in the case of foundation walls as well as for vertical retaining walls as shown on Figure 28.

9.   SLOPE STABILITY

The site is relatively flat with an average slope of less than 1%. No significant permanent cut slopes or embankments are planned in the vicinity of Design Class I and II structures. The excavation slopes planned within these areas are temporary slopes. These slopes are shown on Figure 7. Outside areas of Design Class I and II structures, permanent slopes will be required in the upper sand such as for site grading and for drainage ditches. These slopes should be 2-1/2 horizontal to 1 vertical. Adequate measures must be taken to protect these slopes against erosion.

9.1   Design Criteria and Analysis

The infinite slope method was used in the slope stability analysis for the upper sand, and seismic forces were neglected.

Based on the infinite slope method a minimum factor of safety of 1.1 was considered acceptable for the temporary slopes and 1.3 for the permanent slopes. The factor of safety for slopes in the upper sand was calculated using the following equation for cohesionless materials:

$$F.S. = \frac{\tan \overline{\phi}}{\tan i}$$

where $\overline{\phi}$ = effective angle of internal friction
      i = the angle of the slope.

For cuts in caliche and Gatuna which are dry materials with cohesion, the factor of safety was calculated using the following equation (Duncan and Buchignani[15]):
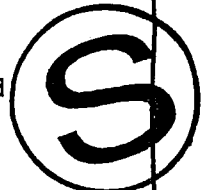
$$F.S. = A\frac{\tan \bar{\phi}}{\tan i} + \frac{B\bar{c}}{\gamma H}$$

where $\bar{c}$ = cohesion
γ = total unit weight
H = Assumed depth of sliding mass in vertical direction.

A = 1 for dry slopes

B = 2.5 for 1/2 horizontal to 1 vertical slope.

The angle of internal friction of the in-situ sand was found to be 33°. However due to the loose nature of the sand, an angle of 30° was used in this analysis. The analysis showed that for sand with slopes of 2 horizontal to 1 vertical the factor of safety is 1.15 and for sand with slopes of 2-1/2 horizontal to 1 vertical the factor of safety is 1.4.

For proposed excavated slopes in the caliche and Gatuna Formation on Figure 7, a weighted average of the strength parameters was used with an angle of internal friction of 39° and a cohesion of 0.7 ksf. The total height for proposed slope is 19 feet (4 feet in caliche + 15 feet in Gatuna). The total unit weight used in the analysis is 102 pcf. For a slope of 1/2 horizontal to 1 vertical in the caliche and Gatuna the factor of safety is 2.1 assuming that the depth of the sliding mass is 10 feet.

10.   FOUNDATION CONSTRUCTION

10.1   Site Grading

Because the construction area is relatively flat, the major site preparation items will be clearing and stripping. The construction area is covered by light woods and grasses which can be cleared easily. During the field investigation, the surface soils were found to be dry and capable of supporting truck mounted equipment. However, repeated movement of rubber-tired equipment over the dry surface will tend to loosen the upper sandy soils and reduce trafficability. It will be desirable to use a layer of 12 inches of caliche as surface for temporary access roads and 6 inches of caliche for work areas. Actual thickness should be determined by experience in the field.

All grading for Design Class I and II and other structures with more than 1.5 ksf loading will require complete removal of the upper sand down to the caliche surface within the limits and to the depths discussed in subsection 10.2.

(1-79) WIPP ENG-1-2
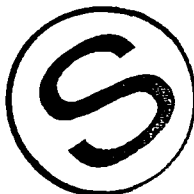
For non-Class I and II lightly loaded structures with less than 1.5 ksf loading the foundation preparation and treatment are discussed in Section 7 of this report. If pockets of clayey or silty material are encountered at foundation level, this material must be removed and replaced with select compacted sand meeting the gradation and compaction criteria given in subsection 10.4.

10.2   Extent of Excavation and Backfill

Most of the structures will have their foundations resting on caliche, Gatuna, or compacted backfill carried down to the caliche surface. The estimated depth of excavation is between 6 and 27 feet as shown on Figure 7. All temporary slopes in the sand should be 2 horizontal to 1 vertical or flatter and in the caliche and Gatuna Formation they should be 1/2 horizontal to 1 vertical. For foundations on comapacted backfill, the bottom of excavation should be determined by extending a line from the periphery of structures at foundation grade with a downward slope of 1 horizontal to 1 vertical or flatter. The intersection of this line with the caliche surface provides the limits of the excavation at the bottom as shown on Figure 7. In addition a minimum distance of 5 feet will be kept between the edge of the structures and the base of excavation slope. This distance was selected for construction consideration as well as for providing adequate compaction near the structure edges.

Excavation of the upper sand can be accomplished by the use of scrapers. Rippers will be required for excavation into caliche and the Gatuna Formation.

Figure 7 shows the location and limits of excavation and structural backfill associated with Design Class II structures at the site. The structural backfill should be compacted to at least 95% of the maximum density determined in accordance with ASTM D 1557. The engineering properties of the sand backfill are provided in Tables 3 and 5. The results of the foundation analysis discussed in Section 7 show that when the sand backfill is compacted to the required density, it will provide an adequate support for all Design Class II structures with large factors of safety for bearing capacity and with tolerable settlements for static and dynamic conditions.

Backfill in non-load bearing areas of the site should be compacted to at least 90% of the maximum dry density determined in accordance with ASTM D-1557.
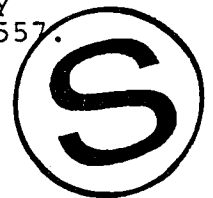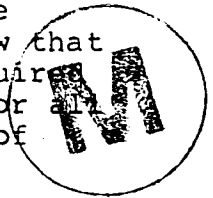
10.3   Dewatering, Slope Protection and Excavation
         Inspection

         During construction, the loose sand should be
         excavated to the top of caliche, approximately at
         Elevations 3390 to 3410. Because the ground water
         level is at a great depth, the only water that might
         be encountered will be surface water due to rainfall.
         Therefore, the only dewatering provision will be to
         intercept and dispose of any surface run-off. No
         permanent dewatering system is required. Sump pump
         and drainage ditches will be adequate to maintain a
         dry excavation and prevent damage to soils during
         construction from rainfall and run-off. The pump
         should be capable of handling rainfall accumulating
         in the excavation.

         Exposed excavations in sand will be susceptible to
         erosion. Therefore, precautions must be taken to
         protect these slopes. All surface run-off should be
         routed around the excavation. Treatment should be
         applied where necessary to minimize slope erosion
         from wind and run-off. Temporary and permanent
         slopes and the slopes of drainage ditches in the
         upper sand can be protected with gunite, asphalt,
         concrete lining, chemical emulsion or seeding with
         native desert grasses depending on the economics of
         the alternative methods.

         The following provisions will be made for the
         identification and removal of unsuitable materials
         for Design Class I and II structures:

         (1) A person experienced in foundation construction
             should inspect all foundation excavations just
             prior to placement of concrete to confirm and
             document that the recommended foundation
             elevation has been reached in the bearing
             stratum. All visual classifications of soils
             should be in accordance with ASTM D 2488.

         (2) The foundation soil will be excavated to the
             caliche surface beneath all Class I and II
             structures. Any loose sand and silty sand at the
             foundation grade will be identified by a person
             experienced in foundation construction and
             removed down to sound caliche. As each section
             of the foundation is approved, approval will be
             documented.

         (3) Since the caliche is hard, there is no special
             requirement for foundation protection. The
             surface of the caliche should be throughly clean
             from all loose material. Surface cracks in the
             caliche should be filled with cement slush grout
             before placement of footings or mat foundations
             directly on the caliche surface.
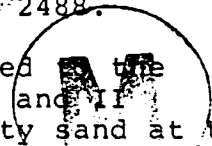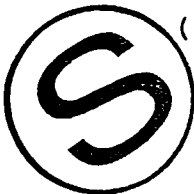
Because the caliche is relatively incompressible, foundation rebound should be negligible. The potential for heave should also be negligible because the caliche and Gatuna materials are non-plastic. Where required, structure settlements can be monitored by optical survey procedures provided that levels are carried back to a deep bench mark that is outside any areas that might be subject to subsidence due to underground construction.

10.4   Structural Backfill

Backfill may be obtained from the excavated upper sand. Only sands with less than 20% passing the U.S. standard sieve No. 200 should be used as structural backfill. Most of the sand excavated from the site is suitable for backfill. The sand should be stockpiled for later use as backfill. During the excavation, undesirable clay or silty material should be removed from the select backfill.

The lift thickness required to place and compact the backfill to the required degree of compaction with the available compaction equipment should be determined in a test fill. The material should be moisture conditioned to the range of $\pm 2\%$ of the optimum moisture content, and compacted to at least 95% of the maximum density determined by ASTM D 1557.

Backfill material within 2 feet of structures and in areas where large construction equipment cannot be used or where there is a danger of damage to structures must be compacted to the specified density with hand-operated equipment. Thinner lifts of 4 inches or less in thickness will be required to do this.

Only suitable material should be placed in the structural backfill. Suitable material for structural backfill should be select sand and silty sand having less than 20% by weight passing the No. 200 U.S. Standard Sieve size. The backfill should not contain any brush, root, peat, sod, or other organic, perishable, or deletrious material, snow, ice, or frozen soil. The bottom of the excavation for Design Class I and II structures should be thoroughly cleaned of all loose material, inspected and documented by a person experienced in foundation construction before placement of sand backfill.

Structural backfill must be placed in horizontal lifts and compacted with a heavy vibratory roller. In confined areas, thinner lifts and hand-operated compactors should be used. The backfill should be raised on both sides of the footings at about the same elevation. The actual lift thickness and the number of passes of the proposed rollers will have to be established by means of a test fill carried out at

the start of backfill operations. All structural backfill must be moisture conditioned just before compaction. All lift surfaces should be inspected and any soft or yielding material should be replaced or corrected.

Field density tests in the compacted backfill and related laboratory compaction testing are required to determine the degree of compaction. Field density tests must be taken in accordance with ASTM D 1556 at a depth of at least 12 in. below the adjacent grade. A compaction test should be carried out in accordance with ASTM D 1557 out for each field density test. These tests will have to be performed at a frequency appropriate for the work in progress but not less than one field density and one compaction test per 1,000 cubic yards of backfill for Class I and II structures and one field density and one compaction test per 2,500 cubic yards of backfill for other structures. In no case should tests for structural backfill be made less frequently than once per shift while backfill is being placed.

To evaluate the applicability of the nuclear tests (ASTM D 2922 and D 3017) for measuring the field density, they should be used in the test fill program in parallel with the sand cone test (ASTM D 1556). If proven satisfactory, these nuclear tests can be used in the backfill provided regular checks are made against densities measured by the sand cone procedure on a basis of at least one out of every ten tests.

All work performed in connection with the placement and compaction of structural backfill must be implemented under a quality assurance program.

A test fill will have to be constructed in order to determine compaction procedures required for Design Class I and II structures to achieve the specified density with the Contractor's equipment. The purpose of the test fill program is to determine the appropriate lift thickness and the number of passes of the vibratory roller required to achieve a minimum of 95% of ASTM D 1557.

The gradation requirements for the fill are discussed in the beginning of this Section. The test fill must be constructed on acceptable caliche foundation. Prior to placement of fill, all low spots and depressions will have to be filled with sand and compacted to the required density to provide a reasonably uniform and horizontal surface. The compaction equipment should be a 10-ton vibratory roller or equivalent and should operate at 1 1/2 mph. The roller should compact when moving forward and not in reverse. The fill should be about 50 ft by 120 ft in plan with side slopes not steeper than 1.5 on 1. The test fill can be divided into three equal
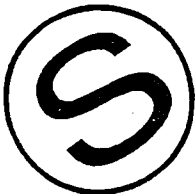
sections. Each section would then be compacted with the number of passes specified below. The moisture content of the fill material should be in the range of 2% above to 2% below the optimum moisture content. Materials having the required moisture content should be placed in 9 inch loose lifts. The lifts will be compacted with the following number of passes:

Section	No. of Passes
1	4
2	6
3	8

The test fill should be raised in a reasonably horizontal plane but sloped sufficiently to drain. At least two lifts should be placed before testing. Additional lifts will then have to be added until 8 lifts have been placed. Additional fills may have to be constructed to evaluate lift thickness or type of vibratory rollers to achieve the required density if this cannot be determined satisfactorily with the initial fill.

The field density tests must be conducted a minimum of 12 inches below the fill surface. At least six field density tests should be made on each lift of each section. Enough material must be obtained from each field density test for compaction tests. The results of all field control tests will then be documented to show compaction equipment, fill section, lift thickness, number of passes, compaction and field density test results, percent compaction, gradation, test location, and elevation.

One compaction test must be performed in accordance with ASTM D 1557 at the location of each field density test. The field density tests should be carried out in accordance with ASTM D 1556. Gradation tests should be made on material from each field density location in accordance with ASTM D 422. The results of all field control tests should be documented and analyzed and the recommended backfill procedure should be used to meet the construction requirements.
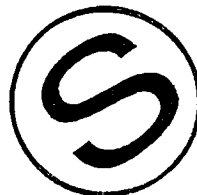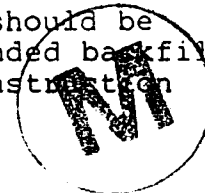
REFERENCES

1. Geologic Characterization Report, Waste Isolation Pilot Plant Site, Southeastern New Mexico, 1978, Powers et al editors, Sandia Laboratories Albuquerque, New Mexico.

2. Seismic Evaluation Report, WIPP Project Document No. 97-D-510-01, Rev. 1, Bechtel National Inc., San Francisco, California (December 1978).

3. Sergent, Hauskins & Beckwith, Seismic Refraction Survey Report, WIPP Site, report to Bechtel Inc., Document No. VP-21-B-02-5-0 (April 1979).

4. Harding-Lawson Associates, Geophysical Investigation P and S Wave Velocities, WIPP Site, report to Bechtel, Inc., Document No. VP-21-B-04-3-0 (1979).

5. R.S. Ladd and M.L. Silver, "Recommended Procedure for Preparing Reconstituted Coarse-Grained Soil Specimens", Preliminary Draft for ASTM Specifications, (1974).

6. H.B. Seed and I.M. Idriss, "Soil Moduli and Damping Factors for Dynamic Response Analyses." Earthquake Engineering Center Report No. EERC-70-10, University of California, Berkeley (December 1970).

7. V.P. Drnevich and B.O. Hardin, "Proposed Standard for Modulus and Damping of Soils by the Resonant Column Method", ASTM Committee D18.09, (May 1974).

8. A.S. Vesic, "Analysis of Ultimate Loads of Shallow Foundations," Journal of Soil Mechanics and Foundations Division, ASCE Proceedings 99, No. SM 1, pp. 45-73 (January 1973).

9. T.W. Lambe and R.V. Whitman, Soil Mechanics, John Wiley & Sons, Inc., New York (1969).

10. H.B. Seed and M.L. Silver, "Settlements of Dry Sands during Earthquakes," Journal of Soil Mechanics and Foundations Division, ASCE Proceedings 98, No. SM4, pp. 381-397 (April 1972).

11. P.B. Schnable, J. Lysmer, and H.B. Seed, "SHAKE, a Computer Program for Earthquake Response Analysis of Horizontally Layered Sites," Earthquake Engineering Research Center Report No. EERC 72-12, University of California, Berkeley, California (December 1972).

12. Bechtel Power Corporation, Topical Report: Seismic Analyses of Structures and Equipment for Nuclear Power Plants, BC-TOP-4-A, Rev. 3, San Francisco, California (November 1974).

13. R.B. Peck, W.E. Hanson, and T.H. Thornburn, <u>Foundation Engineering</u> 2nd Edition, John Wiley & Sons, Inc., New York (1974).

14. H.B. Seed and R.V. Whitman, "<u>Design of Earth Retaining Structures for Dynamic Loads</u>," ASCE Specialty Conference on Lateral Stress in the Ground and Design of Earth Retaining Structures, pp. 103-147, (1970).

15. J.M. Duncan and A.L. Buchignani, <u>An Engineering Manual for Slope Stability Studies</u>. Department of Civil Engineering, University of California, Berkeley (March 1975).

16. Sergent, Hauskins & Beckwith, <u>Report of Subsurface Exploration and Laboratory Testing - WIPP, 3 Volumes</u> (May 1979).

    Vol. 1   Subsurface Exploration & Laboratory Testing, Document No. VP-21-B-02-2-1
    Vol. 2   Visual Soil Classification, Document No. VP-21-B-02-3-1
    Vol. 3   Laboratory Soil Test Results by Dames and Moore, Document No. VP-21-B-02-4-1

17. Photographs of Test Pits, WIPP, 1979, by Sandia.

TABLE. 1.
SOIL TEST RESULTS SUMMARY

JOB NO. 12484   PROJECT WIPP PROJECT   FEATURE FOUNDATION INVESTIGATION - PHASE I   DATE May 1979

HOLE, TEST PIT OR TRENCH NO.	SAMPLE	DEPTH FT. FROM	TO	LABORATORY CLASS.	GRAVEL (%)	SANDS (%)	FINES (%)	LL	PL	PI	SPECIFIC GRAVITY G	WATER CONTENT (%)	TOTAL UNIT WEIGHT PCF	DRY UNIT WEIGHT PCF	OPTIMUM WATER (%)	MAX DRY DENSITY PCF	TEST	WATER (%)	DRY DENSITY PCF	c̄ KSF	φ̄ DEG	DRY DENSITY PCF	K FT/YR	CONSOL-IDATION TEST	REMARKS
B-1	BL-1	0'	9'	SP-SM	0	88	12				2.59														
	P-7	12.1	13.75	SM								9.1	94.5	86.6											Cyclic Triaxial
B-2	S-1	1	2.5	SP-SM	0	92	8				2.63														
	S-2	4'	5.5	SC	0	78	22	22	NP	--	2.47	12	---	---											
	S 4	9	10.5	SM	2	69	29	31	NP	--	2.65	5	---	---											
B-3	S-1	1	2.5	SP-SM	0	93	7				2.65	3	---	---											
	S-3	7.83	8.75	SM	6	61	33	NV	NP	--	2.78	19	---	---											
	P-2	17.67	19.33	SM	0	84	16				2.63 2.65						CD	8.9 6.5	87.4 90.0	0.15	37				Cyclic Triaxial Resonant Column
	P-3	19.33	20.5	SM								5.9 6.5	97.6 101.7	92.2 95.5											
	P-4	20.5	22	SM	0	71	29										CU	10.2 7.6	104.5 111.4	2.82	35				
B-4	P-1	3	5	SM	0	87	13				2.65	4.0	104.0	100.0								100.0	3710		
	S-2	5	6.5	SM	0	86	14	NV	NP	---	2.66	6	---	---											
	P-2	10.5	12.17	SM	0	72	28	NV	NP	---	2.63						CD	9.3 6.4 8.4	83.8 91.3 84.4	0.14	32				
	S-5	19.5	21	SM	0	69	31				2.66	7	--	--											
	P-3	21.5	22	SANDSTONE	2	77	21				2.68														
B-5	BL-1	0	8	SM	0	83	17				2.62														
	S-2	4.5	6	SC	0	76	24				2.66	15	---	---											

SPECIFIC GRAVITY
(a) = MINUS NO 4
(b) = PLUS NO 4

COMPACTION
(1) = ASTM D698
(2) = ASTM D1557
(3) = 20,000 FT. LBS/CU. FT.
(4) = MAXIMUM - MINIMUM
(5) = OTHER (SEE TEXT)

TRIAXIAL COMPRESSION TESTS
UC UNCONFINED COMPRESSION
UU UNCONSOLIDATED UNDRAINED
CU CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)
CU CONSOLIDATED UNDRAINED
CD CONSOLIDATED DRAINED
CR CYCLIC CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)

OTHERS
* VISUAL CLASSIFICATION
** IN-PLACE DENSITY TEST
NV- Non-Valid
NP- Non-Plastic
BL- Bulk-Sample
P- Pitcher Sample
S- Split Spoon Sample

Best Copy Available
HOSCO

TABLE 1

SHEET 2 OF 10

SOIL TEST RESULTS SUMMARY

JOB NO. 12484   PROJECT WIPP PROJECT   FEATURE FOUNDATION INVESTIGATION - PHASE 1   DATE May 1979

HOLE, TEST PIT, OR TRENCH NO.	SAMPLE	DEPTH FT. FROM	TO	LABORATORY CLASS.	MECH. GRAVEL (%)	SANDS (%)	FINES (%)	ATT. LL	PL	PI	SPECIFIC GRAVITY G	NAT. WATER CONTENT (%)	TOTAL UNIT WEIGHT PCF	DRY UNIT WEIGHT PCF	COMP. OPT. WATER (%)	MAX. DRY DENSITY PCF	SHEAR TEST	INITIAL WATER (%)	INITIAL DRY DENSITY PCF	C KSF	Ø DEG	PERM. DRY DENSITY PCF	K FT/YR	CONSOL. TEST	REMARKS
B-5	S-3	8.17	9.83	SM	1	58	41	25	NP	--	2.71	10	---	---											
	P-1	10	11.75	SM	0	80	20	NV	NP	--	2.61						CU	7.6 / 7.5	88.8 / 87.4	0	43				
B-6	BL-1	0	4	SM	0	87	13				2.85														
	S-1	1	2.5	SM				NV	NP	--	2.71	3	--	--											
	P-1	6.5	8.5	SM/ML	0	55	45	NV	NP	--							UU	11.1	83.1						t=1.6ksf at 20.01% strain and σ₃=1ksf Sat.
	P-3	11.25	12.92	SM	0	55	45	31	28	3	2.67						CD	7.0 / 6.6	97.5 / 97.1	0.65	40				
B-7	S-3	9	10.5	SC	3	72	25	42	NP	--	2.79	12	--	--											
B-8	BL-1	0	6	SP-SM	0	90	10				2.82														
	P-5	8	10	SM								8.7 / 8.7	100.9 / 104.4	92.8 / 96.0											Cyclic Triaxial Resonant Column
	P-6	10	12	SM							2.61	7.8 / 12.6	102.8 / 109.1	95.4 / 96.9								95.4	336		Cyclic Triaxial
	P-8	12.92	13.5	SM								6.8	97.1	90.9											Resonant Column
B-9	BL-1	0	4	SP-SM	0	90	10				2.68														
	S-1	1	2.5	SP-SM	0	89	11				2.74	3	--	--											
	P-1	3	5	SP-SM	0	88	12				2.63						CD	3.4 / 6.0	97.6 / 102.5	0	33				
	P-2	8.67	10.25	SM	0	75	25	33	30	3	2.59	10.6 / 13.7	93.1 / 96.3	84.2 / 84.7											Cyclic Triaxial Resonant Column
	S-3	14	15.5	SM	2	75	23	36	NP	--	2.69	11	--	--											
	P-3	19	20	SM								7.1	100.0	93.4											Resonant Column Chemical Analysis

SPECIFIC GRAVITY
(a) = MINUS NO. 4
(b) = PLUS NO. 4

COMPACTION
(1) = ASTM D698
(2) = ASTM D1557
(3) = 20,000 FT. LBS/CU. FT.
(4) = MAXIMUM - MINIMUM
(5) = OTHER (SEE TEXT)

TRIAXIAL COMPRESSION TESTS
UC UNCONFINED COMPRESSION
UU UNCONSOLIDATED UNDRAINED
CU CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)
CU CONSOLIDATED UNDRAINED
CD CONSOLIDATED DRAINED
CR CYCLIC CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)

OTHERS
* VISUAL CLASSIFICATION
** IN-PLACE DENSITY TEST
NV- Non-Valid
NP- Non-Plastic
BL- Bulk-Sample
P- Pitcher Sample
S- Split Spoon Sample

Best Copy Available
HOSCO

SOIL TEST RESULTS SUMMARY

TABLE I

JOB NO. 12484   PROJECT WIPP PROJECT   FEATURE FOUNDATION INVESTIGATION - PHASE I   DATE May 1979

HOLE, TEST PIT, OR TRENCH NO.	SAMPLE	DEPTH FT FROM	DEPTH FT TO	LABORATORY CLASS.	GRAVEL (%)	SANDS (%)	FINES (%)	LL	PL	PI	SPECIFIC GRAVITY G	WATER CONTENT (%)	TOTAL UNIT WEIGHT PCF	DRY UNIT WEIGHT PCF	OPTIMUM WATER (%)	MAX. DRY DENSITY PCF	TEST	INITIAL WATER (%)	INITIAL DRY DENSITY PCF	c̄ KSF	φ̄ DEG	DRY DENSITY PCF	K FT/YR	CONSOL-IDATION TEST	REMARKS
B-10	BL-1	0	6	SP-SM	0	88	12				2.65														
	S-2	4.5	6	SM	0	75	25	NV	NP	--	2.66	14	--	--											
	P-1	11.5	12.92	SM	0	71	29	NV	NP	--		9.4 / 16.3	94.7 / 106.5	86.6 / 91.6											Cyclic Triaxial Resonant Column
	S-5	15.96	17.25	SM	1	72	27	36	NP	--	2.71	12	--	--											
B-11	S-1	1	2.5	SC	0	83	17				2.78														
	S-3	9	10.5	SM	1	71	28				2.79	15	--	--											
	P-3	15	17	SM	0	66	34				2.63														
	P-4	17	18.5	SM								6.5 / 7.9	106.0 / 101.2	99.5 / 93.6											Cyclic Triaxial Resonant Column
B-12	P-2	3	5	SM	0	84	16				2.65	7.0	110.4	103.2								103.2	947		
	P-4	7	9	SM/ML	0	52	48	NV	NP	--	2.59														
B-13	BL-3	0	7	SP-SM / SM	0 / 0	89 / 84	11 / 16	NV	NP	--	2.79		.		5.3	110.0									
	S-1	1	2.5	SP-SM	0	92	8				2.74	2	--	--											
	P-1	3	5	SM								12.4	127.7	113.6								113.6	92		
B-15	S-1	1	2.5	SP-SM	0	91	9				2.70														
	S-3	5.5	7	SM	0	87	13				2.70	4	--	--											
	S-6	14	15.5	SC	1	75	24	NV	NP	--	2.52	12	--	--											
	S-7	19	19.96	SM		72	28	31	NP	--	2.77	8	--	--											

SPECIFIC GRAVITY
(•) = MINUS NO. 4
(b) = PLUS NO 4

COMPACTION
(1) = ASTM D698
(2) = ASTM D1557
(3) = 20,000 FT. LBS/CU. FT.
(4) = MAXIMUM - MINIMUM
(5) = OTHER (SEE TEXT)

TRIAXIAL COMPRESSION TESTS
UC UNCONFINED COMPRESSION
UU UNCONSOLIDATED UNDRAINED
CŪ CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)
CU CONSOLIDATED UNDRAINED
CD CONSOLIDATED DRAINED
CR CYCLIC CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)

OTHERS
✳ VISUAL CLASSIFICATION
✳✳ IN-PLACE DENSITY TEST
NV- Non-Valid
NP- Non-Plastic
BL- Bulk Sample
P- Pitcher Sample
S- Split Spoon Sample

TABLE I

SHEET 4 OF 10

SOIL TEST RESULTS SUMMARY

JOB NO. 12484   PROJECT WIPP PROJECT   FEATURE FOUNDATION INVESTIGATION-PHASE I   DATE May 1979

HOLE, TEST PIT, OR TRENCH NO.	SAMPLE	DEPTH FT FROM	DEPTH FT TO	LABORATORY CLASS.	MECH. GRAVEL (%)	MECH. SANDS (%)	MECH. FINES (%)	ATT. LL	ATT. PL	ATT. PI	SPECIFIC GRAVITY G	NAT. WATER CONTENT (%)	NAT. TOTAL UNIT WEIGHT PCF	NAT. DRY UNIT WEIGHT PCF	COMP. OPTIMUM WATER (%)	COMP. MAX DRY DENSITY PCF	SHEAR TEST	SHEAR INITIAL WATER (%)	SHEAR INITIAL DRY DENSITY PCF	C KSF	φ DEG	PERM. DRY DENSITY PCF	PERM. K FT/YR	CONSOL- IDATION TEST	REMARKS
B-16	P-1	3	5	SM							2.65	8.0	114.6	106.1								106.1	300		
	P-2	8	8.83	SM/ML	0	52	48	NV	NP	--	2.60	24.2	119.4	96.1								96.1	7		
	P-3	16	17.17	SM							2.66	10.7	96.2	86.9								86.9	872		
	P-4	17.17	17.17	SM								6.4	109.6	103.0											Cyclic Triaxial
B-17	BL-1	0	4	SM	0	85	15				2.70														
	P-1	6	8	SM								14.2	109.3	95.7											Cyclic Triaxial
	P-2	8	10	SM								9.4	109.5	100.1											Cyclic Triaxial
B-18	BL-1	0	3	SP-SM	0	90	10				2.74														
	P-2	14	16	SM							2.61	11.5	89.2	80.0								80.0	648		
	P-4	18	20	SM/ML	0	48	52				2.65	19.2	108.2	90.8								90.8	586		
B-19	S-1	1	2.5	SP-SM	0	91	9	NV	NP	--	2.61	7	--	--											
	S-3	9	10	SM	0	63	37	NV	NP	--	2.67	11	--	--											
B-20	BL-1	0	3	SP-SM	0	90	10				2.70				10.0	110.0									
	BL-2	0	3	SP-SM	0	91	9				2.68														
	P-1	3	5	SM								4.7	95.9	91.6										×	
B-21	S-1	1	2.5	SP-SM	0						2.68	4	---	---											
	S-3	8	8.67	SM		75	24	NV	NP	--	2.78	10	---	---											
	S-4	14	15.5	SC	12	55	30				2.75														

SPECIFIC GRAVITY
(a) = MINUS NO. 4
(b) = PLUS NO. 4

COMPACTION
(1) = ASTM D698
(2) = ASTM D1557
(3) = 20,000 FT. LBS/CU. FT.
(4) = MAXIMUM - MINIMUM
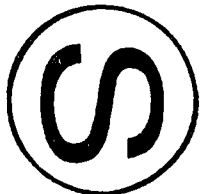(5) = OTHER (SEE TEXT)
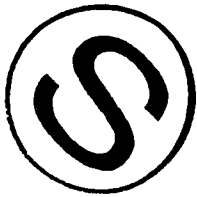
TRIAXIAL COMPRESSION TESTS
UC UNCONFINED COMPRESSION
UU UNCONSOLIDATED UNDRAINED
C̄U CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)
CU CONSOLIDATED UNDRAINED
CD CONSOLIDATED DRAINED
CR CYCLIC CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)

OTHERS
* VISUAL CLASSIFICATION
** IN-PLACE DENSITY TEST
NV- Non-Valid
NP- Non-Plastic
BL- Bulk Sample
P- Pitcher Sample
S- Split Spoon Sample

TABLE 1

SOIL TEST RESULTS SUMMARY

JOB NO. 12484 PROJECT WIPP Project FEATURE Foundation Investigation - Phase I DATE May 1979

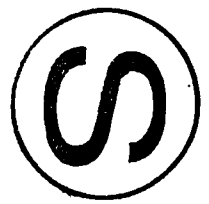| HOLE, TEST PIT, OR TRENCH NO.	SAMPLE	DEPTH FT. FROM	DEPTH FT. TO	LABORATORY CLASS.	MECHANICAL ANALYSIS GRAVEL (%)	SANDS (%)	FINES (%)	ATTERBERG LIMITS LL	PL	PI	SPECIFIC GRAVITY G	NATURAL WATER CONTENT (%)	TOTAL UNIT WEIGHT PCF	DRY UNIT WEIGHT PCF	COMPACTION OPTIMUM WATER (%)	MAX DRY DENSITY PCF	SHEAR DATA TEST	INITIAL WATER (%)	INITIAL DRY DENSITY PCF	C KSF	φ DEG	PERMEABILITY DRY DENSITY PCF	K FT/YR	CONSOLIDATION TEST	REMARKS
B-21	S-6	24	25.46	SM	1	80	19				2.65	5	-	-											
B-22	BL-1	0	3	SP-SM	0 / 0	88 / 89	12 / 11				2.78 / 2.87				6.2	110.3									
	P-1	3	5	SM	0	85	15				2.68	6.1	111.4	105.0								105.0	919		
	S-2	5.17	6.67	SM	1	70	29	29	20	9	2.78	11	-												
B-23	P-2	3	5	SM	1	84	15				2.66	5.3	103.3	98.1				10.2	88.2			98.1	2205		t=5.7 ksf at 4.67% Strain and σ3=1 ksf Nat.Moist.
	P-4	7	9	SM	0	65	35				2.66	18.9	100.2	84.3			UU	11.5	88.4			84.3	62		t=5.5 ksf at 6.11% Strain and σ3=1 ksf Sat.
	S-6	10	10.92	SC	0	71	29	24	NP		2.57	9	-	-											
B-24	S-1	1	2.5	SP-SM	0	91	9				2.67														
	S-3	5	6.5	SC	6	62	32	25	NP	-	2.76	16	-	-											
	S-6	14	14.46	SM								12	-	-											
B-26	BL-1	0	3	SP-SM	0	91	9				2.75														
	P-6	7.5	9.5	SM								5.8	102.4	96.8											Cyclic Triaxial
	P-7	9.5	11.5	SM								12.3	99.7	88.8											Cyclic Triaxial
B-27	BL-1	3	4	SM	0	84	16				2.87														
	P-4	6.67	8.67	SM													UU	14.8	84.8						t=2.4 ksf at 1.71% Strain and σ3=2 ksf Nat.Moist.
	P-5	8.67	10.5	SM													UU	11.1	82.4						t=3.1 ksf at 6.54% Strain and σ3=6 ksf Sat.

SPECIFIC GRAVITY
(a) = MINUS NO. 4
(b) = PLUS NO. 4

COMPACTION
(1) = ASTM D698
(2) = ASTM D1557
(3) = 20,000 FT. LBS/CU. FT.
(4) = MAXIMUM - MINIMUM
(5) = OTHER (SEE TEXT)

TRIAXIAL COMPRESSION TESTS
UC UNCONFINED COMPRESSION
UU UNCONSOLIDATED UNDRAINED
C̄U CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)
CU CONSOLIDATED UNDRAINED
CD CONSOLIDATED DRAINED
CR CYCLIC CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)

OTHERS
* VISUAL CLASSIFICATION
** IN-PLACE DENSITY TEST
NV- Non-Valid
NP- Non-Plastic
BL- Bulk Sample
P- Pitcher Sample
S- Split Spoon Sample

TABLE 1

SHEET 6 OF 10

SOIL TEST RESULTS SUMMARY

JOB NO. 12484   PROJECT WIPP Project   FEATURE Foundation Investigation - Phase I   DATE May 1979

HOLE, TEST PIT OR TRENCH NO.	SAMPLE	DEPTH FT FROM	TO	LABORATORY CLASS.	MECH. GRAVEL (%)	SANDS (%)	FINES (%)	ATT. LL	PL	PI	SPECIFIC GRAVITY G	NATURAL WATER CONTENT (%)	TOTAL UNIT WEIGHT PCF	DRY UNIT WEIGHT PCF	COMP. OPTIMUM WATER (%)	MAX DRY DENSITY PCF	SHEAR TEST	INITIAL WATER (%)	INITIAL DRY DENSITY PCF	C KSF	φ DEG	PERM. DRY DENSITY PCF	K FT/YR	CONSOL. TEST	REMARKS
B-28	BL-1	0	4	SP-SM	0	91	9				2.69				10.5	110.6									Chemical Analysis Resonant Column
	P-5	8.17	10.17	SM								12.1	104.4	93.1											Cyclic Triaxial
	P-6	10.17	11.83	SM	0	70	30	NV	NP	-															
B-29	BL-1	0	6	SP-SM	0	92	8				2.54				13.2	108.6						103.2 Compacted Sample	220		Resonant Column (2 Tests) Cyclic Triaxial(8 Tests)
	P-7	12.5	13.5	SM																					Chemical Analysis
B-30	BL-1	0	4	SP-SM	0	90	10				2.48														
	P-5	7.5	9.5	SC								17.4 16.4	109.3 108.0	93.1 92.8											Cyclic Triaxial Resonant Column
	P-6	9.5	10.58	SC													UU	18.0	94.1						t=3.3 ksf at 3.09% Strain and σ₃=1 ksf Nat.Moist.
	P-7	10.58	12.58	SC								14.2	95.1	83.3											Cyclic Triaxial
B-31	BL-1	0	4.25	SP-SM	0	89	11				2.66				8.0	112.2	CD	8.0 7.8 8.0	105.3 105.8 105.7	0	33				Resonant Column
B-32	BL-1	0	10	SM	0	64	36				2.72				12.8	108.7	CD	13.1 13.1 13.0	102.4 102.4 102.4	0	33.5	103.3 Compacted Sample	181		Cyclic Triaxial-12 Tests Resonant Column-2 Tests
	P-8	15	15.75	SM	0	63	37																		
B-33	BL-1	0	6	SP-SM	0	81	9				2.73														
	P-5	8	9	SM								11.1	99.3	89.4											Cyclic Triaxial
	P-6	9	10.08	SM	0	67	33	NV	NP	-		6.6	93.7	87.9											Cyclic Triaxial
	P-7	10.08		SM								3.8	99.6	96.0								96.0	644		

Best Copy Available
HOSCO

TABLE 1
SOIL TEST RESULTS SUMMARY

JOB NO. 12484   PROJECT WIPP Project   FEATURE Foundation Investigation – Phase I   DATE May 1979

HOLE, TEST PIT, OR TRENCH NO.	SAMPLE	DEPTH FT. FROM	DEPTH FT. TO	LABORATORY CLASS.	MECHANICAL ANALYSIS GRAVEL (%)	SANDS (%)	FINES (%)	ATTERBERG LIMITS LL	PL	PI	SPECIFIC GRAVITY G	NATURAL WATER CONTENT (%)	TOTAL UNIT WEIGHT PCF	DRY UNIT WEIGHT PCF	COMPACTION OPTIMUM WATER (%)	MAX. DRY DENSITY PCF	SHEAR DATA TEST	INITIAL WATER (%)	DRY DENSITY PCF	c̄ KSF	φ̄ DEG	PERMEABILITY DRY DENSITY PCF	K FT/YR	CONSOLIDATION TEST	REMARKS
B-34	BL-1	0	6	SP-SM	0	89	11				2.74				12.8	110.4	CD	12.2 / 12.7 / 12.8	103.9 / 104.9 / 104.7	0	33				Resonant Column-2 Tests
	P-6	11	12.33	SM	0	66	34					8.9	103.1	94.7											Cyclic Triaxial
B-35	BL-1	0	5	SP-SM	0	88	12				2.47														Cyclic Triaxial
	P-5	8	10	SM								11.9 / 11.8 / 11.0	102.7 / 101.7 / 101.7	91.8 / 91.0 / 91.6											Resonant Column-2 Tests
	P-6	10	11	SM								9.2 / 6.8	111.3 / 106.4	101.3 / 99.6											Cyclic Triaxial
	P-8	11.67	13.67	SM								5.9 / 5.2 / 4.8	110.1 / 111.6 / 109.1	104.0 / 106.0 / 104.1											Cyclic Triaxial
	P-9	13.67	15.42	SM								6.6	98.5	92.4											Cyclic Triaxial
	P-10	15.42	16.75	SM								8.6	100.0	92.1			UU	6.4	89.3						Resonant Column UU-t=1.6ksf at 1.05% Strain and σ₃=1ksf Nat. Moist.
B-36	BL-1	0	3.5	SP-SM	0	89	11				2.55														
	P-8	17.5	19.5	SM	0	73	27										CD	5.9 / 5.2 / 4.8	104.0 / 106.1 / 104.1	4.04	28				
B-37	BL-1	0	3	SP-SM	0	91	9				2.81														
	P-5	11.33	13.33														UU	5.6	94.4						t=9.5ksf at 7.39% Strain and σ₃=6ksf Nat. Moist.
B-37A	BL-1	0	3	SP-SM	0		11				2.78														
B-38	BL-1	0	3.5	SP-SM	0	89	11				2.72														
	P-5	11.42	13.42	SM	0	69	31										UU	8.1	85.1						t=1.8ksf at 10.01% Strain and σ₃=6ksf Sat.

SPECIFIC GRAVITY
(a) = MINUS NO. 4
(b) = PLUS NO 4

COMPACTION
(1) = ASTM D698
(2) = ASTM D1557
(3) = 20,000 FT. LBS/CU. FT.
(4) = MAXIMUM - MINIMUM
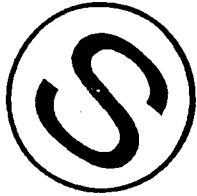(5) = OTHER (SEE TEXT)

TRIAXIAL COMPRESSION TESTS
UC UNCONFINED COMPRESSION
UU UNCONSOLIDATED UNDRAINED
C̄U CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)
CU CONSOLIDATED UNDRAINED
CD CONSOLIDATED DRAINED
CR CYCLIC CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)

OTHERS
* VISUAL CLASSIFICATION
** IN-PLACE DENSITY TEST
NV- Non-Valid
NP- Non-Plastic
BL- Bulk Sample
P- Pitcher Sample
S- Split Spoon Sample

TABLE 1
SHEET 8 OF 10

SOIL TEST RESULTS SUMMARY

JOB NO. 12484   PROJECT WIPP Project   FEATURE Foundation Investigation - Phase I   DATE May 1979

HOLE, TEST PIT, OR TRENCH NO.	SAMPLE	DEPTH FT. FROM	DEPTH FT. TO	LABORATORY CLASS.	MECH. GRAVEL (%)	MECH. SANDS (%)	MECH. FINES (%)	LL	PL	PI	SPECIFIC GRAVITY G	NATURAL WATER CONTENT (%)	NATURAL TOTAL UNIT WEIGHT PCF	NATURAL DRY UNIT WEIGHT PCF	COMPACTION OPT. WATER (%)	COMPACTION MAX. DRY DENSITY PCF	SHEAR TEST	SHEAR INITIAL WATER (%)	SHEAR INITIAL DRY DENSITY PCF	C KSF	φ DEG	PERM. DRY DENSITY PCF	K FT/YR	CONSOL. TEST	REMARKS
B-39	BL-1	0	7	SP-SM	0	88	12				2.54														
B-40	BL-1	0	4	SP-SM	0	89	11				2.67														
B-47	P-5	9.75	10.42	SM	0	69	31										UU	15.8	89.4						t=17.0 ksf at 2.34% Strain and σ₃=1 ksf Sat.
	P-7	11.08	12.08	SM								17.2	89.1	76.0											Cyclic Triaxial
B-48	P-3	11.67	13.67	SM	0	66	34																		
B-49	P-4	8.5	10.5	SM/ML	0	54	46										CD	27.4 / 26.2 / 31.4	86.3 / 88.9 / 83.6	2.58	31				
	P-5	10.5	11.17	SM								29.3	117.3	90.7											Resonant Column
	P-7	13.17	15.17	SM								20.6	105.9	87.8											Cyclic Triaxial
	P-8	15.17	17.17	SM	0	58	42																		
	P-9	17.17	19.17	SM													UU	9.1	92.7						t=1.4 ksf at 0.81% Strain and σ₃=2 ksf Sat.
B-50	P-5	11.17	13.17	SM	0	60	40	32	19	13							CD	19.9 / 12.7	100.0 / 101.6	0.94	33				
	P-7	14.75	15.75	SM								7.1	95.6	89.3											Cyclic Triaxial
B-51	P-6	11.25	12.5	SM													UU	9.7	97.9						t=13.0 ksf at 2.63% Strain and σ₃=2 ksf Sat.
B-52	P-4	5.67	7.67	SM													UU	10.1	83.7						t=6.5 ksf at 3.44% Strain and σ₃=2 ksf Nat. Moisture

SPECIFIC GRAVITY
(a) = MINUS NO. 4
(b) = PLUS NO 4

COMPACTION
(1) = ASTM D698
(2) = ASTM D1557
(3) = 20,000 FT. LBS/CU FT.
(4) = MAXIMUM - MINIMUM
(5) = OTHER (SEE TEXT)

TRIAXIAL COMPRESSION TESTS
UC UNCONFINED COMPRESSION
UU UNCONSOLIDATED UNDRAINED
CU CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)
CU CONSOLIDATED UNDRAINED
CD CONSOLIDATED DRAINED
CR CYCLIC CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)

OTHERS
* VISUAL CLASSIFICATION
** IN-PLACE DENSITY TEST
NV- Non-Valid
NP- Non-Plastic
BL- Bulk Sample
P- Pitcher Sample
S- Split Spoon Sample

HOSCO

TABLE 1
SOIL TEST RESULTS SUMMARY

JOB NO. 12484   PROJECT WIPP Project   FEATURE Foundation Investigation - Phase II   DATE May 1979

HOLE, TEST PIT OR TRENCH NO.	SAMPLE	DEPTH FT. FROM	DEPTH FT. TO	LABORATORY CLASS.	MECH. GRAVEL (%)	MECH. SANDS (%)	MECH. FINES (%)	LL	PL	PI	SPECIFIC GRAVITY G	NAT. WATER CONTENT (%)	NAT. TOTAL UNIT WEIGHT PCF	NAT. DRY UNIT WEIGHT PCF	COMP. OPTIMUM WATER (%)	COMP. MAX DRY DENSITY PCF	SHEAR TEST	SHEAR INITIAL WATER (%)	SHEAR INITIAL DRY DENSITY PCF	C KSF	φ DEG	PERM. DRY DENSITY PCF	PERM. K FT/YR	CONSOL. TEST	REMARKS
TP-1	-	0	-	SP								3.6	105.2	101.5											
	BL-1	3.2	3.7	SP-SM	0	90	10				2.64	7.0	103.5	96.9											Relative Density
	BL-2	8.6	9.1	Caliche								18.7	108.4	91.3											
	BL-3	13.7	14.2	Caliche								8.1	103.4	95.7											
	BL-4	21.0	21.5	Sandstone								5.7	110.5	104.5											
TP-2	-	0	-	SM								3.0	105.2	102.1											
	BL-1	3	3.5	SM	0	87	13				2.64	7.2	117.9	110.0											Relative Density
	BL-2	5.5	6	Caliche								17.3	109.9	93.7	16.1	109.9	CU	16.7 / 15.8 / 16.5	103.3 / 105.0 / 103.7	2.0	34	103.7	3		Compacted to 95% Maximum density
	BL-3	11	11.5	Sandstone								13.7	117.4	103.3	12.9	117.8	CU	13.1 / 13.0 / 12.8	110.4 / 110.4 / 111.2	0.4	31	111.2	34		Compacted to 95% maximum density
TP-3	-	0	-	SP								3.5	112.4	108.6											
	BL-1	3.2	3.7	SM	0	81	19				2.65	3.6	119.7	115.5											Relative Density
	BL-2	5.5	6.0	Caliche								11.0	109.7	98.8											
	BL-3	5.2	5.7	Caliche								12.8	108.5	96.2											
	BL-4	9.2	9.7	Caliche								5.7	115.7	109.5											
	BL-5	8.5	9	Caliche								11.3	112.8	101.3											
	BL-6	16.2	16.7	Sandstone								12.6	114.0	101.3											

SPECIFIC GRAVITY
(a) = MINUS NO. 4
(b) = PLUS NO. 4

COMPACTION
(1) = ASTM D698
(2) = ASTM D1557
(3) = 20,000 FT. LBS/CU. FT.
(4) = MAXIMUM - MINIMUM
(5) = OTHER (SEE TEXT)

TRIAXIAL COMPRESSION TESTS
UC UNCONFINED COMPRESSION
UU UNCONSOLIDATED UNDRAINED
$\overline{CU}$ CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)
CU CONSOLIDATED UNDRAINED
CD CONSOLIDATED DRAINED
CR CYCLIC CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)

OTHERS
* VISUAL CLASSIFICATION
** IN-PLACE DENSITY TEST
NV- Non-Valid
NP- Non-Plastic
BL- Bulk Sample
P- Pitcher Sample
S- Split Spoon Sample

TABLE 1   SHEET 10 OF 10

SOIL TEST RESULTS SUMMARY

JOB NO. **12484**   PROJECT **WIPP Project**   FEATURE **Foundation Investigation – Phase II**   DATE **May 1979**

HOLE, TEST PIT, OR TRENCH NO.	SAMPLE	DEPTH FT FROM	DEPTH FT TO	LABORATORY CLASS.	GRAVEL (%)	SANDS (%)	FINES (%)	LL	PL	PI	SPECIFIC GRAVITY G	NATURAL WATER CONTENT (%)	NATURAL TOTAL UNIT WEIGHT PCF	NATURAL DRY UNIT WEIGHT PCF	OPTIMUM WATER (%)	MAX. DRY DENSITY PCF	TEST	INITIAL WATER (%)	INITIAL DRY DENSITY PCF	C̄ KSF	ø̄ DEG	DRY DENSITY PCF	K FT/YR	CONSOL-IDATION TEST	REMARKS
TP-4	BL-1	0 / 3	– / 3.5	SP / SP-SM	0	91	9				2.63	3.6 / 3.5	111.2 / 110.7	107.3 / 107.0											Relative Density
	–	–	5																						Chemical Analysis
	BL-2	7.5	8	SM	0	79	21				2.65	13.2	106.4	94.0											Relative Density
	–	–	10																						Chemical Analysis
	BL-3	13.5	14	Caliche								7.0	111.1	103.8	13.3	112.0	CD	13.1 / 12.9 / 13.0 / 12.7	105.8 / 106.6 / 106.4 / 104.8	2.0	35	106.4	10		Compacted to 95% Maximum density
	BL-4	29.5	30	Sandstone								7.0	122.6	114.6	13.4	111.0	CD	13.0 / 12.3	105.6 / 106.3	1.6	36	106.3	66		Compacted to 95% maximum density
TP-5	BL-1	0 / 3.5	– / 4	SP / SP-SM	0	89	11				2.62	3.3 / 3.4	101.4 / 112.6	98.2 / 108.9											Relative Density
	BL-2	10.6	11.1	Caliche								11.5	114.1	102.3											
	BL-3	9.9	10.4	Caliche								11.9	108.5	96.9											
	BL-4	13.9	14.4	Caliche								5.1	117.3	111.6											
	BL-5	20.9	21.4	Sandstone								7.4	113.1	105.3											
Hand Dug 116 Ft North 55° East of B-25	–	3.0	3.5	SP								4.1	104.6	100.5											

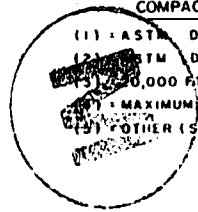**SPECIFIC GRAVITY**
(a) = MINUS NO. 4
(b) = PLUS NO. 4

COMPACTION
(1) = ASTM D698
(2) = ASTM D1557
(3) = 20,000 FT. LBS/CU. FT.
(4) = MAXIMUM - MINIMUM
(5) = OTHER (SEE TEXT)

TRIAXIAL COMPRESSION TESTS
UC UNCONFINED COMPRESSION
UU UNCONSOLIDATED UNDRAINED
CU CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)
CU CONSOLIDATED UNDRAINED
CD CONSOLIDATED DRAINED
CR CYCLIC CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)

OTHERS
* VISUAL CLASSIFICATION
** IN-PLACE DENSITY TEST
NV- Non-Valid
NP- Non-Plastic
BL- Bulk Sample
P- Pitcher Sample
S- Split Spoon Sample

Table 2

## ENGINEERING PROPERTIES OF IN-SITU SOIL AND SHALLOW ROCK

Test	Unit	Sand	Caliche	Gatuna
Specific gravity	(Dimensionless)	2.47-2.87	2.57-2.79	2.52-2.77
In-situ moist density	(pcf)	101-128	89-120	95-115
In-situ dry density	(pcf)	97-115	76-102	83-111
Compressive strength $(\sigma_1-\sigma_3)$ (psf)				
Natural moisture		-	4,700-13,000	3,300-19,000
Saturated		-	3,100-34,000	2,800
Consolidated undrained effective shear strength				
$\bar{c}$	(ksf)	-	0	2,82
$\bar{\phi}$	(deg)	-	43	35
Consolidated drained effective shear strength				
$\bar{c}$	(ksf)	0	0.14-2.58	0.15-4.04
$\bar{\phi}$	(deg)	33	31-33	28-40
Modulus of elasticity				
Static	(ksf)	140-690	1,870-14,110	2,160-2,740
Dynamic	(ksf)		See Table 7	
Shear modulus				
Static	(ksf)		See Table 7	
Dynamic	(ksf)		See Table 7.	
Damping ratio	(%)	See Figures 23 through 25		
Poisson's ratio	(Dimensionless)	0.34	0.35	0.34
Permeability	(Ft/yr)	92-3,710	7-648	587-872

Note: The compressive strength results were rounded to the nearest
100 psf. The modulus of elasticity was rounded to the nearest
10 ksf. The densitites were rounded to the nearest 1 pcf.
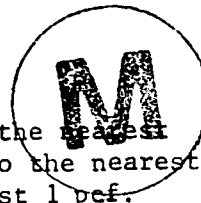
Table 3
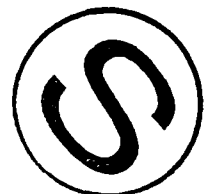## ENGINEERING PROPERTIES OF SAND BACKFILL

Test		Unit	
ASTM D 1557 maximum dry density		(pcf)	109 –112
ASTM D 2049 maximum dry density		(pcf)	113 –118
ASTM D 2049 minimum dry density		(pcf)	84 – 90
Optimum moisture content		(%)	5.3–13.2
Consolidated drained effective shear			
strength			
	$\bar{c}$	(psf)	0
	$\bar{\phi}$	(deg)	33–33.5
Modulus of elasticity			
Static		(ksf)	520
Dynamic			See table 7
Shear modulus		(ksf)	See table 7
Damping ratio		(%)	See Figure 23
Poisson's ratio		(Dimensionless)	0.34

Note: The dry density was rounded to the nearest 1 pcf, and the modulus of elasticity to the nearest 10 ksf.
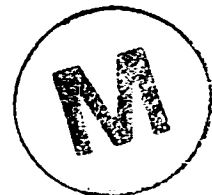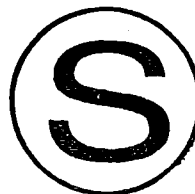
Table 4

DESIGN PROPERTIES OF IN-SITU SOIL AND SHALLOW ROCK

Test	Unit	Sand	Caliche	Gatuna
Specific gravity	(Dimensionless)	2.68	2.68	2.65
Average moist density	(pcf)	110.0	102.0	103.0
Average dry density	(pcf)	103.0	90.0	96.0
Effective shear strength				
$\bar{c}$	(ksf)	0	0.9	0.65
$\bar{\phi}$	(deg)	30	33	40
Modulus of elasticity				
Static	(ksf)	140	3,600	2,100
Dynamic		See Table 7		
Shear modulus				
Static	(ksf)	50	1,300	800
Dynamic		See Table 7		
Damping ratio	(%)	See Figures 23 through 25		
Poisson's ratio	(Dimensionless)	0.34	0.35	0.34
Shear Wave Velocity	(ft/sec)	500	1,300	1,600
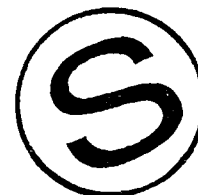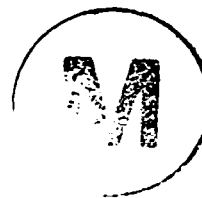
Table 5

DESIGN PROPERTIES OF SAND BACKFILL

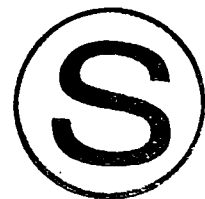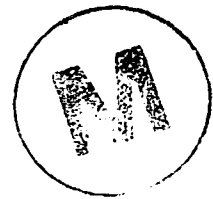| Test	Unit	
Specific gravity	(Dimensionless)	2.63
ASTM D 1557 maximum dry density	(pcf)	110.0
Optimum moisture content	(%)	9.8
Average moist density	(pcf)	121.0
Effective shear strength		
$\bar{c}$	(ksf)	0
$\bar{\phi}$	(deg)	33
Modulus of elasticity		
Static	(ksf)	500
Dynamic		See Table 7
Shear modulus		
Static	(ksf)	200
Dynamic		See Table 7
Damping ratio	(%)	See Figure 23
Poisson's ratio	(Dimensionless)	0.34

Table 6

NEAR SURFACE SEISMIC VELOCITIES AND ELASTIC MODULI

Material	Depth (Ft)	"P" Wave Velocity (ft/sec)	"S" Wave Velocity (ft/sec)	Moist Density lbs/ft^3	Poisson's Ratio	Young's Modulus (ksf)	Shear Modulus (ksf)	Bulk Modulus (ksf)
In-situ Sand	0–13	1,100–1,800	450–900	110	0.34	1,900–7,400	700–2,800	1,900–7,700
Caliche	7–20	2,000–4,000	1,000–1,900	102	0.35	8,600–30,900	3,200–11,400	9,500–34,300
Gatuna	12–43	2,900–4,700	1,300–2,200	103	0.34	14,500–41,500	5,400–15,500	15,100–43,200
Santa Rosa	32–53	3,300–5,200	1,300–3,000	132	0.34	18,500–98,600	6,900–36,800	19,300–102,700
Dewey Lake Redbeds	40–100	3,500–9,400	1,800–3,900	142	0.38	39,500–185,500	14,300–67,200	54,900–257,700
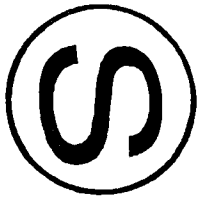
Table 7

DESIGN DYNAMIC AND STATIC SHEAR AND ELASTIC MODULI

| | ELASTIC MODULUS (ksf) | | SHEAR MODULUS (ksf) | |
| | | E max* | | G max* |
Material	Static	Dynamic	Static	Dynamic
Sand Backfill	500	3,500 (σ_3= 1,000 psf)	200	1,300 (σ_3= 1,000 psf)
Sand Backfill	-	4,700 (σ_3= 2,000 psf)	-	1,800 (σ_3= 2,000psf)
Sand Backfill	-	8,700 (σ_3= 6,000 psf)		3,200 (σ_3= 6,000 psf)
In-situ Sand	140	2,300	50	800
Caliche	3,600	14,400	1,300	5,300
Gatuna	2,100	22,000	800	8,200

*Notes:

1. The variation of the dynamic shear moduli, G, with strain will be determined from $G_{Design} = \dfrac{G_\gamma}{G_{\gamma =10^{-5}\%}} \times G_{max}$ where G_γ and $G_{\gamma = 10^{-5}\%}$ are provided in Figures 20 through 22.

2. The variation of the dynamic elastic moduli, E, with the dynamic shear moduli, G, will be determined using E= 2G (1+ μ) where μ is given in Tables 4 and 5.
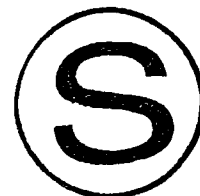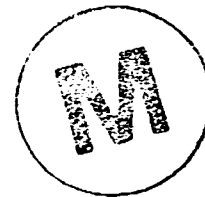
3. σ_3= Confining pressure.

Table 8

RELATIVE DENSITY OF IN-SITU SAND

Test Pit	Sample	Depth Ft.	In-Situ Dry Density γ_d pcf	Minimum Density γ_{min} pcf	Maximum Density γ_{max} pcf	Relative Density D_r %	Elastic Modulus E psi
TP-1	BL-1	3.2 -3.7	97	89	113	39	-
TP-2	BL-1	3.0 -3.5	110	84	118	81	-
TP-4	BL-1	3.0 -3.5	107	86	115	78	
TP-5	BL-1	3.5-4.0	109	90	114	81	3,200

Note: The densities were rounded to the nearest 1 pcf.
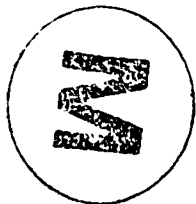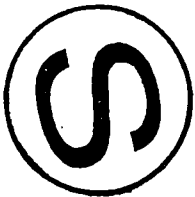
Table 9

D_{10} SIZE AND COEFFICIENT OF UNIFORMITY OF TESTED SAND BACKFILL

Boring	Sample	D_{10} mm	Cu	Comments
B-1	BL-1	.035	7.7	
B-3	BL-1	-	-	
B-5	BL-1	.0025	80	
B-6	BL-1	.017	16.5	
B-8	BL-1	.074	2.8	
B-9	BL-1	.07	2.9	
B-10	BL-1	.076	2.6	
B-13	BL-3	.066	3.2	
B-13	BL-3	.02	10.8	
B-17	BL-1	.015	12.7	
B-18	BL-1	.075	2.9	
B-20	BL-1	.075	2.8	
B-20	BL-2	.08	3.1	
B-22	BL-1	.065	3.2	
B-22	BL-2	.068	3.1	
B-26	BL-1	.079	2.8	
B-27	BL-1	<.001	>220	
B-28	BL-1	.075	2.4	used for cyclic tests
B-29	BL-1	.088	2.2	used for cyclic tests
B-30	BL-1	.05	3.4	
B-31	BL-1	.062	3.4	used for cyclic tests
B-32	BL-1	.031	6.1	used for cyclic tests
B-33	BL-1	.045	4.9	
B-34	BL-1	.044	6.4	used for cyclic tests
B-35	BL-1	.07	4.3	
B-36	BL-1	.072	2.9	
B-37	BL-1	.072	2.9	
B-37A	BL-1	.062	3.5	
B-38	BL-1	.064	3.4	
B-39	BL-1	.045	4.7	
B-40	BL-1	.065	3.2	

NOTE:

1. Cu = coefficient of uniformity $\dfrac{D60}{D10}$

   where,

   D60= particle size corresponding to 60 percent by weight passing on the particle size distribution curve.

   D10= particle size corresponding to 10 percent by weight passing on the particle size distribution curve.
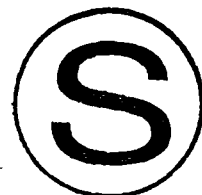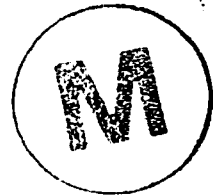
Table 10

CHEMICAL ANALYSES OF FOUNDATION MATERIALS*

| Boring Test Pit No.	Sample	Depth Ft	Material	Silica	Ferric Oxide	Aluminum Oxide	Calcium Oxide	Magnesium Oxide	Loss on Ignition	Sodium Oxide	Potassium Oxide	Sulfur Trioxide	Carbonate	Chloride
B-28	BL-1	0-4	Brown Sand	93.24	1.02	2.83	0.16	0.20	1.44	0.22	0.72	0.042	0.80	0.05
TP-4	-	5	Tan Sand	89.62	1.46	3.81	1.36	0.27	1.88	0.34	0.88	0.045	0.59	0.04
TP-4	-	10	White Sand	84.64	0.87	2.86	5.24	0.26	5.02	0.19	0.55	0.115	4.72	0.03
B-29	P-7	12.5-13.5	Caliche	55.12	2.78	3.57	17.74	2.40	16.71	0.28	1.17	0.101	16.19	<0.01
B-9	P-3	19-20	Gatuna	79.76	4.17	5.92	2.14	1.71	3.96	0.31	1.70	0.065	3.25	0.05

*Shown as a percentage by weight.
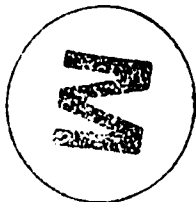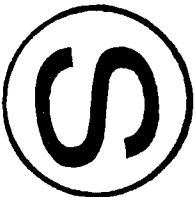
Table 11

PLATE LOAD TEST RESULTS

Test Pit	Test No.	Material	Depth (Ft.)	Dry Density (pcf)	Water Content (%)	Plate Diameter (In)	Modulus of Subgrade Reaction (pci)	Elastic Modulus (psi)
TP-3	1	Sand	3.2	-	-	30	200	4,800
TP-3	2	Sand	3.2	115	3.6	18	225	2,800
TP-3	3	Caliche top	5.5	100	11.0	18	2,500	31,000
TP-3	4	Caliche top	5.2	96	12.8	12	3,200	26,000
TP-3	5	Caliche middle	9.2	109	5.7	12	11,890	98,000
TP-3	6	Caliche middle	8.5	101	11.3	12	1,850	15,000
TP-3	7	Gatuna top	16.2	-	-	12	2,290	19,000
TP-3	8	Gatuna top	16.2	101	12.6	12	1,850	15,000
TP-5	1	Sand	3.8	-	-	30	165	3,400
TP-5	2	Sand	3.6	109	3.4	30	155	3,200
TP-5	3	Caliche top	10.6	102	11.5	12	1,620	13,000
TP-5	4	Caliche top	9.9	97	11.9	12	2,990	25,000
TP-5	5	Caliche middle	13.9	-	-	12	5,560	46,000
TP-5	6	Caliche middle	13.9	112	5.1	12	4,420	37,000
Hand-dug Hole, 116 ft NE of B-25		Sand	3.0	100	4.1	30	47	1,000

Note: The dry densities were rounded to the nearest 1 pcf.
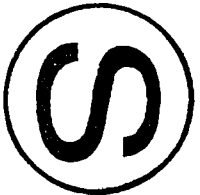
Table 12

NET APPLIED PRESSURES AND
NET ULTIMATE BEARING PRESSURES

Structures	Foundation Dimensions (ft x ft)	Net Ultimate Bearing Pressure (Ksf)	Net Applied Static Pressure (Ksf)	Factor of Safety
Waste Handling Building				
Hot Cell Mat	40x113	116	4	29
Braced Bay Footing	12x76	73	5	15
Column Footings	18x18	78	5	16
Column Footings	8x8	68	5	14
Storage Exhaust Filter Building				
Column Footings	12x12	72	3	24
Column Footings	6x12	64	3	21
Emergency Generator Building				
Column Footings	12x12	34	3	11
Column Footings	6x12	29	3	10
Continuous Footings	5x22	28	3	9
Administration Building (Control Room)				
Column Footings	8x8	21	2	10
Column Footings	6x6	19	2	9

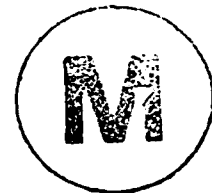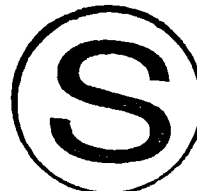## Table 13

SETTLEMENTS OF FOUNDATIONS

Structures	Foundation Dimensions (ft x ft)	Net Applied Static Pressure (Ksf)	Settlement (in)
Waste Handling Building			
Hot Cell Mat	40x113	4	1.0
Braced Bay Footing	12x76	5	0.5
Column Footings	18x18	5	0.4
Column Footings	8x8	5	0.2
Storage Exhaust Filter Building			
Column Footings	12x12	3	0.2
Column Footings	6x12	3	0.1
Emergency Generator Building			
Column Footings	12x12	3	0.5
Column Footings	6x12	3	0.4
Continuous Footings	5x22	3	0.4
Administration Building (Control Room)			
Column Footings	8x8	2	0.2
Column Footings	6x6	2	0.2
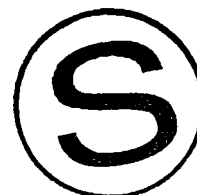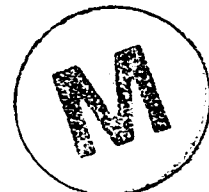
TABLE 14

FIELD RESISTIVITY SURVEY

Resistivity Location	Depth (Ft)	Reading (Ohms)	Resistivity (Ohms-cm)
R - 1	5	0.12×10^2	11,490
	10	0.065×10^2	12,448
R - 2	5	0.405×10^2	38,781
	10	0.11×10^2	21,066
R - 3	5	0.145×10^2	13,884
	10	0.065×10^2	12,448
R - 4	5	0.30×10^2	28,726
	10	0.08×10^2	15,321
R - 5	5	0.47×10^2	45,005
	10	0.09×10^2	17,236
R - 6	5	0.20×10^2	19,151
	10	0.065×10^2	12,448

Notes:

1. The average electrical resistivity was determined to depths of 5 and 10 feet using a "Vibroground" Model 293 instruments and an equally spaced electrode configuration "Wenner Method".

2. Resistivity locations R-1 through R-6 are shown on Figure 1 of this report.
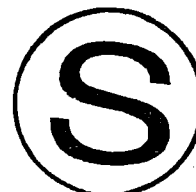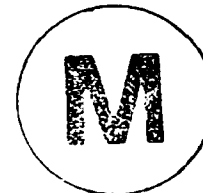
Table 15

LABORATORY RESISTIVITY

Resistivity Location	Sample Origin & Depth		Laboratory Resistivity (ohms - cm)
R-1	Test Trench TP-3	0 -2½'	10,253
		2½-4'	4,638
		4 -8'	4,964
		8 -12'	3,137
R-2	Test Pit TP-4	0 -4'	13,920
		4 -11'	9,275
		11 -18'	6,729
		18 -24'	4,246
		24 -30'	3,395
R-3	Boring B-13, BL-3	0 -7'	6,401
	S-2	5 -6½'	insufficient sample
	S-4	9 -10½'	insufficient sample
R-4	Boring B-14, S-1	1 -2½'	insufficient sample
	S-2	4½-5½'	insufficient sample
R-5	Test Trench TP-5	0 -3'	14,368
		3 -7'	3,005
		7 -13'	3,984
		13 -16'	1,763
		16 -18'	1,763
R-6	Boring B-53, S-1	1 -2½'	insufficient sample
	S-3	5½-7'	insufficient sample
	S-4	8 -9'5"	insufficient sample

Notes:

(1) Laboratory resistivity tests were performed according to California Highway Department Test Procedure 643, Part 4, Laboratory Method of Determining Minimum Resistivity.

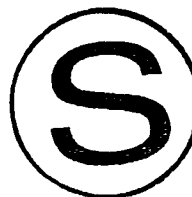(2) Resistivity locations R-1 through R-6 are shown on Figure 1 of this report.

Table 16

pH, SULPHATES & CHLORIDES
(Samples from Resistivity Test Locations)

Resistivity Location	Sample Origin & Depth		pH	Sulphates (ppm)	Chlorides (ppm)
R-1	TP-3,	0 -2 1/2'	7.7	8.75	53.3
		2 1/2-4'	8.8	2.50	16.6
		4 -8'	8.6	17.50	23.3
		8 -12'	9.2	18.80	33.3
R-2	TP-4,	0 -4'	7.1	2.50	20.0
		4 -11'	8.8	0.25	33.3
		11-18'	9.2	0.80	33.3
		18-24'	8.4	7.50	20.0
		24-30'	9.1	16.20	60.0
R-3	B-13,	BL-3, 0 -7'	8.0	1.75	46.6
		S-2, 5 -6 1/2'	7.8	1.00	86.6
		S-4, 9 -10 1/2'	9.3	20.00	20.0
R-4	B-14,	S-1, 1 -2 1/2'	9.1	5.00	80.0
		S-2, 4 1/2-5 1/2'	8.6	7.50	13.3
R-5	TP-5,	0 -3'	7.6	5.00	26.6
		3 -.'	8.4	0.50	20.0
		7 -13'	8.4	15.80	13.3
		13-16'	8.5	150.00	36.6
		16-18'	8.4	5.00	20.0
R-6	B-53,	S-1, 1 -2 1/2'	8.0	6.25	33.3
		S-3, 5 1/2-7'	7.4	2.50	33.3
		S-4, 8 -9'5"	8.5	33.80	20.0

Notes:

1. Test Procedures:

    (a) California Highway Department Test Procedure 643, Part 3, Method of Determining pH of Soil.

    (b) California Highway Department Test Procedure 417, Method of Testing Soils and Waters for Sulphate Content.

    (c) California Highway Department Test Procedure 422, Method of Testing Soils and Waters for Chloride Content.

2. Resistivity locations R-1 through R-6 are shown on Figure 1 of this report.