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LOS ALAMOS SCIENTIFIC LABORATORY
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University of California
LOS ALAMOS • NEW MEXICO

Geologic Studies and Material Properties
Investigations of Mesita de Los Alamos

UNITED STATES
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LOS ALAMOS SCIENTIFIC LABORATORY
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Geologic Studies and Material Properties
Investigations of Mesita de Los Alamos

Compiled by

M. Dean Keller

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GEOLOGIC STUDIES AND MATERIAL PROPERTIES
INVESTIGATIONS OF MESITA DE LOS ALAMOS

Compiled by
M. Dean Keller

ABSTRACT

Investigations were conducted to verify the competence of the foundation material for the Los Alamos Meson Physics Facility and to provide relevant quantitative data for use by the architect-engineers in their facility design work. These investigations included geologic history, seismic probability, physical characteristics of the near-surface rocks, ground vibration characteristics, and deformation characteristics.

1. INTRODUCTION AND SUMMARY

The Los Alamos Meson Physics Facility, encompassing as its primary entity an 800-MeV linear proton accelerator of unprecedented current-carrying capability, is planned for location on Mesita de Los Alamos, a site on the Pajarito Plateau in Los Alamos County. The accelerator and its associated facilities will constitute a significant adjunct to the research capabilities of the Los Alamos Scientific Laboratory and to the scientific community at large, under the auspices of the Atomic Energy Commission. When the facility is completed, the scientific staff will embark upon extensive research programs concerned with seeking new insights into, and

understanding of, the complexities of the atomic nucleus and related problems.

The characteristics of and the geographical location for this accelerator and its supporting laboratories have been carefully considered during the past few years. Preliminary studies have all indicated that Mesita de Los Alamos is a desirable location for an accelerator of this type. The geologic history of the area, the stability of the mesa, and the load-bearing qualities of the tuffaceous rock ensure a firm and quiescent base for this half-mile long machine. However, detailed quantitative descriptions of all relevant geologic features

must be obtained to permit the architect-engineers to proceed with designs for the various buildings and accelerator structures with the assurance that not only will the required stability be realized, but also that the design will be accomplished in an economical manner.

The seismic probability of the Los Alamos area as determined by history, geology, and field research is presented in this report, and the probability of other geological hazards is discussed. The geological units of Mesita de Los Alamos are described along with data

pertinent thereto such as density, moisture content, temperature gradient, and discontinuities.

Tests were conducted to determine vibratory characteristics of the volcanic tuff on Mesita de Los Alamos. Driving forces duplicated for this study included heavy truck traffic, impact loading, and the detonation of high explosives. Microseisms were recorded.

The deformation and creep characteristics of the volcanic tuff were determined by load tests, and correlation to analytical methods was achieved.

2. GEOLOGY OF LOS ALAMOS

History

Griggs has described the geology of the Los Alamos area,¹ and Smith et al. have summarized the area's volcanic history.²

Los Alamos is situated on mesas of the Pajarito Plateau on the east flank of the Jemez Mountains (Fig. 2-1). The area has had intermittent volcanic activity for several million years. The culmination of this activity was

the cataclysmic eruptions that produced the welded tuff deposits of the Bandelier Formation. The lower member of this formation was erupted from vents that were later obscured by a major collapse of the Toledo caldera. This caldera, in turn, was modified and obscured by eruptions of the upper member of the Bandelier Formation and the consequent collapse to produce the Valles caldera.

The upper member of the Bandelier Formation caps the mesas in the vicinity of Los Alamos. These mesas have been incised to depths as great as 800 ft by east-flowing streams. This depth of downcutting and the radiometric age of 1.1 million years for the upper member of the Bandelier Formation³ permit calculating a rate of downcutting by the major streams of $5 \text{ to } 8 \times 10^{-4}$ ft per year.

Subsequent to the development of the Valles caldera, rhyolite domes were extruded within the caldera and the youngest of these--South Mountain, which has a radiometric age of about 400,000 years⁴--is partially blanketed by a pumice sheet that issued from El Cajete. An exposure of the El Cajete pumice in a road cut on State Highway 4 about 1 mile west of the Triple H Ranch is shown in Fig. 2-2. A presumed correlative unit of this sheet locally known as "popcorn-tuff" is exposed in a

Fig. 2-1. Los Alamos area.

Fig. 2-2. El Cajete pumice.

road cut on State Highway 4 about 400 ft northwest of its intersection with West Jemez Road (Fig. 2-3).

It is estimated that the El Cajete pumice may be as young as 50,000 to 100,000 years.³ Samples of the two pumice deposits were unsuitable for radiometric dating. Subsequent to the volcanic activity of El Cajete, the area has been dormant except for some thermal springs and fumaroles.

Seismic Probability

The Los Alamos area has had a long history of faulting and, presumably, earthquakes principally related to the development of the Rio Grande graben which lies immediately east of the Jemez Mountains. Most of the modern earthquake history of New Mexico has been along the Rio Grande graben in the area south of Albuquerque. The Valles caldera and the older Toledo caldera

Fig. 2-3. Popcorn tuff.

developed by collapse on ring fractures, and the area probably suffered some earth shaking during their development.

The Pajarito fault extends along the east front of the Jemez Mountains immediately west of Los Alamos. This fault is part of a zone of faults that forms the western boundary of the Rio Grande graben and, as such, has probably had a long history of movement. The inferred development of the Pajarito fault is shown in Fig. 2-4, which also shows a gravity profile across the west side of the Rio Grande graben. The basement rocks have been displaced more than the precaldern volcanic rocks, which in turn have been displaced more than the Bandelier Formation. The Pajarito fault also displaces the pumice sheet (popcorn tuff) that is believed to be correlative with the El Cajete pumice. The pumice sheet (popcorn tuff) is preserved on the downthrown block and has

Fig. 2-4. Geologic cross section and gravity profile from the Jemez Mountains to the Rio Grande.
(Section cut on Figure 2-7.)

been completely eroded from the uplifted block.

The present-day scarp on the Pajarito fault has a maximum height of about 400 ft where the fault splits into two strands in the vicinity of Canon de Valle, and the height of the two scarps total about 500 ft. This 400 to 500 ft of displacement may all be younger, or only a small fraction of it may be younger, than the pumice sheet, but the displaced pumice sheet permits the determination of a maximum frequency of fault displacement. If it is assumed that the pumice sheet is 50,000 years old and it has been offset the maximum of 500 ft and if it is further assumed that the displacement has been by small increments that may average 10 ft, then one such increment has occurred every 1,000 years. Should the pumice be older and the postpumice displacement appreciably smaller than assumed, the frequency could be drastically reduced.

Considerable evidence suggests that the rate of faulting is very much lower, if it has not ceased altogether. Pinnacles eroded in the soft formation are common features in the vicinity of the Jemez Mountains. Ross and Smith⁵ described those in the Bandelier tuff (welded volcanic ash of the Bandelier Formation) as

...conical shaped pinnacles...that are locally known as tent rocks because of

their resemblance to conical tents or tepees. There is a tendency for the formation of these in nearly all the non-welded parts of the ash-flow tuff deposits but they are particularly outstanding in two members of the Bandelier Formation. The Bandelier Formation occupies an area of about 400 square miles and at least one of the tent rock members is present over most of this area. Some of the tent rocks are cones only a few feet high while others are 100 or more feet high. . . There are all gradations in size and of varying degrees of perfection in symmetry. Some cones are capped by large boulders that were rafted from the ground surface traversed by the ash flow during its emplacement. Some of these boulders weighing as much as several hundred pounds tend to retard erosion and produce boulder-capped pinnacles. Not rarely this results in whole colonies of pinnacles capped by boulders of varying size.

Boulder-capped pinnacles eroded in the Puye conglomerate are prominent in Rendija Canyon (Fig. 2-5). These pinnacles are 10 to 60 ft high and the capping boulders are from 2 to 5 times the diameter of the neck supporting them.

These pinnacles, particularly the boulder-capped ones, are unstable landforms and probably could not survive a strong earthquake in their vicinity. The height of

Fig. 2-5. Boulder-capped pinnacles eroded in Puye conglomerate in Rendija Canyon.

the pinnacles indicates a long period of ground stability. A 60-ft pinnacle would require 75,000 to 120,000 years to develop with the indicated rate of downcutting for the major canyons in the area.

The continuously inhabited Indian dwellings in the area and the absence of mention of earthquakes in the Spanish and Mexican records show that earthquakes have either not occurred in the area or have been so minor as to not deserve mention. Houses of unreinforced adobe construction (Fig. 2-6) in Indian pueblos have little lateral strength and would probably be severely damaged or destroyed during a strong earthquake. The prehistoric Indian ruins in the area also attest to a long, continued period of ground stability.

A recent instrumental study of New Mexico earthquakes by Sanford⁶ shows very low seismicity compared to such regions as southern California. During the 2-1/2 year period covered by his study, no shocks as strong as magnitude 3.5 were recorded. Predictions of activity based on the observed shocks are much lower than the known historical activity.

An instrumental record of ground stability in the area was desirable despite the uncertainty of predicting seismicity from short periods of observation and despite

the geologic and cultural evidence indicating an absence of geologically recent faulting. Therefore, four portable continuous-recording seismograph stations were installed at four locations (Fig. 2-7) in the vicinity of Los Alamos and operated for a 9-day period (March 25 to April 2, 1967).

Three of the stations were instrumented with one vertical and two horizontal Electrotech EV-17 seismometers, and the fourth with only a vertical seismometer. The seismometers were placed about 10 ft apart in holes about 1 ft deep. The signals from the seismometer were amplified and recorded on tape recorders, which were powered by thermoelectric generators.

All of the sites were noisy during the working day and seismic events could be identified with certainty only between the hours of 1900 and 0600. The quality of the seismograms is shown in Fig. 2-8.

Approximately 13 seismic events were recorded during the 9 days of operation. Of these events, one-half are teleseisms originating at great distances from Los Alamos. Most of the remaining events originated at least 100 km away; some of these could not be clearly identified as earthquakes. No local (less than 20 km away) seismic events were recorded. The nearest event was an

Fig. 2-6. Partially ruined adobe building at San Ildefonso Pueblo.

Fig. 2-7. Seismograph stations and sample localities.

Fig. 2-8. Portion of seismogram recorded at the most northerly station. Note the high noise level in the upper part, recorded during working hours. The event shown in the lower part originated at about 200 km from the station. The time ticks are 1 min. apart.

earthquake of small magnitude that occurred about 20 km west of the seismograph location.

Local events of magnitude at least as low as 1.0 would have been recorded with the instrument settings used in this study. Because no such events were recorded, we conclude that the instrumental study of ground stability, although of far too short a term to be definitive, corroborates the geologic and cultural evidence of very stable ground in the Los Alamos area.

The likelihood of severe seismic damage to installations in the vicinity of Los Alamos seems remote. There was no active, creeping fault at Los Alamos during the observation period. Sanford reported⁶ that his data resulted in a prediction for west-central New Mexico of a strongest shock of magnitude 3.5 for a 25-year period. He acknowledges that "this contradicts the known seismicity of the region because at least four shocks of this magnitude occurred within 50 km of Socorro from 1960 to 1961." In view of the seeming absence of earthquakes at Los Alamos, Sanford's figure of a maximum shock

of magnitude 3.5 per 25-year period might be taken as an upper limit for the seismicity at Los Alamos.

Other Physical Characteristics

Geologic hazards of possible significance in planning building construction in the vicinity of Los Alamos, other than seismic hazards, include ground compaction due to withdrawal of water, landslides, and renewed volcanism. Of these the possibility of renewed volcanism in the area probably deserves the most serious study.

Ground compaction due to withdrawal of water is not considered to be a significant geologic hazard because pumping to the present time has lowered the water table significantly only in the vicinity of the well fields. The aquifers are silty sandstones, sandstones, and conglomerates and, as such, are not particularly susceptible to compaction when the water is withdrawn.

Landslides, except for isolated rock falls from the mesa rims, are an unlikely hazard at Los Alamos because of the dry climate, the deep water table, and the rock formation characteristics. The Bandelier Formation is a nearly flat-lying cohesive unit with a low specific gravity, producing a relatively small load on the underlying formations. These rocks are essentially dry to a depth of several feet below the canyon bottoms. Thus, ground and climatic conditions conducive to landslide development are generally not present at Los Alamos. Landslides, except rock falls at the base of the mesa-rim cliffs, are virtually nonexistent topographic features in the area, which indicates that favorable conditions for their development have not existed in the geologically recent past.

The area of the Jemez Mountains has a long history of volcanism² and it is possible that volcanism has not ended. This volcanism has been characterized² by alternating periods of dormancy and activity, some of which produced catastrophic eruptions.

Whether the present quiescent period is a dormant interval or marks the end of volcanism in the area is unknown. Smith⁴ has stated that when he began work in the

area about 20 years ago there were active fumaroles within the Valles caldera and boiling water issued from some of the thermal springs. The fumaroles have now ceased to play and the temperature of the springs is much lower. The failing of the fumaroles might be due to lowering of ground temperatures, which would also account for the cessation of the fumaroles.

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3. NEAR-SURFACE ROCKS

The Pajarito Plateau, formed by a series of ashflows and ashfalls of rhyolite tuff, is dissected into a number of finger-like mesas by eastward-flowing intermittent streams that are tributary to the Rio Grande. Mesita de Los Alamos is one of these mesas bounded on the north by Los Alamos Canyon and on the south by Sandia Canyon. The canyon floors are 200 to 400 ft below the surface of the mesa. There is a small east-west ridge along the northern edge of the mesa, and the surface of the mesa slopes southward from the ridge at an altitude of about 7,035 to about 6,920 ft along the southeastern edge of the mesa.

The geologic investigation was made in conjunction with the drilling of 25 test holes by Albuquerque Testing Laboratory to determine the load-bearing capacities of the tuff. The major structure of the facility is the injector building to be built near test holes TH-1 and TH-2 (Fig. 3-1). A concrete tunnel (beam channel) covered with crushed tuff will extend from near test hole TH-2 to TH-12 to the experimental building, located between TH-12 and TH-14. The foundations of these structures are approximately 6,966 ft above mean sea level. Most of the test holes in a line with the beam channel were drilled and tested to an altitude of 6,960 ft. In the eastern part of the proposed tunnel and experimental building

area, core recovery was poor or penetration tests indicated low bearing capacities; hence, the holes were bored to greater depths.

A geologic map and cross sections of the upper surface of the mesa were compiled. Temperature measurements were made in three of the test holes, and continuous temperature recordings were made to a depth of 3 ft in two types of soil and one type of tuff. Moisture and density measurements were made along the walls of the test holes. Gamma ray and caliper logs were run in the test holes to aid in the stratigraphic correlation of the tuff.

Geology

Mesita de Los Alamos is composed of the Bandelier Formation of Pleistocene age. Underlying the Bandelier Formation are rocks of the Santa Fe Group of middle (?) Miocene to Pleistocene (?) age. In the Los Alamos area the rocks of the Santa Fe Group are, in ascending order, the Tesuque Formation, the Puye conglomerate, and the basaltic rocks of Chino Mesa (Fig. 3-2).

The Tesuque Formation is composed of siltstone and sandstones with some conglomerate and clay lenses. The formation exceeds a total thickness of 2,400 ft in the area. The Puye conglomerate is composed of the Totavi Lentil,

GEOLOGIC MAP AND SECTIONS SHOWING LOCATION OF TEST HOLES
AT MESITA DE LOS ALAMOS, LOS ALAMOS COUNTY, NEW MEXICO

SECTION B-B'

Fig. 3-1. Geologic map and sections showing location of test holes at Mesita de Los Alamos.

Fig. 3-2. West-east cross section (A-A').

a poorly consolidated channel fill deposit, and an overlying fanglomerate member composed of volcanic debris interbedded with the basaltic rocks of Chino Mesa. The total thickness of the Puye conglomerate and the basaltic rocks of Chino Mesa at well T-3 in Los Alamos Canyon immediately north of Mesita de Los Alamos exceeds 640 ft.

Bandelier Formation

The Bandelier Formation consists of ashfalls and ashflow rocks that are draped over older rocks, filling the lows and initially forming a smoother surface than that of the older rocks. The Bandelier Formation is composed of three members. In ascending order they are: the Guaje Member, an ashfall pumice about 35 ft thick, found in the subsurface at Mesita de Los Alamos; the Otowi Member, a friable ashflow about 235 ft thick, partly exposed in the lower wall of Los Alamos Canyon adjacent to Mesita de Los Alamos; and the Tshirege Member, a series of ashflow tuffs which cap Mesita de Los Alamos. Only units of the Tshirege Member were studied.

Tshirege Member

The Tshirege Member is composed of a series of rhyolite tuff ashflows that contain quartz and sanidine crystals and crystal fragments; some mafic minerals; and rock fragments of pumice, rhyolite, and latite in a fine ash matrix. The ashflows vary from a nonwelded to a welded tuff.¹ The Tshirege Member is about 300 ft thick under the ridge at Mesita de Los Alamos. Five units of the Tshirege Member (units 1a, 1b, 2a, 2b, and 3) described by Baltz et al.² are present.

Structure

The units of the Tshirege Member dip gently eastward at Mesita de Los Alamos. Units 2b and 3 form the upper part of the mesa dipping 3 to 6° toward the east-southeast.

Unit 2b outcrops at TH-1 and TH-2. The dip of the unit causes the contact between units 2b and 3 to cross the foundation elevation between TH-7 and TH-8. Thus,

due to the dip of the two units, the foundation of the buildings and tunnel west of TH-7 will be in unit 2b and the foundation east of TH-7 will be in unit 3 (Fig. 3-1).

The units 2b and 3 are broken by a normal strike slip fault between TH-8 and TH-9 (Fig. 3-1). The units on the east have moved down about 14 ft relative to the units west of the fault. Slickensides in the fault zone indicate the units on the east have moved about 14 ft south relative to the units west of the fault. Surface expression of the fault is a gouge zone 6 to 18 in. wide that stands 2 to 3 ft above the land surface for a distance of about 20 ft along the southern flank of the east-west trending ridge. This gouge zone consists of reddish tuff that has been welded by the movement, thus forming a more competent rock than unit 3, which forms the ridge. Along the crest of the ridge the gouge zone narrows and is eroded level with the land surface. The fault plane is nearly vertical as indicated by the remnants of the gouge zone in Los Alamos Canyon.

The cause of the fault is unknown. It may be due to compaction of the underlying members of the Bandelier tuff as the upper units of the Tshirege Member were laid down or it may be related to the tectonic forces that formed the Rio Grande depression.

The numerous joints in units 2b and 3 probably are tension joints formed during the cooling of the ashflows. The most prominent and numerous joints are nearly vertical and some are slightly open. Near-surface joints are filled with clay and weathering products of the tuff.

Units 1a and 1b. The lower part of the Tshirege Member consists of two ledge-forming layers of pumiceous tuff breccia similar in lithology, but slightly different in color and weathering characteristics. The lower layer is unit 1a (Fig. 3-3), a massive orange-weathering pumiceous tuff breccia forming a near vertical cliff above the alluvium in Sandia Canyon. This lower unit contains pumice fragments of obsidian and rhyolite in a fine glassy ash matrix. The weathered outer 1 to 3 in. of tuff is casehardened, protecting the unweathered rock from erosion. The thickness of unit 1a varies because of the irregular erosion surface at the top of the Otowi Member

Fig. 3-3. Stratigraphic sections of the Tshirege Member of the Bandelier tuff at Mesita de Los Alamos and Mesita del Buey.

upon which it rests; it may be as much as 80 ft thick near the center of the mesa.

Unit 1b (Fig. 3-3) rests conformably on unit 1a and weathers to a dull grayish-brown. It is a tuff breccia with a fine-grained pink ash matrix similar to the underlying unit 1a; however, the pumice fragments are smaller and 15 to 20% of the material consists of granule-sized quartz-crystal fragments and fragments of dense volcanic rocks. Unit 1b is slightly less resistant to erosion than the unit 1a and forms a ledge set back from the lower unit. At some places both layers form near-vertical cliffs where the units can be distinguished by a soft bed of pumice at the base of the upper unit, which weathers to a persistent notch in the cliffs. The thickness is fairly uniform, ranging from 21 to 23 ft along the southern edge of Mesita de Los Alamos.

Units 2a and 2b. The lower unit 2a (Fig. 3-3) is a light-gray pumiceous tuff consisting of moderately welded pumiceous ash containing fragments of pumice, dense rhyolite, and latite fragments as much as 3 in. long. The

unit weathers to a dull gray with a casehardened rind several inches thick. It forms a steep smooth slope set back from unit 1 and is separated from the overlying unit 2b by an erosional unconformity. The thickness ranges from 47 to 51 ft along the southern edge of the mesa.

The overlying unit 2b (Fig. 3-3) is a light pink to brown-weathering, moderately welded tuff (porosity from 30 to 50% by volume) composed of quartz and sanidine crystals and fragments in a matrix of light-pink fine-grained ash. Some rock fragments of pumice and latite are 1/2 in. long. The unit is resistant to erosion and forms ledges and benches above the steep rounded slopes of basal unit 2a. The unit ranges from 76 to 83 ft thick along the southern edge of the mesa, where it forms the uppermost rim of Sandia Canyon. Along Los Alamos Canyon, unit 2b forms a bench along the wall. Unit 2b underlies the proposed foundation of the injector building and tunnel in the western part of the mesa.

Unit 3. This unit (Fig. 3-3) rests conformably on

unit 2b and grades downward into it. Unit 3 consists primarily of light-gray, light-tan, and white nonwelded to moderately welded pumiceous tuff breccia (porosity from 30 to 60% by volume). The rock is composed of fine pumice fragments and glassy shards and contains numerous layers of pebble and cobble-sized pumice fragments and some gray dense rhyolite. Most of the unit is relatively soft and has eroded to form smooth, round slopes with a rind of casehardened material several inches thick.

The unit has eroded along the southern and extreme eastern edges of the mesa. The thickest section of unit 3 (about 75 ft thick) occurs near the center of the east-west trending ridge along the northern edge of the mesa. This unit forms the foundation of the east end of the tunnel and the experimental building.

Physical Characteristics

Density. The density measurements of units 2b and 3 were made by a density gauge manufactured by Nuclear-Chicago Corporation. A probe emitting gamma rays was lowered into the test holes and pulses from the probe, counted by a scaler at 5-ft depth intervals, were converted to densities.

The bulk density of the tuff of unit 2b as determined by density logging ranged from 100 to 120 lb/ft³; the density of unit 3 ranged from 80 to 110 lb/ft³. Differential cooling of an ashflow unit results in density changes of the unit in a vertical section (Table 3-I); rapid cooling of an ashflow after emplacement results in a more porous and pumiceous tuff at the upper and lower contacts.

Penetration tests, and bearing capacities determined from cores, indicate that bearing capacity of the tuff increases with density.

Moisture content. The moisture content of the bore walls of some test holes was determined with a neutron-scattering moisture probe manufactured by Nuclear-Chicago Corporation. Pulses from the probe were counted with a portable scaler and converted to moisture content.

The moisture content of the tuff beneath the soil zone or near surface tuff in TH-1, TH-2, TH-3, TH-4, and TH-5 was below 5% by volume. The moisture content of

the other holes was not determined because water was added during drilling. Measurements in six test holes drilled in January 1964 indicate that the moisture content of the tuff below foundation altitude was less than 5% by volume.³ Other studies⁴ of moisture distribution in the soil and tuff in the Los Alamos area indicate that the natural moisture content of the tuff beneath the mesas is generally less than 5% by volume in areas where the natural soil cover and drainage have not been disturbed.

Temperature. Test holes in tuff inhale and exhale air in response to atmospheric pressure changes. Inflatable plastic packings were placed in the test holes to eliminate the effect of transient air movement during the measurement of bore-wall temperatures. Standard calibrated laboratory thermometers were placed between the packers and bore walls at selected depths. Thermometer carriers had an oil reservoir around the bulbs with sufficient heat capacity to maintain constant temperature during removal from the test holes. The temperatures of three test holes given in Table 3-II were measured by this technique.

The lower temperatures measured in TH-1 are due in part to the physical characteristics of the tuff. TH-1 is completely in unit 2b, which is less permeable to air than unit 3 in TH-7 and TH-11. The tuff at TH-7 and TH-11 also receives more solar heat because these test holes are on the south-facing slope of the east-west ridge, where the combination of higher permeability and higher solar radiation probably accounts for the slightly higher temperatures. The cause of the 1 to 2° temperature increase of the tuff at depth in TH-7 and TH-11 is unknown.

Temperatures were recorded at 2-hr intervals at depths of 0.5, 1, 2, and 3 ft on the surface of the mesa in a sandy soil overlying the light gray pumice tuff of unit 3 (Table 3-III), a clay soil (Table 3-IV), and in the light gray pumice tuff of unit 3 (Table 3-V). Concurrent air temperatures are also shown in each table. Daily temperature changes appear to affect the soil or tuff only to a 1-ft depth. Daily temperature extremes in the fall and early spring probably extend slightly deeper.

Table 3-I

Density of the Tuff in Test Holes TH-1, TH-7, and TH-11 as
Determined by Density Logging

TH-1			TH-7			TH-11		
Depth (ft)	Unit of Tshirege	Density (lb/ft ³)	Depth (ft)	Unit of Tshirege	Density (lb/ft ³)	Depth (ft)	Unit of Tshirege	Density (lb/ft ³)
Member			Member			Member		
3	2b	114	5	3	94	5	3	80
6	2b	112	10	3	105	10	3	97
9	2b	115	15	3	107	15	3	105
12	2b	113	20	3	105	20	3	105
15	2b	110	25	3	110	25	3	105
-	-	-	30	3	102	30	3	108
-	-	-	34	2b	110	35	3	105
-	-	-	39	2b	109	40	3	105
-	-	-	-	-	-	45	3	90
-	-	-	-	-	-	49	3	103

Table 3-II

Temperature Gradients in the Tuff in Test Holes TH-1,
TH-7, and TH-11 (June 1966)

TH-1		TH-7		TH-11	
Depth (ft)	Temperature (°F)	Depth (ft)	Temperature (°F)	Depth (ft)	Temperature (°F)
3	60	3	62	3	63
8	53	8	61	8	56
13	50	18	52	13	63
16	50	28	52	23	53
-	-	38	54	33	54
-	-	-	-	43	54

Table 3-III
Air Temperature and Temperature at Depth in a Sandy Soil
Tuff (Unit 3) (June 7-13, 1966)

Time	Air Temp. (°F)	Soil Temp. at 0.5 ft (°F)	Air Temp. (°F)	Soil Temp. at 1.0 ft (°F)	Air Temp. (°F)	Tuff Temp. at 2.0 ft (°F)	Air Temp. (°F)	Tuff Temp. at 3.0 ft (°F)
00:00	58	72	50	63	60	63	60	61
02:00	57	71	50	62	50	63	58	61
04:00	51	68	50	62	54	63	54	61
06:00	50	68	50	63	54	63	54	61
08:00	74	65	54	62	70	63	70	61
10:00	75	64	65	62	73	63	77	61
12:00	76	68	70	62	78	63	76	61
14:00	77	72	64	62	78	63	70	61
16:00	72	74	57	62	-	-	74	61
18:00	74	75	57	62	75	63	72	61
20:00	66	74	50	62	64	63	62	61
22:00	61	74	50	62	62	63	57	61

Table 3-IV
Air Temperature and Temperature at Depth in a Clay Soil
(June 13-20, 1966)

Time	Air Temp. (°F)	Soil Temp. at 0.5 ft (°F)	Air Temp. (°F)	Soil Temp. at 1.0 ft (°F)	Air Temp. (°F)	Soil Temp. at 2.0 ft (°F)	Air Temp. (°F)	Soil Temp. at 3.0 ft (°F)
00:00	59	72	60	70	60	66	53	64
02:00	58	71	54	70	53	66	50	64
04:00	54	70	52	70	53	66	50	64
06:00	60	68	52	69	56	66	50	64
08:00	76	68	74	68	80	66	65	64
10:00	80	68	81	68	80	66	71	64
12:00	81	69	82	68	80	66	74	64
14:00	85	71	80	68	80	66	72	64
16:00	90	74	80	68	78	66	68	64
18:00	82	75	84	70	76	66	68	64
20:00	62	75	68	70	70	66	62	64
22:00	60	74	60	70	64	66	52	64

Table 3-V
Air Temperature and Temperature at Depth in a Pumiceous
Tuff (Unit 3) (June 7-13, 1966)

Time	Air Temp. (°F)	Tuff Temp. at 0.5 ft (°F)	Air Temp. (°F)	Tuff Temp. at 1.0 ft (°F)	Air Temp. (°F)	Tuff Temp. at 2.0 ft (°F)	Air Temp. (°F)	Tuff Temp. at 3.0 ft (°F)
00:00	60	67	56	66	44	61	58	60
02:00	54	65	54	66	44	61	59	60
04:00	51	62	53	65	44	61	58	60
06:00	48	60	50	64	45	61	52	60
08:00	53	60	54	64	48	61	58	60
10:00	64	60	62	62	54	61	68	60
12:00	72	61	66	62	58	61	76	60
14:00	80	62	72	62	66	61	78	60
16:00	85	68	-	-	73	62	88	60
18:00	84	70	79	64	66	62	84	60
20:00	72	71	72	65	61	62	74	60
22:00	64	70	58	66	52	62	64	60

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1. W. D. Purtymun and F. C. Koopman, "Physical Characteristics of the Tshirege Member of the Bandelier Tuff with Reference to Use as Building and Ornamental Stone," U.S. Geological Survey, Open-File Report, 1965.
2. F. H. Baltz, J. H. Abrahams, Jr., and W. D. Purtymun, "Preliminary Report on the Geology and Hydrology of Mortandad Canyon near Los Alamos, New Mexico, with Reference to Disposal of Liquid Low-Level Radioactive Wastes," U.S. Geological Survey, Open-File Report, 1963.
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4. J. H. Abrahams, Jr., J. E. Weir, Jr., and W. D. Purtymun, "Distribution of Moisture in Soil and Near-Surface Tuff on the Pajarito Plateau, Los Alamos County, New Mexico," U.S. Geological Survey, Professional Paper 424-D, 1961, pp. D-142 to D-145.

4. GROUND VIBRATION CHARACTERISTICS

Units and Definitions

Velocity = v μ /sec (maximum).

Frequency = f cps.

Period = T sec.

Displacement = d μ (zero to peak).

Acceleration = a μ /sec/sec (maximum).

$$f = 1/T.$$

$$v = 2\pi f d = 2\pi \left(\frac{1}{T}\right) d = \mu/\text{sec}.$$

$$d = \frac{vT}{2\pi} = \frac{(\text{microns/sec})(\text{sec})}{2\pi} = \mu$$

$$a = 4\pi^2 f^2 d = 4\pi^2 \left(\frac{1}{T}\right)^2 d = \mu/\text{sec}/\text{sec}.$$

$$g = 9.8 \times 10^6 \mu/\text{sec}/\text{sec}.$$

Equipment and Instrumentation

The U. S. Coast and Geodetic Survey supplied and operated the following equipment:

Type No. 21 seismometer, National Geophysical Corporation.

Model L5 amplifiers, National Geophysical Corporation.

Model VR3300 magnetic tape recorder, Central Electronics Company, Inc.

Model 4D camera, National Geophysical Corporation.

Model 905 WWV radio receiver, Beckman Instruments, Inc.

Model TS 100 timer, W. F. Sprengnether Instrument Company.

The response curve for this equipment is essentially flat between 1.5 and 80 cps with a negligible correction factor between 1.0 and 1.5 cps (Fig. 4-1). For 16 and 100-cps cutoff filters, the NGC-21 seismometer is flat to 10 and 100 cps, respectively (Fig. 4-2). The computer programs used in analyzing the data were limited to a range of 1.0 through 16.0 cps since the frequencies beyond this limit attenuated rapidly and were, therefore, well within the flat portion of the instrument response curve. A 16-cps cutoff filter was based on the seismometers. The data were recorded with the Model 4D camera, which recorded simultaneously 13 channels of information.

Fig. 4-1. Seismograph velocity sensitivity versus frequency, from calibrations.

Fig. 4-2. Amplifier relative voltage sensitivity versus frequency.

Impact Tests

Procedure

These tests were conducted with a cast carbon-steel block measuring 0.3 x 1.0 x 5.8 m and weighing 12,000 kg. Two seismograph recorders were located 107 and 137 m from where the weight was dropped. The initial test was conducted by lifting the weight 0.3 m with a crane with the 5.8- x 1.0- m side parallel to the ground. The weight was then dropped by releasing the crane brake, and the resulting ground motion was recorded. This test was repeated, and then the weight was dropped 0.9 m and then 1.5, each test being conducted twice.

Computation of Results

The records for the 0.3-m test, trial 2, were digitized. This test was selected because its record had fewer truncated points due to instrument saturation than other tests. Selected wave forms for the 0.3-, 0.9-, and 1.5-m tests were measured to determine the period and amplitude of each cycle. From these data the frequency, velocity, acceleration, and ground displacement were calculated using the CDC 6600 digital computer. The number of occurrences for each frequency was plotted (Fig. 4-3). Over 90% of the frequencies recorded were between 6 and 16 cps; the predominant frequency was approximately 10 cps.

The average acceleration for selected cycles was calculated and plotted for each test (Fig. 4-4). The ground acceleration for the 1.5-m drop was not appreciably greater than that for the 0.9-m drop, and it was slightly less for the vertical orientation nearest the drop site. This may be due to the weight being dropped on the same spot for each test, which pulverized the tuff. The impact for the 1.5-m drop was thereby cushioned. The seismograph nearest the drop point recorded the largest ground acceleration. The difference can be observed in Fig. 4-4.

Fig. 4-3. Frequency versus number of frequency occurrences.

Fig. 4-4. Ground acceleration versus height of drop.

The digitized record of the 0.3-m drop was processed using a computer program for a Fourier series analysis. The Fourier spectra, plotted by a Stromberg - Carlson 4020 plotter (Figs. 4-5 and 4-6), indicate that for frequencies of approximately 9 to 12 cps the greatest Fourier coefficient amplitudes result. Frequencies of 4 to 5 cps also had relatively high amplitudes, but the 9- to 12-cps range was more prevalent in the impact tests.

Conclusions

There was a difference in prevalent frequencies between the impact- and high-explosive test data. The impact-test spectra were relatively flat, whereas the high-explosives test spectra attenuated rapidly from the peak frequency. The most plausible explanation for this difference is the proximity of the seismograph recorders to the drop site.

High-Explosive Detonation Tests

These tests were conducted to determine the vibratory characteristics of Mesita de Los Alamos with respect to shock forces. Using proper correlation between detonations and seismic disturbances, these tests were interpreted to determine resonant characteristics of the mesa for the inception of seismic response spectra. In addition, the velocity of the shock waves in the tuff was calculated and a means of predicting the effects of detonations at a distance was established for the Los Alamos area.

Procedure

The high explosive was Composition C, with an energy rating 1.4 times that of TNT. As shown in Fig. 3-2, the charges were detonated on Mesita de Los Alamos approximately 15 m from the county line. The sizes of these charges in the order fired were 2.3, 4.5, 6.8, 9.1, 9.1, 13.6, 13.6, and 3.6 kg. Seismographs orientated in the vertical, transverse, and radial directions were at three stations, positioned as follows:

Fig. 4-5. Peak amplitude values, Fourier spectra. Seismograph located 107 m from impact.

Fig. 4-6. Peak amplitude values, Fourier spectra. Seismograph located 137 m from impact.

Station	Distance from Shot (m)
2	996
1	1559
3	2303

The charges were placed in holes approximately 1 m in diameter and approximately 1 m deep drilled to firm tuff. Backfill, consisting of the excavated material, was placed over each charge to couple it to the earth; coupling was adequate to obtain signals for analysis, but a considerable amount of energy was dissipated into the atmosphere. A transducer close to the charges transmitted a detonation signal through wires to the seismograph. This signal appears as a blip on channel 11 of the seismograph record, and indicates zero time for the detonation.

Computation of Results

The data obtained from the seismometers were recorded by the Model 4D camera (Figs. 4-7 and 4-8). Two methods were used to derive results from these data. In one method the seismic wave forms were measured and the results were processed through the CDC 6600 computer to calculate velocity, displacement, and acceleration. The wave forms were matched; i. e., the corresponding wave forms for the different sizes of explosive charges were compared to obtain the data presented in this section. The wave forms for the stations at various distances from the shots were not matched, but the same time increment was compared, thus yielding a comparison of average values at distances from the shots (Table 4-I). The ground acceleration versus the size of charge (Fig. 4-9) and the ground acceleration versus the distance from the detonation (Fig. 4-10) were plotted from the data given in Table 4-I. For the second method the seismic records for the 4.5-kg Composition C detonation were digitized and a computer program was used to calculate Fourier spectra (Figs. 4-11, 4-12, and 4-13).

Limitations

The seismograph records used for the Fourier spectra analysis and for the other graphs presented here

were not segregated according to the type of signal.

These records represent compression, shear, and surface waves. Reflection of waves from various discontinuities has, no doubt, changed their forms. It is believed, however, that the spectra developed do represent the distribution of frequencies that would be experienced from a shock excitation. A different origin for the shock force would possibly yield slightly different spectra, but with similar characteristics. The curves developed from Table 4-I would not be affected greatly by reflection of waves because the same waves for the various size charges were studied.

A limitation that must be considered when comparing formulas developed using the test data is that the coupling of the charges to the ground was not as positive as the coupling attained during underground tests. The slope of curves, however, should be characteristic of the medium (tuff) being tested.

Conclusions

Seismic Records. The time at which the compression, shear, and surface waves arrived is shown in Figs. 4-7 and 4-8. Station 2 was 996 m from the point of detonation, and the initial compression wave arrived 0.82 sec after detonation. The velocity of this shock wave through the tuff (all unit 3) was 1215 m/sec. Based on this velocity, the reflected "P" wave traveled 1460 m. This would indicate that the reflection occurred at the 5500-ft elevation, at the transition from the Puye conglomerate to the Tesuque Formation (Fig. 3-2). The "P" wave is not strong due to the imperfect coupling of the detonation to the tuff, and the surface waves obliterate the reflected "P" waves when they arrive.

Station 1, 1559 m from the point of detonation, was in unit 2b of the Bandelier tuff, as was the point of detonation. This unit is a more competent material than unit 3, having a modulus of elasticity of approximately $14,000 \text{ kg/cm}^2$ compared to approximately $1,400 \text{ kg/cm}^2$ for unit 3. The velocity of the compression wave in unit 2b is 1470 m/sec.

Fig. 4-7. Seismic records: test 3, phase 2; 4.5 kg Composition C.

Fig. 4-8. Seismic records: test 3, phase 6; 13.6 kg Composition C.

Table 4-I
High-Explosive Detonations

		Seismometer Orientation								
		Vertical			Transverse			Radial		
		STA. 2	STA. 1	STA. 3	STA. 2	STA. 1	STA. 3	STA. 2	STA. 1	STA. 3
		(IV)	(4V)	(8V)	(2T)	(5T)	(9T)	(3R)	(7R)	(10R)
2.3 kg	Vel.	26.34	8.03	2.52	18.64	9.63	3.76	18.64	10.97	3.01
	Disp.	0.679	0.266	0.076	0.432	0.289	0.122	0.473	0.335	0.104
	Acc.	1064.	254.	84.	830.	342.	126.	780.	371.	91.
4.5 kg	Vel.	43.92	12.86	5.89	33.81	14.23	4.61	39.98	17.75	4.35
	Disp.	1.201	0.434	0.180	0.850	0.427	0.170	1.049	0.588	0.151
	Acc.	1598.	396.	194.	1392.	499.	134.	1565.	596.	127.
6.8 kg	Vel.	63.95	18.01	9.01	49.69	20.82	7.57	55.41	27.77	6.93
	Disp.	1.745	0.637	0.286	1.228	0.631	0.269	1.580	0.898	0.250
	Acc.	2392.	526.	287.	2077.	735.	229.	2044.	878.	199.
9.1 kg	Vel.	84.77	25.35	11.81	59.43	30.64	10.98	82.06	35.80	10.06
	Disp.	2.470	0.910	0.385	1.546	1.027	0.394	2.511	1.178	0.377
	Acc.	2961.	730.	369.	2361.	961.	334.	2794.	1108.	261.
13.6 kg	Vel.	150.24	39.08	14.76	84.17	51.02	18.78	120.98	51.97	14.33
	Disp.	4.537	1.396	0.526	2.192	1.715	0.703	3.840	1.738	0.572
	Acc.	5041.	1134.	424.	3359.	1600.	522.	4099.	1577.	370.

Vel. μ /sec
Disp. μ
Acc. μ /sec/sec

Fig. 4-9. H. E. detonation tests: ground acceleration versus size of charge.

Fig. 4-11. Peak amplitude values: Fourier spectra, vertical seismograph orientation.

Fig. 4-10. H. E. detonation tests: ground acceleration versus distance from detonation.

Fig. 4-12. Peak amplitude values: Fourier spectra, transverse seismograph orientation.

Fig. 4-13. Peak amplitude values: Fourier spectra, radial seismograph orientation.

At Station 3, 2303 m from the point of detonation, the compression waves were more difficult to detect. The arrival of the surface waves was, however, obvious. The proposed initial points for the compression, shear, and surface waves, are shown in Figs. 4-7 and 4-8. The average velocity of the compression waves in the unit 2b and unit 3 tuff was calculated as 1600 m/sec.

Attenuation Characteristics. Empirical formulas for predicting earth motion resulting from the detonation of underground nuclear devices were developed by the United States Coast and Geodetic Survey.¹ The energy ratings of the detonations for the study conducted on Mesita de Los Alamos were below those of detonations for which the formulas were developed. Also, the ground coupling of the detonations on Mesita de Los Alamos was not comparable to that from which the formulas were developed. A comparison was made, however, to see to what extent our accelerations deviated from those predicted by the empirical formulas. The accelerations, chosen from our

records, were the average of the three peak accelerations, usually surface waves. One formula² developed by the U. S. Coast and Geodetic Survey follows:

$$A = K_a W^{0.54} R^{-1.4}$$

where

K_a = constant = 0.0103 for tuff,

W = size of detonation in tons equivalent TNT (1 ton = 907.18 kg),

R = distance to detonation, in kilometers,

A = acceleration in g's where 1 g = $9.8 \times 10^8 \mu\text{sec/sec}$.

The comparison of predicted accelerations to those recorded by our seismometers is given in Table 4-II. The comparison becomes more favorable as the size of the charge increases and as the distance from the detonation decreases. This formula could be used to conservatively predict the effects of detonations within Los Alamos County on the Meson Physics Facility. At a distance of 7 km, an equivalent of 0.05 tons of TNT well-coupled to the earth will cause an acceleration of less than 130×10^{-6} g. This is less than the disturbance caused by a loaded truck passing the site and is approximately one-eighth the perception threshold for acceleration.

Table 4-II
Acceleration ($g \times 10^{-6}$)

Equivalent TNT (Tons)	Comparison	Distance to Detonation (km)		
		0.996	1.559	2.303
0.007	C. & G. S. Formula	705	379	219
	Measured	276	66	24
0.014	C. & G. S. Formula	1030	553	319
	Measured	557	129	53
0.021	C. & G. S. Formula	1280	666	395
	Measured	1080	151	70

The formula developed by Mickey and the empirical results obtained from the test conducted on Mesita de Los Alamos coincide at approximately 1 km and 0.025 tons of TNT. A conservative formula developed from the empirical results has the form

$$A = 0.0150 W^{0.68} R^{-3.26}$$

using the same nomenclature as the U.S. Coast and Geodetic Survey formula. Note that the greatest change is in R, the distance factor.

The effects predicted by this formula of a loosely coupled detonation of 0.05 tons of TNT at a distance of 7 km would be an acceleration of less than $4 \times 10^{-6}g$, which is less than weekend background noise. This is borne out by the fact that detonations at distant sites were not detected on Mesita de Los Alamos.

Fourier Spectra. Fourier spectra were developed for the seismic records generated by a shock force. Almost all of the data developing this spectra were caused by waves. Generally, the Fourier amplitude coefficient is of a greater magnitude between 3 and 9 cps than below or above this range. The least attenuation occurred between 4 and 6 cps. It is proposed that the predominant frequency of the mesa is approximately 4.5 cps for localized detonations.

Transverse movement is higher than would be expected of a shock wave emitted from a point source. This may be explained by reflection of P, S, and SW waves from the walls of the mesa.

Project Gasbuggy

Procedure

On December 10, 1967, a nuclear device equivalent to 26,000 tons of TNT was detonated at a depth of 4,250 ft, approximately 123.5 km northwest of Mesita de Los Alamos. A U.S. Coast and Geodetic survey team measured ground motions at Mesita de Los Alamos generated by this detonation. Detailed information on equipment and procedure may be obtained from Ref. 4.

Discussion of Results

The maximum amplitude at Mesita de Los Alamos was slightly lower than predicted, and the seismic first arrival times indicated a propagation velocity of about 6 km/sec. At these distances the first arrivals did not penetrate to the Mohorovicic discontinuity. The maximum

motions were probably propagated as a surface wave, progressing at velocities of 3.5 to 4.8 km/sec. The vibrations had decayed to one-tenth of maximum at about 95 sec. The motion had not returned to normal preshot background at 4 min after zero.

The Gasbuggy seismogram was analyzed for maximum motions at dominant visual frequencies. This could be grossly equivalent to a Fourier spectra analysis. Dynamic response spectra for single-degree-of-freedom elastic systems have been used by structural engineers to design earthquake resistant buildings. These spectra are usually computed with seismic data from earthquakes as an input. A method for estimating response spectra has been devised that uses the maximum earth motions. A spectra envelope can be made using 2 times maximum acceleration, 1.5 times maximum particle velocity, and 1 times maximum displacement. This approaches the response for a system with 5 to 10% critical damping.

Fig. 4-14 was constructed using this method. This graph infers that a one-degree-of-freedom oscillator (or building) with a natural frequency of 1 Hz would experience ground motions in terms of acceleration, velocity, and displacement of $1.0 \times 10^{-4}g$, 1.5×10^{-2} cm/sec, and 2.2×10^{-3} cm from Gasbuggy. Similarly, a 4-Hz building would respond 1.5×10^{-3} cm, 5.3×10^{-2} cm/sec, and 2.1×10^{-3} cm.

The visual dominant spectra for all traces were derived. The average frequency for the maximum motions was 2.6 Hz with lower and upper limits of 2.2 and 3.1 Hz. The vibrations decrease rapidly with increased frequency.

Conclusions

The frequencies of maximum motion from Gasbuggy were between 2 and 3 Hz. The amplitudes of vibrations on Mesita de Los Alamos generated by the Gasbuggy detonation were generally as predicted by formulas derived from previous nuclear detonations.

Fig. 4-14. Response spectra - Los Alamos area,
Gasbuggy experiment.

Microseisms

Microseisms of natural or artificial origin will affect the operation of the Meson Physics Facility only if their amplitude is excessive. In his final report, Hof stated³ that he was unable to find anyone having machine-tool problems caused by natural microseisms. It does not appear that there is a possibility of any disturbance to equipment in the Meson Physics Facility from microseisms of natural origin, so this phase of the report is limited to a determination of frequency characteristics and the calculation of ground-motion characteristics for the more intense microseisms generated by artificial means. The difference in intensity of these microseisms for working hours and for nights and weekends is noted as well as the intensity in two different tuffs.

Frequency

A random sampling of frequencies through a 10-sec period indicated a predominance of frequencies in the 5-

to 7-cps range. This sampling was taken for the vertical component only; the horizontal components appeared to be similar (Table 4-III).

Velocity: Zero to Peak Displacement and Acceleration.

Table 4-III presents values for velocity, displacement, acceleration, and period for week end and working-hours microseisms. Unit 3 and unit 2b are the geologic members of the Bandelier tuff previously described.

Conclusions

The microseisms recorded on Mesita de Los Alamos are comparable to those presented in the Hof report.³ In most cases, the microseisms recorded on Mondays are twice the magnitude of those recorded on Sundays. There appears to be no discernible difference in the magnitude of the microseisms in the more competent (unit 2b) as compared to the more friable (unit 3) tuff.

Table 4-III
Microseisms
Microseisms, Sunday, October 23, 1966, 11:15 A. M.

Geologic Member	Orientation	Velocity	Displacement	Acceleration	Period
Unit 3	Vertical	1.14	0.027	72	0.15
Unit 2b	Vertical	1.14	0.025	72	0.10
Unit 3	Transverse	1.37	0.043	53	0.20
Unit 2b	Transverse	1.37	0.030	60	0.14
Unit 3	Radial	2.51	0.070	115	0.20
Unit 2b	Radial	1.36	0.032	57	0.15

Microseisms, Monday, October 24, 1966, 10:10 A. M.

Unit 3	Vertical	2.51	0.076	99	0.16
Unit 2b	Vertical	1.83	0.047	72	0.16
Unit 3	Transverse	2.97	0.071	103	0.15
Unit 2b	Transverse	3.20	0.107	96	0.21
Unit 3	Radial	2.05	0.068	61	0.21
Unit 2b	Radial	3.65	0.116	116	0.20

Velocity	μ /sec.
Displacement	μ .
Acceleration	μ /sec/sec.
Period	sec.

Traffic Effects

The transportation of large quantities of shielding to the experimental building will be necessary. The vibrations generated by heavy truck traffic could be of a magnitude requiring engineering design of equipment supports to attenuate these vibrations. The effects of heavy truck traffic were studied.

Procedure

A trailer truck was loaded with two 12,200-kg weights. The total weight of truck and weights was 26,592 kg. This truck traveled along an unimproved road past two seismograph stations. The seismographs were oriented perpendicular to the road at 37 and 89 m from the center line of the road, respectively. After records were taken of vibrations caused by truck traffic, the truck was stopped at a point closest to the seismograph stations. It was assumed that the braking action of the truck would generate a greater magnitude of vibration than the passing of the trucks.

Computation of Results

The data obtained from the seismometers and amplifiers were recorded simultaneously on 13 channels by the Model 4D camera. Magnetic tape records were also obtained on a CEC Model UR 3300 recorder but were not digitized because they were not considered necessary for this report. Two methods were used to derive results from these data. In one method the seismic wave forms were measured and the results were processed by the CDC 6600 computer to calculate values of velocity, displacement, and acceleration. The maximum values of vibration for the braking action are given in Table 4-IV. In the other method the records for the truck passing the stations were digitized, and a computer program was used to generate Fourier spectra for this event. These spectra are illustrated in Figs. 4-15 and 4-16.

Conclusions

The vibrations generated by truck braking action are 30 to 40 times greater than from natural microseisms

of the area. It appears that attenuation of vibrations was greater in the more competent tuff (unit 2b) than in the more friable tuff (unit 3).

The vibrations generated by the truck braking are of the same order of magnitude as those of the truck passing. The road was rough, however, and much of the vibration was generated when the truck tires hit holes in the road. For a smooth road⁴ vibrations from the truck passing were an order of magnitude lower than vibrations from the braking action. Braking action on a smooth road resulted in vibrations approximately one-half of the magnitude of the same action on a rough road.⁴ A study of the frequency of vibrations indicated a predominance between 8 and 14 cps.

The Fourier spectra for all stations indicate a response across the spectra. However, a general peaking around 2.3, 4.5, 8.5, and 13 cps was noted, and the attenuation at these frequencies was not as pronounced as the attenuation at other frequencies. No amplification of signals was noted at any frequency, although the forcing function may be considered repetitive.

References

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Table 4-IV
Truck Passing and Braking

Geologic Member	Seismometer Orientation	Velocity (μ/sec)		Displacement (μ)		Acceleration (μ/sec/sec)	
		37 m	89 m	37 m	89 m	37 m	89 m
<u>Truck Braking</u>							
Unit 3	Vertical	47.1	47.1	1.27	0.93	2020	1870
Unit 2b	Vertical	30.9	17.3	0.74	0.38	1520	773
Unit 3	Transverse	21.7	18.1	0.62	0.75	980	568
Unit 2b	Transverse	3.9	3.0	0.07	0.09	254	118
Unit 3	Radial	27.2	18.1	0.61	0.63	1220	645
Unit 2b	Radial	42.7	19.1	0.88	0.47	2070	862
<u>Truck Passing</u>							
Unit 3	Vertical	42.7	12.7	1.29	0.39	1410	412
	Transverse	30.0	11.8	0.76	0.30	1170	460
	Radial	30.9	12.7	0.59	0.32	1620	500

Fig. 4-15. Peak amplitude values; Fourier spectra. Seismograph located 37 m from the center line of the road.

Fig. 4-16. Peak amplitude values; Fourier spectra. Seismograph located 89 m from the center line of the road.

5. DEFORMATION CHARACTERISTICS

To correlate the geological composition of Mesita del Buey to that of Mesita de Los Alamos and to describe the geological composition of Mesita de Los Alamos, a stratigraphic section of the Tshirege Member was measured at Mesita del Buey near Pajarito Reservoir No. 2 and the geologic units of the section were correlated with a section previously measured at Mesita de Los Alamos near the proposed location of the experimental building (Figs. 5-1 and 5-2). Correlations of the geologic units were made by the lithology of the units and their weathering characteristics. The units were also traced southward from Mesita de Los Alamos to Mesita del Buey. A datum was established on the prominent marker bed of tuff between units 1a and 1b, which is light orange and weathers to a slight notch. It is a persistent and readily identified geologic feature that is present where the two units are exposed on the plateau.

Units 1b and 2a thicken slightly from Mesita de Los Alamos southward to Mesita del Buey (Table 5-I); unit 2b thins about 47 ft. The difference in unit 2b's thickness on the two mesas may be due to the direction of the ejection of the ash flow that formed this unit and the origin of the flow (an area west of the Pajarito Plateau) and in part to topography that existed in pre-Bandelier time. The thickness of the Tshirege Member decreases rapidly east of the section at Mesita del Buey to the vicinity of White Rock, where the tuff lies directly on the basalt.

The basalt formed a topographic high in pre-Bandelier time. The upper part of unit 3 is eroded on both mesas. The thickness of the remaining part of unit 3 is 45 ft at Mesita del Buey and 98 ft at Mesita de Los Alamos.

Fig. 5-1. Test site locations.

The altitudes of the two test areas at Mesita de Los Alamos are 6,970 and 6,990 ft; the altitude of the test area at Mesita del Buey is 6,935 ft. The test areas on both mesas are in unit 3 but at different stratigraphic positions in the unit (Table 5-II).

Fig. 5-2. Stratigraphic sections of the Tshirege Member of the Bandelier tuff at Mesita de Los Alamos and at Mesita del Buey.

Table 5-I

Thickness of Geologic Units at Stratigraphic Sections			
Geologic Units	Mesita de Los Alamos a/ (ft)	Mesita del Buey b/ (ft)	Remarks
3	98	45	Upper part eroded
2b	76	29	
2a	51	60	
1b	21	23	
1a	74	16	Lower part covered

a/ Altitude at top of section--7,034 ft

b/ Altitude at top of section --6,935 ft

The physical characteristics of units 2a and 3, which will support the Meson Physics Facility structures, apply to both Mesita de Los Alamos and Mesita del Buey.

The foundation material in the Los Alamos area is composed primarily of a welded volcanic ash, usually deposited as an ash flow. As such it is an anisotropic

material which exhibits different characteristics depending on the orientation of the sample being tested. It is remarkable that the foundation material can be cut with a saw or bulldozer, but can maintain its integrity while having a vertical face 100 ft high. Bearing capacity of the more competent tuff may conservatively be estimated at 10 tons/ft². Modulus of elasticity values of from 20,000 to over 150,000 psi are common, depending on the density of the material and the degree of welding.

The degree of welding is classified at Los Alamos as nonwelded, moderately welded, and welded tuff. The transition between nonwelded (porosity as much as 60%) to welded tuff (porosity as small as 15%) is due to compaction and fusing by heat of the crystals, crystal fragments, and rock fragments in the ash matrix. The degree of welding and the amounts and size of pumice, latite, and rhyolite rock fragments determine the physical characteristics of the tuff; i. e., porosity, density, and bearing capacity.

Table 5-II

Thickness of Geologic Units at Test Areas

Geologic Units	Mesita de Los Alamos <u>a</u> / (ft)	Mesita de Los Alamos <u>b</u> / (ft)	Mesita del Buey <u>c</u> / (ft)	Remarks
3	54	34	45	
2b	76	76	29	
2a	51	51	60	
1b	21	21	23	
1a	74	74	16	Lower part covered

a/ Altitude at test area--6,990 ft
b/ Altitude at test area--6,970 ft
c/ Altitude at test area--6,935 ft

Precision Leveling

The equipment used for the precise readings consisted of: a Wild-Herrbrugg Instruments, Inc., N-3 precision level with optical micrometer; a Keuffel and Esser Co., precise level rod; a rod level; and a thermometer.

In general, the procedures and instrumentation of the precise leveling program conformed to U. S. G. S. second-order practice. Field notes included all pertinent data and comments.

The reliability of the reading obtained under field conditions is probably good to the nearest 0.001 ft. Although the optical micrometer scale can be interpolated to 0.0001 ft by estimation between the 0.0005-ft scale graduations, other variables such as bubble creep, air refraction, superimposing of the cross hairs on the eye graduation, eye strain, relief of internal stresses in the tripod, and movement occasioned by the shifting weight of the level man while taking the readings tend to offset the seemingly higher accuracy capabilities of the equipment.

Factors Affecting Accuracy of Theoretical Analysis
Modulus of Elasticity

A deformation analysis of Mesita de Los Alamos should be an elastic analysis based on Hooke's law. Two variables in this analysis are (1) the pressure at various depths and distances from the load and (2) the modulus of

elasticity. The pressures at various depths can be predicted if the characteristics of the foundation are known.

In the unit 3 tuff, it is difficult to obtain a core sample to test for modulus of elasticity because the sample disintegrates when it is disturbed. It is necessary to determine modulus of elasticity values by testing samples taken at or near the surface and by estimating values at depths based upon geological and physical characteristics of the tuff. Samples may be taken close to the surface by excavation with hand tools. This method was used to determine modulus of elasticity values for Pajarito Reservoir No. 2 and for the locations on Mesita de Los Alamos. Because the tuff is an anisotropic material, correct orientation was maintained during the tests. The results of these tests are considered conservative in that lateral pressure was not used to simulate the *in situ* condition of the tuff. The ash matrix contains intrusions of pumice, latite, and rhyolite. These intrusions are usually small, i. e., approximately 6 to 10 in.³, and do not adversely affect deformation characteristics. An intrusion in a tuff sample, however, will greatly affect that sample's strength and modulus of elasticity. Samples which exhibit values much smaller than the average value of a group should be disregarded and a new average value established.

The value of the modulus of elasticity obtained from tests conducted on tuff samples is representative of the

in situ modulus for the sample area, or is conservative.

Strata Depth

The depth of each stratum may be estimated by borings. Indications of changes in strata are a change in the unit weight of the tuff or an increase in resistance to penetration. There is no precise correlation between unit weight and modulus of elasticity.

It is necessary to estimate the modulus of elasticity at various depths by using the modulus of elasticity at the surface, the unit weight of the material, and the results of borings taken in the more competent tuff. This has been done for the analytical analysis of the load test on Mesita de Los Alamos.

Foundation Stiffness

To compare theoretical analysis to empirical data, the stiffness of the foundation must be established. For tests involving water tanks, the load is imposed on the foundation material through a thin steel plate. It may be assumed that the foundation is flexible and that the analytical approach such as the Boussinesq equation for vertical stress or Newmark's chart will apply.^{1,2} Test loads imposed on Mesita de Los Alamos were composed of steel counterweights, which were relatively stiff; therefore, it was necessary to determine what effect this stiffness will have on the theoretical analysis. An analysis by Timoshenko and Goodier³ indicated that for a circular surface footing the theoretical settlement of a rigid footing is approximately 7-1/2% smaller than the corresponding average settlement due to the same total load uniformly distributed under the footing.

The distribution of pressure under the rigid circular footing is not uniform: a concentration of pressure exists around the perimeter of the footing, and the smallest value of pressure is at the center. Since the footing is rigid, this uneven distribution of pressure results in the smaller theoretical settlement. Timoshenko's analysis was based upon the pressure exerted on metal by a rigid die.

Casagrande⁴ states that, for a perfectly elastic soil, the reactions of a rigid footing would be larger on the per-

imeter than in the center of the footing, but for sand the opposite would be true. The volcanic tuff in Los Alamos is more like an elastic than an inelastic material, and a larger stress concentration would be expected around the perimeter of the footing.

The preceding assumptions are based on rigid foundations. It was necessary to consider empirical results to determine the rigidity of the test weights. The pressure coefficients at various depths, assuming a foundation 19 ft x 9 ft 9 in. and using the Newmark chart, are given in Table 5-III. The total pressure at the center of the short edge is approximately 0.8 times the total pressure at the center of the long edge. The total pressure at the corner is approximately 0.6 times the total pressure at the center of the long edge. Assuming that the deformation ratio will be the same, the theoretical distribution of the total deformation around the perimeter of the test load can be determined. In Table 5-IV the measured deformation is compared to an assumed average deformation (with the assumption that the foundation is rigid) and to a theoretical deformation assuming that, for this foundation, the Newmark chart and the Boussinesq equation for vertical stress are valid; i. e., the foundation is flexible. The average percentage difference is twice as much for a rigid as for a flexible foundation. This indicates that redistribution of stress occurs in the tuff, allowing the use of the Newmark chart and the Boussinesq equation for this analysis.

Table 5-III Newmark Pressure Coefficients			
	Center of	Center of	
Depth (ft)	Long Edge	Short Edge	Corner
5	94	80	46
8	78	63	43
10	68	52	39
15	48	38	31
20	32	25	24
30	17	15	15
40	11	11	11
60	5	5	5
Total	353	289	214

Table 5-IV

Mesita de Los Alamos

Location No. 1

Corner	Measured Deformation 11/23/66 (ft)	Average Deformation 11/23/66	Difference From Average	Percentage Difference	Theoretical Distribution Newmark Chart	Difference Deformation and Newmark Theoretical Deformation	Percentage Difference
NW	0.0115	0.0143	-0.0028	-19.5	0.0114	0.0001	0.9
W	0.0162	0.0143	0.0019	13.3	0.0152	0.0010	7.2
SW	0.0097	0.0143	-0.0046	-32.0	0.0114	-0.0017	-15.0
S	0.0172	0.0143	0.0029	20.3	0.0190	-0.0018	-9.5
SE	0.0127	0.0143	-0.0016	-11.2	0.0114	0.0013	11.4
E	0.0112	0.0143	-0.0031	-21.7	0.0152	-0.0040	-26.3
NE	0.0110	0.0143	-0.0033	-23.1	0.0114	-0.0004	-3.5
N	0.0248	0.0143	0.0105	73.3	0.0190	0.0058	30.5
Average	0.0143	0.0143		26.8			13.1

Qualitative Tests

Two loadings, one a structure constructed of iron counterweights (Iron House) and the other a water reservoir, were observed during construction and filling, respectively, to establish procedures for tests to be conducted later. Subsoil information was not available for these projects, nor was it possible to maintain the integrity of the bench marks. It was decided that only qualitative results could be obtained.

Iron House

The characteristics of the tuff under the Iron House are unknown. It may be assumed that it is a competent material. However, fill had been placed over the tuff approximately 6 to 10 ft deep on the south side of the Iron House, with lesser amounts on the east side. Most of the benchmarks did not penetrate the fill; two on the east side of the building did penetrate the fill.

The area covered by the Iron House was approximately 44 x 19 ft, with the load being imposed around the perimeter of the building. Counterweights 19 ft long, 3 ft 3 in. wide, and 2 ft thick were used as a floor. This floor,

no doubt, helped to distribute the load across the width of the building. The total load was in excess of 1,000 tons, or approximately 1.2 ton/ft².

Reference points were 3/4-in. -diam reinforcing bars, 5 ft long, driven into the ground allowing 6 in. to project.

A small amount of creep occurred, but whether in the tuff or in the fill is unknown. Some heaving occurred on the east reference points.

Pajarito Reservoir No. 3

The volcanic tuff at Pajarito Reservoir No. 3 is classified as hard. The tuff near the edge of the mesa is broken and fractured. The tuff under the tank may be assumed to be competent with a modulus of elasticity of approximately 150,000 psi (unit 2b).

The capacity of Pajarito Reservoir No. 3 is one million gallons, or approximately 4,160 tons. The head of water is 35 ft, resulting in a uniform pressure on the bottom of the tank of 2,180 lb/ft².

Reference point N-O, which is located approximately 1 ft from the edge of the tank, was deformed 0.0050 ft

under load. This compares to a deformation of 0.0104 ft for N-1 and 0.0077 ft for E-1, which are located an equal distance from the edge of the tank at Pajarito Reservoir No. 2. Reference point N-2, which is located approximately 9 ft from the edge of the tank, deflected 0.0042 ft. This compares to an average of 0.0054 ft for Pajarito Reservoir No. 2.

Comparison of Calculated to Empirical Deformation

Pajarito Reservoir No. 2

This reservoir is at the intersection of Pajarito Road and Rex Drive (Fig. 5-3). It has a capacity of 1.5 million gallons and imposes a total load of 6,250 tons over an area of 5,740 ft², or about 2,180 lb/ft². The maximum head of water is 35 ft. The radius of the tank is 42 ft, 7 in. The bottom of the tank is at elevation 6,935 ft.

Reinforcing bars were driven into undisturbed tuff at locations E-0, E-1, etc., as shown in Fig. 5-3. Prior to the filling of the tank on September 10, 1966, readings were taken on these bench marks to establish a base for measuring future deformation. Readings were then taken approximately every two weeks to establish a record of creep for the loaded area. On January 11, 1967, the tank water level was lowered to 4.5 ft to measure rebound. The tank was refilled and readings with 29.75 ft of water were taken on January 18, 1967, and with 33.0 ft of water on March 6, 1967.

Assumptions. The water reservoirs impose loads on the soil in agreement with Boussinesq's theory, and the Newmark chart applies. The solution for the theoretical deformation under load was derived using the Newmark chart. The pressure at predetermined depths in the foundation material was computed, and the deformation at these depths was calculated using Hooke's law. This analysis assumed that the foundation material is an isotropic uniform material.

Volcanic tuff of both the units 3 and 2b strata was excavated and tested by the Zia Company's Material Testing Laboratory. The samples were oriented in three

directions for testing: vertical, north-south (horizontal), and east-west (horizontal). Average values for the modulus of elasticity for the unit 3 tuff were as follows:

Vertical	21,390 psi
East-West	15,340 psi
North-South	14,080 psi

Fig. 5-3. Plan, Pajarito Reservoir No. 2.

A value of 20,000 psi for the modulus of elasticity was used in the calculations for deformation for unit 3 tuff. The depth of this stratum was assumed to be 50 ft. Over 75% of the total deformation occurs in the first 50 ft below ground level.

The value for the modulus of elasticity in the vertical direction at the surface of the unit 2b tuff was approximately 100,000 psi. This value was used in the calculations for the tuff from 50 to 75 ft below the surface. Below this stratum a value of 150,000 psi was used, based on borings taken on Mesita de Los Alamos.

Calculated versus Empirical Deformation. To minimize error, five readings representing one month under full load were averaged to obtain the empirical deformation curves given in Fig. 5-4. These curves represent the measured deformation as a function of distance from the center line of the tank in the north and east directions. The calculated deformation curve was derived using the Newmark chart, and the modulus of elasticity determined by tests conducted by The Zia Company.

The agreement between calculated and empirical deformation is approximately the same as the agreement between the deformation recorded in the east and west directions. The agreement is poorest near the tank and becomes more favorable as the distance from the tank increases. Near the the tank the localized effect of the tank ring wall may cause some deviation from theoretical prediction.

In general, the theoretical deformation is within 0.002 ft of the empirical deformation at the greater distances from the tank, and is conservative closer to the tank.

Mesita de Los Alamos

Surplus Atlas silo counterweights were used to impose loads on Mesita de Los Alamos. The dimensions and weights of the counterweights are as follows:

No.	Size	Weight Each (lbs)	Total Weight (tons)
39	19.0 x 3.25 x 1.0 ft	26,887	524.3
21	19.0 x 3.25 x 0.5 ft	14,638	153.
11	9.0 x 3.25 x 1.0 ft	12,712	69.9

The area on which the weights were imposed was first leveled to undisturbed tuff. A 4-in. layer of pulverized tuff was placed and compacted over this area to assure an equal distribution of load to the undisturbed tuff below. Empirical data obtained from the load test were used to determine the type of analysis; i. e., flexible or rigid foundation.

Fig. 5-4. Pajarito Reservoir No. 2, deformation versus distance.

On October 24 and 25, 1966, counterweights weighing 750 tons were placed at location No. 1 on Mesita de Los Alamos. Three counterweights, 19 ft x 3.25 ft, 3 in. x 1 ft, were placed on the bottom followed by five counterweights, 9 x 3.25 x 1 ft, to distribute the load. The remaining counterweights were then placed with the 19-ft dimension in the east-west direction. The total area on which the load was imposed was approximately 185 ft²; the total load imposed was 4.05 ton/ft². Deformation was recorded for five weeks, at which time all creep appeared to have stopped. On December 9 and 10, 1966, the weights were transferred from location No. 1 to location No. 2, where 4.05 ton/ft² were imposed on the tuff. The final reading at this position was taken on December 27, 1966, which allowed two weeks for deformation. It was noted that most of the deformation occurred during the first two-week period at location No. 1, and that little

would be gained by applying the load at location No. 2 for a longer time.

On December 27 and December 28, 1966, the test weights were transferred from location No. 2 to location No. 1. Three counterweights measuring 9 x 3.25 x 1 ft were placed on the bottom followed by three counterweights, 9 x 3.25 x 1 ft to distribute the load. The remaining counterweights were then placed with the long direction running east-west. Additional bench marks were driven into the ground at the four corners of the new configuration prior to the placing of the counterweights. The total area on which the weight was imposed was 87.5 ft²; the load imposed on this area was 8.55 ton/ft².

On March 8, 1967, the test weights were relocated to location No. 2, where 8.55 ton/ft² were imposed on the tuff. Additional bench marks were driven at the four corners of the new configuration as was done at location No. 1. Deformation was recorded and the effect of creep was noted. This test was discontinued when contractor's equipment was moved into the immediate area.

Assumptions. The Newmark influence chart for computation of vertical stress and a computer program using the Boussinesq equation for vertical stress were used to determine the stress at various depths below the surface. The deformations at these depths were determined using Hooke's law.

Values for the modulus of elasticity were obtained from the study conducted by the Albuquerque Testing Laboratory (1966). In the unit 3 tuff, in the vicinity of the test areas, values of 25,600, 30,600, and 23,300 psi were determined from tests. A value of 25,000 psi was used in the calculation for deformation for the unit 3 tuff. At location No. 1 the depth of stratum for unit 3 tuff was assumed to be 50 ft based upon information from test boring No. 12, (Albuquerque Testing Laboratory, 1966). At location No. 2, based upon information from test boring No. 15, the depth of stratum for unit 3 tuff was assumed to be 25 ft. The unit 2b tuff was assumed to have a modulus of elasticity of 160,000 psi based upon the test conducted at elevation 6,932 on a sample from test boring No. 12. Deformation was calculated for a depth of 85 ft after which

the additional increments of deformation were considered negligible.

Calculated versus Empirical Deformation.

Location No. 1, 4.05 ton/ft²

To minimize error, a total of five readings representing a period of over one month was averaged to obtain the empirical deformation curves given in Figs. 5-5 and 5-6. These curves represent the measured deformation as a function of distance from the center line of the test load in the north (Fig. 5-5) and in the east (Fig. 5-6) directions. A review of readings taken on a secondary bench mark indicates that the primary bench mark was not disturbed. Very little creep is indicated by the readings taken during this month, and the difference between the high and low readings for a particular bench mark may be attributed to survey inaccuracy.

The results of this phase of the load test indicate a close correlation between empirical and calculated deformations. The largest deviation noted was 0.001 ft with the exception of the closest east bench mark, which was probably disturbed during erection of the load test weights; a much smaller difference was noted between empirical and calculated values at location No. 2 for a bench mark at a similar location.

Location No. 1, 8.55 ton/ft²

A total of seven readings representing a five-week period were averaged to obtain the empirical deformation curves given in Fig. 5-7. These curves represent the measured deformation as a function of distance from the center line of the test load in the north and in the east directions. A review of readings taken on a secondary bench mark indicates that the primary bench mark was not disturbed. Very little or no creep is indicated by the deformation recorded during this five-week period, and the difference between the high and the low readings for a particular bench mark may be attributed to survey accuracy limitations.

Fig. 5-5. Deformation versus distance: location No. 1 (4.05 ton/ft^2), Mesita de Los Alamos.

Fig. 5-7. Deformation versus distance: location No. 1 (8.55 ton/ft^2), Mesita de Los Alamos.

Fig. 5-6. Deformation versus distance: location No. 1 (4.05 ton/ft^2), Mesita de Los Alamos.

Agreement within 0.001 ft is noted from 10 ft to 40 ft from the center line of the test load. A larger difference is noted 5 ft from the center line of the test load but the deformation-distance curve is steep at this point and the locations of the bench marks with respect to the center line of the test load are not precisely known. The agreement between empirical and analytical results is good.

Location No. 2, 4.05 ton/ft^2

This loading was imposed at location No. 2 for approximately two weeks. The bench marks for location No. 2 were modified to allow more precise readings, and the accuracy of the readings at this location is considered good. The average of two independent readings was used to derive the empirical results shown in Figs. 5-8 and 5-9. Very little creep is indicated for the two-week period. As seen in Figs. 5-8 and 5-9, the agreement between empirical and analytical results is exceptionally good.

Fig. 5-8. Deformation versus distance: location No. 2
(4.05 ton/ft²), Mesita de Los Alamos.

Fig. 5-10. Deformation versus distance: location No. 2
(8.55 ton/ft²), Mesita de Los Alamos.

Fig. 5-9. Deformation versus distance: location No. 2
(4.05 ton/ft²), Mesita de Los Alamos.

Fig. 5-11. Deformation versus distance: location No. 2
(8.55 ton/ft²), Mesita de Los Alamos.

Location No. 2, 8.55 ton/ft²

This loading was imposed on the tuff at location No. 2 on March 8, 1967. Upon inspection of the test load, it was determined that localized failure had occurred and that the weights were leaning to the north and west. The northerly lean was 3-1/4 in. in 12 ft which, after two weeks, had become 2-3/4 in. in 12 ft. The westerly lean was 2-3/4 in. in 12 ft, and after two weeks the test load was plumb in the east-west direction.

Figures 5-10 and 5-11 record the deformation due to the 8.55 ton/ft² loading. The agreement between theoretical and empirical deformation is not as good as for the 4.05 ton/ft² loading, but it is within 0.002 ft at all points 10 ft or more from the center line of the test load. Usually the theoretical deformation is greater than the empirical results and is therefore conservative. Figures 5-10 and 5-11 are based on the readings taken March 10, 1967; readings taken after this date were discounted because of increased construction activity in the area.

Computer Analysis for Deformation. A computer program was written to calculate deformation at various distances from the center of the test load using Boussinesq's equation for vertical stress and Hooke's law. Results of this computer analysis are shown as triangles on Figs. 5-5 through 5-11. The correlation between computer and empirical results is good.

Deformation Under Sustained Loading

The deformation of the tuff at selected bench marks was studied to determine if a creep factor could be associated with the Bandelier tuff. The bench marks selected were those closest to the test load which, therefore, experienced the greatest deformation. By choosing these bench marks, the inaccuracies attributed to surveying are reduced compared to the total deformation.

Pajarito Reservoir No. 2

The test load at this location cannot be considered steady because the water level within the tank is constantly changing. The tank gauge readings are shown on Figs.

5-12 and 5-13. There appears to be no discernible creep recorded at bench mark N-1 or E-1 over the five months of observation. The variations in the readings may be attributed to survey inaccuracies or water-level fluctuations.

Mesita de Los Alamos, Location No. 1

Figures 5-14 through 5-17 are records for bench marks S-O, E-O, N-O, and W-O, respectively, for the test at location No. 1 for the period between October 21, 1966, and March 17, 1967. The reliability of readings taken after March 17, 1967, should be discounted because of construction activity in the area. Particular attention should be given to the deformation under load during (1) the time period October 26 to November 30, 1966, when a test load of 4.05 ton/ft² was imposed on the tuff and (2) the time period between December 28, 1966, and March 3, 1967, when a test load of 8.55 ton/ft² was imposed. Another interesting comparison is one of deformation prior to initial loading, after the 4.05 ton/ft² load was removed.

October 26 to November 30, 1966. During this period E-O and W-O, Figs. 5-15 and 5-17, respectively, show no creep; N-O, Fig. 5-16, shows minor creep under sustained load; and S-O, Fig. 5-14, shows a small negative creep (recovery) under sustained load. The change in readings at N-O and S-O could be attributed to survey inaccuracies or to a redistribution of stress in the tuff. During this five-week period there was no indication of a large amount of deformation due to creep. The load imposed during this period was 4.05 ton/ft².

December 28, 1966, to March 3, 1967. During this nine-week period a load (8.55 ton/ft²) which approached the ultimate bearing capacity of the tuff was imposed. S-O (Fig. 5-14) and N-O (Fig. 5-16) located 5 ft, 4-1/2 in. from the center line of the test load recorded larger deformations than E-O and W-O, which were located 10 ft from the center line of the test load.

Fig. 5-12. Deformation versus time: Pajarito Reservoir No. 2, bench mark N-1.

Fig. 5-13. Deformation versus time: Pajarito Reservoir No. 2, bench mark E-1.

Fig. 5-14. Deformation versus time: location No. 1, Mesita de Los Alamos, bench mark S-O.

Fig. 5-15. Deformation versus time: location No. 1, Mesita de Los Alamos, bench mark E-O.

Fig. 5-16. Deformation versus time: location No. 1, Mesita de Los Alamos, bench mark N-O.

Fig. 5-17. Deformation versus time: location No. 1, Mesita de Los Alamos, bench mark W-O.

For all four bench marks, there is little indication of an increase in deformation due to creep.

Irrecoverable Deformation. At S-O a large (0.01 ft) permanent deformation was recorded between the no-load condition on the undisturbed tuff and the no-load condition after the 4.05-ton/ft² load was removed. This large permanent deformation did not occur from the time the 4.05-ton/ft² load was removed to the time the 8.55-ton/ft² load was removed. For bench marks E-O, N-O, and W-O, the permanent deformation from a no-load condition on undisturbed soil to a no-load condition after the 4.05-ton/ft² load was removed is much smaller (about 0.004 ft). There appears to be no further permanent deformation caused by the application of the 8.55-ton/ft² load.

Mesita de Los Alamos, Location No. 2

Figures 5-18 through 5-20 record the deformation experienced at E-O, N-O, and S-O at location No. 2. E-O was located 10 ft from the center line of the test load; S-O and N-O were located 5 ft, 4-1/2 in. from the center line of the test load. Unfortunately, the test load at location No. 2 was not imposed for a long enough period to obtain a good record of creep deformation. However, all three records indicate that from December 12 to December 27, 1966, a large amount of creep deformation did not take place. From December 28, 1966, to March 3, 1967, a period of approximately 9 weeks during which there was no load imposed at location No. 2, negative creep (recovery) did not take place either. For location No. 2, after the load was removed, bench marks E-O, N-O, and S-O were either at or below the same elevation as prior to the original loading of 4.05 ton/ft². The test load of 8.55 ton/ft² imposed at location No. 2 on March 8, 1967, caused the volcanic tuff to yield. Localized failure caused the test load to lean towards the north 3-1/4 in. in 12 ft and towards the west 2-3/4 in. in 12 ft. After approximately two weeks under load, the north-south lean had been reduced to 2-3/4 in. in 12 ft, and in the east-west direction the test load was plumb. This can be

attributed to a localized failure of the volcanic tuff on the west side of the test load followed by a localized failure on the east side. At N-O and S-O, which were the closest bench marks to the test load, the 8.55 ton/ft² load caused a deformation comparable to that experienced at location No. 1, even though localized failure occurred under the test load. It is proposed that the localized failure resulted in a shear and bearing failure of the tuff around the perimeter of the test load, and that the effect of this failure beyond the perimeter is negligible.

Conclusions

The test conducted on Mesita del Buey and on Mesita de Los Alamos did not answer all of the questions that a designer could ask. The effects of vibratory loading and of numerous repetitive loadings on deformation must still be determined. The tests do give adequate information for the design of foundations to limit deformation with respect to static loading, and thus encompass the majority of loadings to be imposed by the Meson Physics Facility.

Deformation under Load

Newmark's chart or the Boussinesq equation for calculating vertical stress may be used to determine the stress in the tuff. The application of Hooke's law will yield values for deformation due to this stress. Empirical results indicate a correlation between this theoretical approach and actual deformation. Incorrect values for the modulus of elasticity of the foundation material are the more likely reasons for errors. Care must be taken to assure that the modulus of elasticity used in design is representative of the in situ modulus of elasticity of the volcanic tuff in the vicinity of the loaded area.

Deformation under Sustained Loading

Based upon the tests conducted for this study, the following is proposed. The initial deformation under load is comprised of an elastic deformation and a permanent deformation. This permanent deformation is about 20% for the unit 3 tuff; it would be less for the unit 2b

Fig. 5-18. Deformation versus time: location No. 2, Mesita de Los Alamos, bench mark E-O.

Fig. 5-19. Deformation versus time: location No. 2, Mesita de Los Alamos, bench mark N-O.

Fig. 5-20. Deformation versus time: location No. 2, Mesita de Los Alamos, bench mark S-O.

tuff. This permanent deformation occurs with the initial loading and is not recoverable. It is caused by a breakdown of the tuff, probably under the first 2 ton/ft² load.

After the permanent deformation has taken place, usually in the first few hours of loading, the tuff reacts as an elastic material. The effects of vibratory loading on creep is not yet known.

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