Preliminary Geotechnical Investigation

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W.1. INTRODUCTION

This report is submitted pursuant to a preliminary geotechnical investigation performed by this firm at the site of the proposed CHARA Array, located at the southeast corner of Mesa Negra, in the Acoma Indian Reservation, Cibola County, New Mexico. The object of the investigation was to evaluate the physical properties of the subsoils and rock underlying the site and to provide prelimary recommendations for foundation design, slab support, and site grading.

W.2. PROJECT DESCRIPTION

It is understood that the CHARA Array will consist of seven 1-meter aperture telescopes on altitude-azimuth mounts configured in a Y-shaped array enclosed within a 400-meter diameter circle. There are two structures with special construction requirements and seven structures with no special requirements. Those structures requiring special considerations include the Optical Path Length Equalizer (OPLE) building, measuring 29×295 feet, and the Beam Combining Laboratory (BCL), measuring 24×48 feet. If possible, the foundation and slabs of these two structures should be founded on bedrock to provide stability against vibration. It is anticipated that all foundation loads will be relatively light.

The proposed location of the structures were relayed through the offices of Koogle and Pouls Engineering, Inc. It is understood from our telephone conversation that the OPLE Building is very sensitive to low level seismicity and that it may be required that shock mounted bases be designed for this structure, depending on the ambient seismic acceleration.

Should final design details vary significantly from those outlined above, this firm should be notified for review and possible revision of recommendations.

W.3. INVESTIGATION

W.3.1. Subsurface Exploration

The exploration was conducted in concert with personnel with Koogle and Pouls and the use of an Air Force helicopter for transporting equipment to the mesa top. Because of the limited equipment that could be transported, only hand operated equipment was used. Since the site location had not been surveyed at the time of exploration, an approximate location of the site was determined by our crew in order to maximize the time available for exploration. Subsurface exploration consisted of making several potholes with pick and shovel through the surface layer of cobbles and boulders. Where the surface was too rocky to allow excavation, photographs were taken to document the site condition. Representative soils were sampled for subsequent laboratory analyses, and the holes were logged based on visual classification of soils. The results of the exploration are shown in Addendum A.

W.3.2. Seismic Refraction Survey

The refraction seismic survey was performed on October 20, 1993 by Roman Y. Juaregui, P.E., assisted by David Romero, both of this firm. The purpose of the survey was to assist in characterizing the geotechnical conditions and to evaluate the dynamic properties of the subsurface materials. Three 300-foot long compression wave seismic refraction survey lines were completed end to end across the telescope and support building sites. A 120-foot long seismic survey line, including both compression and shear wave setups, was completed near the center of the telescope site. This line was oriented perpendicular to the 300-foot lines at the end of Lines 1 and 2. A brief description of the seismograph equipment and procedures used in the survey are presented in Addendum B. The results of the seismic refraction survey are presented in this addendum, which includes time-travel distance plots, and interpretation of the data collected and analyzed.

Due to the general and approximate nature of the geophysical techniques applied, all depths, locations, and seismic velocities presented should be considered to be approximate. Interpretations of the data from the eight seismic lines are included in Addendum A.

The maximum depth of investigation for the eight lines was about 75 to 100 feet. It should be noted that softer, lower velocity layers or zones of material could underlie moderate and high velocity materials, but would not be identified using the refraction seismic technique. It should also be noted that the subsurface material velocities are average values obtained over distances of at least 25 feet and more typically at least 50 feet. Zones of material could have slower or faster velocities, and therefore be weaker or stronger than the average velocities interpreted from the seismic refraction data.

W.3.3. Laboratory Analyses

The analyses for physical characteristics were conducted on samples collected from the shallow hand dug pits. Moisture content determinations were made on all samples recovered. Selected samples were analyzed for grain-size, moisture density relationships and Atterberg limits. The results of these tests have been delivered to CHARA but are not included in this Appendix.

W.4. SITE CONDITIONS AND SOIL/ROCK PROFILE

W.4.1. Geologic Conditions

Mesa Negra, at elevation 8000 feet, is approximately 1200 feet (370 m) above the adjacent valley of the Malpais National Monument. The mesa top has a resistant caprock layer approximately 80 to 100 feet thick of Tertiary age basalt lava flow and volcanic cinder deposits that overlies the Gallup sandstone formation of Cretaceous age. The sandstone formation, consisting of several cliff-forming sandstone strata, is interbedded with thick shale units. The Gallup formation is underlain by several hundred feet of Mancos Shale formation, consisting of marine shale and claystone that typically forms low angle slopes. At the base of the mesa are the Dakota Sandstone formation of lower Cretaceous age and the Zuni Sandstone of upper Jurassic age which forms the low cliffs facing the Malpais Valley.

Mesa Negra is an erosional feature that is a remnant of an extensive basalt capped surface that includes the Horace Mesa surface at the foot of Mount Taylor that has been dated at 2.5 million years old. It is thought that this surface formed the pedestal for the later formation of the Mount Taylor volcano. The Malpais Valley is a fault-bounded graben structure that is filled to depths of 900 feet with lava flows dating from about 1.5 million to 3000 years old (Love 1993). The age of last movement of the fault that borders the west side of the mesa is thought to be young, possibly Holocene (Hawley & Love 1991).

The mesa terrain at the site location slopes gently to the east toward a broad north-south swale in the middle of the approximately four mile square mesa. This swale is the axis of the McCartys Syncline. Apparently folding occurred after the flow was deposited. A small peak on the east half of the mesa is considered to be a volcanic vent that likely produced some of the lava flows and cinder deposits that cap the mesa. A fault zone forms the boundary between the valley and the foothills at the west side of the mesa. Stereo-pair aerial photographs indicate that there may be an east-west fracture that has dissected the mesa top near the north end of the site and which appears to intersect with the syncline axis that may also be fractured. The presence of the fractures has not been confirmed by inspection, however.

The variable densities of the volcanic deposits are the result of different cooling rates. The thicker flows at the bottom of the cap have cooled slower and have fewer cracks and gas vesicles, while the flows in the upper part are thinner and have numerous cracks and gas vesicles due to rapid cooling. Ejected pyroclastic deposits, such as ash and cinders which have a natural low density, are likely to occur more frequently in the upper thin lava flows as interbedded deposits. The lava flows and pyroclastic materials may have originated from several sources at varying distances.

W.4.2. Surface Conditions

The site surface is covered with a moderate to dense cover of pinion and juniper trees with a ground cover of grass and small shrubs. Line of site is limited to about 100 feet in most areas. In the vicinity of the proposed center of the site structure there are the remains of an apparent old windfall where the trees have been uprooted and laid oriented in a northeast trend, possibly as a result of a downdraft. The surface is gently sloping towards the syncline axis, forming a swale in the center of the mesa. The surface has a covering of desert pavement of mostly cobbles and gravel with occasional small boulders interbedded in the clayey soil. The higher ground typically has the thinnest soil cover. Weathered bedrock surface is exposed on the surface in many areas of the site. The surface slopes southeastward approximately 4 percent toward the middle of the mesa, with the local drainage divide at the western edge of the mesa.

W.4.3. Subsurface Profile

The subsurface soils consist of an approximate one foot layer of silty clay (CL-SC-GC) soils mixed with gravel, cobbles, and boulders. The rock material is comprised of basalt lava flow, welded tuff, and unconsolidated (non-indurated) cinder gravel. The silty clay soil, which has moderate to high potential for swelling, is likely derived from eolian material rather than from weathered rock. The thin soil grades downward at about 6 to 15 inch depth into weathered fractured bedrock. The cinder deposit appears to be interbedded with lava flows near the surface. Below about 10 feet the basalt is moderately fractured and below 30 feet depth the bedrock appears to be comprised of a thick dense basalt layer with columnar jointing. Results of the seismic survey indicate that a layer of low velocity bedrock occurs generally from 4 to 12 feet depth. This may be a combination of poorly indurated cinder deposits and/or a weathered/fractured basalt layer.

W.4.4. Soil Moisture and Groundwater Conditions

The thin clayey soils are well drained because of the slope and the fracture systems in the poorly consolidated volcanic bedrock. The moisture content of the soil samples ranged from 15.6 to 17.1 percent of dry weight. A rainstorm which occured the night before our field exploration had little affect on the actual moisture content below 0.5-inch depth. The effect of the rocky surface is to create a mulch that allows infiltration, conserves moisture, and provides support for the numerous pinons and juniper trees.

Groundwater is expected to occur as thin perched zones: at the base of the basalt layer; and at the base of the several sandstone strata of the Gallup formation observed at intervals in the upper 500 feet. A generalized geologic section of Mesa Negra underlying the CHARA site is shown in Addendum A. The recharge for the perched water below the basalt caprock is expected to occur through fracture systems, possibly along the axis of the McCartys syncline that forms the central swale in the interior of the mesa, and a possible east-west fracture, indicated by aerial photos, that dissects the mesa. The occurrence of Deer Spring, on the escarpment slope at the south end of the mesa, may be the exposure of a perched system that follows the syncline axis.

W.5. DISCUSSIONS AND RECOMMENDATIONS

W.5.1. Analysis of Results

The evaluation of subsurface conditions are based on minimal test pit excavations, observations of the surficial deposits exposed on the site surface and along the west edge of the escarpment, and from the interpretation of the seismic refraction data. The results of the initial seismic survey indicated a variable surface condition that is expected to mask fractures. Since the exact site was only approximated, the exploration is considered as only representative of the site area and not necessarily specific to the site foundation location.

Examination of stereo-pair aerial photographs indicates that an east-west fracture may cross the north leg of the site. The nature of this feature has not been verified during this phase of the investigation.

The data indicates that the soils below a depth of 1 foot typically are suitable for support of foundations and slabs for all non-special structures. Preparation of the top one foot of the natural grade will require the removal of vegetation and the recompaction to minimum moisture density standards as specified for structural fill.

Post-construction moisture increases in the supporting soils for non-special structures could cause some differential foundation movements and, thus, careful site drainage and moisture protection procedures will be necessary, specifically as a result of standing snow drifts and for concern of frost heave.

W.5.2. Dynamic Design

Dynamic material parameters were determined from the results of the shear and compression seismic wave refraction data presented in Addendum B. Poisson's ratio, dynamic modulus (E), and shear modulus (G) were calculated for the subsurface materials. Line 4, which had both compression and shear velocity data, served as the primary source of Poisson's ratio information which was assumed for Lines 1, 2, and 3. The recommended modulus values presented in Table W.1 below for use in preliminary design analysis for loads resulting in low soil strains are based on the modulus values calculated from the field data.

TABLE W.1. Recommended modulus values

Layer	Poisson's Ratio	Young Modulus (E) (ksi)	Shear Modulus (G) (ksi)
#1 (0′–3′ to 14′) at Lines 1, 2, 3, & 4 110 pcf assumed	0.35	20	9
#2 (3′–14′ to 25′-40′) at Lines 1, 2, & 3 130 pcf assumed	0.30	400	150
#2 (6′–47′ to depth) at Lines 2 & 4 130 pcf assumed	0.30	200	75
#3 (25′–40′ to depth) at Lines 1, 2, & 3 140 pcf assumed	0.30	2500	1000

The recommended elastic constants are based upon shear strains of about 10^{-3} percent for foundation design analysis. However, if higher strains should occur, E_{dyn} may need to be reduced for "strain softening" effects on the basis of relationships given by Seed and others, as referenced in Addendum B.

A preliminary damping ratio of 0.05 is suggested to account for internal damping of the soils. Actual design values will be dependent upon the shape and depth of the foundation, and the mass of the system supported by the foundation. Should it become necessary, a soils density of 110 pcf is recommended for use in final calculation of the damping ratio.

W.5.3. Recommendations for site grading

Site grading will encounter the weathered volcanic bedrock within one foot of the surface in most portions of the site. The rock will be rippable in most areas but boulders up to three or four feet will be encountered. Fill materials for use in exterior grading of roads, parking areas and for structures other than the OPLE and BCL structures, may incorporate native materials with the following exceptions: rock materials larger than one foot should not be used in any structural fills within one to three feet of the final subgrade; and rocks larger than six inches should not be used within the upper one foot of final subgrade elevation. Large rocks should be dispersed within a finer grained matrix so that the potential for voids will be eliminated. The grading for the OPLE and the BCL structures will be only on surfaces which have been cut to expose undisturbed bedrock surfaces.

W.5.4. Recommendations for foundations

These recommendations are based on preliminary exploration and prior to the location of the actual building sites. Site specific investigations of the subsurface conditions must be conducted when the exact location of the buildings have been determined, using drill rigs or heavy backhoe equipment, to determine the specific depths of the excavation that is necessary and to verify the interpretations of seismic data.

W.5.4.1. OPLE Building

The principal structure is the OPLE building, which has an inner shelter and an outer shelter that are isolated from each other. Design instruction indicates that the foundations of the inner shelter, which have very light wall loads, should either tie into the bedrock or consist of a floating reinforced slab to avoid differential movement within the 29×295 feet structure. Because of the need to avoid vibration that may originate from other buildings in the complex, it is necessary to place the foundation and slabs of the inner shelter on undisturbed bedrock.

The foundation and slab for the outer shelter structure may be designed as for all non-special structures, as detailed in the sections below.

The foundations for the interior shelter structure should consist of a structural slab that will bridge the variable bedrock conditions. Since the wall loads are very light no special footings are required. The depths of the foundation for bearing purposes may be as shallow as 2 feet; however, concerns for vibration sensitivity may require deeper placement of the foundation. Because of the length of the structure and the varying depths of excavation required to maintain a level grade, variable density bedrock, as indicated by the velocities, ranging from 1100 – 3230 feet/sec., will be expected if the foundation is placed at depths of 3 – 10 feet. Placement of the foundation at depths below 10 feet is expected to encounter variable rock velocities ranging from 3200 – 6200 feet/sec. Although the higher density rock will transmit vibration better, it has a higher frequency and therefore the displacement is less than lower density rock.

Because the response of the bedrock to vibration is not well understood, it is recommended that during the final site-specific exploration of the building sites a vibration monitoring study be conducted to determine the reaction of these various deposits to ground motion. Information gained from monitoring generator or compressor noise at various distances could be used to finalize the foundation depth of the critical structures.

It is expected that any excavation will result in a very uneven surface. A leveling material of granular base course will be required in order to construct a uniform thickness of the slab so that concrete shrinkage stresses will be predictable. The base course shall consist of well-graded, minus (–) 1-inch gravel and sand, with less than 8 percent passing the No. 200 sieve, and be compacted to 95 percent of the maximum dry density. The base course layer should have a minimum thickness of two inches over the bedrock. Any contact of the concrete with the bedrock will result in differential strain between such contacts.

The climate at this elevation has a frost depth on the order of 2 to 3 feet. Construction should avoid the use of fine grained soils that might be inducive to frost heave. The outer shelter slab should provide protection against frost heave for the interior shelter. However, the presence of snowpack may allow sufficient moisture accumulation in minor fills around the outer shelter walls to cause frost heave of the slab unless the exterior is graded for positive drainage.

W.5.4.2. BCL Building

The Beam Combining Laboratory (BCL) building requires a foundation and structural slab similar to the inner shelter of the OPLE building, so recommendations presented above are applicable to this structure also. This structure also requires an enhanced thermal insulation requirement and the mechanical isolation of heating/AC units from the optical table mounts of the BCL. Although the design requirements do not specify an isolated slab for the mechanical units, it is recommended that the foundation for these units be placed on a separate slab that is placed on thick granular fill at one end of the building to mitigate vibration potential.

W.5.4.3. Other Structures

There are seven other structures which have no special construction requirements. These include the Control Building, Machine and Electrical Shops, Garage, Offices, Kitchen/Lounge, Sleeping Quarters, and the Mechanical/Storage Building. The principal concerns for foundation of these structures are: that foundations be placed at a minimum depth of three feet below final subgrade on either select structural fill or properly prepared natural subgrade; and that the exterior of the foundations be graded for positive drainage to protect against frost heave. Floors may be designed using slab-on-grade with the subgrade consisting of a minimum thickness of six inches of minus one-inch well graded granular base course material.

The foundations should be designed for a safe soil bearing pressure of 1500 psf or have a minimum bearing width of sixteen inches. The bearing pressure recommended applies to full dead plus live loads, and can be safely increased by one-third for total loads, including wind or seismic forces. The foundation stem walls should be reinforced to allow for a degree of load redistribution should differential uplift or settlement occur. Vertical movements of footings designed as recommended above are estimated not to exceed 3/4 inch for competent native soils, prepared surficial soils, and engineered fill soils.

W.5.5. Construction Considerations

The excavation to competent bedrock for the critical structures should be verified by inspection. It is expected that bedrock with velocities less than 5,000 ft/sec will be rippable. The bottom of the excavation must be cleaned of disturbed material, particularly loosened boulders. Disturbed material having a thickness greater than 2 inches should be removed prior to placement of the compacted leveling course.

W.6. REFERENCES

New Mexico Bureau of Mines, 1993, personal communication with David Love, geophysicist, on revised ages of volcanic activity in Grants area now in publication, with G. Lindsey, SHB AGRA, 10/18/93

Hawley, J. W. & Love, D. W., 1991. "Quarternary and Neogene landscape evolution: a transect across the Colorado Plateau and Basin and Range provinces in west-central and central New Mexico", in *Field guide to geologic excursions in New Mexico and adjacent areas of Texas and Colorado*, ed. Betsy Julian and Jiri Zadek, Bulletin 137, New Mexico Bureau of Mines and Mineral Resources, Socorro, New Mexico

W.7. ADDENDUM A: TEST BORE HOLE EQUIPMENT AND PROCEDURES

W.7.1. Equipment and Sampling Procedures

Since the site is inaccessible by vehicle, only hand operated equipment was used for sampling. A powered hand auger was ineffective because of very rocky conditions. Samples were taken of the upper 16 inches of soil using pick and shovel. No undisturbed samples were taken.

W.7.2. Boring Records

Exploration activities were conducted by our field engineer/geologist, who examined and classified soils and prepared the boring logs. Soils were classified by observation and modified as necessary to reflect laboratory determinations of physical characteristics.

W.7.3. Terminology Used to Describe the Relative Density, Consistency, or Firmness of Soils

The terminology used on the boring logs to describe the relative density, firmness, or consistency of soils relative to the standard penetration resistance is presented in Tables W.2 – W.4 below. The standard penetration resistance (N) in blows per foot is obtained by ASTM D1586 procedure using 2″ O.D., 1-3/8″ I.D. samplers.

TABLE W.2. Relative Density: (left) Terms for description of relative density of cohesionless, uncemented sands and sand-gravel mixtures.

TABLE W.3. Relative Firmness: (right) Terms for the description of partially saturated and/or cemented soils which commonly occur in the Southwest, including clays, cemented granular materials, silts, and silty and clayey granular soils.

N	Relative Density
0–4	Very loose
5–10	Loose
11–30	Medium dense
31–50	Dense
50+	Very dense

N	Relative Density
0–4	Very soft
5–8	Soft
9–15	Moderately firm
16–30	Firm
31–50	Very firm
50+	Hard

TABLE W.4. Relative Consistency: Terms for the description of clays which are saturated or near saturation.

N	Relative Consistency	Remarks
0–2	Very soft	Easily penetrated several inches with fist
3–4	Soft	Easily penetrated several inches with thumb
5–8	Medium stiff	Can be penetrated several inches with thumb with moderate effort
9–15	Stiff	Readily indented with thumb, but penetrated only with great effort
16–30	Very stiff	Readily indented with thumbnail
30+	Hard	Indented only with difficulty by thumbnail

W.8. ADDENDUM B: REFRACTION SEISMIC EQUIPMENT AND PROCEDURES

W.8.1. Seismic Equipment

Refraction seismic surveys are performed using an EGG Geometrics Nimbus ES-1225 signal enhancement seismograph. This instrument has the capability to simultaneously record 12 channels of geophone data and produce hard copies of that data. Signal enhancement capability permits the use of a sledge hammer as the seismic energy source. A timing sensor is attached to the hammer, and for compression (vertical) waves a metal plate is set securely on the ground surface and struck. Generating shear (horizontal) waves involves setting the plate against a wooden plank or railroad tie oriented horizontal and perpendicular to the axis of the geophone array and striking with the sledge hammer. A truck is usually driven onto the tie in order to effectively couple the tie to the ground. In this investigation, two people were used as ballast and three steel rods were driven through the plank and into the ground to provide the coupling.

Because of the signal enhancement capability, signals from several or many strikes can be added together to increase the total signal available to obtain the seismic record. Although explosives can also be used as a compression wave seismic energy source, a sledge hammer does not require licenses or permits, nor does it involve special limitations or regulations. A cable with 12 geophone takeout positions at 25-foot intervals with vertical and, if needed, horizontal geophones are used. The seismograph system is extremely portable. In areas where vehicle access is not possible, the equipment can be mobilized by hand or packhorse.

W.8.2. Field Procedures

Refraction seismic lines are generally laid out using the 25-foot spacings on the geophone cables. A maximum depth of investigation on the order of 100 feet may be possible using the entire cable as a 300-foot array. Shorter spacings can also be used. For shorter lines with improved near-surface resolutions, 10-foot spacings between geophones result in a 120-foot array with a maximum depth of investigation on the order of 30 – 40 feet. To improve the resolution of near surface interfaces, sledge hammer source positions are generally set at 12.5 feet from the ends of a 25 foot spacing geophone array. Three shots are usually obtained for a refraction line: a foreshot, a backshot, and a midshot. The midshot is usually placed midway between the two center geophones so that it is the same distance from the nearest geophone as the foreshot and backshot. This permits interpretation of near-surface interfaces at the center of a refraction line as well as the endpoints. It also implicitly separates a 12 geophone refraction line into two 6 geophone refraction lines, which permits more refined interpretations of shallow and mid-depth subsurface interfaces.

Compression waves are recorded for general exploration work. Shear waves are also recorded when dynamic soil properties are desired. A shear wave arrival is verified by obtaining two sets of horizontal data that are 180° out of phase. The phase reversal is obtained by either reversing the horizontal geophone orientation or reversing the sledge hammer impact direction. Hard copy printouts of all field data are made and inspected as the information is collected.

W.8.3. Records & Interpretation

The operations are directed by our engineer, who operates the equipment, prepares the records, and examines the data in the field. Seismic data are interpreted in the office. When appropriate, preliminary interpretations are made in the field.

TABLE W.5. Modulus Values from Compression and Shear Wave Velocities

Layer	Line 1	Line 2	Line 3	Line 4
Layer 1, 110 pcf assumed				
Comp vel, fps	1100–1400	1300–1400	1300–1800	1110–1670
Shear vel, fps				360–1180
Poisson ratio	0.35*	0.35*	0.35*	0.37
E modulus, ksi	18 – 30	25 – 30	25 – 48	18 – 78
G modulus, ksi	7 – 11	9 – 11	9 – 18	7 – 39
Layer 2 Depth, ft	3 – 14	5 – 14	4 – 6	1 – 10
Layer 2, 130 pcf assumed				
Comp vel, fps	4300–8500	3600–6600	4500–5300	2360–5580
Shear vel, fps				1680
Poisson ratio	0.30*	0.30*	0.30*	(0.07)
E modulus, ksi	386–1510	271–896	420–590	154
G modulus, ksi	148–580	104–343	161–228	72
Layer 3 Depth, ft	32 – 38	28 – 35	15 – 61+	14 – 16 (Line 4 is Layer 2)
Layer 3, 140 pcf assumed				
Comp vel, fps	15,600	14,600	9800–11,000	9140
Shear vel, fps				4570
Poisson ratio	0.30*	0.30*	0.30	0.33
E modulus, ksi	5520	4780	2130–2730	1680
G modulus, ksi	2130	1840	816–1050	630

Notes:

* Poissons ratio assumed, shear velocity data was not collected on this line.

The dynamic properties measured by the seismic surveys involve shear strains of less than 10^{-3} percent. Should higher shear strains be involved in the problem being analysed, the shear modulus of the material may need to be reduced in accordance with relationships given by Seed et al. (1984).

W.8.4. Reference

Seed, H.D., Wong, R. T., Idriss, I.M., & Tokimatsu, K., 1984, "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils", College of Engineering, University of California, Berkeley, EERC 84/14, September

FIGURE W.1. Line 1: (top left) Geologic Interpretation of Refraction Seismic Data; (top right) Legend; (bottom) Refraction Seismic Time versus Distance. Note: Velocities are in feet per second. Topography, when shown, is approximate.

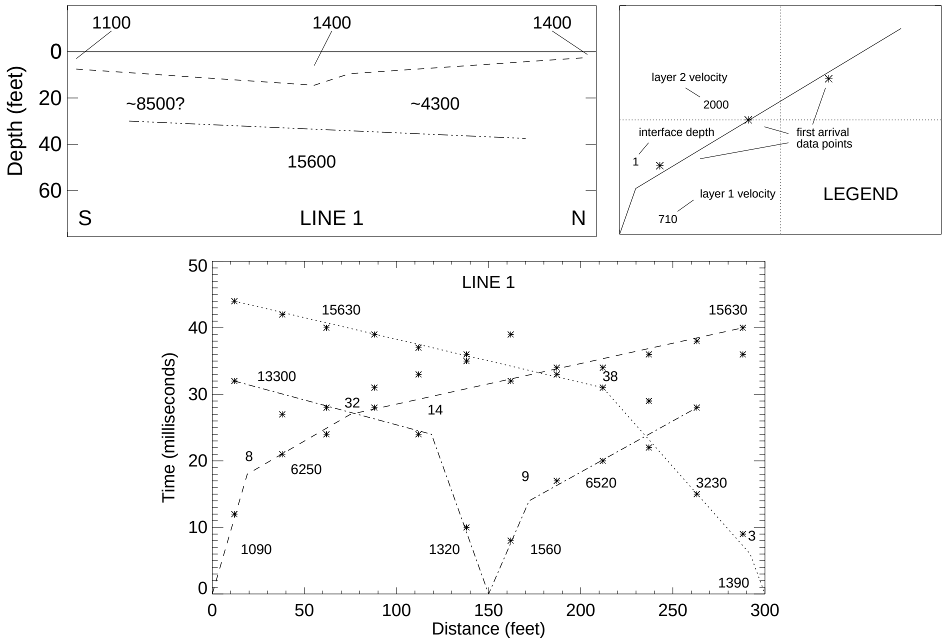

FIGURE W.2. Line 2: (top) Geologic Interpretation of Refraction Seismic Data; (bottom) Refraction Seismic Time versus Distance.

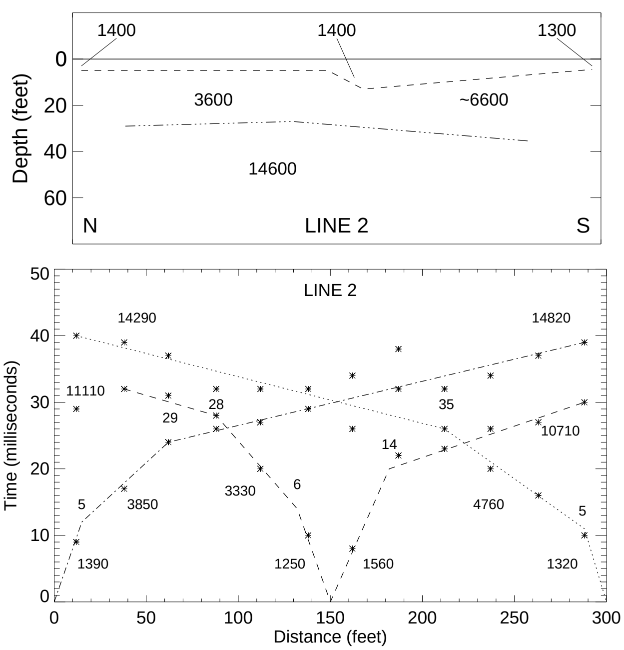

FIGURE W.3. Line 3: (top) Geologic Interpretation of Refraction Seismic Data; (bottom) Refraction Seismic Time versus Distance.

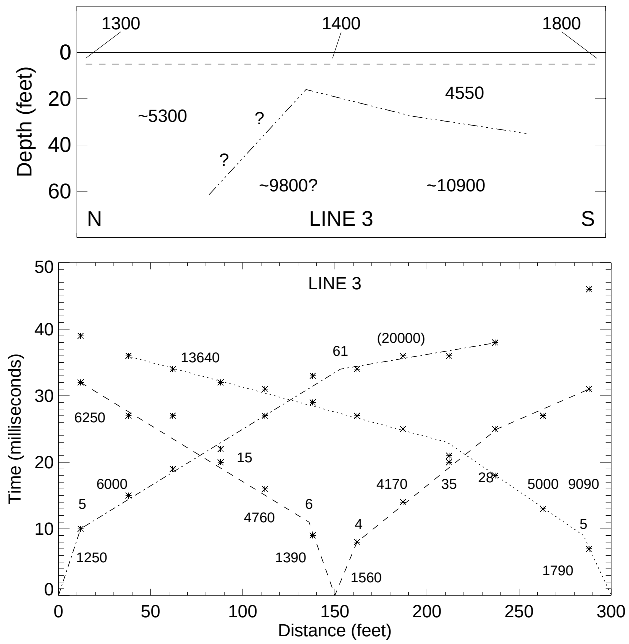

FIGURE W.4. Line 4 (compression): (top) Geologic Interpretation of Refraction Seismic Data; (bottom) Refraction Seismic Time versus Distance.

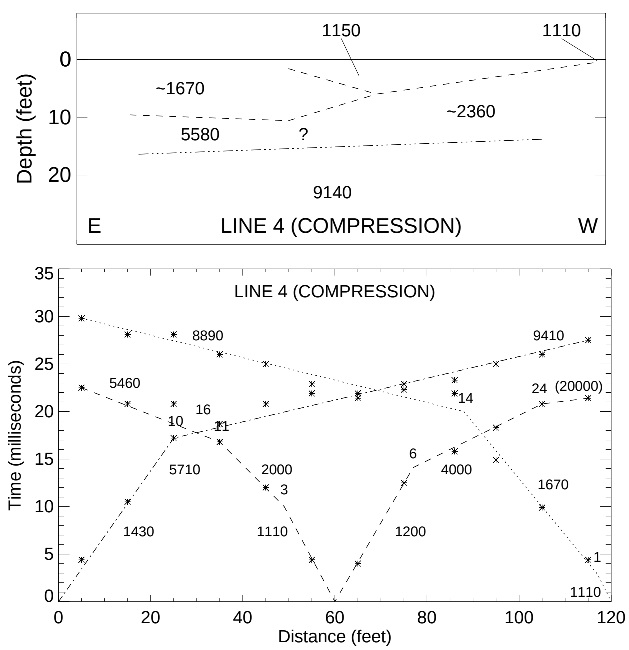

FIGURE W.5. Line 4 (shear): (top) Geologic Interpretation of Refraction Seismic Data; (bottom) Refraction Seismic Time versus Distance.