sandier material as backfill against the accelerator walls and the Klystron wall in order to minimize lateral earth pressures. However, there may not be sufficient select material to use it at all locations desired.

Select sandy material placed as fill for support of the accelerator or the Klystron housing should be compacted to approximately 95 percent of the maximum density determined by the Modified A.A.S.H.O.* method of compaction testing. Any shale placed beneath the structures should be compacted to at least 90 percent of maximum Modified A.A.S.H.O. density. There should be a provision for modifying the degree of compaction based on field observation during construction and further detailed study of compressibility characteristics.

We understand the degree of compaction of the shielding soil is not critical. However, we suggest that a compaction of approximately 90 percent of maximum Modified A.A.S.H.O. density be used. Based on this compaction, we believe that fill slopes for the entire alignment and end station areas would be stable when constructed on 1-1/2:1 (one and one-half horizontal to one vertical) with 10- to 12-foot wide benches every 30 feet vertically. Although the slopes would be generally safe against failure, considerable surface erosion could occur unless special provision were to prevent erosion--this should include compaction of the slopes, probably by rolling up and down the slopes, and suitable planting or other measures. Drainage ditches should be provided at the tops of slopes and along the benches. Periodic maintenance should be provided to keep the ditches clear and to correct any surface sloughing that may occur.

*American Association of State Highway Officials.

Based on a comparison of field density of the material and density of the compacted material, we expect approximately the following volume factors between cut and fill:

Required Compacted Density Percent of Maximum Modified A.A.S.H.O. Density	Cubic Yards of Fill Obtained from One Cubic Yard of Cut	
	Shale	Sandstone
85	1.05	1.09
90	1.00	1.04
95	.96	1.00
100	.92	.96

NOTE: In arriving at the above factors, we have estimated that actual compaction percentage will be about 2 percent greater than the required percentage.

Foundation Support: Foundation loads along the accelerator alignment are expected to be fairly light on the order of a few thousand pounds per linear foot of accelerator. Based on present information, settlement due to the weight of fill placed would be more critical than foundation bearing pressures. Fill placed as recommended above would provide suitable support for the accelerator and Klystron housings. Dead load plus design live load bearing pressures on the order of 4000 pounds or more per square foot could be used. Higher values could be used in the rock cut areas; however, it does not appear that they would be required for support of the accelerator and Klystron housings.

Lateral Pressures: Our present information indicates that about ten feet of fill will bear against the Klystron housing wall. The slope of the fill will impose some additional surcharge. The accelerator housing will be surrounded on three sides by fill. Pressure against the walls will depend on rigidity and shape of the wall, characteristics of the fill as placed, height and slope of fill, drainage, and construction sequence. We have based our estimates on the following:

1. Non-yielding vertical walls will be constructed and the backfill will be placed against them.
2. The sandy or shale backfill will be compacted to approximately 90 percent of maximum Modified A.A.S.H.O. density at a maximum moisture content about 3 percent above optimum.
3. The accelerator housing will have backfill on the sides and tops. The Klystron housing wall will have about 10 feet of backfill against it.
4. Drainage will be provided to prevent the build-up of hydrostatic pressure.

Under these conditions, we believe that lateral earth pressure coefficients of about 0.4 to 0.5 for sandy material, and 0.6 to 0.7 for clayey material could develop.

For present preliminary design of lateral pressure against side walls of the accelerator housing, we recommend that the backfill be considered to act as a fluid with a density of 60 pounds per cubic foot for the sandy soil, and 80 pounds per cubic foot for the clayey soil or clay shale. Since present plans call for a horsehoe-shaped section for the accelerator housing, the actual pressure distribution may be quite complex and be dependent upon the plasticity of the backfill material.

In considering lateral earth pressures against the Klystron housing wall, some allowance should be made for the increased pressures due to compaction equipment, possible surface loads on the fill adjacent to the wall, and the adjacent fill slope. For preliminary design, we recommend using a lateral pressure of 200 pounds per square foot in addition to the equivalent fluid pressure given above.

Settlement: Near Borings 10A, 12, 13, and 14, quite compressible clayey alluvium exists at accelerator floor elevation. For instance, in Boring 10A, where clayey soils were encountered to a depth of 24 feet, we estimate one foot or more of consolidation would take place within the clayey soils under a 75-foot high fill. Accordingly, in the Section "Site Preparation and Filling," we have recommended excavation and replacement of this alluvium under the acclerator housing and other important structural areas.

The shale would also compress under the proposed fill loads. The amount cannot be accurately assessed. For a given load, it will depend largely on the proportion of shale to sandstone, and the degree of weathering and distortion. The greatest consolidation is expected in the shales that have been highly weathered to a clay. We anticipate that in the shale areas settlement on the order of six inches may occur under fill loads of 75 feet. A large portion of this would occur as the fill is placed. We expect that most of the settlement would occur within two years after placement and that subsequent settlement would be a fraction of an inch per year.

Settlement in the Miocene sandstone also cannot be accurately assessed. However, we expect it to be much less than in the clay shale. Settlements under fills up to 75 feet deep are estimated to be on the order of one inch. Where shales are interbedded with the sandstone, the settlement may be greater.

Additional settlement will take place within the fill. The amount and rate will depend on type of fill material, compaction procedures and degree of compaction, water content, and height of fill. If fill is placed as recommended under "Site Preparation and Fill Placement," we believe that settlement of fill can be limited to tolerable amounts. In the areas where alluvium will be stripped, it appears that a maximum of about 45 feet of fill will be required below accelerator grade. Settlement of this fill due to its own weight, if it is sandy as recommended for fill under the accelerator housing, should be substantially complete prior to construction of the accelerator housing. We expect that subsequent settlement of the supporting fill due to the weight of the shielding fill and structure will be limited to a few inches, most of which will take place during placement of shielding fill and within two years afterward.

We recommend that provision be made to establish settelement observation points as fills are placed, in order to record magnitude and time-rate of settlement. Subsequent settlement behavior can thus be better predicted, and where necessary, construction procedures planned to aid in keeping structure settlements within tolerable amounts. As plans are further advanced, specific procedures for settlement observations can be developed.

EAST END AREA:

Foundations: Foundation materials at this end are good, consisting primarily of Miocene sandstone. Some shale or cemented gravel could be encountered, although none was evident in the borings at foundation depth.

Cut slopes of 1:1 (one horizontal to one vertical) may be used, with 10- to 12-foot wide benches at no greater than 30 feet vertically. Cuts less than 10 feet high may be made one-half horizontal to one vertical.

Although some seepage could occur upon cutting, we do not expect it to be a major problem in this area. However, the same type of slope erosion protection should be provided as for slopes along the accelerator alignment.

For major footings on the sandstone, preliminary design bearing pressures of 12,000 pounds per square foot may be used for dead and live loads, increased one-third for wind or seismic loading. If shales or very weakly cemented sands are encountered, some reduction will probably be necessary. Final design values should also consider foundation size and depth; narrow, shallow footings may require smaller bearing pressures. Design bearing pressures of about 6000 pounds per square foot would be appropriate for footings two feet wide and two feet deep, where the material is very weakly cemented.

Rebound during excavation, and subsequent settlement due to structural loading, will be small in the sandstone area. We expect less than one inch for all but extremely heavy loading. For instance, we would expect less than one inch for column loads of several hundred thousand pounds, with foundations imposing up to 12,000 pounds per square foot.

WEST END AREA:

Cuts: This area is in Eocene shale with interbedded layers of harder gray sandstone. We believe the shale can be readily ripped with heavy equipment, but that the sandstone will require some blasting where it is in thick layers. Although the sandstone will serve to strengthen the shale, design cut slopes should be based on shale properties. Permanent cuts in this area will be deeper than elsewhere.

For present designs, we recommend cut slopes no steeper than 1-3/4:1 for slopes over 30 feet high, or 1-1/2:1 for lower slopes, with 10- to 12-foot wide benches at no greater than 30-foot vertical intervals. Sufficient space should be available at the tops of slopes to permit flattening to about 2-1/2:1, in case it should be necessary in some places either during or after cutting.

Drainage: The seepage problem in this area would probably be more severe than in other areas, because of the deep permanent cuts and type of formation. The amount of seepage at time of excavation will depend largely on weather conditions. Provision should be made to install horizontal drains and collection systems. Locations where these are required must be determined in the field as excavation proceeds; this will require careful inspection by trained engineers or geologists. The same type of erosion protection should be provided as for slopes along the accelerator alignment.

Foundation Support and Settlement: Preliminary design bearing pressures on the order of 8000 pounds per square foot may be used for dead plus live loads. In general, foundations in shale should be established at least three feet below grade, to help minimize effects of swelling and shrinking. Somewhat higher bearing pressures would be appropriate at greater depth, and somewhat lower pressures may be required in local areas.

It is expected that some rebound will occur in the shale when the overburden load is removed, with subsequent recompression on reloading. We expect that rebound could be several inches in deep cuts. The amount will depend considerably on the mineralogy of the shale, availability of water to permit swelling, and the depth of cut. Subsequent settlement of heavy building or shielding loads could be several inches. We believe much of the settlement would take place as load is applied. Where structures are to be built in cut areas, they should be built as soon as feasible, to minimize moisture changes and the amount of swell and recompression within the cut area. Deep foundations can help minimize the effects of rebound, swelling, and settlement. It may even be desirable in some cases to overexcavate and replace undesirable material with compacted fill.

We recommend that settlement markers be established in the proposed structure areas as excavation proceeds, and on selected foundations after they are poured, in order to better determine the magnitude and time-rate of rebound and settlement. It will be desirable to establish settlement observation monuments at the bottoms of deep holes in the cut areas, to observe the amount of rebound upon excavation. Specific procedures can be developed as plans are further advanced.

EFFECT OF PROPOSED RESERVOIR:

If the reservoir water level is at Elevation 275, it will be against the accelerator embankment at several locations. We believe that fill slopes below this level should be 2-1/2:1 or flatter, and protected from erosion with rock rip-rap or by other means. Occasional flood water rising above Elevation 275 would probably cause some erosion and surface sloughing of the fill, which would require maintenance, but we do not believe it would cause any major instability.

Long-term storage of water would permit infiltration into the soils and rock under the accelerator. However, the proposed storage level of Elevation 260 is well below the proposed accelerator housing Elevation 275. Although access to water could cause a swelling tendency in some shales, we believe the weight of overlying fills would prevent appreciable swelling. Also, we do not believe that water infiltration or the weight of water in the reservoir would cause appreciable additional consolidation.

The following Plates and Appendixes are attached and complete this report:

Plate 1 - Plot Plan

Plate 2 - Profile Along Center Line of Proposed Linear Accelerator

Appendix A - Field Exploration and Laboratory Testing

Appendix B - Geology Report by Frank W. Atchley, Consulting Geologist

Respectfully submitted,

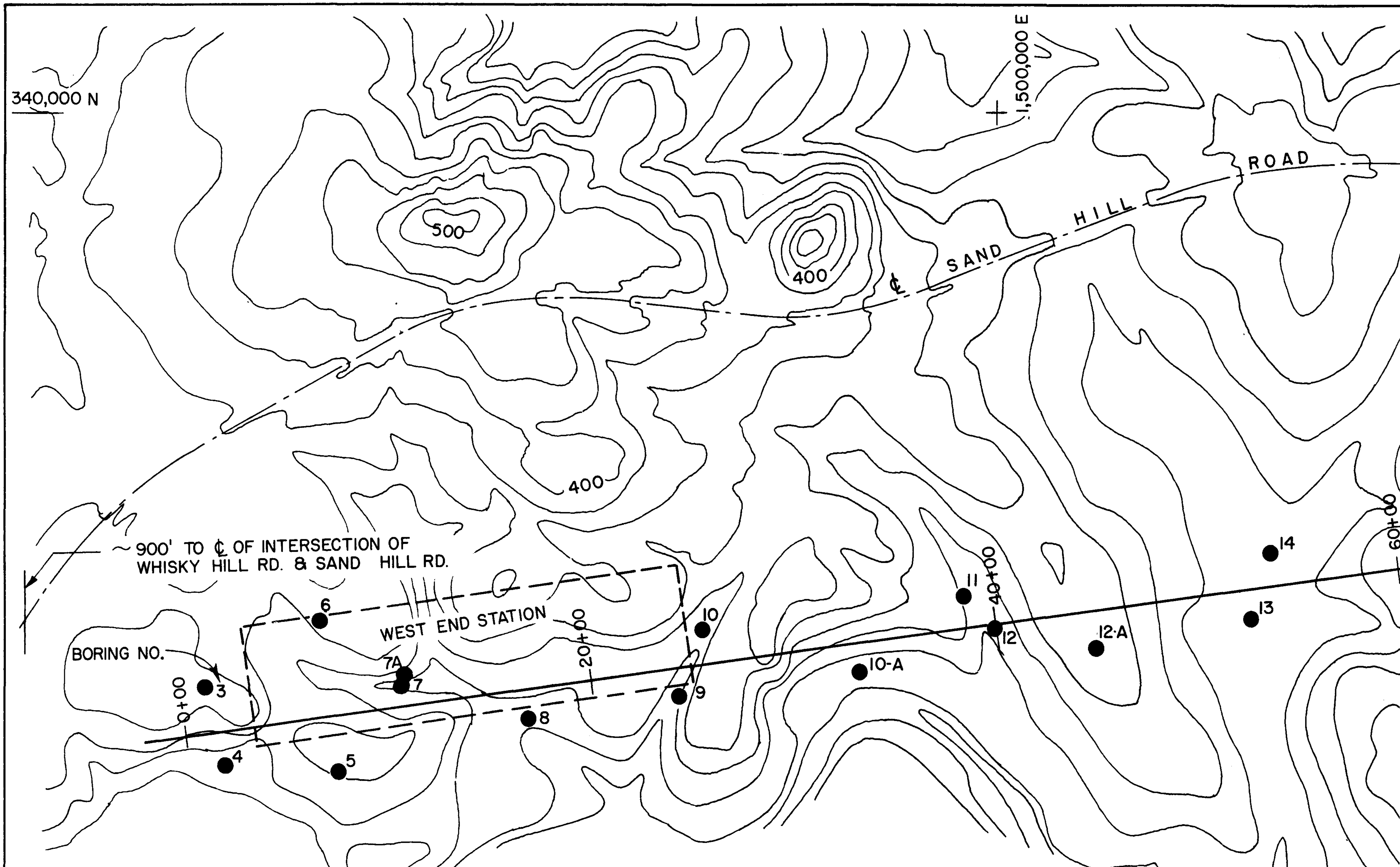
DAMES & MOORE

William W. Moore

William W. Moore

Charles L. Nichols

Charles L. Nichols



Note:
Sta. 0+00 = Intersection of Stanford University Coordinate 1,496,000 E & West end of Accelerator Alignment

PHOTO CONTOUR Map by
R. M. TOWILL, Inc.

Date of Photography: July 19, 1960

Grid is based on California Coordinate System, Zone 3
Mean Sea Level Datum, 1927 Adj.

Produced under the process of U.S. Letter Patent No 2,811,445

PLOT P

500 0 500 10

Scale in Feet

Contour Interval

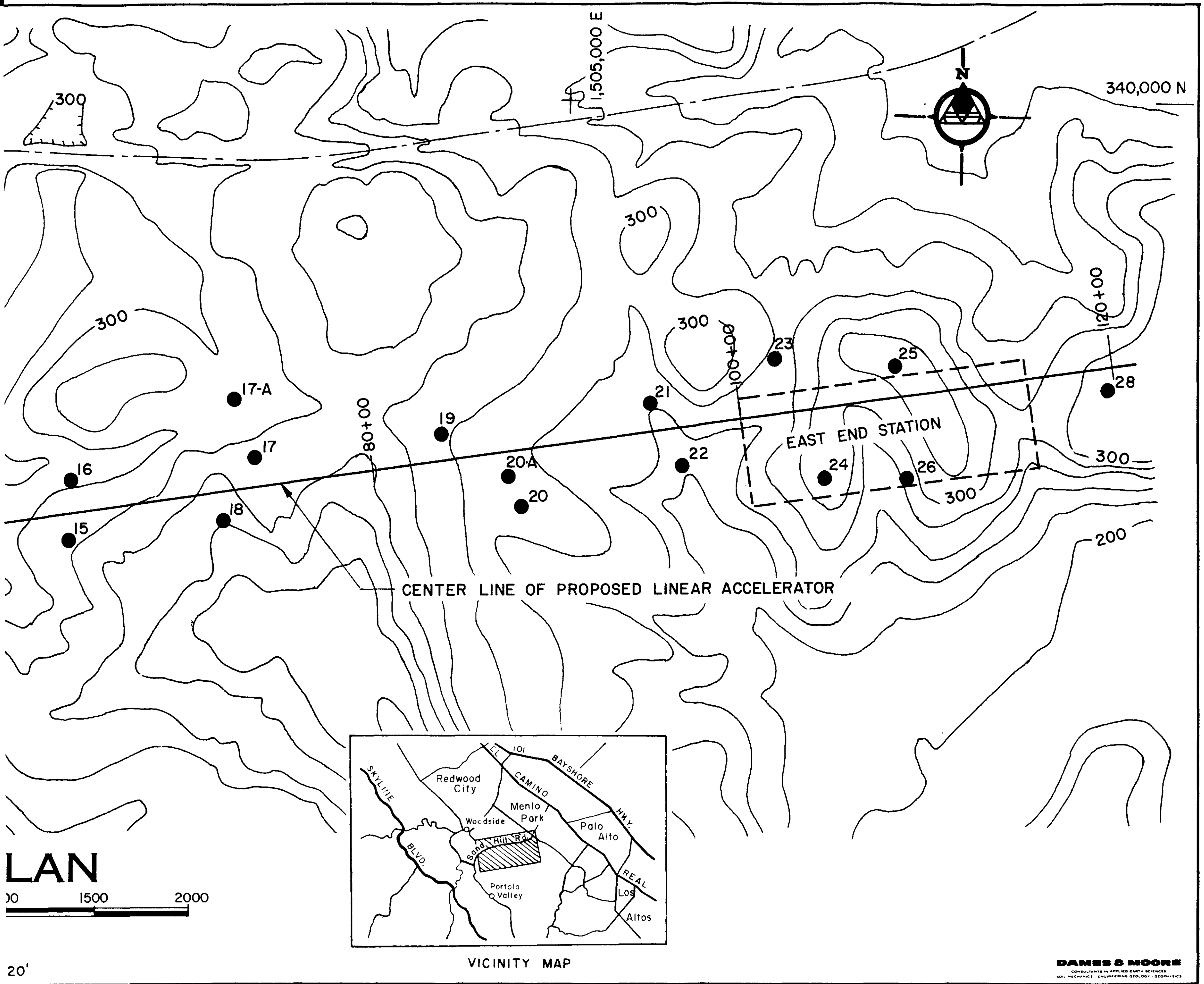
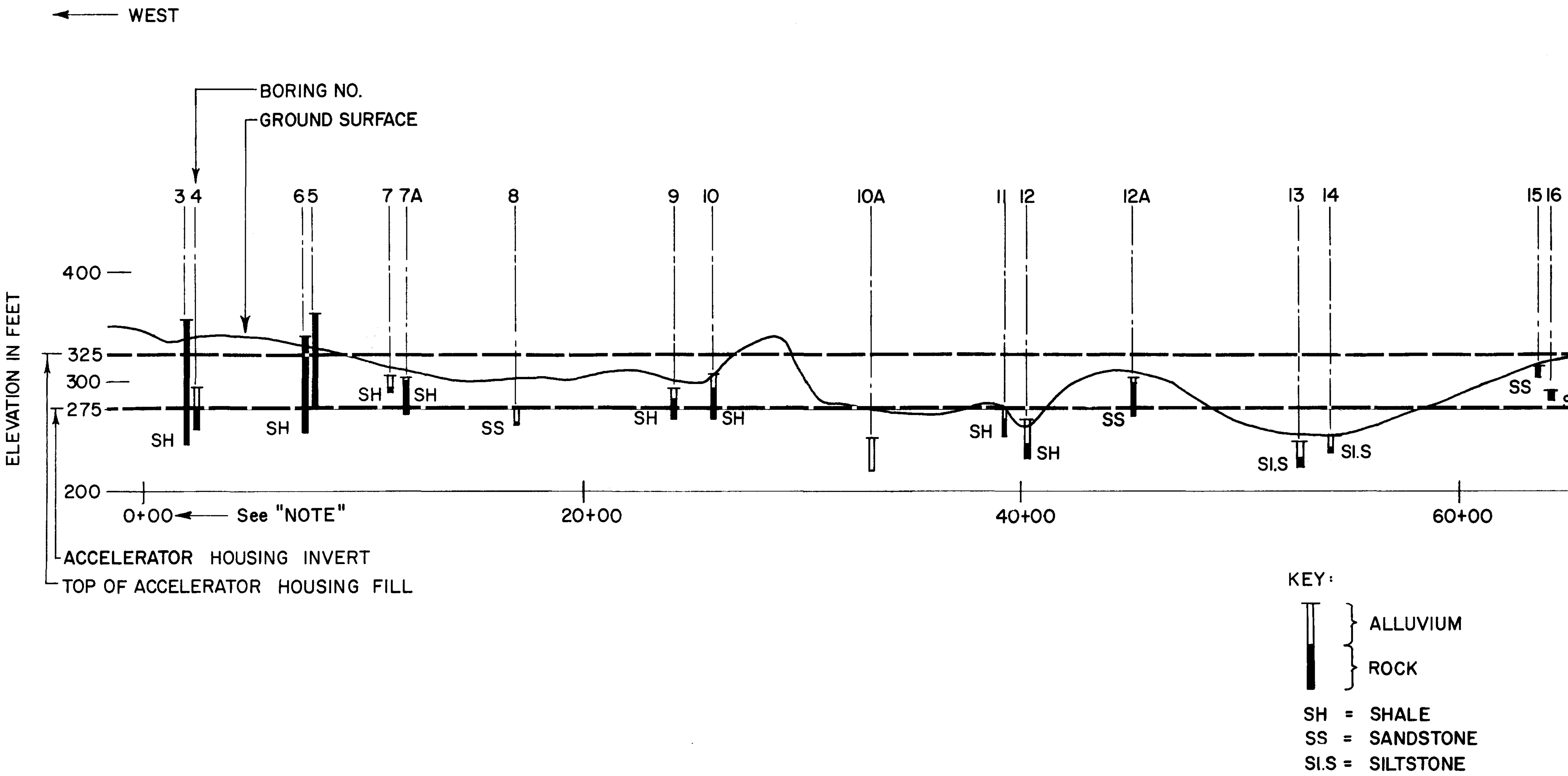
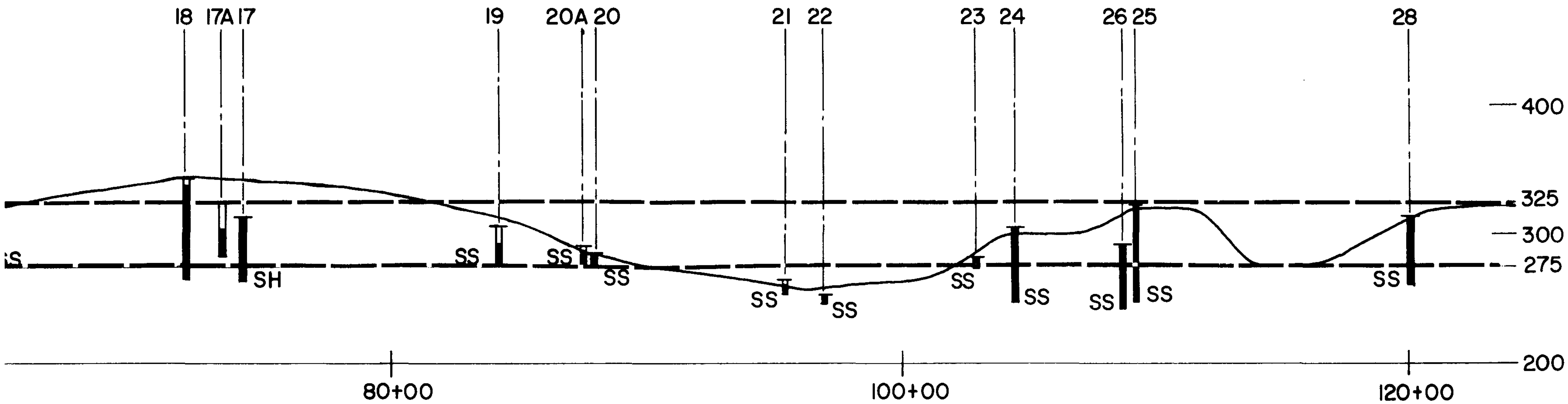

1,505,000 E
340,000 N
N
300
100+00
80+00
120+00
17-A
17
16
15
18
19
20-A
20
21
22
23
24
25
26
28
EAST END STATION
200
CENTER LINE OF PROPOSED LINEAR ACCELERATOR
LAN
00
1500
2000
20'
101
EL CAMINO REAL
BAYSHORE HWY.
SKYLINE BLVD
Redwood City
Woodside
Menlo Park
Palo Alto
Sand Hill Rd.
Portola Valley
Los Altos
VICINITY MAP
DAMES & MOORE

PLATE 1



PROFILE ALONG CENTER LINE OF PR

HORIZONTAL SCALE: 1" =
VERTICAL SCALE: 1" =



NOTE:
Sta. 0+00 = Intersection of Stanford University Coordinate 1,496,000 E & West end of Accelerator Alignment

:OPOSED LINEAR ACCELERATOR

500'
100'

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APPENDIX A

FIELD EXPLORATION AND LABORATORY TESTING

FIELD EXPLORATION:

Subsurface conditions along the accelerator alignment and at both end station areas were investigated by drilling 28 test borings and by trenching. Rotary-bucket drilling equipment was used for the relatively shallow borings and rotary-wash equipment for the deeper borings and also, in some instances, for the borings in which hard rock was encountered which the rotary-bucket equipment could not drill. The soils and rock encountered were logged by your engineers, with periodic site visits and advice on logging and sampling provided by Dames & Moore. Dr. Frank W. Atchley, Consulting Geologist, also made frequent site visits to observe the drilling and to obtain geological information. Representative bulk samples and undisturbed core samples were obtained for visual examination and laboratory testing. Undisturbed core samples were obtained utilizing the Dames & Moore Type D Sampler shown on Plate A6. Rock was also cored using both tungsten carbide and diamond bits with NX size core barrels. Graphical representation of the soil and rock encountered in the borings is presented on Plates A 1A through A 1W, Log of Borings. The nomenclature used in classifying the soils is defined by the Soil Classification Chart shown on Plate A2.

The drilling program was planned by John A. Blume & Associates, Engineers, and by Dames & Moore with additional advice by Dr. Atchley. Originally proposed Borings 1, 2, and 27 were not drilled.

Some trenching was done under the supervision of Dr. Atchley to aid him in delineating rock types and distribution and other detailed geology. The locations of the trenches are shown in Dr. Atchley's report in Appendix B.

LABORATORY TESTING:

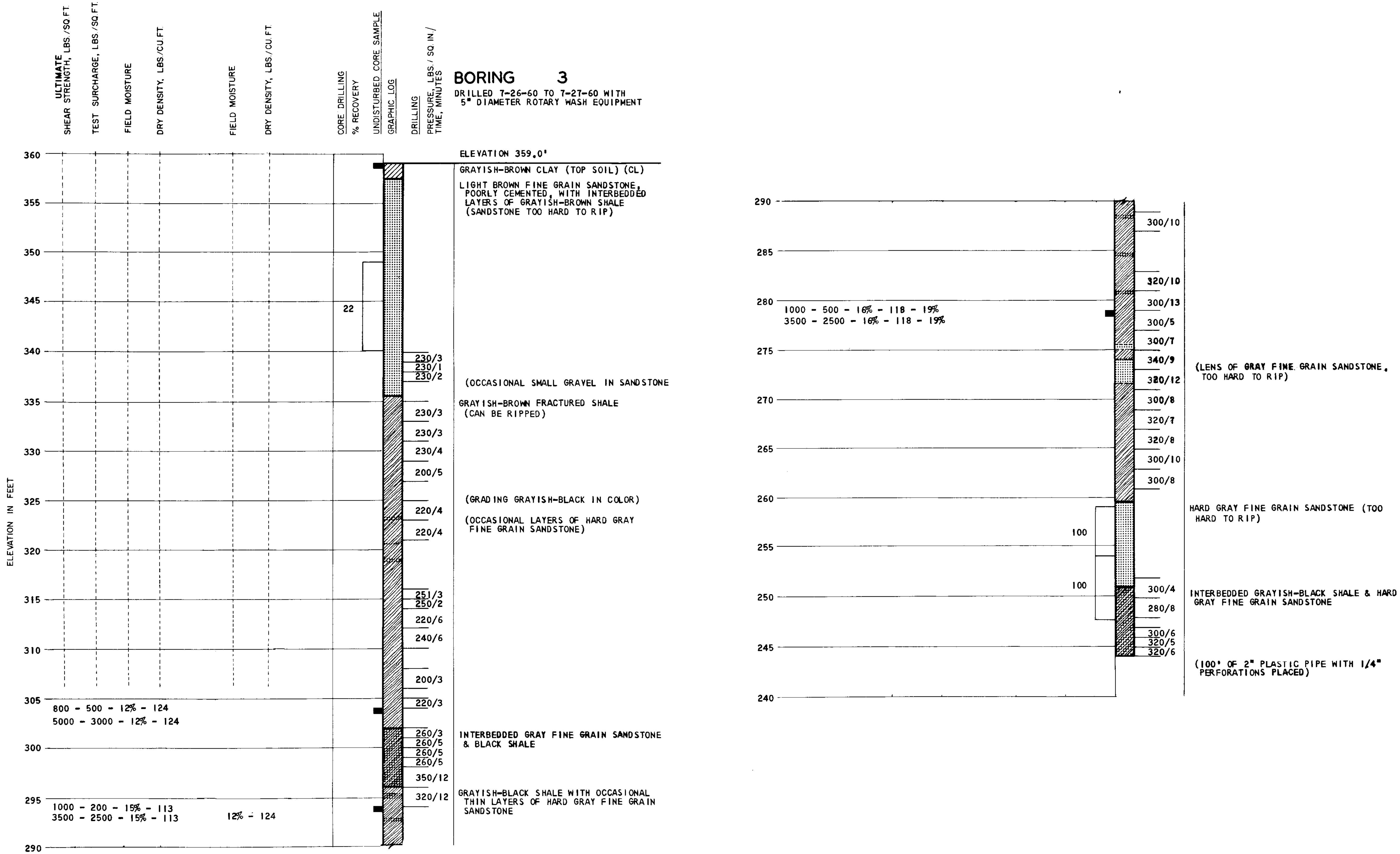
Selected undisturbed core samples obtained during the drilling program were tested in double direct shear, strain control, to aid in evaluating the stability of slopes and the supporting capacity of soil and rock encountered under the accelerator and target building structures. The results of the shear tests and moisture-density determinations are presented to the left of the appropriate boring logs. The method of performing these shear tests is shown on Plate A7, Method of Performing Direct Shear Tests. No rock samples were tested in triaxial shear because the samples became disturbed upon removal from the soil rings as a result of the friable or fractured nature of the rock.

To provide data for use in settlement calculations, consolidation (confined compression) tests were performed. The results of the consolidation tests are presented on Plates A 3A through A 3C, Consolidation Test Data. The methods used are explained on Plate A8, Method of Performing Consolidation Tests.

Compaction tests were performed on representative samples of sandstone and shale by the Modified A.A.S.H.O. Method shown on Plate A9, Method of Performing Compaction Tests. The results of the tests are presented on Plates A 4A through A 4C, Compaction Test Data. Representative direct double shear tests were performed on cores compacted to varying densities at varying moisture contents and under surcharges applicable to expected field conditions. The results are tabulated on Plate A5, Shear Test Data, Compacted Cores.

The following Plates are attached and complete this Appendix:

Plates A 1A through A 1W	- Log of Borings (Borings 3 through 26, and 28)
Plate A2	- Soil Classification Chart
Plates A 3A, A 3B, and A 3C	- Consolidation Test Data
Plate A4, A 4B, and A 4C	- Compaction Test Data
Plate A5	- Shear Test Data, Compacted Cores
Plate A6	- Soil Sampler, Type D
Plate A7	- Method of Performing Direct Shear Tests
Plate A8	- Method of Performing Consolidation Tests
Plate A9	- Method of Performing Compaction Tests



LOG OF BORING

DAMES & MOORE
CONSULTANTS IN APPLIED EARTH SCIENCES
SOIL MECHANICS · ENGINEERING GEOLOGY · GEOPHYSICS

PLATE A1A

BORING 4

DRILLED 7-28-60 WITH 24" DIAMETER ROTARY BUCKET EQUIPMENT

ELEVATION 294.0'

ELEVATION IN FEET	ULTIMATE SHEAR STRENGTH, LBS/SQ FT	TEST SURCHARGE, LBS/SQ FT	FIELD MOISTURE	DRY DENSITY, LBS./CU.FT.	FIELD MOISTURE	DRY DENSITY, LBS./CU FT
295						
~292					18%	110
290						
~287					13%	114
285						
~282					15%	101
280						
275						
~271	1500	500	13%	118	17%	124
270						
265						
260						
255						
250						
245						
240						
235						

UNDISTURBED CORE SAMPLE

GRAPHIC LOG

ALLUVIUM:

- GRAYISH-BROWN CLAY WITH SOME ROOTS (TOP SOIL)(CL)
- BROWN CLAY (CL)
- (GRADING TO DARK BROWN SILTY CLAY)
- BROWN SANDY CLAY WITH MANY ROCK FRAGMENTS (CL)
- (ROCK FRAGMENTS TO 6" DIA.)
- GRAY CLAYEY SAND (SOME SEEPAGE) (SC)

BROWNISH-GRAY FRACTURED SHALE (SOME SEEPAGE IN FRACTURES)(SOME RAVELING) (CAN BE RIPPED)

GRAYISH-BLACK FRACTURED SHALE (GRADING DRYER)

(THIN LAYERS OF GRAY FINE GRAIN SANDSTONE)

(BUCKET IS DRIFTING TO ONE SIDE BECAUSE OF SANDSTONE ON ONE SIDE OF HOLE)

GRAY FINE GRAIN SANDSTONE (TOO HARD TO DRILL WITH BUCKET, CANNOT BE RIPPED)

NOTE: WATER LEVEL AT ELEV. 263', 7-29-60

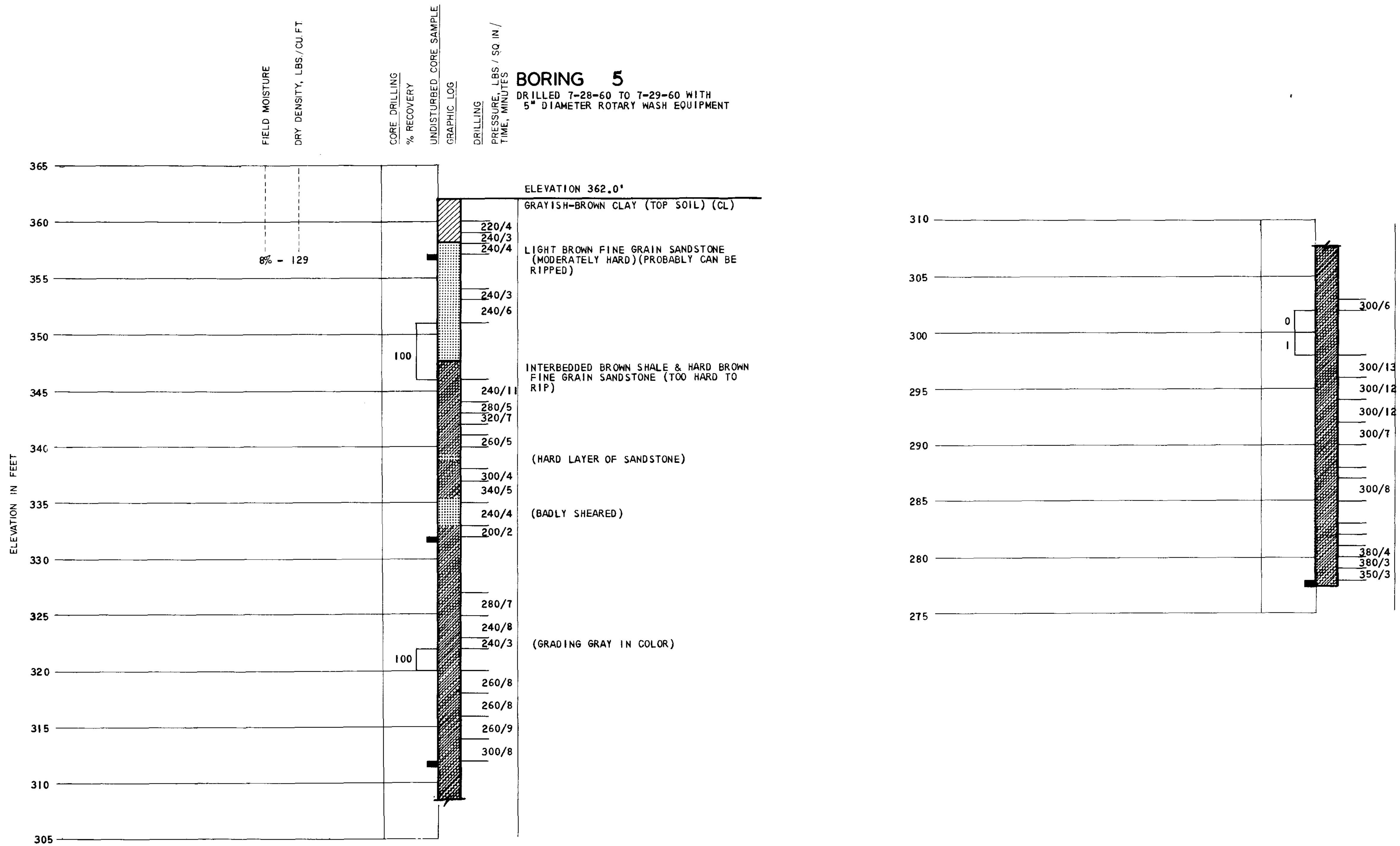
LOG OF BORING

DAMES & MOORE

SOIL MECHANICS ENGINEERS

PLATE A1B

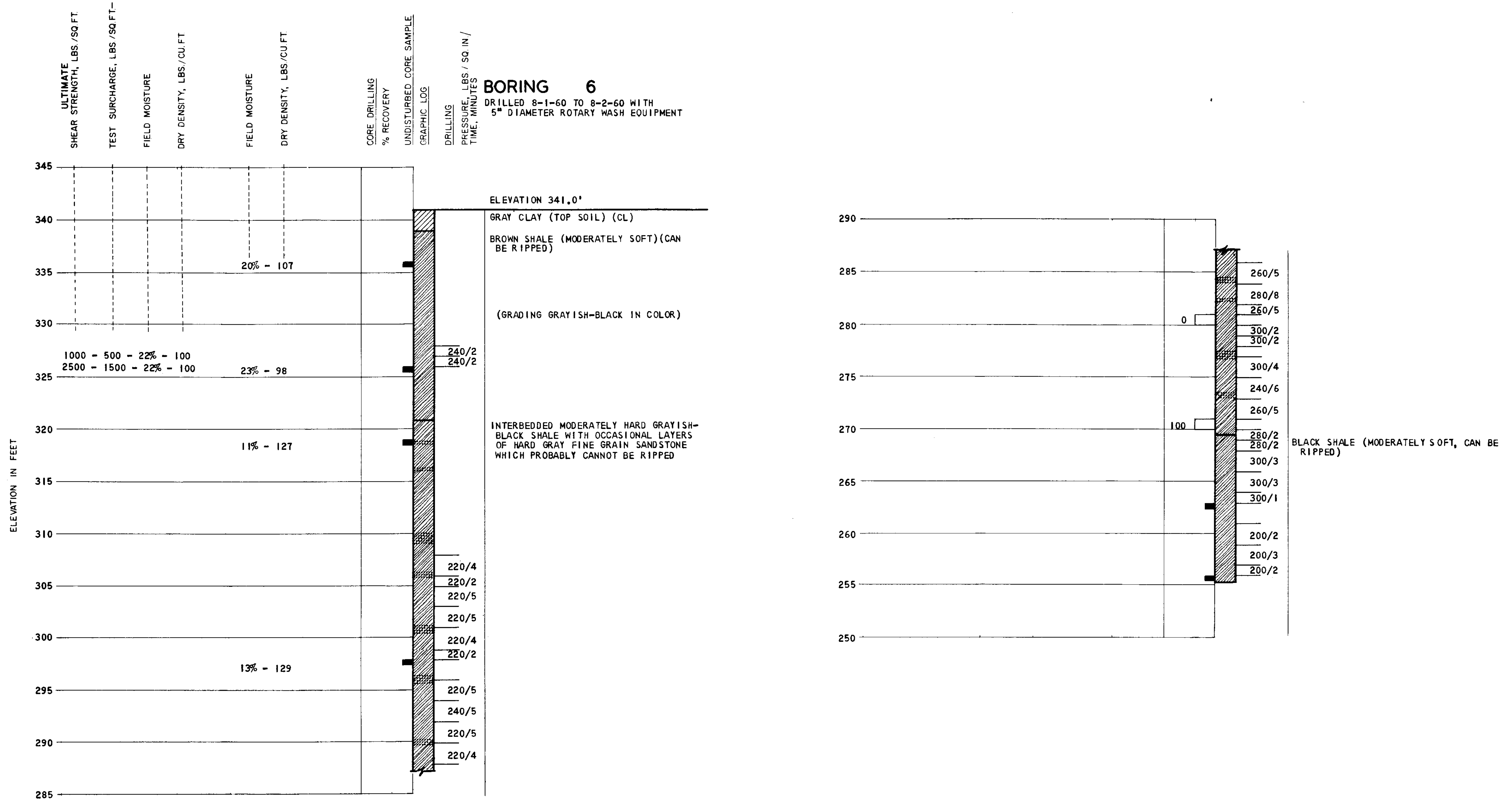
BY ______ DATE ______ CHECKED BY ______ DATE ______ PLATE ______ OF ______



LOG OF BORING

DAMES & MOORE

PLATE A1C



LOG OF BORING

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PLATE A10

651-BI

K. Johnson