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Introduction

This report presents the results of foundation investigations and ground response analyses of the existing and proposed State Capitol Building sites in downtown Sacramento.

The work reported herein was authorized by the Joint Rules Committee of the California Legislature on June 5, 1974, in a memorandum to Mr. Howard Ullrich, Director, Department of Transportation. The study was conducted in accordance with Interagency Agreement LCB 9928, dated May 1, 1974, between the Rules Committee and the Department.

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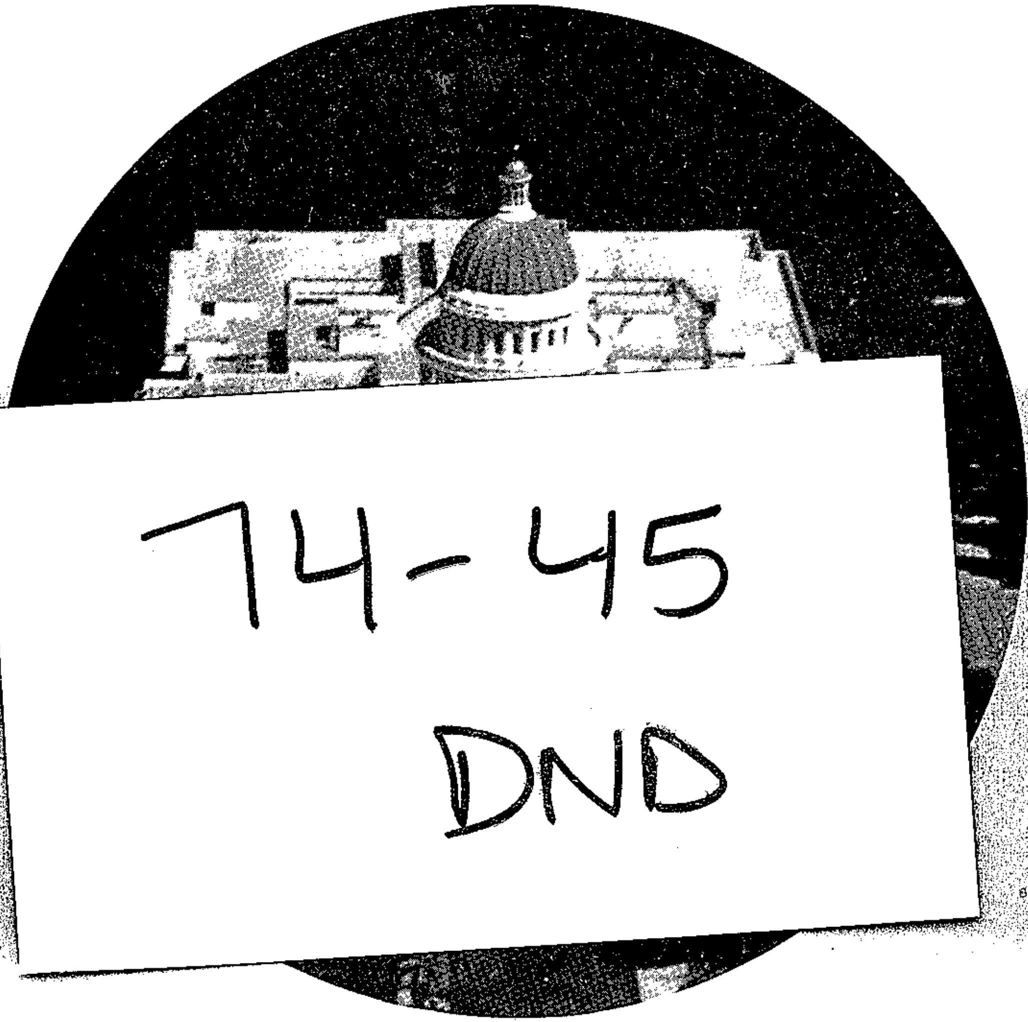

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# FOUNDATION AND SEISMIC INVESTIGATION FOR THE EXISTING AND PROPOSED STATE CAPITOL BUILDING SITES

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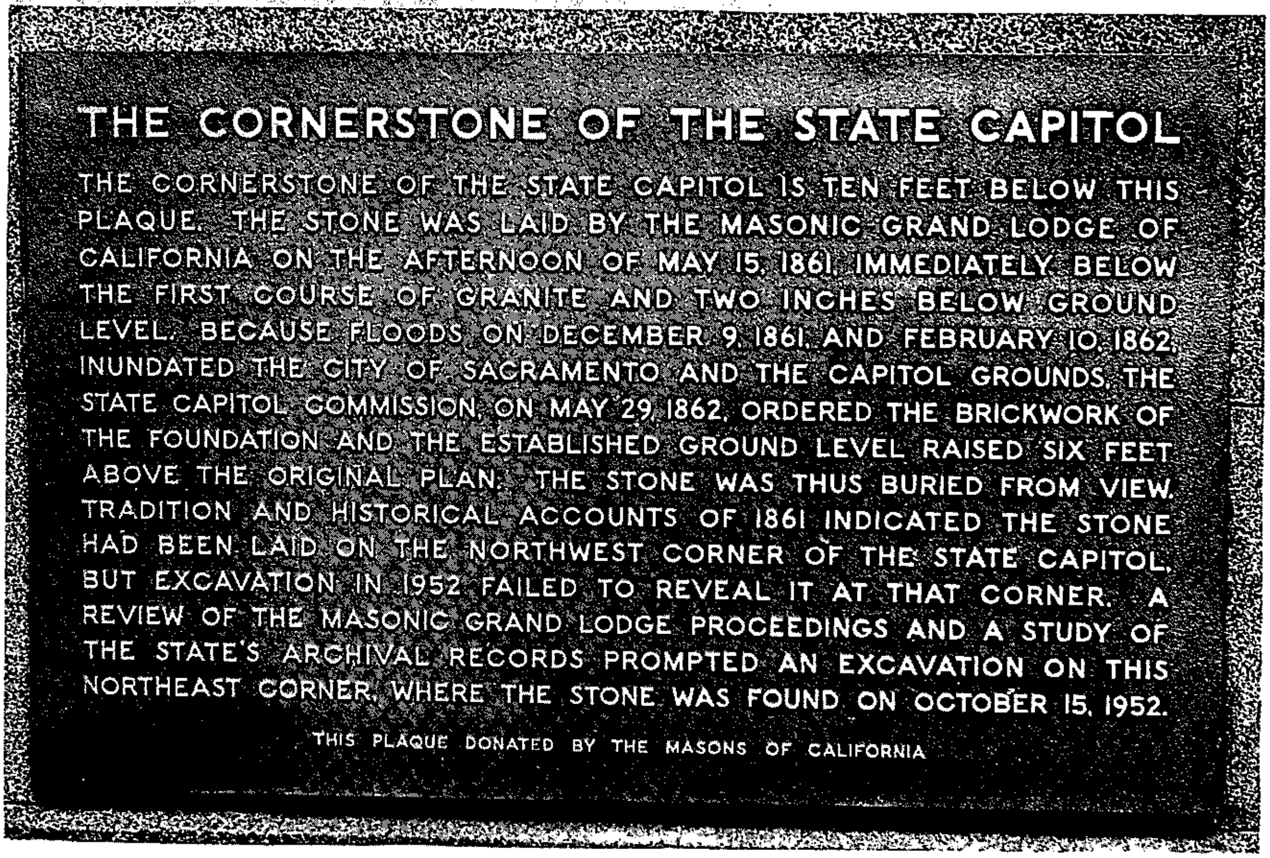
THE CORNERSTONE OF THE STATE CAPITOL
THE CORNERSTONE OF THE STATE CAPITOL IS TEN FEET BELOW THIS PLAQUE. THE STONE WAS LAID BY THE MASONIC GRAND LODGE OF CALIFORNIA ON THE AFTERNOON OF MAY 15, 1861, IMMEDIATELY BELOW THE FIRST COURSE OF GRANITE AND TWO INCHES BELOW GROUND LEVEL. BECAUSE FLOODS ON DECEMBER 9, 1861, AND FEBRUARY 10, 1862, INUNDATED THE CITY OF SACRAMENTO AND THE CAPITOL GROUNDS, THE STATE CAPITOL COMMISSION, ON MAY 29, 1862, ORDERED THE BRICKWORK OF THE FOUNDATION AND THE ESTABLISHED GROUND LEVEL RAISED SIX FEET ABOVE THE ORIGINAL PLAN. THE STONE WAS THUS BURIED FROM VIEW. TRADITION AND HISTORICAL ACCOUNTS OF 1861 INDICATED THE STONE HAD BEEN LAID ON THE NORTHWEST CORNER OF THE STATE CAPITOL, BUT EXCAVATION IN 1952 FAILED TO REVEAL IT AT THAT CORNER. A REVIEW OF THE MASONIC GRAND LODGE PROCEEDINGS AND A STUDY OF THE STATE'S ARCHIVAL RECORDS PROMPTED AN EXCAVATION ON THIS NORTHEAST CORNER, WHERE THE STONE WAS FOUND ON OCTOBER 15, 1952.
THIS PLAQUE DONATED BY THE MASONS OF CALIFORNIA

STATE OF CALIFORNIA—BUSINESS AND TRANSPORTATION AGENCY
RONALD REAGAN, Governor

DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
TRANSPORTATION LABORATORY
5900 FOLSOM BLVD., SACRAMENTO 95819

October 26, 1974

State Capitol Project
Interagency Agreement
No. LCB9928

Mr. Howard C. Ullrich
Director of Transportation

Dear Sir:

In accordance with the provisions of Interagency Agreement No. LCB9928 between the Department of Transportation and the Joint Rules Committee of the California Legislature, submitted herewith is the report:

FOUNDATION AND SEISMIC INVESTIGATION
FOR
THE EXISTING AND PROPOSED
STATE CAPITOL BUILDING SITES

Study made by . . . . . . . . . . . . . . . . . Foundation Section

Under general direction of . . . . . . . . . . Raymond A. Forsyth

Project Director . . . . . . . . . . . . . . . . . R. H. Prysock

Project Engineer . . . . . . . . . . . . . . . . Kenneth A. Jackura

Project Assistant . . . . . . . . . . . . . . . . Douglas J. Kuhl

irreg p. illus

Very truly yours,

ROBERT J. DATEL
State Highway Engineer

By

John L. Beaton
Chief Engineer, Transportation Laboratory

(Oct 26) 1974

1. Earthquakes and buildings
2. California -- Capitol and capitol

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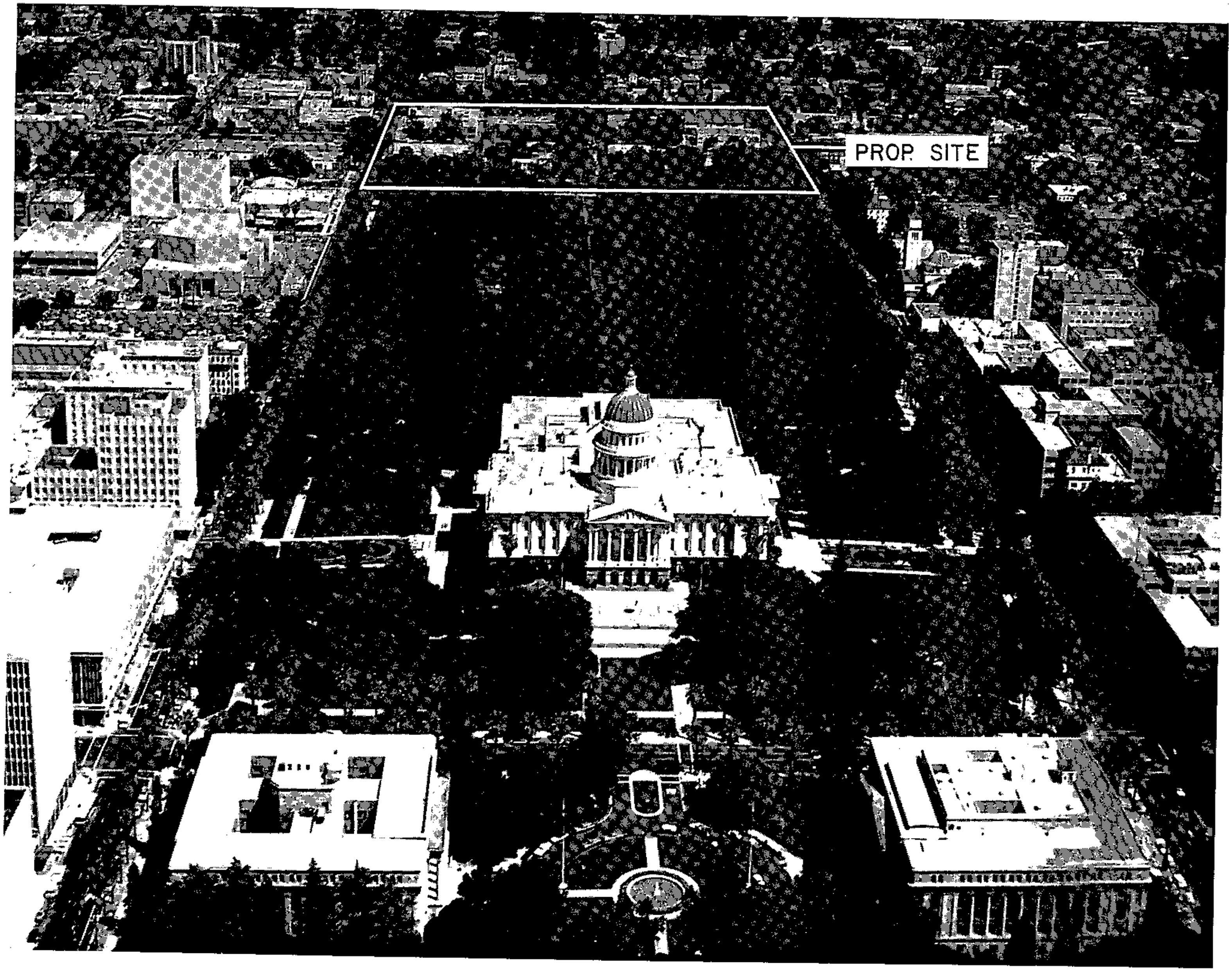
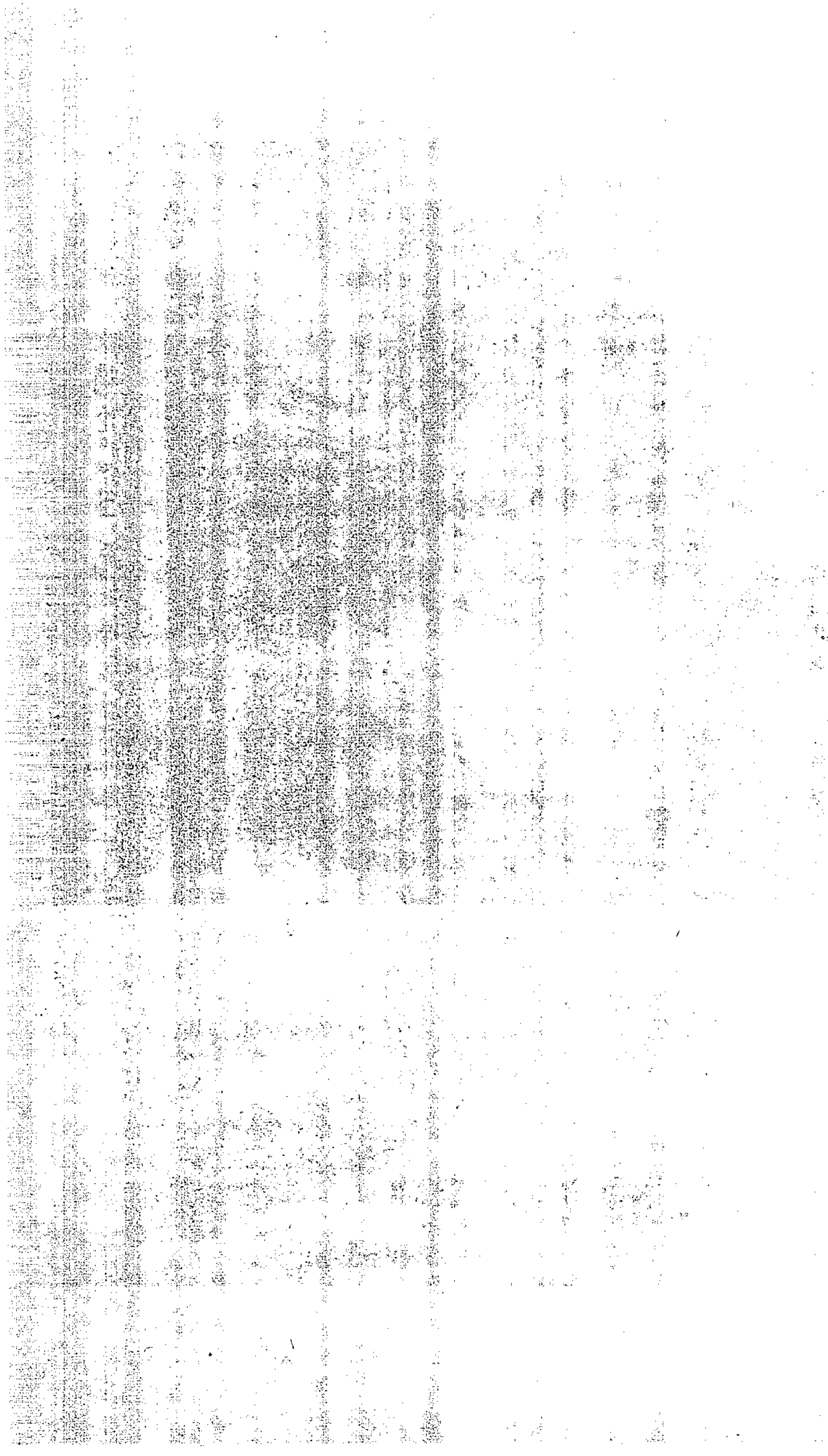



i Looking East at the Existing Capitol Building and the Proposed Site in Background



## ACKNOWLEDGEMENTS

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Appreciation is extended to the entire Transportation Laboratory's Foundation Section staff who, nearly to the man, participated in the accumulation of material and data and the preparation of information presented in this report. Other laboratory personnel who contributed time and energy to the project include members of the Concrete, Structural Materials, as well as the Drafting and Illustration and the Photography and Reproduction Units of the General Services Section. Individual recognition in their respective areas of expertise is afforded Elgar Stevens, geology and geophysical investigation; Chris Masklee, computer applications; Wilfred Yee, analysis of existing foundations; Bennett John, groundwater considerations; Ben Squires, settlement analysis; Ray Leech, field exploration; and Wes Gray, laboratory testing.

The following persons and their various organizations are mentioned as acknowledgement of their invaluable assistance in providing access to essential information concerning the existing Capitol structure: Capitol Building Manager Ira Billick and Assistant Manager Ron Neal and Capitol Civil Engineer Phil Lee; Karl Stricker and Fred Zancai of the Office of Architecture and

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Construction, Dr. W. N. Davis, Jr., Chief Archivist, and his staff of the State Archives, and Eugene Leyval, Director of Assembly Space.

Finally, we wish to gratefully acknowledge the efforts in consulting roles of Drs. Carl Rhomsted and C. K. Shen of the University of California at Davis in the area of soil foundation analysis and dynamic soil and structural response. Special appreciation is extended to Dr. Joseph Paduana of the California State University at Sacramento, for his consultation in the area of existing Capitol foundation evaluation. Dr. Paduana also painstakingly searched a multitude of old records related to original foundation design and construction of the existing building and prepared Appendices C and D.

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## TABLE OF CONTENTS

## TABLE OF CONTENTS

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# INTRODUCTION

This report presents the results of foundation investigations and ground response analyses of the existing and proposed State Capitol Building sites in downtown Sacramento.

The work reported herein was authorized by the Joint Rules Committee of the California Legislature on June 5, 1974, in a memorandum to Mr. Howard Ullrich, Director, Department of Transportation. The study was conducted in accordance with Interagency Agreement LCB 9928, dated May 1, 1974, between the Rules Committee and the Department.

## Construction Concepts

A combination of new and rehabilitative construction involving the State Capitol Building is presently being considered by the California Legislature. The proposed construction consists of two alternative concepts which, for purposes of this report, may be described as follows:

Concept 1

Construct a new building at the four square block proposed site bounded by 15th, 17th, L, and N streets directly east of Capitol Park, <u>or</u>, construct an additional wing to the existing building. Also, the West Wing of the existing building would be strengthened to the extent necessary for its intended use.

Concept 2

Completely dismantle and reconstruct to modern standards the West Wing of the existing building. In addition, a new wing <u>possibly</u> would be added to provide additional space requirements.

As used in this report, the term "West Wing" refers to the original Capitol structure which was built in the 1860's. The term "East Wing" refers to the relatively newer structure (sometimes referred to as the Annex) which was completed in 1952 to meet additional space requirements. The term "existing Capitol Building" is used herein to include both the East and West Wings. The above terminology is consistent with general usage and is defined in the Office of Architecture and Construction report "Seismic Study; West Wing, California State Capitol" dated June, 1972.

Construction features at the proposed site probably would include underground parking under the building itself as well as the remainder of the site. Also, that portion of the site not occupied by the building would be made into a mall or small plaza at or very near present ground elevation. This would mean that State Route 160, comprised of 15th and 16th Streets, would require reconstruction to lower the grades into depressed sections thereby allowing traffic to pass under the site. An alternate to the depressing of the two streets would be to relocate Route 160 and close 15th and 16th Streets at L and N Streets.

A new wing at the existing site would either extend to the east from the present East Wing, or extend around the north, east, and south sides of the East Wing. For either alternate involving a new wing, parking under the East Wing would be expanded to include the area under the new wing. Foundation problems of undue difficulty would not be expected although special attention should be directed to tunnels in the event the West Wing is vacated by working personnel thereby requiring relocation or extensions of subsurface works. Also, the effect of new construction on the foundation system of the existing East Wing would require consideration.

Complete reconstruction of the West Wing would be essentially new construction from a foundation viewpoint. However, the effects on the adjacent East Wing foundation of removing the present load and then applying a new construction load would have to be evaluated.

Strengthening the West Wing for some form of limited use would include the foundation system to the extent necessary to realize balanced objectives between superstructure and substructure. This would mean that if the building itself is strengthened to withstand specific earthquake loadings, the foundation system should be capable of resisting the same forces.

## Purpose of Investigation

The purpose of this study was to analyze foundation soil and groundwater conditions at the two sites to provide information for use in planning and preliminary design of new and rehabilitative construction associated with either of the above two concepts.

## Scope of Investigation

The scope of the investigation was based on the above construction concepts and related potential problems involving foundation soils and groundwater. Major elements of the study were:

1. A seismicity study of the Sacramento area to determine the most appropriate earthquakes for use in ground response analyses.

2. Bearing capacity analyses, considering earthquake loading, of the West and East Wings of the existing Capitol Building.

3. Ground response analyses for the existing and proposed sites to develop response spectra for design use.

4. An evaluation of soil liquefaction potential and ground settlement due to vibratory densification.

5. A groundwater analysis to estimate construction dewatering requirements and to assess the companion problem of ground subsidence immediately outside the construction limits.

# SUMMARY AND RECOMMENDATIONS

This section summarizes findings from the study and presents recommendations concerning foundations most suitable for the construction previously described as Concepts 1 and 2. Special problems or major considerations associated with contemplated construction are given appropriate discussion. It should be emphasized that this investigation was intended to provide information for planning and preliminary design. Consequently, a detailed design effort directed toward a particular structure with specific dimensions and loadings will require further soil investigation, planned and executed in accordance with in-depth needs.

## PROPOSED SITE

### Subsurface Soil Conditions

Borings at the proposed site show that the upper 5 to 10 feet of soil is a loosely compacted fill material consisting of a mixture of sand, silt, and in some areas, varying amounts of rock and brick rubble. In its present state, this material is unsuitable as foundation material due to its moderate compressibility and apparent low to moderate compressive strengths. Underlying this material is a 20 to 30-foot thick stratum of silty clay of alluvial origin. The deposit exhibits low to medium compressibility. Unconsolidated-undrained compressive strengths of samples from above the water table generally ranged from 2 to 4 TSF, whereas samples from immediately below the water table yielded undrained compressive strengths of 0.4 to 1 TSF. Below the water table, undrained compressive strength values were somewhat scattered but did appear to increase with depth. The engineering properties of this stratum indicate that it is unsuitable in its present state for founding any major structure of moderate to heavy loads.

Underlying the silty clay is a 16 to 23-foot thick stream bed deposit of sand, gravel, and cobbles. Due to the inability to retrieve samples of this deposit during field exploration, its engineering properties can only be estimated. Based on the excellent performance of two 18-story state office structures founded on a similar gravel stratum at 8th and 9th Streets, it would appear that the material behaves as a medium dense to dense sandy gravel with low compressibility and high bearing strength. It is suggested, however, that further field investigation of this particular stratum be undertaken before a high degree of competence is placed in its total ability as a foundation material. The remaining layers of soil lying below the sand and gravel consist of various mixtures of sand, silt, and clay. Standard penetration blow counts ranging from 40 to more than 100 indicated relative densities in excess of 80% below a depth of 50 feet.

## Site Groundwater and Dewatering Considerations

Water table fluctuations at this site vary seasonally with river stage. During the monitored period (1968-1972) groundwater levels varied from Elevation +4 to -5, corresponding to depths of 14.5 and 23.5 feet, respectively, below the ground surface.

As a consequence of the high water level, a pump-down test was conducted to determine permeability characteristics of the sandy gravel stratum discussed earlier, and the water table drawdown characteristics. Results indicated a permeability coefficient of 700 feet/day with rapid recharge potential. Permeability of the soils under the overlying and sand and gravel is estimated at 0.01 to 0.1 feet/day.

Pumping from the gravel stratum will create an essentially flat drawdown curve which means the excessive settlement of the outlying areas during periods of pumping is likely. It is therefore concluded that fairly long sheet piling, penetrating to approximately 60-foot depths, should be used during the excavation phase of construction. Dewatering by pump wells located inside the area enclosed by sheet piling is recommended. Drawdown outside the sheeting would then be negligible with respect to the settlement problem, and groundwater recharging would not be necessary. Permanent sheet piling will not replace the need for permanent pumping to maintain dry conditions below the water table. Therefore, subsurface structures should be thoroughly waterproofed.

## Earthquake Considerations

Ground response at the proposed site was evaluated using two postulated design earthquakes. These design earthquakes represent maximum credible events of magnitudes 7 and 8+ associated with the Midland and San Andreas fault systems, respectively. The Midland fault is about 24 miles from the site whereas the San Andreas fault system is about 80 miles distant. The magnitude and distance of each earthquake was established based on available geologic features and historical data.

Results of ground motion analyses indicate the maximum acceleration in rock-like material underlying the proposed site to be 0.18g. At ground surface, the maximum acceleration response due to such bedrock motion was computed as 0.23g. Structural design response spectra associated with each design earthquake and three values of structural damping are presented in Appendix H. Due to potential embedment of a portion of the proposed structure, a response spectrum was developed for a depth of 28 feet below ground surface. Comparison of results indicated no significant difference in response spectra at the two levels. A generalized maximum value for vertical acceleration is 0.67 times the maximum horizontal acceleration. Therefore, a maximum vertical acceleration at ground surface of about 0.13g should be anticipated for the design earthquakes used in this investigation.

Soil-structure interaction during an earthquake was researched and the available data indicated that the magnitude of uncertainties associated with source mechanism, travel path geology, and analytical treatment of soil layering and wave propagation is probably much greater than potential mechanical interaction effects (i.e., effect of deformability of supporting medium on structural response). There are some data in the literature which indicate that low structures of massive weight with varying depths of embedment (nuclear power plants and pads for space vehicles or heavy mechanical equipment) are most sensitive to mechanical interaction.

Laboratory cyclic loading tests of undisturbed soil samples were conducted to determine liquefaction potential under design earthquake loading conditions. Samples taken from depths of 15, 23, 46, and 55 feet were tested. Detailed information concerning the tests and analysis is presented in Appendix I. From analysis of the test results, it was concluded that liquefaction was unlikely during the design seismic events. It was further concluded that transient excess pore pressures developed during dynamic loading would not be sufficiently high to result in foundation soil bearing failures.

Potential settlement due to seismic densification of the fill layer immediately below ground surface and the gravel stratum underlying the proposed site was estimated by considering design earthquake induced shear strains. It was concluded that seismic densification of this material will be negligible.

## Foundation Recommendations

Depending on the structure type and intended use, structural embedment depths will vary. Minimum excavation depths of 25 feet are recommended to bypass unsuitable material. If 25 to 30 feet of excavation is acceptable, then a floating type of foundation would be entirely adequate for supporting major loads. Since it was not possible to obtain samples for testing, the bearing capacity of the sand and gravel stratum can only be estimated. However, this stratum can conservatively be assumed to possess a minimum effective angle of shearing resistance of 35° ($c = 0$) with a more realistic value being between 40° and 45°. Relative densities are estimated to be a minimum of 50%; hence, with overburden removal and the reloading stress of the structure, a total settlement of 1/2 to 1 inch can be expected based on the performance of the 8th and 9th Street structures. Time-settlement characteristics recorded for these two 18-story office structures showed a maximum settlement over a 2-year period of less than 1 inch. Embedment depth for these two structures was approximately 30 feet.

## Construction Considerations: Fifteenth and Sixteenth Streets

Consideration is being given to reconstruction of those portions of 15th and 16th Streets, within the site bounds, as depressed sections. This would require excavation to a depth of 25 to 30 feet below present ground surface (Elevation +18±) and would place the bottom of the excavation at the top of the highly pervious sand and gravel stratum (Elevation -10±). Construction dewatering problems would be very similar to those already described for the proposed building. Again, to minimize ground subsidence and volume of water to be pumped, sheet piling should be used to fully penetrate the aquifer and substantially reduce the inflow of water into the area excavated.

Due to the very pervious nature of the sand and gravel stratum, it is recommended that the depressed sections be designed as "boat sections", either as full gravity or a combination gravity and hold-down piles to resist the buoyancy or uplifting force of the water. A boat section would eliminate the need for a permanent subdrainage system to collect and transport relatively large volumes of water to a pumping station and subsequent disposal. The success with the Interstate 5 depressed section along the Sacramento River and the lack of maintenance problems often associated with permanent subdrainage systems further support a boat section as a preferred design.

A cursory review was made of the possibility of relocating State Route 160 as an alternative to depressing 15th and 16th Streets. While such a relocation may be feasible, it would appear to be difficult due to established traffic flow patterns associated with the 15th - 16th Street/I-80 Interchange.

## EXISTING SITE

### Subsurface Soil Conditions

Boring records show fill material to extend from 5 to 10 feet below ground surface. This material is similar to that found at the proposed site; a sand-silt mixture interspersed with some rock and brick rubble. It is moderately compressible and in a loosely compacted state.

Underlying the fill material is a clayey silt, approximately 15-30 feet or more in thickness. This material exhibits low to medium compressibility characteristics. Unconsolidated-undrained compressive strengths are similar to those determined for the proposed site; 2-4 TSF above the water table and 0.4 - 1.0 TSF just below the water table. Below the water table, undrained compressive strengths were scattered but appeared to increase with depth as was noted for the proposed site. Consolidated-undrained triaxial compression tests indicate representative values of angle of shearing resistance, Ø, and unit cohesion, c, of 24 degrees and 1000 PSF, respectively, based on total stresses. A sand-gravel stratum extends from 8 to 17 feet beneath the clayey silt followed by layers of clayey sand, sands, and silt mixtures. A competent sand-gravel layer was encountered at a depth of about 100 feet in one boring.

## Site Groundwater and Dewatering Considerations

Water level during the July exploratory drilling period stood at Elevation +2 (20 feet below ground surface). Estimated maximum fluctuations in groundwater level are from Elevation +8 to -1 (16 to 25 feet below ground surface).

No pump-down tests were conducted in this area. However, the gravel stratum encountered at the 25-foot depth is estimated to have permeability values smaller than those recorded at the proposed site. Permeability estimates range from 10 to 100 feet per day. As a consequence, any proposed excavation to or into this layer may require sheet piling for seepage control, and adequate bracing and possibly underpinning to prevent potential damage to the existing Capitol facilities. Lowering of the water table 10 feet from its July 1974 level is not considered a potential settlement hazard to grounds or buildings outside the existing Capitol site if pumping rates and depths are carefully controlled. Maximum ground surface settlement due to 10 feet of drawdown is an estimated 0.5 inches. Permeability for the upper 20 feet of material is estimated at 0.01 to 0.1 ft./day.

## Earthquake Considerations

Ground response at the existing site was evaluated in the same manner as for the proposed site with the exception that a response spectrum below ground surface was not obtained. Maximum bedrock acceleration was found to be 0.18g with a maximum value at ground surface of 0.21g. Structural design response spectra generated for both the design earthquakes using three values of structural damping are presented in Appendix H.

As with the proposed site, soil-structure interaction is not considered to be a problem.

Cyclic loading tests of undisturbed soil samples retrieved from depths of 15, 23, 46, and 55 feet were conducted. Appendix I lists the results. It is concluded from this investigation that liquefaction during seismic excitation is not likely to occur and seismic settlement within the more granular soil strata will be insignificant.

## Evaluation of East Wing Foundations

Detailed calculations were made to assess the static and seismic loading capabilities of the East and West Wing foundations. Input information for the analyses was obtained from the Office of Architecture and Construction and reviews of original foundation construction plans.

The East Wing is a steel framed building with reinforced concrete shear walls supported by a grid of reinforced concrete footings. Footings are dimensioned variably so as to equalize foundation soil pressures, and comprise about 48% of the gross building area. Maximum dead load plus live load soil pressures are estimated at 3.7 KSF (kips per square foot). Based on triaxial compression tests of undisturbed soil samples, the ultimate bearing capacities are approximately 5.6 to 6.1 KSF.

Even though the East Wing has thus far performed satisfactorily, differential settlements in excess of 1/2 inch are recorded, and the total settlement at the southeast corner is close to 1 inch. A large part of the total settlement occurred during construction. Total settlement at the west end of the East Wing is minor due to the preconsolidation effects of the existing West Wing, and a larger amount of fill that overlayed this area prior to construction of the East Wing.

The foundation material of the East Wing area is adequate to support the maximum predicted vertical and lateral loads. Based on the California Hospital Code requirements for seismic loads in the Sacramento area, the estimated minimum factor of safety against soil bearing capacity failure is above one for the most heavily stressed shear wall footing. Overall building stability against overturning moments and sliding forces due to seismic loading is satisfactory. The reinforced concrete footings are adequate for vertical and lateral loads based on a structural analysis of two heavily loaded shear wall footings. No recommendations for increasing structural strength capabilities are included since adequate collapse resistance is still present.

## Evaluation of West Wing Foundations

The West Wing consists of a network of unreinforced bearing brick walls supported on tapered concrete footings. Original construction plans specified footings to be embedded to depths of approximately 5 feet (Elevation +10.2). High flood stages during the 1860's necessitated fill material to be brought in to raise the ground level outside the structure to a height of 13 feet above the surface of the surrounding streets. Thus, ground elevation outside the wall is approximately +28 feet.

Two test pits were excavated within the basement of the old structure to verify footing dimensions and embedment depths and to evaluate soil, brick, and mortar conditions. Based on laboratory test results, the soil bearing capacity values range from 6.6 to 8.4 KSF for the footing sizes investigated. Under maximum design earthquake loading conditions (California Hospital Code) the estimated range of factors of safety against soil bearing failure varied from 0.7 to 0.8. For DL plus LL loading conditions, a factor of safety slightly above 1.0 was computed. Even allowing for sampling disturbance, which tends to underestimate the soil shear strength, the calculated factors of safety are still quite low. It should be noted that for purpose of analysis, it was necessary to assume (whether valid or invalid) that the superstructure is structurally capable of transmitting possible seismic forces to the foundation. The earthquake hazards that exist in the West Wing structure, however, are considered in a report of the Office of Architecture and Construction dated June, 1972.

Stability against overturning was found to be satisfactory. However, tension stresses, developing within the east - west corridor walls and between the brick wall and concrete footing would result in mortar and brick separation. According to the Unified Building Code (Section 2314): "all elements within structures located in Seismic Zones 2 and 3 which are of masonry or concrete shall be reinforced so as to qualify as reinforced masonry or concrete. . . ". Thus, the basement brick walls and concrete footings do not satisfy this code requirement.

The overall quality of materials used for construction of the foundation system was rather poor, compared to present standards, based on visual observation and laboratory test results. Two laboratory tests on concrete cores from footings showed compressive strengths of approximately 290 and 580 PSI. Although calculated maximum concrete compressive stresses are estimated to be about 110 PSI, these laboratory compressive strengths are considerably below the 1973 Unified Building Code minimum standard of 2,500 PSI. Test results of four individual brick specimens indicated compressive strengths ranging from 1,850 to 4,800 PSI. These tests indicate the bricks are of good quality and, in general, conform to the Unified Building Code minimum compressive strength requirement of 2,500 PSI. Test results of the brick assemblies showed the mortar between the bricks to be of poor quality. Shear testing of the joints indicated an ultimate shear stress value of 40 PSI under zero normal stress loading conditions. The Unified Building Code minimum allowable value for mortar is 20 PSI.

Investigation of the basement walls uncovered evidence of entire load-bearing cross walls removed during remodeling, and large openings cut into various load bearing walls for ducts and conduit. Some cracking was also observed. A crack in the northwest corner is attributed to differential settlement due to softer material underlying this section. Leveling nets conducted in July 1974 indicated no apparent tilt or sag in the West Wing with the exception of the north face which sagged downward to the west by slightly more than 1/2 inch. This observation is consistent with level readings taken in 1868, soon after construction of the support foundation.

From the evidence gathered during this investigation, it is concluded that the existing foundation of the West Wing does not meet the required seismic resistant capabilities of present earthquake standards. Therefore, a strengthening program for the West Wing (Concept 1) should include the foundation system to ensure compatible seismic resistant capabilities between superstructure and substructure. No specific recommendation for foundation strengthening are presented herein since substructure and superstructure should be considered together in developing an overall renovative plan.

## Foundation Recommendations

Soil conditions at this site are not entirely suitable for the use of spread footing or mat type of foundations for the support of large structures with moderate to heavy loads. Settlement records of the last 23 years show a maximum settlement of 1 inch for the East Wing despite an applied building load less than the excavated overburden. A rational explanation for this behavior is that soil rebound occurred during excavation followed by recompression during construction loading. This experience is useful for predicting probable soil behavior during reconstruction of the West Wing, where loads involved will be even greater. Thus, even though the soil strata under the West Wing will have been preloaded after dismantling, settlements can be expected if shallow foundations were to be used. Moreover, the much heavier West Wing with its high domes, irregular shape, and unusual structural features is more susceptible to damage by differential settlements than the more flexible frame structure of the East Wing.

It should also be noted that approximate estimates of total settlements that have occurred for the West Wing range from 6 to 12 inches, or more (Appendix J). In addition, recorded history during construction of the West Wing (Appendix C) shows that the building experienced serious differential settlements resulting in cracks in the brick walls. The consultants engaged to investigate this problem suggested that the structure should have been founded on piles reaching down to incompressible soil, so that there would be little or no settlement. It is noted that the West Wing construction period had lasted almost 12 years, thus permitting some time for adjustment of the masonry structure to differential settlements. Under modern-day methods of more rapid construction, the effects of differential settlement on the West Wing would have been even more serious. With regard to foundation selection for the East Wing, the designers would normally have used piles carried to the underlying gravels (Appendix D). However, they reached the conclusion that pile driving adjacent to the West Wing with its inadequate footings would have resulted in irreparable damage due to vibrations of pile driving. Finally, it is noted that except for some floating type of foundations, most major buildings in the downtown Capitol area are founded on piles.

On the basis of these experiences, it is recommended that new wing construction, or reconstruction of the West Wing if decided upon, utilize pile foundations founded on a suitable bearing stratum. All previous borings indicate that a sand and gravel stratum exists at a depth varying from about 20 to 30 feet below the base of existing foundations. The thickness of this stratum under the West Wing has not been determined, nor has it been adequately determined for any new wing construction. For this reason the pile foundation recommendation is contingent upon findings from additional exploratory borings that should be made prior to any construction. The borings should go deep enough to define the thickness of the sand and gravel stratum and the nature of the underlying material within the seat of settlement of any proposed construction. Special attention should be given to soil conditions below the northwest portion of the West Wing, where difficulties were encountered during original construction.

During design of pile foundations, consideration should be given to the need for partial predrilling before driving piles to minimize the effects of possible heave and vibrations on adjacent structures and to reduce the level of noise pollution during possible legislative sessions in the East Wing. Also, it is strongly recommended that pile bearing capacity be determined by controlled and carefully interpreted field pile load tests. Before commencing pile driving for any new wing, the West Wing should be adequately strengthened (Concept 1), or dismantled or in a state of reconstruction (Concept 2).

Another consideration related primarily to reconstruction is the effect on the East Wing of soil rebound resulting from dismantling the West Wing. Accurate predictions of soil rebound are difficult; but estimates based on consolidation rebound test data indicate possible rebound of up to 1.5 inches at the westernmost portion of the East Wing and practically no rebound at the easternmost portion. A thorough program of field instrumentation is recommended, therefore, to monitor possible movements during dismantling and rebuilding of the West Wing.

## Subsurface Structures

Extensions and/or relocations of existing underground structures, such as tunnels, will be subjected to potential periods of high water; thus, consideration must be given to complete waterproofing and buoyancy effects. Soil to the 20 foot-depth is considered competent enough that heaving or swelling due to seasonal changes will be of minor importance.

$K_o$ - tests conducted on undisturbed samples retrieved at the 15-foot depth indicated a coefficient of lateral earth pressure at rest of 0.4, but $K_o$ values may vary from 0.4 to 0.6. Determination of earth pressures for design should consider material variation, soil-structure interaction, and seismic effects.

# APPENDICES

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# APPENDIX A

## AREAL GEOLOGY AND GROUNDWATER

# APPENDIX A

## AREAL GEOLOGY AND GROUNDWATER

### Geology

Sacramento is located on the eastern side of the broad Sacramento Valley, in the northern half of the Great Valley of California. The valley floor, although seemingly flat, displays various topographic forms. In the vicinity of Sacramento there are river terraces and natural levees on the westward sloping alluvial plains, developed by the adjacent Sacramento and American Rivers.

Geologically, the Sacramento Valley is an elongate, northwest-trending trough filled with a thick sequence of marine and continental sediments, ranging in age from Cretaceous to Recent. Deposits on the eastern flanks of the trough are supported at depth by crystalline rock of the Sierra Nevada mountains. In the vicinity of Sacramento, the sedimentary sequence is many thousands of feet thick. The lower portions, having been indurated and consolidated, now exhibit characteristics of competent rock.

A stratigraphic sequence for the Sacramento area has been developed using local gas and water well logs (1). Sediments encountered are all relatively flat-lying and range from Recent River Deposits at the surface, through the Pleistocene Victor formation into the Plio-Pleistocene Laguna formation. The Laguna formation, encountered below 100 feet, is a extremely heterogeneous assemblage of silt, clay, sand, and minor lenses of gravel deposited on broad flood plains by meandering sluggish streams. Somewhat compact light gray to yellowish-brown clayey silts to silty fine sands are most abundant; clean, well sorted sand occurs chiefly in relatively thin zones. Gravel beds, which are scarce, are mostly poorly sorted and of low permeability.

The Victor formation ranges in thickness from 30 to 100 feet and is encountered at depths of 10 to 30 feet. It is also a heterogeneous assemblage of silt, sand and gravel, and clay, transported by shifting streams. The area south of the American River generally has a layer of "clay and gravel" at a depth of 30 to 60 feet that is part of this formation. The Victor formation appears to be in hydraulic continuity with the Recent River Deposits. The River Deposits are 10 to 30 feet thick, ranging in size from boulders and cobbles to silt and clay. They generally occur as geologically unconsolidated, clean, well-sorted sand and gravel. Lenses of silt and clay are common. These upper formations are considered good groundwater aquifers and have a rapid recharge rate. Groundwater level fluctuates about a depth of 12 to 18 feet.

Only two minor faults have been recognized near Sacramento (2). Both are considered to be of Quaternary age without historical displacement. One is located 15 miles northeast, the other 20 miles northwest of Sacramento. They are both about 10 miles long and trend north-northwest. No earthquake epicenters have been reported for the immediate Sacramento area. The nearest are in Solano County, twenty miles west, with a magnitude of 6.0 to 6.9. Others are thirty miles west to northwest in Napa and Yolo counties, with magnitudes of 4.0 to 4.9.

## Groundwater Records at Proposed Site

Figure A1 illustrates monthly water level fluctuations between the Sacramento River and a well located at 15th and N Streets (proposed site) for a period of approximately 3-1/2 years commencing September 1968. Pumping along Front Street adjacent to the Sacramento River during the I-5 construction was conducted periodically during the first six months of the illustrated period. However, water level fluctuations at the 15th Street location were minor.

The 3-1/2-year recorded water history occurred during the periods of some of Sacramento's wettest years, hence, is a good indicator of maximum water level fluctuations near peak and ebb river stages. Results indicate maximum water table variations of approximately 10 feet. Damping effects due to distance and soil conditions results in the lag period noted between rapid rising peak river stages and groundwater levels. However, slow rises and ebbs in river stage are followed quite closely by groundwater levels.

The peak water levels during the indicated periods show maximum elevations of approximately +5 feet, with lows of about -4 feet elevation. Long term transient conditions always showed a minimum of 3 to 5 foot river head. During drilling operations in mid-July, 1974, water levels near 15th and N Streets were at an elevation of +5 feet. River stage during and just preceding this period was at Elevation +6 feet, thus, water levels were commensurate with river stage and head. It is not known at this time whether groundwater pumping was taking place in the vicinity of 15th and N Streets during the recorded period. Consequently, no explanation for head variations recorded then, but not observed during this investigation, was found.

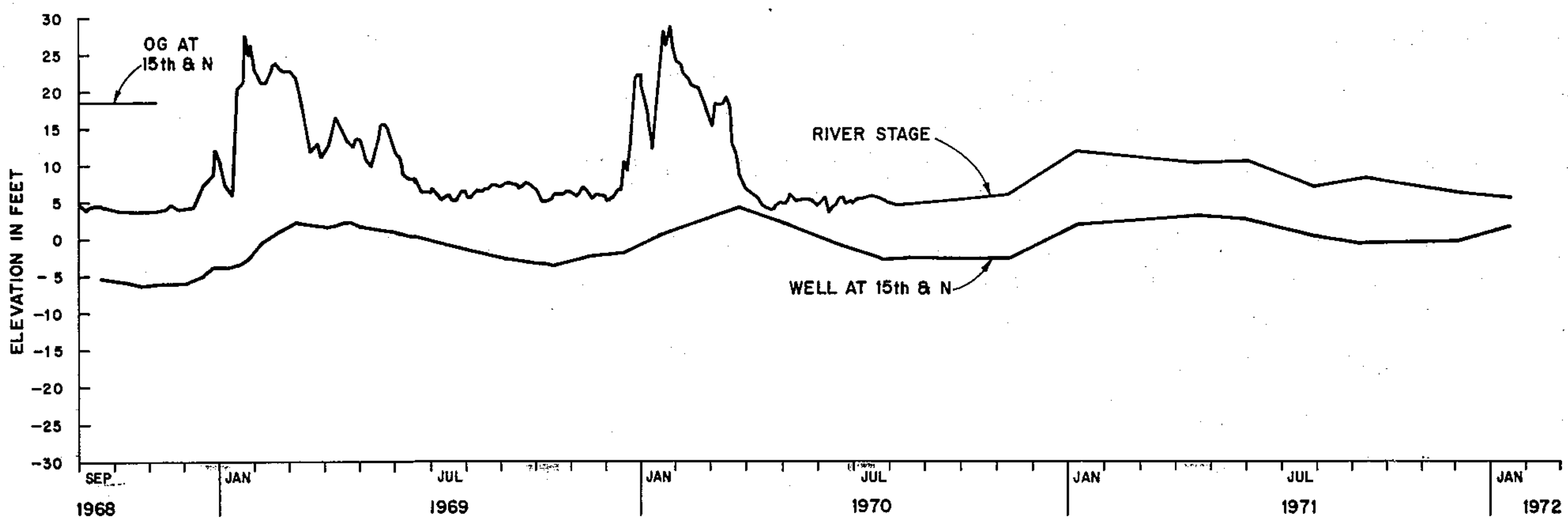


FIG. A1 MONTHLY WATER LEVEL FLUCTUATIONS BETWEEN SACRAMENTO RIVER AND WELL AT 15TH AND "N" (PROPOSED SITE)

## References

A1. Olmsted, F. H., and Davis, G. H., 1961, Geologic Features and Groundwater Storage Capacity of the Sacramento Valley, Calif., Geological Survey Water-Supply Paper 1497.

A2. Olmsted, F. H., and Davis G. H., 1972, Provisional Fault Map of California, California Division of Mines and Geology, Seismic Safety Information Sheets 72-1, 72-3.

# APPENDIX B

## SITE SEISMICITY

# APPENDIX B

## SITE SEISMICITY

The seismicity of a particular location within a given area can be considered as the relationship between the number of earthquake occurrences whose epicenters lie within the area, their magnitudes, and the severity of the ground response at the selected location. Seismic events occur randomly with time and, in the case of large magnitude earthquakes, may be experienced only a very few times over a period of hundreds of years. Consequently, evaluation of seismicity relies entirely on past earthquake records. Unfortunately, historical earthquake records are far from complete. Only about 40 years of actual recorded ground motions exist and perhaps 200 years of semi-reliable verbal accounts are available. Nevertheless, a substantial amount of information regarding the seismic vulnerability of a given area can be gleaned from the analysis of this information.

### Earthquake Ground Motion

Within the continental United States and adjacent parts of Mexico enough data have been collected to show a positive correlation between the length of surface fault rupture and earthquake magnitude, Figure B1, (Bonilla (1)). In addition, Schnabel and Seed (2) have developed curves relating maximum acceleration in rock and predominant period to distance from causative fault for given earthquake magnitudes, Figures B2 and B3.

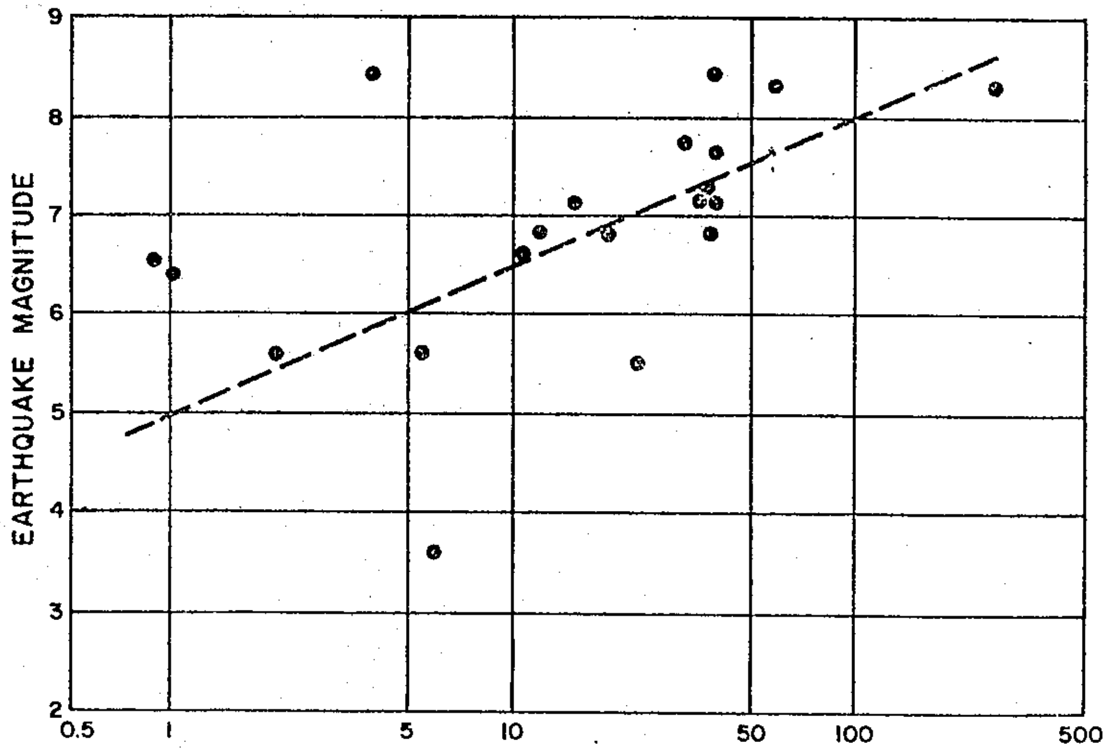


Fig.B1 LENGTH OF SURFACE RUPTURE, MAIN FAULT - miles (2)

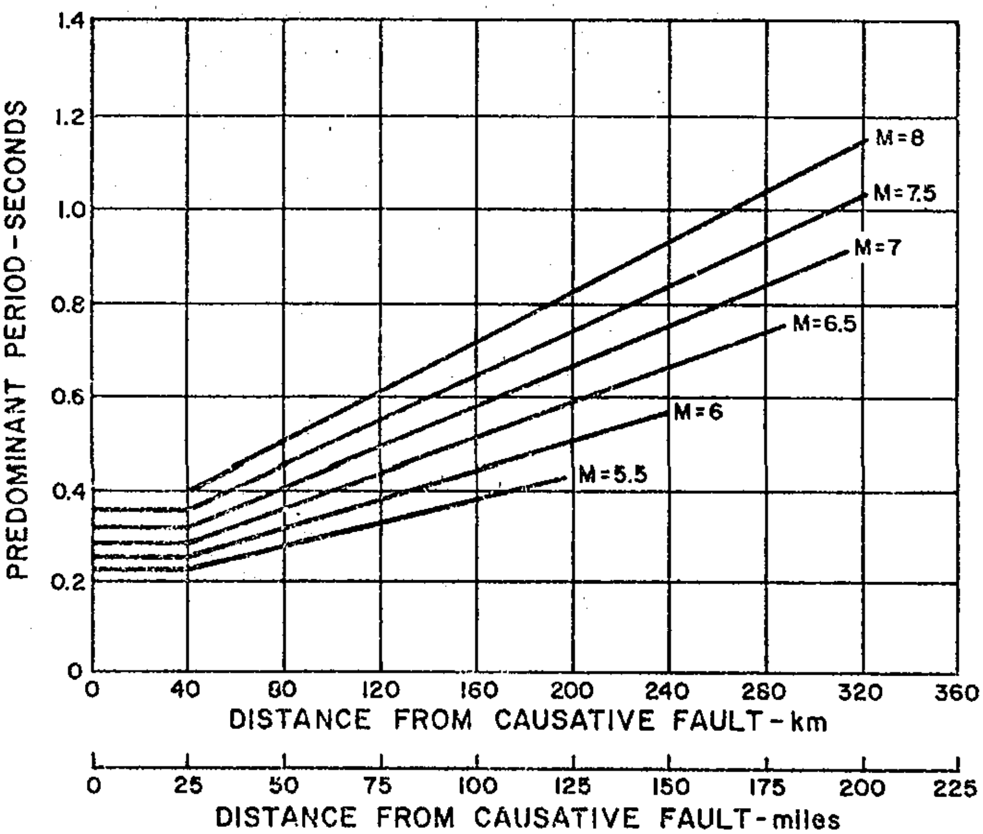


Fig.B2 PREDOMINANT PERIODS FOR MAXIMUM ACCELERATIONS IN ROCK (2)

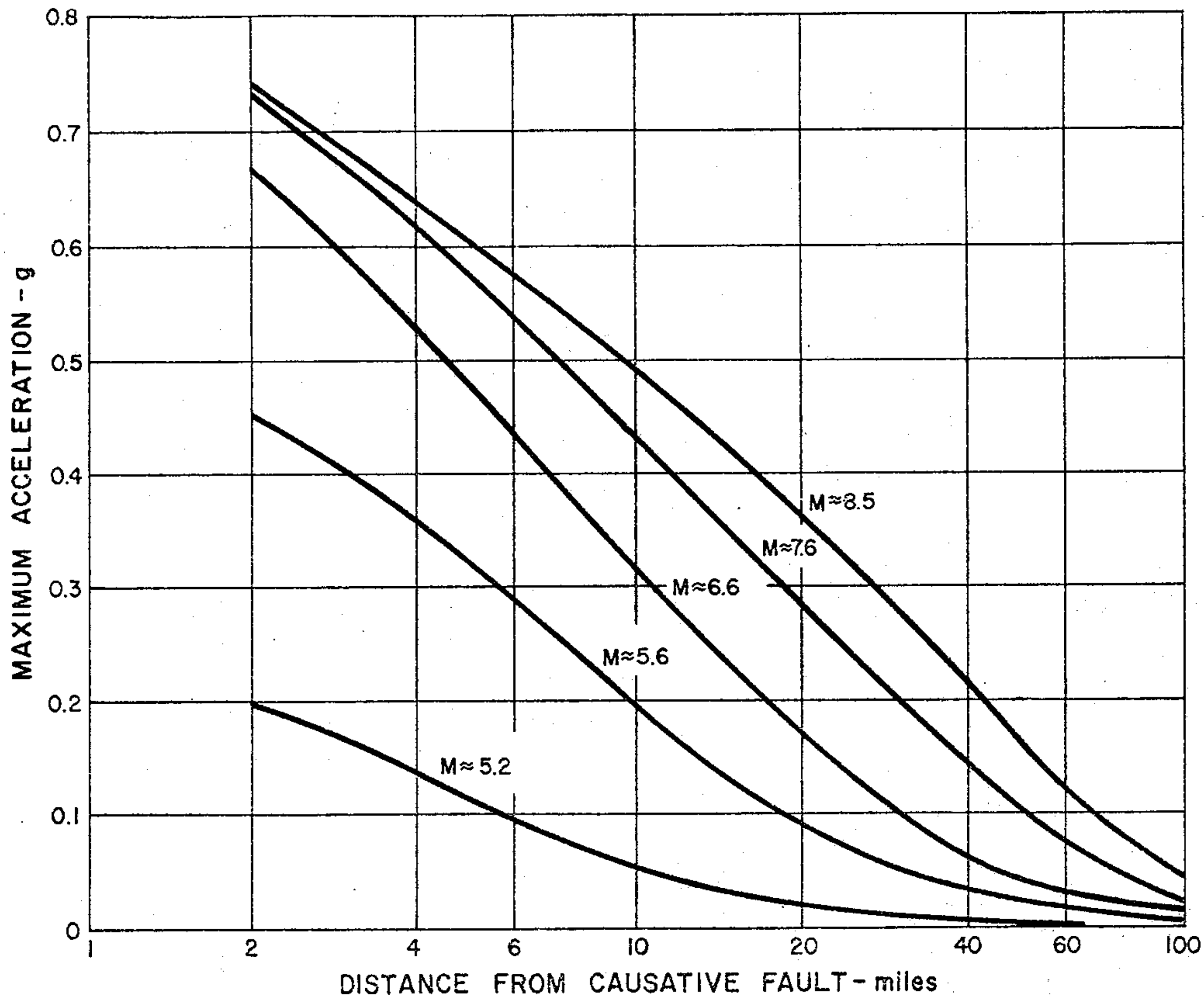


Fig. B3 AVERAGE VALUES OF MAXIMUM ACCELERATIONS IN ROCK (2)

Utilizing these relationships, it is possible to approximate parameters characterizing bedrock motion at any location resulting from a given earthquake.

## Method of Analysis

Two approaches to the problem of determining the seismicity of a given project site have enjoyed considerable popularity in recent years. The first, and certainly the more rigorous, involves an indepth statistical analysis of the historical data pertaining to a selected study area within which the project site is located. This method assumes that:

1) Historical seismic activity is distributed somewhat uniformly over the study area.

2) The time period over which the historical data apply provides a representative number of events of each magnitude.

3) The study area includes all fault systems which might have a significant effect on the project site.

The reliability of this method, therefore, depends on the degree of validity of the above assumptions. The second approach, more simplified in nature, often yields results more valid than the rigorous approach. It consists of individual evaluation of critical fault systems which historically have produced significant ground motion at the project site.

Locating the existing and proposed sites on the earthquake epicenters map prepared by Greensfelder (3), Figure B4, it is at once apparent that the surrounding area has exhibited minimal

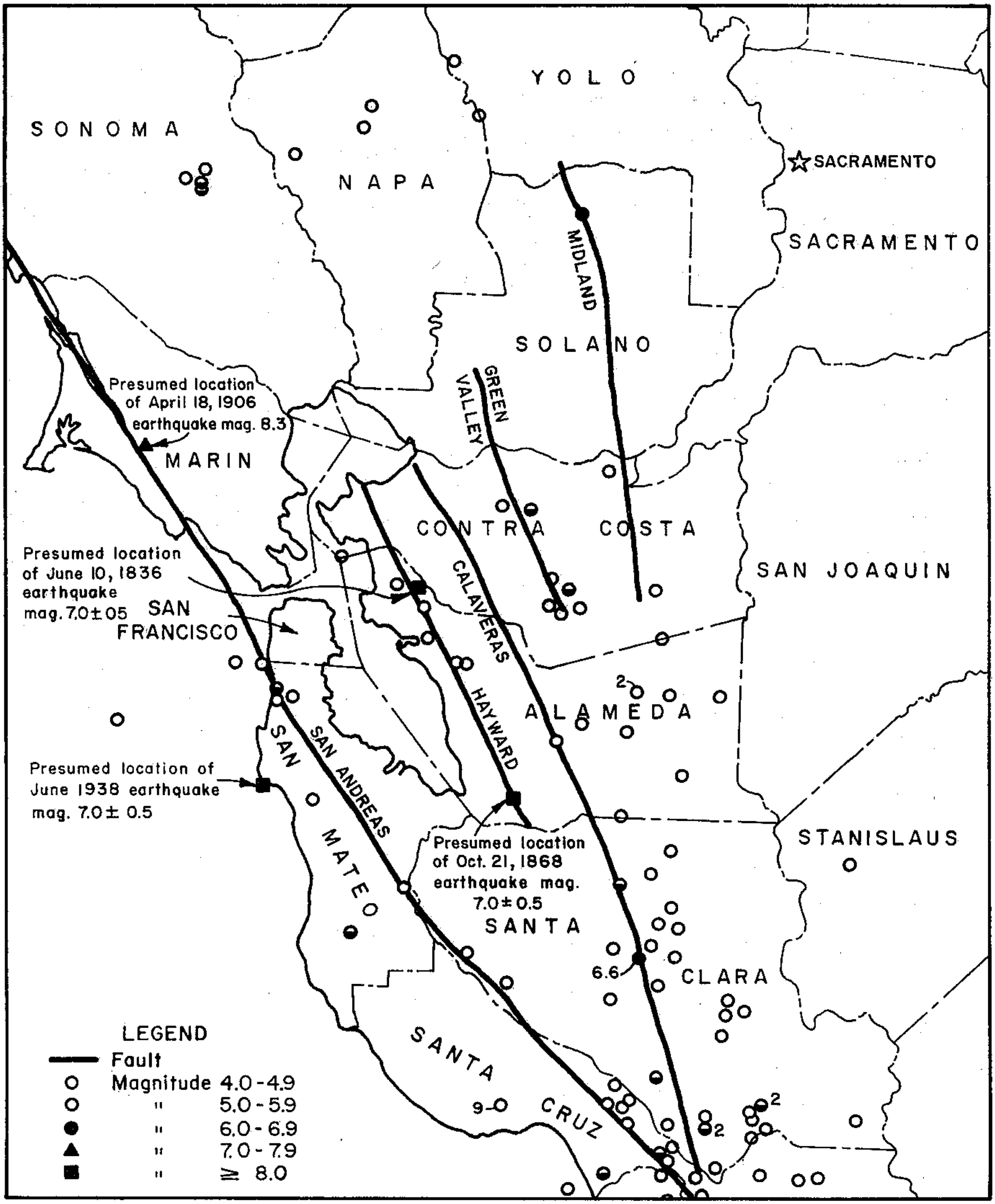


Fig.B4 PORTION OF EPICENTER MAP BY GREENSFELDER WITH A NUMBER OF SIGNIFICANT FAULTS SHOWN. (3)

seismic activity compared to other areas within the State. To the east and west, however, at a distance of approximately 80 to 90 miles, lie areas with extremely active histories. In order to apply the statistical seismicity approach to the site, inclusion of these active fault zones within the study area is essential. This, however, creates a situation where seismic activity throughout the study area is far from uniform. For this reason, the second approach, concentrating on critical fault systems, was selected for this study.

## Seismic History of the Sacramento Area

Probably the most intense earthquake ground shaking experienced in the Sacramento area in the recent past occurred on April 21, 1892. This was the result of a rupture along the Midland fault between Winters and Vacaville. The magnitude of the event has been estimated to be in the low sixes. Although no record of the actual ground motion in the vicinity of Sacramento exists, publications of the period have documented the visible effects.

The following is a quotation from Edward S. Holden (4):

> "April 21, 1892, Sacramento. There was another severe earthquake shock at 9:45 o'clock this morning, lasting twenty seconds. Buildings got a lively shaking and plastering fell from many ceilings.
>
> Several old chimneys toppled over and much glassware was broken in the crockery stores. The State capitol building suffered. A large portion of one of the plaster statues over the portico, 150 feet from the ground, fell and struck 40 feet from the building. The gigantic building trembled violently and the occupants in the State offices were badly frightened, and there was general exodus of clerks.
>
> It was discovered that a crack was made in the ceiling, extending from one end of the building to the other and going through the office of the superintendent of

public instruction into the assembly chamber. The beautiful ceiling of the latter, which is formed of stucco work, tipped with gold, was rent in places, as were also the Corinthian columns supporting the gallery. Books were thrown from the shelves and general disorder reigned.

The public schools were dismissed. All the pupils got out without creating a panic. At the city prison the greatest excitement prevailed. A number of police officers rushed into the street, expecting the collapse of the old building. Jailer John McManus stood at his post, with key in hand, prepared to turn loose the prisoners in case the building showed signs of collapsing."

Another account was reported in Woodland Daily Democrat of April 21, 1892:

"The first news came from Sacramento. The Deputy State Librarian reported that the State Capitol rocked wildly and the inmates were panic stricken. Several of the large statuary on the top of the building were thrown to the ground with such force that they were buried in the ground. The books in the State Library were thrown from the shelving and the floors literally covered with debris. (The State Library at that time was in the State Capitol building.)

All the elegant decorations in the Assembly Chamber are ruined. Nearly every building on J and K Streets show the effect of the shock. Chimneys were cracked and in many instances thrown down and they crushed through the sidewalks. The Sutter Club and the Metropolitan and Clunie buildings have been deserted for two nights. The courthouse on Second and I Streets was damaged to the extent of $3,000."

Although records indicate this to be the most severe ground shaking, other important earthquakes have induced similar effects in the City of Sacramento. Table B1 lists the noteworthy events along with selected commments taken from various publications. Roman numerals indicate probable Modified Mercalli intensities. A majority of these data was accumulated and presented in a report by Perry Byerly to the Sacramento Municipal Utility District (6).

Table B1 - Chronological Listing of Selected Seismic Events Relevant to Sacramento

1850, August 4.
"Smart shocks in Sacramento." Reference calls it V.

1857, January 9. (The great Fort Tejon earthquake)
"Very severe at Sacramento."
"San Francisco Bulletin", January 9, 1857: "By Magnetic Telegraph: Earthquake felt in Sacramento: A smart shock of earthquake was experienced here this morning at half past seven o'clock. No damage was caused."

1864, May 20.
"Very severe at Sacramento."

1868, October 21. (The Hayward earthquake)
V in Sacramento where plaster was cracked. (Water in river receded, shoaling ships, then came back with a rush.)

1869, December 27. (Origin in Nevada, near Virginia City)
Ref. says no damage in Sacramento.
The "Sacramento Union" for this date says "Door bells were rung and chandeliers and everything else that could do so swing to and fro." No damage mentioned.

1872, March 25. (The great Owens Valley earthquake)
Sacramento: "Severe, but no damage done."

1889, May 19. (Centered near Antioch)
Sacramento: "quite severe --- no damage."

1892, April 19. (Centered in Solano County)
VI at Sacramento where many chimneys were thrown down and windows were broken. Books were thrown from shelves.

1902, May 19, M=?
Probably V at Sacramento and Ione.

1906, April 18. (Centered in Marin County, M=8.3)
V at Sacramento.

1909, June 22, M=?
VI at Sacramento: Plaster down.

1915, October 2. (Pleasant Valley, Nevada, earthquake, M=7.6)
V at Sacramento.

1933, June 25. (Center near Wabuska, Nevada)

1948, December 29. (Verdi earthquake)
V at Sacramento where objects were disturbed.

1952, July 21. (Kern County Earthquake)
V at Sacramento where small objects were disturbed.

1954, July 6. (Fallon earthquake, M=7.1)
V at Sacramento.

1954, December 16. (Fairview Peak - Dixie Valley, Nevada, earthquake, M=7.1)
Low VI at Sacramento where a reservoir roof partially collapsed, plaster cracked, power cables were broken, vases overturned.

1966, September 12. (Verdi epicenter)
VI at Sacramento where plaster was cracked, furniture was shifted, windows were cracked.

## Site Bedrock Response

Judging from the historical data, the assumption was made that earthquakes significant to the Sacramento area have in the past and will continue to originate along one of the following fault zones:

San Andreas
Hayward
Calaveras
Midland
Russell Valley

Table B2 lists a summary of seismic activity in terms of magnitude and number of events associated with the above faults. The table does not include events of magnitude less than 4. From these data and the relationships presented in Figures B1, B2, and B3, ground motion parameters were estimated characterizing bedrock response beneath the two sites under study due to the maximum historical earthquake associated with each of the faults under consideration. The results are presented in Table B3. Similarly, ground motion parameters, corresponding to bedrock motion at the sites as a result of maximum credible earthquakes assigned to each fault system by Greensfelder (3) were evaluated and are displayed in Table B4.

These values can be described as indicative of the most severe bedrock motion realistically possible at the existing and proposed sites as a result of seismic activity along the listed faults. It should be emphasized, however, that these values stem from relationships based on idealized situations and a very limited amount of actual recorded data.

Table B2 - Seismic Activity Summary (from Ref. 3)

| Fault | Length of Fault Considered (miles) | 1934-1971 | | 1836-1971 | | |
|---|---|---|---|---|---|---|
| | | 4-4.9 | 5-5.9 | 6-6.9 | 7-7.9 | 8-8+ |
| San Andreas | 200 | 105 | 46 | 3 | 1 | 1 |
| Hayward | 50 | 6 | - | - | 2 | - |
| Calaveras | 120 | 45 | 8 | 1 | - | - |
| Midland | 50 | 3 | - | 1 | - | - |
| Russell Valley | 20 | 25 | 5 | 3 | - | - |

Table B3 - Maximum Historical Earthquake Characteristics

| Fault | Estimated Maximum Magnitude | Estimated Duration of Strong Motion (sec.) | Estimated Predominant Period (sec.) | Estimated Distance to Causative Fault (miles) | Estimated Maximum Rock Acceleration (g) |
|---|---|---|---|---|---|
| San Andreas | 8.25 | 35 | .55 | 80 | .07 |
| Hayward | 7.0 | 24 | .39 | 65 | .05 |
| Calaveras | 6.6 | 20 | .36 | 60 | .04 |
| Midland | 6.2 | 20 | .32 | 24 | .12 |
| Russell Valley | 6.0 | 18 | .33 | 85 | .02 |

Table B4 - Maximum Credible Earthquake Characteristics

| Fault | Estimated Maximum Magnitude | Estimated Duration of Strong Motion (sec.) | Estimated Predominant Period (sec.) | Estimated Distance to Causative Fault (miles) | Estimated Maximum Rock Acceleration (g) |
|---|---|---|---|---|---|
| San Andreas | 8.25 | 35 | .55 | 80 | .07 |
| Hayward | 7.6 | 30 | .42 | 65 | .06 |
| Calaveras | 7.6 | 30 | .41 | 60 | .07 |
| Midland | 7.0 | 24 | .32 | 24 | .18 |
| Russell Valley | 6.5 | 21 | .35 | 85 | .02 |

Table B5 - Design Earthquake Characteristics

| Design Earthquake | Source Fault (miles) | Magnitude | Distance to Source Fault | Maximum Rock Accel-eration-g | Predominant Period (sec.) |
|---|---|---|---|---|---|
| DE1 | Midland | 7.0 | 24 | .18 | .35 |
| DE2 | San Andreas | 8.25 | 80 | .07 | .55 |

Based on the available historical evidence and estimated ground response at the study sites for maximum historical earthquakes, it is concluded that no earthquake during the last 150 years has produced bedrock accelerations greater than .12 g at the existing and proposed sites. Furthermore, the results of an identical analysis based on maximum credible earthquakes, leads to the assertion that it is extremely unlikely that a bedrock acceleration greater than .18 g will ever be experienced at the study sites due to earthquake excitation.

## Design Earthquakes

Restating the findings of the site seismicity analysis, the maximum bedrock acceleration anticipated in the study is approximately .18 g, corresponding to a magnitude 7 earthquake along the Midland Fault. It has been shown by Housner (5), however, that for long period structures (multi-story) a large distant earthquake may have a stronger effect on structural response than a smaller event occurring much nearer the building site. Due to this circumstance, and the knowledge that historically earthquakes originating along the San Andreas fault system have been extremely devastating this ground response investigation will also consider motions resulting from a maximum credible event emanating from the San Andreas fault.

In summary, the design earthquakes selected for evaluating ground response in this study are representative of a large nearby earthquake (Midland fault) and a larger distant earthquake (San Andreas fault). These design earthquakes will be referred to as DE1 and DE2, respectively. Characteristics of bedrock motion likely at the two study sites in response to the design earthquakes are shown in Table B5.

The selection of maximum credible events as design earthquakes was based primarily on a recognized need for the State Capitol Building to remain functional immediately after a major earthquake. This need has been discussed in Reference 7 from which the following quotation was taken:

> "The extent to which the property owner, in this case the people of California, is willing to increase construction costs to achieve a higher degree of earthquake safety depends also on decisions made as to the importance according to the numbers of people exposed per day, the value of the structure, and how vital a function it serves in event of an emergency. Therefore hospitals are given an Importance Factor of 1 in recognition of their vital role in relief of disaster. It may be assumed that the State Capitol would similarly be assigned an Importance Factor of 1, considering the nature of the building and its function as the seat of State government. State law now also requires that hospitals be designed to withstand major earthquakes without damage. It seems only reasonable that the State Capitol should be given equivalent protection. Not only are a large number of people exposed to risk daily, but the legislators performing their duties therein influence the operation and thus the cost-loss picture for the entire State. They and the other leaders of State government who are also frequently exposed to earthquake risk, are important human beings who must be available to act in the case of a disaster. The State Capitol Building should in fact provide a model for earthquake protection throughout the State."

The California Hospital Code, which addresses itself to functionally important structures, states that dynamic

> "analyses shall be based upon the ground motion prescribed for the site in a geology-seismology report. The report shall consider the seismic exposure of the site and the probable maximum seismic event that may be postulated with a reasonable confidence level within a 60-year period."

During this investigation, the strictly statistical approach implied in the California Hospital Code was not followed because of the limited data and nonuniformly located historical seismic events within a given study area, as already mentioned. Also, considering the fact that the West Wing of the existing Capitol is about 100 years old and probably would be used many more years if it were not for earthquake hazards, a 60-year time period was judged to be an unrealistically short life for a State Capitol building. For these reasons, attention was given to the previously listed fault systems to select design earthquakes.

Recorded data for the faults show that since 1836 one event of magnitude 6-6.9 has occurred on the Midland fault and one event of magnitude 8+ has occurred on the San Andreas system. Estimating a structure life of approximately 100 years, it is reasonable to expect that events of similar or greater magnitude could occur on the respective faults, as shown by Greensfelder.

Probability concepts were utilized in establishing some basis of estimating maximum probable bedrock acceleration levels. These results indicated bedrock acceleration levels of 0.2 and 0.3 g's can be expected 1.7 and 0.8 times respectively over a 100 year period. The probability concept assumes, for a given study area, that a particular event is given equal probability to occur anywhere within the study area but, in actuality, may not be valid for this study due to the distance between causitive fault and study site. Hence, a realistic approach based on maximum credible events was utilized to reduce the maximum probable bedrock acceleration levels to reasonable values.

## References

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2. Seed, H. B. and Schnabel, P. B., Accelerations in Rock for Earthquakes in the Western United States, Lecture Note, Earthquake-Resistant Design of Engineering Structures Course, University of California, Berkeley, June 19-30, 1972.

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6. Byerly, Perry, Report to the Sacramento Municiple Utility Distriction Seismic Hazards at the Clay Site, August 30, 1967.

7. Botich, M. E., Cates, Norman and Ridgers, A. H., Report on Evaluation of the State Architect's Study West Wing, California State Capitol, Evaluation Report, VTN Consolidated, Inc., 1973.

# APPENDIX C

REVIEW OF PREVIOUS FOUNDATION DESIGN
AND CONSTRUCTION - WEST WING

## APPENDIX C

## REVIEW OF PREVIOUS FOUNDATION DESIGN AND CONSTRUCTION WEST WING

### The Capitol Building

The present California State Capitol consists of two portions: the older Capitol Building referred to in recent years as the West Wing, and the newer portion known as the East Wing. The foundation of the West Wing supports a massive and imposing masonry structure weighing approximately 86,000,000 pounds, not including the weight of the foundation itself, and covering an area approximately 320 feet long by 160 feet wide, using the greatest dimensions. The height of the building, not including the dome, is over 90 feet. The height of the building from grade at its base to the ball atop the dome is more than 230 feet. Construction of the West Wing was started in 1860, and the main features of the building were completed in 1871 (1). Fortuitously, the original plans and specifications for the foundation and documentary evidence related to the foundation were preserved and subsequently found in the State Archives.

### Foundation Selection

After selection of the site within a four-block area bounded by L, N, 10th and 12th Streets in Sacramento, the State legislators created a Board of State Capitol Commissioners to supervise the construction of a State Capitol building. On July 14, 1860, the

Commissioners selected the plan of M. Frederick Butler for the Capitol design. Three days later, Reuben Clark was appointed superintending Architect for the construction. At the outset, the Capitol Commissioners were duly impressed with the importance of selecting a good foundation for the building and were determined to pile it, irrespective of cost (2). By order of the Commissioners, Clark initiated a series of borings to determine the character of the underlying soil. According to one of the Commissioners (2), Clark reported that he had found as good a foundation subsoil for the building as could possibly be needed, and that piling was not recommended. On July 30, 1860, the Commissioners adopted the first set of specifications (3) requiring concrete foundations at a depth of 5 feet below the ground surface. This type of foundation is now more commonly known as a plain concrete wall footing.

## Foundation Contracts

In September, 1860, Michael Fennell was awarded a contract for construction of the foundation and basement story walls in accordance with his approved plans (4) and the Capitol Commissioners' first set of specifications. On September 24, 1860, ground was broken for the new structure. Construction was started for the northern part of the foundation, but progress was slow. In February 1861, the architect Clark reported that 75% of the trenches for the foundation had been excavated and that 45% of the concrete foundations had been completed (5). Brickwork for the basement walls of the northern portion was started in February, 1861. It was not until May 15, 1861, that the cornerstone of the State Capitol was laid at the northeast corner of the foundation brick walls (6). In May, 1861, the contract with Fennell was broken. One month later, a second set of specifications (7) was written to invite bids for completion of the foundation work. In July, 1861, Blake and Connor were awarded (8) a contract for the completion of the foundation and basement story walls in accordance with their approved plans (9) and a second set of specifications. In 1861-62, during one of the wettest winters in California's history, four heavy floods overflowed the levee banks of the American River and inundated the city, the Capitol grounds, and the foundation walls. In February, 1862 the Commissioners directed the architect Clark to report on the conditions of the foundations of the Capitol building and the flood damages incurred. After Clark had examined the site, he reported that (10):

> "In these examinations I found the soil to be from three to three feet six inches in depth, under which is a bed of firm clay twenty-five feet in depth. Below the clay there is a bed of gravel filled with boulders. There is also a stratum of hard pan, averaging from three to four feet in thickness, passing through the bed of clay about eleven feet below the surface of the ground. The trenches for the reception of the foundations are excavated to the depth of five feet below the surface of the ground, making a cutting of from eighteen inches to two feet into the clay."
>
> "Those excavations were made both during the dry and rainy seasons, and there was no perceptible difference in the consistency of the clay. Consequently, the foundations are perfectly secure and will receive no injury from the effects of the water, but on the contrary, both concrete and brick will become much harder by having been surrounded by water."

In contrast, the contractors Blake and Connors reported (11) that mud and water exceeding one foot in depth had surrounded the foundation walls and covered the site. Under these conditions, they requested additional time before recommencing their work, so that the walls already constructed would have had time to become dry and the ground would become firm and in good condition.

As a result of the flood conditions, a third set of specifications dated May 19, 1862, raised the "established ground line," or planned terrace surrounding the Capitol walls, to a height of 13 feet above the surface of the surrounding streets, and the basement story floor was specified to be 3 feet above the established ground line. By that time, the foundation work had been nearly completed.

Foundation Plans and Specifications

Except for differences in proportioning of ingredients of concrete mixtures for the foundations and some minor changes, the second set of specifications reads essentially the same as the first. To indicate the quality of materials and workmanship required for the foundation, the following excerpts are quoted from the second set of specifications (7):

> "Height of Stories The ground line of the building is to be established to the height of 2 inches above the stone now set on the northeast corner of the building, which is 7 feet 6 inches above the present surface of the surrounding streets, and the basement floor is to be 3 feet 1 inch about the established ground line."

> "Digging and Grading Excavations are to be made for the reception of all the various foundations, to a level of the bottom of those now laid, and the foundations now dug are to be trimmed to the required forms, where required. The earth is to be filled in around all the foundations and solidly rammed, and all the surplus earth to be graded under the basement floor and around the building, as may be directed."

> "Foundations All the various foundations are to be formed of concrete 3 feet in thickness, including the layer of cobble stones, and all to be of the forms and widths shown and figured on the sections marked 'AA,' 'BB,' 'CC,' etc. The bottoms of all the trenches are

to be made perfectly level and covered with a layer of cobble stones, solidly bedded and rammed down; the spaces between the stones to be clear from all earth, rubbish or small stones, for the reception of the concrete."

"The concrete to be formed as follows: to 1 measure of the best Rosendale cement, well manipulated with 1-1/4 measures of clear, sharp, river sand, add 1-3/4 measures of granite chips broken in small pieces (i.e., the average size not to exceed a cubic inch) and 1-1/2 measures of fine gravel, clear from all foreign substances; and all to be thoroughly manipulated and rammed into the trenches immediately after mixing. No layer to exceed 10 inches in thickness. The sides of the concrete to be formed by means of plank boxes, which can be removed for the formation of other sections after the concrete has sufficient consistency for their removal."

"Brickwork All the bricks are to be of the usual dimensions, sound square, hard burnt, and of the best quality, and no soft or salmon bricks to be used in any portion of the work."

"The joints of all brickwork in the cellar are to be smoothly struck."

"All the walls are to be commenced on the concrete, battering upward, as shown in the sections at large. In the execution of the work, care must be taken to effect a perfect bond, and every 5th course to be headers."

"Mortar The mortar for all brick work, from the line of the concrete to the established ground line, is to be formed of 2 parts of Rosendale cement to 1 part of fresh lime; ... and all to be well manipulated in a mill, with a proper proportion of clean, sharp, river sand, free from all foreign substances, and all to be well screened, if required."

With regard to this set of specifications, it should be noted that: (1) excavations would have had to be carried to a depth of 5 feet below the ground surface existing at that time, in order to make the base of the concrete foundations level with the bottom of those already laid; (2) the established ground line (the planned terrace level surrounding the building) was subsequently raised, as noted previously, after the heavy winter floods of 1861-62; and (3) "basement floor" as used in the specifications corresponds to the present first floor. In the 1860's-1880's, the present basement was used for waste disposal (6).

Except for minor changes, the foundation plans (9) submitted by Blake and Connor in 1861 are similar to those (4) submitted by Fennell in 1860. Figures C1 and C2 were prepared from the original plans to show, respectively, the general outline of the foundations and three typical cross-sections, A-A, B-B, and C-C. In general, the foundation consists of a network of unreinforced load-bearing walls supported on slightly-tapered plain concrete footings. The annular basement wall under the rotunda and dome of the building is step-tapered to a width of 10 feet 4 inches, just above the concrete foundation. As shown, the depth of all concrete foundations is constant at 3 feet, while the width varies to accommodate the different thicknesses of the supported walls.

## Wall Cracks and Supporting Buttresses

According to one of the Capitol Commissioners (2), the architect Clark and the Commissioners had observed a crack in the north wall in 1862, but it never troubled Clark. In September, 1865, Clark was granted a leave of absence because of illness, and the following January, Gordon Cummings was appointed Architect. Soon after his appointment, Cummings had noticed two cracks in the north wall, one on each side of the north portico (12). Presumably, the cracks had occurred in the basement wall. In October, 1866, he reported this condition to the Commissioners, who considered the matter of sufficient importance to engage the services of Major George Elliott, of the United States Engineers, and H. Kenitzer, an architect.

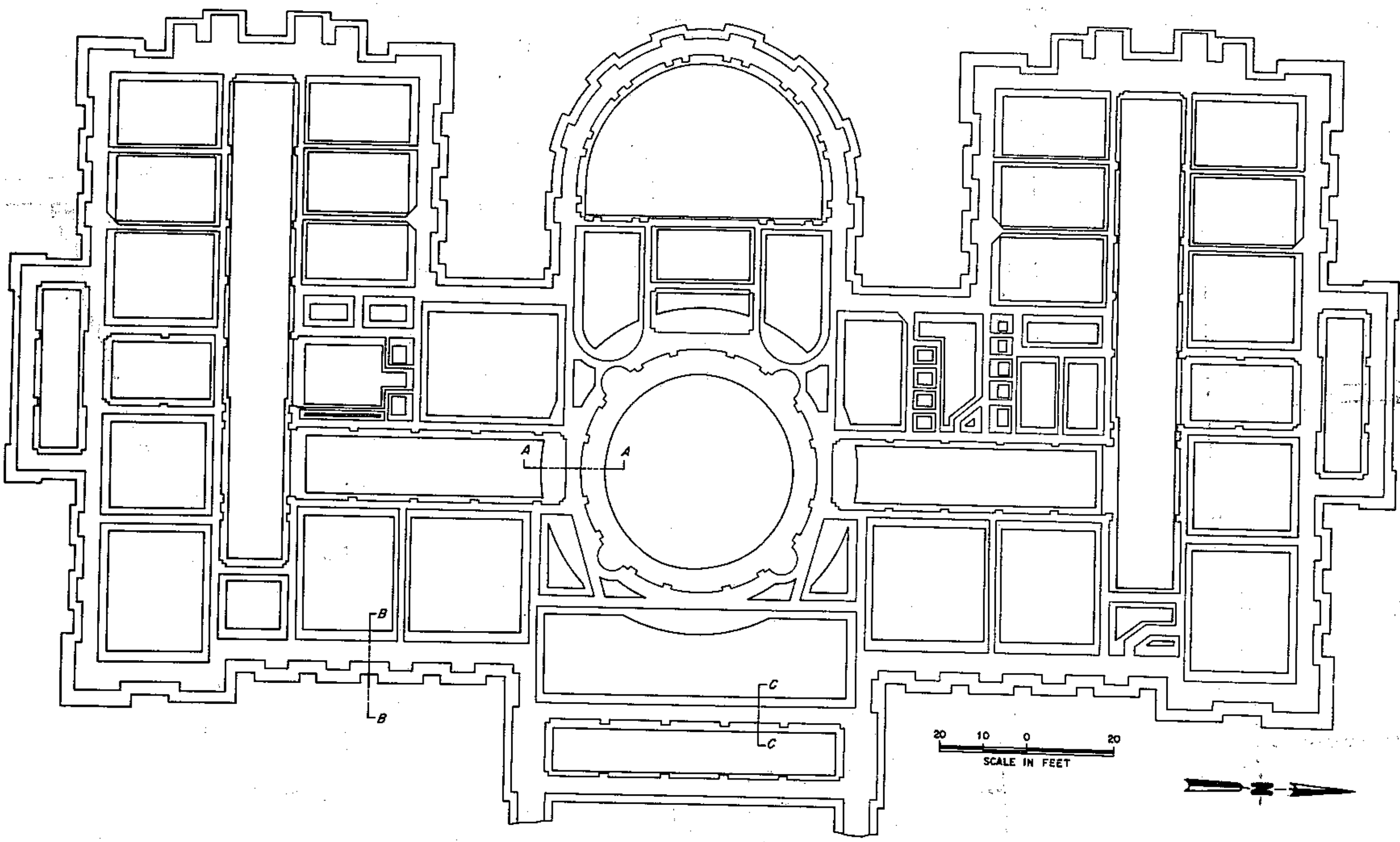



Fig. CI FOUNDATION PLAN WEST WING - CAPITOL BUILDING
(After Blake & Connor, 1861)

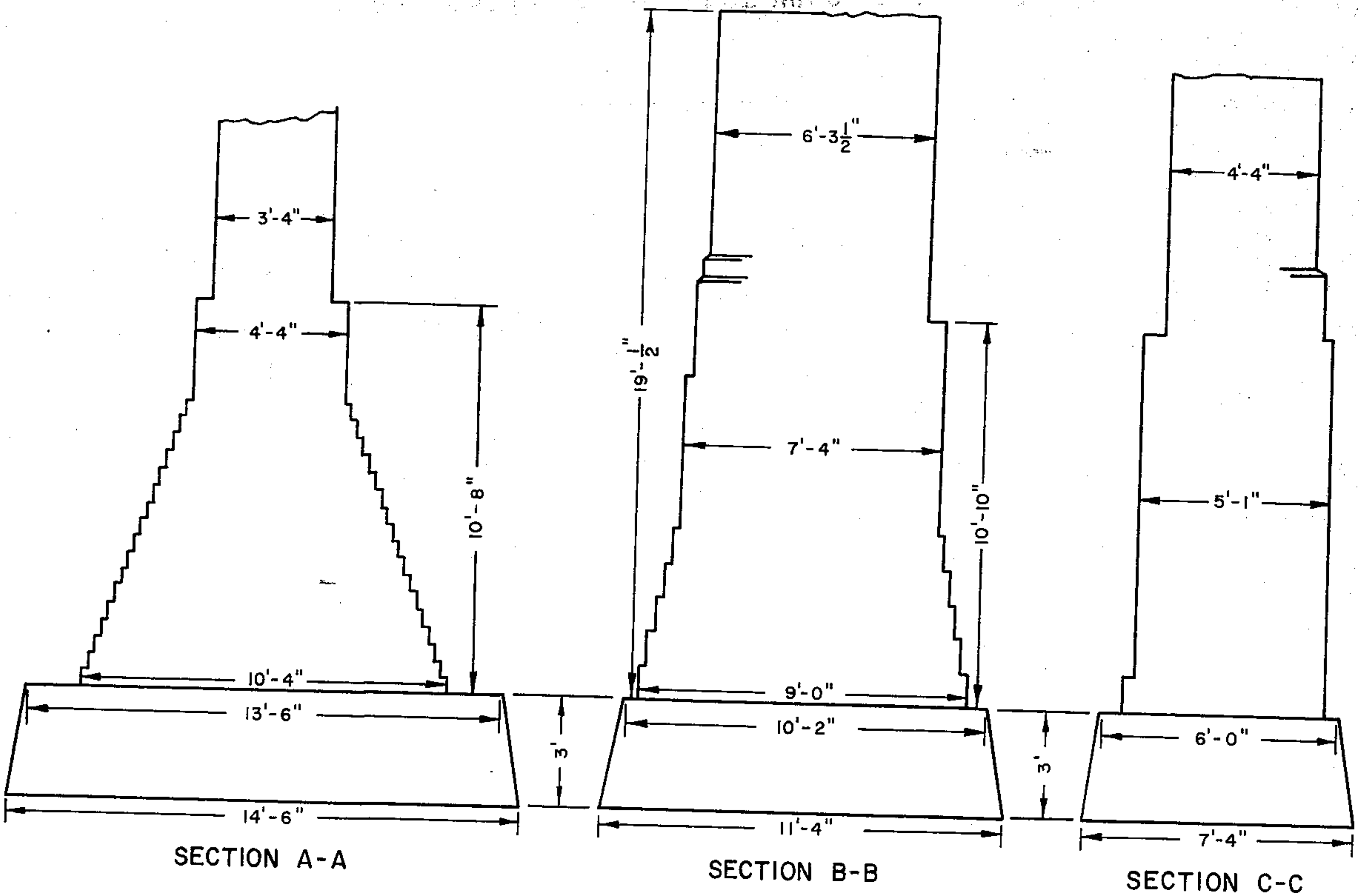



Fig. C2 TYPICAL FOUNDATION SECTIONS
West Wing-Capitol Building
(After Blake & Connor, 1861)

In a report (13) dated October 30, 1866, Elliott and Kenitzer described the subsurface for the site as a compressible clay resting at a moderate depth on an incompressible substratum. They suggested that, for these conditions, it would have been better to have used for the foundation bed a grillage and platform resting on piles reaching down to the incompressible soil, so that there would be little or no settlement. Moreover, the type of foundation already adopted could not prevent settlement, but only limit it and make it uniform by constructing the bearing surfaces of the foundations proportionate to the loads to be carried. Elliott and Kenitzer considered the bearing surfaces for all the walls as probably being too small for the compressible soil, and especially so at the masonry corners. Thus, they attributed the cause of the cracks to differential settlement of the partially completed structure.

Under the circumstances, Elliott and Kenitzer recommended that buttresses be constructed at the northeast and northwest corners of the building, which would provide additional support and tend to produce uniformity in any further settlement that might occur. The buttresses would in no way mar the appearance of the Capitol building, as they would be entirely beneath the ground surface after the area was graded. Although the Elliott-Kenitzer report refers to a sketch of the buttresses, no drawings showing either the exact location of the cracks or details of the butresses have yet been found, if still existent.

In December, 1866, Elliott recommended that a series of borings be made around the Capitol building to determine the character of the soil below the surface and the depth of an incompressible stratum (14). During the following month 8 borings were made around the Capitol building, Figure C3, one near each corner and one near the center of each side (15). Figures C4 and C5 show,

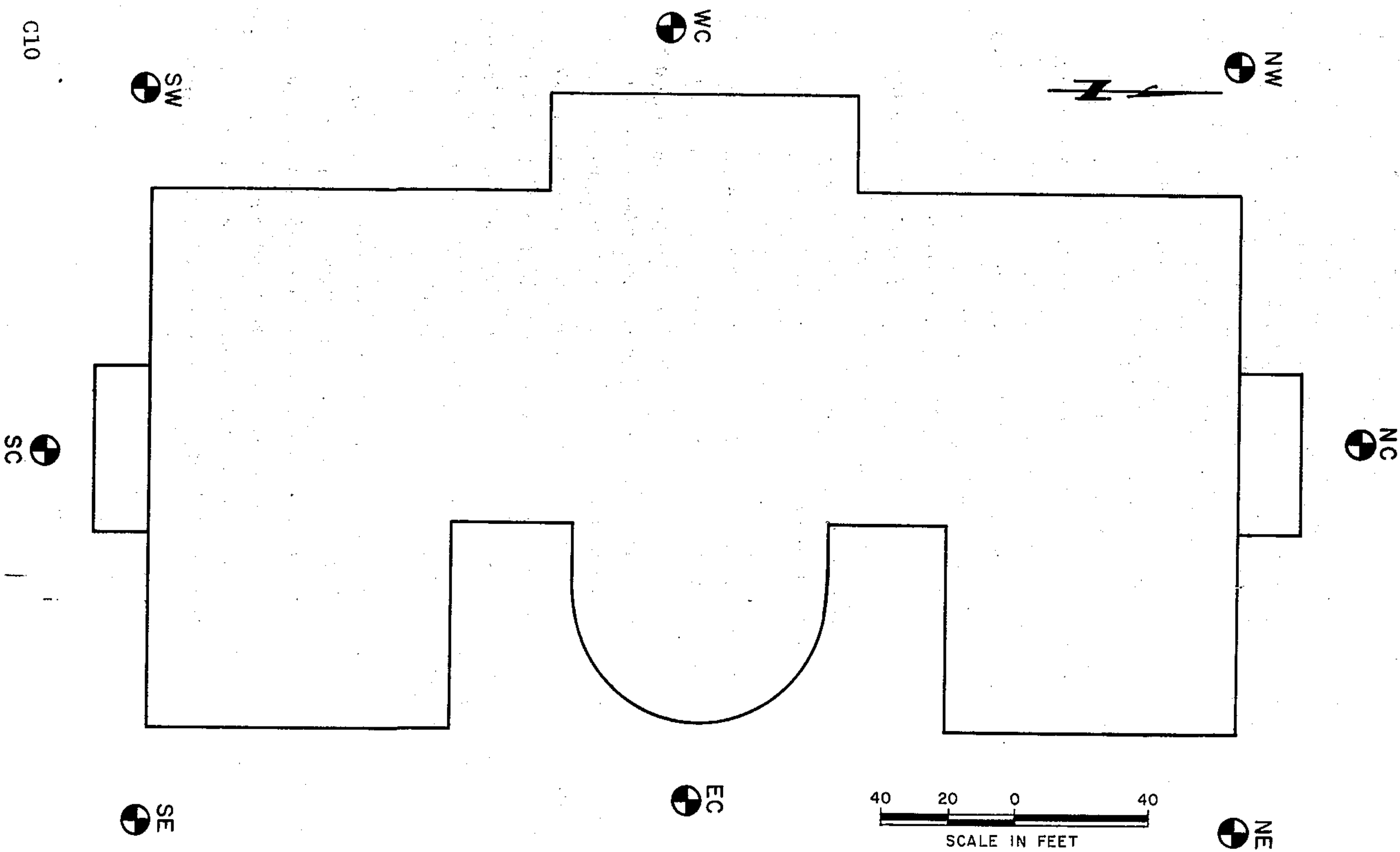



Fig. C3 - BORING LOCATION PLAN - WEST WING (1866)
(Reference 15)

NOTE Based on 1974 foundation investigation, most probable elevation of 1866 ground surface was Elev +15'(±) with respect to USGS datum

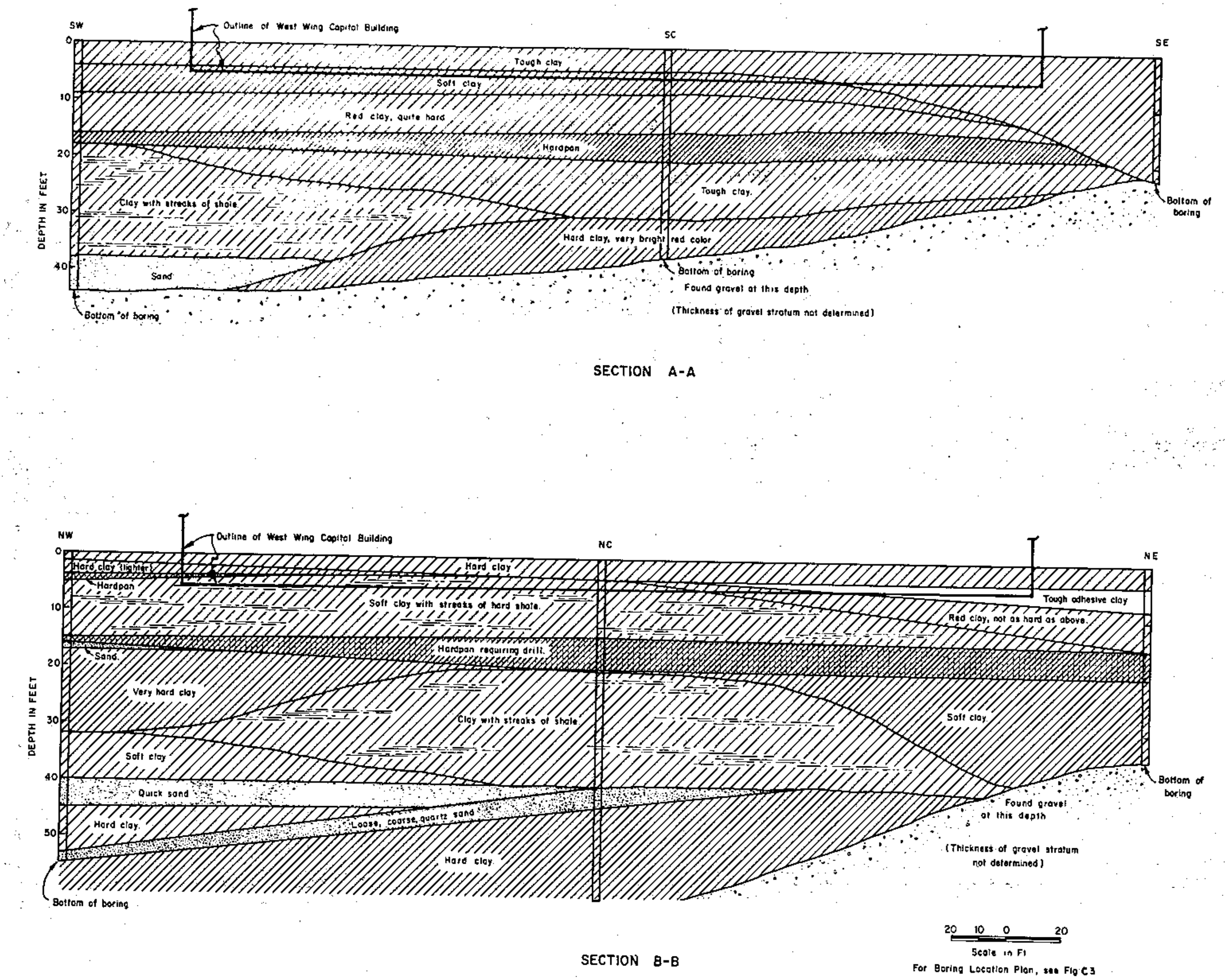
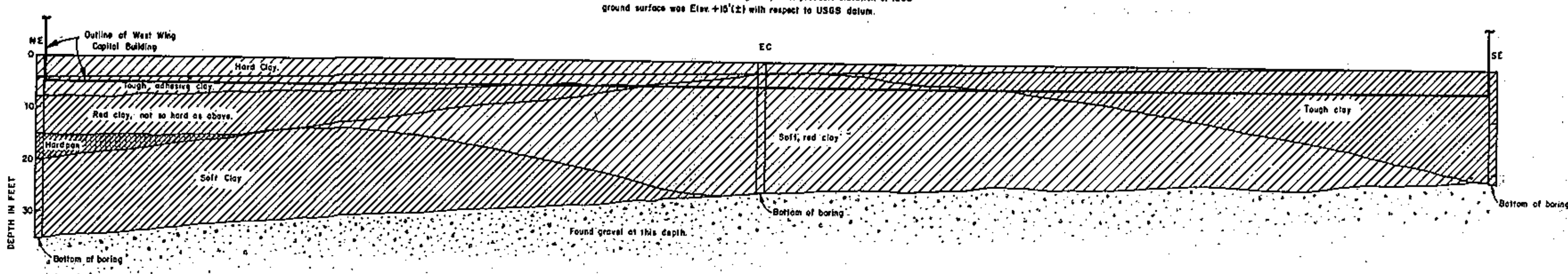
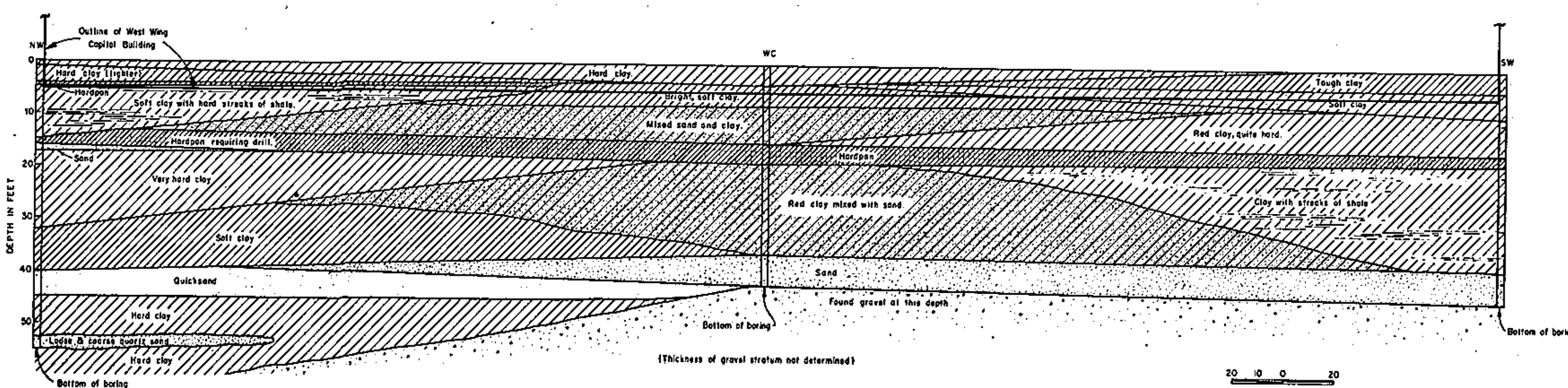



Fig. C4 - SOIL PROFILES FROM BORINGS MADE IN 1866
WEST WING - CAPITOL BUILDING
(Reference 15)

NOTE: Based on 1974 foundation investigation, most probable elevation of 1866 ground surface was Elev. +15'(±) with respect to USGS datum.




SECTION C-C




SECTION D-D

For Boring Location Plan, see Fig. C-3

Fig. C5 - SOIL PROFILES FROM BORINGS MADE IN 1866
WEST WING - CAPITOL BUILDING
(Reference 15)

respectively, the soil profiles in the East-West and North-South directions. Except for 2 borings, a gravel stratum was reached within 22 to 44 feet below the ground surface existing at that time. Below and adjacent to the northwest corner and the center of the north wall, no gravel was reached at bored depths of 55 and 60 feet, respectively.

It should be noted that the buttresses proposed by Elliott and Kenitzer were substantially the same as the one initiated by the architect Cummings and partly completed at the northeast corner, i.e., "a footing of one hundred and ten surface feet of concrete and granite added to each corner, and brick angle buttresses built into and incorporated with the walls" (12). Cummings had also reported that:

> "From careful observations taken with the instrument from established benchments at regular periods of two weeks, between January first and September twenty-four (1867), when the whole weight for this year was placed on the corners, I found the subsidence to be scarcely perceptible, being less than one quarter of an inch, while in the centre of the building it is nearly two inches in the same time, producing now almost a perfect level."

Cummings was satisfied that the defect had been remedied.

In 1868, a Joint Committee of the State legislature reported (2) on the foundation conditions of the State Capitol, including the basement wall cracks. The Committee had been instructed to investigate and report on the safety and stability of the foundations of the Capitol building and the possible danger from river floods. The findings of the Committee cover nearly 100 pages and include the reports and testimonies of a large number of engineers, architects, mechanics, builders and contractors. The testimony was varied and sometimes conflicting.

As noted from the following quote from the report, these experienced men presented various concepts to account for the presence of the wall cracks:

> "Some thought that in this instance it could be assigned to the compressibility of the soil beneath; some thought they may have been caused by the shrinkage of the material; and others thought that they were owing to a combination of causes, to wit: the compressibility of the soil beneath, and unequal distribution of the weight upon the different parts of the foundation; whilst more thought that these fissures were attributable solely to the unequal distribution of the weight."

Other differences were related to: the nature of the subsoil, the number and size (3/8 to 3/4 of an inch wide) of the cracks, and the amount of differential settlement between the various parts of the structure. It was generally agreed, however, that the maximum settlement had occurred at the northwest corner of the building, where a boring went to a depth of 55 feet without disclosing any gravel or boulders.

From the evidence presented, the Committee concluded that the foundation of the Capitol building was safe and secure and that the cracks were not detrimental.

It should be noted that in November, 1869, Cummings reported (16) that the cracks had nearly disappeared. Subsequently, the buttresses and exterior basement walls were covered with fill material. In 1872, fill material was hauled to the site with as many as 182 2-horse teams making some 10 trips per day. The grading of the grounds with two terraces around the Capitol building was essentially completed by November, 1873 and continued in existence until relandscaped in 1951-52 (1).

## Foundation Investigation, 1948

At the request of the State Architect, an investigation was conducted in February and March, 1948, of the soil conditions for a proposed addition to the State Capitol, just east of the existing West Wing. Eleven power borings were made, and the results of the investigation are discussed in Appendix D of this report.

In May, 1948, two hand borings were made adjacent to one of the footings of the existing West Wing (17). The boring logs describe the soil under the footing as a brown silty clay. Laboratory tests were performed on 2-inch samples thus obtained, to determine soil parameters for evaluating the adequacy of the existing foundation. Shear strength parameters, c and Ø, were determined from direct shear tests and used to calculate the bearing capacity of the soil under the existing footing. In the analyses the additional parameters used were:

size of footing = 9.25 ft. x 10.5 ft.
height of surcharge = 7 ft.
unit load on existing footing = 5.4 kips per sq. ft.

The foundation report (17) summarized the results of the analyses in tabular form:

SUMMARY OF RESULTS
(After Reference 17)

| Shear Strength Parameters | | Krey's Method | | Terzaghi & Peck Method | |
|---|---|---|---|---|---|
| Ø Degrees | c lbs/sq.ft. | Bearing Capacity lbs/sq.ft. | Factor of Safety | Bearing Capacity lbs/sq.ft. | Factor of Safety |
| 14 | 400 | 9940 | 1.8 | 6500 | 1.2 |
| 12 | 487 | 9200 | 1.7 | 6900 | 1.3 |
| 14 | 487 | 11200 | 2.1 | 7100 | 1.3 |

The report (17) noted that a factor of safety of three is often recommended in design, while the maximum factor of safety indicated by the analyses was only two. Thus, it concluded that the unit load on the foundation soil under the existing footing of the West Wing had exceeded the safe design loading calculated by approved design methods based on soil tests.

## Foundation Alterations

In 1949 construction was started on the adjacent East Wing of the State Capitol, resulting in some alterations to the existing West Wing. These alterations consisted of the reconstruction of the central portion of the east wall of the West Wing, and the demolition of the semicircular part of the building just east of the rotunda.

Two design and construction plans (18) give details of reconstructing the central portion of the east wall, including the addition of two reinforced concrete spread footings and a new buttress wall in the basement. Essentially, the construction procedure for the foundation alterations consisted of:
(1) placing two new wall footings and concrete walls in recessed areas up to the underside of the present second floor; (2) installing jack bearing pads and hydraulic jacks between the top of the new concrete wall and the existing brick wall above; (3) after the concrete had attained sufficient strength, the brick wall load was taken up by jacking to a predetermined load; (4) with the jacks maintaining the load, and after settlement had occurred, pipe shores were wedged on each side of the jacks, and the space between the top of the new wall and the underside of the existing brick wall was filled with dry-pack; (5) after the dry-pack had aged sufficiently to carry the required load, the jacks were removed and the remaining space

around the pipe shores were filled with concrete and dry-pack. After the load was satisfactorily transferred to the new foundations, the existing semicircular construction on the east side of the reconstructed wall was removed.

Except for passageway connections, the East Wing was designed to be structurally independent and separated from the older West Wing, as described in Appendix D of this report.

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3. Minutes of the State Capitol Commissioners, "Specifications for the Construction of the Foundations and the Erection of the Basement Story Walls," California State Archives, July 30, 1860, 8 pp.

4. Two Plans of the California State Capitol entitled "FOUNDATION PLAN" and "BASEMENT PLAN," contracted by Michael Fennell, filed with the Secretary of State, Sept. 22, 1860.

5. Report of Reuben Clark to the Board of State Capitol Commissioners, California State Archives, no date (presumably Feb., 1861 from contents of report), 2 pp. handwritten.

6. Bowman, J. N., "Cornerstone of the State Capitol, Its Discovery on October 15, 1952," California Historical Society Quarterly, Vol. XXXIII, No. 4, Dec., 1954, pp. 329-335.

7. "Specifications of Labor, Materials and Mechanical Workmanship to be Furnished for the Erection of the Basement Story Walls of the State Capitol of California," California State Archives, June, 1861, 9 pp.

8. Appendix to Journals of Senate & Assembly, 14th Session, State of California, "Report of the Board of State Capitol Commissioners," Sacramento, State Printer, 1863, pp. 3-14.

9. Two plans of the California State Capitol entitled "FOUNDATION PLAN" and "BASEMENT PLAN," submitted by R. Blake and P. E. Connor, Contractors, filed with the Secretary of State, July 29, 1861.

10. Minutes of the State Capitol Commissioners, Communication of Reuben Clark to the Board of State Capitol Commissioners, California State Archives, Feb. 24, 1862, 4 pp.

11. Minutes of the State Capitol Commissioners, Communication of Blake and Connor to the Board of State Capitol Commissioners, California State Archives, Feb. 19, 1862, 2 pp.

12. "Biennial Report of the State Capitol Commissioners from November 1, 1865, to November 1, 1867," Sacramento, State Printer, 1867, pp. 4-20.

13. Report of George H. Elliott and H. Kenitzer to Governor F. F. Low, Chairman, State Capitol Commissioners, California State Archives, Oct. 30, 1866, 4 pp. handwritten.

14. Communication of George H. Elliott to Governor F. F. Low, Chairman, State Capitol Commissioners, California State Archives, Dec. 10, 1866, 3 pp. handwritten.

15. Minutes of the State Capitol Commissioners, California State Archives, Jan., 1867, pp. 237-240.

16. Appendix to Journals of Senate and Assembly, 18th Session, State of California, Vol. III, "Biennial Report of the State Capitol Commissioners from November 1, 1867 to November 1, 1869," Sacramento, State Printer, 1870, pp.9-12.

17. Foundation Report, State of California, Division of Highways, Materials and Research Department, July 8, 1948, 12 pp.

18. Two Plans, Sheets S18 and S19, entitled, "State Capitol Addition, California State Capitol, Sacramento, California," Division of Architecture, Department of Public Works, State of California, May 2, 1949.

# APPENDIX D

## REVIEW OF PREVIOUS FOUNDATION DESIGN AND CONSTRUCTION - EAST WING

## APPENDIX D

## REVIEW OF PREVIOUS FOUNDATION DESIGN AND CONSTRUCTION EAST WING

### State Capitol Addition

The newer building of the California State Capitol, now known as the East Wing and sometimes referred to as the Capitol Annex, was constructed during the period June, 1949, to January, 1952 (1, 2). The foundation of the East Wing supports a structure weighing approximately 48,000,000 pounds, not including the weight of the foundation itself, and covering an area of approximately 60,000 square feet. The East Wing is about 100 feet high. It consists of five floors, a basement, and penthouses constituting a sixth floor. The basement serves primarily as a garage, but also provides space for mechanical and service facilities. Although designed to be structurally independent and separated from the adjacent older West Wing, the East Wing connects directly into the main corridors of the original Capitol and into the two legislative chambers.

### Subsurface Investigation

At the request of the State Architect, an investigation was conducted in February and March, 1948, of the soil conditions at the site of the proposed East Wing, just east of the West Wing Capitol building (3). Eleven power borings were made with 2-inch diameter California-type samplers at the locations shown on Figure D1. Laboratory tests were made on samples obtained

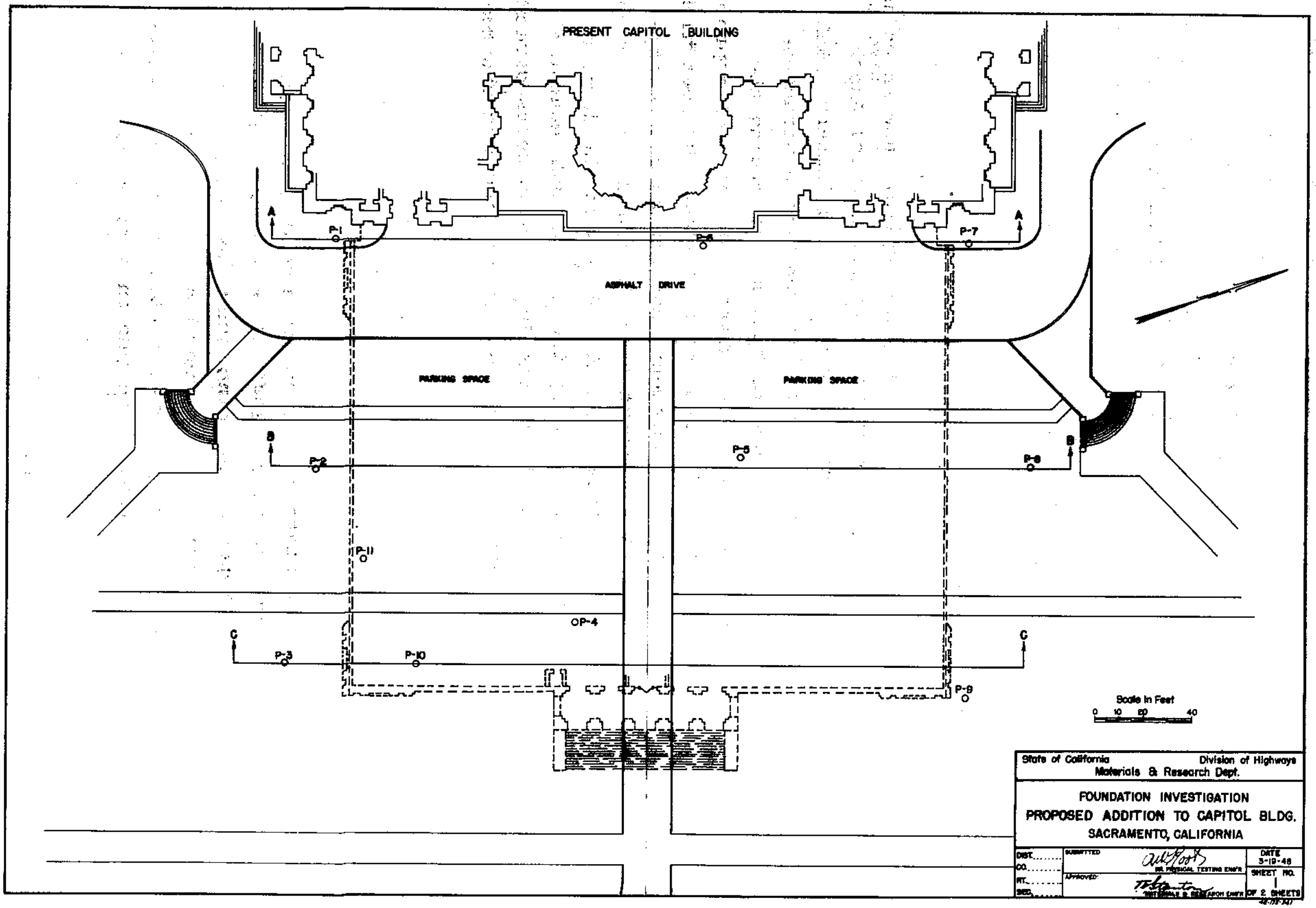

PRESENT CAPITOL BUILDING
ASPHALT DRIVE
PARKING SPACE
PARKING SPACE
P-1
P-2
P-3
OP-4
P-5
P-6
P-7
P-8
P-9
P-10
P-11
A
B
C
Scale in Feet
0 10 20 40
State of California
Division of Highways
Materials & Research Dept.
FOUNDATION INVESTIGATION
PROPOSED ADDITION TO CAPITOL BLDG.
SACRAMENTO, CALIFORNIA
DIST.
CO.
RT.
SEC.
SUBMITTED
APPROVED:
DATE
3-19-48
SHEET NO.
1
OF 2 SHEETS

Fig. D-1

from the various borings, including moisture contents, density, mechanical analysis, consolidation and direct shear tests. Figure D2 also shows the soil profiles with plotted results of the test data. In a report (3) dated March 31, 1948, the soil conditions at the site were described as follows:

> "The surface soil on the site is man-made fill consisting of sandy soil, the layer varying in thickness from three feet on the easterly side to fifteen feet adjacent to the Capitol building. The in-place soil underlying this embankment material is a river deposited soil of undetermined depth, the upper stratum of which is clayey silt with scattered irregular lenses of clayey silt and sand. The lower limit of this layer of silty soil varies from Elevation 00 to Elevation -12 (U.S.G.S. datum). Below the clayey silt layer is a stratum of coarse sand and gravel of variable thickness; in some of the borings the depth of the gravel was not determined as churn drilling and casing would have been required in order to penetrate further into the gravel formation. In Boring No. P-8 the bottom of the hole at Elevation -18 was still in sand and gravel; in contrast, at the southeast corner of the site the sand and gravel lay between Elevation -12 and -17, with stiff silty clay from Elevation -17 to -23, and clean sand between Elevation -23 and -35; from Elevation -35 to the bottom of Boring No. P-3, Elevation -45, the soil varied from clayey silt and sand to fine clayey sand."

The report (3) also summarized the bearing capacity and settlement analyses made for proposed alternate type of footing foundations located within the layer of clayey silt; the results are described in the following section of this Appendix.

## Foundation Selection

According to the Division of Architecture (1, 4), their office would normally have used piles carried down into the underlying gravels to support a structure such as the proposed East Wing on the soil formations shown in Figure D2. They (1) recognized

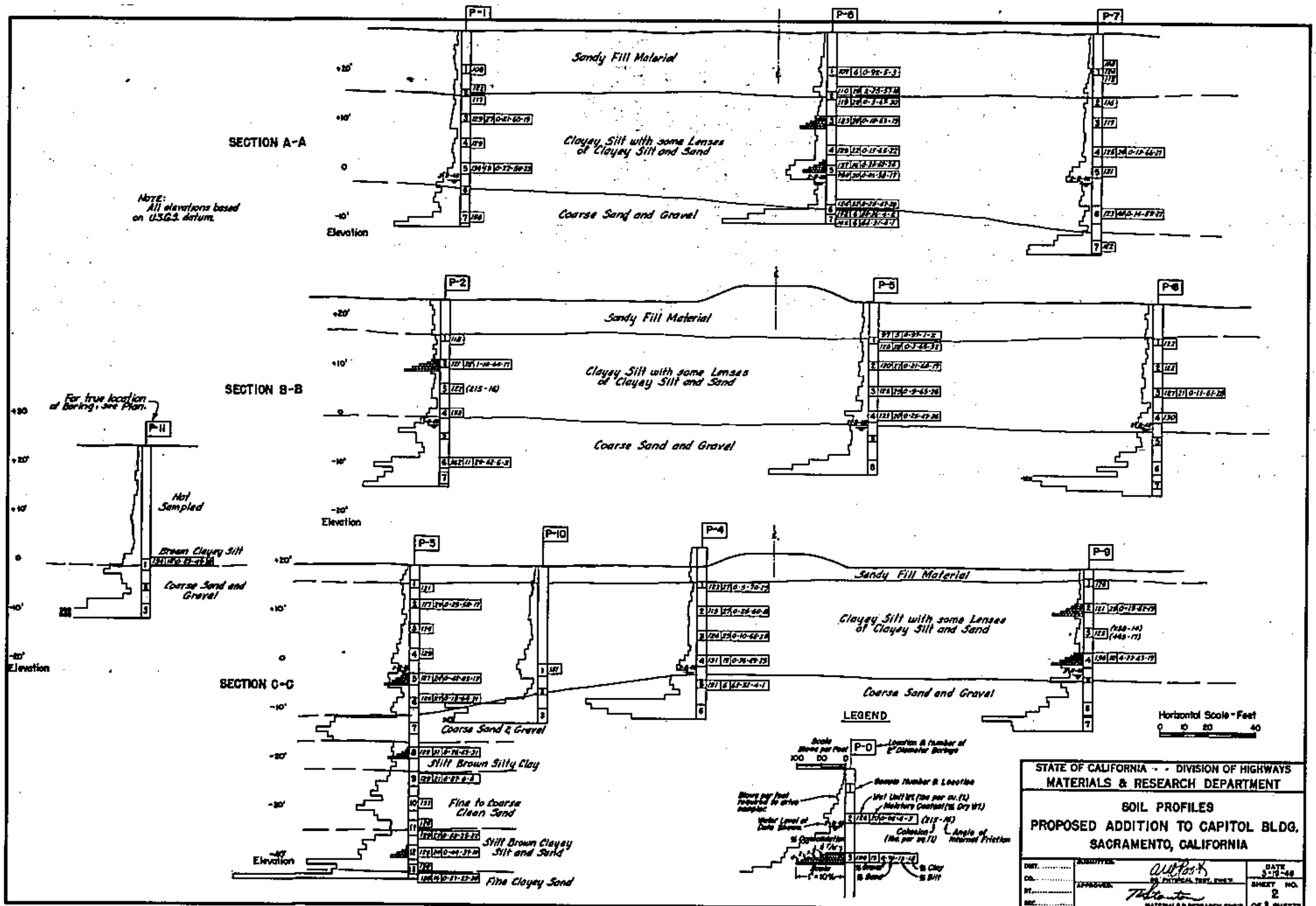



Fig. D-2

Soil Boring Profiles

that much of the area under and around the Capitol Park was at one time or other the bed of a river, or a slough, during the geological history of the Sacramento area, and that most of the surrounding concrete buildings are built on pile foundations driven 30 or more feet below the ground level. However, their analyses (4) of the foundation conditions of the existing adjacent West Wing influenced their selection of a foundation type for the proposed East Wing.

The Division of Architecture was aware of the large cracks, which had occurred during construction of the north wall of the old West Wing. Further, their analyses (1, 4) indicated that the soil under the footings of the West Wing was overloaded due to dead load only without regard to live load or seismic loads. Thus, they reached the conclusion that the driving of piles adjacent to the existing West Wing with its inadequate footings would have resulted in irreparable damage due to the vibrations set up by the pile driving. This problem led to a consideration of a footing type of foundation for the proposed East Wing.

Another problem posed during design of the East Wing was the wide column spacing of approximately 25 feet each way as governed by the basement garage requirements (4). It was evident that the use of normal weight concrete floors for such relatively long spans would have resulted in very large footings for low allowable soil pressures. This problem was solved by using a continuous steel frame structure with reinforced lightweight concrete floors and exterior walls. The net result was a substantial reduction in the dead weight of the entire building.

Initially, two types of footings were considered: the isolated square footing and a grid type extending each way between columns, with the bottoms of the proposed footings at approximately Elevation 8 (3, 4). This elevation is within the layer of clayey silt as shown by the soil profiles in Figure D2. The results (3) of the subsurface investigation indicated that below this elevation, the moisture content of the clayey silt layer varied from 10 to 40%, with an average of 25%. The in-place soil density varied from 88 to 118 pounds per cubic foot dry weight, with an average of 103 lbs. per cu. ft. Direct shear tests indicated the following variations in shear strength parameters; c from 215 to 448 lbs. per cu. ft. and Ø from 13 to 16 degrees. The following quote from the foundation report (3) summarized the results of the bearing capacity analyses:

> "The tentative building design being studied by the Division of Architecture contemplates interior columns spaced 25 feet in each direction, with a load of 550 kips at each column. Two alternate footing designs are being considered for the columns: (1) 14 ft. x 14 ft. pedestal, and (2) monolithic footing consisting of 8 ft. wide concrete strips in both directions. The unit loads on the foundation soil would be 2800 lbs. per sq. ft. and 1630 lbs. per sq. ft., respectively, for the two types of footings. As the test data summarized above indicate that the safe bearing power of the soil is one ton per sq. ft. or less, the loading on the soil under the 14 ft. x 14 ft. pedestal would be excessive; however, the soil should safely support the 8 ft. wide strip 'grid' type of footing without shear failure."

Settlement analyses based on consolidation tests were also made to predict the probable settlements for both the square and grid type of footings. The foundation report (3) summarized the results as follows:

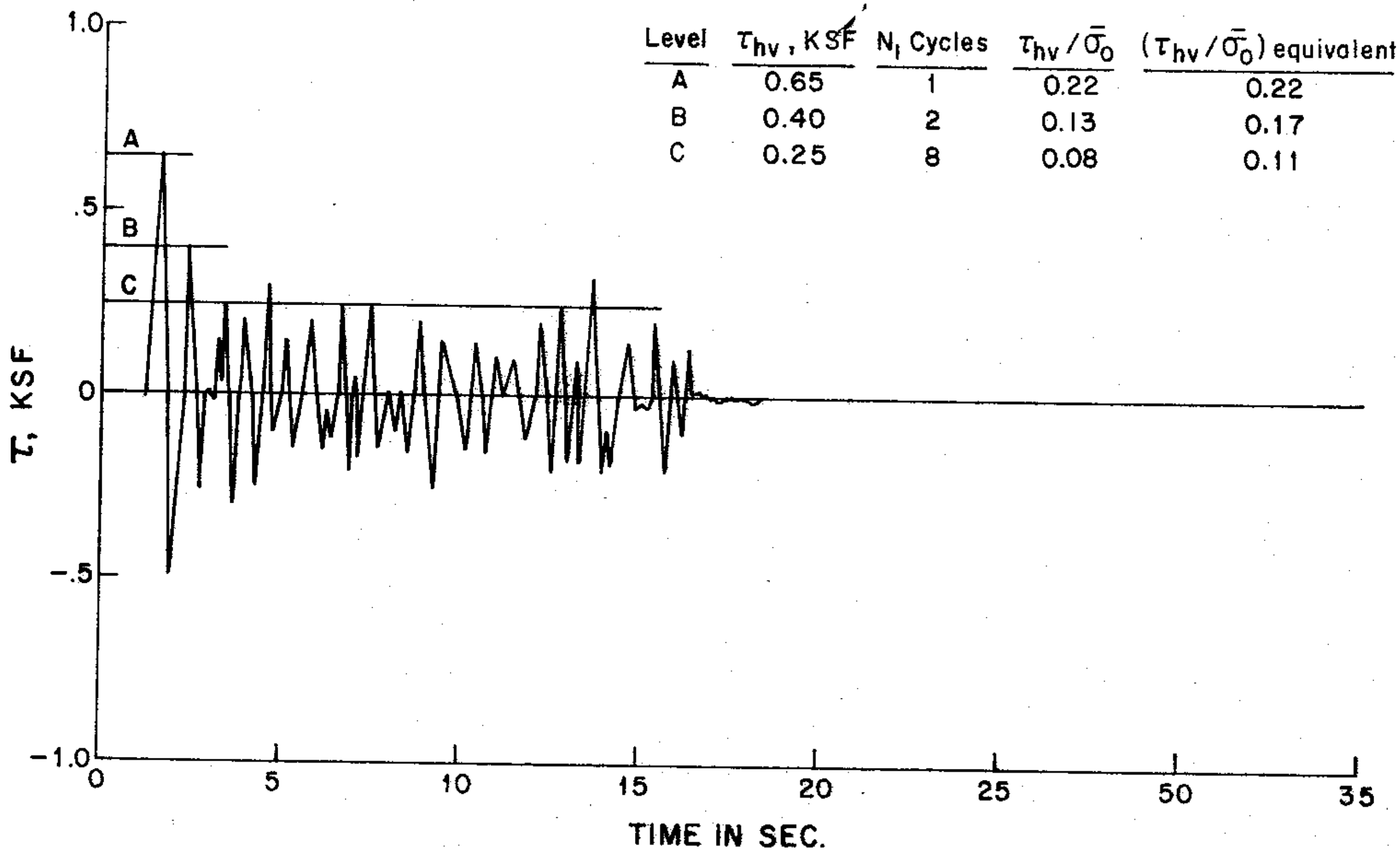
> "The maximum settlement was computed at Boring P-3 at the southeast corner of the site, where the depth of compressible soil is greatest. The maximum differential

settlement would be between this point and Boring P-2, where the depth of compressible soil is the minimum. For the 14 ft. x 14 ft. pedestals with loading of 2800 lbs. per sq. ft. on the soil, the maximum ultimate settlement due to consolidation of the foundation soil is calculated to be approximately 0.15 ft., and the minimum settlement about 0.05 ft., so the maximum differential settlement would be 0.1 ft. For the grid type of strip footings with loading of 1630 lbs. per sq. ft. on the soil, the settlements would be much less. These values are for settlement due to consolidation only and include no estimate of any subsidence caused by plastic flow of the soil under the footings, which for the heavier foundation loading might be considerable; for the lighter loading, the settlement due to plastic flow should be very slight."

From a consideration of these analyses, the grid type of footing was selected for the design of the East Wing foundation.

In May, 1948, two hand borings were made adjacent to one of the existing footings of the West Wing, at locations inaccessible to the power rig. Laboratory tests (5) were performed on 2-inch diameter samples thus obtained; the test results are discussed in the preceding part of this report. However, for purposes of comparison, some pertinent qualitative results should be here noted. The soil samples obtained in 1948 by hand boring adjacent to an existing footing of the West Wing consisted of a clayey silt similar to the material at the proposed footing levels for the East Wing. However, the test results also showed that the soil under the existing footing of the West Wing has a higher shear strength and was more consolidated than the soil in the same horizon outside the area of the West Wing, thus indicating, as expected, that some consolidation and corresponding strength increase had occurred under the footing of the existing West Wing.

With regard to soil conditions at the proposed site for the East Wing, the foundation report (5) noted that:




| Level | $\tau_{hv}$, KSF | $N_l$ Cycles | $\tau_{hv}/\bar{\sigma}_0$ | $(\tau_{hv}/\bar{\sigma}_0)$ equivalent |
|---|---|---|---|---|
| A | 0.65 | 1 | 0.22 | 0.22 |
| B | 0.40 | 2 | 0.13 | 0.17 |
| C | 0.25 | 8 | 0.08 | 0.11 |

FIG.I6 TIME HISTORY OF SHEAR STRESS AT 29.8 FEET
-PROPOSED SITE (CASTAIC E.Q. RECORD)

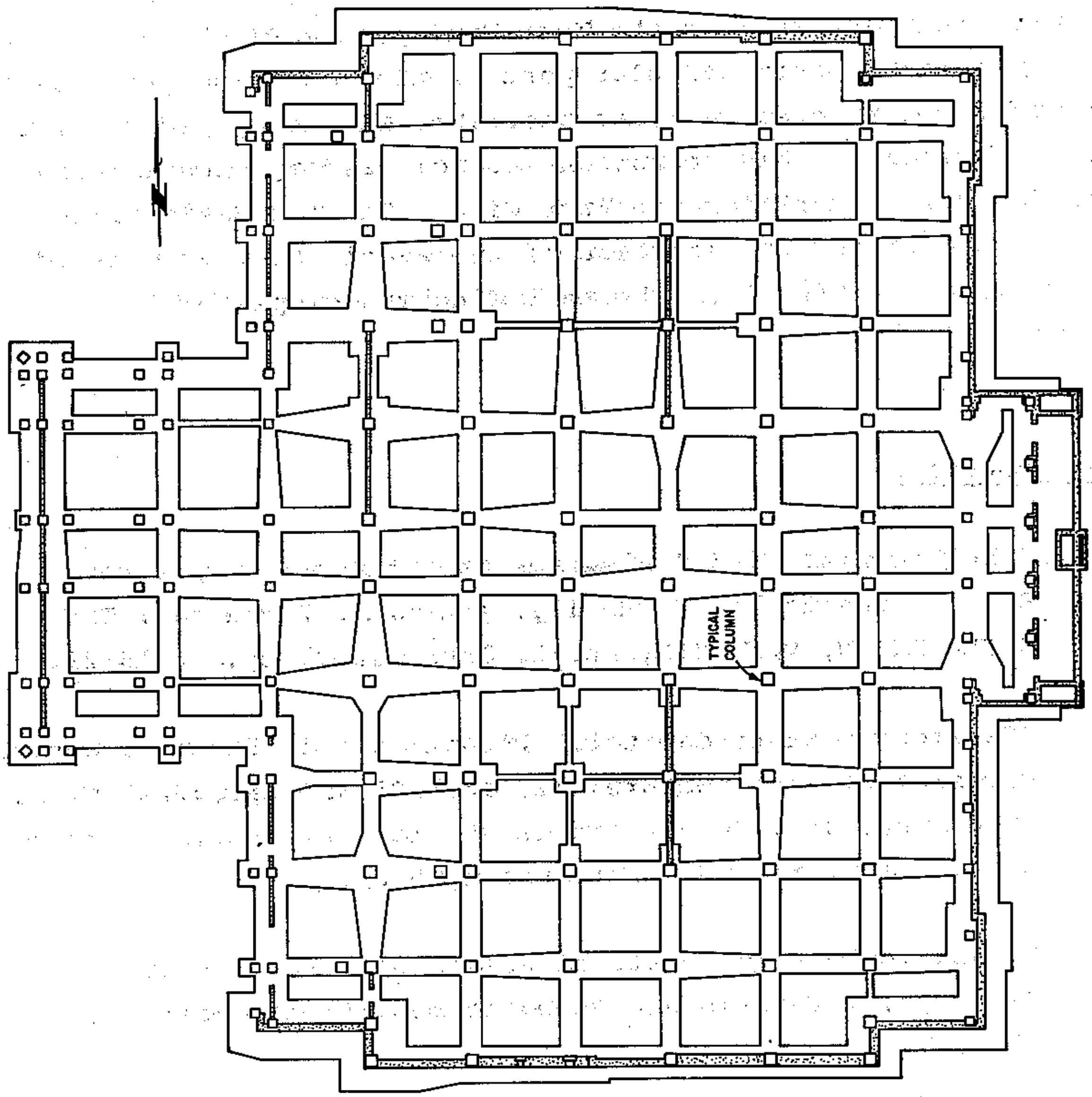



Fig. D3 FOUNDATION PLAN – EAST WING
Scale 1" ≃ 40'

## Foundation Construction

The first contract for the construction of the East Wing was awarded in June 1949, for the excavation and foundation work (2). Preliminary operations required reconstructing the central part of the east wall of the West Wing just east of the rotunda, and removing the semicircular part of the West Wing just east of the reconstructed wall, as described in the preceding part of this report. This operation was followed by excavation and foundation work for the East Wing. The foundation work was completed and the erection of the steel work was started in September, 1949 (1). The East Wing was completed in January, 1952.

## References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3. Foundation Report, State of California, Division of Highways, Materials and Research Department, Mar. 31, 1948, 14 pp.

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5. Foundation Report, State of California, Division of Highways, Materials and Research Department, July 8, 1948, 12 pp.

6. Plan, Sheet S13, entitled, "State Capitol Addition, California State Capitol, Sacramento, California," State of California, May 2, 1949.

# APPENDIX E

## FIELD INVESTIGATIONS - EXISTING SITE

## APPENDIX E

## FIELD INVESTIGATIONS - EXISTING SITE

Field investigations of the existing site were directed toward gathering information necessary to analyze the foundations of both the East and West Wings, and to develop preliminary design criteria for a possible new wing. The field work consisted of:

1. A general inspection of the building (both wings) as a whole, and a more detailed inspection of the West Wing basement and the parking area under the East Wing.

2. A thorough review of available previous foundation design, specification, and construction records for both wings (Appendices C and D) to determine foundation type, planned dimensions, and construction methods.

3. A program of boring and sampling on the grounds surrounding the building to delineate subsurface soil stratification and to obtain samples for laboratory testing.

4. Geophysical testing in selected boreholes to determine values of dynamic soil properties for use in ground response analysis.

5. Excavation of two pits in the basement of the West Wing to inspect the foundations and extract core samples for testing, to check actual dimensions and elevations, and to sample foundation soils for classification and testing.

6. Measurement of elevations of known points on the first floor of the West Wing and on the parking area under the East Wing for comparison with previous measurements.

Inspection of West Wing Basement

A walk-through inspection of the West Wing basement was made on June 13, 1974, in the company of the Building Manager. This inspection included practically every room in the basement as well as the narrow (±3') space separating the two wings. During the inspection, attention was focused on qualitatively appraising the condition of exposed masonry walls and searching for evidence indicative of poor foundation behavior. Unfortunately, the concrete strip footings supporting the massive bearing walls were well below basement floor level and hence could not be inspected.

The basement walls showed no evidence of watermarks or accumulation of water although some deterioration of mortar between bricks was detected at several locations. Outer portions of mortar in some instances could be dislodged by thumbnail and inner portions by penknife. There were no visible continuous wall cracks in the areas inspected and only a few isolated bricks were cracked. Most of the walls along the northern portion of the building were covered with fibre board masking the masonry from view. There were no visible separations of walls, floors, and ceilings at the joints. A peculiar construction feature of inverted arches in some of the walls was noted but no explanation was found.

There was evidence of work for remodeling purposes subsequent to original construction. Original window openings in outside walls had been "bricked in" some years ago when additional fill was placed outside the building. Of a more serious nature, there was a surprising number of instances where large openings had been cut through walls for ducts and conduits; in some cases entire load bearing cross walls in rooms had been removed for space utilization. There were no visible ill effects, however.

Most of the basement room floors were of concrete with tile covering. The floor in one area was of wood. Although much unevenness was apparent in some of the concrete floors, there were no large visible cracks.

Ceilings consisted of brick arches spanning I-beams. Again, some deterioration of mortar was noted except in rooms in the northern portion of the building where hung ceilings had recently been installed blocking the arches from view.

After the inspection, it was decided that a test pit should be excavated in one of the basement rooms to expose the foundations for detailed inspection and core sampling. A room at the south-west corner of the building was selected and a pit excavated (Pit No. 1, to be discussed later). Unsatisfactory core sample recovery plus the need for further foundation inspection prompted consideration of another pit. Consequently, another walk-through inspection was made, this time concentrating on the northern portion of the basement, to select another room in which to excavate. During this inspection, a chance discovery was made of a continuous crack in the north wall, extending from above the hung ceiling to below the floor. This location was selected for Test Pit No. 2.

Test Pit No. 1

Test Pit No. 1 was excavated during July 19-24 in the north-west corner of the southwest room (Room 8) of the basement.

The location relative to the entire West Wing is shown in Figure E1 and the pit layout relative to Room 8 is shown in Figure E2. Locating the pit in the corner of the room permitted portions of two different foundations to be exposed by the excavation. The completed pit was eight feet square and eight feet deep.

Excavation was fairly rapid considering all digging was done by hand. The tile-covered plain concrete floor slab, varying in thickness from two to four inches, was easily fractured with hand picks and presented no problems in removing the required eight-foot square area. Soil was removed to a depth of about five feet at which point timber sheeting was installed and subsequently advanced with the excavation. Vertical cuts in the soil could be made prior to installing the sheeting. As soil was removed from the pit it was stored over the remaining floor area of the same room for later use in backfilling.

To a depth of about four feet, the soil consisted of a loose moist brown clayey silt which afforded easy digging. From the top of the concrete footing to the bottom of the excavation, the soil was a firm to hard moist brown clayey silt which presented more difficult digging.

Upon completion of the excavation, the foundation system was found to agree very well with the original contractor's drawings. Figures E3 and E4 show sections through the exposed foundations and compare original planned dimensions and geometry with actual measurements. As can be seen, discrepancies are of a minor nature.

Individual bricks (red, measuring 4" x 2 3/4" x 7 1/2") comprising the walls down to the concrete footings appeared to be in sound condition with no visible cracks, spalling, or apparent

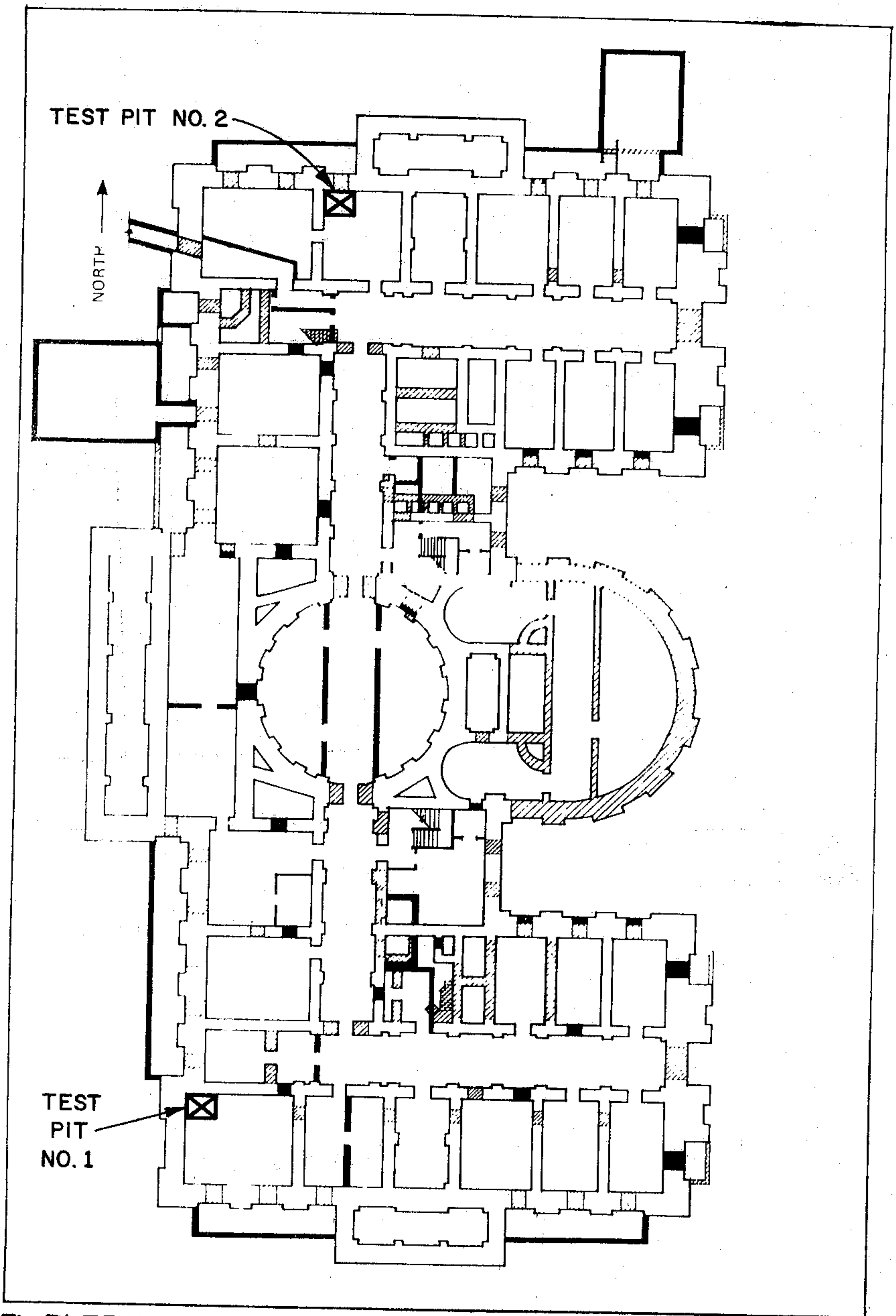



Fig.E1 TEST PIT LOCATIONS IN BASEMENT OF WEST WING

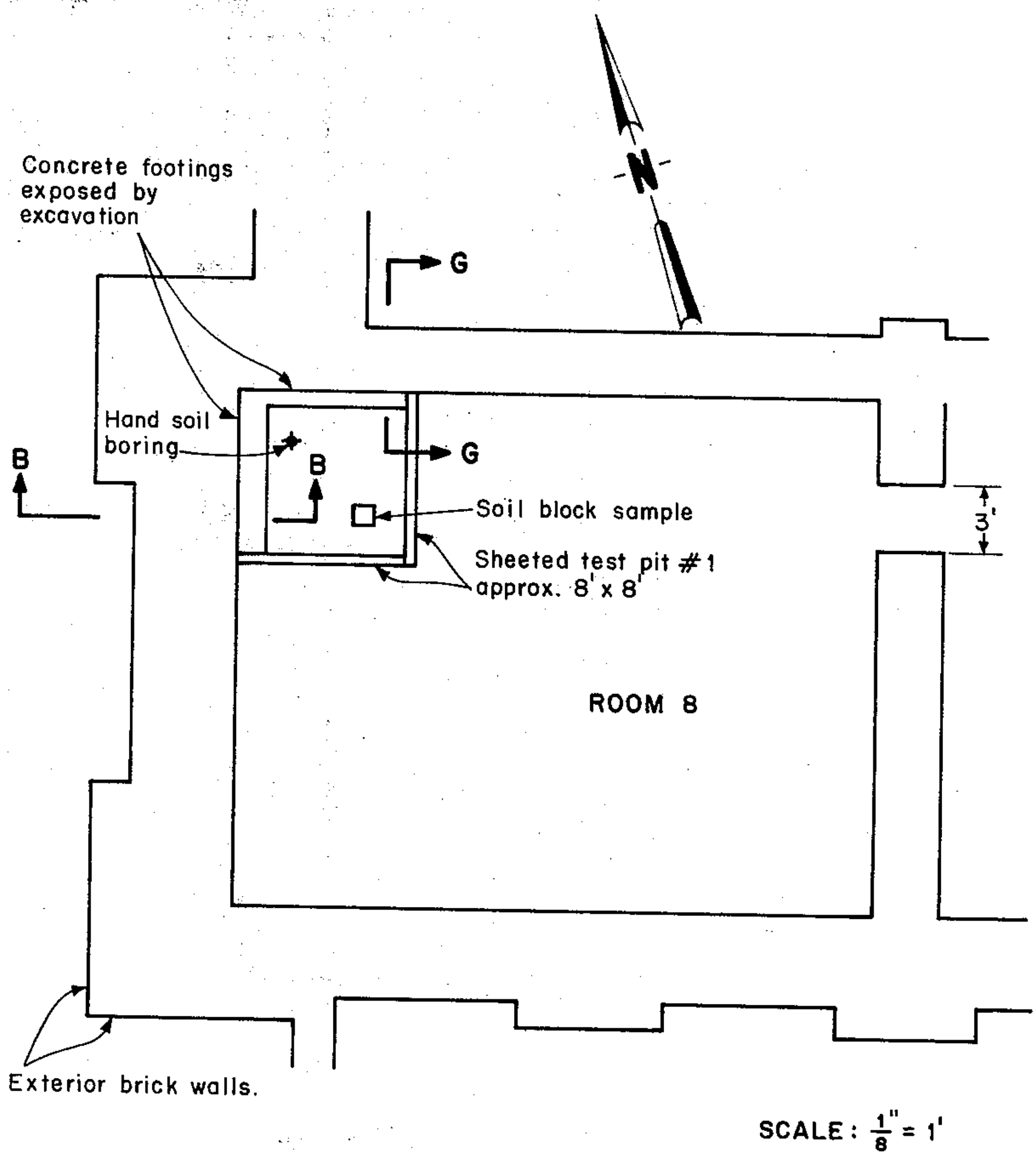
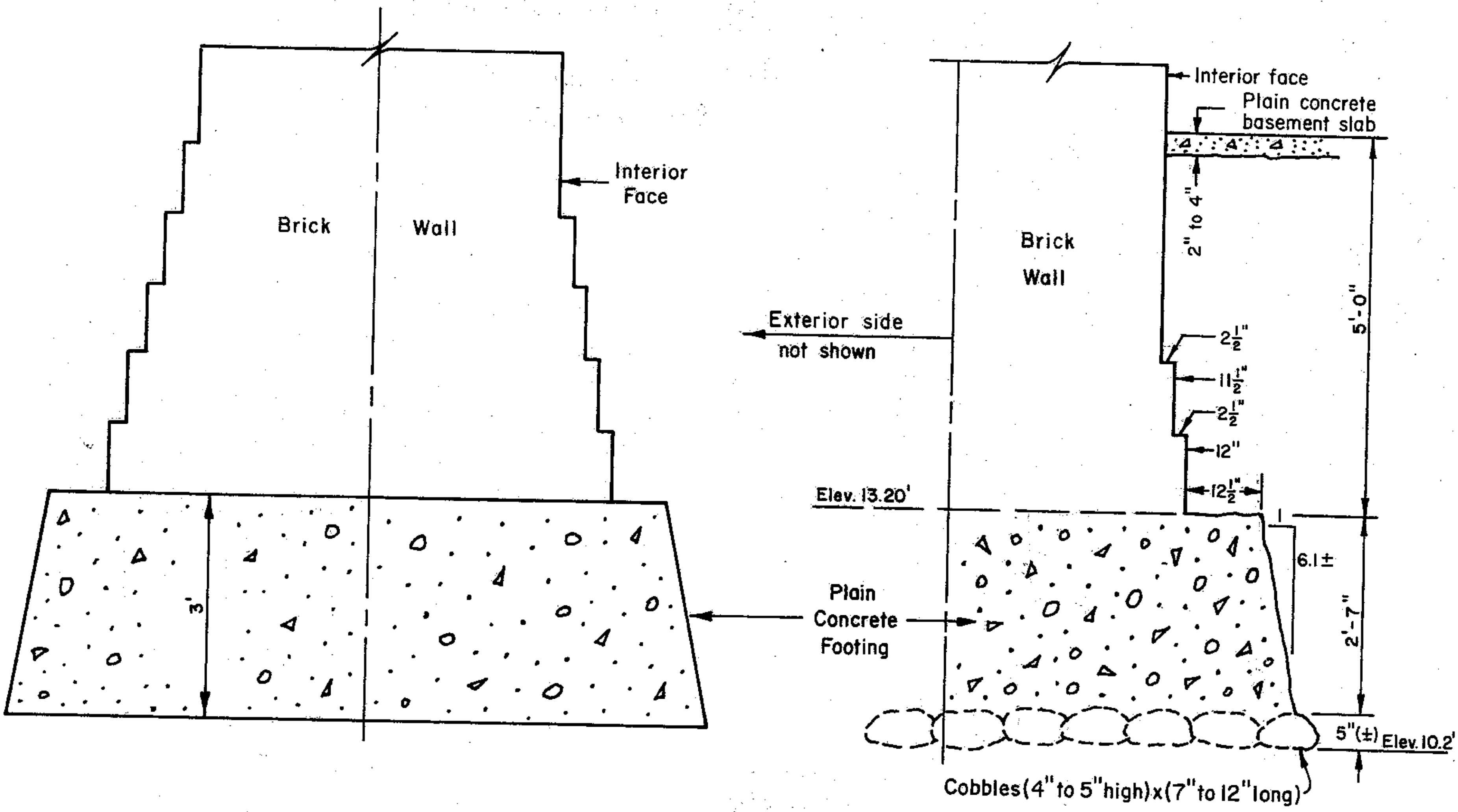



FIG. E2 TEST PIT NO. 1 - WEST WING BASEMENT

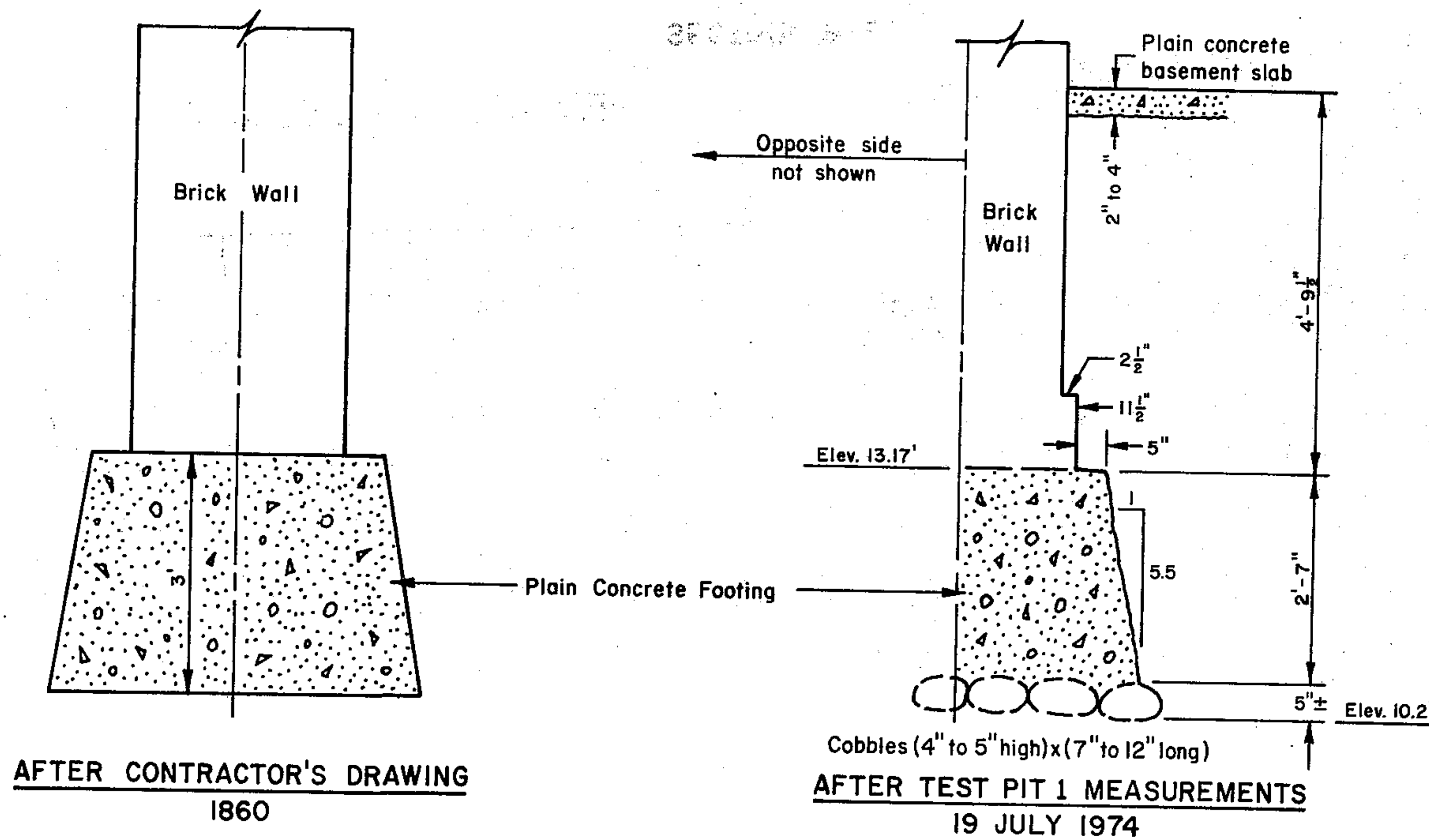



AFTER CONTRACTOR'S DRAWING
1860

AFTER TEST PIT 1 MEASUREMENTS
19 JULY 1974

SECTION B-B
1/2" = 1 ft.

FIG. E3 FOUNDATION WALL AND FOOTING DIMENSIONS
WEST EXTERIOR WALL OF WEST WING




**SECTION G-G**
1/2"=1 ft.

FIG. E4 FOUNDATION WALL AND FOOTING DIMENSIONS
NORTH WALL OF ROOM 8, WEST WING

serious deterioration. Only a few of the bricks exhibited minor chips. Although the lower portions of the walls were moist from soil water, there was no visible evidence of wall deterioration. Figure E5 shows the typical appearance of foundation walls. The mortar between the bricks (dirty white, 3/8" thick) had undergone some deterioration as evidenced by fairly large void spaces between several bricks and some hairline cracks through the mortar. At several random locations the mortar was easily spalled into powder when lightly rubbed with a blunt tool. Intact pieces of mortar removed with little effort were brittle and could be broken by hand.

The exposed concrete footings are shown in Figure E6 with more detailed views in Figures E7 and E8. The lines of demarcation between the three one-foot layers of concrete were clearly visible during the inspection of the footings and verified reported construction methods. Each of the exposed footings contained about a one-foot square area of marked deterioration from which granite chips (1" to 2") and pebbles (1" to 2 1/2") were easily removed with fingers. The deterioration appeared to be in the cement mixture surrounding the aggregate. In one area, the surrounding soil had penetrated around the aggregate and into the concrete face to a depth in excess of two inches; the area was easily penetrated with fingers. There was widespread evidence of segregation of the concrete mix during construction; the larger and heavier aggregate were found more frequently in the lower portions of the one-foot layers. The larger aggregate consisted of two distinct types of rock; a granite which was prone to severe weathering, and a denser, more competent material.

The layer of cobbles underlying the concrete footings were oval shaped, had smooth surfaces, and ranged in size from four to five inches high and seven to 12 inches long. They were not cemented together with mortar and contained no apparent fractures.

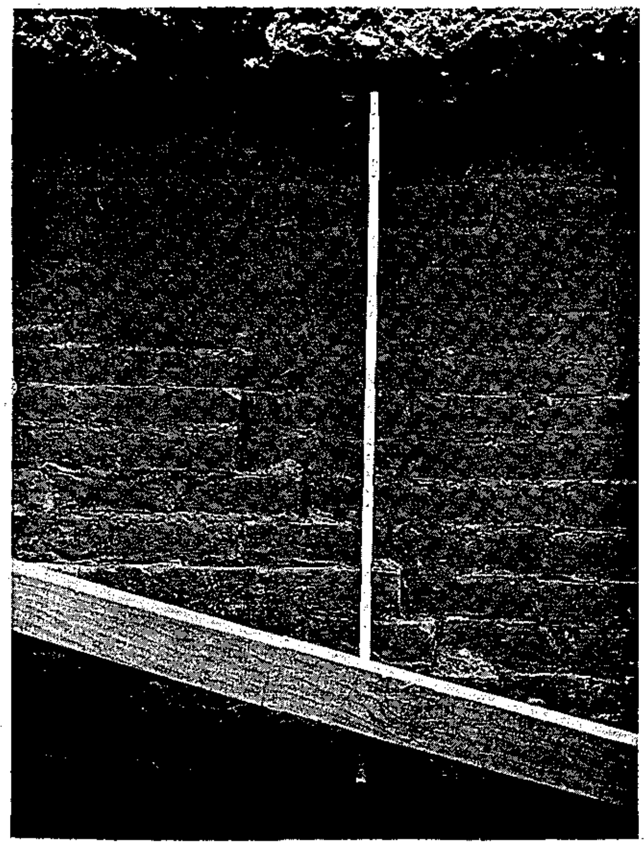


Figure E5 Exposed brick foundation wall, Pit No. 1.

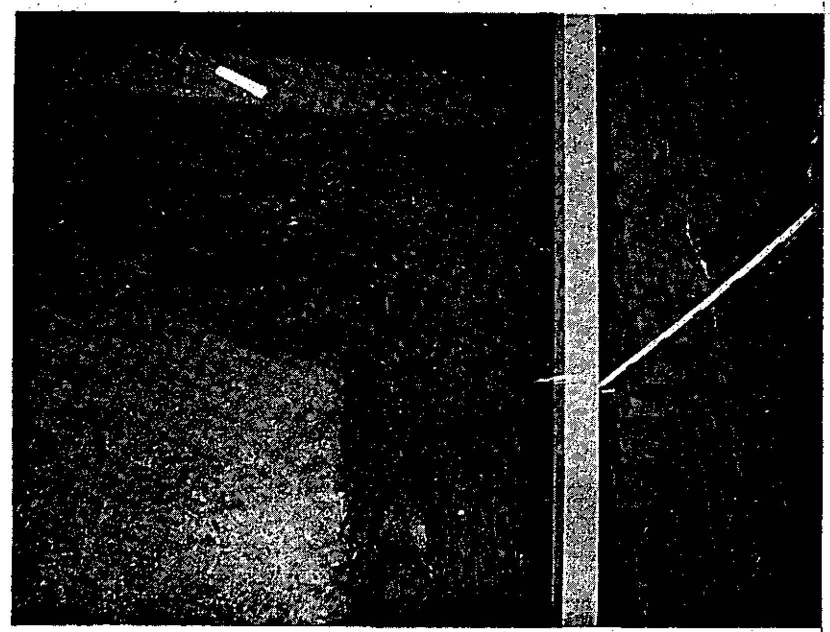


Figure E6 Concrete footings, Pit No. 1 (hammer on west wall).

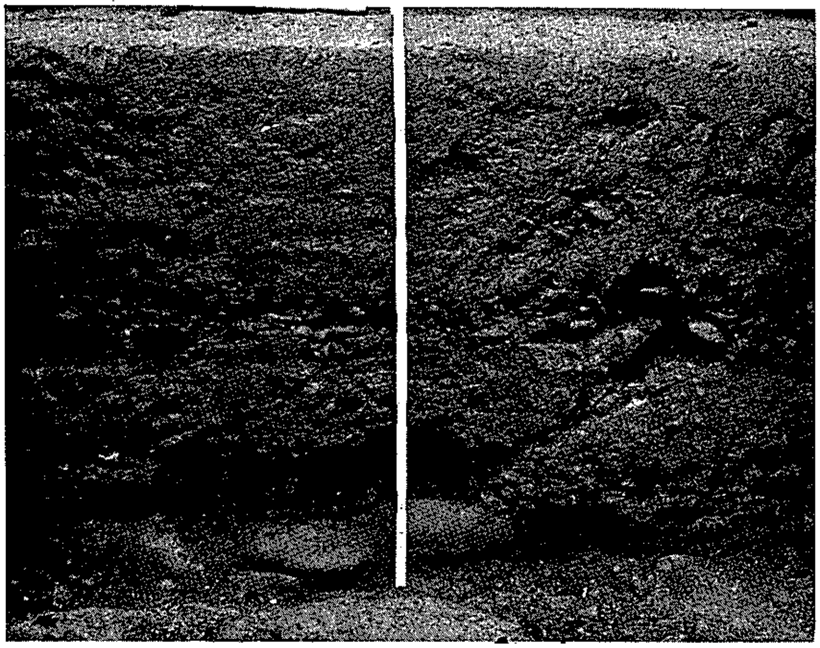


Figure E7 Concrete footing under west wall.

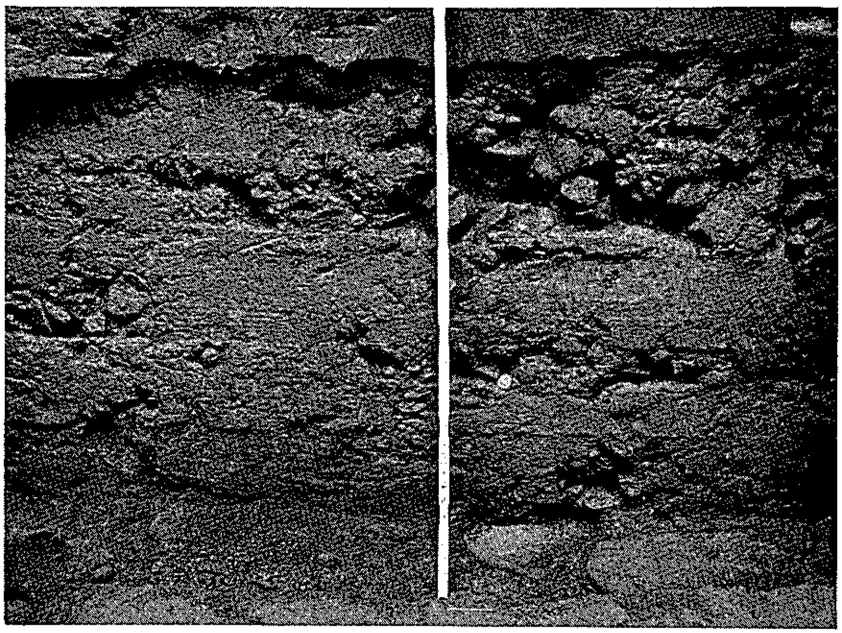


Figure E8 Concrete footing under north wall of Room 8.

Attempts to core the concrete footings met with limited success although two short 8-inch diameter samples were extracted. The problems encountered were created by the poor quality of the concrete as anchor bolts placed in the concrete to secure the coring rig would pull out under small loads. Also weakly cemented aggregate became dislodged in large quantity making it necessary to try another location. The two cores that were obtained are shown in Figure E9. As can be seen, large voids are present, especially in the core from the west wall footing. Apparently, the voids are the result of poor placement methods that allowed segregation of aggregate and cement to occur.

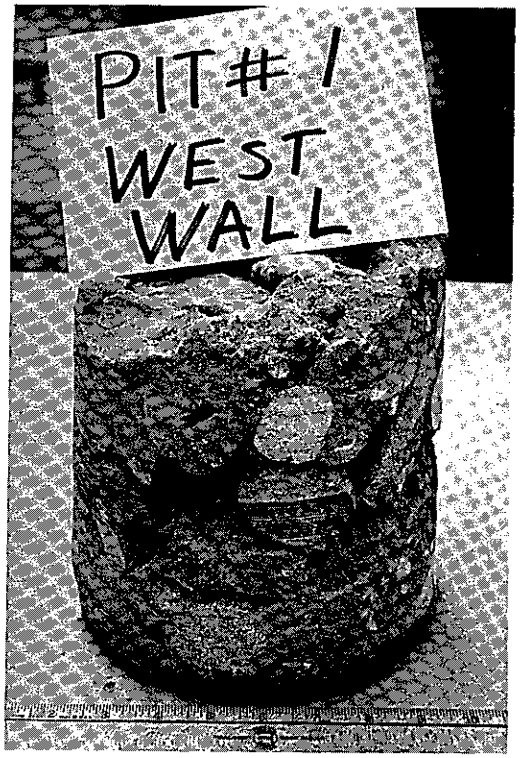
One intact brick assembly section was removed from each of the two exposed walls by the method shown in Figure E10. During transportation to the laboratory, the brick assembly section from the north wall separated at the mortar joint.

A California type hand sampler was used to sample the soil to a depth of 12 feet below the foundation level. The soil was primarily a firm moist brown clayey silt as found in the lower portion of the excavation. Also, a one-foot cube block sample, Figure E11, was cut from the pit floor for later testing.

After all inspection and sampling operations were complete, the voids from the removed cores and bricks were filled with dry pack mortar. The entire pit was then backfilled and compacted using the stockpiled excavated material, and the concrete floor slab was repaired.

## Test Pit No. 2

Test Pit No. 2 was excavated during July 31-August 6 in the northwest corner of the room (Room 18) just east of the north-



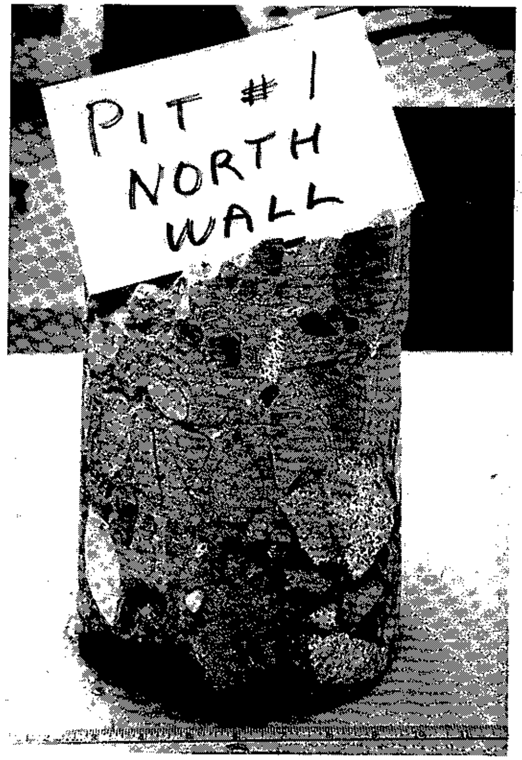




Figure E9 Cores from concrete footings, Pit No. 1.

Figure E10 Removal of brick sample, Pit No. 1.

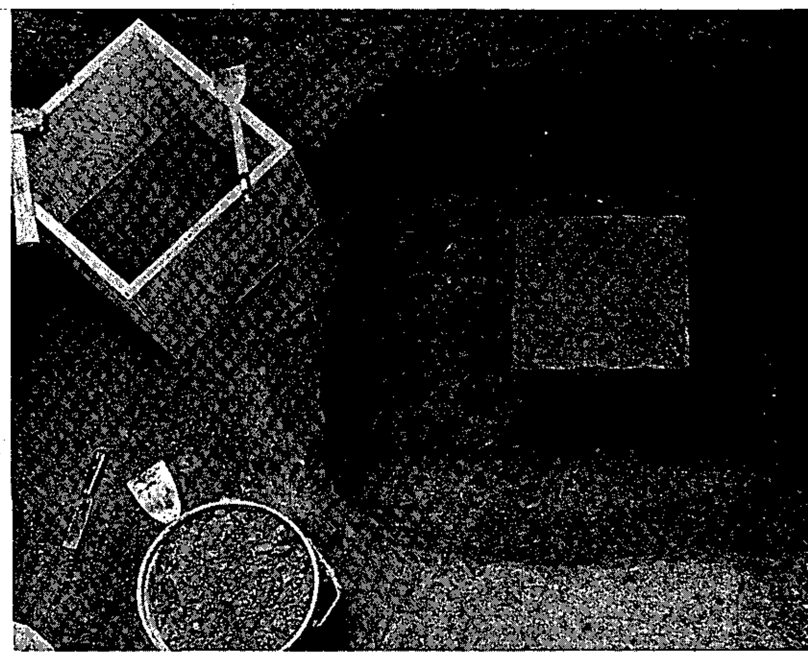


Figure E11 Hand carved block soil sample in bottom of Pit No. 1.

west corner room of the basement. The location is shown with respect to the entire West Wing in Figure E1 and the pit layout relative to the room is shown in Figure E12.

Excavation was conducted in an identical manner as for Pit No. 1. Again, no problem was encountered in breaking through the concrete slab basement floor and excavating the eight-foot square by eight-foot deep pit. The first foot of soil beneath the basement slab consisted of a silty sand with some bricks, brick fragments, and mortar. This was followed by a one-foot layer of medium brown clayey silt. At this depth a hard dark bluish plastic clay was encountered and extended to a depth of 5.5 feet. The remaining 2.5 feet, to the bottom of the pit, consisted of a hard dark brown clayey silt.

As was the case with Pit No. 1, completion of the excavation revealed a foundation system which agreed quite well with the original contractor's drawings. Figures E13 and E14 show cross-sections of the two foundation walls and their dimensions as measured following excavation. Of major interest, and the primary reason for locating the second pit at this location, was a long vertical crack in the north wall extending from above the suspended ceiling of the room down and into the concrete footing. Figure E15 illustrates the position of the crack with respect to the foundation walls and Figures E16 and E17 show photographs of the crack. During excavation the work crew reported that the crack was visible in the hard clay layer and continued across the entire width of the pit. It was subsequently mapped as shown in Figure E18. Apparently, the crack is one of several which occurred during construction of the Capitol as discussed in detail in Appendix C. Other than the crack, no apparent spalling or deterioration of the bricks beneath the basement slab was evident. Some breakdown in

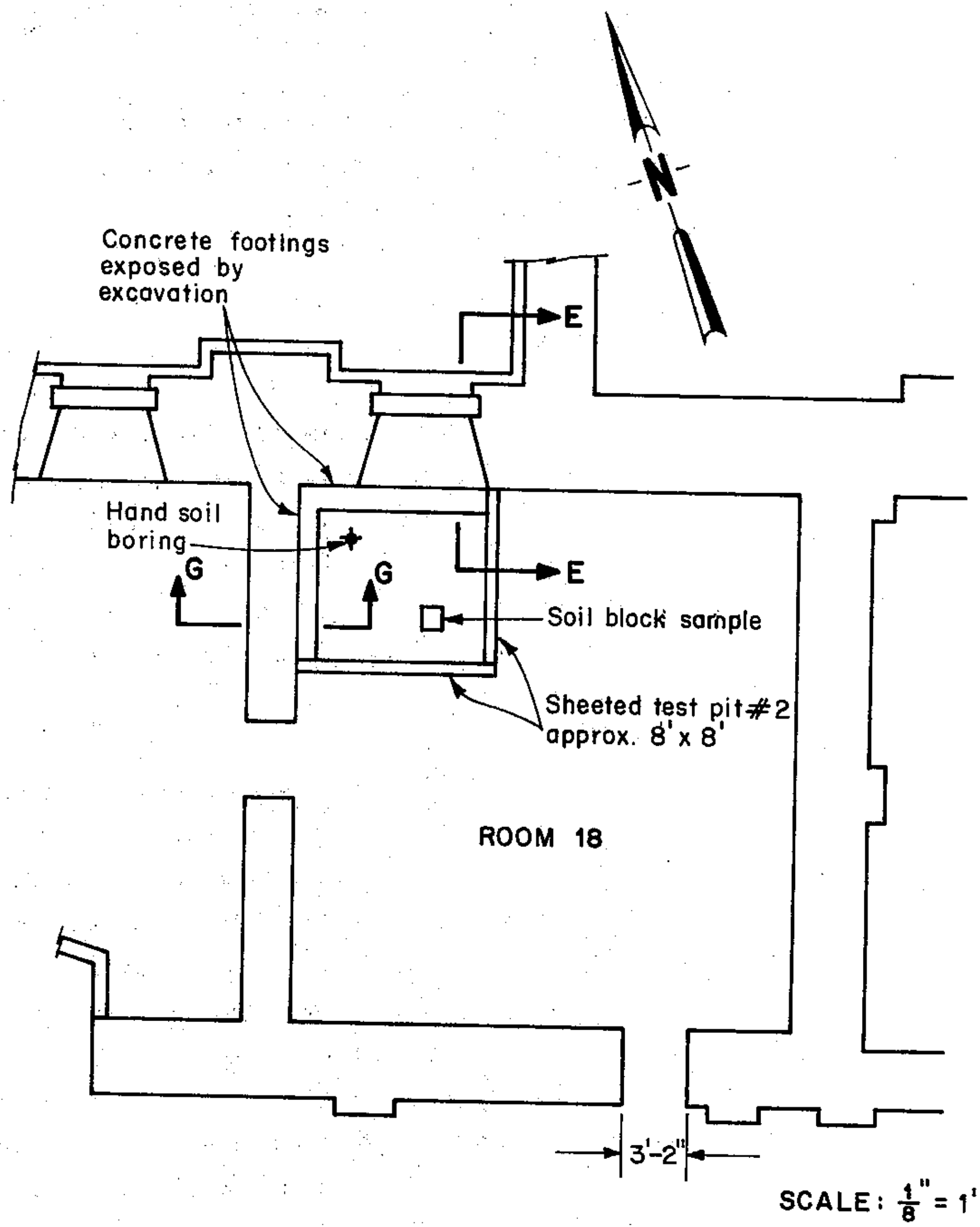
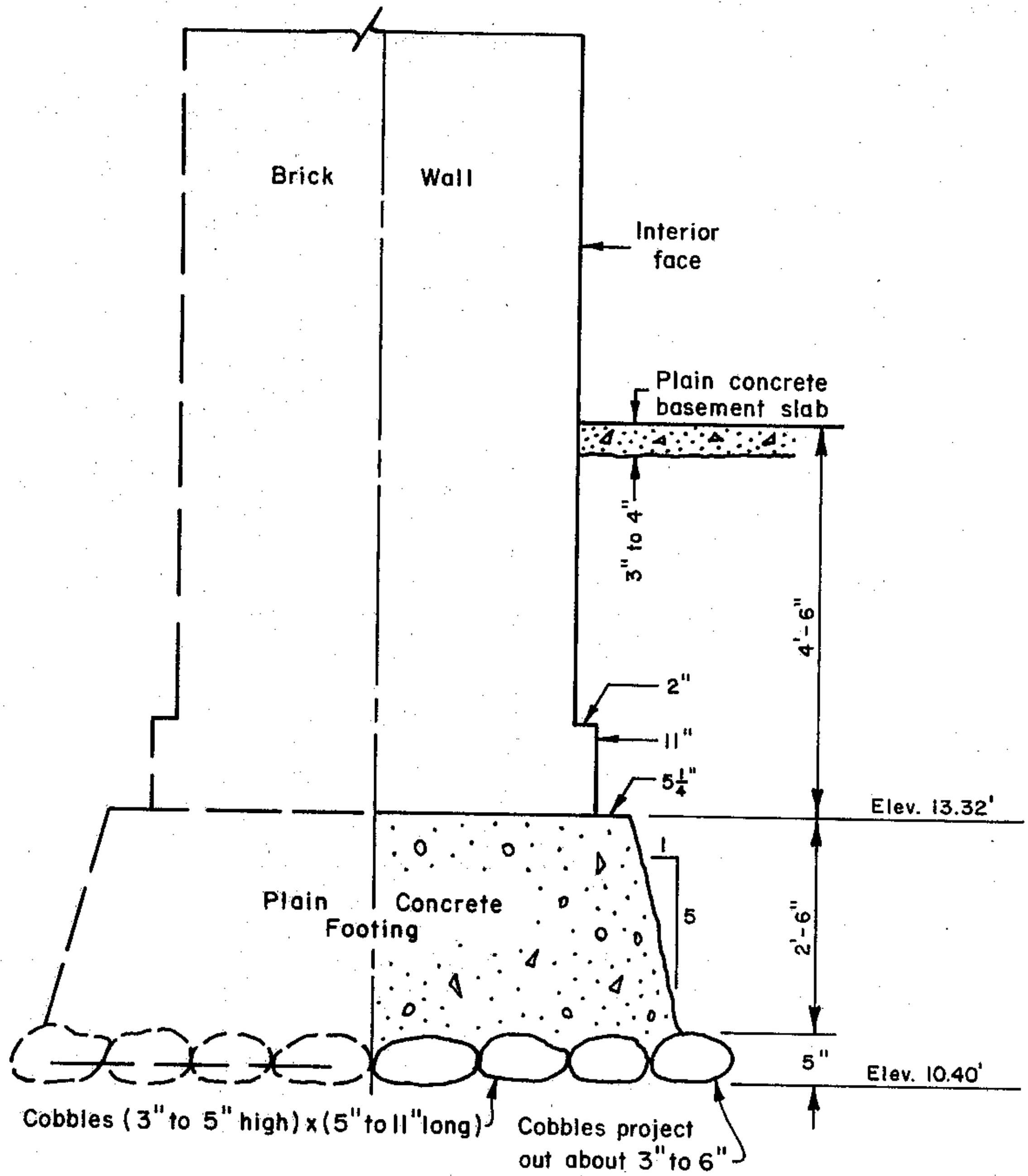
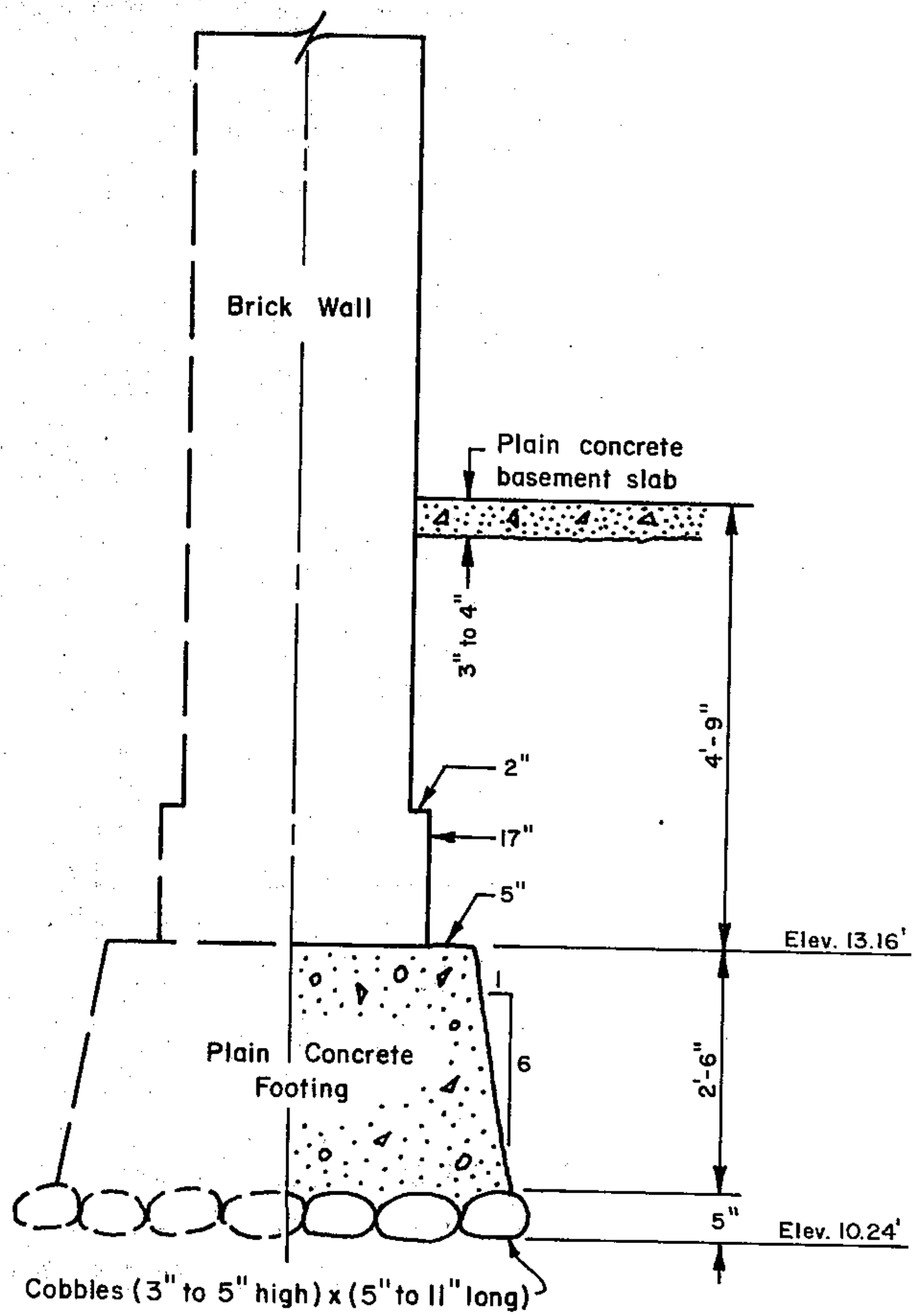



FIG.E12 TEST PIT NO. 2 - WEST WING BASEMENT




SECTION E-E, TEST PIT NO. 2
1/2" = 1 ft.

FIG. F13 FOUNDATION WALL AND FOOTING DIMENSIONS
NORTH EXTERIOR WALL OF WEST WING




SECTION G-G, TEST PIT NO. 2

1/2" = 1 ft.

FIG. E14 FOUNDATION WALL AND FOOTING DIMENSIONS WEST WALL OF ROOM 18, WEST WING

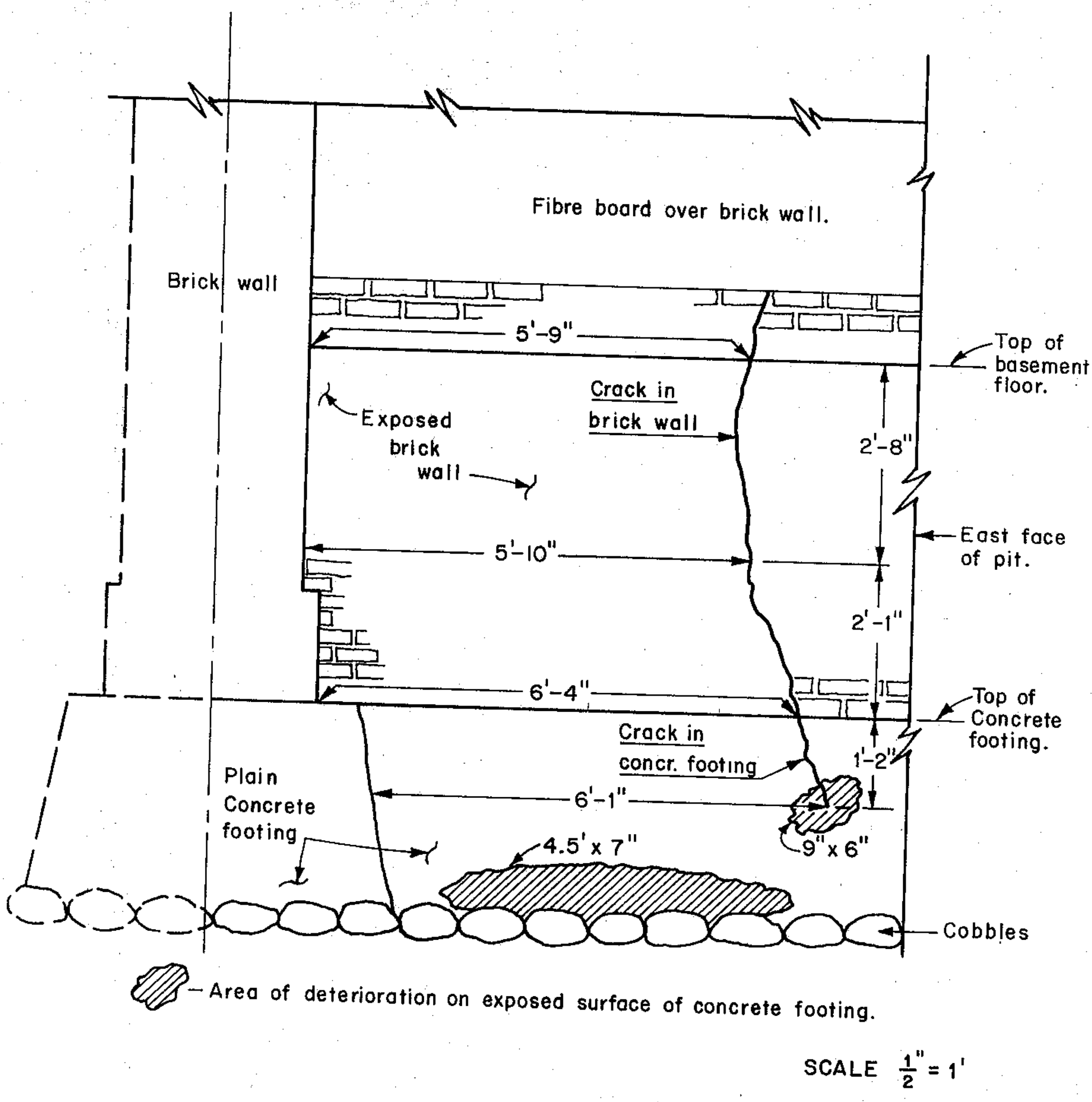



FIG. E15 LOOKING NORTH SHOWING CRACK IN NORTH EXTERIOR WALL AND FOOTING, PIT NO. 2

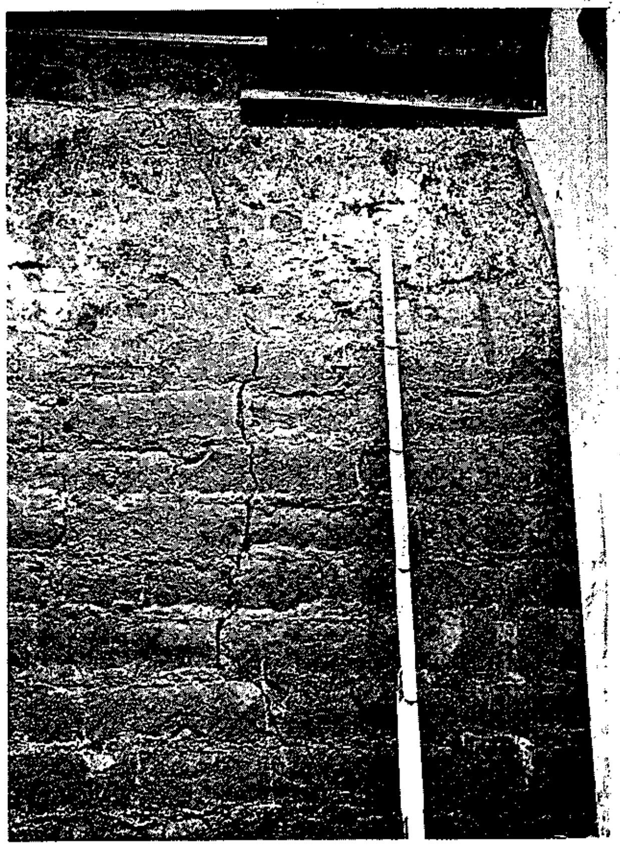


Figure E16 Crack in brick foundation wall, Pit No. 2.

Figure E17 Continuation of wall crack into footing, Pit No. 2.

mortar between bricks was found. In some spots mortar was easily spalled into powder form with a pen knife and removed pieces of intact mortar were brittle and could be broken by hand. The lower portions of the brick walls were moist from contact with moist soil but no well defined water marks could be found. The condition of the concrete footings underlying the north and west walls appeared somewhat deteriorated as may be noted from Figures E19 and E20. Several areas of serious cement paste breakdown were found in both footings, where aggregates and granite chips could be easily removed with fingers. The lines of demarcation between three one-foot layers of concrete comprising the footings were clearly visible as "cold construction joints".

The layer of cobbles underlying the concrete footings was comprised of smooth oval shaped stones varying in size from three to five inches high and five to eleven inches long. One cobble was found to be split along its side. There was no evidence of embedment of the cobbles in mortar.

Attempts at coring the concrete footing met with much the same problems as encountered at Pit No. 1. Poor quality mortar made it impossible to anchor coring equipment and allowed weakly cemented aggregate to become dislodged, ruining the sample for testing purposes. Two cores were eventually retrieved, however, one of which is shown in Figure E21.

Three assemblies of brick and mortar from the north and west walls below the basement slab were removed intact and transported to the laboratory for testing. The two samples from the north wall are shown in Figure E22.

Soils to a depth of 12 ft. below the foundation level were sampled using the California type hand sampler. The soil was

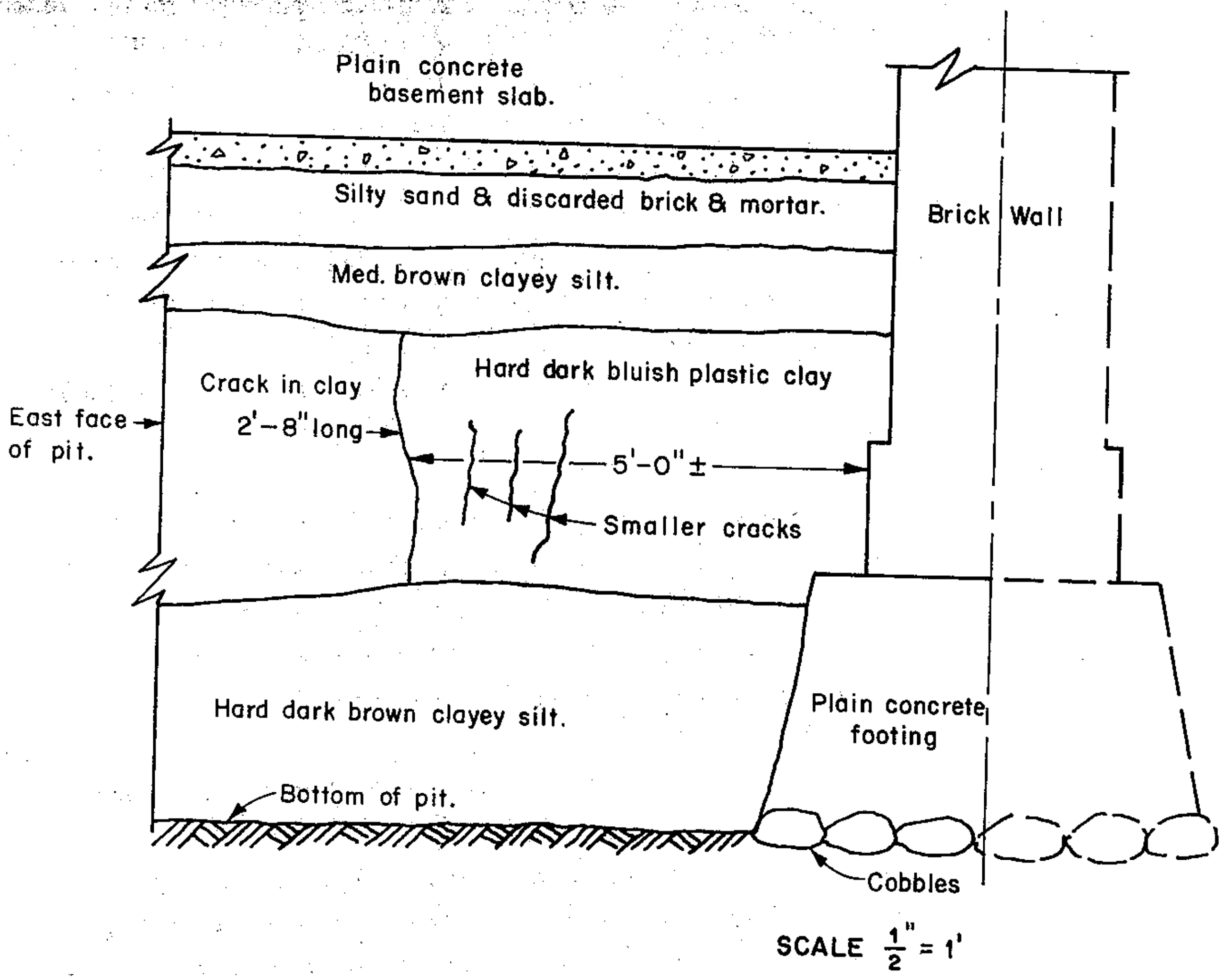



FIG. E18 LOOKING SOUTH SHOWING CRACK IN CLAY STRATUM, PIT NO. 2

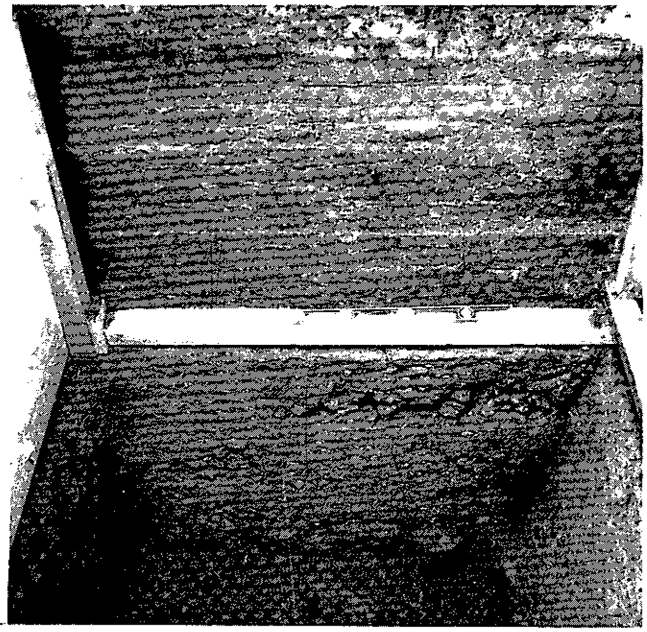


Figure E19 Exposed foundation wall and footing, Room 18 west wall, Pit No. 2.

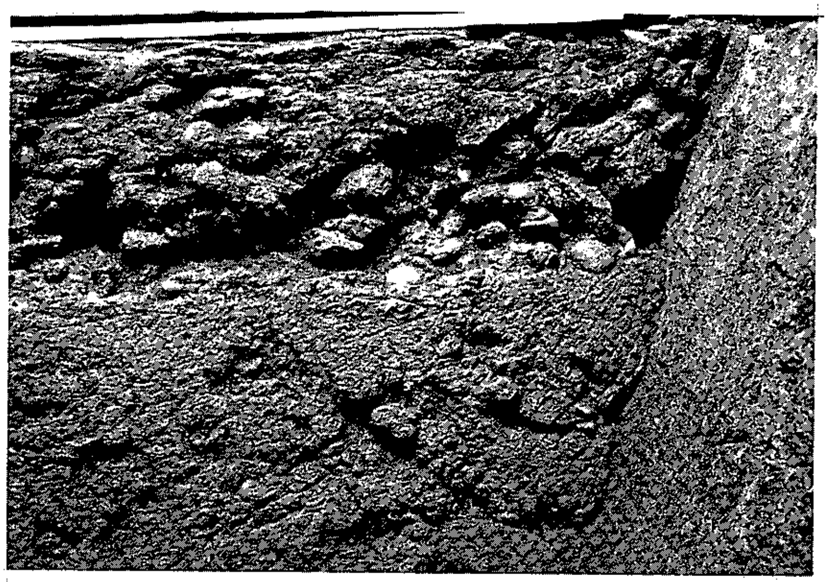


Figure E20 Concrete footing under Room 18 west wall, Pit No. 2.


Figure E21 Core from concrete footing under Room 18 west wall, Pit No. 2.

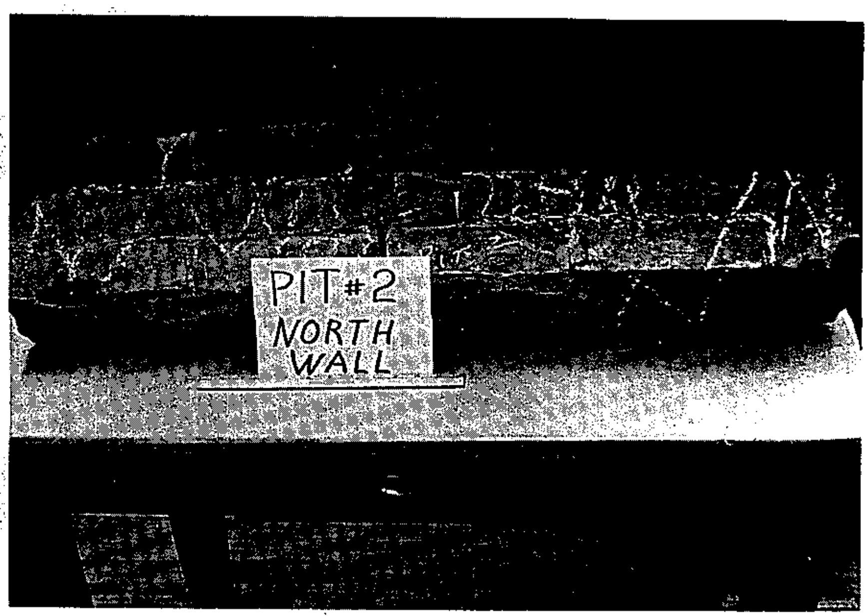



Figure E22 Samples from north brick foundation wall, Room 18, Pit No. 2.

found to be a silty clay, very similar to that encountered in the lower portion of the test pit. A one-foot cube block sample, Figure E23, was cut from the pit floor for later testing just prior to backfilling.

## West Wing Settlement Data

Elevation readings were made on July 25, 1974, on six reference points on the West Wing first floor. These points, located along the interior east wall under the dome, were read initially in October, 1951, in connection with construction of the East Wing. The maximum settlement during the 23-year period was about 1/2 inch, but was not considered appreciable considering instrumental and observational errors in reading levels.

During a general inspection of the building exterior to check for visible signs of sag or tilt in the structure, additional levels were read at random locations along ledges, steps, porticos and similar "level" surfaces. The results showed no sag or tilt, except for the north portico where a series of readings showed the portico and steps to slope downward to the west by about 1/2 inch. Although the measurement itself is of no great significance, the observation is consistent with level readings taken March 5, 1868 (see Reference C2), which showed that the north wall of the building sloped downward to the west with the lowest point at the northwest corner.

## Inspection of East Wing Parking Garage

An inspection of the parking garage, the floor of which is about three feet lower than the West Wing basement floor, was made

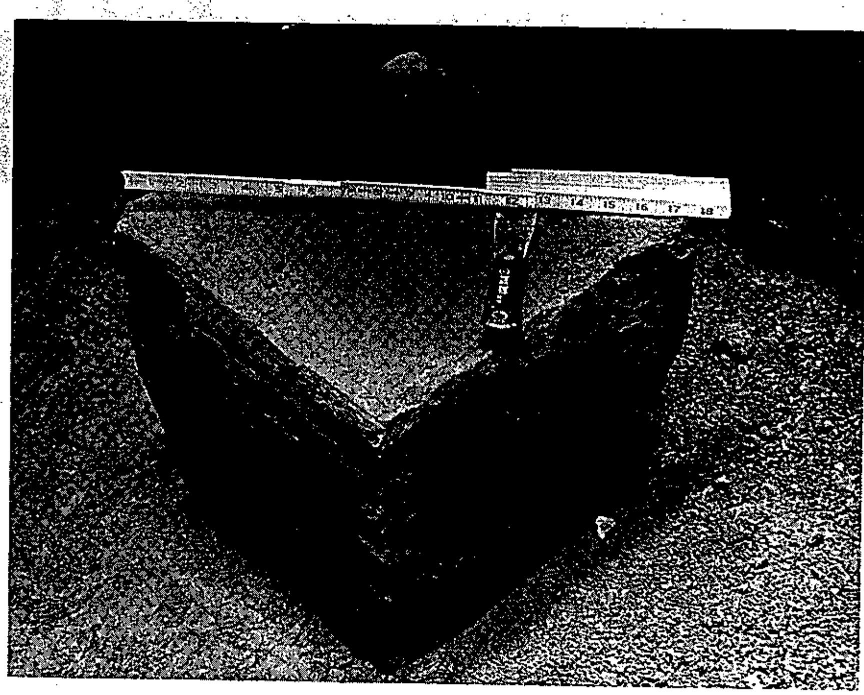


Figure E23    Hand carved block soil sample from bottom of Pit No. 2.

on July 22, 1974. There were no signs of significant distress although two series of wall cracks were observed. One series of five diagonal cracks was noted in the north wall of the transformer room. These cracks extended more or less radially outward from a set of double doors. The west wall of the transformer room contained another set of seven diagonal cracks fairly evenly spaced over the length of the wall.

The above cracks are believed to be the result of local differential settlement caused by the relatively large loads of the transformers. It should be emphasized that the observed cracks were very narrow, or thin, and that the transformer room walls are not a part of the main shear walls for the East Wing.

## East Wing Record Settlement Data

Early during construction of the East Wing, twelve settlement reference points were established in the concrete basement floor of the parking garage. Nine of these points were located around the perimeter of the new structure, one was very near the center, and the remaining two were located about mid-distance between the building center and western most wall. These reference points (chiseled X's in the floor) were used to monitor settlement during construction as loads were gradually applied to the foundation soils. Locations of the points are shown on Figure E24.

Initial elevation readings on the points were made in February, 1950, and seven subsequent sets of readings were made periodically through February, 1953, the last set having been made approximately one year after completion of the structure.

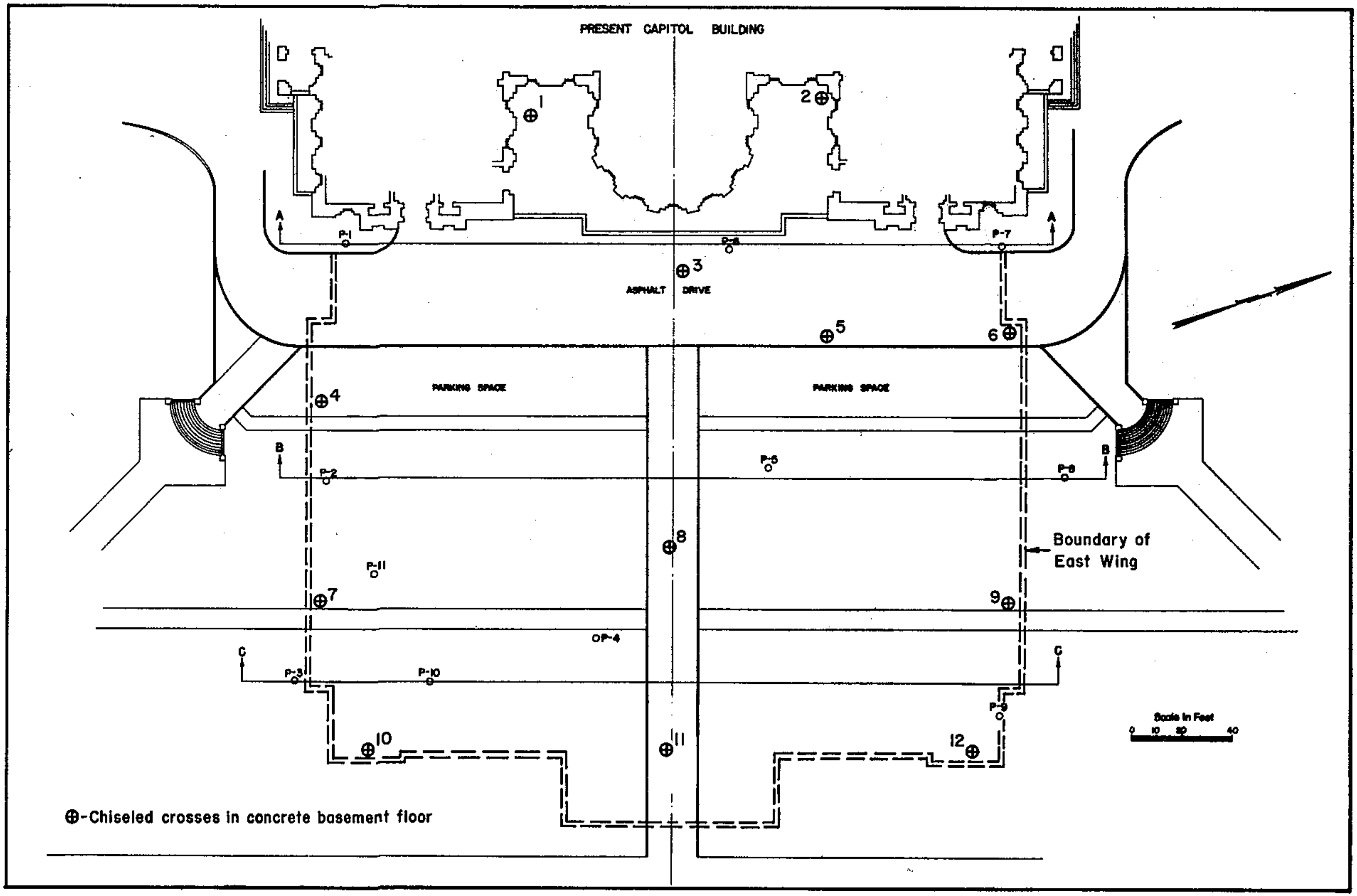



Fig. E-24 EAST WING SETTLEMENT REFERENCE POINTS

The above points were found during this investigation and elevations were read on August 25, 1974, to determine the 22-year settlement pattern of the building. The data are shown in Figures E25 and E26, which contain elevation versus time plots for each of the twelve reference points. All the plots are similar with respect to time-rate of settlement, although plots for Points 3 and 8 are less pronounced. During the first three months very little settlement occurred; it is doubtful that construction had progressed during this three-month period to the point that heavy loads were being applied to the foundation soils. Actually, six of the plots show small gains in elevation, a discrepancy that probably falls within the accuracy of measuring methods. Beyond three months, settlement began to increase and continued beyond completion of the structure. Further review of the plots shows that Points 2, 5, and 6, under the northwest portion of the structure, experienced very similar behavior with regard to both time and total settlement (±1/2"). The same is true for Points 3 and 8 (total settlement = ±0.1") located under the middle portion of the building. It is also noted that Points 7 and 10, located under the southeast corner show similar behavior with total settlements of 0.6" and 1.0", respectively.

No clear-cut relation could be found between total settlement and thickness of compressible stratum. It was generally evident, however, that for reference points above the same thickness of compressible material, distance of point from the West Wing was related to total settlement. This relationship is explained by the combined preloading effects of the adjacent West Wing and the large amount of fill that covered the area now occupied by the western portion of the East Wing.

Eleven differential settlements were checked for the entire recording period. Results were generally between 1/4 and 1/3 inch, although two values of 1/2 and .6 inch were found. Distances between points checked for differential settlement ranged from 50 to 130 feet.

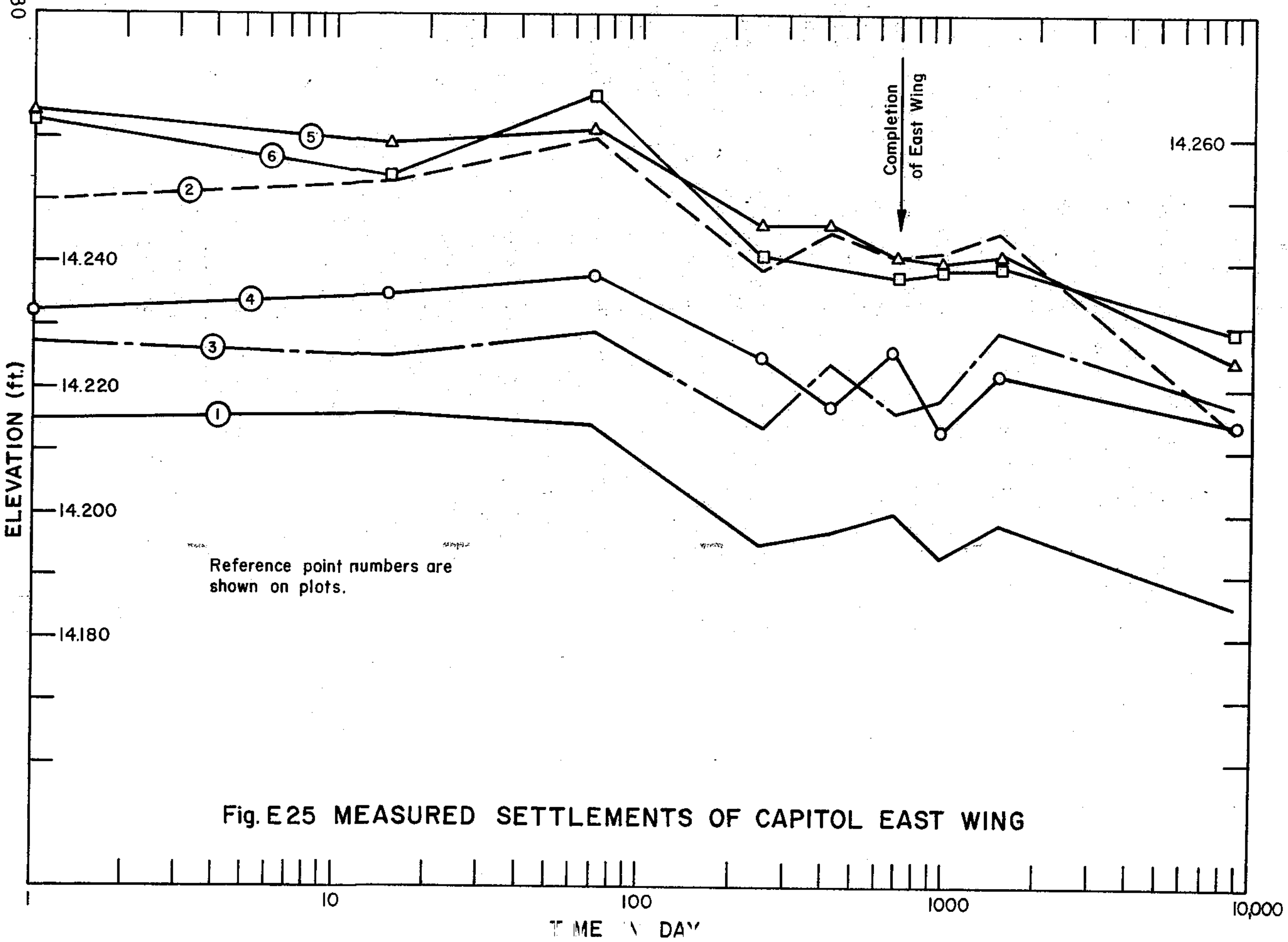



Fig. E25 MEASURED SETTLEMENTS OF CAPITOL EAST WING

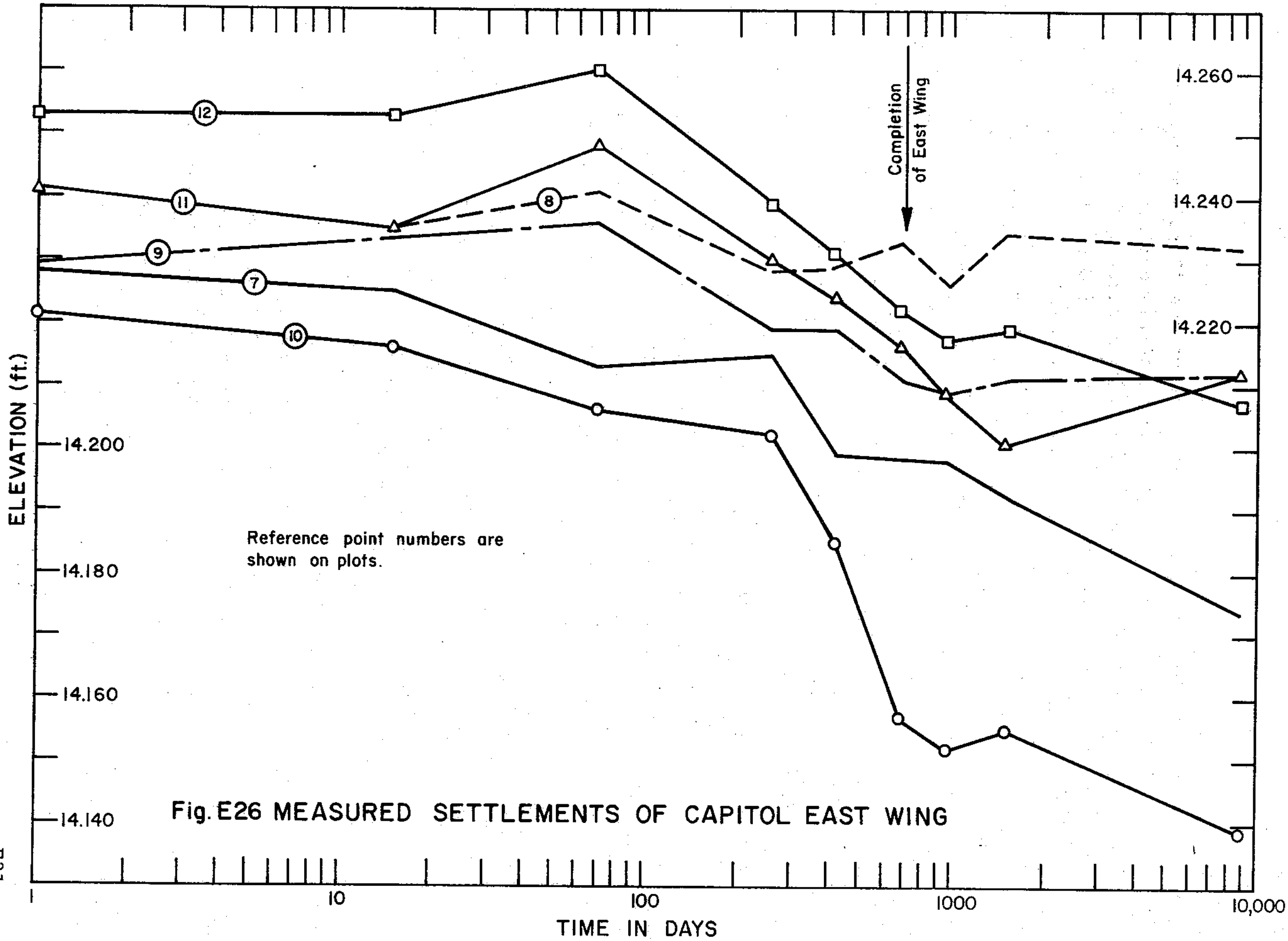



Fig. E26 MEASURED SETTLEMENTS OF CAPITOL EAST WING

It is interesting to note that 10 of the 12 reference points experienced less than 1/4 inch of settlement after the structure was completed. The maximum post-construction settlement was about 1/3 inch. Therefore, a very large portion of the total settlements previously discussed occurred during construction of the building. It was also noted that for those points having the greatest settlements for the entire recording period, about 75 to 85% of the settlement occurred during construction. A check between the same points previously used to determine differential settlement showed a maximum post-construction differential settlement of .3 inch with the majority being less than .2 inches.

## Boring and Sampling

Boring and sampling of subsoils began at the existing site on June 11, 1974, and continued through July 9. Simultaneous drilling operations were conducted with truck-mounted rigs from Districts 02 and 10 and the Transportation Laboratory to meet scheduled deadlines.

Five boring locations around the perimeter of the building were selected as a basic network for delineating soil stratification, performing penetration tests, and obtaining samples for laboratory testing. These borings, designated A1-A5 and shown in plan on Figure E27, ranged in depth from 80 to 220 feet. None of the borings, however, were sampled or logged below a depth of 162 feet. Two additional shallow borings (A1-B, A2-B) were made to retrieve extra samples at selected depths. A special boring, R1, was made near Boring A5. This boring, to a depth of 250 feet, was not logged, since it was made solely for geophysical testing purposes. Samples were obtained with California-type samplers lined with two-by four-inch brass tubes, and with the "standard" split-barrel sampler. Individual boring profiles with summarized laboratory test data are included as Figures E28-E35.

Information on the area subsoils provided by the above series of borings was augmented by data derived from an earlier series of relatively shallow sample borings made in 1948, in connection with construction of the East Wing of the existing structure. The 1948 series, 11 in number, ranged from 35 to 65 feet in depth and represent the foundation soils beneath the structure, or immediately adjacent thereto. Locations and profiles of the 1948 borings are shown on Figures D1 and D2. Information was combined from all the above borings to prepare generalized soil profiles shown in Figures E36 and E37.

Analysis of the various boring soil profiles indicates that, with some sandy and gravelly exceptions, the area soils are principally silts to a depth of about 60 feet below ground surface. Below the 60 foot depth are predominantly sandy strata. The deposition of the alluvial soils in this area appears to be lenticular in nature as specific layers seldom extend laterally from one boring location to another.

The entire area is blanketed by lightly compacted silty sand some 5 to 12 feet in thickness. The surface mantle is underlain generally by a thin layer of soft, silty clay changing to firm to stiff, slightly to moderately plastic, clayey silts from 15 to 30 feet in thickness.

Underlying the clayey silts is a sandy gravel stratum which is only partially defined in terms of thickness and lateral extent. This stratum has been encountered at several boring locations

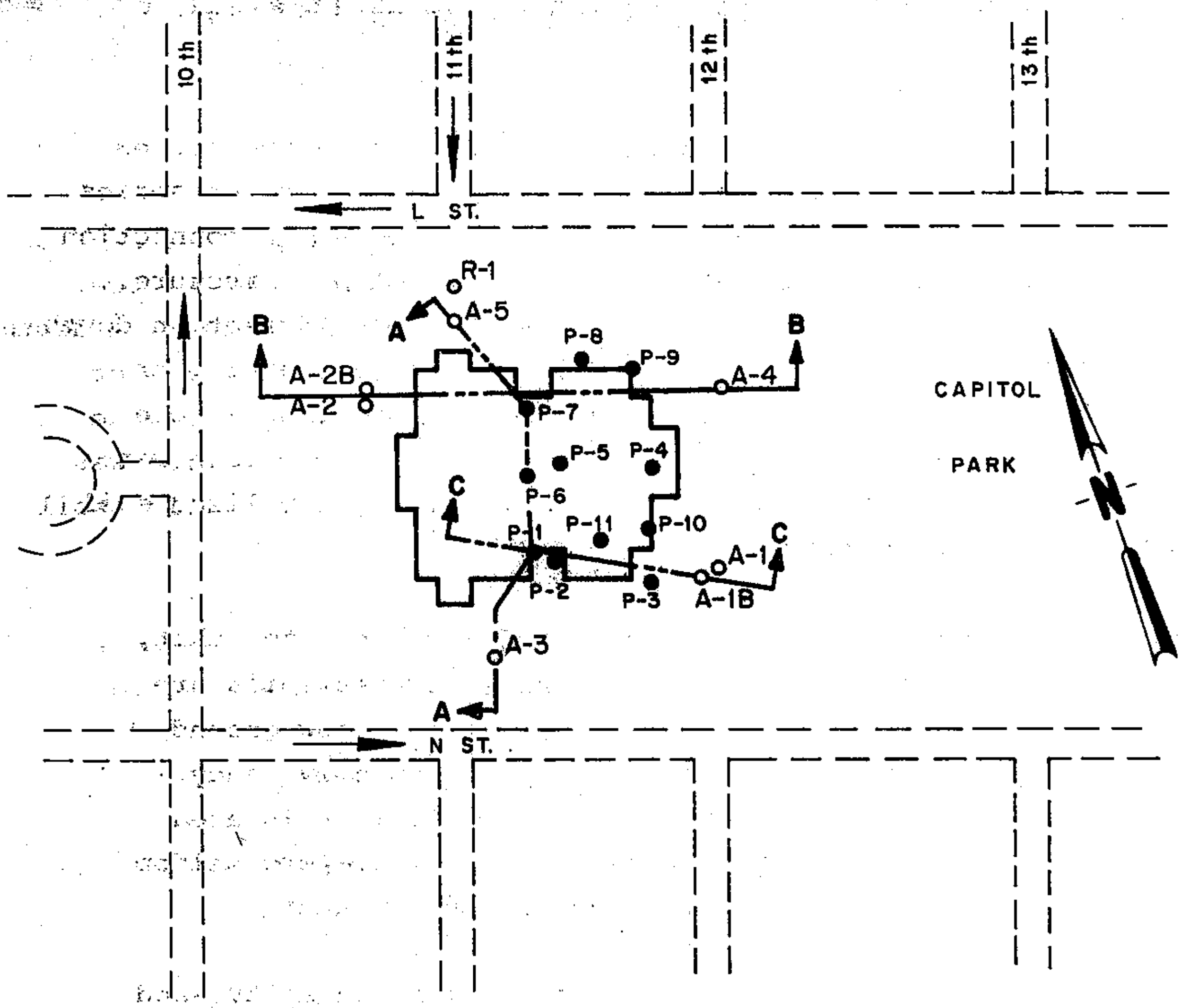
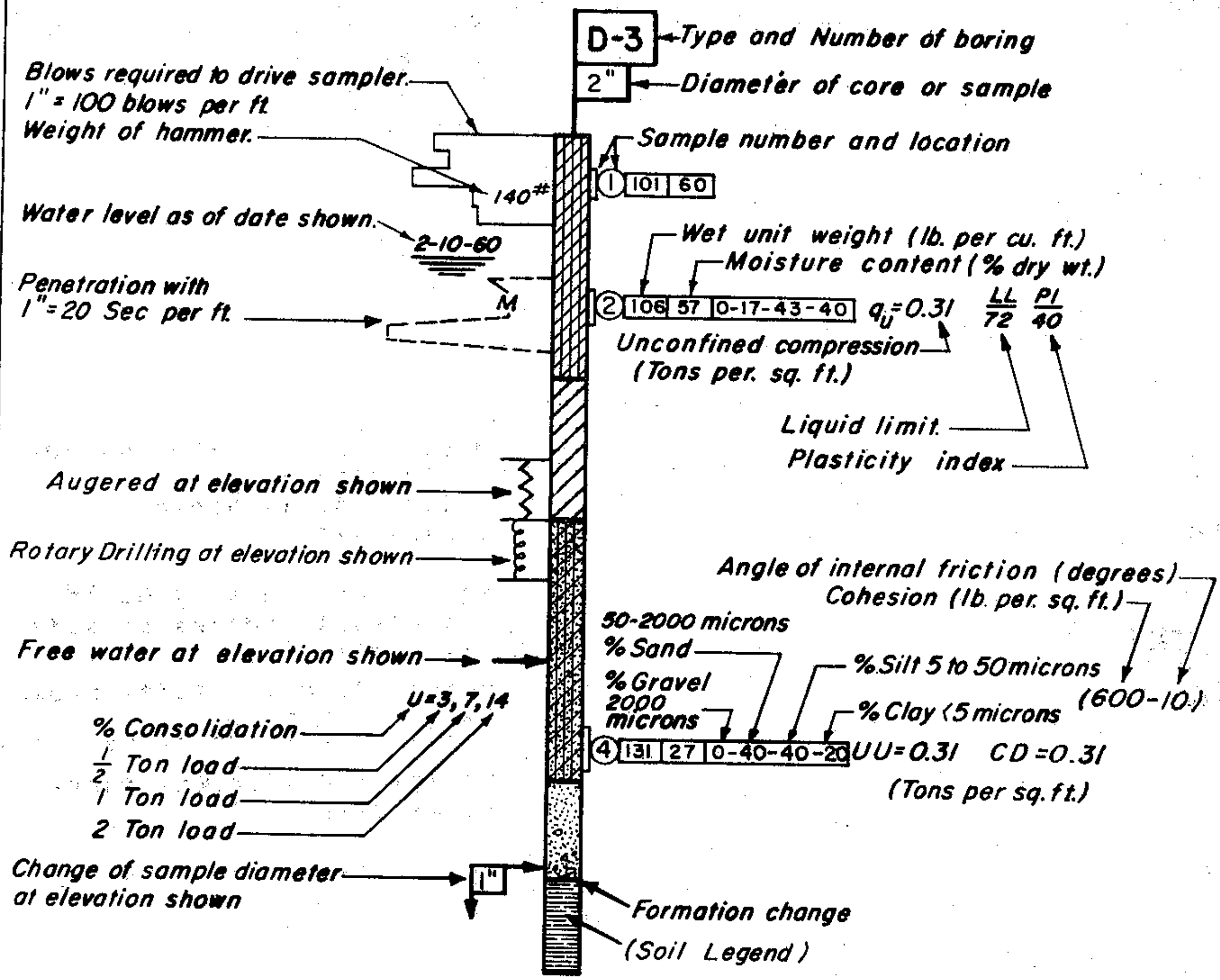



FIG.E27 PLAN MAP OF EXISTING STATE CAPITOL SHOWING APPROXIMATE LOCATIONS OF BORINGS MADE IN 1948 (•) AND 1974 (○)

# BORING LEGEND
## CROSS-SECTION & PROFILE SHEETS




SPT - Standard Penetration Resistance Test

STRENGTH TESTS

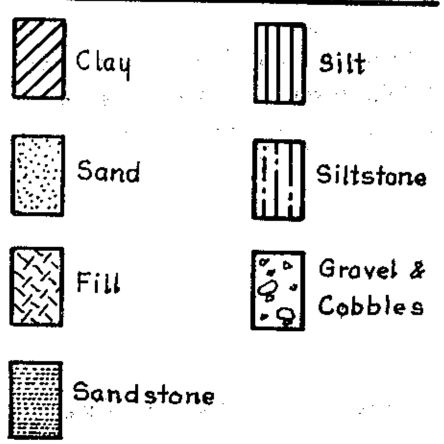
$q_u$ - Unconfined Compression

UU - Unconsolidated Undrained

CU - Consolidated Undrained

CD - Consolidated Drained

### SOIL LEGEND




STATE OF CALIFORNIA — HIGHWAY TRANSPORTATION AGENCY
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
TRANSPORTATION LABORATORY

**Fig. E28 BORING LEGEND**

| DATE | SUBMITTED BY: | DWG. NO. |
|---|---|---|
| | SR. MAT'LS. & RES. ENG'R. | 2695 |
| DR. BY M.I. D.D. / CK. BY | APPROVED BY: SUP'R. MATLS. & RES. ENG'R. - FDN | SHEET NO OF SHTS. |

SE Corner State Capitol Bldg.

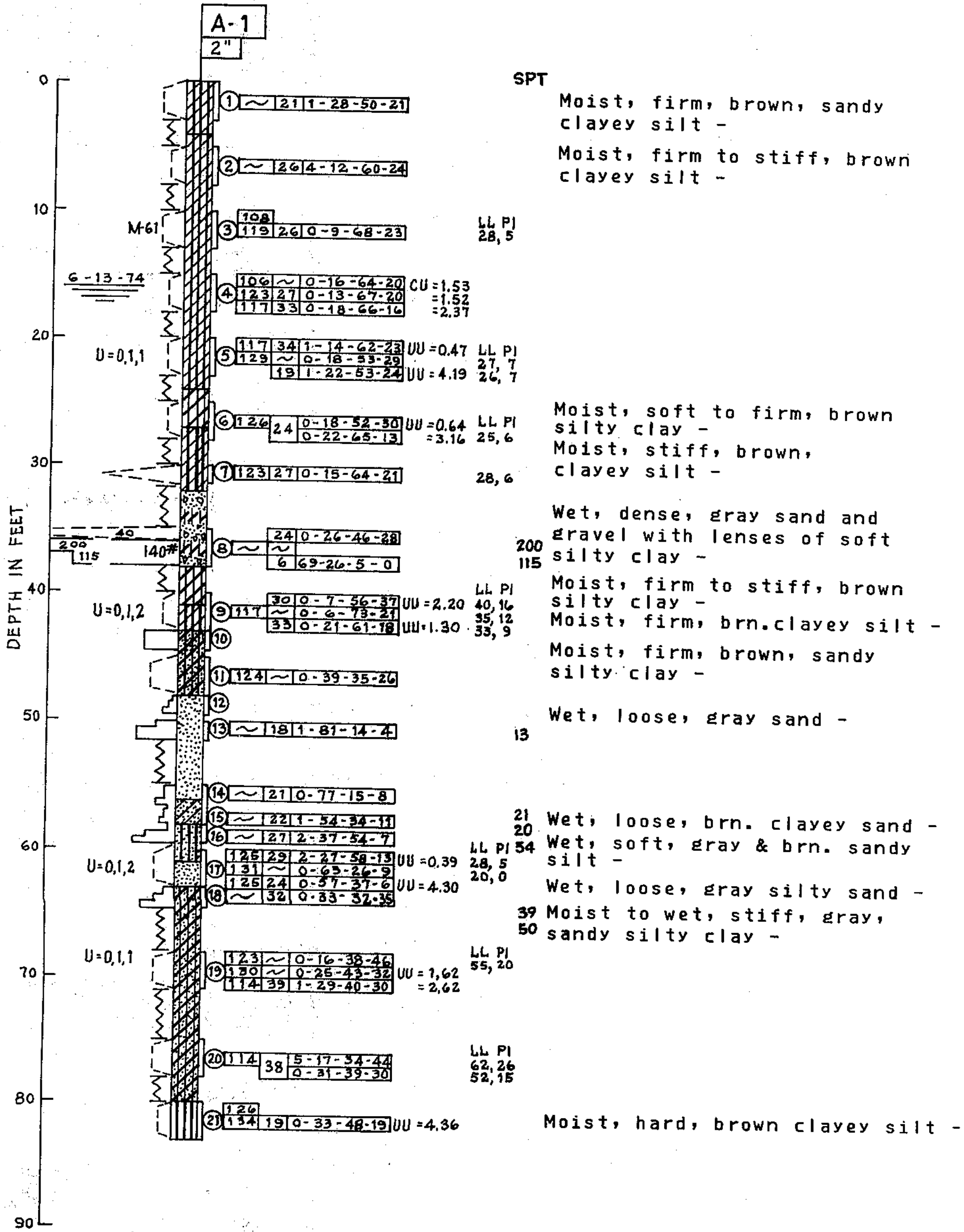
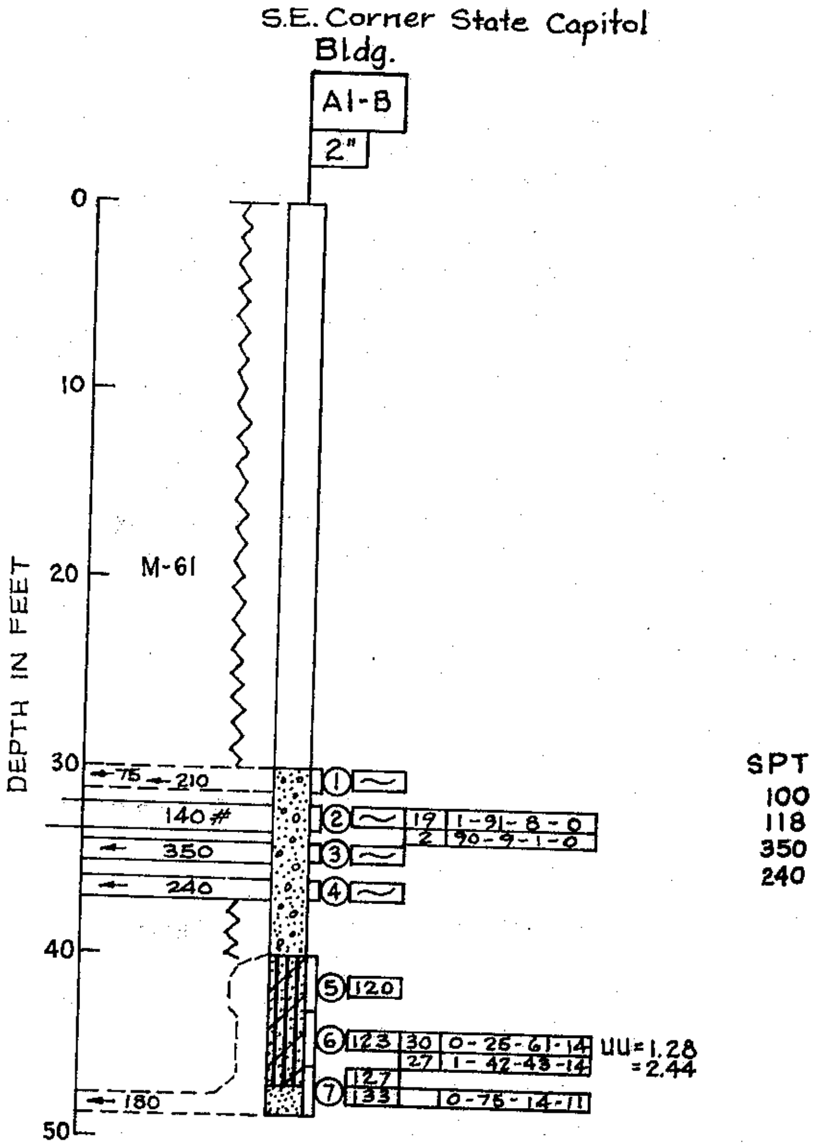





Figure E29




Wet, dense, brown, sand, gravel and rock -

Moist, firm to stiff, brown clayey sandy silt -

Wet, dense, brown fine sand-

Figure 30

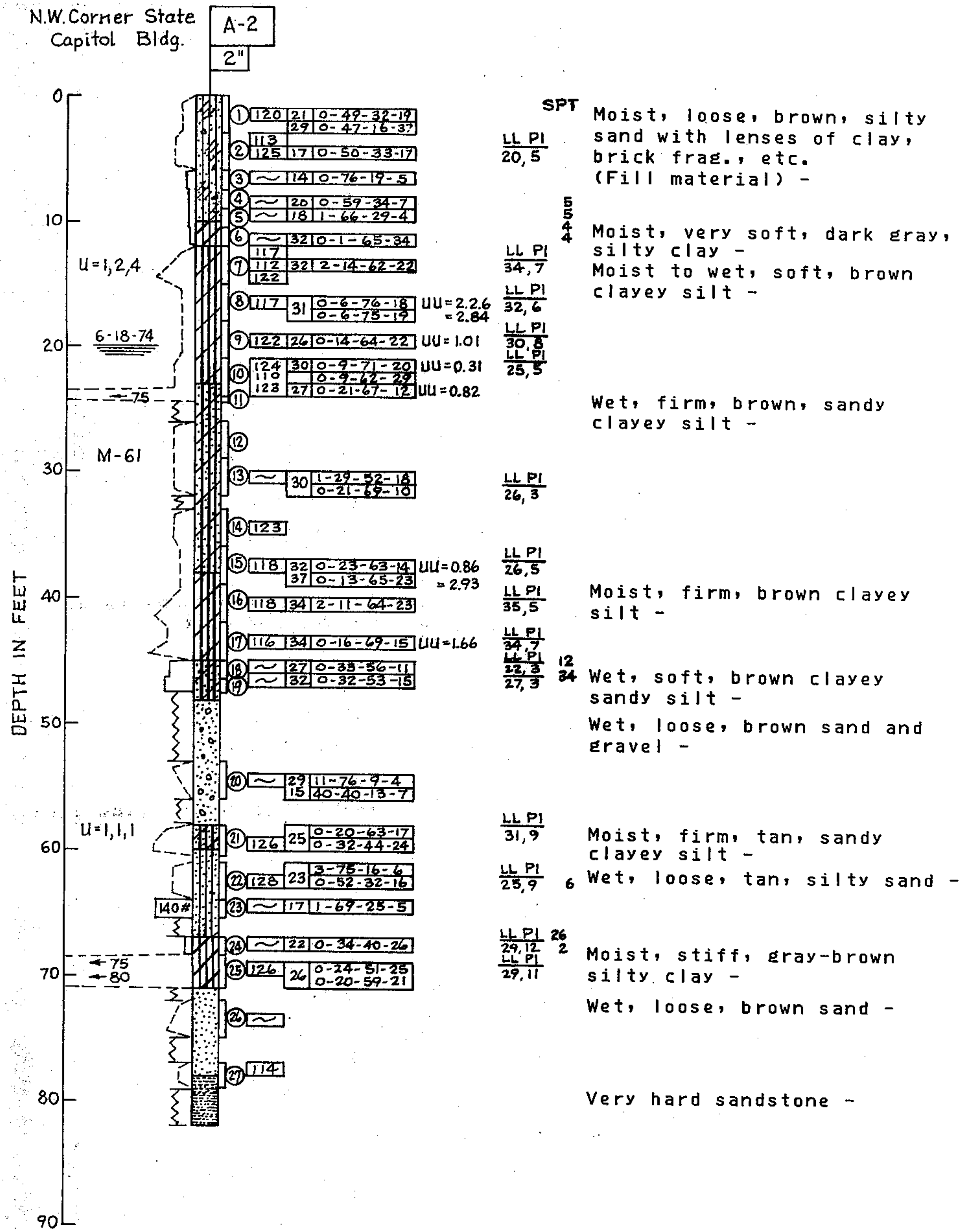
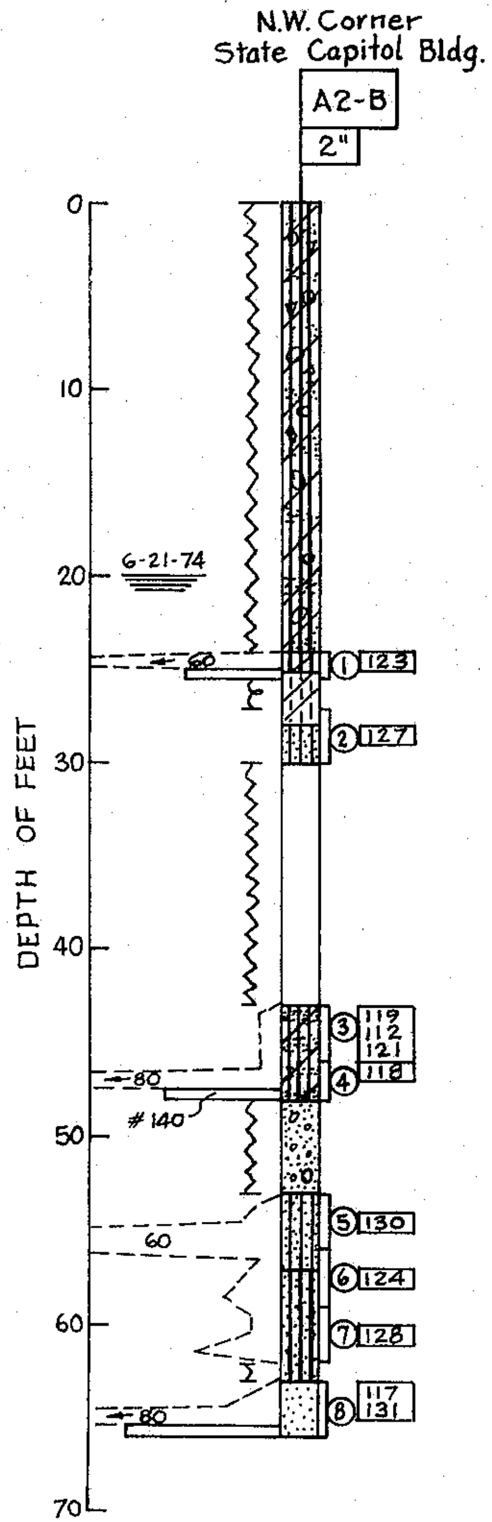

N.W. Corner State Capitol Bldg.
A-2
2"
DEPTH IN FEET
U=1,2,4
6-18-74
M-61
U=1,1,1
140#
SPT
Moist, loose, brown, silty sand with lenses of clay, brick frag., etc. (Fill material) -
Moist, very soft, dark gray, silty clay -
Moist to wet, soft, brown clayey silt -
Wet, firm, brown, sandy clayey silt -
Moist, firm, brown clayey silt -
Wet, soft, brown clayey sandy silt -
Wet, loose, brown sand and gravel -
Moist, firm, tan, sandy clayey silt -
Wet, loose, tan, silty sand -
Moist, stiff, gray-brown silty clay -
Wet, loose, brown sand -
Very hard sandstone -

**Figure E 31**




SPT Moist to wet, soft, brown clayey silt with some sandy lenses, cobbles, brick etc., as in Boring A2 -

Very wet, very soft, brown clayey silt -
Moist, firm to hd, reddish brn. weathered rock hardpan -
Moist, dense, brn. silty sand-
(Not sampled-material similar to Boring A-2)

Silt, clay and loose fine sand inter bedded -

Sand, gravel and cobbles -

Moist, firm gray-brown silty sand -

Moist, firm, gray-brown clayey silt and sand -

160 Moist, dense, tan, fine sand -

Figure E32

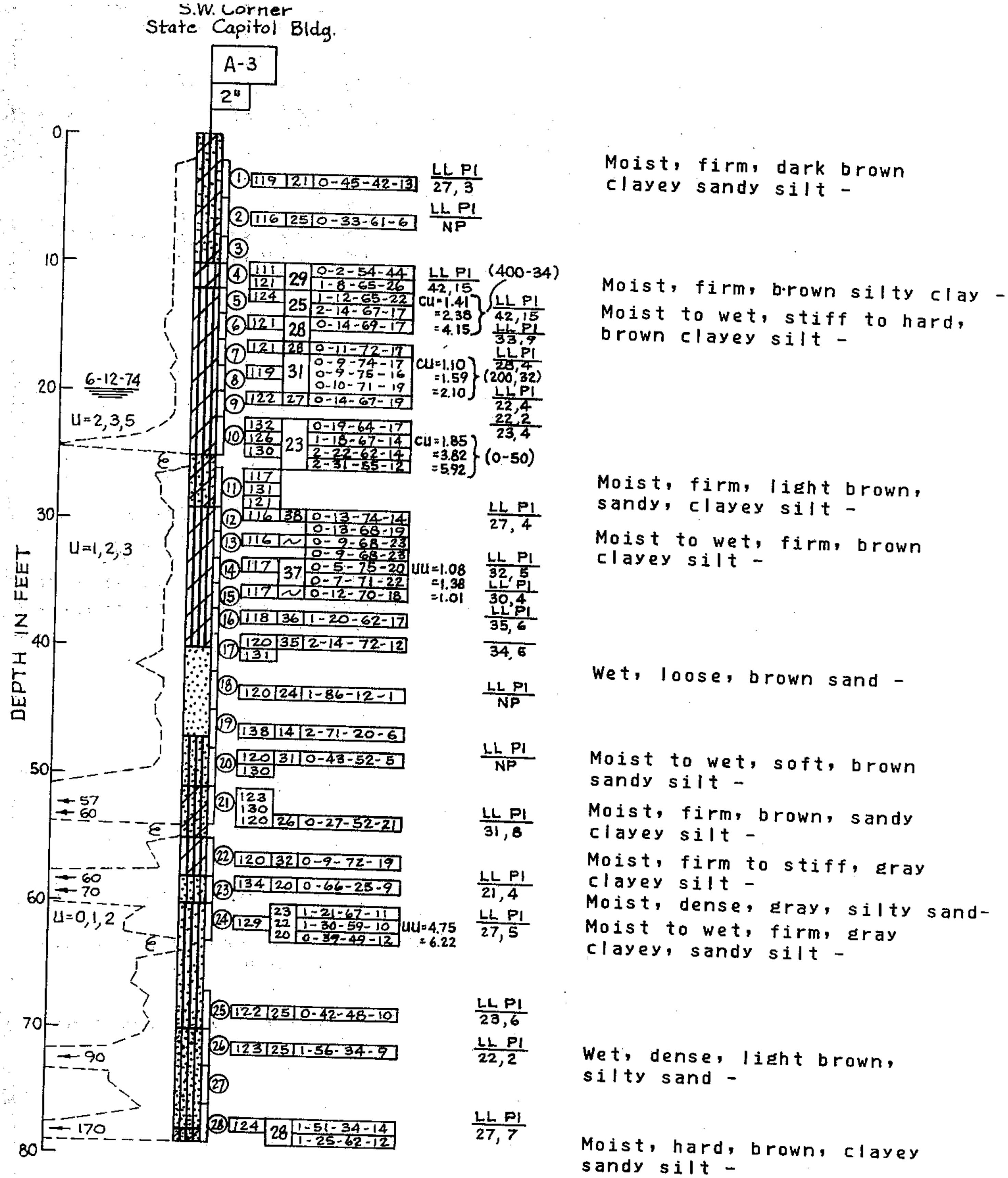
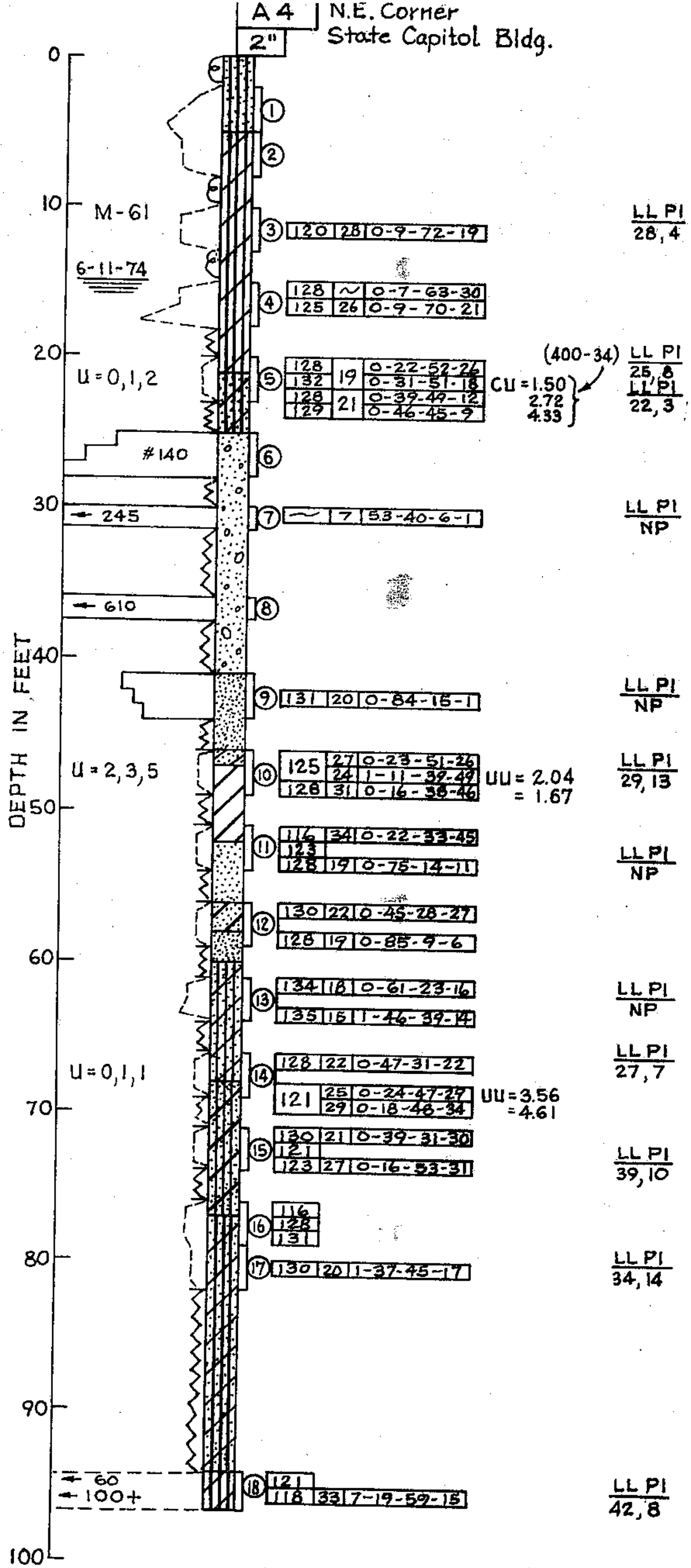

S.W. Corner
State Capitol Bldg.
A-3
2"
DEPTH IN FEET
6-12-74
Moist, firm, dark brown clayey sandy silt -
Moist, firm, brown silty clay -
Moist to wet, stiff to hard, brown clayey silt -
Moist, firm, light brown, sandy, clayey silt -
Moist to wet, firm, brown clayey silt -
Wet, loose, brown sand -
Moist to wet, soft, brown sandy silt -
Moist, firm, brown, sandy clayey silt -
Moist, firm to stiff, gray clayey silt -
Moist, dense, gray, silty sand-
Moist to wet, firm, gray clayey, sandy silt -
Wet, dense, light brown, silty sand -
Moist, hard, brown, clayey sandy silt -

Figure E33




Moist, soft, brown sandy silt -

Moist to wet, soft to firm, brown clayey silt -

Moist, firm to stiff, reddish brown, sandy clayey silt -

Wet, dense, brown, sand, gravel and cobbles -

Wet, loose, brown, fine sand -

Moist, stiff, brn, sandy clay
Moist, stiff, gray clay -

Moist, loose, brown sand -

Moist, firm, gray, sandy clay

Moist, dense, brown sand-

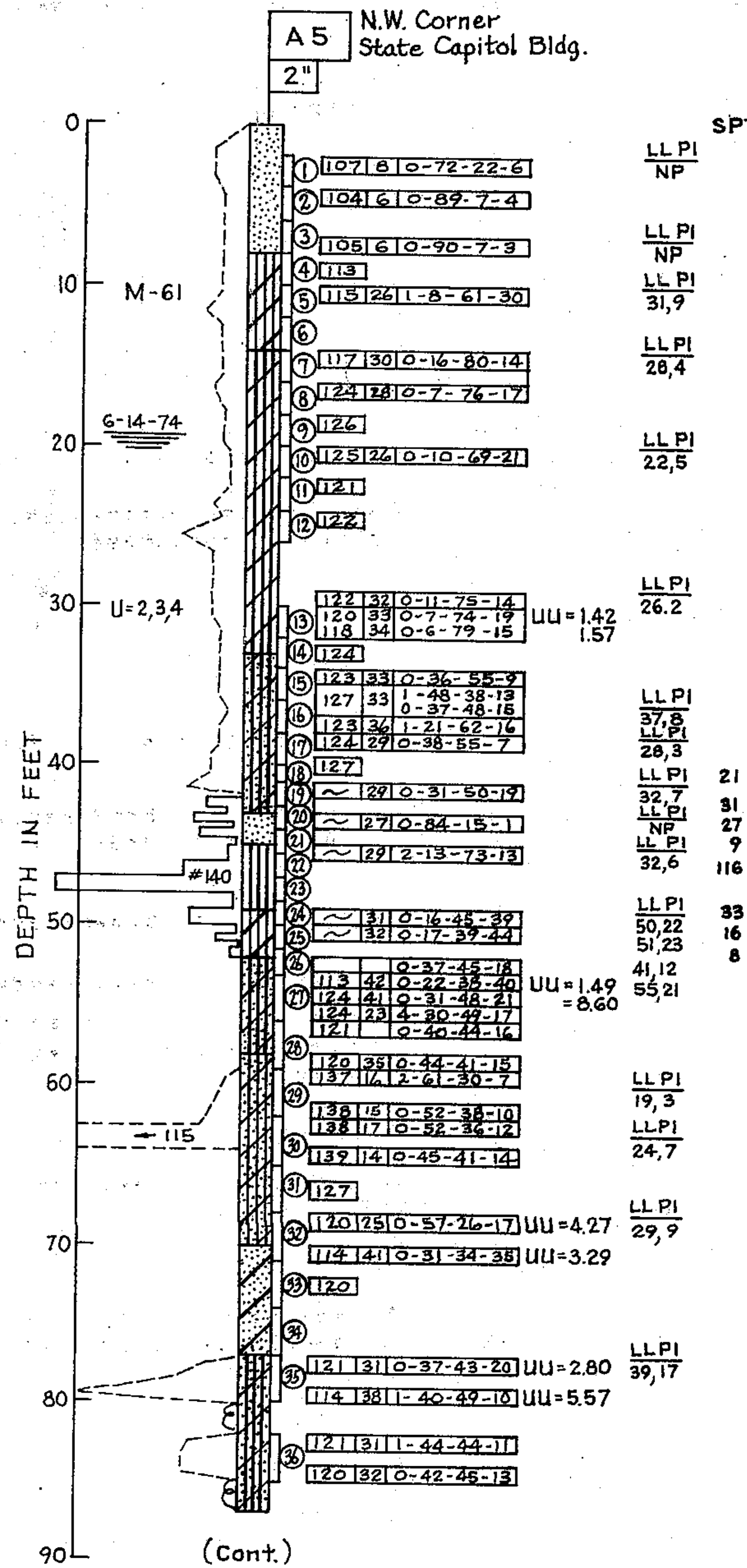
Moist, firm, brown, silty clayey sand -

Moist, hard, gray, sandy silty clay -

Moist, firm to stiff, gray, sandy clayey silt -

Moist, hard, brown, clayey silt -

Figure E 34




Moist, loose, brown sand -

Moist, firm, dark gray, silty clay -

Moist to wet, soft to firm, brown clayey silt -

Moist to wet, firm, brown, clayey sandy silt -

Wet, dense, gray sand -

Moist, stiff, gray silt -

Moist, stiff, gray, silty clay -

Moist, stiff to hard, gray, sandy clayey silt -

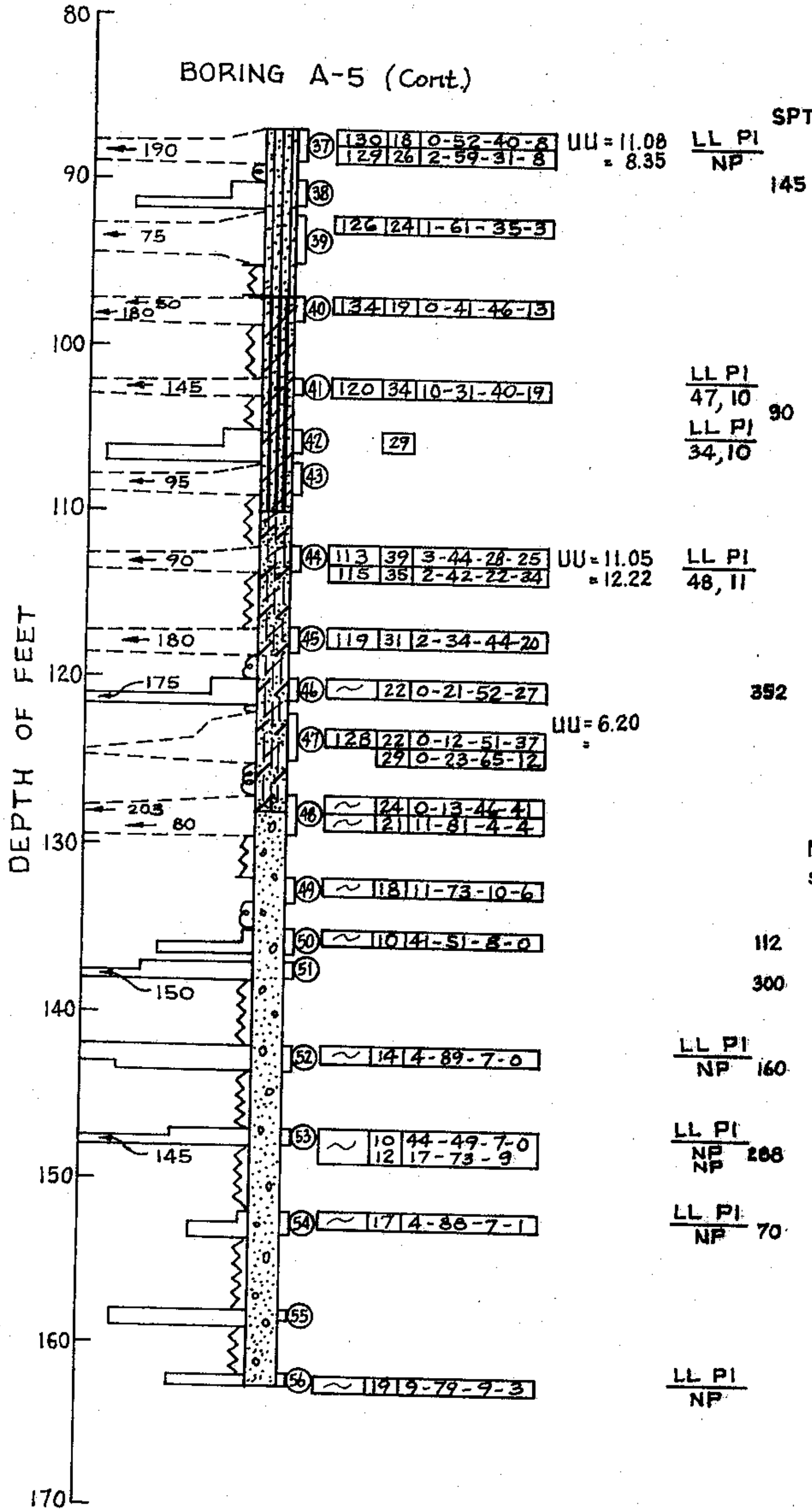
Wet, dense, gray clayey silty sand -

Moist, hard, gray sandy clay -

Moist, hard, gray, clayey sand and silt -



Figure E35



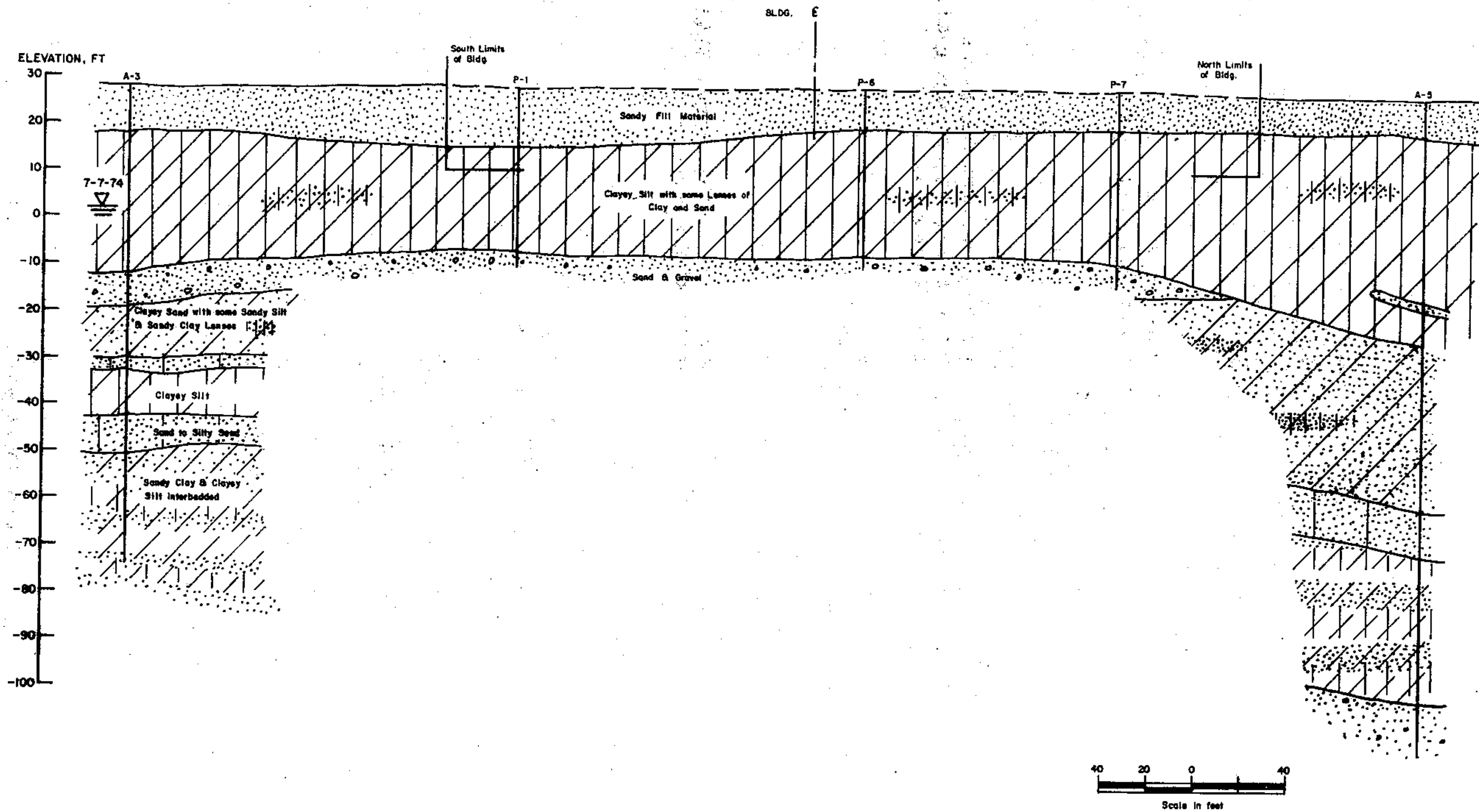

Moist, dense, dark gray, silty micaceous sand -

Moist, hard, gray, clayey, sandy silt -

Moist, hard, gray, interbedded sandy clay and clayey silt -

Moist to wet, dense, gray sand and gravel -

Figure E35 (cont'd)




SECTION A-A

FOR BORING LOCATION PLAN, SEE FIG. E27

Fig. E 36 SOIL PROFILE FROM BORINGS MADE IN 1948 & 1974
EXISTING CAPITOL BUILDING

(1866, 1948, 1974) which, together, include the area now occupied by the East Wing and various locations around the perimeter of both wings except along the north wall and northwest corner of the West Wing. It thuse appears that the stratum is continuous under the entire East Wing and at least a portion of the West Wing. The stratum either feathers out or plunges rather steeply under the north and northwest portion of the West Wing. No borings, of course, have been recorded for the area actually occupied by the West Wing.

Unfortunately, all six of the eight borings made in 1866 (Appendix C) that encountered the stratum were terminated immediately with no indication of stratum thickness. Similarly, ten of the eleven 1948 borings were terminated at from 3 to 15 feet into the stratum, leaving its thickness still undetermined. The remaining boring, P3, did fully penetrate the stratum which was 5 feet thick at that location. Borings A1 and A4 of this investigation showed sand and gravel thicknesses of 10 and 16 feet, respectively, near the southeast and northeast corners of the East Wing. Boring A2, located west of the northwest corner of the West Wing, penetrated a sand and gravel stratum 5 to 10 feet thick. Boring A3, south of the West Wing Southwest corner, revealed a 7-foot thick coarse sand stratum but with much less gravel content than was found at other boring locations.

Depth to the sand and gravel stratum varies, but east of the West Wing it was generally found at Elevation -5 to -8. Around the south and west sides of the West Wing the stratum was found at Elevation -20 to -30. Boring A5, north of the West Wing, encountered sand and gravel at Elevation -100± which extended beyond the bottom of the boring at Elevation -135±.

Below the sand and gravel, dense sands and/or silty sands predominate with varying clay fractions. Information on these soils,

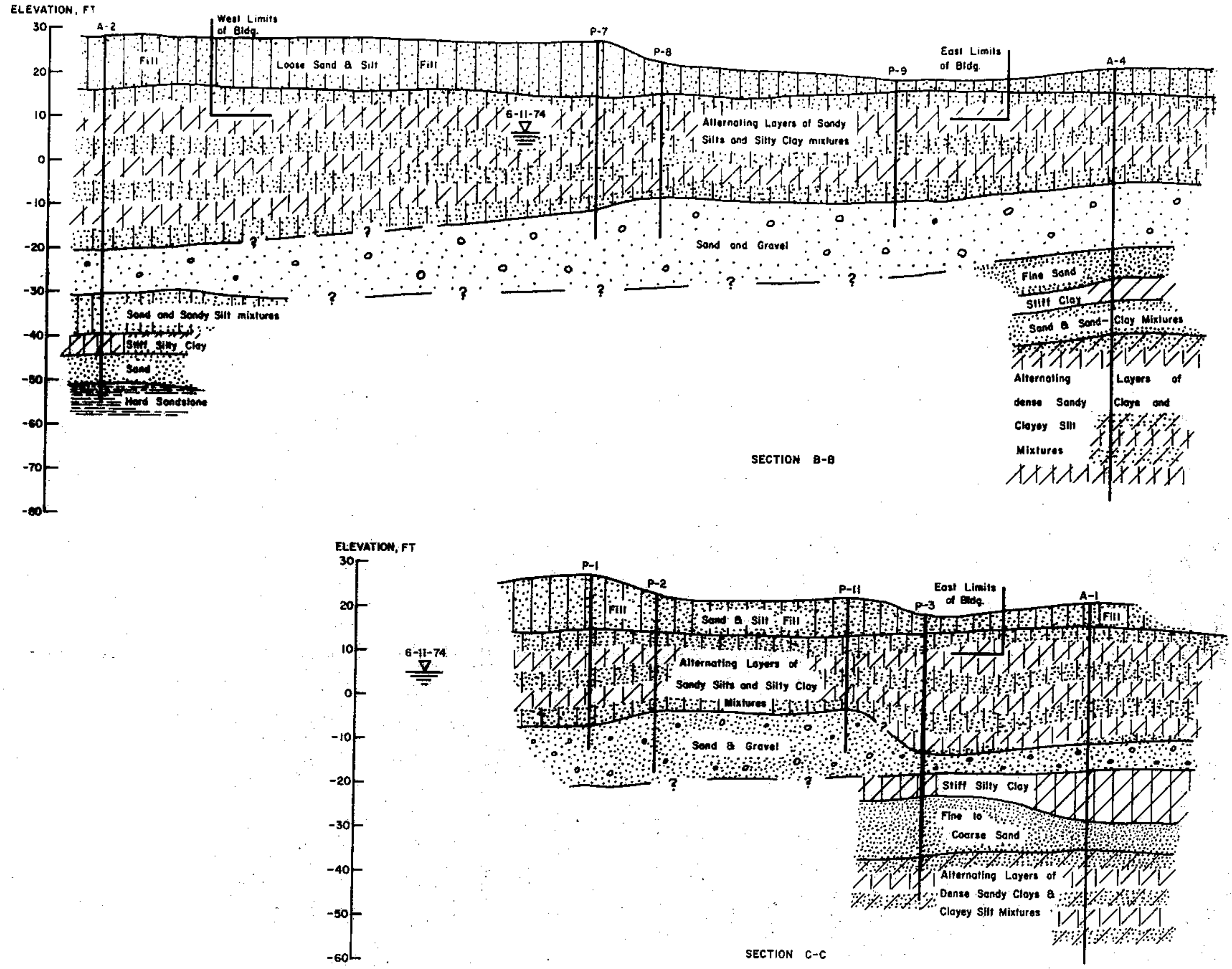



Fig. E37 SOIL PROFILE FROM BORINGS MADE IN 1948 & 1974
EXISTING CAPITOL BUILDING

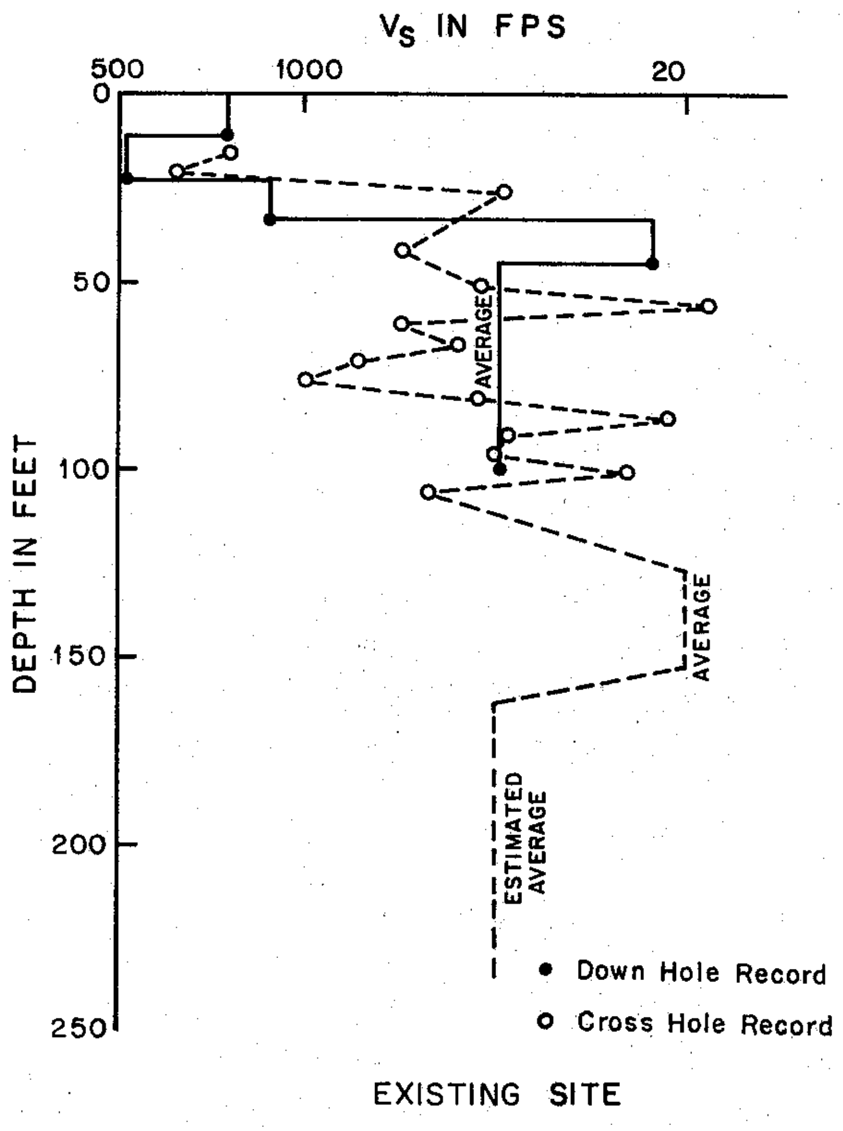



Fig.E38 SHEAR WAVE VELOCITY AVERAGES DETERMINED FROM FIELD GEOPHYSICAL SURVEY METHODS based entirely on the preliminary borings from this investigation (except for the 1948 Boring P3), is limited with respect to strata continuity, and depth below Elevation -50.

Free water was recorded at about Elevation +8 during the period of exploration in June-July, 1974.

## Geophysical Investigation

Geophysical testing at the existing site included seismic refraction, cross hole seismic, and down hole seismic tests. All tests were performed at boring location A-5 by Transportation Laboratory personnel and equipment. The purpose of the tests was to determine compressional (P) and shear (S) wave velocities at various depths below the site.

Two seismic refraction lines were run. Energy was supplied by striking an aluminum plate with a sledge hammer. Compressional wave arrival times were recorded on a Bison Model B Two-Channel Timer.

Cross hole compressional and shear wave velocities were obtained to a depth of 220 feet using test holes located 35 feet apart. An Electrotech Model M4E was used to record the seismic events. Blasting caps were detonated at various levels to supply the energy. A three-dimensional detector geophone was lowered to the same depth as the energy source and fixed in place by inflating an attached neoprene packer with compressed nitrogen gas.

A down hole seismic survey was conducted and results to a depth of 100 feet were obtained. Shear waves were generated by striking the ends of a weighted wooden plank, lying horizontal, with a wooden mallet. The compressional waves resulted from striking an aluminum plate in an idential manner.

The data accumulated from the three test methods were reduced and the results analyzed and compared, where possible. Figure E38 presents the results in the form of a graph showing shear velocity versus depth below ground surface.

# APPENDIX F

## FIELD INVESTIGATIONS - PROPOSED SITE

## APPENDIX F

## FIELD INVESTIGATIONS - PROPOSED SITE

At the proposed site, field investigations were directed toward obtaining information pertinent to the development of preliminary foundation design criteria for a new building. The field work consisted of:

1. A program of boring and sampling throughout the site to delineate soil stratification and to obtain samples for laboratory testing.

2. Geophysical testing at selected boreholes to determine values of dynamic soil properties for use in ground response analysis.

3. A groundwater pump test to evaluate drawdown characteristics and aquifer permeability.

### Boring and Sampling

Boring and sampling was initiated at the proposed site on June 15 and continued through August 7. The drilling operation was conducted using the same equipment, personnel and procedures as at the existing site, with the exception that an additional drill rig, from District 05, was employed.

In total, seven boring locations were selected for soil stratification investigation. Soil samples for laboratory testing and standard penetration test results were obtained at various depths as borings progressed. The borings, shown in plan on Figure F1, ranged in depth from 70 to 220 feet. None of the borings were sampled or logged below a depth of 165 feet. At location 5, three boreholes were made in order that additional samples of selected soil layers could be retrieved. Individual boring profiles for all the holes sampled, with summarized laboratory test data are presented as Figures F2-F11. From these data, a generalized soil cross-section was developed and is shown in Figure F12.

Adjacent to location 7, a special boring, (R2) was made to a depth of 230 feet for purposes of geophysical testing. Its proximity to Boring 7, 35 feet, negated the need for logging.

In association with a planned groundwater pump test, an additional set of six borings were made. These included a 12-inch diameter pump well (P-1B) augered just south of the alley bounded by 15th, 16th, and N Streets and Capitol Avenue, and five 4-inch observation wells, two to the east of the pump well (0-1, 0-2) and three to the north (0-3, 0-4, 0-5). All are shown on Figure F1.

Analysis of the above boring records indicates that surface soils to a depth of 5 to 10 feet are heterogeneous in character, consisting of sand and silt with some brick and rubble. A generalized profile of underlying soils consists of soft to firm, slightly plastic silts and clays to depths of 25 to 30 feet, followed by dense sand, gravel and cobbles extending to depths of about 55 feet or more below ground surface. Due to the size of cobbles in this lower stratum, it was impossible to retrieve any sort of representative sample of the material. This sand and gravel stratum is underlain by a layer of sandy silt or silty sand

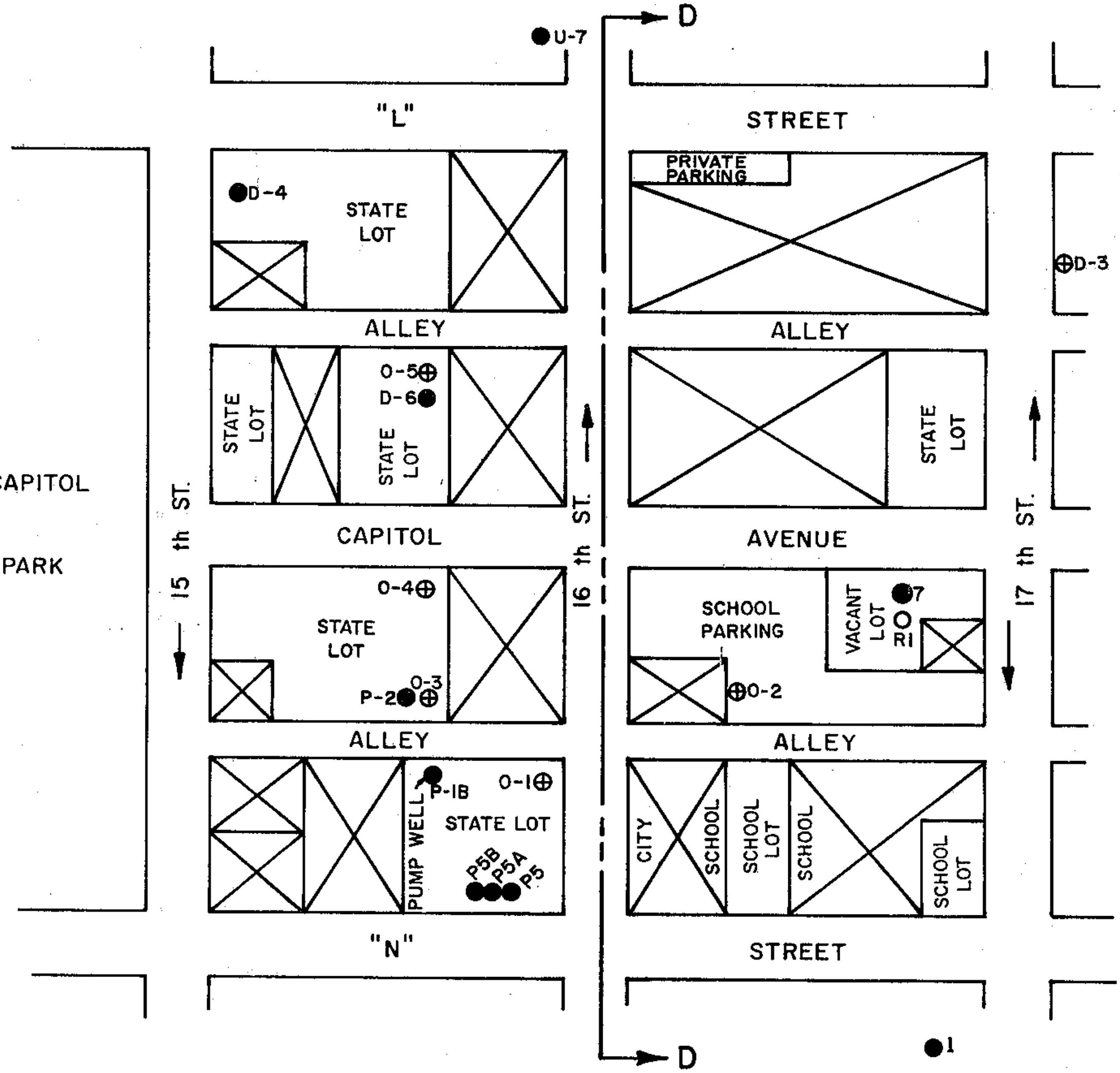
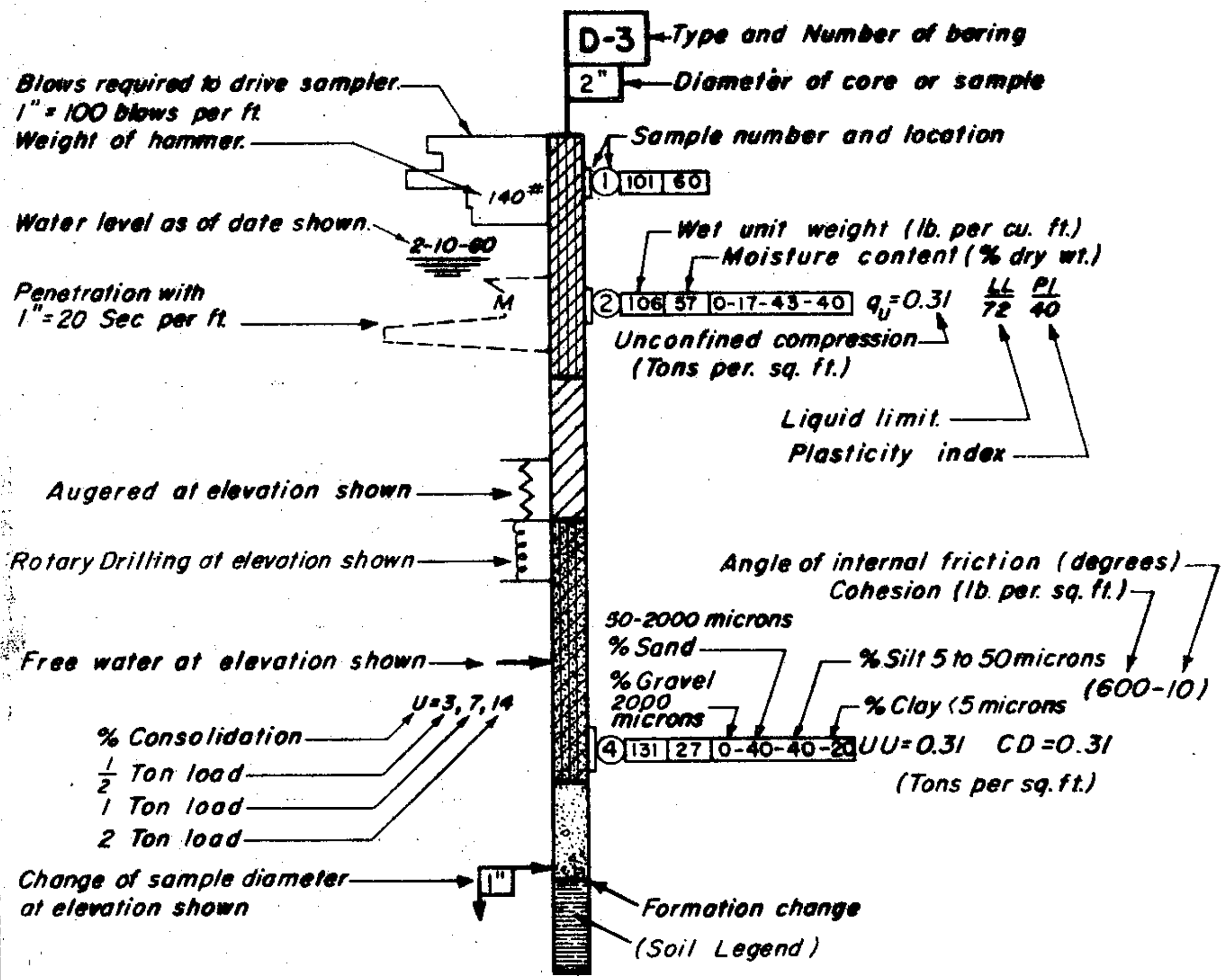



Fig.F1 PLAN MAP OF PROPOSED STATE CAPITOL SITE SHOWING APPROXIMATE LOCATION OF SAMPLE BORINGS (●) AND OBSERVATION WELLS (⊕) - 1974.

# BORING LEGEND

## CROSS-SECTION & PROFILE SHEETS

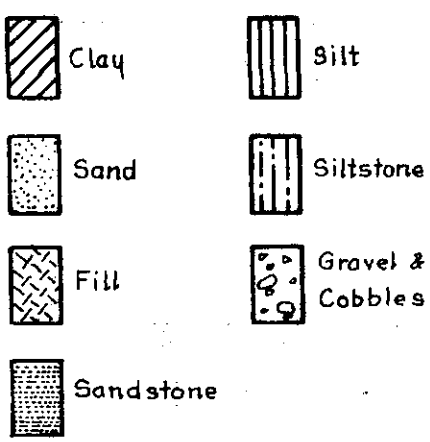



SPT - Standard Penetration Resistance Test

STRENGTH TESTS

$q_u$ - Unconfined Compression
UU - Unconsolidated Undrained
CU - Consolidated Undrained
CD - Consolidated Drained

## SOIL LEGEND




STATE OF CALIFORNIA — HIGHWAY TRANSPORTATION AGENCY
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
TRANSPORTATION LABORATORY

**Fig. F2 BORING LEGEND**

| | | | |
|---|---|---|---|
| DATE | | SUBMITTED BY: SR. MAT'LS. & RES. ENG'R. | DWG. NO 2695 |
| DR BY M.I. D.D. | CK BY | APPROVED BY: SUP'R. MAT'LS. & RES. ENG'R. - FDN | SHEET NO OF SHTS. |

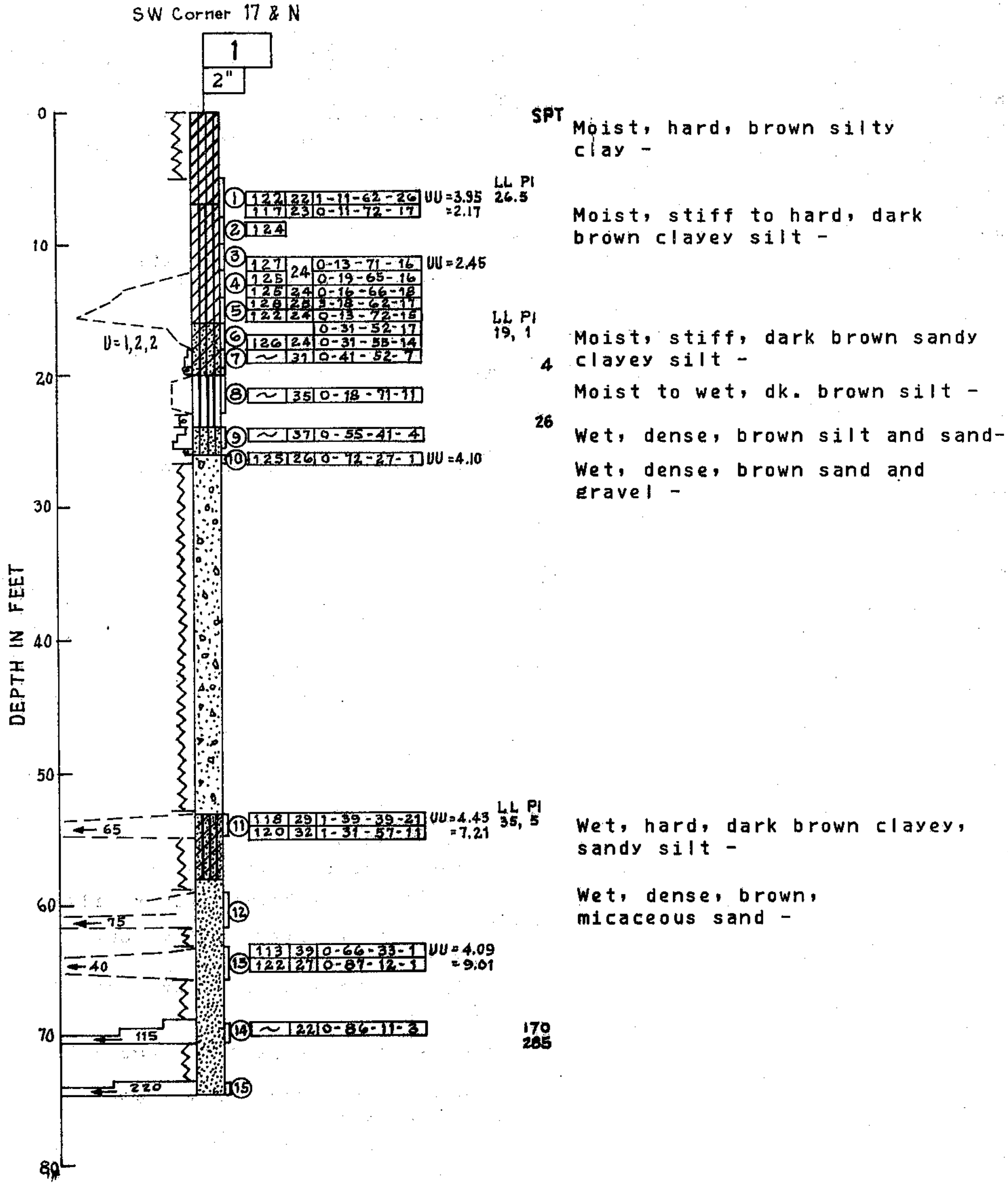

SW Corner 17 & N
1
2"
DEPTH IN FEET
0
10
20
30
40
50
60
70
80
SPT
Moist, hard, brown silty clay -
LL PI
26.5
UU=3.95
=2.17
Moist, stiff to hard, dark brown clayey silt -
UU=2.45
LL PI
19, 1
U=1,2,2
Moist, stiff, dark brown sandy clayey silt -
4
Moist to wet, dk. brown silt -
26
Wet, dense, brown silt and sand-
UU=4.10
Wet, dense, brown sand and gravel -
LL PI
35, 5
UU=4.43
=7.21
Wet, hard, dark brown clayey, sandy silt -
65
Wet, dense, brown, micaceous sand -
75
UU=4.09
=9.01
40
115
170
285
220

Figure F3

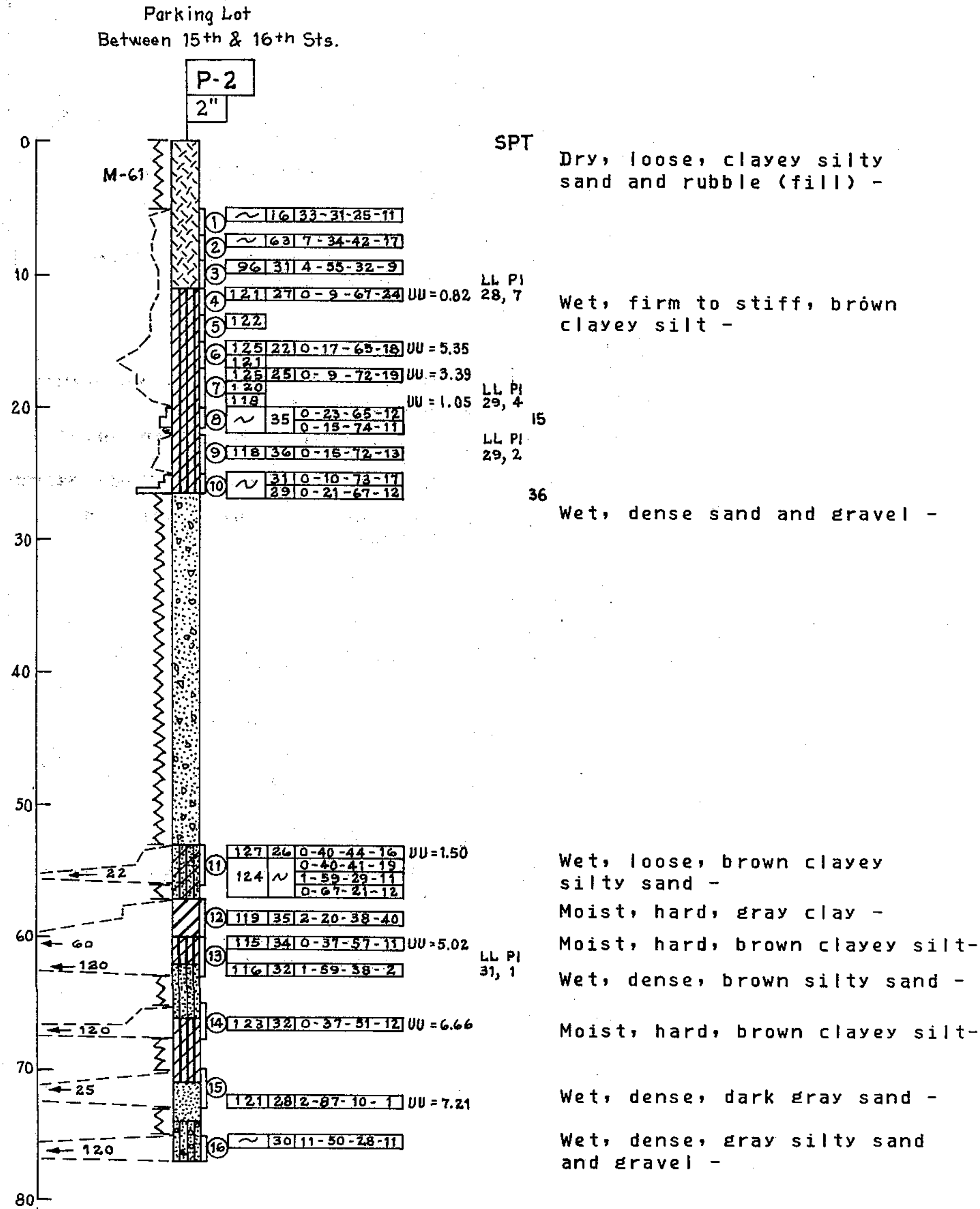

Parking Lot
Between 15th & 16th Sts.
P-2
2"
SPT
M-61
Dry, loose, clayey silty sand and rubble (fill) -
16 33-31-25-11
63 7-34-42-17
96 31 4-55-32-9
121 27 0-9-67-24 UU=0.82
LL PI 28, 7
Wet, firm to stiff, brown clayey silt -
122
125 22 0-17-65-18 UU=5.35
121
125 25 0-9-72-19 UU=3.39
120
118 UU=1.05
LL PI 29, 4
35 0-23-65-12
0-15-74-11
15
LL PI 29, 2
118 36 0-16-72-13
31 0-10-73-17
29 0-21-67-12
36
Wet, dense sand and gravel -
127 26 0-40-44-16 UU=1.50
124 0-40-41-19
1-59-29-11
0-67-21-12
22
Wet, loose, brown clayey silty sand -
119 35 2-20-38-40
Moist, hard, gray clay -
60
115 34 0-37-57-11 UU=5.02
LL PI 31, 1
Moist, hard, brown clayey silt-
120
116 32 1-59-38-2
Wet, dense, brown silty sand -
120
123 32 0-37-51-12 UU=6.66
Moist, hard, brown clayey silt-
25
121 28 2-87-10-1 UU=7.21
Wet, dense, dark gray sand -
120
30 11-50-28-11
Wet, dense, gray silty sand and gravel -
0
10
20
30
40
50
60
70
80

Figure F4

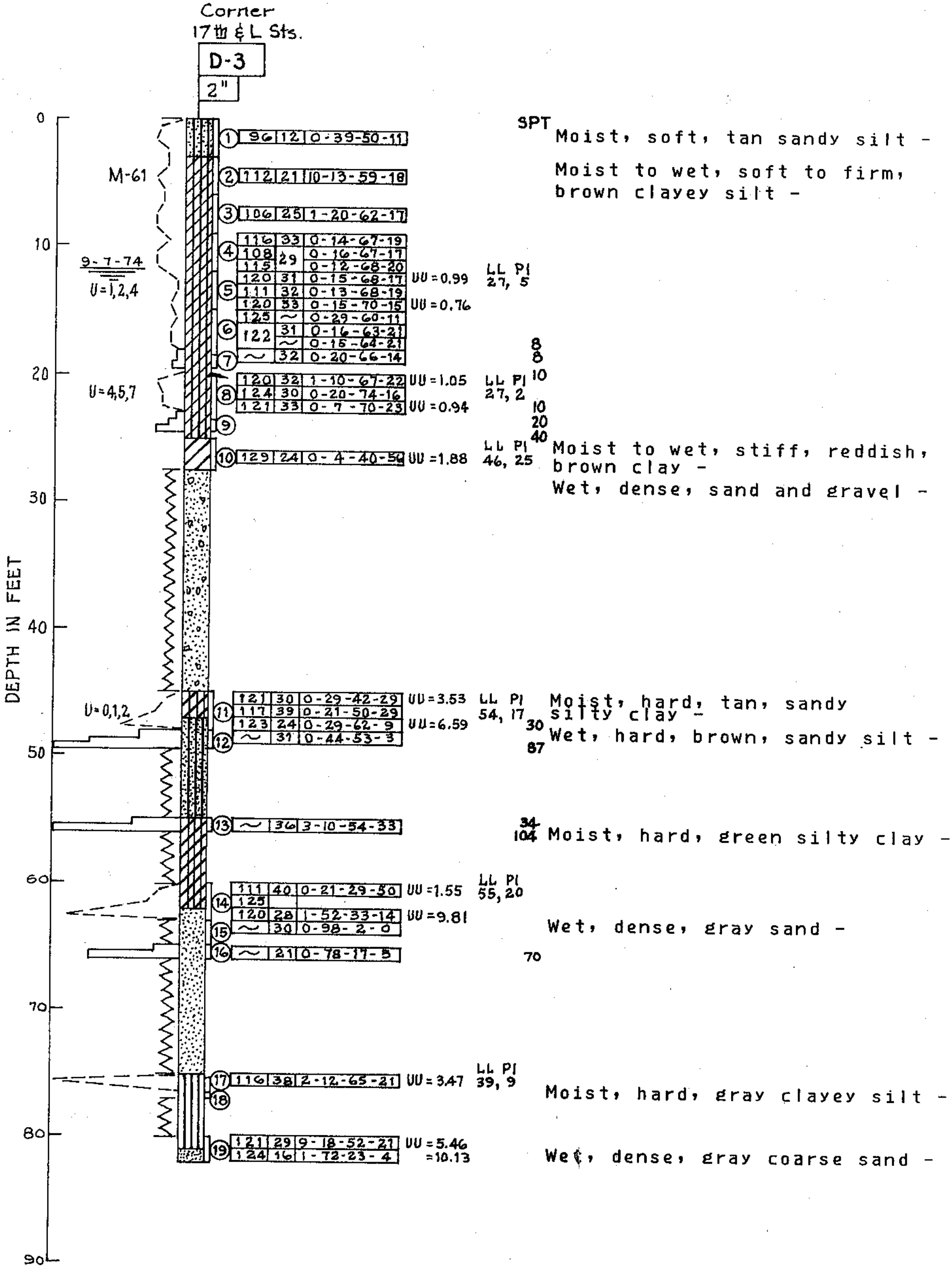

Corner
17th & L Sts.
D-3
2"
DEPTH IN FEET
0
10
20
30
40
50
60
70
80
90
M-61
9-7-74
U=1,2,4
U=4,5,7
U=0,1,2
SPT
1 96 12 0-39-50-11
2 112 21 10-13-59-18
3 106 25 1-20-62-17
4 116 33 0-14-67-19
108 29 0-16-67-17
115 0-12-68-20
5 120 31 0-15-68-17 UU=0.99
111 32 0-13-68-19
120 33 0-15-70-15 UU=0.76
6 125 ~ 0-29-60-11
122 31 0-16-63-21
~ 0-15-64-21
7 ~ 32 0-20-66-14
LL PI
27, 5
8
8
8 120 32 1-10-67-22 UU=1.05
124 30 0-20-74-16
121 33 0-7-70-23 UU=0.94
LL PI
27, 2
10
10
20
40
9
10 129 24 0-4-40-56 UU=1.88
LL PI
46, 25
11 121 30 0-29-42-29 UU=3.53
117 39 0-21-50-29
LL PI
54, 17
12 123 24 0-29-62-9 UU=6.59
~ 31 0-44-53-3
30
87
13 ~ 36 3-10-54-33
34
104
14 111 40 0-21-29-50 UU=1.55
125
LL PI
55, 20
15 120 28 1-52-33-14 UU=9.81
~ 30 0-98-2-0
16 ~ 21 0-78-17-5
70
17 116 38 2-12-65-21 UU=3.47
LL PI
39, 9
18
19 121 29 9-18-52-21 UU=5.46
124 16 1-72-23-4 =10.13
Moist, soft, tan sandy silt -
Moist to wet, soft to firm, brown clayey silt -
Moist to wet, stiff, reddish, brown clay -
Wet, dense, sand and gravel -
Moist, hard, tan, sandy silty clay -
Wet, hard, brown, sandy silt -
Moist, hard, green silty clay -
Wet, dense, gray sand -
Moist, hard, gray clayey silt -
Wet, dense, gray coarse sand -

Figure F5

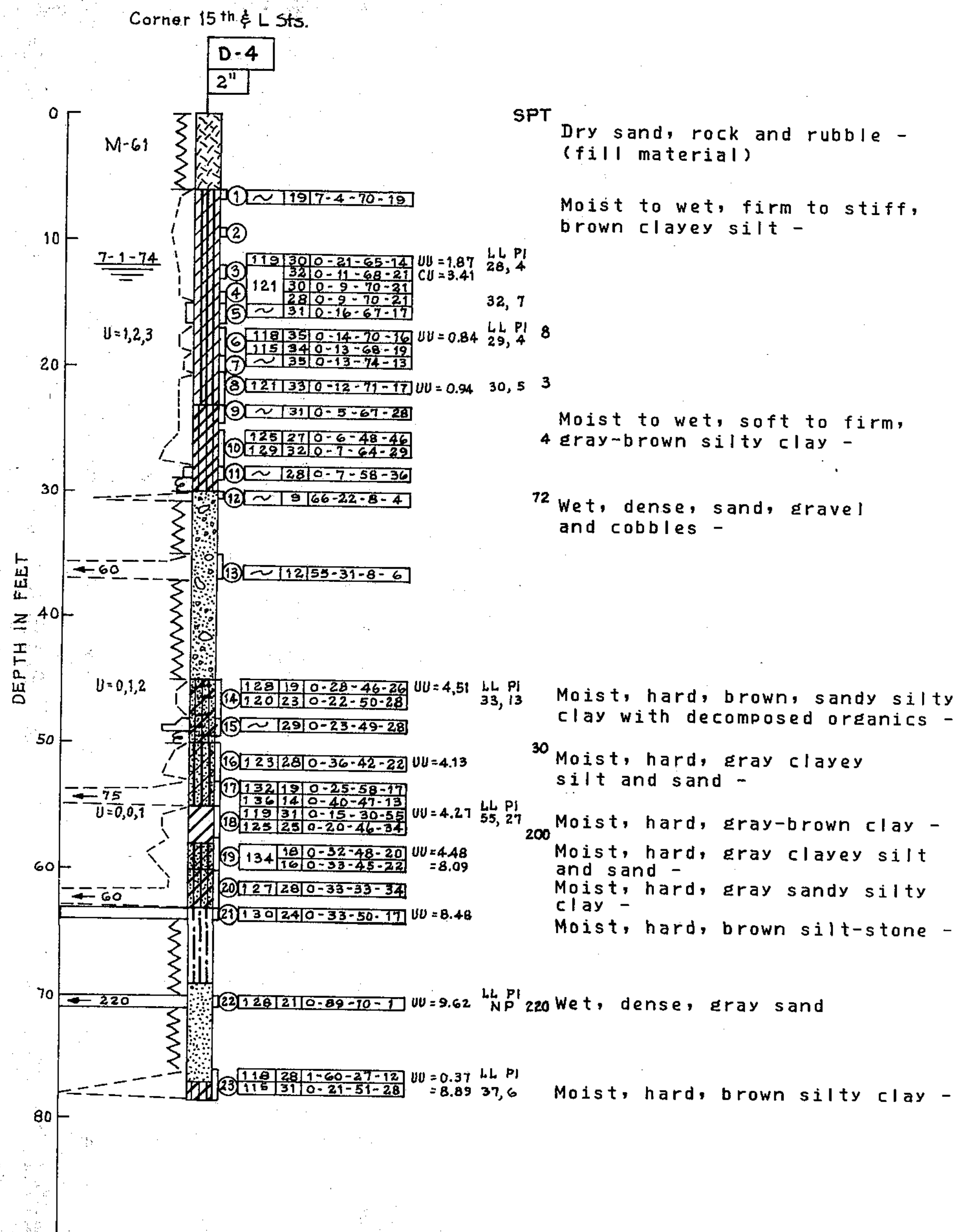
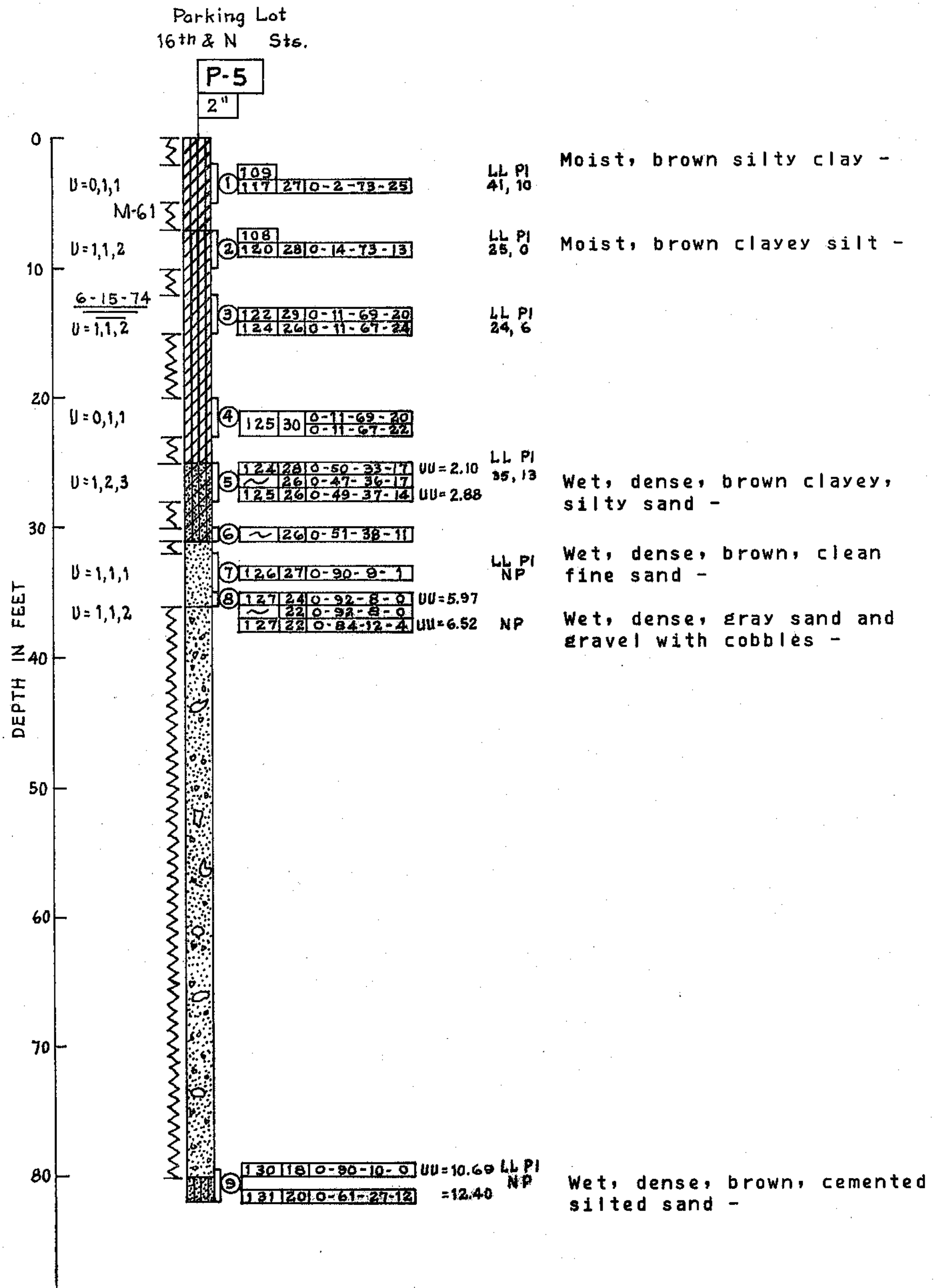

Corner 15th & L Sts.
D-4
2"
DEPTH IN FEET
M-61
7-1-74
U=1,2,3
U=0,1,2
U=0,0,1
SPT
Dry sand, rock and rubble - (fill material)
Moist to wet, firm to stiff, brown clayey silt -
Moist to wet, soft to firm, gray-brown silty clay -
Wet, dense, sand, gravel and cobbles -
Moist, hard, brown, sandy silty clay with decomposed organics -
Moist, hard, gray clayey silt and sand -
Moist, hard, gray-brown clay -
Moist, hard, gray clayey silt and sand -
Moist, hard, gray sandy silty clay -
Moist, hard, brown silt-stone -
Wet, dense, gray sand
Moist, hard, brown silty clay -

Figure F6



Parking Lot
16th & N Sts.
P-5
2"
DEPTH IN FEET
0
10
20
30
40
50
60
70
80
90
U=0,1,1
M-61
U=1,1,2
6-15-74
U=1,1,2
U=0,1,1
U=1,2,3
U=1,1,1
U=1,1,2
109
117 27 0-2-73-25
LL PI
41, 10
Moist, brown silty clay -
108
120 28 0-14-73-13
LL PI
25, 0
Moist, brown clayey silt -
122 29 0-11-69-20
124 26 0-11-67-24
LL PI
24, 6
125 30 0-11-69-20
0-11-67-22
124 28 0-50-33-17 UU=2.10
26 0-47-36-17
125 26 0-49-37-14 UU=2.88
LL PI
35, 13
Wet, dense, brown clayey, silty sand -
26 0-51-38-11
126 27 0-90-9-1
LL PI
NP
Wet, dense, brown, clean fine sand -
127 24 0-92-8-0 UU=5.97
22 0-92-8-0
127 22 0-84-12-4 UU=6.52
NP
Wet, dense, gray sand and gravel with cobbles -
130 18 0-90-10-0 UU=10.69
131 20 0-61-27-12 =12.40
LL PI
NP
Wet, dense, brown, cemented silted sand -


Figure F7

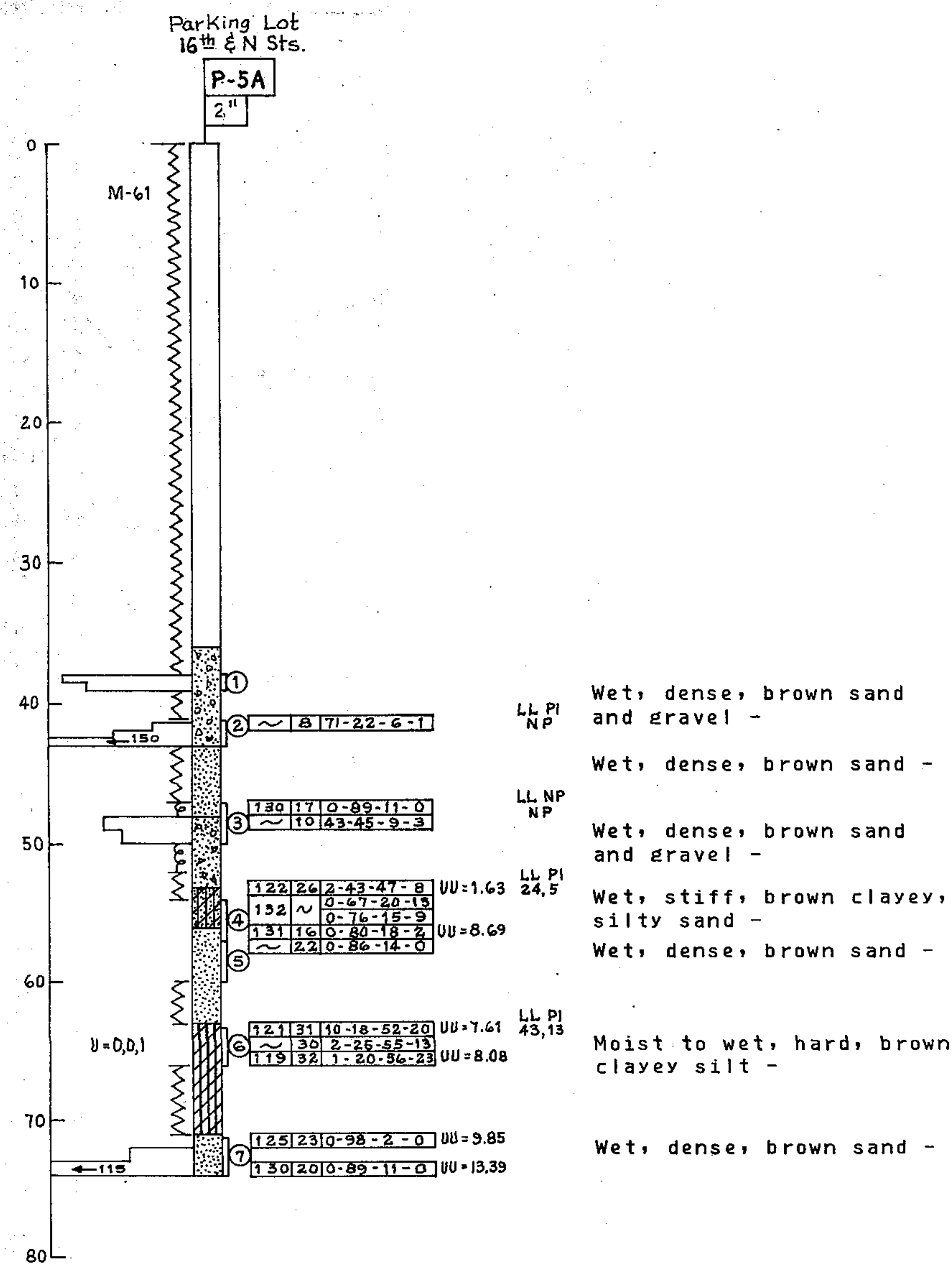
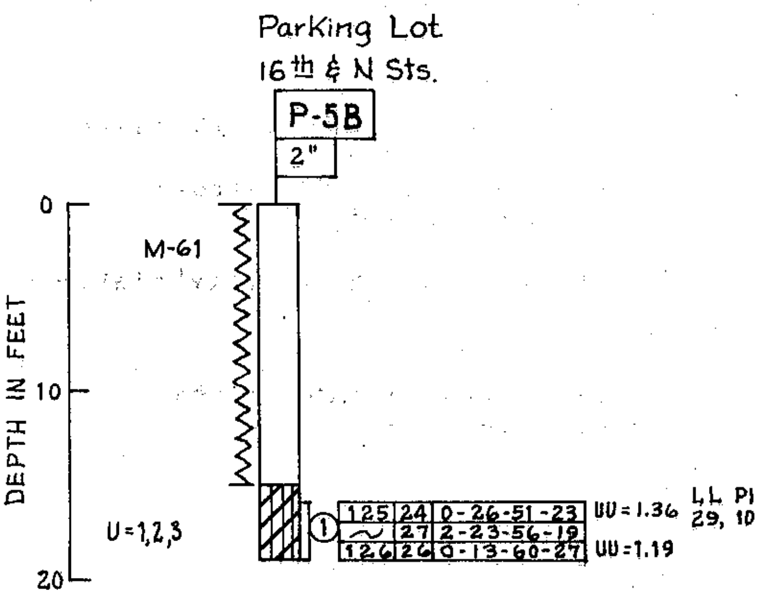

Parking Lot
16th & N Sts.
P-5A
2"
M-61
150
LL PI
NP
LL NP
NP
LL PI
24,5
LL PI
43,13
Wet, dense, brown sand and gravel -
Wet, dense, brown sand -
Wet, dense, brown sand and gravel -
Wet, stiff, brown clayey, silty sand -
Wet, dense, brown sand -
Moist to wet, hard, brown clayey silt -
Wet, dense, brown sand -
UU=1.63
UU=8.69
UU=7.61
UU=8.08
UU=9.85
UU=13.39
U=0,0,1
115

Figure F8




Moist, firm, brown silty clay -

Figure F9

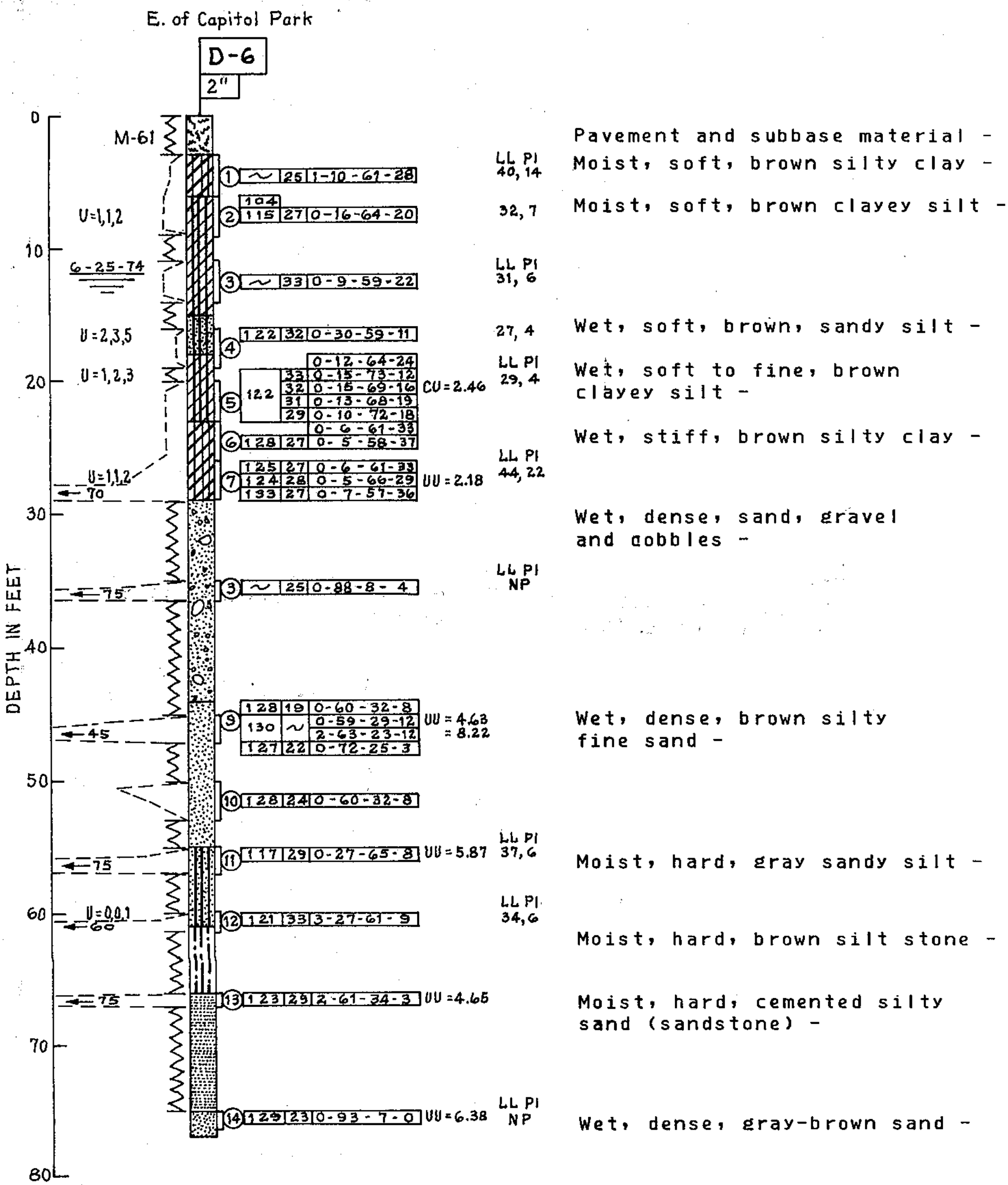
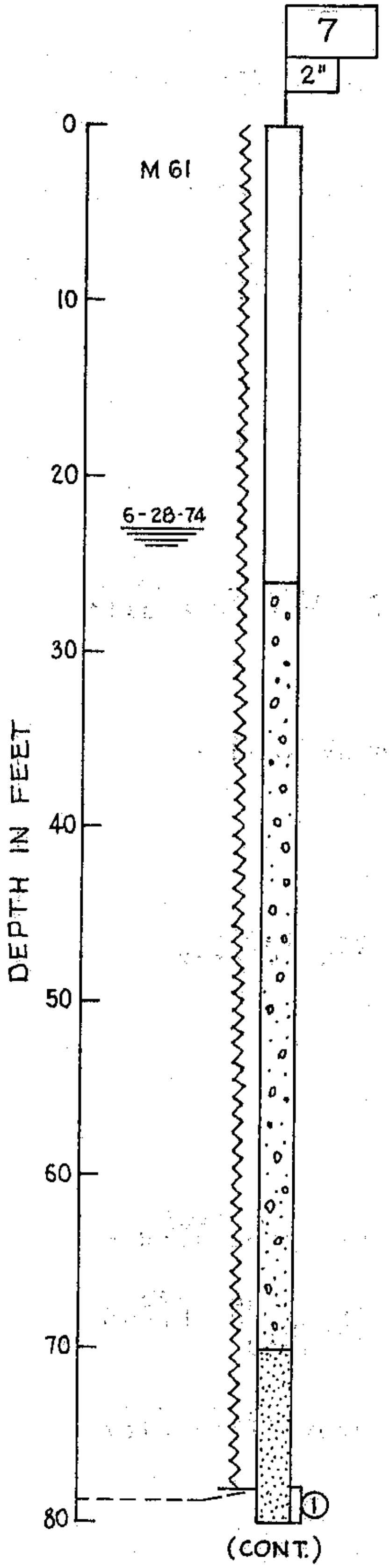

E. of Capitol Park
D-6
2"
DEPTH IN FEET
M-61
U=1,1,2
6-25-74
U=2,3,5
U=1,2,3
U=1,1,2
70
75
45
75
U=0,0,1
60
75
CU=2.46
UU=2.18
UU=4.63
=8.22
UU=5.87
UU=4.65
UU=6.38
Pavement and subbase material -
Moist, soft, brown silty clay -
Moist, soft, brown clayey silt -
Wet, soft, brown, sandy silt -
Wet, soft to fine, brown clayey silt -
Wet, stiff, brown silty clay -
Wet, dense, sand, gravel and cobbles -
Wet, dense, brown silty fine sand -
Moist, hard, gray sandy silt -
Moist, hard, brown silt stone -
Moist, hard, cemented silty sand (sandstone) -
Wet, dense, gray-brown sand -

Figure F10

Vacant Lot - Btw. 16 & 17 Sts.




No Sampling

Gravel penetrated - no sampling

Fine gray sand - no samples

Figure F11

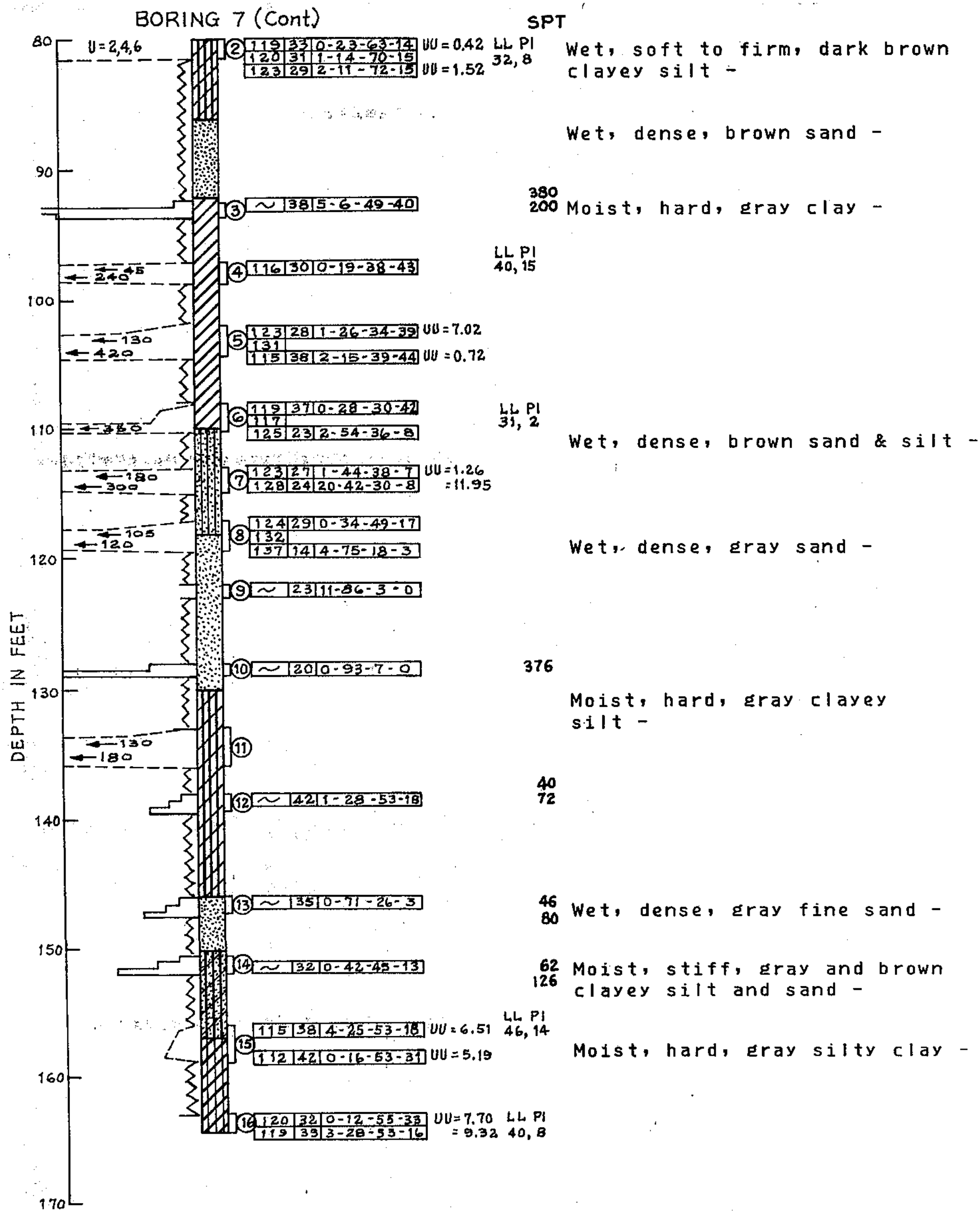
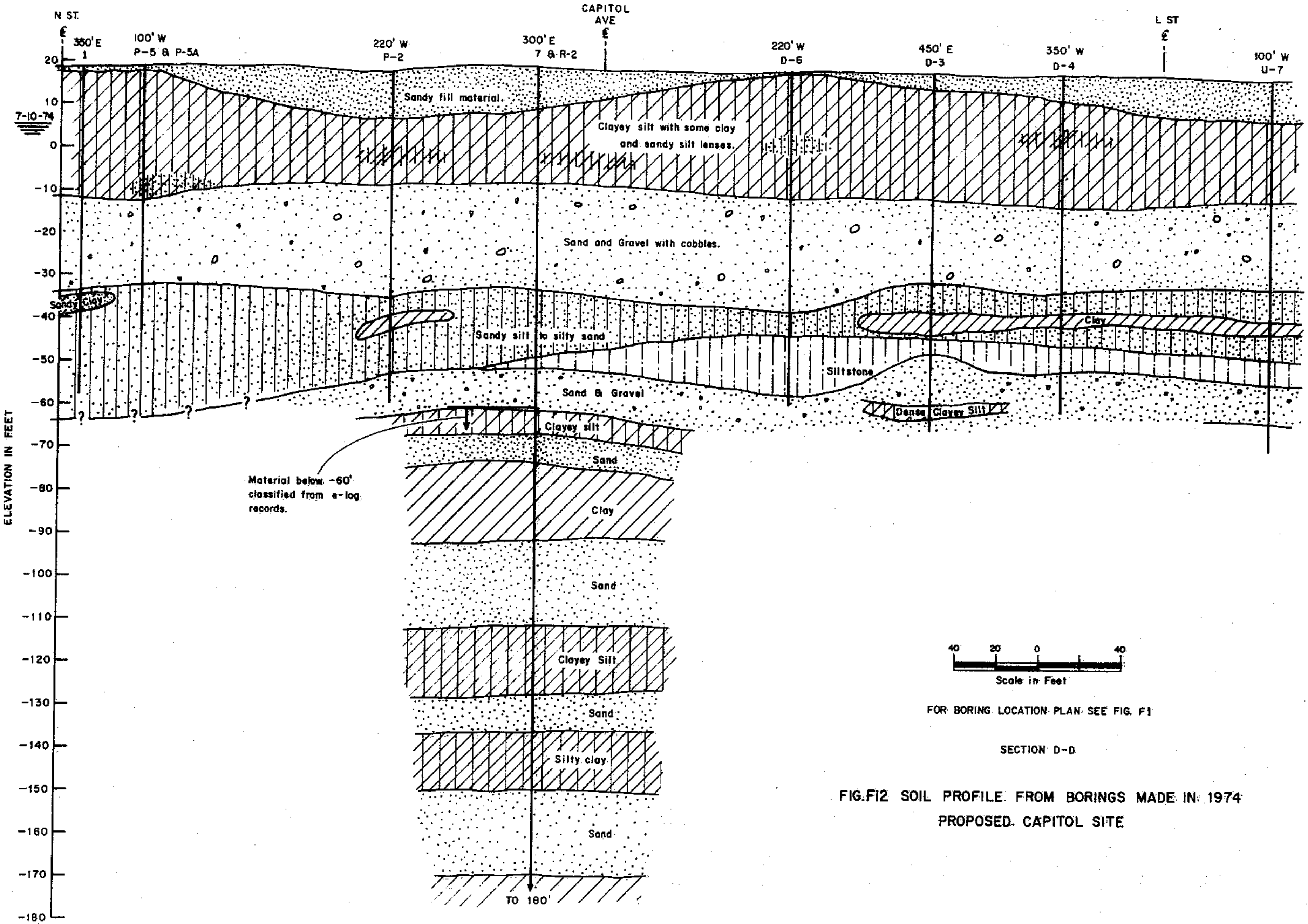



Figure F11 (cont'd)




FOR BORING LOCATION PLAN SEE FIG. F1

SECTION D-D

FIG. F12 SOIL PROFILE FROM BORINGS MADE IN 1974
PROPOSED CAPITOL SITE

varying in thickness from about 8 to 25 feet. This layer includes two clay lenses 2 to 4 feet thick. The silty sand stratum is underlain in the northern portion of the site by a 8- to 12-foot thick hard silt (or siltstone). Underlying the siltstone is another layer of sand and gravel, the thickness of which was determined as about 10 feet at Boring 7. The deep boring, 7, indicates this second sand and gravel layer is underlain by alternating layers of hard, fine-grained soils with dense, wet granular soils extending to depths of over 160 feet below ground surface.

Groundwater elevations were recorded at about Elevation +5 during the period of the exploration in July 1974.

## Geophysical Investigation

Field seismic tests were performed at boring location 7 using several different methods. The tests were conducted in order to determine shear (S) and compressional (P) wave velocities of the various soils underlying the site. Along with Transportation Laboratory personnel and equipment, the Applied Nucleonics Company of Los Angeles supplied a crew to demonstrate recently developed equipment and procedures for geophysical investigation.

The Applied Nucleonics Company of Los Angeles used a vibrator to create standing waves and accelerometers to determine ground wave lengths. The results of their testing are listed in Table F1.

The same standing wave was used by Transportation Laboratory personnel to record shear velocities using geophones and an oscilloscope. The procedure was to use two geophones, one beside the vibrator and another located at a variable distance away.

TABLE F-1

STATE CAPITOL TEST RESULTS

| Test/Run | Frequency (Hz) | Effective Sampling Depth m(ft) | Wave Velocities m/sec (ft/sec) $V_R$ | $V_S$ | $V_P$ | Shear Modulus $N/m^2 x10^{-8}$ (Ksi) | Modulus of Elasticity $N/m^2 x10^{-8}$ (Ksi) |
|---|---|---|---|---|---|---|---|
| 1-3 (a) | 24 | 3.7 (12) | 176 (576) | 190 (622) (c) | - | 0.69 (10.0) | 1.79 (26) |
| 1-2 (a) | 15 | 5.5 (18) | 165 (540) | 178 (583) (c) | - | 0.61 (8.80) | 1.59 (23) |
| 1-1 (a) | 11 | 9.2 (30) | 201 (660) | 218 (713) (c) | - | 0.91 (13.2) | 2.34 (34) |
| 3-2 (b) | 20 | 5.2 (17) | - | 207 (680) | - | 0.83 (12.0) | 2.14 (31) |
| 3-1 (b) | 10 | 11.0 (36) | - | 220 (720) | - | 0.92 (13.4) | 2.41 (35) |
| 2-5 | Explosives | - | - | - | 381 (1250) | - | - |
| 2-4 | Explosives | - | - | - | 360 (1180) | - | - |
| 2-3 | Explosives | - | - | - | 381 (1250) | - | - |
| 2-2 | Explosives | - | - | - | 436 (1430) | - | - |
| 2-1 | Explosives | - | - | - | 305 (1000) | - | - |
| Average Values | - | - | $180 \pm 18$ ($600 \pm 60$) | $200 \pm 22$ $660 \pm 70$) | $375 \pm 23$ ($1225 \pm 25$) | $(7.93 \pm 1.0) x10^7$ ($11.5 \pm 1.5$) | $(2.07 \pm .3) x10^8$ ($30 \pm 4$) |

Notes
(a) Vertical shaking
(b) Horizontal shaking
(c) Calculated using $\nu = 0.3$ and $V_R/V_S = 0.925$

The signals from both geophones were viewed on a dual channel oscilloscope while the travelling geophone was advanced until the two signals were coincident. At that time, the length of the wave was measured on the ground while the frequency was measured on the oscilloscope.

Results were obtained by this method at two frequencies. The wave lengths were 22.3 feet at a frequency of 23.8 Hz and 34 feet at a frequency of 15 Hz. Corresponding shear velocities were 531 and 510 feet per second at depths of 11 and 17 feet, respectively. These results compare well with those obtained by the Applied Nucleonics Company.

A refraction line run at this same location showed a three layer condition, with P-wave velocities of 1100 fps to a depth of 10 feet; 3000 fps for the next 15 feet; and 5400 fps below the 25 feet depth.

Cross-hole compressional and shear wave velocities were obtained to a depth of 220 feet using test holes located approximately 35 feet apart at boring location 7. Waves were generated by detonating blasting caps at various depths. Arrival times were recorded using an Electrotech Model M4E and a three-dimensional detector geophone lowered to the same depth as the energy source. Problems arising from the geophone hole caving below a depth of 110 feet necessitated that remaining tests be conducted with the geophone fixed at this depth. The scatter in subsequent test results, however, led to considerable skepticism regarding their validity. Nevertheless, the proposed site test results are presented in Figure F13.

It is recommended that in selecting shear velocities for use in earthquake ground response analysis, test results for the

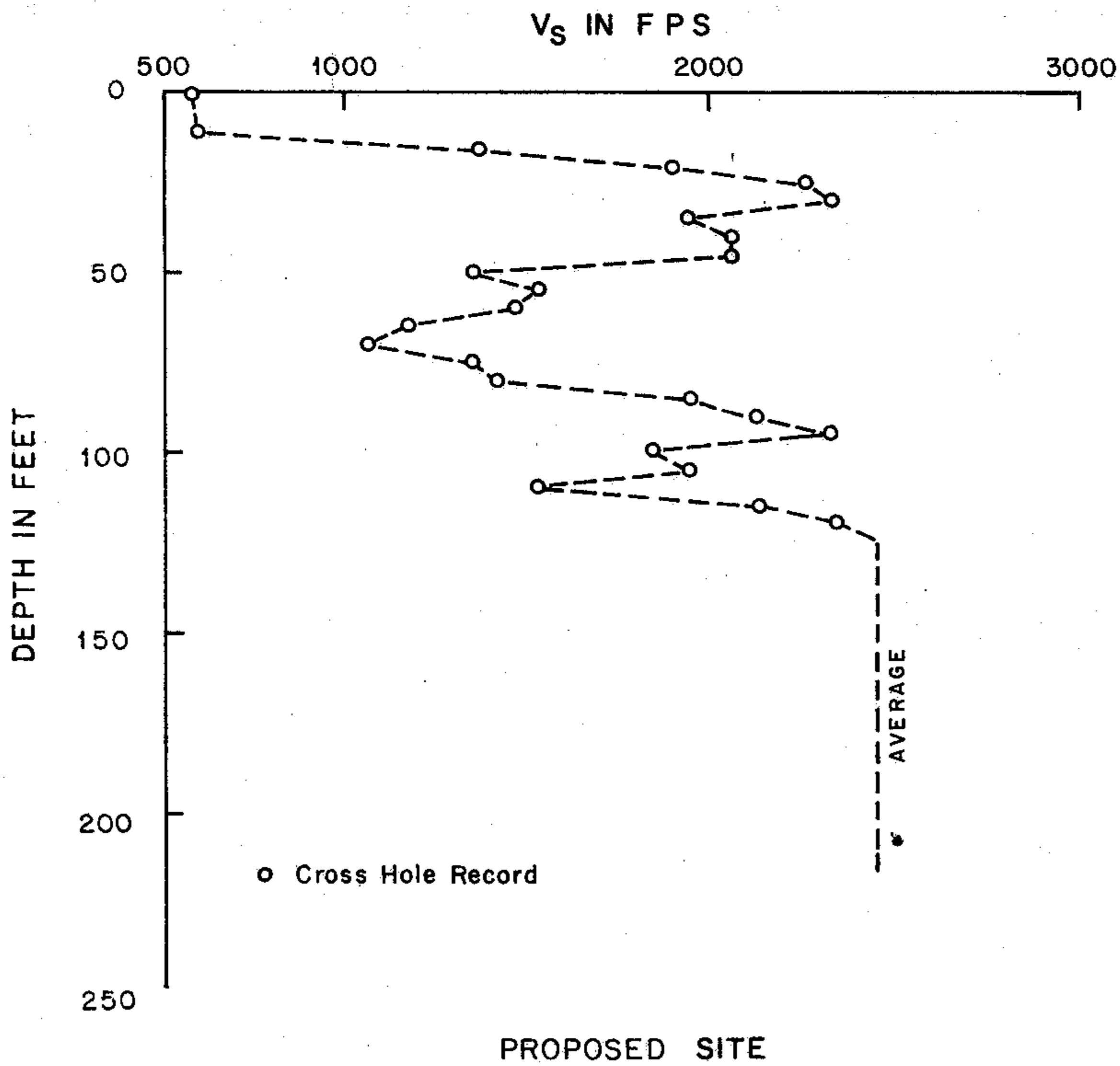



Fig. F13 SHEAR WAVE VELOCITY AVERAGES DETERMINED FROM FIELD GEOPHYSICAL SURVEY METHODS proposed site be accepted to a depth of 110 feet and for depths below 110 feet, test results from the existing site be used. Boring logs for the two locations indicate quite similar soil types and layering below the 110 foot depth. It is believed that any error introduced by assuming identical dynamic soil properties in these common soils is acceptable as compared to that associated with the uncertainties surrounding the cross hole test results for the proposed site.

## Drawdown Test

A pump test to determine construction dewatering requirements and the effects of dewatering on surrounding structures was conducted during July 10-13.

As discussed earlier, (Boring and Sampling) preparations for the drawdown test included boring a 12-inch pump well to a depth of 70 feet at a location just south of the alley bounded by 15th, 16th and N Streets and Capitol Avenue, Figures F14 and F15. Five 4-inch observation wells at varying distances from the pump well, two to the east and three to the north, were also made, Figure F16. All five observation wells were bored to depths of approximately 70 feet. Figure F17 shows the layout of the drawdown test equipment.

Soil conditions at the pump well consist of a silty-clay layer extending to a depth of approximately 27 feet followed by a sand, gravel and cobble aquifer about 23 feet thick. The aquifer is confined from below by an essentially imperveous layer of silty clay. The ground water table was at a depth of 12 feet below ground surface prior to pumping.

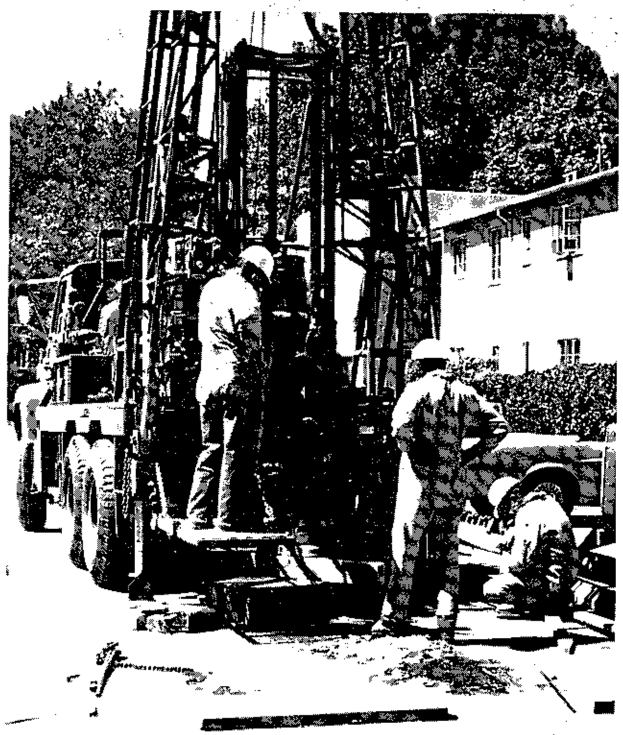


Fig. F14 Pump well drilling operation.

Fig. F15 Measurement of depth to ground water at pump well.

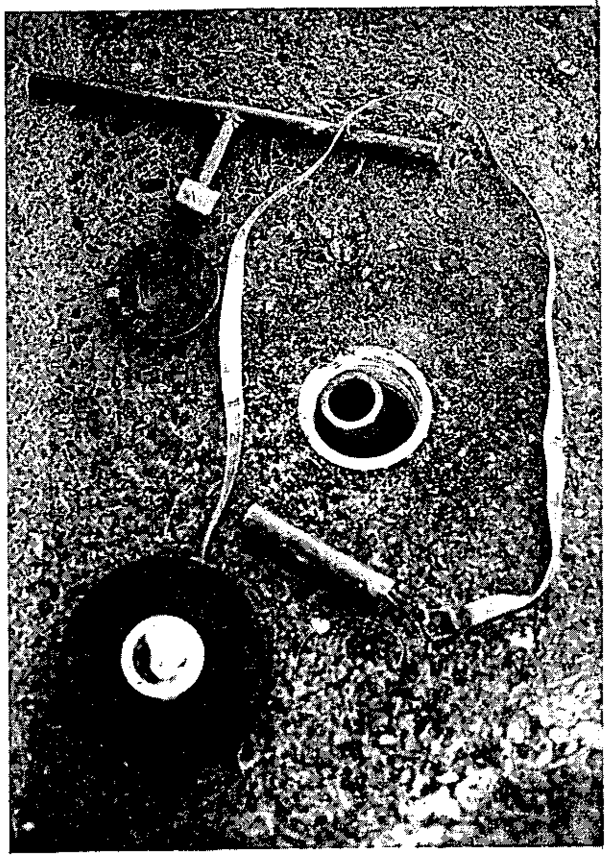


Fig. F16 Observation well and device for measuring depth to water surface.

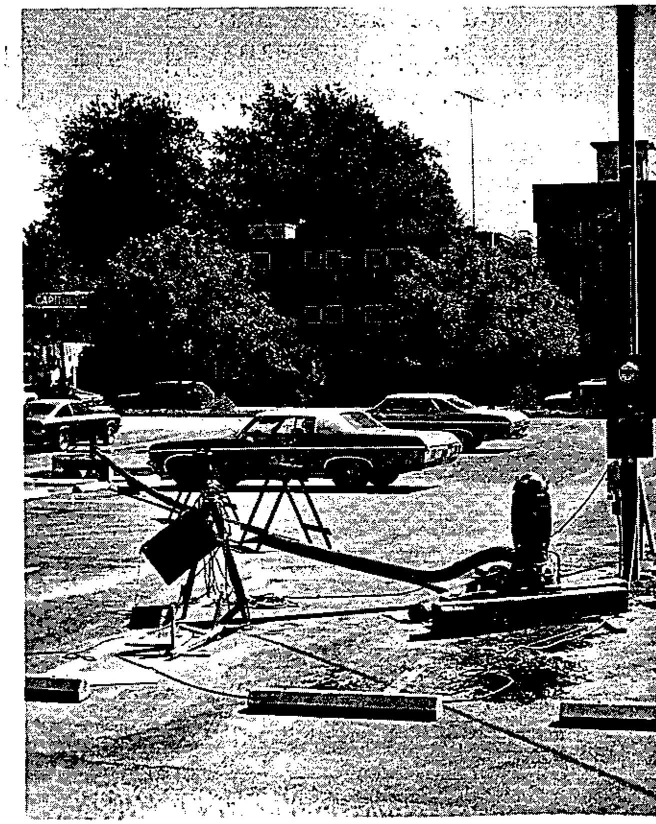


Fig. F17 Layout of the drawdown test equipment.

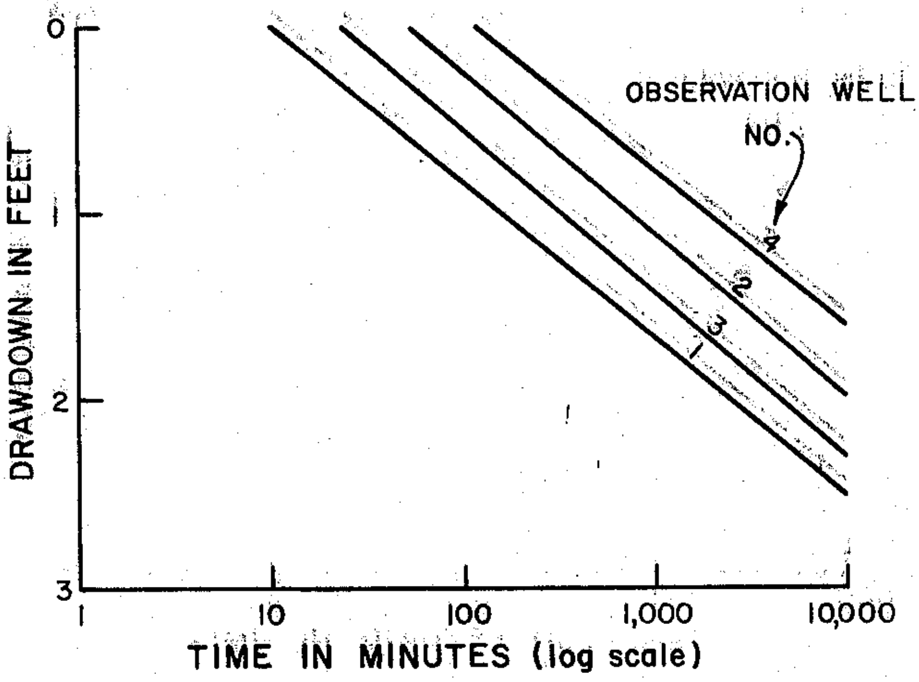



Fig. F18 TIME VS DRAWDOWN CHARACTERISTICS FOR NOTED OBSERVATION WELLS

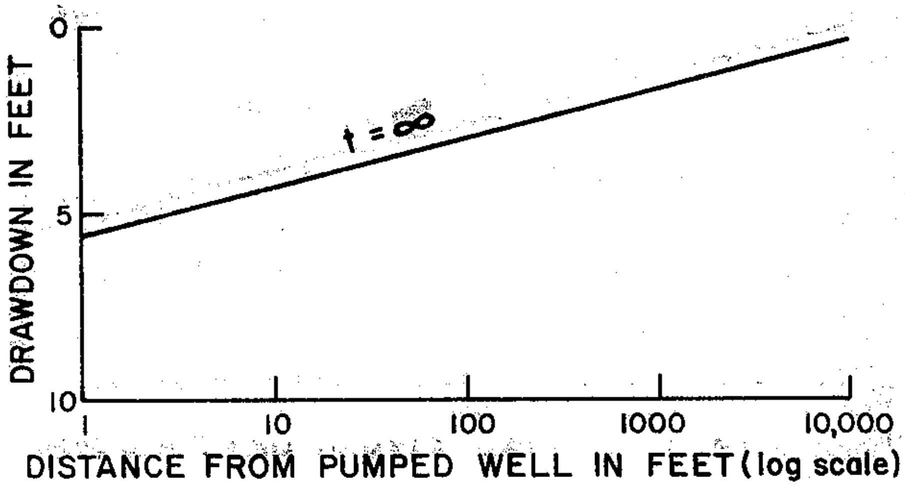



Fig. F19 DRAWDOWN VS DISTANCE FROM PUMPED WELL (PROPOSED SITE)

Pumping at the test well commenced at 1700 on July 10, 1974, and operation continued uninterrupted for 68 hours. The rate of discharge was held nearly constant at 280 gpm. Water level measurements were taken at observation wells and the pump well according to a predetermined time schedule. The first few sets of readings were made at short time intervals, with those in the later stages of the test taken at four-hour intervals. Drawdown verses elapsed time and distance from the pump well are presented in Figures F18 and F19, respectively. Using the Thiem equation and the drawdown data, the permeability of the gravel aquifer was estimated at 700 feet per day. Based on the grading analysis and permeability rates available in the literature for silts and clays, a permeability of 0.1 feet per day has been estimated for the soils under and overlying the aquifer.

Analysis of test data shows that large scale dewatering for construction of a building or depressed streets would result in a very flat drawdown curve extending a considerable distance outside the limits of the proposed site. The associated problem of ground subsidence outside anticipated construction limits was therefore studied.

Prior to the drawdown test, elevations of several fixed points out to a distance of 100 feet from the pump well were measured to determine if the test itself would produce any ground subsidence. Readings made prior to rebound showed no perceptible change in elevations. However, the test drawdown was of very short duration compared to construction periods and was accomplished with only one well. Longer drawdown periods involving lines of wells would be expected to produce measurable ground subsidence, especially in view of the fact subsurface soil conditions are very similar to those some 12 blocks to the west where the Interstate 5 construction dewatering produced subsidence of about one inch.

A quantitative analysis using laboratory consolidation test data showed that 1/2 to one inch of settlement can be expected over an area of several blocks surrounding the site. However, the economic feasibility of such large scale dewatering, considering the extremely pervious nature and recharge characteristics of the sand and gravel aquifer, is doubtful because of the volume of water involved. Therefore, in any dewatering efforts sheet piling would be a very important consideration to limit the lateral extent of drawdown and thereby eliminate or minimize ground subsidence, and also to minimize the amount of water to be pumped. To achieve the desired results, sheet piling about 60 feet in length would be required to completely penetrate the sand and gravel stratum, and pumping from inside the sheeting would be a necessity. The importance of sealing off the aquifer is illustrated by previous experiences during Interstate 5 construction.

# APPENDIX G

## LABORATORY TESTING

APPENDIX G

LABORATORY TESTING

Samples retrieved from the existing and proposed sites for laboratory testing included a variety of materials. Undisturbed soil samples were obtained at both locations using the 2" California sampler. Additional undisturbed soil samples were removed from test pits excavated within the basement of the West Wing. In strata where ~~it~~ was not possible to retrieve undisturbed soil samples, jar samples were taken for visual inspection. Concrete cores from the footings and intact portions of the subsurface brick walls were removed from sections of the West Wing foundation. The following is a discussion of the laboratory procedures used to determine the engineering properties of the materials sampled and a summary of the corresponding test results.

## Soil Classification Testing

Representative samples of the various soils encountered at the two study sites were classified by routine mechanical analyses and Atterberg limit tests. In addition, in place moistures and densities were determined as well as several specific gravities. A summary of these data is shown on the boring records presented in Appendices E and F.

## Soil Strength Testing

Both unconsolidated-undrained (UU) and consolidated-undrained (CU) triaxial compression tests were performed on various undisturbed soil samples for determination of strength characteristics. The UU test specimens were set up at their approximate overburden pressures and sheared, undrained, at an axial strain rate of 1% per minute. CU test samples were set up at their natural water contents and back pressured to approximately 80 psi, while maintaining an effective confining pressure of 3 psi. Consolidation pressures equal to or exceeding effective overburden pressures were applied prior to shearing for a period of at least 24 hours. Pore pressures were monitored continuously during testing, while axial strain rates of approximately 0.3% per minute were maintained.

Loosely compacted fill material comprising the upper soil layers at both study sites was not tested for strength as it was judged unsuitable as a foundation material from the ease of boring and inspection of samples and classification test results.

UU strength capabilities of the naturally deposited soils from the base of the fill material to the groundwater table at both sites averaged 2-4 TSF. Reduced strengths, 0.4-1.0 TSF, were obtained for soils at the water table, with strengths generally increasing below this depth. Moderately fine grained soils (silts and silty clays) from both sites exhibited reasonably consistant results, with UU strengths of at least 4 TSF and some exceeding 10 TSF. Inconsistent and generally low UU results were obtained for the sandier soils. This is presumed to be a result of sample disturbance.

CU testing was restricted to samples retrieved from depths of 16-30' at the existing site. Material here is nearly identical to that at the same depth at the proposed site, hence, it is assumed that strength characteristics are similar. Total stress test results indicate an average cohesion (c) of 0.3 TSF. Angle of internal friction, Ø, varied from a low of 14°. Effective stress results indicate an average Ø' of 34°.

## Lateral Earth Pressure at Rest

Coefficients of lateral earth pressure at rest ($K_o$) were determined from undisturbed 2" tube samples taken from the 15-foot depth at the existing site. These samples were extracted from above the water table, but were saturated by backpressuring prior to testing to simulate critical peak groundwater conditions. Samples were consolidated to their effective overburden conditions. Tests were conducted by compressing the specimens in a drained state while constantly monitoring the displaced sample volume. Chamber pressure was adjusted manually to maintain original cross-sectional area (Poisson's ratio equal to zero). $K_o$ values were estimated from the following relationship:

$$K_o = \sigma_3'/\sigma_1'$$

These values were computed for an axial strain range to 0 to 1.5%. The resulting average $K_o$ value was 0.37. It is assumed that this value applies to the material from a depth of 12 to 30 feet, as this strata is considered fairly homogeneous.

## Consolidation Testing

Consolidation tests were conducted on undisturbed 2" tube samples retrieved from various depths below the proposed and existing sites. Samples were inundated and loaded at 24-hour time increments, with loads increasing from 1/4 to 8 tons per square foot. Time-settlement and e-log p plots were prepared and maximum past pressures were estimated in order to determine over-consolidation ratios. Rebound characteristics were estimated from the unloading cycle of the e-log p plots.

Consolidation test results were erratic, probably due to sample disturbance. Indications are that the upper silts and clayey silts (10'-50' depth) are over-consolidated with over-consolidation ratios (OCR) varying from 1 to 3. Investigation, however, revealed that no known past geological or physical event has occurred which would substantiate these results. Therefore, settlement calculations were based on those test results indicating normally consolidated materials. Initial void ratios for the sandy-silts were approximately 0.7 and those of the silty clays approximately 1.1. Rough approximations of the coefficients of consolidation ($C_v$) for the silty clays and sandy silts are 4 and 30 ft.$^2$/day, respectively. Estimated permeability rates are 0.001 and 0.01 ft./day, respectively.

## Testing of West Wing Foundation Samples

A total of four brick assemblies with brick-mortar bonds intact were obtained from the West Wing foundation walls. As shown in Figure G1, each assembly consisted of three bricks, two on the bottom and one overlapping the lower butt joint. Three of these assemblies were failed in shear and the fourth in compression. Before testing, excess mortar was carefully removed from the free surfaces of the bricks. The lower bricks were partially embedded in the high strength, fast setting compound to provide uniform bearing. The top brick was capped with the same compound, and a steel plate the same size as the brick was embedded in the capping compound to assure uniform load distribution, Figure G2. During shear testing the lower bricks were restrained laterally, while a normal force was applied to the steel plate and an increasing shearing force was applied to the end of the upper brick until failure, Figure G3. For the compression test the normal force was increased until failure occurred, Figure G4.

The three shear tests were conducted using normal forces of 0, 30, and 60 psi. The results of these tests are listed in

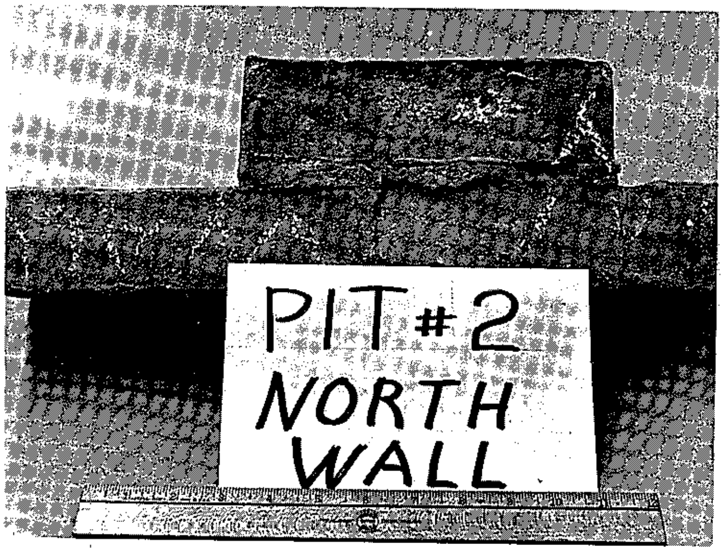



Figure G1 Section of West Wing foundation wall.

Figure G2 Foundation wall section just prior to shear testing.

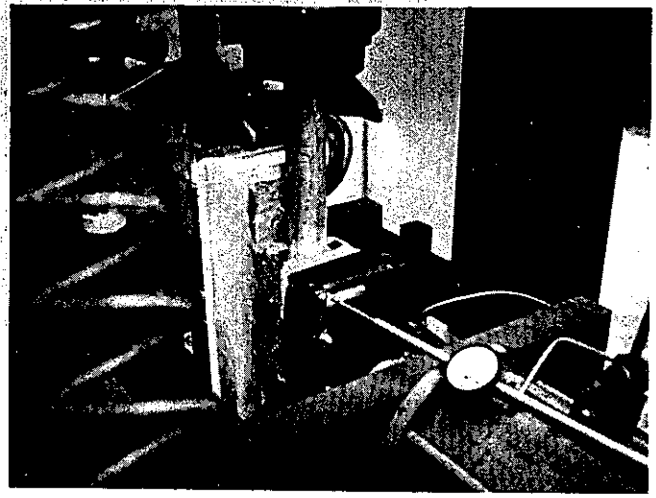


Figure G3 Foundation wall section during shear testing.

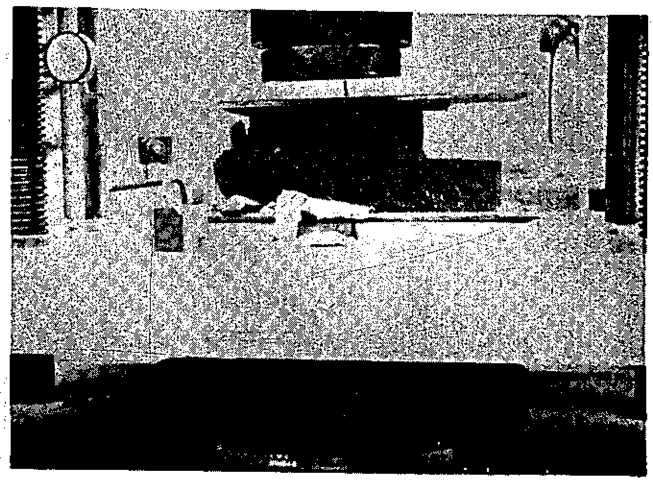


Figure G4 Foundation wall section following compression testing.

Table G1. Prior to testing at 60 psi normal force, visual examination of the brick assembly revealed that approximately half of the brick-mortar bond area was fractured. This is a likely explanation for the low shear strength listed in Table G2.

Compressive strengths of individual bricks, two from Test Pit No. 1 and two from Test Pit No. 2, were also determined. These bricks were tested in a horizontal orientation, as shown in Figure G5. The results are given in Table G2.

Two 8-inch diameter concrete cores were tested for compressive strength. One core was taken from the concrete footing supporting the north wall at Test Pit No. 1, and the other core was taken from the concrete footing supporting the west wall at Test Pit No. 2. The concrete was obviously of poor quality with numerous voids. Flat bearing surfaces at the ends of the cores were prepared using the high strength casting compound mentioned earlier, Figure G6. Figure G7 shows one of the cores just prior to testing. The test results for both cores are presented in Table G3.

It should be emphasized that the concrete cores and brick assemblies tested in the laboratory may not be representative of the overall quality of foundation materials. For example, some 8 to 10 unsuccessful attempts (at different locations along the exposed footings) were made to retrieve concrete cores. The cores actually retrieved, therefore, may be of somewhat better quality concrete that would be generally found in the footings. Also, one of the brick assemblies separated at the mortar joint during transportation to the laboratory. Inspection of the separation faces revealed that a large void had existed where the mortar had not completely covered the brick. The quality

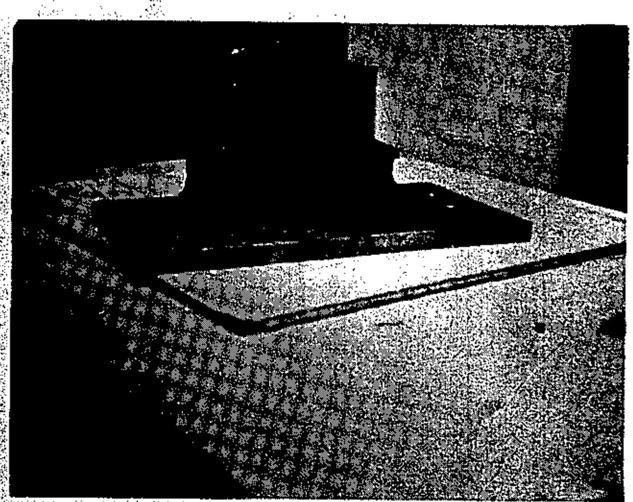


Figure G5 Test specimen following individual brick compression test.

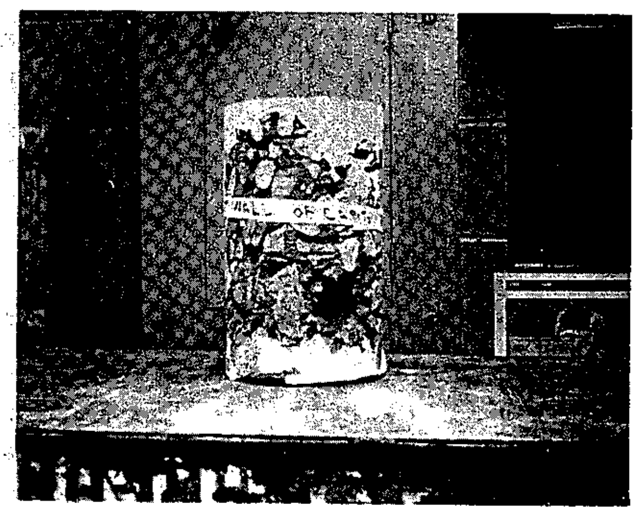


Figure G6 West Wing foundation footing core after capping with casting compound.

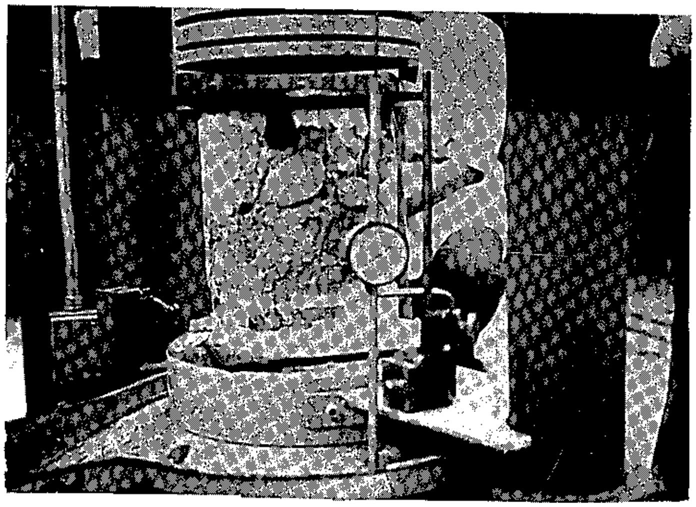


Figure G7  Footing core in compression testing.

Table G1 Brick Assembly Test Results

| Type of Test | Normal Stress psi | Ultimate Shear Stress psi | % CaO in Mortar | Compressive Strength psi | Secant Modulus at 1/2 Ultimate psi |
|---|---|---|---|---|---|
| Shear[1] | 0 | 40 | 12.3 | - | - |
| Shear[1] | 30 | 86 | 15.5 | - | - |
| Shear[1] | 60 | 87 | 15.2 | - | - |
| Compression[2] | - | - | - | 1,200 | 65,000 |

[1]Shear tests were conducted on assemblies from Pit No. 2

[2]Compression test was conducted on assembly from Pit No. 1

Table G2 Individual Brick Test Results

| Specimen Identification | Compressive Strength psi | Secant Modulus at 1/2 Ultimate Stress psi |
|---|---|---|
| Pit No. 1 | | |
| Brick 1 | 3,200 | 160,000 |
| Brick 2 | 1,850 | 110,000 |
| Pit No. 2 | | |
| Brick 1 | 3,100 | 230,000 |
| Brick 2 | 4,800 | 260,000 |

Table G3 Concrete Footing Core Test Results

| Core Location | Ultimate Compressive Strength psi | Section Modulus at 1/2 Ultimate psi | Cement Content Sacks/Yd.$^3$ | Unit Weight lbs./ft.$^3$ |
|---|---|---|---|---|
| North Wall (Test Pit 1) | 580 | 300,000 | 2.5 $\pm$ 1/2 | - |
| West Wall (Test Pit 2) | 290 | 370,000 | 2.6 $\pm$ 1/2 | 126 |

of the mortar itself (a lime mortar, according to a report by VTN Consolidated, Inc., dated March, 1973) and the workmanship associated with it appeared to be the factors detracting from the quality of the brickwork.

# APPENDIX H

## GROUND MOTION ANALYSIS

## APPENDIX H

## GROUND MOTION ANALYSIS

Most of the current methods for determining the effects of local soil conditions on ground response during an earthquake are based on the upward propagation of shear waves from the underlying rock foundation. Analytical procedures which incorporate nonlinear soil behavior have yielded results in general agreement with field observations, particularly at sites where the depth to bedrock is large and changes gradually over a large area. The development of these procedures has been extremely valuable to engineers by providing an analytical method of estimating the amplitude and frequency charactertistics of ground surface motion at a specified site prior to actual seismic excitation.

### Method of Analysis

The ground response investigation presented herein extensively utilized the computer program "SHAKE 3" (1). The program, developed by Schnabel, Lysmer, and Seed, computes the response associated with the upward propagation of shear waves through a system of homogeneous, viscoelastic layers of infinite horizontal extent. The basis of the program is the continuous solution to the wave-equation adopted for use with transient motions through the Fast Fourier Transform Alogarithm. The nonlinearity of the shear modulus and damping is accounted for by the use of equivalent linear soil properties using an iterative procedure to obtain strain values compatible with modulus and damping. In general, the analytic procedure employed to conduct a ground response analysis incorporated the following steps:

1. Determine the characteristics of bedrock motion likely to develop at the site due to the project design earthquakes using empirical relationships between the site-causative fault distance and the motion parameters; maximum acceleration, predominant period, and duration of strong motion.

2. Select a number of accelerograms with motion characteristics similar to those computed for bedrock at the site from strong motion accelerograms based on previous earthquakes or generated artificially.

3. Determine the soil stratification at the site and the dynamic soil properties of the various materials (field and laboratory testing).

4. Adjust the motion parameters to fit those of bedrock at the site for each of the accelerograms individually. Use the resulting time-history of motion as the input earthquake in "SHAKE 3" together with the site profiles.

Soil Conditions

Although extensive field exploration indicated essentially the same pattern of soil stratification at both locations, the divergent foundation requirements of the two alternate Capitol construction concepts dicate that the seismic ground response of the sites be evaluated separately. The high degree of similarity in findings between exploratory borings, however, did facilitate the consolidation of site soil conditions into two representative soil profiles, one for each site, Figure H1. The depth to rock-like material in each case was estimated to be in the range of 250 to 350 feet. Dynamic soil properties were established from the results of field and laboratory tests which were discussed earlier in this report.

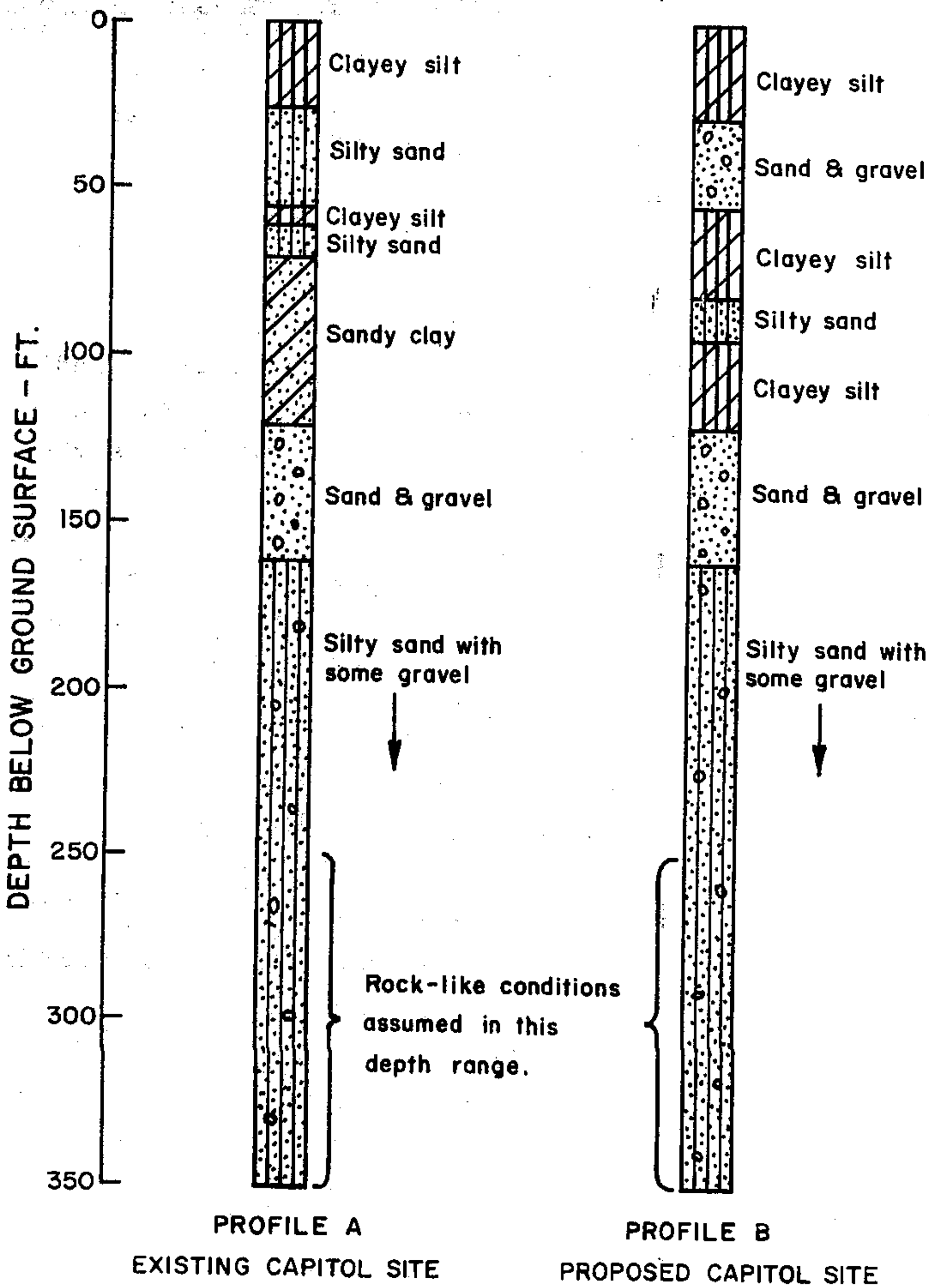
## Bedrock Excitation Motion

Accelerograms based on actual records from Taft, Castaic and El Centro were selected for use as the basis for developing bedrock excitation motions representing D.E. 1 (Design Earthquake, see Appendix B). To represent D.E. 2, accelerograms based on Taft, Pasadena, and El Centro records were used. The parameters of the earthquake records were modified to simulate bedrock motion expected in the vicinity of the Capitol due to the design earthquakes. Table H1 summarizes the required adjustments.

TABLE H1

EXCITATION MOTION ADJUSTMENTS

| Design Earthquake | Acceleration Record Title | Maximum Acceleration Original | Maximum Acceleration Adjusted | Predominant Period Original | Predominant Period Adjusted |
|---|---|---|---|---|---|
| | Taft | .44 | .18 | .35 | .35 |
| D.E. 1 | Castaic | .27 | .18 | .45 | .35 |
| | El Centro | .30 | .18 | | .35 |
| | Taft | .44 | .07 | .35 | .55 |
| D.E. 2 | Pasadena | .02 | .07 | .65 | .55 |
| | El Centro | .30 | .07 | .20 | .55 |




SOIL PROFILES USED IN GROUND MOTION ANALYSIS

Figure H-1

## Ground Response

The modified earthquake records were used as the bedrock excitation motions applied to the soil profiles representing soil conditions at the site. Information obtained from the resulting ground response included; surface ground motion and the associated response spectra, maximum acceleration at soil layer boundaries and soil layer stress-time histories.

## Structural Response Spectra

A response spectrum is an effective way of portraying the maximum potential effect of a given ground surface motion on different possible structural systems. A response spectrum is a plot of the maximum acceleration, velocity or displacement of a set of single degree of freedom structures of varying natural period and damping to a specified base excitation motion. The result is a family of curves for different values of damping and the peak values of the velocity and acceleration curves tend to identify the predominant period content of the base motion. In this study, response spectra are presented which represent the direct numerical average of the response spectra generated using each of the three separate motions which had been scaled to yield the "same" design earthquake. Separate average spectra are presented for each damping value and each soil profile. It is the considered opinion of the authors that each individual spectrum has peaks associated with "local resonances" unique to the digitized record and mathematical model utilized which should not be considered alone. The average spectrum represents a more realistic assessment of the maximum probable responses at the site. In the remaining sections of this report these average response spectra will be referred to simply as "response spectra".

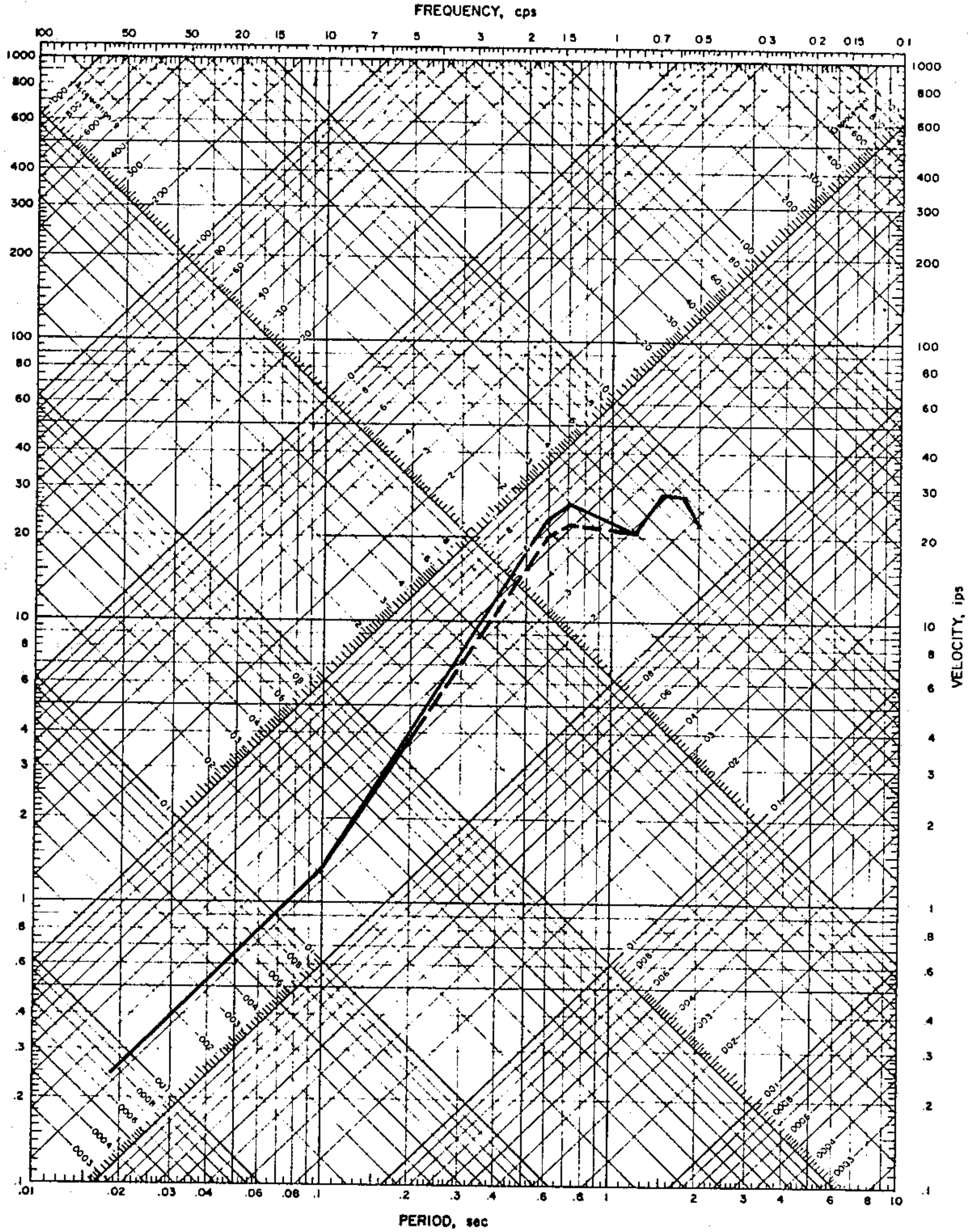
## Results of Ground Motion Analysis

Due to the uncertainty over the depth to "rock like material" at the study sites, an initial study was undertaken to determine the significance of a variable bedrock depth. A comparison has been made of response spectra generated by applying the excitation motions representative of the D.E. 1 event to Profile A, assuming bedrock depths of 250 and 350 feet. A value of 5% structural damping was used for both spectra. Figure H2 presents this comparison. Note that these response spectra vary significantly only over a range of periods from .6 to 1.0 seconds. The maximum acceleration differential between curves is approximately 20%, with the curve representing a bedrock depth of 350 feet exhibiting the larger spectral values. From these results it was decided that for the remaining portion of the ground response analysis a depth to bedrock of 350 feet would be assumed.

Tables H2 and H3 summarize the results obtained from the ground response analysis of the existing and proposed sites. It should be pointed out, that in all cases surface acceleration reflects a slight amplification of input bedrock motion. Furthermore, the degree of amplification is greatest in connection with the large distant earthquake, D.E. 2.

Figures H3 through H6 present the response spectra associated with the data listed in Table H2 and H3. Structural damping curves of 2%, 5%, and 10% are present for each spectra.

The results presented in Figure H7 represent a comparison of structural response at ground surface vs. a depth of 28 feet at the proposed site during the D.E. 1 event (5% damping). These data is associated with possible relatively deep embedment of the Capitol structure being considered for this location, and indicates that no significant difference in response spectra is obtained by using the time history of motion at each level.



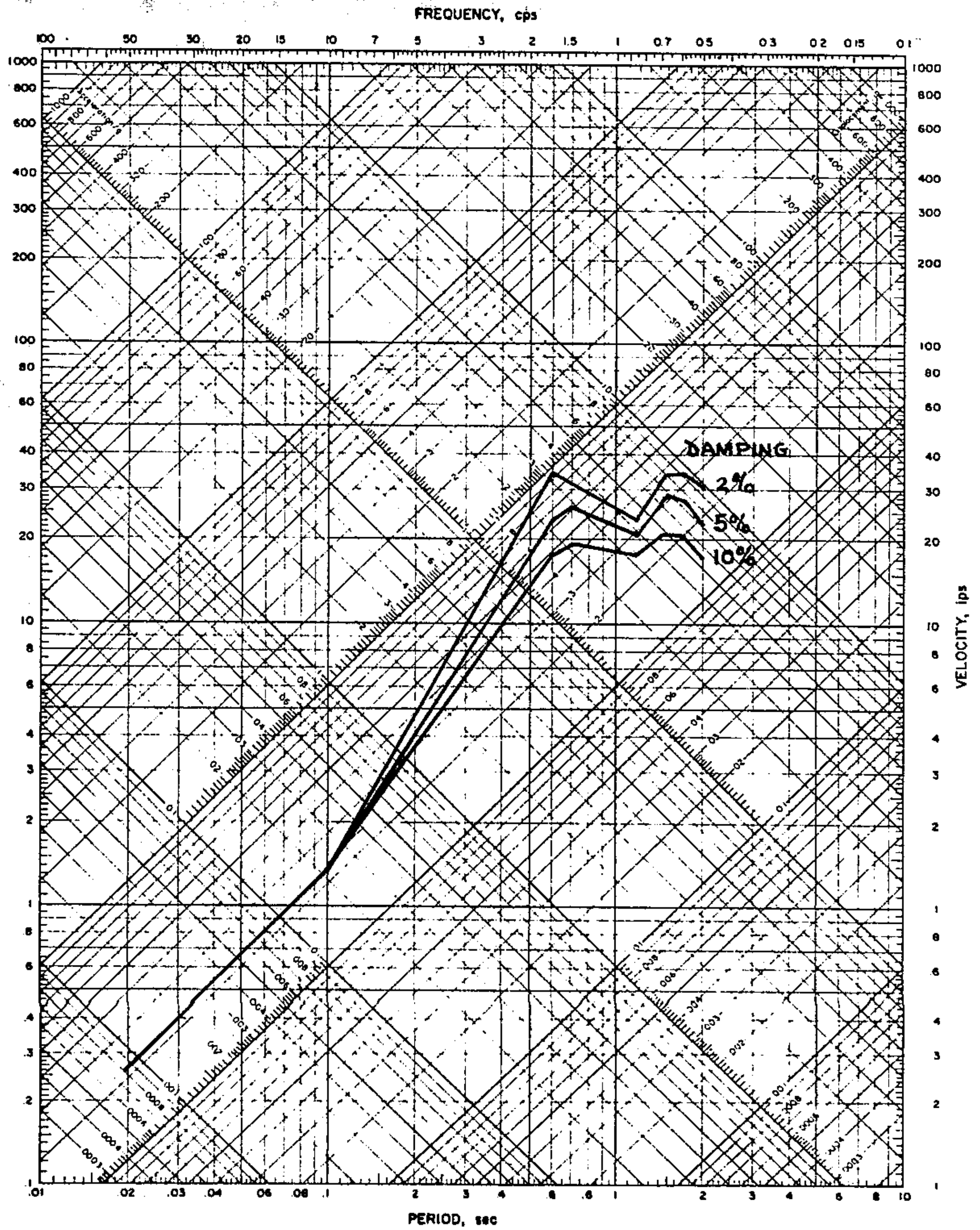

# COMPARISON OF RESPONSE SPECTRA

## BEDROCK DEPTH 350 FT. VS 250 FT.

Profile A-D.E.I (Midland) —— depth to bedrock = 350'

5% damping — — depth to bedrock = 250'

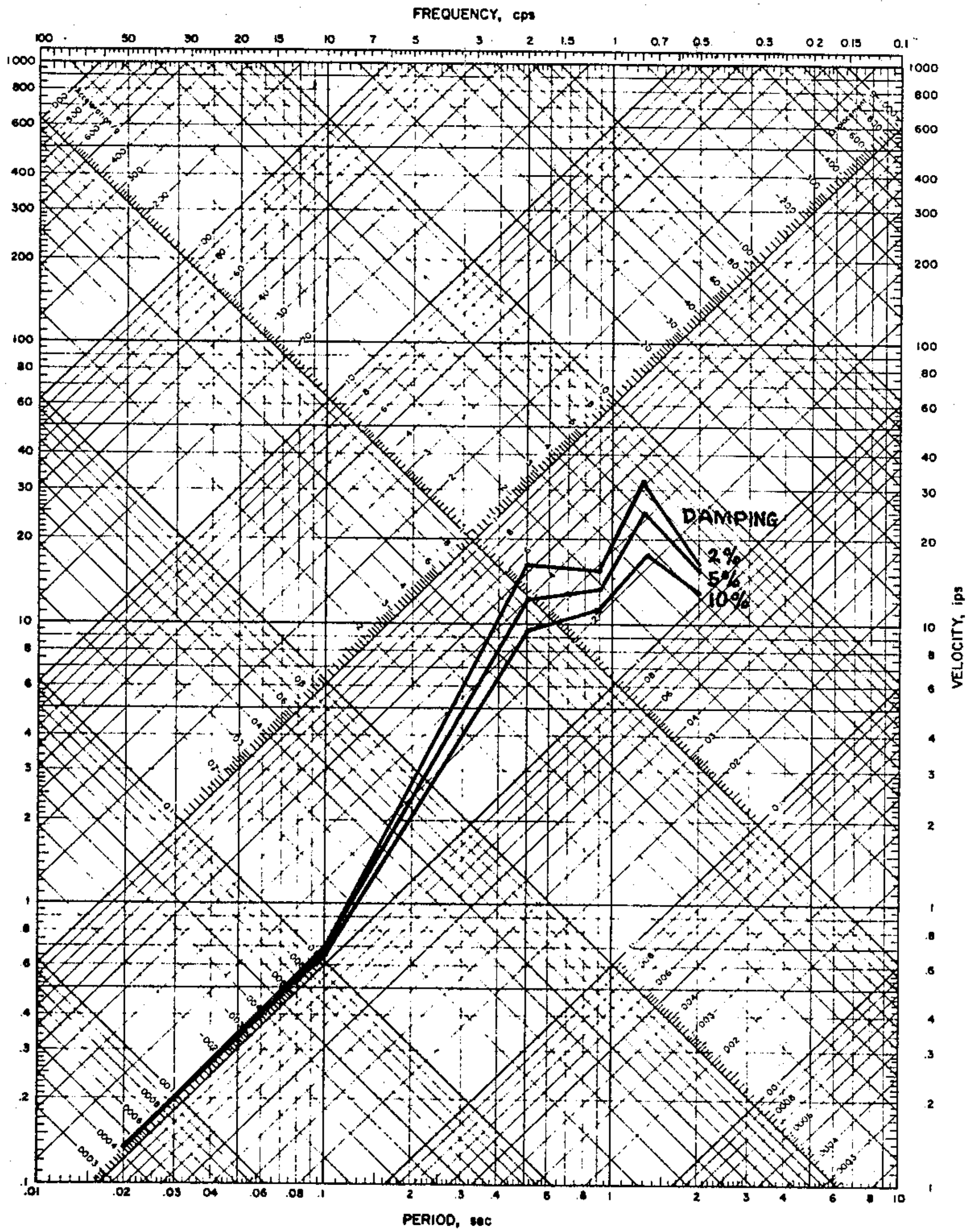
Figure H2




**RESPONSE SPECTRA**

**Profile A-D.E.1 (Midland)**

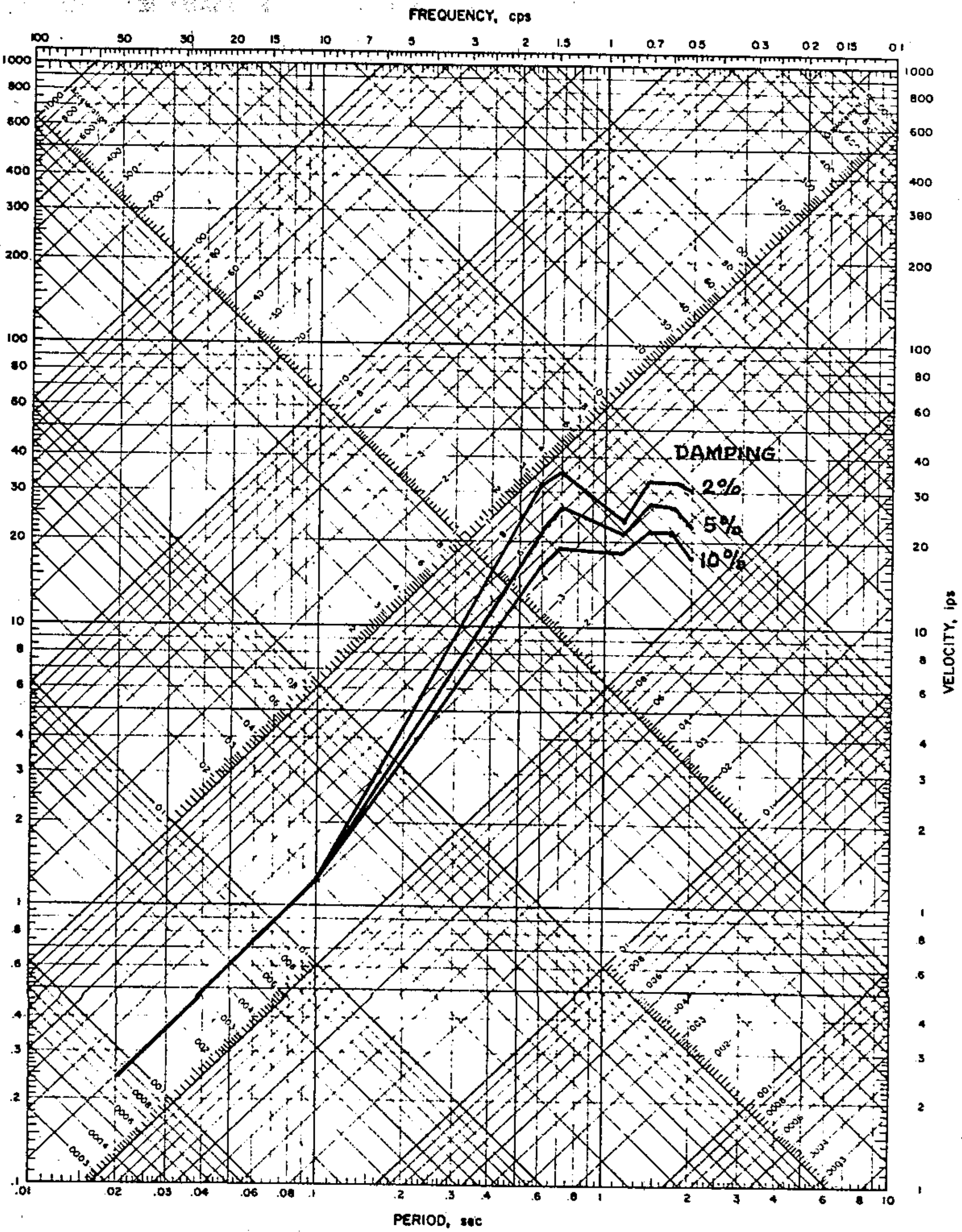
**Figure H3**




# RESPONSE SPECTRA

## Profile A-D.E. 2 (San Andreas)

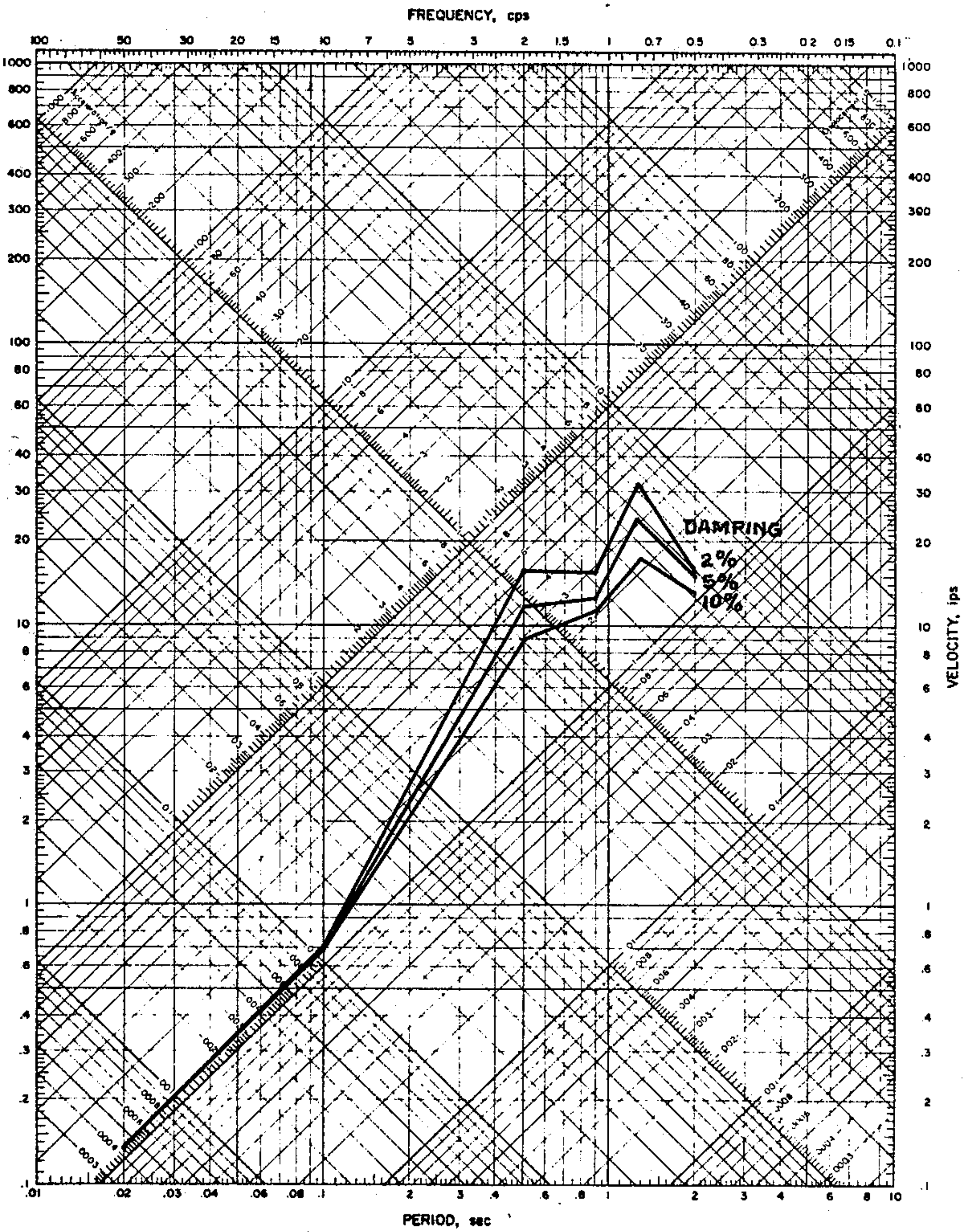
**Figure H4**




# RESPONSE SPECTRA

## Profile B-D.E.I (Midland)

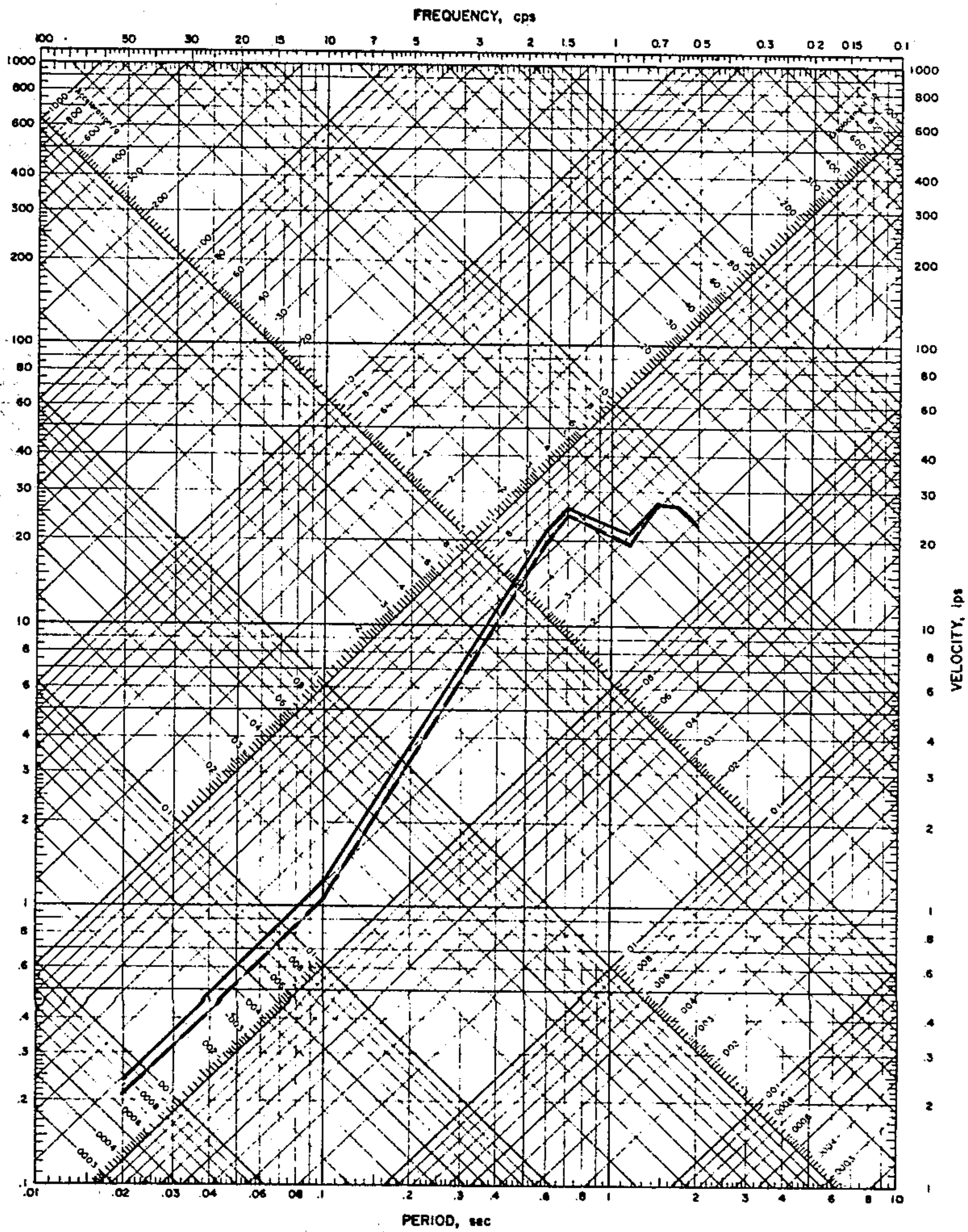
Figure H5




# RESPONSE SPECTRA

**Profile B-D.E.2 (San Andreas)**

**Figure H6**




# COMPARISON OF RESPONSE SPECTRA
## GROUND SURFACE VS 28 FT. DEPTH

Profile B-D.E.1 (Midland)
5% damping

—— response at surface
— — response at 28 ft. depth

Figure H7

TABLE H2

SUMMARY OF GROUND RESPONSE

PROFILE A

| DESIGN CHARACTERISTICS | | | RECORD AND MODIFICATIONS | | | OUTPUT | |
|---|---|---|---|---|---|---|---|
| DESIGN EARTHQUAKE No. | RICHTER MAGNITUDE | DISTANCE TO FAULT FROM SITE (miles) | DIGITIZED EARTHQUAKE RECORD USED | APPROX. MAXIMUM ROCK ACCEL-ERATION (g) | APPROX. PREDOM-INANT PERIOD (sec) | MAXIMUM GROUND SURFACE ACCELERATION (g) | NATURAL PERIOD OF SOIL DEPOSIT (sec) |
| D.E. 1 | 7.0 | 24 | Taft | .18 | .35 | .20 | 1.731 |
| D.E. 1 | 7.0 | 24 | Castaic | .18 | .35 | .23 | 1.767 |
| D.E. 1 | 7.0 | 24 | El Centro | .18 | .35 | .23 | 1.738 |
| D.E. 2 | 8.25 | 80 | Taft | .07 | .55 | .12 | 1.445 |
| D.E. 2 | 8.25 | 80 | Pasadena | .07 | .55 | .14 | 1.522 |
| D.E. 2 | 8.25 | 80 | El Centro | .07 | .55 | .11 | 1.409 |

TABLE H3

SUMMARY OF GROUND RESPONSE

PROFILE B

| | DESIGN CHARACTERISTICS | | RECORD AND MODIFICATIONS | | | OUTPUT | |
|---|---|---|---|---|---|---|---|
| DESIGN EARTHQUAKE No. | RICHTER MAGNITUDE | DISTANCE TO FAULT FROM SITE (miles) | DIGITIZED EARTHQUAKE RECORD USED | APPROX. MAXIMUM ROCK ACCEL- ERATION (g) | APPROX. PREDOM- INANT PERIOD (sec) | MAXIMUM GROUND SURFACE ACCELERATION (g) | NATURAL PERIOD OF SOIL DEPOSIT (sec) |
| D.E. 1 | 7.0 | 24 | Taft | .18 | .35 | .20 | 1.763 |
| D.E. 1 | 7.0 | 24 | Castaic | .18 | .35 | .21 | 1.810 |
| D.E. 1 | 7.0 | 24 | El Centro | .18 | .35 | .19 | 1.763 |
| D.E. 2 | 8.25 | 80 | Taft | .07 | .55 | .11 | 1.467 |
| D.E. 2 | 8.25 | 80 | Castaic | .07 | .55 | .14 | 1.553 |
| D.E. 2 | 8.25 | 80 | El Centro | .07 | .55 | .11 | 1.430 |

## Summary and Conclusions

The response at the two proposed Capitol construction sites has been evaluated for two postulated design earthquakes. The magnitude and distance of each earthquake was established based on available geologic features and historical data. These design earthquakes, D.E. 1 and D.E. 2, represent maximum credible events occurring along the Midland and San Andreas fault systems, respectively.

The maximum acceleration in rock-like material underlying the study sites was estimated at .18 g. Maximum ground surface accelerations of .21 g at the existing site and .23 g at the proposed site were computed. In both cases the maximum values were in response to the D.E. 1 earthquake.

It is recommended that the response spectra presented in Figures H3 through H6 be used as the basis for estimating the seismic design levels for the proposed structures.

Natural periods of soil deposits at the two sites ranged from 1.7 to 1.8 seconds for D.E. 1 earthquake and 1.4 to 1.5 seconds for D.E. 2 event. Predominant periods of motion for these earthquakes were .35 and .55 seconds, respectively. The spacing of the predominant periods of earthquakes and fundamental periods of soil profiles would indicate no "unusual soil amplification effect", although, relatively longer period structures may develop larger force levels than historical data indicate for the area.

## References

1. Schnabel, P. B.; Lysmer, J.; and Seed, H. B.; SHAKE, A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites, University of California, Berkeley, Report No. EERC 72-12, 1972.

# APPENDIX I

## LIQUEFACTION AND SEISMIC DENSIFICATION POTENTIAL

# APPENDIX I

## LIQUEFACTION AND SEISMIC DENSIFICATION POTENTIAL

This appendix presents an evaluation of the potential for seismically induced liquefaction and excess transient pore pressure build-up in soil strata below the water table at both sites. Seismic densification potential of the sand and gravel stratum at the proposed site, and of the surface fill layer common to both sites was also evaluated.

### LIQUEFACTION AND EXCESS TRANSIENT PORE PRESSURE

Liquefaction and transient pore pressures were studied by comparing estimated field seismic stress values with stress values determined from laboratory cyclic load tests. Field seismic stress levels were estimated by two methods; a simplified procedure based on an average but uniform number of stress cycles associated with a design earthquake, and a second method employing a shear stress time history determined by computer utilizing a recorded accelerogram. Laboratory tests were conducted on 27 soil samples recovered from depths between 15 and 55 feet below ground surface. Most of the samples tested were sandy silts as shown by the gradation curves of Figure I1.

#### Cyclic Load Tests

The cyclic load testing procedure consisted of first applying a backpressure of 4 KSC (kilograms per square centimeter) to

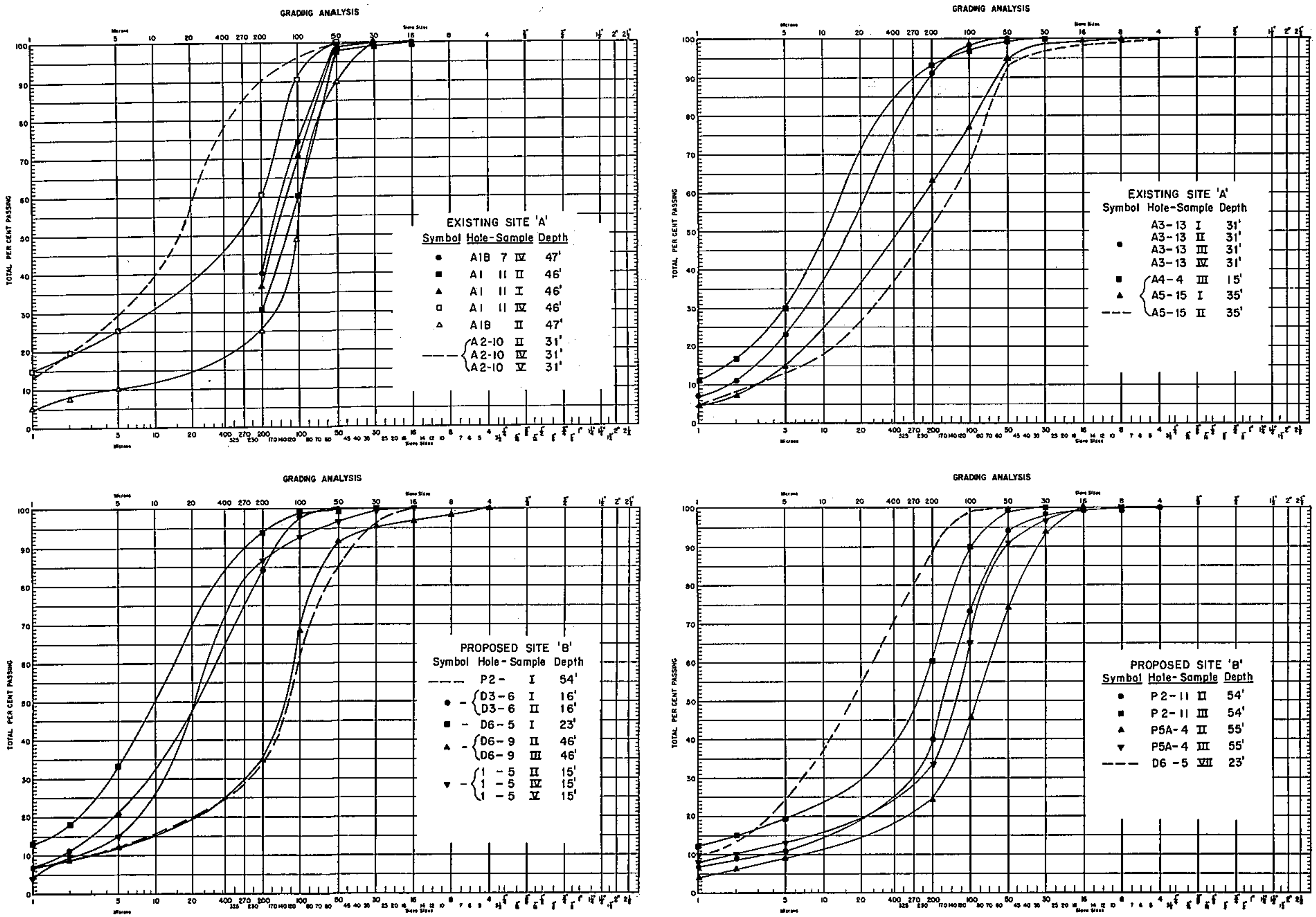



FIG. II GRADING CHARACTERISTICS OF UNDISTURBED SAMPLES TESTED UNDER LABORATORY CYCLIC LOADING CONDITIONS each sample to ensure saturation, and then allowing it to consolidate isotropically to its field effective overburden pressure ($\sigma_{3c} = \bar{\sigma}_o$) . A symmetrical pulsating deviator stress ($\pm\sigma_{dc}$) was then applied and tracings were recorded of the variations in sample pore pressure, deviator load, and axial strain during the duration of the test. To minimize errors, a single transducer was used to monitor chamber and pore water pressures.

The testing program was divided into series, a series being comprised of samples from the same soil layer. The samples in a given series were subjected to pulsating loads of different magnitudes until axial strains (double amplitude) of 20 percent were reached. The number of stress cycles, N, required to produce axial strains of 2, 5, 10, and 20 percent were taken from the tracings recorded during the tests, and plotted versus the cyclic stress ratio, ($\sigma_{dc}/2\sigma_{3c}$). Also, the pore pressure ratio,($\Delta u/\sigma_{3c}$) , was plotted against N to illustrate pore pressure build-up during cyclic loading. A typical plot of both relationships is shown in Figure I2.

To compensate for inadequacies in cyclic load testing equipment and procedures, differences between field and laboratory stress conditions, and relative density variations, Seed and Peacock (Ref. 1) have developed a correction factor for laboratory cyclic stress ratios. The "corrected" or "adjusted" cyclic stress ratios are used in analytical studies. A correction factor of 0.6 was determined for this investigation.

## Estimating Field Stress Ratios

Two methods were used to estimate field stress ratios during design seismic loading. The first (Simplified Method) was

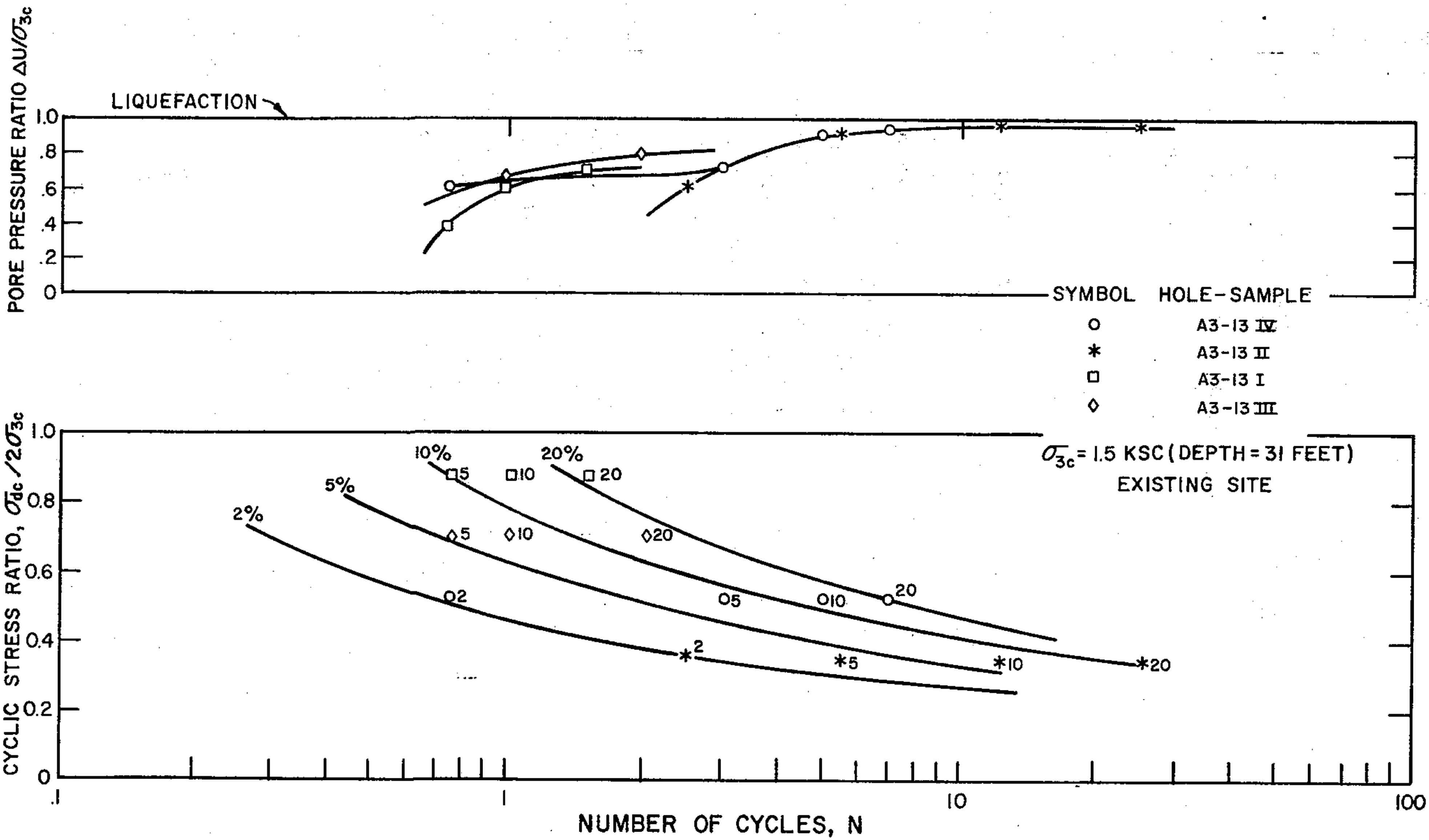



Fig.I2 RELATIONSHIP BETWEEN LABORATORY CYCLIC STRESS RATIO AND PORE PRESSURE RATIO VERSUS NUMBER OF CYCLES TO YIELD VARIOUS % OF AXIAL STRAINS.

developed by Seed and Idriss (Ref. 2) and considers earthquake magnitude and maximum ground acceleration. This method associates an average number of uniform equivalent stress pulse cycles to a design earthquake, thus reducing random acceleration pulses to a constant and more comparative form. The relationship used for establishing the simplified stress ratio is:

$$\frac{\tau_{ave}}{\gamma_h} = 0.65 \frac{A_{max}}{g} \times r_d$$

where: $r_d$ = stress reduction factor (less than 1 and dependent on depth below ground surface)

$A_{max}$ = maximum ground surface acceleration

$g$ = acceleration due to gravity

$\gamma_h$ = unit weight of soil column

The number of significant stress cycles ($N_c$) was taken as 10 for this investigation, based on a duration of ground shaking associated with an M = 7.0 earthquake.

The second method used in estimating field stress ratios (Computer Method) is based on a computer determined shear stress-time history utilizing the Castaic based earthquake accelerogram referred to in Appendix H. This earthquake accelerogram was used for several reasons; only minor adjustments to frequency and peak acceleration were necessary to typify design earthquake conditions; it exhibited the largest resultant ground acceleration; and peak acceleration occurred at the beginning of the record. The shear stress time histories shown in Figures I3-I6 represent two depths at each study site. The relationship between shear stress and number of stress cycles was evaluated by considering three levels of stress;

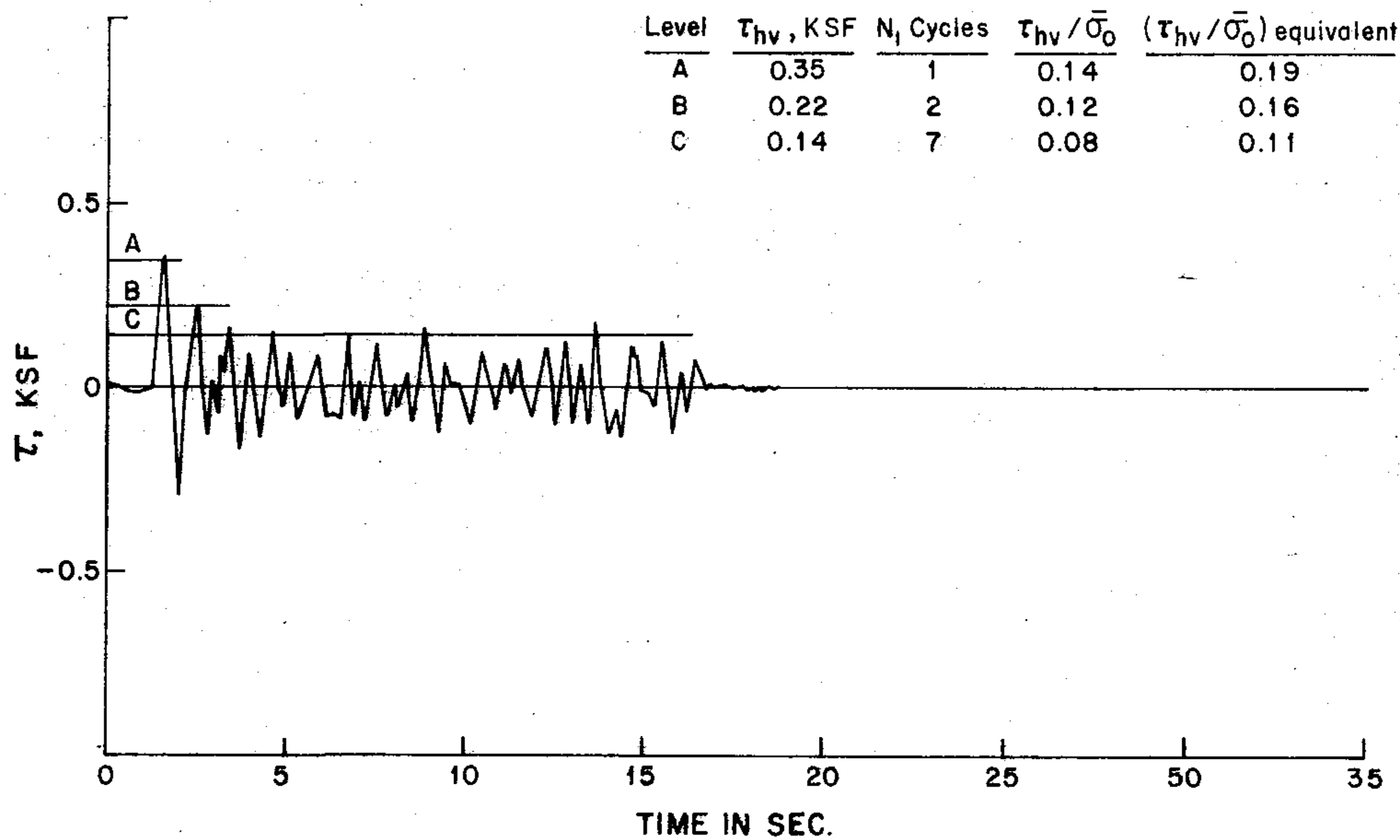
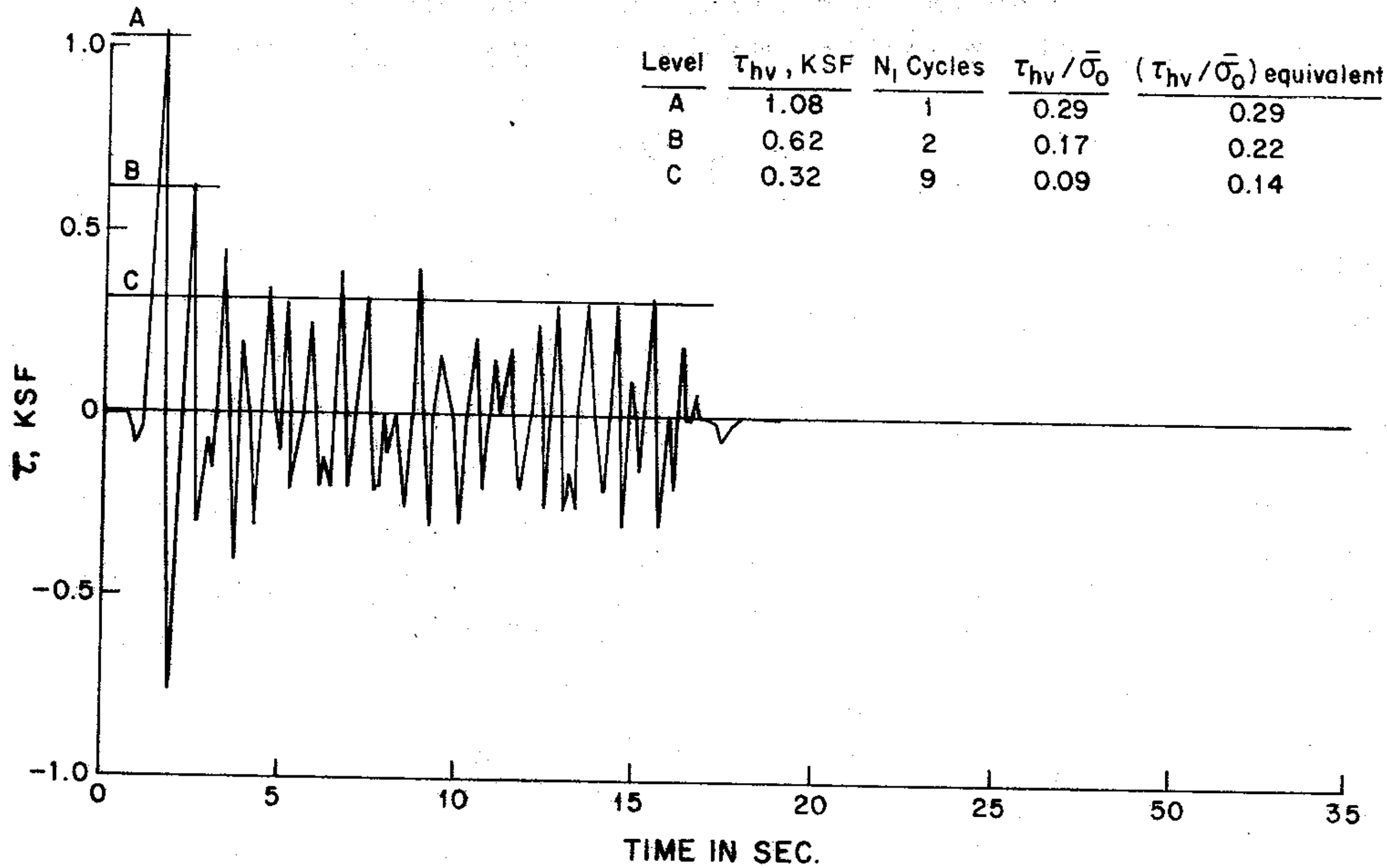
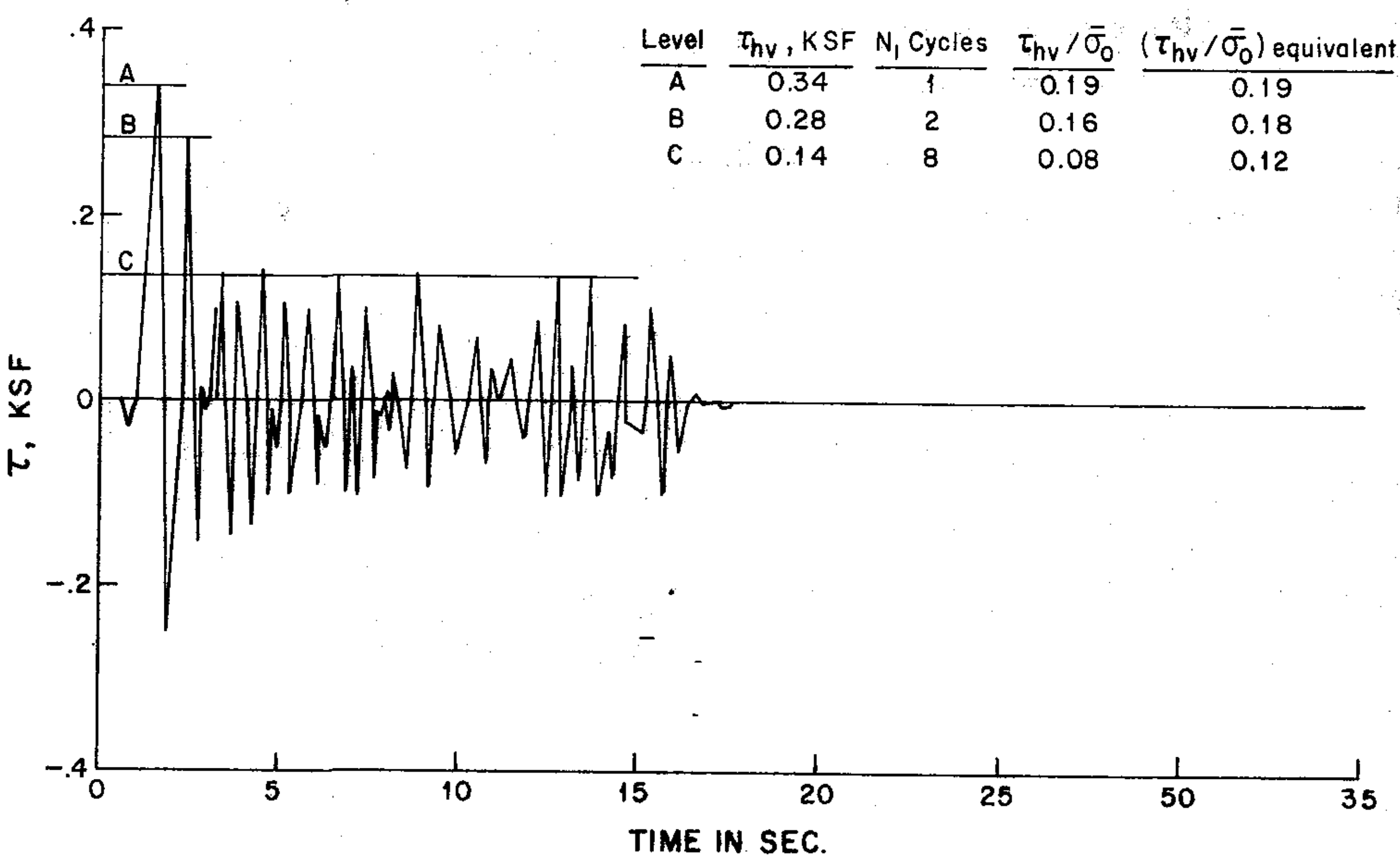



FIG. I3 TIME HISTORY OF SHEAR STRESS AT 17.5 FEET
-EXISTING CAPITOL SITE (CASTAIC E.Q. RECORD)



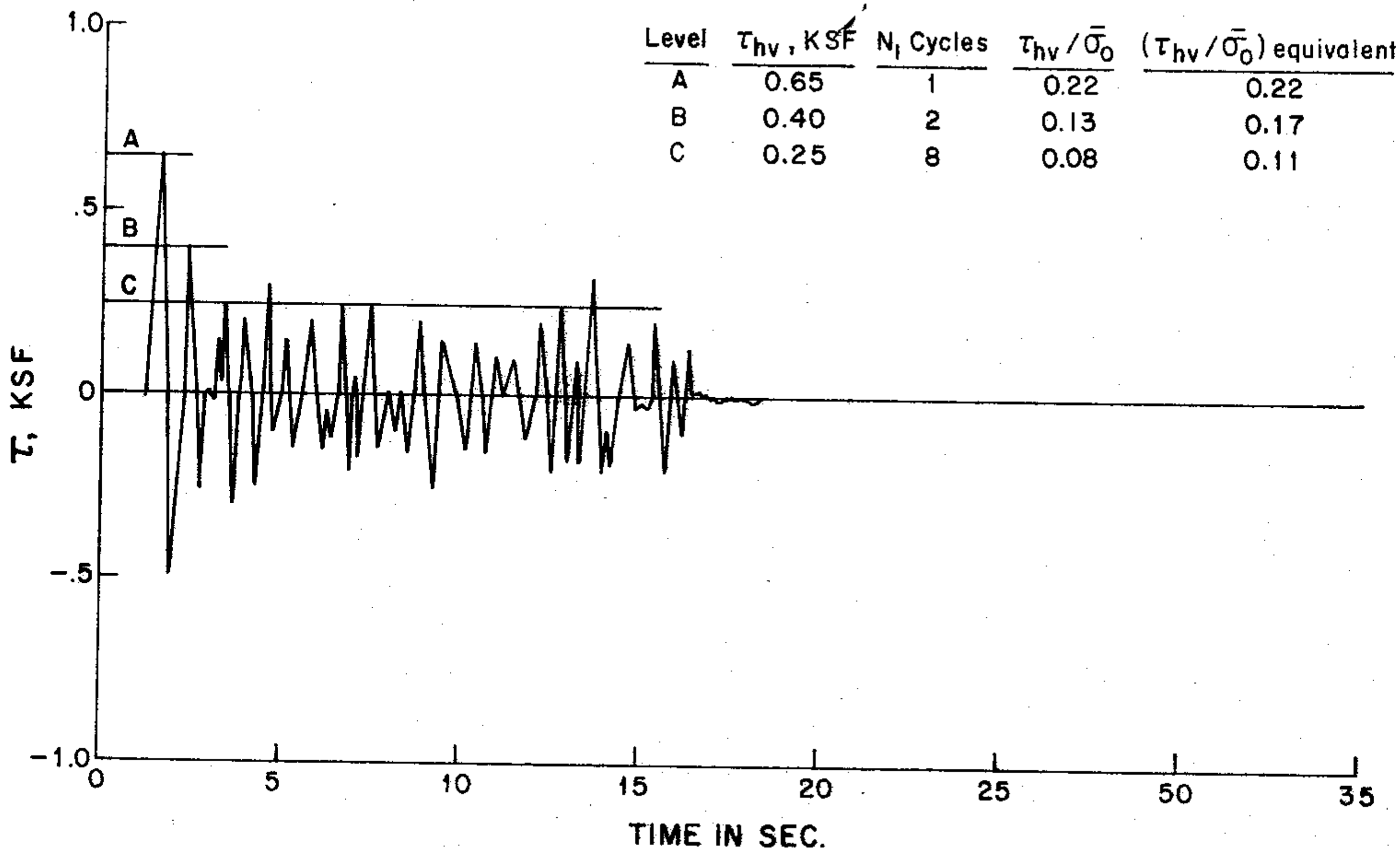

| Level | $\tau_{hv}$, KSF | $N_l$ Cycles | $\tau_{hv}/\bar{\sigma}_0$ | $(\tau_{hv}/\bar{\sigma}_0)$ equivalent |
|---|---|---|---|---|
| A | 1.08 | 1 | 0.29 | 0.29 |
| B | 0.62 | 2 | 0.17 | 0.22 |
| C | 0.32 | 9 | 0.09 | 0.14 |

FIG. 14 TIME HISTORY OF SHEAR STRESS AT 50.5 FEET -EXISTING CAPITOL SITE (CASTAIC E.Q. RECORD)




| Level | $\tau_{hv}$, KSF | $N_l$ Cycles | $\tau_{hv}/\bar{\sigma}_0$ | $(\tau_{hv}/\bar{\sigma}_0)$ equivalent |
|---|---|---|---|---|
| A | 0.34 | 1 | 0.19 | 0.19 |
| B | 0.28 | 2 | 0.16 | 0.18 |
| C | 0.14 | 8 | 0.08 | 0.12 |

FIG.15 TIME HISTORY OF SHEAR STRESS AT 15 FEET
- PROPOSED SITE (CASTAIC E.Q. RECORD)




| Level | $\tau_{hv}$, KSF | $N_l$ Cycles | $\tau_{hv}/\bar{\sigma}_0$ | $(\tau_{hv}/\bar{\sigma}_0)$ equivalent |
|---|---|---|---|---|
| A | 0.65 | 1 | 0.22 | 0.22 |
| B | 0.40 | 2 | 0.13 | 0.17 |
| C | 0.25 | 8 | 0.08 | 0.11 |

FIG.I6 TIME HISTORY OF SHEAR STRESS AT 29.8 FEET -PROPOSED SITE (CASTAIC E.Q. RECORD) the peak stress condition (A) and two intermediate stress conditions (B and C). Number of cycles were determined on the basis of intercepted stress pulses. Since stress level A contains only the peak stress pulse, it is anticipated that it would create greater material disturbance and excess pore pressures than if it was comprised of a number of smaller stress pulses. Stress levels B and C likewise include this peak stress pulse and therefore experience its effects. In general, it can be stated that the effect of any individual stress pulse on a soil will be greater if it is preceded by larger stress pulses. For this reason, the original stresses defining levels B and C were adjusted upward by averaging the maximum stresses of the intercepted pulses. These new defining stresses are the equivalent or average stresses associated with pulses intercepting levels B and C. Normalizing these equivalent stresses by dividing by the effective overburden pressure $(\bar{\sigma}_0)$, the equivalent field stress ratio, $(\tau_{hv}/\bar{\sigma}_0)_e$, is obtained. These equivalent field stress ratios are listed on Figures I3-I6 together with additional data regarding stress levels A, B, and C.

## Comparison of Laboratory and Estimated Field Stress Ratios

Figures I7 and I8 present a comparison of laboratory adjusted stress ratios and estimated field stress ratios as determined using the simplified method. Values are compared at 10 stress cycles. Laboratory test results shown for this number of stress cycles were taken from curves extrapolated from actual test points, as shown in Figure I2. Note that for only one sample does the estimated field stress ratio apprpach a value large enough to creat 2% strain.

Figures I9 and I10 present similar graphs comparing laboratory adjusted stress ratios with equivalent field stress ratios as

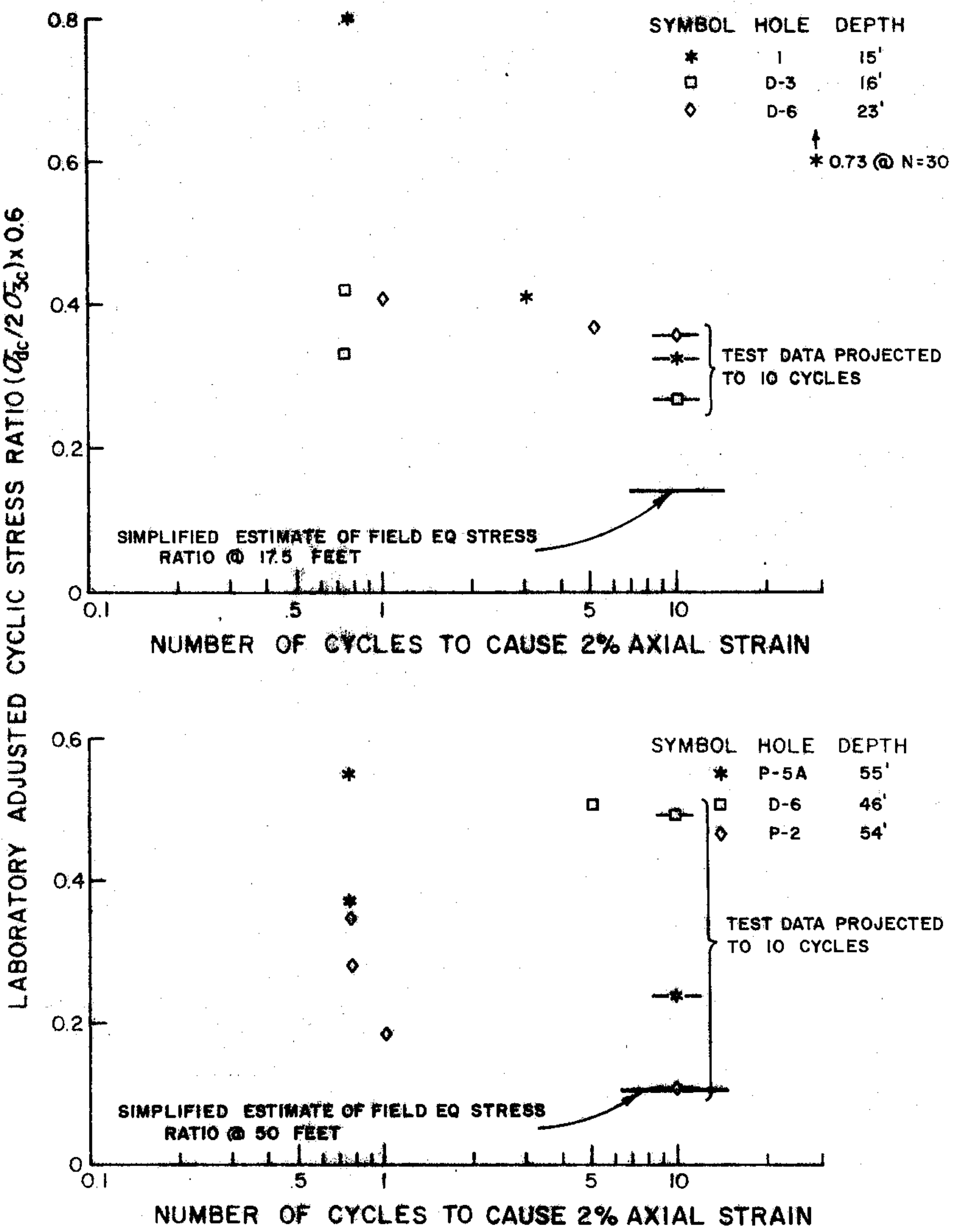



Fig.17 COMPARISON OF FIELD EARTHQUAKE (EQ) STRESS RATIO AT NOTED DEPTHS TO LABORATORY ADJUSTED CYCLIC STRESS RATIO. (EXISTING SITE)

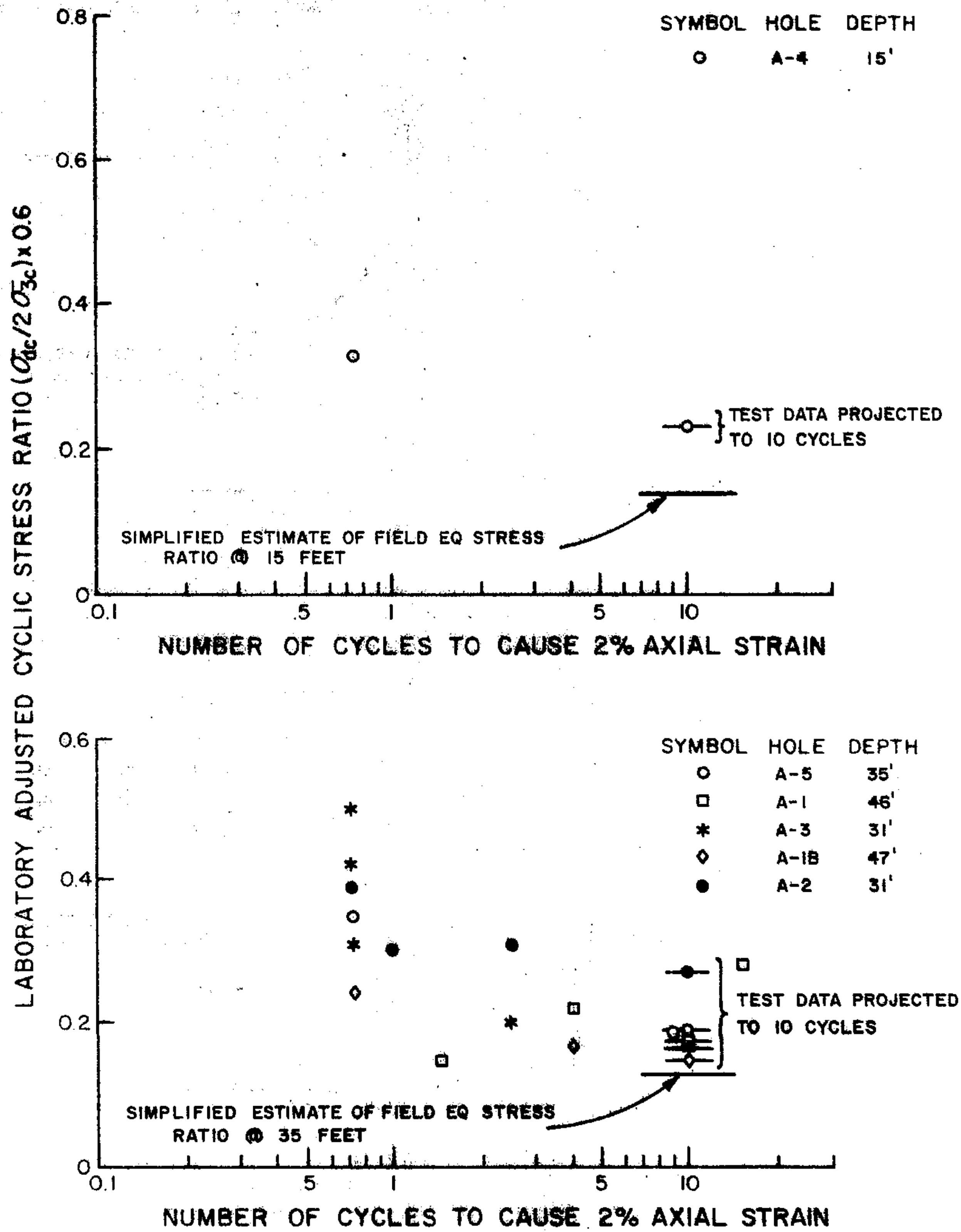



Fig.18 COMPARISON OF FIELD EARTHQUAKE (EQ) STRESS RATIO AT NOTED DEPTHS TO LABORATORY ADJUSTED CYCLIC STRESS RATIO. (PROPOSED SITE)

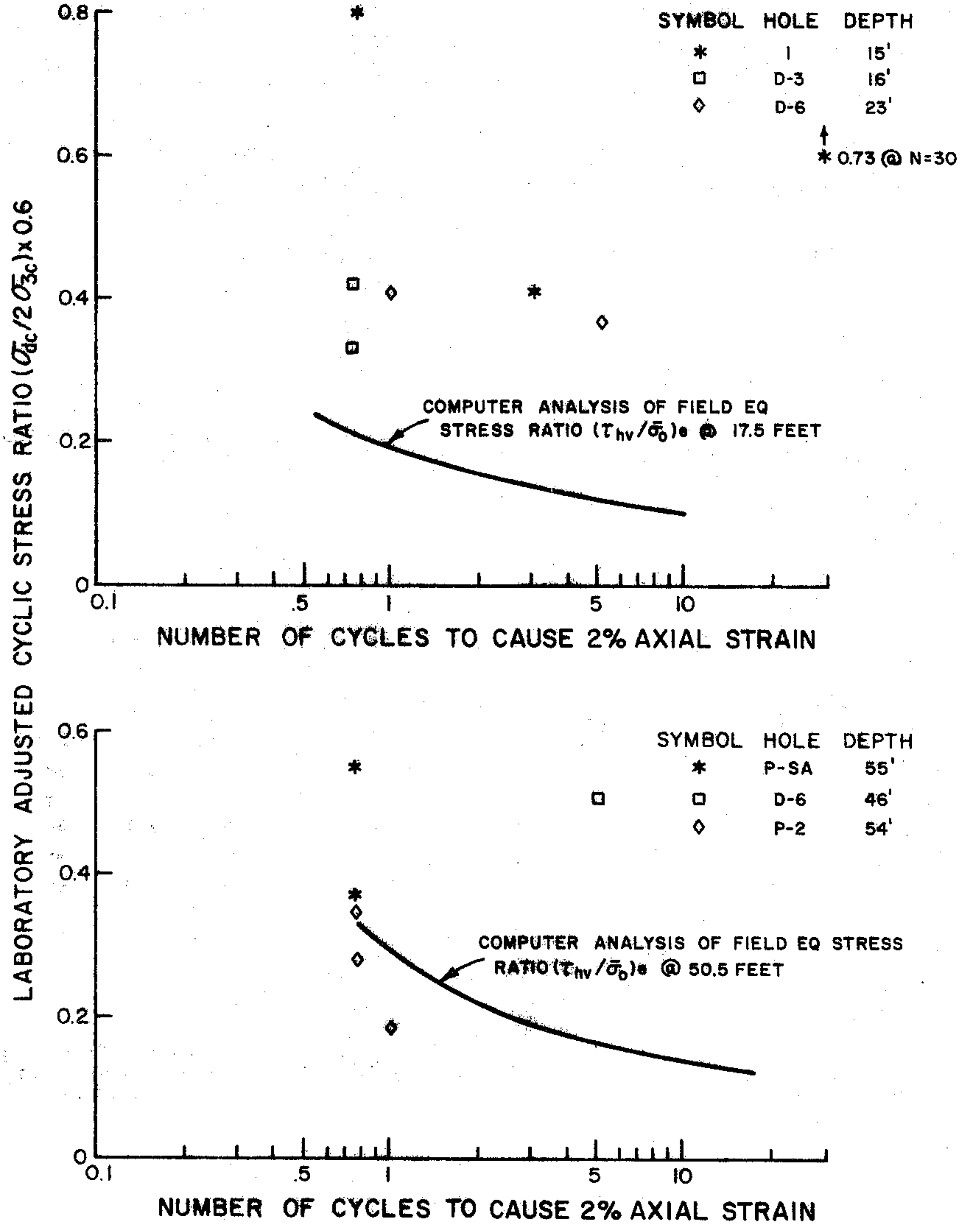



Fig.I9 COMPARISON OF FIELD EARTHQUAKE (EQ) STRESS RATIO AT NOTED DEPTHS TO LABORATORY ADJUSTED CYCLIC STRESS RATIO. (EXISTING SITE)

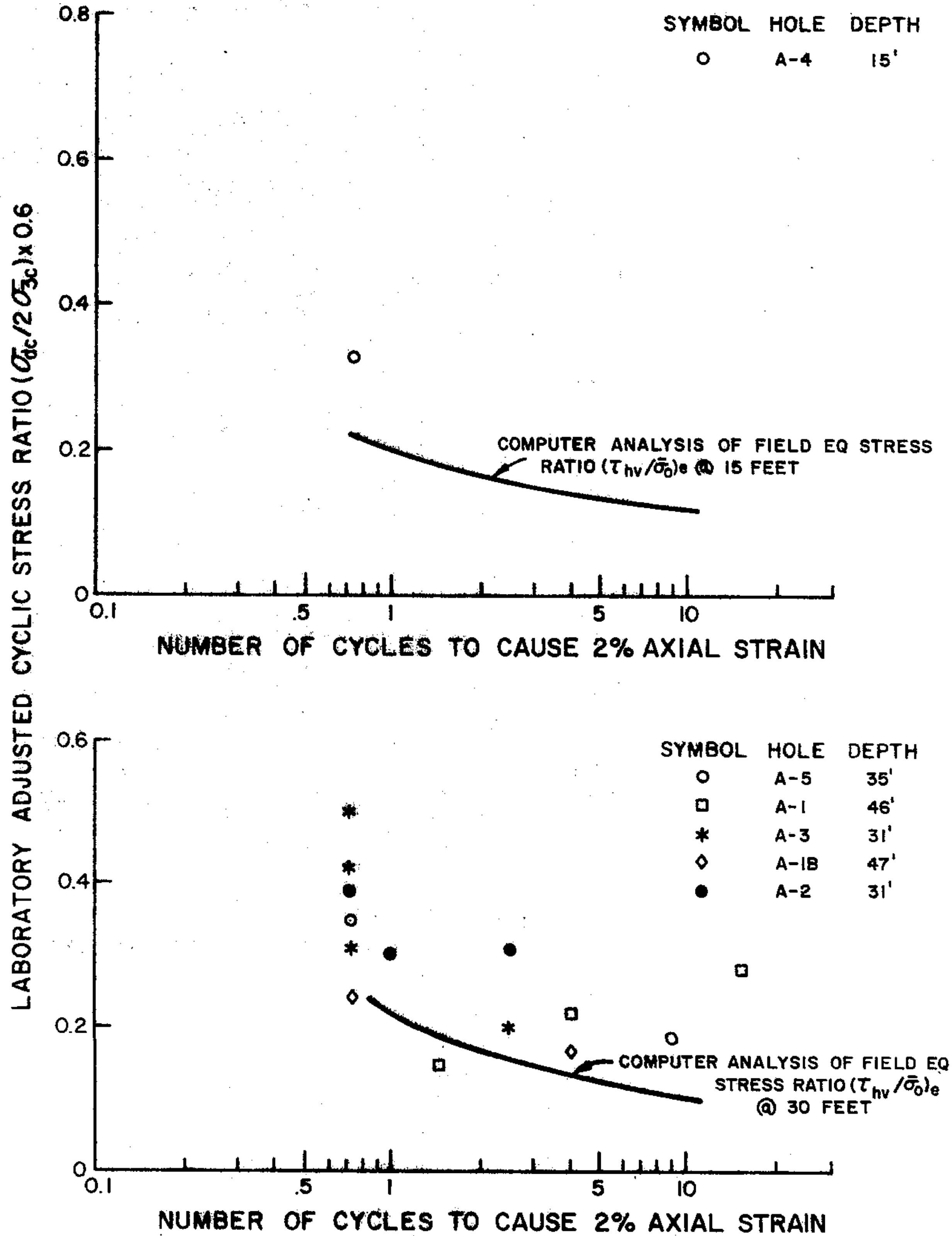



Fig.I10 COMPARISON OF FIELD EARTHQUAKE (EQ) STRESS RATIO AT NOTED DEPTHS TO LABORATORY ADJUSTED CYCLIC STRESS RATIO (PROPOSED SITE) defined in the description of the Computer Method. Here, equivalent field stress ratios indicate a 2% or greater strain potential for only a few samples.

## Conclusions

Two governing failure criteria were selected for the evaluation of soil strength during seismic loading. The first, is complete liquefaction or a zero effective confining pressure condition. The second criterion is field shear strains equivalent to 2% laboratory axial strain. Two percent strain was selected for several reasons; one, it was a low level of strain that could reasonably be approached in the field, and two, laboratory excess pore pressures were approaching levels which, if occurring in the field, would significantly reduce material strengths.

Complete liquefaction was never observed in any of the samples during cyclic load testing. Comparison of laboratory and estimated field stress ratios indicate a few instances where earthquake induced stresses reach levels sufficient to cause field strains equivalent to 2% laboratory axial strain. All of the test points in this category correspond to depths below 45 feet. The depths from which these samples originate help to minimize the significance of these low values, however. It is concluded that field strains developing as a result of design earthquakes will not reach levels high enough to create foundation problems.

Averaging the pore pressure data presented in Figure I11 indicates an average increase in pore pressure during cyclic load testing of 50%. Due to differences in $K_o$ values and stress conditions between laboratory and field, and lower levels of anticipated strain as seen from a comparison of the lab and field stress

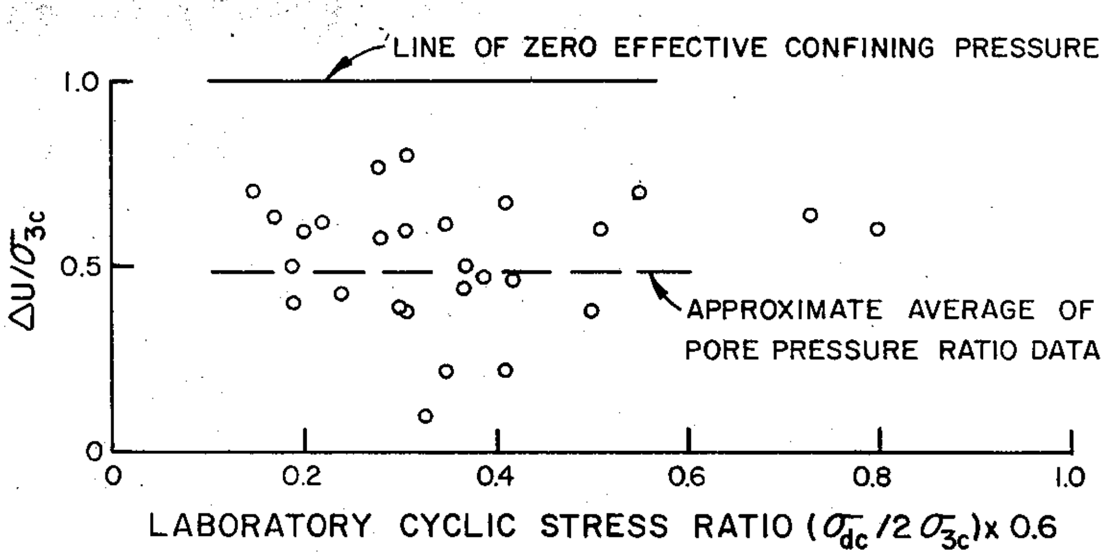



Fig.I11 COMPARISON OF PORE PRESSURE RATIO ($\Delta U/\sigma_{3c}$) TO THE LABORATORY ADJUSTED CYCLIC STRESS RATIO AT 2% AXIAL STRAIN (ALL DATA).

ratios on Figures I7-I10, actual field pore pressures are expected to be much less. Some excess pore pressures will persist in certain strata following seismic loading, however, these pressures will be small and will not appreciably effect soil strength capabilities.

As a final note, it must be stated that at best the results and conclusions presented here reflect a reasonable approximation of what can be expected to occur in the field under earthquake induced loading conditions. Assumptions and generalizations made in this analysis have continually reflected a conservative approach. Consequently, estimated field stresses are somewhat higher than will actually develop.

## SEISMIC DENSIFICATION

Seismic densification estimates were made by theoretical calculations relating field shear moduli and anticipated field shear stress levels to an average field shear strain condition. These anticipated shear strain levels were then compared to information in the literature relating vertical straining potential to number of significant stress pulse cycles (Ref. 3).

Densification (settlement) of the gravel stratum for relative density conditions exceeding 35% was estimated by consideration of potential earthquake induced shear strains and information published by Silver and Seed (Ref. 4) who list densification potential for dry, cohesionless soils. Since the strata of particular concern for this investigation is submerged, but extremely pervious, it is believed that differences for the level of field shear strain anticipated would be minor and direct comparison to saturated soils would apply. Pulsating accelerations are not considered an important factor in the densification of granular soil; however, shear stresses associated with horizontal accelerations are important. As a consequence, estimation of settlement potential was made by computing the maximum average equivalent uniform shear stress at the 35 foot depth (Ref. 2) and estimating the shearing strain associated with this stress level by the following relationship;

$$\gamma_{xy} = \frac{\tau_{ave}}{G} \ldots\ldots 1$$

where $\gamma_{xy}$ = shear strain in the vertical planes

$\tau_{ave}$ = maximum average equivalent uniform shear stress = 0.50 KSF

$G$ = shear modulus of gravel stratum (estimated from field shear wave measurements as $1 \times 10^4$ KSF at a shear strain level of approximately 0.005%).

Use of Equation 1 resulted in computed dynamic shear strain levels of less than 0.005%. Comparing this value to information on Figure I12 (based on particle size between 0.4 mm to 1.0 mm diam.) and assuming a similar relationship exists for gravel size particles, the field vertical strain potential was found to be less than 0.01%; thus, insignificant. Figure I12 was based on deformation for 10 cycles. This is equivalent to the significant number of stress cycles associated with the design earthquake.

In addition to the vibratory densification estimates of the subsurface sandy-gravel stratum, additional computations were made

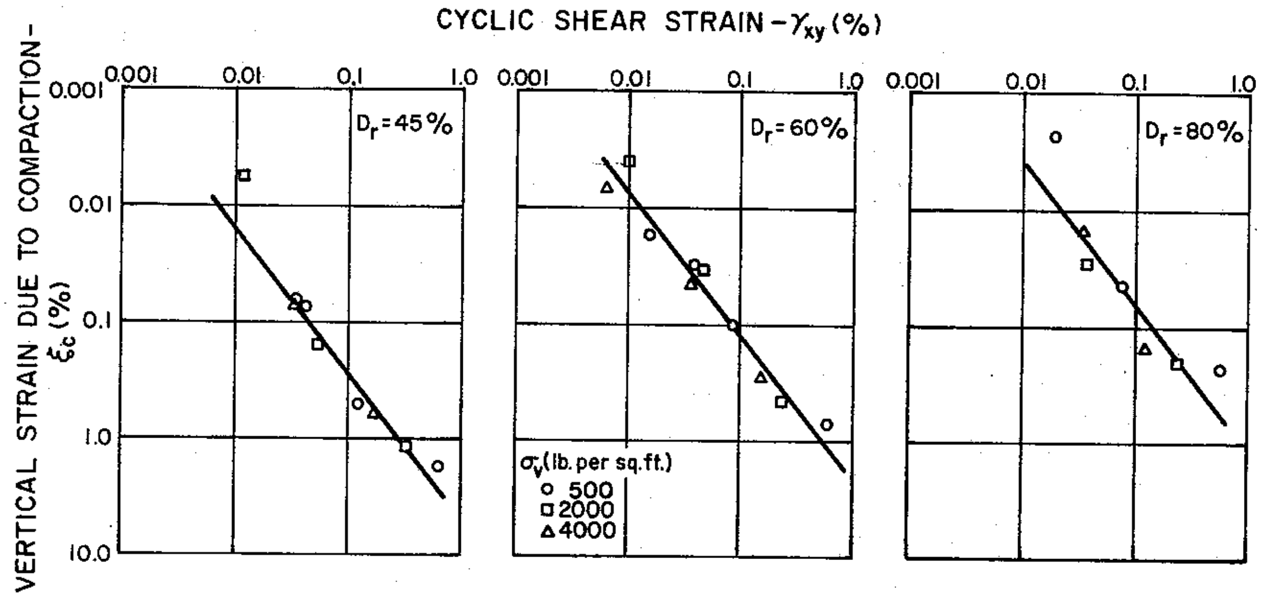



Fig. I12 VERTICAL STRAIN IN 10 CYCLES OF SHEAR STRAIN AT VARIOUS AMPLITUDES. (Silver and Seed, 1971) for the loosely compacted sandy fill. This surface stratum is of a variable thickness at both sites, but for computational simplicity, a representative thickness of 8 feet was used. Field seismic surveys showed a fairly uniform shear wave velocity at both sites of 500-600 feet per second (fps). Thus, using 550 fps as an average, and the following relationship relating shear modulus (G) to shear velocity ($V_s$);

$$G = \frac{V_s^2 \cdot \gamma_t}{g} \quad \ldots\ldots\ 2$$

where $\gamma_t$ and g are the soil unit weight and acceleration due to gravity, respectively, we obtain a value of $G \approx 0.9 \times 10^3$ KSF at an anticipated shear strain of 0.005%. Also, estimating the average shear stress at 4 feet by Equation 1 we get $\tau_{ave} \simeq$ 0.063 KSF. Utilizing the relationship;

$$\gamma_{xy} = \frac{\tau_{ave}}{G}$$

where $\gamma_{xy}$ = shear strain in a vertical plan, we find that the maximum shear strain in the field is approximately 0.005%, hence, less than 0.01% vertical densification of this stratum (see Figure I12) if relative densities are approximately 45%. Figure I12 shows increases of approximately 30% in vertical strain for roughly each 15% decrease in relative density, thus, anticipating fill relative densities of 30% would still result in vertical densification potential of less than 0.1%.

## References


1. Seed, H. B., and Peacock, W. H., "Applicability of Laboratory Test Procedures for Measuring Soil Liquefaction Characteristics Under Cyclic Loading," Earthquake Engineering Research Center Report 70-8, November 1970 (PB 198 016).

2. Seed, H. B., and Idriss, I. M., "Simplified Procedure for Evaluating Soil Liquefaction Potential," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, SM-9, 1970, pp. 1249-1273.

3. Lee, K., and Albaisa, A., "Earthquake Induced Settlements in Saturated Sands," Journal of the Geotechnical Engineering Division, Vol. 100, No. GT4, April 1974.

4. Silver, M. L., and Seed, H. B., "Volume Changes in Sands During Cyclic Loading," Journal of the Soil Mechanics and Division, ASCE, September, 1971.

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# APPENDIX J

## FOUNDATION CONDITIONS - EXISTING SITE

# APPENDIX J

## FOUNDATION CONDITIONS - EXISTING SITE

This appendix reports the results of the analysis for the existing West Wing and East Wing foundations, and any proposed new construction at the existing Capitol Site. For the West and East Wings, the investigation includes analyses of: foundation loads; overall stability of the wing; bearing capacity of soil under footings; structural analyses of footings; and settlement analyses of the wing.

The appendix is divided into three parts:

Part I - West Wing Analysis
Part II - East Wing Analysis
Part III - Possible New Construction - Existing Site

In order to determine the foundation loads, it was necessary in each instance to make the assumption, whether valid or invalid, that the superstructure is structurally capable of transmitting possible seismic forces to the foundation. In the case of the West Wing, however, the earthquake hazards that exist in that structure are considered in detail in a report by the Office of Architecture and Construction dated June, 1972.

### Part I - WEST WING ANALYSIS

The foundation of the West Wing consists of a network of slightly tapered plain concrete wall footings, as shown in Figures C1 and

C2 of Appendix C of this report. The depth of all footings is constant at 3 feet; while the width varies to accommodate the different thicknesses of the supported brick walls.

Foundation Loads

The dead loads (DL) and live loads (LL) were estimated using source information taken from the files of the Office of Architecture and Construction. Seismic forces were estimated based on both the Zone 3, Uniform Building Code (UBC) requirements and the recently adopted Hospital Code requirements for California. Convincing reasons for using the California Hospital Code requirements in an analysis of the State Capitol are presented in a seismic study report dated March, 1973, prepared by VTN, Consolidated, Inc. (John H. Wiggins, Jr., consultant). Because calculated wind loads were only a small percentage of the estimated seismic forces, wind loads were not considered in any further analysis.

Table J1 summarizes the dead loads (DL) and live loads (LL) acting on top of the footings at Elevation +13.2 for the various walls of the West Wing.

The loads are expressed in units of kips per lineal foot of wall. For each wall, Table J1 also gives the base width of the supporting footing at Elevation +10.2. For the purpose of foundation analysis, all loads were transmitted to Elevation + 10.2 at the base of the footings. The total dead weight of the building to the top of the footings is approximately 86,200 kips, while the weight of the footings themselves is approximately 7,400 kips. Thus, the calculated total DL at the base of the footings is 93,600 kips. It should be noted that the LL of the West Wing constitutes less than 15% of the building DL. The footing area of the West Wing constitutes almost half (48%) of the entire 36,800 sq. ft. of area under the West Wing.

With regard to seismic forces, the total base shear based on Zone 3, UBC requirements, was estimated to be 8,620 kips at the level of the top of the footings, Elevation +13.2. In comparison, estimates based on the California Hospital Code require almost double this value for the total base shear. For the purpose of analysis, these loads were also transmitted to Elevation +10.2, the base elevation of the foundation. The lateral inertia forces of the footings were estimated in addition to the dynamic pressure increment of the backfill material retained by the exterior walls of the West Wing for two separate earthquake loading conditions: in the north-south direction and in the east-west direction. These loads were used for the overall stability analysis of the West Wing.

## Overall Stability of the West Wing

The results of the overall stability analysis of the West Wing are summarized in Table J2. Table J2 shows that the factor of safety against overturning for the West Wing building would be satisfactory, provided the loads can be safely transmitted to the foundation and the resulting bearing pressures (considered in the next section of this Appendix) are within the allowable values for the supporting soil. In contrast, the calculated factor of safety against sliding is quite low for the California Hospital Code requirement, but satisfactory for the Zone 3, UBC requirements. It should be noted, however, that any passive earth pressure resistance against sliding was neglected in the analysis, because such lateral ground resistance usually is available only after considerable movement has occurred. In addition, the key effect of footings under cross walls (perpendicular to the direction of earthquake forces) was also neglected in the analysis. The inclusion of these resisting forces in the analysis would tend to increase the factor of safety against sliding.

Table J1 West Wing DL and LL at Top of Footing
(Elevation + 13.2)

| Location | DL (kips/ft) | LL (kips/ft) | Total DL+LL (kips/ft) | Base Width of Ftg.(ft) |
|---|---|---|---|---|
| West Wall | 49.1 | 3.3 | 52.4 | 9.5 |
| Walls Under Dome | 105.4 | 3.0 | 108.4 | 14.5 |
| North-South Corridor Walls | 31.9 | 5.0 | 36.9 | 4.8 |
| North-South Walls | 44.0 | 2.8 | 46.8 | 7.5 |
| East West Corridor Walls | 28.3 | 4.0 | 32.3 | 4.8 |
| Interior Partition | 18.7 | 2.4 | 21.1 | 4.8 |

Table J2 Overall Stability of West Wing

| Direction | FS Against Overturning | FS Against Sliding | Code |
|---|---|---|---|
| East-West EQ | 9 | 1.6 | UBC, Zone 3 |
| North-South EQ | 22 | 1.8 | UBC, Zone 3 |
| East-West EQ | 5 | 1.0+ | CA HOSPITAL |
| North-South EQ | 12 | 1.1 | CA HOSPITAL |

Table J3 Vertical Pressures Under West Wing Footings
(Seismic Condition: California Hospital Code Requirements)

a) East - West Earthquake

| Location | DL (ksf) | LL (ksf) | DL&LL (ksf) | +EQ (ksf) | Total (ksf) |
|---|---|---|---|---|---|
| North & South Walls | 6.56 | .38 | 6.94 | 5.28 | 12.2 |
| East-West Corridor Walls | 6.57 | .83 | 7.40 | 8.20 | 15.6 |
| Walls Under Dome | 8.13 | .20 | 8.33 | 0.99 | 9.3 |

Table J3 Vertical Pressures Under West Wing Footings (con't.)

b) North - South Earthquake

| Location | DL (ksf) | LL (ksf) | DL&LL (ksf) | +EQ (ksf) | Total (ksf) |
|---|---|---|---|---|---|
| West Walls | 5.90 | .35 | 6.25 | 2.20 | 8.5 |
| North-South Corridor Walls | 6.72 | 1.04 | 7.76 | 4.35 | 12.1 |
| Walls Under Dome | 8.13 | .20 | 8.33 | .97 | 9.3 |
| Interior Partition | 4.60 | .50 | 5.10 | 2.63 | 7.7 |

Table J4 Results of Bearing Capacity Analyses of Soil Under West Wing Footings

| Shear Strength Parameters c (psf) | Shear Strength Parameters Ø Degrees | Footing Location | Ultimate Bearing Capacity (ksf) | Factor of Safety DL+LL | Factor of Safety DL+LL+EQ CA Hosp. Code |
|---|---|---|---|---|---|
| 1100 | 0 | West Wall | 6.6 | 1.1 | .8 |
| 1100 | 4 | North Wall | 8.4 | 1.2 | .7 |

Table J5 Mortar, Brick, and Concrete Stresses

| Material | Calculated Max. Stresses for DL+LL (psi) | Calculated Max. Stresses for DL+LL+EQ (psi) | Laboratory Strength Tests (psi) | Allowable Stresses UBC or Calif. Hosp. Code (psi) | Type |
|---|---|---|---|---|---|
| Mortar | - | 20 | 40-87 | 20 | Shear |
| Concrete | 48 | 68* | 290-580 | 2500(ult.) | Compression |
| Concrete | 36 | 51 | 27-38** | 80 | Tension |
| Concrete | 36 | 51 | 34-48*** | 100 | Shear |
| Brick | 175 | 200 | 1200(assembled) | 115-225 (assembled) | Compression |

*For California Hospital Code requirements, concrete compression is 110 psi.

**Concrete tension 1.6 $\sqrt{f'c}$ UBC Section 2615(h).

***Concrete shear 2.0 $\sqrt{f'c}$ UBC Section 2615(h).

## Bearing Capacity of Soil Under West Wing Footings

Tables J3(a) and J3(b) summarize the vertical pressures at the base of the various footings of the West Wing, Elevation +10.2, for DL and LL and, also, the vertical pressures resulting from the California Hospital Code requirements for earthquake conditions. A similar table was prepared for the Zone 3, UBC requirements, but is not included herein.

As noted in Table J3(a), based upon the California Hospital Code requirement, a new uplift pressure would occur, at the base of the footings for the east-west corridor walls (comparing DL = 6.57 KSF versus EQ = $\pm$8.20 KSF). For these conditions the analysis shows that a net uplift would also occur at the top of the footings at the interface between the brick wall and the concrete footing. But there are no ties (dowels) at this interface, only mortar.

According to the UBC requirements under Section 2314 Earthquake Regulations, (K) Design Requirement:

> "All elements within structures located in Seismic Zones Nos. 2 and 3 which are of masonry or concrete shall be reinforced so as to qualify as reinforced masonry or concrete under the provisions of Chapters 27 and 26. Principal reinforcement in masonry shall be spaced 2 feet maximum on center in buildings using a moment-resisting-space frame."

The basement brick walls and concrete footings of the West Wing do not satisfy the code requirement.

Table J4 gives a summary of the ultimate bearing capacity of the soil under the existing footings. Shear strength parameters, c and Ø, were determined from triaxial compression shear tests on samples taken from test pits and used to calculate the soil bearing capacity. In the analysis the additional parameters used were:

height of surcharge = 7 ft.
unit weight of soil = 120 pcf

Table J4 shows that the factors of safety against bearing capacity failure for the West Wing are slightly above one for DL + LL and unsatisfactory for California Hospital Code requirements. Even allowing for sampling disturbance, which tends to underestimate the soil shear strength, the calculated factors of safety are still quite low. For the Zone 3, UBC requirements, calculated factors of safety were also calculated, but not included herein. These also were unsatisfactory.

Structural Analysis of West Wing Footings

Table J5 summarizes calculated and allowable stresses, and laboratory test strengths of the mortar, brick and concrete. The calculated seismic stresses are for the UBC, Zone 3 requirements.

Table J5 shows that the calculated concrete stresses in the north wall footing for DL + LL are within the limits of the laboratory compressive strength and the interrelated strengths for shear and tension. For the UBC, Zone 3 requirements, the computed shearing and tensile stresses do not meet the required laboratory determined strengths. Therefore, assuming the laboratory compressive strength as valid, the ultimate capacity of the concrete footing is unsatisfactory for either the UBC, Zone 3 or the California Hospital Code requirements. The quality of the concrete in the footing is below today's standards.

The maximum computed compressive stress of the brick assemblies was in the wall supporting the dome. The stresses are satisfactory since the brick assembly tested was of very high strength. The maximum shearing stress of the mortar in the west wall is also satisfactory for both DL + LL and UBC Zone 3, and California Hospital Code requirements. Except for the brick compressive and mortar shearing strengths, none of the laboratory test strengths meet present code requirements. Laboratory strength tests are described in Appendix G.

## Settlement Analysis of the West Wing

The soil under the West Wing foundation has been determined to consist of a thick layer of normally consolidated clayey silt underlain by layers of sand and gravel, and sandy silt. The thickness of each layer has not been determined since no borings were taken within the building area. However, the clayey silt layer is estimated to vary from 20 to 30 feet. (See Figures E36, E37).

Settlement analysis from consolidation test data of clayey silt samples from Test Pits 1 and 2 indicate that the West Wing has settled 6 to 12 inches or more since 1861. Recent borings made outside of the building area indicate that there might be other underlying compressible soil layers within the seat of settlement below the sand and gravel. (See Figures E36, E37). Historical records indicate that very large settlements occurred during the construction of the West Wing. This resulted in cracks in the brick walls and construction delays. (See Appendix C).

Consideration should be given to soil rebound and recompression if it is decided that the West Wing should be reconstructed.

Although the clayey silt under the West Wing will have been preloaded after dismantling, rebound and recompression can be expected of up to 2 inches if shallow foundations are used. Accurate predictions of soil rebound and recompression are difficult. Estimates were based on consolidation rebound tests data from undisturbed soil samples in Test Pits 1 and 2, using the original building loads and considering only the recompression of the clayey silt stratum above the sand and gravel. Additional settlements would result from recompression of any compressible strata that might exist below the sand and gravel within the seat of settlement.

## Part II - EAST WING ANALYSIS

The East Wing structure is a steel frame-reinforced concrete shear wall building supported by a grid type of reinforced concrete footing, as shown in Figure D3 of Appendix D. The depth of the interior continuous footings is 3'-6" and 2'-0" for the exterior footings. The base of the footings is at the same Elevation +10.2 as the base of the West Wing footings at the interface of the two buildings. The interior footings are slightly tapered both vertically and horizontally. The reasons for the selection of this type of foundation are discussed in Appendix D.

### Foundation Loads

The dead loads (DL) and live loads (LL) were estimated using source information taken from the files of the Office of Architecture and Construction. Seismic forces were estimated based on the recently adopted Hospital Code requirements for California. Because calculated wind loads were only a small percentage of the estimated seismic forces, wind loads were not considered in any further analysis.

Table J6 summarizes the dead loads (DL) and live loads (LL) on top of the footings at Elevation +13.7 for the easterly and westerly sections of the East Wing. Figure J1 defines the designated easterly and westerly sections.

For purpose of foundation analysis, all loads were transmitted to Elevation +10.2 at the base of the footings. The total dead weight of the building to the top of the footings is approximately 48,300 kips, while the weight of the footings themselves is 13,500 kips. Thus the calculated total DL at the base of the footings is 61,800 kips. It should be noted that the LL of the East Wing constitutes about 28% of the building DL. Also, the footing area of the East Wing constitutes almost half (48%) of the entire 58,800 sq. ft. of area under the East Wing. The easterly section of the East Wing covers 32,200 sq. ft. of the total building area.

Table J7 gives a summary of the weight of soil removed during construction of the East Wing and the building loads. In addition to the excavation load within the building outline of the East Wing, 14,300 kips of soil were removed at the south ramp location and 6,100 kips at the north ramp location.

With regard to seismic forces, the total base shear based on the California Hospital Code was estimated to be 5,500 kips at the level of the top of the footings, Elevation +13.7. For purpose of analysis, these loads were transmitted to Elevation +10.2, the base elevation of the foundation. In addition, an estimate was made of the lateral inertia forces of the footings themselves. These loads were used for the overall stability analysis of the East Wing.

## Overall Stability of the East Wing

The results of the overall stability analysis of the East Wing are summarized in Table J8. The factors of safety for the north-south and east-west seismic forces are the same. Table J8 shows that the East Wing is safe against overturning or sliding.

## Bearing Capacity of Soil Under East Wing Footings

Table J9 summarizes the vertical pressure at the base of the various shear wall footings of the East Wing, Elevation +10.2 for DL and LL. Also summarized are the vertical pressures resulting from the California Hospital Code requirements for earthquake conditions. The location of the shear walls is shown in Figure J1.

Table J10 gives a summary of the estimated ultimate bearing capacity of the soil under the most heavily stressed East Wing footing. The shear strength parameters used in the analysis were obtained by correlation with triaxial compression test results of soil samples taken from borings outside the area of the East Wing foundation.

In the analyses the additional parameters used were:

height of surcharge = 4 ft.
unit weight of soil = 120 pcf.

Table J10 shows that the range of factors of safety against bearing capacity failure for shear wall A footing is 1.5 to 1.6 for DL + LL and above one for the California Hospital Code requirements. It should be noted, however, that these values are for the most heavily stressed shear wall footing, and that the factors of safety for other footings would be predominately greater.

Table J11 gives a summary of steel and concrete working stresses for shear wall footings G2 and G3 for DL + LL and the California Hospital Code requirements. The compressive strength of concrete used for design was 2500 PSI.

Table J6 East Wing DL and LL At Top of Footing

| Description and Elevation | Westerly Section DL(kips) | Easterly Section DL(kips) | Westerly Section LL(kips) | Easterly Section LL(kips) |
|---|---|---|---|---|
| Bldg. Loads to Elev. +13.7 | 24,800 | 23,500 | 7,500 | 6,200 |
| Ftg Loads to Elev. +10.2 | 7,400 | 6,100 | - | - |
| Basement Slab Load Elev. +13.7 | 2,100 | 2,000 | 2,800 | 2,700 |
| Soil Load Between Ftgs to Elev. +10.2 | 5,300 | 7,600 | | |
| Total Loads at Elev. +10.2 | 39,600 | 39,200 | 10,300 | 8,900 |

Table J7 Construction Loads

| Type | Westerly Section (kips) | Easterly Section (kips) | Total (kips) |
|---|---|---|---|
| Excavation Load within bldg outline | -48,700 | -37,100 | -85,800 |
| Building Load (including LL, basement slab and ftg) | +44,600 | +40,500 | +85,100 |
| Net Construction Load | - 4,100 | + 3,400 | - 700 |

Table J8 Overall Stability of East Wing

| F.S. Against Overturning | F.S. Against Sliding | Code |
|---|---|---|
| 26 | 3.1 | Calif. Hospital |

Table J9 Maximum Soil Pressures Under Shear Wall Footings

| Shear Walls | DL (kips) | LL (kips) | DL+LL (ksf) | EQ (ksf) | DL+LL+EQ (ksf) |
|---|---|---|---|---|---|
| B | 3.14 | 0.48 | 3.62 | 1.14 | 4.76 |
| G2 | 2.70 | 0.62 | 3.33 | 1.75 | 5.08 |
| G3 | 2.30 | 0.31 | 2.61 | 0.77 | 3.38 |
| M | 1.72 | 0.46 | 2.18 | 2.67 | 4.85 |
| A | 3.46 | 0.27 | 3.73 | 2.08 | 5.81 |
| C | 2.39 | 0.71 | 3.10 | 0.05 | 3.15 |
| N | 2.12 | 0.41 | 2.26 | 0.52 | 2.78 |

Table J10 Bearing Capacity of Soil Under East Wing Footing

| Footing Location* | Ultimate Bearing Capacity (ksf) | Factor of Safety DL+LL | Factor of Safety DL+LL+EQ |
|---|---|---|---|
| Shear Wall A | 5.6 to 6.1 | 1.5 to 1.6 | 1.0 to 1.1 |

*Most heavily stressed shear wall footing.

Table J11 East Wing Footing Stresses

| Shear Wall Footings | Steel Tension (psi) | Concrete Compression (psi) | Concrete Shear (psi) | Group Loading |
|---|---|---|---|---|
| G2 | 16,800 | 790 | 60 | DL+LL |
| G2 | 25,400 | 1,100 | 100 | DL+LL+EQ |
| G3 | 25,000 | 1,130 | 110 | DL+LL |
| G3 | 31,500 | 1,400 | 140 | DL+LL+EQ |
| Allowable Stresses | 18,000 | 1,125 | 100 | DL+LL |
| | 24,000 | 1,495 | 133 | DL+LL+EQ |

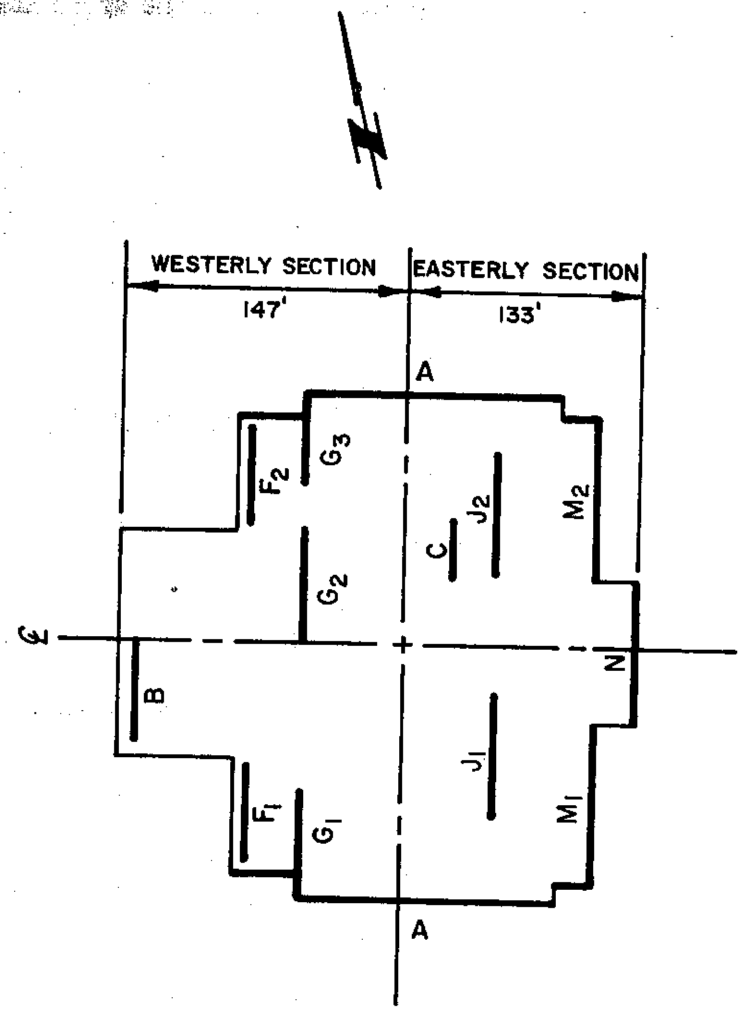


Fig.J-1 SHEAR WALLS - EAST WING

The results show that the steel is overstressed for G2 and G3 footings. The concrete is slightly overstressed for G3 footing. However, structural failure is not imminent for the G2 or G3 footings. For the seismic loading conditions, the allowable stresses may be increase 33-1/3 percent.

## Settlement Analysis of the East Wing

The soil under the East Wing foundation consists of a thick layer of normally consolidated clayey silt underlain by sand and gravel as determined from 1948 borings within the building area. The clayey silt layer varies from about 15 to 30 feet. (See Figures E36, E37). The thickness of the underlying sand and gravel layer has not been adequately determined.

Settlements of the East Wing were estimated using 1948 consolidation test data, and recent rebound consolidation test data for samples from the West Wing test pits. The analysis considered only the upper layer of clayey silt. Settlements of up to 2 inches were estimated for the upper clayey silt layer. Settlement records of the last 23 years show a maximum settlement of 1 inch, despite an applied building load less than the excavated overburden. A rational explanation for this behavior is that soil rebound occurred during excavation followed by recompression during construction loading. Analysis of 1948 consolidation test data showed that the East Wing area had been fully consolidated under the existing fill prior to construction of the East Wing.

Considerations should be given to soil rebound and recompression at the western portion of the East Wing if the West Wing is to be dismantled and reconstructed. Soil rebound and recompression of up to 1-1/2 inches for the upper clayey silt layer is possible if the area of the West Wing is unloaded and reloaded with about the same building loads. It should be noted that predictions of soil rebound and recompression are difficult and estimates are based on consolidation rebound test data from undisturbed soil samples in Test Pits 1 and 2.

Part III - POSSIBLE NEW CONSTRUCTION - EXISTING SITE

One of the proposed alternatives for the West Wing is reconstruction as described in a report of the Office of Architecture and Construction dated June, 1972. If this alternative is chosen, considerations should be given to the effect on the East Wing of soil rebound resulting from dismantling the West Wing.

Accurate predictions of soil rebound are difficult, but estimates based on consolidation rebound test data of the upper clayey silt layer indicate possible rebound of up to 1-1/2 inches at the westernmost portion of the East Wing and practically no rebound at the easternmost portion. This does not include rebound of any underlying compressible layers within the seat of settlement below the sand and gravel. Therefore, a program of field instrumentation to monitor possible movements of the East Wing during dismantling and reconstructing is strongly recommended.

The building loads are much heavier for the West Wing than the East Wing. The East Wing, despite the fact that the net construction load was negative due to excavation, experienced a one-inch settlement. This experience is useful for predicting probable soil behavior during reconstruction of the West Wing, where loads involved will be even greater. Thus, even though the soil strata under the West Wing will have been preloaded after dismantling, differential settlements of slightly over one-inch between the center and corners of the building can be expected if shallow foundations are used. This does not include differential settlements of any underlying compressible soil layer within the seat of settlement below the sand and gravel. Therefore, pile foundation founded on a suitable bearing stratum is recommended for the reconstruction of the West Wing.

Pile foundation is recommended for any new wing because records showed that the West Wing experienced serious differential settlements. The East Wing would have been founded on piles to the underlying gravels except that pile driving vibrations would have resulted in irreparable damage to the West Wing. Most downtown Capitol buildings are founded on piles.

All previous borings indicate that a sand and gravel stratum exists at a depth varying from 20 to 30 feet below the base of the existing foundations. The thickness of this stratum under the West Wing has not been determined nor has it been adequately determined for any new wing construction. For this reason the pile foundation recommendation is contingent upon findings from additional exploratory borings that should be made prior to any construction. The bottom of the borings should be at sufficient depth to define the thickness of the sand and gravel stratum and the nature of the underlying material within the seat of settlement of any proposed construction. Special attention should be given to soil conditions below the northwest portion of the West Wing where difficulties were encountered during original construction.

During the design stage for the pile foundations, considerations should be given to the need for partial predrilling to minimize the possible effects of heave and pile driving vibrations on adjacent structures. Also, field pile load tests are strongly recommended for determining pile bearing capacities.

Before commencing pile driving for any new wing, the West Wing should be adequately strengthened (Concept 1), or dismantled or in a state of reconstruction (Concept 2).