Document 21
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GEOTECHNICAL INVESTIGATION

PEDESTRIAN OVERCROSSING
LAS VEGAS BOULEVARD AND TROPICANA AVENUE

CLARK COUNTY, NEVADA

Prepared For

CRSS ENGINEERING
Denver, Colorado

April, 1993

Reno/Sparks, Nevada
Las Vegas, Nevada
Phoenix, Arizona

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April 14, 1993
Project No. 2-765-01-4

Mr. Steven Linenberger
CRSS Incorporated
216 16th Street Mall, Suite 1700
Denver, Colorado 80202-5131

Re: Pedestrian Overcrossing - Las Vegas, Nevada

Dear Linenberger:

We are pleased to present the results of our geotechnical investigation for the proposed pedestrian overcrossings at Las Vegas Boulevard and Tropicana Avenue in Las Vegas, Nevada. This report reflects review comments provided by CRSS or March 31, 1991. Soils on this site consist of loose silty sands and sandy silt extending to depths of approximately 10 feet. Below this depth materials are generally dense with competent caliche horizons encountered at depths of approximately 15, 25, and 37 feet below existing grades. The following reports presents geotechnical recommendations for design and construction of the project including alternative designs for shallow spread footings and shallow end bearing drill shafts. At the date of this report, the northern crossing of Las Vegas Boulevard has been shifted approximately 200 feet south of its original planned (and explored) location. Due to the potential of variable thickness and hardness inherent in caliche deposits, additional exploration of the new location is recommended.

We wish to thank you for the opportunity to provide this investigation, and look forward to working with you on the remainder of the project. Please feel free to contact us with any questions.

Sincerely,

SEA, Incorporated
Consulting Engineers

PROFESSIONAL ENGINEER-STATE OF NEVADA
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LAS VEGAS BOULEVARD AND TROPICANA AVENUE
LAS VEGAS, NEVADA

INTRODUCTION

Presented herein are the results of our geotechnical investigation for the proposed pedestrian overcrossing to be located at the intersection of Las Vegas Boulevard and Tropicana Avenue, Las Vegas, Nevada. The project is entirely contained in Sections 28 and 29, Township 21N, Range 61E, M.D.M. The objectives of this study were to:

1. Determine general soil, caliche, and groundwater conditions pertaining to design and construction of the proposed overpass system.

2. Provide recommendations for design and construction of the project, as related to these geotechnical conditions.

The area covered by this investigation is shown on Plate 1 - Plot Plan. The investigation included field exploration, laboratory testing, and engineering analysis to determine the physical and mechanical properties of the various on-site materials. Results of our field exploration and testing programs are included in this report and form the basis for all conclusions and recommendations.

PROJECT DESCRIPTION

The proposed project is to consist of four pedestrian overcrossings, one over each leg of the intersection. The overcrossings will be built with a structural steel substructure and a reinforced concrete substructure. Each crossing will be accessed by stairs, escalators and elevators. The mechanical equipment for the elevators will be housed in a control room above grade but under the stairs/escalator. The overcrossings will range in length from about 180 feet on South Las Vegas Boulevard to a maximum of 230 feet on east Tropicana maximum column loads are anticipated to be 425 kips, dead load plus live load. At the date of this report, the northern crossing of Las Vegas Boulevard has been shifted apprximately 200 feet south of its original planned location.

SITE CONDITIONS

The intersection of Las Vegas Boulevard (north-south) and Tropicana Avenue (east-west) is one of the busiest intersections in Las Vegas. The Tropicana Hotel is located on the southeastern corner, and the Excalibur Hotel is located on the southwestern corner. On the northeastern corner the MGM Grand Hotel is under construction. The northwestern corner is vacant.

The land slopes east-northeast at about 60 feet per mile. Although the intersection itself is relatively flat, Tropicana Avenue drops in elevation just east of the intersection towards Koval Lane. In the vacant lot to the northwest, the Tropicana Wash drainage flows in a buried reinforced concrete box culvert that runs just north of Tropicana Avenue and crosses Las Vegas Boulevard, north of the pedestrian overcrossing. A portion of the original channel of Tropicana Wash, about ten feet deep and 20 feet wide, remains unfilled on the vacant lot.

EXPLORATION

The foundation areas were explored in late February and early March, 1993 by drilling a series of 8 test borings with a soils sampling drill rig. The maximum depth of exploration was 51 feet below the existing ground surface. The locations of the test borings are shown on Plate 1 - Plot Plan and described in Table 1.

TABLE 1- Foundation Borehole Location

B-1 is located in the vacant lot on the northwestern corner, west of the buried reinforced concrete box culvert, west of Las Vegas Boulevard.

B-2 is located in the vacant lot on the northwestern corner, north of the buried concrete culvert, north of Tropicana Avenue.

B-3 is located in the asphalt of the valet parking lot of the Excalibur Hotel, south of Tropicana Avenue, and west of Las Vegas Boulevard.

B-4 is located in the asphalt at the entrance to the employees' parking lot of the Tropicana Hotel, south of Tropicana Avenue, and east of Las Vegas Boulevard.

B-5 is located in fill north of Tropicana Avenue south of the MGM Hotel structure under construction, and east of Las Vegas Boulevard.

B-6 is located in asphalt pavement on Las Vegas Boulevard in the right hand, northbound lane, north of Tropicana Avenue, near the MGM Grand Hotel.

B-7 is located in asphalt pavement on Las Vegas Boulevard in the right hand, southbound lane, south of Tropicana Avenue, near the MGM Grand Hotel.

B-8 is located in asphalt pavement on Las Vegas Boulevard in the right hand, northbound lane, south of Tropicana Avenue, near the Tropicana Hotel.

The native soils were sampled in-place by use of a standard 3 inch O.D. split spoon sampler driven by a standard 350 pound drive hammer with a 30 inch stroke. The number of blows to drive the sampler 12 inches into undisturbed soil is an indication of the density and consistency of the material. This test can be roughly correlated with the Standard

Penetration Test (ASTM D1586) by correcting for sampler diameter and hammer weight. Blowcounts shown on the logs are uncorrected, field data.

An engineering geologist examined and classified all soils in the field. Logs of the test borings are presented as Plate 2 and a graphic soils classification chart has been included as Plate 3. Representative samples were returned to our Las Vegas laboratory for testing.

LABORATORY TESTING

Samples of each significant soil type were analyzed in SEA's materials testing laboratory to determine their grain size distribution and plasticity (Plate 4). Results of these tests were used to classify the soils according to the Unified Soils Classification System (Plate 3) and to verify the field logs. Classification in this manner is an indication of the soil's mechanical properties and governs the remainder of the testing program. All tests were conducted in accordance with ASTM Standards.

Direct shear tests were performed on representative samples of sandy clay - clayey sand screened to remove particles larger than the No. 4 sieve. Tests were run on remolded and saturated samples at various lateral pressures so as to plot the Mohr's Circle Envelope. Results of these tests are shown on Plate 5 - Direct Shear Test and were used in calculation of bearing capacities, friction factors, and lateral soil pressures.

Consolidation testing was performed on native sandy clay - clayey sand. These results (Plate 6) were used to estimate settlement characteristics of the soils and to arrive at an allowable bearing capacity for spread footing foundations.

GEOLOGIC AND GENERAL SOIL CONDITIONS

The site has been mapped (Nevada Bureau of Mines and Geology, 1987, Map 3Bg) as being underlain by Quaternary to Tertiary consolidated sediments. Typically, these materials are white and light gray to light and pale red, fine sands interstratified with silt, pebbly sand, and pebble to small cobble gravel. The materials are moderately to well consolidated and often strongly cemented. Caliche layers are common and have variable textures and fabrics. The unit is thought to have been deposited by stream channels.

Soils encountered in our borings are consistent with the general geology of the area. The site is overlain by interbedded, weakly to moderately cemented, silty sand, sandy silt and sandy-silty-clayey gravel soils that extend to depths ranging from 13.5 to about 17 feet below ground surface, at the time of our exploration. At these depths, typically around 15 feet, a well developed caliche layer 2-1/2 to 5 feet or more in thickness was encountered in all eight borings. The caliche is underlain by soil with various degrees of cementation that extends to the depth of exploration. These include dense silty gravels, sandy silts, clayey gravels, clayey sands, and thin interbeds of high plastic clay, all with various degrees of cementation. A second caliche layer occurs across the site at depths starting at 20 to 28 feet below existing ground. A third caliche unit was found below 37 feet, however, it is less continuous than the two upper layers. In general most soils are compact to very dense, or for cohesive soils, very stiff to very hard below depths of approximately 10 feet.

Groundwater was encountered in all eight borings. The depth to groundwater ranged from 18.5 to 25 feet below ground elevation, at the time of exploration.

GEOLOGIC HAZARDS

Much of the Western United States is a region of moderate to intense seismicity related to movement of the crustal masses (plate tectonics). By far, the most active regions, outside of Alaska, center around the San Andreas fault system of western California. Other seismically active areas include the Wasatch Front in Salt Lake City, Utah, which forms the eastern boundary of the Basin and Range physiographic province, and the eastern front of the Sierra Nevada Mountains, which is the western margin of the province. The Las Vegas area lies near the western boundary of the basin and range of the Sierra Nevada, within the western extreme of the Basin and Range. It must be recognized that there are probably few regions in the United States not underlain at some depth by older bedrock faults.

The Las Vegas Valley lies within Seismic Zone 2, an area with some potential for earthquake damage. The published geologic map (Nevada Bureau of Mines and Geology, 1987, map 3Bg) shows several faults within a one mile radius of the project. The nearest trends northwest and is approximately 1000 feet east of the easternmost foundation area. The ages of these faults are not known. Based on the geologic map they appear to be of Quaternary age. The criteria for evaluation of Quaternary earthquake faults are not currently regulated by Clark County or the State of Nevada. As a consequence, most geological consultants in Nevada rely on methods and criteria established by the State of California. In California, the Alquist-Priolo Act of 1972 defined active faults as those with evidence of displacement within the past 11,000 years (Holocene time). Those faults with evidence of displacement during Pleistocene time (11,000 to 2,000,000 years before present) are generally considered potentially active. Based on the geologic map, the faults in the vicinity of the project are considered potentially active. Potentially active is a rather alarming and unfortunate term in that it suggests a higher degree of risk than is justified, in most cases.

No faults were observed on the site either at the surface or in the borings. Because the site area is underlain by dense cemented soils, little amplification of ground motion would be expected during an earthquake. Liquefaction potential is minimal due to the types of materials present.

A moderate potential for dust generation is present if grading is performed in dry weather. Some potential for flash flooding may be present. No other geologic hazards were identified.

DISCUSSION AND RECOMMENDATIONS

General Information

The site is overlain by interbedded weakly to moderately cemented silty sand, sandy silt and sandy gravel soils that extended to a depth of about 15 feet. At this depth a well developed caliche layer 2-1/2 to over 5 feet thick was found in all 8 borings. The caliche is underlain by soils with various degrees of cementation that extend to the depth of exploration. A second caliche layer occurs across the site at depths starting at 20 to 28 feet below existing ground. A third caliche unit was found at depths below 37 feet. This unit is less continuous than the two upper layers.

The recommendations provided herein, and particularly under **Site Preparation, Grading and Filling, Foundation Design, Site Drainage** and **Quality Control** are intended to minimize risks of structural distress related to consolidation or expansion of native soils and/or structural fills. These recommendations, along with proper design and construction of the structure and associated improvements, work together as a system to improve overall performance. If any aspect of this system is ignored or poorly implemented, the performance of the project will suffer.

All structures should be designed for seismic zone 2. A site coefficient(s) of 1.2 should be used for design of the structures. Structural areas referred to in this report include all areas of foundations, mechanical pits, concrete slabs, asphalt pavements, as well as pads for any minor structures. All compaction requirements presented in this report are relative to ASTM D1557-78. For the purposes of this project, fine grained soils are defined as those with more than 40 percent by weight passing the number 200 sieve. Clay soils are defined as those with more than 30 percent passing the number 200 sieve and a plastic index greater than 15. Granular soils are those not defined by the above criteria. Sufficient quality control should be performed to verify that the recommendations presented in this report are followed.

Any evaluation of the site for the presence of surface or subsurface hazardous substances is generally beyond the scope of this investigation. When suspected hazardous substances are encountered during routine geotechnical investigations, they are noted in the exploration logs and immediately reported to the client. Soil samples were checked for the presence of hydrocarbon contaminates using a portable photoionization meter. No contamination was identified during our exploration.

Site Preparation

All vegetation should be stripped and grubbed from structural areas and removed from the site. Any debris and existing asphalt should also be removed. Existing fill will need to be removed from areas of shallow spread footings or concrete slabs-on-grade. For drilled shaft foundations the fill can remain in place. All areas to receive structural fill or structural loading should be densified to at least 90 percent relative compaction. Caliche, where encountered, should be cleaned of loose material but not compacted.

Trenching and Excavation

Temporary trenches with near vertical side walls should be stable to a depth of approximately 3.5 feet. Excavations to greater depths will require shoring or laying back of sidewalls to maintain adequate stability. Regulations amended in Part 1926, Volume 54, Number 209 of the Federal Register (Table B-1, October 31, 1989) require that the temporary sidewall slopes be no greater than those presented in Table 2.

TABLE 2 - Maximum Allowable Temporary Slopes

Soil or Rock Type	Maximum Allowable Slopes[1] For Deep Excavations Less Than 20 Feet Deep[2]	
Stable Rock	Vertical	(90 degrees)
Type A [3]	3H:4V	(53 degrees)
Type B	1H:1V	(45 degrees)
Type C	3H:2V	(34 degrees)

NOTES:

1. Numbers shown in parentheses next to maximum allowable slopes are angles expressed in degrees from the horizontal. Angles have been rounded off.
2. Sloping or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineer.
3. A short-term (open 24 hours or less) maximum allowable slope of 1H:2V (63 degrees) is allowed in excavations in Type A soil that are 12 feet or less in depth. Short-term maximum allowable slopes for excavations greater than 12 feet in depth shall be 3H:4V (53 degrees).

These Regulations, including the classification system and the maximum slopes, have been adopted and are strictly enforced by the State of Nevada, Department of Industrial Relations, Division of Occupational Safety and Health. In general, Type A soils are cohesive, non-fissured soils with an unconfined compressive strength of 1.5 tons per square foot (tsf) or greater. Type B are cohesive soils with an unconfined compressive strength between 0.5 and 1.5 tsf. While those designated as Type C have an unconfined compressive strength below 0.5 tsf. Numerous additional factors and exclusions are included in the formal definitions. The client, owner, design engineer and contractor shall refer to Appendix A and B of Subpart P of the previously referenced Federal Register for complete definitions and requirements on sloping and benching of trench sidewalls. Appendices C through F of Subpart P apply to requirements and methodologies for shoring.

On the basis of our exploration, the native soils down to depths of caliche are predominately Type B. All trenching should be performed and stabilized in accordance with local, state and OSHA standards.

Trench backfill should include no particles larger than 4 inches in maximum dimension. In general, bedding and initial backfill 12 inches over the pipe will require import, but native granular soil will provide adequate final backfill as long as oversized particles are excluded. Bedding and initial backfill should conform to the requirements of the utility having jurisdiction. Trenches should be backfilled in maximum 8-inch thick loose lifts in all structural areas. Each lift should be densified to a minimum of 90 percent relative compaction (ASTM D1557-78). Localized zones of perched groundwater may be encountered in deeper excavations.

Grading and Filling

Fine grained native clay soils should be placed as fill only in nonstructural areas. Native granular soils will be suitable for structural fill provided any particles larger than 4 inches are removed. If imported structural fill is required on this project, it should meet the guideline specifications of Table 3.

TABLE 3 - Guideline Specification for Imported Structural Fill

Sieve Size	Percent by Weight Passing
4 Inch	100
3/4 Inch	70 - 100
No. 40	15 - 70
No. 200	5 - 30

Percent Passing No. 200 Sieve	Maximum Liquid Limit	Maximum Plastic Index
5 - 10	50	20
11 - 20	40	15
21 - 30	35	10

These recommendations are intended as guidelines to specify a readily available, prequalified material. Adjustments to the recommended limits can be provided to allow the use of other granular, non-expansive material. Any such adjustments must be made and approved by the geological engineer, in writing, prior to importing fill to the site.

Any structural fill or backfill within the foundation areas should be placed in maximum 8 inch thick (loose) lifts, each densified to at least 95 percent relative compaction. All other structural fill, and utility trench backfill in all other structural areas, should be densified to a minimum 90 percent relative compaction. Nonstructural fill should be densified to a least 85 percent relative compaction to minimize consolidation and erosion.

Overcrossing Foundation Design

The near surface soils are relatively poor foundation materials for large loads. Foundations bearing directly in these materials could experience considerable levels of settlement. Three foundation alternatives were considered: Spread footings on structural fill, shallow end-bearing shafts, and deep drilled shafts. Each is discussed below:

Spread Footings (Alternate 1)

Footings underlain by a minimum of 3 feet of structural fill can be designed for a net maximum allowable bearing pressure of 2500 pounds per square foot. The net allowable bearing pressure is that pressure at the base of the footing in excess of the adjacent overburden pressure. This allowable bearing value should be used for dead plus ordinary live loads. Ordinary live loads are defined as being that portion of the design live load which will be present during the majority of the life of the structure. Design live loads are those loads which are produced by the use and occupancy of the building such as by moveable objects including people or equipment. This bearing value may be increased by 1/3 for total loads. Total loads are defined as the maximum load imposed by the required combinations of dead load, design live loads, snow loads, and wind or seismic loads. With this allowable bearing pressure, total settlements of approximately 3/4-inch should be anticipated with differential settlements of approximately one-half of this amount.

It is anticipated that overexcavation will be necessary for the placement of 3 feet of structural fill beneath footing. This overexcavation should extend, at least, 1.5 feet beyond the perimeter of the footings. Any overexcavated fine grained soils should be reused as fill only in nonstructural areas.

Lateral loads, such as wind or seismic, may be resisted by passive soil pressure and friction on the bottom of the footing. The coefficient of friction is 0.45 for mass concrete bearing in structural fill. Design values for active and passive equivalent fluid pressures are 40 and 350 pounds per cubic foot per foot of depth, respectively. These design values are based on spread footings bearing on structural fill and backfilled with native soil. All exterior footings should be placed a minimum 2 feet below adjacent finish grade for frost protection.

Shallow Drilled Piers Bearing in Caliche (Alternate 2)

The site is underlain fairly uniformly by a well developed, resistant caliche layer at depths of approximately 15 feet. The top of this unit varies in depth from about 13.5 to 17 feet below ground elevation at the time of our exploration. This caliche unit is typically 3 to 5 feet in thickness and underlain by compact weakly cemented silty and sandy gravels, probably representing poorly developed caliche. In the event that adequate bearing is not encountered at the anticipated depths it will be necessary to extend the drilling to next substantial caliche layers, a depth of about 25 feet. At this depth it would be necessary to coat the sidewalls of the excavation with bentonite in order to decrease skin friction and allow development of adequate end-bearing.

Our calculations indicate that the proposed column loads can be supported on 4 foot diameter drilled shafts, end bearing in the caliche. The shaft should be drilled to penetrate 4 inches into the resistant material. Based on these conditions, the net maximum allowable bearing pressure is 20,000 pounds per square foot for piers of 4 foot diameter or larger. The net allowable bearing pressure is that pressure at the base of the pier in excess of adjacent overburden pressure. Concrete should be poured stiff (slump of 2 to 4 inches) to decrease skin friction. If more than one shaft is used per pier the center to center spacing should be at least one and one half times the diameter to minimize overlapping stresses in the foundation caliche.

This allowable bearing value should be used for dead plus ordinary live loads. Ordinary live loads are defined as being that portion of the design live load which will be present during the majority of the life of the structure. Design live loads are those loads which are produced by the use and occupancy of the building such as by moveable objects including people or equipment. This bearing value may be increased by 1/3 for total loads. Total loads are defined as the maximum load imposed by the required combinations of dead load, design live loads, snow loads, and wind or seismic loads. With this allowable bearing pressure, total settlements of approximately 3/4-inch should be anticipated with differential settlements of approximately one-half of this amount.

Due to their high stiffness, drilled shaft foundations have an inherent capacity to dissipate large lateral loads. Lateral load analysis was performed using the finite element method of Bowles (1988) and a minimum concrete 28 day compressive strength of 3000 psi. This analysis indicates that lateral deflections of approximately 1/4 inch would be expected from lateral loads of 30 kips. A graph of lateral load versus deflection is presented as Plate 7. This analysis was based on a free head condition with top of pier at or below ground surface. No effects of a pier cap of any configuration were considered. The analysis was based on the following design soil profile

TABLE 4 - Design Profile For Shallow Drill Piers

Depth-feet	Modulus of Subgrade Reaction (Ks)	Effective Unit Weight
0 - 8	$38 + 25 Z^1$ KCF	120 PCF
8 - 15	$75 + 49 Z^1$ KCF	120 PCF
15 - 16	$225 + 90 Z^1$ KCF	120 PCF

Where Z = depth in feet.

Deep Drilled Shafts (Alternate 3)

Deep drilled shafts were given preliminary consideration but were not found to be economical for this project. Deep drilled shafts are supported primarily by skin friction. It is generally considered that excessive settlement must occur before end bearing strength is fully mobilized. Our calculations indicate depths of about 45 feet would be necessary to develop sufficient skin friction capacity with a 4 foot diameter shaft.

Foundation Design for Other Structures

The concrete floor slab for the escalator equipment pit can bear directly in native soils prepared and densified as previously described under **Site Preparation.** If bearing pressures within any portion of this pit exceeds 1500 pounds per square foot, overexcavation and replacement of native soils may be required. The depth and width of overexcavation will be dependent upon foundation load, size and depth.

Control Room

The control room for the elevator can best be supported on spread footings. Footings bearing in native soils densified to at least 90 percent relative can be designed for a net maximum allowable bearing pressure of 1500 psf. If foundations are underlain by at least 2 feet of structural fill, this bearing pressure can be increased to 2500 psf. Overexcavation would likely be required for this alternate. Any foundation overexcavation should extend a minimum of 1.5 feet beyond the perimeter of the foundation

The net allowable bearing pressure is that pressure at the base of the footing in excess of the adjacent overburden pressure. This allowable bearing value should be used for dead plus ordinary live loads. Ordinary live loads are defined as being that portion of the design live load which will be present during the majority of the life of the structure. Design live loads are those loads which are produced by the use and occupancy of the building such as by moveable objects including people or equipment. This bearing value may be increased by 1/3 for total loads. Total loads are defined as the maximum load imposed by the required combinations of dead load, design live loads, snow loads, and wind or seismic loads. With this allowable bearing pressure, total settlements of approximately 3/4-inch should be anticipated with differential settlements of approximately one-half of this amount.

Escalator Pit

It is our understanding that the escalator pit will be supported by four drilled shaft foundations. The foundations criteria provided for the main structures are generally appropriate for this purpose. Shafts should be a minimum of 2 feet in diameter in order to maintain the end bearing capacity of 20,000 psf.

We anticipate that the escalator pit walls will be acting as retaining walls. The following recommendations are for retaining walls with vertical backfaces horizontal backfills and no long term surcharge adjacent to the top of the wall. While these recommendations may be suitable for other conditions, the geological engineer should be consulted. Equipment pit walls can be designed without granular backfill if designed using equivalent fluid density (active plus hydrostatic) of 78 pounds per cubic foot per foot of wall height, unless it is known that groundwater buildup will not be a problem behind the wall. For the latter case of no groundwater behind the wall, an equivalent fluid density of 40 pounds per cubic foot of wall height may be used, if native backfill is used.

Where relatively clean (5 percent or less passing the number 200 sieve) granular backfill is employed behind the wall, an equivalent fluid density of 30 pounds per cubic foot per foot of wall height can be employed for active pressure design if the thickness of granular fill is at least one half the height of the wall. If the granular backfill thickness is less than one half the wall height, an equivalent fluid density of 55 pounds per cubic foot per foot of wall height should be employed. In no case should the granular backfill thickness behind the wall be less than 2 feet.

Passive pressures can be used in design for pit walls where appropriate. An equivalent fluid density of 350 pounds per cubic foot per foot of wall length developing passive pressure can be used for native soil.

Slope Stability and Erosion Control

No major cut or fill slopes are planned for this project. Temporary (during construction and permanent (after construction) erosion control of disturbed areas will be required in accordance with the Clark County Standards. The contractor shall prevent dust from being generated during construction in compliance with all applicable county, state, and federal regulations.

Site Drainage

Adequate surface drainage should be provided away from the structure foundation and equipment pits. Ponding of water on finish grade or at the edge of pavements should be prevented by proper grading.

Concrete Slabs

All concrete slabs should be directly underlain by 6 inches of aggregate base with a minimum R-Value of 60. Type II aggregate base is the preferred alternate although other materials may be acceptable. Aggregate base courses should be densified to, at least, 95 percent relative compaction.

A modulus of subgrade reaction (K-Value) of 250 pounds per cubic inch should be used for design of concrete slabs. This is based on a standard 30-inch diameter load and should be adjusted for smaller or larger load areas. Type V cement should be used for all concrete work in contact with soil.

The Las Vegas area is a region with exceptionally low relative humidity. As a consequence, concrete flatwork is prone to excessive shrinking and curling. Concrete mix proportions and construction techniques, including the addition of water and improper curing, can adversely affect the finished quality of the concrete and result in cracking, curling, and spalling of slabs. We recommend that all placement and curing be performed in accordance with procedures outlined by the American Concrete Institute. Special considerations should be given to concrete placed and cured during hot or cold weather conditions. Proper control joints and reinforcing should be provided to minimize any damage resulting from shrinkage.

Asphaltic Concrete

Some patching of existing asphaltic concrete may be required for this project. Asphalt patching in parking lots should consist of a minimum of 4 inches of full depth asphaltic concrete. Roadways including Las Vegas Boulevard and Tropicana Avenue should be patched with a minimum of 6 inches of asphalt concrete.

ANTICIPATED CONSTRUCTION PROBLEMS

Drilled shaft foundations will require careful quality control to verify that foundation conditions are as anticipated and that adequate foundation bearing is available. Field decisions may require deep excavations. Localized zones of perched water may require minor pumping drilled shaft excavations.

QUALITY CONTROL

All plans and specifications should be reviewed for conformance with this geotechnical report and approved by the Geological Engineer prior to submitting to the building department for review.

The recommendations presented in this report are based on the assumption that sufficient field testing and construction review will be provided during all phases of construction. We should review the final plans and specifications for conformance with the intent of our recommendations. Prior to construction, a pre-job conference should be scheduled to include, but not be limited to, the Owner, Architect, Civil Engineer, the General Contractor, Earthwork and Materials Sub-Contractors, Building Official and Geological Engineer. The conference will allow parties to review the project plans, specifications, and recommendations presented in this report and discuss applicable material quality and mix design requirements. All quality reports should be submitted to, and reviewed by, the Geological Engineer.

During construction, we should have the opportunity to provide sufficient on-site observation of preparation and grading, overexcavation, fill placement, foundation installation and paving. These observations would allow us to verify that the geotechnical conditions are as anticipated and that the Contractor's work is in conformance with the approved plans and specifications.

STANDARD LIMITATION CLAUSE

This report has been prepared in accordance with generally accepted geotechnical practices. The analyses and recommendations submitted are based upon field exploration performed at the locations shown on Plate 1 - Plot Plan, of this report. This report does not reflect soils variations that may become evident during the construction period, at which time re-evaluation of the recommendations may be necessary. We recommend our firm be retained to perform construction observation in all phases of the project related to geotechnical factors to insure compliance with our recommendations. The owner shall be responsible for distribution of this geotechnical investigation to all designers and contractors whose work is related to geotechnical factors.

Equilibrium water level readings were made on the date shown on Plate 2 -Log of Borings, of this report. Fluctuations in the water table may occur due to rainfall, temperature, seasonal runoff, or adjacent irrigation practices. Construction planning should be based on assumptions of possible variations.

This report has been prepared to provide information allowing the Architect or Engineer to design the project. In the event of changes in the design or location of the project from the time of this report, recommendations should be reviewed and possibly modified by the Geological Engineer. If the Geological Engineer is not accorded the privilege of making this recommended review, he can assume no responsibility for misinterpretation or misapplication of his recommendations or their validity in the event changes have been made in the original design concept without his prior review. The Geological Engineer makes no other warranties, either expressed or implied, as to the professional advice provided under the terms of this agreement and included in this report.

REFERENCES

Baumer, Otto W., 1983, Soil Survey of Washoe County, Nevada, South Part, U.S.D.A. Soil Conservation Service.

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1

     CRSS PEDESTRIAN OVERCROSSING 4' DIA x 16' L DRILLED SHAFT

          ++++++++++++++++++ THIS OUTPUT FOR DATA FILE:  CRS.DTA

     NO OF NP =  20   NO OF ELEMENTS, NM =  9   NO OF NON-ZERO P, NNZP =  1
     NO OF LOAD CASES, NLC =  3                      NO OF CYCLES NCYC =  5
     NODE SOIL STARTS JTSOIL =  2
     NONLINEAR (IF > 0) =  1                NO OF BOUNDARY CONDIT NZX =  0
            NO OF SOILS =  3                         LIST BAND IF > 0 =  0
                                                        IMET (SI > 0) =  0

     MOD OF ELASTICITY E =   488500. KSF
     GROUND LINE REDUCTION FACTOR FOR PILES, REDFAC =  .70


 MEMNO   NP1   NP2   NP3   NP4   LENGTH     WIDTH    INERTIA,FT**4

     1     1     2     3     4     2.000     4.000  .12560E+02
     2     3     4     5     6     2.000     4.000  .12560E+02
     3     5     6     7     8     2.000     4.000  .12560E+02
     4     7     8     9    10     2.000     4.000  .12560E+02
     5     9    10    11    12     2.000     4.000  .12560E+02
     6    11    12    13    14     2.000     4.000  .12560E+02
     7    13    14    15    16     2.000     4.000  .12560E+02
     8    15    16    17    18     2.000     4.000  .12560E+02
     9    17    18    19    20     2.000     4.000  .12560E+02

     THE INITIAL INPUT P-MATRIX ENTRIES
     NP        P(NP,LC)
        ++ LOAD CASE =  1
      2        25.000
        ++ LOAD CASE =  2
      2        30.000
        ++ LOAD CASE =  3
      2        35.000

     THE ORIGINAL P-MATRIX WHEN NONLIN > 0 ++++++
         1          .00       25.00
         2          .00         .00
         3          .00         .00
         4          .00         .00
         5          .00         .00
         6          .00         .00
         7          .00         .00
         8          .00         .00
         9          .00         .00
        10          .00         .00

     THE NODE SOIL MODULUS, SPRINGS AND MAX DEFL:
     NODE    SOIL MODULUS    SPRING,K/FT   MAX DEFL,FT
```

3 of 8

4 of 8

```
    1          .0         .0         .0000
    2        38.0      153.1         .0500
    3        88.0      704.0         .0500
    4       138.0     1104.0         .0500
    5       188.0     1504.0         .0500
    6       238.0     1706.7         .0500
    7       140.0     1381.3         .0500
    8       238.0     1904.0         .0500
    9       336.0     2409.3         .0500
   10       225.0     1048.0         .0500

  MEMBER MOMENTS, NODE REACTIONS, DEFLECTIONS, SOIL PRESSURE, AND LAST USED P-MATRIX FOR LC =  1
MEMNO  MOMENTS--NEAR END 1ST, K-FT    NODE   SPG FORCE, KIPS   ROT, RADS    DEFL, FT   SOIL Q, KSF    P-, K-FT     P-, KIPS
  1        -.018     49.997            1           .00         -.00152      .01893*        .00         .00        25.00
  2      -49.992     95.102            2          2.43         -.00151      .01589         .60         .00          .00
  3      -95.109    122.071            3          9.07         -.00149      .01289        1.13         .00          .00
  4     -122.064    127.065            4         10.98         -.00145      .00995        1.37         .00          .00
  5     -127.066    110.770            5         10.65         -.00141      .00708        1.33         .00          .00
  6     -110.768     79.815            6          7.33         -.00137      .00430        1.02         .00          .00
  7      -79.815     44.493            7          2.18         -.00134      .00158         .22         .00          .00
  8      -44.489     13.292            8         -2.06         -.00132     -.00108         .26         .00          .00
  9      -13.290       .003            9         -8.95         -.00131     -.00372        1.25         .00          .00
                                      10         -6.64         -.00131     -.00634        1.43         .00          .00
  SUM SPRING FORCES =      24.99 VS SUM APPLIED FORCES =      25.00 KIPS

     (*) = SOIL DISPLACEMENT > XMAX SO SPRING FORCE AND Q = XMAX*VALUE ++++++++++++
    NOTE THAT P-MATRIX ABOVE INCLUDES ANY EFFECTS FROM X > XMAX ON LAST CYCLE ++++++++++

  MEMBER MOMENTS, NODE REACTIONS, DEFLECTIONS, SOIL PRESSURE, AND LAST USED P-MATRIX FOR LC =  2
MEMNO  MOMENTS--NEAR END 1ST, K-FT    NODE   SPG FORCE, KIPS   ROT, RADS    DEFL, FT   SOIL Q, KSF    P-, K-FT     P-, KIPS
  1         .018     60.002            1           .00         -.00182      .02271*        .00         .00        30.00
  2      -60.003    114.122            2          2.92         -.00181      .01907         .72         .00          .00
  3     -114.128    146.481            3         10.89         -.00179      .01547        1.36         .00          .00
  4     -146.472    152.489            4         13.18         -.00174      .01194        1.65         .00          .00
  5     -152.481    132.921            5         12.78         -.00170      .00850        1.60         .00          .00
  6     -132.926     95.777            6          8.80         -.00165      .00515        1.23         .00          .00
  7      -95.779     53.391            7          2.62         -.00161      .00190         .27         .00          .00
  8      -53.391     15.949            8         -2.47         -.00159     -.00130         .31         .00          .00
  9      -15.944       .001            9        -10.75         -.00158     -.00446        1.50         .00          .00
                                      10         -7.97         -.00157     -.00761        1.71         .00          .00
  SUM SPRING FORCES =      29.99 VS SUM APPLIED FORCES =      30.00 KIPS

     (*) = SOIL DISPLACEMENT > XMAX SO SPRING FORCE AND Q = XMAX*VALUE ++++++++++++
    NOTE THAT P-MATRIX ABOVE INCLUDES ANY EFFECTS FROM X > XMAX ON LAST CYCLE ++++++++++

  MEMBER MOMENTS, NODE REACTIONS, DEFLECTIONS, SOIL PRESSURE, AND LAST USED P-MATRIX FOR LC =  3
MEMNO  MOMENTS--NEAR END 1ST, K-FT    NODE   SPG FORCE, KIPS   ROT, RADS    DEFL, FT   SOIL Q, KSF    P-, K-FT     P-, KIPS
  1         .001     70.017            1           .00         -.00213      .02650*        .00         .00        35.00
  2      -70.001    133.139            2          3.41         -.00212      .02225         .85         .00          .00
  3     -133.148    170.891            3         12.70         -.00208      .01805        1.59         .00          .00
```

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 4      -170.898     177.898        4        15.37       -.00203      .01393        1.92        .00        .00
 5      -177.897     155.072        5        14.91       -.00198      .00991        1.86        .00        .00
 6      -155.077     111.743        6        10.26       -.00192      .00601        1.43        .00        .00
 7      -111.742      62.289        7         3.06       -.00188      .00221         .31        .00        .00
 8       -62.288      18.607        8        -2.89       -.00185     -.00152         .36        .00        .00
 9       -18.602       -.001        9       -12.54       -.00184     -.00520        1.75        .00        .00
                                   10        -9.30       -.00184     -.00888        2.00        .00        .00
SUM SPRING FORCES =      34.99 VS SUM APPLIED FORCES =      35.00 KIPS

   (*) = SOIL DISPLACEMENT > XMAX SO SPRING FORCE AND Q = XMAX*VALUE ++++++++++++
  NOTE THAT P-MATRIX ABOVE INCLUDES ANY EFFECTS FROM X > XMAX ON LAST CYCLE ++++++++++
```

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     DATA FILE NAME: CRS.DTA

     CRSS PEDESTRIAN OVERCROSSING 4' DIA x 16' L DRILLED SHAFT


     FOR INPUT SUBGRADE REACTION Ks =     38. +      25.*Z**  1.000   XMX =   .050
        THE NODE Ks VALUES ARE:
          1          .0      .0000
          2        38.0      .0500
          3        88.0      .0500
          4       138.0      .0500
          5       188.0      .0500
          6       238.0      .0500

     FOR INPUT SUBGRADE REACTION Ks =    140. +      49.*Z**  1.000   XMX =   .050
        THE NODE Ks VALUES ARE:
          7       140.0      .0500
          8       238.0      .0500
          9       336.0      .0500

     FOR INPUT SUBGRADE REACTION Ks =    225. +      90.*Z**  1.000   XMX =   .050
        THE NODE Ks VALUES ARE:
         10       225.0      .0500
1

     CRSS PEDESTRIAN OVERCROSSING 4' DIA x 16' L DRILLED SHAFT

          +++++++++++++++++ THIS OUTPUT FOR DATA FILE:  CRS.DTA

     NO OF NP =  20   NO OF ELEMENTS, NM =  9   NO OF NON-ZERO P, NNZP =  1
     NO OF LOAD CASES, NLC =  3                       NO OF CYCLES NCYC =  5
     NODE SOIL STARTS JTSOIL =  2
     NONLINEAR (IF > 0) =  1                  NO OF BOUNDARY CONDIT NZX =  0
            NO OF SOILS =  3                           LIST BAND IF > 0 =  0
                                                          IMET (SI > 0) =  0

     MOD OF ELASTICITY E =   488500. KSF
     GROUND LINE REDUCTION FACTOR FOR PILES, REDFAC =  .70


 MEMNO   NP1   NP2   NP3   NP4   LENGTH     WIDTH    INERTIA,FT**4

     1     1     2     3     4    2.000     4.000  .12560E+02
     2     3     4     5     6    2.000     4.000  .12560E+02
     3     5     6     7     8    2.000     4.000  .12560E+02
     4     7     8     9    10    2.000     4.000  .12560E+02
     5     9    10    11    12    2.000     4.000  .12560E+02
     6    11    12    13    14    2.000     4.000  .12560E+02
     7    13    14    15    16    2.000     4.000  .12560E+02
     8    15    16    17    18    2.000     4.000  .12560E+02
```

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9    17    18    19    20     2.000      4.000  .12560E+02

THE INITIAL INPUT P-MATRIX ENTRIES
NP        P(NP,LC)
   ++ LOAD CASE =  1
 2        10.000
   ++ LOAD CASE =  2
 2        15.000
   ++ LOAD CASE =  3
 2        20.000

THE ORIGINAL P-MATRIX WHEN NONLIN > 0 ++++++
    1          .00       10.00
    2          .00         .00
    3          .00         .00
    4          .00         .00
    5          .00         .00
    6          .00         .00
    7          .00         .00
    8          .00         .00
    9          .00         .00
   10          .00         .00

THE NODE SOIL MODULUS, SPRINGS AND MAX DEFL:
NODE    SOIL MODULUS    SPRING,K/FT   MAX DEFL,FT
   1              .0            .0        .0000
   2            38.0         153.1        .0500
   3            88.0         704.0        .0500
   4           138.0        1104.0        .0500
   5           188.0        1504.0        .0500
   6           238.0        1706.7        .0500
   7           140.0        1381.3        .0500
   8           238.0        1904.0        .0500
   9           336.0        2409.3        .0500
  10           225.0        1048.0        .0500

BASE SUM OF NODE SPRINGS =   11980.0 K/FT NO ADJUSTMENTS

 MEMBER MOMENTS, NODE REACTIONS, DEFLECTIONS, SOIL PRESSURE, AND LAST USED P-MATRIX FOR LC =  1
MEMNO  MOMENTS--NEAR END 1ST, K-FT     NODE   SPG FORCE, KIPS   ROT, RADS     DEFL, FT   SOIL Q, KSF    P-, K-FT     P-, KIPS
  1          -.001      19.999          1            .00        -.00061      .00757*         .00         .00        10.00
  2        -20.000      38.039          2            .97        -.00060      .00636          .24         .00          .00
  3        -38.042      48.829          3           3.63        -.00060      .00516          .45         .00          .00
  4        -48.829      50.828          4           4.39        -.00058      .00398          .55         .00          .00
  5        -50.828      44.308          5           4.26        -.00057      .00283          .53         .00          .00
  6        -44.308      31.926          6           2.93        -.00055      .00172          .41         .00          .00
  7        -31.926      17.797          7            .87        -.00054      .00063          .09         .00          .00
  8        -17.797       5.316          8           -.82        -.00053     -.00043          .10         .00          .00
  9         -5.317        .001          9          -3.58        -.00053     -.00149          .50         .00          .00
                                       10          -2.66        -.00052     -.00254          .57         .00          .00
 SUM SPRING FORCES =      10.00 VS SUM APPLIED FORCES =      10.00 KIPS
```

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     (*) = SOIL DISPLACEMENT > XMAX SO SPRING FORCE AND Q = XMAX*VALUE ++++++++++++
    NOTE THAT P-MATRIX ABOVE INCLUDES ANY EFFECTS FROM X > XMAX ON LAST CYCLE ++++++++++


  MEMBER MOMENTS, NODE REACTIONS, DEFLECTIONS, SOIL PRESSURE, AND LAST USED P-MATRIX FOR LC =  2
MEMNO  MOMENTS--NEAR END 1ST, K-FT    NODE   SPG FORCE, KIPS   ROT, RADS    DEFL, FT   SOIL Q, KSF    P-, K-FT     P-, KIPS
  1            .009     30.001         1           .00        -.00091     .01136*        .00          .00        15.00
  2         -30.002     57.061         2          1.46        -.00091     .00954         .36          .00          .00
  3         -57.064     73.241         3          5.44        -.00089     .00773         .68          .00          .00
  4         -73.236     76.245         4          6.59        -.00087     .00597         .82          .00          .00
  5         -76.241     66.461         5          6.39        -.00085     .00425         .80          .00          .00
  6         -66.463     47.888         6          4.40        -.00082     .00258         .61          .00          .00
  7         -47.889     26.696         7          1.31        -.00081     .00095         .13          .00          .00
  8         -26.696      7.974         8         -1.24        -.00079    -.00065         .15          .00          .00
  9          -7.972       .000         9         -5.37        -.00079    -.00223         .75          .00          .00
                                      10         -3.99        -.00079    -.00380         .86          .00          .00
  SUM SPRING FORCES =      14.99 VS SUM APPLIED FORCES =      15.00 KIPS

     (*) = SOIL DISPLACEMENT > XMAX SO SPRING FORCE AND Q = XMAX*VALUE ++++++++++++
    NOTE THAT P-MATRIX ABOVE INCLUDES ANY EFFECTS FROM X > XMAX ON LAST CYCLE ++++++++++


  MEMBER MOMENTS, NODE REACTIONS, DEFLECTIONS, SOIL PRESSURE, AND LAST USED P-MATRIX FOR LC =  3
MEMNO  MOMENTS--NEAR END 1ST, K-FT    NODE   SPG FORCE, KIPS   ROT, RADS    DEFL, FT   SOIL Q, KSF    P-, K-FT     P-, KIPS
  1           -.002     39.998         1           .00        -.00122     .01514*        .00          .00        20.00
  2         -39.999     76.079         2          1.95        -.00121     .01271         .48          .00          .00
  3         -76.083     97.659         3          7.26        -.00119     .01031         .91          .00          .00
  4         -97.657    101.655         4          8.79        -.00116     .00796        1.10          .00          .00
  5        -101.656     88.615         5          8.52        -.00113     .00566        1.06          .00          .00
  6         -88.616     63.853         6          5.86        -.00110     .00344         .82          .00          .00
  7         -63.852     35.595         7          1.75        -.00107     .00126         .18          .00          .00
  8         -35.594     10.631         8         -1.65        -.00106    -.00087         .21          .00          .00
  9         -10.634       .003         9         -7.16        -.00105    -.00297        1.00          .00          .00
                                      10         -5.32        -.00105    -.00507        1.14          .00          .00
  SUM SPRING FORCES =      19.99 VS SUM APPLIED FORCES =      20.00 KIPS

     (*) = SOIL DISPLACEMENT > XMAX SO SPRING FORCE AND Q = XMAX*VALUE ++++++++++++
    NOTE THAT P-MATRIX ABOVE INCLUDES ANY EFFECTS FROM X > XMAX ON LAST CYCLE ++++++++++
```

LOG OF BORING

LOG NO.: B-1

GROUND ELEVATION.:

LOGGED BY: N. Saines

GROUND WATER DEPTH: 19.9 feet

DATE: 02-25-93

DATE MEASURED: 03-02-93

TYPE OF BORING: Rotary

NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	Graphic Log	DESCRIPTION
This may be filled in wash. Interpreted as fill to 8', hard at 6.5'.						GP		0-4.5: Dry, loose, light brown Sandy Gravel.
					2			
	1-A	■		22				
					4			
						SP		4.5-8.0: Dry, loose, yellow brown Gravelly Sand.
	1-B	■		19				
					6			
Drilling easier.					8	CL/SC		8.0-12.0: Slightly moist, medium dense, gray Sandy Clay-Clayey Sand - partially cemented.
					10			
	1-C	■		40				
					12	SM		12.0-15.0: Slightly moist, medium dense, gray Silty Sand - partially cemented.
					14			
Sample from shoe.	1-D	■		30-1"				15.0-19.5: Caliche - Light brown Sandy Silty Gravel - cemented. Strongly cemented rock, mostly gray limestone fragments cemented together. Partially cemented 17-18 feet.
					16			
					18			
Slightly musty smell - like sewage.						GM		19.5-25.0: Moist, medium dense, light brown Silty Gravel - partially cemented.
	1-E	■		12	20			
					22			
Softer 24-25'.					24			
No recovery.		■		50-3"				25.0-31.5: Caliche
					26			

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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LAS VEGAS, NEVADA
PHOENIX, ARIZONA

CRSS
Pedestrian Crossing
Clark County, Nevada

Sheet 1 of 2
Project No. 2-765-01-4
Plate 2-A

LOG OF BORING

LOG NO.: B-1
GROUND ELEVATION.:
LOGGED BY: N. Saines
GROUND WATER DEPTH: 19.9 feet
DATE: 02-25-93
DATE MEASURED: 03-02-93
TYPE OF BORING: Rotary

NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	DESCRIPTION
							25.0-31.5: Caliche
Penetration rate: 10-12 min./ft.					28		
Caliche cuttings.	1-F						
					30		
					32	GM	31.5-38.0: Moist, medium dense, light brown Silty Gravel with trace of clay. Partially cemented or gravelly.
					34		
	1-G	■		23			
					36		
Fines plugging bit.					38	ML	38.0-45.5: Moist, stiff, brown Silt with trace of pebbles, changing to mottled white and brown Silt. Partially cemented from 41.5 to 43.5 feet.
					40		
	1-H	■		62			
					42		
					44		
				12-6"			
	1-I	■		40-1"		GM	45.5-46.0: Moist, dense, gray Silty Gravel.
					46		46.0-50.5: Caliche - cemented sand and gravel.
Hard drilling to 50'.					48		
					50		
					52		

Graphic Log

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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PHOENIX, ARIZONA

CRSS
Pedestrian Crossing
Clark County, Nevada

Sheet 2 of 2
Project No. 2-765-01-4
Plate 2-A

LOG OF BORING

LOG NO.: B-2

GROUND ELEVATION.:

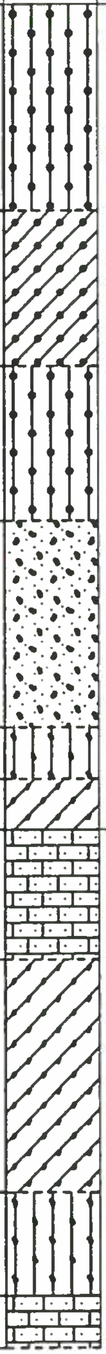
LOGGED BY: N. Saines

GROUND WATER DEPTH: 23.9 feet

DATE: 02-25-93

DATE MEASURED: 03-02-93

TYPE OF BORING: Rotary



NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	Graphic Log	DESCRIPTION
						SM		0-4.0: Slightly moist, loose, yellow-brown Fine Silty Sand.
	2-A	■		9	2			
					4	SC		4.0-7.0: Slightly moist, loose, yellow-brown Fine Clayey Sand - trace of silt.
	2-B	■		39				
					6			
						SM		7.0-10.0: Silty Sand - white in color.
Easy drilling.					8			
Harder drilling.	2-C	■		39	10	GP		10.0-14.0: Slightly moist, medium dense, yellow-brown Sandy Gravel - trace of silt.
					12			
				35-6"	14	GM		14.0-15.0: Silty Gravel.
	2-D	■		15-2"		GC		15.0-16.0: Moist, moderately dense, light gray Clayey Gravel - pebbles to 2" diameter.
Very hard drilling.					16			16.0-18.5: Caliche
Caliche chips.	2-E							
					18	GC		18.5-23.0 Moist, moderately dense, light brown Clayey Gravel - pebbles to 2" diameter. Partially cemented or gravelly.
				28-6"				
	2-F	■		25-3"	20			
Hard.					22			
						GM		23.0-25.0: Dense Silty Gravel.
				30-1"	24			
No recovery.		■						25.0-33.0: Caliche
					26			

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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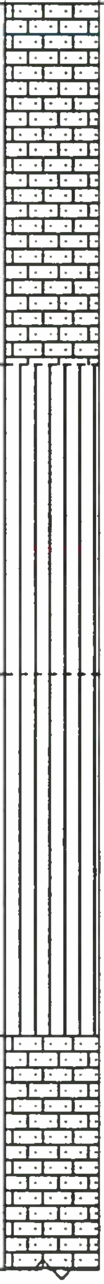
Clark County, Nevada

Sheet 1 of 2
Project No. 2-765-01-4
Plate 2-B

LOG OF BORING

LOG NO.: B-2
LOGGED BY: N. Saines
DATE: 02-25-93
TYPE OF BORING: Rotary
GROUND ELEVATION.:
GROUND WATER DEPTH: 23.9 feet
DATE MEASURED: 03-02-93



NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	Graphic Log	DESCRIPTION
Caliche Cuttings.	2-G							25.0-33.0: Caliche
Penetration rate: greater than 8 min./ft.					28			
					30			
					32			
						ML		33.0-39.0: Moist, stiff, light brown Silt - minor sand, trace of clay. Gravelly from 36.5 to 37.5 feet.
					34			
	2-H	■		32				
Hard drilling.					36			
					38			
				6-6"		ML		39.0-46.0: Dry, stiff, light brown Silt. Partly cemented.
Easy drilling.	2-I	■		50-5"	40			
					42			
Easy drilling.								
Hard drilling.					44			
	2-J	■		50-6"				
Very hard drilling.					46			46.0-50.5: Caliche
					48			
	2-K							
Caliche Cuttings.					50			
					52			

EXPLANATION
Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.
Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.
PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location
▨ Thin Wall Shelby Location

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LAS VEGAS, NEVADA
PHOENIX, ARIZONA

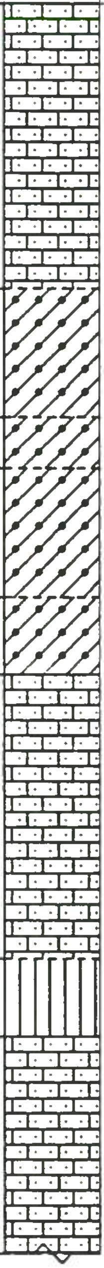
CRSS
Pedestrian Crossing
Clark County, Nevada

Sheet 2 of 2
Project No. 2-765-01-4
Plate 2-B

LOG OF BORING

LOG NO.: B-3
LOGGED BY: N. Saines
DATE: 02-25-93
TYPE OF BORING: Rotary

GROUND ELEVATION.:
GROUND WATER DEPTH: 21.60 feet
DATE MEASURED: 03-02-93



NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	Graphic Log	DESCRIPTION
	3-G							
Caliche cuttings - penetration rate: 15 min./ft.								11.6-31.5: Caliche
					28			
Penetration rate: 12 min./ft.					30			
						SC		31.5-34.0: Moist, medium stiff, light brown, fine to medium Clayey Sand, with some coarser grains.
Cuttings below Caliche					32			
	3-H/3-I	■		10				
					34	SC		34.0-35.0: Gravelly Clayey Sand
						SC		35.0-37.5: Clayey Sand
					36			
						SC		37.5-39.0: Gravelly Clayey Sand
					38			
								39.0-44.5: Caliche
					40			
					42			
Very hard drilling								
				27-6"	44	ML		44.5-46.0: Slightly moist, hard, light brown, partially cemented Silt with some coarser material.
	3-J	■		30-2"				
					46			46.0-50.2: Caliche
Hard drilling					48			
					50			
					52			

EXPLANATION
Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.
Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.
PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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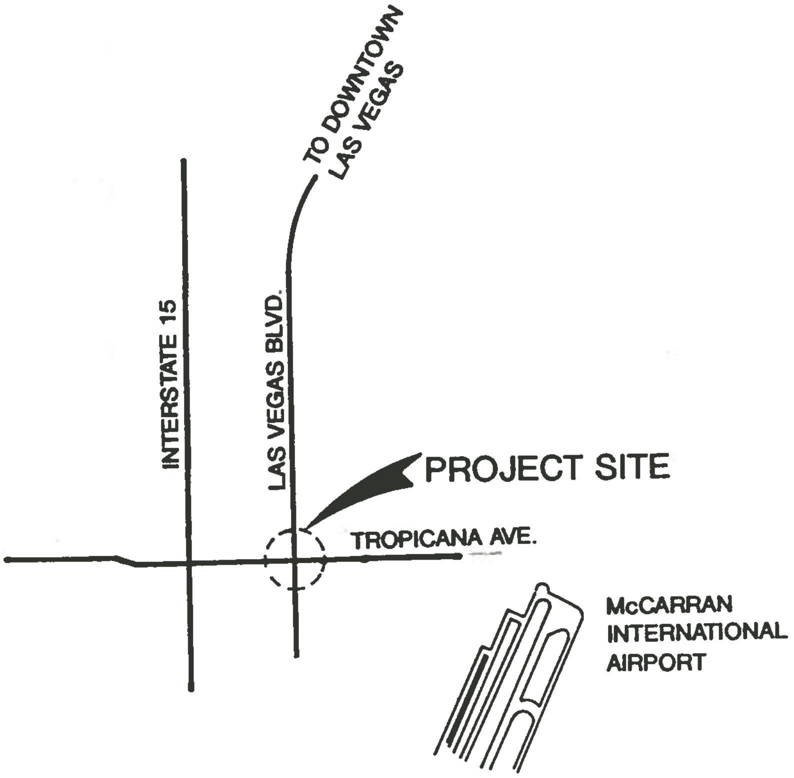
Plate 2-C



VICINITY MAP

LEGEND

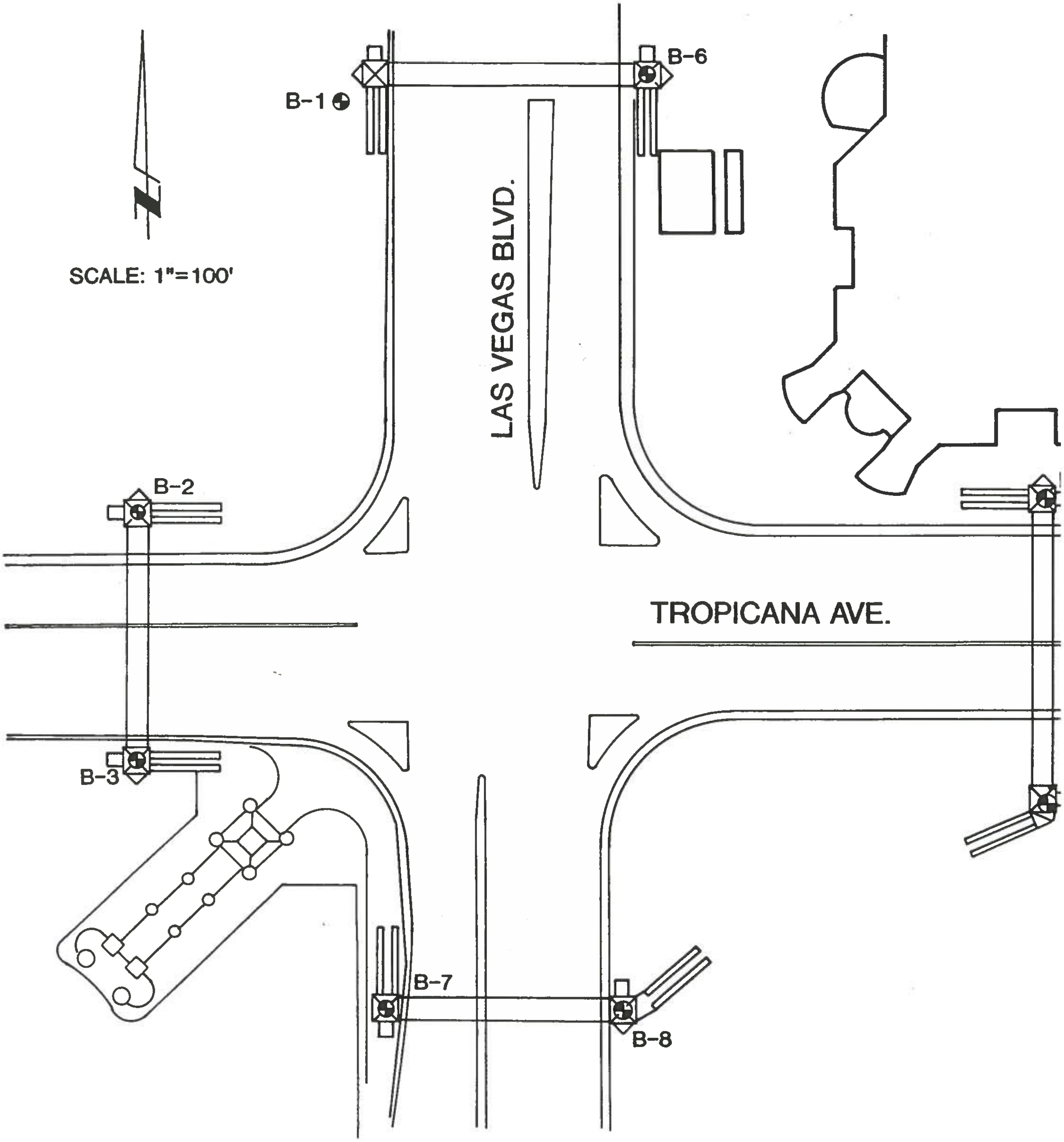
B-1 ● BORING LOCATION

OVERCROSSING
AND TROPICANA DRIVE

NEVADA

Project No. 2-765-01-4

Plate No. 1



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PEDESTR
LAS VEGAS BL

LOG OF BORING

LOG NO.: B-4

LOGGED BY: N. Saines

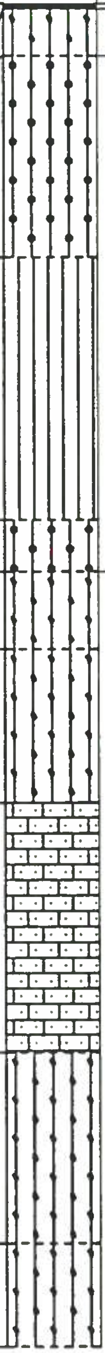
DATE: 03-01-93

TYPE OF BORING: Rotary

GROUND ELEVATION.:

GROUND WATER DEPTH: 18.5 feet

DATE MEASURED: 03-02-93

NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	Graphic Log	DESCRIPTION
						AC		0-0.1: Asphaltic Concrete.
						GM		0.1-1.0: Aggregate Base.
						SM		1.0-4.9: Moist, brown to light brown Silty Sand with some coarser grains.
					2			
	4-A	■		4				
					4			
	4-B	■		5		ML		4.9-10.0: Moist, gray Silt. More plastic, more clay, reddish at top of sample.
					6			
					8			
					10	SM		10.0-11.0: Dry, loose, gray, slightly to partially cemented Sand.
	4-C	■		21				
						GM		11.0-12.5: Dry, loose, greenish gray Silty Gravel.
					12			
						GM		12.5-15.5: Slightly moist, dense, light brown Silty Gravel. Cemented at bottom of sample. Lenses of Silty Sand, cemented.
					14			
	4-D/4-E	■		30-2"				15.5-20.3: Hard Caliche
Cemented gravel or caliche sample from 16'					16			
Penetration rate: 9 min./ft.					18			
					20	GM		20.3-24.0: Moist, loose, very light brown Silty Gravel. Trace of clay, pebbles to 2" diameter.
	4-F			14				
					22			
Caving a little, had to inject water								
					24	GM		24.0-27.0: Moist, loose, brown Silty Gravel. Sandy in places.
	4-G			10				
					26			

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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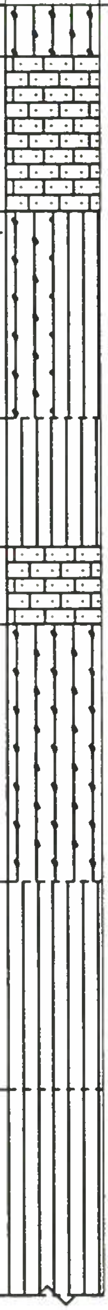
LOG OF BORING

LOG NO.: *B-4*
GROUND ELEVATION.:
LOGGED BY: *N. Saines*
GROUND WATER DEPTH: *18.5 feet*
DATE: *03-01-93*
DATE MEASURED: *03-02-93*
TYPE OF BORING: *Rotary*

NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	DESCRIPTION
						GM	
					28		27.0-30.0: Caliche
Hit water beneath caliche came up to about 25'					30	GM/ML	30.0-34.0: Moist, stiff to medium dense to soft, brown Silty Gravel to Silt.
	4-H/4-I	■		32			
					32		
					34	ML	34.0-36.5: Moist, stiff, mottled light brown and gray-green, plastic Clayey Silt.
	4-J/4-K	■		23			
					36		36.5-38.0: Caliche
					38	GM	38.0-43.0: Wet, loose, light brown to gray Silty Sandy Gravel with pebbles to 2" diameter.
					40		
	4-L	■		50-6"			
					42		
						ML	43.0-47.0: Moist, stiff, light brown Silt with traces of clay and coarser grains. Cemented from 46.5 to 47.5 feet.
					44		
	4-M/4-N	■		23			
					46		
						ML	47.0-51.0: Moist, hard, brown Silt with traces of clay and coarser grains. Partially cemented.
					48		
					50		
	4-0	■		41			
					52		



EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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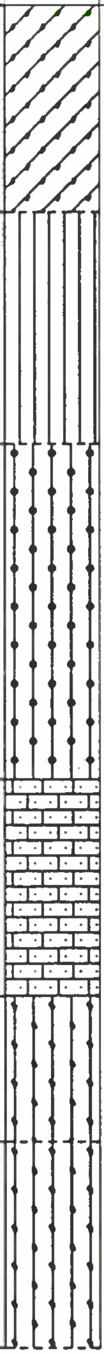
LOG OF BORING

LOG NO.: B-5
GROUND ELEVATION.:
LOGGED BY: N. Saines
GROUND WATER DEPTH: 23.6 feet
DATE: 02-26-93
DATE MEASURED: 03-02-93
TYPE OF BORING: Rotary

NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	DESCRIPTION
						GC	0-4.0: Slightly moist, medium dense, light brown Clayey Gravelly Fill.
					2		
	5-A	■		28			
					4		
						ML	4.0-8.5: Moist, stiff, greenish gray Clayey Silt, some pebbles. Changes to brown at 8 feet.
	5-B	■		16	6		
					8		
						SM	8.5-15.0: Moist, loose, mottled light brown and gray, fine to medium Silty Sand. Grayer at 12 feet.
					10		
	5-C	■		27			
					12		
					14		
							15.0-19.2: Hard Caliche.
					16		
					18		
						GM	19.2-22.0: Moist, loose to medium dense, light brown Sandy Silty Gravel with lenses of high plastic Gravelly Clayey Sand. Silty or clayey from 22.0 to 23.5 feet.
		■			20		
	5-D			17	22		
						GM	22.0-27.0: Moist, loose to medium dense, light brown Sandy Silty Gravel, changing to moist, stiff Silt with coarse materials present.
Fines clogging bit - slower drilling					24		
	5-E	■		32	26		

Graphic Log



EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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Plate 2-E

LOG OF BORING

LOG NO.: B-5
GROUND ELEVATION.:
LOGGED BY: N. Saines
GROUND WATER DEPTH: 23.6 feet
DATE: 02-26-93
DATE MEASURED: 03-02-93
TYPE OF BORING: Rotary

NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	Graphic Log	DESCRIPTION
						GM		
					28 30			27.0-31.5: Caliche
					32 34	ML		31.5-35.0: Slightly moist, stiff, light brown, partially cemented Silt.
	5-F	■		36	36 38	GC		35.0-38.5: Slightly moist, dense, light brown, partially cemented Clayey Sandy Gravel.
Fines plugging bit.	5-G	■		36	40 42	ML		38.5-44.0: Slightly moist, stiff, light brown, partially cemented Silt.
	5-H/5-I	■		10	44	GC/ML		44.0-46.0: Moist, mottled light brown to gray Clayey Gravel, some pebbles, changing to moist, stiff, partly cemented Silt.
	5-J	■		21	46 48 50	ML		46.0-51.0: Moist, medium stiff, brown, partly cemented Silt.
					52			

EXPLANATION
Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.
Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.
PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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Plate 2-E

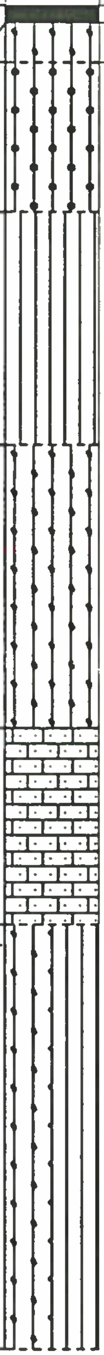
LOG OF BORING

LOG NO.: B-6
GROUND ELEVATION.:
LOGGED BY: N. Saines
GROUND WATER DEPTH: 23.2 feet
DATE: 03-02-93
DATE MEASURED: 03-02-93
TYPE OF BORING: Rotary

NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet
					2
	6-A	■		5	
					4
	6-B	■		6	
					6
					8
					10
2" limestone pebble in shoe	6-C	■		32	
					12
					14
					16
					18
					20
		■			
	6-D			50-4"	
					22
					24
	6-E	■		12-6"	
					26

Soil Class	DESCRIPTION
AC	0-0.33: Asphaltic Concrete
GM	0.33-1.1: Aggregate Base
SM	1.1-4.0: Moist, soft, brown Fine Silty Sand.
ML	4.0-8.5: Slightly moist, soft to medium stiff, mottled brown and green Silt, small roots present. Changes to greenish gray at 7 feet.
GM	8.5-14.0: Moist, dense, light yellow and gray, slightly cemented Silty Sandy Gravel with large clasts to 2" diameter.
	14.0-17.8: Caliche
GM/ML	17.8-26.5: Moist, very dense, light brown Silty Gravel, interbedded with Sandy Silt and Gravelly Clayey Sand.

Graphic Log



EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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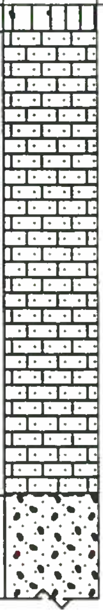
Plate 2-F

LOG OF BORING

LOG NO.: B-6
GROUND ELEVATION.:
LOGGED BY: N. Saines
GROUND WATER DEPTH: 23.2 feet
DATE: 03-02-93
DATE MEASURED: 03-02-93
TYPE OF BORING: Rotary

NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	Graphic Log	DESCRIPTION
				20-1"		GM/ML		26.5-35.5: Caliche
Penetration rate: 6 min./ft.					28			
					30			
Very hard drilling.					32			
					34			
A little easier drilling.						GP		35.5-37.5: Partially cemented sands and gravel.
					36			
No sample - hole crooked, driller afraid to lose tool.								
					38			
					40			
					42			
					44			
					46			
					48			
					50			
					52			

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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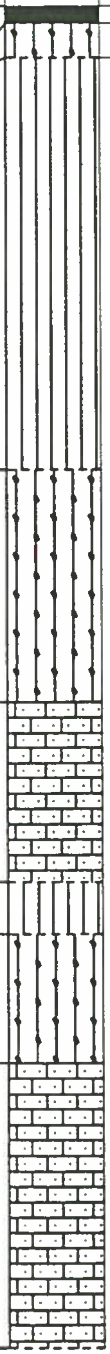
Plate 2-F

LOG OF BORING

LOG NO.: B-7
GROUND ELEVATION.:
LOGGED BY: N. Saines
GROUND WATER DEPTH: 24.6 feet
DATE: 03-02-93
DATE MEASURED: 03-02-93
TYPE OF BORING: Rotary



NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	DESCRIPTION
						AC	0-0.33: Asphaltic Concrete
						GM	0.33-1.0: Aggregate Base
						ML	1.0-9.0: Dry, loose, light brown Sandy Silt with gypsum veins. Changes to slightly moist, gray-green Silt, more plastic.
					2		
Sample 7-B is a gypsiterous sample from shoe.	7-A/7-B	■		7			
					4		
	7-C	■		12			
					6		
					8		
						GM	9.0-13.5: Slightly moist, loose, light brown Silty Gravel.
					10		
	7-D	■		20			
					12		
							13.5-17.0: Hard Caliche
					14		
					16		
						ML	17.0-18.0: Gray Silt.
					18	GM	18.0-20.5: Dry, dense, light brown, white sulfates Clayey Gravel partly cemented.
	7-E	■		41-6"			
				20-2"			
					20		
							20.5-27.5: Caliche
					22		
					24		
					26		

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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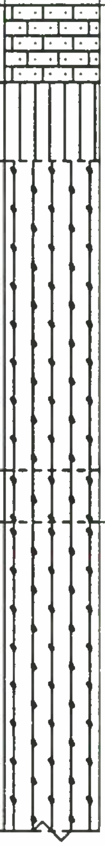
Plate 2-G

LOG OF BORING

LOG NO.: B-7
GROUND ELEVATION.:
LOGGED BY: N. Saines
GROUND WATER DEPTH: 24.6 feet
DATE: 03-02-93
DATE MEASURED: 03-02-93
TYPE OF BORING: Rotary

NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	Graphic Log	DESCRIPTION
								
Water coming into hole below caliche.					28	ML		27.5-29.0: Moist, medium stiff, light brown Silt.
	7-F/7-G	■		8		GM		29.0-35.0: Moist, loose, light brown, sulfates Silty Sandy Gravel. Cemented from 33.0 to 34.0 feet.
					30			
					32			
					34			
	7-H	■		50-2"		GM		35.0-36.0: Wet, hard, light brown, partially cemented Silty Gravel
					36	GM		36.0-42.0: Wet, loose, light brown Silty Gravel, partly cemented to cemented, large clasts to 2" diameter.
					38			
					40			
	7-I	■		50-1.5'				
					42			
					44			
					46			
					48			
					50			
					52			

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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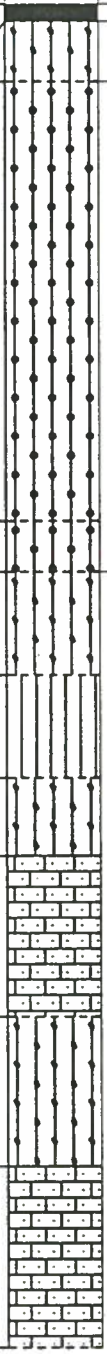
Plate 2-G

LOG OF BORING

LOG NO.: B-8
GROUND ELEVATION.:
LOGGED BY: N. Saines
GROUND WATER DEPTH: 25.0 feet
DATE: 03-02-93
DATE MEASURED: 03-04-93
TYPE OF BORING: Rotary



NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	DESCRIPTION
						AC	0-0.33: Asphaltic Concrete
						GM	0.33-1.5: Aggregate Base
						SM	1.5-10.0: Slightly moist, loose, yellow-brown <u>Silty Sand</u>. Gravelly at 6.8 feet, and silty at 8 feet.
					2		
	8-A			5			
					4		
No sample at 5' hit something very hard - stone blocking shoe? 5-10' afraid to drive because of utilities.		■		43			
					6		
					8		
					10		
	8-B	■		12		SM	10.0-11.0: Dry, medium dense, light brown <u>Silty Sand</u> with small pebbles. Gravelly from 11.0 to 13.0 feet.
						GM	11.0-13.0: <u>Silty Sandy Gravel</u>.
					12		
						ML	13.0-15.0: Gray <u>Silt</u>.
					14		
	8-C	■		8-6"		GM	15.0-16.5: Moist, hard, partly cemented <u>Silty Gravel</u>.
				20-3"	16		16.5-19.6: <u>Caliche</u>
					18		
						GM	19.6-22.5: Moist, medium stiff, light brown <u>Silty Gravel</u>.
					20		
	8-D	■		20			
					22		
							22.5-30.0: <u>Caliche</u>
					24		
					26		

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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Plate 2-H

LOG OF BORING

LOG NO.: *B-8*
GROUND ELEVATION.:
LOGGED BY: *N. Saines*
GROUND WATER DEPTH: *25.0 feet*
DATE: *03-02-93*
DATE MEASURED: *03-04-93*
TYPE OF BORING: *Rotary*

NOTES	Sample Number	Location	PID (ppm)	Number of Blows	Depth in Feet	Soil Class	Graphic Log	DESCRIPTION
					28			
	8-E	■		13	30	GM		30.0-34.0: Moist, loose, light brown <u>Silty Gravel</u>, pebbles to 1" diameter. Cemented around 33.0 feet.
					32			
					34	ML		34.0-36.0: Moist, soft, yellow-brown <u>Silt</u>, strongly cemented in places.
	8-F/8-G	■		16				
					36			
					38			
					40			
					42			
					44			
					46			
					48			
					50			
					52			

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

PP: Pocket Penetrometer Measurement (tons per square foot)

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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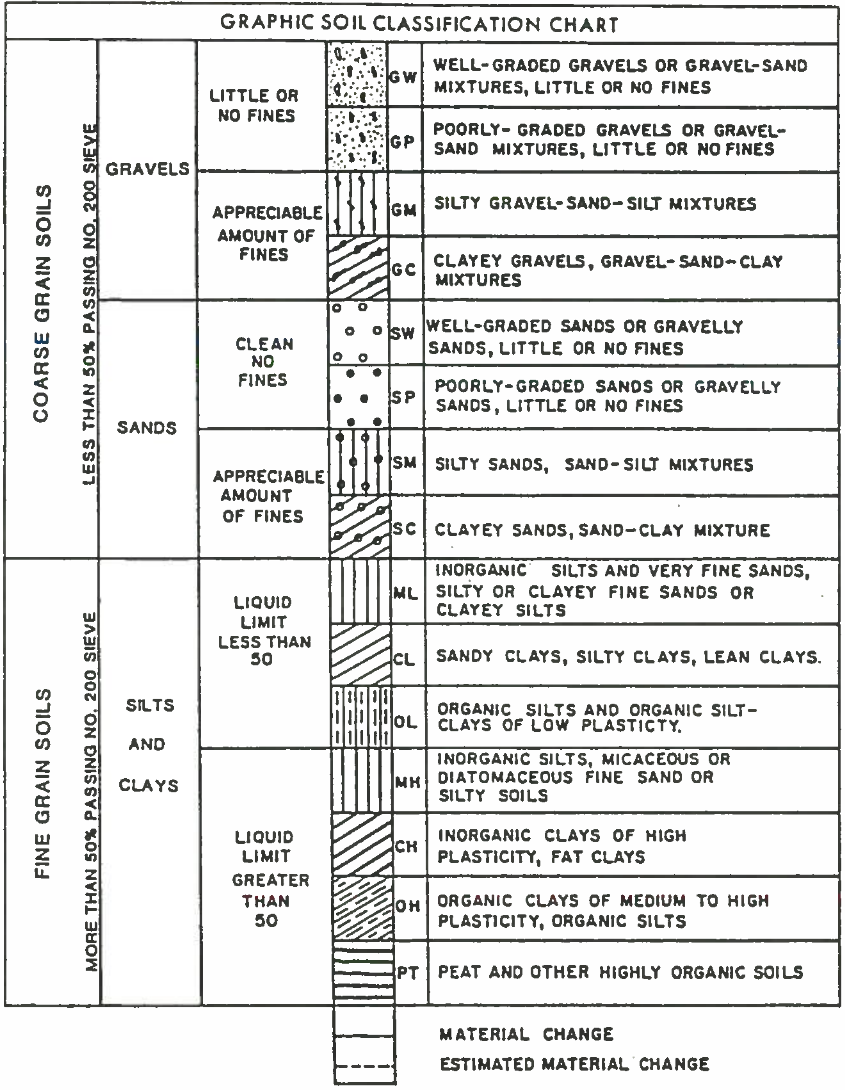
Plate 2-H

GRAPHIC SOIL CLASSIFICATION CHART



COARSE GRAIN SOILS	LESS THAN 50% PASSING NO. 200 SIEVE	GRAVELS	LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY- GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			APPRECIABLE AMOUNT OF FINES	GM	SILTY GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
		SANDS	CLEAN NO FINES	SW	WELL-GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
			APPRECIABLE AMOUNT OF FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURE
FINE GRAIN SOILS	MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS
				CL	SANDY CLAYS, SILTY CLAYS, LEAN CLAYS.
				OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICTY.
			LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
				PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

MATERIAL CHANGE

ESTIMATED MATERIAL CHANGE

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Size Range
Boulders	Over 12 in. (300 mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
Gravel	3 in. to #4 sieve (75mm to 2mm)
Sand	#4 to #200 sieve (2mm to .074 mm)
Silt or Clay	Passing #200 sieve (0.074mm)

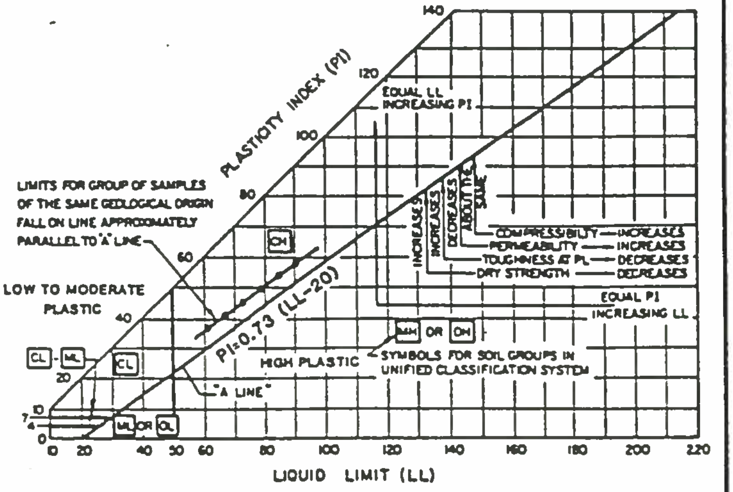
RELATIVE DENSITY OF GRANULAR SOILS:

N-Blows/ft.	Relative Density
0-4	Very Loose
5-10	Loose
11-30	Compact
31-50	Dense
greater than 50	Very Dense

CONSISTENCY OF COHESIVE SOILS:

Unconfined Compressive Strength, /Qu, psf	N-Blows/ft.	Consistency
less than 500	0-1	Very Soft
500-1,000	2-4	Soft
1,000-2,000	5-8	Firm
2,000-4,000	9-15	Stiff
4,000-8,000	16-30	Very Stiff
8,000-16,000	31-60	Hard
greater than 16,000	greater than 60	Very Hard

PLASTICITY CHART



DATAPRINT N30980

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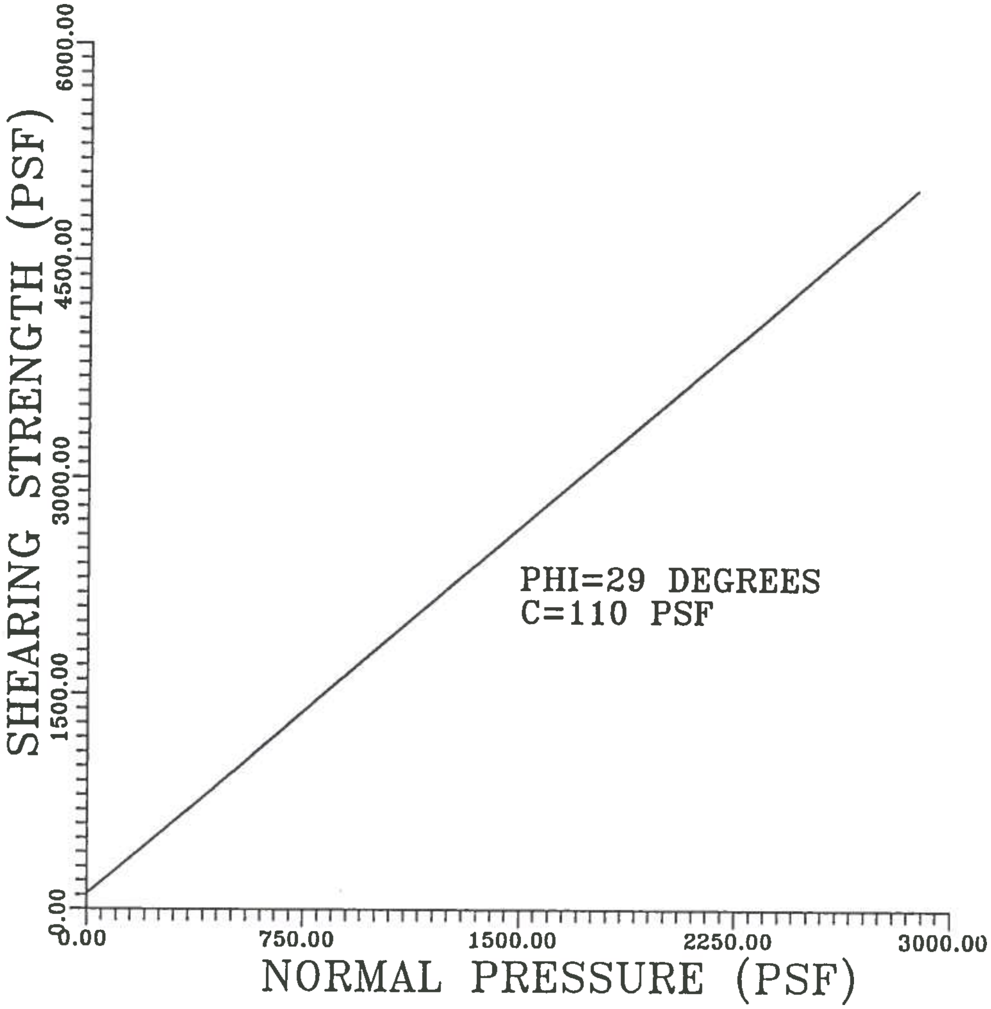
Plate 3



SAMPLE: 1C
SOIL TYPE: SANDY CLAY - CLAYEY SAND
DRY UNIT WEIGHT: 107.1 PCF
IN PLACE MOISTURE: 13.6 PERCENT
TEST MOISTURE: SATURATED

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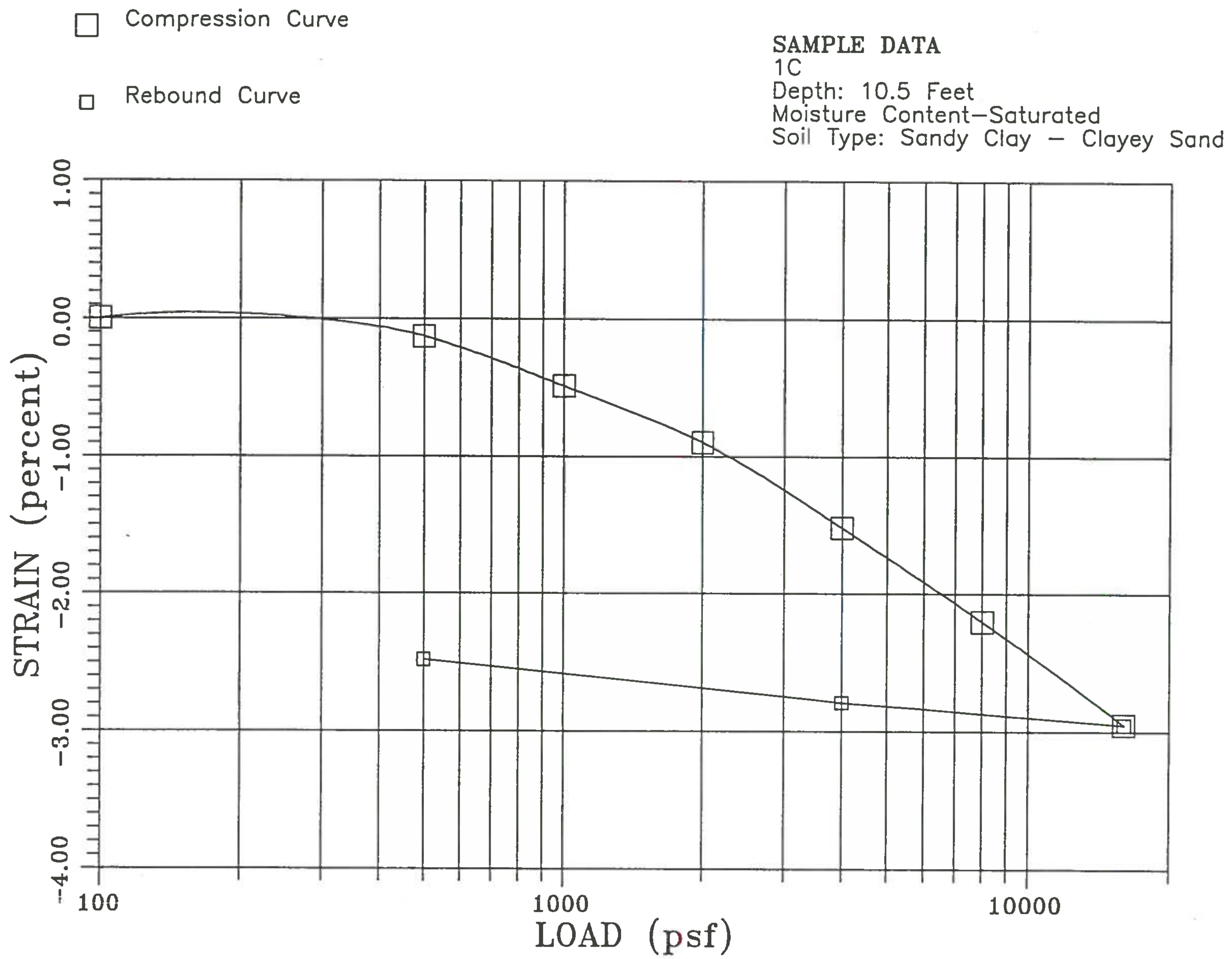
DIRECT SHEAR TEST RESULT
PEDESTRIAN CROSSING
LAS VEGAS BLVD. & TROPICANA DR.
CLARK COUNTY, NEVADA

Project No. 2-765-01-4

PLATE 5



SAMPLE DATA
1C
Depth: 10.5 Feet
Moisture Content–Saturated
Soil Type: Sandy Clay – Clayey Sand

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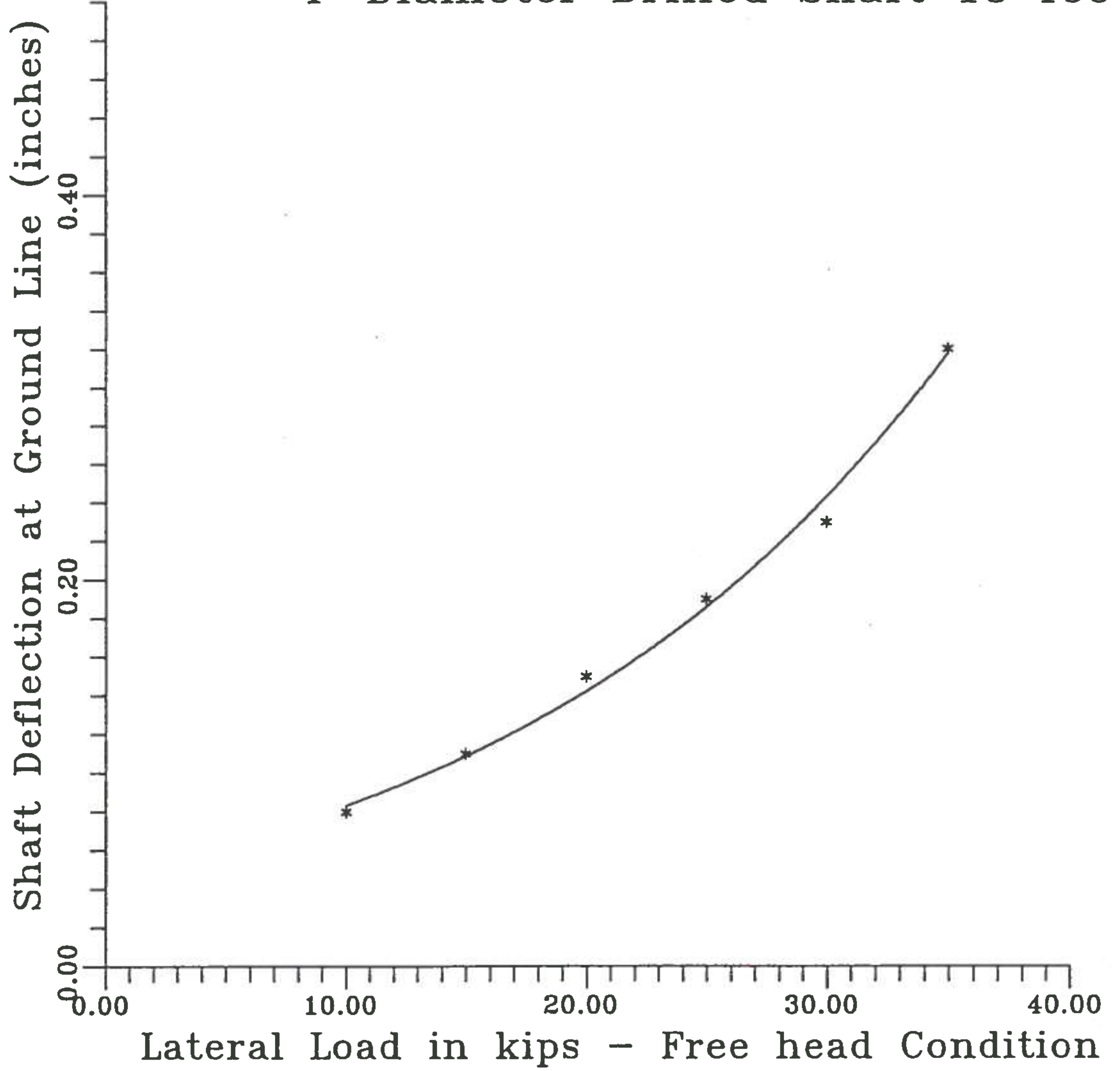
CONSOLIDATION TEST RESULT
PEDESTRIAN CROSSING LAS VEGAS BLVD. & TROPICANA DR.
CLARK COUNTY, NEVADA

PROJECT NO. 2-765-01-4

PLATE 6

156041

C.R.S.S.
PREDESTRIAN OVERCROSSING
4' Diameter Drilled Shaft 15 feet long



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RENO/SPARKS, NEVADA
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DEFLECTION OF LATERALLY LOADED PIERS
PEDESTRIAN CROSSING LAS VEGAS BLVD. & TROPICANA DR.
CLARK COUNTY, NEVADA

PROJECT NO. 2-765-01-4

PLATE 7

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JOB NO. 2-765-01-4

SHEET 1 **OF** 8

PROJECT CRSS **DATE** 3-12-93

SUBJECT Laterally Load Short Pier **DESIGNED** DH **CHECKED**

1.) Design Soils Profile

a) use B-4 as worst case for lateral loads

Depth	N	Nc	C	$\bar{q}$	Ø	N_q	Hansen N_γ
0-8	5	7	.120	.48	29	17	13.0
8-15	23	26	.120	.90	35	34	48
15-16	120	120	.120	1.86	40°	64	79

2) estimates Ks as per Bowles methodology!

$K_{s_{0-8'}} = 24(.5\gamma B N_\gamma + \gamma Z N_q)$

$= (24)(.5)(.12)(4)(13) + (.12)(17)(Z')\ 24$

$= 75 + 49Z'$ use $\boxed{38 + 25Z'}$ 0-8

SF = 2

$K_{s_{8-15'}} = (24)((.5)(.120)(4)(48) + (.12)(34)Z')$

$K_{s_{8-15}} = 276 + 98Z'$ use

SP = 2 $\boxed{140 + 49Z'}$ 8-15

$K_{s_{15-16'}} = (24)(.5)(.120)(4)79 + (24)(.12)(64)Z'$

$K_s = 455 + 184$ use $\boxed{225 + 90Z'}$

0-8' $K_s = 38 + 25Z'$

8-14' $K_s = 140 + 49Z'$

15-16' $K_s = 225 + 90Z'$

FORM 230

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JOB NO. 2-765-01-4
SHEET 2 OF 8

PROJECT CRSS
DATE 3-12-93
SUBJECT Laterally Load Short Pier
DESIGNED DH CHECKED

3) Calculate Es, assume 3000 psi Concrete

a) $E_c = 57{,}000\sqrt{3000 \text{ psi}} \div 1000 \times 144$

$= 450{,}000$ KSF

b) $E_{steel} = 4{,}200{,}000$ KSF

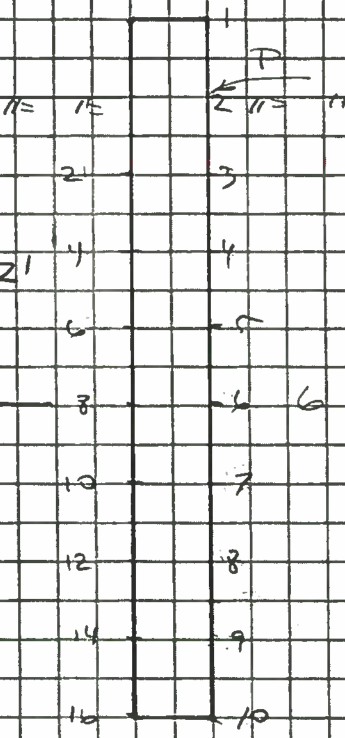
Assume 1% $(.99)(450{,}000) + (.01)(4{,}200{,}000)$

$E_{composite} = \boxed{488{,}500 \text{ KSF}}$

3) $I = \frac{4^4\pi}{64} = \boxed{12.6 \text{ ft}^4}$

5) Set up FEM using Bowls program FADBMLP1

NP = 20
NM = 9



Ks = 38 + 25 Z'

Ks = 140 + 49 Z'

Ks = 225 + 90 Z'

6) Summary of Results (from attached output)

a) Dia = 4' min

b) Length = 16'

c)

P	Deflection	Max. Moment
10	.08"	50.8 K.ft
15	.11"	76.2 K.ft
20	.15"	101.7 K.ft

FORM 230

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JOB NO. 2-765-14
SHEET 6 OF 8

PROJECT CRSS- Ped. Overcrossing DATE 3-11-93
SUBJECT Foundation Design DESIGNED DH CHECKED

IV As per my conversation with Ken Hawkins CRSS (3-11-93) they want to use shallow drilled shafts where possible and deeper shafts where necessary

1) Refine bearing capacity for worst case and check for punching

2) Check thickness of high strength (caliche) strata.

Boring	Depth	N	Nc	φ *	C
B-1	15-19.5	40+	40+	40°	0
	19.5-	14	14	32°	0
B-2	15-18.5	70	70	40°	0
	18.5+	56	56	40°	0
B-3	12-31+	∞	∞	40°	0
B-4	15-20	46+	46+	40°	0
	20+	12	12	31	0
B-5	15-19	?	?	(est) 40°	0
	19+	17	17	32°	0
B-6	14-17.5	?	?	40°	0
	17.5+	50+	50	40°	0

* Fig 19.5, PH&T

FORM 230

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JOB NO. 2-765-14

SHEET 7 OF 8

PROJECT CRSS DATE 3-11-93

SUBJECT Foundation Design DESIGNED DH CHECKED

Boring	Depth	N	Nc	ϕ	C
B-7	14-28	[illegible]	40+	40°	0
B-8	16-20	[illegible]		40°	0
	20+	20	20	32°	0

3) Based on the above conditions they all look good

a) B-5 is about the worst case

4) Assume we drill 1 ft into caliche leaving only 3 feet.

5) The most conservative approach is to ignore caliche and calc B.C. with $\phi = 32°$

a) Use eq 19.7 of Bowles

$$Q_a = \frac{A_p}{SF}(L)(\gamma)(N_q) + (4)(\gamma)BN_\gamma$$

$$A_p = \left(\frac{5}{2}\right)^2\pi = 19.6\ ft^2$$
5'

$SF = 3$ $B = 5$

$L = 16.0$ $N_q = 29$

$\gamma = .120$ $N_\gamma = 28$

$$Q_a = \frac{19.6}{3}(16)(.120)(29) + (4)(.12)(5)(28)$$

$$Q_a = 364 + 6.7 = 370\ K$$

(Max column load = ~ 260 kips as of 3-11-93)

JOB NO. 2-765-14

SHEET 8 OF 8

PROJECT CRSS DATE 3-11-93

SUBJECT Foundation Design DESIGNED DH CHECKED

6) From this we see that we are in good shape for Ø = 32°, C = 0 at 5' dia

Try 4' dia. and use 19' depth to bottom of Caliche

3) Ap = 12.6 ft², B = 4'

4) Qult = (12.6)(19)(.120)(29) + (.4)(.12)(4)(28)

Qult = 830 + 5.3

[illegible] = $\frac{835}{260}$ = 3.2 = very good

7) So this look good - use 4' dia,

Say min 3 inches into caliche

$\frac{260}{\left(\frac{4}{2}\right)^2 \pi}$ = 20 KSF - very conservative
use for other diameters

8) Since the soils underlying the caliche would support the loads we can be sure that the caliche won't punch.

In addition settlement should be neglible

9) We should probably pour these stiff

to decrease skin friction so that full end bearing is developed with minimal "friction hang" type settlement.