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August 18, 1988
K936-3, 11766

American Land Cruise
5959 Blue Lagoon Drive
Miami, Florida 33126

Attention: Mr. Jack Beaver

RE: GEOTECHNICAL INVESTIGATION
PROPOSED CRUISE AMERICA FACILITY
769 - 66TH AVENUE
OAKLAND, CALIFORNIA

Gentlemen:

In accordance with your request, we have performed a geotechnical investigation for the proposed Cruise America Facility. The accompanying report presents the results of our field investigation, laboratory tests, and engineering analysis. The soil and foundation conditions are discussed and recommendations for the soil and foundation engineering aspects of the project are presented. The conclusions and recommendations contained herein are based upon applicable standards of our profession at the time this report has been prepared. Copies of this report are furnished only to provide the factual data which were gathered and which were summarized in the report.

We refer you to the text of the report for detailed recommendations. If you have any questions concerning our findings, please call us.

Very truly yours,

KALDVEER ASSOCIATES, INC.

REGISTERED PROFESSIONAL ENGINEER
RONALD L. BAJUNIEMI
No. 112
EXPIRES 12-31-89
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GEOTECHNICAL INVESTIGATION

For
PROPOSED CRUISE AMERICA FACILITY
OAKLAND, CALIFORNIA

To
American Land Cruise
5959 Blue Lagoon Drive
Miami, Florida 33126

August 1988

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GEOTECHNICAL INVESTIGATION
FOR
PROPOSED CRUISE AMERICA FACILITY
OAKLAND, CALIFORNIA

INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Cruise America Facility. The proposed facility will be located at 769-66th Avenue in Oakland, California, as shown on the Site Plan, Figure 1. The purpose of our investigation was to evaluate the foundation soils and provide recommendations concerning the geotechnical engineering aspects of the project.

Based on the information indicated on the Site Plan as well as on our conversations with Mr. Perry Haviland of Haviland Associates and Mr. Nasr Farag of SEDCO, the project structural engineer, it is our understanding that the development will consist of a two-story combination office building, vehicle repair and body shop structure, a car wash and storage building and associated parking. The main structure will be roughly L-shaped with a total plan area of about 17, 500 square feet. The vehicle repair and body shop portion of this structure will include below grade service pits. Building loads will be typical for this type of structure. The car wash and storage building will be lightly loaded with plan dimensions of 65 by 90 feet. Excavation will be required for the below grade portion of the repair and body shop.

SCOPE

The scope of work of this investigation included a site reconnaissance, subsurface exploration, laboratory testing, engineering analyses of the field and laboratory data and the preparation of this report. The data obtained and the analyses performed were for the purpose of providing design and construction criteria for site earthwork, building foundations, slab-on-grade floors, retaining walls and pavements.

This report has been prepared in accordance with generally accepted geotechnical engineering practices for the exclusive use of American Land Cruise and their consultants for specific application to the proposed facility at 769-66th Avenue, Oakland. In the event that there are any changes in the nature, design or location of the buildings or if any future additions are planned, the conclusions and recommendations contained in this report shall not be considered valid unless 1) the project changes are reviewed by Kaldveer Associates and 2) conclusions and recommendations presented in this report are modified or verified in writing.

SITE INVESTIGATION

Subsurface exploration was performed using a truck-mounted, 6-inch diameter continuous flight auger. Three exploratory borings were drilled on May 25, 1988, to a maximum depth of about 26½ feet, eight borings were drilled on June 14, 1988 to maximum depths of about 31½ feet, and four borings were drilled during the period of July 26 through July 27, 1988 to maximum depths of about 76½ feet. The approximate locations of the borings are shown on the Site Plan, Figure 1. Logs of the borings and details regarding the field investigation are included in Appendix A. The results of our laboratory tests are discussed in Appendix B.

A. Surface

The site is roughly rectangular in shape, essentially level and has maximum plan dimensions of approximately 350 by 800 feet. At the time of our field investigation, the site was being used as a heavy equipment storage and maintenance yard. Three structures occupied the eastern half of the site, a chain-link fence encircled the property and the majority of the site was paved with asphaltic concrete. The site was void of vegetation, except adjacent to the railroad tracks on the eastern boundary where a moderate growth of grasses and weeds and a few trees were observed. The site is bounded on the north by 66th Avenue, and on the south by Damon Slough. Southern Pacific Railway Company tracks form the eastern boundary of the site.

B. Subsurface

The majority of the site is paved with approximately 2 inches of asphaltic concrete underlain by about 4 inches of baserock material. The soils encountered in our exploratory borings below the pavement section are variable but can generally be described as loose clayey gravel fill with sand interbeds underlain by soft compressible bay deposits. The fill extends to depths of about 5 to 12 feet, is relatively weak and potentially compressible. The bay deposits (clay), vary in thickness from about 0 to 12 feet, and are soft to firm, relatively weak and potentially compressible. Underlying the bay deposits are interbedded sand, clay and gravel which extended to the maximum depth explored of about 76½ feet. These materials were typically medium dense to dense and very stiff to hard. A strong petroleum hydrocarbon odor was observed during the drilling of Borings 2, 3, 6, 6a, 11, 12 and 14 at depths of about 2 to 10 feet. Borings 3, 6 and 6a were terminated at shallow depth. Detailed descriptions of the soils encountered in each of the exploratory borings are presented on the boring logs in Appendix A.

The attached boring logs and related information depict location specific subsurface conditions, encountered during our field investigation. The approximate locations of the borings were determined by pacing and should be considered accurate only to the degree implied by the method used. The passage of time could result in changes in the subsurface conditions due to environmental changes.

C. Groundwater

Free groundwater was encountered in Borings 1, 2, 4, 5, 7, 8, 11, 12, and 13 at depths of about 4 to 18½ feet at the time of drilling. Borings 1 and 2 were left open for periods of 3 to 5 hours at which time groundwater was measured at depths of about 5 to 6 feet. All other borings were backfilled immediately after drilling. The borings may not have been left open for a sufficient period of time to establish equilibrium groundwater conditions. In addition, fluctuations in the groundwater level could occur due to change in seasons, variations in rainfall, and other factors.

D. Geology and Seismicity

The project vicinity is underlain by unconsolidated sediments and marsh deposits consisting of gravel, sand, silt and clay. Historical data indicates the site is in an area which was once marshland and has been recently filled.

The project site is located in one of the most seismically active regions in the United States. Significant earthquakes have occurred in the San Francisco Bay Area and are believed to be associated with crustal movements along a system of subparallel fault zones that generally trend in a northwesterly direction. The site is located approximately 18 miles northwest, 2½ and 10½ miles southeast, respectively, of the active San Andreas, Hayward and Calaveras fault zones.

Earthquake intensities will vary throughout the Bay Area, depending upon the magnitude of earthquake, the distance of the site from the causative fault, and the type of materials underlying the site. The site will be subjected to strong ground shaking. However, during such an earthquake, the hazard associated with surface ground rupture is considered to be low.

CONCLUSIONS AND RECOMMENDATIONS

It is our opinion that the site is suitable for the proposed development from a geotechnical engineering standpoint. The conclusions and recommendations presented in this report should be incorporated in the design and construction of the project to avoid possible soil and/or foundation related problems. The presence of relatively weak and potentially compressible loose gravel fill and soft Bay deposits, and the high groundwater table are the primary consideration for foundation design.

The anticipated building loads and any new fills placed at the site could cause settlement due to the consolidation and compression of the bay deposits and the loose gravel. Based on the anticipated structural loads, we estimate total settlement of spread footings supporting the 2-story building will be about 2½ inches and differential settlement between adjacent footings will be about 2 inches. We estimate total settlement of spread footings supporting the car wash/storage structure to be about 1 inch, and differential settlement between adjacent footings to be about 3/4 inches. Estimated settlement due to placement of any new fill at the site will be about ½ to 1 inch per foot of new fill.

Because of the anticipated settlements, we recommend that the two-story building be supported on a driven pile and grade beam foundation. Settlement of a pile supported structure should be negligible. Piles should extend through the loose gravel fill and soft bay deposits and into the competent underlying materials. If the estimated magnitude of the total and differential settlements for the planned car wash/storage building are tolerable, it can be supported on spread footing foundations. The settlement should be anticipated and addressed by the project structural engineer in foundation design and selection.

The first level floor slabs of the planned buildings can be either structurally supported by the pile foundation or supported directly on the gravel fill. If the slab(s) is supported directly on the fill, it should be structurally separated from the pile foundation to allow independent movement of the slab and building as settlement occurs. In addition, if the slabs are supported on the existing fill, they should be reinforced with reinforcing bars in lieu of wire mesh to minimize differential movement of the slab.

Groundwater levels were measured during our exploration program at an elevation at or above the planned bottom of the vehicle repair and service pits. Therefore, we recommend the service pits be designed to resist hydrostatic forces. The design groundwater level should be taken as a depth of 2 feet below existing grade. Dewatering of the excavation and subsurface drainage systems will be required during construction.

We recommend that our firm review the final design plans and specifications to check that the earthwork and foundation recommendations presented in this report have been properly interpreted and implemented in the design and specifications. We can assume no responsibility for misinterpretation of our recommendations, if we do not review the plans.

A. Earthwork

1. Clearing and Site Preparation

The site should be cleared of all obstructions including building foundations and slabs, underground tanks, designated underground utilities, fencing and debris. Removed asphaltic concrete and baserock material can be used as fill provided the asphaltic concrete is broken up to meet the size requirements presented in Item A.5, "Material for Fill". Excavations resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with suitable material compacted to the requirements given below under Item A.6, "Compaction". We recommend backfilling operations for any excavations to remove deleterious material be carried out under the observation of the geotechnical engineer.

After clearing, the portions of the site containing surface vegetation or organic laden topsoil should be stripped to an appropriate depth to remove these materials. At the time of our field investigation, we estimated that a stripping depth of approximately 2 inches would be required adjacent to the railroad tacks on the eastern boundary; deeper stripping could be required to remove tree roots. The amount of actual stripping should be determined in the field by the soil engineer at the time of construction. Stripped materials should be removed from the site or stockpiled for later use in landscaping, if desired.

2. Excavation

Excavation for the construction of the planned service pits should be performed under dry conditions and will require temporary sloping of the excavation sides. Temporary excavation slopes should be no steeper than 2:1 (horizontal to vertical) and sloughing should be anticipated. Maintenance and stability of the excavation slopes will be the contractor's responsibility.

3. Dewatering

The service pit excavations and utility trenches should be appropriately dewatered as necessary during construction. Groundwater will be encountered in the excavation and temporary dewatering measures will be required to lower the water level to an elevation at least 2 feet below the bottom of the deepest excavation. However, the contractor should be responsible for the dewatering design and implementation of any dewatering system. Dewatering should continue until all structural and/or trench backfill is placed.

4. Subgrade Preparation

After the completion of cleaning and stripping soil exposed in areas to receive structural fill, slabs-on-grade or pavements should be scarified to a depth of 6 inches, moisture conditioned to slightly above optimum water content and compacted to the requirements for structural fill, except for the service pits.

After completion of the excavation of the service pits, a subgrade stabilization fabric such as Mirafi 700X or equivalent should be placed in the bottom of the excavation. A 6-inch blanket of clean crushed rock should then be placed over the stabilization fabric. The crushed rock should be compacted using hand operated vibratory compaction equipment.

5. Fill Material

On-site soil below the stripped layer and having an organic content of less than 3 percent by volume can be used. All fill placed at the site including on-site soils should not contain rocks or lumps larger than 6 inches in greatest dimension with not more than 15 percent larger than 2.5 inches. In addition, any imported fill should be predominantly granular with a plasticity index of 12 or less.

6. Compaction

Structural fill less than 5 feet thick should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation

D1557-78. The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Structural fill or wall backfill greater than 5 feet deep should be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding 8 inches in uncompacted thickness.

7. Pipe Bedding

The method of bedding selected for the pipelines associated with the facility depends on the subgrade conditions at the bottom of the pipeline trench. However, as a minimum, we recommend that all pipes be founded on a 6-inch layer of approved well-graded bedding material such as 3/4-inch Class 2 Aggregate Base. Where the subgrade is unstable, an increase in the thickness of the bedding material could be required. The need for increasing the thickness of the bedding material should be determined in the field by the geotechnical engineer at the time of construction.

The trench bottom should be free of lumps or hollows and graded so as to provide uniform support along the length of the pipe. Where couplings or bells are located, holes should be excavated to provide a minimum of 2 inches below each connection.

8. Trench Backfill

Pipeline trenches should be backfilled with fill placed in lifts of approximately 8 inches in uncompacted thickness. However, thicker lifts can be used provided the method of compaction is approved by the geotechnical engineer and the required minimum degree of compaction is achieved.

If on-site soil is used as trench backfill it should be compacted to at least 85 percent relative compaction by mechanical means only (no jetting will be allowed). Imported sand can be used for trench backfill if it is compacted to at least 90 percent relative compaction and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction.

The upper 3 feet of trench backfill in slab and pavement areas should be compacted to at least 90 percent relative compaction for on-site soils, and 95 percent where imported sand backfill is used. In addition, the upper 6 inches of all trench backfill in pavement areas should be compacted to at least 95 percent relative compaction.

9. Surface Drainage

Positive surface gradients should be provided adjacent to the buildings so as to direct surface water away from foundations and slabs toward suitable discharge facilities. If new fills are placed in the pavement areas, total and differential settlement must be considered in design of the surface gradients as well as gravity flow pipelines. Ponding of surface water should not be allowed adjacent to the structure or on pavements.

10. Construction During Wet Weather Conditions

If construction proceeds during or shortly after wet weather conditions, the moisture content of the on-site soils could be appreciably above optimum. Consequently, subgrade preparation, placement and/or reworking of on-site soil as structural fill might not be possible. Alternative wet weather construction recommendations can be provided by the geotechnical engineer in the field at the time of construction, if appropriate.

11. Guide Specifications

All earthwork should be performed in accordance with the Guide Specifications - Site Earthwork presented in Appendix C. These specifications are general in nature and the final specifications should incorporate all recommendations presented in this report.

B. Foundation Support

1. Friction Piles
a. Vertical and Uplift Capacity

We recommend that the main building be supported on a driven pile and grade beam foundation system. The car wash and storage building should be supported on driven piles if the anticipated settlement for spread footings previously discussed exceeds the tolerable limits for the structure. Piles will gain support from skin friction between the pile shaft and the surrounding soil. Capacities for 12-inch square piles are illustrated on Figure 2. The capacity can be increased by one-third for all loads including wind or seismic loads. Pile lengths should be estimated based on Figure 2.

An allowable uplift capacity for individual piles of 80 percent of the vertical downward capacity shown on Figure 2 should be used for design. Piles designed should have a minimum center-to-center spacing of 2.5 pile widths.

Production pile lengths should be established and pile capacities verified by an indicator pile program as discussed under Item B.1.b.

b. Indicator Piles

To establish production pile lengths and blow count criteria, we recommend that a minimum of 10 indicator piles be driven at the start of construction within the perimeter of the proposed main building and a minimum of 5 indicator piles within the perimeter of the car wash and storage building if necessary. We recommend indicator pile lengths be 5 feet longer than the anticipated final length to allow for variations in the subsurface conditions. At least one pile should be driven at the location of each of the test borings drilled for the building. Additional indicator piles should be driven at locations approved by our engineer in the field. The indicator piles should be driven with the same equipment and in the same manner that will be used during production driving. In addition, to avoid excessive cutoffs in these areas, production pile lengths will have to be modified based on the indicator program.

c. Pile Driving Criteria and Installation

All piles should be driven to their design tip elevations in order to develop adequate skin friction capacities. Any piles that terminate shorter than their anticipated lengths will be evaluated on an individual basis. We should note, however, that in no case should driving be terminated without the approval of the geotechnical engineer.

The pile driving hammer should be a minimum rated energy of 40,000 foot pounds for the 12-inch square precast-prestressed concrete piles. It is possible for a very large hammer to cause damage to the particular pile it is driving; therefore, we recommend that our office and the structural engineer approve the hammer type and capacity selected by the contractor. In addition, the method of handling and picking up of the piles should be established by the pile driving contractor and should be approved by both the structural engineer and the geotechnical engineer prior to construction.

d. Specification and Construction Observation

As an aid in developing the pile specifications for this project, guide specification for pile foundations are included in Appendix D. We recommend that our office review the final foundation plans and specifications to assure that the recommendations presented in this report have been properly incorporated into the contract documents. To assure that piles are extended to adequate depths and to assure that they have encountered sufficient resistance to develop the required supporting capacities, we recommend that our firm observe the driving of the piles installed at the site.

2. Spread Footing Foundations

If the estimated settlements are tolerable for the car wash and storage building, the structure could be supported on conventional continuous and isolated spread footings bearing on the gravel fills. Footings should be at least 12 inches wide and should be founded at least 18 inches below lowest adjacent finished grade. Footings located adjacent to other footings or utility trenches should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trench.

At the above depths, the footings should be designed for an allowable bearing pressure of 1000 pounds per square foot due to dead loads, 1500 pounds per square foot due to dead plus live loads and 2000 pounds per square foot for all loads including wind or seismic. These allowable bearing pressures are net values; therefore, the weight of the footing can be neglected for design purposes. We recommend that we observe the footing excavations prior to placing reinforcing steel or concrete, to check that footings are founded on appropriate material.

3. Building Floor Slabs

The first level floor slab of the 2-story building can be either supported directly on the existing fill materials or by the pile and grade beam foundation system. We recommend that the floor slab of the car wash/storage building be a slab-on-grade.

a. Slabs-On-Grade

Slabs-on-grade can be supported directly on the existing fill, however, the upper two feet of fill should be removed and compacted in accordance with Item A.4, "Compaction". Slabs-on-grade used with a pile foundation should be structurally separated from the foundation. All slab-on-grade subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support. Slabs-on-grade should be reinforced with reinforcing bars in lieu of wire mesh to minimize differential movement of the slab.

If migration of moisture through the slab is undesirable, a moisture barrier should be provided between the slab and subgrade. We recommend 4 inches of free draining gravel covered with an impermeable membrane be placed between the subgrade soil and the slab. The membrane should be covered with 2 inches of sand for protection during construction. The sand should be lightly moistened just prior to placing the concrete. Alternatively, a capillary break consisting of 6 inches of free draining gravel could be used.

b. Structurally Supported Slabs

If the first level floor slab of the 2-story building is structurally supported, it should be totally supported by the pile and grade beam foundation. Reworking of the existing fill will not be required. If migration of moisture through the structurally supported slab is undesirable, a moisture barrier should be provided as discussed above.

4. Service Pit

The service pits of the repair and body shop must be designed to resist both lateral earth pressures, hydrostatic forces if not drained and any additional lateral loads caused by surcharging.

We recommend that unrestrained drained walls be designed to resist an equivalent fluid pressure of 35 pounds per cubic foot. This assumes a level backfill. Restrained drained walls should be designed to resist an equivalent fluid pressure of 35 pounds per cubic foot plus an additional uniform lateral pressure of 6H pounds per square foot where H = height of backfill above the top of the wall footing in feet.

The recommended lateral pressures assume that the walls are fully-backdrained walls to prevent the build-up of hydrostatic pressures. Adequate drainage may be provided by means of a system of subdrains. For the subdrain system, the top of the perforated pipe should be below the bottom of the adjacent service pit. The pit walls should be water proofed to prevent migration of moisture through the walls.

Undrained restrained below grade walls below the design groundwater depth of 2 feet should be designed to resist an equivalent fluid pressure of 85 pounds per cubic foot. In addition, the walls should be designed for a uniform pressure of 8H where H = the height of the below grade portion of the wall.

Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third or one-half the anticipated surcharge load for unrestrained or restrained walls, respectively.

Retaining wall backfill less than 5 feet deep should be compacted to at least 90 percent relative compaction using light compaction equipment. Backfill greater than 5 feet deep should be entirely compacted to at least 95 percent relative compaction. If heavy compaction equipment is used, the walls should be appropriately designed to withstand loads exerted by the heavy equipment and/or temporarily braced.

To avoid hydrostatic uplift forces, the bottom of the pit should be underlain by a gravel layer and a perforated pipe installed with perforations down placed below the gravel layer. This subdrain should be designed to flow to an appropriate discharge facility. The bottom of the pits should be water proofed.

Service pits should be supported 1) on driven pile foundations, 2) hung from the pile supported floor slab of the building or 3) a structural slab. If the pits are pile supported, the piles should be designed in accordance with the recommendations presented previously under Item B.1, "Friction Piles". If service pit walls are supported on a structural slab, the slab should be designed with a bearing capacity of 500 pounds per square foot. Lateral load resistance for the walls can be developed in accordance with the recommendations presented below under Item B.5, "Lateral Loads".

5. Resistance to Lateral Loads

a. Pile Foundations

Lateral load resistance for the proposed two-story building can be developed by a passive resistance equal to 1) an equivalent fluid weighing 200 pounds per cubic foot acting against the vertical face of grade beams and 2) an equivalent fluid weighing 400 pounds per cubic foot acting against the projected area of the pile shafts. The upper 2 feet of soil should be neglected when calculating total lateral resistance of the piers.

b. Spread Footings

Lateral load resistance for the proposed car wash and storage building can be developed by friction between the foundation bottom and the supporting subgrade. A friction coefficient of 0.40 is considered applicable. As an alternative, a passive resistance equal to an equivalent fluid weighing 350 pounds per cubic foot acting against the vertical face of the foundations could be used. If foundations are poured neat against the soil, the friction and passive resistance can be used in combination.

6. Special Considerations

Due to the settlements and differential settlements that could occur between the various facilities constructed at the site, we recommend that service connections for utility lines into the buildings be designed with enough flexibility to withstand as much of the anticipated total and differential settlement as is possible. In addition, maintenance of the utility pipelines should be anticipated. We also recommend that gravity flow pipelines be designed for the anticipated settlements. To avoid additional loads imposed on any buried utility lines which are "hung" from the structural slab and/or grade beams, we recommend that consideration be given to the following: (1) hang the pipes as close as possible to the bottom of the slab and use "knockouts" through the grade beams, (2) use light weight aggregate backfill in the trenches, or (3) do not backfill the utility trench. In addition, landscaping berms should not be placed over underground utility lines.

C. Pavements

One "R" (resistance) value test was performed on a representative bulk sample of the surface materials at the site. The results of this test are presented in Appendix B and indicate an "R" value of 41. We developed the following alternative preliminary pavement sections using Procedure 301-F of the State of California Department of Public Works, Division of Highways, R-value test results and assumed traffic indices. Pavement designs for pavement lives of 1 to 5 years, 6 to 10 years and 11 to 20 years are also included.

RECOMMENDED PAVEMENT DESIGN ALTERNATIVES

Location	Anticipated Pavement Life (years)	Pavement Components		Total Thickness (inches)
		Asphaltic Concrete (inches)	Aggregate Base Class 2 (inches)	
Automobile Parking and Access Areas (T.I. = 4.0 for 20-year life)	11-20	2.0	4.0	6.0*
Heavy Truck Access (T.I. = 6.5 for 20-year life)	11-20	3.5	7.0	10.5
	6-10	3.5	6.0	9.5
	1-5	3.5	6.0	9.5*

* Minimum recommended pavement section

The traffic indices used in our pavement designs are considered reasonable values for the proposed development and should provide the indicated pavement lives with only a normal amount of pavement maintenance. Selection of the design traffic parameters, however, was based on engineering judgement and not on an equivalent wheel load analysis developed from a traffic study or furnished to us.

Asphaltic concrete, aggregate base and preparation of the pavement subgrade should conform to and be placed in accordance with the Guide Specifications - Asphalt Paving presented in Appendix E.

In areas where the pavements will abut planted areas, the pavement baserock layer should be protected against saturation from irrigation. Planned concrete curbs should extend to the bottom of the baserock layer, forming a cut-off wall between the planter and the pavement section.

D. Construction Observation

The analysis and recommendations submitted in this report are based in part upon the data obtained from the ten soil borings. The nature and extent of variations between the borings may not become evident until construction. If variations then become apparent, it will be necessary to re-evaluate the recommendations of this report.

We recommend that our firm be retained to provide geotechnical engineering services during site grading and foundation installation. This is to observe compliance with the design concepts, specifications and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered.

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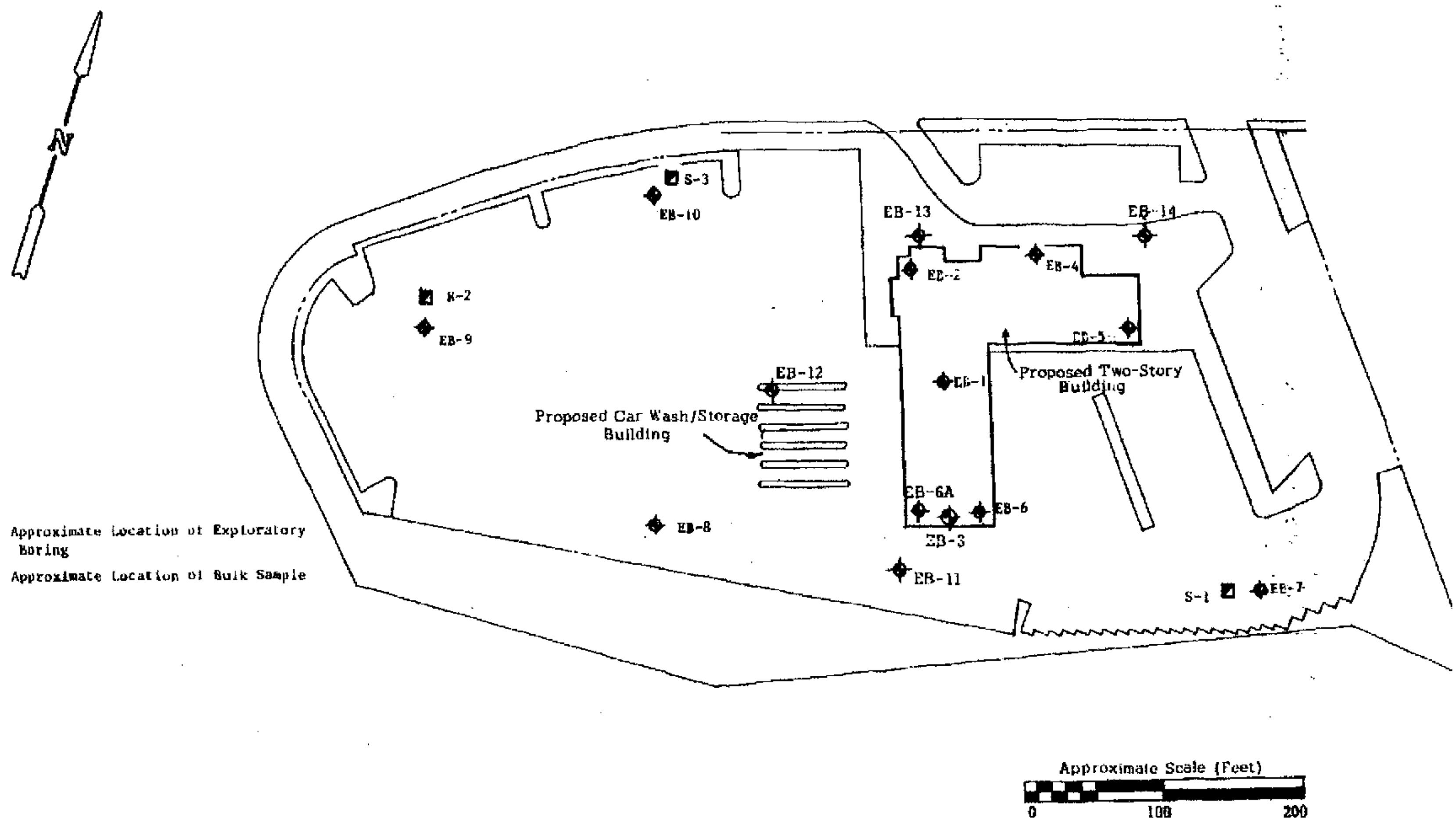

EB-1 Approximate Location of Exploratory Boring

S-1 Approximate Location of Bulk Sample

Base: "Site Plan" by Haviland Associates, Architects, dated January 4, 1988.

Kaldveer Associates
Geoscience Consultants
A California Corporation

SITE PLAN		
PROPOSED CRUISE AMERICA FACILITY Oakland, California		
PROJECT NO.	DATE	Figure 1
K936-3	August 1988	