REPORT
GEOTECHNICAL INVESTIGATION
CENTURY AERO CLUB PROJECT
VAN NUYS AIRPORT
VAN NUYS, CALIFORNIA
FOR CASTLE & COOKE AVIATION

URS JOB NO. 29405192
JUNE 20, 2008

URS

June 20, 2008

Castle & Cooke Aviation
7415 Havenhurst Place
Van Nuys, CA 91406

Attention: Mr. Ross Young

Subject: Report of Geotechnical Investigation
Century Aero Club Project
Van Nuys Airport
Van Nuys, California
URS Project No. 29405192

Dear Mr. Young:

URS Corporation is pleased to present our report entitled "Geotechnical Investigation" for the Century Aero Club Project in the Van Nuys Airport, Van Nuys, California. This report summarizes the results of our investigation and contains geotechnical recommendations for design and construction of the project.

Based on the results of our geotechnical investigation, we believe the proposed Project is feasible from a geotechnical point of concern, provided the recommendations in this report are incorporated in the design and implemented during earthwork and construction of the Project.

If you have any questions regarding this report, please contact us. We look forward to being of further assistance as construction begins.

Very truly yours,

URS

Garry Lay, P.E., G.E.
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1.0 INTRODUCTION

This report presents results of a geotechnical investigation performed by URS Corporation (URS) for the Century Aero Club Project in the Van Nuys Aiport, Van Nuys, California. The location of the project site relative to existing topographic features is shown in the Vicinity Map, Figure 1. This investigation was performed in accordance with the 2007 California Building Code and the 2008 City of Los Angeles Building Code. This report includes our conclusions and geotechnical recommendations for design and construction of the Project. Environmental consideration is outside our scope of our investigation. Conclusions and recommendations presented in this report are based on our current knowledge of the proposed construction; subsurface conditions encountered at exploration locations; results of our laboratory testing. These conclusions and recommendations should not be extrapolated to other areas without our prior review.

2.0 PROJECT DESCRIPTION

The proposed Century Aero Club Project is located in the south side within the Van Nuys Airport and is to the east of the intersection of Saticoy Street and Havenhurst Place/Havenhurst Avenue. The Project site is currently occupied by various existing buildings and the ground surface is paved with asphalt. The proposed Project consists of the design and construction of a two acre asphalt ramp for aircraft parking and an optional 1,000 square feet customer service building. Building loads were unknown at the time of this report’s preparation, therefore column loads of less than 100 kip were assumed. In additional to the above proposed improvement, a percolation test was performed to investigate the percolation rate for the storm drain drainage.

3.0 PURPOSE AND SCOPE OF SERVICES

The purpose of the current geotechnical investigation was to explore and evaluate the subsurface conditions at the Project site. The goal of our investigation was to identify the key geotechnical and geologic issues that could potentially impact the proposed project and to develop geotechnical recommendations for design and construction of the Project. Our scope of services included performing the following tasks:

- Review of available geologic and geotechnical data pertinent to the project site;
- Field marking of boring locations and Underground Services Alert (USA) of Southern California notification to identify subsurface utilities and obtain clearance for drilling at the site;
- Exploration of subsurface conditions by drilling and sampling two geotechnical borings to depths of approximately 51½ and 56½ feet below the ground surface (bgs) with a limited access drill dig;
- Performed an in-situ percolation test to evaluate the percolation rate of the near-surface soils.
- Performed geotechnical laboratory tests on selected soil samples obtained from the borings to evaluate index, consolidation characteristic, expansion potential and California Bearing Ratio of the soils;
- Engineering analyses and geologic/seismic hazard evaluation to develop geotechnical recommendations for design and construction of the proposed Project; and
- Preparation of this report containing our findings and recommendations including:
  a. Brief description of the proposed Project;
  b. Description of the field exploration and laboratory testing programs;
  c. Discussion of the site geologic conditions;
  d. Results of geologic and seismic hazards evaluation;
  e. Discussion of the site surface and subsurface geotechnical conditions;
  f. Preliminary earthwork recommendations, preparation for support of foundations;
  g. Recommendations for temporary excavations;
  h. Preliminary recommendations for type and depth of foundations for structural support;
  i. Preliminary geotechnical parameters for design of foundations, including the allowable bearing capacity and estimated settlements under assumed loading for structural design purposes;
  j. Seismic shaking design parameters;
  k. Pavement recommendations; and
  l. Construction monitoring recommendations.

4.0 FIELD INVESTIGATION PROGRAM

The geotechnical investigation included reviewing available geotechnical and geologic information, completing subsurface exploration, and performing field percolation testing.

4.1 REVIEW OF EXISTING DATA

Available geologic geotechnical documents listed in Section 12.0, References, were researched and reviewed as part of this study. This include the geotechnical investigation reports for the Air Traffic Control Tower performed by URS (as Dames & Moore), 1999 and for City Helicopter Maintenance Facility (URS 2002) for pertinent information. Pervious available geotechnical borings and laboratory data within the project site are presented in Appendix C.

4.2 FIELD EXPLORATION PROGRAM

4.2.1 General

A field exploration program consists of drilling and sampling two geotechnical borings (B-1 and B-2), approximately 51½ and 56½ feet bgs. The borings were drilled using a limited access drill rig, equipped with 8-inch diameter hollow-stem augers, operated by our subcontractor, Pacific Drilling of San Diego, California. The drilling was conducted under the technical supervision of a geologist from our Los Angeles office. Boring B-1 is at the proposed location of the ramp and B-2 is at the proposed location of the building. The locations of the borings are shown on Figure 2.

4.2.2 Boring Log and Samples

Relatively undisturbed ring-lined soil samples from a Modified California sampler and unlined soil samples from a Standard Penetration Test (SPT) sampler (per ASTM D 1586) were obtained by driving the samplers 18 inches into the subsurface soils using a 140-pound hammer falling 30 inches. All blow counts were recorded at 6-inch intervals. The number of blows required to drive the samplers the final 12 inches was recorded on the logs of borings. Bulk samples from the near-surface soils were collected from all borings.

Upon completion, the boring was backfilled with the cuttings to within approximately 6 inches of the existing ground surface. The surface was then patched with cold-patch asphalt to match existing ground condition.

Our representative visually classified the soils encountered in accordance with the Unified Soil Classification System (USCS), and maintained detailed log of the boring. A Key to the Log of Boring describing the Unified Soil Classification System is presented in Figure A-1 and the log of boring is presented in Figures A-2 and A-3 of Appendix A. It should be noted that the interface between subsurface materials on the log of boring represent approximate boundaries. The actual transition between subsurface materials can be gradual.

4.3 PERCOLATION TEST

4.3.1 Percolation Test Preparation

One percolation test was performed for this Project. The percolation test hole was drilled to a depth of 4 feet below the existing ground surface using an 8-inch-diameter hollow-stem auger. Preparation of the percolation test hole began by placing approximately two inches of pea-sized gravel in the bottom of each hole. A four-inch-diameter perforated PVC pipe was then placed in each hole and surrounded by angular gravel. The hole was pre-soaked overnight prior to testing. The approximate location of the percolation test hole is shown on Figure 2.

4.3.2 Percolation Testing

Upon completion of the overnight presoak, the hole was refilled with water to approximately 12-inches above the bottom of the hole (top of pea gravel) and the water level was measured every 5 minutes for a four-hour period. If the water level within the percolation test hole lowered to 9 inches above the bottom of the test hole, additional water was added to refill the test to approximately 12-inches above the bottom of the hole. The stabilized measurement (steady rate) was used to determine the percolation rate.

4.3.3 Percolation Test Results

The percolation rate for test was 80 minutes per inch. This value can be used for preliminary design for the storm drain drainage. The engineer should use the appropriate conversion factors and safety factors deemed necessary for the drainage design.

5.0 LABORATORY TESTING

Geotechnical soil samples obtained from the borings were carefully sealed and packaged in the field to reduce moisture loss and disturbance. The samples were delivered to our laboratory located in Los Angeles, where they were further examined and classified. Laboratory testing was performed on selected samples to confirm (and to modify if necessary) the visual classification of the soils based on the field identification, and to evaluate their physical and engineering properties. Tests performed included:

- soil classification (ASTM D 2488);
- moisture content and dry density (ASTM D 2937);
- grain size analysis (ASTM D 422);
- wash sieve analysis (ASTM D 1140);
- expansion index (ASTM D 4829);
- consolidation (ASTM D 2435); and
- California Bearing Ratio test (ASTM D1883-05 and D4429).

A description of the laboratory testing and the test results are presented in Appendix B of this report. For convenience, test results of moisture and dry density determination, and fines content of soils tested are also shown on the Logs of Borings in Appendix A. Four direct shear tests were performed during the geotechnical investigation for the Air Traffic Control Tower and are attached in Appendix C. Corrosivity tests results are presented in a separate report prepared by Schiff Associates and is attached in Appendix D.

6.0 GEOLOGIC CONDITIONS

6.1 SITE GEOLOGY

The proposed facility lies in the San Fernando Valley, which is an east-trending structural trough within the Transverse Ranges geologic province of southern California. The mountains that bound it to the north and south are actively deforming anticlinal ranges bounded on their south sides by thrust faults. As these ranges have risen and been deformed, the San Fernando Valley has subsided and filled with sediment. Portion of the valley, where the project is located, has received sediment from the Pacoima and Tujunga washes. These washes are associated with large river systems that have their sources in the steep, rugged San Gabriel Mountains, which are comprised of crystalline bedrock. The rivers have deposited a broad alluvial fan composed of sand, silt, and gravel that blankets most of the surrounding site vicinity.

The Project site lies on an older inactive portion of the Pacoima/Tujunga alluvial fan that appears to have been cut off from its upstream source area by uplift of the Northridge Hills. This fan surface may have been abandoned when continuing uplift of the Northridge Hills deflected the Pacoima Wash drainage to the east. Although this surface is older than any other part of the Pacoima/Tujunga fan, it may have formed in early to mid-Holocene time.

6.2 SUBSURFACE CONDITIONS

The Project site is mantled by about 9 feet of artificial fill consisting of silty sand to sandy silt. No documentation of the placement of these fill materials were available. This fill is relatively soft/loose and contains some debris.

Below the fill to final depths explored is alluvium. The upper 15 feet of the alluvium is a layer of fine-grained materials of soft to stiff silt or clay with varying amount of sand. Below the fine-grained materials is approximately 10 feet of granular materials of medium dense silty sand. Underlying the granular materials to final depths (56½ feet) explored is stiff to very stiff lean clay or silt with varying amount of silty to clayey sand. Due to the anticipated thickness of the alluvium in the vicinity, no bedrock was encountered in our borings.

6.3 GROUNDWATER

Groundwater was not encountered during our subsurface investigation to a maximum depth of 56½ feet bgs. Based on regional data, the historical highest groundwater level in the project vicinity is approximately 60 to 70 feet below the existing ground surface (CDMG, 1997). The depth to groundwater may fluctuate, depending on factors such as rainfall in the site vicinity.

6.4 SEISMICITY AND FAULTING

The table below lists the active faults in the vicinity of the Project site. An active fault is defined as a fault that has a historic seismic record (activity in the last 100 years) or displaces Holocene (11,000 years and younger) deposits. The recurrence rates and maximum magnitudes assigned to these sources were based on historic seismicity and geologic data published by the U.S. Geological Survey, California Division of Mines & Geology (currently known as California Geological Survey), Southern California Earthquake Center, and several local geologists. Because there are no active or potentially active faults known to be present crossing the project site, the potential for surface fault rupture is considered unlikely. However, with the presence of active faults in the region, the site could be subjected to future strong ground shaking that may result from earthquakes on local to distant sources.

Active faults located within a ten-mile radius of the Project site include the Verdugo, Sierra Madre, Northridge, Santa Susana, and Santa Monica-Hollywood-Raymond fault system. Each of these faults are capable of generating strong ground motion at the site during an earthquake.

Fault	Maximum Earthquake Magnitude(M_w)[1]	Estimated Closest Distance from Site mi^2 (km^2)[2]
Verdugo	6.7	5.5 (8.8)
Sierra Madre	6.7	6.3 (10.2)
Northridge	6.9	6.6 (10.7)
Santa Susana	6.6	7.0 (11.2)
Santa Monica-Hollywood-Raymond	6.6	9.4 (15.1)

Notes:
1. Based on data from EQFAULT (Version 3.00)
2. Distance shown represents distance to seismic sources at depth and does not always represent distance to projections of fault planes at ground surface

6.5 GEOLOGIC AND SEISMIC HAZARDS

Geologic and seismic hazards are those hazards that could impact a site due to local and surrounding area geologic and seismic conditions. Geological hazards include subsidence, landslides, poor soil conditions (expansive or collapsible soil), and potential methane gas. Seismic hazards include phenomena that occur during an earthquake such as ground shaking, surface fault rupture, liquefaction, differential seismically-induced settlement, lateral spread displacement, ground lurching, tsunami, seismic induced flooding, and seiche. The potential impact of these hazards to the site has been assessed and is summarized in the following sections.

6.5.1 Geologic Hazards

6.5.1.1 Subsidence

The extraction of water or petroleum from sedimentary rocks or deposits can cause the permanent collapse of the pore space previously occupied by the removed fluid. The compaction of subsurface sediment caused by fluid withdrawal will cause subsidence of the ground surface. If the volume of water or petroleum removed is sufficiently great, the amount of resulting subsidence may be sufficient to damage nearby engineered structures. The project site is situated well outside any oil field and the area is not known to be in an area with significant ground water pumping. Therefore, the potential for subsidence is not considered a significant geologic hazard to the project.

6.5.1.2 Landslides

The Project site is situated within a relatively flat lying alluvial plain and the potential for landslides induced by seismic shaking is not anticipated to pose a significant seismic hazard to the proposed Project. The Seismic Hazards Zone map for the Van Nuys Quadrangle indicates that the Project site do not lie within areas designated as having the potential for earthquake-induced landsliding (California Division of Mines and Geology, 1998). Therefore, the potential for landslides is not considered a significant geologic hazard to the project.

6.5.1.3 Poor Soil Conditions (Expansive or Collapsible Soil)

Expansive soils are fine-grained soils (generally high plasticity clays) that can undergo a significant increase in volume with an increase in water content and a significant decrease in volume with a decrease in water content. Changes in the water content of a highly expansive soil can result in severe distress to structures constructed upon the soil. The near-surface soils at the project site consist primarily of silty sand or sandy silt. Based on our laboratory expansion index test result of 8, the excavated material is anticipated to be non-expansive.

Collapsible soils are those that undergo settlement upon wetting, even without the application of additional load. The process of collapse with the addition of water is known as hydrocompaction. Hydrocompaction occurs when water weakens or destroys the bonds between soil particles and severely reduces the bearing capacity of the soil. Typical collapsible soils are lightly colored, are low in plasticity and have relatively low densities. Collapsible soils are typically associated with alluvial fans, windblown materials, or colluvium. Laboratory tests performed on the site soils indicate a low potential for collapse. Based on the results of the laboratory tests performed, collapsible soils are not considered to pose a significant hazard to the proposed project.

6.5.1.4 Methane

The proposed Project does not lies within an area delineated by the City of Los Angeles (2004) as a “Methane Zone” and “Methane Buffer Zone”, which are zones with the potential for seepage of methane gas to occur in buildings. Therefore, the potential for the existence of methane within the Project site is low.

6.5.2 Seismic Hazards

Ground Shaking

By far the most severe seismic hazard with the potential to affect the Project is strong seismic shaking that can be expected from future earthquakes in the site region. The degree of shaking that is felt at a given site depends on the magnitude and distance of the earthquake, and on the type of subsurface material on which the site is situated. Although the site could be subject to significant ground shaking in the event of a major earthquake, this hazard is common to southern California, and possible damage caused by shaking can be reduced by proper structural design and construction.

Surface Fault Rupture

As stated in Section 6.4, no known active or potentially active faults have been recognized as crossing the Project site, and the California Geological Survey does not delineate any part of the proposed project area as being within an Alquist-Priolo Earthquake Fault Zone (California Division of Mines and Geology, 1986 & 1997). Therefore the potential for surface fault rupture at the site is considered to be low.

Liquefaction

Liquefaction is defined as significant and relatively sudden reduction in stiffness and shear strength of saturated sandy soils caused by a seismically induced increase in pore water pressures. Potential for seismically induced liquefaction exists whenever relatively loose, sandy soils exist with high groundwater level and/or potential for long duration, high seismic shaking.

The California Geological Survey has designated certain areas within California as potential liquefaction hazard zones. These are areas considered at greater risk of liquefaction-related ground failure during a seismic event, based upon mapped surficial deposits and the presence of a relatively shallow groundwater table. Based on the Seismic Hazard Map for the Van Nuys Quadrangle (CDMG, 1998), the Project site lies outside the Liquefaction Hazard Zone. Moreover, the historic high groundwater table is deeper than 50 feet bgs. According to Youd et. al 2002, no evidence shows that soil will liquefy below 50 feet bgs, therefore, liquefaction potential at the Project site is considered to be low. Settlement caused in unsaturated soils (dry sand settlement) is estimated to be less than 1 inch provided that earthwork is performed to mitigate loose/soft foundation soils, according to Section 7.2.

Lateral Spread Displacement

According to publications by Bartlett and Youd (1999), conditions such as free-face, sloping ground surfaces and liquefiable layers are factors contributing to lateral spread displacement of the ground during strong motion events. The site has very low susceptibility of liquefaction; therefore, risk of lateral spread displacement is remote.

Differential Seismically-induced Settlement

Differential seismic settlement occurs when seismic shaking causes one type of soil to settle more than another type. It may also occur within a soil deposit with relatively homogeneous properties if the seismic shaking is uneven, which could occur due to variable geometry, for example, and variable depth of the soil deposit. Based on our investigation, the subsurface soils are found to be uniform throughout the Project site, therefore, the potential of differential seismically-induced settlement is considered low.

Ground Lurching

Ground lurching is permanent displacement or shift of the ground in response to seismic shaking. Ground lurching occurs in areas with high topographic relief, and usually occurs near the source of an earthquake, where shaking and permanent ground displacements are highest. These displacements can result in permanent cracks in the ground surface, which are sometimes confused with surface fault ruptures. Cracks from lurching do not extend to great depths, usually only several feet to tens of feet below the ground surface, depending on specific site conditions. With the flat topography at the Project site, we judge that ground lurching does not represent a potential hazard to the proposed structure.

Tsunamis

Tsunamis are great sea waves (commonly called a tidal wave) produced by a significant undersea disturbance. Due to the site's distance from the coast, the seismic hazard potential for tsunamis is considered negligible at the Project.

Seismic-Induced Flooding

Earthquake induced flooding occurs when nearby water retaining structures, such as dams or storage tanks, are breached or damaged during an earthquake. The site is located approximately 6 miles south of the Los Angeles Reservoir and is within its flood or inundation hazard zone according to the Los Angeles County Safety Element (1990). Based on this information, The project team should take this into consideration and if necessary, provide appropriate mitigation measures to reduce the risk of earthquake induced flooding within the vicinity of the site.

Seiche

A seiche is an oscillation of a body of water in an enclosed or semi-enclosed basin, such as a reservoir, harbor, lake, or storage tank, resulting from earthquakes or other large environmental disturbances. Given its distance to the nearest reservoir, there appears to be little risk of seiche impacting the site.

7.0 DISCUSSIONS AND RECOMMENDATIONS

7.1 GENERAL

Based on the findings of our investigation, the proposed project is feasible from a geotechnical engineering standpoint, provided that the recommendations of this report are followed; and the designs, grading and construction are properly and adequately executed.

With respect to seismic hazards, no faults are known to exist within the project site. The possibility of surface rupture of the site due to faulting is low. Although the site could be subject to significant ground shaking in the event of a major earthquake, this hazard is common to southern California. Liquefaction is not a project design issue because of the deep groundwater level. Possible damage caused by seismic shaking and unsaturated sand settlement is low at the project site.

The proposed building can be supported by shallow foundation with appropriate soil mitigation earthwork. Based on the findings of our field exploration, the project site is mantled by about 9 feet of artificial fill. The layer of undocumented artificial fill was generally observed to be soft and loose. It is not considered to be a suitable bearing material for foundations and must be removed entirely and replaced by primary structural fill, based on City of Los Angeles Building Code Section 1805, if the proposed building is to be supported by shallow foundations. Underlying the fill to the final depths explored (56½ feet) are alluvial deposits of clays, sands, and silts. In general, the alluvial soils become denser/stiffer with depths.

The ramp pavement area is also underlain by approximately 9 feet of soft/loose artificial fill. It is a general policy of the City of Los Angeles's Department of Building and Safety that new structures or new structural fill (primary or secondary) shall not be constructed on undocumented fill and therefore all artificial fill will have to be replaced by secondary structural fill. If any of the undocumented fill is to be left-in-place, the owner may have to apply a "waiver" to the City of Los Angeles.

Recommendation for earthwork, foundation design, seismic design, floor slab support, pavement design, and corrosion protection considerations are presented in the following sections of this report.

7.2 EARTHWORK

Any required earthwork should be performed in accordance with the applicable portion of the grading code of the 2008 City of Los Angeles Department of Building and Safety, as well as the recommendations of this report, and should be performed under the observation and testing of a geotechnical engineer.

It is the responsibility of the contractor to select the proper equipment necessary for site excavation and backfilling. Excavations should be observed by the Geotechnical Engineer-of-Record. The exposed subgrade should be inspected and approved by inspectors from the City of Los Angeles. No great variations in subsurface conditions are anticipated. However, if conditions encountered during construction appear to differ from those encountered in the exploratory borings, URS should be notified so as to consider the need for modifications.

7.2.1 Site Preparation

It is the responsibility of the contractor to notify and coordinate with Underground Services Alert (USA) before any proposed earthwork. All active or inactive utilities within the construction limits should be identified for relocation, abandonment, or protection prior to grading. Any pipes greater than 2 inches in diameter to be abandoned in-place should be filled with sand/cement slurry after review of their location and approval by the Geotechnical Engineer-of-Record.

Where shallow excavations are proposed, existing pavement, debris, organic materials, deleterious materials and artificial fill should be removed and disposed of outside the construction limits under observation of the Geotechnical Engineer-of-Record.

7.2.2 Overexcavation and Subgrade Preparation

Building

If the building is to be constructed, it is our recommendation that the undocumented artificial fill (upper 9 feet) should be removed entirely. It should be pointed out that deeper and/or poorer quality fill soils could exist between boring locations and final depths should be determined based on actual field observation. The overexcavation depth should be equal to the depth of the artificial fill, or 5 feet below the bottom of the foundations, whichever is greater. The overexcavation is made to receive a minimum of five feet of primary structural fill below the foundations and slabs-on-grade. The area of removal should extend at least 5 feet beyond the edge of foundations, or equal to the depth of removal, whichever is greater.

Following the excavation, the exposed subgrade should be proofrolled to locate any loose or soft zones. Proofrolling will involve making several passes with heavy rubber-tired equipment over the area under consideration, and observing the reaction of the subgrade under the wheel loads. Upon completion of proofrolling, a field representative of the Geotechnical Engineer-of-Record should perform probing and/or field density testing to evaluate the extent of loose or soft zones, if any. All observed isolated loose or soft zones less than 12 inches in depth should be compacted in-place. Upon completion of proofrolling, the excavation subgrade should be scarified a minimum of 8 inches deep and compacted in-place, achieving a minimum subgrade relative compaction of 90 percent of the maximum dry density per ASTM D-1557.

If large area of loose/soft bottom is encountered, we recommend a layer of geogrid should be placed to stabilize the bottom before placing the primary structural fill. Such additional subsurface improvement requirements should be determined in the field by the Geotechnical Engineer-of-Record during foundation subgrade preparation activities. Upon completion of the required overexcavation, backfill should be placed in accordance with recommendations presented later in this report.

No fill should be placed until approval is obtained from the City of Los Angeles field inspector assigned to the Project. Earthwork recommendations for the ramp are provided in Section 7.6.

7.2.3 Temporary Excavations

Excavations during construction should comply with the current California and Federal OSHA requirements. For design purposes, a Cal/OSHA soil type C can be assumed for the silty soils. It should be noted that this assessment of Cal/OSHA soil type for temporary excavations is based on engineering classifications of the subsurface materials encountered in widely spaced exploratory borings. The Contractor should have a geotechnical professional evaluate the soil conditions encountered during excavation to determine permissible temporary slope inclinations.

Based on Cal/OSHA requirements, unsupported slopes for temporary excavation in the existing soils may be cut up to a maximum height of 20 feet, with a slope no steeper than 1½:1 (Horizontal:Vertical). In areas where soils with little or no cohesion are encountered, shoring or flatter excavation slopes may be necessary. Shoring should be used for excavations with vertical cut, or where unsafe conditions are anticipated for cut slopes.

Surcharge loads from vehicles, airplanes and stockpiled material should be kept away from the top of temporary excavations at a distance equal to at least one half of the excavation depth. During wet weather, runoff water should be prevented from entering the excavation, and collected and disposed of outside the construction limits. To prevent runoff from adjacent areas from entering the excavation, a perimeter berm should be constructed at the top of the slope.

7.2.4 Slot Cutting

Existing hangers are located adjacent to the proposed excavation at the north side of the property and the required over-excavation may encroach the existing hangers. Slot cuttings will be required for such condition. An *A-B-C* slot sequence with a maximum slot width of 8 feet should be utilized. The slots were typically 5 feet deep. The A slots were excavated first, leaving the B and C slots intact. The A slot excavations were then backfilled with engineered fill. The procedure was repeated with the B (then C) slots until all the required excavations have been completed and replaced with primary structural fill. All slot-cutting operations should be under observation of the Geotechnical Engineer-of-Record.

7.2.5 Fill and Backfills

7.2.5.1 Onsite Sources

Most of the existing fill materials to be excavated will be silty sand or sandy silt. Based on our laboratory expansion index test result of 8, the excavated material is anticipated to be non-expansive. These materials would be suitable for use in compacted primary and secondary structural fill, provided that any deleterious materials (debris) and rocks over 3 inches in greatest dimension are removed. We recommend that the geotechnical engineer be allowed to review the types of materials encountered in the excavations in order to confirm their re-usability. Laboratory tests such as expansion index and sieve analyses should be performed before its use. As necessary, some mixing or blending of soils may be required in order to achieve a suitable fill consistency.

7.2.5.2 Import Materials

Import fill should be predominantly granular in nature, with an Expansion Index of less than 20. New fill should contain no rocks in excess of 3 inches in maximum dimension, and no more than 35% of fines passing a standard No. 200 sieve. In addition, any trench bedding materials should conform to Sections 306-1.2.1 and 306-1.3 of the Green Book, or similar standards. All new fills shall be free of hazardous, organic and inorganic debris. No soil should be imported to the site without prior approval by the Geotechnical Engineer-of-Record. All fill and backfill materials should be observed and tested by the Geotechnical Engineer-of-Record in order to determine their suitability.

7.2.5.3 Compaction Criteria

Primary and secondary structural fills and utility trench backfills may be placed during construction of this project. All areas to receive fill should be placed in loose lifts not exceeding 8 inches in thickness, brought to within 3 percent wet of the optimum moisture content in-place, and compacted to at least 95 percent of the maximum dry density per ASTM D 1557 using mechanical compaction equipment. Densification by flooding or jetting should not be allowed. All structural fills (primary and secondary) should be tested by a representative from the office of the Geotechnical Engineer of Record, and the results of tests should be presented in a fill compaction report.

No fill should be placed, spread or rolled during unfavorable weather. When the work is interrupted by rain, operations should not be resumed until field tests by the Geotechnical Engineer-of-Record have indicated that conditions are appropriate for fill placement.

7.2.5.4 Skrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 10 and 15 percent should be anticipated when excavating and recompacting the existing fill onsite.

7.3 SHALLOW FOUNDATION DESIGN

7.3.1 Spread Footing Foundation

Spread footings should have a minimum dimension of 2 feet square and a minimum embedment depth of 2 feet below the lowest adjacent grade founded over a minimum of 5 feet of the primary structural fill. Footings resting on primary structural fill and with the above minimum dimensions may be designed using an allowable bearing pressure of 2,000 pounds per square foot (psf). The bearing pressure applies to dead plus live loads and includes a calculated factor of safety of at least 3. The allowable bearing pressure values may be increased by one-third for short-term loading due to wind or seismic forces. The edge pressure of any eccentrically loaded footing should not exceed the allowable bearing value.

7.3.2 Settlement

On the basis of the recommended allowable bearing values in Sections 7.3.1, and the earthwork recommendations in Section 7.2, static total settlement is estimated to be about 1 inch. Static differential settlement is estimated to be half of the total static settlement. Seismic-induced settlement is estimated to be about 1 inch. Majority of the static settlement is expected to occur shortly after construction.

7.3.3 Lateral Load Resistance

Resistance to lateral loads may be provided by frictional resistance between the bottom of concrete foundations and the underlying primary structural fill, and by passive soil pressure against the sides of the foundations. The allowable coefficient of friction between poured-in-place concrete foundations and the underlying primary structural fill may be taken as 0.25. An allowable lateral bearing pressure of 300 psf, per foot of depth can be used, provided that there is positive contact between the vertical bearing surfaces and the primary structural fill. These recommended values have included a safety factor of at least 1.5. Friction and lateral pressure resistance may be combined, provided that either value is limited to two-third of the allowable.

7.4 SLABS-ON-GRADE

All floor slabs should be at least 4 inches thick and should be reinforced at a minimum with ½-inch diameter (#4) deformed reinforcing bars spaced a maximum of 16 inches on center each way. The actual design of slab and reinforcement should be determined by the project structural engineer. As a general requirement, all slabs-on-grade should be supported on a subgrade prepared in accordance with Section 7.2.2 of this report. A layer of sand at least 2 inches thick should be placed under the slab to promote uniform curing of the concrete. For design of slabs and estimating their deflections, a modulus of subgrade reaction (k) of 100 pci may be used for re-compacted materials.

In addition, moisture barrier, consisting of a plastic or vinyl membrane placed between two layers of clean sand, each at least two inches thick, is recommended beneath all floor slabs to be overlain by moisture-sensitive floor covering.

7.5 SEISMIC SITE COEFFICIENTS

For determination of the site coefficients, a site class type E in accordance with Table No. 1613.5.2 of the 2007 CBC may be assumed to represent the upper 100 feet of subsurface conditions. The seismic design parameters for the project in accordance with the 2007 CBC are presented in the table bel0ow:

Table 3 – Seismic Design Parameters

Mapped Spectral Accelerations for Short Periods per Figure 1613.5(3), S_S	1.81 g
Mapped Spectral Accelerations for One Second Period per Figure 1613.5(4), S_1	0.66 g
Site Coefficient per Table 1613.5.3(1), F_a	0.9
Site Coefficient per Table 1613.5.3(2), F_v	2.4
Maximum Considered Earthquake Spectral Response Accelerations for Short Period, S_{MS}	1.63 g
Maximum Considered Earthquake Spectral Response Accelerations for One Second Period, S_{M1}	1.58 g
5% Damped Design Spectral Response Acceleration at Short Periods, S_{DS}	1.09 g
5% Damped Design Spectral Response Acceleration at One Second Periods, S_{D1}	1.05 g

7.6 RAMP PAVEMENT

A California Bearing Ratio (CBR) test was conducted on the near-surface on-site soils of Boring B-1 at the project site. The result indicated a CBR-value of 7 and is used for pavement section design of the ramp. The analysis follows the Federal Aviation Administration (FAA) Criteria for Pavement Design and was performed with the Flexible Pavement Design Spreadsheet provided by the FAA. The following table summarizes recommended minimum pavement sections based on 100,000 pounds aircrafts with 1,350 equivalent annual departures, which should be evaluated by the project designer/civil engineer.

<table>
<tr><td colspan="2" rowspan="3">Layer</td><td colspan="3">Design Aircraft</td></tr>
<tr><td colspan="3">DUAL100 – 100,000lbs</td></tr>
<tr><td colspan="3">Thickness (inches)</td></tr>
<tr><td colspan="2">Asphalt</td><td>4</td><td>8.5</td><td>12.5</td></tr>
<tr><td colspan="2">Base</td><td>6.5</td><td>0</td><td>0</td></tr>
<tr><td rowspan="2">Geo-composite Layer</td><td>Upper 1” Gravel</td><td>12</td><td>12</td><td>6</td></tr>
<tr><td>Bottom 1” Gravel</td><td>6</td><td>6</td><td>6</td></tr>
<tr><td colspan="2">Total Thickness</td><td>28.5</td><td>26.5</td><td>24.5</td></tr>
</table>

To provide uniform support, all pavement areas should be underlain by a “geo-composite layer” below the base layer. The “geo-composite layer” should consist of a 6-inch thick layer of gravel at the bottom, overlain by a layer of geogrid, and then another layer of gravel in accordance to the above table for different design aircraft. Geogrid used should be bi-directional and manufactured by either Tensar such as BX-1200, or by Mirafi such as BasXgrid 12. The gravel can be crushed miscellaneous base materials.

The aggregate base course materials should conform to the Caltrans Class II (37.5 mm) aggregate base standard with a minimum CBR-value of 80. All base materials should be compacted to a minimum of 95 percent of the maximum dry density per ASTM D-1557.

Alternatively, resurfacing can be performed to provide the equivalent pavement section without removing the existing pavement section. It is calculated based on 1.5 inch of base equivalent to 1 inch of asphalt. At the northern area where boring B-1 is located, the existing pavement section consists of 3 inches of asphalt concrete over 4 inches of aggregate base. At the southern area where boring B-2 is located, the existing pavement section consists of 3 inches of asphalt concrete over 2 inches of aggregate base. To provide the equivalent pavement section, an additional layer of concrete asphalt will be required to overlay on top of the existing pavement section. The recommended overlaying thicknesses for each area are as follows:

Area	Design Aircraft SINGLE60 – 100,000lbs Overlay Asphalt Thickness (inches)
Northern area (B-1)	3
Southern area (B-2)	6

8.0 CONSTRUCTION CONSIDERATIONS

8.1 RIPPABILITY

The surface materials to be excavated are generally soft to firm sandy silt. Although local pockets of stronger materials may be present, excavations are not anticipated to be difficult with a proper choice of equipment.

8.2 GROUNDWATER

Based on the findings of our field exploration and the available information on groundwater conditions, groundwater is not anticipated to be within pertinent depths of the proposed construction. Consequently, neither wet construction methods nor dewatering are deemed necessary for the Project.

9.0 DESIGN REVIEW

The geotechnical aspects of the project should be reviewed by the Geotechnical Engineer-of-Record during the design process. The scope of services may include assistance to the design team in providing specific recommendations for special cases, reviewing the foundation design and evaluating the overall applicability of the recommendations presented in this report, reviewing the geotechnical portions of the project for possible cost savings through alternative approaches and reviewing the proposed construction techniques to evaluate if they satisfy the intent of the recommendations presented in this report.

10.0 CONSTRUCTION MONITORING

All earthwork and foundation construction should be monitored by a qualified engineer/technician under the supervision of a licensed Geotechnical Engineer-of-Record including:

- Site preparation including site stripping, removal of subsurface structures, and bottom observation;
- Temporary excavations;
- All foundation excavations;
- Placement of all structural (primary and secondary) fills and backfills; and
- Observation of subgrade preparation for paved and building areas.

The Geotechnical Engineer-of-Record should be present to observe the soil conditions encountered during construction, to evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and to recommend appropriate changes in design or construction if conditions differ from those described herein.

11.0 LIMITATIONS

URS warrants that our services have been performed within the limits prescribed by our clients, with the usual thoroughness and competence of the geotechnical engineering profession in southern California at this time. No other warranty or representation, expressed or implied, is included or intended in this report.

–o0o –

We appreciate the opportunity to be of service to you on this project and trust this report meets your needs at this time. Should you have any questions, please contact us.

Respectfully submitted:

URS

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Manager, Geotechnical Department

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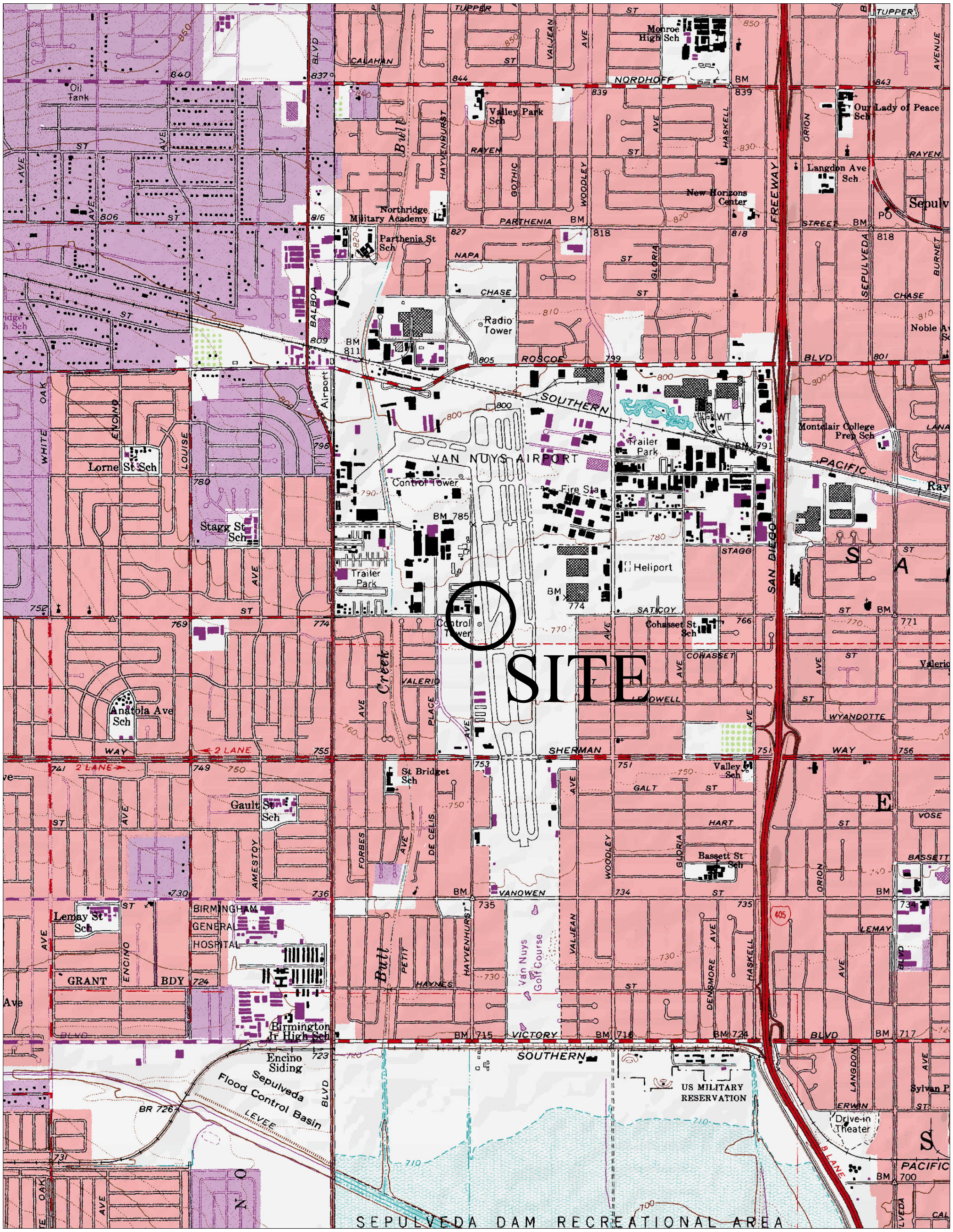
FIGURES



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VICINITY MAP

Century Aero Club Project
Van Nuys Airport
Van Nuys, California

FOR: Castle & Cooke Aviation

REFERENCE: USGS 7.5 Minute Series Topographic Map,
"Van Nuys, California". Quadrangle, 1972.

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FIGURE 1

G:\Da Wu\Van Nuys Airport\CAD\Plot Plan.dwg

EXPLANATION

- B-1 URS Boring Location (2008)
- DM-2 Dames & Moore Boring Location (1999)
- Location of Percolation Test

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PLOT PLAN

CENTURY AERO CLUB PROJECT
VAN NUYS, CALIFORNIA
FOR: CASTLE AND COOKE AVIATION

0 100 200
SCALE, feet

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Figure 2