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CITY OF ELK GROVE
PLANNING

GEOTECHNICAL INVESTIGATION

EAST ELK GROVE PARISH

BRADSHAW ROAD, APN #121-022-008

ELK GROVE, CALIFORNIA

REPORT PREPARED FOR:

JACKSON PROPERTIES

OUR JOB NUMBER: SGG-0038

July 19, 2007

NEIL O. ANDERSON
AND ASSOCIATES
50 Goldenland Court, Suite 100 • Sacramento, CA 95834

NEIL O. ANDERSON
AND ASSOCIATES

GEOTECHNICAL
ENVIRONMENTAL
INSPECTIONS & TESTING
LABORATORY SERVICES
POOL ENGINEERING
POST TENSION DESIGN

July 19, 2007
Our Project Number: SGG-0038

Jackson Properties
Attn: Todd Kraenzel
5665 Power Inn Road Suite 140
Sacramento, CA 95824

Subject: **GEOTECHNICAL INVESTIGATION**
EAST ELK GROVE PARISH
BRADSHAW ROAD, APN #121-022-008
ELK GROVE, CALIFORNIA

Dear Mr. Todd Kraenzel:

The following report presents the findings and conclusions of our geotechnical investigation conducted at the subject site. The purpose of the report was to provide recommendations for building foundations, floor slabs, site grading, utility construction, and pavement sections, as indicated in our revised proposal dated June 12, 2007 and accepted June 27, 2007. Recommendations for this project have been provided in the body of the report. Coordination between our office and your grading contractor will help reduce the potential for soil related problems.

Key information regarding this geotechnical report is presented on the following page. This information sheet has been provided to aid you in assessing the limitations of this geotechnical investigation as well as to indicate when additional information from our office may be required.

We appreciate the opportunity of working with you on this project and look forward to providing our services in the future. Please contact us if you have any questions.

Sincerely,
NEIL O. ANDERSON & ASSOCIATES, INC.

Ryan King, Staff Engineer
EIT No. 122451, B.Sc.

Robert E. Holmer, Principal Engineer
Registered Geotechnical Engineer 2672

REGISTERED PROFESSIONAL ENGINEER
ROBERT HOLMER
No. 2672
Exp. 12/08
GEOTECHNICAL
STATE OF CALIFORNIA

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KEY INFORMATON REGARDING YOUR GEOTECHNICAL REPORT

➢ *The Applicability of Geotechnical Reports is Limited*

Geotechnical reports are written to provide test results, observations, and professional opinions regarding a specific site for a specific project. Reports are tailored to the client and are influenced by each client's risk management strategies, economical constraints, and personal preferences. Since each report is a "custom fit" for a particular client, reports should not be transferred to anyone else without first consulting the geotechnical engineer.

Each geotechnical report considers only the construction information and site boundaries that existed at the time of the investigation. Modification of construction plans, such as a change in the shape, size, weight, location, or intended use of a project, nullifies the recommendations contained in the report, unless the geotechnical engineer indicates otherwise. A geotechnical report can not be used for an adjacent site. Time and money can often be saved by consulting with the geotechnical engineer when circumstances change from those which existed when the report was written.

➢ *Site Conditions Can Change*

The conditions which existed at the time of a geotechnical investigation can change. Investigations can only report conditions at a particular time and place and no guarantee exists to ensure that recommendations will apply after natural or man made changes occur. Examples of some possible changes include: earthquakes, floods, fluctuations in groundwater, construction on or *next* to the site, and the addition or removal of soil. In addition, even the mere passing of time can affect site conditions. Consult with the geotechnical engineer to verify site conditions have not changed since the geotechnical report was completed.

➢ *Geotechnical Findings Are Comprised Primarily of Professional Opinions*

Even if typical 6 inch borings were spaced 5 feet apart across an entire site (typical borehole spacings are on the order of at least 10's or 100's of feet apart), *less than one percent* of the soil or rock on the site would actually be explored. From this limited exploration, the geotechnical engineer is called on to provide an opinion regarding the subsurface conditions across the site, provide appropriate foundation recommendations, and predict the response of subsurface materials to numerous scenarios using information from samples that may or may not be representative of the entire site. Obviously, most of the geotechnical report is based on the professional opinion of the geotechnical engineer. The actual subsurface conditions may significantly differ from those which were encountered during the geotechnical investigation. Consequently, the most effective method of managing the risks associated with a project is to retain the geotechnical engineer who provided the report throughout construction of the project.

➢ *Contact Your Geotechnical Engineer When in Doubt*

Time, money, and confusion can all be saved by simple explanations at critical moments. Please contact your geotechnical engineer whenever there is any doubt regarding subsurface conditions or their effect on part or all of any project.

GEOTECHNICAL INVESTIGATION
EAST ELK GROVE PARISH
ELK GROVE, CALIFORNIA

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GEOTECHNICAL INVESTIGATION

EAST ELK GROVE PARISH

BRADSHAW ROAD, APN #121-022-008

ELK GROVE, CALIFORNIA

OUR JOB NUMBER: SGG-0038

1.0 INTRODUCTION

This report presents the findings, conclusions, and recommendations of a geotechnical investigation conducted for the proposed development of a new Catholic K-8 school and parish to be constructed at APN #121-022-008 in Elk Grove, California. We understand that the proposed site will encompass approximately 18 acres. Construction will consist of wood or steel stud frame, concrete tilt-up, and/or masonry structures supported on spread footing foundations with concrete slab on grade floors. An existing house and detached garage will be razed to accommodate the proposed construction. Maximum foundation loads (dead plus live) for these structures are anticipated to be in the range of 1 to 5 kips per linear foot for perimeter and interior wall loads and 5 to 25 kips for isolated column loads. Since the site is relatively flat, we expect that grading will consist of minor cuts and fills, less than 3 feet in vertical extent. The buildings will be surrounded by rural residential homes. The project will also include surface parking, drives, paved play areas, fire lanes, and associated landscaping.

The geotechnical study conducted at this site was prepared for the use of the architect and engineer for application to the design of the building and grading plans in accordance with generally accepted geotechnical engineering practices. No warranty is expressed or implied. This report presents the results of this study.

2.0 GENERAL SITE CONDITIONS

At the time of our investigation the majority of the site was undeveloped and covered with grasses and weeds that had been disked. An existing vacant residential home and detached garage were located in the northeast corner of the site. The site was relatively flat across the parcel. The site was bordered by Bradshaw Road to the east, and rural residential homes to the north, west, and south. Bradshaw Road shows signs of rutting. This arterial street is well traveled and would be expected to shows signs of fatigue. The general area that surrounds the site was rural residential.

3.0 GENERAL GEOLOGIC CONDITIONS

A geologic map of the area was reviewed and indicated the surface soils are described as Pleistocene age (10,000 to 1.8 million years ago) alluvium deposits of the Riverbank Formation. The site is located in Seismic Zone 3[1] and all structures should be designed accordingly. The closest active Class C fault is the Foothills fault zone located a distance of 28.1 kilometers. The CBC currently considers non-blind thrust faults for seismic design parameters. The closest Class B fault is the Concord – Green Valley fault located at a distance of 73.2 kilometers[2].

The California Geological Survey assigns a probabilistic (10% probability of exceeding that motion in a 50 year period) peak horizontal ground acceleration for surface soil at the subject site of 0.186g based on longitude and latitude coordinates[3]. A liquefaction evaluation was outside the scope of our services, however, due to the relatively low site acceleration and dense soil conditions, the potential for seismically induced surface distress is considered low.

Following is a table of the 2001 California Building Code Soil Parameters[2] which may be used for design of structures at the subject site:

2001 CALIFORNIA BUILDING CODE SEISMIC DESIGN PARAMETERS	
Seismic Zone 3, Z	0.30
Soil Type, S	Sc
Seismic Source Type	B
Seismic Coefficient, Ca	0.33
Seismic Coefficient, Cv	0.45

4.0 FIELD EXPLORATION AND LABORATORY TESTING

The field investigation conducted at this site consisted of drilling 12 exploratory test holes carried to depths of 5 to 15 feet. The test holes were drilled with a truck mounted Mobile B24 drill rig, utilizing 4-inch continuous flight auger. The locations of the test holes are shown on the Location Map, Plate No. 1. The locations of the test holes were determined by pacing from existing site features; hence, accuracy can be implied only to the degree that this method warrants.

[1] California Building Code, 2001 Edition, International Conference of Building Officials, Whittier, CA
[2] Blake, T.F., 1998a, UBC Seismic Version 1.03
[3] http://www.consrv.ca.gov/cgs/rghm/pshamap/psha12138.html

Sampling of the drilled test holes was performed at various depths using a California Modified 2.5 inch o.d. split spoon sampler with stainless steel tube liners. The sampler was driven by a 140 pound hammer with a 30-inch drop. Blow counts required to drive the sampler every 6 inches for a total of 18 inches were recorded.

Soil samples obtained from the test holes were preserved in stainless steel tubes until the samples could be tested in the laboratory. Samples were taken to the laboratory of Neil O. Anderson & Associates, Inc., Sacramento, California and used for performing various laboratory tests. Tests performed consisted of unit weights, moisture contents, gradation, expansion index, R-values, and Atterberg limits. A summary of the test results are presented on the Log of Boring sheets, Plates 2 through 13.

5.0 SOIL CONDITIONS

Visual classification of each soil stratum encountered according to ASTM D2488 (Visual – Manual Procedure) was made in the field by a representative from our office at the time the test holes were drilled. The samples obtained were checked in the laboratory by a geotechnical engineer and classification verified according to ASTM D2487. A classification and graphical representation of each soil encountered is presented on the Log of Boring sheets. The test boring legend is presented on Plate No. 14.

The soils encountered in the test holes varied slightly over the site. The upper soils generally consisted of silty sand and clayey sand that extended to depths of between 3 and 6 feet below the existing ground surface. The silty, clayey sand layer was underlain by 1.5 to 2 feet of hardpan which in turn was underlain by silty, clayey sand to the maximum depth explored of 16.5 feet. For a more detailed description of the soils encountered in the test holes see the Logs of Boring sheets.

Four samples of the sandy clay were tested in our laboratory for Atterberg Limits and exhibited liquid limits of 24 to 27, plasticity indexes of 7 to 12, and contained 52 to 72 percent silt and clay-sized particles (passing the No. 200 sieve). Four samples of the clayey sand were tested in our laboratory for Atterberg Limits and exhibited liquid limits of 22 to 30, plasticity indexes of 10 to 16, and contained 40 to 49 percent silt and clay-sized particles (passing the No. 200 sieve). Two samples of the silty sand were deemed non-plastic and contained 41 and 42 percent silt and clay-sized particles (passing the No. 200 sieve). Two samples of the underlying sandy clay were also tested for expansion index and rendered values of 19 and 27. Laboratory testing indicates the clay soil has low expansion potential. The laboratory test results are summarized in the Log of Test Borings and graphically illustrated in Appendix B.

Groundwater was not encountered in any borings at the time the borings were drilled. Groundwater conditions in the future could change due to rainfall, construction activities, irrigation, or other factors. The evaluation of these factors is beyond the scope of this study.

Test boring logs show subsurface conditions at the date and location indicated and it is not warranted that they are representative of subsurface conditions at other locations and times.

5.1 Percolation Tests

Percolation tests were performed at four test locations at the future grass playfields. Percolation tests were performed by first drilling a 4 foot deep, 6-inch diameter test hole. A base layer of pea gravel was poured in the bottom of the test hole. A 5 foot long, 2-inch diameter perforated PVC pipe was placed in the test hole. Pea gravel was poured around the PVC pipe up above the perforated portion of the pipe. The test hole was then filled with water so that the test hole could saturate for 24 hours. After the pre-saturation period, water was poured into the pipe and the water level was measured over a period of time to measure the percolation rate. The following table shows measured percolation rate for each test hole.

PERCOLATION TESTS	
Percolation Test No.	Percolation Rate (min./inch)
P.T. #1	5.44
P.T. #2	17.00
P.T. #3	4.41
P.T. #4	7.03

6.0 DESIGN STUDIES AND RECOMMENDATIONS

From a soil engineering standpoint, our office concludes that the site is suitable for construction of the proposed structures; however, all of the conclusions and recommendations presented in this report should be incorporated into the design and construction to help reduce the potential for soil and foundation problems. Our main concern for construction of the buildings is the demolition of the existing structures.

6.1 Demolition

The existing house, barn and other structures located on the northeast portion of the site will be demolished to accommodate the proposed construction. Following demolition, the concrete slab floors, footing foundations, exterior concrete flatwork, and pavement sections should be completely removed.

After removal of the building slabs, foundations, and flatwork, any loose soil should be removed and the resulting excavations should be scarified to a depth of 8 inches, moisture conditioned to at least 2 percent above optimum moisture content, and compacted to at least 90 percent of maximum density as determined by ASTM D1557, modified proctor density. Any underground utilities that will be abandoned, and are smaller than 4" in diameter may be left in place. Utilities 4" in diameter or larger should be removed, grouted solid, or crushed in place and back-filled. The stumps of any trees should be removed. During stump removal all roots greater than 1/2 inches in diameter should be grubbed out. Voids resulting from concrete, asphalt, stump and root or utility removal should be cleaned out of all loose soil and debris and then scarified, moisture conditioned, and re-compacted as specified in Appendix A. Voids should then be backfilled with engineered fill as specified in Appendix A. Water wells, septic tanks, and leach lines should be removed and/or capped in accordance with the Sacramento County Department of Environmental Health rules and regulations. We recommend project bidding to include a line item for foundation and utility demolition.

6.2 Grading

The site should be initially cleared of all vegetation, trees, roots, debris, and deleterious material as outlined in Appendix A, Engineered Fill Specifications. Areas that are covered with light vegetation consisting of native weeds and grasses may be blended into the soil. Areas of moderate or heavy vegetation should not be blended into surficial soils, but should be cut and removed from the site. An engineer from our office is required to determine the degree of vegetation prior to grading operations. Voids resulting from the removal of any buried structures (such as irrigation structures or pipes, foundations, septic systems or water lines) should be cleaned of all loose soil and debris so that they may be backfilled during filling operations. All wells shall be abandoned in accordance with Sacramento County requirements. After clearing operations and any cuts have been made, the subgrade thus exposed shall be scarified a minimum of 8 inches and compacted as indicated in Appendix A. Fill placed on building pads and in pavement areas should be non expansive and placed as engineered fill as recommended in Appendix A. Soils encountered on the site should be suitable for use as engineered fill.

6.3 Winterization and Construction Equipment Mobilization

Cohesive soils located across the site can trap moisture from winter rains within the upper zones of the subgrade. This is known to cause unstable "pumping" subgrade conditions which can hinder the movement of grading equipment if construction is occurring in the winter or early spring. This should be taken into consideration when planning the site construction schedule. If it is desired to proceed with site grading during wet conditions, our office can provide recommendations for subgrade stabilization.

6.4 Spread Foundations

If grading is accomplished as specified, foundations for the proposed buildings may consist of shallow, spread or continuous foundations bearing on compacted native soil, engineered fill, or a combination of both. Foundations may be designed using a bearing capacity of 2000 pounds per square foot (psf) for dead plus live loads. If a higher bearing capacity is desired, we recommend the foundations either be carried to the underlying hardpan, or supported on a minimum 2 feet of engineered fill. The engineered fill may consist of either lean mix concrete (2 sack mix), or over-excavated and compacted soil as specified in Appendix A. With the foundations supported on either hardpan or a minimum 2 feet of engineered fill, a bearing capacity of 3000 psf for dead and live loads may be used in design. The above bearing capacities may be increased by 1/3 for temporary wind and seismic loads.

The minimum width of all foundations should be 12 inches. Foundations should be embedded a minimum depth of 18 inches bellowing surrounding grade.

Potential settlement, either immediate or long term, of foundations constructed on compacted native soils and loaded in the manner described above, should be less than 1 inch total and ½ inch differential across the width of the buildings. Care should be taken to understand settlements may vary based on actual loads and footing sizes.

To ensure footings have adequate support, special care should be taken when footings are located adjacent to trenches. The bottom of such footing should be at least 1 foot below an imaginary plane with an inclination of 1.5 horizontal to 1.0 vertical extending upward from the nearest bottom edge of the adjacent trench.

Lateral resistance for spread footing may be provided by assuming a passive pressure acting against the side of the footings equal to 300 pounds per cubic foot (pcf) equivalent fluid pressure. Lateral resistance may also be provided by computing friction between the bottom of the footing and the soil. A coefficient of friction of 0.30 should be utilized. If footings are cast against firm native soil, passive and frictional resistance may be combined