

**PRELIMINARY GEOLOGIC AND GEOTECHNICAL
INVESTIGATION
FOR THE PROPOSED
DIAMOND DEVELOPMENT PROJECT
CITY OF LAKE ELSINORE, CALIFORNIA**

Project No. 690-000-03
November 13, 2008

Prepared for:

**JIC-CP, Diamond Development, LLC
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By:

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November 13, 2008
Project No.: 690-000-03

Attention: Mr. Adam Dodge

Subject: Preliminary Geologic and Geotechnical Investigation
Proposed Diamond Development Project
City of Lake Elsinore, California

References: See attached List of References

1.0 INTRODUCTION

Neblett & Associates, Inc. (N&A) performed a preliminary geologic and geotechnical investigation for the proposed Diamond Development Project, located in the City of Lake Elsinore, California. The purpose of this investigation was to obtain geologic and geotechnical information pertinent to the subject site and to develop preliminary recommendations for site grading, foundation design and construction for the proposed Project, to be included in an environmental impact report (EIR) submittal.

This report discusses the regional and local geologic setting and addresses geotechnical factors pertinent to the proposed development including seismic hazards, liquefaction, and flood hazards. The approximate 1 inch = 200 feet topographic map (Plate 1, map pocket), prepared by Wilson-Mikami Corporation, was utilized to plan this investigation and develop the conclusions and recommendations presented in this report. Proposed building pad locations and elevations are not included on this topographic map.

2.0 SITE LOCATION

The Diamond Development Project encompasses a total of approximately 55-acres of vacant lots and partially developed lots along Diamond Drive, and surrounding the Lake

Elsinore Storm Baseball Stadium (Figure 1 – Site Location Map). The study area includes:

- A portion of the Mission Trail Plaza development, a mixture of big-box stores and individual mini-mall type shops south of the Stadium.
- A trapezoidal shaped vacant lot bounded by the Mission Trail Plaza to the north, Diamond Drive to the east, Pete Lehr Drive to the south and the Lake Elsinore Inlet Channel to the west.
- A triangular shaped vacant lot bounded on the east by the Albertsons/Do-it Center big box stores, on the south by Malaga Road and on the west by Diamond Drive.
- The current parking lot for the Elsinore Storm Baseball Stadium graded as part of the John Laing Homes' Summerly Development (N&A, 2007).

3.0 PROJECT DESCRIPTION

The Diamond Development Project will include a mix of commercial, tourist, and entertainment facilities with Diamond Stadium the centerpiece. It is our understanding the subject project will also include retail, office, hotel and residential. The density and square footage of these uses is currently unknown.

3.0 PREVIOUS INVESTIGATIONS

South Coast Geologic Services (1991), Zeiser Geotechnical Inc. (1992), Pacific Soils Engineering (PSE, 1993) and N&A performed investigations for the project area. Essentially, the reports were prepared to address the proposed residential and commercial development of this portion of the Lake Elsinore Back Basin. This portion of the basin was originally programmed for development of a baseball stadium and big-box retail stores. Figure 2 is an Aerial Photo showing the regional features on this portion of Lake Elsinore.

Information regarding the construction of the Mission Trail Plaza was not made available to N&A, and will be evaluated further as plans become available.

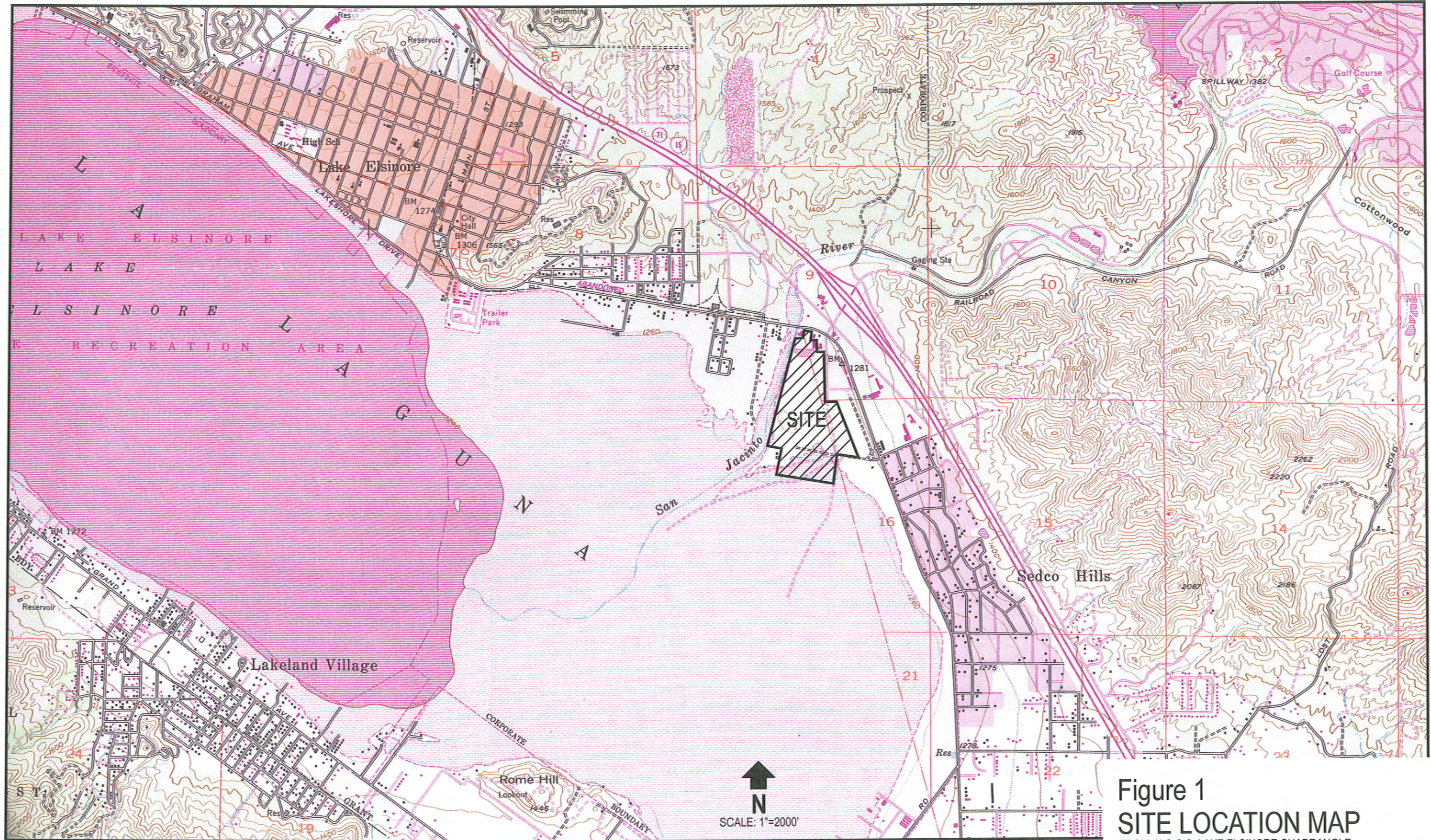


Figure 1
SITE LOCATION MAP

FROM: U.S.G.S. LAKE ELSINORE QUADRANGLE

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Rough grading for the Stadium was completed by PSE in 1993 (References). Spoil materials (oversized rock, gravelly material, etc.) was stockpiled in the triangular shaped lot east of Diamond Drive as part of the grading activities (Plate 1 – Geologic Map).

N&A was retained in 1997 to evaluate this overall portion of the back basin from a geologic/geotechnical aspect, including preliminary recommendations for site grading, foundation design and construction and issues related to faulting and liquefaction. As a result of these investigations, N&A identified and delineated a subsurface fault zone that is located southwest of the project site area and trending in a northeast-southwest en echelon fashion (Plate 2 – Area Geologic Map). The subsurface fault zone was determined by a variety of methods to be inactive within the Summerly Development residential sphere. These reports were reviewed and approved by the County of Riverside Geologist (N&A, 2005).

N&A observed and provided recommendations during the mass grading of the adjacent John Laing Homes Summerly development (N&A, 2007). Those activities included the construction of the Stage 1 and Stage 2 design areas, the San Jacinto River alignment, and the Links at Summerly Golf Course. The Stadium parking lot, a portion of the present project, was included in that development.

The subsurface investigations for these previous studies included seismic refraction/reflection surveys, hollow-stem auger borings, hollow-stem auger continuous core borings, cone penetrometer tests (CPT), monitoring wells, and fault trenches.

Pertinent data from these earlier investigations by others for the subject site are included in this report for reference. An Area Geologic Map, including the approximate locations of previous subsurface explorations, is attached as Plate 2 for reference.

4.0 SCOPE OF WORK

In order to evaluate the geologic and geotechnical conditions at the subject site, Neblett & Associates Inc., performed the following:

- Review of pertinent geologic and geotechnical literature and aerial photographs.

- Subsurface exploration utilizing a hollow-stem auger drilling rig. The exploration included six borings to depths of 51 feet below site grades, soil sampling and logging by our geologist (Appendix A).
- CPT soundings with a direct-push rig. Three CPT's were excavated to depth of 36.5 feet (CPT-1), 47 feet (CPT-2), and 42 feet (CPT-3 – Appendix A).
- Laboratory testing of representative soil samples obtained from the borings. The laboratory testing program included in-situ moisture/density, maximum density/optimum moisture content, grain-size distribution, direct and remolded shear, consolidation, soil expansion and soluble sulfate testing. The results of density/moisture content tests are posted to the Log of Geotechnical Borings presented in Appendix A. Laboratory test results are presented in Appendix B.
- Seismic hazard evaluation including 2007 CBC Seismic Design Parameters (Appendix C).
- Site specific liquefaction analysis (Appendix D).
- Preparation of this report, summarizing the results of the investigation, including geotechnical recommendations for site grading and foundation design and construction.

5.0 SITE GEOLOGY

5.1 Geologic Setting

The Lake Elsinore area is located in the northwestern portion of the Peninsular Ranges geomorphic province, characterized by northwest-southeast trending mountain ranges and intervening basins and valleys (Figure 3 – Regional Geologic Map; Plate 2 – Area Geologic Map). This area is also situated within the northwest-trending Elsinore Fault Zone (EFZ), which separates the relatively stable Perris structural block on the northeast from the Lake Elsinore Mountains on the southwest. In general, the EFZ consists of several sub-parallel, right-lateral strike-slip faults within a 1 to 4 km wide zone (Hull, Nicholson, 1992). This zone forms a trough separating low hills on the northeast side and

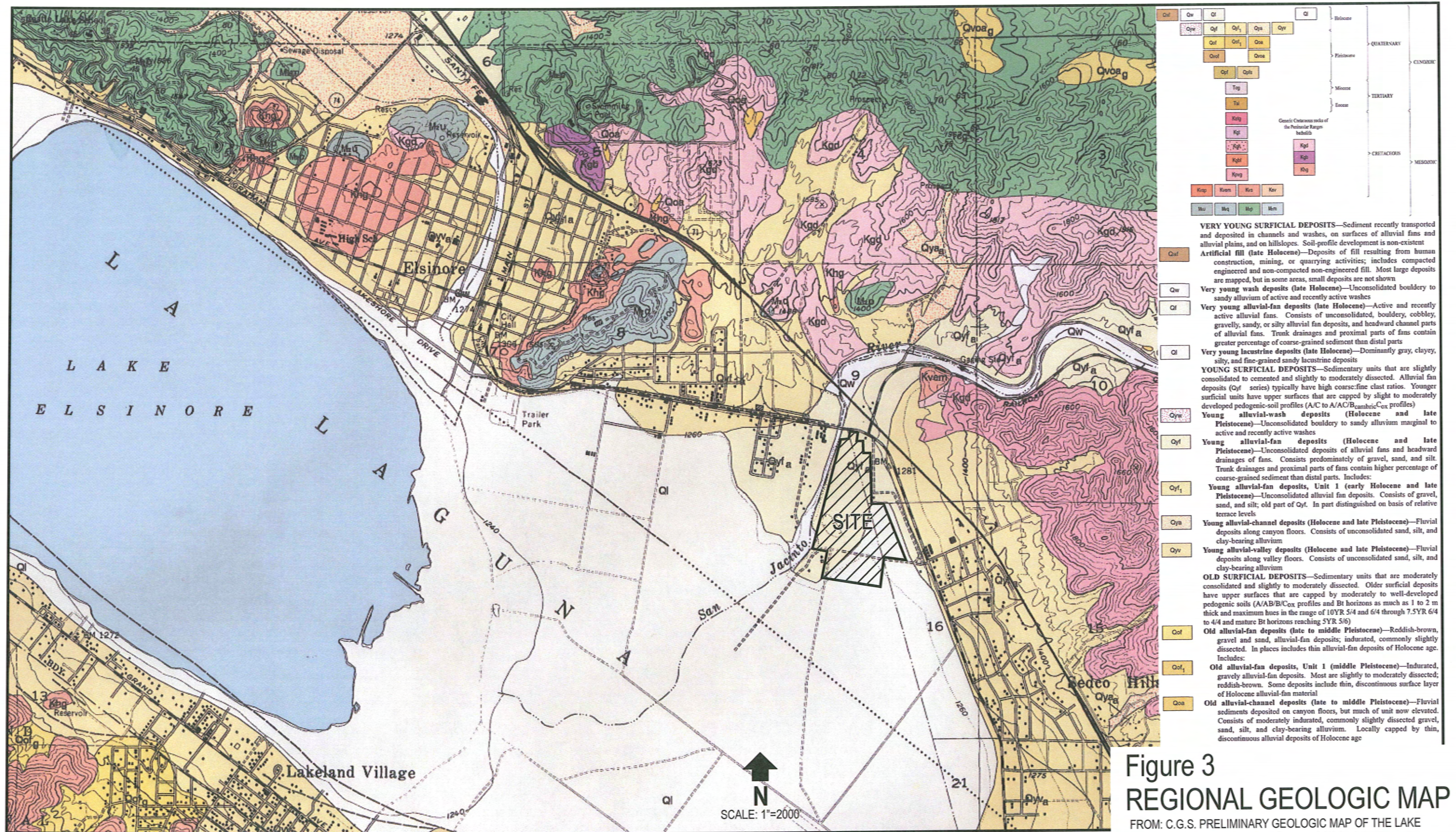


Figure 3
REGIONAL GEOLOGIC MAP

FROM: C.G.S. PRELIMINARY GEOLOGIC MAP OF THE LAKE
ELSINORE 7.5 QUADRANGLE (Morton & Weber)



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the rugged and steep Elsinore Mountains on the southwest (Plate 2 – Area Geologic Map).

Near Lake Elsinore the right-lateral strike-slip EFZ generally consists of a series of en-echelon fault strands -- the Glen Ivy North Fault (GINF) zone along the north shore of the lake, and the Wildomar and Willard faults along the south shore. The lake is the surface expression of a closed depression formed as a pull-apart basin at a major right-step between the Wildomar and Willard faults on the south side of the lake, and the GINF zone on the north (Morton, and Weber Jr., 1991).

The basement bedrock is mainly granodiorite of the Southern California Batholith. Medium to coarse-grained, granitic and non-marine sandstone conglomerates, siltstones and beds of clay of the Pauba Formation overly the basement complex (Dept. of Water Resources, 1981; Mann, 1951, 1955). The Pauba formation, is in turn, overlain by floodplain deposits. Older floodplain deposits within and immediately southeast of the lake consist of inter-bedded sand, silt and clay with occasional stringers of gravel. Overlying these deposits are clays, silts and fine sands formed in the sink and terminus of the San Jacinto River. These deposits are lacustrine in origin, of Holocene age, and are relatively poorly consolidated near the surface, but more consolidated at depth.

5.2 Geologic Units

Plate 1 (map pocket) shows the conceptual site plan superimposed with the geology of the subject site. Plate 1 was prepared based upon field mapping, subsurface exploration, and literature review.

The surficial unit underlying the site includes younger fan deposits (Qyf, Morton and Weber Jr., 1991 - Figure 3) to at least fifty (51) feet below grade. Also mapped are very young wash deposits (Qw), very young lacustrine deposits (Ql), undocumented artificial fill (afu), certified fill by others (caf-others), certified fill by Pacific Soils (caf-PSE), and certified fill by N&A (caf-N&A). A brief description of these units is included below:

- Younger fan deposits are emplaced as alluvial fans and include poorly sorted yellowish brown sands and gravels
- Very young wash deposits are generally encountered near the active stream channels. The wash deposits are mapped offsite of the subject project area.
- Very young lacustrine deposits have been mapped throughout the Back Basin area, and underlie the current stadium parking lot. The younger fans (Qyf) are underlain by the lacustrine deposits at depth. The lacustrine deposits vary widely from fine to coarse sands and gravels, to silts and clays.
- Undocumented artificial fill material underlies the triangle-shaped lot east of the stadium and consists of rock and soil spoil material generated from the grading for the stadium. This material tends to be coarse grained sands, cobbles and boulders.
- Certified fill materials by others include reworked soils of near-source origin used during grading of the Mission Trail Plaza shopping center north of the subject site. This material is largely comprised of sands.
- Certified fill materials placed under the observation of and testing by Pacific Soils during construction of the stadium and associated roadways and parking areas (PSE, 1994). The material was derived from near-site sources and is anticipated to be comprised primarily of sands and gravels.
- Certified fill soils placed under the observation and testing by N&A during construction of the stadium parking lot as part of the neighboring John Laing Homes Summerly Project (N&A, 2007). The material was derived from the back-basin area and is comprised of sands and gravels.

5.3 Landslides

Literature review, site mapping, aerial photo analysis, and field exploration indicate that landslides do not exist on or adjacent to the subject site.

5.4 Groundwater

Groundwater Levels

Groundwater was encountered in some of the exploratory borings drilled within the study area. The groundwater depths from existing grades recorded in the various borings and CPT's are presented on the pertinent boring logs (Appendix B). In general, subsurface water was encountered at shallower depths in the west portion of the site, due to their proximity to the lake and lake inlet. No water was encountered in borings NB-1 through NB-3. It appears that groundwater may be perched at several elevations, due to the presence of less pervious soil layers deposited under deltaic conditions. Based on available data, the historical high groundwater level at this site is on the order of 25 feet below existing site grades.

Impact of Groundwater

The groundwater levels observed during boring excavation and verified with CPT soundings are summarized in the table on the following page:

Groundwater Summary

Boring/CPT Excavation	Depth of Boring/CPT	Depth to First Encountered Ground Water (During Drilling) (ft.)	Boring/CPT Elevation (ft.-msl)	Groundwater Elevation (ft.-msl)
NB-1	51	NA	1267	NA
NB-2	51	NA	1262	NA
NB-3	51	NA	1270	NA
NB-4	51	42	1262	1220
NB-5	51	40	1270	1220
NB-6	51	40	1273	1233
CPT-1	36.5	NA	1263	NA
CPT-2	47.0	40	1269	1229
CPT-3	42.0	36.5	1269	1232.5

Generally, variations in groundwater level may occur due to fluctuations in rainfall, run-off and other factors. Considering the depth to groundwater and the anticipated remedial grading in fill areas, the groundwater is not anticipated to affect the grading for the proposed development.

6.0 SEISMIC HAZARD EVALUATION

6.1 Surface Rupture and Strong Ground Motion

The Elsinore Fault system between Glen Ivy and Lake Henshaw exhibits relatively large-scale strike-slip along a braided system of en-echelon normal, reverse and strike-slip faults that have produced, in some areas, pull-apart basins and horsts (Kennedy, 1977; Mann, 1955; Millman et al., 1986). Weber (1977) indicated that the entire Elsinore Fault

system has probably been active in late Quaternary time. Figure 4 is a portion of the State of California Special Studies Zones Map.

The trace of the Elsinore Fault Zone (EFZ) is geomorphically apparent north of Lake Elsinore and south in the Temecula area from satellite images (Lamar, 1992) and standard aerial photographs. The major branches of the EFZ occur near Lake Elsinore. These are, from north to south, the Glen Ivy North Fault, the Glen Ivy South Fault, the Willard Fault, the Wildomar Fault and the Wolf Valley Fault. The most active segments of EFZ north of Lake Elsinore are the Glen Ivy South, the Glen Ivy North (from its northwest end to the southeast along the present Lake Elsinore eastern shoreline) and the Wildomar Fault to the south (Weber, 1977). N&A completed assessments of this fault to determine its location and activity, if any. These investigations determined locations of the faults present in the project area and that these faults were inactive. The results of the Fault Investigation completed by this firm are included in the references (N&A, 2005). The 2007 CBC Seismic Design Parameters and Peak Ground Acceleration values for the project are presented in Appendix C.

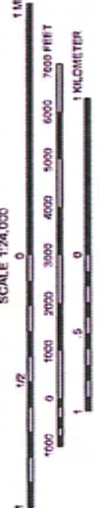
As described above, the subject site does not lie within the latest Alquist-Priolo Special Studies Zones (A-P) of the CDMG (SP-42, 1999; Figure 4). The nearest A-P zoned fault lies approximately 9,500 feet (1 $\frac{3}{4}$ miles) southwest of the proposed Diamond Development project area. It is very unlikely the subject site will experience fault related surface rupture. However, the subject site may experience strong ground motion as the result of regional seismic activity during the project life.

6.2 Liquefaction

Liquefaction is a phenomenon in which saturated, cohesionless soils lose strength during relatively severe earthquake ground motions, with potential adverse effects to buildings and road structures. In general, during ground motion, saturated fine sands and silty sands tend to compact and decrease in volume if drainage is impeded, resulting in an increase in pore water pressure. If the pore water pressure becomes equivalent to the overburden pressure, the effective stress becomes zero and, consequently, the cohesionless soil loses



TOPOGRAPHIC BASE BY U.S. GEOLOGICAL SURVEY 1955
PHOTO REPRODUCED 1973



MAP EXPLANATION

Potentially Active Faults

1900 C
Faults considered to have been active during Quaternary time; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.

--- Aerial photo lineaments (not field checked); based on youthful geomorphic and other features believed to be the results of Quaternary faulting.

Special Studies Zone Boundaries

These are delineated as straight-line segments that connect encircled turning points so as to define special studies zone segments.

--- Seaward projection of zone boundary.

STATE OF CALIFORNIA
SPECIAL STUDIES ZONES
Delineated in compliance with
Chapter 7.5, Division 2 of the California Public Resources Code

ELSINORE QUADRANGLE

OFFICIAL MAP

Effective: January 1, 1980

James F. Davis State Geologist

REFERENCES USED TO COMPILE FAULT DATA

Elsinore Quadrangle

Saich, D.P., 1979, Fault Evaluation Report FEO-79 (Elsinore fault zone, Prado Dam to Lake Elsinore, Riverside County). Unpublished report, California Division of Mines and Geology, 20 p., with supplement, 15 p. (see figures 4 and 5).

Weber, J.H., 1977, Seismic hazards related to geologic factors, Elsinore and Chino fault zones, California. California Division of Mines and Geology Open File Report 77-14, 94 p., plates 2A and 2B.

IMPORTANT - PLEASE NOTE

- 1) This map may not show all potentially active faults, either within the special studies zones or outside their boundaries.
- 2) Faults shown are the basis for establishing the boundaries of the special studies zones. The identification of these potentially active faults and the location of such fault traces are based on the best available data. Traces have been drawn as accurately as possible at this map scale; however, the quality of data used is varied.
- 3) Fault information on this map is not sufficient to serve as a substitute for the geologic site investigations (special studies) required under Chapter 7.5, Division 2, Section 2623 of the California Public Resources Code.
- 4) Fault information on this map is not sufficient to serve as a substitute for the geologic site investigations (special studies) required under Chapter 7.5, Division 2, Section 2623 of the California Public Resources Code.

Figure 4

STATE OF CALIFORNIA SPECIAL
STUDIES ZONES



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its shear strength and is considered to be in a liquefied state. Factors known to influence the potential for liquefaction include soil type and depth, grain-size distribution, relative density, groundwater level, degree of saturation, and both the intensity and duration of ground shaking.

The site may experience strong shaking and ground accelerations as the result of a nearby seismic event.

During the current investigation, six exploratory borings (Borings NB-1 through NB-6) were drilled to depths of 51 feet below grade (fbg) to obtain additional site data for engineering evaluation. In Borings NB-4 through NB-6, ground water was encountered at depths ranging from 40 fbg to 42 fbg. Also, CPT soundings were advanced near the locations of soil borings NB-2, NB-3, and NB-5. Groundwater elevations were determined via pore pressure dissipation tests in CPT-2 and CPT-3, with corresponding depths of 40 fbg and 36.5 fbg, respectively.

The borings and CPT's indicate that the sub-surface materials consist predominantly of silty sands and sands with occasional gravel, clayey silts, and gravelly sands. The silty sands and gravelly sands are generally dense to very dense, and include minor zones of clayey silt and silty sand layers.

As part of the current investigation, liquefaction potential was evaluated using the results of CPT soundings and the results are included in Appendix D. Based on our evaluation, minor zones of potential soil liquefaction are present below depths on the order of 25 feet.

6.3 Earthquake-Induced Landslides

Site mapping, aerial photo analysis, and field exploration indicate that landslides do not exist on or adjacent to the subject site. The lack of relief at the site indicates the area is relatively stable in the recent geologic past (Holocene) and has not been subject to earthquake-induced large-scale land sliding. Therefore, the potential for earthquake induced, large-scale, landsliding at the subject site is considered nil.

6.4 Seiche

Seiche is defined as a standing wave oscillation generated in a closed or semi-closed body of water caused by wind, tidal current, and earthquake. Seiche potential is highest in large, deep, steep-sided reservoirs or water bodies. Lake Elsinore lacks significant potential for damaging seiche because it is very shallow, and because of flood control devices constructed by the Army Corps of Engineers including the berm fill at the southern end of the lake, and the proposed finish residential pad grades. Based on the above considerations, the potential for seiche effects to the project site is considered remote.

7.0 EVALUATION, CONCLUSIONS, AND RECOMMENDATIONS

7.1 General

Based on the results of our field and laboratory investigation, combined with engineering analysis and our experience and judgment, it is our opinion that the site is considered suitable for the proposed development from a geologic and geotechnical engineering standpoint, provided the recommendations presented herein are incorporated into the design and implemented during construction.

Preliminary geotechnical recommendations for site grading and foundation design and construction are presented below. As conceptual grading and foundation plans are completed, they should be forwarded to our office for review to verify conformance with the intent of these recommendations, and to enable us to provide additional/revised recommendations, as appropriate.

7.2 Major Geotechnical Factors

The major geotechnical factors which should be considered in the design and construction of the planned commercial development include the following:

- (a) Site borings indicate that the on-site soils in the planned project site are generally loose and disturbed to approximately 5 feet. Surface materials may include organic materials which should be stripped and wasted from the site. The onsite

soils, in their present condition, are unsuitable for providing acceptable support for new fill placement and structures. Therefore, prior to new fill placement, it will be necessary to implement remedial grading over the entire site, consisting of excavation of the site soils to a depth of at least 5 feet from existing grades and replacement with compacted engineered fill.

- (b) The triangle shaped lot east of the stadium is underlain with undocumented artificial fill which will require complete removal prior to fill placement. Based on findings in borings NB-1 and NB-2, removals in this lot could extend to 12 to 13 feet below existing site grades.
- (c) Documentation relative to the material types, placement conditions, and relative compaction of the fill soils placed by others during grading for the Mission Trail Plaza were not available for our review and evaluation. Based on subsurface exploration at Boring NB-6, it appears that this fill may be on the order of 5 feet in depth. Full depth removal of this existing fill material and replacement with engineered compacted fill is recommended in order to provide more uniform and adequate support for future development.
- (d) Groundwater was encountered in some of the exploratory borings in the area north of the stadium, as discussed in Section 5.4.
- (e) Considering the depth to static groundwater and the planned new fill placements, groundwater will not impact the grading operations.
- (f) The proposed structures should be designed and constructed to the prevailing seismic design standards.

7.2 Liquefaction Hazard

The results of our liquefaction evaluation indicate that there is potential for soil liquefaction in some of the sandy soil horizons below general depths of 25 feet below existing site grades.

Typical measures to mitigate the potential adverse effects due to site liquefaction include the following, in order of decreasing effectiveness:

- Improvement of liquefiable soils:
Dynamic compaction, compaction grouting, or removal and replacement with approved compacted fill would minimize liquefaction potential of the underlying soils and associated adverse effects to structures.
- Structures supported on deep foundations:
Use of pile/grade beam system with structurally supported floor slab would help mitigate temporary loss of foundation support due to liquefaction of the underlying site soils, although there may be some temporary loss of lateral support.
- Structures supported on mat foundations:
A mat foundation may suffer distress in the form of differential movement due to temporary loss of foundation support during a major seismic event. However, it may be feasible to re-level mat supported structures utilizing compaction grouting/mud jacking procedures. Mat foundations typically outperform conventional footings in reducing the adverse effects associated with dynamic differential settlement.
- Structures supported on conventional foundations:
Continuous and spread footings incorporating redundant systems (e.g. grade beams) would provide some measure of mitigation against the adverse effects due to site liquefaction. However, the structure may experience extensive distress.

The mitigation measure selected by the Owner should be based on cost, feasibility and level of acceptable risk. It is our opinion that shallow conventional foundations incorporating redundant systems may be used for lightly loaded structures. Consideration should be given to supporting structures with relatively high structure loading (e.g., hotel, entertainment center) on a mat foundation system.

Preliminary recommendations are provided herein for both shallow conventional and mat foundations to help mitigate potential adverse effects due to soil liquefaction, although

not completely eliminate potential problems. Deepened foundation (i.e. caisson/grade beam) recommendations can be provided, if requested, and would need to be based on more detailed analysis, including additional field exploration and laboratory testing.

7.4 Site Grading

General

Grading for the subject site can be accomplished utilizing conventional earth-moving equipment. Specific grading recommendations for the subject site should be developed during tentative grading plan review.

Site Clearing and Preparation

- (i) Prior to the start of grading and earthwork operations, all excess vegetation, surface trash, debris and other deleterious materials should be removed and wasted away from the site.
- (ii) Debris resulting from the demolition of any existing structures and improvements should be removed and hauled away from the site. Native soils disturbed during demolition and clearing operations should be excavated full depth.
- (iii) Any underground utilities to be abandoned within the zone of proposed construction should be cut-off a minimum of 5 feet from the area of future building. The ends of cut-off lines should be plugged a minimum of 5 feet with concrete exhibiting minimum shrinkage characteristics to prevent water migration to or from hollow lines. Capping of lines may also be required should the plug be subject to any line pressure.

Alternately, deep hollow lines may be left in place, provided they are filled with concrete. No filled line should be permitted closer than 2 feet from the bottom of footings.

Local ordinances relative to abandonment of underground utilities, if more restrictive, will supersede the above minimum requirements.

- (iv) Precautions should be taken during the performance of all work described in the following sections, at all times, to protect the work site, particularly excavated areas, from flooding, ponding and inundation due to poor or improper temporary surface drainage. During periods of impending inclement weather, temporary provisions should be made to adequately direct surface drainage away from all sources and off the work site and to provide adequate pumps and sumps to handle any flow into the excavations.
- (v) All site preparation and grading should be made under the observation of the Project Geotechnical Engineer or Project Engineering Geologist, and/or their field representatives.

Remedial Grading

Remedial grading will include the full depth removal of fill soils placed by others across the northerly site portion and existing undocumented fill soils underlying the triangle-shaped lot east of the stadium.

To provide more uniform and acceptable support for structure foundations and improvements, it is also recommended that some remedial grading include the near surface site native soils. As a minimum, near surface native soils should be over-excavated at least 5 feet below existing or proposed grades, whichever depth is greater, and replaced with engineered compacted fill.

The recommended depth of remedial grading may be greater, and would be dependent upon review of conceptual plans and design loading conditions, when available. The minimum lateral extent of over-excavation beyond building structure perimeters should be to a distance equivalent to the depth of remedial grading or 10 feet, whichever distance is greater.

The depth of over-excavation should be reviewed by the Project Geotechnical Engineer during construction. Any sub-surface obstruction, buried structural elements, and unsuitable materials encountered during grading, should be immediately brought to the

attention of the Project Geotechnical Engineer for proper exposure, removal and processing, as required.

Exposed excavation bottoms should be observed and approved by the Project Geotechnical Engineer or his representative, prior to fill placement.

The excavated on-site soils may be used as compacted fill provided they are free of organics, debris, over-size rocks greater than 6 inches in maximum dimension, and other deleterious matter.

Temporary Excavations

Remedial grading recommended for the proposed development will include over-excavation to depths of at least 5 feet below existing site grades to remove unsuitable site soils. The plan showing the building configuration and footprint is not available at this time. If structures are to be located at or close to the property limits, temporary shoring may be necessary to facilitate planned excavations. If the buildings are located away from the property limits, temporary excavations may be made by using sloping back-cuts.

Temporary excavations in site soils deeper than 4 feet should be sloped no steeper than 1H:1V or shored in accordance with Cal OSHA requirements.

The design of shoring should include surcharge effects of existing structures and anticipated traffic, including construction equipment, when loading is within a distance from the shoring equal to the depth of excavation.

Our office should review soil conditions exposed during excavations to verify acceptability of temporary slopes. Final temporary back cut slope design will be dependent upon the soil conditions encountered, construction procedures and schedule. Temporary back-cuts should not to be left open for extended periods of time.

Fill Placement, Moisture Control, and Compaction

After the approval of site clearings and removals, loose surficial material should be removed to expose competent removal bottoms. All removal bottoms must be approved by the Project Geologist and/or Geotechnical Engineer.

- (i) The approved exposed surface should be scarified 6 inches in depth, thoroughly moisture conditioned to about 1 to 2 percentage points wet of optimum moisture content and compacted to minimum of ninety (90) percent of the laboratory maximum density (ASTM: D1557) using heavy vibratory equipment.
- (ii) Approved stockpiled, on-site or imported soils should be spread in thin lifts not exceeding six (6) inches in thickness. The soils utilized for fill should be moisture conditioned to about 1 to 2 percentage points over optimum moisture content, and each lift rolled and compacted to minimum of 90 percent of the applicable laboratory maximum dry density (ASTM: D1557), until the desired grades are achieved.
- (iii) Import soils, if required, should be evaluated by the Project Geotechnical Engineer for suitability prior to delivery. Import soils should be free of organics, trash, debris, over-size rocks greater than 6 inches in maximum dimension, or other objectionable materials, and should exhibit engineering properties similar to the on-site soils.
- (iv) Plans and specifications should indicate that the grading contractor shall notify the Project Geotechnical Engineer not less than 72 hours in advance of the location of any soils proposed for import. Each proposed import source shall be sampled, tested and approved prior to delivery of soils for use on the site.

Field Testing and Observations

All site preparation, over-excavation, remedial grading, and earthwork should be performed under the observation and testing of the Project Geotechnical Engineer or his representative.

If it is determined during grading that site soils require over-excavation to greater depths for obtaining proper support for the proposed construction, this additional work should be performed in accordance with the recommendations of the Project Geotechnical Engineer.

The fill should be tested at the time of placement to ascertain that the required compaction is achieved. An adequate number of field density tests should be performed to verify compliance of earthwork to the geotechnical recommendations and the project requirements. The frequency of field testing should be in accordance with the recommendations of the Project Geotechnical Engineer and the requirements of governing jurisdiction.

The field density testing should be done using the Sand Cone Method (ASTM: D 1556) or the Nuclear Gauge Test Procedure (ASTM: D2216), or other test procedure, as recommended by the Project Geotechnical Engineer.

Quality control testing performed to determine the acceptability of the fill compaction should be based on the laboratory maximum dry density and optimum moisture content determined in accordance with ASTM: D 1557 test procedure.

Soil Shrinkage

Losses due to soil shrinkage are anticipated during the recommended remedial grading. For preliminary planning purposes, an estimated shrinkage factor of 15 percent may be assumed. This estimated shrinkage factor does not include losses due to vegetation removal/stripping, any demolition and foundation removal, etc.

In addition, site subsidence associated with the placement of engineered compacted fill is anticipated to be on the order of 0.1 feet.

7.5 Foundation Design

Presented below are preliminary recommendations for use in the design of shallow conventional footings and mat foundations. These recommendations should be confirmed based on conceptual plans and structure details and loading conditions, when available.

Conventional Footings

Lightly loaded structures (e.g., one and two story wood frame construction) may be supported on shallow conventional footings. These should be designed based on the following:

Allowable Bearing Pressure (1)	=	1,500 psf
Minimum Footing Depth (2)	=	18 inches
Minimum Footing Width	=	12 inches
Passive Soil Pressure (3)	=	250 psf/ft.
Friction Coefficient	=	0.40 (ultimate)
Minimum Footing Reinforcement	=	Two No. 4 bars, one each at top and bottom

- (1) Allowable bearing pressures may be increased by one-third for short term loading due to wind or seismic forces.
- (2) Footing depth is from lowest adjacent finished soil grade.
- (3) Value applies to level soil condition

In order to mitigate the potential adverse effects due to soil liquefaction, it is recommended that the footings incorporate redundancy and be designed to provide sufficient rigidity to span an unsupported distance of at least 15 feet. Isolated pad footings should be tied by grade beams in at least two orthogonal directions.

Mat Foundation

Consideration should be given to supporting structures with relatively high loads on mat foundation bearing on compacted engineered fill. Presented below are the recommended geotechnical criteria for the design of mat foundations for structure support:

Allowable Bearing Pressure:	1,500 lb./sq.ft
Footing Depth (1):	Perimeter of mat should include a deepened footing, minimum 12 inches depth.
Lateral Soil Resistance:	250 psf/ft. of embedment to a maximum of 2,000 lb/sq.ft.
Coefficient of Friction:	0.40 (Ultimate)
Modulus of Subgrade Reaction	150 pci
Mat Slab Thickness:	9 inches thick (minimum).
Mat Reinforcement (2):	To be determined by the Project Structural Engineer. Minimum No. 3 bars at 18 inches on center, both-ways, top and bottom.
(1)	Below lowest adjacent exterior soil grade
(2)	In order to mitigate the adverse effects due to soil liquefaction, it is recommended that the mat foundation be designed to provide sufficient rigidity to span an unsupported distance of at least 15 feet.

Foundation design details such as concrete strength, reinforcements, etc. should be established by the Project Structural Engineer.

Settlement

Total static settlements for conventional footings supporting column and wall loads on the order of 30 kips and 3 kips/lineal foot, respectively, and when designed in accordance with the above recommendations are not anticipated to exceed 1-inch. Differential settlement under static load is not expected to exceed 3/4 inch in 30 feet. A majority of these settlements will occur during construction, with the remaining settlements expected to be long-term.

Potential settlements for structures supported by mat foundation would be dependent upon the mat configuration and applied pressure at base of mat, and can be provided at a later date, based on conceptual information, when available.

Due to the presence of potentially liquefiable layers below 25 feet depths at this site, there is potential for liquefaction-induced settlement during a severe seismic event. Liquefaction settlements under seismic conditions are estimated to be on the order of one

inch total and are expected to occur over a large area. Generally, the differential settlements at level sites are expected to be small even if the total settlements are large. In accordance with Recommended Procedures for Implementation of DMG Special Publication 117 (March 1999), liquefaction-induced differential settlements on the order of one-half of the total settlement may be considered for design. The potential liquefaction-induced settlements are in addition to the estimated static settlements for structures.

For preliminary design purposes, lightly loaded structures should accommodate potential total and differential settlement (static plus liquefaction-induced) on the order of 2.0 inches and 1.0 inches in a 30 feet distance, respectively.

General Remarks

- i) Foundation depths should not be allowed to be affected adversely, such as through erosion, softening, digging, landscaping, etc.
- ii) Where foundations encroach closer than five (5) feet horizontally from the flow line of drainage swales, the footing should be deepened sufficiently to maintain the required embedment depth below the adjacent flow line.
- iii) Foundation details such as thickness, concrete strength, reinforcements, etc. should be established by the Project Structural Engineer considering the loading and potential differential settlements. The recommended reinforcements should be considered minimum. More restrictive criteria based on structural design considerations or Code requirements shall govern.
- iv) Foundation excavations should be observed and approved by the Project Geotechnical Engineer prior to the placement of reinforcement or concrete. Forming of footing excavations may be required. Excavations should be free of slough and debris and thoroughly moisture conditioned prior to placing concrete.
- v) Excavated material from footing and utility trenches should not be placed in slab-on-grade areas unless properly compacted and tested.

Seismic Design Considerations

The site, as is all of Southern California, is within a zone of seismic activity. Strong ground motion from an earthquake generated along active faults should therefore be anticipated at this site. The proposed development should be designed and constructed to the prevailing standards regarding seismic design. Seismic design should be based on current and applicable CBC requirements, as appropriate. Seismic design parameters based on the 2007 CBC are included in Appendix C.

7.6 Floor Slab-on-Grade

It is anticipated that, upon completion of grading, the compacted engineered fill will consist predominantly of granular, low-expansive soils (silty sands and sandy silts). Slab sub-grade soils should be thoroughly moisture conditioned, prior to slab construction. Structural and/or construction loading considerations will govern slab design and should be evaluated by the Project Structural Engineer. Testing should be conducted on the finish grade pad to determine the expansive nature of the surface soils at completion of rough grading.

In areas where a moisture sensitive floor covering will be used, the floor slab-on-grade should be underlain by a moisture/water vapor retarder, as recommended in Section 7.7 below.

7.7 Moisture/Water Vapor Mitigation for Concrete Slab-on-Grade

In order to reduce the potential for moisture/water vapor migration up through the slab and possibly affecting floor covering, a moisture/vapor retarder is recommended under concrete floor slab-on-grade. The moisture barrier should be properly installed, lapped and sealed in accordance with the manufacturer's specifications. Punctures and rips should be repaired prior to placement of sand.

As a minimum, this moisture/water vapor retarder should consist of 10-mil thick polyethylene ("Visqueen"), lapped and sealed, and placed mid-height within a 4-inch

coarse sand layer. This moisture/water vapor retarder should be installed in accordance with manufacturer's specifications.

The following recommendations are based on the tentative guidelines by the American Concrete Institute (ACI, April 2001) to reduce the potential moisture/water vapor intrusion in concrete slab-on-grade. Based on our review of available literature, it appears that the ACI procedure would be more effective to help reduce potential moisture/water vapor migration up through concrete slab-on-grade.

Recommendations based on the ACI guidelines are presented below:

- The moisture/water vapor retarder should consist of high strength polyethylene membrane and should meet or exceed the ASTM: E-1745-97 Class C material requirements for water vapor permeance, tensile strength and puncture resistance. The vapor retarder should consist of "Moistop Plus" (Fortifiber Building Products Systems) or "Vapor Block" VB 15 (Americover, Inc.), or approved equal. The vapor retarder should be underlain by a capillary break comprised of minimum 4 inches thick pea gravel layer. The gravel layer should be placed and compacted on approved soil sub-grade.
- The membrane should be placed on approved gravel layer and properly lapped and sealed. Membranes intersecting utility pipes, sewer lines, ducts or drains must be properly wrapped around the penetrations and sealed. All punctures and rips in the membrane should be repaired prior to placement of concrete, following manufacturer's recommendations. The vapor retarder should be installed in general accordance with the procedures outlined in ASTM: E-1643, and in conformance with the installation procedures recommended by the manufacturer.
- To minimize slab curling, a low slump concrete (low shrinkage mix design) should be used for the slab construction, as determined by the Project Structural Engineer.

The moisture/water vapor protection for concrete slab-on-grade should be selected based on cost and construction considerations, and considering potential future problems resulting from improper and uncontrolled landscape irrigation practices. Regardless of the moisture/water vapor retarder option selected, it should be emphasized that proper control of irrigation and landscape water adjacent to the structure is of paramount importance.

7.8 Concrete Design

Based on the test results presented in Table 1 in Appendix B, sulfate exposure for concrete is considered negligible (UBC Table 19-A-4). Concrete mix design, including water-cement ratio should be in accordance with 1997 UBC Table 19-A-4. Based on Table 19-A-4, special sulfate resistant concrete is not required. Additional sulfate testing should be performed during or near the completion of rough grading.

7.9 Utility Trench Backfill

Bedding materials should consist of sand with a sand equivalent value not less than 30 and should comply with the City of Lake Elsinore guidelines.

Existing on-site soils may be used for trench backfill, provided they are free of organic material and rocks over 6 inches in maximum dimension.

Backfill of all exterior and interior trenches should be placed in thin lifts not exceeding 4 inches and mechanically compacted to achieve a relative compaction of not less than 90% based on ASTM: D1557. Care should be taken not to damage utility lines.

Utility trenches should not be located within the influence of footings. This is defined as a zone located below the footing and a line sloping at an inclination of 1:1 (horizontal to vertical) outward from the outside edge of footings. If utility lines are located within the zone of footings, the backfill should be compacted to a minimum 95 percent relative compaction or slurry backfilled (minimum 1-1/2 sack cement- sand mix).

Trenches must be shored or sloped back as required by the local regulatory agency, the State of California Division of Industrial Safety Construction Safety Orders, and Federal OSHA requirements.

7.10 Site Drainage

It should be noted that potential problems may develop when drainage is altered through construction of retaining walls, paved walkways, and patios. Conditions which will lead to ground saturation must be avoided.

- All roof and surface drainage should be directed away from structures and their appurtenances to approved drainage facilities. Ponding of water should be avoided. For graded soil areas, a minimum gradient of 2 percent away from structures should be maintained.
- The recommended drainage patterns should be established at the time of fine grading and maintained throughout the life of the structure or, if altered, should be replaced with a properly designed area drain system.
- Irrigation activities at the site should be monitored and controlled to prevent over-watering. Planter areas adjacent to structures should be avoided. If utilized, such planters should include measures to contain irrigation water and prevent moisture migration into walls and under foundations and slabs-on-grade.

8.0 ASPHALT CONCRETE PAVEMENT

Preliminary asphalt concrete pavement sections for the planned development were developed utilizing the Cal Trans Method of Design and the Structural Section Design Guide for California Cities and Counties.

The following recommended pavement sections were computed assuming an "R" value of 20 for site soils compacted as sub-grade material, and assumed traffic indices. The actual "R" value will depend on the soil conditions exposed at the sub-grade elevations. During rough grading, "R" value testing should be performed on the pavement sub-grade soils to confirm the pavement design basis. Further analysis and evaluation are necessary,

if the design traffic index and the sub-grade “R” value are different from those used in our analyses.

Pavement Utilization	Traffic Index	Asphalt Pavement Section (inches)		
		AC	AB	Total
Circulation Driveways	6.0	4.0	6.0	10.0
Parking Areas	4.0	3.0	4.0	7.0

Note: AC = Asphalt Concrete, AB = Aggregate Base

Aggregate base should consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB) and should comply with the specifications outlined in the “Standard Specifications for Public Works Construction”, (“Green Book”). The base material should be compacted to minimum 95 percent of the maximum laboratory density, determined in accordance with ASTM: D-1557.

The soil sub-grade should be compacted to minimum 90 percent relative compaction. The sub-grade soils should exhibit a firm and unyielding surface, in addition to the recommended minimum compaction. Final compaction and testing of pavement sub-grade should be performed just prior to placement of aggregate base.

9.0 PLAN REVIEW, OBSERVATIONS, AND TESTING

Neblett & Associates, Inc. should be provided with conceptual, rough grading and foundation plans, when available, for our review to evaluate the preliminary recommendations presented herein and to develop additional/revised recommendations, as appropriate.

Field observation and testing during grading must be performed by a representative of this firm to verify conformance of the earthwork to the requirements of the regulating agencies, the project specifications, and the recommendations presented in this report. Any earthwork performed in connection with the subject project without the full knowledge of, and not under the direct observation of this firm, will render the recommendations of this report invalid.

10.0 LIMITATIONS

This report has been prepared for the exclusive use of JIC-CP, Diamond Development, LLC and their design consultants relative to the design and construction of the proposed project. This report is not intended for other parties, and it may not contain sufficient information for other purposes.

The Owner or Owner Representative should make sure that the information and preliminary recommendations presented in this report are brought to the attention of the Project Architect and Project Engineer and incorporated into the project plans.

The findings contained in this report are based upon our evaluation and interpretation of the information obtained from limited borings and/or test pits and the results of the laboratory testing and engineering analysis. The opinions and recommendations provided were based on the assumption that the geotechnical conditions, which exist across the site, are similar to those observed in the test excavations. The condition and characteristics of the sub-surface materials at locations and depths other than those excavated and observed may be different and no representations are made as to their quality and engineering properties. Should any conditions encountered during construction differ from those described herein, this office should be contacted immediately for evaluation of the actual conditions and for appropriate recommendations prior to continuation of work.

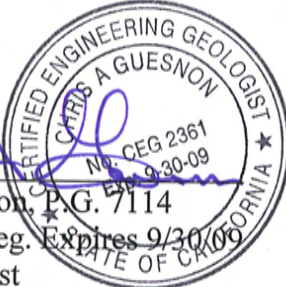
The findings and recommendations presented herein were obtained in accordance with currently accepted professional engineering principles and practice in the field of geologic and geotechnical engineering, and reflect our best professional judgment. We make no other warranty, either express or implied.

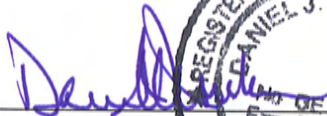
We thank you for the opportunity of being of service to you. We look forward to working with you and project team during the design and construction of the subject project.

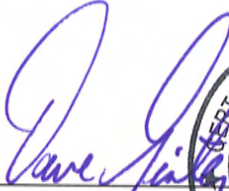
If you have any questions, or you require additional information, please contact the undersigned.

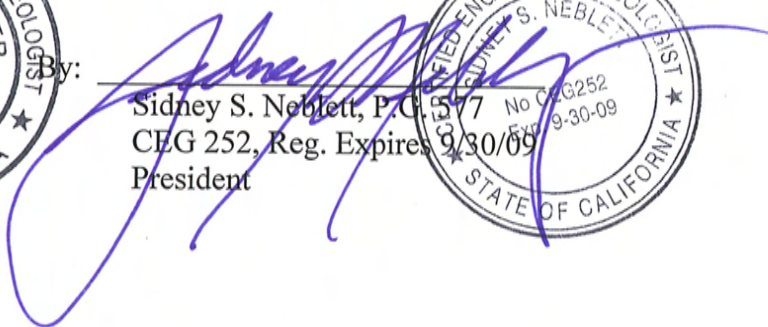
Respectfully submitted,

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File: 690-000-03 111308 JIC-CP ElsinoreDiamondFeasibilityReport.doc

REFERENCES

1. South Coast Geologic Services, inc., Geotechnical Feasibility Report for a Proposed $\pm$ 125 Acre Regional Shopping Center to be located Along the Future Extension of Railroad Canyon, East Lake Development, City of Lake Elsinore, California, dated January 17, 1991 (Project No. 90069.1).
2. Zeiser Geotechnical, inc., Preliminary Subsurface Feasibility Investigation for the Proposed 2,900 Acre East Lake Development Project, City of Lake Elsinore, California, dated June 29, 1992 (PN 90495-00).
3. Zeiser Kling Consultants, inc., Report of Feasibility-Level Geotechnical Investigation, 23-Acre Baseball Stadium Parcel, And Adjacent Commercial Site, Southwest of Sylvester and Mission Trail, Lake Elsinore, California, dated March 22, 1993 (PN 92393-01).
4. Pacific Soils Engineering, inc., Geotechnical Review of Rough Grading Plan for a Baseball Stadium, Sportstown and Big Box Commercial Centers and Associate Roadways and Foundation Recommendations for the Baseball Stadium, Lake Elsinore, California, dated July 2, 1993 (Work Order 101872-GP).
5. Pacific Soils Engineering, inc., Rough Grading Compaction Report for Structural Areas Only, Baseball Stadium, Lake Elsinore, California, dated August 25 1993 (Work Order 101872-RG).
6. Pacific Soils Engineering, inc., Results of Percolation Testing for Existing Playing Field, Proposed Baseball Stadium, Lake Elsinore, California, dated September 20, 1993 (Work Order 101872-RG).
7. Pacific Soils Engineering, inc., Final Rough Grading Compaction Report, Baseball Stadium, Parking Lots and Adjacent Streets, Lake Elsinore, California, dated May 19, 1994 (Work Order 101872-RG).
8. Neblett & Associates Inc., Geotechnical Review and Hydrogeologic Assessment for Concept Mass Grading Plan, Liberty Development Project, Tract 28766, Lake Elsinore, California, dated May 27, 1999 (Project No. 142-S).
9. Neblett & Associates Inc., Preliminary Geotechnical Investigation and 40-scale Grading Plan Review, Tentative Tract 31920, Phase I Residential and Golf Course Development, City of Lake Elsinore, County of Riverside California, dated December 15, 2004 (Project No. 420-000-05).
10. Neblett & Associates Inc., Additional Fault Investigation and Response to County of Riverside Report Review, Tentative Tract 31920, Summerly Site, City of Lake Elsinore, California, dated October 18, 2005 (Project No. 420-000-03)
11. Preliminary Geotechnical Investigation and 40-Scale Grading Plan Review, Tentative Tract 31920, Stage 2 of Summerly Development, City of Lake Elsinore, County of Riverside, California, dated February 7, 2006, Project No. 420-001-05.

12. Neblett & Associates, Inc., Final Rough Grade Compaction Report, Stage 1 of the Summerly Development Project, Tract 31920, Lake Elsinore, California, dated May 11, 2007 (Project No. 420-000-07).
13. Neblett & Associates, Inc., Rough Grade Compaction Report, Stage 2 of the Summerly Development Project, Tract 31920, Lake Elsinore, California, dated May 11, 2007 (Project No. 420-000-07).
14. Neblett & Associates, Inc., Rough Grade Compaction Report, Parcels 18 & 19, Stage 2, Summerly Development Project, Tract 31920, Lake Elsinore, California, dated November 30, 2007 (Project No. 420-001-07).