APPENDIX G

Geologic and Soils Investigation
SubSurface Designs, Inc.

SubSurface
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PRELIMINARY GEOLOGIC AND SOILS ENGINEERING INVESTIGATION

PROPOSED HIGH SCHOOL DEVELOPMENT

11023 LURLINE AVENUE

CHATSWORTH, CALIFORNIA

FOR

SIERRA CANYON HIGH SCHOOL FOUNDATION

ATTN: MR. SANJE RATNAVALE

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INTRODUCTION

This report presents the results of our Preliminary Geologic and Soils Engineering Investigation performed at 11023 Lurline Avenue in the Chatsworth area of Los Angeles, California. The purpose of the investigation was to determine the subsurface conditions as they relate to the proposed construction of a new high school. This report follows our previous site investigation which was limited to the construction of a new gymnasium structure and parking areas on the subject and adjacent properties.

SCOPE

This investigation is based upon:

- A topographic site plan by Iacobellis and Associates, Inc., a licensed land surveyor, that was utilized as our base map. This map appears to accurately reflect topographic conditions as observed at the subject property.
- A review of preliminary plans by Parallax Associates, Inc.
- The review of eight (8) hollow-stem auger borings and eight (8) backhoe trenches. The explorations were excavated to a maximum depth of forty-one feet (41'). The materials encountered were logged by a representative of this office. The explorations were backfilled with the excavated materials. However, backfill was not compacted and should be monitored for future settlement.
- Preparation of the enclosed Site Plan which locates the proposed development and our explorations (see APPENDIX I).
- Preparation of graphic logs of explorations placed at the subject property (see APPENDIX I)
- Laboratory testing and analysis of samples obtained during placement of the excavations (see APPENDIX II).
- Calculations which may include, but are not limited to, bearing value, lateral pressure, active earth pressure, slope stability (see APPENDIX III).
- A search of earthquake, fault, and seismic parameters and conditions relevant to the subject site utilizing the EQSEARCH, EQFAULT, and UBCSEIS computer programs (see APPENDIX IV).
- The review available maps and previously prepared reports by this office and others (see APPENDIX V).
- Preparation of this report.

The data that supports the following SUMMARY OF FINDINGS, CONCLUSIONS and RECOMMENDATIONS are contained within Appendices I through V.

The scope of our exploration is limited to the areas explored for the proposed development as delineated on the enclosed Site Plan. This report should not be considered as a comprehensive evaluation of the entire property. This report has not been prepared for use by other parties or for other purposes (or developments), and may not contain sufficient information for other than the intended use. If construction is delayed more than one year, this office should be contacted to perform an update and to verify the current site conditions.

LOCATION AND TOPOGRAPHY

The subject property is located at 11023 Lurline Avenue in the Chatsworth area of California. The subject property is a partially developed, irregular shape parcel situated along the west side of Lurline Avenue, approximately 1000 feet south of the 118 Freeway. Improvements to the northern portion of the site consists of a two-story single family residence detached garage, swimming pool and pool house. The detached garage is located directly to the east of the residence. The swimming pool is located to the south of the residence and the pool house is located southwest of the residence. The subject property slopes gently to the south from the northern property line. Access to the residence is via a paved driveway that extends from Lurline Avenue several hundred feet west to the structure. See enclosed Vicinity Map for location of subject property (see APPENDIX I).

Slope areas are covered with a sparse to moderate growth of weeds, scattered shrubs and moderate size eucalyptus trees. For specific topographic conditions, refer to the attached Geologic Map, Plate A and Geologic Cross-Sections, Plate B (see APPENDIX I).

PROPOSED DEVELOPMENT

It is our understanding that the proposed development will consist of constructing a high school on the subject property. Structures will consist of classroom and administrative buildings, auditorium, gymnasium and aguatics center and associates structures. The proposed structures will be one, two and three stories in height. Additionally, the majority of the southern portion of the subject property will include a subterranean parking level.

Retaining walls will necessary for the construction of the subterranean parking areas. Temporary cuts up to twelve (12') in height are anticipated during construction of walls. Grading will consist of excavating to establish the desired grade elevations, temporary cuts for the retaining walls and backfilling the retaining walls. In addition, foundation excavations will be made for the support of the proposed structures and retaining wall(s). All existing improvements on the site, including the single-family residence, would be removed to accommodate the proposed project. For reference, the locations of proposed structures are shown on the attached Site Plan, Plate A.

Anticipated structural loads will be on the order of two to five kips per lineal foot with column loads not exceeding 200 kips. These loads are based upon estimates and may need to be modified when construction plans are available. Final building plans have not been prepared and await the conclusions and recommendations of this investigation.

SUMMARY OF FINDINGS

Research

A representative from this office conducted research of available geotechnical engineering reports prepared for the subject property and adjacent properties at the City of Los Angeles on January 28, 2003 and January 30, 2003. In addition, research of available maps and publications prepared for the area was conducted. SubSurface Designs, Inc. has reviewed the referenced reports and has incorporated applicable information from these sources into this report.

It should be noted that other reports may have been prepared for the subject site in the past but were not found during records research, or were not submitted to the governing reviewing agency, and thus are not part of the public record.

Previous Studies

SubSurface Designs, Inc., conducted subsurface exploratory studies on the subject property in 2000. The purpose of the investigation was to evaluate subsurface conditions for the proposed construction of a classroom and a parking area on the subject site. Site exploration consisted of excavating, logging and sampling eight backhoe trenches. SubSurface Designs, Inc., presented findings, conclusions and recommendations in the referenced report dated December 11, 2000.

SubSurface Designs, Inc., conducted additional exploratory studies on subject property in 2003. The purpose of the investigation was to evaluate subsurface conditions for the proposed construction of a gymnasium structure and three parking areas on the subject site.

Site exploration consisted of excavating, included logging and sampling eight (8) hollow-stem auger borings that were drilled on the site. SubSurface Designs, Inc., presented findings, conclusions and recommendations in the referenced report. The proposed gymnasium was to be located within the south-central portion of the site. The parking areas were situated northeast, northwest and west of the proposed gymnasium. Retaining walls were planned for the construction of the parking and driveway areas. The proposed development was never initiated.

Field Investigation

A field investigation of the site was conducted on October 25, 2000, November 7, 2000 and January 27, 2003. This included logging and sampling eight (8) backhoe explorations and eight (8) hollow-stem auger borings that were drilled on the site. The backhoe explorations were excavated to a maximum depth of twelve feet (12') and the borings were excavated to a maximum depth of forty-one feet (41') below an adjacent grade. For reference, excavations previously reviewed by this firm are located on the attached Site Plan, Plate A (see APPENDIX I). Exploration logs of the subsurface excavations are appended herein (see (APPENDIX I).

The subsurface conditions encountered in these explorations were logged in detail by a representative of this office. Further, representative samples of the earth materials encountered were obtained.

Undisturbed samples were obtained within the test borings with a Modified California (M.C.) ring sampler (ASTM D 3550 with a shoe similar to ASTM D 1586. The M.C. sampler has a 3" outside diameter and a 2.37" inside diameter. The SPT sampler has a 2.00" outside diameter and a 1.37" inside diameter. The samples were obtained by driving the sampler with successive drops of the Kelly bar or 140 pound hammer dropping 30 inches in accordance with ASTM D 1586. The soil is retained in the brass rings of 2½" outside diameter and 1" in height.

Bulk samples were obtained for testing and analysis. All undisturbed and bulk samples were sent to the laboratory for examination, testing, and classification, using the Unified Classification system and group symbol.

Earth Materials

Earth fill materials were within the slope areas and in the areas of the proposed parking. These fills extend up to eight and one-half feet (8.5') as encountered with our explorations. Older alluvium was encountered beneath the earth fills and areas not affected by past grading to the depths explored

The earth materials encountered on the subject property are briefly described below. For approximate depths and more detailed descriptions, refer to the enclosed Exploration Logs Figure E.1 through E.8 (see APPENDIX I).

Earth Fill (ef)

The earth fill consists of a brown to dark brown, loosely to moderately compact, slightly moist to moist, silty sand (SM) to Clayey Sand (SC).

Older Alluvium (Qoa)

The referenced geologic map by Dibblee (1992) deposits the subject site and surrounding areas as being underlain by older alluvial deposits (Qoa) of Pleistocene geologic age. The map also depicts the level southern portion of the property as being underlain by younger alluvial deposits. However, for the purposes of this report, the alluvial deposits have not been differentiated.

The alluvial deposits observed in the test trenches consist of a light reddish-brown to yellowish-brown, moist, moderately dense, slightly silty sand (SM) and gravelly sand (SP). The gravel layers are generally discontinuous, subhorizontal and range from 12 to 18 inches thick.

The earth fill materials and alluvial deposits were visually classified in accordance with the Unified Soils Classification System. Earth material profiles can only be obtained from individual explorations placed on the subject property. Care should be exercised when using these profiles to determine changes in depth or thickness of the earth materials between the explorations.

Site Drainage & Groundwater

Drainage within the site comprises essentially of sheet flow runoff of precipitation derived primarily within property boundaries. Groundwater was not encountered to the maximum depth of the explorations. However, historic high groundwater data is available within the Seismic Hazard Evaluation of the Oat Mountain Quadrangle, Seismic Hazard Zone Report 005, dated 1997 (Revised 2001), prepared by the State of California Division of Mines and Geology. Plate 1.2 of the aforementioned report illustrates that the depth to the historic high groundwater level approximately ½ mile to the south is on the order of eighty feet (80') below the ground surface.

It must be noted that fluctuations in the level of the groundwater may occur. The depth to groundwater, if encountered in the explorations, is only valid for the date of exploration. Changes may occur in this groundwater level due to climatic conditions and/or alterations in the existing groundwater recharge area (i.e. changes in landscaping irrigation rates, surface drainage and surface water infiltration conditions).

SEISMIC EVALUATION

General

The subject property is not located within the confines of an "Earthquake Fault Zone" (Formerly the Alquist-Priolo Earthquake Fault Zoning Act). These are zones delineated by the California Division of Mines and Geology depicting active faults, within which special earthquake studies have to be accomplished prior to construction of habitable structures. Although the site is not located within a State designated "Earthquake Fault Zone" it is located in an active seismic region where large numbers of earthquakes occur each year.

For planning purposes, the City of Los Angeles Safety Element of the General Plan designates the subject site within Fault Rupture Study Area. These are zones approximately ½-mile wide that extend along identified active and potentially active faults. The subject site has been placed within the Fault Rupture Study Area due to its close proximity to the Northridge Hills Fault Zone.

The potential exists throughout Southern California for strong ground motion similar to that which occurred during the 1994 Northridge Earthquake. Earthquakes with a magnitude of 5.0 and greater have occurred in Southern California throughout historic time. Strong ground shaking from a moderate to major earthquake can be expected during the lifetime of the structure. This may result in significant damage to the structure, hardscape and adjacent slopes. Since there are so many variables associated with ground movement during an intense earthquake, it is almost impossible to predict the impact of a seismic event to a particular site.

A search of earthquake, fault, and seismic parameters and conditions relevant to the subject site utilizing the EQSEARCH, EQFAULT, and UBCSEIS computer programs may be found in APPENDIX IV.

Northridge Hills Fault Zone

The Northridge Hills Fault (NHF) is little-studied fault that trends southeast across the San Fernando Valley from the vicinity of Chatsworth, or more specifically the Santa Susana Pass, in the western Santa Susana Mountains.

The fault appears to become a series of imbricate faults in the vicinity of Northridge in the western San Fernando Valley and then turns nearly due east and disappears under thick alluvium in the east-central valley. Thus, the exact location of the fault is not known. In the northwest part of the valley, the fault is defined by discontinuous but aligned topographic expressions (Wentworth and Yerkes, 1971) in the form of low anticlinal hillocks. An apparent north-south trending surface trace of this or a related fault is present in the Cretaceous Chico formation north of the town of Chatsworth. This fault well may be associated with, or a part of, the fault system related to uplift of the Santa Susana Mountains.

The Northridge Hills fault is believed to be either more than one fault plane, or splintering faults which align and possibly blend with the fault complex in Santa Susana Pass and Simi Valley to the west. Near the town of Northridge, the Northridge Hills fault is buried beneath alluvium, and locations are interpreted from oil company data and from topographic patterns.

Subsurface data indicate Miocene (5-22.5± million years old) strata has been vertically displaced between 500 and 1000 feet with a corresponding variance in the thickness of the alluvium overlying the Miocene rock units. This condition strongly suggests that the Northridge Hills fault was subjected to post-Miocene fault slippage and seismic events with activity continuing during the Pleistocene (10,000 - 1.8 million years old). The fault has offset Saugus Formation beds of Quaternary age, yet there is no evidence suggesting that more recent beds of Holocene age (<11,000 years old) have been displaced nor is there any evidence of surface fault rupture. Therefore, the fault is not considered to be active.

Seismic data from Allen et al. (1971), Greensfelder (1972), Wentworth et al. (1973), and other observers indicate that this fault should be considered "potentially active". Seven significant aftershocks of the 1971 San Fernando earthquake of magnitude between 3.2 and 4.3 were plotted within 2 miles of this fault, with three aftershocks centered on the fault. Wentworth et al. (1973) state, "... Late Quaternary age for the fault is indicated by its displacement of terrace deposits along the Northridge Hills". The variance of hundreds of feet of thickness of Pleistocene and Holocene alluvium on opposite sides of the fault suggest intermittent seismic activity to at least late Pleistocene and possibly into Holocene.

Greensfelder (1972) postulated an estimated maximum earthquake magnitude of 7.5 for the Northridge Hills Fault and a maximum credible bedrock acceleration of 0.5g for the northern side of the San Fernando Valley.

In proposing this bedrock acceleration, consideration also was given to the location of the Oakridge fault, Santa Susana-San Fernando-Sierra Madre Fault zone and the San Andreas. Wentworth, et al. (1973) suggested a possible 6.5 magnitude earthquake on the Northridge Hills Fault and lists other authors as giving a range from 5.8 to 6.8. Wentworth et al. (1973) assigned an estimated maximum acceleration of .42g for the Northridge Hills Fault. Considering the variety of interpretations of the Northridge Hills Fault: (1) as to whether it is a single fault rupture (State Water Rights Board, 1962) or a multiple plane fault, and (2) as to its length of 12 miles (Wentworth et al., 1973) to 30 ± miles (Greensfelder, 1972); the fault appears to be capable of producing a seismic event of magnitude 6.5 or greater with a probable upper limit of 7.5.

The inferred maximum bedrock acceleration utilizing data produced by Greensfelder (1972) and Wentworth et al. (1973) ranges from .42g to .5g. Wentworth et al. (1973) suggested a possible maximum acceleration of 0.49g attributed to the Burro Flats Reservoir fault in the Chatsworth area, which would thus tabulate a range of 0.49 to 0.5g for the northern San Fernando Valley.

Recent Seismic Activity

The most recent largest earthquake within the specified search radius and time period is the Northridge Earthquake. The 6.7 magnitude Northridge Earthquake occurred on January 17, 1994 at 4:31 a.m., PST, and created strong ground shaking for approximately 10 seconds in the Los Angeles area resulting in wide spread, random damage. The Northridge Earthquake did not occur on the Northridge Hills Fault (NHF). The Northridge Earthquake occurred along a previously unrecognized south dipping thrust fault known as the Frew Fault. The causative fault, as defined by a pattern of aftershocks, moved under an area roughly 19-miles across its front (approximately east-west in orientation) and 13 miles from front to back (approximately north-south in orientation). Although slip magnitude was approximately 9- to 10-feet, surface rupture along the causative fault did not occur as a result of the earthquake.) The epicenter of the Northridge Earthquake is located approximately 4.8 miles east of the subject property. The earthquake produced an estimated repeatable high ground acceleration of 0.466g on the subject property.

Active Faults within Close Proximity to the Site

Malibu Coast Fault Zone

The Malibu Coast Fault, which is considered a "potentially active" to "active" earthquake fault, is situated approximately 15 miles south of the subject site and has a maximum probable event magnitude of 6.9. The Malibu Coast Fault Zone (MCFZ) parallels the coast and forms a zone of folding and complex faulting varying in width up to approximately 2300 feet. The MCFZ extends in an east-west direction for approximately 48 miles along the Malibu coastline. The MCFZ is the western extension of the Hollywood-Santa Monica Fault. The zone may be as wide as 1 mile and dips to the north between 30° and 70°.

Late Quaternary (100,000± years ago) fault breaks have been noted by Yerkes and Wentworth (1964) in relation to a 2-foot offset of Upper Pleistocene (100,000± years) terrace deposits along a portion of the Malibu Coast Fault.

A Corral Canyon marine terrace deposit was also reportedly offset 17 vertical feet along a complex north-dipping thrust fault. Other offsets of a Corral Canyon marine terrace have also been noted by John Merrill and Birkeland. The foundation excavation easterly of Malibu Canyon exposed Monterey shale thrust over a Corral Canyon marine terrace deposit, a condition indicating relatively recent movement. Faulting associated with the corral terraces is common in the Malibu area. Fresh scarps offset terrace deposits and are dated between 131,000 ±15,000 and 104,000 ±5,000 years before present. The faulting observed may have occurred between 125,000 and 15,000 years before present. Recent fault movement within the past 2,000 years is suggested by the exposure west of Trancas Canyon.

Seismicity studies in 1987 associated with the proposed General Motors Corporation design center near Pepperdine University indicate that strands of the Malibu Coast Fault trend through the terraced area. The fault has been dated at this location as approximately 6,000 years and therefore, is considered to be active.

San Gabriel Fault

The San Gabriel Fault zone transects the northeastern part of the Ventura basin and can be traced from the Frazier Mountains area, about 30 miles northwest of Saugus, to the eastern part of the San Gabriel Mountains, a distance of about 90 miles. The San Gabriel Fault Zone is located approximately 11 miles to the northeast and has a maximum probable event magnitude of 7.0. The San Gabriel fault is a high-angle right-lateral strike-slip fault, which extends about 46 miles north westward across Los Angeles County. The western half consists of a single fault. The eastern half consists of two branches that split near Big Tujunga Ranger Station. This fault has about 37 miles of right-lateral displacement, based on exposures of several rock units that have been offset. Some evidence of Holocene displacement has been noted in a few trenches across the trace of this fault, only between Saugus and Castaic. Thus, the San Gabriel Fault is "active" and is certainly capable of producing a large magnitude earthquake. The slip rate is on the order of 1mm/yr to 5 mm/yr. Slip rate and recurrence interval probably vary significantly along the length of the San Gabriel fault zone.

Santa Monica-Hollywood Fault

The Hollywood fault is the eastern 1-4 kilometer long segment of the Santa Monica-Hollywood fault system that forms the southern margin of the Santa Monica Mountains. The fault traverses the cities of Beverly Hills, West Hollywood and Hollywood, where the Santa Monica Mountains are referred to as the Hollywood Hills. Movement on the Hollywood fault over geologic time is thought to be responsible for the growth of the mountains which is why earlier researchers characterized the Santa Monica-Hollywood fault system as primarily a northward-dipping reverse fault. However, recent studies by Dolan et al. (1997, 2000) have shown that this fault is primarily a strike-slip rather than a reverse fault. The strong lateral component of movement on this fault is indicated, for example by its linear trace and steep, 80- to 90- degree dips (reverse faults typically have irregular, arcuate traces and shallow dips). Since other west-trending faults in the Transverse Ranges exhibit a left-lateral component of movement, lateral slip on the Hollywood fault is most likely also left-lateral.

The Sant Monica-Hollywood fault system has not produced any damaging historical earthquakes. Subsurface study of the main fault by Dolan et al. (2000) suggests that this fault moves infrequently. One, and possibly two surface-rupturing earthquakes on this fault were interpreted to have occurred in the last about 10,000 to 22,000 years ago, with the most recent earthquake occurring most likely between 7,000 and 9,500 years ago (Dolan et al., 2000). These data suggest that the fault either has a slow rate of slip or that it breaks in large-magnitude events. Based on its length, the Hollywood fault is thought capable of generating a magnitude 6.6 earthquake. However, if it breaks together with the Santa Monica fault to the west, larger magnitude earthquakes can be expected. Given the extensive urbanization of this portion of the Los Angeles Basin, an earthquake in this fault has the potential to be economically and socially devastating to southern California.

In the early geologic maps of the area (Hoots, 1931), the Hollywood fault is shown as a buried structure along the sharp break in slope between the Santa Monica Mountains (i.e., the Hollywood Hills), to the north, and the Los Angeles basin to the south. Hoots did not observe the fault, but surmised that it was buried by the alluvial deposits mantling the valley floor. Hoots (1931) shows the inferred traces of the fault north of the subject site. This interpretation changed very little over the next 60 years. Dibblee (1991) also shows the fault near the base of the mountains along a similar alignment as Hoots (1931).

However, in the last few years, the location of the Hollywood fault has been reevaluated in part as a result of Dolan et al's work (1992, 1997, 2000), and recent fault studies conducted by various consultants in the cities of the Los Angeles and West Hollywood. Dolan and his co-authors (1992, 1997) looked at 5-foot contour topographic maps of the area prepared by the U.S. Geological Survey in the mid-1920's. Although substantial portions of the cities of Hollywood, West Hollywood, and Beverly Hills had already been developed by the mid-1920', much of this development was conducted without extensive modification to the existing landscape (unlike more recent developments where mechanized grading is used to significantly to alter the natural ground surface). As a result, fault scarps and other topographic features are locally preserved beneath the pavement in these cities, In some areas, the location of the fault is expressed at the surface by a series of linear scarps and faceted south-facing ridge. In other areas, the location of the fault cannot be based on topography alone because the scarps are broad features more than 50 to 200 feet wide. However, by combining these geomorphic data with subsurface data obtained from geotechnical and groundwater studies, Dolan et al. (1997) attempted to better constrain the location of the major traces of the Hollywood fault.

In the Beverly Hills area and western portion of West Hollywood, west of Laurel Canyon, the Hollywood fault trends N55-65E, while in the Hollywood area, it trends approximately N80E (Dolan et al., 1997). Based on borehole and trenching data, the main fault is known to dip steeply to the north (Dolan et al., 1997). Secondary, south-dipping faults (some with possible normal dip-slip motion) in the hanging wall, north of the main fault, have been exposed locally (WLA, 1998; Dolan et al., 1997).

In 1997, Dolan et al. (1997) inferred two traces of the fault in the immediate vicinity of the site. The first of these inferred traces was located north of Sunset Boulevard, near the contact between the bedrock and the alluvium, where there is a sharp break in the slope. This is approximately the same area where both Hoots (1931) and Dibblee (1991) placed the Hollywood fault.

The second fault trace inferred by Dolan et al., (1997), based primarily on groundwater data, was located approximately 500 to 700 feet south of the Sunset Boulevard and Doheny Drive intersection (and within 100 feet south of Sunset Boulevard - Horn Avenue intersection). This southern fault is interpreted as the main strand of the Hollywood fault in this area. In a more recent evaluation of the Hollywood fault (Dolan et at., 2000), the northern inferred trace is not shown. This reflects the results of several recent investigations along Sunset Boulevard, between Doheny Drive and La Cienega Boulevard (Harza, 1998; WLA 1998a, 1998b 1999; ECI, 1999a 1999b) that indicate that the Hollywood fault is not located at the base of the bedrock outcrops along Sunset Boulevard, but rather to the south.

These studies have also demonstrated that the Hollywood fault is a complex zone of faulting that includes several inactive, moderately to steeply dipping secondary faults that impact the older alluvial apron but do not extend upward into the younger sediments. In addition to these inactive north dipping structures, the fault zone locally includes south-dipping secondary normal faults within the hanging wall that may still be active. Although these types of faults have been identified within the Hollywood Fault Zone, their location and extent have yet to be resolved.

Raymond Hill Fault

The Raymond Hill Fault forms the boundary between the Raymond (ground water) basin and the San Gabriel Valley. The fault trends generally in an east-west direction through an intensely urbanized area. Urbanization has altered most of the geologic features, making it difficult to assess the degree of hazard posed by the fault. The Raymond Hill Fault Zone is located approximately 23 miles to the southeast and has a maximum probable event magnitude of 6.5.

Active Faults with Historic Surface Rupture

San Andreas Fault

The 1857 Fort Tejon Earthquake (8.3±M) occurred along a portion of the San Andreas Fault Zone north of San Bernardino. At that time, the fault moved laterally approximately 18 to 30 feet. The San Andreas Fault extends in a northwest-southeast direction for over 600 miles through California. The main trace of the fault is situated approximately 29 miles north of the subject property. Numerous magnitude 7.0± earthquakes have occurred at the rate of approximately one every ten years along the fault system south of San Bernardino. Recurrence rates along this fault indicate that a large magnitude event, similar to the 1857 Fort Tejon Earthquake, occurs on the average of every 105 to 132 years. A large magnitude earthquake along the San Andreas Fault System is anticipated to have a magnitude on the order of 8± and is anticipated to result in one to three minutes of ground shaking.

Sierra Madre Fault Zone

The 1971 San Fernando Earthquake (6.4M) occurred along the San Fernando Fault Zone (SFFZ). The SFFZ is the western extension of the Sierra Madre Fault Zone located further to the east. The site is situated approximately 7.5 miles southwest of the SFFZ which has a maximum probable event magnitude of 7.0. The Sierra Madre Fault is considered to be the nearest "active fault", as it is the closest known fault to have known surface rupture during Holocene time (the last 11,000 years). Although the epicenter of the Northridge Earthquake was closer to the subject site, the earthquake occurred along the Frew Fault, which did not have surface fault rupture.

The Sierra Madre Fault Zone consists of a fault complex, which is located along the southeasterly margin of the transverse ranges province. The complex extends approximately 75 miles along the southern front of the San Gabriel Mountains from Cajon Pass to San Fernando and along a portion of the Santa Susana Mountains.

This province is characterized by west-trending structural features, unlike the majority of southern California which is dominated by northwest trending features. The steep southern face of the San Gabriel Mountains was recognized as a fault scarp as early as 1905. The fault system was given a name, "Sierra Madre Fault", in 1924 by Kew. At that time, the fault was thought to be a normal fault, similar to the fault which parallels the eastern flank of the Sierra Nevada (the Owens Valley Fault). It was not until 1930 that workers demonstrated that the fault was, in fact, a reverse or thrust fault. Tunneling operations conducted in conjunction with studies by the Metropolitan Water District of Southern California in 1970 exposed a low angle thrust fault with granitic bedrock thrust over alluvial materials.

The 1971 San Fernando earthquake demonstrated the activity of the western portion of the fault system. Since 1971, several investigations along the southern flank of the San Gabriel mountains have exposed granitic bedrock thrust over alluvium within the Sierra Madre Fault Zone, which demonstrates Quaternary thrust faulting (fault activity which has displayed movement within the last 2 to 3 million years). Historic activity on the Sierra Madre Fault system has been limited to the 1971 San Fernando earthquake on the west and the recently noted micro-seismic activity on the eastern portion. Both of these segments of the fault zone have been included within an Earthquake Fault Zone. The central portion of the fault system has not been included within an Earthquake Fault Zone. Recent exploration on the Sierra Madre Fault indicates that the fault is detectable and may be identified by direct observation and subsurface exploration as Holocene (less than 11,000 years old). Surface displacement is evident along portions of the fault. Therefore, it appears that the State Geologist requirements for including the fault in an Earthquake Fault Zone are met for the majority of the Sierra Madre Fault system.

Activity of the Sierra Madre Fault Zone has been a subject of controversy since the San Fernando earthquake of 1971. The non-inclusion of the majority of the fault zone within an Earthquake Fault Zone and the controversy over its activity has led to a wide variation of geotechnical investigations on properties near the fault zone. Whether the Sierra Madre Fault is "potentially active" or "active", as recent work tends to reflect, it should be recognized as a potential hazard.

Newport-Inglewood Structural Zone

The 1933 Long Beach Earthquake (6.3M) occurred along the Newport-Inglewood Fault Zone. The Newport-Inglewood Fault is located approximately 21 miles to the southeast and has a maximum probable event magnitude of 4.20. The Newport-Inglewood Structural Zone, located 21 miles south of the subject site, is one of several large predominantly right-lateral strike-slip fault zones that parallel the San Andreas Fault in southern California. The Newport-Inglewood Zone of deformation has been intensely investigated in the subsurface since the early 1920's by the petroleum industry which referred to it as the "Newport-Inglewood Uplift." Faults of the Newport-Inglewood Fault Zone are predominantly defined in the subsurface from oil well data and groundwater data. Very little geologic evidence of surface faulting has been found within the zone and very few instances of documentation of surface faulting exist. Even following the 1933 Long Beach Earthquake, which had a Richter magnitude of 6.3, no evidence of surface faulting was found or reported.

However, recent work along the north branch of the Newport-Inglewood Zone in the west Newport Mesa suggests that recent geologic units have been offset within the man-made fill, a condition indicating that surface faulting occurred very recently, probably during the 1933 earthquake. Therefore, the Newport-Inglewood Structural Zone is classified as active and appears to be capable of creating a maximum probable event magnitude of 6.9.

Horizontal Ground Acceleration

Predicted maximum credible and probable event magnitudes, credible and repeatable high ground accelerations, and fault distances from the site have been estimated by using a method developed by Campbell & Bozorgnia (1994). Mean plus one standard deviation values have been determined for each known active fault zone within a 50 mile radius of the subject property. By definition, a mean plus one standard deviation" value has an 84 percent probability of non-exceedance. For additional information refer to the attached results obtained from EQFAULT, APPENDIX IV.

The program EQSEARCH was used to determine all historical earthquakes with event magnitudes ranging from 4.0 to 9.0 on the Richter Scale within a 50 mile search radius of the subject property over the past 100 years. EQSEARCH effectively performs searches of a historical-earthquake catalog using an abbreviated (M=4.0 and above, and latitude from 30.0 to 36.5) and supplemented Southern California version of the California Division of Mines and Geology computerized earthquake catalog for the State of California. Search parameters (i.e., geographic limits, limiting dates, and limiting magnitudes) are specified and the user selects one of 14 available acceleration-attenuation relations. Site specific peak-horizontal-acceleration probability of exceedance is also estimated from the historical search. For each historical earthquake in the search area, EQSEARCH prints latitude, longitude, date, depth and Richter magnitude. EQSEARCH computes site-acceleration, site-Modified-Mercalli Intensity, and approximate earthquake-to-site distance in both miles and kilometers.

Strong motion records were recovered from a total of 193 stations of the Strong Motion Instrumentation Program (SMIP) after the Northridge Earthquake. These stations include more than 250 ground-response stations, 400 buildings, and 50 other structures.

The epicentral distance of the stations ranges from 5km for the closet (Tarzana) to about 270km for the farthest Imperial County) There were no ground response recording stations within close proximity to the subject property. The maximum horizontal ground acceleration associated with this event was 1.8g. This acceleration was recorded at the Tarzana Nursery, located 5km south of the epicenter (approx. 13km southeast of the subject site).

UBC Seismic Coefficients

California Building Code designs and City of Los Angeles amendments where applicable are intended to accommodate horizontal accelerations up to 0.4g in Seismic Zone 4. Since the project is located within this zone, the structure(s) should, at a minimum, be designed to accommodate this acceleration. However, the project Structural Engineer should be made aware of the maximum acceleration value for the "mean plus one standard deviation" condition to determine if any additional structural strengthening is warranted.

It should be noted that the Structural Engineer often utilizes the repeatable high ground acceleration (RHGA) values in their analysis, which is approximately equal to 65 percent of the maximum probable accelerations. The RHGA values for each nearby known active fault zone are delineated on the attached results obtained from EQFAULT in APPENDIX IV.

The following values were obtained from the program UBCSEIS for Seismic Zone 4 (Z = 0.4), Seismic Source Type B and Soil Profile Type SD:

UBC Seismic Coefficients					
Na:	1.0	**Ca:**	0.44	**Ts:**	0.694
Nv:	1.2	**Cv:**	0.76	**To:**	0.139

Seismic Hazards Maps

This office has reviewed the Seismic Hazards Map of the Oat Mountain Quadrangle prepared by the California Division of Mines and Geology. The purpose of this map is to delineate areas that may be subject to liquefaction and/or landsliding during a strong seismic event.

As defined, a liquefaction area is an area where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693© would be required. As defined, an Earthquake-Induced Landslide area is an area where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693© would be required. According to the Seismic Hazards Map of the Oat Mountain Quadrangle, the site is not located within an area of study for earthquake-induced liquefaction or within an area subject to earthquake-induced landsliding.

Inundation and Tsunamis

Exhibit G of the Safety Element of the Los Angeles City General Plan dated November 26, 1996 illustrates the potential areas that may be affected by inundation and/or tsunamis. According to Exhibit G, the site is approximately four miles away from the area of potential inundation from the Los Angeles Reservoir. Due to the location of the property relative to coastal areas, the subject property would not be impacted by potential tsunamis. The west portion of the subject property in the area of the proposed parking lot is located directly to the south of a buried tank. It is our understanding that the tank is under ownership and maintenance of the Department of Water and Power. As the tank is buried to nearly its full height, inundation of the subject property from the tank is considered remote. However, an evaluation of the potential of water over-toping the tank or tank rupture is beyond the scope of this investigation.

CONCLUSIONS

General

It is the professional opinion of this office that construction of the proposed gymnasium and parking areas is feasible provided that the recommendations contained herein are followed. In addition, all applicable elements of the City of Los Angeles Building Codes shall be followed.

Specific recommendations were provided in the referenced report by this office dated December 11, 2000, for the construction of a classroom building on the northern portion of the subject property. The recommendations provided in that report relating to grading and construction remain unchanged. However, the discussion in this report relating to seismicity are more specific and supercede the discussions previously presented.

The site displayed no evidence of geologic instability during reconnaissance and mapping of the subject property. Review of geologic maps indicate no known landslide structures within or immediately adjacent to the subject property. Further, stereoscopic examination of aerial photographs indicates no topographic anomalies suggesting the site has been affected by landslide activity.

Based upon our field observations, laboratory testing and analysis, the older alluvium found in the explorations should possess sufficient strength to support the proposed gymnasium and retaining walls. The parking areas are anticipated to be supported by either compacted fill or older alluvium. The artificial fill, encountered in the explorations excavated on site are not considered suitable for foundation support as these materials may posses adverse deformational characteristics.

Expansive Soils

Soil index tests have been performed in order to determine the potential expansiveness of the supporting earth material. The results of the testing are presented in Table I-1 of APPENDIX II. Based upon a review of the earth materials encountered within the exploratory openings, laboratory testing and research the soils are considered to possess a low potential for expansion.

Excavation Characteristics

Subsurface exploration was performed through the use of a backhoe and a drill rig excavating into the underlying soils. Excavating into the underlying earth materials during construction should be possible with conventional excavating equipment.

RECOMMENDATIONS

The proposed development will include the construction of a high school on the subject property. Structures will consist of classroom and administrative buildings, auditorium, gymnasium and aguatics center and associates structures. The proposed structures will be one, two and three stories in height. Additionally, the majority of the southern portion of the subject property will include a subterranean parking level. Retaining walls will necessary for the construction of the subterranean parking areas. To achieve the proposed grades for the development typical cut fill grading will be performed. The foundations and slabs-on-grade for most of the proposed structure are anticipated to be supported by the older alluvium.

Portions of the subject property are underlain by a 54" diameter Metropolitan Water District (MWD) water line. It is our understanding that portions of this pipe are relatively shallow across the site. Prior to certification of the project grading plan, the exact location and depth of the existing water line beneath the proposed construction areas shall be identified to the satisfaction of MWD.

1. All existing earth fill encountered on site shall either be removed, or be removed and replaced as certified compacted fill. It is anticipated that the proposed swimming pool will be supported over a cut/fill transition. It is recommended that the proposed swimming pool be supported on a relatively uniform blanket of certified compacted fill. The compacted fill shall extend a minimum of four feet (4') below the bottom of the pool. Grading shall be carried fourth as described in the GRADING AND EARTHWORK section below.

2. The proposed structures associated with the development shall be supported by foundations extending into the underlying older alluvium. Based upon the current plan conventional foundations are anticipated. In some cases deepened foundations may be required. Foundations should be designed as outlined in the FOUNDATIONS section below.

3. The portions of the proposed structures that span the MWD water line may need to be supported by pile foundations deriving support from the underlying alluvium. The need for pile foundations shall be determined by the project structural engineer to the satisfaction of MWD. Pile foundation need for support of structures shall be designed as outlined in the FOUNDATIONS section below.

4. Retaining walls up to twelve feet in height will be required for the proposed subterranean parking areas. Retaining walls shall be designed and backfilled as outlined in the RETAINING WALLS section below.

5. The proposed swimming pool shall be designed as a free standing pool deriving support from the proposed certified compacted fill. The pool walls shall be designed to support the water without bearing from the adjacent soil. The pool walls shall be designed to support the backfill or existing soil when the pool is empty, addition to any surcharge. The proposed swimming pool shall be equipped with a hydrostatic relief valve to reduce the potential for the buildup of hydrostatic pressures.

6. Slabs-on-grade shall be designed as outlined in the FLOOR SLABS section below.

   Prior to the placement of concrete slabs, the expansive soils encountered on the subject property shall be pre-moistened until the moisture content reaches at least 120% of the optimum moisture content to a depth of twelve inches (12"). The pre-moistened soils should be tested, and verified to be 120% of optimum moisture content, prior to the placement of the sub-grade.

   Following our testing and verification of moisture content, the sub-grade, polyethylene plastic, and sand **must** be placed within one day.

7. Preliminary design recommendations for pavement is presented in the PAVING FOR PRIVATE DRIVEWAYS AND PARKING AREAS section below.

8. The site shall be maintained as outlined in the DRAINAGE AND MAINTENANCE section below.

It should be noted that, the recommendations contained within this report may be more restrictive than applicable building codes. All recommendations of this report which are in addition to or more restrictive than those outlined in a subsequent review letter, by your governing reviewing agency, shall be incorporated into the plans.

GRADING AND EARTHWORK

Proposed Grading

Proposed grading will consist of excavating to establish the desired elevations, temporary cuts for the retaining walls, backfilling the retaining walls, as well as the removal and recompacted of the existing earth material in the area of the proposed swimming poll and driveways.

In addition, foundation excavations will be made for the support of the proposed gymnasium and retaining wall(s). All grading shall be carried forth as outlined herein. See RETAINING WALLS section below for Wall Backfill specifications.

Hillside

1. Prior to commencement of work, a pregrading meeting shall be held. Participants at this meeting will be the contractor, the owner or his representative, and the soils engineer. The purpose of this meeting is to avoid any misunderstanding of any recommendations set forth in this report that could cause delays in the project.

2. Prior to the commencement of grading a surveyor should be retained to layout the proposed grading. This should, as a minimum, consist of locating all proposed keys, tops of cuts, toe of fills, stability fills, setbacks, easements and areas requiring over excavation of the cut portions of any building pads. All staking shall be setback from the proposed grading area at least five feet (5').

3. Sidehill fills should have a key placed at the toe of the proposed fill slope. This key should be cut a minimum of three feet into the older alluvium. The base of the key shall be sloped back into the hill. The key should be a minimum of twelve feet wide. Where slopes are steeper than 5:1 (5 horizontal to 1 vertical), horizontal benches shall be cut into older alluvium in order to provide both lateral and vertical stability.

4. Sidehill fills shall have backdrains installed at the compacted fill/older alluvium contact to prevent future porewater pressure buildup. Backdrains shall be placed in accordance with the BACKDRAINS section below.

5. All areas to receive compacted fill, including all removal areas, keys, and benches, shall be reviewed and approved by the soils engineer or his representative prior to placing compacted fill.

6. The grade that is determined to be satisfactory for the support of the filled ground shall then be scarified to a depth of at least six inches (6") and moistened as required. The scarified ground should be compacted to at least 90 percent of the maximum laboratory density.

   Materials excavated uphill from where fills are to be placed, shall not be cast over the slope into the fill area. Materials shall be channeled down a ramp to the area to receive compacted fill and then spread in horizontal layers. As compacted fills are placed, this ramp will be trimmed out to expose the dense, tight materials approved by the soils engineer. The minimum vertical height of bench in approved materials shall be three feet (3'). This will maintain the proper benching, as fill is placed up the slope. The ramp will be shifted periodically during the grading operations to allow for complete removal of the loose fill materials and for the proper benching.

7. The fill soils shall consist of select materials approved by the project soils engineer or his representative. These materials may be obtained from the excavation areas and any other approved sources, and by blending soils from one or more sources. The material used shall be free from organic vegetable matter and other deleterious substances, and shall not contain rocks greater than eight inches (8") in diameter nor of a quantity sufficient to make compaction difficult.

8. The suitable fill material shall be placed in approximately level layers six inches (6") thick, and moistened as required. Each layer shall be thoroughly mixed to ensure uniformity of moisture in each layer.

   When the moisture content of the fill is three percent (3%) or more below the optimum moisture content, as specified by the soils engineer, water shall be added and thoroughly mixed in until the moisture content is within three percent (3%) of the optimum moisture content.

   When the moisture content of the fill is three percent (3%) or more above the optimum moisture content, as specified by the soils engineer, the fill material shall be aerated by scarifying or shall be blended with additional materials and thoroughly mixed until the moisture content is within three percent (3%) or less of the optimum moisture content. Each layer shall be compacted to 90 percent of the maximum density, as determined by the latest version of ASTM D 1557, using approved compaction equipment.

9. Review of the fill placement should be provided by the soils engineer or his representative during the progress of grading. In general, density tests will be made at intervals not exceeding two feet (2') of fill height or every 500 cubic yards of fill placed.

10. The contractor shall be required to obtain a minimum compaction of 90 percent out to the finish face of 2:1 fill slopes. Compaction on slopes may be achieved by over building the slope and cutting back to the compacted core or by direct compaction of the slope face with suitable equipment. Direct compaction on the slope faces shall be accomplished by back-rolling the slopes in three foot (3') to four foot (4') increments of elevation gain.

11. The on-site materials may experience a shrinkage of five percent (5%).

12. During the inclement part of the year, or during periods when rain is threatening, all fill that has been spread and awaits compaction shall be compacted before stopping work for the day or before stopping because of inclement weather. These fills, once compacted, shall have the surfaces sloped to drain to an area where water can be removed. Work may start again, after the rainy period, once the site has been reviewed by the soils engineer and he has given his authorization to resume. Loose materials not compacted prior to the rain shall be removed and aerated so that the moisture content of these fills will be within three percent (3%) of the optimum moisture content. Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if deemed necessary by the soils engineer.

Backdrains

To minimize the potential for future porewater pressure buildup behind the proposed compacted fill backdrains shall be installed at the compacted fill/older alluvium contact. Backdrains shall consist of four inch (4") perforated pipes; placed with perforations down. The pipe should be encased with at least one foot (1') of gravel around the pipe. The minimum cover on the pipe should be one foot (1'). The gravel should consist of three-quarter inch (3/4") to one inch (1") crushed rock. The first drain shall be placed no higher than three feet (3') above the front cut of the key excavation. Additional backdrains shall be placed at intervals roughly equivalent to five feet of vertical rise in elevation or where deemed necessary by the project soils engineer.

Each drain shall be placed into a trench excavated along the back of a horizontal bench at the compacted fill/older alluvium contact. The trench bottom shall slope downward to each exit drain with a minimum gradient of two percent (2%). The exit pipe shall consist of a four-inch (4") diameter non-perforated pipe. This pipe need not be encased in gravel. It shall exit at a minimum gradient of two percent (2%) to the finish face of the fill slope. Exist drains shall be placed at intervals not exceeding one hundred feet (100'). A cutoff wall consisting of concrete or soil cement shall be placed at the junction of the perforated pipe and the exit drains to stop seepage and force the water being removed into the perforated pipe.

Flatland

1. Prior to commencement of work, a pre-grading meeting shall be held. Participants at this meeting will consist of the contractor, the owner or his representative, and the soils engineer. The purpose of this meeting is to avoid misunderstanding of the recommendations set forth in this report that might cause delays in the project.

2. Prior to placement of fill, all vegetation, rubbish, and other deleterious material should be disposed of off site. The proposed structures should be staked out in the field by a surveyor. This staking should, as a minimum, include areas for over-excavation, toes of slopes, tops of cuts, setbacks, and easements. All staking shall be offset from the proposed grading area at least five feet (5').

   The proposed construction areas should be excavated down to the older alluvium.

3. The natural ground, which is determined to be satisfactory for the support of the filled ground, shall then be scarified to a depth of at least six inches (6") and moistened as required. The scarified ground should be compacted to at least 90 percent of the maximum laboratory density.

4. The fill soils shall consist of materials approved by the project Soils Engineer or his representative. These materials may be obtained from the excavation areas and any other approved sources, and by blending soils from one or more source. The material used shall be free from organic vegetable matter and other deleterious substances, and shall not contain rocks greater than eight inches (8") in diameter nor of a quantity sufficient to make compaction difficult.

5. The approved fill material shall be placed in approximately level layers six inches (6") thick, and moistened as required. Each layer shall be thoroughly mixed to attain uniformity of moisture in each layer.

When the moisture content of the fill is three percent (3%) or more below the optimum moisture content, as specified by the Soils Engineer, water shall be added and thoroughly mixed in until the moisture content is within three percent (3%) of the optimum moisture content.

When the moisture content of the fill is three percent (3%) or more above the optimum moisture content as specified by the Soils Engineer, the fill material shall be aerated by scarifying or shall be blended with additional materials and thoroughly mixed until the moisture content is within three percent (3%) or less of the optimum moisture content.

Each layer shall be compacted to 90 percent of the maximum density as determined by the latest version of ASTM D 1557, using approved compaction equipment.

6. Review of the fill placement should be provided by the Soils Engineer or his representative during the progress of grading. In general, density tests will be made at intervals not exceeding two feet (2') of fill height or every 500 cubic yards of fill placed.

7. The on-site materials can experience a shrinkage of five percent (5%).

8. During the inclement part of the year, or during periods when rain is threatening, all fill that has been spread and awaits compaction shall be compacted before stopping work for the day or before stopping because of inclement weather. These fills, once compacted, shall have the surfaces sloped to drain to one area where water may be removed.

Work may start again, after the rainy period, once the site has been reviewed by the Soils Engineer and he has given his authorization to resume. Loose materials not compacted prior to the rain shall be removed and aerated so that the moisture content of these fills will be within three percent (3%) of the optimum moisture content.

Surface materials previously compacted before the rain, shall be scarified, brought to the proper moisture content, and re-compacted prior to placing additional fill, if deemed necessary by the Soils Engineer.

FOUNDATIONS

It is recommended that the proposed structures and retaining walls be supported by foundations extending into the older alluvium. All earth materials derived from the excavations of foundations shall be removed from the site or placed as certified compacted fill. Fill temporarily stockpiled on site should be placed in a stable area, away from slopes, excavations and improvements. Earth materials shall not be cast over any descending slopes in an uncontrolled manner.

Conventional

The minimum continuous footing size is twelve inches (12") wide for one story structures and fifteen inches (15") wide for two story structures. Pad foundations shall be a minimum of twenty-four inches (24") square. All depths of embedment for footings are to be measured from the lowest adjacent grade or into the specified bearing material.

Foundation Design Values					
Foundation Type	Bearing Material	Embedment Depth	Bearing Value (psf.)	Coefficient of Friction	Passive Resistance (pcf.)
Continuous	Older Alluvium	24"	2000	0.38	300
Pad	Older Alluvium	24"	2000	0.38	300

The depths specified in the above table are minimum embedment depths required by this office. However, the project structural engineer may need to make the depths deeper to accommodate specific structural loads. The bearing values given above are net bearing values; the weight of concrete below grade may be neglected. These bearing values may be increased by one-third (⅓) for temporary loads, such as wind and seismic forces.

Based upon past experience, all continuous footings shall be reinforced with a minimum of four #4 bars, two placed near the top and two near the bottom. Reinforcing recommendations are minimums and may be revised by the structural engineer.

Lateral loads may be resisted by friction at the base of the foundations and by passive resistance within the older alluvium. The coefficient of friction shall be used between the base of the foundation and the recommended bearing material. When combining passive and friction for resistance of lateral loads, the passive component should be reduced by one-third. For isolated poles, the allowable passive earth pressure may be doubled.

All footing excavation depths will be measured from the lowest adjacent grade of recommended bearing material. Footing depths will not be measured from any proposed elevations or grades. Any foundation excavations that are not the recommended depth into the recommended bearing materials will not be acceptable to this office.

Friction Piles

The minimum friction piles diameter is twenty four inches (24"). All friction piles should extend into the alluvium a minimum of five feet (5'). The friction piles may be proportioned using a skin friction value of 550 pounds per square foot of shaft exposed to the alluvium. Further, friction piles shall be considered fixed at an embedment depth of two feet (2') into the recommended bearing material.

All friction piles should be tied in both horizontal directions with grade beams. The City of Los Angeles Department of Building and Safety requires continuous inspection of all friction piles excavations.

Lateral Load Design (Friction Piles)

Lateral loads may be resisted by passive resistance within the alluvium. The passive resistance may be assumed to act as a fluid with a density of 350 pounds per cubic foot. A maximum passive earth pressure of 4000 pounds per square foot may be assumed. For isolated poles, the allowable passive earth pressure may be doubled.

SETTLEMENT

Settlement of the proposed structures will occur. Typical settlements of ½" to ¾" between walls, within 20 feet or less of each other and under similar loading conditions, are considered normal. Total settlement on the order of ¾" should be anticipated. Differential settlement of the proposed structures is not expected to exceed ¾".

Future settlement of the structure due to long term deformation and natural occurrences are still possible. However, any site drainage improvements, such as those outlined in the DRAINAGE AND MAINTENANCE section below, will result in a lower risk of future structural hazards.

FLOOR SLABS

Floor slabs should be reinforced with minimum #3 reinforcing bars, placed at sixteen inches (16") on center each way. Floor slabs may be supported directly on the older alluvium or certified compacted fill. Although precautions can be taken, the recommendations are not intended to stop movement, only to reduce cracking as a result of expansion and contraction of the soil.

For crack control in secondary concrete slabs, the maximum control joint spacing should be eight feet (8'). A closer control joint spacing would provide greater crack control. Additional control joints at curves and angle points are recommended.

Where there are floors which may be affected by moisture, they should be protected by a polyethylene plastic vapor retarder. This retarder should be covered with a one inch (1") layer of sand to prevent punctures in the vapor retarder and to aid in the cure of the concrete. It should be noted that this type of barrier will not preclude moisture damage to wood floors or vapor sensitive flooring. Further, if this type of vapor retarder is used, the minimum thickness should be 10 mil.

Prior to the placement of concrete slabs, the expansive soils encountered on the subject property shall be pre-moistened until the moisture content reaches at least 120% of the optimum moisture content to a depth of twelve inches (12").

The pre-moistened soils should be tested, and verified to be 120% of optimum moisture content, prior to the placement of the sub-grade. Following our testing and verification of moisture content, the sub-grade, polyethylene plastic, and sand **must** be placed within one day.

Footing trench spoils should either be removed from the slab areas or compacted into place by mechanical means and tested for compaction.

EXCAVATION EROSION CONTROL

During inclement periods of the year, when rain is threatening (between November 1, and April 15, per the Los Angeles Building Code, Sec. 7002.), an erosion control plan shall be implemented and approved by the City of Los Angeles, to reduce the potential of site erosion. The following are several recommendations prepared by this office. The following recommendations are valid for any time of the year that rain threatens an excavation.

Open Excavations

All open excavations shall be protected from inclement weather. This is required to keep the surface of the open excavation from becoming saturated during rainfall. Saturation of the excavation may result in a relaxation of the soils which may result in failures.

Hillside Excavations

All hillside excavations shall be covered during the rainy seasons. Stakes, ropes, and sandbags, along with plastic may be employed to help facilitate the coverage of the excavations. Coverage of the open excavations shall over-extend from the edges of the excavations in all directions.

The project Civil Engineer shall be consulted for the limits of coverage. If possible, slopes around the open excavations shall be trimmed to slope away from the open excavation, so water runoff will not drain into the excavation. Any trees or planters that might cause failure around the open excavations, due to the saturated hillside, shall be anchored safely. After the rain has ceased, the excavations shall be reviewed by the project soil engineer and geologist for safety prior to recommencement of work.

Open Trenches

No water shall be allowed to pond or saturate open trenches. All open trenches shall be covered with plastic and sandbags. Areas around trenches shall be sloped in such a way that water will not runoff into the trenches. After the rain has ceased, trenches shall be reviewed by project soil engineer for safety prior to recommencing work. All footing excavations must be reviewed by the project soil engineer again, prior to pouring concrete.

Grading in Progress

During the inclement part of the year, or during periods when rain is threatening, all fill that has been spread and awaits compaction shall be compacted before stopping work for the day or before stopping because of inclement weather. These fills, once compacted, shall have the surfaces sloped to drain to one area where water may be removed.

Work may start again, after the rainy period, once the site has been reviewed by the project soils engineer. Loose materials not compacted prior to the rain shall be removed and aerated so that the moisture content of these fills will be within three percent (3%) of the optimum moisture content. Surface materials previously compacted before the rain, shall be scarified, brought to the proper moisture content, and re-compacted prior to placing additional fill, if deemed necessary by the Soils Engineer.

Additionally, it is suggested that all stock-piled loose fill materials, not compacted prior to anticipated rainfall, shall be covered with plastic. This action will keep the loose fill from being saturated with water, and will allow the grading to resume when the rain stops. It is always easier and less time consuming to increase moisture content of the fill than to aerate the fill to achieve optimum moisture.

All of the above recommendations shall be considered as part of the erosion control plan for the subject property. However, these recommendations shall and will not supersede, nor limit any erosion control plans produced by the Project Civil Engineer.

EXCAVATIONS

Excavations ranging in height up to twelve feet (12') will be required for the retaining walls. Conventional excavation equipment may be used to make these excavations. Excavations should expose older alluvium. These soils are suitable for vertical excavations up to ten feet (10'). Excavations that are higher than ten feet (10') in height, and all loose surficial material, shall be trimmed back at a gradient of 1:1 (h:v). This should be verified by a representative of this office during construction so that modifications can be made if variations in the soil occur.

Soil exposed in the proposed cuts should be kept moist, but not saturated, to reduce the potential for raveling and sloughing that may occur during construction.

All excavations should be stabilized within 15 days of initial excavation. If this time is exceeded, the project soils engineer must be notified, and modifications, such as shoring or slope trimming may be required. Water should not be allowed to pond on top of the excavation, nor to flow toward it. All excavations should be protected from inclement weather. The top of the excavations should be barricaded to ensure that no vehicular surcharge be allowed within five feet (5') of the top of cut.

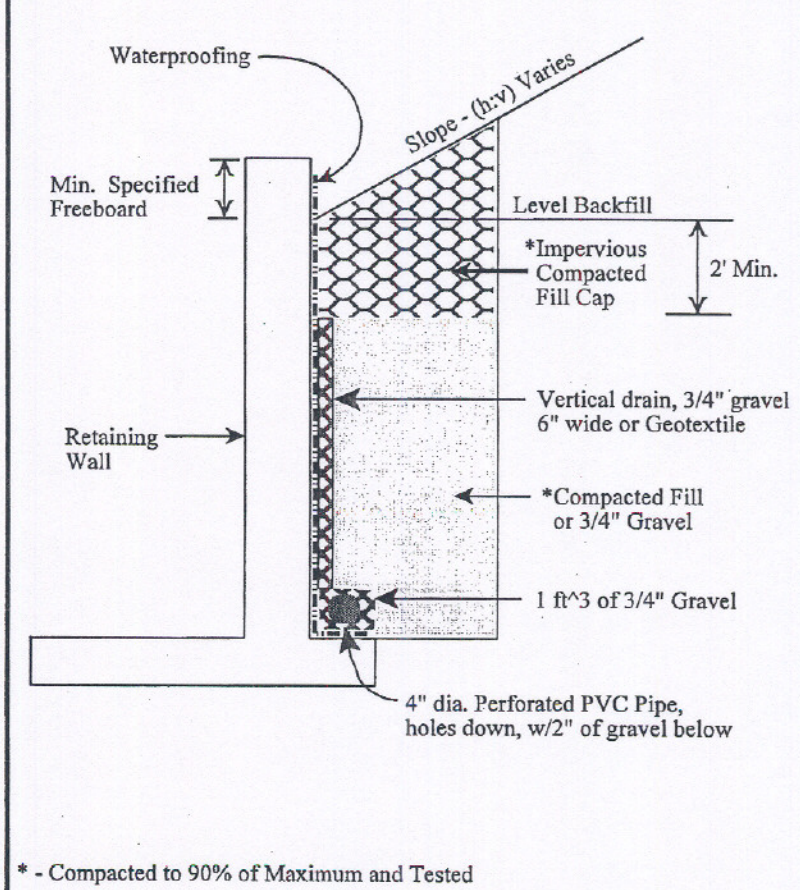
Construction methods shall meet the requirements of the Construction and General Industry Safety Orders, the Occupational Safety and Health Act, California OSHA in addition to other public agencies having jurisdiction.

RETAINING WALLS

Cantilever Walls

Retaining walls should be designed to resist an active earth pressure such as that exerted by compacted backfill. Retaining walls should be designed to resist an active earth pressure such as that exerted by compacted backfill or retained alluvium. Retaining walls up to fifteen feet in height may be designed per the following table.

Surface Slope of Retained Material (h:v)	Equivalent Fluid Weight lb/ft³.
Level	30
5 to 1	32
4 to 1	35
3 to 1	38
2 to 1	43
1½ to 1	55
1 to 1	80



Retaining Wall Detail

The wall pressure stated above assumes:

1. The wall has been backfilled with non-expansive soils, compacted to 90 percent of the maximum density and tested as outlined in the wall backfill section of this report.

2. The gradient behind the wall conforms to the above table, and there is no structural surcharge.

3. Adequate drainage is provided behind the wall to minimize the buildup of hydrostatic pressures.

a. A perforated pipe, with perforations placed down, shall be installed at the base of the wall footing. The pipe shall be encased in at least one foot (1') of three-quarter inch (¾") gravel. All drainage from this pipe should be transferred to street drainage via non-erosive devices or to an approved drainage area via non-erosive devices.

b. The back of the wall shall be waterproofed and a continuous vertical drain (geotextile or gravel) shall be placed on the backside of the wall.

A concrete-lined swale should be placed behind the wall that can intercept surface runoff from up slope areas. This surface runoff shall be transferred to street drainage via non-erosive devices or to an approved drainage area via non-erosive devices. A minimum freeboard of eight inches (8") shall be maintained at all times. Any slough, debris or trash that accumulates behind the wall should be removed immediately.

All excavations shall be reviewed by this office to ascertain if there are any conditions encountered that are different from those observed in the explorations and modeled by the calculations. If changes are observed additional recommendations will be made at that time. All excavations must be stabilized within fifteen days (15) or less.

Foundation design parameters, as given in the preceding section, may be used for retaining walls. All loose material shall be cleaned from the foundation excavations. Water shall not be allowed to pond or drain into or through the footing trench excavations. Proper compaction of the backfill is recommended to provide lateral support to adjacent properties.

Basement Walls

Basement walls for the proposed subterranean parking level should be designed to resist a trapezoidal distribution of lateral earth pressure. The lateral active earth pressure for the basement wall will be similar to that recommended for braced excavations. The "at rest" lateral earth pressure will be 65 pounds per cubic foot. Further, the maximum pressure developed should be taken as 29H. In addition to lateral earth pressure, this wall should be designed to resist the surcharge imposed by the proposed structures, footings, any adjacent buildings, or by adjacent traffic surcharge.

All required backfill adjacent to subterranean walls should be compacted to at least 90 percent of the maximum density or backfilled with gravel. Proper compaction of the backfill is recommended to provide lateral support to adjacent properties. Even with proper compaction of required backfill, settlement of the backfill may occur because of the significant depth of the backfill. Accordingly, utility lines, footings, or false work should be planned and designed to accommodate such potential settlements. All drainage requirements listed in the RETAINING WALL section of the above referenced report shall apply.

Wall Backfill

1. Walls to be backfilled must be reviewed by the project Soils Engineer prior to commencement of the backfilling operation or placement of the wall backdrain system.

2. After the wall backdrain system has been placed and the back side of the wall has been waterproofed, fill may be placed, if sufficient room allows, in layers not exceeding four inches (4") in thickness and compacted to 90 percent of the maximum density, as determined by the latest version of ASTM D 1557.

3. If the wall backfill consists of a granular free-draining material, a vertical gravel blanket at the face of the wall, or similar vertical drainage system, will not be required.

4. If the onsite soils are used for wall backfill, and they have an expansion index of 30 or greater, a vertical gravel drain blanket, six inches (6") thick along the back side of the wall from top to bottom, shall be required.

5. Where space does not permit compaction of material behind the wall, a granular backfill shall be used. This granular backfill shall consist of one-half inch (½") to three-quarter inch (¾") of crushed rock.

6. All granular free-draining wall backfills shall be capped with a clayey compacted soil within the upper two feet (2') of the wall for a depth of two feet (2'). This compacted material should start below the required wall freeboard.

7. Where slopes are steeper than 5:1 (h:v) benching shall be required into competent materials as determined by this office in the field at the time of grading.

DRAINAGE AND MAINTENANCE

Specific

A comprehensive drainage system must be designed and incorporated into the final plans. In addition, any pads must be maintained and planted in a way that will allow this drainage system to function as intended. The following are specific drainage, maintenance, and landscaping recommendations.

1. Pad Drainage

   (a) Positive pad drainage shall be incorporated into the final plans. All drainage from the roof and pad shall be directed so that water does not pond adjacent to the foundations or flow toward them. All drainage from the site shall be collected and directed via non-erosive devices to a location approved by the building official. **No alteration of this system shall be allowed.**

(b) Planters should not be placed adjacent to the structures. However, if planters are placed adjacent to the structure they shall be designed to drain away from the structure. All planters shall have a sufficient number of area drains to collect water and transferring it away from the foundation. Care should be taken to not saturate the soils, i.e. leaking irrigation lines or excessive landscape watering.

2. Landscaping (Planting)

It is recommended that a landscape architect be consulted regarding planting adjacent to the development. Plants surrounding the development shall be of a variety that requires a minimum of watering. It will be the responsibility of the property owner to maintain the planting. Alterations of planting schemes shall be reviewed by the landscape architect.

3. Irrigation

An adequate irrigation system will be required to sustain landscaping. Any leaks or defective sprinklers shall be repaired immediately. To mitigate erosion and saturation, automatic sprinkling systems shall be adjusted for rainy seasons. A landscape irrigation specialist should be consulted to determine the best times for landscape watering and the maximum amount of water usage.

4. Rodent Control

The property owner must undertake and maintain a program which eliminates or controls burrowing animals. This must be an ongoing program in order to provide protection to the slope's stability. The uncontrolled burrowing by rodents has proven to be one of the major causes for surficial slope stability problems.

PAVING FOR PRIVATE DRIVEWAYS AND PARKING AREAS

Asphaltic concrete paving will be required for the parking areas and driveways. The existing grade in this area is underlain by earth fills and older alluvium. Therefore, these fill soils shall be removed and recompacted as outlined above.

Pavement sections are given below for various types of vehicle use. These pavement sections may be used for preliminary design criteria.

Vehicle Use	Assume T.I.	Pavement Section (in.)	Base Course Section (in.)
Passenger Cars	3.5	3	5
Medium Weight Trucks	6.0	4	9
Truck and Trailer	9.0	6	14

The above sections are based upon "Traffic Indexes" (T I's) for cars, light trucks and trucks with trailers. Multiple sections are provided so that the entire site is not designed for a "Traffic Index" associated with truck and trailer. The appropriate design section must be utilized in the appropriate area of the proposed development. Failure to use the correct "Traffic Index" on the subject property will result in a greatly reduced life of the paving section. If it is determined that "Traffic Indexes" other than those shown in the above table are site appropriate, this office should be contacted for additional section calculations.

Base course should consist of crusher-run base or decomposed granite. The base course should be brought to optimum moisture content and re-compacted to ninety five percent (95%) of the maximum density. The maximum density is determined by the latest version of ASTM D 1557.

Additional field and laboratory testing will be required, near the completion of grading, to determine the engineering characteristics of the material located at grade in the areas to receive paving. The results of this work, along with the required paving sections will be provided in letter form.

REVIEWS

Plan Review and Plan Notes

The final construction and/or grading plans shall be reviewed and approved by the consultants. This is required to determine if the recommendations of the report have been properly understood and carried forth in the design drawings.

The final plans should reflect the following:

1. The Soils Engineering Investigation by SubSurface Designs, Inc., as a part of the plans.

2. Plans must be reviewed and signed by the soils engineer.

3. All grading must be reviewed by the project soils engineer.