INTRODUCTION

This report presents results of a Geotechnical Engineering study performed for proposed improvements to the athletic fields at Adolfo Camarillo High School in the City of Camarillo (see Vicinity Map in Appendix A). Proposed improvements will include installation of synthetic turf surfaces and subdrainage systems to replace natural turf surfaces on the athletic fields, a new bathroom building adjacent to the baseball field, and three ticket booths with attached entry gates at the entrances to the football field. Existing asphalt walkways around the football field will be replaced with concrete sidewalks ranging in width from 6 to 12 feet. New parking spaces will be added southeast of the eastern end of the track, including some in an area where leach lines reportedly currently exist. An existing asphalt service road will be replaced with new asphalt paving, and new sidewalk will run between the western parking lot and the baseball/softball fields. Water and sewer lines will connect the new restroom near the baseball field to existing utilities.

Current plans indicate that a minimum of 12 inches of soils are to be compacted below the drain system that will underlie the fields. Where flat panel drains will be located within the drainage grid, a trench about 18 inches wide will be cut about 3 to 4 inches deeper than adjacent subgrade soils. Subgrade soil elevation will be 6 inches below the finished base grade elevation (before synthetic turf is placed). The panel drains are 12 inches wide and approximately 2 inches high, and are to be wrapped with a filter sock and backfilled with a minimum of 0.5 inches of clean washed sand.

The panel drains are to flow at a gradient of 0.6% toward the perimeter of the field where they will be collected within a trench with a depth and design that will depend on the soil characteristics and groundwater conditions at the site. The trench will run parallel to and under the sidelines toward a storm drain outlet.

The synthetic turf will be supported by 6 inches of permeable base (rock) material on the subgrade soils and panel drain sand cover.

The all-weather track surface will be underlain by asphalt pavement above compacted aggregate base materials and compacted subgrade soils. Surface flow will be directed inward to a drain running parallel to the track edge. Storm water will flow from the track edge drain at a 2% gradient toward and into the larger trench that gathers the athletic field flat panel drain waters.

The water gathered within the trench will either infiltrate into the subsurface or will be piped to a storm drain system.

The one-story bathroom building will be a reinforced CMU block structure that will be approximately 498 feet in plan view. It is proposed to support it with a conventional foundation system and a slab-on-grade floor.

The ticket booths are expected to range from 50 to 70 square feet, and to have attached 10-foot tall entry gates supported by steel tube columns on pier footings. The one-story ticket booths will be constructed with reinforced CMU block, and will utilize conventional foundation systems with slab-on-grade floors. There will be 8-foot high freestanding reinforced CMU walls adjacent to the ticket booths at the entry gates.

It is understood that there may be 6-foot high CMU and/or concrete site walls, some of which may be retaining, but none that retain more than 6 feet. There may also be fences that range in height from 8 to 18 feet high in various areas of the site.

PURPOSE AND SCOPE OF WORK

The purpose of the geotechnical study that led to this report was to analyze the soil conditions of the site with respect to the proposed improvements. These conditions include surface and subsurface soil types, expansion potential, settlement potential, bearing capacity, and the presence or absence of subsurface water. The scope of work included:

1. Performing a reconnaissance of the site.
2. Drilling, sampling, and logging 5 hollow-stem-auger borings to study soil and groundwater conditions.
3. Drilling and logging 2 hollow-stem-auger borings for infiltration testing.
4. Laboratory testing soil samples obtained from the subsurface exploration to determine their physical and engineering properties.
5. Consulting with owner representatives and design professionals.
6. Analyzing the geotechnical data obtained.
7. Preparing this report.

Contained in this report are:

1. Descriptions and results of field and laboratory tests that were performed.
2. Conclusions and recommendations pertaining to site grading and infiltration potential.

GENERAL GEOLOGY

The site lies within the Oxnard Plain, which in turn lies within the western Transverse Ranges geomorphic province. The Oxnard Plain and the Transverse Ranges are characterized by ongoing tectonic activity. In the vicinity of the subject site, Tertiary and Quaternary sediments have been folded and faulted along predominant east-west structural trends.

There are several faults located within the region, including the Camarillo Fault that is mapped along an east-west trend through the athletic field areas. As such, the project area is located within the "Fault Rupture Hazard Zone" delineated by the State of California (CDMG. 1972, Revised 1999) for the Camarillo Fault. However, the Camarillo Fault is not considered capable of generating a large seismic event. The nearest known fault capable of generating significant earthquakes is the Simi-Santa Rosa Fault, which is located approximately 1.4 miles north of the subject site.

The site is underlain by alluvial sediments consisting of loose to medium dense silty sands, fine to medium sands, and firm to very stiff sandy clays. Boring No. B-4 encountered artificial fill consisting of stiff silty clay with varying sand content. In addition to the artificial fill, bedrock consisting of the Saugus Formation was encountered and consisted of silty fine-grained sandstone. Boring B-2 from 2009 site studies for the aquatic center also encountered fill when advanced from the main campus level near the top of the walkway down to the football field.

The site is not within any of the Liquefaction Hazard Hazard Zones designated by the California Geological Survey (CGS, 2002).

No landslides were observed to be located on or trending into the subject property during the field study, or during reviews of the referenced geologic literature.

SEISMICITY AND SEISMIC DESIGN

Although the site is not within a State-designated "fault rupture hazard zone", it is located in an active seismic region where large numbers of earthquakes are recorded each year. Historically, major earthquakes felt in the vicinity of the subject site have originated from faults outside the area. These include the December 21, 1812 "Santa Barbara Region" earthquake, that was presumably centered in the Santa Barbara Channel, the 1857 Fort Tejon earthquake, the 1872 Owens Valley earthquake, and the 1952 Arvin-Tehachapi earthquake.

It is assumed that the 2016 CBC and ASCE 7-10 guidelines will apply for the seismic design parameters. The 2016 CBC includes several seismic design parameters that are influenced by the geographic site location with respect to active and potentially active faults, and with respect to subsurface soil or rock conditions. The seismic design parameters presented herein were determined by the U.S. Seismic Design Maps "risk-targeted" calculator on the USGS website for the jobsite coordinates (34.2156˚ North Latitude and -119.0102° West Longitude). The calculator adjusts for Soil Site Class D, and for Occupancy (Risk) Category I (for non-habitable structures). (A listing of the calculated 2016 CBC and ASCE 7-10 Seismic Parameters is presented below and in Appendix C.)

Summary of Seismic Parameters – 2016 CBC

Site Class (Table 20.3-1 of ASCE 7-10 with 2016 update)	D
Occupancy (Risk) Category	I
Seismic Design Category	E
Maximum Considered Earthquake (MCE) Ground Motion	
Spectral Response Acceleration, Short Period – S_s	2.146g
Spectral Response Acceleration at 1 sec. – S_1	0.787g
Site Coefficient – F_a	1.00
Site Coefficient – F_v	1.50
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	2.146g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	1.181g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	1.430g
One Second Spectral Response – S_{D1}	0.787g
Site Modified Peak Ground Acceleration - PGA_M	0.809g
Values appropriate for a 2% probability of exceedance in 50 years	

The Fault Parameters table in Appendix C lists the significant “active” and “potentially active” faults within a radius of about 34 miles from the subject site. The distance between the site and the nearest portion of each fault is shown, as well as the respective estimated maximum earthquake magnitudes, and the deterministic mean site peak ground accelerations.

SOIL CONDITIONS

Evaluation of the subsurface indicates that soils are generally alluvium consisting of loose to medium dense silty sands, fine to medium sands, and firm to very stiff sandy clays. Boring No. B-4, which was located near the northeast corner of the football field, encountered approximately 7 feet of artificial fill consisting of stiff silty clay with varying sand content. Artificial fill was also encountered to a depth of 8.5 feet in Boring B-2, which was drilled during 2009 studies for the aquatic center at the main level of the campus. Saugus Formation bedrock was encountered below the fill in Boring B-4, and consisted of silty fine-grained sandstone. Saugus Formation was also encountered below the fill in the 2009 boring (B-2), and consisted of interbeds of clayey silty sands with caliche, silty sands with gravels, and silty clay.

Near-surface alluvial soils encountered within the fields in Boring Nos. B-1 through B-4 are generally characterized by low blow counts and in-place densities, but low compressibilities. Testing indicates that near-surface soils within the field area lie in the “very low” expansion range because the expansion index equals 0. [A locally adopted version of the classification of soil expansion, Table 18-I-D, is included in Appendix B of this report.] It appears that soils can be cut by normal grading equipment.

Groundwater was not encountered to a depth of 51.5 feet during drilling for a feasibility study conducted for a proposed pool complex (see Site-Specific Bibliography). Mapping of historically high groundwater levels by the California Geological Survey (CGS, 2002a) indicates that groundwater has been at least 55 feet below the ground surface near the subject site.

The subject site is not located within any of the Liquefaction Hazard Zones delineated by the California Division of Mines and Geology (2002b). As a result, it appears that the hazard posed by liquefaction to the proposed improvements is low.

Samples of near-surface soils were tested for pH, resistivity, soluble sulfates, and soluble chlorides. The test results provided in Appendix B should be distributed to the design team for their interpretations pertaining to the corrosivity or reactivity of various construction materials (such as concrete and piping) with the soils. It should be noted that sulfate contents (61 mg/Kg) are in the "S0" ("negligible") exposure class of Table 19.3.1.1 of ACI 318-14; therefore, it appears that special concrete designs will not be necessary for the measured sulfate contents.

Based on criteria established by the County of Los Angeles (2013), measurements of resistivity of near-surface soils (6,000 ohms-cm) indicate that they are "moderately corrosive" to ferrous metal (i.e. cast iron, etc.) pipes.

GEOTECHNICAL CONCLUSIONS

The site is suitable for the proposed athletic field improvements from a Geotechnical Engineering standpoint provided that the recommendations contained in this report are successfully implemented into the project.

Infiltration of storm water may be feasible for this campus. More detailed findings after infiltration testing is completed.

GEOTECHNICAL RECOMMENDATIONS FOR FIELD AND TRACK SURFACE IMPROVEMENTS

All proposed grading should conform to the 2016 California Building Code.

Plans and specifications should be provided to Earth Systems prior to grading. Plans should include the grading plans, drainage plans, and applicable details.

The existing ground surface should be initially prepared for grading by removing all grass and vegetation, large roots, debris, other organic material, and non-complying fill. Organics and debris should be stockpiled away from areas to be graded, and ultimately removed from the site to prevent their inclusion in fills. Voids created by removal of such material should be properly backfilled and compacted. No compacted fill should be placed unless the underlying soil has been observed by the Geotechnical Engineer.

Proposed areas of athletic field improvements or areas to receive fill should be overexcavated to a depth of one foot. The resulting surface should then be scarified an additional 6 inches, moisture conditioned, and recompacted. This will result in at least 12 inches of compacted fill below the flat panel drains, and 18 inches of compacted fill below the areas between the drains. Compaction should be verified to be a minimum of 90% of the maximum dry density obtained by the ASTM D 1557 test method.

Proposed areas of track surface replacements (and underlying asphaltic concrete pavement), exterior slabs-on-grade, or sidewalks should be overexcavated to a depth of one foot. The resulting surface should then be scarified an additional 6 inches, moisture conditioned, and recompacted. Compaction should be verified to be a minimum of 95% of the maximum dry density obtained by the ASTM D 1557 test method.

Once subgrade elevations are achieved and flat panel drains are installed, a permeable filter fabric, such as Mirafi 140N, should be placed over the subgrade soils and panel drains. Permeable base should be placed over the filter fabric and compacted to a minimum of 95% of the maximum dry density obtained by the ASTM D 1557 test method.

The bottoms of all excavations should be observed by a representative of this firm prior to processing or placing fill.

On-site soils may be used for fill once they are cleaned of all organic material, rock, debris, and irreducible material larger than 8 inches.

Fill and backfill should be placed at, or slightly above optimum moisture in layers with loose thickness not greater than 8 inches.

Shrinkage of soils affected by compaction is estimated to be about 10% based on an anticipated average compaction of 92%. Shrinkage from removal of any existing subsurface structures is not included in these figures.

Utility trench backfill should be governed by the provisions of this report relating to minimum compaction standards. In general, on-site service lines may be backfilled with native soils compacted to 90% of the maximum dry density. Backfill of offsite service lines will be subject to the specifications of the jurisdictional agency or this report, whichever are greater.

Compaction tests shall be made to determine the relative compaction of the fills, subgrade soils, and utility trench backfills in accordance with the following minimum guidelines: one test for each two-foot vertical lift, one test for each 1,000 cubic yards of material placed, one test per two-foot vertical lift per 250 lineal feet of utility trench backfill, and four tests at finished subgrade elevation of each field.

It is recommended that Earth Systems be retained to provide Geotechnical Engineering services during the site development, drain installation, and grading phases of the work to observe compliance with the design concepts, specifications and recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

GRADING RECOMMENDATIONS FOR BUILDINGS, ENTRY GATES, AND PAVEMENTS

It should be noted that the location provided to Earth Systems for the future 498 square-foot restroom building is within the Fault Rupture Hazard Zone for the Camarillo Fault, and an evaluation of the fault rupture hazard may be required. However, if the size precludes the requirement for hazard evaluation, or an acceptable location for the restroom is located outside the fault zone, a conventional foundation system would be acceptable.

Grading at a minimum should conform to the 2016 California Building Code.

The existing ground surface should be initially prepared for grading by removing all vegetation, trees, large roots, debris, other organic material, and non-complying fill. Non-complying fill would include the gravel and piping of the leach lines that reportedly exist southeast of the eastern end of the track around the perimeter of the football field. Organics and debris should be stockpiled away from areas to be graded, and ultimately removed from the site to prevent their inclusion in fills. Voids created by removal of such material should be properly backfilled and compacted. No compacted fill should be placed unless the underlying soil has been observed by the Geotechnical Engineer.

Once the gravel and piping is completely removed from the existing leach lines, the excavations should be deepened and widened until firm native soils are encountered in each direction.

Overexcavation and recompaction of soils in the building areas will be necessary to decrease the potential for differential settlement and provide more uniform bearing conditions. Soils should be overexcavated to a depth of 4.5 feet below finished subgrade elevation throughout the entire building area, and to a distance of 5 feet beyond the perimeter of each building. The resulting surface should then be scarified an additional 6 inches, moisture conditioned, and recompacted to at least 90% of the maximum dry density. The intent of these recommendations is to have a minimum of 5 feet of compacted soil below the building.

Overexcavation and recompaction of soils under and around pier footings for the entry gates will also be necessary. Soils should be overexcavated to a depth of 4.5 feet below finished subgrade elevation, and to a distance of 3 feet on either side of the footing edges. The resulting surface should then be scarified an additional 6 inches, moisture conditioned, and recompacted to at least 90% of the maximum dry density.

Areas outside of the building area to receive fill, exterior slabs-on-grade, sidewalks, or paving should be overexcavated to a depth of 1.5 feet below finished subgrade elevation. The resulting surface should then be scarified an additional 6 inches, moisture conditioned, and recompacted. Because the expansion index of on-site soils is in the “very low” range, no aggregate base will be required below sidewalks. (Recommendations for structural paving sections for pavements subjected to vehicular traffic are provided elsewhere in this report.)

The bottoms of all excavations should be observed by a representative of this firm prior to processing or placing fill.

On-site soils may be used for fill once they are cleaned of all organic material, rock, debris, and irreducible material larger than 8 inches.

Fill and backfill should be placed at, or slightly above optimum moisture in layers with loose thickness not greater than 8 inches. Each layer should be compacted to a minimum of 90% of the maximum dry density obtainable by the ASTM D 1557 test method. The upper one foot of subgrade below areas to be paved should be compacted to a minimum of 95% of the maximum dry density.

Import soils used to raise site grade should be equal to, or better than, on-site soils in strength, expansion, and compressibility characteristics. Import soil can be evaluated, but will not be prequalified by the Geotechnical Engineer. Final comments on the characteristics of the import will be given after the material is at the project site.

If pumping soils or otherwise unstable soils are encountered during the overexcavation, stabilization of the excavation bottom will be required prior to placing fill. This can be accomplished by various means. The first method would include drying the soils as much as possible through scarification, and working thin lifts of "6-inch minus" crushed angular rock into the excavation bottom with small equipment (such as a D-4) until stabilization is achieved. Use of a geotextile fabric such as Mirafi 500X, or Tensar TX-160, or an approved equivalent, is another possible means of stabilizing the bottom. If this material is used, it should be laid on the excavation bottom and covered with approximately 12 inches of "3-inch minus" crushed angular rock prior to placement of filter fabric (until the bottom is stabilized). The rock should then be covered with a geotextile filter fabric before placing fill above. It is anticipated that stabilization will probably be necessary due to the existing high moistures of the soils, and due to the shallow groundwater depth. Unit prices should be obtained from the Contractor in advance for this work.

Utility trench backfill should be governed by the provisions of this report relating to minimum compaction standards. In general, on-site service lines may be backfilled with native soils compacted to 90% of the maximum dry density. Backfill of offsite service lines will be subject to the specifications of the approved project plans or this report, whichever are greater.

Utility backfill operations should be observed and tested by the Geotechnical Engineer to monitor compliance with these recommendations.

Utility trenches running parallel to footings should be located at least 5 feet outside the footing line, or above a 2:1 (horizontal to vertical) projection downward from a point 9 inches above the outside edge of the bottom of the footing.

Compacted native soils should be utilized for backfill below structures. Sand should not be used under structures because it provides a conduit for water to migrate under foundations.

GEOTECHNICAL DESIGN PARAMETERS FOR BUILDINGS AND SITE WALLS

Conventional Spread Foundations

Conventional continuous footings and/or isolated pad footings may be used to support structures. For one-story buildings, perimeter and interior footings should have minimum depths of 12 inches.

Footings should bear into firm recompacted soils. as recommended elsewhere in this report. Foundation excavations should be observed by a representative of this firm after excavation, but prior to placing of reinforcing steel or concrete, to verify bearing conditions.

Conventional continuous footings may be designed based on an allowable bearing value of 2,000 psf. This value has a factor of safety of 3.

Isolated pad footings may be designed based on an allowable bearing value of 2,300 psf. This value has a factor of safety of greater than 3.

Allowable bearing values are net (weight of footing and soil surcharge may be neglected) and are applicable for dead plus reasonable live loads.

Bearing values may be increased by one-third when transient loads such as wind and/or seismicity are included.

Lateral loads may be resisted by soil friction on floor slabs and foundations and by passive resistance of the soils acting on foundation stem walls. Lateral capacity is based on the assumption that any required backfill adjacent to foundations and grade beams is properly compacted.

Resistance to lateral loading may be provided by friction acting on the bases of foundations. A coefficient of friction of 0.60 may be applied to dead load forces. This value does not include a factor of safety.

Passive resistance acting on the sides of foundation stems equal to 380 pcf of equivalent fluid weight may be included for resistance to lateral load. This value does not include a factor of safety.

A minimum factor of safety of 1.5 should be used when designing for sliding or overturning.

For building foundations, passive resistance may be combined with frictional resistance provided that a one-third reduction in the coefficient of friction is used.

Footing designs should be provided by the Structural Engineer, but the dimensions and reinforcement he recommends should not be less than the criteria set forth in Table 18-I-D for the “very low” expansion range.

Soils should be lightly moistened prior to placing concrete. Testing of premoistening is not required.

Drilled Pier Foundations

A pier and grade-beam foundation system may be used to support the proposed entry gates and site walls. Foundation piers should be designed as friction piles. No allowance should be taken for end bearing.

Piers may consist of drilled, reinforced cast-in-place concrete caissons (cast-in-drilled-hole “CIDH” piles). Piers may be drilled or hand-dug. Steel reinforcing may consist of “rebar cages” or structural steel sections.

As a minimum, the new piers should be at least eighteen inches (18”) in diameter and embedded into compacted fill, firm native soil, or a combination of both. The geotechnical engineer should be consulted during pier installation to determine compliance with the geotechnical recommendations.

For vertical (axial compression) and uplift capacity, the attached pile capacity graphs may be used. Drilled pier diameters of 1.5, 2.0, and 2.5 feet were analyzed, and the results are presented on the attached charts. Side resistance is not allowed to increase beyond a depth equal to 20 pile diameters. Upward resistance is taken as two-thirds of the downward resistance. The downward and upward capacity graphs for drilled piers are presented in Appendix D.

The load capacities shown on the attached charts are based upon skin friction with no end bearing. These allowable capacities include a safety factor of 2.0 and may be increased by one-third when considering transient loads such as wind or seismic forces.

Reduction in axial capacity due to group effects should be considered for piers spaced at 3 diameters on-center or closer.

All piers should be tied together laterally (in both directions) at the top with grade beams. The size, spacing, and reinforcing of grade beams should be determined by the Structural Engineer.

Lateral (horizontal) loads may be resisted by passive resistance of the soil against the piers. An equivalent fluid weight (EFW) of 380 psf per foot of penetration in the compacted fill (upper 5 feet) and an EFW of 300 pcf in the underlying firm native soils. These resisting pressures are ultimate values. The maximum passive pressure used for design should not exceed 4,200 psf. An appropriate factor of safety should be used for design calculations (minimum of 1.5 recommended).

For piers spaced at least three diameters apart, an effective width of 2 times the actual pier diameter may be used for passive pressure calculations.

Assuming 18-inch diameter piers of reinforced concrete that are fixed against rotation at the head, the “point of fixity” was estimated to be located at least 6 feet below the final ground elevation based on commonly accepted engineering procedures (Lee, 1968). If 24-inch diameter piers are used, the “point of fixity” was estimated to be located at least 7 feet below the final ground elevation. If 30-inch diameter piers are used, the “point of fixity” was estimated to be located at least 8.5 feet below the final ground elevation.

The geotechnical engineers, or their representatives, should be present during excavation and installation of all piers to observe subsurface conditions, and to document penetration into load supporting materials (i.e. either compacted fill or firm native soil).

Since the piers are designed to rely completely on intimate frictional contact with the soil, any casing (if used) should be removed during placement of concrete. The bottoms of pier excavations should be relatively clean of loose soils and debris prior to placement of concrete.

Installed piers should not be more than two percent (2%) from the plumb position.

Slabs-on-Grade

Concrete slabs should be supported by compacted structural fill as recommended elsewhere in this report.

It is recommended that perimeter slabs (walks, patios, etc.) be designed relatively independent of footing stems (i.e. free floating) so foundation adjustment will be less likely to cause cracking.

Slab designs should be provided by the Structural Engineer, but the reinforcement and slab thicknesses should not be less than the criteria set forth in Table 18-I-D for the “very low” expansion range. Current plans call for 4-inch thick concrete reinforced with No. 3 bars on 18-inch centers. These specifications are considered appropriate for the soil conditions. (Note that structural paving sections for areas to be exposed to vehicular traffic are presented elsewhere in this report.)

Areas where floor wetness would be undesirable should be underlaid with a vapor retarder (as specified by the Project Architect or Civil Engineer) to reduce moisture transmission from the subgrade soils to the slab. The retarder should be placed as specified by the structural designer.

Soils should be lightly moistened prior to placing concrete. Testing of premoistening is not required. Premoistening of slab areas should be observed and tested by this firm for compliance with these recommendations prior to placing of sand, reinforcing steel, or concrete.

Retaining Walls

Conventional cantilever retaining walls backfilled with compacted on-site soils may be designed for active pressures of 40 pcf of equivalent fluid weight for well-drained, level backfill.

Restrained retaining walls backfilled with compacted on-site soils may be designed for at-rest pressures of 60 pcf of equivalent fluid weight for well-drained, level backfill.

These pressures are based on the assumption that backfill soils will be compacted to 90% of the maximum dry density determined by the ASTM D 1557 Test Method.

For retaining walls, passive resistance may be combined with frictional resistance without reduction to the coefficient of friction.

Because walls will not retain more than 6 feet, seismic forces do not need to be added to the design.

The lateral earth pressure to be resisted by the retaining walls or similar structures should also be increased to allow for any other applicable surcharge loads. The surcharges considered should include forces generated by any structures or temporary loads that would influence the wall design.

A system of backfill drainage should be incorporated into retaining wall designs. Backfill comprising the drainage system immediately behind retaining structures should be free-draining granular material with a filter fabric between it and the rest of the backfill soils. As an alternative, the backs of walls could be lined with geodrain systems. The backdrains should extend from the bottoms of the walls to about 18 inches from finished backfill grade. Waterproofing may aid in reducing the potential for efflorescence on the faces of retaining walls.

Compaction on the uphill sides of walls within a horizontal distance equal to one wall height should be performed by hand-operated or other lightweight compaction equipment. This is intended to reduce potential “locked-in” lateral pressures caused by compaction with heavy grading equipment.

SETTLEMENT CONSIDERATIONS

Maximum settlements of about one inch are anticipated for foundations and floor slabs designed as recommended. (It should be noted that these values do not include potential seismic- or liquefaction-induced settlements.) Differential settlement between adjacent load bearing members should be expected to range up to about one-half the total settlement.

If the preliminary recommendations for foundation design and construction are followed, settlement of the piers should not exceed approximately 0.5 inch under static conditions. Differential settlement of neighboring pier footings of varying loads, depths or sizes may be as high as fifty% of the total static settlement over a distance of about 30 feet.

DESIGN VALUES FOR FENCEPOST PIER FOOTINGS IN NON-COMPACTED AREAS

Pier footings to support fence posts that are drilled into native soils may be designed for passive pressures of 100 psf per foot below natural grade. This value is based on presumptive parameters provided in the California Building Code for clay soils.

PRELIMINARY ASPHALT PAVING SECTIONS FOR TRACK RESURFACING

Assuming a Traffic Index of 5 for areas to be used for asphalt below track resurfacing, and using the measured R-Value of 29, paving sections should have a minimum gravel equivalent of 1.14 feet. This can be achieved by using 3 inches of asphaltic concrete on 6 inches of Processed Miscellaneous Base (PMB) compacted to a minimum of 95% of the maximum dry density on subgrade soils compacted to a minimum of 95% of the maximum dry density.

For new fire lanes or drive lanes in parking areas with a Traffic Index of 6.5, paving sections should have a minimum gravel equivalent of 1.48 feet. This can be achieved by using 4 inches of asphaltic concrete on 9 inches of Processed Miscellaneous Base (PMB) compacted to a minimum of 95% of the maximum dry density on subgrade soils compacted to a minimum of 95% of the maximum dry density.

The preliminary paving sections provided above have been designed for the type of traffic indicated. If the pavement is placed before construction on the project is complete, construction loads, which could increase the Traffic Indices above those assumed above, should be taken into account.

PRELIMINARY CONCRETE PAVING SECTIONS

Concrete paving sections provided below have been based on an assumed design life of 20 years and have been calculated for the measured R-Value of 29 (approximately equivalent to a coefficient of subgrade reaction of k = 150 pounds per cubic inch) using design methods presented by the American Concrete Institute (ACI 330R-87). For an assumed Traffic Index of 5 (for light traffic), the following minimum unreinforced paving section was determined:

1. Concrete thickness = 5 inches
2. Aggregate base thickness under concrete = 4 inches
3. Compressive strength of concrete, fc = 3,500 psi at 28 days

4.	Modulus of flexural strength of 3,500 psi concrete =	530 psi
5.	Maximum spacing of contraction joints, each way=	12.5 feet

For an assumed Traffic Index of 6.5 (for traffic that includes fire trucks), the following minimum unreinforced paving section was determined:

1.	Concrete thickness =	6 inches
2.	Aggregate base thickness under concrete =	4 inches
3.	Compressive strength of concrete, fc =	3,500 psi at 28 days
4.	Modulus of flexural strength of 3,500 psi concrete =	530 psi
5.	Maximum spacing of contraction joints, each way=	15 feet

If additional resistance to cracking is desired beyond that provided by the contraction joints, steel reinforcement can be added to the pavement section at approximately two inches below the top of concrete; however, reinforcement is not required.

STORM WATER INFILTRATION FEASIBILITY TESTING

On August 22, 2019, a set of two 8-inch diameter infiltration borings (P-1 and P-2) were drilled to depths of about 7 and 18 feet below the existing ground surface to determine the soil profile and allow installation of plastic casing for infiltration testing (see Site Plan in Appendix A for infiltration boring locations). All infiltration borings were bottomed into native Alluvium (see Logs of Borings in Appendix A).

After drilling was completed, 3-inch diameter slotted PVC casings were lowered into the boreholes. The annuli between the casings and boring walls were then filled with pea gravel. The falling-head borehole infiltration test procedure was used for infiltration testing. Approximately 2 feet of water was added to the bottom of each of the holes to start the tests, and the drop in the water surface monitored by taking periodic measurements. Readings were taken at reasonable time intervals based on infiltrating rate, and after each of these intervals, water was added to return the water level to its original depth above the hole bottom for the next test interval. The tests were run until the infiltration rates were reasonably stable.

It should be noted that the rate the water surface drops in a borehole is a percolation rate, which is related to, but is not an infiltration rate. Percolation rate ignores the wetted soil surface area into which the water is infiltrating and does not account for the volume of water infiltrated. An infiltration rate considers both factors. Hence, percolation rates (in unit length per unit time) are an overestimation of infiltration rates (also in unit length per unit time).

Earth Systems uses the Porchet equation to account for the wetted surface area and volume of water infiltrated to estimate an infiltration rate. Forms of the equation can be found in the Riverside County - Low Impact Development BMP Design Handbook (2001), the South Orange County Version, Technical Guidance Documents Appendices (2017), or in a paper by J.W. Van Hoorn, "Determining Hydraulic Conductivity with the Inversed Auger Hole and Infiltrometer Methods." The Porchet equation in its most simple form is the volume of water infiltrated divided by the product of the change in time and the wetted surface area. By substitution, the equation can be shown to be equal to:

Infiltration Rate (inches /hr.) = $(\Delta H * r * 60)/[\Delta t * (r + 2H_{avg})]$

where: ΔH = Change in water level (inches)

Δt = Change in time (minutes)

r = Radius of test hole (inches)

H_{avg} = Average height of water in test hole (inches)

The above equation does not account for the gravel pack in the annulus between the borehole wall and the slotted pipe fitted in the test hole. Ignoring the gravel pack inflates the amount of water infiltrated and, hence, yields an unconservative infiltration rate. A method to account for the volume occupied by the gravel (and the slotted pile) and adjust the infiltration rate accordingly is presented in Caltrans Test 750. Earth Systems makes this additional adjustment to our test data. The equation is:

Correction Factor = $n * [1 - (O/D)^2] + (I/D)^2$

Where: n = Pea gravel porosity

O = Outside diameter of slotted pipe (inches)

D = Test hole diameter (inches)

I = Inside diameter of slotted pipe (inches)

Earth Systems has determined an average porosity for the pea gravel used in our testing. The other values are simple measurements.

There are many factors that influence the infiltration rate. Clear water was used in our tests, whereas deleterious material will likely be contained in the storm water. Variations in soil conditions within the limits of the proposed infiltration system will likely affect infiltration characteristics. The designer who utilizes the infiltration results should consider these factors, as well as apply a factor-of-safety to the infiltration rate to account for future disposal bed siltation.

Based on the infiltration testing results in Appendix E, the measured test infiltration rates for the depths tested and boring locations are summarized in the following table:

Boring	Boring Depth (feet)	Infiltration Rate (inch/hour)	Infiltration Rate (cm/sec)
P-1	7	1.58	1.115×10^{-3}
P-2	18	0.14	9.878×10^{-5}

The designer of the proposed infiltration system beneath the synthetic turf should also consider that a minimum of 2 feet of compacted soil will be present below the bottom of the synthetic turf system. The infiltration rates provided above are for the native soils at the depths tested. Compaction of the native soils will reduce the infiltration rate of the upper 2 feet of soils underlying the 6-inch thick layer of Class II Permeable Base. The designer of the proposed infiltration system should consider the use of gravel-filled drains that extend below the compacted native soils to allow the storm water to infiltrate into the underlying native soils.

ADDITIONAL SERVICES

This report is based on the assumption that an adequate program of monitoring and testing will be performed by Earth Systems during construction to check compliance with the recommendations given in this report. The recommended tests and observations include, but are not necessarily limited to the following:

1. Review of the grading plans during the design phase of the project.
2. Observation and testing during site preparation, grading, placing of subdrainage systems and engineered fill, and permeable base.
3. Consultation as required during construction.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The analysis and recommendations submitted in this report are based in part upon the data obtained from the borings drilled on the site. The nature and extent of variations between and beyond the borings may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

The scope of services did not include any environmental assessment or investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater or air, on, below, or around this site. Any statements in this report or on the soil boring logs regarding odors noted, unusual or suspicious items or conditions observed, are strictly for the information of the client.

Findings of this report are valid as of this date; however, changes in conditions of a property can occur with passage of time whether they are due to natural processes or works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur whether they result from legislation or broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of 1 year.

In the event that any changes in the nature, design, or locations of the improvements are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing.

This report is issued with the understanding that it is the responsibility of the Owner, or of his representative to ensure that the information and recommendations contained herein are called to the attention of the Architect and Engineers for the project and incorporated into the plan and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

As the Geotechnical Engineers for this project, Earth Systems has striven to provide services in accordance with generally accepted geotechnical engineering practices in this community at this time. No warranty or guarantee is expressed or implied. This report was prepared for the exclusive use of the Client for the purposes stated in this document for the referenced project only. No third party may use or rely on this report without express written authorization from Earth Systems for such use or reliance.

It is recommended that Earth Systems be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If Earth Systems is not accorded the privilege of making this recommended review, it can assume no responsibility for misinterpretation of the recommendations contained herein.

SITE-SPECIFIC BIBLIOGRAPHY

Earth Systems Southern California, December 24, 2009, Engineering Geology and Geotechnical Engineering Feasibility Report for Proposed Pool Complex at Camarillo High School, Camarillo, California (Job No. VT-24393-01).

Earth Systems Southern California, October 27, 2011, Engineering Geology and Geotechnical Engineering Report for Proposed Aquatic Center at Camarillo High School, 4660 Mission Oaks Boulevard, Camarillo, California (Job No. VT-24393-02).

GENERAL BIBLIOGRAPHY

American Concrete Institute (ACI), 2009, ACI 318-14.

California Building Standards Commission, 2016, California Building Code, California Code of Regulations Title 24.

California Division of Mines and Geology (C.D.M.G.), 1972 (Revised 1999), Fault Rupture Hazard Zones in California, Special Publication 42.

C.D.M.G., 1975, Seismic Hazards Study of Ventura County, California, Open File Report 76-5-LA.

C.D.M.G., 2002a, Seismic Hazard Zone Report for the Camarillo 7.5-Minute Quadrangle, Ventura County, California, Seismic Hazard Zone Report 054.

C.D.M.G., 2002b, State of California Seismic Hazard Zones, Camarillo Quadrangle, Official Map, February 7, 2002.

California Geological Survey (C.G.S.). 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A.

County of Los Angeles Department of Public Works, July 1, 2013, Manual for Preparation of Geotechnical Reports.

Dibblee, Jr., Thomas W., and Helmut E. Ehrenspeck, 1990, Geologic Map of the Camarillo and Newbury Park Quadrangles, Ventura County, California, Dibblee Foundation Map No. DF-28.

Idriss, I.M., and Boulanger, R.W., 2008, Soil Liquefaction during Earthquakes, Earthquake Engineering Research Institute, MNO-12.

Jennings, C.W. and W.A. Bryant, 2010, Fault Activity Map of California, C.G.S. Geologic Data Map No. 6.

NCEER, 1997, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Technical Report NCEER-97-0022.

Pradel, D., 1998 Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 124, No. 4, April.

Pyke, R., Seed, H. B. And Chan, C. K., 1975, Settlement of Sands Under Multidirectional Shaking, ASCE, Journal of Geotechnical Engineering, Vol. 101, No. 4, April, 1975.

Seed, H. B., and Silver, M. L., 1972, Settlement of Dry Sands During Earthquakes, ASCE, Journal of Geotechnical Engineering, Vol. 98, No. 4.

Seed, H.B., 1987, Design Problems in Soil Liquefaction, Journal of the Geotechnical Engineering Division, ASCE, Volume 113, No. 8.

Southern California Earthquake Center (SCEC), 1999, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California.

Tokimatsu, Kohji and H. Bolton Seed, 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, August 1987, New York, New York.

Ventura County Planning Department, October 22, 2013, Ventura County General Plan Hazards Appendix.

Weber, F. Harold, Jr. and others, 1973, Geology and Mineral Resources of Southern Ventura County, California, C.D.M.G., Preliminary Report 14.

Youd, T.L., C.M. Hansen, and S.F. Bartlett, 2002, Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement, in Journal of Geotechnical and Geoenvironmental Engineering, December 2002.