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January 20, 2026

Howard Maxwell, Architect
Cal Poly Humboldt-Facilities Management
1 Harpst Street
Arcata, CA 95521

Subject: Geotechnical Recommendations, Parking Lot G14, Cal Poly Humboldt, Arcata, California

Dear Howard Maxwell:

Introduction

This report presents our geotechnical recommendations for proposed improvements to the G14 Parking Lot at Cal Poly Humboldt. SHN previously performed a geotechnical investigation at the site in 2023 for the proposed Energy Research and Sustainability Center Project (which will not be constructed), and presented the results of that field investigation, including conclusions and recommendations, in our report titled *Geotechnical Investigation Report, Alternative Site for the Proposed Energy Research and Sustainability Center Project, Cal Poly Humboldt, Arcata, California,* dated July 5, 2023 (Project #023098). In addition, we provided supplemental geotechnical recommendations for the Energy Research and Sustainability Center Project in our letter reports: *Supplemental Geotechnical Recommendations, Existing Basement of the Mary Warren House, Alternative Site for the Proposed Energy Research and Sustainability Center Project, Cal Poly Humboldt, Arcata, California* and dated August 9, 2024, and *Supplemental Geotechnical Recommendations, Proposed Energy Research and Sustainability Center Project, Cal Poly Humboldt, Arcata, California*, dated March 3, 2025.

In 2017, SHN performed a geotechnical investigation for reconstruction of the existing G14 Parking Lot located north of (and adjacent to) the subject site and presented the results in our report titled *Geotechnical Investigation Report, Parking Lots G14 and G15 Reconstruction Project, Humboldt State University, Arcata, California, Revision 2*, dated April 19, 2017 (Project #016325). The 2017 report was initially prepared for improvements that have since been developed. Data from the 2023 subsurface investigation and laboratory testing within the area have been analyzed to develop geotechnical recommendations for design and construction of the improvements for the proposed parking lot.

Discussion and Conclusions

It is our opinion that the project site can be developed as proposed provided the geotechnical recommendations in this letter report are followed. As discussed in our 2023 report, the main geotechnical considerations affecting design and construction of project development at this site are:

- the unengineered fill that was encountered to a depth up to 3 feet,

- the disturbed soils that would occur following the removal of large trees and demolition of the existing buildings and improvements,
- the basement excavation at the former location of the Mary Warren House in the southeastern corner of the site, and
- the need to provide uniform foundation support under the proposed development.

The site is in a seismically active area and is likely to experience strong seismic ground shaking during the design life of the proposed project. The intensity of ground shaking from earthquakes will depend on several factors, including the distance from the site to the earthquake focus, the magnitude and duration of the earthquake, and the response of the underlying soil. At a minimum, it will be necessary to design and construct the project in accordance with the earthquake-resistant provisions of the most recent building codes.

All geotechnical-related work should be performed in accordance with the recommendations of the Geotechnical Engineer-of-Record (GEOR) during construction. Where the recommendations of this report and the cited sections of Title 24 are in conflict, the Owner and Architect should request clarification from the GEOR. The recommendations in this report should not be waived without the consent of the GEOR for the project. The following subsections present recommendations for the geotechnical-related work.

Seismic Design Parameters

Based on the subsurface conditions encountered at our 2023 exploration locations, laboratory test results, and our interpretation of soil conditions within 100 feet of the ground surface, we classify the site as a Site Class D consisting of a “stiff soil profile” in accordance with Chapter 20 of American Society of Civil Engineers (ASCE) 7-22. On this basis, the mapped and design spectral response accelerations were determined using the Structural Engineers Association of California (SEAOC) and California Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps (Accessed 1/19/26) in conjunction with the site class and the site coordinates 40.871835° N, -124.078283 ° W). Calculated values for ASCE 7-22 are presented in Table 1.

Table 1. ASCE 7-22 Spectral Acceleration Parameters
Parking Lot G14, Cal Poly Humboldt, Arcata, California

Parameter	0.2 Seconds	1 Second
Maximum Considered Earthquake Spectral Acceleration (MCE_R)	S_S = 3.44	S_1 = 1.17
Site Class	D	
Site-modified spectral acceleration	S_{MS} = 3.59	S_{M1} = 2.41
Numeric seismic design value	S_{DS} = 2.39	S_{D1} = 1.61
Seismic Design Category (SDC)	E	
Site modified peak ground acceleration (PGA_M)	1.39	

Site Preparation and Grading

General

All grading and site work should be performed in accordance with the 2025 California Building Code (CBC), Title 24, Chapter 33 (Safeguards During Construction), Appendix J (Grading), and Chapter 18 (Soils and Foundations), and with the recommendations of the Geotechnical-Engineer-of-Record (GEOR) during construction. Where the recommendations of this report and the cited sections of Title 24 are in conflict, the owner should request clarification from the GEOR. The recommendations of this report should not be waived without the consent of the GEOR for the project. Recommendations for additional work and construction monitoring are contained in later sections of this report.

Site preparation includes the demolition and removal of existing surface and subsurface improvements, including any utilities, hardscapes, and removal of debris and loose soil caused by demolition. All active or inactive utility lines within the construction areas should be relocated or abandoned. Pipelines to be abandoned in place should be filled with a sand-cement slurry. Site preparation operations should extend at least 5 feet beyond the limits of the proposed site improvements. Where the removal of large trees is required, it will be necessary to remove all major root systems, then backfill the excavations with properly placed engineered fill compacted to at 90 percent relative compaction[1]. The cleared vegetation and debris should be removed from the site. Any soil with more than 2 percent organic material by dry weight should also be removed from the site. Alternatively, soil strippings with more than 2 percent organic material can be stockpiled onsite for reuse in landscape areas.

Existing and former cesspools, septic tanks, and so on, may exist at the site. If cesspools are encountered, they should be excavated to a minimum depth of 5 feet below planned soil subgrade, and the remaining portions cleaned of their contents and backfilled with lean concrete. Septic tanks and/or other underground improvements should be completely removed, and the resulting cavities filled with engineered fill under the direction of the Geotechnical Engineer. Other methods may be appropriate as field conditions dictate.

Subgrade Evaluation and Preparation

Following the surface stripping and backfilling of excavations as described above, the Geotechnical Engineer or qualified representative should review the exposed subgrade surfaces that will support the pavement section. Typically, a firm, unyielding subgrade, compacted to a minimum of 95 percent relative compaction, is required prior to the placement of the recommended pavement section. Because of the variability of the unengineered fill materials and the disturbed soils that occur following the removal of large trees and demolition of the existing buildings and improvements, we anticipate that there will be some areas where the exposed subgrade is not firm and unyielding. Pumping subgrade soils may be encountered with heavy, rubber-tired construction vehicles driving on the exposed subgrade soils.

1 Relative compaction refers to the in-place dry density of a soil expressed as a percentage of the maximum dry density of the same soil, as determined by the ASTM D1557 compaction test procedure. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

Prior to the placement of the pavement section, we recommend that the entire site be proof rolled to determine the suitability of the subgrade for construction of the recommended pavement section. Where mentioned, “proof-rolling” means an inspection during which the Geotechnical Engineer or their qualified representative observes a fully loaded dump truck driving over the improved areas, to ensure that “pumping” and other signs of failure are not evident.

For subgrade areas that exhibit mild pumping, install a layer of triaxial geogrid (Tensar InterAx™ NX750 or equivalent) on the exposed subgrade soils. Follow all geogrid manufacturers’ installation procedures (that is, subgrade preparation, geogrid overlap, and so on). Extend geogrid at least 36 inches beyond the perimeter of soft soil areas. In areas where excessive pumping is observed, 24 inches of over-excavation extending 36 inches beyond the pumping areas may be required. Following over-excavation, install a layer of triaxial geogrid (Tensar InterAx™ NX750 or equivalent) at the base of the over-excavation and backfill the area using crushed aggregate base material in 8-inch loose lifts compacted to a minimum of 90 percent relative compaction to subgrade elevation.

We expect that the site soils will be excavatable with conventional grading and trenching equipment. If grading commences in the winter or spring, or after a period of excessive rainfall, it is likely that the surficial soils will become saturated due to the presence of fine-grained material. Wet or saturated soil may cause difficulties in access with grading and trenching equipment and difficulties in loading, spreading, and compaction of fill material. Moisture conditioning and/or aerating of the site soils may be required. The time required for drying can be reduced by disking, ripping, or otherwise aerating the soil.

The contractor shall be responsible for the stability of all temporary excavations and should comply with the applicable Occupational Safety and Health Administration (OSHA) regulations (California Construction Safety Orders, Title 8). The contractor should periodically monitor all open cuts for evidence of incipient stability failures.

Select Engineered Fills

Select engineered fill should have less than 2 percent by dry weight of vegetation and deleterious material and should meet the gradation requirements presented below in Table 2.

Table 2. Fill Gradation Criteria
Parking Lot G14, Cal Poly Humboldt, Arcata, California

Sieve Designation	Percent Passing by Dry Weight
3-inch (50 mm)[a]	100
2½-inch (37.5 mm)	85 minimum
¾-inch (19 mm)	70 minimum
No. 4 (4.75 mm)	35 minimum, 40 maximum
No. 200 (75 µm)[b]	5 minimum, 40 maximum

a. mm: millimeters

b. µm: micrometers

Fine-grained soil with a liquid limit greater than 35 and a plasticity index greater than 12 should not be used as engineered fill. Crushing and/or removal of rock particles greater than 3 inches in size will be required.

Engineered fill imported to the site should have a low corrosion potential, which is defined as a minimum resistivity of 2,000 ohms-centimeters (ohms-cm) and maximum sulfate and chloride concentrations of 250 parts per million (ppm).

River-run material shall not be used as engineered fill; crushed, angular material is recommended with at least 75 percent of the material (as determined by the material's dry weight) containing a minimum of one fractured face.

Engineered fill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to a minimum of 90 percent relative compaction. The Geotechnical Engineer or qualified representative should approve all fill prior to placement. A qualified field technician should be present to observe fill placement and perform field density tests in accordance with ASTM-International (ASTM) D 6938 at random locations throughout each lift to verify that the specified compaction is being achieved.

Excavation Stability Considerations

Excavations should be made in accordance with OSHA specifications and conditions. Excavations deeper than 4 feet below ground surface (BGS; or shallower if excavations appear unsafe) should be laid back to a safe slope inclination or supported by an appropriate shoring system. Excavated soils should be placed a minimum of 10 feet away from the edge of any below-grade excavation to reduce surcharge loads on the temporary cut slopes. If shoring systems are used, the effects of the soil stockpile on the shoring system should be taken into account during design if the soils are placed in the area between the top of the excavation and a 1H:1V (horizontal to vertical) projection from the toe of the excavation to reduce the potential of a shoring failure.

Similarly, heavy equipment should be operated in a safe manner and should be kept an adequate distance from unshored excavation sidewalls to prevent a cut slope stability hazard. If shoring is used, surcharge loads from heavy equipment should be considered in the design calculations to prevent a surcharge failure during construction. For an unshored excavation, a heavy equipment exclusionary zone should be established based on soil type, depth of excavation, presence of groundwater, and configuration of the open cut. As a general guideline, heavy equipment should be excluded from a zone located between the top of the excavation and a 1H:1V projection from the bottom toe of the adjacent excavation sidewall. This may be modified in the field for specific geotechnical conditions.

Utility Trench Backfill

Unless concrete bedding is required around utilities, bedding should consist of sand having a sand equivalent (SE) of at least 30. The bedding should extend from 6 inches below to 1 foot above the conduit or pipe. Sand bedding should not be jetted or ponded into place and should be mechanically compacted to a minimum 90 percent relative compaction.

In areas to support improvements such as slabs and pavements and adjacent to structure foundations, backfill placed above the bedding in utility trenches (including culvert and sprinkler lines) should be properly placed and adequately compacted to minimize settlement and provide a stable subgrade. If possible, the trench backfill should be compacted following rough grading but prior to final grading and compaction.

Onsite inorganic soils meeting the requirements for engineered fill may be used as trench backfill. Backfill consisting of onsite soils should be placed in layers not exceeding 8 inches in loose thickness, water-conditioned, and compacted to a minimum of 90 percent relative compaction as described for engineered fill. Trench backfill need only be compacted to 85 percent relative compaction in landscape areas or in areas more than 5 feet beyond the limits of structures, pavements, concrete slabs-on-grade, sidewalks, or other flatwork.

Retaining Walls

Retaining walls should be designed to resist static earth pressures, seismic earth pressures, and surcharge pressures. Retaining wall backfill should be placed and compacted according to the recommendations above in Site Preparation and Grading, and drainage should be provided behind walls according to the recommendations that follow. Retaining wall footings may be founded in the competent native soils at the site or engineered select fill. If footings are to be supported by engineered fill, the backfill should meet the material and placement recommendations above in Site Preparation and Grading and Select Engineered Fills. All footings should be founded at least 18 inches below the lowest adjacent finished grade as recommended in Section 5.6 Foundations of our July 5, 2023 report. Such foundations may be designed for an allowable bearing capacity of 2,500 pounds per square foot (psf) for dead plus long-term live loads. This value may be increased by one-third to account for short-term effects of wind and seismic loading.

Provided any new foundations are constructed in accordance with these recommendations, we estimate that total post-construction settlement will be 0.5 inch or less under static conditions; the differential settlement will be about half of the total settlement.

The resistance to lateral loadings may be calculated using a friction factor of 0.30 between the bottom of the footings and the native soil or engineered fill. Where the footings were poured neat and the adjacent ground surface paved or covered with concrete slabs, a passive resistance of 300 pound per cubic foot (pcf) equivalent fluid weight may be developed between the footings and the adjacent soil. Where the adjacent ground surface is not paved, the upper 1 foot should be neglected in determining the available passive resistance.

Active earth pressures may be used for design of unrestrained retaining walls where the top of the wall is free to translate or rotate. To develop active earth pressures, the walls should be capable of deflecting by at least 0.004H (where H is the height of the wall). At-rest earth pressures should be used for design of retaining walls where the wall top is restrained such that the deflections required to develop active soil pressures cannot occur or are undesirable. Cantilever walls retaining firm native soil or engineered fill may be designed for active or at-rest lateral earth pressures for various backfill slopes using the equivalent fluid unit weights presented in Table 3, Equivalent Fluid Unit Weight (pcf).

**Table 3. Equivalent Fluid Unit Weight (pcf)
Parking Lot G14, Cal Poly Humboldt, Arcata, California**

Backfill Slope	At-Rest Conditions	Active Conditions
Level	60	36
3H:1V	81	46
2H:1V	89	55

Lateral earth pressures for backfill slopes other than those given above can be estimated by interpolation. The lateral earth pressures should be applied to a plane extending vertically upward from the base of the heel of the retaining wall to the ground surface.

The lateral earth pressures given above apply where the wall backfill is fully drained, is not subject to traffic or other surcharge loads, and the backfill is not subject to heavy compaction equipment within a distance of one-third the height of the backfill. Lateral surcharge pressures are discussed later in this section.

If retaining wall backfill will be subject to passenger vehicle or light truck traffic loading within a distance of H/2 from the top of the wall (where H is the wall height), the wall should be designed to resist an additional uniform lateral pressure of 72 psf (equivalent to an additional 2 feet of backfill) applied to the back of yielding walls (active conditions), or 124 psf applied to the back of non-yielding walls (at-rest conditions). Surcharge loads imposed by greater loads or unusual loads within a distance of H of the back of the wall should be considered on a case-by-case basis.

Surcharge loads on retaining walls resulting from adjacent building foundations parallel to the retaining wall can be approximated by the following expression:

$\Delta p_h = (4p/\pi)(x^2z/R^4)$

where:

Δp_h = the lateral stress on the wall at depth z
p = magnitude of the footing load (lbs/ft)
x = centerline distance from the footing load to the wall
z = depth below surface
$R^4 = x^4 + z^4$ = the radius from the location on the wall where Δp is measured to the footing load on the surface

Surcharge loads imposed by greater loads or unusual loads within a distance of H of the back of the wall should be considered on a case-by-case basis.

In addition to the active lateral soil pressure, retaining walls supporting more than 6 feet of backfill should be designed to resist additional dynamic earth pressures during earthquake loading. The additional dynamic pressure increment may be calculated using an additional equivalent fluid pressure of 19 pcf for level backfill, 27 pcf for back slopes of 3H:1V, and 55 pcf for back slopes of 2H:1V. The dynamic pressure increment should be applied to the wall as a triangular distribution so the resultant force acts at a distance of 0.33H above the base of the wall (where H is the height of the wall). Under the combined effects of static and dynamic loading, a safety factor of 1.1 against sliding or overturning is acceptable. The dynamic component of the lateral earth pressure was calculated using the Mononabe-Okabe equation and, therefore, assumes that sufficient deformation of the wall will occur during seismic loading to develop active soil conditions.

A drainage system should be constructed on the backside of all retaining walls. The drainage system for backfilled walls should consist of a 4-inch-diameter perforated pipe surrounded by Class 2 Permeable Material complying with Section 68 of the California Department of Transportation (Caltrans) Standard Specifications, latest edition. Alternatively, the perforated pipe may be surrounded by clean coarse gravel or drain rock, provided the gravel or rock is completely separated from the surrounding soil by an engineering filter fabric such as Mirafi 140N or similar fabric. The section of permeable material should be at least 12 inches wide and should extend up the back of the wall to within about 18 inches of finished grade. The drainage material should be capped with compacted fine-grained soil, soil-cement, or other relatively impermeable material or barrier. The pipe should be polyvinyl chloride (PVC) Schedule 40 or acrylonitrile butadiene styrene (ABS) with a standard dimension ratio (SDR) of 35 or better. Perforations in the drainpipe should be ¼ inch in diameter. The perforated pipe should be placed holes-down near the bottom of the section of permeable material and should discharge by gravity to a suitable outlet. Accessible subdrain cleanouts should be provided and maintained on a regular basis.

Backfill within 5 feet, measured horizontally behind new retaining wall structures, should be compacted with relatively light, hand-operated compaction equipment to reduce the potential for the creation of relatively large compaction-induced stresses on the wall. If large or heavy compaction equipment is used, compaction-induced stresses could result in increased lateral earth pressures on retaining walls in addition to those presented in this letter report. In this case, the walls may need to be temporarily braced.

Backfill material should be brought up uniformly around below-grade structures (that is, backfill should be at the same elevation all around the structure as the backfill is placed and compacted). The elevation difference of the backfill surface around the structure should not be greater than 2 feet.

Light Pole Foundations

The planned parking lot lighting should be supported on drilled, reinforced, cast-in-place, concrete friction pier foundations. The piers for the parking lot lighting should be at least 16 inches in diameter and bottomed at least 8 feet below adjacent grade. The piers should be designed using an allowable skin friction of 700 psf of surface area per foot of depth for dead plus long-term live loads. This value may be increased by one-third when evaluating total loads including wind and seismic forces. Eighty percent of these values may be used in determining uplift resistance. The upper 2 feet should be neglected in determining axial capacities.

Please note that the above values are recommended minimum pier dimensions. Other structural criterion, such as the need to resist lateral forces, may force the pier design diameters and depths to be greater.

Resistance to lateral loads can be obtained from passive earth pressure acting on pier faces. A passive earth pressure of 250 pcf (triangular distribution) may be used. Passive pressure can be assumed to act on a width equal to 2 times the pier diameter, to take advantage of edge effects. The upper 2 feet should be neglected in determining lateral capacities.

The bottoms of the pier excavations should be substantially free of loose cuttings and soil slough and tamped prior to the installation of reinforcing steel and the placement of concrete. In addition, any significant amounts of accumulated water in the pier excavations should be pumped out prior to placing concrete or displaced using the tremie method when placing concrete. The Geotechnical Engineer, or approved representative, should observe the pier excavations to evaluate whether the piers are founded in the supportive material and whether the pier excavations are properly prepared. The pier depths recommended above may require adjustment if differing conditions are encountered during excavation. Pier excavations should be filled with concrete as soon as practical after drilling to minimize the potential for caving.

Pavement Design

Pavement construction should conform to the requirements of the Caltrans Standard Specifications, latest edition. Recommendations for both asphalt concrete and Portland cement concrete pavements are given in this section.

Our recommendations for flexible asphalt pavement sections in the proposed parking lot are based on an R-Value of 17 from our 2017 report, assumed traffic indices (T.I.) of 6.0 for the heavily traveled driveway and truck service areas in the parking lot, 4.5 for automobile parking areas, and the California Standard Procedure for Flexible Asphalt Pavement. We recommend design pavement sections consisting of:

Driveway and Truck Service Areas

Asphalt Concrete (AC) Pavement Thickness:	0.25 feet
Class 2 Aggregate Base Thickness:	0.90 feet

Parking Stalls and Very Light Traffic Areas

AC Pavement Thickness:	0.20 feet
Class 2 Aggregate Base Thickness:	0.60 feet

Aggregate used for asphalt concrete surfacing should conform to the grading specified in Section 39 of the Caltrans Standard Specifications for 9.5 millimeters (mm) or 12.5 mm (⅜ inch or ½ inch) maximum, medium grading. Asphalt concrete surfacing should be placed in a single lift. Base rock aggregate should comply with the minimum requirements for Class 2 Aggregate Base rock specified in Caltrans Standard Specifications Section 26 and should be compacted to at least 95 percent relative compaction; crushed, angular material should have at least 50 percent of the material (as determined by the material's dry weight) containing a minimum of two fractured faces.

We recommend that exterior concrete pavements consist of at least 6 inches of aggregate base rock beneath at least 6 inches of concrete. For durability and wear resistance, all Portland cement concrete pavements should have a minimum compressive strength of 4,000 pounds per square inch (psi). A modulus of subgrade reaction, k_v (30-inch circular plate) of 150 psi may be used for design of Portland cement concrete pavements. Contraction joints should be constructed at 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt concrete pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 10H:1V. Recommendations for subgrade preparation and aggregate base compaction for concrete pavement are the same as those for asphalt concrete described above.

Paved areas should be sloped and adequately drained to prevent surface water or subsurface seepage from saturating the pavement subgrade soil. All curbs surrounding landscape areas should be embedded at least 6 inches into the soil subgrade to minimize the migration of water beneath pavements.

Closure

During construction, we recommend that a representative of our firm confirm site conditions during site grading and installation of foundations. If subsurface conditions differ significantly from those disclosed by our 2023 investigation, we should be given the opportunity to re-evaluate the applicability of our conclusions and recommendations. Some alteration of recommendations may be appropriate.

The recommendations provided in this report are based on the assumption that we will be retained to provide the construction monitoring described above in order to evaluate compliance with our recommendations. If we are not retained for these services, SHN cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of this report by others. Furthermore, if another geotechnical consultant is retained for follow-up service to this report, SHN will at that time cease to be the GEOR.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of a property can occur with the passage of time, whether due to natural processes or the works of man, on this or adjacent properties. In addition, changes in applicable standards of practice can occur, whether from legislation or the broadening of knowledge.

Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of 3 years. In addition, this report should not be used and is not applicable for any property other than that evaluated.

If you have any comments or concerns, please call me at (707) 459-4518.

Sincerely,

SHN

REGISTERED PROFESSIONAL ENGINEER
JOHN H. DAILEY
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John H. Dailey, PE, GE
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