ATTACHMENT E

September 15, 2022
G5069A

Sara A. Clark
Shute, Mihaly and Weinberger, LLP
396 Hayes Street
San Francisco, CA 94102

SUBJECT: **Engineering Geologic and Geotechnical Evaluation**
RE: Proposed Sargent Quarry
Gilroy, CA

Dear Ms. Clark,

At your request, Cotton, Shires and Associates, Inc. (CSA) is providing you with this summary of opinions pertaining to the geologic and geotechnical aspects of the geotechnical investigation, draft environmental impact report, and reclamation plan for the proposed Sargent Quarry, Gilroy, California. We understand that the proposed quarry operators would be mining aggregate from bedrock sources (not an alluvial source), which will result in hillside excavations with the potential for slope destabilization, groundwater alterations, and surface drainage changes. We reviewed the following documents as part of our evaluation:

1) Sargent Quarry Mining & Reclamation Plan (M&RP), prepared by Freeman Associates, dated February 2022;

2) Revised Geotechnical Slope Stability Analysis Report for Proposed Sargent Ranch Quarry Site, prepared by Sierra Geotechnical Services, dated September 12, 2016;

3) Geologic Hazards Assessment and Preliminary Slope Stability Evaluation, Sargent Ranch Quarry Site, prepared by Sierra Geotechnical Services, dated January 20, 2015; and

4) Draft Environmental Impact Report (SCH#2016072058), Sargent Ranch Quarry, prepared by County of Santa Clara with Technical Assistance by Environmental Science Associates (ESA), dated July 2022.

Executive Summary

Based on our review of the documents listed above, we conclude the geologic and geotechnical investigation by SGS failed to provide a sufficient number of grain size tests in the documents reviewed, or boring log analyses to form opinions regarding the percentages of gravel, sand and clay available for market, or for use as compacted fill on the reclaimed slopes. Additionally, SGS failed to gather sufficient data to enable appropriate slope stability analyses, and thus, there is insufficient data to form opinions regarding the future static and seismic stability of the permanent slopes and the proposed stockpiles. Thus, we recommend a supplemental geotechnical investigation be performed that should include, but not necessarily be limited to: 1) additional small diameter borings in the areas to be mined, and in the proposed stockpile area; 2) use of the borings to confirm or modify subsurface geologic conditions presented on the engineering geologic cross sections; 3) conversion of the exploratory boreholes to multi-stage piezometers for groundwater and/or perched groundwater modeling; 4) the procurement of high-quality undisturbed samples of all earth units to be modeled in slope stability analyses, and bulk samples of the material to be mined for grain size testing. Bulk samples should also be obtained on the materials to be used as compacted buttress fills and to be compacted in the stockpile area; 5) additional laboratory testing and ore analyses, as noted in the comments below; and 6) additional static and seismic slope stability analyses (as noted in the comments below). Without this information, conclusions reached in the EIR and associated documents are not fully supported by substantial evidence.

If the above supplemental analyses demonstrate adequate stability of reclaimed slopes, and project approval is granted, we recommend that the project geotechnical consultant make sufficient site visits during the lifetime of the project to include, but not be limited to, the following tasks: 1) engineering geologic mapping of all cutslopes; 2) compaction testing of all buttress and stockpile fills; 3) mapping of as-built drainage measures; 4) mapping and descriptions of additional as-built mitigation measures that were needed during excavation; 5) supplemental stability analyses, if needed, related to changed conditions; and 6) the preparation of an as-built report to the County which summarizes these tasks, as well a statement regarding whether the quarry slopes were constructed as designed, and their anticipated long term stability. This information is necessary to document any changes from the investigated conditions to the as-built conditions, and that the grading and restoration of the quarry slopes was constructed as designed by the project geotechnical consultant, or if changes were implemented, that these changes meet the stability criteria of the geotechnical report and the reclamation plan.

1.0 Geotechnical Comments on SGS Investigation Reports

Earth Material Properties from Laboratory Testing

a. The M&RP (Section 3.5.1.1 and 3.5.1.2) indicates the mined material is expected to contain 50% sand, 25% gravel and 25% fines (clay and silt), with the sand and gravel being the primary resources being mined. However, we did not find an explanation of how these estimates were made, nor did we find sufficient data to enable the grain size breakdown. The 2016 geotechnical report provides the results of only four particle size laboratory tests.

> The viability of the proposed project and the estimated quantity of materials that will be needed for market, for buttress fills, or to be stockpiled on site is highly dependent on the proportions of gravel and sand versus fines. Consequently, the grain-size data relied upon for these estimates should be included and accompanied with an explanation to support the estimated percent gradation. If there are no additional grain size tests utilized aside from the four tests presented in the 2016 geotechnical report, additional grain size testing should be performed in the supplemental geotechnical report.

b. The geotechnical report included surface mapping and a subsurface exploration program that included 11 large diameter borings and 43 test pits. Laboratory testing by the Geotechnical Consultant included 2 expansion index tests, 3 Atterberg limit tests, 4 direct shear tests and 4 particle size tests. However, testing performed on two of the direct shears, one of the Atterberg tests and two of the particle size tests had sample numbers not listed on the boring or test pit logs, and it is not clear where they were obtained. There does not appear to have been any remolded strength tests on the proposed compacted materials to be used as the fill buttress for the final reclaimed 3H:1V slopes, and no unit weight testing appears to have been done. We also did not see the results of a Hoek-Brown analysis to estimate the rock mass strength, as was indicated in the M&RP (4.5.2 Page 79, item CS-2).

> The amount of laboratory testing for slope stability analyses completed and included in the DEIR and appendices is atypical for a project of this size and scope. Consequently, the supplemental geotechnical report should include more index testing, as well as unit weight testing; consolidated, undrained triaxial strength tests with pore pressure

readings on undisturbed samples; unconfined compression tests on rock-like materials; remolded torsional ring shear testing of the clay beds and clay gouge encountered in the faults and shears for peak and residual strengths; and hydrometer particle size tests on the clay beds and clay gouge materials for use in correlations. In addition, bulk samples of the materials to be used as compacted fill for the reclaimed 3H:1V buttresses and the permanent stockpile area should be acquired, blended, and tested for grain size, maximum dry unit weight, and optimum moisture content, and remolded to the specified relative compaction and testing for consolidated, undrained triaxial strength testing. Triaxial tests with pore pressure readings are preferred over direct shear tests as there is complete control over the drainage which allows for both drained and undrained strength parameters, and less chance of gravel size particles impacting the results. The undisturbed samples should be acquired using small diameter drilling methods (instead of large diameter drilling rigs or test pits) utilizing pitcher barrels fitted with Shelby tubes to minimize the amount of sample disturbance. In addition, Hoek-Brown analyses should be performed to estimate the strengths of the rock units.

The clay beds are described generally as very stiff to hard, and thus may be over-consolidated. Softening and strength loss of over-consolidated clays may occur after the materials are unloaded (in this case by mining) and subject to surface water infiltration. In addition, small changes in the cohesion value can significantly impact the safety factor, especially for the long failure surfaces analyzed in the slope stability analyses. Thus, slope stability analysis should use fully softened strengths (with no cohesion) for the clay beds typically derived from torsional ring shear tests or correlations with liquid limits and clay fraction from hydrometer testing (e.g., Stark and Eid, 1997).

Many of the clay earth materials at the site were described as either faulted or sheared, and the clay particles are likely aligned and at (or very near) residual strength. For these types of materials, the use of a cohesion value is not recommended (e.g. Stark, Choi and McCone, 2005) and the use of residual strengths from torsional ring shear tests or from correlations should be used (e.g. Stark and Idriss, 2021)

After testing and analyzing the laboratory results, the Geotechnical Consultant should prepare a comprehensive explanation of how the shear strength parameters were selected for each earth unit modeled in slope stability analyses. The selected shear strengths should be based on site-specific laboratory test results, and/or index test results correlated with shear strengths in accordance with procedures outlined in published papers.

c. For the slope stability analyses, two strength envelopes were used: Ø=32° and C = 300 psf for "cross bedding" cases, and Ø=12° and C = 375 psf for "daylight" cases (exposed clay bedding planes, or dip slope conditions).

The cross bedding strengths (Ø=32° and C = 300 psf) appear to have been taken from one direct shear test with only two points. For a project of this scope and size, multiple strength tests should be performed, and as noted above in Comment 1b, triaxial testing is preferred. The clay bedding plane strength appears to have been estimated using back-calculation analyses performed on L-L′ and on G-G′. Although there is a large diameter boring on L-L′, the boring did not appear to penetrate the basal shear surface. No subsurface explorations were projected onto G-G′. Thus, while back-calculations can be very important data for estimating earth strength parameters, both the subsurface geometry and groundwater conditions for the two back-calculated cases are largely unknown, and the resulting back-calculation analyzed strengths may not be representative of actual conditions. Sufficient additional borings should be excavated in these areas to better define the stratigraphy, obtain shear gouge samples for residual strength correlations as noted above, and to install piezometers prior to back-calculation analyses.

Slope Stability Analyses

d. The slope stability analyses performed by the Geotechnical Consultant used either Bishops method (which solves for moment equilibrium) or the Janbu method (which solves for force equilibrium).

The current standard of practice is to use stability methods that satisfy both moment and force equilibrium (e.g., Spencers Method), which will provide more accurate calculations of the safety factors of the reclaimed slopes.

e. Slope stability analysis results for 19 different cases were provided in Table III, and the stability analysis graphic plots were provided in an appendix. None of the appendix plots are denoted by a case number, and the plots are not in the same order listed on Table III. We attempted to identify the cases by using other information given on the plots, however, we are not certain we correctly paired all of the plots with the correct case numbers.

In the supplemental geotechnical report, the case numbers should be added to the plots for clarity, and a brief discussion of the purpose, results and conclusions of each case should be provided.

f. The plots that we suspect represent Cases 9a and 9b appear to show a saturated unit weight of 12.0 pcf for one of the units.

This is likely a data entry mistake, and we suspect 120.0 pcf was intended to represent the unit weight. This should be corrected in the supplemental geotechnical report.

g. Although the geotechnical report mentions that the proposed reclamation slopes were analyzed, we could not find supporting stability analyses of the proposed final remediated slopes (i.e., 3H:1V fill buttresses overlying a bedrock cut at 2H:1V slopes) to verify their long-term stability. Without this information, the EIR's conclusions about the future stability of the slopes are either unsupported or unclear.

For a supplemental geotechnical report, the Geotechnical Consultant should perform stability analyses of the final remediated slopes under static and seismic conditions using remolded consolidated undrained triaxial strength tests on materials that are planned to be used for compacted fill. The stability model should include bedrock, compacted fill and topsoil, the 2H:1V cut into bedrock, and the 3H:1V buttress fill wedge with keyways and benches.

h. The 2015 report provides estimates of seismic shaking at the site from earthquake activity. This information was not included in the 2016 report. The ground motion that has a 10% probability of being exceeded in 50 years (475 year return period) has a peak ground acceleration of 1.40g, and the motion that has a 2% chance in 50 years (2,475 year return period) has a peak ground acceleration of 2.2 g. These values are considered high.

This information should be included in a supplemental geotechnical report and incorporated into the EIR.

i. Seismic shaking was modeled using horizontal and vertical pseudo-static seismic coefficients of 0.15.

This type of pseudo-static analysis is typically performed as a screening analysis using a seismic coefficient calculated based on Mw, distance from the fault and maximum horizontal acceleration (CGS, SP117A) prior to using methods that estimate seismic deformations with site specific seismic parameters (e.g., Bray et al, 2007 and 2019). Deformations are important due to the high expected site accelerations mentioned in the 2015 geotechnical report, and due to six stability cases that were analyzed (with daylighting clay beds on 2H:1V mined slopes) that show static safety factors below one (i.e. static failure is likely). These types of slopes could undergo large deformations during future earthquakes even if remediated with 3H:1V buttress fills. Thus, the magnitude of seismic deformation for permanent slopes should be provided in the supplemental geotechnical report in order to show how much displacement could occur during a large seismic event.

j. Groundwater information was taken from 11 large diameter holes and 3 air hammer holes.

No piezometers were installed to measure long term and/or perched groundwater levels. Thus, the phreatic surface for the back-calculation analyses of current conditions, and forward analyses that modeled proposed mining cutslopes, are supported by very few data points. Additional small diameter borings are recommended as part of a supplemental geotechnical investigation, and should be converted to piezometers after the drilling is finished. We recommend they be grouted with multiple vibrating wire piezometers at various elevations to better define the groundwater table, and/or perched groundwater levels.

k. Out of the 19 cases analyzed, only one planar failure was modeled. However, the large diameter boring logs show the earth materials are highly sheared and faulted in various areas, and the report notes that landslide backscarps in a few areas appeared to originate along fault planes and fractures (page 7). Thus, predominate discontinuities are likely to play a key structural role in the formation of landslides.

Based on this information, the analysis of structurally controlled landslides should be included in a supplemental geotechnical report in order to demonstrate that all possible landslide configurations have been appropriately characterized and analyzed. This analysis typically includes a stereonet summary of joints, fractures, shears, bedding or faults to identify major discontinuities and preferred orientations, and kinematic analyses of the major discontinuities to identify and analyze potentially unstable blocks. The results of weaker bedding planes/discontinuities should be used in the slope stability analysis to optimize these weaker planes using block and/or wedge shaped failure surfaces.

l. We could not find the location of cross Section N-N′ on the site geologic map (Figure 3).

The location of Section N-N′ should be provided in a supplemental geotechnical report.

m. The M&RP shows that a permanent stockpile up to 90 feet thick of overburden materials will be located between Pit 1 and the processing plant near the northeast corner of the proposed project (Figure 9 of the MR&P). It appears that the analysis of Civil Section A-A′ (Cases 9a and 9b) addresses the stability of this proposed feature; however, we could not find the location of this section on the geotechnical site map, and it does not appear that any subsurface borings were drilled in the stockpile area. The Case 9a and 9b stability plots for Civil Section A-A′ model only one unit. Table III suggests that lab strengths were used in this analysis; however, the two strength envelopes used (Ø=12°, C = 375 psf, and Ø=12°, C = 675 psf) match the back-calculation analysis results of native materials performed on Sections L-L′ and G-G′ (Cases 5 and 6).

The subsurface conditions of the stockpile area should be investigated during the work for a supplemental geotechnical report to define the limits of soil and bedrock. The static safety factor and estimated seismic deformations of the stockpile and surrounding slopes should be analyzed on a cross section that depicts the stockpile material, and the underlying soil and bedrock. The strengths of the stockpile material should be based on remolded consolidated undrained triaxial strength tests on the blended materials that are anticipated to be used. The strengths of the underlying soil and bedrock should be derived using the same techniques discussed above in Comment 1b. The Geotechnical Consultant should provide recommendations for construction of the stockpile that show the depths of the keyway and benches, slope gradients, compaction requirements and subsurface drainage in order to ensure the future stability of the stockpile.

2.0 <u>Sargent Quarry Mining and Reclamation Plan (M&RP) Comments</u>

n. Inconsistency - The M&RP includes the 2015 Sierra Geotechnical Report as Appendix D, and another separate PDF document in the EIR package (but not in the M&RP) is the 2016 Sierra Geotechnical Services report. We note the text and graphics of the 2015 geotechnical report are consistent with the excavation phases discussed and shown graphically in the M&RP. However, the text (Page 2) and graphics (Figure 3) of the newer 2016 report are not consistent with the M&RP phases. Also, the 2015 report includes ground motion information and data from electrical resistance surveys, but this information is not included in the 2016 report.

> A supplemental geotechnical report should be consistent with the M&RP and should contain (or provide references to) all analyses and data that were relied upon for their recommendations and conclusions.

o. Inconsistency - The M&RP indicates that mining will result in 2H:1V temporary slopes with 10′ benches at 40 feet vertical spacing (3.6.2, Page 44). These mined 2H:1V slopes will then be overlain with fill buttresses at a maximum gradient of 3H:1V using crushed rock (less than 6-inches in size) compacted to 95% relative compaction in 8- to 12-inch lifts, and the upper foot of material will be soil for planting purposes (3.6.9 Page 46). A later entry (4.5.1 Page 76) indicates the backfill materials can include clay overburden up to 5 feet below the ground surface compacted to 90% relative compaction and the upper 5 feet will be granular material suitable for plant growth.

> In a supplemental geotechnical report, the geotechnical consultant should include a typical sketch showing the intended details of the fill buttresses. The Consultant should also provide recommendations to ensure a uniform, well graded/blended fill material and include the anticipated USC soil type, the topsoil thickness, the keyway and bench details, the compaction requirements of the fill and topsoil and drainage details. These details should be incorporated into future stability analyses of the buttress fills.

p. Inconsistency - There appears to be a discrepancy regarding the Phase 4 base elevation (El) between the current version of the M&RP and both the 2015 and the 2016 geotechnical reports. The base elevation of Phase 4 on M&RP Figure 14 (Section A-A′) and Figure 15 (Section D-D′) is shown to be El. 315′. These figures were prepared by Triad Holmes Assoc. and are dated 12/1/2020. However, Figure 14 of the 2015 geotechnical report (which is attached to the current M&RP as Appendix D) is an earlier design sheet by Triad/Holmes Assoc. dated 8/13/2014 that shows the base of the Phase 4 pit to be El. 245′. In the 2016 geotechnical report, both engineering geologic cross sections L-L′ and M-M′ show the base of Phase 4 cut to also be at El 245′.

This discrepancy should be addressed in a supplemental geotechnical report and revised M&RP.

q. The M&RP indicates that after reclamation is complete, the quarry will remain suitable for future cattle ranching, agriculture <u>or any use permitted</u> by the County General Plan and Zoning ordinance. The preferred use is restoring the property back to cattle ranching (4.3.1 Pages 64 and 65). The 2016 geotechnical report states that the site is presently undeveloped open space, and it is understood that the end use will be the same. Thus, slope stability will not represent a hazard to structures or human occupancy.

Due to housing demand, many old quarries in the Bay Area (which were never intended to be developed) have been converted to residential housing sites, and slope stability failures and property damage after the development was completed are not uncommon and have resulted in safety hazards, damaged residences, litigation, and expensive repairs. The level of slope stability risk associated with housing is much higher than open space for cattle ranching, thus, the geotechnical consultant should comment on whether the analyses and recommendations in their geotechnical report are considered suitable for possible future residential construction, and consider providing suitable setbacks from the toe and top of slopes based on slope stability analyses.

r. The 2016 geotechnical report states that due to limited geometric data available with respect to the complexities of the site, geologic inspections during pit excavations are considered essential to identify field conditions that differ from those anticipated (Page 11). The M&RP (3.6.12 Page 47) states that geotechnical site visits and reports to the County will be made each time a 30-foot bench has been completed for the first three years, and from time to time after that. In Section 4.7.3, a financial assurance cost estimate is presented for reclamation. In Table 13 (Page 103) five geotechnical monitoring visits are proposed for the reclamation cost estimate.

We agree that site visits by the project geotechnical engineer are essential during excavations to observe the exposed geologic conditions in order to ensure that the site conditions are as anticipated, and that the slopes are stable and performing adequately. These site visits should include engineering geologic mapping of new exposures, concentrating on discontinuities, earth material contacts, seepage and signs of slope instability. Exposed conditions may warrant additional laboratory testing, analyses and alteration of remediation plans. We also consider it essential that the project geotechnical engineer inspect and test the compaction of the buttress fills as they are being constructed. Based on our understanding of the proposed mining (3.6.1 Page 43), there will be areas of the quarry undergoing reclamation (e.g. buttress construction) while other areas are being excavated. Thus, geotechnical site visits will likely be needed more often than every time a 30-foot bench is completed. We expect that as the quarry pits become deeper, more slope stability hazards are likely to emerge, and as this is expected to be a 30-year project, regular site visits and mapping should take place over the lifetime of the project. We recommend that the project Geotechnical Engineer perform the number of visits required throughout the lifetime of the project sufficient for them to provide an as-built report to the County upon project completion. The report should certify that the cuts and buttress fills were made in accordance with their recommendations and provide a statement regarding the anticipated future stability of the slopes. The as-built report should also include the results of compaction testing, the locations of keyways and subdrains, additional mitigation measures that were needed during excavation, maps and/or sections showing these measures, any additional laboratory testing or slope stability analyses performed, and an engineering geologic map showing the cumulative results of their cutslope observations and mapping. The financial assurance cost estimate should be revised to account for this additional monitoring and reporting.

3.0 Geologic Comments on Faulting/Seismicity

s. Sargent Fault Zone – The main trace (northern trace) of the active Sargent fault zone has been mapped just north of the Processing Plant, Permanent Overburden Stockpile Area, and Pit #1. This northern trace has been zoned according to the State Alquist-Priolo Fault zoning Act as a Holocene active fault. A secondary splay fault has been mapped just south of these three areas, and extends in a NW-SE trend along the southern edge of Pit #1. Two traces of the fault are shown on the Geologic Map by SGS based on published map location interpretations (i.e., USGS OFR 97-210; and USGS Modified by RWS). In addition, a third trace (aerial photo lineament) is shown on their geologic map in the vicinity of the published map traces. One of the traces extends directly through Pit #1. Their report also doocuments many faults and shears in their subsurface exploration, and many other faults are shown on their Geologic Map aside from the Sargent fault. Our primary concerns with regard to the Sargent fault, and other splay faults, is: 1) to avoid placing permanent structures for human occupation across these faults; and 2) the potential adverse impacts of these shear zones on slope stability, for both temporary and permanent slopes. Faults often provide a plane of weakness upon which landslides mobilize.

Quarry excavations will alter the stability state of the site slopes and these planes of weakness should be identified and characterized during quarrying.

t. Seismic Hazards Mapping Act – The Draft EIR mentions that the site was not part of the Seismic Hazards Mapping Program. Our review of the Geologic and Geotechnical Reports reveals that the SGS investigation was performed with a scope of work that is consistent with a site containing Landslide Hazards, as typically defined by the State Seismic Hazards Mapping Program.

u. Siting of Structures – The processing plant/business offices that will contain human occupancy at 40-hour weeks/2000-hours/year should be cleared by the Project Geologist with a geologic investigation to assure that these structures are not constructed atop the active Sargent fault, or an associated splay fault.

Limitations

Our services consist of professional opinions and recommendations made in accordance with generally accepted engineering geology principles and practices. No warranty, expressed or implied, or merchantability of fitness, is made or intended in connection with our work, by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings. Our findings and preliminary conclusions may change as additional data is made available.

We trust that this report provides you with the information that you need at this time. If you have any questions, or need additional information, please contact us.

Very truly yours,
COTTON, SHIRES AND ASSOCIATES, INC.

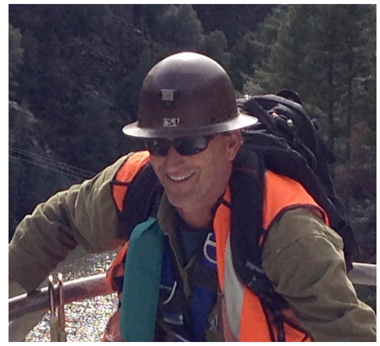
John M. Wallace
Principal Engineering Geologist, CEG 1923

David T. Schrier

David T. Schrier
Principal Geotechnical Engineer
GE 2334

JMW:DTS:DRM

CURRICULUM VITAE – **JOHN M. WALLACE**



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Registration

California Professional Geologist, PG 6151, February 8, 1995
California Certified Engineering Geologist, CEG 1923, February 8, 1995

Education

M.S. Geology: San Jose State University, San Jose, California, 1991
B.S. Geology: University of Southern California, Los Angeles, California, 1985

Professional History

Staff to Principal Engineering Geologist, 1990 - Present; Cotton, Shires and Associates, Inc., Los Gatos, California.

Field Geologist, 1986-1988; Electrowatt Engineers/Gibbs and Hill, North Fork Stanislaus Hydroelectric Project, Murphys, California.

Field Geologist, 1986; United States Geological Survey, Denver, Colorado.

Representative Experience

Mr. Wallace has over 34 years of experience in the fields of geology and engineering geology, working on projects in both northern and southern California as well as Colorado, Utah, Idaho, Hawaii, and North and South Dakota. Mr. Wallace has performed geologic mapping and evaluation of steep rock slopes affecting more than 30 penstocks, 20 dams and powerhouses, 60 canals, and 4 tunnels primarily within PG&E's hydro-generation facilities in the northern, central and southern Sierra Nevada, in addition to extensive experience in hydro-projects such as mapping dam abutments, tunnels and penstock alignments, as well as tunnel, dam abutment, and portal rock bolting. Many of these projects involved using rock climbing techniques to safely access steep rock slopes. Mr. Wallace has been involved in numerous rock slope instability investigations on steep rock slopes within the City of San Francisco, involving steep rock slope mapping, characterization, and identifying mechanisms of rock slope failure.

He has extensive experience in coastal geologic processes, coastal landslide investigation, characterization, and mitigation, and recently performed detailed geologic investigations of coastal bluff properties in San Luis Obispo County, San Mateo County, Santa Cruz County, Mendocino County and Santa Barbara County. In addition, he has recently investigated several large, active landslides that severely distressed roadways and residential areas, including the Sycamore Ranchito Landslide in Santa Barbara, the Northbeach Rockslide in San Francisco, the Ocean Trails Landslide in Rancho Palos Verdes, and the Montellano Landslide in Los Angeles. He has investigated large landslides in Utah and Idaho, including the Green Hollow Landslide in Cedar City, and the North Alto Via Landslide in Boise. These projects involved detailed surface and subsurface investigation, instrumentation, and analysis. Mr. Wallace has also been involved with geologic mapping and siting studies for several fault and landslide constrained reservoirs, and recently mapped unstable coastal bluffs in Santa Barbara, San Luis Obispo, Mendocino, Bodega Bay, Capitola, Aptos, Montara and Pacifica.

Mr. Wallace's extensive experience on a wide variety of large- and small-scale investigations has provided a solid background for performing peer reviews for various communities over several decades, including Portola Valley, Cupertino, and San Francisco. Mr. Wallace has also been involved in very large, select peer reviews on large landslides and proposed developments in Rancho Palos Verdes, Rolling Hills Estates, and for the California Coastal Commission.

As a field geologist with Electrowatt/Gibbs and Hill from 1986 to 1988, Mr. Wallace participated in the exploration and construction phases of the North Fork Stanislaus Hydroelectric Project, where he was involved in siting studies for four dam sites (including one thin-arch concrete dam, one concrete-face rockfill dam, and two concrete gravity dams) and over ten miles of pressure tunnel and shafts. His responsibilities included geologic mapping, exploratory drilling and core logging, rock bolt support layout for dam abutments, geotechnical instrumentation installation and monitoring, exploratory trench logging, and extensive tunnel mapping of 10 miles of pressure tunnels and shafts, tunnel rock bolt support layout, and pressure grouting supervision.

Mr. Wallace's current duties include: research and compilation of pertinent geologic data; photogeologic mapping from aerial photographs; large-scale and regional engineering geologic field mapping; coordination, logging, and analysis of subsurface exploration programs, including downhole logging of large-diameter exploratory borings; geologic mapping of precipitous rock slopes using rock climbing techniques; installation and monitoring of slope inclinometers and piezometers; the final preparation of technical reports, maps and cross sections; attendance at and giving technical talks at professional conferences, and expertise witness testimony.

Mr. Wallace has considerable experience as an expert witness for a variety of geologic issues, including landsliding, debris flows, rock characterization, seacliff instability, and rockfalls. Mr. Wallace has testified in 6 trials, 1 binding arbitration, and been deposed on 13 separate occasions as an expert witness.

Professional Affiliations

Association of Engineering Geologists
Earthquake Engineering Research Institute

Professional Short Course Instructor: 2012 - 2015

University of Wisconsin-Madison, College of Engineering and Department of Engineering and Professional Development; *Slope Stability and Landslides, Course #904.* Yearly 3-day professional development course. Professor James M. Tinjum Program Director.

Selected Publications/Abstracts

HISTORY AND MECHANISMS OF ROCK SLOPE INSTABILITY ALONG TELEGRAPH HILL, SAN FRANCISCO CALIFORNIA, 2015, (with Dale R. Marcum), Published Paper accepted for the 49th U.S. Rock Mechanics/Geomechanics Symposium.

GEOLOGIC ENGINEERING TOUR OF SAN FRANCISCO AND THE SAN FRANCISCO PENINSULA, 2015, (with R. GOODMAN, D. MARCUM and E. Medley) American Rock Mechanics Association, 49th US Rock Mechanics/Geomechanics Symposium, Guide Book co-author and field trip co-leader.

DEEP ROCK TOPPLING DISTRESS AT BELDEN TUNNEL AND SIPHON, SIERRA NEVADA, CALIFORNIA, 2011, (with D. Marcum), Paper submitted and accepted for the 13th International Conference and Field Trips on Landslides, Kyoto, Japan.

WOODLEAF ROCKFALL, NORTHERN SIERRA NEVADA, CALIFORNIA: KEEPING THE POWERHOUSE OPERATING AFTER A NEAR MISS, 2011, (with D. MARCUM), Paper submitted and accepted for the 13th International Conference and Field Trips on Landslides, Kyoto, Japan.

THE HIDDEN COMPLEXITY OF A DEEP-SEATED LANDSLIDE IN RICHMOND, CALIFORNIA, 2011, (with JOHNSON, Philip L.), Abstract submitted and accepted for the 11th International & 2nd North American Symposium on Landslides, Banff, Canada.

DETAILED GEOLOGIC MAPPING UNCOVERS PREHISTORIC LANDSLIDE DAM IN THE RIDGE BASIN, CALIFORNIA, 2011, (with JOHNSON, Philip L.), Abstract submitted and accepted for the 11th International & 2nd North American Symposium on Landslides, 2012, Banff, Canada.

COMPLEX INTERPLAY BETWEEN TOPPLING, SLIDING, AND STRESS CHANGES ASSOCIATED WITH A MASSIVE LANDSLIDE, SANTA BARBARA, (with SHIRES, Patrick O., DURDELLA, Milton J., SNEDDON, Tim P.), in Program with Abstracts, 2009 Association of Engineering Geologists Annual Convention, South Lake Tahoe.

ROCKFALL HAZARD EVALUATION AT THE KERN INTAKE, KERN CANYON PENSTOCK, KERN RIVER, CALIFORNIA: 2003 (with William D. Page, Dale R. Marcum and Joseph M. Durdella), in Program with Abstracts, Association of Engineering Geologists, 2003, Annual Meeting, Page 70.

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