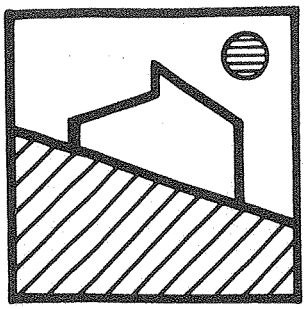
Appendix E.1a Geology Report Peer Review



Grover
Hollingsworth
and Associates, Inc.

THIRD-PARTY GEOLOGIC AND SOILS ENGINEERING REVIEW

Proposed Parking Structure Retaining Walls
and Pedestrian Bridge
APNs 2385-018-001, -002, -003 and -011; 2384-007-005,
2385-019-016, -015, -014, -013, -017, -049, -050 and -051;
Arbs 1 and 2, Fraction of Lot 35, Tract 6293, Arbs 1 and 2, Lot 1111,
and Arb 45, Portion of Lot 1112, Tract 1000, and Arbs 1 and 2, Lot 65,
and Lots 63, 64, 66, 67, 68 and 69, Tract 7442;
3675, 3693, 3703, 3709, 3707, 3717, 3719, 3705 and 3700 N. Coldwater Canyon Avenue; 12908, 12916, 12924 and 12930 W. Hacienda Drive;
and 3686 and 3678 N. Potosi Avenue
Studio City, California.
For
HARVARD-WESTLAKE SCHOOL

October 23, 2015 GH16870-G

Engineering Geology Geotechnical Engineering
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THIRD-PARTY GEOLOGIC AND SOILS ENGINEERING REVIEW
PROPOSED PARKING STRUCTURE RETAINING WALLS
AND PEDESTRIAN BRIDGE
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3675, 3693, 3703, 3709, 3707, 3717, 3719, 3705 AND 3700 N. COLDWATER CANYON AVENUE; 12908, 12916, 12924 AND 12930 W. HACIENDA DRIVE; AND 3686 AND 3678 N. POTOSI AVENUE
STUDIO CITY, CALIFORNIA.
FOR
HARVARD-WESTLAKE SCHOOL

INTRODUCTION

Harvard-Westlake School ("School") has proposed to construct a parking structure with a rooftop synthetic-turf athletic field on the west side of Coldwater Canyon Avenue, adjacent to the School's main campus. The development site for the parking structure will be created through the use of soil-nail and conventional retaining walls. A pedestrian bridge will connect the parking garage to the main School campus. The School has retained geotechnical consultants to evaluate the suitability of the site given the project scope and design. A detailed analysis has been conducted by Byer Geotechnical, Inc. We understand that the School's paramount concern as it relates to this project is the safety of those who will occupy the structure, the safety of the surrounding community, and protecting the use of Coldwater Canyon Avenue as a public thoroughfare. For that reason, the School retained Grover-Hollingsworth and Associates, Inc., to review the existing geotechnical findings and determine, in an unbiased manner, whether the site's geologic and geotechnical engineering conditions support the safe construction and operation of the project.

As part of our work, we participated with Byer Geotechnical, Inc., in the most recent phase of subsurface exploration. The following report summarizes findings of our geologic and soils engineering exploration and third-party review of the May 18, 2015, Byer Geotechnical, Inc., report for the subject property. The purpose of our exploration was to

provide a third-party review of the geologic and geotechnical conditions on the subject property with particular emphasis on the shear strengths of the earth materials. As indicated above, our subsurface exploration and testing were performed in conjunction with Byer Geotechnical Inc., the geologic and geotechnical consultant for the project.

It is the intent of this report to aid in the design and completion of the project and to reduce potential or perceived risks associated with the project. This report is prepared for the use of the client and authorized agents and should not be considered transferable. Prior to use by others, the site and this report should be reviewed by Grover-Hollingsworth and Associates, Inc. Following review, additional work may be required to update this report.

EXPLORATION

As a part of our third-party review, exploration was conducted in areas of the proposed project, as shown on the Byer Geotechnical, Inc. ("BGI") Geologic Map included in their "Geologic and Soils Engineering Report," Project BG 21898, dated May 18, 2015 ("Byer Report"). The boring and test pit locations were selected to supplement numerous previous explorations by Geotechnical Professionals, Inc., and BGI.

The field exploration was conducted between September 29 and October 7, 2014, with the aid of truck-mounted bucket-auger drill rigs, hollow-stem auger drill rig, and a hand labor. Exploration included drilling eight (8) borings and excavating three (3) test pits to depths of 6½ to 76 feet. Samples were obtained from the test pits and borings and taken to our laboratory. Downhole observation of the earth materials encountered in the borings and test pits was performed by staff and project geologists with our firm. Excavation spoils and samples from the hollow-stem-auger borings were visually logged by the principal engineer. Excavations were backfilled and tamped but should not be considered compacted.

Office tasks included laboratory testing, review of reports from previous consultants, and the preparation of this report and a prior report dated March 25, 2015, that contains boring and test pit logs, as well as laboratory test data from our firm.

THE PROJECT

The project includes a three-level parking structure with a practice field on the top level on the west side of Coldwater Canyon Avenue and a pedestrian bridge to connect the parking structure to the campus on the east side of Coldwater Canyon Avenue. Permanent soil-nail walls are planned to support excavations into the ascending slope, which will be approximately 87 feet high (not including any fencing atop the walls). A debris basin will be created west of the southwest portion of the parking structure. Deflection walls and a drainage channel are planned above the northwest and northern portions of the structure. The project is diagrammatically shown on the Revised Geologic Map in the Byer Report.

SITE DESCRIPTION

The subject site, as described in the Byer Report, consists of several contiguous parcels located on the west side of Coldwater Canyon Avenue, approximately one-quarter of a mile south of Ventura Boulevard on the north flank of the Santa Monica Mountains in the Studio City area of the city of Los Angeles, California. The subject site also includes a portion of the School campus on the east side of Coldwater Canyon Avenue.

Past grading on the parcels west of Coldwater Canyon Avenue has consisted of cut and fill operations to create level building pads. The pads are accessed via a concrete-paved driveway that ascends west from Coldwater Canyon Avenue. A northern pad was created by placing compacted and undocumented fill in an east-trending secondary canyon. A 2:1 compacted fill slope descends to the east below the pad to a retaining wall above the driveway. The slope to the west of the northern pad ascends offsite, at gradients ranging from 2:1 to 3:1, to a north-trending ridge.

The southern pad was created by cutting into an existing secondary ridge and placing compacted fill in the southeastern portion of the pad. A 6- to 8-foot-high vertical cut was created along the west edge of the pad. A 1½:1 cut slope ascends above the vertical portion 25 feet to a natural ascending 2:1 slope. Past grading of the south portion of the site included grading to accommodate a driveway to two additional pads that formerly each supported residences situated in the axis of an east-draining canyon. Slopes ascend offsite to the southwest about 350 feet at a 2:1 to a 1½:1 gradient and are locally as steep as 1:1.

Vegetation on the pads and cut and fill slope consists of scattered weeds and shrubs. Moderate to dense native vegetation is present on non-graded slopes.

Surface drainage is by sheetflow runoff down the contours of the land to the two canyons and to the east to Coldwater Canyon Avenue.

The State of California has included the western portion of the School site and a narrow strip of land west of Coldwater Canyon Avenue in a liquefaction hazard zone. That designation was likely made based on the mapped presence of alluvium and assumed depth of groundwater. The potential for liquefaction is adequately addressed in the Byer Report and is not considered to be a hazard to the planned improvements.

EARTH MATERIALS

Fill was observed in most of the recent test pits and borings. The fill encountered is similar to that described in the BGI report. The depth of fill at the eastern bridge abutment is 7¾ feet as indicated by boring GHB-8.

Natural residual soil was encountered in several of the recent test pits and borings, with an observed thickness of 2 to 5 feet. The soil composition observed is similar to that described by BGI. The soil depth in boring GHB-8 at the eastern bridge abutment is 4¼ feet.

Alluvium was encountered in several of the recent test pits and borings. Alluvium was not found at the eastern bridge abutment due to the presence of a buried ridge. The alluvium composition is similar to that described by BGI.

Bedrock included in the Upper Miocene Age Modelo Formation by Hoot (1930)) underlies the property and was encountered in all of the test pits and borings, except GHTP-3. The bedrock consists primarily of siltstone, shale and mudstone with occasional siliceous shale and sandstone beds. Significant portions of the bedrock are diatomaceous and the siltstone ranges from sandy to clayey in nature. As discussed by BGI, the diatomaceous bedrock was primarily encountered on the central and northern portions of the site, while the mudstone bedrock was primarily encountered on the southern portion of the site.

GROUNDWATER

Seeps, springs, or groundwater were not encountered during the recent exploration or during the earlier explorations by BGI or GPI west of Coldwater Canyon Avenue. Although groundwater was previously encountered by BGI in the area of the school pool (east of Coldwater Canyon Avenue), groundwater was not encountered in the area of the planned eastern bridge abutment in boring GHB-8 due to the fact that this abutment is located on a buried bedrock ridge.

LOCAL GEOLOGIC STRUCTURE

The bedrock encountered in our recent test pits and borings and in earlier borings by BGI is common to this area of the Santa Monica Mountains. According to regional geologic maps, bedding in this area generally dips moderately to steeply to the north by northwest.

Bedding planes mapped in the excavations and within outcrops generally dip steeply to the northwest. This orientation is generally consistent with the regionally mapped trends. Bedding planes dip steeper and toward the south in the southern portion of the site. As discussed by BGI, the south-dipping beds are likely over turned. The bedding will not surcharge the planned soil-nail walls.

Most shear planes mapped dip moderately to steeply in various directions. However, a relatively consistent orientation of shear planes dipping gently to moderately to the north was encountered in boring GHB-3. This orientation is considered to be adverse with respect to the north-facing slope above and south of boring GHB-3. This adverse shear orientation has been considered by BGI and DRS Engineering in the soil-nail wall design. We concur that the BGI and DRS Engineering design adequately considered and incorporated the adverse shear condition.

The geologic structure is favorably oriented for stability of the site and the project with respect to sliding along bedding. Based upon the recent exploration, shear planes are potentially adversely oriented in the vicinity of GHB-3 with respect to future north-facing cuts or retaining walls. The City of Los Angeles has queried a northeast-trending landslide on the slope above GHB-3. Although evidence of a landslide was not found in boring GHB-3, the presence of adversely oriented shear surfaces will necessitate special design

considerations as discussed above. The special design considerations will adequately address the City's concern, and we have described such considerations below.

ENGINEERING CONSIDERATIONS

The shear strength testing performed by our firm reveals considerable variability in the strength of the bedrock. This variability is partially due to differences in the composition of the bedrock, partially due to the degree of weathering (depth below the bedrock surface), and partially due to the degree of fracturing of the bedrock. In general, the bedrock strength increases somewhat with depth. The cut bedrock samples, which are unfractured, generally have higher strengths than the driven samples, which can contain fractures (either natural or resulting from sampling).

We have reported both peak and ultimate shear strengths for the bedrock samples. Based on the "Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California," ASCE/SCEC dated June 2002, peak shear strength values can be used for "fine-grained, low-plasticity materials (liquid limit=40) that have not experienced significant previous shear deformations and are unlikely to be subject to significant weathering over the life of the project. In contrast, the guidelines state that "stiff clay and clayey bedrock materials (e.g., claystone, shale) of high plasticity (liquid limit=60) fail at shear stresses that are typically intermediate between fully softened and residual strength (provided they had not been subject to significant previous shear deformations). For these materials, the use of ultimate strength parameters is typically required. We have determined the liquid limit for six (6) bedrock samples representing the range of material types encountered. The liquid limits range from 60 to 103. The plastic limit ranges from 33 to 66. The plasticity indices (the differences between the liquid limits and plastic limits) range from 25 to 37. These values suggest that the bedrock material is highly plastic silt. Our field observations suggest that the bedrock is less plastic than suggested by the testing. We believe that the results are significantly influenced by the diatomaceous content of the bedrock which significantly increases moisture contents over more typical shale and mudstone.

Nonetheless, in keeping with the tests and associated results suggesting high plasticity, BGI has utilized the ultimate shear strength results for the earth materials in their static stability analyses. We believe that this is appropriate even though, as discussed above, we believe

the high liquid limits found for the bedrock are primarily due to the diatomaceous nature of the materials rather than their plasticity. The nearly lower-bound strengths selected by BGI are conservative since the use of strength parameters closer to the average could be justified based on the high number of samples tested and the fact that failure surfaces modeled in the analyses extend through many different bedrock layers.

The bedrock is in the low expansion range based on expansion index testing by GPI (EI < 50). GPI suggests that the bedrock is potentially moderately to highly expansive based on Atterberg Limits testing. The Atterberg Limits tests do not directly measure the expansiveness of the soil. We believe that the erroneous correlation being discussed by GPI is based on the unusually high liquid limit of the bedrock due to its diatomaceous composition.

CONCLUSIONS

We have reviewed the Byer Report and it is our professional opinion that the Byer Report provides an accurate description of the earth materials that underlie the site and their distribution. The only unfavorable structural condition revealed by the exploration to date is the presence of north-dipping shear surfaces in boring GHB-3. The strength of the clay material along these surfaces has been determined in our laboratory and that strength has been utilized in the BGI stability analyses.

The shear strength testing performed by our firm reveals considerable variability in the strength of the bedrock. This variability is partially due to differences in the composition of the bedrock, partially due to the degree of weathering (depth below the bedrock surface), and partially due to the degree of fracturing of the bedrock. In general, the bedrock strength increases somewhat with depth. The cut bedrock samples, which are unfractured, generally have higher strengths than the driven samples, which can contain fractures (either natural or resulting from sampling). The shear test results by our firm and those by BGI are believed to provide a much more accurate representation of the material properties than those by GPI. BGI has used nearly lower-bound ultimate shear strength parameters in their static analyses although higher strength parameters could be justified based on the number of samples tested and the fact that the failure surfaces analyzed extend through multiple bedrock layers.

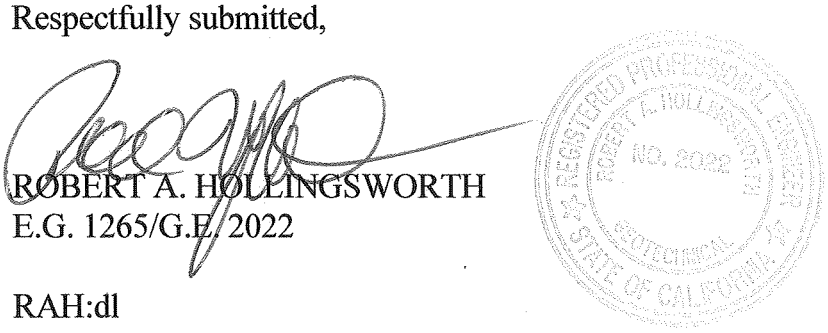
The slope stability analyses performed by BGI through, below, and above the soil-nail walls capture the critical conditions for analysis of those sections. As expected, the longest soil nails are associated with the southern wall due to the adverse shear surfaces encountered during the recent phase of exploration.

We agree with BGI that although portions of the project are included in a liquefaction hazard zone by the State, the presence of shallow bedrock in the area of the main structure and the eastern bridge support precludes the occurrence of liquefaction beneath the structure. Further, all building and bridge foundation elements are to derive support from bedrock.

In summary, it is our professional opinion that sufficient exploration and testing have been performed to accurately characterize and model the site conditions. The number of borings and shear strength tests performed significantly exceed those for similar projects in hillside areas. We believe that the project can be safely and successfully completed by following the recommendations in the Byer Geotechnical Report.

Please call this office with any questions.

Respectfully submitted,



ROBERT A. HOLLINGSWORTH
E.G. 1265/G.E. 2022

RAH:dl

xc: (1) Harvard-Westlake (Attention: David Weil)
(1) David Weil via email