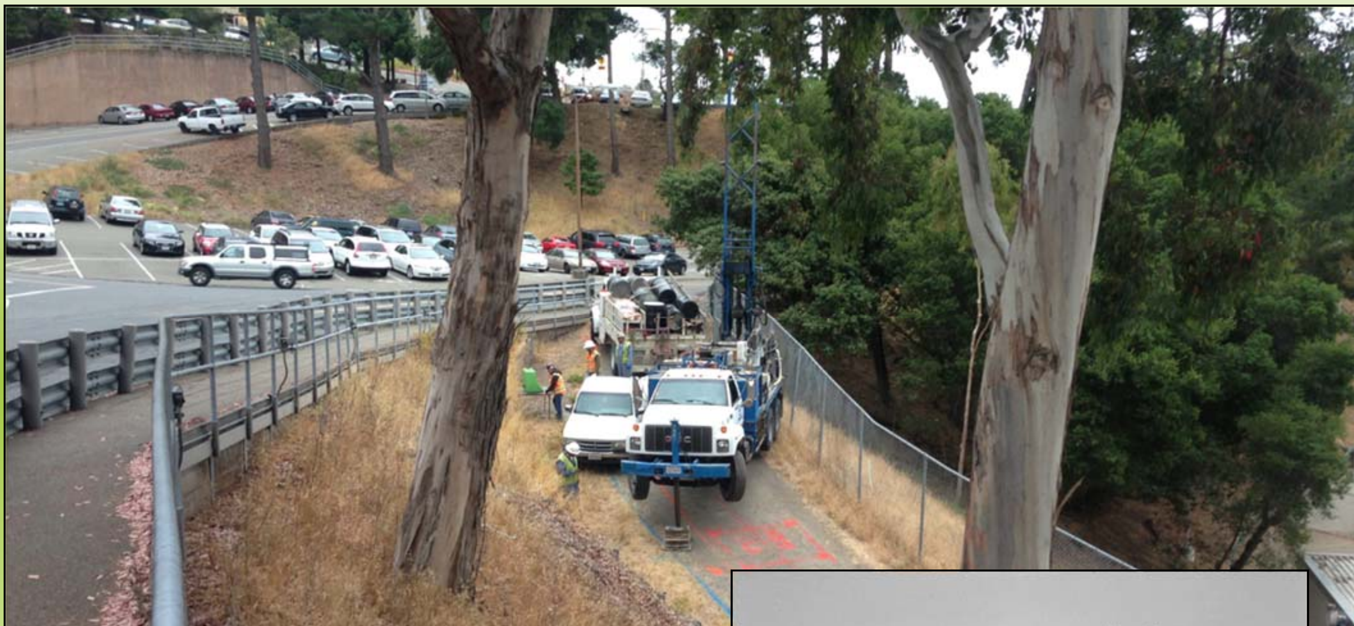
Geotechnical Investigation Report

Integrative Genomics Building (IGB) Project
Conceptual Design Phase
Lawrence Berkeley National Laboratory
Berkeley, California



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A3GEO

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EXECUTIVE SUMMARY

In 2014, A3GEO, Inc. conducted a geotechnical investigation for the proposed Integrative Genomics Building (IGB) Project at the Lawrence Berkeley National Laboratory (LBNL). As part of our study we reviewed available literature, data and historical photography; drilled seven geotechnical/geologic boreholes; acquired downhole geophysical data in three boreholes; installed one piezometer and one inclinometer; performed laboratory tests; analyzed and interpreted the collected information/data; performed engineering analyses; developed conclusions and recommendations for the project and prepared this report.

The IGB is anticipated to be approximately 76,000 gross square feet, located on four floors. The IGB site is presently a nearly-level parking lot that has come to be known as the "Bevatron flat"; the first floor of the IGB will be approximately at grade. As envisioned in the Project Conceptual Design Report (CDR), gravity columns for the IGB will be supported on spread footings whereas concrete core walls that are part of the lateral system for the building will be supported on thickened mats with micropiles at the edges of the mat to resist transient seismic uplift loads. The CDR also shows separate modular utility plant (MUP) located along the east side of the Bevatron flat northeast of the IGB. At this location, existing retaining walls bound three sides of a rectangular pad about 15 feet above the level of the Bevatron flat; the base of a steep (about 1-½:1; horizontal to vertical) slope bounds the east side of the elevated pad.

As documented in this report, the IGB site shown in the CDR is generally well-suited for the planned construction. The IGB site presently contains localized fills less than about 20 feet deep overlying bedrock; the envisioned Project would improve the onsite fill materials in order to optimize foundation performance. The IGB site is relatively free of geologic hazards other than earthquake groundshaking; a hazard shared through the region that is routinely mitigated through the seismic design provisions of the California Building Code. The IGB site is situated on level ground unaffected by previous landsliding and there is little to no potential for ground failure to occur beneath the site. Earthquake fault rupture is not a significant concern as the IGB site is at least 1,000 feet away from the closest known or suspected active fault trace.

Siting of the MUP at the location shown in the CDR would be complicated by known landslide deposits located directly upslope. Deep landslide-related movements occurred in 1973 upslope of the planned MUP site triggered by prolonged heavy rainfall. Later in the 1970s, stabilization measures implemented by LBNL were effective in arresting ongoing slope movements and this known landslide deposit has been stable for the past 35+ years. Additional structural stabilization measures were implemented by LBNL in the early 1990s to enhance the landslide deposit's seismic stability. However, increased knowledge and advancing standards of engineering practice show the seismic stabilization measures implemented in the 1990s cannot presently be relied upon to restrain the existing landslide deposits during a large (i.e. design-level) seismic event. Development of the MUP site shown in the CDR would also be complicated by interpreted landslide deposits that extend down to and below the level of the Bevatron flat.

This report presents geotechnical conclusions and recommendations for conceptual design purposes. Included in our recommendations (Section 6.0) are basic criteria for seismic design (per the California Building Code), foundations, retaining walls, tiebacks, ground improvement, underdrainage and expansive soil mitigation. Considerations associated with the siting of the MUP are discussed in Section 5.05, which includes a preliminary analysis of earthquake-induced landslide forces and displacements conducted in accordance with up-to-date State of California guidelines. As with other significant landslides at LBNL, our preliminary analyses generally show that the forces required to restrain the landslide deposits are quite large. On the other hand, predicted downslope displacements without added restraint(s) are generally limited allowing for siting of buildings outside of a "safe" setback zone. These preliminary lateral force and displacement analyses are intended to inform future planning/design efforts with respect to the siting of the MUP and/or other improvements in areas north of the IGB.

1.00 INTRODUCTION

This report presents the results of A3GEO's geotechnical investigation for the proposed Integrative Genomics Building (IGB) Project at the Lawrence Berkeley National Laboratory (LBNL). We prepared this geotechnical investigation report in accordance with LBNL Master Task Agreement (MTA) No. 7105895 Task Order No. 7109435. The location of LBNL is shown on the Vicinity Map, Plate 1. At the time of this report, the IGB project (Project) was in the conceptual design phase.

1.01 Overview

1.01.1 Project and Site Description

We obtained information about the Project conceptual design from LBNL's A/E consultant team, which includes Smithgroup (architecture) Rutherford & Chekene (structural engineering) and BKF (civil engineering). As currently envisioned, the Project will include a new research facility (the IGB), a modular utility plant, site retaining walls, landscaping, paved parking and an access road. The conceptual layout of the IGB and associated improvements are shown on Plate 2.

The aerial photograph presented on Plate 3 shows the IGB site at the southern end of large nearly-level pad paved with asphalt concrete. Until recently the IGB site was occupied by a large circular building (Building 51) housing the Bevatron, a large particle accelerator built in the early 1950s. This report refers to the large paved area that includes the IGB site as the Bevatron flat; the elevation of the Bevatron flat is approximately +710 feet, UC/LBNL datum. The east and south sides of the Bevatron flat are bounded by retaining walls that are about 15 feet high. Above the tops of the walls are small level areas and graded slopes that extend up to Smoot and McMillan Roads (Plate 3).

1.01.2 Site History

Plate 4 shows the pre-development topography of LBNL, which can be generally characterized as hillsides and ridgelines punctuated by valleys. As shown on Plate 4, the IGB site is located along the southern flank of a primary east-west trending valley commonly known as Blackberry Canyon. Prior to initial development (i.e. before about 1948) the IGB site was traversed by a southeast-northwest trending tributary drainage (Plate 5).

The pre-development natural topography in the vicinity of the IGB site has been extensively modified by grading. The photograph on Plate 6, taken in June 1949, shows the Bevatron site at an early stage of development. The Bevatron flat was created by cutting and filling with the original (1949) cut/fill transition passing through the northern portion of the IGB site. Development of the site in 1949 also involved grading for the Bevatron Warehouse (now Building 46), which is located on a separate cut/fill pad near Elevation +810 feet about 200 feet east of the Bevatron flat. Soil derived from excavation cuts along the eastern and southern sides of the Bevatron flat and from the Building 46 site was used to fill part of Blackberry Canyon; the fill placed in the deepest portion of the canyon is roughly 80 to 90 feet deep.

The Bevatron complex included a variety of below-grade improvements installed during the original construction and in the years following. The accelerator itself was circular in plan and covered with heavy shielding blocks. Below the accelerator was a roughly circular basement with tunnels that extended to the eastern limit of the pad (wind tunnels). Northwest of the circular basement was an irregularly-shaped motor-generator basement (MG basement). Heavy and/or settlement-sensitive elements of the Bevatron complex were supported on drilled piers or belled caissons. Other tunnels, drains and underground utilities existed in various areas of the site. The 2011 Aerial Photograph on Plate 7 shows the general location of Bevatron and wind tunnel backfill as well as the MG basement.

In 1949 and 1950, multiple landslides occurred in steep (approximately 1.5:1, horizontal to vertical) excavation cuts made along the pad's eastern and southern perimeter. The locations of these slides are documented in construction photographs and memos; some of these slides can be seen on the August and November 1949 photographs presented on Plates 8 and 9. The landslides that occurred during that time were reportedly replaced with compacted fill buttresses; horizontal drains (hydraugers) were also installed at that time to drain subsurface water from beneath areas upslope.

During the winter of 1972-1973, a large landslide damaged Building 46, which was interpreted to toe-out above the level of the retaining walls that bound the east side of the Bevatron flat (Plate 10). The portion of the landslide upslope of Building 46 was excavated as part of emergency stabilization measures and later replaced with compacted fill. However, the portion of the landslide below Building 46 was not excavated but remains in place. The stability of the lower portion of the 1973 landslide was later enhanced by removing about 10 feet of soil from the upper part of the landslide; this project created the elevated roadway and parking area that now exists directly west of Building 46. A second project involved the installation of 51 drilled piers and tiebacks along an access road about midway between the Bevatron flat and Building 46. Two smaller landslides located south of the Building 46 landslide have also been the subject of subsequent stabilization/repair projects

1.01.3 State of California Seismic Hazard Zonation

A portion of the IGB site is within a State-designated zone of required investigation for earthquake-induced landsliding, as are most of the hillside areas directly upslope of the site. The State's minimum criteria required for project approval within zones of required investigation are defined in CCR Title 14, Section 3724, which requires "evaluation of site-specific seismic hazards based on geological and geotechnical conditions, in accordance with current standards of practice." The California Geological Survey (CGS) provides guidance to lead agencies, practitioners and reviewers in CGS Special Publication 117A (SP117A). In discussing the areal extent of mapped hazards, SP117A notes:

> *Although past earthquakes have caused ground failures in only a small percentage of the total area zoned, a worst-case scenario of a major earthquake during or shortly after a period of heavy rainfall is something that has not occurred in northern California.*

The SP117A guidelines outline two levels of analysis for earthquake-induced landslide hazards. A screening investigation assesses whether pre-existing landslide deposits or other potentially hazardous slope features with the potential to affect the site may be present. A substantially more rigorous quantitative evaluation is recommended for sites where existing landslide deposits, subsurface water and/or susceptible landforms are suspected or known to exist, which is the case in the vicinity of the proposed IGB site. More information on the State's seismic hazard zonation program can be found at http://www.conservation.ca.gov/cgs/shzp/Pages/shmppgminfo.aspx.

1.02 Project Description

1.02.1 IGB

The IGB is anticipated to be approximately 76,000 gross square feet, located on four floors. The first floor of the IGB will be approximately "at grade" (Elevation +710 feet). As envisioned in the Conceptual Design Report (CDR), gravity columns will be supported on spread footings whereas concrete core walls that are part of the lateral system for the building will be supported on thickened mats with micropiles at the edges of the mat to resist transient seismic uplift loads. Ground floor slabs-on-grade will be 5-inch concrete reinforced except in areas with sensitive equipment where the slabs will be thickened to 8 inches and isolated from adjacent construction. As currently planned, the IGB will be constructed on a nearly level site and will not be in contact with retaining walls that bound the Bevatron flat.

1.02.2 Modular Utility Plant

The CDR shows separate modular utility plant (MUP) located along the east side of the Bevatron flat northeast of the IGB. At this location, existing retaining walls bound three sides of a rectangular pad about 15 feet above the level of the Bevatron flat; the base of a steep (about 1-½:1; horizontal to vertical) slope bounds the east side of the pad. As currently envisioned, the existing retaining walls that bound the three sides of the pad will be removed and a new 15-foot high wall will be built to retain the base of the slope, which was affected by landsliding in 1949 and 1973.

1.02.3 Site Development

As currently planned, the southeast corner of the IGB intersects the existing retaining walls that bound the Bevatron flat. At this location, the existing retaining wall will be partially demolished and reconfigured maintain a separation between the IGB and the adjacent ground. The new and existing retaining walls will bound the northern edge of a new access road and building entrance at the second floor level (about Elevation +725 feet). At the first floor level (about Elevation +710 feet), there will be a landscaped entry and courtyard and limited parking for visitors and building maintenance.

1.03 This Investigation

1.03.1 Purpose and Scope

We conducted our geotechnical investigation for the purposes of characterizing geotechnical, geologic and seismic conditions and providing geotechnical engineering recommendations in support of the Project conceptual design. The scope of our geotechnical investigation included:

- Compiling and reviewing existing data;
- Drilling three new borings in upslope areas where landslide deposits are present;
- Drilling four new borings on the Bevatron flat;
- Collecting downhole geophysical data (suspension logging and televiewer);
- Installing one inclinometer and one piezometer;
- Conducting baseline inclinometer measurements;
- Performing engineering geologic field mapping;
- Compiling, reviewing and interpreting new and compiled data;
- Characterizing geologic, seismic and geotechnical site conditions;
- Analyzing slope stability and seismic displacements;
- Consulting with project team members;
- Developing conclusions and recommendations for the geotechnical aspects of the Project; and
- Preparing this geotechnical investigation report.

1.03.2 Report and Appendices

In preparing this report, it was our objective to provide: 1) concise descriptions of geotechnical, geologic and seismic conditions for LBNL's and the IGB design team's use; and 2) sufficient detail pertaining to our geotechnical and engineering geologic analyses to allow for third-party technical reviews. Supporting information, data and interpretations deemed most relevant to our concept-level IGB investigation are presented in the plates, figures, and appendices that accompany this report.

1.03.3 Report Limitations and Exclusions

This report was prepared for the exclusive use of LBNL and design team members in support of the IGB conceptual design phase. This report is not considered appropriate for final design as: 1) details involving the IGB project design are likely to evolve in ways that cannot be anticipated at this time; 2) new geotechnical information relevant to the IGB project design may come to light through future research, onsite observations and/or monitoring; 3) additional explorations, analyses, conclusions and/or recommendations may be advisable or necessary based on design changes or other factors; and 4) specific details relevant to the construction phase have been intentionally excluded from this report, including: a) discussions of construction considerations intended for the Contractor's use; and b) recommendations pertaining to geotechnical observation and testing.

1.03.4 Supplemental Compilations

There is a substantial quantity of information available pertaining to the IGB site and vicinity, which we catalogued and organized in association with this conceptual-level geotechnical and geologic study. We prepared three supplemental compilations to make this information more readily available for future use:

Previous Subsurface Data – includes site plans, boring logs, laboratory data, and observations related to previous projects.

Historical Photographs – includes site development photographs obtained from LBNL's archives.

Previous Plans and Calculations – includes design and survey-related data from previous projects.

Due to their size, we have chosen to submit our three supplemental compilations as separate "stand alone" files rather than as appendices to this conceptual-level report.

2.00 METHODS OF INVESTIGATION

2.01 Review of Existing Information

2.01.1 General Information

We reviewed maps and literature published by the U.S. Geological Survey (USGS), California Geological Survey (CGS), and California Division of Mines and Geology (CDMG) relating to geologic and seismic conditions at the Project site. These and other published materials used in our study are listed in Section 8.01, General References.

2.01.2 Geotechnical Reports and Correspondence

We reviewed subsurface data, maps, interpretations and other information contained in geotechnical reports and files from LBNL's geotechnical database. The reference list presented in Section 8.02 includes identifying information on the reports and correspondence that we reviewed along with the number of the LBNL file in which the reference was found. Many of the files are also available on-line (organized by date) at https://sites.google.com/a/lbl.gov/berkeley-lab-geotechnical-reports-and-studies/). Reports prepared by A3GEO and LCI after 2010 were obtained from our files and do not have an LBNL reference number.

The approximate locations of previous geotechnical/geologic borings for which logs are available are shown on the Site Plan, Figure 1. Borings logs and other relevant data from the referenced reports are included in the supplemental compilation document titled "Previous Subsurface Data."

2.01.3 Historical Photographs

We reviewed historical photographs of LBNL to assess pre-development geomorphology, historical landslides and site development history. Among the aerial photographs we reviewed are an east-facing oblique aerial photograph of the site area from 1935 and a stereo-paired set of vertical aerial photographs from 1939, both of which predate development associated with the lab. In all, we examined seven sets of vertical aerial photographs using a stereoscope; identifying information pertaining to these photographs is presented Section 8.03, Aerial Photographs.

We also reviewed historic photographs from LBNL's online photo archive, which can be accessed at http://photos.lbl.gov/. Within this archive are a large number of historical photographs taken of the Bevatron site before, during and after development. Most of these pictures were taken in 1949 and 1950 and show the grading that took place to develop the site and the various landslides that occurred within the excavation cuts at the upslope site perimeter. Compiled photographs from the archives are presented in the supplemental reference document titled "Historical Photographs." Selected photographs from the LBNL photo archives are also presented on the plates that accompany this report.

2.01.4 LBNL-Provided Plans and Structural Calculations

We reviewed plans, structural calculations and other LBNL-provided information relevant to the project. The reference list presented in Section 8.04 includes identifying information on the reports and correspondence that we reviewed, which included:

- A drawing showing below-grade elements of the Bevatron (Huber and Knapik, et al, 1961);
- Structural calculations for the Phase 1 Slope and Seismic Stabilization Project (PFFA, 1992);
- Plans for the Phase I Slope and Seismic Stabilization Project (C+D, 1992);
- Plans for the Phase II Slope and Seismic Stabilization Project (Harza, 1994);
- Structural calculations for tiebacks installed during Bevatron demolition (Cartwright, 2010a);
- Plans for tiebacks installed during Bevatron demolition (Cartwright, 2010b); and
- Survey data/drawings showing the locations of remaining Bevatron caissons (Cartwright, 2012).

Relevant information from the preceding bulleted items is included in the supplemental compilation document titled "Previous Plans and Calculations." Other relevant LBNL-provided information that we reviewed included:

- Earthwork reports containing field density test results by Consolidated Engineering Laboratories (22 reports with dates from July 2, 2009 to January 27, 2012); and
- A plan showing the extent of localized cement treatment (5% Portland cement) performed prior to the paving of the Bevatron flat.

The locations and identification numbers of caissons installed along the access road east and upslope of the IGB and MUP sites are shown on Figure 1A.

2.01.5 LBNL Environmental Reports and Data

We reviewed parts of LBNL's RCRA Facility Investigation Report (RFI Report), which includes groundwater data, interpretive geologic maps, interpretive cross sections and other geologic information. The RFI Report can be reviewed at the Berkeley Public Library downtown branch together with supplemental report "Modules" A through D. The RFI Report is also currently available online at http://www2.lbl.gov/Community/SeismicPhase2B/GeoTech/RCRA-Facility-Investigation-Report_Sept-2000.pdf . We also reviewed selected logs of LBNL environmental borings, which are available on-line (organized by date) at https://sites.google.com/a/lbl.gov/berkeley-lab-geotechnical-reports-and-studies/.

2.02 Field Explorations and Laboratory Testing

2.02.1 Borings

Between July 11 and 29, 2014 we explored subsurface conditions by drilling seven borings at the approximate locations shown on the Site Plan, Figure 1. The borings were drilled by Pitcher Drilling Company, Inc. of East Palo Alto using truck-mounted rotary wash drilling equipment. During drilling, an A3GEO engineer logged the subsurface materials encountered and obtained samples for examination and laboratory testing. Borings B-1, B-2, B-4 and B-5 were drilled for the purpose of landslide characterization and were sampled on a continuous (or near-continuous) basis. Borings B-6, B-7 and B-8 were drilled in the vicinity of the IGB footprint and were sampled intermittently. Boring B-3 was attempted but could not be advanced due to an unidentified obstruction at a depth of about 3 feet; subsurface utilities and geometric constraints prevented us from completing this boring. The location at which Boring B-3 was attempted is roughly midway between Borings B-2 and B-4; this location is indicated on the Site Plan, Figure 1.

Core samples were reexamined in the laboratory by LCI geologists, who assisted in the interpretation of geologic conditions and augmented the field logs with structural notations. Finalized logs of the borings are attached in Appendix A together with explanatory information and descriptions of our drilling and logging methods.

2.02.2 Downhole Geophysical Surveys

Between July 17 and 25, 2014, NORCAL Geophysical Consultants, Inc. (NORCAL) collected downhole geophysical data in borings B-2, B-4 and B-5. The primary purpose of the downhole surveys was to: 1) obtain oriented imagery of the borehole walls to assist in the evaluation of structural discontinuities; and 2) develop profiles of shear wave velocity versus depth for seismic analyses (landslide displacements and building code Site Class). NORCAL Professional Geophysicist William J. Henrich (PGp No 893) conducted the following types of downhole surveys:

- Suspension P- and S- wave velocity profiling;
- Acoustic borehole televiewer (BHTV) logging; and
- Caliper logging.

NORCAL's August 25, 2014 report is attached as Appendix B. The report presents descriptions of NORCAL's investigative methods interpretations and includes P- and S-wave velocity profiles and BHTV discontinuity plots for borings B-2, B-4 and B-5.

2.02.3 Inclinometer and Piezometer Installations

Boring B-1 was completed by installing inclinometer casing to allow for the future monitoring of subsurface slope movements. The inclinometer casing consists of approximately 66 feet of 2.75-inch-diameter plastic pipe with two orthogonal sets of internal vertical grooves, which are traversed by an inclinometer probe. During installation, the primary set of grooves (A-axis) was oriented in the general direction of anticipated downslope movement. The second set of grooves (B-axis) is oriented perpendicular to the primary set. The casing, which is sealed at the bottom, was grouted in place using a tremie pipe. During this operation, a weight was lowered to the bottom of the casing to counterbalance the uplift forces of the fluid grout. Following grouting, the inclinometer installation was completed with a flush-mount surface enclosure.

Boring B-4 was completed as a standpipe piezometer to allow for the future monitoring of groundwater depths/elevations. The standpipe consists of 81 feet of 2-inch-diameter plastic pipe, the bottom 60 feet of which is slotted. The top of the piezometer installation is fitted with a flush-mount surface enclosure. Other details pertaining to piezometer construction are summarized in the following table

Piezometer Construction Details

Approximate Depth Interval	Annular Backfill
Surface to 1 foot	Concrete and Surface Enclosure
1 foot – 15 feet	Neat Cement Grout
15 feet - 17 feet	Bentonite Pellet Seal
17 feet – 81 feet	No. 3 Monterey Sand

Borings B-2 and B-5 through B-8 were backfilled with cement-bentonite grout. Following grouting, the pavement at the locations of borings B-5, B-6 and B-7 was patched with asphalt concrete (cold mix).

2.02.4 Geologic Field Mapping

In July 2014, LCI Geologists conducted surface reconnaissance mapping in the areas upslope of the Bevatron flat. The surface reconnaissance included: 1) mapping of surficial deposits (including artificial fill and landslides); 2) collecting structural information (orientation of bedding and discontinuities) from rock exposures and outcrops; and 3) evaluation of roads, curbs, sidewalks and other cultural features for indications of movement and/or distress. During the geologic field mapping, the approximate limits of the 1973 Building 46 landslide were marked on the ground in paint by LBNL's surveyor (Bates & Bailey).

Geologic field mapping was performed on topographic basemap constructed using Berkeley Lab Facilities Division "Q Sheets." The basemap is in the University of California grid projection (units in feet) with Grid North oriented about 16.7 degrees west of True North. Our geologic field mapping was performed at a map scale of 1 inch equals 50 feet.

The Site Geologic Map we prepared for this study is presented on Figure 2. In developing this map, we considered the data acquired through field reconnaissance coupled with the previous consultant studies described in preceding sections.

2.02.5 Inclinometer Baselining and Groundwater Depth Measurement

We utilized LBNL's inclinometer probe to make baseline measurements within the inclinometer casing installed within Boring B-1. As currently planned, subsequent measurements will be made using this same probe during the winter of 2014-2015 to check for slope movement and evaluate the depth of any significant movement that occurs. Identifying information for the probe is as follows: *Slope Indicator Company Digitilt AT Probe Serial No. 50330200; LBNL Property No. 6783063.* Baseline data acquired by the probe is stored within Slope Indicator's proprietary software on the accompanying tablet computer (part of *LBNL Property No. 6783063)*.

The rotary wash drilling method utilizes fluids that preclude the measurement of natural groundwater depths/elevations at the time of drilling. On September 4, 2014, we measured the depth/elevation of groundwater within the standpipe piezometer installed in Boring B-4; the data obtained from this measurement is summarized in table that follows.

Groundwater Depth/Elevation Measurement – Boring B-4

Date of Measurement	Groundwater Depth	Approximate Ground Surface Elevation	Approximate Groundwater Elevation
September 4, 2014	28.15 feet	+ 756 feet	+728 feet

This groundwater depth was made near the end of summer during a period of relative drought and water levels at this location are expected to vary.

2.02.6 Laboratory Testing

Our geotechnical laboratory testing program focused on determinations of soil plasticity, grain size and shear strength. The following geotechnical laboratory analyses were performed on samples retrieved from the borings:

- Atterberg Limits (ASTM D-4318)
- Particle size analysis (ASTM D-422)
- Unconsolidated-Undrained Triaxial Shear Strength (ASTM D-2850)
- Drained Residual Torsional Shear Strength (ASTM D6467)
- Drained Fully Softened Peak Torsional Shear Strength (ASTM D7608)

The results of geotechnical laboratory tests are included on the boring logs presented in Appendix A at the appropriate sample depths. Geotechnical laboratory data sheets are attached in Appendix C.

We also screened for naturally-occurring corrosive materials by conducting a suite of geochemical laboratory tests on samples obtained from a depth 6 and 7 feet in Borings B-6 and B-7, respectively. The geochemical laboratory tests included measurements of:

- Resistivity (100% saturated) per ASTM G57;
- Chloride ion concentration per Caltrans 422 (modified);
- Sulfate ion concentration per Caltrans 417 (modified);
- pH per ASTM G51; and
- Redox potential per Standard Methods 2580B.

The corrosivity test results are presented on the Corrosivity Test Summary in Appendix C.

3.00 GEOLOGIC CONDITIONS

3.01 Regional Geology and Seismicity

3.01.1 Geologic Setting

The site is in the northern portion of the Coast Ranges geomorphic province of California, which is characterized by northwest-trending mountain ranges and valleys that generally parallel major regional geologic structures such as the San Andreas and Hayward faults. The region is at the boundary between the North American and Pacific tectonic plates, which are in motion relative to each other. The nature of this motion has changed over time. Within the region, basement rocks that were accreted to the North American plate have been subducted, uplifted, folded and faulted by compressional and transverse displacements.

The oldest widespread rocks in the region are from the Mesozoic Era (225 to 65 million years ago). Mesozoic rocks of the Great Valley Complex predominate east of the Hayward fault, which is located near the base of the Berkeley Hills. Locally, the Hayward fault zone juxtaposes sedimentary rocks of the Great Valley Complex with similar age sedimentary, metamorphic and volcanic rocks of the Franciscan Assemblage. Franciscan rocks predominate between the Hayward fault and the San Andreas fault, which passes through the San Francisco peninsula, Pacific Ocean and Marin County headlands father to the west.

The Great Valley Complex and Franciscan Assemblage rocks are locally overlain by diverse sequences of Cenozoic Era (younger than 65 million years) sedimentary and volcanic rocks. Virtually all of the rocks in the region have been extensively deformed by repeated episodes of folding and faulting. During the late Miocene and early Pliocene (11.2 to 3.6 million years ago) an extended period of compression occurred that resulted in the folding, faulting and uplifting of the Berkeley Hills. Quaternary-age (younger than 2.5 million years) deposits cover much of the gently-sloping plain that exists between the Berkeley Hills and San Francisco Bay. Within flatland areas, alluvial deposits predominate. Near the base of the hills, Quaternary-age colluvium and landslide deposits locally overlie bedrock and alluvial deposits.

3.01.2 Bay Area Active Faults

San Francisco Bay Area includes a series of major active northwest-trending faults, which include the San Andreas, Hayward, Rodgers Creek, Calaveras, San Gregorio, Concord-Green Valley, West Napa, and Greenville faults (Plate 11). These major regional faults are near-vertical in orientation, and generally exhibit right-lateral, strike-slip movement (which means that movement along these faults is predominantly horizontal, and when viewed from one side of the fault to the other, the opposite side of the fault is observed as being displaced to the right). Faults that are defined as active exhibit one or more of the following: (1) evidence of Holocene-age (within about the past 11,000 years) displacement, (2) measurable seismic fault creep, (3) close proximity to linear concentrations or trends of earthquake epicenters, and/or (4) tectonic-related geomorphology. Potentially active faults are defined as those that have evidence of Quaternary-age displacement (within the past 11,000 to 2 million years), but have not been definitively shown to lack Holocene movement.

The closest known active fault to the site is the Hayward fault. The Hayward fault is zoned by the CGS as active; the closest mapped active trace of the fault is about 1,000 feet (0.2 mile) southwest of the IGB site (Plate 12). The Hayward fault is about 74 miles long, trending northwest from San Jose through several East Bay cities into San Pablo Bay. Further northward of San Pablo Bay is the Rodgers Creek fault, which is offset slightly eastward of the Hayward fault (Plate 11). Both Hayward and Rodgers Creek faults are considered to be interconnected by a series of en echelon fault strands, that are inferred to step eastward beneath San Pablo Bay. To the south, the Hayward fault also is considered to merge with the Calaveras fault, which lies to the south of San Jose. The Calaveras fault extends northward and merges with other unnamed faults within San Ramon Valley, which is located east of the Hayward fault.

Approximate distances and directions to major active Bay Area faults from the project site are shown in the following table (Jennings and Bryant, 2010; CDMG, 1982).

Approximate Distances and Directions to Active Faults

Active Fault	Approximate Distance from Site (miles)	Approximate Direction from Site
Hayward	0.2	Southwest
Calaveras	13.8	East
Rodgers Creek	22.5	Northwest
Concord-Green Valley	13.4	Northeast
San Andreas	18.7	Southwest
Greenville	18.1	Northeast
West Napa	19.6	North
San Gregorio	23.8	Southwest

3.01.3 Bay Area Seismicity

The greater San Francisco Bay Area region is characterized by a high level of seismic activity. Historically, this region has experienced strong ground shaking from large earthquakes, and will continue to do so in the future. Since 1800, five earthquakes with Moment Magnitudes (M) of 6.5 or greater have occurred in the Bay Area (Bakun, 1999). These include the 1) 1836 M6.5 event east of Monterey Bay; 2) 1838 M6.8 event on the Peninsula section of the San Andreas fault; 3) 1868 M6.8-7.0 Hayward event on the Southern Hayward fault; 4) 1906 M7.9 San Francisco event on the San Andreas fault; and 5) 1989 M6.9 Loma Prieta event in the Santa Cruz Mountains.

In 2003, The Working Group on California Earthquake Probabilities (WGCEP, 2003), in conjunction with the United States Geological Survey (USGS), published an updated report evaluating the probabilities of significant earthquakes occurring in the Bay Area over the next three decades, (2002-2031), which has since been updated on a state-wide scale in 2008 for the time span of 2007 through 2036. The WGCEP 2008 report indicates that there is a 0.63 (63 percent) probability that at least one magnitude 6.7 or greater earthquake will occur in the San Francisco Bay region before 2037. This probability is an aggregate value that considers seven principal Bay Area fault systems and unknown faults (background values – WGCEP, 2003). The findings of the WGCEP 2008 report are summarized in the following table:

WGCEP (2008) Probabilities

Fault System	Probability of At Least One Magnitude 6.7 or Larger Earthquake in 2007-2036
Hayward/Rodgers Creek	0.31
San Andreas	0.21
Calaveras	0.07
San Gregorio	0.07
Concord-Green Valley	0.03
Greenville	0.03
Mount Diablo Thrust	0.01
Background *(2002-2031)	0.14*

The published background values are not explicitly stated in the WGCEP (2008) and thus the WGCEP (2003) values were used. The background values indicate that between 2002 and 2031 there is a 14 percent chance that an earthquake with a magnitude of greater than 6.7 may occur in the Bay Area on a fault system not characterized in the study. It should be noted differences between the 2008 and 2003

WGCEP generally fall within the magnitude of error, and major differences in background values are not expected.

3.02 Local Geologic Conditions

3.02.1 Geologic Structure

Bedrock units mapped in the vicinity of the IGB include the Great Valley Complex, the Orinda Formation and Moraga Volcanics. The Great Valley Complex and overlying Orinda Formation differ in age by more than 100 million years. The structural unconformity between these two units is interpreted by most to be an unnamed fault that is presently neither active nor potentially active. This fault generally trends northwest-southeast and is interpreted to dip back into the slope (e.g., down to toward the northeast) and away from the Hayward fault. Borings drilled for this investigation did not encounter Great Valley Complex rocks and the precise location of the fault relative to the IGB is not currently known. In the Building 51 area, Great Valley Complex rocks crop out directly southwest of the traffic circle at the east end of Chu Road. Orinda Formation rocks were exposed in multiple excavations during the Building 51 (Bevatron) demolition. The fault is therefore inferred to lie between these two locations.

The Orinda Formation is locally overlain by and occasionally interfingers with the Moraga Volcanics, which were deposited during a period of volcanism that ended about 8.4 million years ago (Jones and Curtis, 1991). All of the bedrock units at LBNL have been uplifted, folded and faulted since they were deposited producing complex geologic structures that in some areas are not well understood. The USGS regional geologic map on Plate 13 (Graymer, 2000) shows Orinda Formation (map symbol Tor) and Moraga Volcanics (map symbol Tmb) folded into a syncline that has been displaced by the inactive Wildcat fault in the eastern part of LBNL. The sediments of the Orinda Formation include materials that are weak, compressible and subject to landsliding and erosion. Accordingly, natural slopes within the Orinda Formation are typically less steeply inclined than those within the more resistant Great Valley Complex and Moraga Formation, except where locally incised by landslides or flowing water.

The Moraga Formation (map symbol Tmb) generally overlies the Orinda Formation and generally caps the peaks and ridgelines at higher elevations within and above LBNL (Plate 13). The Moraga Formation rock found at lower elevations is commonly discontinuous, consisting of distinct volcanic bodies or lenses resting upon the Orinda Formation. These relationships are locally complex and have been studied extensively due to the Moraga Formation's higher permeability, which locally controls and directs shallow groundwater flow (LBNL/Parsons, 2000). Within the northern portion of LBNL (i.e. in the vicinity of the IGB), Lawson and Palache (1901) and many subsequent investigators interpret that the upper Orinda Formation was deposited contemporaneously with volcanic flows and pyroclastics of the Moraga Formation and that these two units locally interfinger

Geologic conditions in the vicinity of the IGB site are controlled in part by the complex stratigraphic and structural relations between the Orinda and Moraga Formations. In the vicinity of the IGB site, structural discontinuities involving the juxtaposition Orinda and Moraga formation rocks have been alternatively interpreted as evidence of faulting (e.g. HLA, 1982) or landsliding (e.g. LBNL/Parsons, 2000). Geologic maps by HLA (Plates 14 and 15) generally show alternating bands of Orinda Formation (map symbol To) and Moraga Volcanics (map symbol Tm) in upslope areas east of the IGB site; HLA interprets these bands as roughly parallel tilted beds of Orinda and Moraga formation rocks that dip northeast into the hillside consistent with regional mapping.

3.02.2 Orinda Formation

The Orinda Formation is the predominant bedrock unit in the developed areas of LBNL and is present beneath the IGB site and within the slopes south and east of the Bevatron flat. The Orinda Formation was deposited within an inland basin at a time when hills of Franciscan Assemblage rocks were present to the west. The Orinda Formations is described by Graymer as follows:

> *"Distinctly to indistinctly bedded, nonmarine, pebble to boulder conglomerate, conglomeratic sandstone, coarse- to medium-grained lithic sandstone, and green and red siltstone and mudstone. Conglomerate clasts are subangular to well rounded, and contain a high percentage of detritus derived from the Franciscan complex."*

The conglomerates were deposited under alluvial fan conditions, and the sandstone, siltstone and claystone were deposited as floodplain and channel material (Jones and Curtis, 1992).

3.02.3 Moraga Formation

The Miocene Moraga Formation consists of as many as five distinct flows typically defined by basaltic and andesitic composition (Wahrhaftig and Sloan, 1989). Early studies by Lawson and Palache (1901) refer to the volcanic deposits in the vicinity of the IGB site as the Campan series and are described as fresh-water deposits interbedded with lavas and tuffs. Others also describe similar clastic deposits at or near the base of eruptive sequences (Lawson, 1901; Wahrhaftig and Sloan, 1989; and Clements, 1963). Potassium-argon ages of the volcanic flows vary from 10.2 million years (Ma) to 8.5 Ma (Curtis, 1989). The basal member of the volcanics, defined as an amygdaloidal andesite is interpreted to have been deposited over a broad alluvial flood plain with later flows and tuffs being confined to narrow channels, ravines and valleys (Lawson and Palache, 1901; Wahrhaftig and Sloan, 1989). Locally, the Moraga Formation rests depositionally on the Orinda Formation, and/or at its base interfingers with the Orinda Formation. Along the McMillan Road exposure northeast of the IGB site, the Moraga Formation consists of a series of bedded highly fractured and weathered, subangular to subrounded agglomerate, andesite, altered ash and tuff, and basalt.

3.02.4 Hydrogeology

The hydrogeology of the IGB Site area is described in LBNL's RFI Report (LBNL/Parsons, 2000) as follows:

> *"Groundwater flow directions generally follow the slope of the surface topography. However, at some locations flow directions deviate due to contrasts in subsurface hydraulic conductivity or artificial drainage features such as building subdrains, subhorizontal hillside drains (hydraugers), and slope stability wells. Hydraulic conductivity testing and groundwater well yields show that the Moraga Formation is relatively permeable, and constitutes the main water-bearing unit at LBNL. In contrast, the underlying Orinda Formation is relatively impermeable. Measured hydraulic conductivities in the other units at LBNL are generally intermediate between these two formations." (RFI Report, Module A)*

In the late 1800's, various natural springs that existed at the base of Moraga Volcanic units were tapped to provide water for the University of California (now UC Berkeley) campus. Among the water sources mapped in 1875 by Frank Soule, UC Professor of Engineering, are two springs near the southern end of the location now occupied by LBNL Building 46 (Soule, 1875).

A preliminary geological report prepared for the Bevatron site by Chester Marliave, consulting geologist and registered civil engineer working for UC, noted:

> *"Seeps come out of the ground in many places, and even now several weeks since any rains have fallen there are four seeps issuing from the ground in the vicinity of the Bevatron. There are two known permanent springs in the area where tunnels have been drilled into the hillside, and pipes leading out from the caved entrances have been flowing water for many years"* (Marliave, 1948).

Marliave (1948) further noted that at the site of the future Bevatron *"Both the older Cretaceous sediments and the later Orinda sediments dip towards the hills and thus tend to hold back the absorbed water till the water table rises and allows it to seep out to the surface."*

3.02.5 Landsliding prior to Site Development

The earliest geologic map available for this area (Lawson and Palache, 1900) does not show landslide deposits. Lawson and Palache map faults offsetting bedrock units in the northern part of Blackberry Canyon (labelled Wolsey Canon) but no faults in the vicinity of the IGB site. Similarly, the pre-development geologic map and cross sections prepared in 1948 by Marliave does not show landslide deposits. However, the text that accompanies Marliave's map indicates that old landslide deposits existed at the site prior to development:

> *"There appears to have been considerable landsliding in this amphitheater in which the Bevatron is to be located and during periods of heavy rainfall the deep overburden and the underlying Orinda sediments become quite soft from the absorbed water;"* and *"The deep fill downstream of the Bevatron will have a maximum depth of 90 feet which will be underlain by soft pervious clays that are now sliding along the general contact with the Cretaceous* (i.e. Great Valley Complex) *sediments."*

More geologic recent maps prepared by consultants for LBNL (e.g. HLA, 1982) show known (i.e. historic) landslides, interpreted faults and bodies of Quaternary colluvium but do not map landslide deposits from before the lab was developed.

LBNL's RFI Report (LBNL/Parsons, 2000) presents an alternative interpretation that all of the Moraga Volcanic rocks in the vicinity of the IGB site are paleolandslide deposits of unspecified age. The geologic maps and cross sections presented in the RFI Report generally show paleolandslide deposits beneath LBNL Buildings 46 and 71, but not within the intervening canyon or beneath the Bevatron flat. As shown on Plate 16, LBNL/Parsons (2000) maps two paleolandslide deposits upslope (east) of the IGB site.

> A paleolandslide deposit composed of Moraga Formation rocks is mapped beneath the northern half of Building 46. This mapped deposit is about 340 feet wide beneath Building 46 and about 340 feet long in an upslope-downslope direction. As mapped, this deposit lies within and would have been displaced by the 1973 Building 46 landslide.
>
> A paleolandslide deposit composed of Moraga Formation rocks is also mapped beneath the southern portion of Building 46. This mapped deposit is about 120 feet wide beneath Building 46 and extends diagonally upslope towards the southeast beneath Buildings 17 and 27.

The inferred paleolandslide deposits mapped by LBNL/Parsons roughly coincide with the locations of Moraga Formation rocks shown on geologic maps prepared by other LBNL consultants. Geologic maps prepared previously by LBNL/Parsons (2000), HLA (1982) and other LBNL consultants do not show paleolandslide deposits or pre-development landslides beneath or intersecting the Bevatron flat.

3.03 Historic Landslides

3.03.1 Landslides during Site Development

The IGB site is within a parcel known as the Wilson Tract, which was annexed to the lab in 1948. A 1947 photograph of the Wilson Tract is presented Plate 17; Plate 18 shows elevation contours from a 1948 topographic survey drawing of the Wilson Tract (LBNL/Parsons, 2000). The first building built within the Wilson Tract was the Central Research Laboratory, which is now part of the Building 50 complex (west of the IGB site). In 1948, a new road was built leading from upslope areas of the lab down into Blackberry Canyon (Plate 19). The Building 50 area (west of the IGB site) is situated along a north-south trending bedrock ridge; development of the Building 50 area generally involved excavating the site to grade and placing the excavated soil as fill beneath Smoot Road, "J" Lot, the Cafeteria Lot and in the general area of the IGB site (Plates 19 through 21).

Virtually all of the grading to construct the Bevatron, Building 46 and the intervening section of McMillan Road was performed in 1949. Initial grading for McMillan Road and the Building 46 pad was underway in

February of 1949 (Plate 22). An April 1949 photograph that looks west at the location of the IGB site (Plate 23) shows that vegetation within the northwest-trending drainage had been cleared and concrete pipe segments that would later be placed at the bottom on fill were present onsite. In June 1949, grading for McMillan Road and the Building 46 pad were essentially complete (Plate 24) and grading of the cut slope east of the IGB site was partly complete.

An August 25, 1949 photograph (Plate 25) shows one of the initial slope failures that occurred in the cut slope upslope and east of the IGB site. Correspondence from later that year (Rossi, 1949) identifies this slope failure as Slide #1 with the following description:

> *Slide #1: "This slide, developed during the week of August 7, 1949, while extensive in area is not deep and actually consists of top soil and the sub-soil slipping down over the unweathered Orinda bedrock."*

The September 1949 photograph on Plate 26 shows Slide #1 and multiple other smaller slides in the cut slope southeast and south of the IGB site (including a slide identified in an attachment to Rossi, 1949 as "Slide A").

Two photographs taken in October of 1949 (Plates 27 and 28) show excavations that extend below the level of the Bevatron flat. Plate 25 and 26 generally show the open cut excavations made to construct the MG Basement, wind tunnels and other below-grade structures associated with the Bevatron as well as free-standing formed retaining walls surrounding the flat's southern perimeter.

The November 1949 photograph on Plate 29 shows the circular Bevatron excavation in the area of the IGB site as well as shows two additional slides in the west-facing cut slope north of Slide #1, which Rossi (1949) identifies and describes as follows:

> *Slide #2: "This slide, developed the week of September 11, 1949, is considered quite serious and is being studied very thoroughly. The slide plane is deep and it has been determined from field studies that this is an old slide which has begun to move again because the building excavation removed the toe of the slide."*
>
> *Slide #3: "This slide, developed to its present proportions during the week of November 13, was caused by underground water seepage and over-steepened slopes required for the installation of the Building wall. The water seepage has kept the cut bank soft and there has been progressive crumbling of the slope."*

By early December 1949 (Plate 30), Slide #3 and parts of Slide #1 appear to have been filled in. A second December 1949 photograph (Plate 31) shows the completed fill slope within the deepest part of Blackberry Canyon, which is several hundred feet northwest of the IGB site. A photograph taken in March of 1950 (Plate 32) shows the framing for the Bevatron building (Building 51) was essentially complete and two relatively fresh landslides in the cut slope above the Bevatron flat north of the primary Blackberry Canyon drainage (upslope of the location now occupied by LBNL Building 64).

3.03.2 1951 and 1952 Landslides

As described in the preceding section, multiple landslides occurred in the cut slopes made to construct the Bevatron flat during initial grading. For the most part, the landslides in the 1949-1950 timeframe occurred within excavated cut slopes at the site perimeter and did not extend a significant distance offsite.

The March 1952 photograph on Plate 33 shows the scarp of a much larger slide traversing the hillside upslope of the Building 46 cut slope. A letter prepared by Chester Marliave several years later (Marliave, 1955) includes a sketch map showing the 1952 landslide margin with the notation "slide of 3-10-52" along with a smaller slide near the northern end of Building 46 with the notation "1951 Slide". Marliave's letter, which pertains to an old water tunnel and not directly to the 1951 or 1952 landslides, notes that in the Building 46 area:

> *"The bedrock formation is comprised of brecciated lava rocks and pervious gravels and sands all of which are interbedded with clays. This entire mass is conducive to sliding where it becomes saturated with the winter rains"*

A report by an Investigative Committee of the U. S. Atomic Energy Committee (UCAEC, 1973) notes that *"in the latter part of 1951 a small slide formed behind the north end of Bldg. 46; later on March 10, 1952, it was encompassed in a much larger slide zone that extended the full length of the building."*

3.03.3 <u>1973 Landslide</u>

In 1973, a large landslide occurred involving much of the same area as the 1952 Landslide. The 1973 Landslide caused significant damage to Building 46 effectively shifting the northern two-thirds of Building 46 approximately 1.5 feet to the west. The approximate dimensions of the landslide are illustrated in the photograph on Plate 34 (USAEC, 1973).

Reports prepared at the time generally indicate that "significant" landslide movement began in January of 1973 and extending until mid-March (HLA, 1973; USAEC, 1973). A March 29, 1973 geologic review letter by Burton Marliave (registered Engineering Geologist) notes:

> *"It is now believed the most of the slide was comprised of older slide debris from the moraga volcanics and that this probably was not a tongue of inplace volcanics but the remains of an older slide. The depth of the slide from numerous drill holes put down appears to be 10 to 30 feet with the base of the slide moving on a clayey or gougy zone on the surface of the underlying Orinda formation."*

HLA's April 1973 report presents a similar interpretation, noting that:

> *"The geology of the area suggests that the landslide had its original movement many years ago. This is evidenced by the volcanic mass at the toe of the slide which is no longer connected to the closest volcanic outcrops located behind the south portion of Building 46."*

The geologic cross sections by HLA (1973) generally show the lower and upper portions of the landslide slip surface at the top of Orinda formation rock with a displaced layer of Moraga Volcanics in the middle of the landslide (beneath Building 46). The HLA (1973) report notes that:

> *"Movement of the slide below the road west of Building 46 is not apparent on the surface. No large cracks or significant bulging of the slide tow are present; however, survey measurements indicated movement on the slope above Building 51, but not inside the building."*

3.03.4 <u>Building 51 Slide</u>

Geotechnical reports from the early 1990's (GRC, 1993; Harza, 1994) refer to a relatively small landslide on the slope south of the mapped location of the 1973 Building 46 Landslide. GRC's 1993 report titled "Landslide at Fire Trail Access Gate" indicates that the landslide occurred at the location of a previous slope repair, noting that:

> *"In 1980, a slide repair consisting of earthwork, including keying and benching, was performed for the lower slope below the fire trail. The embankment slope failed again as evidenced by the cracking and lateral movement along the existing trail."*

A December 22, 1980 report by HLA summarizes observations and testing services made during the previous repair (HLA, 1980).

GRC's 1993 investigation includes subsurface data from five borings and ten temporary stabilization piers (caissons) drilled along the outboard edge of the access road as well as a Site Plan that shows the locations of tensional cracks within Smoot and McMillan roads. GRC's (1993) cross sections generally show the slide to have a maximum thickness of about 25 feet; Harza (1994) maps two slides within this general area: 1) a surficial slide area about 50 feet wide and 60 feet long (horizontal dimension) that extends from the outboard edge of the access road down to the level pad at the base of the slope; and 2) a deeper landslide about 120 feet wide and 120 feet long that extends from the east side of McMillan Road down to a level about 10 feet above the base of the slope.

3.04 Geotechnical Improvements

3.04.1 <u>Bevatron Mass Grading</u>

A report issued in September of 1949 (Dames & Moore, 1949) provides documentation on the fill placed to construct the Bevatron flat. The report indicates that:

> *"Fill placement commenced on about April 3, 1949 and continued with various interruptions and delays to termination on or about August 30, 1949. During the early period of construction, the work was hampered considerably by seasonal rainfalls and flow from several springs. During this period, considerable amounts of available fill soils were too moist in their field condition to be compacted to the required density. In view of the limited storage space available for soil not in condition to be used immediately, it was decided in consultation with the various groups concerned to allow this wet soil to be placed in the outer 10 to 15 feet (later increased to 20 feet) where loadings would be relatively light."*

The report notes that inspection and control of the fill was *"not continuous"* and documents a total of 70 field density test results. Based on these results, Dames & Moore (1949) concludes: *"except in the outer 10 to 20 feet of the embankment, required densities of 90 percent of the Modified A.A.S.H.O. laboratory test densities or better were generally achieved."* A Plate titled "Compacted Fill Data" shows most of the field density tests in areas of deep fill with no tests shown south of the UC 10+00N gridline (i.e. in the vicinity of the IGB).

3.04.2 <u>Stabilization/Repair of Pre-1973 Landslides</u>

A report issued in April of 1973 (USAEC, 1973) summarizes works and studies associated with the stability of slopes in the vicinity of the Building 46 slide area prior to the winter of 1972-1973. Included in the summary are the following descriptions of stabilization measures implemented in the pre-1973 timeframe:

> *"By the end of 1952, 32 horizontal drains had been installed into slopes behind the Bevatron (Bldg. 51), Bldg. 64 and Bldg. 46. Three vertical wells were drilled in the hillside behind Bldg. 46. A subdrain was installed below the concrete pavement behind Bldg. 46 to a maximum depth of 10 feet to intercept seepage."*

Most of the landslides that occurred in cut slopes surrounding the Bevatron pad were reportedly *"reconstructed with buttress fills"* (HLA, 1973). In 1958, the 1:1 horizontal to vertical cut slope behind Building 46 was reportedly cut "*back to 1-½:1, removing 4000+ cu. yds."* (USAEC, 1973).

3.04.3 <u>Stabilization/Repair of the 1973 Landslide</u>

Stabilization/repair measures implemented in response to the 1973 Landslide included:

- Removing landslide materials upslope of Building 46 (labeled "area of recent excavation" on Plate 14) to reduce driving forces on the lower portion of the slide. About 40,000 cubic yards of soil was removed (HLA, 1973).

- Structurally modifying Building 46 to isolate the southern end of the building from the portion on the active landslide. This effort included cutting the foundations on the west and east sides of the building, jacking out-of-line bents back into place and pouring new foundations (USAEC, 1973).
- Installing fourteen horizontal drains (hydraugers) in and below the slide area to reduce hydrostatic pressures; horizontal drains were also reportedly installed to tap the bottoms of vertical wells. The lengths of the horizontal drains ranged from 140 to 400 feet (HLA, 1973).
- Removing about 10 feet of soil from beneath McMillan Road and the parking lot on the west side of Building 46 (HLA, 1976b).

3.04.4 Phase I Seismic Slope Stabilization Project

A 1992 geotechnical investigation report by Kaldveer Associates, Inc. (Kaldveer, 1992) presents data, interpretations, analyses and recommendations for a project to enhance the seismic stability of the portion of the 1973 landslide that was not previously removed and replaced. The seismic analyses utilized pseudostatic methods, in which a uniform out-of-slope horizontal acceleration is applied to the landslide mass to model the destabilizing effect of earthquake ground shaking. Using this model, Kaldveer (1992) calculated the resisting forces needed to maintain seismic slope stability for pseudostatic acceleration values ranging from 0.125 and 0.200 times the acceleration of gravity (g). The supporting structural calculation package (PFFA, 1992), shows that added resisting force upon which the Phase I design is based includes a factor of safety of 1.15 on the force determined using a pseudostatic acceleration of 0.15g.

Kaldveer's (1992) Phase I stabilization design resists this dynamic out-of-slope force using a system of 51 caissons equipped with tiebacks. A 2014 photograph showing tops of Phase 1 caissons is presented in Plate 35; the caisson locations and numbering system are shown on Figure 1A. The Phase I caissons are numbered from south to north starting with Caisson 1, which is near the southern margin of the 1973 Landslide. The plans (C+D, 1992) show 42-inch diameter caissons spaced on 5-foot centers. The structural calculations (PFFA, 1992) indicate that the loading criteria used are based on two "design profiles". Plans and structural calculations for the Phase I stabilization system are included in the supplemental reference document titled "Previous Plans and Calculations". Information from the Phase 1 plans and calculations is summarized in the following table:

Design Feature	Design Profile I	Design Profile II
Caisson Numbers	1 - 37	38-51
Caisson Depth	58 feet	56 feet
Tieback Strands	2 @ 0.6 inches	3 @ 0.6 inches
Tieback Unbonded Length	30 to 45 feet	45 feet
Tieback Bond Length	30 feet	45 feet
Tieback Test Load	97.3 kips	140.6 kips
Tieback Lock Off Load	29.3 kips	44 kips
Design Lateral Load	75.9 klf	66.7 klf

The Phase I plans (C+D, 1992) include hydrauger drains and other features related to the project. The revision list on the plans includes the date 1/96 next to the notation "as built changes." Construction-phase records documenting caisson installation and tieback installation and testing have not been located for Phase I.

3.04.5 Phase II Slope Stabilization Project

A 1996 construction observation report for Phase II (Harza, 1996) documents an additional 19 caissons that were installed south Phase I in July of 1995 (between 1992 and 1996, Kaldveer was acquired by Harza). The tops of the Phase II caissons are presently below-grade (Plate 35); the caissons are numbered 1A to 19A from north to south (Figure 1A). Plans for the Phase II stabilization system (Harza 1994b) are included in the supplemental compilation titled "Previous Plans and Calculations;" however design details for the caissons have not been located. Caissons 1A through 19A were logged by a Harza geologist during drilling; the logs of the caissons are included in the supplemental compilation titled "Previous Subsurface Data." Neither the construction report (Harza, 1996) nor the Phase II plans (Harza, 1994b) show tiebacks being a part of the Phase II design.

Other geotechnical improvements documented in the Harza (1996) report include a group of hydraugers drilled from the level pad at the base of the slope (near Elevation +125 feet) and the reconstruction of a shallow landslide on the face of the slope with compacted fill reinforced with Tensar BX1100 textile at 3-foot vertical intervals.

3.04.6 Bevatron Demolition Project

Most of the pre-existing below-grade improvements at the IGB site were removed during the demolition of the Building 51-Bevatron complex. The demolition project included the removal of basement walls, slabs, tunnels, utilities and related below-grade improvements with the exception of deeply-embedded portions of existing piers/caissons. Existing upslope retaining walls at the perimeter of the Building 51-Bevatron complex that were left in place presently bound the east and south sides of the Bevatron flat. Some of the walls include drilled and grouted anchors (tiebacks) that were installed in association with the demolition project. Structural calculations and plans for the tieback installations (Cartwright Engineers 2010a and 2010b, respectively) are included in the supplemental compilation titled "Previous Plans and Calculations." A 2014 photograph showing tiebacks within the "South Room" is presented on Plate 36.

Plates 35 through 40 present photographs taken during the Bevatron demolition project. Plate 35 shows the locations of the features identified on the plans by Cartwright Engineers (2012b) as the North Room and South Room. The central portion of the Bevatron structure including below-grade wind tunnels can be seen on Plates 37 and 38. Plans for the Bevatron show circular wind tunnel foundations bottomed at Elevation +695.5 feet surrounding a narrower but deeper circular underpinning tunnel bottomed at approximately +689 feet. These elevations are 14.5 feet and 21 feet, respectively, below the elevation of the Bevatron flat.

Demolition and excavation activities associated with the removal of below-grade piers/caissons are shown on Plates 39 and 40. The locations of existing piers/caissons that were encountered and cut off were documented by survey prior to backfilling; survey data documenting the locations of buried piers/caissons (Cartwright Engineers, 2012) are included in the supplemental compilation titled "Previous Plans and Calculations."

Specifications for the demolition project included requirements for fill materials, compaction, and compaction control (QA/QC). Fill placement and compaction was intermittently monitored by a material testing and inspection firm, Consolidated Engineering Laboratories (CEL). Onsite inspection reports prepared by CEL personnel were provided to us by LBNL to review. In April 2012, the surface of the Bevatron flat was paved in Asphalt concrete. Prior to paving, portions of the subgrade were treated with cement (Plate 41) to mitigate soft and locally overwet soils at the site perimeter

4.00 GEOLOGIC INTERPRETATION AND ANALYSES

4.01 Overview of Geologic Findings

In the following sections, we present the results of interpretations and analyses based on new and existing subsurface data, historical photographs and direct onsite observations coupled with published materials and our personal knowledge of local historical, geotechnical and engineering geologic conditions from past work at LBNL. Geologic findings of significance to the IGB Project include the following:

- Most of the proposed IGB site is within an area that was cut to grade in 1949. The 1949 cut-fill transition passes through the northern portion of the IGB site (Figure 2); interpretation of pre-development topographic drawings suggest that the 1949 fill is at least 5-10 feet deep at the far northern end of the IGB.

- The IGB site is at the location previously occupied by the Bevatron, which included a variety of below-grade elements. Most of the IGB site is underlain by fill placed in association with the Bevatron demolition; the lower portions of drilled piers/caissons that were not removed during demolition also remain beneath the site.

- Existing landslide deposits in the direct vicinity of the IGB site are interpreted to “toe out” or “day-light” above the level of the retaining walls at the perimeter of the Bevatron flat. Accordingly, landslide-related forces are not a concern for new or existing retaining walls at the perimeter of the Bevatron flat adjacent to the IGB.

- Existing deeper landslide deposits to the north of the IGB site (i.e. in the vicinity of the 1973 Landslide) may intersect and/or extend below the Bevatron flat. Accordingly, landslide-related forces are a localized concern for new or existing retaining walls at the perimeter of the Bevatron flat northeast of the IGB.

4.02 Site Plan and Geologic Map

The Site Plan presented on Figure 1 shows the locations of borings drilled for this investigation, the logs of which are attached in Appendix A. Figure 1 also shows the locations of existing borings for which we have logs, which are included in the supplemental compilation document entitled “Subsurface Data from Previous Investigations.” The boring locations and landslides on Figure 1 were compiled from a variety of source materials, should be considered approximate, and not necessarily represent a complete dataset of subsurface information available.

The Geologic Map presented on Figure 2 presents our interpretation of the surficial geology in the vicinity of the IGB. The Site Geologic Map focuses upon bedrock units, landslide deposits and artificial fill placed during mass grading and initial site development. Fill placed to repair landslides and backfill within excavations beneath the Bevatron flat are purposely not shown on Figure 2 so that underlying bedrock and landslide relations can be better displayed. The locations of landslides that are known to have occurred during or after initial site development (i.e. after 1948) are shown on Figure 2 to provide context. Also shown on Figure 2 are the shallow and deeper Building 51 landslides introduced in Section 3.04.3. The locations of the landslides shown on Figure 2 were documented by maps contained in previous consultant-prepared reports and/or by historical photography. Figure 2 also shows the locations of geologic cross sections A-A’ through D-D’, which we developed to further interpret geologic relations.

4.03 Geologic Cross Sections

We organized this section to first introduce the lateral extent of the landslide deposits and basic geology presented in each of the geologic sections A-A’ through D-D’ (Figures 2 and 3). After this introduction is a synthesized discussion of the mapped landslides, their geometry, and extent using all of the cross sections and subsurface data available at the time this report was being prepared.

4.03.1 Geologic Cross Section A-A'

Cross Section A-A' is oriented northwest-southeast so that it intersects the existing 15-foot-high retaining wall near the northeast corner of the IGB (Figures 2 and 3). This section was constructed using borehole data collected as part of this study (A3GEO/LCI, 2014 B-1) and using past consultant reports (Dames and Moore, 1948; 1956; HA, 1965; HLA, 1976b; GRC, 1993; HARZA, 1994; A3GEO/AKA, 2011; 2012a; 2012b). Along the length of the section, bedrock is primarily northeast-dipping Orinda Formation, which is juxtaposed by a buried northwest-striking, and inferred east-dipping fault against Great Valley Complex in the northwest portion of the section (Figure 3). Moraga Volcanics are mapped at the top of the slope. Quaternary alluvium is present within the former Blackberry Canyon drainage and is now overlain by compacted fill (circa 1949) northwest of the proposed IGB. Uphill of the Building 47 pad, a thin (12-15 ft thick) veneer of colluvium is mapped in boreholes B-12, B-13, and B-1 (HLA, 1976b). This thin colluvium is mapped above Orinda Formation and Moraga Volcanics, which appear to be dipping to the east and into the slope (Figure 3). The A-A' section is oriented through the Building 51 landslide (GRC, 1993; HARZA, 1994), but otherwise this section has relatively few landslides (Figure 3).

4.03.2 Geologic Cross Section B-B'

Cross Section B-B' is oriented along a potential axis of pre-development landsliding based on an analysis of pre-development (1948) topography (Figures 2 and 3). This section was constructed using borehole data collected as part of this study (A3GEO/LCI, 2014 B-2 and B-5) and using past consultant reports (Dames and Moore, 1956; HA, 1965; HLA, 1973; 1976b; Kaldveer, 1992; AKA, 2012a). Along the length of the section bedrock consists of primarily northeast-dipping Orinda Formation that is overlain by slightly east-dipping to subhorizontal Moraga Volcanics uphill of Building 46. Based on geologic bedding inclinations from mapping and in downhole geophysics, as wells as boreholes, we infer that the Moraga Volcanics and Orinda Formation are slightly folded into a broad anticline in the southeastern part of the section. Upslope of Building 46, Moraga Volcanics are overlain by 15 to 20 ft of colluvium and artificial fill and are largely outside of the 1973 landslide repair. In the northwestern portion of B-B', a thin (10 to 15 ft thick) section of Quaternary alluvium and colluvium is present within the former drainage and is now overlain by 50 to 70 ft of compacted fill (circa 1949) north of the proposed IGB. The B-B' section is oriented through the left margin of the 1973 landslide (HLA, 1973) and through 1949 Slide #1 (Figures 2 and 3).

4.03.3 Geologic Cross Section C-C'

Cross section C-C' trends east-west through the central axis of the 1952 and 1973 Landslides utilizing previous and new borehole information, along with observations of past historic failures, to constrain the limits of the potential landslide related deposits (Figures 2 and 4). This section was constructed using borehole data collected as part of this study (A3GEO/LCI, 2014 B4) and using past consultant reports (Dames and Moore, 1948; 1956; HLA, 1965; 1973; 1976; A3GEO/AKA, 2011; 2012a; 2012b; HLA, 1976). Along the length of the landslide, bedrock is primarily northeast-dipping Orinda Formation, composed of a series of interbedded, siltstones, claystones, shales, and sparse marl beds. Moraga Volcanics are mapped at the top of the slope (generally above the 1973 headscarp) and as a thin approximately 25 ft thick interbed within Orinda Formation directly upslope of Building 46. On the western most portion of the section Quaternary alluvium is mapped within the former Blackberry Canyon drainage and overlain by compacted fill (circa 1949).

4.03.4 Geologic Cross Section D-D'

Cross Section D-D' is north-south oriented and intersects the 1973 Landslide as wells as borings drilled along the access road about midway up the slope east of the Bevatron Flat (Figures 2 and 4). This cross section is intended to help correlate interpretations between each of the cross sections; and was constructed using borehole data collected as part of this study (A3GEO/LCI, 2014 B-1, B-2, and B-4) and using past consultant reports (Dames and Moore, 1948; 1956; HA, 1965; HLA, 1973; 1976b; Kaldveer, 1992; GRC, 1993; Harza, 1994). Along the length of the section, bedrock is primarily northeast-dipping Orinda Formation. Moraga Volcanics are mapped in the northern most portion of the section. Several thin fill bodies have been placed within former swales and on slopes.

4.04 Landslides

4.04.1 1973 Landslide

The original 1973 landslide was approximately 450 feet long, up to 260 feet wide, and 25-57 feet deep and is intersected by cross sections B-B', C-C' and D-D' (Figures 2, 3 and 4). As discussed in Section 3.04.3, a large portion of the landslide up slope of Building 46 was removed and replaced with compacted fill as part of the landslide repair and stabilization (HLA, 1976). However, a portion of the original landslide remains in place beneath and downslope of Building 46 (Figure 4). As shown in section C-C' and D-D' (Figure 4), the remaining 1973 landslide mass is approximately 260 feet long, up to 260 feet wide, and up to 40 feet deep.

The basal slide plane for the 1973 slide is well-constrained base on multiple borehole observations and inclinometer measurements, including: B-4 (A3GEO/LCI, 2014), B-40 (HLA, 1976), SI-1 and SI-5 (HLA, 1976a), and B-1 (HLA, 1973) (Figure 5). These data indicate the 1973 landslide material is composed of primarily Moraga Volcanics and movement along the basal slide plane is coincident with the contact between Moraga Volcanics (Qls) and the underlying Orinda Formation. For example, near the toe of the 1973 slide, we identified in Boring B-4 (A3GEO/LCI, 2014) the basal slide plane—composed of a 0.5-foot-thick reddish brown laminated clay juxtaposing landslide debris above from intact hard Orinda Formation below a depth of approximately 18 feet. At the adjacent SI-5 inclinometer, movement was recorded along the basal slide plane at a similar depth (HLA, 1973).

Further upslope the basal slide plane for the 1973 slide was identified in borehole B-40 (HLA, 1976) at a depth of approximately 57 feet (several feet below the groundwater table) where Moraga Volcanics (i.e. andesite) overlies Orinda Formation and is separated by a soft saturated sandy clay along the basal slide plane. The movement along the basal slide plane (at the Moraga/Orinda contact) was recorded in SI-1 (adjacent to Building 46) at a depth of about 46 feet (HLA, 1973). Section D-D' (Figure 4) illustrates that the 1973 landslide may be slightly thicker further south near B28 (HLA, 1973) and appears to thin along the southern margin of the slide near B-2 (this study) and B29 (HLA, 1973) (Figure 6). Together, these data suggest that the basal slide plane for the intact portion of the 1973 slide is generally flat to slightly inclined to the west (10-18°) and occurs at or near the Moraga/Orinda contact.

4.04.2 Building 51 Landslide

The Building 51 landslide is shown in Sections A-A' and D-D' (Figures 2, 3 and 4). As discussed in Section 3.03.4, the Building 51 landslide is composed to two landslides, including: (1) a shallow surficial slide composed of grayish brown to reddish brown gravelly lean/fat clay, and poorly graded gravel (B-1, this study); and (2) a potential deeper landslide composed of the Orinda Formation (dark reddish brown claystone (B-1, this study) and dark greenish gray to grayish red siltstone to sandstone (B-6 and B-7, GRC, 1993). The shallow surficial slide and subsequent repair are not shown on Section A-A' because our focus for this study is characterizing the stability of the deeper landslide. The deeper Building 51 landslide is shown on Sections A-A' and D-D' as approximately 75 to 110 feet long, 120-130 feet wide, and up to 30 feet deep.

The geometry and lateral extent of the deeper Building 51 landslide is reasonably well constrained based on multiple boreholes, caisson holes, deformed cultural features, and historical photography. The basal slide plane was identified at approximately 21-22 feet depth in Boring B-1 (this study) and marked by a transition from dark reddish brown gravelly fat clay to a dark reddish brown claystone with polished slickensides (although no distinct shear was identified)(Figure 3). Similarly, GRC (1993) discuss the potential slide plane in both boreholes (e.g. GRC, 1993 Boring B-6) and the caisson holes as a zone of brown to gray slightly moist highly plastic soft clay with abundant slickensides within Orinda Formation at approximate 25 feet depth. This potential deeper landslide plane is mapped over a distance of 30 feet between caisson holes C-3 to C-10 along the fire access road suggesting this clay layer is subhorizontal, has some lateral continuity, and is not an isolated clay shear within Orinda Formation. GRC (1993) also mapped a series of crescent shaped tensional cracks along McMillan Road (from the intersection of McMillan and Smoot Roads to GRC Boring B-4) (Figures 2 and 3). These tensional cracks appear to be the result of movement of the deeper Building 51 landslide; and constrain the width of the landslide and delineate the uphill extent of the landslide.

Lastly, review of historical photography during the initial construction of the Bevatron building (dated September 29, 1949; photo Bev.-133 and Bev.-137) help to constrain the location of the deeper Building 51 landslide toe. These photographs illustrate the eastern and southeastern slopes of the Bevatron Flat. At the base of the slope is light to dark gray banded bedrock (Unit 1) that we infer to be Orinda Formation that is dipping 30-40 degrees to the northeast to a height of approximately 20 to 25 feet above the base of the Bevatron Flat. Unit 1 is overlain by a poorly exposed light gray deposit (Unit 2) with weak subhorizontal bedding that is approximately 10 to 15 feet thick. Unit 2 is overlain by a 5- to 10-foot-thick uniformly dark gray deposit (Unit 3) that is inferred to be surficial fill or colluvium. At the base of the Unit 2, are a series of pipes that extend out of and down the slope to a series of oil drums. The middle pipe has a dark colored halo around it suggesting it may be moist. These observations suggest these pipes are hydroaugers installed at the base of the Unit 2. The base of Unit 2 appears to be the toe of the deeper Building 51 landslide, which is roughly coincident with the mapped landslide toe by Harza (1994) at approximately elevation 733 to 740 feet.

Together with the borehole data, this information suggests that the deeper Building 51 landslide has a steep curvilinear geometry between boreholes B-7 and McMillan Road and becomes very low angle east of B-7 before exiting the slope, 23 to 30 feet above the Bevatron Flat (Figures 2, 3 and 4).

The northern and southern limits of the deeper Building 51 landslide are more poorly constrained (Figure 4). We infer that the southern extent of the deeper Building 51 landslide could coincide with the limit of extensional fractures mapped along McMillan Road by GRC (1993), although the landslide may extend further to the south beneath colluvium. The basal slide plane for the deeper landslide may also extend to the north and join the basal slide plane of the 1973 landslide (as shown in D-D'; Figure 4). The available data used to infer the presence of a possible deeper landslide beyond the extent of the Building 51 landslide identified by GRC (1993) and Harza (1993) are discussed below.

4.04.3 Deeper Pre-Development Landsliding

As shown in Figures 2 through 6, a deeper landslide mass is mapped beneath the more surficial 1973 and Building 51 landslides. Evidence for this deeper landslide includes a broad group of data, including: (1) interpretation of historical topography, (2) observations in boreholes/caissons, and (3) excavation mapping. Below we provide a synthesis of each of these observations to help establish the possible extent of the deeper landslide.

Interpretation of pre-development 1948 topography (Plate 16 and Figures 3 and 4) suggests a deeper landslide mass may extend beneath the northern portions of the Bevatron Flat. The original pre-development topography (circa 1948) is gently northwest-sloping in each section with no distinct breaks in slope consistent with a toe of a large landslide. However, a break in slope (that can be interpreted as a landslide toe) is shown in the original topography between boreholes B8 (Dames and Moore, 1956) and B6 (HLA, 1965) (Figure 3). This observation supports the interpretation that a deeper landslide that may project out of slope further downhill (at a location now buried by circa 1949 fill). It is important to note that in the vicinity of the 1973 and Building 51 landslides historical topography does not show a significant change in the original slope suggestive of the 'bulge' typically observed at a landslide toe. This observation suggests that these historical slides possibly represent older landslides that have been reactivated by grading of the Bevatron Flat.

Similar to the historical topography, several key observations in the available borehole data suggest the presence of a possible deeper landslide. First and foremost, in Boring B-5 (this study) we identified a highly weathered rubbly interval of andesite (interpreted as Moraga Volcanics) from the surface to a depth of 27 feet (Figures 2 and 3). Although originally interpreted as fill, close inspection revealed the clasts contain weathered plagioclase crystals and has filled vesicles consistent with andesite of the Moraga Volcanics. These data suggest the Moraga Volcanics in Boring B-5 likely represent a block of andesite that has been translated down the slope through landslide processes (similar to the 1973 landslide). Interestingly, only Orinda Formation is mapped in the other boreholes in this area and other recent boreholes drilled in the vicinity for other purposes did not collect samples above 30 feet in depth (Figure 2).

Second, we have identified a series of possible landslide slide slip surfaces in multiple boreholes at key locations within intact Orinda Formation that permit the geometries shown on Figures 3 and 4. In no place along the inferred deeper basal slide plane did we need to “force” a landslide slip surface through a borehole with no discontinuity at that depth. Evidence used to infer the deeper landslide shear plane included: slickensided intervals of siltstone and/or claystone, intensely fractured zones, and moist flat clay seams. For example, we identified a series of clay seams, shears, and/or possible deeper landslide shear planes at 22, 27, and 29 feet in depth in Boring B-2 (this study) (Section B-B’, Figure 3). HLA (1973) identified a slickensided interval of siltstone at a similar depth (19 feet) and sheared siltstone at 32 and 26 feet in depth in B-29. Another example can be made in Section C-C’ where (at approximately 32-ft depth in B-4) we identified a 0.1-foot-thick clay seam that could represent a possible deeper landslide constrained to the Orinda Formation below the 1973 landslide.

Additional evidence for possible deeper bedrock- involved landsliding is provided by excavation mapping performed by A3GEO/LCI in 2012 during the removal of facilities on the Bevatron Flat (Figure 2). In the floor of the Injector excavation, A3GEO/LCI (2012) mapped a 0.5-ft-thick soft highly polished and slickensided clay shear zone at approximately 704 ft elevation within Orinda Formation (Figure 4). The base of the clay zone strikes northeast and dips 20-32 degrees northwest (out of slope). This dip direction is inconsistent with the northeast dip of beds within the Orinda and is more suggestive of a possible landslide failure plane.

Based on these data (among others) we infer a deeper (possibly bedrock involved) landslide may extend beneath the Bevatron Flat north of the Building 51 landslide (Figure 2).

4.04.4 <u>Deeper Pre-Development Landslide Geometry</u>

As shown in the geologic cross sections, the geometry of the deeper landslide is poorly constrained, except beneath the Building 51 landslide. The follow discussion explains the information and rationale used to draft the geometries shown on each cross section. As shown in the A-A’ section (Figures 2 and 3), both historical photography and subsurface data suggest the deeper landslide projects out of the slope above the base of the Bevatron Flat (see description of the Building 51 landslide for further details). The inferred deeper pre-development landslide does not extend beneath the proposed IGB footprint.

Further north in the Section B-B’ (Figure 4), we infer the basal slide plane extends at a low-angle geometry beneath the Bevatron Flat based on: (1) a zone of clay seams, and shears in Boring B-2 (this study) at 22, 27, and 29 feet in depth, (2) the anomalous Moraga Volcanics identified in Boring B-5 (this study), and (3) based on historical topography, which suggests the landslide toe may project out of slope near boring B8 (Dames and Moore, 1956). Beneath the eastern edge of the Bevatron Pad, the geometry of the slide is poorly constrained and we infer two possible basal slide plane geometries. Available information suggests the basal slide plane of the deeper slide merges with the 1973 landslide just upslope of Boring B-2 (Figure 4).

In the C-C’ section, the deeper slide plane geometry is based on available borehole and inclinometer data (Figure 5). For example, at approximately 32-foot depth in B-4 we identified a 0.1-ft-thick clay seam that may represent a possible deeper landslide below the basal slide plane of the 1973 landslide and is within the Orinda Formation. In the adjacent B-37 borehole, two zones of clay 5 to 6 inches thick were logged by HLA (1976b) between 32 and 34 feet in depth. These clays are described as soft moist to saturated and may represent a possible deeper slide plane within Orinda Formation. Conversely, these clay shears have a steep northeast dip of 76 degrees, which if correct is not likely a slide plane. Available borehole and inclinometer data also suggest that the inferred deeper slide plane possibly merges with the 1973 slide plane, either in Boring B-4 or further east at borehole B-40 (HLA (1976). No description of deeper shearing is noted within boreholes B-40 or SI-1 (although the descriptions are limited). Further to the west, it is unclear if the inferred slide plane projects to the base of the retaining wall at the eastern edge of or projects deeper below the Bevatron Flat. Unfortunately, available boreholes in the vicinity were drilled for a different purpose and were not logged at shallow depth; the results from logging the cuttings are of only limited use (i.e. B-2 and B-3, A3GEO/AKA 2012a).

In Section D-D' (Figure 4), the lateral extent of the deeper landslide is shown as correlated with the other cross sections and additional borehole data along the section. We infer that the southern extent of the Building 51 landslide could coincide with the limit of extensional fractures mapped along McMillan and Smoot Roads by GRC (1993), although the landslide may extend further to the south beneath colluvium (Figure 4). The deeper landslide may also extend to the north based on discontinuities consistent with a landslide shear plane mapped at approximately 20-foot depth in B29 (HLA, 1973) and in EB-3 (Kaldveer, 1992). We also tentatively infer the deeper landslide extends beneath the northern margin of the 1973 landslide (Figure 4) based on the C-C' section. The basal slide plane is projected beneath B27 (HLA, 1973) and through a zone of intensely fractured and slickensided claystone at approximately 27 ft depth in EB-1 (Kaldveer, 1992).

Collectively, these data suggest the possible presence of a deeper possibly bedrock involved landslide that may extend beneath the Bevatron Flat north of the Building 51 landslide (Figure 2). It is important to note, the geometry and extent of the deeper landslide is poorly constrained, especially beneath the Bevatron Flat (Figure 2). Available historical topographic maps (Plate 16) are of insufficient detail to map the boundaries of the slide. Further data collection is likely required that specifically targets this potential landslide to confirm the geometry and extent of the slide.

4.05 Geologic Hazard Assessment

4.05.1 Earthquake Ground Shaking

The Project is located within the seismically-active San Francisco Bay Area and it is likely that the site will experience strong earthquake shaking during the life of the project. Strong earthquake shaking is a hazard shared throughout the region and direct effects of earthquake ground motions on structures are addressed through the structural design provisions of the California Building Code (CBC). Earthquake ground shaking can also produce ground failures as a result of geotechnical losses in strength (e.g. liquefaction or seismic softening) and/or inertial effects (landsliding or lateral spreading). Geotechnical parameters for code-based seismic design are presented in the recommendations section of this report.

In 2000, a suite of probabilistically-derived ground motions were developed for the nearby UCB main campus by URS Corporation (URS). This standardized suite of ground motions (response spectra and acceleration time histories) was updated in 2003 and again in 2008. The 2008 update included ground motions for LBNL for four return periods (72, 475, 949 and 2,475 years). The results for a 475-year return period (10 percent probability of exceedence in 50 years) are summarized in the following table.

LBNL Probabilistic Ground Motions (URS, 2008)

475-year Return Period Spectral Accelerations		
0.01-Second Period	**0.2-Second Period**	**1-Second Period**
0.86g	2.02g	0.85g

We used the 2008 URS-derived spectral accelerations for our conceptual-level analyses to estimate probable landslide displacements.

The USGS recently updated the national seismic hazard maps as described in Open File Report (OFR) 2014-1091 http://earthquake.usgs.gov/hazards/products/conterminous/. OFR 2014-1091 provides information on the sources and information used to estimate ground motions in the United States, and California, in particular. The 2014 updated seismic hazard map incorporates a new seismic source model (i.e., active faults and seismic sources) developed by the Southern California Earthquake Center and the CGS based on the Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3, www.scec.org/ucerf/; WGCEP, 2013). A significant difference between the 2008 and 2014 models is that UCERF-3 incorporates many more multi-segment ruptures than in previous versions allowing for larger ruptures along potentially linked faults (Frankel and others, 1996, 2002; Petersen and others, 2008). All of these models involved a major update in the methodology for calculating earthquake recurrence.

In the new 2014 USGS OFR 2014-1091 report, comparisons are made between the 2008 and 2014 models, which suggest that there could be an increase in ground motion hazard at LBNL. Online tools are not yet available to calculate site-specific ground motions using updated 2014 models; thus, it is presently unclear how design earthquake ground motions at the site may change in the future. Moving forward beyond conceptual design, LBNL should consider further review of the new ground motion hazard maps and the maps implications with respect to final design of the IGB site. For instance, additional ground motion analysis may be required to understand the full range of uncertainty in ground shaking hazard at the proposed IGB site.

4.05.2 Surface Fault Rupture

The closest known active fault is the Hayward fault, the nearest trace of which is mapped (CDMG, 1982) about 1,000 feet west of the site. The various other faults that have been mapped closer to the site (including at/near the contact between the Great Valley Complex and the Orinda Formation) are not considered active. The IGB site is well outside of the official Alquist-Priolo Fault Hazard Zone that surrounds the Hayward fault. In our opinion, the overall potential for significant fault-related offsets to occur at the IGB site is low.

4.05.3 Inundation

The IGB site is located in the Berkeley Hills at Elevation +710 feet; inundation by sea level rise, tsunami or seiche is therefore not a concern. There are no lakes or open bodies of water within the Blackberry Canyon watershed and the reservoirs that are present consist of tanks that are not particularly large. In our opinion, the overall potential for significant inundation to occur at the IGB site is negligible.

4.05.4 Liquefaction/Densification

Liquefaction is a phenomenon whereby certain types of susceptible soils can lose strength, compress and gain mobility (i.e. flow) in response to earthquake ground shaking. Saturation is a prerequisite for liquefaction to occur and the soils that are most susceptible to liquefaction are clean sands, silts and gravels in a loose to medium dense condition. Densification can occur where these same types of soils are above groundwater. Current and ongoing research has demonstrated that cohesive silts and clays of low plasticity can also exhibit seismic strength degradation behavior that is in some ways similar to liquefaction. The range of conditions over which this behavior occurs is the subject of continuing research; however, there appears to be general agreement that soils with a Plasticity Index (PI) of 7 or less are susceptible to earthquake-induced strength loss, whereas soils having a PI of 18 or greater are not.

Based on our review of the available data, it appears that the fill that underlies the IGB site is predominantly cohesive and was compacted under intermittent engineering control. Borings drilled in the vicinity of the IGB site generally indicate that the fill is underlain by Orinda Formation bedrock and/or colluvium comprised of soils that are predominantly cohesive. The available data suggests that soils susceptible to seismically-induced liquefaction and densification are generally absent beneath the IGB site. Accordingly, we judge that the overall potential for significant liquefaction to occur beneath the IGB site or in adjacent upslope areas is essentially nil.

4.05.5 Landsliding

As documented throughout this report, there is a history of landsliding in the vicinity of the IGB site and planned retaining walls on the east side of the MUP site may be affected by landslide deposits that existed prior to the development of the Bevatron flat. Based on the available data, we interpret that existing landslide deposits do not intersect the planned IGB site or the retaining walls (new or existing) adjacent to it. Based on this interpretation, we judge that the overall potential for existing landslide deposits to significantly affect the IGB itself is generally low. However, from an engineering geologic perspective, we judge that there to be a significant potential for existing landslide deposits to significantly affect: 1) the planned MUP site; and 2) the planned access driveway southeast of the IGB. Potential earthquake-induced landslide displacements and forces are examined further in subsequent sections of this report.

5.00 GEOTECHNICAL ANALYSES AND EVALUATIONS

5.01 General Conclusions and Findings

Based on the available data, we conclude that the planned IGB project is feasible from a geotechnical standpoint. The site of the IGB itself appears to relatively free of geologic hazards other than strong earthquake shaking; however, existing landslide deposits and steep cuts/fills upslope have some potential to affect the design of the access road south of the IGB at the second-floor level.

Existing landslide deposits upslope of the MUP site is a significant consideration for the siting and design of the MUP. This report includes an initial analysis of seismically-induced landslide displacements and forces intended for conceptual design and future planning purposes.

A principal geotechnical consideration for the IGB project is the presence of existing fill materials beneath the site. As envisioned at this time, the IGB and MUP will be supported on shallow foundations that bear upon improved ground and/or bedrock. Geotechnical analyses and evaluations pertaining to the project conceptual design are discussed in the sections that follow.

5.02 IGB Foundations

5.02.1 Foundation Support

The IGB site includes existing fill materials placed during the initial grading of the Bevatron flat (1949) and in association with the Bevatron Demolition Project (2009-2012). Pre-existing fill materials are generally considered unsuitable for the direct support of new foundations unless it can be documented that the materials were placed under adequate engineering controls. In geotechnical practice, such controls typically include:

- Geotechnical observation of subgrades before fill is placed to confirm that firm natural materials are present;
- Geotechnical observation during fill placement to verify lift thicknesses and uniformity between field density test locations;
- A sufficient number and distribution of field density tests to verify that specified compaction levels (relative to laboratory 100% compaction values) have been achieved; and
- A construction report documenting how the fill was placed and any field changes or exceptions to the specifications.

Relative to the above practices, it is our opinion that the construction report documenting initial site grading (Dames & Moore, 1949) lacks specificity and does not include any field density tests in fill areas near the IGB site.

The earthwork reports (field dailies) submitted by Consolidated Engineering Laboratories (CEL) in association with the Bevatron Demolition Project include the results of field density tests within the area of the IGB site; however, the field dailies provide little direct evidence that the fill was placed on firm natural subgrade materials. Given the complexities and phasing of the Bevatron below-grade demolition, we judge it possible that pockets/layers of disturbed and/or under-compacted materials could be present beneath the locations where IGB foundations are planned.

We judge that IGB can be adequately supported on a shallow foundation system comprised of spread footings and structural mats, essentially as planned, provided that the soils below the foundations are appropriately improved. This report discusses rammed aggregate piers, cement soil mixing and removal and replacement as possible ways to locally improve soils below footings. Excavation and replacement with engineered fill or flowable material would also be geotechnically acceptable. Alternatively the IGB could be supported on drilled piers or another type of deep foundation system that gains support in natural undisturbed materials beneath the existing onsite fill.

Preliminary recommendations for shallow foundations (spread footings and mats) designed to bear upon improved ground are presented in Section 6.02.1.

5.02.2 Uplift Resistance

Micropiles can be used to resist upward tensile loads caused by earthquake ground shaking. As used in this report, the term “micropile” refers to a drilled foundation element consisting of a high-strength steel threadbar surrounded by cement grout. The central threadbar of the micropile typically extends up into the footing, grade beam or mat to make a structural connection. Micropiles that function as tiedown anchors can be post-tensioned off to limit upward movements; in this case, the top of the micropile is designed to be “unbonded” and the threadbar extends through the footing, grade beam or mat so that it can be tensioned and locked off to a specified load.

Micropiles resist axial loads by skin friction, which is significantly enhanced through the technique of post-grouting. Typically, drill holes for micropiles range from about 6 to 12 inches in diameter. Micropiles that are grouted under gravity conditions (i.e. not post-grouted) would typically designed using the same skin friction values that would be used for a conventional drilled pier. Micropiles that are to be post-grouted have grout tubes attached to the central threadbar with specially-designed grout ports over the length of the bond zone. After the initial (gravity) grout has set, grout is pumped into the post-grout tubes under high pressure to fracture and displace the hardened grout outward, which greatly increases skin friction capacity.

Micropile capacities and load-deflection behavior are confirmed by load testing. Specialty micropile contractors have developed a variety of techniques and proprietary systems to construct high capacity micropiles. Plans and specifications prepared by the project Structural Engineer typically include micropile locations, threadbar diameters, design capacities, corrosion protection requirements and testing and acceptance criteria. Other details involving the micropile design are commonly determined by the specialty micropile contractor, subject to the review and approval of the project structural and geotechnical engineers.

Section 6.02.2 of this report presents preliminary geotechnical criteria for use in developing conceptual-level micropile designs.

5.03 IGB Ground Improvement

5.03.1 Ground Improvement below Foundations

The conceptual design of the IGB includes ground improvement beneath shallow foundations. Specialty contractors have developed a variety of techniques and proprietary systems for ground improvement; in our opinion, primary objectives of a ground improvement program for the IGB should include: 1) achieving adequate bearing and acceptable settlement/deflection under the anticipated loads; 2) compatibility with onsite environmental conditions; and 3) overall cost effectiveness. We judge that potentially feasible and appropriate ground improvement technologies for this site are likely to include the following types of systems or similar derivatives:

> ***Rammed Aggregate Piers® - Vibro Piers ® -*** In these types of systems, an auger or mandrel is typically used to create a hole in the soil in which a pier comprised of aggregate is constructed. The aggregate is placed in the hole in lifts and compacted by ramming/vibrating with a high-energy device that displaces the aggregate both downward and outward, densifying the aggregate and the surrounding soil. Additional aggregate is added in incremental lifts and the process is repeated to construct a dense aggregate column.

> ***Cement Soil Mixing (SMX) –*** In the SMX process, admixtures are introduced into the soil using single- or multiple-axis augers to form columns or panels of mixed soil. The admixture can consist of cement, lime, fly ash, slag, or other additives. Once the treated soil sets, it forms a strong and rigid material.

From a geotechnical standpoint, removal and replacement would also be an acceptable means of improving the ground below new foundations, provided that the backfill material can be appropriately engineered to have acceptable bearing and long-term settlement characteristics. For conceptual design, we recommend assuming that removal and replacement would only be appropriate in areas where undisturbed bedrock is shallow. Among the considerations for deeper excavations at this site are: 1) instability concerns and dewatering requirements due to shallow groundwater; 2) environmental concerns associated with the handling, characterization, treatment and offsite disposal of excavated soils and groundwater: and 3) health and safety requirements for temporary shoring and/or the laying back of temporary excavation slopes. We note that some of the preceding considerations also apply to excavations made with drilling equipment.

Section 6.03.1 of this report presents preliminary geotechnical criteria for use in developing conceptual-level ground improvement designs.

5.03.2 Ground Improvement below Slabs-on-Grade

The envisioned ground improvement program includes drilled aggregate piers SMX columns/panels beneath spread footings, but does not require ground improvement below all slab-on-grade areas. As currently envisoned, ground floor level slabs-on-grade will be underlain by a compacted aggregate layer placed on a prepared stable subgrade. Any weak, unstable or otherwise unsuitable soils present at subgrade level, will be selectively overexcavated and replaced with appropriately engineered material. Prior to the placement of aggregate layers, the exposed subgrade will be compacted and confirmed to be firm and non-yielding. The overall intent is provide uniform support for slabs-on-grade that will be relatively lightly-loaded.

Under earthquake conditions, it is possible that localized pockets of non-improved ground below slabs-on-grade will densify and settle. The magnitude and pattern of subgrade settlement is likely to be variable; however, we anticipate that in some cases portions of slabs may be forced to span and that some cracking of slabs-on-grade could occur. The overall risk of slab cracking is greatest for slabs that are heavily loaded. Where risks associated with localized subgrade settlement are considered unacceptable, the ground below slabs-on-grade should be improved in a manner similar to what is recommended for footing and mat foundations.

5.04 Other IGB Design Considerations

5.04.1 Shallow Groundwater

As noted previously this report, hydrogeologic conditions in the vicinity of the site are complex with a long documented history of localized springs, seeps, and locally wet conditions. During the demolition of the Bevatron, groundwater was observed within basements and site excavations and wet conditions at the perimeter of the Bevatron flat were mitigated by cement treatment prior to paving. It should therefore be anticipated that under current conditions, groundwater may at times rise to the level of the Bevatron flat in localized areas.

In general, we recommend that a waterproofing expert be consulted on issues relating to moisture control for buildings. In our opinion, minimum requirements for moisture control should include:1) a gravity underdrainage system beneath the IGB to prevent groundwater from becoming trapped and rising to the level of the ground floor slabs-on-grade; and 2) a heavy duty membrane overlying the underdrainage layer to inhibit water vapor transmission. We anticipate that some below-grade portions of the structure may need to be waterproofed and that hydrostatic pressures may factor into the structural design of deeper elements such as sumps or elevator pits unless a positive means of gravity drainage is provided.

Section 6.03.2 of this report includes preliminary geotechnical criteria for the conceptual-level design of a slab underdrainage - moisture retarder system.

5.04.2 Expansive Soils

Expansive soils shrink and swell with changes in moisture and have the potential to damage improvements unless appropriately mitigated. In local practice, correlations with Plasticity Index (PI) are often used to evaluate expansion potential. For example, “non-expansive fill” is commonly required to have a PI of 15 or less. Expansion potential is mostly a concern for soils that are shallow (i.e. near the ground surface) as deeper soils are typically less affected by seasonal drying.

The PI values obtained from Atterberg Limits determinations conducted on samples from Boring B-6 and B-7 ranged from 18 to 25, which is generally indicative of soils with a moderate to high expansion potential. Expansive soils are generally not a concern for building elements deeper than about 3 feet (below lowest adjacent grade) or in soils that have been improved by SMX. Expansive soils, if present at subgrade level, will need to be selectively overexcavated and replaced with non-expansive material (such as engineered fill or lean concrete).

Section 6.03.3 of this report includes preliminary geotechnical criteria for conceptual-level evaluations of non-expansive soil requirements.

5.04.3 Soil Corrosivity

We screened for the presence of corrosive soils by conducting a suite of geochemical laboratory tests on one sample from the site. Guidelines on the interpretation of the chloride, sulfate and pH test results presented in the following table was obtained from Caltrans (2003); Based on these guidelines, the tested samples would not be considered corrosive.

Corrosion Test Data and Guidelines

Geochemical Test	Sample ID and Test Results		Corrosion Threshold for Structural Elements
	B-6 @ 6 feet	B-7 @ 7 feet	
Resistivity @ 15.5° C (ohm-cm)	881	1713	see below
Chloride (mg/kg or ppm)	12	3	≥ 500
Sulfate (mg/kg or ppm)	230	81	≥ 2,000
pH	7.6	10.2	≤ 5.5

The Caltrans guidelines do not include soil resistivity; the following guidelines are from the National Association of Corrosion Engineers (NACE):

Soil Resitivity (ohm-cm)	Soil Classification
Below 500	Very Corrosive
500 – 1,000	Corrosive
1,000 – 2,000	Moderately Corrosive
2,000 – 10,000	Mildly Corrosive
Above 10,000	Progressively Less Corrosive

Based on the NACE criteria, the sample from Boring B-6 would classify as “Corrosive” and the sample from Boring B-7 would classify as Moderately Corrosive.

A qualified corrosion engineer should be consulted if additional interpretations or recommendations pertaining to corrosion are desired.

5.05 Design Considerations for the MUP

5.05.1 General

We suggest that the preceding discussions relating to the IGB be reviewed and considered in the conceptual design of the MUP. However, we anticipate that performance requirements for the MUP may be less stringent and that certain items considered essential for the IGB may not be essential for the MUP. For example, ground improvement may not be essential beneath a small, settlement-tolerant MUP building with low-to-moderate foundation bearing pressures.

The most critical difference between the IGB and MUP sites is that upslope landslide deposits are a significant consideration for the design and siting of the MUP. The existing 15-foot high wall that presently bounds the east side of the Bevatron flat at the planned location of the MUP retains fill and old landslide deposits (Cross Section C-C'; Figure 4). Relocating this wall farther to the east would need to be accomplished without destabilizing the slope above; particularly that portion of the slope west of the existing Phase I seismic stabilization piers (Figure 1A). Notably, high pressure water lines and a 12KV electrical duct bank are among the critical utilities that underlie the access road directly west of the stabilization piers.

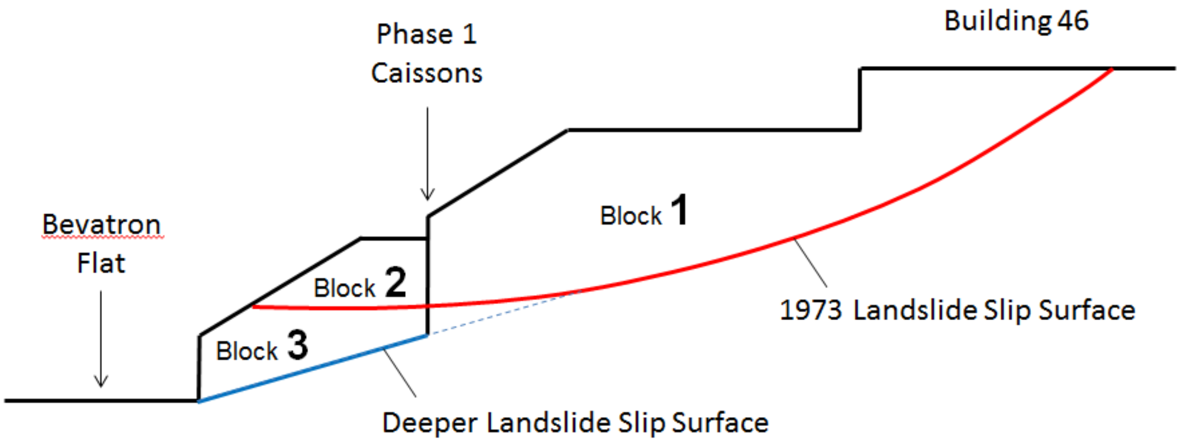
In addition, the results of our analyses show: 1) the landslide deposits upslope of the MUP are not seismically stable; 2) a large earthquake on the nearby Hayward fault is likely to result in significant landslide displacements; and 3) the forces needed to reduce seismic displacements to "structurally compatible" levels are quite large. These general statements apply not only to the landslide deposits downslope of the Phase 1 seismic stabilization piers but also to the portion of the 1973 landslide that remains below Building 46.

In our opinion, landslide-compatible approaches for the design of the MUP include: 1) siting the MUP at a location where existing retaining walls will not need to be removed and maintaining an acceptable "setback" from predicted seismic landslide displacements; or 2) installing/constructing new structural elements to restrain upslope landslide deposits and reduce seismic landslide displacements to structurally-compatible levels. It is conceivable that the MUP could also be sited on or within landslide deposits that are expected to move, provided that utilities and other attachments to the MUP that cross landslide margins are designed to tolerate the predicted seismic displacements. This conceptual approach, however, would require a much better understanding of local geologic conditions and landslide displacement mechanics and may not after all is considered be technically feasible.

5.05.2 Landslide Analysis Overview

We analyzed slope stability and seismic displacements using methods consistent with those presented in the official State SP117A guidelines (CGS, 2008). We used commercially available two-dimensional (2D) geotechnical analysis software (Slide® by rocscience) to analyze slope stability. We analyzed seismic displacements using simplified spreadsheet-based methods developed by Bray and Travasarou (2007). The cross sections used for our 2D slope stability analyses are based on Cross Section C-C' (Figure 4). In order to account for the existing Phase 1 stabilization system and the inferred presence of a deeper landslide slip surface, we segmented the existing landslide deposits into three blocks as shown in the following schematic illustration.

Conceptual 2D Slope Stability Analysis Model



Generalized descriptions of the three blocks follow:

- Block 1 – 1973 Landslide upslope of Phase 1 Caissons
- Block 2 – 1973 Landslide downslope of Phase 1 Caissons
- Block 3 – Deeper Landslide downslope of Phase 1 Caissons

Following an initial calibration step, we used our conceptual 2D slope stability analysis model to calculate a parameter known as the yield acceleration, which is the horizontal acceleration that when applied to the sliding block(s) produces a Factor of Safety (FS) of 1.0. The yield acceleration parameter captures multiple variables, including slope geometry, the weight (mass) of potential sliding blocks, the shear strength along sliding surfaces and the groundwater surface at the time that sliding occurs. The slope stability program also allows the user to input horizontal forces/pressures that resist slope movement. We utilized this capability to model the contribution of the existing Phase 1 stabilization system when calculating the yield acceleration of Block 1. The yield acceleration can be viewed as the horizontal acceleration at which a sliding block just begins to move, and is a critical parameter in simplified seismic slope displacement analysis methods.

We utilized simplified seismic slope displacement analysis methods (Bray and Travasarou, 2007) to develop probabilistic estimates of earthquake-induced slope displacements for a range of yield acceleration values. The Bray and Travasarou (2007) analysis method generally captures variables relating to the magnitude/duration of earthquake shaking and the dynamic response of the sliding mass. Our conceptual-level analyses are based on earthquake ground motions with a 10 percent probability of exceedance in 50 years (475-year return period). The analysis results include probabilistic plots of yield acceleration versus displacement for median (50 percent probability of exceedance) as well as 16 percent and 84 percent probability of exceedance values.

Once probabilistic yield acceleration versus displacement plots are obtained, it is possible to essentially run the process in reverse and determine the yield acceleration needed for a desired displacement value. For our conceptual-level analyses, we assumed that 2-inch median displacement could potentially be tolerated by existing and future structural stabilization elements (e.g. caissons and/or tiebacks). We then used the 2D slope stability analysis model to calculate the additional horizontal force/pressure needed to increase the yield acceleration to the value corresponding to the desired 2-inch median displacement.

The preceding analyses were run for a variety of cases to evaluate probable earthquake-induced slope displacements and the added forces/pressures needed to resist them. Output from our slope stability and seismic displacement analyses are attached in Appendix D.

5.05.3   Cases A and B - Calibration of Slope Stability Model

Analysis Cases A and B were conducted for calibration purposes and focused on evaluating the shearing resistance of existing landslide slip surfaces. Current guidelines require slip surfaces of existing landslides be modeled using fully remolded residual strengths (CGS, Blake, et al); essentially the lowest strength that the material can have after it has been thoroughly sheared by previous landsliding. The laboratory tests conducted for this study included three determinations of fully softened residual strength, producing the following friction angle values.

Fully Softened Residual Strength

Boring	Depth	Material Description	Friction Angle
B-1	21-21.6 feet	Reddish Brown Clay	11 to 12 degrees
B-2	12-12.5 feet	Dark Reddish Brown Clay, trace sand	14 to 16 degrees
B-4	18-18.5 feet	Dark Brown Clay	9 to 10 degrees

For one sample (Boring B-4 at 18-18.5 feet), fully softened peak strength was evaluated at the start of the final cycle of rotational shearing. Peak friction angles of 19 to 20 degrees were obtained from this test.

As a check on these values, we used our analytical model to back-calculate the minimum shear strengths needed to prevent slope failure under gravity loads. For each of these calculations, we assumed that the landslide materials would be fully drained and calculated the shear strength needed to achieve a Factor of Safety (FS) of 1.0. This groundwater assumption is “conservative” in that greater shear strengths would be needed to maintain stability for higher groundwater levels. In all cases, we assumed resistance would be purely frictional (i.e. no cohesion) so that the results could be directly compared to our laboratory test results. Friction angles were back-calculated for the following cases:

> ***Case A:*** Slope indicator plots (HLA, 1976a) indicate that the 1973 Landslide was marginally stable under non-earthquake conditions. In Case A, we back-calculated the minimum strength needed to prevent the movement of Blocks 1 and 2 before the Phase 1 stabilization system was installed. ***Minimum Friction Angle = 14.1 degrees***
>
> ***Case B:*** In Case B, we back-calculated the minimum strength required at the base of Block 3 to prevent the movement of Blocks 2 and 3 following the installation of the Phase 1 Stabilization system. ***Minimum Friction Angle = 14.4 degrees***

For our subsequent slope stability analyses, we modeled the 1973 Landslide slip surface below Block 1 and Block 2 using a friction angle of 14.1 degrees and the deeper slip surface below Block 3 using a friction angle of 14.4 degrees.

5.05.4   Cases C, D and E - Phase 1 Stabilization System

Analysis Cases C and D were conducted as a check on the Phase 1 stabilization system, which was designed and installed in the early 1990s. In performing our analyses, we assumed that the available structural capacity of the existing stabilization system can be approximated by the loading criteria used in its design (approximately 75 kips per lineal foot, klf, or total horizontal resistance). Yield accelerations and seismic displacements were calculated for the following cases:

> ***Case C:*** Case C evaluates seismic displacements for Block 1 and Block 2 acting together with the added resistance provided by the Phase 1 stabilization system. The displacement value calculated in Case C is viewed as a possible lower bound as it includes the buttressing effect of Block 2 even though it is recognized that Block 2 may move independently and pull away from the Phase 1 Caissons retaining Block 1.

Case D: Case D evaluates seismic displacements for Block 1 alone, assuming that Block 2 decouples and provides no buttressing effect. The displacement value calculated in Case D is viewed as possibly realistic and generally appropriate for evaluating the adequacy of the Phase 1 stabilization system, provided that it is understood that greater seismic displacements could result if existing caissons and/or tiebacks fail.

Our Case C analyses produced a yield acceleration of 0.042g corresponding to a median displacement of about 6.2 feet. Our Case D analyses produced a yield acceleration of 0.068 corresponding to a median displacement of about 4.6 feet. Since these values exceed the target displacement criterion, Case E was performed to determine the added horizontal force that would be needed at this location to produce a median displacement value of 2 inches.

Case E: Like Case D, Case E assumes that Block 2 pulls away from the existing Phase 1 Caissons and provides no buttressing effect.

We analyzed Case E for to different scenarios to assess the sensitivity of the landslide thickness input parameter (which affects landslide resonance and seismic loading). Our Case E analyses for a 40-foot-thick landslide show that an additional horizontal force of 425 klf is needed to restrain Block 1 under design-level seismic loading in order for a calculated median displacement value of 2 inches to be achieved. We ran the same analyses for a 30-foot-thick landslide and calculated an additional horizontal force of 505 klf.

For both thickness parameters, we evaluated the reduction in additional horizontal force that would be need if displacements higher than 2 inches could be tolerated by the existing lateral restraint system. A median displacement value of 6 inches resulted in additional horizontal forces of 310 klf and 365 klf for landslide thicknesses of 40 feet and 30 feet, respectively.

5.05.5 <u>Case F – Bevatron Flat Retaining Walls</u>

Analysis Case F was conducted to evaluate seismic displacements and loading criteria for retaining walls at the east side of the Bevatron flat. Our Case F analyses focus on forces exerted by Blocks 2 and 3 acting together; Block 2 toes out on the slope above and therefore exerts no direct force on the retaining walls.

Case F: Case G evaluates seismic forces on a wall constructed at the base of the slope retaining Blocks 2 and 3 (only). For this case, it is again assumed that Block 2 and Block 3 act together and that Block 1 exerts no load.

Our Case F analyses shows that an additional horizontal force of 102 klf is needed to restrain Blocks 2 and 3 under design-level seismic loading in order for a calculated median displacement value of 2 inches to be achieved.

5.05.6 <u>MUP Landslide Analysis Conclusions</u>

Our experience on previous LBNL projects suggests that a stabilization system can be designed to restrain the 102 klf load calculated in Case F (Section 5.05.5). For a 15-foot high wall, the 102 klf load corresponds to a lateral pressure of 6.8 ksf; a 1.2 factor of safety, if applied, would equate to a lateral pressure of a little over 8 ksf. Conceptually, an array of 200-kip tiebacks spaced on 5-foot vertical and horizontal centers would be capable of resisting the landslide-related horizontal thrust; this type of approach appears to us to be feasible as would other similar tieback capacity and spacing combinations. In our opinion, a total horizontal thrust of 100 klf due to seismically-induced landsliding can be assumed for conceptual-level design. We note that this 100 klf value was obtained using simplified analysis methods; future analyses using more complex methods (such as 3D and/or finite element/difference models) may be warranted as part of a future design-level study. Note that the 102 klf load calculated in Case F is based on the assumption that the portion of the 1973 Landslide upslope of the existing Phase 1 seismic stabilization piers is also stabilized, as discussed in Section 5.06.

Alternatively, the MUP could be sited at a different location. For conceptual design, a setback distance of at least 25 feet from existing walls that presently retain landslide deposits can be considered appropriate.

5.06 Future Upslope Stabilization

The analyses in Section 5.05.5 generally indicate that the existing Phase I stabilization system is under-designed relative to current standards. Analysis Cases C and D both predict median (50 percent probability of exceedance) seismic displacements on the order of 6 feet, even after accounting for the 75 klf resisting force provided by the existing caissons and tiebacks. Notably, the calculated 16 percent and 84 percent probability of exceedance values are on the order of twice and half the median displacement value (i.e. 3 and 12 feet, respectively). Considering these results, it appears to us that a design-level earthquake groundshaking could produce downslope landslide-related movements that would: 1) cause existing Phase I stabilization caissons and/or tiebacks to fail; and 2) result in significant damage in areas upslope.

Benefits of a future stabilization project addressing Block 1 include: 1) reducing earthquake-induced landslide hazards to McMillan Road, Building 46, subsurface utilities and other existing features that overlie or are intersected by the 1973 landslide deposit: and 2) increased flexibility in siting future buildings on the Bevatron flat. The results of Case E (Section 5.05.5) suggest that 300 to 500 klf of additional resisting force would be needed at this location to produce seismic displacements compatible with conventional structural restraints (estimated to be between 2 and 6 inches). If higher displacements can be tolerated, the amount of additional resisting force required would be less.

An upslope stabilization project would be a significant undertaking that would, in our opinion, best be looked at in a holistic way. First, the analyses presented in the conceptual-level report for the IGB are based on simplified 2D analytical methods; future analyses conducted using 3D modeling and/or finite element/difference methods could result in lower design forces. Second, our conceptual-level analyses model Block 1 as single mass in its present configuration; other scenarios involving multiple stabilization alignments and/or reduction in landslide mass (by excavation) may be more advantageous. Third, present perceptions of landslide-related risks relate to specific features that are currently exposed to the hazard (e.g. Building 46, roads, utilities, etc.); it may in some cases be advantageous to consider relocating or redesigning certain features in order to reduce or eliminate certain risks. And finally, at this time of this report plans for future development in other areas of the Bevatron flat and on the slope were not yet available; such future plans may reveal opportunities, constraints, risks and/or costs that are not yet understood. Evaluations involving the nature and scope of a possible future upslope stabilization project are beyond the scope of this conceptual-level study for the IGB Project.

6.00 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

6.01 General

The geotechnical recommendations in the sections that follow are preliminary and intended solely for conceptual design purposes. In some cases, additional subsurface investigations, laboratory testing and analyses may be needed to prior to final design. We recommend that we be consulted as future designs are developed so that we can provide geotechnical input and advise on the applicability of the preliminary geotechnical recommendations presented in this report. In addition, the preliminary geotechnical recommendations that follow are purposefully limited to what is needed to support the conceptual design. A subsequent “design-level” geotechnical investigation report should be prepared for the project once details involving the final design are better established. The design-level geotechnical investigation report should contain final geotechnical recommendations for the design of the project as well as discussions and data to be considered by the contractor during the bidding process. In addition, the design-level geotechnical report for the project should include recommendations for construction-phase observation and testing appropriate for the geotechnical aspects of the final design.

6.02 Building Code Seismic Design Parameters

Structures at the site should be designed to resist strong ground shaking in accordance with the applicable building code(s) and local design practice. The IGB and MUP sites are underlain at relatively shallow depths by Orinda Formation bedrock; based on geotechnical considerations, the soils the underlie future buildings will consist of compacted fill or ground that has been improved in an engineered manner.

The downhole suspension profiles in Appendix B generally show the in-place Orinda Formation rock that was logged has a shear wave velocity between about 2,000 and 2,500 feet per second (fps). Based on our review of the subsurface conditions (including the downhole shear wave velocity measurements) , we judge that a “C” Site Class is generally appropriate for the design of the IGB and MUP.

Location-specific seismic design parameters for use with the 2013 California Building Code (ASCE 7-10 Standard) follow.

Site Class
C = Very Dense Soil and Soft Rock

Site Location
Latitude = 37.87730 degrees
Longitude = -122.25079 degrees

Mapped Acceleration Parameters
Short Period, (S_S, Site Class B) = 2.474g
1-Second Period, (S_1, Site Class B) = 1.029g

Maximum Considered Earthquake Spectral Response Acceleration Parameters
Short Period, (SM_S, Site Class C) = 2.474g
1-Second Period (SM_1, Site Class C) = 1.337g

Design Spectral Acceleration Parameters
Short Period (SD_S, Site Class C) = 1.649g
1-Second Period (SD_1, Site Class C) = 0.891g

The USGS Design Maps Detailed Report for the site is attached as Appendix E.

6.02 Conceptual Foundation Design

6.02.1 Spread Footings and Mat Foundations

This section presents preliminary geotechnical recommendations for the conceptual design of spread footings and mat foundations. All foundations should be designed to bear at least 18 inches below lowest adjacent firm finished grade. Continuous and isolated spread footings should have minimum widths of 18 inches and 24 inches, respectively. The following bearing pressures can be used for the conceptual design of spread footings and mats that bear upon improved ground or bedrock:

Preliminary Bearing Pressures for Foundations on Improved Ground or Bedrock

Load Case	Bearing Pressure	Minimum Factor of Safety
Dead Load (DL) Allowable	2333 psf	3.0
Dead Plus Live Load (DL+LL) Allowable	3500 psf	2.0
Total (DL+LL+wind or seismic) Allowable	4667 psf	1.5
Ultimate	7000 psf	1.0

Resistance to lateral loads can be provided by friction along the base of foundations and by passive pressures developing on the sides of below-grade structural elements. Passive resistance in soil can be preliminarily evaluated using an equivalent fluid weight of 350 pounds per cubic foot (pcf), which can be increased by one-third for dynamic loading. Where pavements or floor slabs cover the adjacent ground surface, passive resistance can be assumed to begin at the ground surface. In areas not confined by slabs or pavements, passive resistance should be neglected within 1 foot of the ground surface. A friction coefficient of 0.35 can be used to evaluate frictional resistance along the bottoms of spread footings and mat foundations. The above passive and frictional resistance values include a factor of safety of at least 1.5 and can be mobilized with deformations of less than 1/2- and 1/4-inch, respectively.

6.02.2 Micropiles

The recommendations presented in this section were developed assuming that micropiles will be designed and installed by an experienced pre-qualified specialty subcontractor under a design-build approach. In this case, loads and allowable displacements at the head of the micropile should be provided by the project Structural Engineer and load tests should be performed to verify that the specified criteria are met. The Structural Engineer should also detail pile-structure connections and specify the required level of corrosion protection. We recommend that all micropiles be equipped with double corrosion protection; appropriate corrosion protection should also be provided at the micropile head-anchorage connection.

All micropiles should be load tested to verify that the specified capacity and deflection criteria are met. The magnitude of the required test loads should be considered when evaluating the structural capacity of the micropiles. The central reinforcing bar of the micropile is commonly sized so that the axial stress during load testing does not exceed 90 percent of the bar’s minimum yield strength; however, some publications recommend a maximum test load no greater than 80 percent of yield. We recommend applying a geotechnical factor of safety of 1.5 on the maximum test load when calculating allowable seismic design capacities (compressive and uplift).

This section presents example designs for micropiles intended for conceptual design purposes. The designs that follow are based on a geotechnical assessment, published information, and experience on recent projects. Our example micropile designs include minimum bond lengths based on an assumed average load transfer rate of 10 kips per lineal foot (klf), which should be achievable for micropiles post-grouted under pressure. For conceptual design purposes, we recommend assuming an average top of bond zone elevation of +685 feet (25 feet below the level of the Bevatron flat). Micropiles should be spaced no closer than 3 pile diameters (drill hole diameters) on center; a maximum drill hole diameter of 12 inches can be assumed for conceptual design.

Conceptual Micropile Designs – with Post-Grouting

Central Threadbar Specifications			Conceptual Micropile Designs		
Bar #	Grade	Minimum Yield Strength	Maximum Test Load (90% of Yield)	Seismic Tension/Compression (FS = 1.5)	Bond Length (assumed average 10 klf transfer rate)
#20	75	368 kips	331 kips	221 kips	30 feet*
#20	97	477 kips	429 kips	386 kips	40 feet
#24	97	665 kips	599 kips	399 kips	40 feet
#24	150	830 kips	747 kips	498 kips	50 feet

** all micropiles should have a minimum bond length of 30 feet*

We recommend that one or more experienced specialty micropile contractors be consulted to provide input as preliminary and final micropile designs are being developed.

6.03 IGB Sitework

6.03.1 <u>Ground Improvement</u>

Spread footings and mat foundations should bear upon bedrock or on approved engineered materials that bear directly on bedrock. For conceptual design purposes, we recommend assuming that the existing fill materials and colluvial soils beneath spread footings and mat foundations will be improved using rammed aggregate methods (e.g. Rammed Aggregate Piers® or Vibro-Piers®) or, alternatively, by cement soil mixing (SMX).

The recommendations presented in this section were developed assuming that ground improvement will be designed and installed by an experienced pre-qualified specialty subcontractor under a design-build approach. In this case, loads and allowable displacements at the bottom of footings/mats should be provided by the project Structural Engineer and load tests should be performed to verify that the specified criteria are met.

For conceptual design purposes, it can be assumed that ground improvement beneath footings and mats, on the average, will need to extend to Elevation +690 feet (20 feet below the level of the Bevatron flat). A more accurate estimate of the quantity of ground improvement necessary can probably be made using existing Bevatron plans and data from the Bevatron Demolition Project to evaluate variations in fill depths across the site. We recommend assuming that zones of ground improvement extend down and out from the outboard edges of footings and mats at inclinations no steeper than ½:1, horizontal to vertical.

6.03.2 <u>Underdrainage</u>

For conceptual design, we recommend assuming that the IGB will be underlain by a drainage layer containing a system of pipes (perforated and non-perforated) designed to drain by gravity to an appropriate discharge. Alternatively, the IGB should be appropriately waterproofed to protect against groundwater, which for conceptual design purposes can be assumed to occasionally rise to the level of the Bevatron flat (Elevation +710 feet). Hydrostatic forces on basement-like structures (e.g, sumps, elevator pits) should be evaluated based on a groundwater surface that is: 1) at the elevation of adjacent/nearby perforated pipes within the gravity underdrainage system; or 2) at Elevation +710 feet where no underdrainage system is present.

We recommend that the underdrainage system include a continuous layer of compacted Caltrans Class 2 Permeable Material and a system of 4-inch minimum-diameter SDR 35 or Schedule 40 PVC perforated pipes installed in trenches that are contiguous with the underdrainage layer. The continuous layer of permeable material below the slabs should be at least 8 inches thick. The trenches should be at least 12 inches wide and 12 inches deep. The trenches/pipes should be located within 5 feet inside the building perimeter, no more than 15 feet apart and drain (by gravity) to non-perforated collector pipes and an appropriate discharge facility. The perforated pipes should be placed, perforations down, on a 2-inch-thick layer of permeable material.

6.03.3 Non-Expansive Layer

For conceptual design, we recommend assuming that building slabs-on-grade, flatwork and pavement will be underlain by a layer of non-expansive fill at least 18 inches thick. Granular drainage materials and pavement aggregate base can be counted a part of the 18-inch non-expansive requirement. Non-expansive fill should: 1) be free of 6-inch plus material with no more than 15 percent of material larger than 2.5 inches; 2); be free of organic material, debris and environmental contaminants; 3) have a Plasticity Index of 15 or less; and 4) have a Liquid Limit of 40 or less.

6.04 Retaining Walls (IGB Site)

6.04.1 Lateral Pressures

Retaining walls that are free-to-rotate (i.e. walls unrestrained by adjacent structural elements, wall geometry or tiebacks) can be designed using active soil pressures that increase uniformly with depth (triangular distribution). The following active earth pressure values are considered appropriate where landslide deposits are not present.

Static Lateral Pressures for Free-to-Rotate Site Retaining Walls

Slope Behind Wall	Horizontal Lateral Pressure (psf per foot of depth)	Increase over Level Backslope
Level	45	1.00
3:1	50	1.11
2:1	60	1.33

Retaining walls that are restrained from rotation by adjacent structural elements or wall geometry can be preliminarily evaluated using “at rest” earth pressures that increase uniformly with depth (triangular distribution). The following values are considered appropriate where landslide deposits are not present.

Static Lateral Pressures for Fixed Retaining Walls

Slope Behind Wall	Horizontal Lateral Pressure (psf per foot of depth)	Increase over Level Backslope
Level	60	1.00
3:1	67	1.11
2:1	80	1.33

Retaining walls that are restrained by tiebacks can be preliminarily evaluated using “apparent” earth pressure diagrams based on active earth pressures redistributed into a trapezoidal shape. The following maximum (uniform) lateral pressures shown are considered appropriate where landslide deposits are not present.

Static Lateral Pressures for Tieback Walls

Slope Behind Wall	Uniform Horizontal Lateral Pressure (psf for wall height in feet)	Increase over Level Backslope
Level	25H	1.00
3:1	28H	1.12
2:1	33H	1.32

Retaining walls should be designed to resist increases in lateral pressure caused by vehicle loadings and/or other surcharges that may be applied at the ground surface. The following lateral pressure distributions can be used for the design of retaining walls for a level backfill condition under normal surcharge conditions where landslide deposits are not present.

Increases in Lateral Wall Pressures Caused by Surcharges

Load Condition	Lateral Pressure
Surcharge (vehicles)	100 psf (uniform) – applied over the upper 10 feet of the wall height
Surcharge (general)	0.5 times anticipated surcharge load (uniform) – applied over the full height of the wall

Unusually heavy and/or concentrated surcharge loads should be evaluated on an individual basis.

Where landslide deposits are not present, lateral load increases caused by earthquake shaking can be preliminarily evaluated using the earthquake surcharge pressures presented below.

Increases in Lateral Wall Pressures Caused by Earthquake Shaking

Slope Behind Wall	Uniform Horizontal Lateral Pressure (psf for wall height, H, in feet)
Level	18H
3:1	20H
2:1	24H

6.04.2 Retaining Wall Backdrainage

The lateral forces and pressures presented in the preceding section are only appropriate for retaining walls that are fully drained to prevent the buildup of hydrostatic pressure. Wall drainage may consist of either: (1) holes, slots or gaps in the wall that allow water to freely drain through the wall face; or (2) a wall backdrainage system that collects water from behind the wall and drains it, by gravity, to an appropriate discharge location. Backdrainage should consist of either prefabricated drainage material (Miradrain or an approved alternative) installed in accordance with the manufacturer's recommendations, or a vertical gravel blanket at least 12 inches thick. Additional drainage provisions may be required if seepage conditions are exposed during wall construction. We recommend that a waterproofing consultant be retained to provide any additional recommendations needed pertaining to the waterproofing of retaining walls or below-grade portions of the IGB and/or MUP.

The upper foot of retained soil behind the wall should be backfilled with low permeability soil to limit surface water infiltration into the wall backdrainage system. Concrete paving or a lined V-ditch/gutter should be installed behind the wall above the low-permeability soil that directs water away from the back of the wall and toward a suitable gravity discharge.

Prefabricated drainage material should be in direct contact with the retained soil/rock materials behind the wall and should be designed to drain through weepholes or into a perforated plastic pipe or other approved prefabricated drainage conduit. If prefabricated drainage material is used, the elements comprising the wall backdrainage system should be specified and detailed in accordance with the manufacturer's recommendations. Drainage material should have sufficient crushing strength to support the expected lateral earth pressures. We recommend full slope face coverage with prefabricated drainage panels unless soldier piles are used, in which case minimum 50% slope face coverage is acceptable. Additional drainage provisions may be required if seepage conditions are exposed during wall construction.

Drain rock used to construct vertical gravel blankets should conform to Caltrans specifications for Class 2 permeable material. Alternatively, locally available, clean, ½- to ¾-inch maximum size crushed rock or gravel could be used, provided it is encapsulated in a non-woven geotextile filter fabric, such as Mirafi 140N or an approved alternative. The gravel blanket should drain into a perforated plastic pipe installed (with perforations down) along the base of the walls on a 2-inch-thick bed of drain rock. Plastic pipe should be sloped to drain by gravity to a sump, relief wells or other suitable discharge and a cleanout should be provided at the pipe's upslope end. Perforated and non-perforated plastic pipe used in the drainage system should consist of 4-inch diameter Schedule 40 PVC or an approved equivalent.

6.05 Retaining Walls (MUP Site)

Walls that retain landslide deposits should be designed to resist forces associated with seismically-induced landsliding. For conceptual design purposes, in can be assumed that walls at the east side of the Bevatron flat in the vicinity of the planned MUP site will be subjected to an unfactored total horizontal seismic thrust 100 kips per lineal foot (klf) from landslide deposits.

For conceptual design, it should be assumed that tiebacks will be used to resist seismically-induced landslide loads. New retaining structures adjacent to the base of slope should be installed using top-down methods to avoid destabilizing the slope during construction. Tiebacks should be designed to have bond zones within rock behind the basal surface of the "Possible Deeper Landslide" shown on Cross Section C-C' (Figure 4).

Lateral loads also can be resisted by passive pressure acting on the downslope face of embedded drilled piers, grade beams and/or footings. Passive resistance can be evaluated using an equivalent fluid pressure of 350 pcf, which includes a factor of safety of at least 1.5 and can be applied over two (horizontal) pier/pile diameters.

6.04 Conceptual Tieback Wall Design

Permanent tiebacks should be appropriately protected to resist corrosion; appropriate permanent corrosion protection should also be provided in the area of tieback stressing tail and anchorage. Tiebacks should be inclined downward at an angle of at least 10 degrees below the horizontal and should be designed to be anchored entirely within rock below the depth of interpreted or suspected slope movement. The cross sections presented on Figures 3 and 4 can be used to evaluate depths/elevations of rock for the conceptual design purposes.

The downward component of tieback loads can be resisted by: 1) skin friction acting on the embedded portions of piers; and/or 2) bearing on the bottom of wall footings. For conceptual design, the axial capacity of drilled piers can be estimated using an allowable skin friction value of 750 psf for sustained long-term loads. Any contribution to pier axial capacity from end bearing should be ignored. Footings can be evaluating using the bearing pressures provided in Section 6.02.1.

All tiebacks should have a bond zone length of at least 20 feet and portions of tiebacks that are not within the anchorage zone should be designed to be unbonded. Tiebacks that retain landslide deposits will require significantly longer bond zones; for conceptual design, we recommend assuming an ultimate skin friction value of 4,000 psf for tiebacks that are post-grouted in rock. Tieback hole diameters between 8 inches and 12 inches can be assumed in assessing bond zone lengths for conceptual design purposes.

Tieback steel area and strength should be sized so that neither the design load nor test load exceed allowable limits specified by the tieback manufacturer. Additional guidance on this subject can also be found in the Post Tensioning Institute's publication *"Recommendations for Prestressed Rock and Soil Anchors."* The Structural Engineer should consider the effects of tieback load testing and verify that wall/structure will not be damaged under the maximum test load.

7.00 LIMITATIONS

This report has been prepared for the exclusive use of LBNL and their consultants for specific application to the conceptual design of the Integrative Genomics Building (IGB) Project. The opinions presented herein were developed in accordance with generally-accepted geotechnical and engineering geologic principles and practices. No other warranty, expressed or implied, is made. Note that the findings presented in this report are based, in part, upon data collected by previous investigators. We cannot vouch for the accuracy of the data obtained from others or (consequently) for interpretations that we have made based on existing available data.

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

The findings of this report are valid as of the present date. However, the passing of time will likely change the conditions of the existing property due to natural processes or the works of man. In addition, due to legislation or the broadening of knowledge, changes in applicable or appropriate standards will occur. Accordingly, this report should not be relied upon after a period of three years without being reviewed by this office.

Finally, as previously noted, this report was prepared in support of the conceptual design phase and should not be used for final design.

8.00 REFERENCES

8.01 General References

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Lawson, Andrew C. and Palache, Charles, 1901, "The Berkeley Hills, a Detail of Coast Range Geology," The University of California Bulletin of the Department of Geology, Vol. 2, No. 12, pp. 349-450, Plates 1-17 with oversize map

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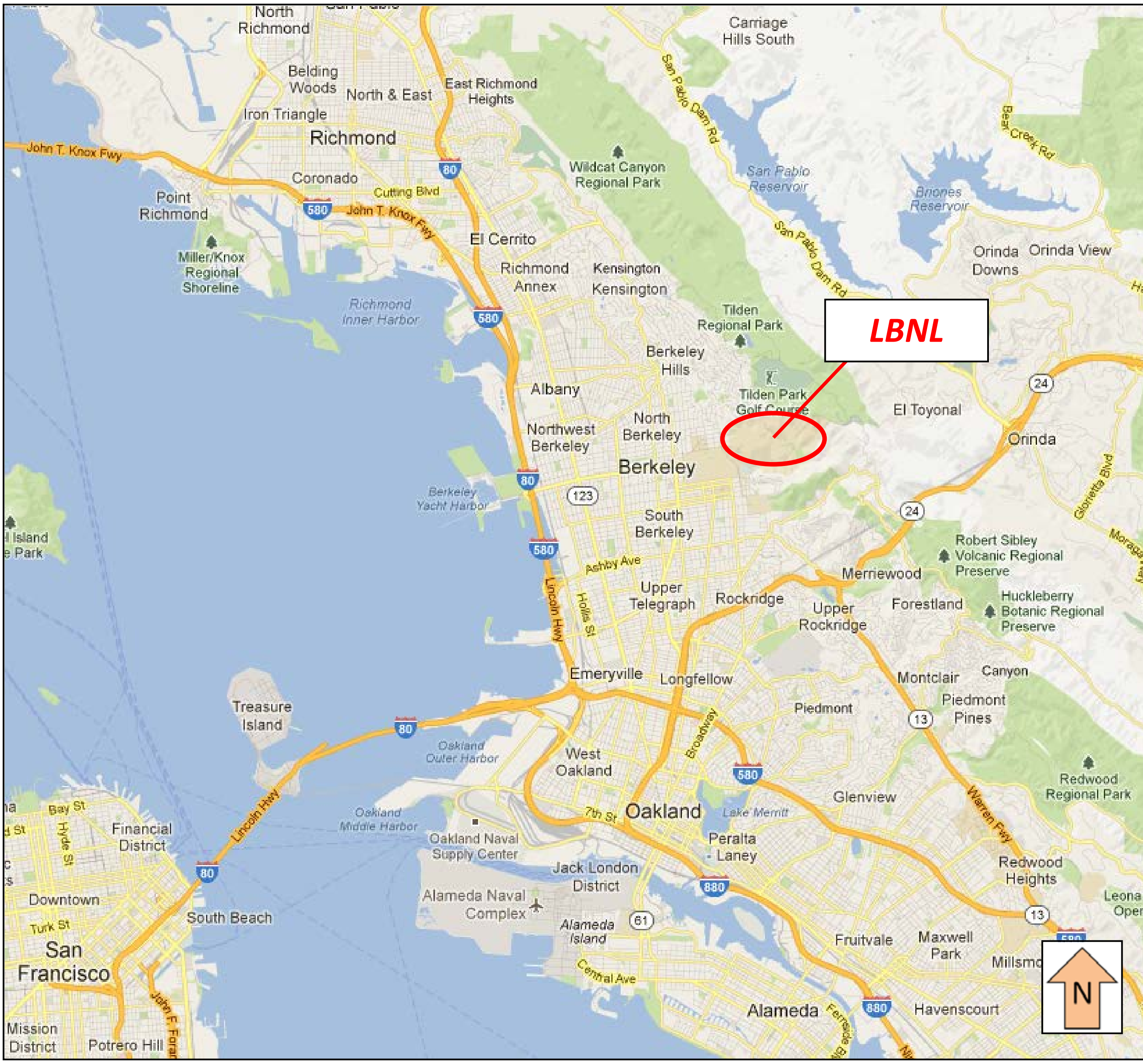
Plates

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Source: Google Maps



LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 1
Vicinity Map

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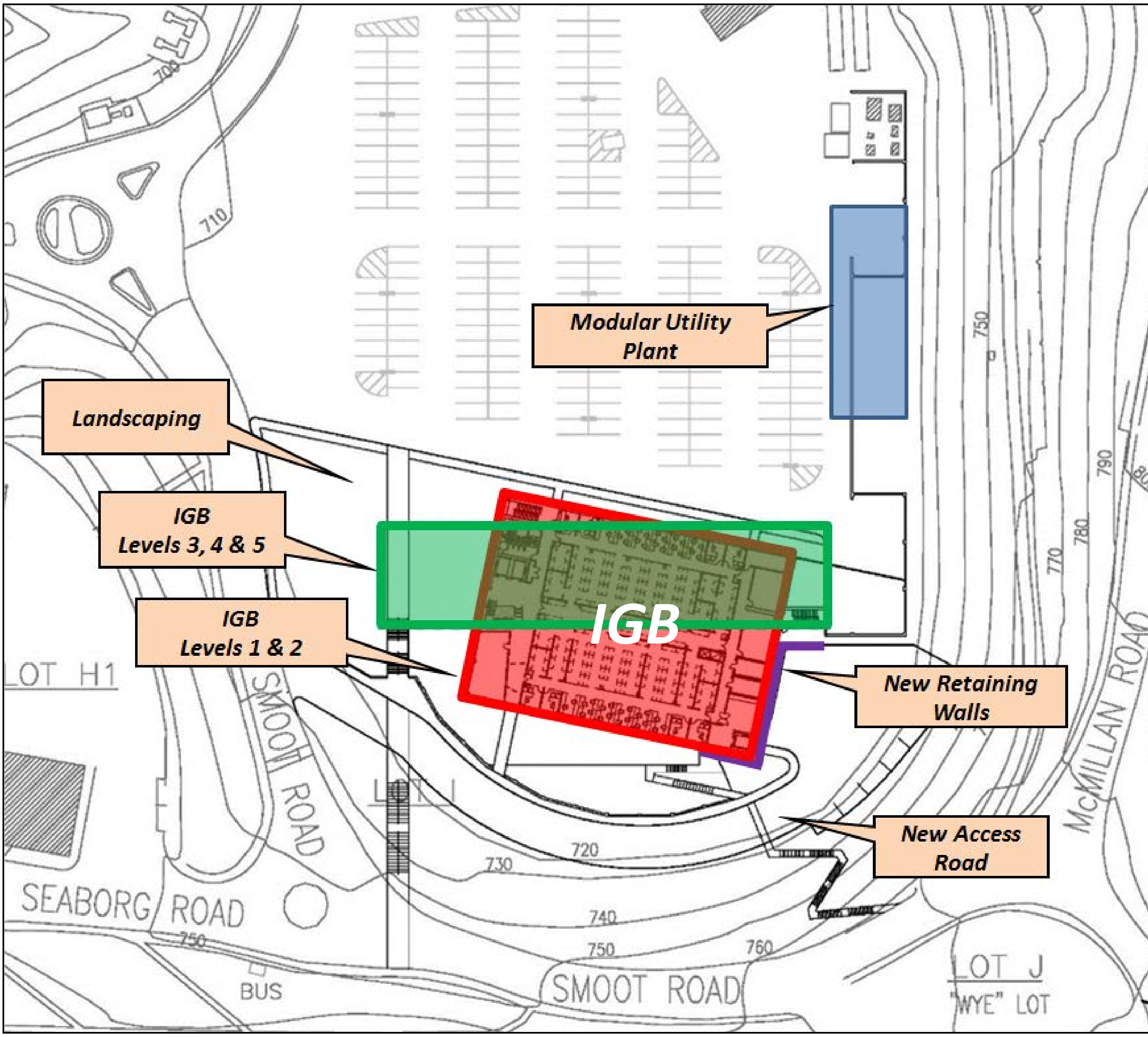
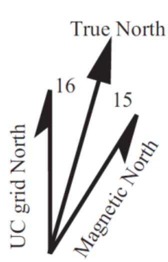


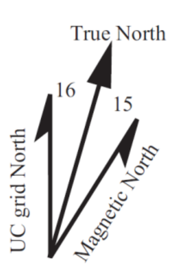


APPROXIMATE MEAN DECLINATION, 2000

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 2
Conceptual IGB Layout

Source: *Google Earth (imagery date 8/28/2012)*

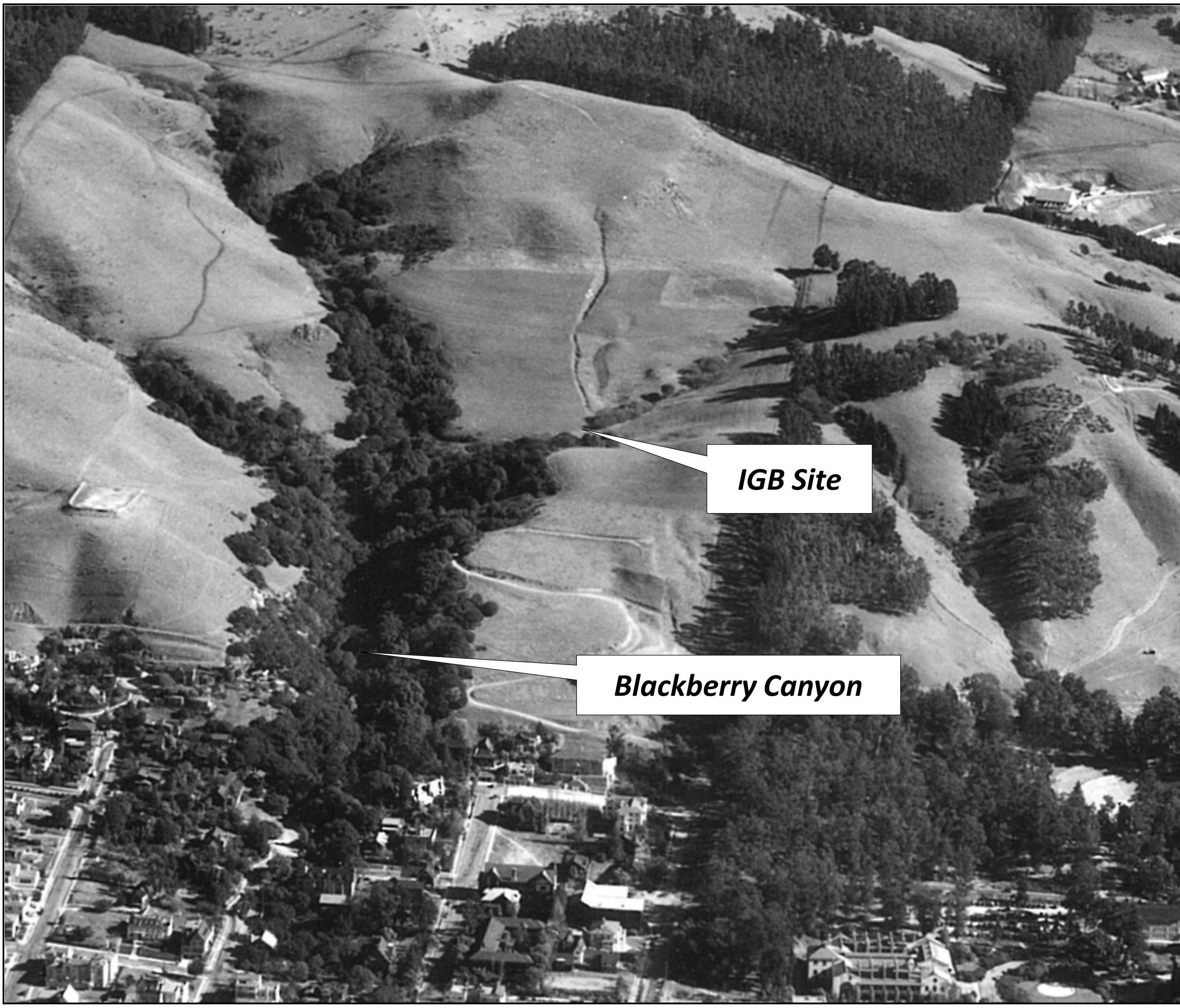


APPROXIMATE MEAN
DECLINATION, 2000

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

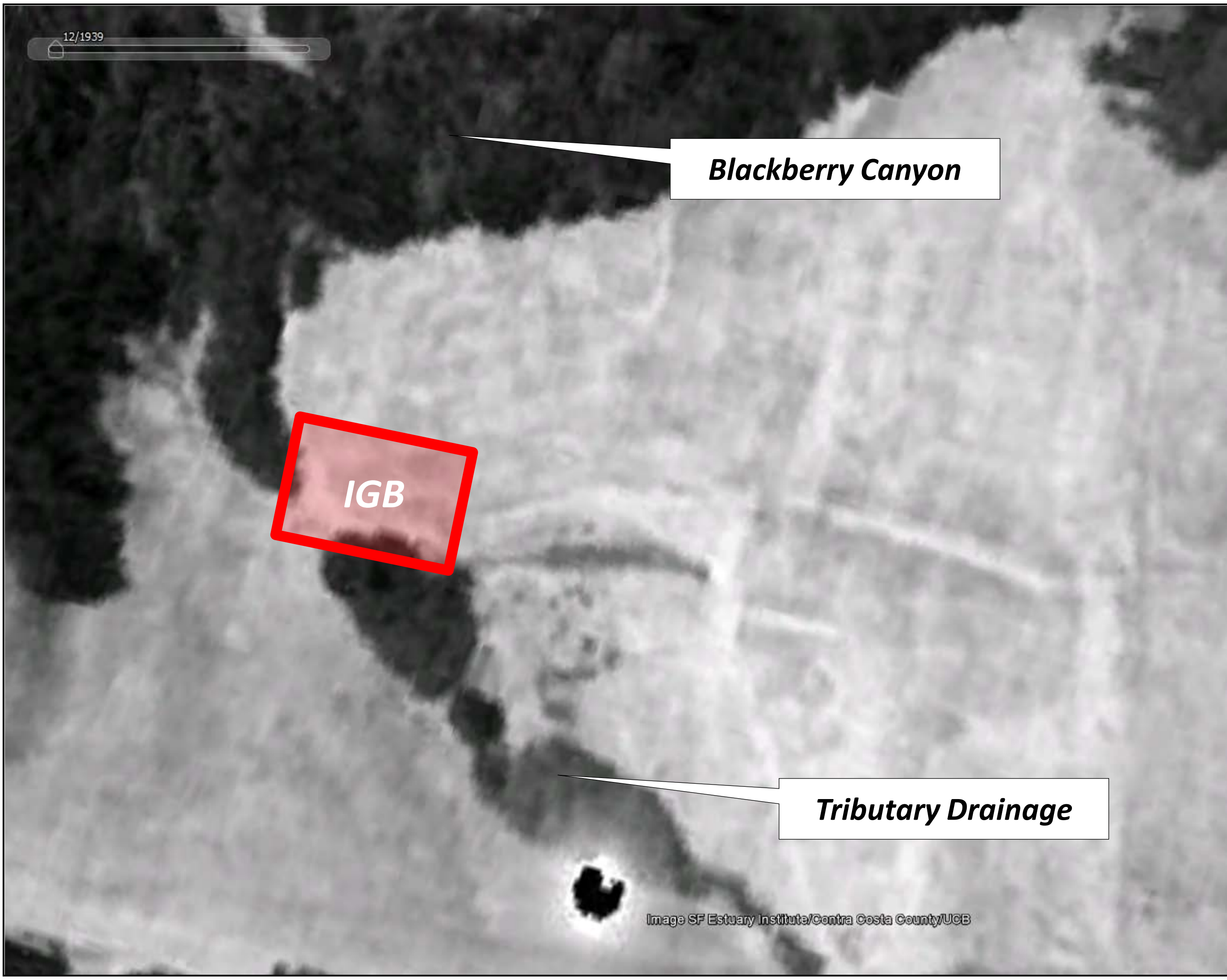
Plate 3
2012 Aerial Photograph

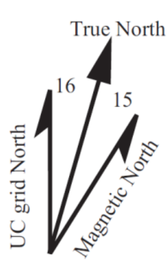
Source: *Pacific Aerial Surveys (1935_USNPS-5-J)*



Photograph looking East

Source: *Google Earth (imagery date 12/31/1938)*





APPROXIMATE MEAN DECLINATION, 2000

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 5
1938 Aerial Photograph

Source: LBNL Photo Archives – Bevatron 44 (imagery date 06/17/1949)

Photograph looking Southeast

XBD201407-00899

Plate 6
June 1949 Photograph

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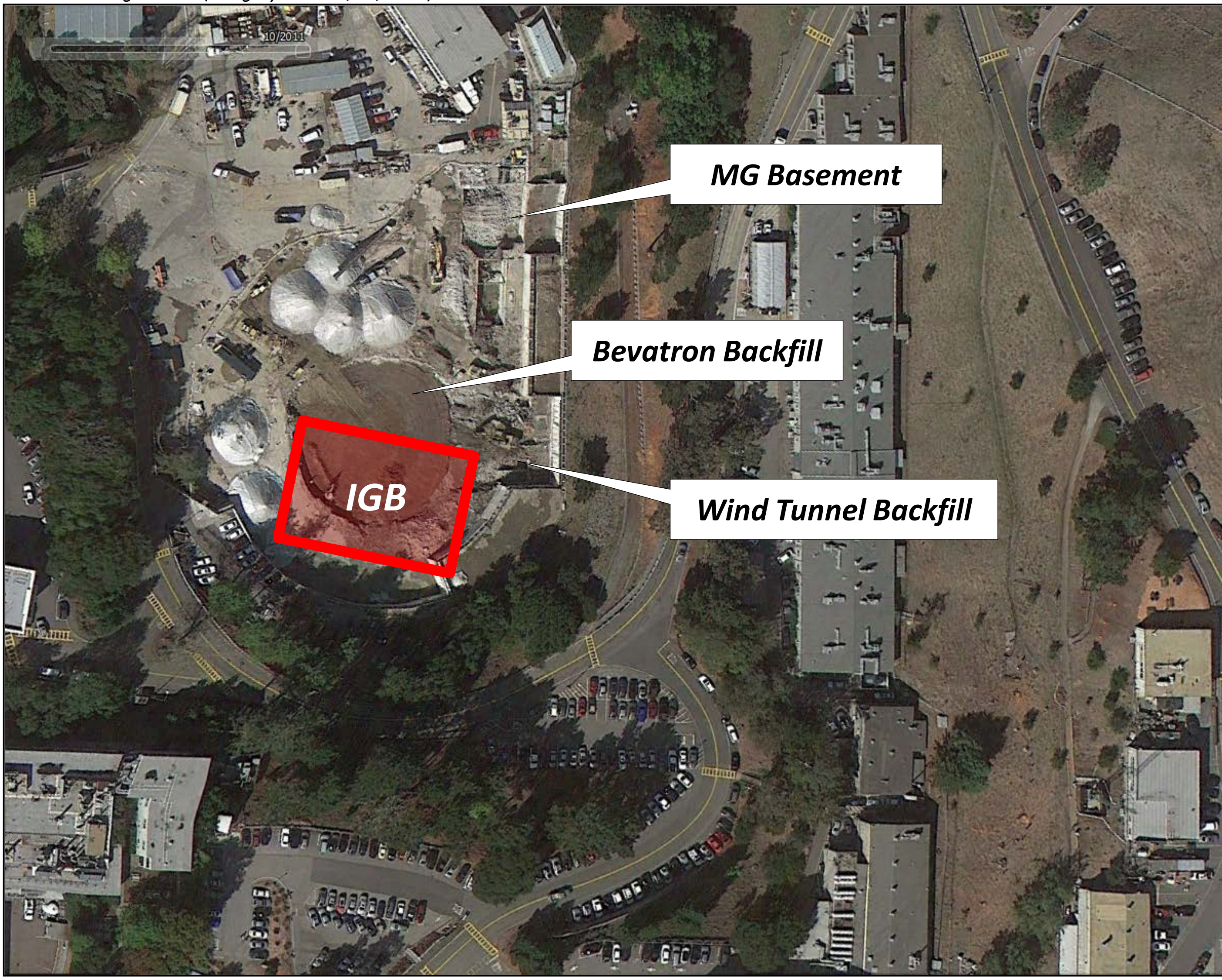
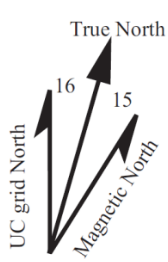
Source: Google Earth (imagery date 10/11/2011)





APPROXIMATE MEAN
DECLINATION, 2000

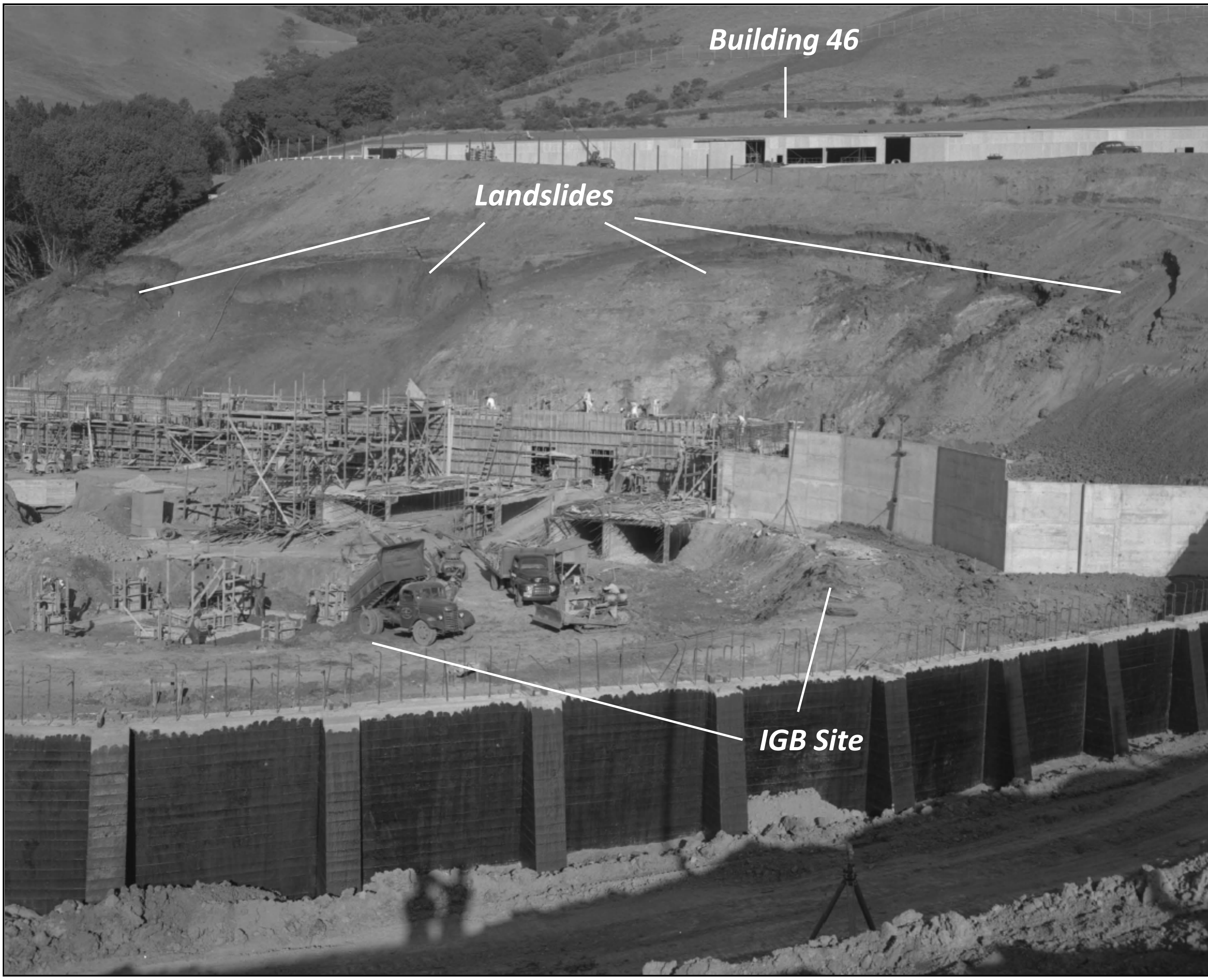
LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 7
2011 Aerial Photograph

Source: LBNL Photo Archives – Bevatron 101 (imagery date 08/29/1949)

Photograph looking Southeast

Source: *LBNL Photo Archives – Bevatron 170 (imagery date 11/17/1949)*

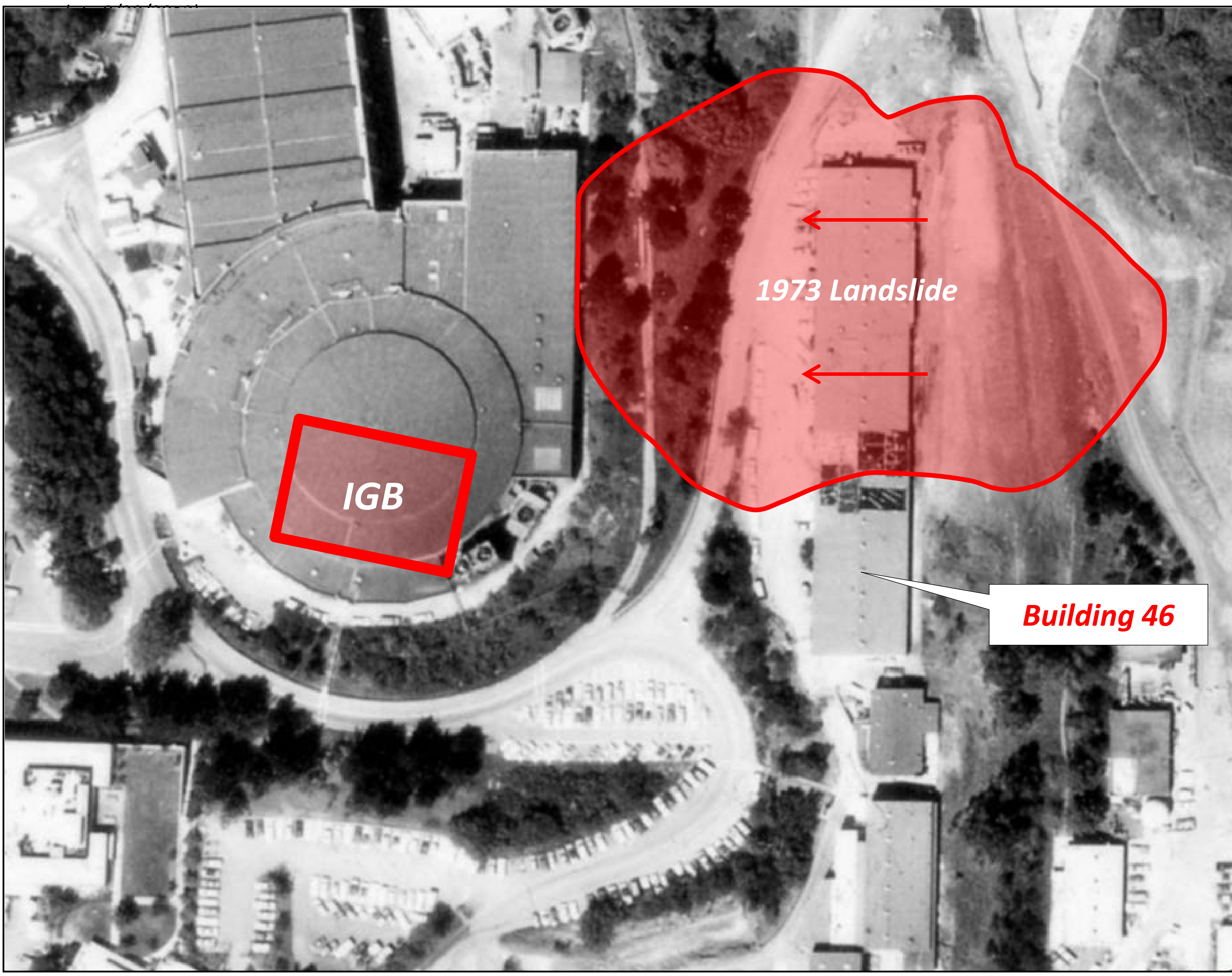


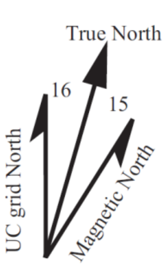
Photograph looking Northeast

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 9 November 1949
Photograph

Source: Pacific Aerial Surveys (Imagery date 4/18/1973)





APPROXIMATE MEAN
DECLINATION, 2000

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 10
1973 Aerial Photograph

A3GEO

Source: Google Earth

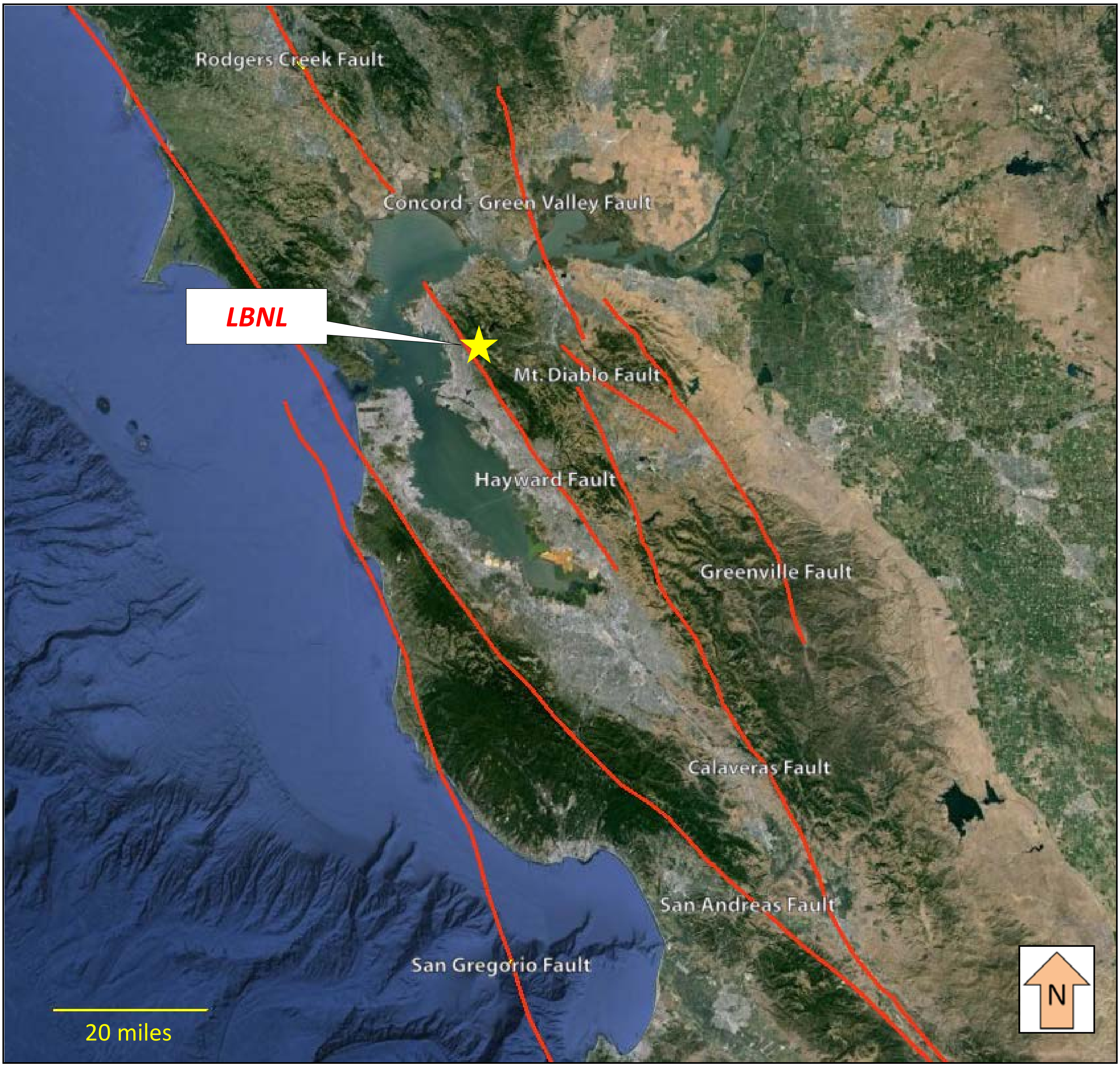


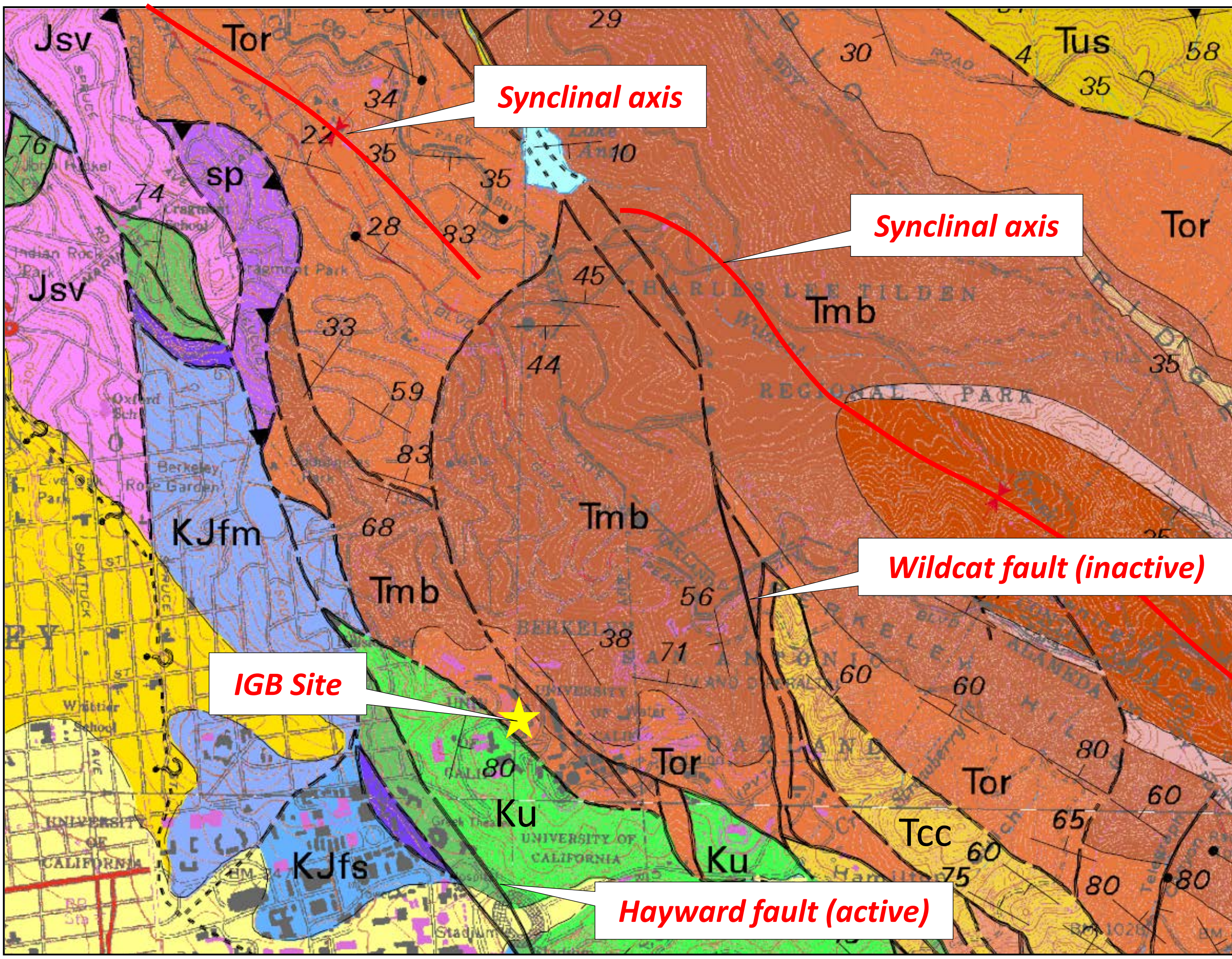
Plate 11
Bay Area Active Faults

Source: Google Earth; CDMG 1982

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 12
Local Active Fault Traces

Source: Graymer, 2000 (USGS Map MF -2342)



BEDROCK UNITS

Jsv	Keratophyre and quartz keratophyre (Late Jurassic)
sp	Serpentinite
	Franciscan Complex
KJfs	Franciscan complex sandstone, undivided (Late Cretaceous to Late Jurassic)
KJfm	Franciscan complex, m élange (Cretaceous Late Jurassic)

BEDROCK UNITS

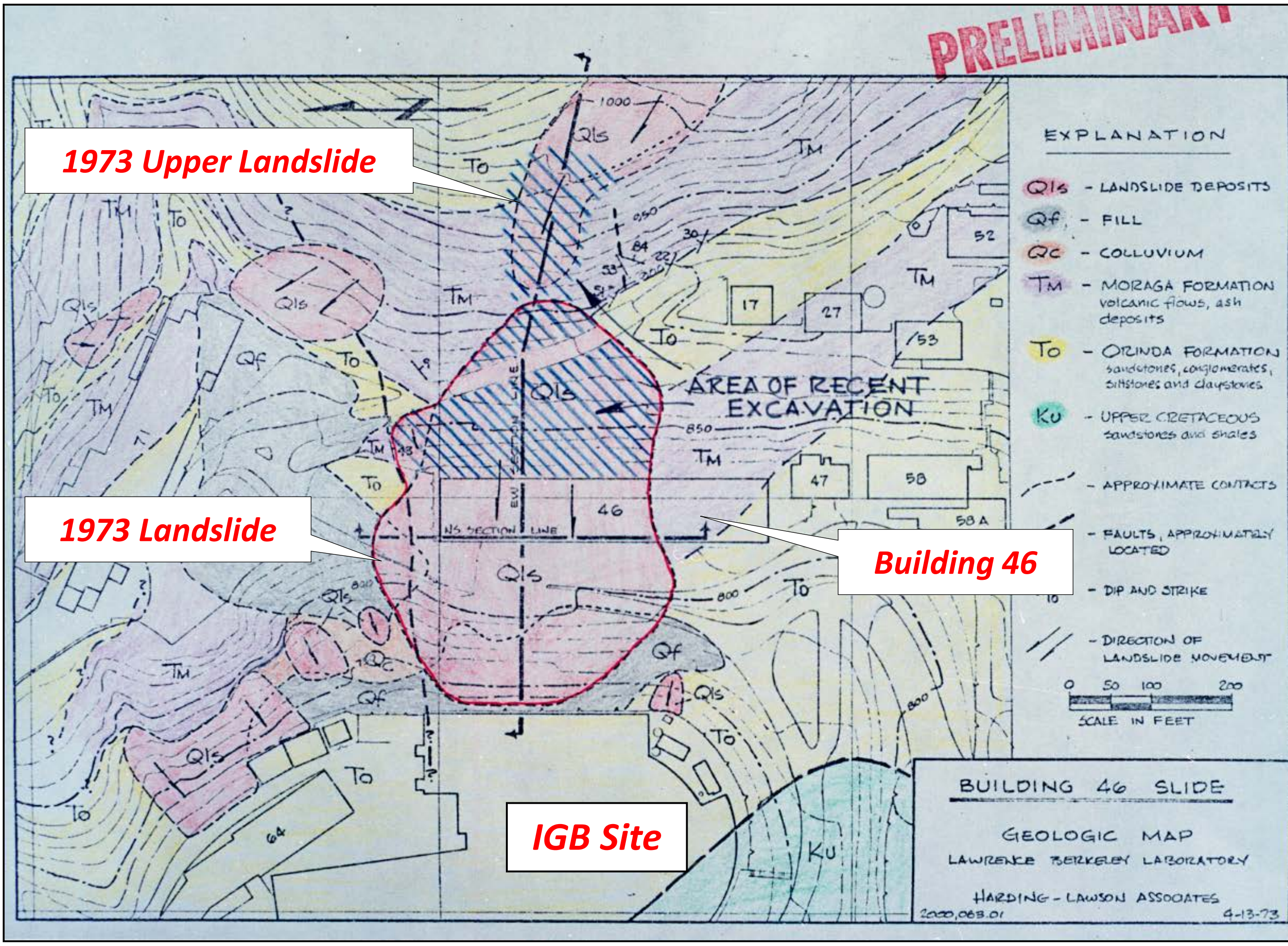
Tmb	Moraga Formation (late Miocene)
Tor	Orinda Formation (late Miocene)
Tcc	Claremont chert (late to middle Miocene)
	Great Valley Complex
Ku	Undivided Great Valley complex rocks (Cretaceous)

35 Strike and dip of bedding

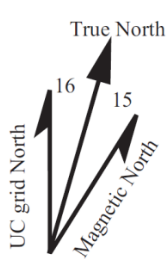
LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 13
Regional Geologic Map

Source: LBNL Photo Archives – (map date 04/13/73)



UC Grid North ←

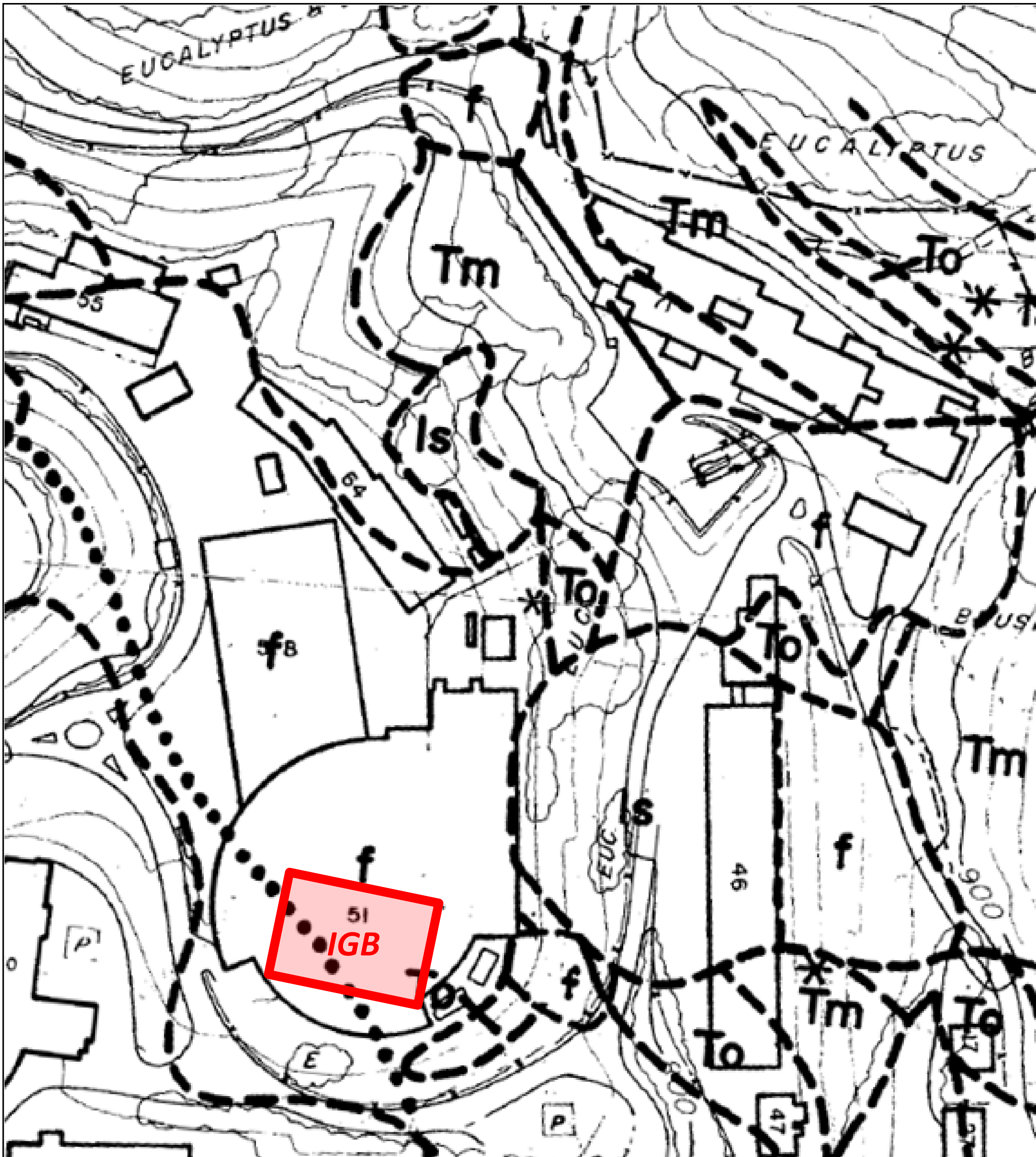


APPROXIMATE MEAN DECLINATION, 2000

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 14
1973 Geologic Map

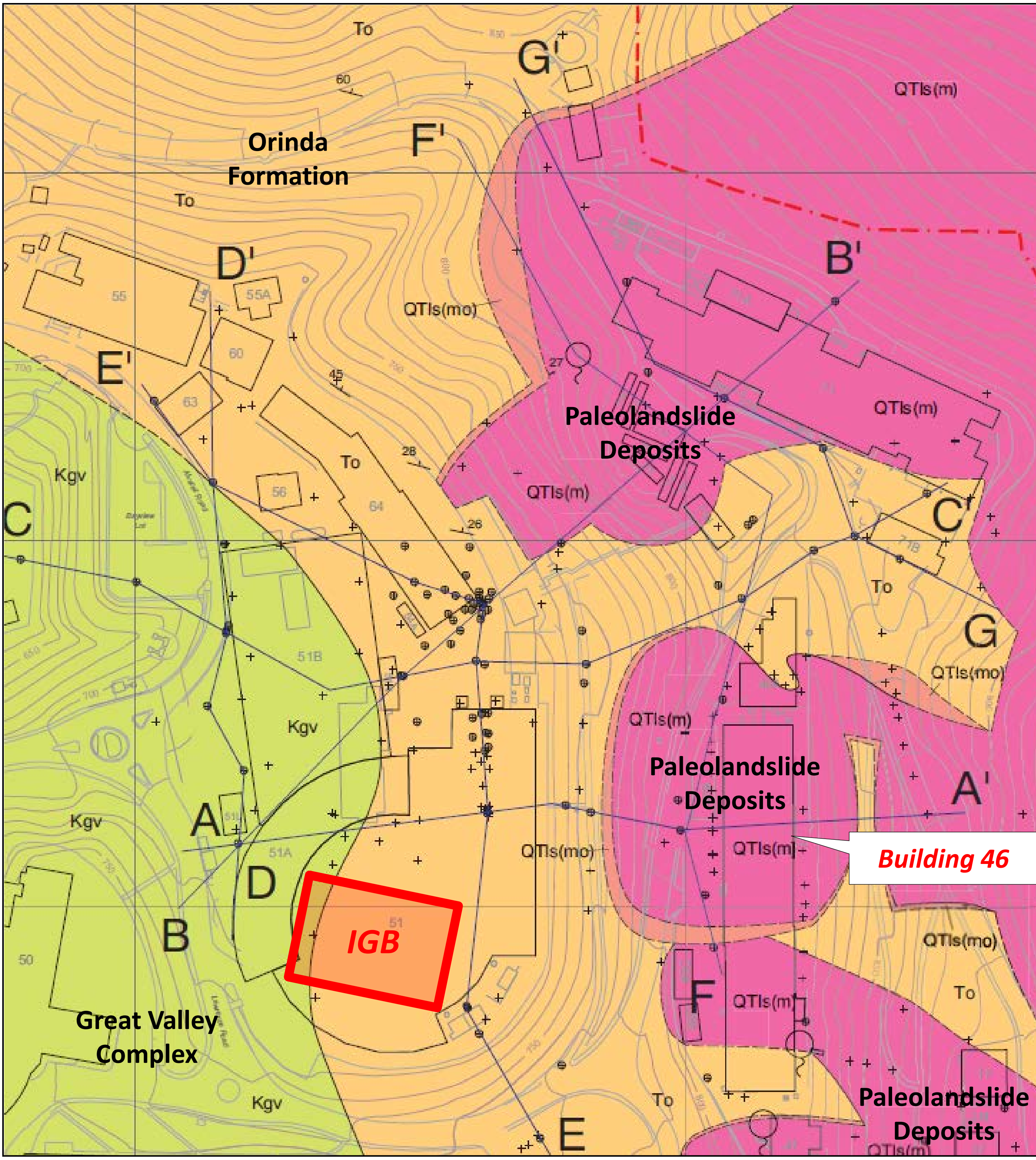
Source: HLA, 1982 (LBNL File #041)



LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 15
1982 Geologic Map

Source: LBNL/Parsons, 2000 (RFI Report)



LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 16
RFI Report Geologic Map

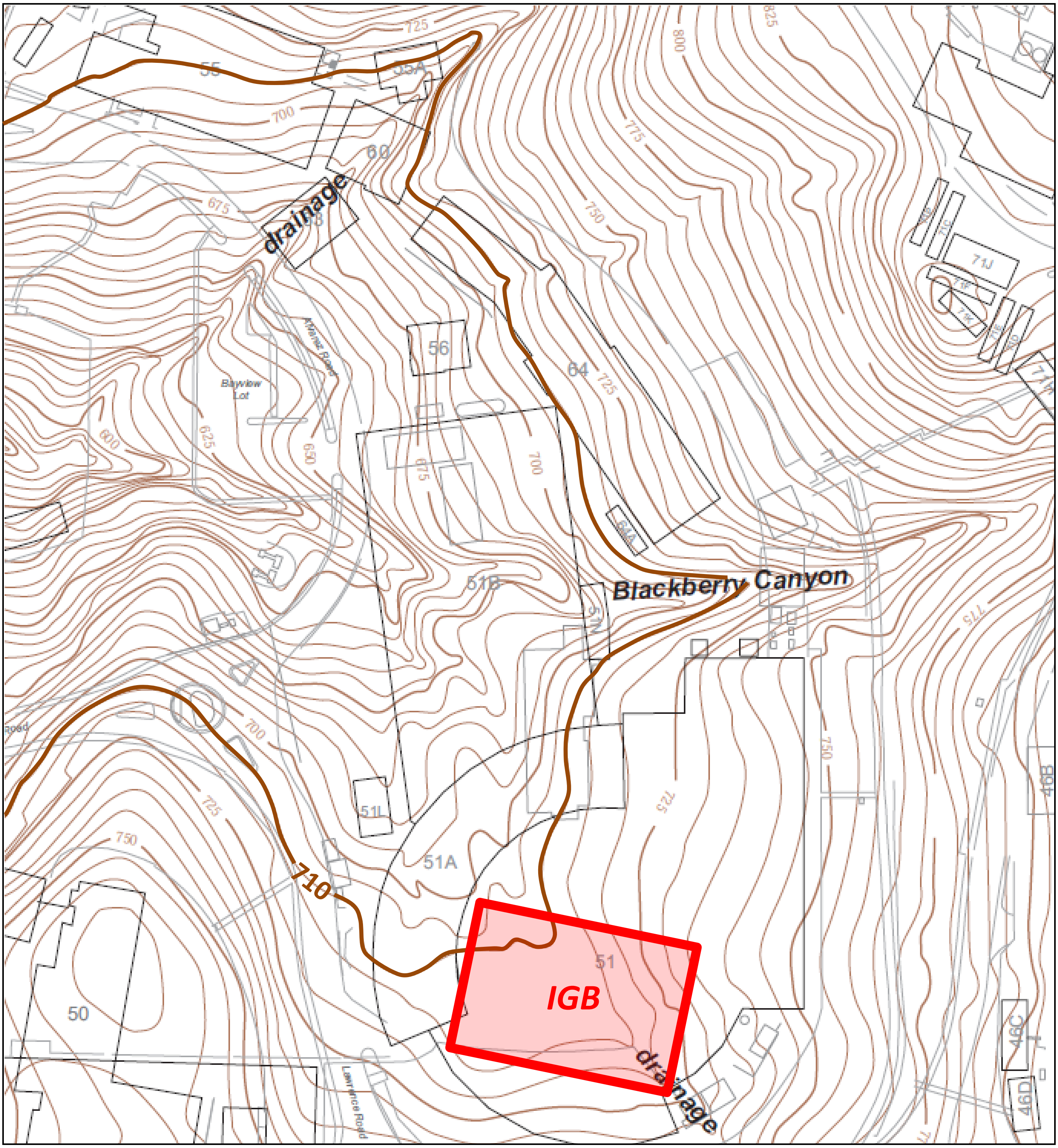
Source: LBNL Photo Archives – BLDG 331 (imagery date 11/1947)

Photograph looking Southeast

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 17
Pre-Development Setting

Source: LBNL/Parsons, 2000 (RFI Report; Module A)

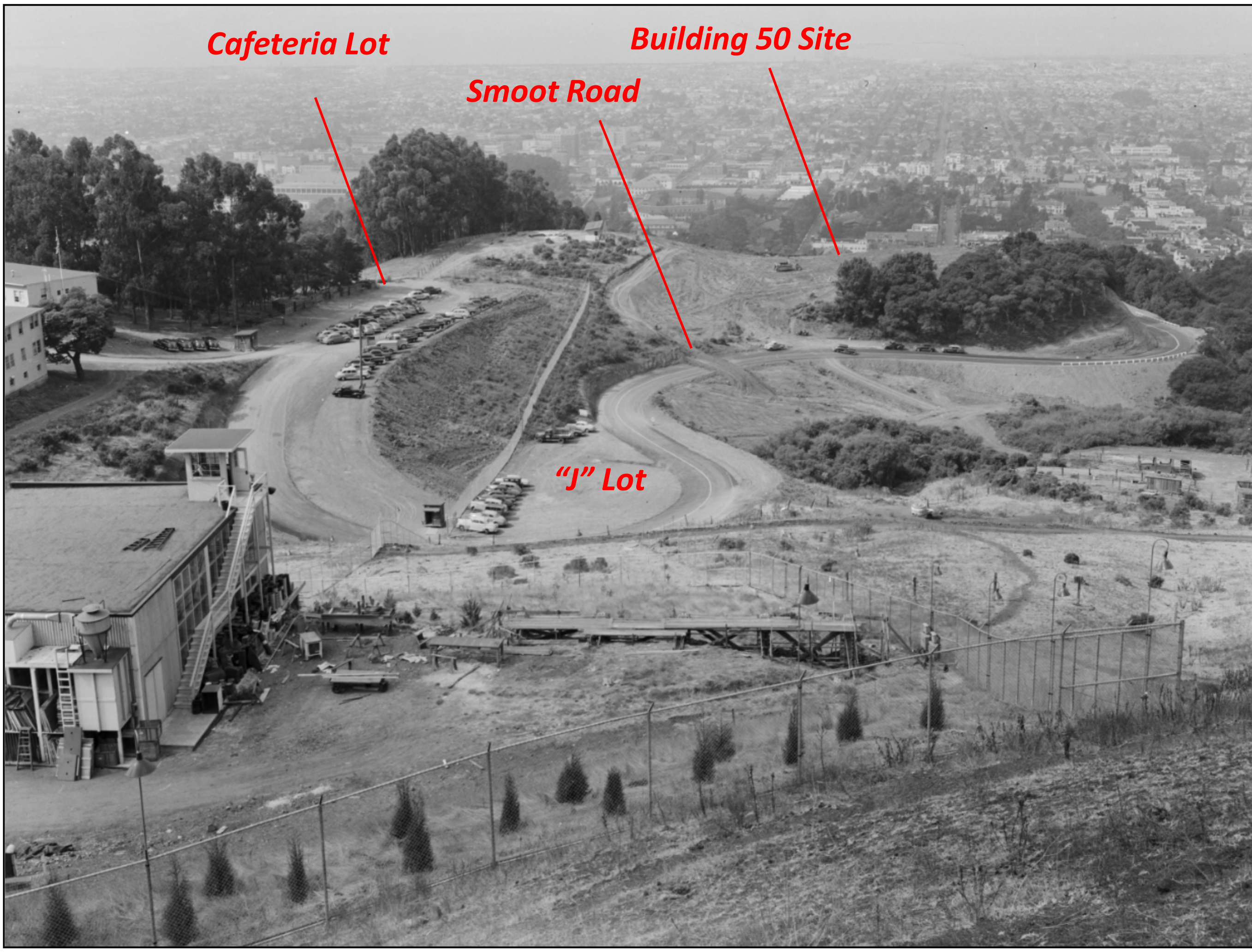


RFI Report citation: *University of California, Berkeley, 1948. Access road to Wilson Tract - location map – Berkeley campus, sheet 1 of 8, Drawing Number 4221, scale 1:1200.*

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 18
1948 Topography

Source: *LBNL Photo Archives – XBD201407-00892 (September 22, 1948)*



Photograph looking Northwest

Source: LBNL Photo Archives – XBD210407-00893 (September 28, 1948)

Photograph looking Northwest

Source: LBNL Photo Archives – XBD201207-00441 *(November 11, 1948)*

Photograph looking Northwest

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 21
November 1948 Photograph

Source: *LBNL Photo Archives* – XBD201212-01751 *(February 7, 1949)*

Photograph looking East

Plate 22
February 1949 Photograph

Source: LBNL Photo Archives – XBD201212-01756 *(April 19, 1949)*

Photograph looking North

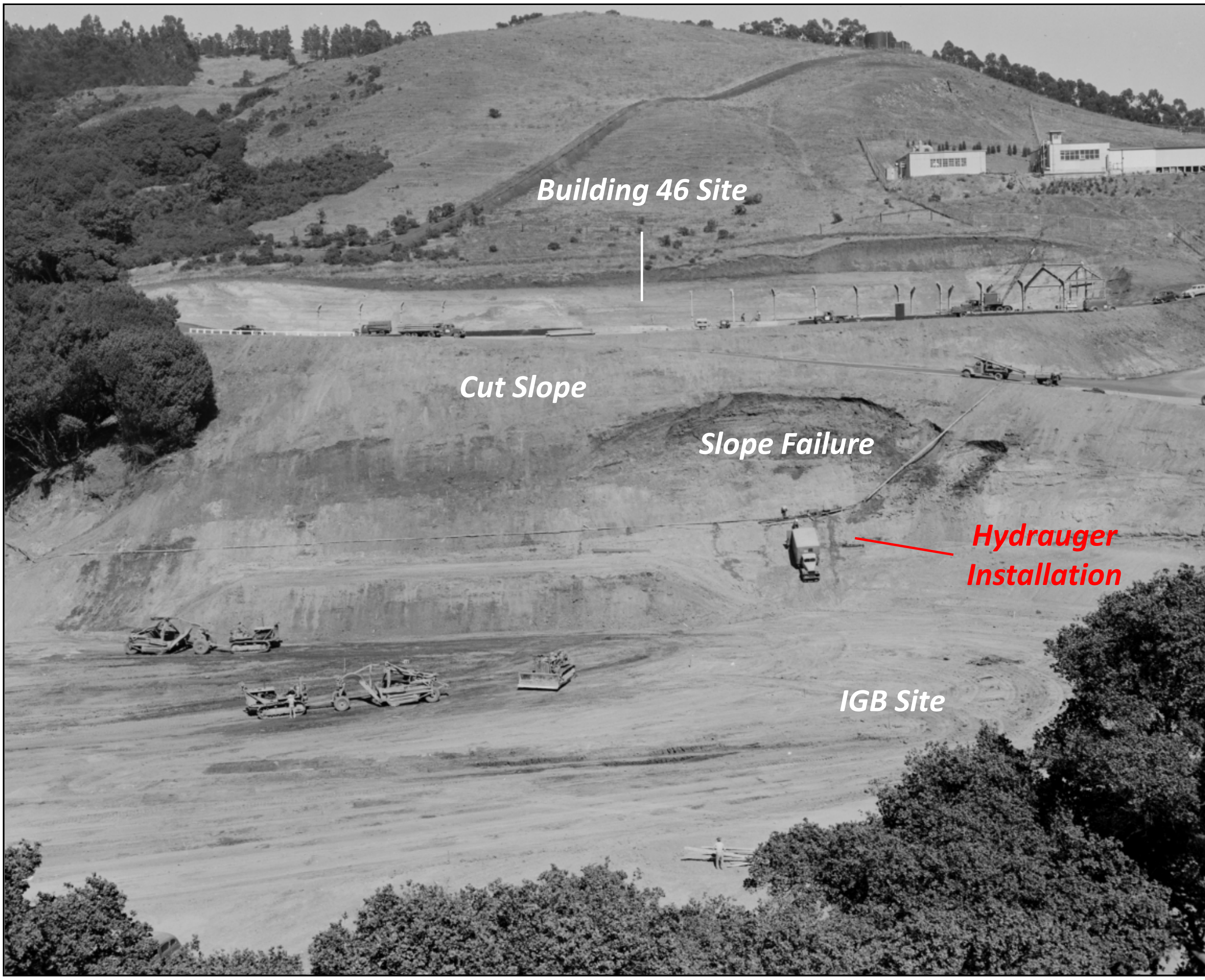
LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 23
April 1949 Photograph

Source: *LBNL Photo Archives* – XBD201212-01763 *(June 27, 1949)*

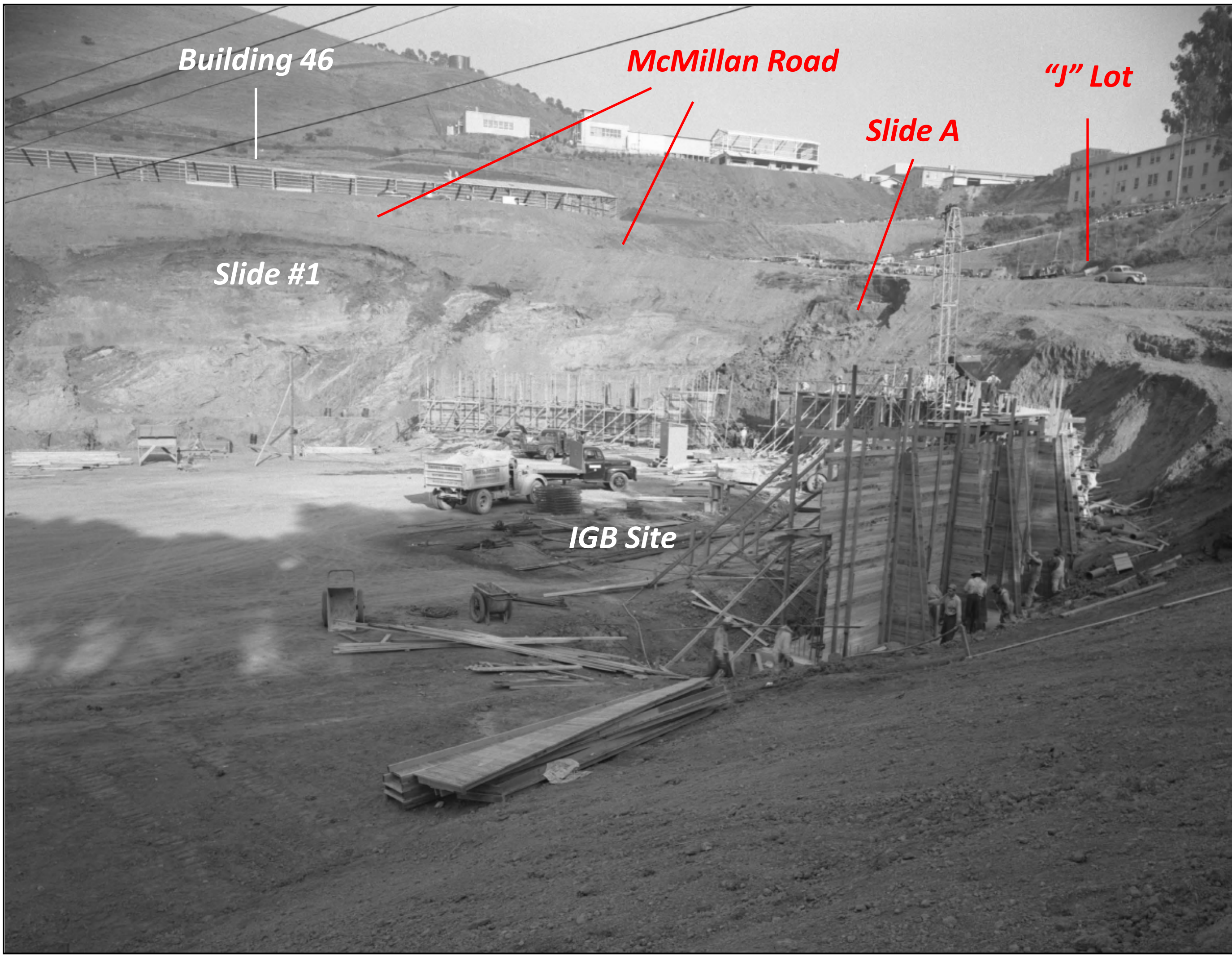
Photograph looking Northeast

Source: LBNL Photo Archives – XBD201407-00863 *(August 25, 1949)*



Photograph looking East

Source: LBNL Photo Archives – XBD201212-01786 *(September 29, 1949)*



Photograph looking Southeast

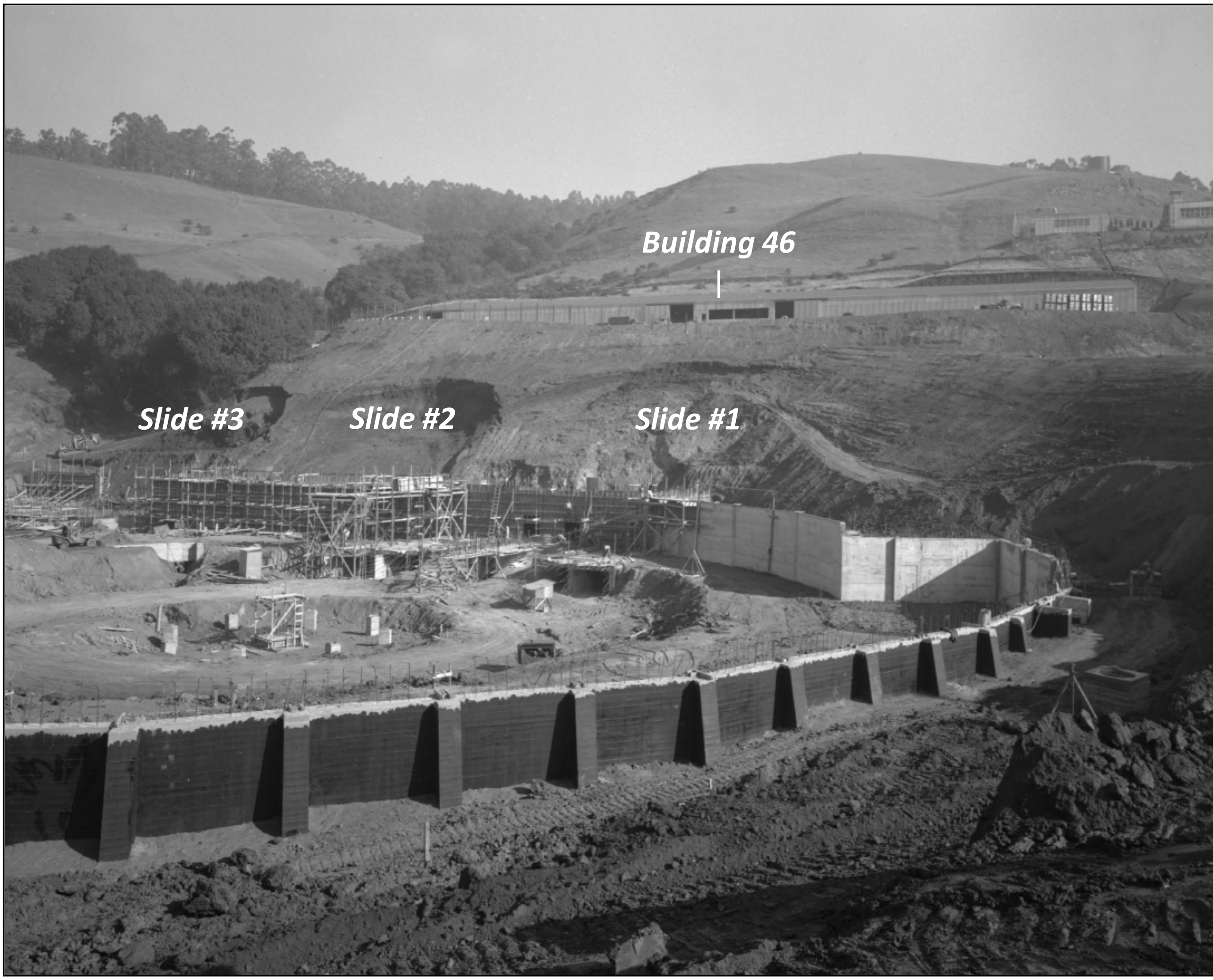
Source: LBNL Photo Archives – XBD201212-01793 *(October 5, 1949)*

Photograph looking Northwest

Source: *LBNL Photo Archives* – XBD201212-01798 *(October 24, 1949)*

Photograph looking West

Source: LBNL Photo Archives – XBD201212-01817 *(November 28, 1949)*

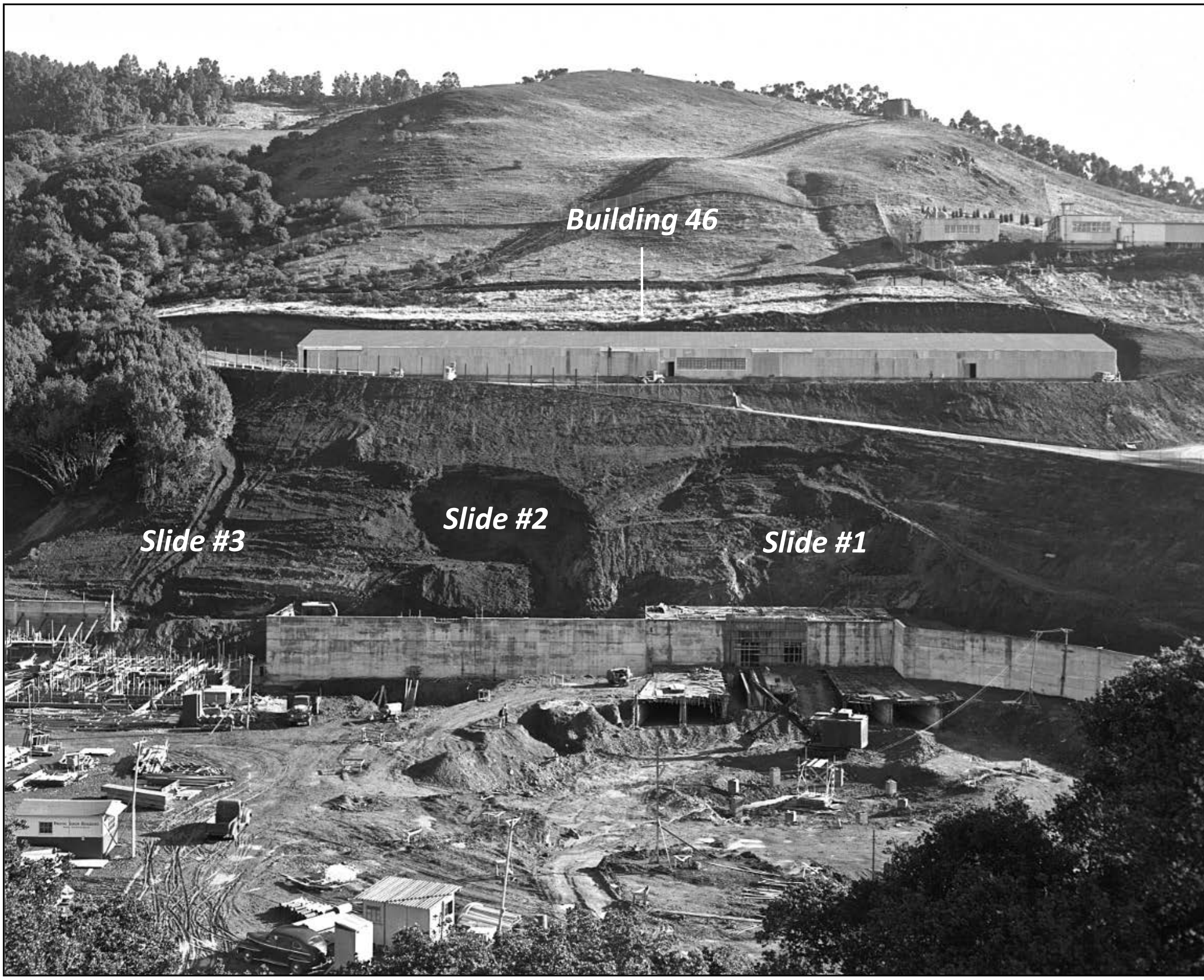


Photograph looking Northeast

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 29
November 1949 Photograph

Source: LBNL Photo Archives – XBD201212-01821. *(December 6, 1949)*



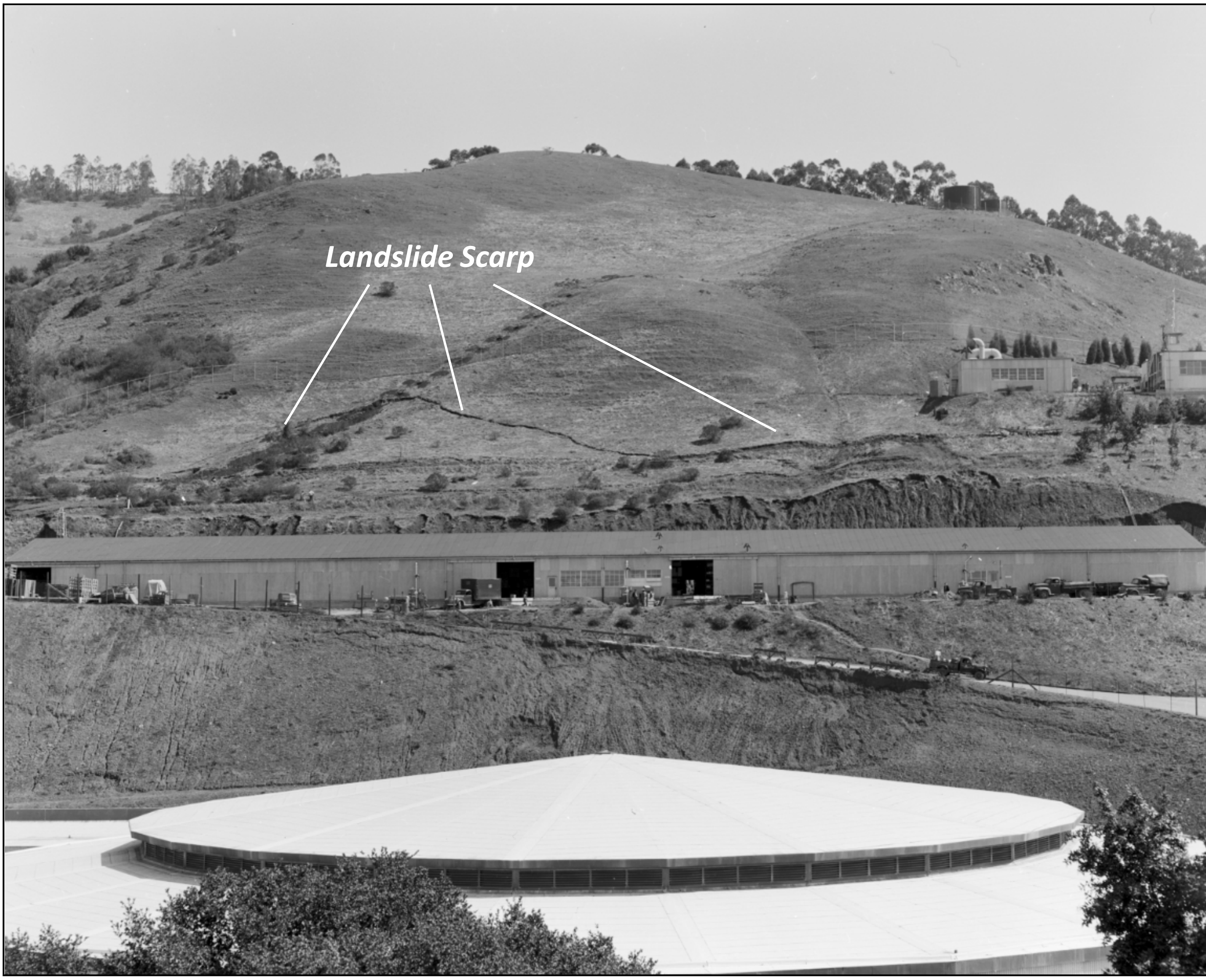
Source: LBNL Photo Archives – XBD201407-00909 *(December 6, 1949)*

Photograph looking Northeast

Source: LBNL Photo Archives – XBD9709-03489 *(March 28, 1950)*

Photograph looking Northeast

Source: LBNL Photo Archives – XBD201407-00878 *(March 14, 1952)*

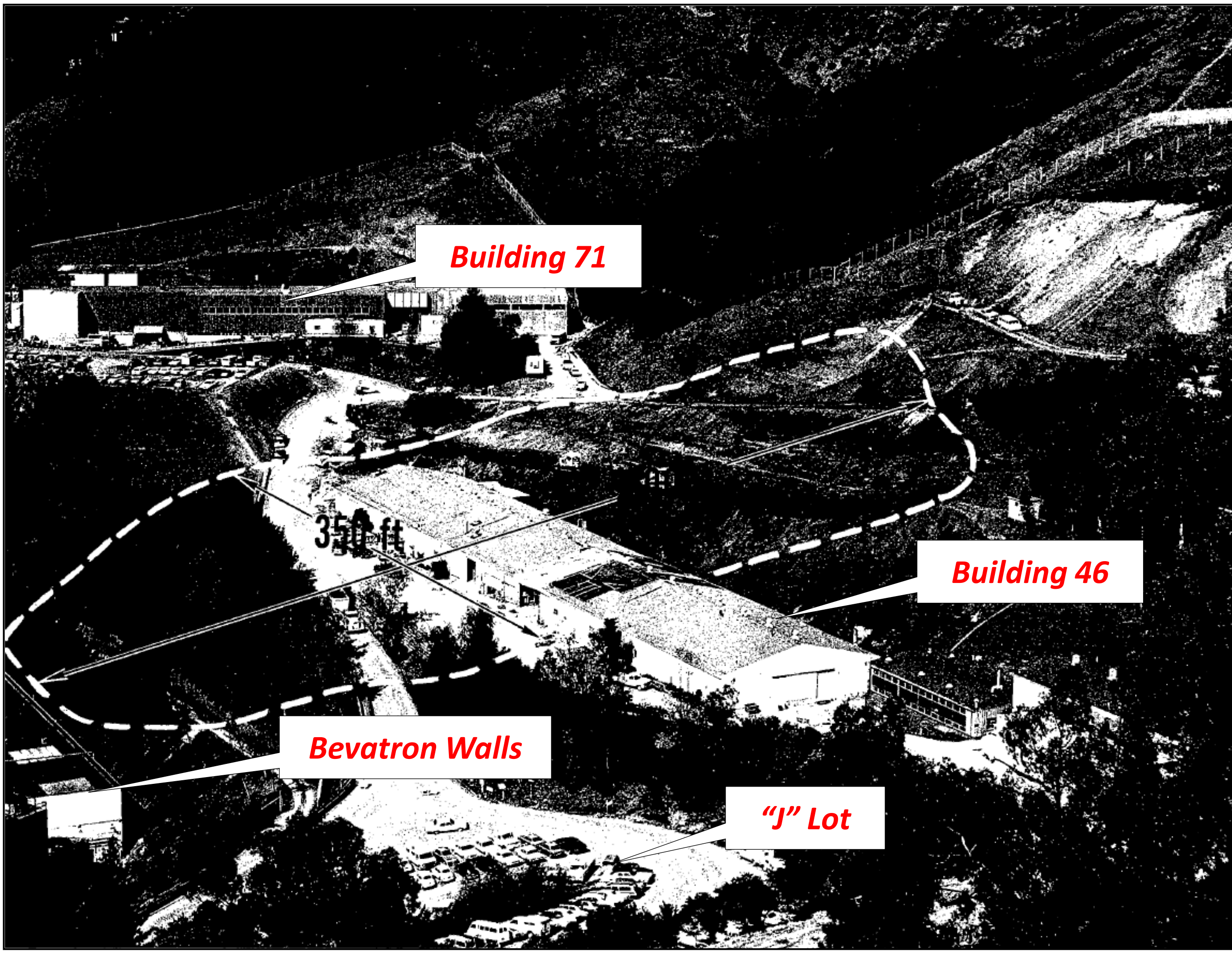


Photograph looking East

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 33
March 1952 Photograph

Source: USAEC, 1973 (Photograph date March 1973)



Photograph looking Northeast

A3GEO

June 11, 2014

Photograph looking North

June 11, 2014

Photograph looking South

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 36
South Room Tiebacks

January 1, 2011

Photograph looking North

Photograph looking Southwest

September 7, 2011

Photograph looking East

Plate 39
Demolition Photograph 3

November 17, 2011

Photograph looking Northeast

April 19, 2012

Photograph looking South

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

Plate 41
Cement Treatment

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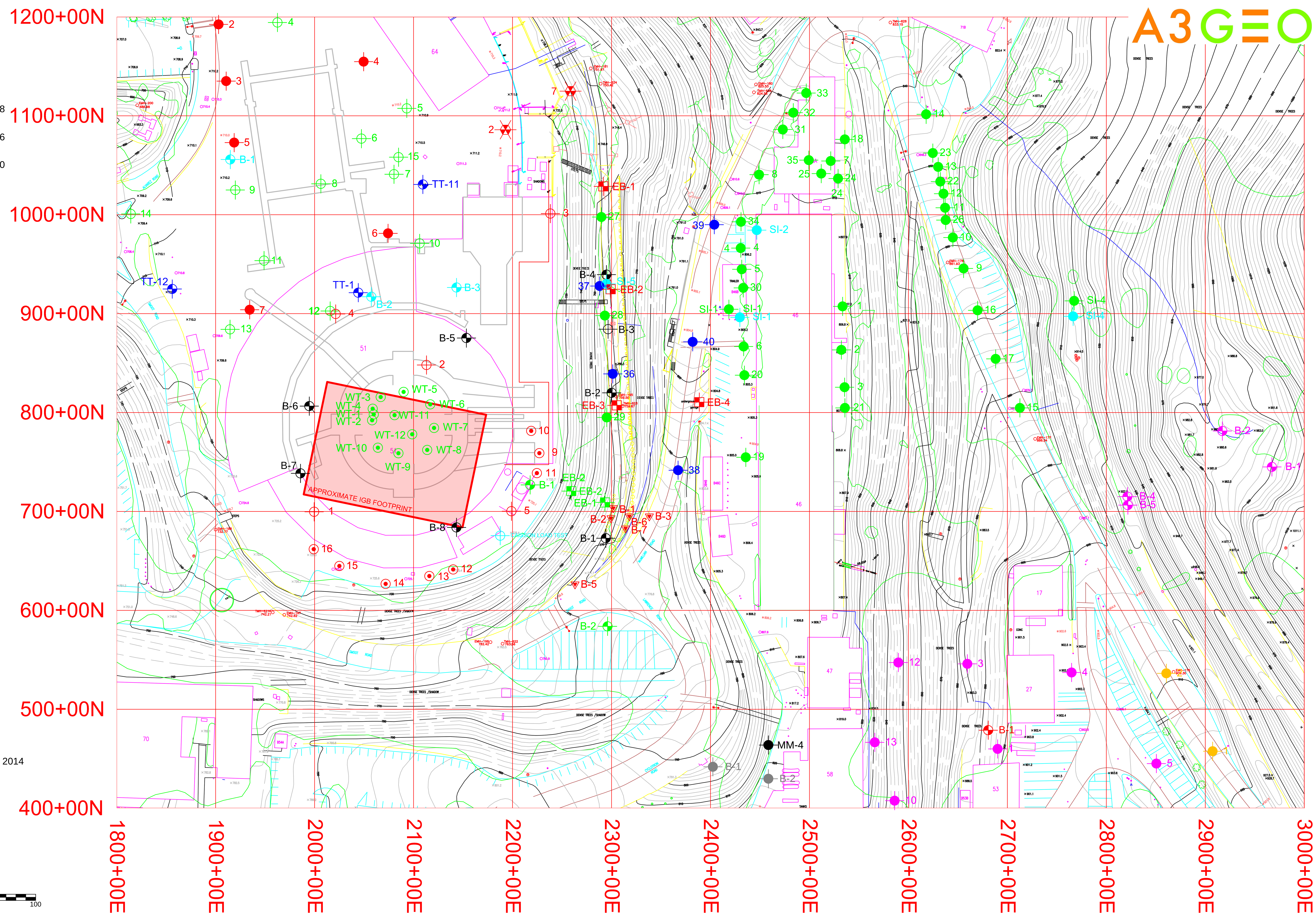
Figures

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

A3GEO, Inc.
PROJECT No. 1100-17B
LBNL - IGB

LEGEND:

9	BORINGS BY DAMES & MOORE 1948
9	BORINGS BY DAMES & MOORE 1956
9	BORINGS BY DAMES & MOORE 1960
9	BORINGS BY HA, 1965
9	BORINGS BY HLA, 1973
9	BORINGS BY HLA, 1976a
9	BORINGS BY HLA, 1976b
9	BORINGS BY HLA, 1976c
9	BORINGS BY HLA, 1983
B-2	BORINGS BY HLA, 1987
MM-4	BORINGS BY HLA, 1988
EB-1	BORINGS BY KALDVEER, 1992
EB-1	BORINGS BY HARZA, 1994
EB-1	BORINGS BY HARZA, 1996
9	ENVIRONMENTAL BORINGS BY LBNL (A3GEO/LCI 2011)
WT-9	BORINGS BY A3GEO/AKA, 2011
B-1	BORINGS BY A3GEO, 2011
B-1	ENVIRONMENTAL PUSH CORES BY LBNL (A3GEO/LCI 2011)
B-1	BORINGS BY A3GEO/AKA, 2012a
TT-9	BORINGS BY A3GEO/AKA, 2012b
B-1	BORINGS BY A3GEO/LCI, 2013
B-1	BORINGS BY A3GEO/LCI, 2014
B-3	BORING ATTEMPED BY A3GEO/LCI, 2014
7	BORINGS BY SCI, 1995
B-7	BORINGS BY GRC, 1993



UC GRID NORTH

SCALE: 1" = 50'
0 50 100

FIGURE 1

SITE PLAN

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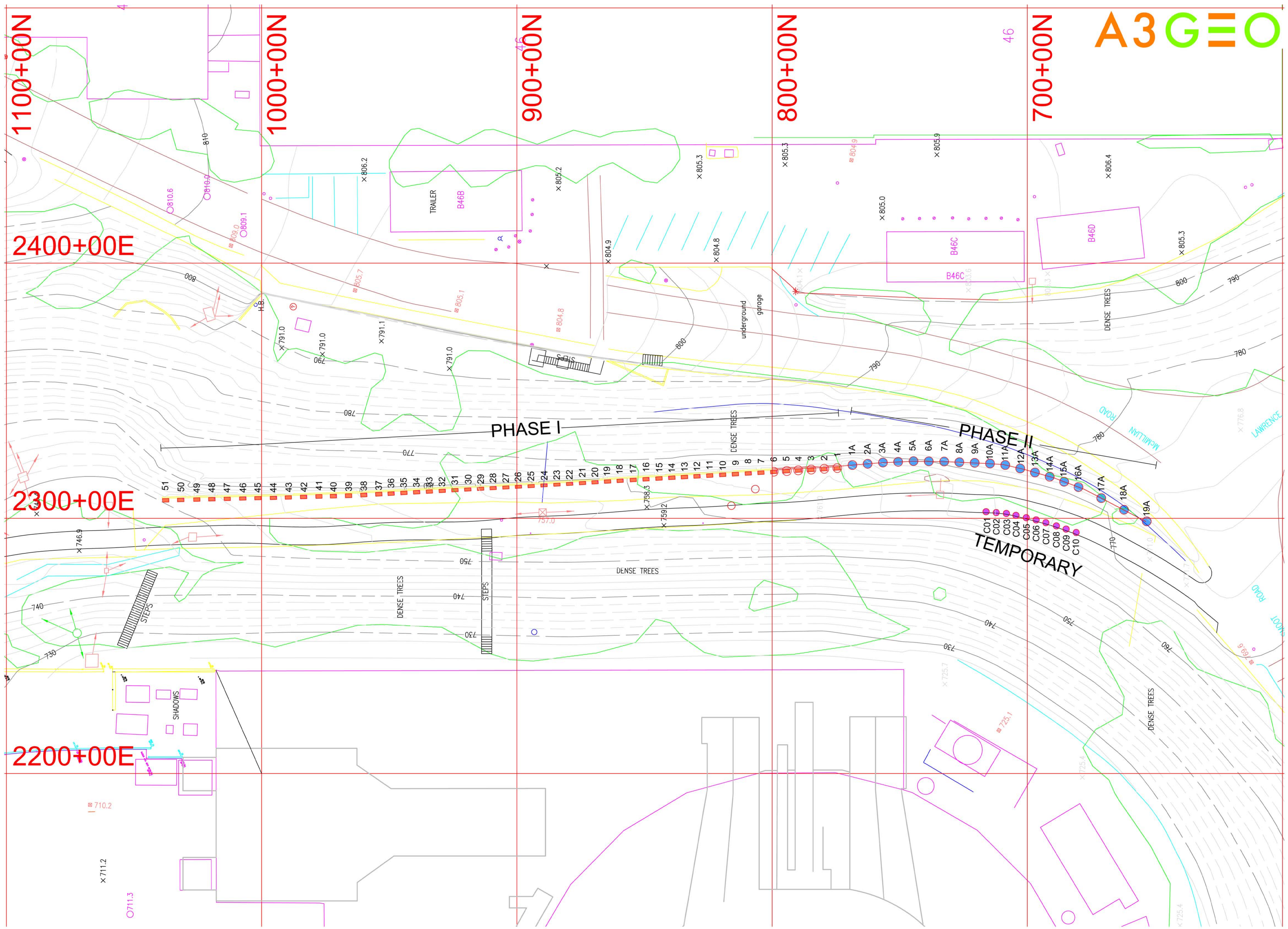
LEGEND

CAISSON LOCATIONS AND NUMBERS

- PHASE I
- PHASE II
- TEMPORARY



UC GRID NORTH

SCALE: 1" = 20'
0 20 40

FIGURE 1A

CAISSON LOCATIONS

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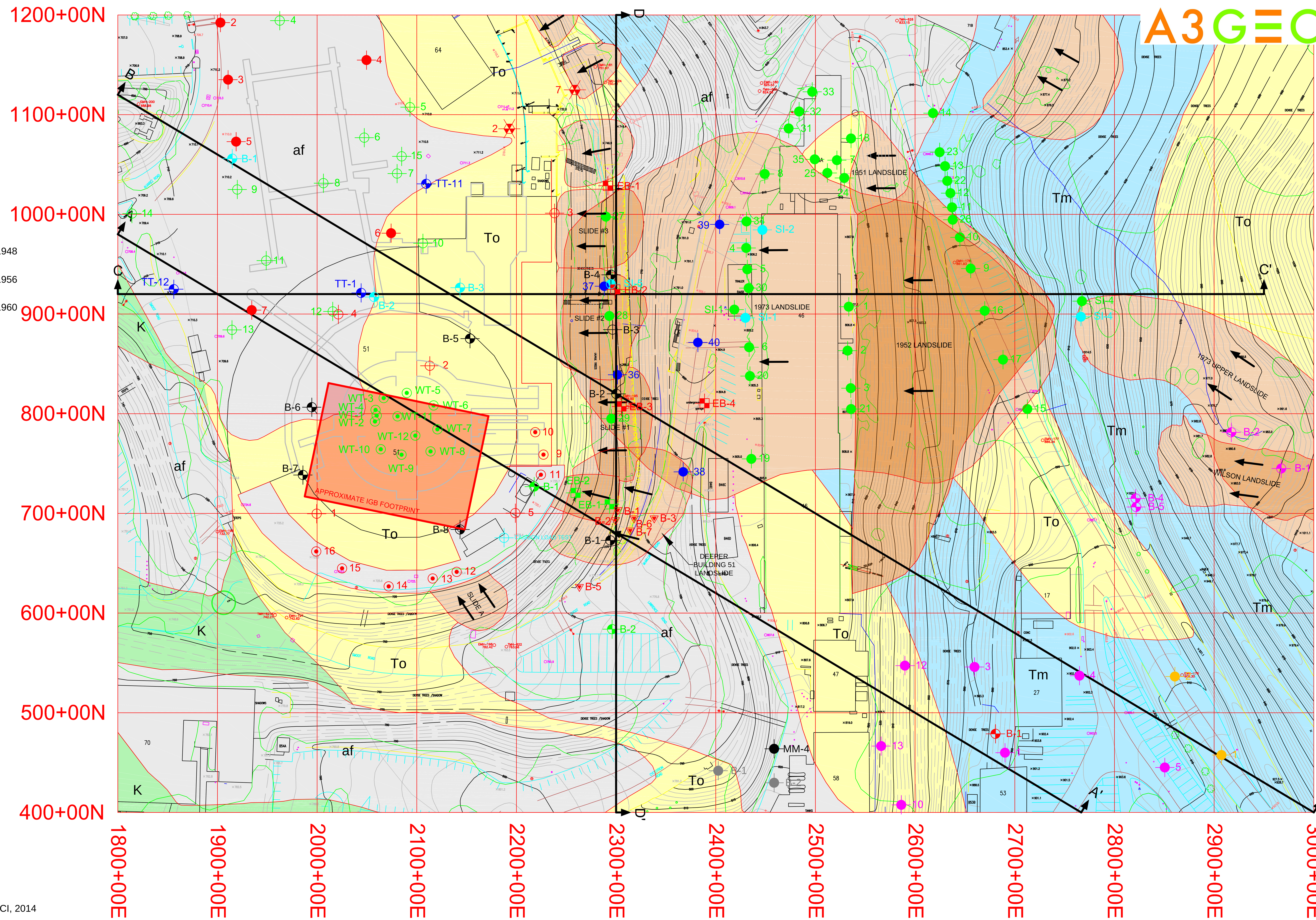

LEGEND:

af	ARTIFICIAL FILL
	LANDSLIDES
Tm	MORAGA VOLCANICS
To	ORINDA FORMATION
K	GREAT VALLEY COMPLEX
9	BORINGS BY DAMES & MOORE 1948
9	BORINGS BY DAMES & MOORE 1956
9	BORINGS BY DAMES & MOORE 1960
9	BORINGS BY HA, 1965
9	BORINGS BY HLA, 1973
9	BORINGS BY HLA, 1976a
9	BORINGS BY HLA, 1976b
9	BORINGS BY HLA, 1976c
9	BORINGS BY HLA, 1983
B-2	BORINGS BY HLA, 1987
MM-4	BORINGS BY HLA, 1988
EB-1	BORINGS BY KALDVEER, 1992
EB-1	BORINGS BY HARZA, 1994
EB-1	BORINGS BY HARZA, 1996
9	ENVIRONMENTAL BORINGS BY LBNL (A3GEO/LCI 2011)
WT-9	BORINGS BY A3GEO/AKA, 2011
B-1	BORINGS BY A3GEO, 2011
B-1	ENVIRONMENTAL PUSH CORES BY LBNL (A3GEO/LCI 2011)
B-1	BORINGS BY A3GEO/AKA, 2012a
TT-9	BORINGS BY A3GEO/AKA, 2012b
B-1	BORINGS BY A3GEO/LCI, 2013
B-1	BORINGS BY A3GEO/LCI, 2014
B-3	BORING ATTEMPED BY A3GEO/LCI, 2014
7	BORINGS BY SCI, 1995
B-7	BORINGS BY GRC, 1993

SCALE: 1" = 50'
0 50 100

UC GRID NORTH

FIGURE 2

GEOLOGIC MAP

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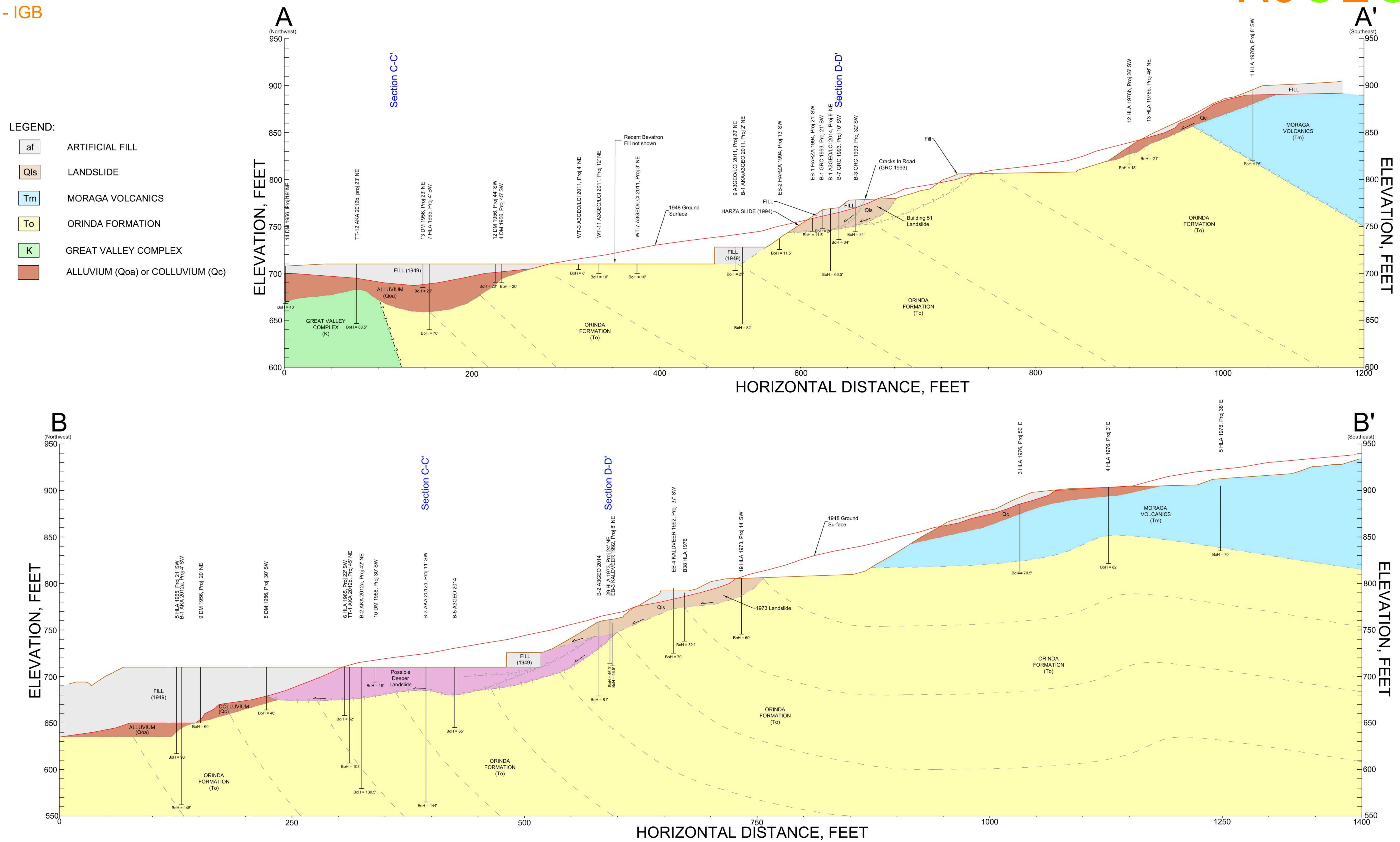

FIGURE 3
CROSS SECTIONS A-A' & B-B'

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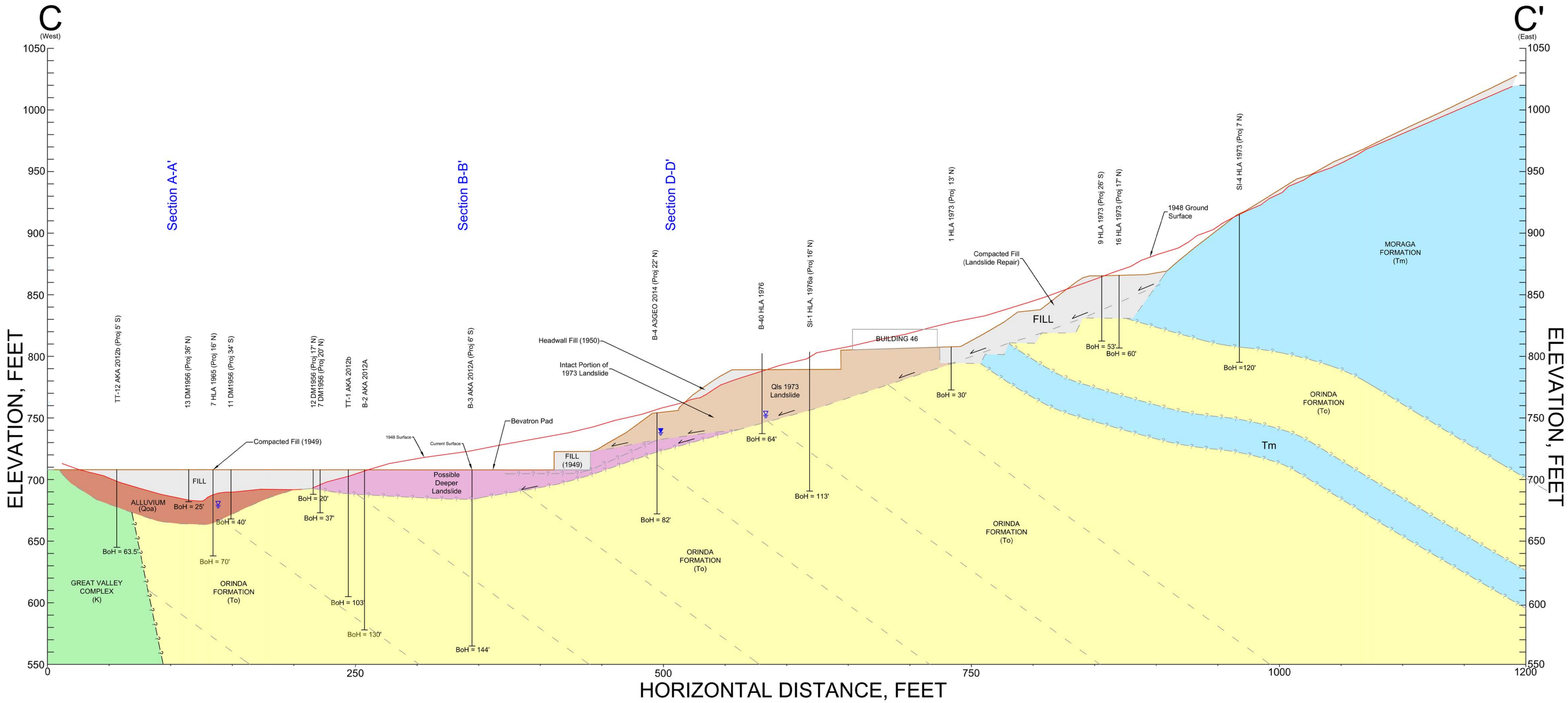

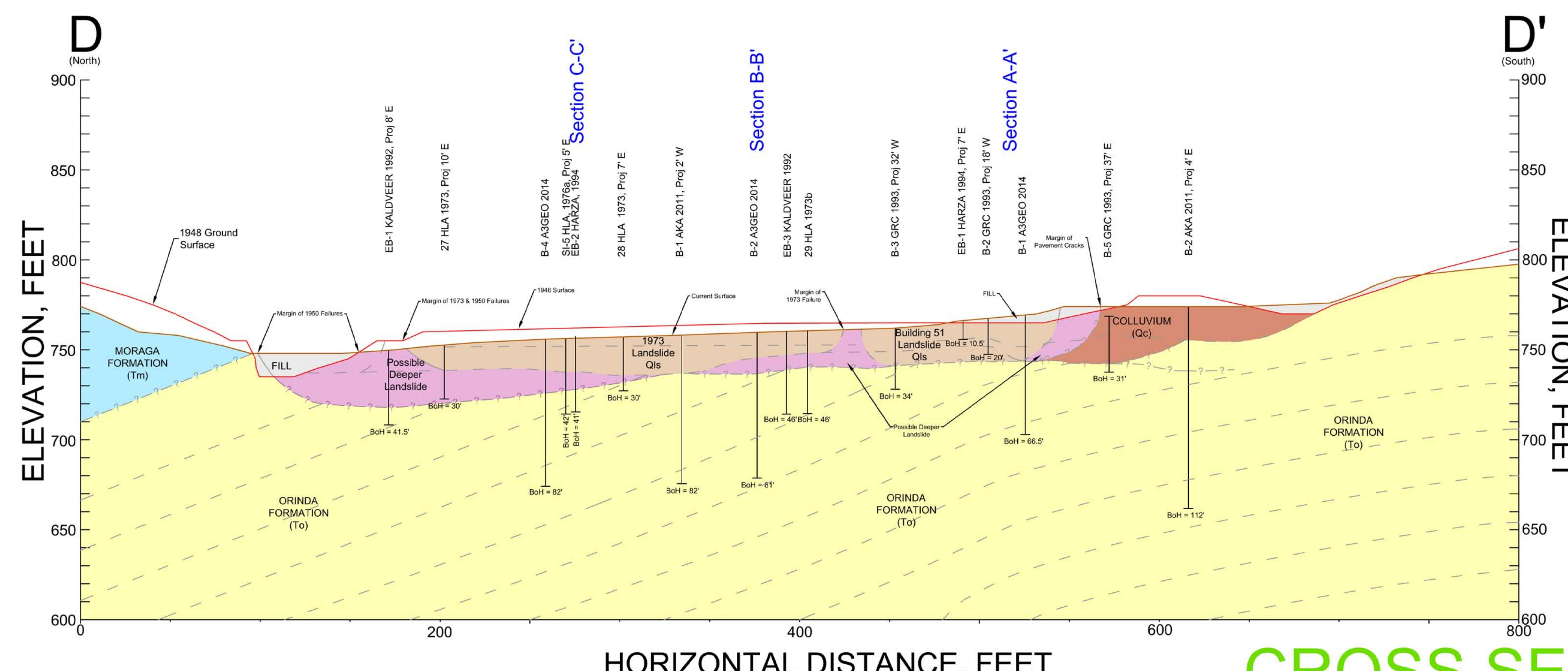

LEGEND:

af	ARTIFICIAL FILL
Qls	LANDSLIDE
Tm	MORAGA VOLCANICS
To	ORINDA FORMATION
K	GREAT VALLEY COMPLEX
	ALLUVIUM (Qoa) or COLLUVIUM (Qc)

FIGURE 4

CROSS SECTIONS C-C' & D-D'

Appendix A

Boring Logs

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

APPENDIX A
LOGS OF BORINGS

Our borings are numbered B-1, B-2 and B-4 through B-8. Summary information pertaining to the borings follows.

Summary of Borings

Boring ID	Location	Surface Elevation	Boring Depth	Bottom Elevation
B-1	Upper Access Road	+770 feet	66.5 feet	+703.5 feet
B-2	Upper Access Road	+760 feet	81 feet	+679 feet
B-4	Upper Access Road	+755 feet	81.75 feet	+673.25 feet
B-3	*(not drilled)*			
B-5	Bevatron Flat	+710 feet	65 feet	+645 feet
B-6	Bevatron Flat	+710 feet	51.5 feet	+658.5 feet
B-7	Bevatron Flat	+710 feet	50 feet	+660 feet
B-8	Above Perimeter Retaining Wall	+725 feet	51 feet	+674 feet

The ground surface elevations in the above table were estimated using the topographic contours shown on LBNL survey drawings (Q-Sheets) and should be considered approximate. Samples were obtained using the following tools.

Sampling Equipment

Tool	Approximate Sample Diameter
HQ wireline rock core	2.5 inches
101 Geobarrel	2.5 Inches
Standard Penetration Test (SPT) drive sampler	1.38 inches
Modified California (Mod Cal) drive sampler	2.5 inches
Pitcher Barrel sampler	3 inches

The SPT and Mod Cal drive samplers were advanced using a 140-pound automatic-trip hammer falling 30 inches. The hammer blows required to drive the sampler the final 12 inches of each 18-inch drive are presented on the boring logs. Where the sampler met early refusal, the number of hammer blows and the corresponding depth of penetration (in inches) are indicated. In all cases, the SPT sampler was driven without liners. The Mod Cal sampler was driven without liners in borings B-1, B-2, B-5 and B-5 and with liners in borings B-6, B-7 and B-8. All of the samples obtained from borings B-1, B-2, B-4 and B-5 were cleaned, placed in cardboard HQ core boxes and sealed in plastic wrap. All of the samples were transported to A3GEO's laboratory for further review and analysis.

An LCI Certified Engineering Geologist reviewed the contents of the HQ core boxes and augmented the field boring logs with engineering geologic data and notations. An A3GEO Geotechnical Engineer reviewed samples to check soil classifications and select suitable specimens for laboratory testing. Soils were classified in general accordance with ASTM D2488, which is based on the Unified Soil Classification System (USCS). The USCS is described on the Key to Exploratory Boring Logs, Figure A1. Rock was classified in general accordance with the Physical Properties for Rock Descriptions described on Figure A2. The attached log depicts interpreted subsurface conditions at the approximate location shown on the Site Plan (Figure 1) on the particular date designated on the log; the passage of time may result in changes in the subsurface conditions. The boring locations indicated on the Site Plan were determined by measuring from existing improvements and should be considered approximate.

UNIFIED SOIL CLASSIFICATION CHART

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS: more than 50% retained on No. 200 sieve	COARSE GRAINED SOILS: 50% or more of coarse fraction on No. 4 sieve	CLEAN GRAVELS	GW	Well graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		GRAVELS WITH SAND	GM	Silty gravels and gravel-sand-silt mixtures
			GC	Clayey gravels and gravel-sand-clay mixtures
	SANDS: more than 50% passing on No. 4 sieve	CLEAN SANDS	SW	Well graded sands and gravelly sand, little or no fines
			SP	Poorly graded sands and gravelly sand, little or no fines
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS: 50% or more passing No. 200 sieve	SILTS AND CLAY: Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
			CL	Inorganic clays or low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAY: Liquid Limit 50% or greater		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic clays
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS			PT	Peat, muck, and other highly organic soils

BOUNDARY CLASSIFICATION AND GRAIN SIZES

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

U.S. Standard Sieve Sizes: No. 200 (0.075 mm); No. 40 (0.425 mm); No. 10 (2 mm); No. 4 (3/16"); 3/4"; 3"; 12"

SYMBOLS

Modified California (MC) Sampler (3" O.D.)	HQ ROCK CORE (RC)	101 Barrel (SS)
Standard Penetration Test: SPT (2" O.D.)	Pitcher Tube (ST)	Water Levels At time of drilling At end of drilling After drilling

ABBREVIATIONS

Item	Meaning
LL	Liquid Limit (%) (ASTM D 4318)
PI	Plasticity Index (%) (ASTM D 4318)
-200	Passing No. 200 (%) (ASTM D 1140)
TXCU	Laboratory consolidated undrained triaxial test of undrained shear strength (psf) (ASTM D 4767)
TXUU	Laboratory unconsolidated, undrained triaxial test of undrained shear strength (psf) (ASTM D 2850)
psf/tsf	pounds per square foot / tons per square foot
psi	pounds per square inch
OD	Outside Diameter
ID	Inside Diameter

NOTES

1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Modified California (MC) blow counts were adjusted by multiplying field blow counts by a factor of 0.63.
3. Recorded blow counts have not been adjusted for hammer energy.

BEDDING OF SEDIMENTARY ROCK		
SPLITTING PROPERTY	THICKNESS	STRATIFICATION
Massive	Greater than 4.0 feet	Very Thick-Bedded
Blocky	2.0 to 4.0 feet	Thick-Bedded
Slabby	0.2 to 2.0 feet	Thin-Bedded
Flaggy	0.05 to 0.2 feet	Very Thin-Bedded
Shaly or Platy	0.01 to 0.05 feet	Laminated
Papery	Less than 0.01 feet	Thinly Laminated

FRACTURING	
INTENSITY	SIZE OF PIECES IN FEET
Very Little Fractured	Greater than 4.0 feet
Occasionally Fractured	1.0 to 4.0 feet
Moderately Fractured	0.5 to 1.0 feet
Closely Fractured	0.1 to 0.5 feet
Intensely Fractured	0.05 to 0.1 feet
Crushed	Less than 0.05 feet

HARDNESS	
Soft	Reserved for plastic material alone
Low Hardness	Can be gouged deeply or carved easily by a knife blade
Moderately Hard	Can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away
Hard	Can be scratched by a knife blade with difficulty; scratch produces little powder and is often faintly visible
Very Hard	Cannot be scratched by a knife blade; leaves a metallic streak

STRENGTH	
Plastic	Very low strength
Friable	Crumbles easily by rubbing with fingers
Weak	An unfractured specimen of such material will crumble under light hammer blows
Moderately Strong	Specimen will withstand a few heavy hammer blows before breaking
Strong	Specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments
Very Strong	Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

WEATHERING:	
	— the physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing
Deep	Moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
Moderate	Slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
Little	No megascopic decomposition of minerals; little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
Fresh	Unaffected by weathering agents. No discoloration or disintegration. Fractures usually less numerous than joints.

KEY TO ROCK DESCRIPTIONS

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BORING NUMBER B-1

CLIENT Lawrence Berkeley National Laboratory (LBNL)
PROJECT NAME Integrative Genomics Building (IGB), Geotechnical Investigation
PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/11/14 **COMPLETED** 7/14/14
GROUND ELEVATION 770 ft **HOLE SIZE** 3.75"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
0.0		Asphaltic concrete							9:45 (7/11/14)
		(SC) CLAYEY SAND: Yellowish brown, some gravel, poorly graded, stiff (gravel content consists of dark grey and light brown rock fragments) [FILL]							
2.5									
5.0			SS					35	10:07
		(CH) SANDY FAT CLAY: Grayish brown to reddish brown, with gravel and silt, subangular to subrounded gravel, plastic fines, soft to medium stiff, moist [FILL, possible landslide repair]							10:11
7.5									
		@9': very moist to wet, soft							10:30-13:30, drilling stopped for mechanical issues
10.0									

(Continued Next Page)

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10.0	(CH) SANDY FAT CLAY: Grayish brown to reddish brown, with gravel and silt, subangular to subrounded gravel, plastic fines, soft to medium stiff, moist [FILL, possible landslide repair]*(continued)*							
12.5	b/w 12'-13': clasts of bluish green fine gravel, from Moraga Formation rocks @13': stiff, no gravel	SPT	5				44	
	(CL) GRAVELLY LEAN CLAY: Dark olive brown to reddish brown, very soft [Qls]	SS					83	
15.0	(GP) POORLY GRADED GRAVEL: Greenish grey, very angular Moraga formation clasts, no clay, grading to gravelly clay below, loose [Qls]	SS					67	
	(CH) GRAVELLY FAT CLAY: Dark reddish brown, some silt, moderate plasticity, weak discontinuous seams, stiff to very stiff, contains small blocks of clay, moist [Qls]							14:10, circulation lost in drilling, placed 18.5' of 5" casing
17.5								15:00
20.0	- color change to yellowish brown							

(Continued Next Page)

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PROJECT NUMBER 1100-17B **PROJECT LOCATION** Berkeley, CA
DATE STARTED 7/11/14 **COMPLETED** 7/14/14 **GROUND ELEVATION** 770 ft **HOLE SIZE** 3.75"
DRILLING CONTRACTOR Pitcher Drilling Co. **GROUND WATER LEVELS:**
DRILLING METHOD Rotary Wash Drilling **AT TIME OF DRILLING** ---
LOGGED BY RES **CHECKED BY** JNB **AT END OF DRILLING** ---
NOTES **AFTER DRILLING** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
20.0		(CH) GRAVELLY FAT CLAY: Dark reddish brown, some silt, moderate plasticity, weak discontinuous seams, stiff to very stiff, contains small blocks of clay, moist [Qls] *(continued)*	SS					71	
		@21': abrupt transition to claystone, possible baked zone between Morage and Orinda Formations - no distinct shear claystone							Residual Torsional Strength (See Appendix C)
		CLAYSTONE: Dark reddish brown, weak, low hardness, moderately weathered, polished and slickensided, no distinct shear [Qls]							
		(no recovery from 22'-23' from 101 core barrel, cuttings retrieved from SPT to 24')	SS					0	15:15
22.5		SILTSTONE/CLAYSTONE: Bluish grey, plastic to friable, low hardness, moderately weathered							
			SPT	55				78	
25.0		@25.5' to 26.5': clay-rich	RC					100	15:35/7:20 (7/14/14)
		@26.5': darker bluish grey silty claystone.							7:45
27.5		@27.5': contact at 20-30 deg inclination, increasing in fine sand content with depth, intensely fractured, polished random slicks, no planar connections							
		SANDY SILTSTONE: Dark bluish grey, plastic to friable, low hardness, moderately weathered	RC					100	
30.0		@30': contact at <10 deg with siltstone/claystone w/ few fractures.							

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PROJECT NUMBER	1100-17B	**PROJECT LOCATION**	Berkeley, CA
DATE STARTED	7/11/14 **COMPLETED** 7/14/14	**GROUND ELEVATION**	770 ft **HOLE SIZE** 3.75"
DRILLING CONTRACTOR	Pitcher Drilling Co.	**GROUND WATER LEVELS:**	
DRILLING METHOD	Rotary Wash Drilling	**AT TIME OF DRILLING**	---
LOGGED BY	RES **CHECKED BY** JNB	**AT END OF DRILLING**	---
NOTES		**AFTER DRILLING**	---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
30.0		SILTSTONE/CLAYSTONE: Bluish grey, plastic to friable, low hardness, moderately weathered, few fractures, iron oxide stain along fractures (staining grades less w/ depth)							
									8:10
32.5									
		@34': weak bedding @ <30 deg	RC					100	
35.0									
		b/w 36.5'-39.5': color change, dark grey to black b/w 36.5'-38': intensely fractured, hard, polished, subangular to angular gravel size fragments							
									8:25
37.5									
		b/w 38'-39': Drill dropped as if through a void, possible loose gravel layer?	RC					11	
		SANDSTONE: Light bluish grey, very fine, grades to sandy siltstone, weak to friable, moderately hard, moderately weathered, blocky to massive							8:45
40.0									

(Continued Next Page)

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GROUND WATER LEVELS:
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LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
40.0		SANDSTONE: Light bluish grey, very fine, grades to sandy siltstone, weak to friable, moderately hard, moderately weathered, blocky to massive *(continued)*	RC					100	
		b/w 41.5'-43.5': no recovery, loss of circulation due to drilling through hydrauger, drill fluid observed flowing from drain pipe at toe of slope below, casing installed to 43.5'							8:55, lost drilling fluid through hydrauger in hillside, drilling stopped in wait for casing delivery; 13:10, placed 43.5' of 4" casing and washed out
42.5									
		SILTSTONE: Light bluish grey, reddish brown laminations, plasitc, soft, moderately weathered, moderately fractured, shaly bedding (SHEAR ZONE?) @44.7': graded contact, not distinct							
45.0		SANDSTONE: Light bluish grey, fine to medium sands, weak to friable, low to moderate hardness, moderately weathered, blocky	RC					110	
									14:00
47.5		@47.4' to 48': thin seam of limestone (possibly quartz), 1"-2" thick, hard, resistant, contact obscured by blocky texture, no clay shears							
		grading to siltstone	RC					102	
50.0									

(Continued Next Page)

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GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
50.0		SILTSTONE: Dark bluish grey, weak clayey veins, soft, plastic to friable, moderately weathered, no clay shears							
		SANDSTONE: Dark bluish grey, shallow bedding, weak to friable, soft to low hardness, moderateley weathered, grading from medium sandstone to coarse granules (Drill dropped straight from 51.5'-53', gravel?) @52': slick clay-coated fracture	RC					89	14:30
52.5		b/w 53'-54.5': No recovery, trace of medium sandstone at top of sample below	RC					62	14:45
55.0		SILTSTONE: Dark bluish grey, sandy, weak to friable, soft to low hardness, moderately weathered, close to intense fracturing, shaley bedding @55.3': sandstone seam, 2"-3" thick, bedding inclined less than 10 deg @ 56' : Sandy Siltstone							
		SILTSTONE/CLAYSTONE: Light gray to dark grey @56.4': light grey, weak to moderateley strong, moderately hard, moderately weathered @56.8': dark grey, intensely fractured, abundant polished slicks but random and discontinuous b/w 57'-63.2': dark grey, friable to weak, low hardness, moderately weathered, closely fractured, thin bedding, competent but weak	RC					97	14:55
57.5		@58.5': veins and fractures within clayey zone							
60.0									15:30

(Continued Next Page)

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GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
60.0		SILTSTONE/CLAYSTONE: Light gray to dark grey @56.4': light grey, weak to moderateley strong, moderately hard, moderately weathered @56.8': dark grey, intensely fractured, abundant polished slicks but random and discontinuous *(continued)*	RC					83	16:00
62.5		LIMESTONE: Light grey to white, hard, resistant	RC					97	
		@63.7': Siltstone seam 2"-3" thick							
		LIMESTONE: Light grey to white, hard, resistant							
65.0		SILTSTONE: Light grey, strong, moderately hard, lightly weathered, thin bedding, closeley fractured gradational contact							
		CLAYSTONE: Dark grey, soft to friable, low hardness, very thin to papery bedding							16:40

Bottom of borehole at 66.5 feet.
1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Inclinometer installed to a depth of 65.25', 7/15/14
3. Groundwater level was obscured by drilling method.

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BORING NUMBER B-2

CLIENT Lawrence Berkeley National Laboratory (LBNL)
PROJECT NAME Integrative Genomics Building (IGB), Geotechnical Investigation
PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/15/14 **COMPLETED** 7/17/14
GROUND ELEVATION 760 ft **HOLE SIZE** 3.75"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
0.0		(CL) LEAN CLAY: Reddish brown, very stiff, moderate plasticity, some fine to medium sand, mottled with olive brown cohesive silty sand [Qls].							7:15 (7/16/14), hand-augered 0'-5'
2.5									
5.0			MC	21				83	
		(CL) SANDY CLAY: Dark brown, very stiff, moderate to low plasticity, fine to medium sand, moist [Qls].	SPT	16				83	
7.5									
		@8': with fine clasts of volcanic rock clasts and weathered tuff. (Moraga Formation)	MC	20				94	8:05, 5" casing installed to 8'
10.0									

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10.0		(CL) SANDY CLAY: Dark brown, very stiff, moderate to low plasticity, fine to medium sand, moist [Qls].*(continued)*	SPT	16				44	
		@11': more angular coarse sand	MC	14				72	
		(CH) FAT CLAY: Dark reddish brown, stiff, moderate to high plasiticity, some silty, moist. [Qls] b/w 12'-12.5' : Stiff to soft b/w 12'-12.5 : Stiff to soft clay, may be old slide plane?							Residual Torsional Strength (See Appendix C)
12.5			SPT	11				61	
			MC	25				89	
		SANDY SILTSTONE: Light brown, thinly laminated, fine sand, , plastic to friable, soft to low hardness, moderately weathered [ORINDA FORMATION]							
15.0		SANDSTONE: Olive brown, medium to coarse sand, friable, soft to low hardness, deeply weathered							9:30
			ST					100	
17.5									TXUU (See Appendix C)
		@ 18' : grades quickly to siltstone, no shearing at contact	RC					108	10:00
		SILTSTONE: Greenish olive brown, plastic, soft to low hardness, moderateley weathered @19.75': color change to red siltstone at 30 deg							
20.0									

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20.0		SILTSTONE: Dark reddish brown w/ green mottling, soft to low hardness, moderately weathered, plastic to friable @ 21' : fine sandy siltstone greenish gray to light blue gray contact dipping 20-30 degrees.							10:40
		SANDY SILTSTONE: Greenish/light bluish gray, fine sand, friable, low hardness, moderately weathered, very thin bedding, veins of overlying claystone @22.5': clay seam at contact dipping 20-30 degrees							
22.5		CLAYEY SILTSTONE: Reddish brown, plastic, soft to low hardness, deeply weathered, some hard fragments							
		SANDY SILTSTONE: Same as sandy siltstone from 21.5'-22.5' @24' ferrous oxide stained contact	RC					105	
25.0		SANDSTONE: Bluish grey, medium to coarse sand, friable, low hardness, moderately weathered							11:05
27.5		@27': weak claystone seam b/w 27'-28': becomes more red and more silty							
		CLAYSTONE/SILTSTONE: Reddish brown/dark bluish grey, alternating laminations, friable, low hardness, moderately weathered @ 29.5' : Abundant fine open fractures, gradational lower boundary/contact.	RC					110	
30.0									

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30.0		SANDY SILTSTONE: Blue gray, fine to medium sand, weak to friable, low hardness, moderateley weathered							
									11:25
		SILTSTONE: Light reddish brown, friable, low hardness, moderateley weathered, closely fractured @ 20 deg (@33.5: 20 deg contact)							
32.5									
		SANDSTONE: Bluish grey, very fine, friable, low hardness, moderately weathered	RC					100	
35.0									
		CLAYSTONE: Light reddish brown, weak, low hardness, moderately weathered							11:45
37.5									
		@ 38' : intensley fractured claystone/siltstone, no clay shears or soft zones.	RC					100	
40.0									

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40.0		CLAYSTONE: Light reddish brown, weak, low hardness, moderately weathered*(continued)* @40': Open fractures, crushed and friable, dry							
		CLAYSTONE/SILTSTONE: Reddish brown/dark bluish grey, alternating laminations, weak to friable, low hardness, little weathering							12:00
42.5		SANDSTONE: Light bluish grey, friable, low hardness, moderately weathered - b/w 44.5'-45.5': color grades to dark grey; @45.5': 45 deg contact that is gradutional to abrupt.	RC					103	
45.0		CLAYSTONE: Reddish brown, friable, low hardness, moderately weathered							
		CLAYSTONE/SILTSTONE: Reddish brown/dark bluish grey, alternating laminations, weak, low hardness, moderately weathered, 46'-48' massive							12:30
47.5		@ 48' : Very dark gray claystone fractured (or laminated shears?) fine planar partings common.	RC					97	
50.0									

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50.0		CLAYSTONE/SILTSTONE: v. dark grey, friable, soft to low hardness, moderateley weathered, closely fractured, laminated shears, fine planar partings, polished slicks, no clay gouge (@50': ~45 deg fracture, change to v. dark grey)*(continued)*							
52.5		CLAYSTONE: Dark grey, friable, low hardness, moderateley weathered, prominent planar partings, intensely fractured, polished fractures (@52.5': fine limetstone) @ 53' : Flakes easily. - b/w 53.5'-54.5': blocks of gray limestone, hard, resistant - b/w 54.5'-56': change to dark grey/black, claystone/siltstone w/ gypsum along fractures	RC					110	13:00
55.0									
57.5		LIMESTONE/DOLOMITE: Light gray, microcrystalline, hard, moderately strong, moderately weathered (extents inferred from clasts able to recover from zone)	RC					5	13:20
60.0									

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60.0		LIMESTONE/DOLOMITE: Light gray, microcrystalline, hard, moderately strong, moderately weathered (extents inferred from clasts able to recover from zone)*(continued)*							
		SILTSTONE: Dark bluish grey, massive, clean, weak to friable, moderate to low hardness, moderate weathering, diffuse bedding w/ limestone at 40-50 deg							14:50, core lost out of barrel, bit plugged from hole settling and material washed out
62.5			RC					100	
		- b/w 62.5'-63': limestone unit							
		- b/w 63'-64': steep fractures filled w/ calcite (?)							
									16:30
65.0			RC					104	
		@66': color change to light bluish gray							7:22 (7/17/14)
67.5									
			RC					100	
		@69': 50 deg contact							
		SANDSTONE: Dark bluish grey, fine to medium, weak, low to moderate hardness, moderate to little weathering, competent blocks, little to no open fractures							
70.0									

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70.0		SANDSTONE: Dark bluish grey, fine to medium, weak, low to moderate hardness, moderate to little weathering, competent blocks, little to no open fractures *(continued)*							
									7:43
72.5									
		@73.5': <10% coarse sand, sand stringers and laminations @20-25 deg	RC					100	
		- b/w 74'-76': light gray, grading to medium grained w/ some coarse sand							
75.0		- b/w 76'-77': increase in coarse sand and gravel clasts							
									8:00
		- b/w 77'-78': weak bedding of fractures at <20 deg							
77.5									
		CONGLOMERATE: Light bluish gray, subrounded fine gravel clasts cemented in well-graded sand matrix, weak to friable, low to moderate hardness, moderate weathering	RC					100	
80.0									

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80.0		CONGLOMERATE: Light bluish gray, subrounded fine gravel clasts cemented in well-graded sand matrix, weak to friable, low to moderate hardness, moderate weathering *(continued)*							
									8:32

Bottom of borehole at 81.0 feet.
1. Stratification lines represent the approxiamte boundaries between material types and the transitions may be gradual.
2. Downhole geophysics logging 7/17/14.
3. Borehole grouted 7/18/14.
4. Liners were not used in the Modified California (MC) sampler.
5. MC blow counts were adjusted by multiplying field blow counts by a factor of 0.63.
6. Groundwater level was obscured by drilling method.

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BORING NUMBER B-4

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PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/18/14 **COMPLETED** 7/22/14
GROUND ELEVATION 755 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
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0.0		(CL) LEAN CLAY: Reddish brown, silty, some fine sand, stiff, moderate plasticity, dry, w/ 10-20% fine sand [Qls]							hand-augered 0'-5'
2.5									
5.0		- b/w 5.5'-8': color change to brown, some dark grey angular gravel size fragments	MC	17				67	13:00 (7/18/14)
7.5			SPT	14				56	
		(CL) GRAVELLY CLAY: Olive brown with some grey and red mottling, stiff to very stiff, well-graded and massive, 20% fine to coarse gravel, angular clasts from Moraga and Orinda formations, moist to wet [Qls]	MC	20				67	
10.0									

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10.0	(CL) GRAVELLY CLAY: Olive brown with some grey and red mottling, stiff to very stiff, well-graded and massive, 20% fine to coarse gravel, angular clasts from Moraga and Orinda formations, moist to wet [Qls]*(continued)*	SPT	15				56	
		MC	16				56	
12.5	@12.5'-14': Soft clay rich, no large Moraga clasts.	SPT	9				44	
	SANDY SILTYSTONE: Bluish grey, wiht with clay, very stiff, soft, plastic to friable, deeply weathered, intensely fractured, very moist [Qls]	MC	20				67	
15.0	@15.25': thin weak clay seam (but not soft gouge)							
		SPT	17				50	
17.5		MC	21				56	13:35
	SILTSTONE/CLAYSTONE: Bluish grey/reddish brown, alternating laminations, soft to firm (possible basal slideplane) [Qls]							Peak and Residual Torsional Strength (See Appendix C)
	SILTSTONE: Grey, some reddish brown clay mottling, friable, low hardness, deeply weathered, becomes more competent with depth [ORINDA FORMATION]	SPT	35				44	
20.0								

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20.0		SILTSTONE: Grey, some reddish brown clay mottling, friable, low hardness, deeply weathered, becomes more competent with depth [ORINDA FORMATION]*(continued)*	MC	50/6"				67	13:55
			SPT	32				39	
22.5		- b/w 22.25'-23.75': soft, plastic, deeply weathered							
			ST						14:50
		SANDSTONE: Bluish gray, fine to medium, trace fine gravel, friable, low hardness, moderateley to deeply weathered [Qls]							TXUU (See Appendix C)
25.0									
		SILTY CLAYSTONE: Dark reddish brown, some gray mottling, soft, plastic, deeply weathered, moist [Qls]	ST						TXUU (See Appendix C)
27.5									15:45, 7:30 (7/21/14)
		SILTSTONE: Dark grey with reddish brown mottling, friable to weak, soft to low hardness, moderately weathered (w/ depth, from massive and well cemented to intense tight fracturing) [Qls]							
		@30.25': color change to light gray across bedding, diffuse soil-like boundary @ >50 deg	RC					92	
30.0									

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30.0		SILTSTONE: Bluish grey, friable, soft to low hardness, moderately weathered							
		@31': 1"-2" clay seam @<10 deg, possible deformation zone							
		- b/w 31'-32.5': brecciated zone w/ clay [POSSIBLE DEEP SLIDE, seam of red clay @32']							7:50
32.5		@33': thin limestone (?) seam							
		@34': thin limestone seam	RC					103	
35.0		@35': joint healed/filled with siltstone material							
		SILTSTONE/CLAYSTONE: Mottled bluish grey/reddish brown, friable to weak, soft to low hardness, moderately weathered							
		@36.25': calcite vein							
		@36.5': hard, wavy and indistinct contact @40-50 deg							8:15
37.5		@38': soft zone, but not landslide shear, same rock and no fractures above and below							
		@38.5': mottled with limestone, cemented and hard, ~3" thick							
		@39' : Limestone bed, cemented and hard.	RC					103	
40.0		@40': limestone, ~6" thick, intenstely fractured							

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40.0		SILTSTONE/CLAYSTONE: Mottled bluish grey/reddish brown, friable to weak, soft to low hardness, moderately weathered*(continued)* @40.5': relatively subhorizontal contact, no clay gouge, increasing bluish gray color, less mottling, increase in silt and fine sand content							
		SILTSTONE: Bluish grey, friable to weak, soft to low hardness, moderately weathered							8:55
42.5		@42.5': mostly gradational contact @<20 deg							
		SANDSTONE: Bluish grey, weak, low hardness, moderately weathered (thin clay contact @10 deg, possible old tectonic deformation, no shears or laminations) @43' : Reddish brown thin clay seam <10 degrees.							
		SILTSTONE: Reddish brown, some siltstone mottling, friable, soft to low hardness, moderately weathered							
		@44.25': increase in veins and precipitations	RC					105	
45.0		- b/w 45'-46': calcite veins, anastamosing							
		- b/w 46'-51': clayey siltstone - b/w 46.5'-48': massive							9:25
47.5		- b/w 48'-50': crushed and soft zone, more clay rich							
		- b/w 49'-50': calicite veins along fractures	RC					105	
50.0									

(Continued Next Page)

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BORING NUMBER B-4

CLIENT Lawrence Berkeley National Laboratory (LBNL)
PROJECT NAME Integrative Genomics Building (IGB), Geotechnical Investigation
PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/18/14 **COMPLETED** 7/22/14
GROUND ELEVATION 755 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
50.0		SILTSTONE: Reddish brown, some siltstone mottling, friable, soft to low hardness, moderately weathered*(continued)*							
		SILTSTONE/CLAYSTONE: Bluish gray/reddish brown, mottled, friable to weak, soft to low hardness, moderateley weathered (despite crushed zones, unlikely to be old landslide debris) @51': calcite veins - b/w 51'-52.25': intact - b/w 52.25'-53': crushed, clay rich, no distinct shears							9:50
52.5		- b/w 53'-54': intact - b/w 54'-54.5': crushed, clay rich, limestone/calcite veins - b/w 54.5'-56.25': intact	RC					100	
55.0		@56': irregular and sharp contact @50-60 deg							
		SILTSTONE: Bluish gray, weak, low hardness, moderately weathered - b/w 56.5'-57.8': no recovery							10:15
57.5		@58.5': crushed, clay rich, calcite veins and clasts - b/w 59'-59.5': weak clay rich shears (?)	RC					72	
60.0									

(Continued Next Page)

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PROJECT NUMBER 1100-17B
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DATE STARTED 7/18/14 **COMPLETED** 7/22/14
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DRILLING CONTRACTOR Pitcher Drilling Co.
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DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
60.0		SILTSTONE: Bluish gray, weak, low hardness, moderately weathered *(continued)*							
		@60.25': soft contact @ 30 deg							
		SILTSTONE/CLAYSTONE: Bluish gray/reddish brown, mottled, weak, soft to low hardness, moderately weathered, bedding at 30 deg							
		- b/w 61'-61.6': no recovery	RC					61	10:50
62.5		- b/w 62.5'-62.7': likely limestone (from clasts at top of recovery)	RC					92	11:20
		- b/w 64'-64.5': brecciated zone, consisting mostly of siltstone with limestone, dipping @45-50 deg							
		- b/w 64.5'-65.75': thin lenses of hard angular limestone, bounded by clay seams, slightly brecciated siltstone zone @~65.5'	RC					113	11:45
65.0		- b/w 65.75'-67.75: reddish brown w/ green mottling, friable to weak, moderate weathering							
		- b/w 66.5'-66.75': no recovery, likely limestone, angular to subround greyish white clasts of limestone intermittently bound by siltstone at top of recovery	RC					88	12:20
67.5		- b/w 67.75'-68: limestone; fractured, sheared, and soft @68' below limestone							
70.0									13:00 hole squeezing and bit plugged, hole reamed to 5"

(Continued Next Page)

CLIENT Lawrence Berkeley National Laboratory (LBNL)
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PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/18/14 **COMPLETED** 7/22/14
GROUND ELEVATION 755 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
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AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
70.0		LIMESTONE: Light olive brown w/ green and white mottling and streaks, some pink surfaces, cemented, moderateley strong, moderateley hard, moderateley weathered (no recovery b/w 69.5'-71.5' inferred from limestone material captured in the bit)*(continued)*	RC					14	
		SILTSTONE/LIMESTONE: Bluish gray/reddish brown, alternating lamina dipping 10-20 deg, friable to weak, soft to low hardness, moderately weathered, thinly fractured							14:45 loss of circulation, 7:15 (7/22/14) still no return of fluid, 5" casing installed to 70'
72.5		@72.5': thin alternating laminations							
		@73.0': very hard contact with claystone below at 60 deg							
		SILTSTONE/CLAYSTONE: Reddish brown, friable to weak, low hardness, moderately weathered	RC					100	
75.0									
		@75.5': Fault/sheared contact							
		@76.25': weather resistant nodules							9:40
77.5									
		@78': fractured and crushed, no clay shear	RC					100	
		@79.5': weak gypsum along fracture faces							
80.0									

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CLIENT Lawrence Berkeley National Laboratory (LBNL)
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PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/18/14 **COMPLETED** 7/22/14
GROUND ELEVATION 755 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
80.0		SILTSTONE/CLAYSTONE: Reddish brown, friable to weak, low hardness, moderately weathered *(continued)*							10:00

Bottom of borehole at 81.8 feet.
1. Stratification lines represent the approxiamte boundaries between material types and the transitions may be gradual.
2. Downhole geophysics logging 7/22/14.
3. Piezometer installed to 81'.
4. Liners were not used in the Modified California (MC) sampler.
5. MC blow counts were adjusted by multiplying field blow counts by a factor of 0.63.
6. Groundwater level was obscured by drilling method.

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BORING NUMBER B-5

CLIENT Lawrence Berkeley National Laboratory (LBNL)
PROJECT NAME Integrative Genomics Building (IGB), Geotechnical Investigation
PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/23/14 **COMPLETED** 7/25/14
GROUND ELEVATION 710 ft **HOLE SIZE** 4.875"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
0.0		ASPHALTIC CONCRETE							12:20 (7/23/14) hand-augered 0'-5'
		AGGREGATE BASE							
		(CL) GRAVELLY LEAN CLAY: Light olive brown, very stiff, plastic fines, angular to subangular gravel and concrete fragments, some medium to coarse sand, dry [FILL]							
2.5									
5.0		(SC) CLAYEY SAND: Dark brown, very dense, well graded, some subangular to angular fine gravel, some ferrous oxide and manganese oxide staining, clasts of Moraga Formation volcanics	MC	23				94	13:10
			SPT	83				56	
7.5									13:30
		(GW) SANDY GRAVEL WITH CLAY: Olive brown, very dense, subangular to subrounded fine to medium gravel in cohesive matrix, volcanic clasts of Basalt/Andesite @12'-13' (Moraga Formation), appears crushed, some ferrous oxide staining	MC	58				78	
10.0									

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GROUND WATER LEVELS:
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DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
10.0		(GW) SANDY GRAVEL WITH CLAY: Olive brown, very dense, subangular to subrouned fine to medium gravel in cohesive matrix, volcanic clasts of Basalt/Andesite @12'-13' (Moraga Formation), appears crushed, some ferrous oxide staining*(continued)*							
									14:45
12.5									
15.0			ST						
		@ 17': coarse clasts of andesite (Moraga Volcanics), inferred from shoe of tube							15:20
17.5									
									15:35
20.0									

(Continued Next Page)

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PROJECT NAME Integrative Genomics Building (IGB), Geotechnical Investigation
PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/23/14 **COMPLETED** 7/25/14
GROUND ELEVATION 710 ft **HOLE SIZE** 4.875"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
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DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
20.0		(GW) SANDY GRAVEL WITH CLAY: Olive brown, very dense, subangular to subrouned fine to medium gravel in cohesive matrix, volcanic clasts of Basalt/Andesite @12'-13' (Moraga Formation), appears crushed, some ferrous oxide staining *(continued)*	ST						7:40 (7/24/14)
		@21.5': coarse clasts of andesite (Moraga Volcanics), inferred from shoe of tube							
22.5									
25.0			ST						8:10
		GRAVELLY SAND WITH CLAY/CLASTIC SANDSTONE: Dark olive brown with black and light brown mottling, moderately strong, hard to very hard, same volcanic clasts	SPT	30				44	
		CLAYSTONE/SILTSTONE: Light grey/reddish brown b/w 25.75'-27': sandy, plastic, soft, deeply weathered							
		b/w 27'-28': mottled, plastic to friable, soft to low hardness, moderateley weathered							
27.5		@28': bedding appears subhorizontal @28.1': thin stiff clayey seam b/w 28.2'-31': mottled laminations 2"-6" thick @ 10-50 deg, plastic, soft, moderately weathered, grading to more silt and fine sand	MC	23				100	
		@29.4': very thin clay seam, very soft, @<10 deg @29.6': silty sand lense, 1" thick	RC					60	8:45
30.0									

(Continued Next Page)

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PROJECT LOCATION Berkeley, CA
DATE STARTED 7/23/14 **COMPLETED** 7/25/14
GROUND ELEVATION 710 ft **HOLE SIZE** 4.875"
DRILLING CONTRACTOR Pitcher Drilling Co.
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DEPTH (ft)	MATERIAL DESCRIPTION
30.0	CLAYSTONE/SILTSTONE: Light grey/reddish brown b/w 25.75'-27': sandy, plastic, soft, deeply weathered *(continued)* @30.5': small fault @ 70-80 deg, 2-3 cm offset, surrounded by thin laminations to 31' b/w 31'-33': friable to weak, soft to low hardness, moderately weathered @31.5': reddish seam, 20 deg dip, clay(?) @32': clay seams @ 40 deg
32.5	SILTY SANDSTONE: dark bluish grey/reddish brown, laminated with clay/claystone seams, some vertical, friable, low hardness, moderately weathered @33.7': thin clay seam @33.8': gypsum nodules, parallel to bedding @ 40 deg @34': joint @45', contact @ 25 deg sandy along contact.
	SANDY SILTSTONE/ CLAYSTONE: Light bluish gray/reddish brown, laminations and bedding @ 40 deg, plastic to friable, soft to low hardness, moderately weathered
35.0	b/w 35'-36': massive reddish brown claystone with some siltstone mottling, friable, low hardness, vertical red clay seams b/w 36'-36.5': sheared and soft @37': hard fragments at contact
37.5	SILTSTONE: Light bluish grey b/w 37'-39.2': massive, weak, soft to low hardness, moderately weathered @37.5': clay seam, 2"-3" thick, very soft, subhorizontal @<10 deg [POSSIBLE DEEP SLIDE?] @38': clay seam, 2-3mm thick @ 70 deg
40.0	

Interval (ft)	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
31-32	RC					150	10:10
32-35	RC					106	10:35
35-36	RC					100	11:10
36-37	RC		2.0 0.75			100	11:30
37-40	RC					142	11:45

(Continued Next Page)

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PROJECT NUMBER 1100-17B
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DATE STARTED 7/23/14 **COMPLETED** 7/25/14
GROUND ELEVATION 710 ft **HOLE SIZE** 4.875"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
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DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
40.0		SILTSTONE: Light bluish grey *(continued)* b/w 40'-41.3': dark grey/brown, slickensided angular to subrounded rock fragments, clayey, very soft @41.3': dark grey, rubbly, very hard siltstone clasts b/w 41.4'-42.3': light bluish grey/reddish brown, very thinly laminated, friable to weak, soft to low hardness, moderately weathered							12:15
42.5		42.3'-44.5': light bluish grey, massive siltstone, friable to weak, moderately weathered @44.5': small vertical fault, calcium carbonate concretions in fault zone b/w 44.7'-45.3': light bluish grey/reddish brown, massive, very thinly laminated, plastic to friable, soft, moderately weathered	RC					95	
45.0		@45.3': fracture with soft clay fill, deeply weathered b/w 45.5'-45.8': very soft, sheared, deeply weathered @46': thin red soft fat clay seam within massve siltstone (clay lined fracture?)							12:50
47.5		LIMESTONE: Whitish grey, very hard, fresh, weak, subhorizontal bedding, gravelly base. @47.8': Subhorizontal bedding (?) SILTSTONE: Light greenish grey, massive (unless otherwise noted) @48.6': sheared, very soft, moist @48.9': very hard siltstone/dolostone, 1"-2" thick 49'-50.2': massive, hard, no apparent bedding, possible subhorizontal laminations	RC					100	
50.0									

(Continued Next Page)

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BORING NUMBER B-5

CLIENT Lawrence Berkeley National Laboratory (LBNL)
PROJECT NAME Integrative Genomics Building (IGB), Geotechnical Investigation
PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/23/14 **COMPLETED** 7/25/14
GROUND ELEVATION 710 ft **HOLE SIZE** 4.875"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
50.0		SILTSTONE: Light greenish grey, massive (unless otherwise noted)*(continued)* b/w 50.2'-50.7': intensely fractured, hard angular fragments in soft clayey filling @50.4': very hard limstone/dolostone, 1"-2" thick b/w 50.7'-53': weak to friable, low hardness, closely fractured, moderately weathered, grading more sandy @52': vertical fracture with calcium carbonate							13:30
52.5		b/w 53'-53.7': fractures/shears @ 45 deg, coarse sandy siltstone with clayey fill, deeply weathered 53.7'-54.2': red clay/claystone zone, low hardness, soft and plastic contacts above and below at 50-60 deg b/w 54.2'-60': return to greenish grey clayey siltstone, massive, friable, very hard, moderately weathered, thin laminate subhorizontal siltstone, gypsum along fractures	RC					103	
55.0		@55': massive below @55.9': fracture with soft clayey fill, moist @56.9': fracture with soft clayey fill, moist @57': fracture @ 45 deg, open, no filling							14:35
57.5		b/w 57.5'-58.5': contact @ 60 deg, light greenish grey soft and clayey siltstone grading to darker sandy siltstone b/w 58.5'-60': greenish grey sandy siltstone, friable, soft to low hardness, moderately weathered @59.5': fracture @ 45 deg, soft	RC					97	
60.0									

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PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/23/14 **COMPLETED** 7/25/14
GROUND ELEVATION 710 ft **HOLE SIZE** 4.875"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
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DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
60.0		SILTSTONE: Light greenish grey, massive (unless otherwise noted)*(continued)* b/w 60'-65': greenish grey siltstone (no sampling, inferred from cuttings)							15:35
62.5			AU					0	
65.0									7:45 (7/25/14)

Bottom of borehole at 65.0 feet.
1. Stratification lines represent the approxiamte boundaries between material types and the transitions may be gradual.
2. Downhole geophysics logging 7/25/14.
3. Borehole grouted 7/25/14.
4. Liners were not used in the Modified California (MC) sampler.
5. MC blow counts were adjusted by multiplying field blow counts by a factor of 0.63.
6. Groundwater level was obscured by drilling method.

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BORING NUMBER B-6

CLIENT Lawrence Berkeley National Laboratory (LBNL)
PROJECT NAME Integrative Genomics Building (IGB), Geotechnical Investigation
PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/28/14 **COMPLETED** 7/29/14
GROUND ELEVATION 710 ft **HOLE SIZE** 3.875"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
0.0		ASPHALT							14:45 (7/28/14), hand-augered 0'-5'
		(GW) AGGEGATE BASE							
		(CL) SANDY LEAN CLAY WITH GRAVEL: Dark and light brown mix, stiff, moderate to high plasticity, fine to coarse sand, fine to coarse gravel, with silt, heavy oxidation and ferrous oxide staining 0.5'-3.5' [FILL]							
2.5									
5.0		b/w 5'-10': dark brown and reddish brown mottled with yellow brown, uniformly mixed, stiff [FILL]	MC	9				61	15:10 LL=48,PL=18,PI=30; <#4=78%, <#40=66%, <#200=52% Corrosivity Analysis (See Appendix C)
7.5									
10.0									

(Continued Next Page)

GEOTECH BH COLUMN TERM LEFT ALIGNED (2) - A3GEO DATA TEMPLATE.GDT - 9/30/14 10:15 - A:\A3GEO PROJECTS\1100 - LBNL\1100-17B_IGB GEOTECHNICAL INVESTIGATION\A3GEO BORING LOGS\IGB BORING LOGS.GPJ

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DATE STARTED 7/28/14 **COMPLETED** 7/29/14
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10.0		(CH) FAT CLAY b/w 10'-15': dark brown to black, firm/medium stiff, high plasticity, soft and moist fines, weathered granular clasts (Moraga Formation), some oxidation and ferrous oxide staining, moist [HOLOCENE ALLUVIUM/FILL?]	MC	8				72	15:40 LL=58,PL=18,PI=40; <#4=100%, <#40=94%, <#200=77%
12.5									
15.0		b/w 15'-20': gravelly, with fine to coarse sand, medium stiff, very high plasticity, few fine clasts from Orinda and Moraga Formations, very moist/saturated [QUATERNARY ALLUVIUM/FILL?]	SPT	6				106	7:30 (7/29/14) LL=50,PL=18,PI=32; <#4=78%, <#40=71%, <#200=58%
17.5									
20.0									

(Continued Next Page)

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PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/28/14 **COMPLETED** 7/29/14
GROUND ELEVATION 710 ft **HOLE SIZE** 3.875"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
20.0		(CH) FAT CLAY *(continued)* b/w 20'-25': greyish brown mottled with yellowish brown, stiff, trace fine angular clasts, massive, moderate to high plasticity, some oxidation and ferrous oxide staining, moist	SPT	21				72	7:45
22.5									
25.0		b/w 25'-30.5': reddish brown and yellowish brown, alternating, firm to medium stiff, high plasticity, trace fine clasts, massive, saturated	SPT	12				94	7:55
27.5									
30.0									

(Continued Next Page)

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BORING NUMBER B-6

CLIENT Lawrence Berkeley National Laboratory (LBNL)
PROJECT NAME Integrative Genomics Building (IGB), Geotechnical Investigation
PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/28/14 **COMPLETED** 7/29/14
GROUND ELEVATION 710 ft **HOLE SIZE** 3.875"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
NOTES
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DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
30.0		(CH) FAT CLAY *(continued)*							8:05
		CLAYSTONE [ORINDA FORMATION] b/w 30.5'-35': very dark grey, very plastic, soft, deeply weathered, numerous polished and slickensided fractures, some fine and hard claystone clasts mixed in, friable in sections, moist	SPT	25				67	
32.5									
35.0		b/w 35'-40': grey, plastic to friable, soft, moderately weathered, platey partings, no biotite or muscovite, trace oxidation and ferrous oxide staining, dense and competent, dry	MC	54/10"				80	8:25
37.5									
40.0									

(Continued Next Page)

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DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
40.0		CLAYSTONE [ORINDA FORMATION]*(continued)* b/w 40'-50': grey, plastic to friable, soft, deeply weathered, intensely fractured, soft fractures at 40.3' & 40.7', common partings, no biotite or muscovite, interlocking structure, trace oxidation, dry to moist @40.8': light grey seam	SPT	27				67	8:45
42.5									
45.0			SPT	61				39	9:00
47.5									
50.0									

(Continued Next Page)

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DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
50.0		CLAYSTONE [ORINDA FORMATION]*(continued)* b/w 50'-51.2': plastic, soft, deeply weathered, intensely fractured, gypsum-lined thin fractures that are polished and slickensided, massive with common partings between fractures	SPT	28				94	9:25
		SANDSTONE: Silty and very fine grained, moderately hard, friable, calcium carbonate cementation (Recovered from shoe of sampler)							

Bottom of borehole at 51.5 feet.
1. Stratification lines represent the approxiamte boundaries between material types and the transitions may be gradual.
2. Borehole grouted immediately upon completion.
3. MC blow counts were adjusted by multiplying field blow counts by a factor of 0.63.
4. Groundwater level was obscured by drilling method.

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BORING NUMBER B-7

CLIENT Lawrence Berkeley National Laboratory (LBNL)
PROJECT NAME Integrative Genomics Building (IGB), Geotechnical Investigation
PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/28/14 **COMPLETED** 7/28/14
GROUND ELEVATION 710 ft **HOLE SIZE** 3.875"
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GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
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LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
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AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
0.0		ASPHALTIC CONCRETE							8:50 (7/28/14), hand-augered 0'-3'
		AGGREGATE BASE							
		(SC) CLAYEY GRAVEL WITH SAND: Dark olive brown, medium dense, fine to coarse sand, fine to coarse gravel, subangular to subrounded gravel up to 2", mostly fine gravel content, moderately plastic fines, some silt, mix of soil and rock fragments [FILL]							
2.5									
			SPT	11				50	9:15 LL=44,PL=20,PI=24; <#4=67%, <#40=52%, <#200=38%
5.0		(CL) - loose, angular to subangular gravel							
			MC	7				39	9:35
									Corrosivity Analysis (See Appendix C)
7.5									
		b/w 9'-10.5': very loose to loose, grading more sandy							9:50
10.0			MC	4				44	

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DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
10.0									
		(SC) CLAYEY SAND: Light olive brown, loose, cohesive, well graded, some subangular to angular gravel up to 1.5", moist [FILL]	SPT	10				56	9:55 LL=49,PL=25,PI=24; <#4=51%, <#40=31%, <#200=21%
12.5									
		(CL)							
			MC	8				67	10:10
15.0		(CH) FAT CLAY: Yellowish brown, medium stiff to stiff, moderate plasticity, with fine sand, some oxidation [NATIVE ALLUVIUM]							LL=63,PL=21,PI=42; <#4=100%, <#40=96%, <#200=85%
17.5									
		(CL) SANDY LEAN CLAY: Light olive brown, stiff to very stiff, low plasticity, fine sand, trace fine gravel clasts (Moraga Formation), very oxidized and ferrous oxide stained, some rock structure, some fat clay pockets [NATIVE CHANNEL ALLUVIUM/FILL??]	MC	26				56	10:25
20.0									

(Continued Next Page)

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20.0		(CL) SANDY LEAN CLAY: Light olive brown, stiff to very stiff, low plasticity, fine sand, trace fine gravel clasts (Moraga Formation), very oxidized and ferrous oxide stained, some rock structure, some fat clay pockets [NATIVE CHANNEL ALLUVIUM/FILL??]*(continued)*	SPT	14				56	
22.5									
		LEAN CLAY: Dark grey, stiff, moderate plasticity, moist							10:50
25.0			MC	13				67	
		- grading to weak and soft claystone with polished partings							
27.5									
									11:00
			MC	35/11"				53	
		CLAYSTONE: Dark grey b/w 29'-32': friable to moderately plastic, angular and very hard claystone to shale-like fragments and clasts up to 2", some fine sand, (stuck on large clast?, clast from shoe appears to be basalt or very dirty grey sandstone)							
30.0									

(Continued Next Page)

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DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
30.0		CLAYSTONE: Dark grey*(continued)*							
32.5		b/w 32'-36': friable to weak, very hard, fine shale-like partings, calcium carbonate partings, deeply weathered, very stiff/dense, competent	MC	47/11"				56	11:15
35.0		b/w 36'-40': very dark grey to black, friable, moderately hard, deeply weathered, very fine grained and shale-like, no partings but conchoidal, little polished slickensiding, very competent and hard	MC	35/5"				120	11:30
37.5									
40.0									

(Continued Next Page)

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DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
40.0		SANDY SILTSTONE: Grey to dark grey, very hard and resistant, very dense, very competent, no biotite or muscovite (closer in origin to Orinda Formation)	MC	35/3"				67	11:50
42.5									
45.0			MC	35/1"				0	12:10
47.5									
50.0			SPT	34/1"				100	

(Continued Next Page)

Bottom of borehole at 50.0 feet.

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DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
									12:35

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BORING NUMBER B-8

CLIENT Lawrence Berkeley National Laboratory (LBNL)
PROJECT NAME Integrative Genomics Building (IGB), Geotechnical Investigation
PROJECT NUMBER 1100-17B
PROJECT LOCATION Berkeley, CA
DATE STARTED 7/29/14 **COMPLETED** 7/29/14
GROUND ELEVATION 725 ft **HOLE SIZE** 3.875"
DRILLING CONTRACTOR Pitcher Drilling Co.
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** JNB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ROCK RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
0.0		(SW) GRAVELLY SAND: Yellowish brown, loose to medium dense, well graded, trace fines, gravel from 1/4" to 3/4", dry [FILL]							11:30 (7/29/14), hand-augered 0'-5'
		(CL) GRAVELLY LEAN CLAY: Olive brown, stiff, some fine sand, low plasticity, gravel up to 1.5", deep ferrous oxide staining, damp [FILL]							
2.5		- grading less gravel							
5.0			SPT	19				61	12:10
		(CL) SANDY LEAN CLAY: Dark brown to black, stiff, fine sand, low plasticity, mottled with olive brown clay, some ferrous oxide staining, moist [FILL]							
7.5									
10.0									

(Continued Next Page)

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10.0		(CH) FAT CLAY: Mixed and mottled reddish brown and olive brown and dark brown, stiff, high plasticity, deeply weathered, ferrous oxide staining, moist [FILL] (contact below inferred)	SPT	10				72	12:20
12.5									
		SANDSTONE: Very dark grey, very fine grained, some silt, friable to weak, low hardness, partly cemented, moderately weathered (possibly just a block derived from an Orinda fomation unit in fill or siltstone layer) [ORINDA FORMATION]							
15.0			MC	39				78	12:40
		CLAYSTONE/SILTSTONE b/w 16.25'-19.5': very dark grey, fine sand, friable, soft to low hardness, deeply weathered, platey partings, likely dolostone/limestone content causing for hard drilling throughout 16.25'-19.5' [ORINDA FORMATION]							
17.5									
		b/w 19.5'-25': dark brown to black, very plastic, soft, deeply weathered, intensely fractured, dark bluish grey clasts of limestone and gypsum clasts							
20.0									

(Continued Next Page)

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20.0		CLAYSTONE/SILTSTONE *(continued)*	SPT	17				61	13:00
22.5									
25.0		LIMESTONE/DOLOSTONE: Dark grey to grey, hard fragments, gypsum lined fractures (highly disturbed/crushed in sampling)	MC	35/6"				133	13:15
		CLAYSTONE/SILTSTONE b/w 25.5'-30': dark grey, very friable, soft, deeply weathered (inferred/projected from sample at 30')							
27.5									
30.0									

(Continued Next Page)

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30.0		CLAYSTONE/SILTSTONE *(continued)* b/w 30'-35': dark grey, very friable soft, deeply weathered, platey partings, calcium carbonate and gypsum-lined fractures	SPT	50/1"				300	13:30
32.5									
35.0		b/w 35'-40': grey/reddish brown, laminated, plastic to friable, soft, platey partings, abundant polished and slickensided fractures	SPT	53				83	13:45
37.5									
40.0									

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40.0		CLAYSTONE: Dark greyish brown, with fine sand, friable to weak, soft to low hardness, deeply weathered, intensely fractured with random calcium carbonate lining and polishing, possible dolomite fragment in shoe	MC	53/11"				64	14:05
42.5									
45.0		CLAYSTONE/SILTSTONE: Bluish grey/reddish brown, laminated/alternating beds, plastic to friable, soft to low hardness, moderately weathered, abundant platey partings, slickensided facings and fractures with ferrous oxide staining and calcium carbonate and gypsum lining	SPT	81				67	14:25
47.5									
50.0									

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50.0		CLAYSTONE/SILTSTONE: Bluish grey/reddish brown, laminated/alternating beds, plastic to friable, soft, deeply weathered, abundant platey partings, fractures with ferrous oxide staining and calcium carbonate and gypsum lining, dry	SPT	35/0.0"				100	14:45
		SILTSTONE: Greenish grey, soft, friable, deeply weathered, platey partings, dry (from shoe of sampler)							

Bottom of borehole at 51.0 feet.
1. Stratification lines represent the approxiamte boundaries between material types and the transitions may be gradual.
2. Borehole grouted immediately upon completion.
3. MC blow counts were adjusted by multiplying field blow counts by a factor of 0.63.
4. Groundwater level was obscured by drilling method.

Appendix B

NORCAL Geophysical Logging Investigation Report

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

August 25, 2014

Mr. Wayne Magnusen
A3GEO, Inc.
1331 Seventh St., Unit E
Berkeley, CA 94710

Subject: Borehole Geophysical Logging Investigation
Lawrence Berkeley National Laboratory
Integrated Genome Building
Berkeley, California
NORCAL Job No. 14-1080.02B

Attention: Wayne Magnusen, P.E.

This report summarizes the findings of a follow up borehole geophysical investigation performed by NORCAL Geophysical Consultants, Inc. at the subject site for A3GEO, Inc. The investigation was conducted on various single day site visits spanning the time period July 17 through 25, 2014 by NORCAL Professional Geophysicist William J. Henrich (PGp No. 893). Mr. Robert Speidel Project Engineer of A3GEO provided background information, coordination and on-site logistical support.

The purpose of the borehole geophysical investigation was to measure P- and S-wave velocities and map borehole discontinuities within local sandstone, siltstone and mudstone bedrock. These data will be used by others to assess bedrock characteristics that may affect slope stability within the project footprint.

1.0 SCOPE

Geophysical borehole logging was conducted in three boreholes labeled as B-2, -4 and -5. The geophysical logging methods consisted of suspension P- and S-wave velocity profiling, acoustic televiewer (BHTV) and caliper logging.

2.0 BOREHOLE CONDITIONS

Within the bedrock section, all boreholes were advanced with an HQ rotary diamond core method. The bedrock consisted of poorly consolidated, moderate to highly weathered, fractured marine sandstone, siltstone with thin mudstone interbeds. The boreholes contained a steel conductor casing ranging from 4- to 28-ft below ground surface (bgs) as a procedure to seal off fill, alluvium and highly weathered bedrock materials. Total depths of the boreholes ranged from 65 to 81-ft bgs. Borehole stability varied from good as in no caving or sloughing to very poor. The latter condition was noted in Borehole B-4 which as a result of several episodes of formation squeezing required the borehole to be reamed out to 6-inch in diameter.

3.0 BOREHOLE GEOPHYSICAL LOGGING METHODOLOGY

Complete descriptions of the methodology, data acquisition, data analysis procedures and results for the suspension P- and S-wave and televiewer logging are presented in Appendices A and B, respectively. Specific survey data-log plots for each of these logging methods are presented at the end of each Appendix as well as other supporting tables and illustrations.

Caliper logs are a measure of the borehole diameter versus depth. The tool was used both as a survey technique to assess borehole stability and quantify the relative consolidation of bedrock. The caliper tool consists of three interconnected mechanical arms that are spring loaded against the borehole wall. The horizontal deflections of the arms gauge the borehole diameter in units of inches with depth. The logging measurement was made in the uphole direction at a speed of approximately 10-ft per minute. The data sampling rate for this instrument was every 0.2-ft.

NORCAL conducted the borehole geophysical investigation using a digital ***Robertson Geologging,* Ltd.** Model ***MICROLOGGER2 System***. This system consisted of a control console, a computer, the logging tools, and a winch. The borehole logging tools consisted of an ***Oyo-Robertson Geologging,* Ltd.** Suspension P- and S-wave velocity tool, acoustic televiewer (BHTV) and a mechanical three-arm caliper.

4.0 INTERPRETATION and DISCUSSION

1) Suspension P- and S-wave Velocity Profiles

The results of our Suspension P- and S-wave Velocity Profiles are presented in Appendix A, labeled as A-1 through A-3. On the basis of interval velocities, the P- and S-wave velocities for all three boreholes ranged from 4000 to 8000 feet per second (fps) and 2000 to 3500 fps, respectively. Borehole B-4 of the survey showed the greatest velocity variability (see Suspension P- and S-wave Velocity Profiles, Appendix A, A-2) with depth. This was due to the presence of a relatively high velocity layer from 45- to 60-ft bgs. In general, the average S-wave velocity in consideration of all boreholes was approximately 2200 fps; the average P-wave velocity was approximately 7000 fps.

No P-wave energy was detected in the upper section of Borehole B-2 (see velocity profile in Appendix A, A-1). We often see this effect in unsaturated (above the static water table) fractured rock. Even though the fluid column was created by the addition of water during the borehole survey, it is possible that some air is entrapped in the fractures. If the secondary void spaced within the fracture contains 10 percent by volume air the P-wave can be attenuated. Because of the transverse nature of propagation, the S-wave propagation is unaffected.

2) Televiewer Discontinuity Analysis

Results of our identification and Dip (Dip azimuth and magnitude) of borehole discontinuities are presented in Appendix B as BHTV Discontinuity Analysis Plots, Boreholes B-2, -4 and -5. These image plots show the bedrock discontinuities classified as discontinuous fractures, open fractures, "crushed/shear" zones and bedding. In general, the number of discontinuities that could be identified from the BHTV image logs was limited. This was due to poor or scattered returned acoustic signals due to the high degree of fracturing, weathering and poor consolidation of most of the bedrock.

5.0 STANDARD CARE

The scope of NORCAL's services for this project consisted of using geophysical logging techniques to measure P- and S-wave velocities and map borehole discontinuities. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the level of skill ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

We appreciate the opportunity to provide our services to A3GEO, Inc. for this project. If you have any questions, or require additional geophysical services, please do not hesitate to call on us.

Sincerely,

NORCAL Geophysical Consultants, Inc.

William J. Henrich PGp
Professional Geophysicist-893

Enclosures: Appendix A: Suspension P- and S-Wave Logging
Appendix B: Televiewer Logging

Appendix A:

Suspension P- and S- Wave Logging

APPENDIX A

SUSPENSION P- AND S-WAVE VELOCITY SURVEY

The Suspension tool is a highly specialized downhole methodology that measures P- and S-wave velocities at discrete depths. The following presents a narrative on its operation, data reduction procedures, velocity profiles and complete velocity data table.

1) Methodology

We measured downhole compressional (P-) and shear (S-) wave velocities using an OYO-Robertson Model 3403 digital suspension logging system. The tool is equipped with a dipole seismic energy source located near the base of the probe and a pair of geophones (detectors R-1 and R-2) located within the middle to the upper section of the probe. A schematic diagram depicting the probe configuration and equipment attachments is shown in Figure 1. The distance from the energy source to the first geophone was 10.3-ft (3.14 meters) when assembled with a detachable 2-meter isolation tube. The in-line distance between the geophone pair is 3.28-ft (1.0 meter). Each geophone contains one horizontal and one vertical oriented element. The horizontal phone elements preferentially record the shear wave. The vertical geophone elements record first arriving P-wave energy.

Suspension seismic data are collected at discrete depths in the fluid-filled portion of the borehole. At each measurement depth, the energy source is activated via commands from the surface control console. This activation causes a metal solenoid to strike a plate (anvil) mounted inside the probe housing. This energy transmits through the fluid to the borehole wall which produces a seismic wave ("flexure") in the adjacent formation. As this wave propagates radially into the formation a seismic interaction between the seismic wave and the borehole wall creates tube waves together with a refracted compressional P-wave that travels up the borehole to the two recording geophones.

When assembled with a 2-meter isolation tube, the suspension logging tool measures approximately 23-ft in length (Figure 1). The measuring point of the tool is taken at the center of the pair of receiver geophones. This measuring point is approximately 15-ft from the probe tip. Therefore, the maximum depth of our survey given a non-sloughing borehole will always be reported 15-ft less than the total depth of the borehole.

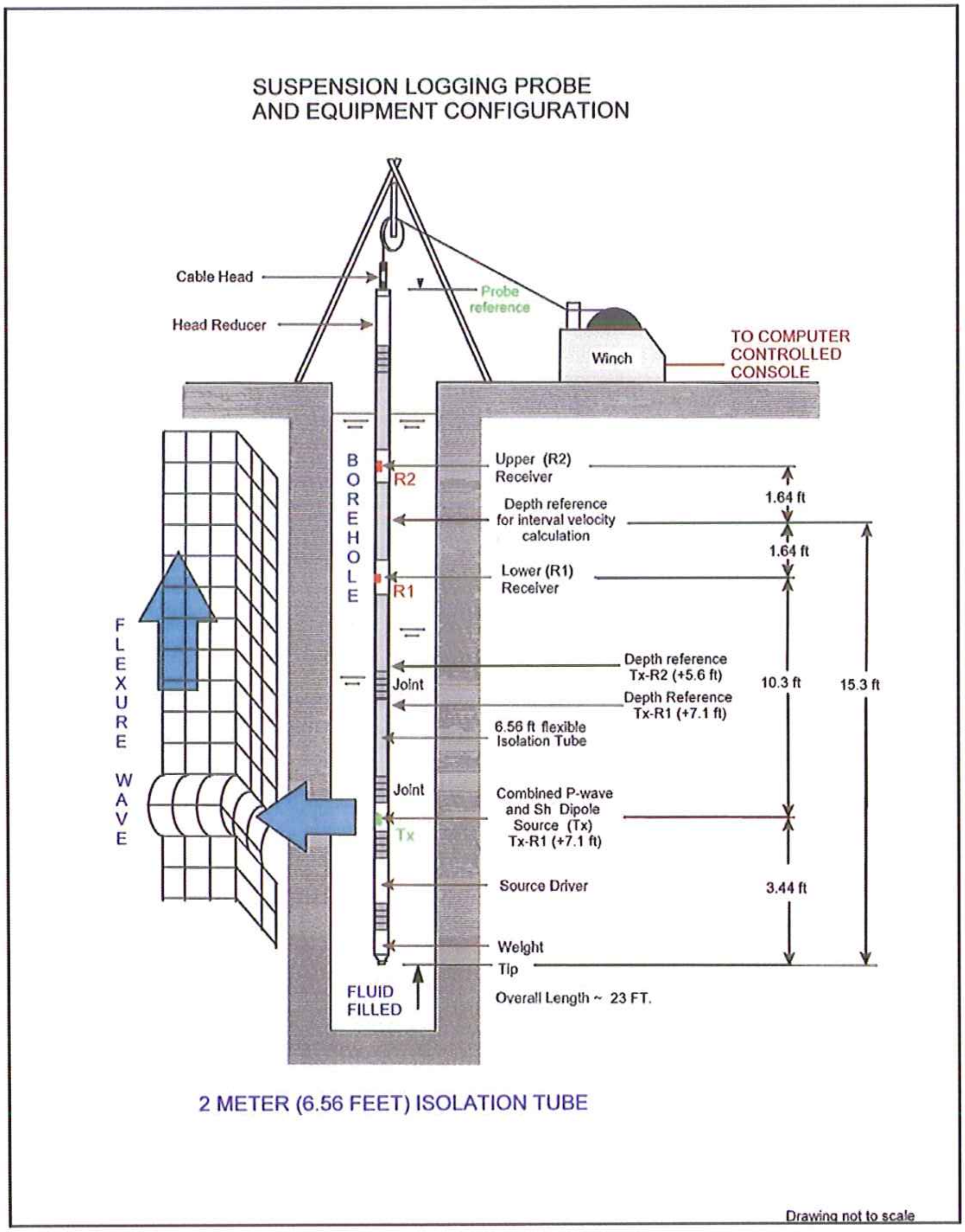


Figure 1: Schematic of suspension logging tool.

2) Data Acquisition

Typically, we measured seismic suspension velocities at stationary 1.0- to 1.5-ft intervals. The surveys began at the bottom of each borehole interval and proceeded up the borehole to the tip of the drill rod casing shoe or HWT steel casing. At each measurement station, we cycled the energy source to fire 2 times in succession into each of the geophone elements. This cycling stacks the seismic energy resulting in an improved signal-to-noise ratio. We also recorded S-wave data using a 1.2 KHz low pass filter. This filtering reduces high frequency interference from the onset of earlier arriving P-wave energy. We recorded P-wave waveforms using a 10 KHz low pass filter. At some measurement depths, we made essentially duplicate records by offsetting the depth by 0.1 feet. This was performed for one of two reasons: 1) determine repeatability and 2) modify recording times and/or stacking number to improve the waveform record at that depth position.

3) Data Analysis

Suspension P- and S-wave velocities were calculated with the interpretation computer software ***Glog SUS***, Version 1.12 published by ***Oyo Corporation*** (2000). An example suspension waveform interpretation of arrival times and velocity determination is presented in Figure 2. The example records are from Borehole CFR-4. Each record shows six detector (geophone) traces. The upper four traces are related to horizontal detector elements labeled R1 and R2. The red traces result from a left strike or impact of the dipole source (anvil) to the probe housing (cycle 1); the green traces result from a right strike (cycle 2) of the dipole source. By superimposing and pairing the respective left and right strike detector traces, phase reversals associated with the arrival times of the S-wave energy can be identified. The lower two traces (blue color) are related to the vertical detector elements which are preferentially aligned to record P-waves. With P-wave energy, the direction of the dipole strike can be in either direction. P-wave arrival times are determined by noting the first breaks on the set of vertical detector traces. Note that at a minimum, a complete suspension waveform record requires at least three recording cycles.

All seismic waveform records were analyzed for P- and S-wave arrival times in this manner. Interval seismic P- and S-wave velocities in meters per second are calculated by dividing the detector spacing (R1-R2 spacing = 1 meter) by the difference in interpreted arrival times in microseconds. Two separate S-wave velocities (dipole source striking left then right) are calculated at each depth measurement station. We averaged the results of these two S-wave interval velocities and presented a single S-wave value at each measurement station.

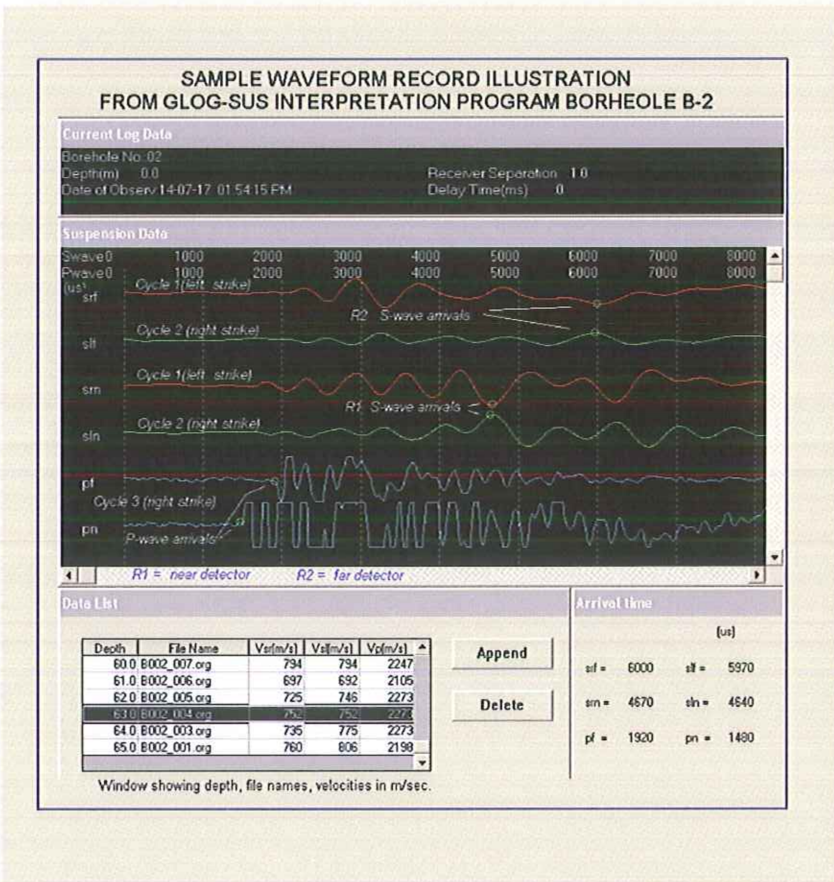


Figure 2 : Sample record showing P- and S- waveforms

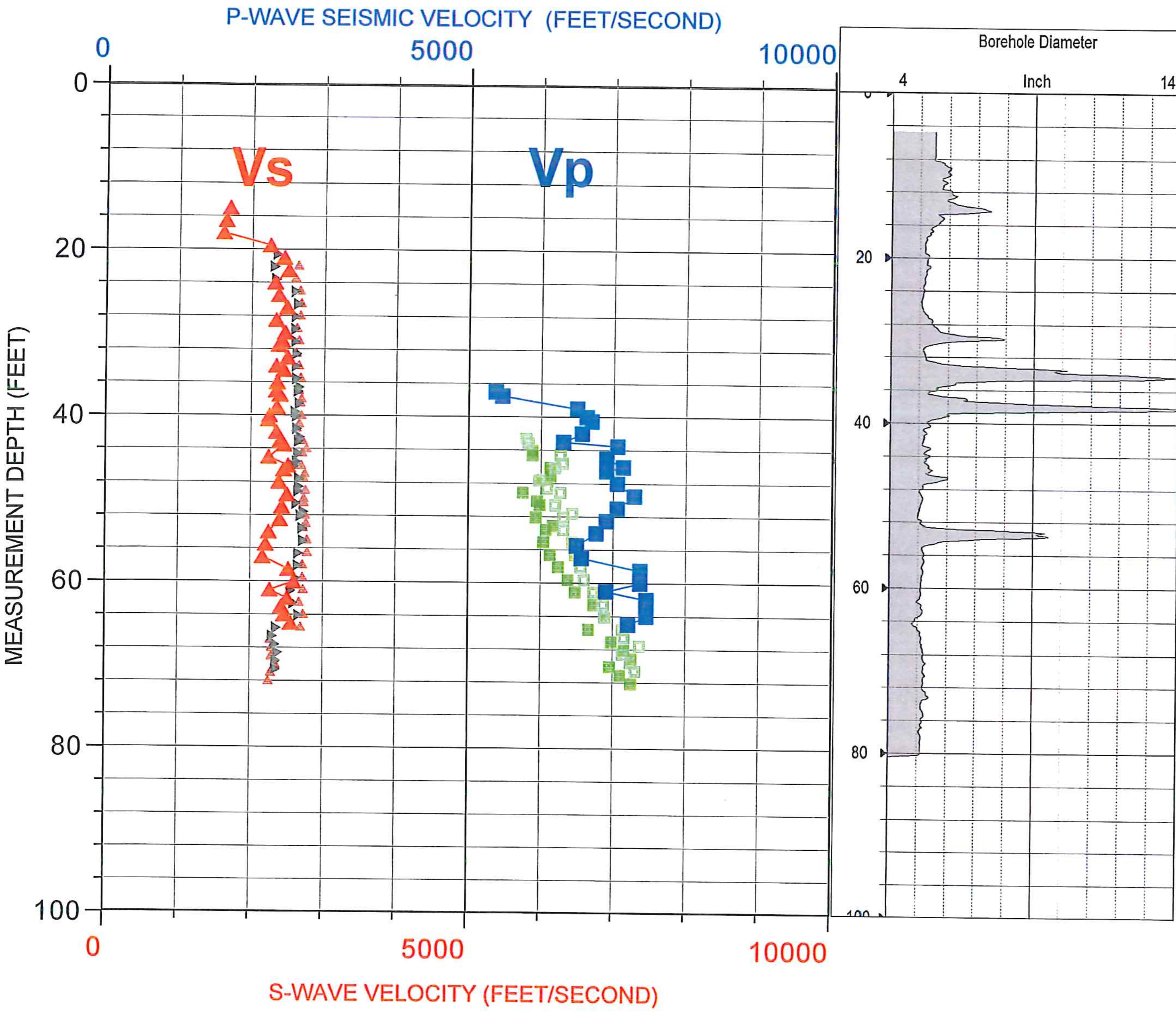
As an internal data analysis check, we also computed direct P-wave and S-wave velocities from the interpreted arrival time data. These "direct" velocities are determined by taking the in-line distances from source (Tx) to lower (R1) and upper (R2) detectors and dividing by respective P- and S-wave arrival times. The very small time delays due to offset distances from the source to the borehole wall and borehole wall to detectors are neglected. Note that the depth references for direct velocities are taken as the mid-point between the source and successive detectors. As a consequence, reference depths of the direct velocity computations will always plot several feet lower than the depths of the interval velocities. The interval and direct velocities are then comparatively plotted on depth versus velocity graphs. When significant velocity variations are noted between the different computations of the respective P- and S-waves, we reinterpret the arrival times within the ***Glog-SUS*** program so that the final interpreted interval velocities, to the extent permissible by the detector response, converges more closely to the trends and magnitude of direct velocities.

In the very upper sections of some boreholes, P-wave arrivals were uncertain (e.g. B-2. It may be the case that in these instances that the uppermost section of bedrock is highly weathered and unsaturated and thus may have P-wave velocities that are less than the borehole fluid (approximately 5000 fps). It is our experience that in this borehole environment the P-wave response appears attenuate and in most cases does not register on the far detector. Therefore, we no do report P-wave velocities in borehole sections with under these recording conditions.

4) Results

The results of the Suspension Log P- and S-wave surveys are illustrated by the plots that follow this appendix labeled A-1 through A-3. The results include all sources to near and far detector P- and S-wave velocity combinations (see Legend to distinguish the various symbols denoting interval and direct velocities). We have highlighted the interval P- and S-wave velocities on the profiles (see red and blue colored symbols) as these velocities should be used to calculate elastic moduli values for the subsurface layering. This is because the interval velocity method compensates for any delayed arrival time errors and stand-off ray paths and therefore is the most accurate. Caliper logs have been plotted to the right of the velocity profile. This comparison illustrated the correlation of relatively low velocities to borehole enlargements and conversely, intact borehole wall to relatively higher velocities.

Data tabulation, in terms of depth, arrival times and various derived interval direct velocities follow Plots A-1 through A-3 as a series of tables (Suspension Velocity Tables B-2, -4 and -5).



P- & S-WAVE VELOCITY LEGEND

- *Vs- R1-R2 interval
- Vs- Tx-R1 direct
- Vs- Tx-R2 direct
- *Vp- R1-R2 interval
- Vp- Tx-R1 direct
- Vp- Tx-R2 direct

*Interval velocities should be used to calculate elastic moduli values

NORCAL

SUSPENSION P- AND S-WAVE VELOCITY PROFILE BOREHOLE B-2

LOCATION: LBL INTEGRATED GENOME BUILDING

CLIENT: A3GEO, Inc.

JOB #: 14-1080.02B

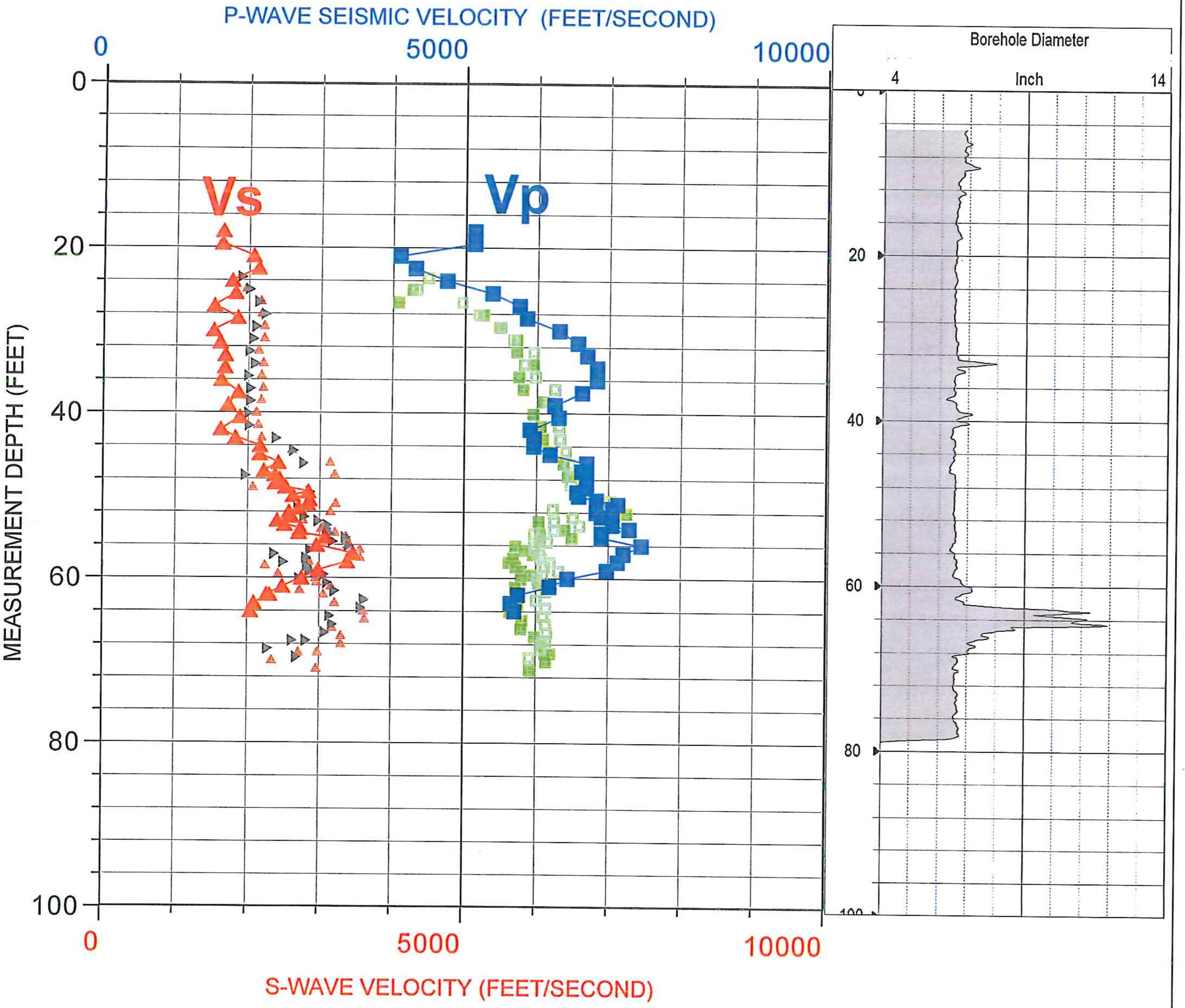
NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: AUG. 17, 2014

DRAWN BY: W HENRICH

APPROVED BY: WJH

A-1



P- & S-WAVE VELOCITY LEGEND

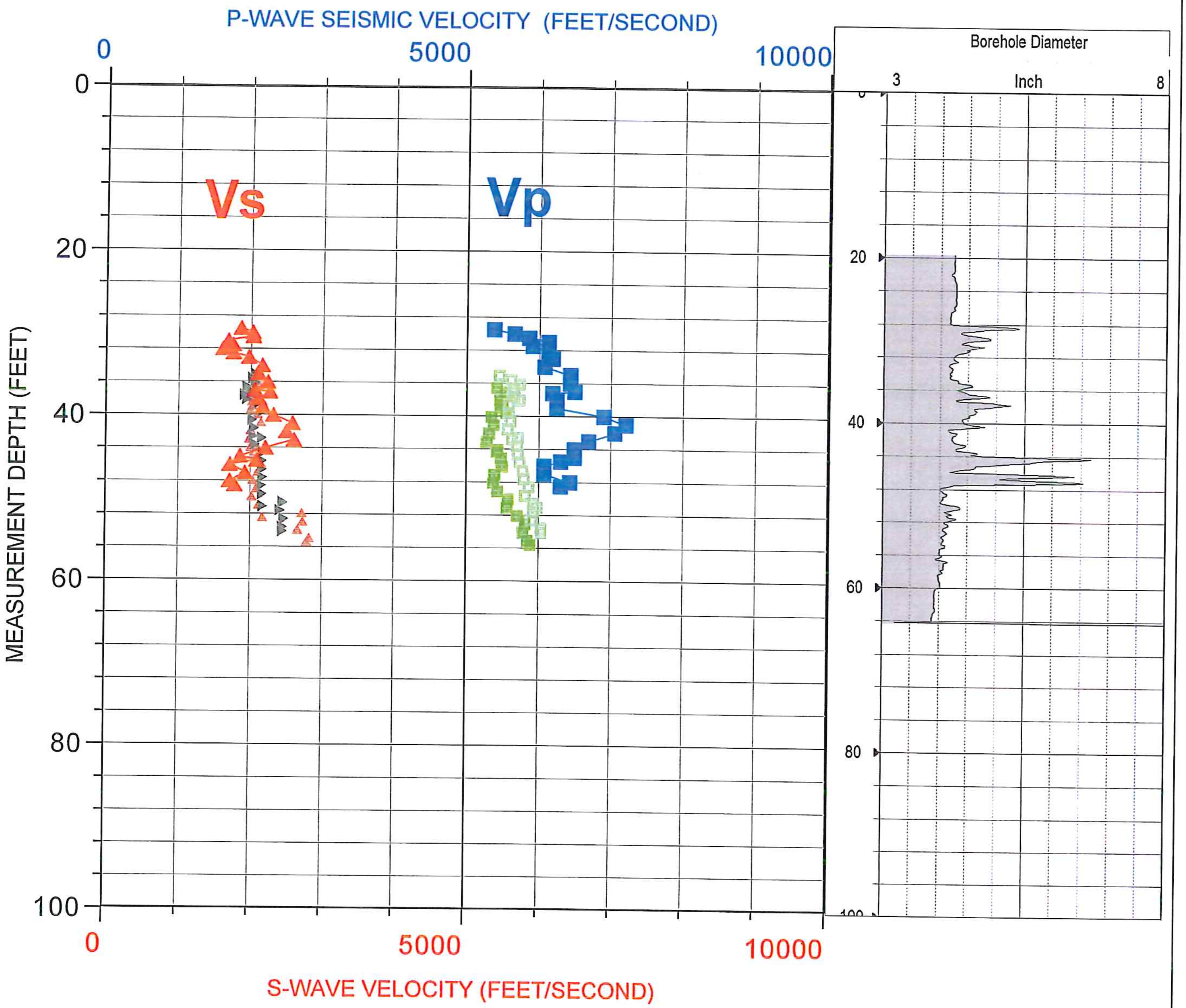
- *Vs- R1-R2 interval
- Vs- Tx-R1 direct
- Vs- Tx-R2 direct
- *Vp- R1-R2 interval
- Vp- Tx-R1 direct
- Vp- Tx-R2 direct

*Interval velocities should be used to calculate elastic moduli values

NORCAL

	SUSPENSION P- AND S-WAVE VELOCITY PROFILE BOREHOLE B-4
	LOCATION: LBL INTEGRATED GENOME BUILDING
	CLIENT: A3GEO, Inc.
JOB #: 14-1080.02B	NORCAL GEOPHYSICAL CONSULTANTS INC.
DATE: AUG. 17, 2014	DRAWN BY: W HENRICH / APPROVED BY: WJH

A-2



P- & S-WAVE VELOCITY LEGEND

- *Vs- R1-R2 interval
- Vs- Tx-R1 direct
- Vs- Tx-R2 direct
- *Vp- R1-R2 interval
- Vp- Tx-R1 direct
- Vp- Tx-R2 direct

*Interval velocities should be used to calculate elastic moduli values

NORCAL

SUSPENSION P- AND S-WAVE VELOCITY PROFILE BOREHOLE B-5

LOCATION: LBL INTEGRATED GENOME BUILDING

CLIENT: A3GEO, Inc.

JOB #: 14-1080.02B

NORCAL GEOPHYSICAL CONSULTANTS INC.

A-3

DATE: AUG. 17, 2014

DRAWN BY: W HENRICH

APPROVED BY: WJH

SUSPENSION LOG DATA AND VELOCITY TABLE BOREHOLE B-2, LBNL, INTEGRATED GENOME BUILDING

DEPTH	Vs ARRIVAL TIMES				Vp ARRIVAL TIMES		INTERVAL VELOCITIES					DIRECT TRAVEL VELOCITIES				Offset Depth References	
(ft bgs)	R2-right (us)	R1-right (us)	R2-left (us)	R1-left (us)	R2 (us)	R1 (us)	Vs-rt (m/s)	Vs-left(m/s)	Vp (m/s)	Vs-ave (fps)	Vp (fps)	Vs Tx-R2 (fps)	Vs Tx-R1(fps)	Vp Tx-R1(fps)	Vp Tx-R2 (fps)	Tx-R1	Tx-R2
15	6045	4080	6045	4150			509	528		1701		2641	2364			22.1	20.6
16.5	6205	4165	6150	4205			490	514		1647		2604	2322			23.6	22.1
18	6210	4125	6095	4110			480	504		1614		2668	2344			25.1	23.6
19.5	5530	4025	5495	4090			664	712		2257		2682	2612			26.6	25.1
21	5470	4105	5425	4105			733	758		2446		2672	2647			28.1	26.6
22.5	5415	4105	5480	4175			763	766		2508		2624	2620			29.6	28.1
24	5510	4050	5495	4130			685	733		2326		2655	2612			31.1	29.6
25.5	5470	3995	5480	4185			678	772		2379		2618	2620			32.6	31.1
27	5415	4105	5455	4130			763	755		2490		2655	2632			34.1	32.6
28.5	5455	4035	5470	4090			704	725		2344		2682	2625			35.6	34.1
30	5400	4065	5455	4125			749	752		2462		2658	2632			37.1	35.6
31	5435	4040	5380	4070			717	763		2428		2696	2671			38.1	36.6
31.5	5410	3970	5410	4090			694	758		2382		2682	2655			38.6	37.1
33	5360	4050	5380	4070			763	763		2503		2696	2671			40.1	38.6
34	5455	4040	5485	4110			707	727		2352		2668	2617			41.1	39.6
34.5	5425	4020	5445	4165			712	781		2449		2631	2637			41.6	40.1
36	5370	3985	5435	4040			722	717		2361		2718	2642			43.1	41.6
37	5380	3975	5350	3965	2360	1750	712	722	1639	2352	5377	2773	2686	5886	5805	44.1	42.6
37.5	5445	4050	5390	4050	2350	1750	717	746	1667	2400	5469	2711	2665	5886	5830	44.6	43.1
39	5400	3955	5400	4065	2185	1680	692	749	1980	2364	6496	2700	2660	6131	6270	46.1	44.6
40	5380	3920	5435	3995	2175	1680	685	694	2020	2262	6627	2750	2642	6131	6299	47.1	45.6
40.5	5490	4040	5490	4020	2210	1720	690	680	2041	2247	6696	2732	2615	5988	6199	47.6	46.1
42	5370	3980	5370	3980	2290	1790	719	719	2000	2359	6562	2761	2676	5754	5983	49.1	47.6
43	5370	4010	5370	4010	2250	1730	735	735	1923	2411	6309	2739	2676	5954	6089	50.1	48.6
43.5	5275	3980	5385	4010	2185	1720	772	727	2151	2459	7057	2739	2668	5988	6270	50.6	49.1
45	5440	3985	5440	3985	2210	1735	687	687	2105	2254	6906	2758	2640	5937	6199	52.1	50.6
46	5340	4020	5255	3965	2130	1670	758	775	2174	2515	7133	2773	2737	6168	6432	53.1	51.6
46.5	5305	3930	5320	4025	2170	1695	727	772	2105	2459	6906	2728	2702	6077	6313	53.6	52.1
48	5340	3930	5275	3945	2170	1705	709	752	2151	2397	7057	2788	2726	6041	6313	55.1	53.6
49.5	5215	3915	5255	3940	2130	1680	769	760	2222	2508	7290	2791	2737	6131	6432	56.6	55.1
51	5370	4025	5370	4025	2115	1650	743	743	2151	2438	7057	2728	2676	6242	6478	58.1	56.6
52.5	5385	4025	5385	4025	2090	1615	735	735	2105	2411	6906	2728	2668	6378	6555	59.6	58.1
54	5445	3975	5445	4010	2075	1590	680	697	2062	2259	6765	2739	2637	6478	6602	61.1	59.6
55.5	5535	4065	5575	4090	2035	1530	680	673	1980	2220	6496	2682	2573	6732	6732	62.6	61.1
57	5530	4005	5495	4005	1995	1495	656	671	2000	2177	6562	2743	2612	6890	6867	64.1	62.6
58.5	5320	4025	5355	4060	1990	1545	772	772	2247	2533	7372	2703	2684	6667	6884	65.6	64.1
60	6010	4750	6010	4750	1920	1475	794	794	2247	2605	7372	2289	2378	6983	7135	67.1	65.6
61	6140	4705	6155	4710	1915	1440	697	692	2105	2279	6906	2309	2320	7153	7154	68.1	66.6
62	5990	4720	6060	4720	1860	1420	787	746	2273	2515	7457	2304	2358	7254	7366	69.1	67.6
63	6000	4670	5970	4610	1920	1480	752	735	2273	2439	7457	2362	2395	6959	7135	70.1	68.6
64	6080	4720	6020	4730	1890	1450	735	775	2273	2477	7457	2299	2374	7103	7249	71.1	69.6
65	6065	4750	6040	4800	1875	1420	760	806	2198	2569	7211	2264	2366	7254	7307	72.1	70.6

Vs & Vp Interval Velocities

COLUMN HEADER LEGEND

DEPTH: Reference point of the Interval Velocity Measurement

see red triangle & blue squares on graph

Vs ARRIVAL TIMES: Shear Wave

R2-right (u: Upper Detector arrival time in micro seconds due to right side strike of the dipole source
R1-right (u: Lower Detector arrival time in micro seconds due to right side strike of dipole source
R2-left (us) Upper Detector arrival time in micro seconds due to left side strike of the dipole source
R1-left (us) Lower Detector arrival time in micro seconds due to left side strike of dipole source

Vp ARRIVAL TIMES: Compression P-Wave

R2 (us): Upper Detector arrival time in micro seconds
R1 (us): Lower Detector arrival time in micro seconds

INTERVAL VELOCITY:

Vs-rt (m/s) Interval Shear wave velocity derived from right dipole strike in Meters/sec.
Vs-left(m/s Interval Shear wave velocity derived from left dipole strike in Meters/sec.
Vp (m/s): Interval P-wave velocity in Meters/sec.
Vs-ave.(fps Average interval Shear wave velocity in feet/sec.
Vp (fps): Interval P-wave velocity in feet/sec.

DIRECT TRAVEL VELOCITIES:

Vs Tx-R2(fp Shear wave velocity = inline distance from source to upper detector divided by travel time measurement at the upper detector
Vs Tx-R1(fp Shear wave velocity = inline distance from source to lower detector divided by travel time measurement at the lower detector
Vp Tx-R1(fp P-wave velocity = inline distance from source to the lower detector divided by travel time measurement at the lower detector

SUSPENSION LOG DATA AND VELOCITY TABLE BOREHOLE B-4, LBNL, INTEGRATED GENOME BUILDING

DEPTH	Vs ARRIVAL TIMES				Vp ARRIVAL TIMES		INTERVAL VELOCITIES					DIRECT TRAVEL VELOCITIES				Offset Depth References	
(ft bgs)	R2-right (us)	R1-right (us)	R2-left (us)	R1-left (us)	R2 (us)	R1 (us)	Vs-rt (m/s)	Vs-left(m/s)	Vp (m/s)	Vs-ave (fps)	Vp (fps)	Vs Tx-R2 (fps)	Vs Tx-R1(fps)	Vp Tx-R1(fps)	Vp Tx-R2 (fps)	Tx-R1	Tx-R2
18	7300	5230	7390	5480	3050	2410	483	524	1563	1652	5128	1969	1919	4274	4492	25.1	23.6
19.5	7010	5040	7010	4970	3160	2520	508	490	1563	1637	5128	2182	2027	4087	4335	26.6	25.1
21	6610	5100	6610	4950	2760	1960	662	602	1250	2074	4101	2191	2154	5255	4964	28.1	26.6
22.5	6440	4850	6340	4870	2640	1880	629	680	1316	2147	4318	2229	2250	5479	5189	29.6	28.1
24	6800	4950	6710	4870	2490	1800	541	543	1449	1778	4754	2229	2121	5722	5502	31.1	29.6
25.5	6840	5040	6840	5040	2410	1800	556	556	1639	1824	5377	2150	2079	5722	5685	32.6	31.1
27	7070	4890	6990	4890	2300	1730	459	476	1754	1534	5755	2220	2033	5954	5957	34.1	32.6
28.5	6780	5060	6760	4950	2350	1790	581	552	1786	1859	5860	2191	2104	5754	5830	35.6	34.1
30	7110	4970	7050	4890	2290	1770	467	463	1923	1526	6309	2220	2015	5819	5983	37.1	35.6
31.5	6970	4910	6950	4950	2190	1690	485	500	2000	1616	6562	2191	2045	6095	6256	38.6	37.1
33	7130	5080	6950	5100	2220	1730	488	541	2041	1688	6696	2124	2045	5954	6171	40.1	38.6
34.5	6990	5080	7030	5040	2180	1700	524	503	2083	1685	6834	2150	2021	6059	6284	41.6	40.1
36	6990	5040	6990	4930	2170	1690	513	485	2083	1637	6834	2201	2033	6095	6313	43.1	41.6
37.5	5865	4130	5945	4175	2165	1670	576	565	2020	1872	6627	2624	2406	6168	6328	44.6	43.1
39	5425	3545	5425	3515	2140	1615	532	524	1905	1732	6250	3155	2647	6378	6402	46.1	44.6
40.5	5285	3535	5160	3450	2120	1600	571	585	1923	1896	6309	3219	2790	6438	6462	47.6	46.1
42	7375	5320	7160	5195	2110	1555	487	509	1802	1634	5912	2083	1983	6624	6493	49.1	47.6
43	5735	3975	5760	3945	2115	1565	568	551	1818	1836	5965	2788	2486	6581	6478	50.1	48.6
44	5005	3560	5005	3435	2060	1510	692	637	1818	2180	5965	3234	2881	6821	6650	51.1	49.6
45	5045	3545	5020	3505	1950	1420	667	660	1887	2177	6191	3164	2872	7254	7026	52.1	50.6
46	5270	3920	5295	3955	2195	1705	741	746	2041	2439	6696	2780	2716	6041	6241	53.1	51.6
47	5085	3650	5140	3640	2105	1610	697	667	2020	2238	6627	3038	2802	6398	6508	54.1	52.6
47.5	4740	3370	4820	3440	2190	1700	730	725	2041	2387	6696	3229	2998	6059	6256	54.6	53.1
48	4640	3315	4640	3295	2075	1585	755	743	2041	2457	6696	3383	3121	6498	6602	55.1	53.6
48.5	4680	3310	4650	3280	2190	1700	730	730	2041	2395	6696	3399	3114	6059	6256	55.6	54.1
49	4550	3275	4585	3275	2290	1800	784	763	2041	2538	6696	3405	3160	5722	5983	56.1	54.6
49.5	4280	3150	4300	3130	2260	1760	885	855	2000	2854	6562	3576	3383	5852	6062	56.6	55.1
50	4447	3245	4525	3245	2225	1727	832	781	2008	2646	6588	3439	3205	5964	6157	57.1	55.6
50.5	4295	3135	4250	3125	2290	1810	862	889	2083	2872	6834	3583	3425	5691	5983	57.6	56.1
51	5005	3860	4995	3840	2290	1830	873	866	2174	2853	7133	2869	2887	5628	5983	58.1	56.6
51.5	5990	4760	5990	4805	2265	1800	813	844	2151	2718	7057	2261	2387	5722	6049	58.6	57.1
52	5125	3865	5085	3810	2255	1775	794	784	2083	2589	6834	2893	2834	5803	6075	59.1	57.6
52.5	5780	4505	5690	4465	2210	1735	784	816	2105	2625	6906	2444	2518	5937	6199	59.6	58.1
53	5050	3690	5070	3725	2240	1775	735	743	2151	2425	7057	2964	2842	5803	6116	60.1	58.6
53.5	5025	3750	5025	3705	2170	1705	784	758	2151	2530	7057	2981	2869	6041	6313	60.6	59.1
54	4785	3600	4700	3495	2250	1800	844	830	2222	2746	7290	3174	3079	5722	6089	61.1	59.6
54.5	5215	4050	5250	4005	2275	1800	858	803	2105	2725	6906	2743	2740	5722	6022	61.6	60.1
55	4745	3650	4635	3605	2265	1790	913	971	2105	3091	6906	3070	3124	5754	6049	62.1	60.6
56	4550	3390	4490	3440	2230	1790	862	952	2273	2976	7457	3229	3231	5754	6143	63.1	61.6
57	4050	3085	4010	3090	2280	1825	1036	1087	2198	3483	7211	3627	3644	5644	6009	64.1	62.6
58	4075	3130	4060	3075	2230	1770	1058	1015	2174	3401	7133	3646	3596	5819	6143	65.1	63.6
59	4565	3480	4585	3480	2245	1775	922	905	2128	2997	6982	3189	3160	5803	6102	66.1	64.6
60	4570	3380	4535	3355	2230	1720	840	847	1961	2767	6434	3317	3197	5988	6143	67.1	65.6
61	4690	3390	4680	3355	2225	1695	769	755	1887	2500	6191	3317	3093	6077	6157	68.1	66.6

62	5100	3615	5085	3695	2230	1660	673	719	1754	2284	5755	2990	2834	6205	6143	69.1	67.6
62	5445	4035	5435	4025	2250	1680	709	709	1754	2326	5755	2728	2642	6131	6089	69.1	67.6
63	6175	4650	6205	4615	2255	1675	656	629	1724	2108	5656	2360	2301	6149	6075	70.1	68.6
64	5305	3725	5320	3710	2315	1740	633	621	1739	2057	5705	2977	2702	5920	5918	71.1	69.6

Vs & Vp Interval Velocities
see red triangle & blue squares
on graph

COLUMN HEADER LEGEND

DEPTH: Reference point of the Interval Velocity Measurement

Vs ARRIVAL TIMES: Shear Wave

R2-right (us Upper Detector arrival time in micro seconds due to right side strike of the dipole source

R1-right (us Lower Detector arrival time in micro seconds due to right side strike of dipole source

R2-left (us): Upper Detector arrival time in micro seconds due to left side strike of the dipole source

R1-left (us): Lower Detector arrival time in micro seconds due to left side strike of dipole source

Vp ARRIVAL TIMES: Compression P-Wave

R2 (us): Upper Detector arrival time in micro seconds

R1 (us): Lower Detector arrival time in micro seconds

INTERVAL VELOCITY:

Vs-rt (m/s): Interval Shear wave velocity derived from right dipole strike in Meters/sec.

Vs-left(m/s: Interval Shear wave velocity derived from left dipole strike in Meters/sec.

Vp (m/s): Interval P-wave velocity in Meters/sec.

Vs-ave.(fps: Average interval Shear wave velocity in feet/sec.

Vp (fps): Interval P-wave velocity in feet/sec.

DIRECT TRAVEL VELOCITIES:

Vs Tx-R2(fp Shear wave velocity = inline distance from source to upper detector divided by travel time measurement at the upper detector

Vs Tx-R1(fp Shear wave velocity = inline distance from source to lower detector divided by travel time measurement at the lower detector

Vp Tx-R1(fp P-wave velocity = inline distance from source to the lower detector divided by travel time measurement at the lower detector

SUSPENSION LOG DATA AND VELOCITY TABLE BOREHOLE B-5, LBNL, INTEGRATED GENOME BUILDING

DEPTH	Vs ARRIVAL TIMES				Vp ARRIVAL TIMES		INTERVAL VELOCITIES					DIRECT TRAVEL VELOCITIES				Offset Depth References	
(ft bgs)	R2-right (us)	R1-right (us)	R2-left (us)	R1-left (us)	R2 (us)	R1 (us)	Vs-rt (m/s)	Vs-left(m/s)	Vp (m/s)	Vs-ave (fps)	Vp (fps)	Vs Tx-R2 (fps)	Vs Tx-R1(fps)	Vp Tx-R1(fps)	Vp Tx-R2 (fps)	Tx-R1	Tx-R2
29.5	6860	5115	6850	5085	2510	1900	573	567	1639	1870	5421	2130	2076	5377	5458	36.6	35.1
30	6970	5370	6980	5350	2440	1860	625	613	1724	2031	5538	2020	2036	5656	5615	37.1	35.6
30.5	6860	5160	6860	5330	2390	1830	588	654	1786	2037	5628	2028	2073	5860	5732	37.6	36.1
31	7145	5195	7250	5320	2420	1885	513	518	1869	1691	5464	2032	1957	6132	5661	38.1	36.6
31.5	7000	5000	7090	5335	2430	1875	500	570	1802	1755	5493	2026	2003	5912	5638	38.6	37.1
32	7230	5230	7375	5320	2420	1885	500	487	1869	1619	5464	2032	1923	6132	5661	39.1	37.6
32.5	7160	5300	7250	5390	2390	1855	538	538	1869	1765	5553	2004	1957	6132	5732	39.6	38.1
33	6770	5015	6710	5140	2455	1925	570	637	1887	1980	5351	2106	2121	6191	5580	40.1	38.6
34	6450	4910	6525	5035	2455	1915	649	671	1852	2165	5379	2153	2183	6076	5580	41.1	39.6
35	6965	5475	6985	5370	2445	1935	671	619	1961	2116	5323	2012	2034	6434	5603	42.1	40.6
36	6910	5475	6910	5425	2460	1950	697	673	1961	2247	5282	1990	2057	6434	5569	43.1	41.6
37	6950	5355	6950	5370	2400	1895	627	633	1980	2067	5435	2012	2045	6496	5708	44.1	42.6
37.2	6610	5175	6575	5125	2430	1900	697	690	1887	2275	5421	2113	2166	6191	5638	44.3	42.8
38	6870	5300	6860	5335	2405	1880	637	656	1905	2121	5479	2026	2073	6250	5696	45.1	43.6
39	6515	5015	6530	5015	2400	1875	667	660	1905	2177	5493	2162	2182	6250	5708	46.1	44.6
40	6515	5125	6560	5125	2385	1910	719	697	2105	2323	5393	2113	2171	6906	5744	47.1	45.6
41	6545	5285	6555	5285	2370	1915	794	787	2198	2594	5379	2046	2173	7211	5781	48.1	46.6
42	6545	5285	6530	5170	2360	1895	794	735	2151	2508	5435	2093	2182	7057	5805	49.1	47.6
43	6640	5380	6570	5320	2335	1845	794	800	2041	2615	5583	2032	2168	6696	5867	50.1	48.6
44	6545	5045	6545	5085	2355	1850	667	685	1980	2218	5568	2130	2176	6496	5817	51.1	49.6
45	5725	3985	5800	4020	2310	1805	575	562	1980	1865	5706	2732	2468	6496	5931	52.1	50.6
45.5	6515	4930	6515	4970	2290	1770	631	647	1923	2096	5819	2182	2187	6309	5983	52.6	51.1
46	5915	4005	5895	4005	2310	1770	524	529	1852	1727	5819	2743	2427	6076	5931	53.1	51.6
47	5755	4025	5770	4105	2320	1780	578	601	1852	1934	5787	2672	2482	6076	5905	54.1	52.6
48	5835	3955	5820	3885	2270	1760	532	517	1961	1721	5852	2834	2460	6434	6035	55.1	53.6
48.5	5745	3955	5810	3935	2270	1750	559	533	1923	1791	5886	2795	2464	6309	6035	55.6	54.1

Vs & Vp Interval Velocities
see red triangle & blue squares
on graph

COLUMN HEADER LEGEND

DEPTH: Reference point of the Interval Velocity Measurement

Vs ARRIVAL TIMES: Shear Wave

R2-right (us): Upper Detector arrival time in micro seconds due to right side strike of the dipole source
R1-right (us): Lower Detector arrival time in micro seconds due to right side strike of dipole source
R2-left (us): Upper Detector arrival time in micro seconds due to left side strike of the dipole source
R1-left (us): Lower Detector arrival time in micro seconds due to left side strike of dipole source

Vp ARRIVAL TIMES: Compression P-Wave

R2 (us): Upper Detector arrival time in micro seconds
R1 (us): Lower Detector arrival time in micro seconds

INTERVAL VELOCITY:

Vs-rt (m/s): Interval Shear wave velocity derived from right dipole strike in Meters/sec.
Vs-left(m/s): Interval Shear wave velocity derived from left dipole strike in Meters/sec.
Vp (m/s): Interval P-wave velocity in Meters/sec.
Vs-ave.(fps): Average interval Shear wave velocity in feet/sec.
Vp (fps): Interval P-wave velocity in feet/sec.

DIRECT TRAVEL VELOCITIES:

Vs Tx-R2(fps: Shear wave velocity = inline distance from source to upper detector divided by travel time measurement at the upper detector

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Appendix B:

Televiewer Logging

APPENDIX B

BOREHOLE IMAGING TELEVIEWERS

Optical and acoustic televiewers are oriented imaging tools used to map borehole discontinuities. The output of these tools is presented as an unwrapped (unfolded cylinder on a two-dimensional surface) image plot with superimposed fracture/joint identification associated with the orientation and dip angles of all interpreted discontinuities are presented in Appendix B and referenced in the following section.

1) Methodology-Data Acquisition

The occurrence and orientation of borehole discontinuities (fractures, bedding, geologic contacts, etc.), rock textures, and other descriptive geologic information can be viewed with an OPTV imaging tool. The OPTV tool uses a digital optical sensor to produce radial images at a resolution down to 0.004 feet. These radial images are then composited sequentially via computer software to produce continuous color (unwrapped) video-like images on a field computer screen. The tool can operate in either dry or water-filled portions of the borehole providing that the water-filled portion is optically clear. The final "unwrapped" radial images are referenced to magnetic north as determined by an on-board magnetic compass. In addition, to the magnetic compass bearing, the inclination and azimuth of the borehole was recorded by a combination three-axis magnetic-inclinometer sensor package.

Because a viscous drilling fluid was required to maintain borehole integrity during this survey, we relied exclusively on the BHTV method to image the borehole wall. The BHTV tool is an ultrasonic acoustic send and receive device. Sidewall borehole images are created by measuring variations of thousands of two-way travel times and amplitudes of reflected ultrasonic pulses as the device is moving up the borehole. The BHTV logging technique requires a water column to act as a medium to transmit and receive acoustic signals to and from the borehole wall. The data sampling rate for the BHTV tool is every 0.004 foot. The left margin of the borehole images plot corresponds to the direction of magnetic north as determined by an on-board magnetic compass. In addition to the magnetic compass, the inclination of the borehole was recorded by an omni-directional three-axis accelerometer. Image data were conducted in the up hole direction. Logging speed was approximately 4 feet per minute.

2) Data Analysis

We used the computer program *WELLCAD* (Version 4.4, ALT, Luxemburg) to produce merged BHTV image plots and to calculate orientations of interpreted discontinuities (e.g. fractures). Corrections for the magnetic declination in the survey area required adding 14 degrees to the magnetic compass bearings in order to orient the borehole images to true north (NOAA, Magnetic Declination Map, 2010). Since borehole diameter is a major reduction parameter in determining dip magnitude, we input caliper log measurements. Discontinuities analysis was performed interactively on sections of the unwrapped acoustic amplitude images as viewed on a computer monitor. An interpretable discontinuity on a two-dimensional unwrapped borehole televiewer log appears as a recognizable sinusoidal shaped trace that usually extends across the full width of the borehole image. The sinusoidal shape is a manifestation of a planar discontinuity intercepting a three-dimensional cylindrical borehole. Planar discontinuities can be geologic features that include discrete fractures or joints, bedding planes, and planar intrusions such as veins and geologic contacts. Identified discontinuity traces on the image logs were fitted with a bendable sinusoid that overlies the trend of the trace. *WELLCAD* then calculates a plane that represents the orientation of the discontinuity in terms of dip direction and dip magnitude based on the position of the sinusoid overlay. The process is repeated for every significant discontinuity until the entire borehole is interpreted. At this stage, apparent dip direction and dip magnitude of the discontinuities are converted to true geographic dip azimuth and dip magnitude by factoring in the borehole tilt (inclination) and azimuth at the depth of the discontinuity.

We assigned a descriptive hierarchy to the fractures/joints as follows: open fractures/joints, discontinuous fractures-joints, shear zone and bedding. Open fractures/joints are significant features having continuous traces and are associated with borehole washouts as indicated by the caliper log. These features have apparent apertures well in excess of 1 millimeter (mm). Discontinuous fractures/joints as the name implies are related to incomplete traces and typically displayed apparent open apertures of approximately one to two mm. We also included some features in this category that were at the threshold of detection (less than one mm) and nearly continuous. We used the "shear zone" classification sparingly to zones that exhibited highly fractured, broken rock intervals.

3) Presentation and Results

Televiewer image log plots can be found in Appendix B. These plots contain a series of illustrations from left to right across the page as follows:

- Unwrapped, BHTV image corrected to true North. A caliper log trace was superimposed over the OPTV log colored white. Lower column header ("Sinusoids") depicts the interpreted sinusoid curves fitted over traces of visible discontinuities.

- "DIPS" plot. The DIPS plot shows tadpole symbols that represent identified discontinuities on the aforementioned image plots in terms of dip direction and dip angel magnitude. The discontinuity dip angle is depicted by the tadpoles position on the depth versus degrees (0° to 90°) plot where 0° represents horizontal and 90° represents vertical. Dip direction is depicted by the position of the symbol's tail as if it were positioned on a 360° compass face where north is the tail pointing vertically up the page, east is the tail pointing 90° to the right of vertical, south is pointing vertically down the page and west is 90° from vertical to the left of the page. Various colored tadpole symbols convey the classification of the interpreted discontinuities as follows: grey (triangle symbols) = "crushed-shear" zone, red = open fractures/joints, light orange = discontinuous fractures/joints and green = bedding. Hachured colorized features on the Sinusoid overlay show the vertical extent of the feature.

- Borehole deviation in terms of azimuth (bearing) direction and tilt (borehole angle with respect to vertical).

Each tadpole symbol represents a discontinuity at depth. The plane is defined by its dip direction and dip magnitude. The latter is the angle made from horizontal plane.

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NORCAL GEOPHYSICAL CONSULTANTS, INC.

BHTV DISCONTINUITY ANALYSIS

COMPANY:	A3GEO	DATE:	July 17, 2014
WELL ID:	B-2	CASING:	steel to 8 ft bgs
FIELD:	LBNL-IGB	JOB NO.	14-1080.02B
COUNTY:	ALAMEDA	STATE:	CA

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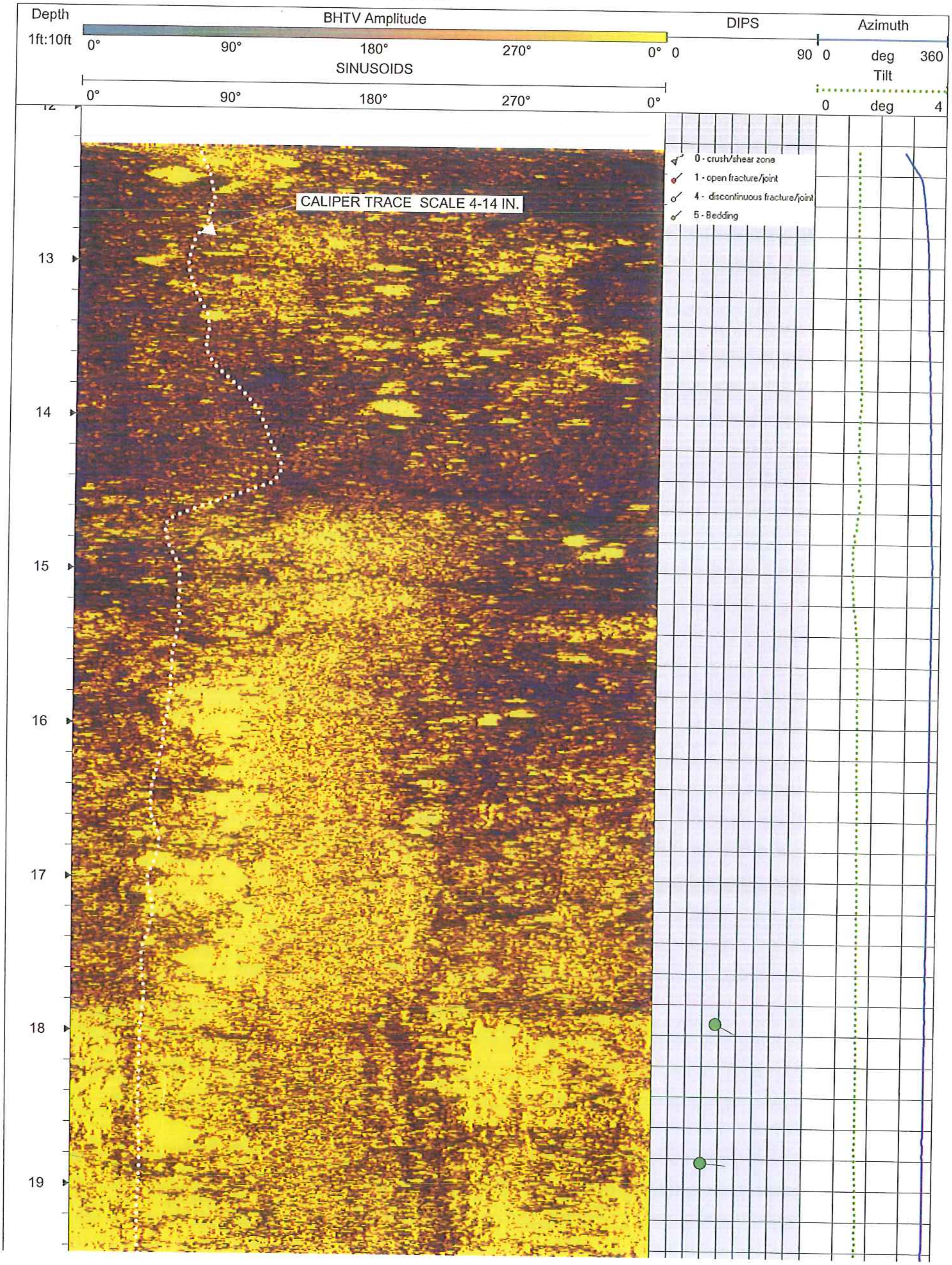

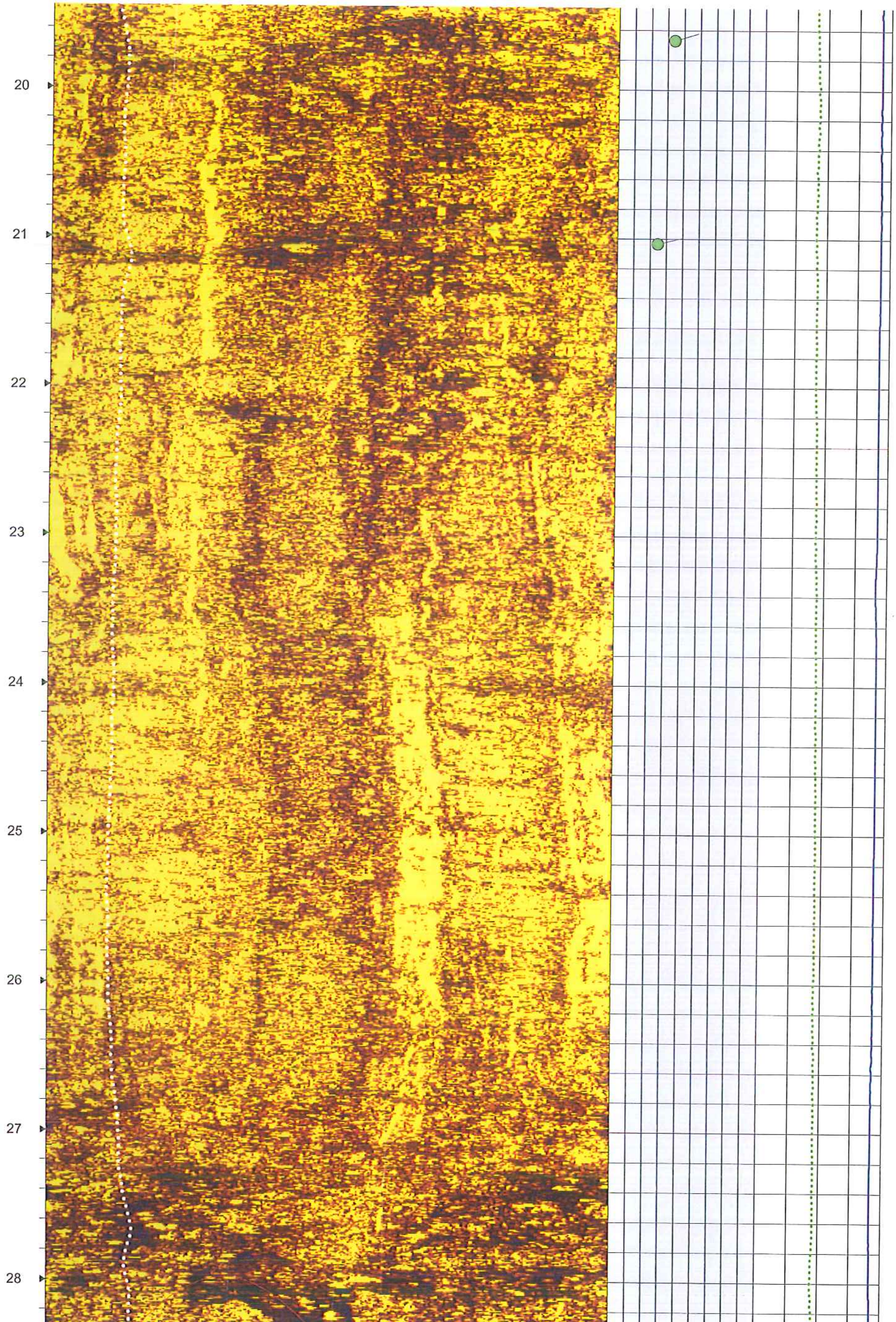

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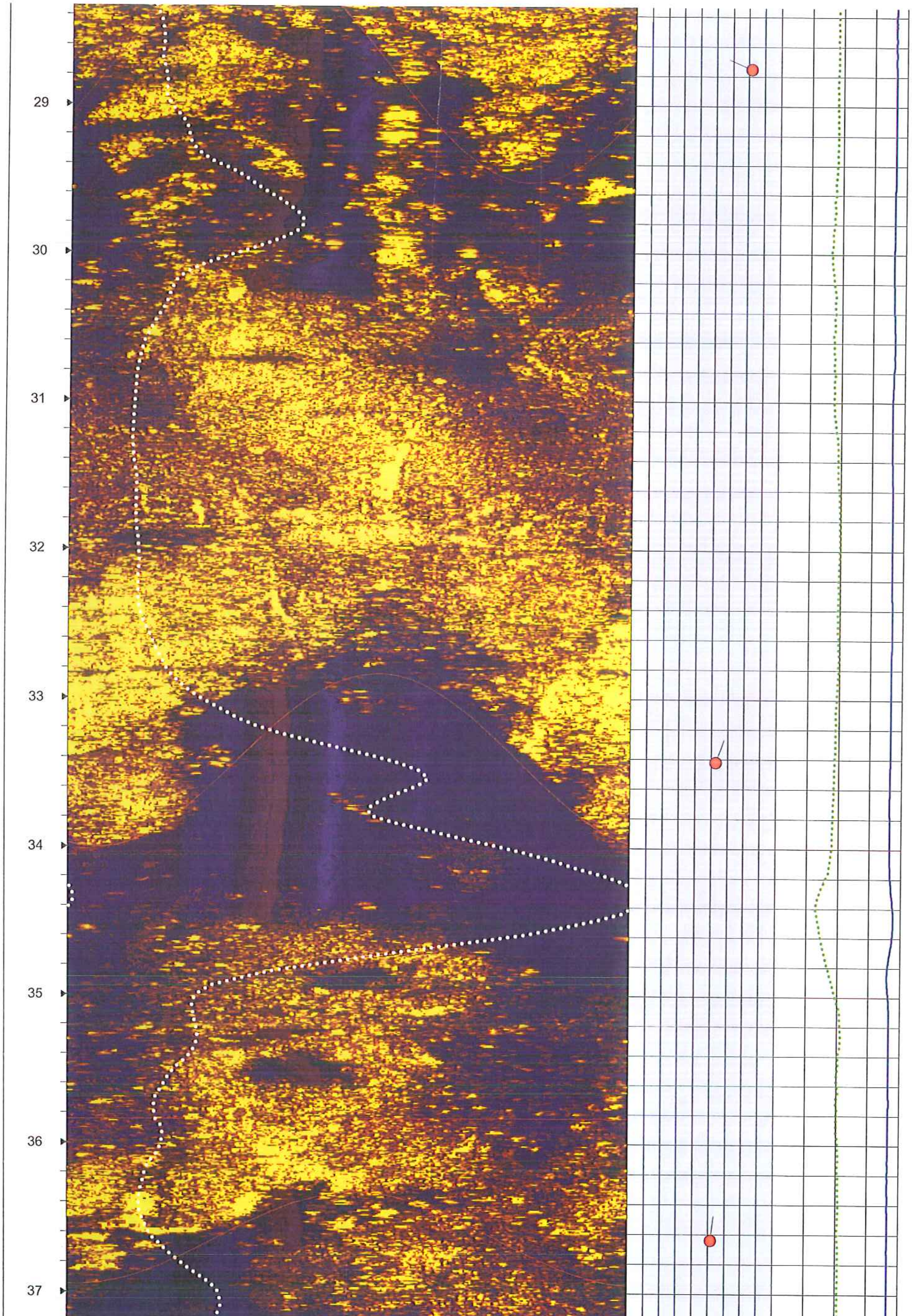

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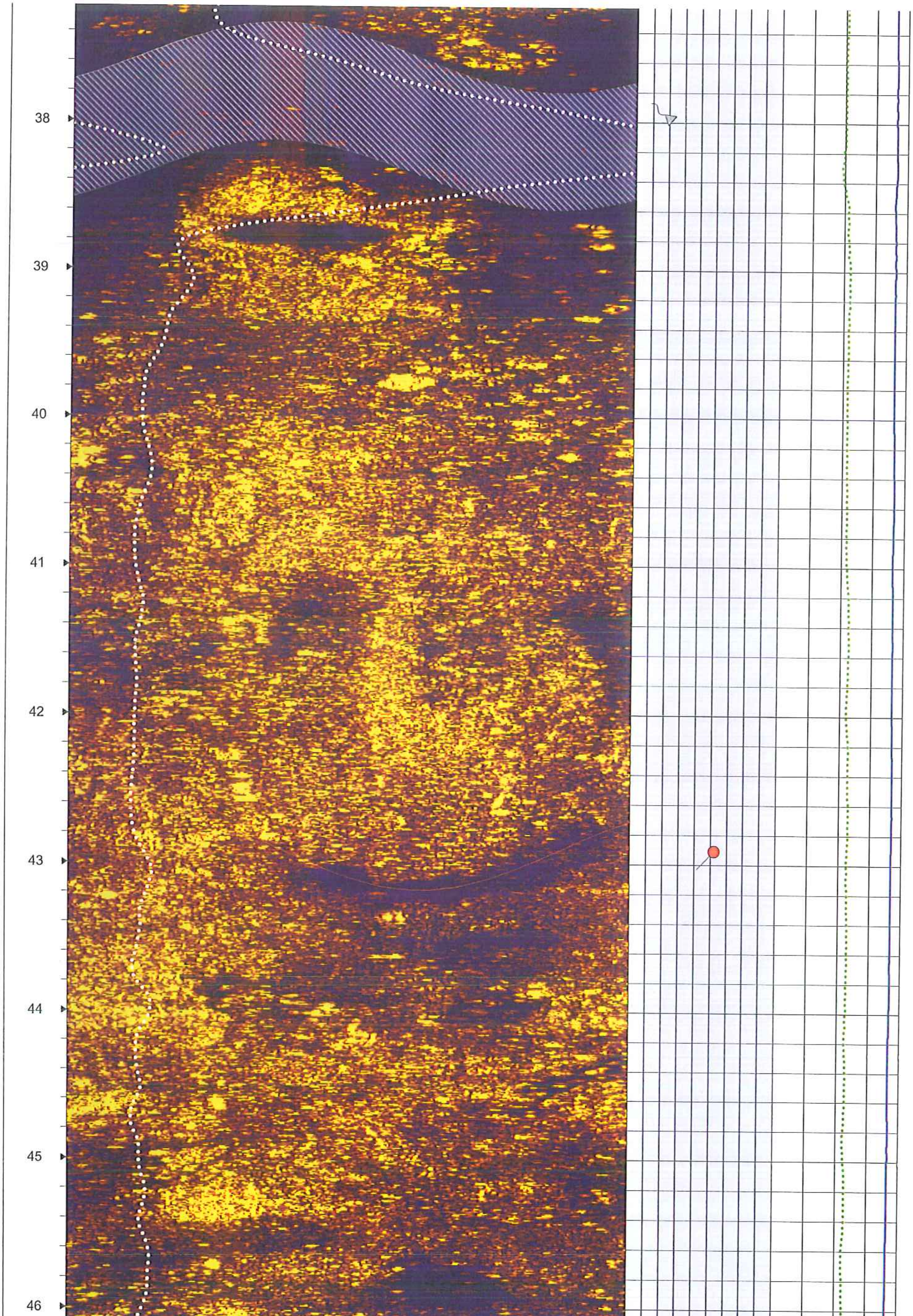
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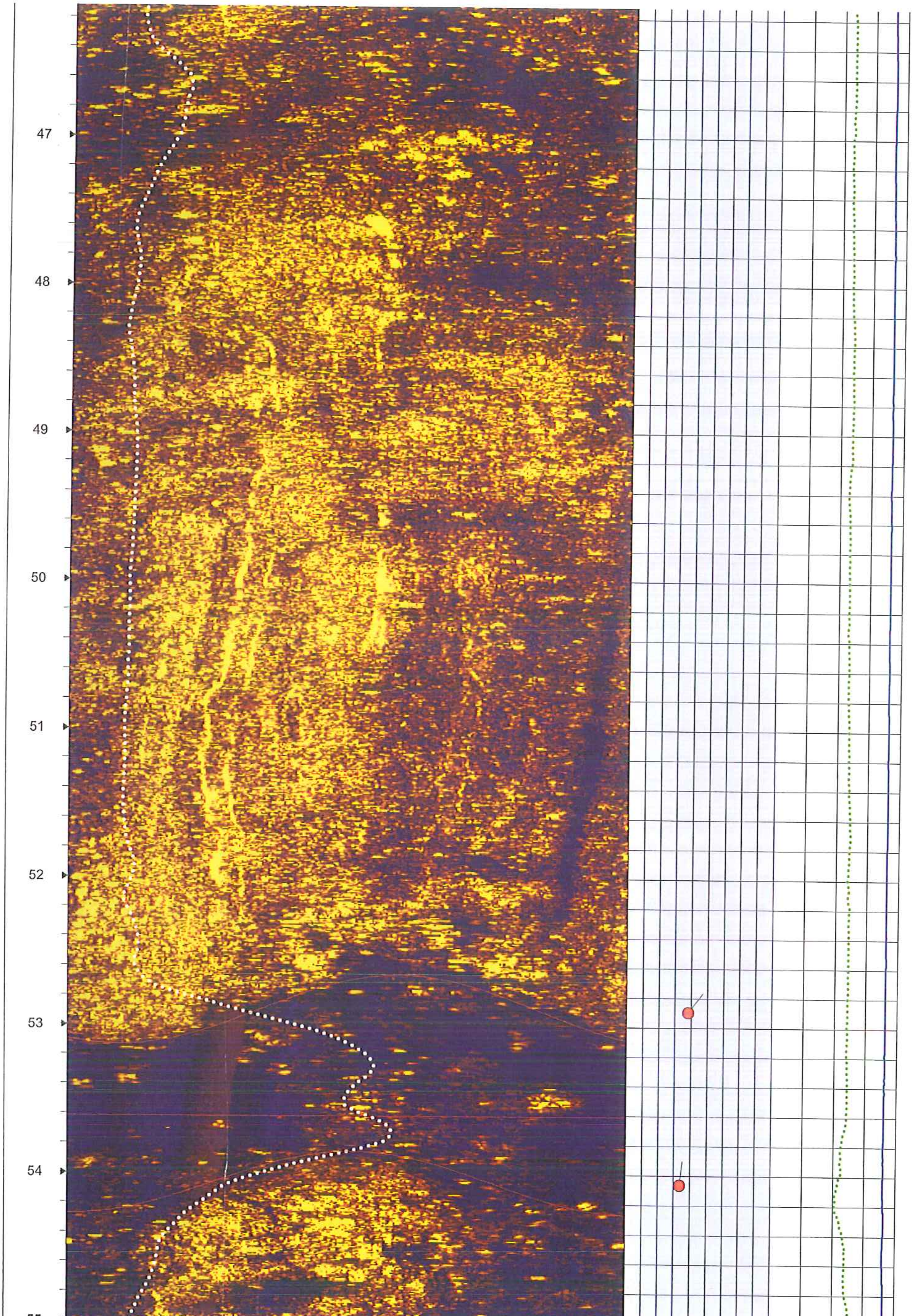
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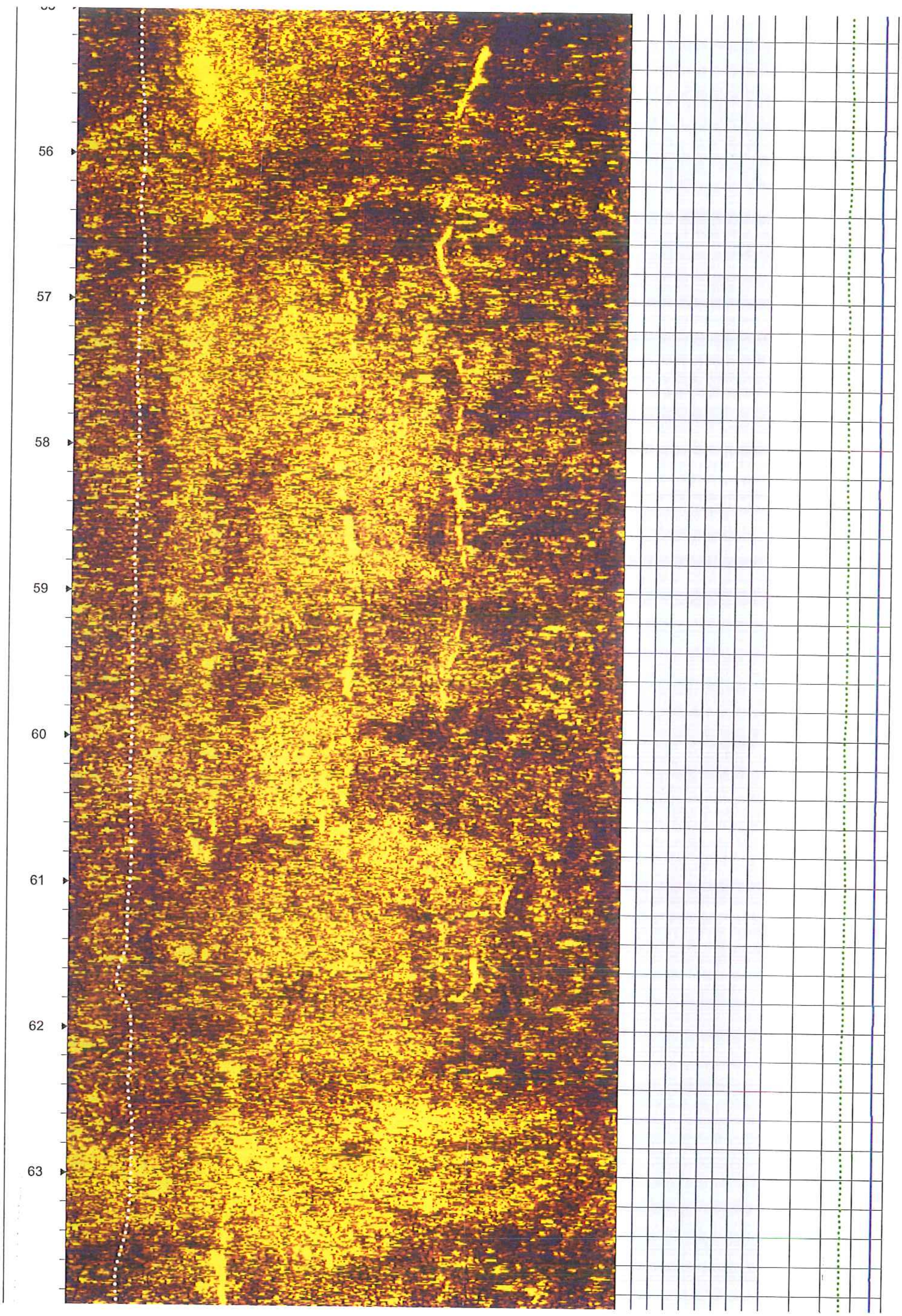
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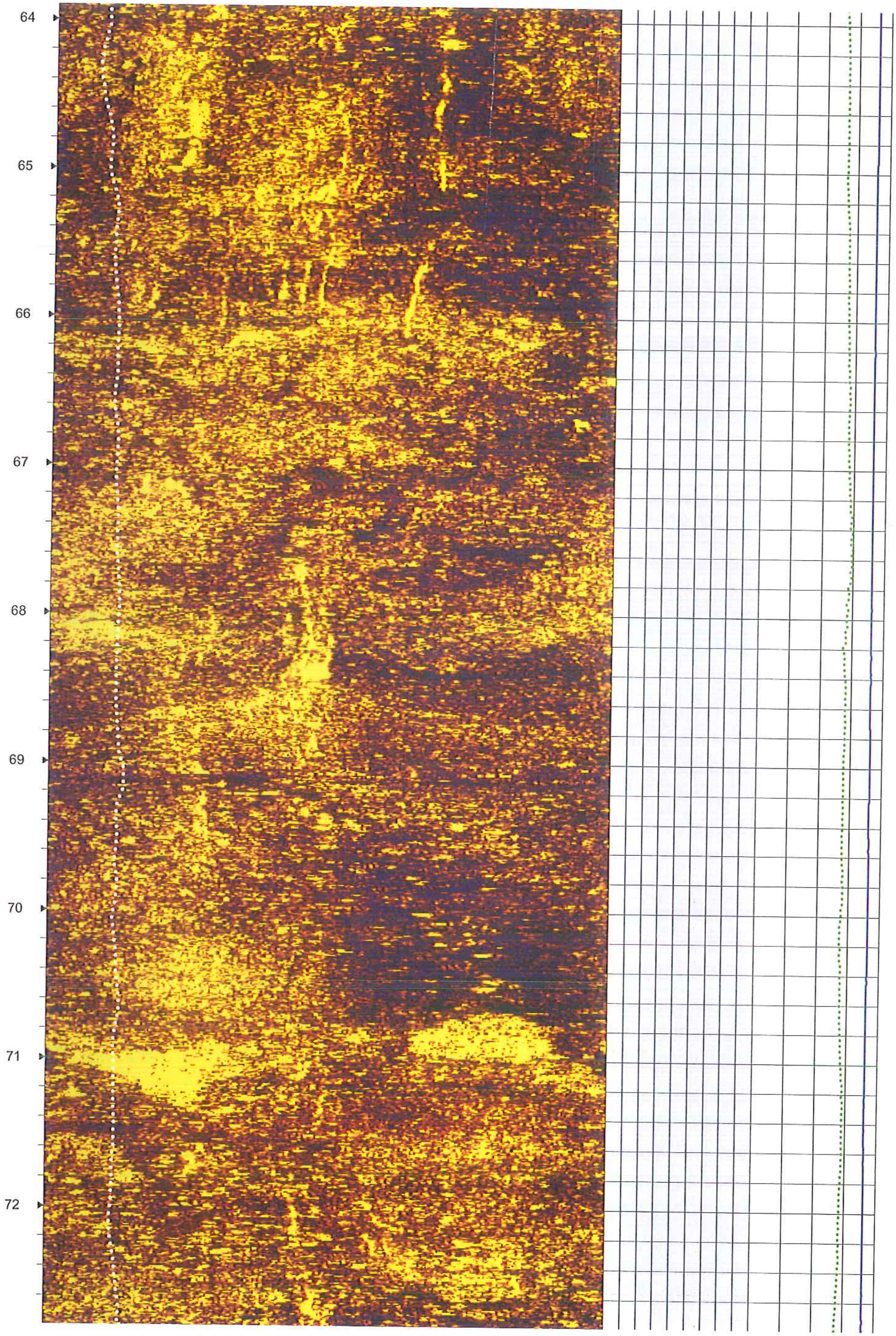
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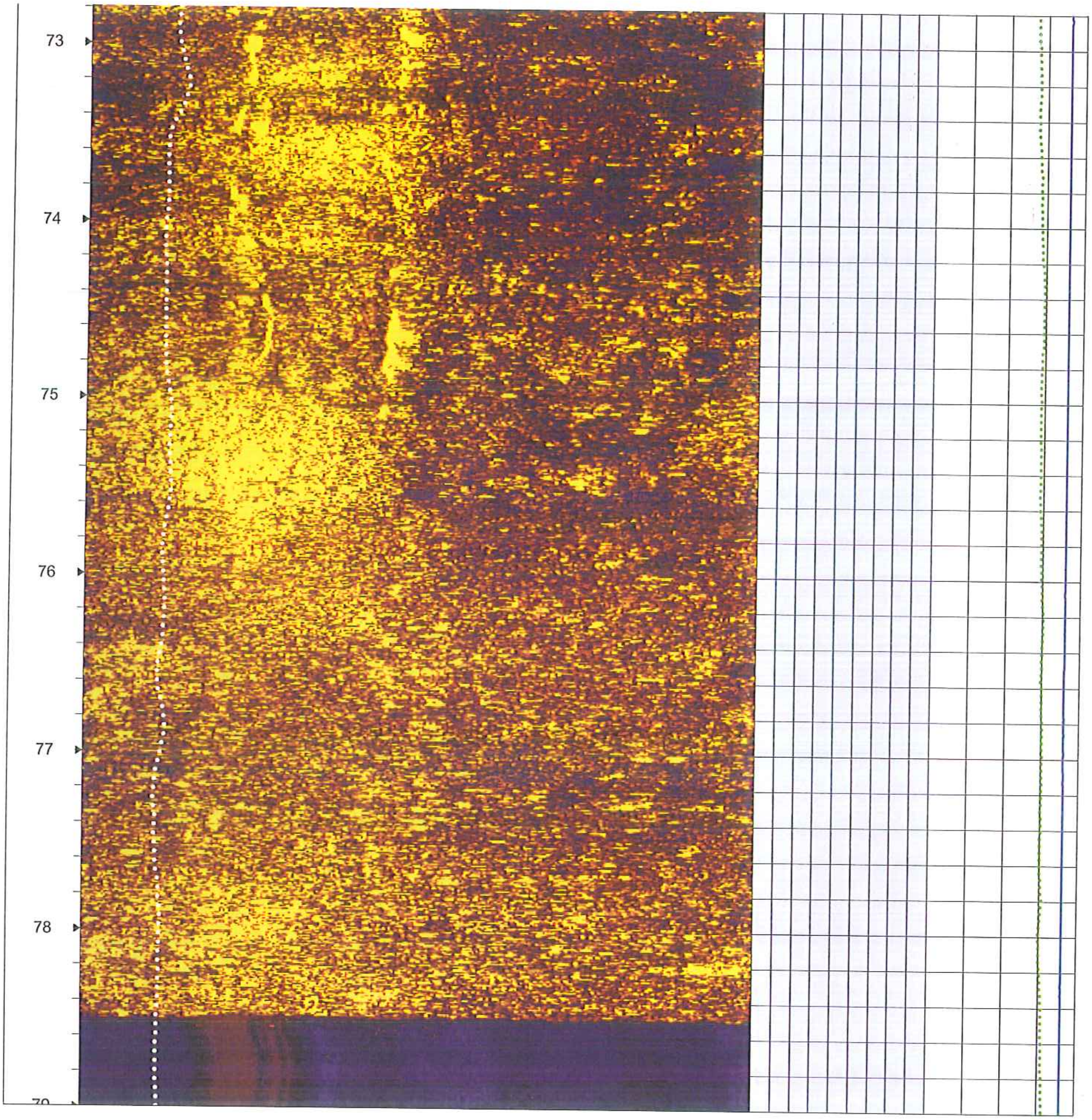
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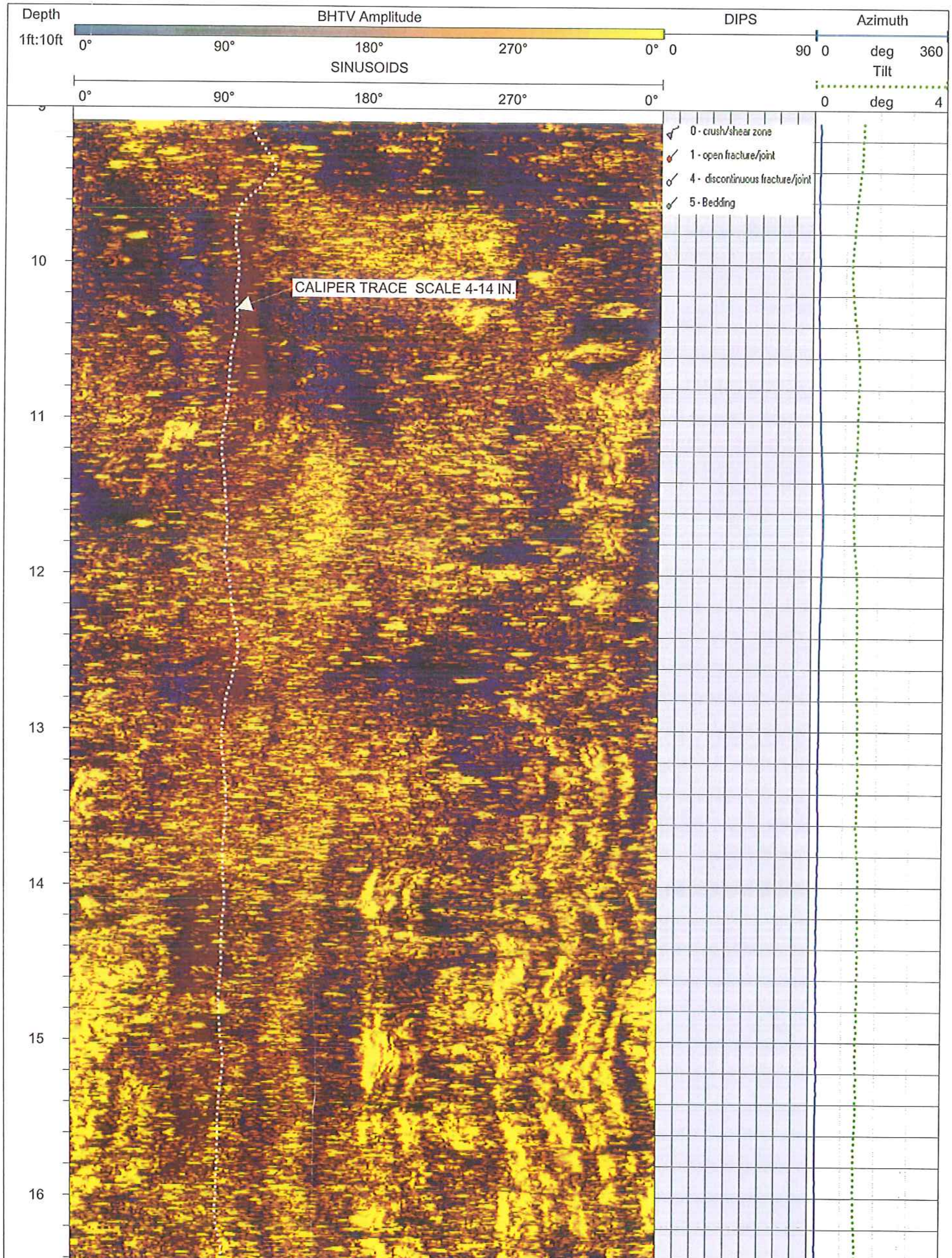
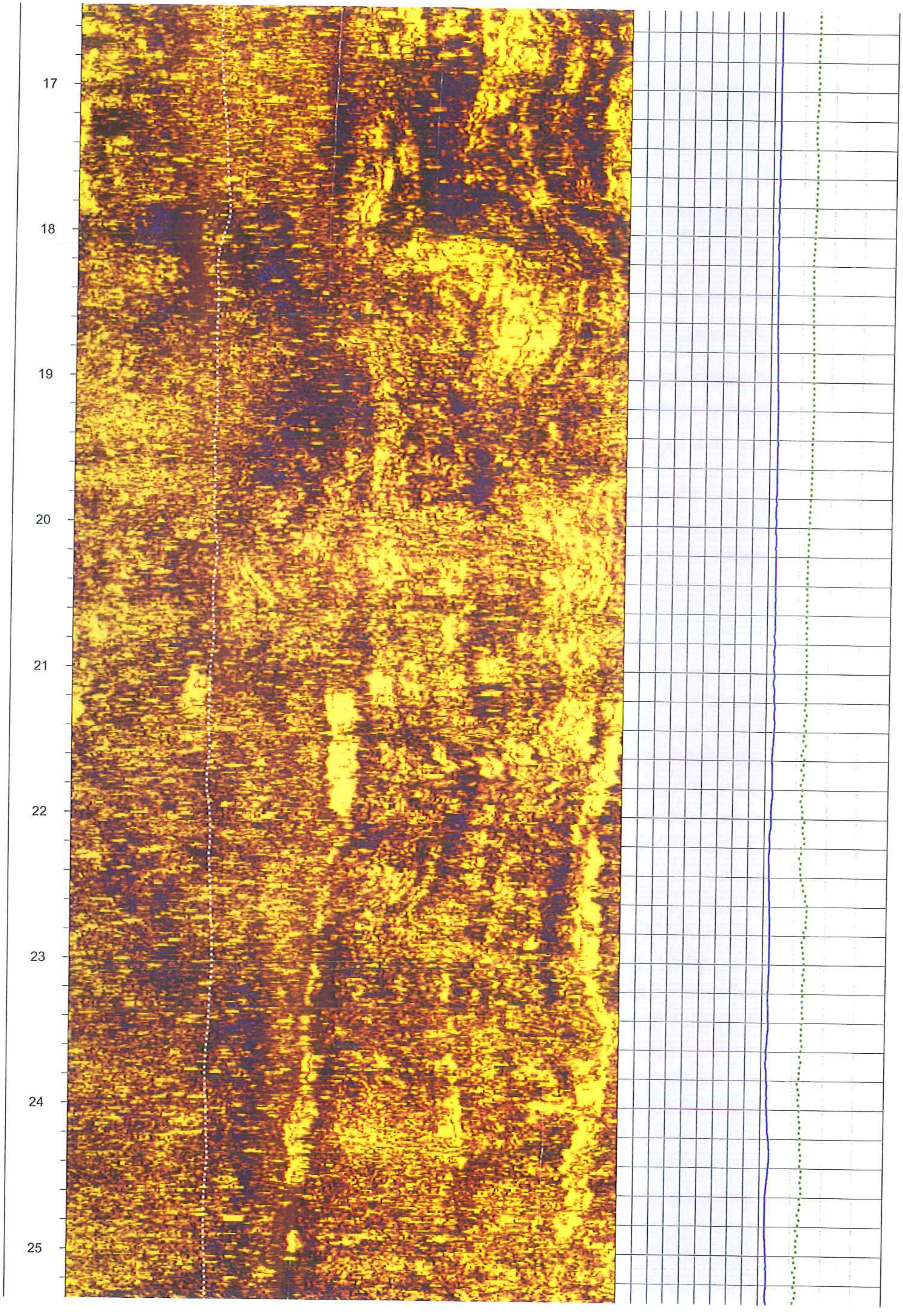
NORCAL NORCAL GEOPHYSICAL CONSULTANTS, INC.	BHTV DISCONTINUITY ANALYSIS	COMPANY: A3GEO WELL ID: B-4 FIELD: LBNL-IGB COUNTY: ALAMEDA	DATE: July 22, 2014 CASING: steel to 3 ft bgs JOB NO. 14-1080.02B STATE: CA

NOTES:

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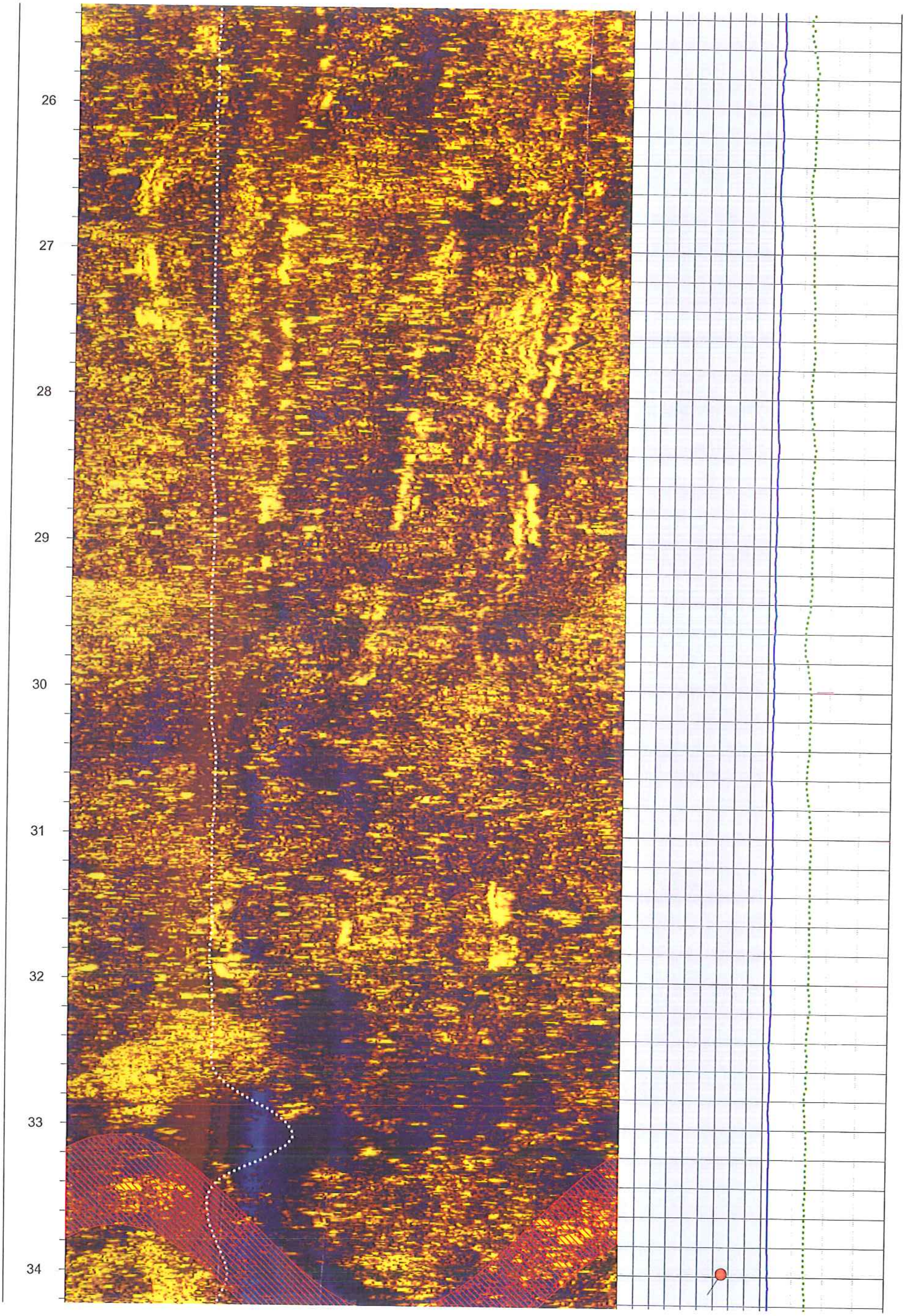
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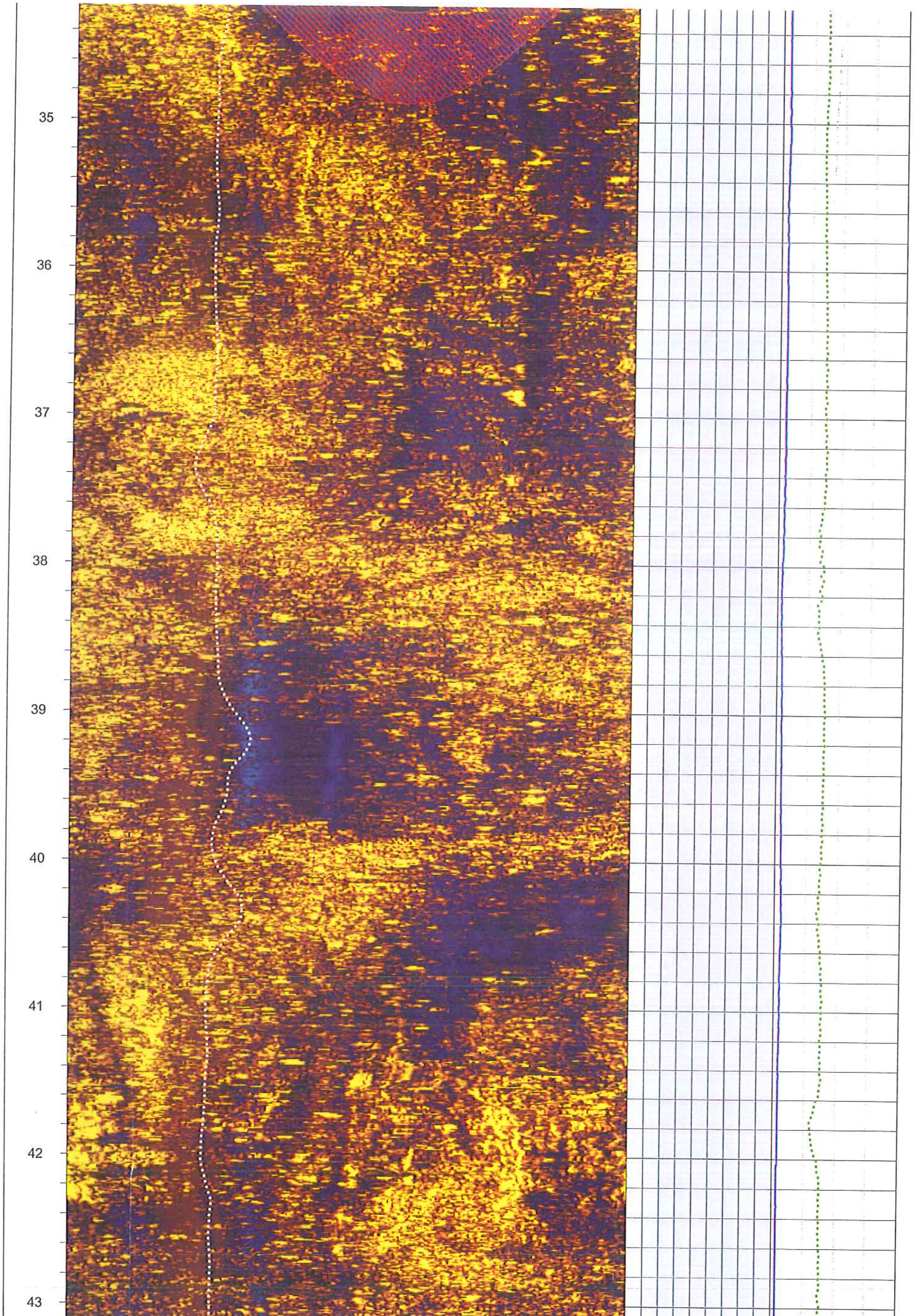
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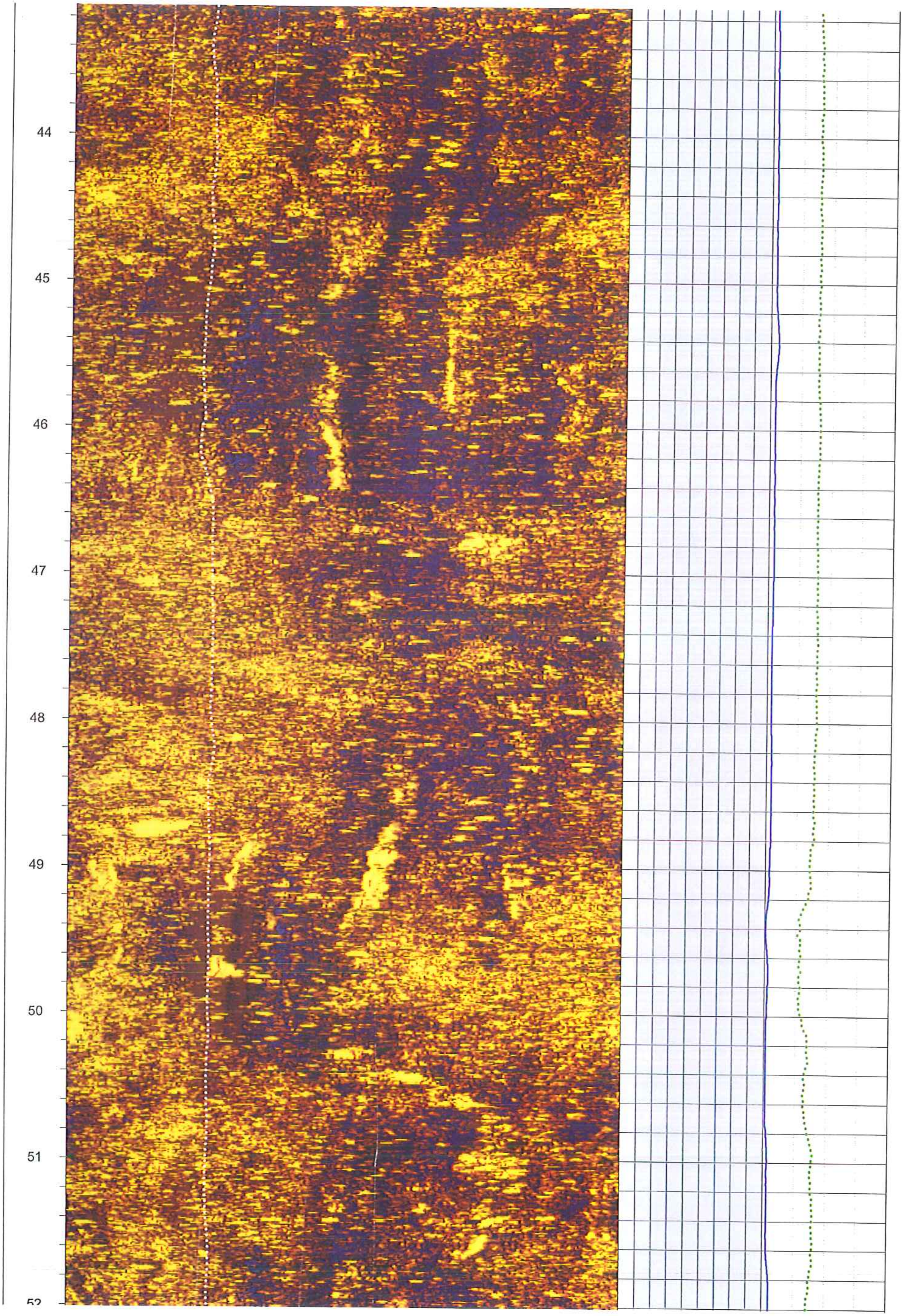
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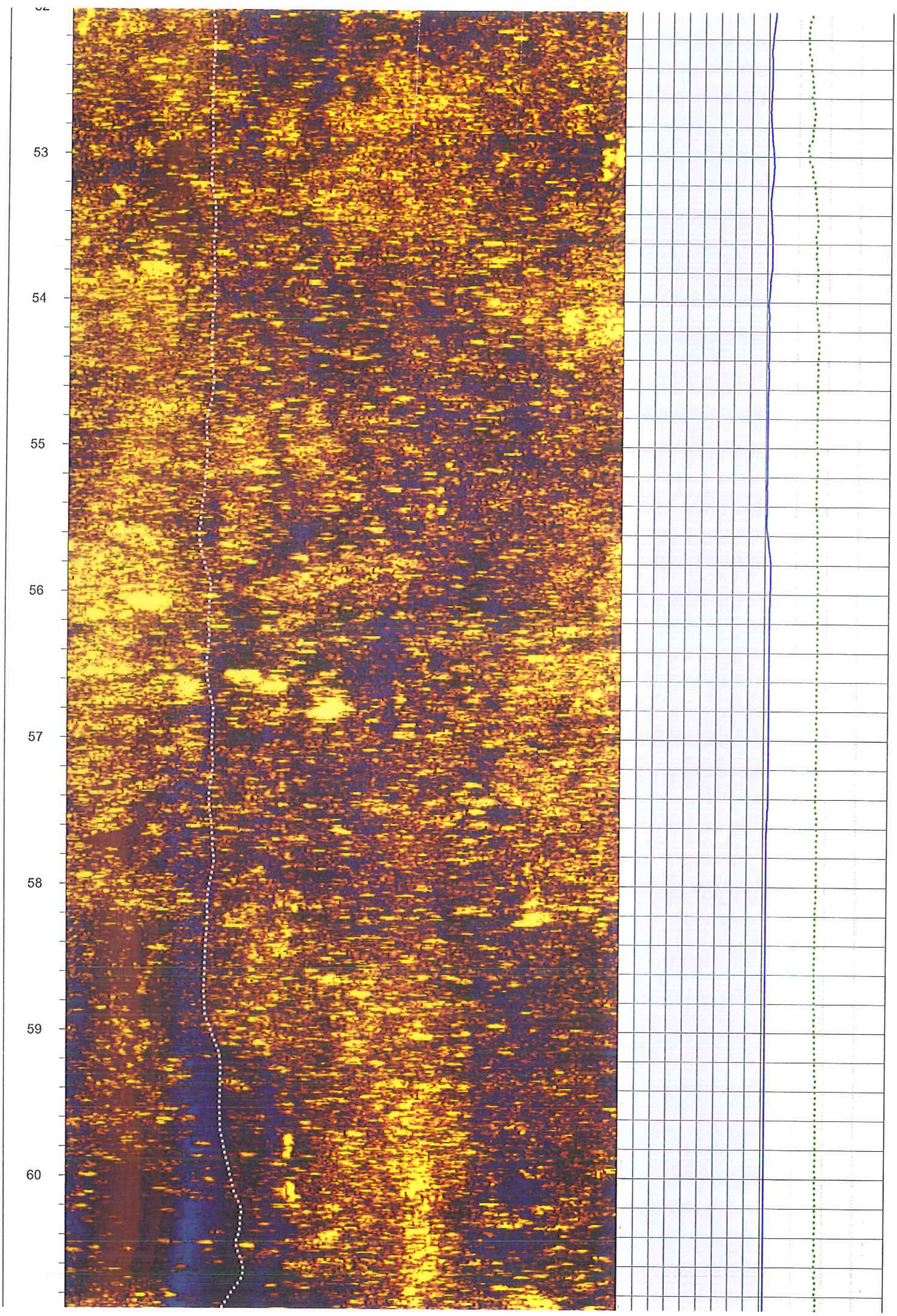
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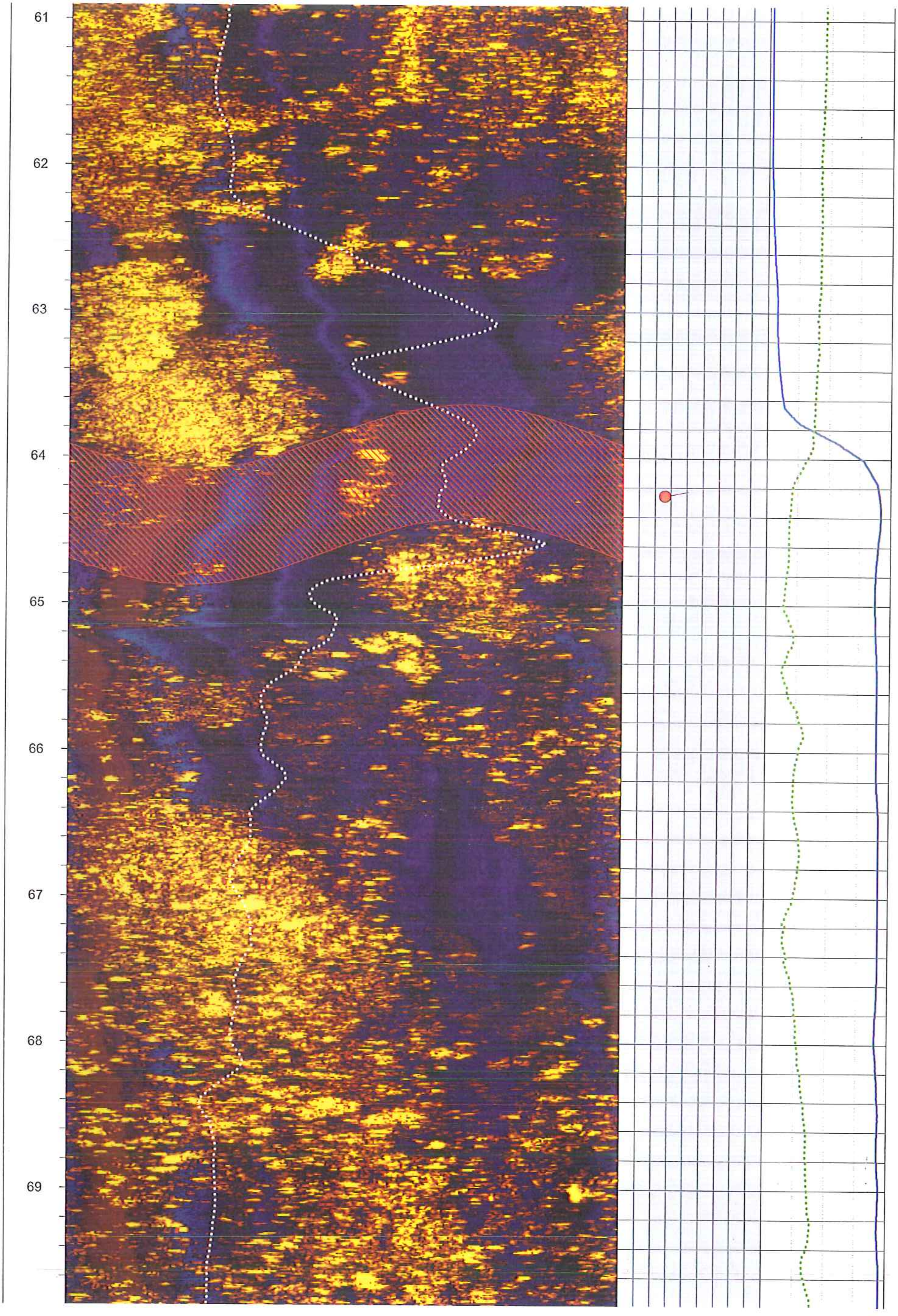
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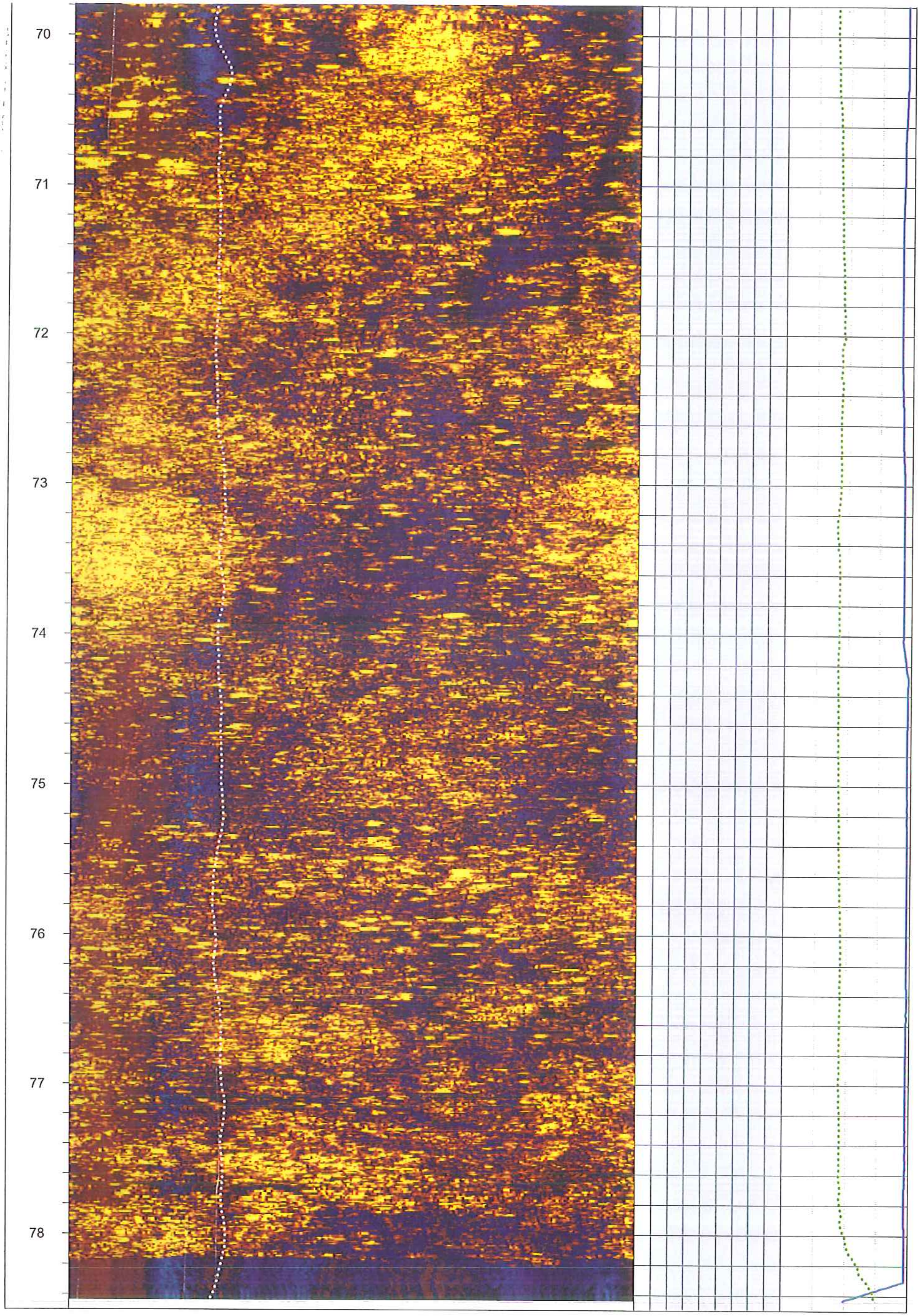
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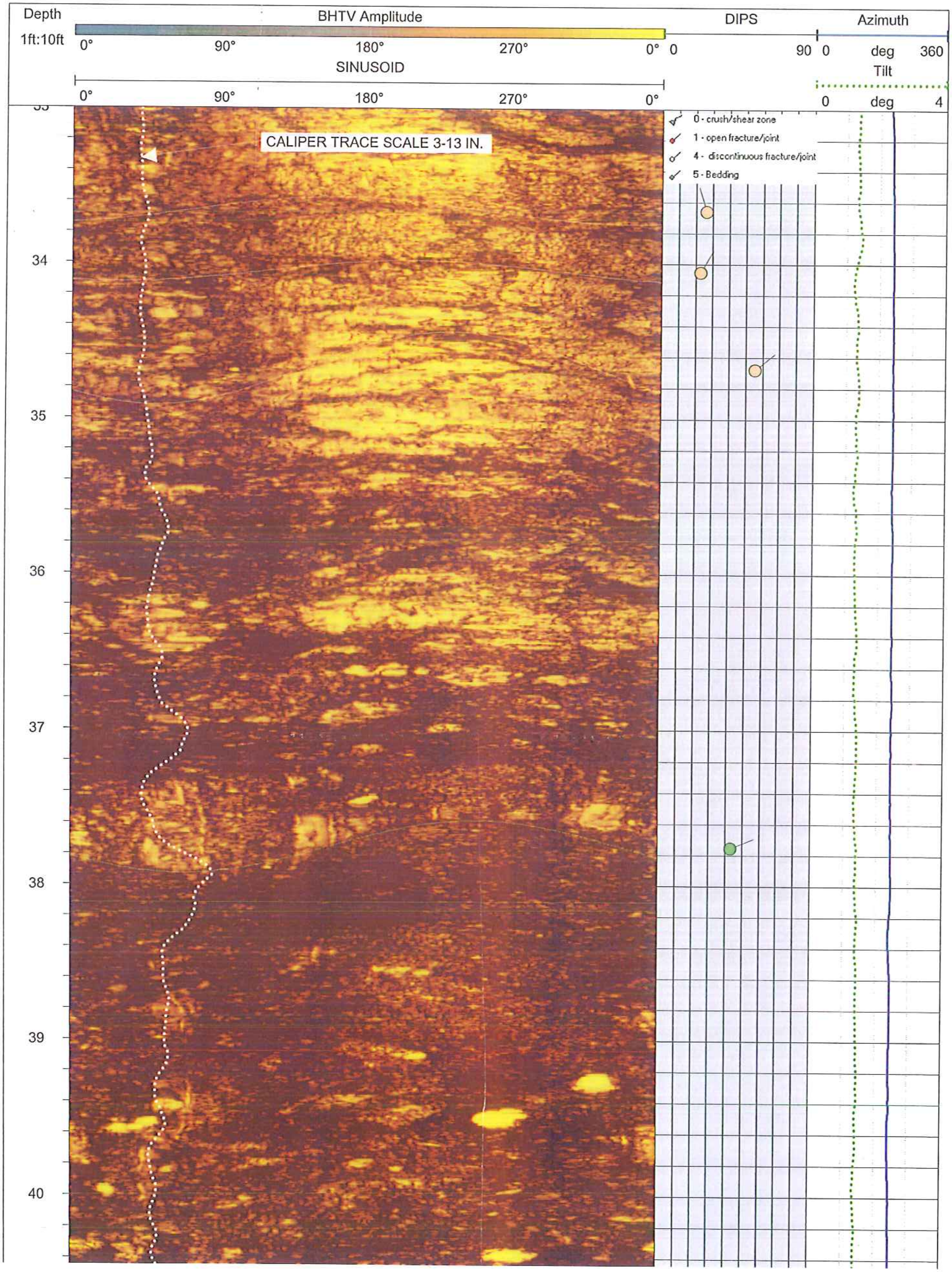
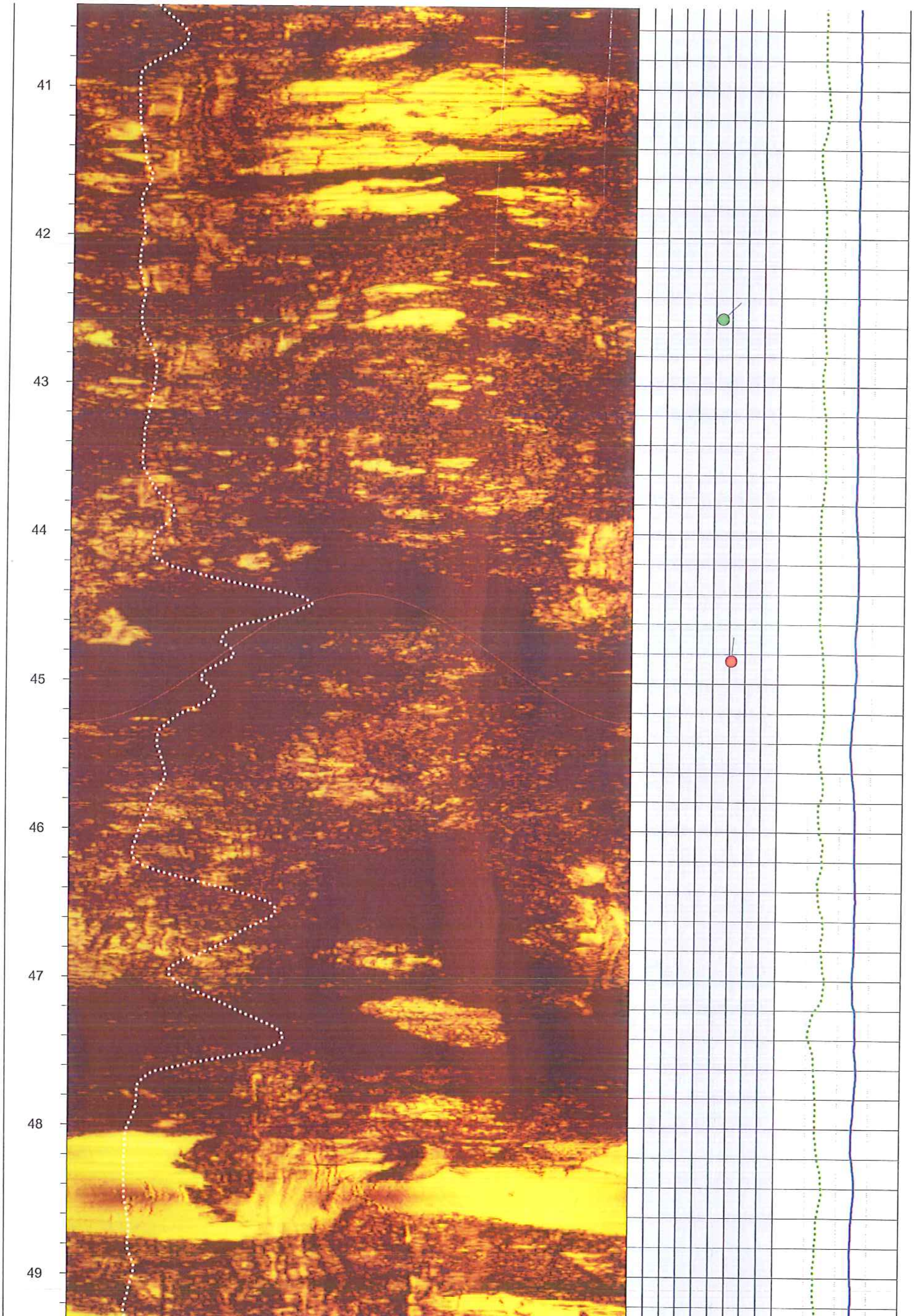
NORCAL NORCAL GEOPHYSICAL CONSULTANTS, INC.	BHTV DISCONTINUITY ANALYSIS	COMPANY: A3GEO WELL ID: B-5 FIELD: LBNL-IGB COUNTY: ALAMEDA	DATE: July 25, 2014 CASING: steel to 28 ft bgs JOB NO. 14-1080.02B STATE: CA

NOTES:





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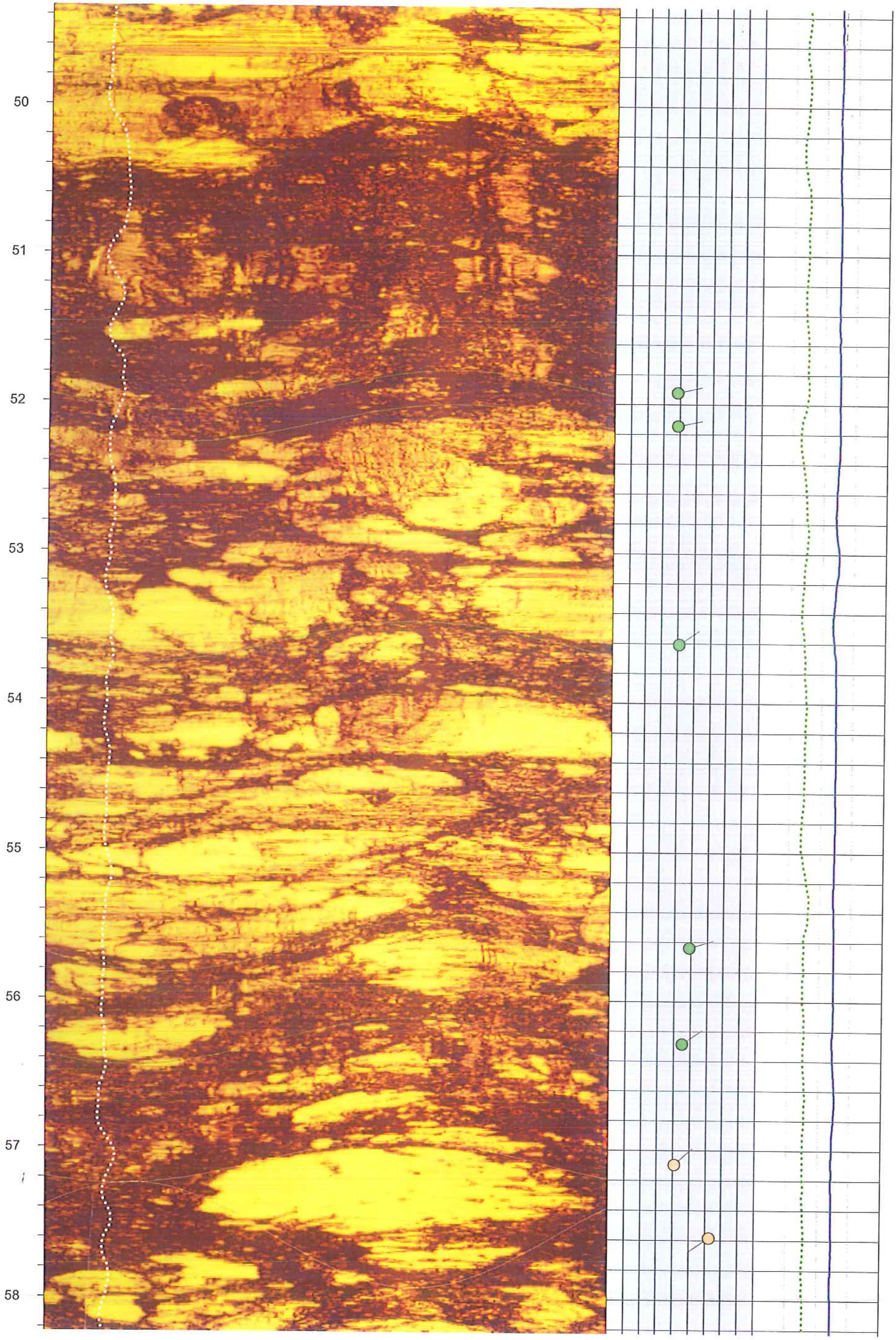
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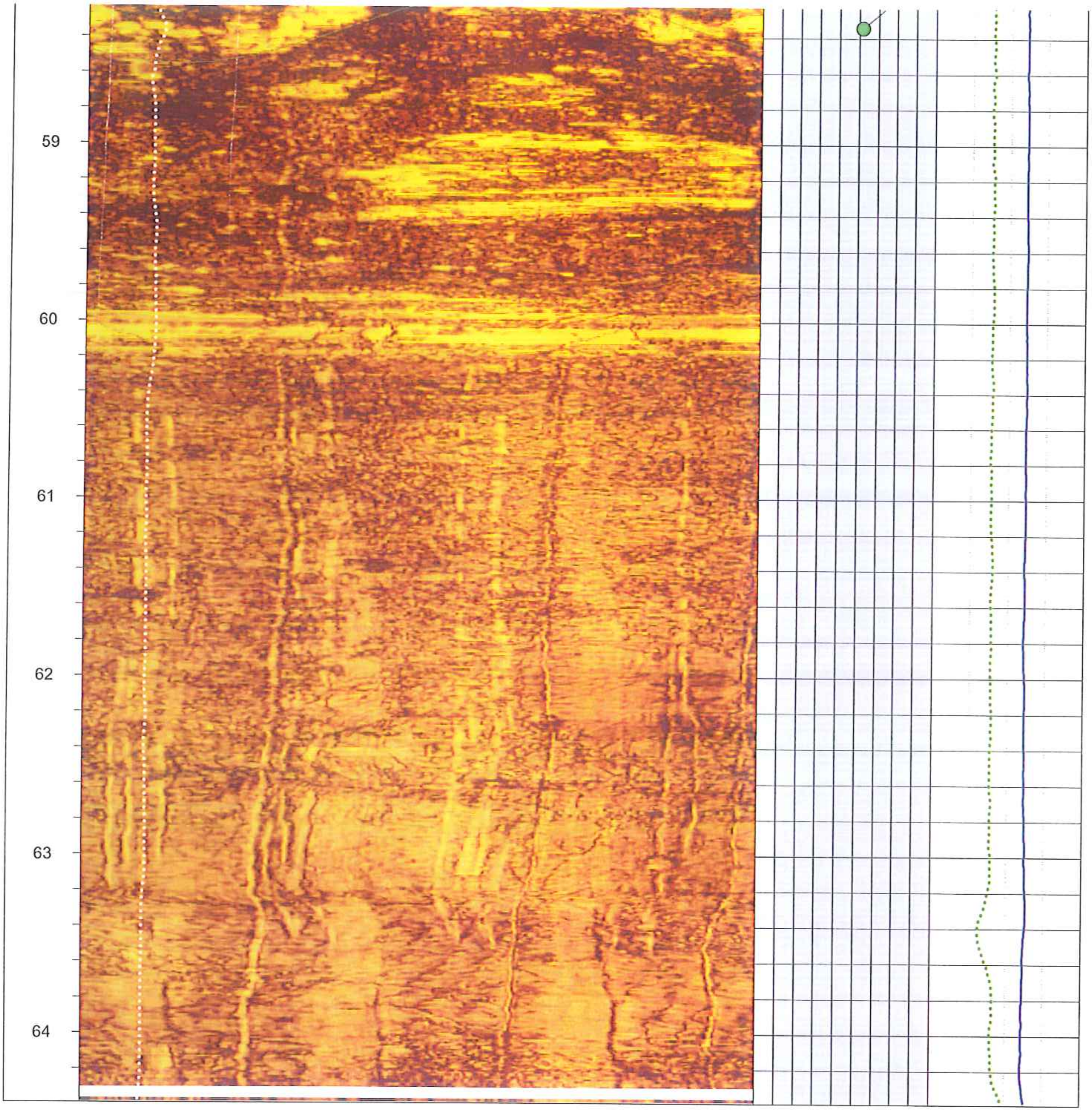
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Appendix C

Laboratory Test Data

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

B. HILLEBRANDT SOILS TESTING, INC.

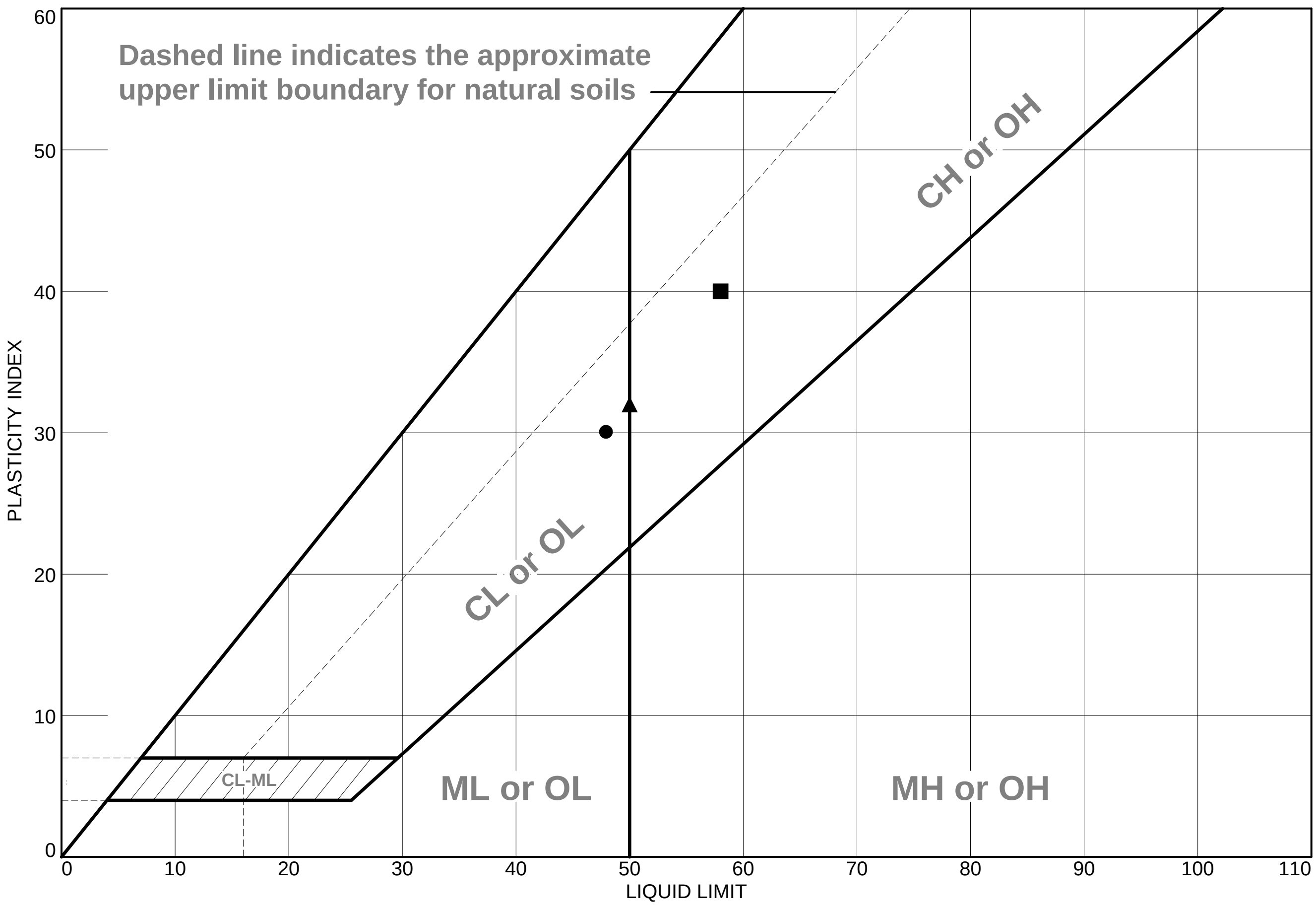
29 Sugarloaf Terrace, Alamo, CA 94507 - Tel: (510) 409-2916 - Fax: (925) 891-9267 - Email: soiltesting@aol.com

LAB RESULTS SUMMARY FORM

Project Number: 1100-17B **Project Name:** IGB **Results Due By:**
Requested By: DM **Request Date:** 9/24/14 **Throw Samples Out On:**

Boring #	Sample Depth (feet)	Dry Density (pcf)	Moisture Content (%)	Unconfined Compressive (ksf)	Atterberg			-200			Compaction		Pocket Penetrometer (tsf)	Torvane (tsf)	Remarks
					Liquid Limit	Plastic Limit	Plasicity Index	Passing #4 Sieve (%)	Passing #40 sieve (%)	Passing #200 sieve (%)	Maximum Dry Density (pcf)	Optimum Moisture (%)			
B-6	5.5				48	18	30	78	66	52					
B-6	11.0				58	18	40	100	94	77					
B-6	15.0				50	18	32	78	71	58					
B-7	3.0				44	20	24	67	52	38					
B-7	10.5				49	25	24	51	31	21					
B-7	15.0				63	21	42	100	96	85					

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark grayish brown sandy lean CLAY with gravel	48	18	30	65.5	51.7	CL
■	Grayish brown fat CLAY with sand	58	18	40	94.1	77.0	CH
▲	Dark brown to dark gray gravelly fat CLAY with sand	50	18	32	70.7	58.2	CH

Project No. 1100-17B **Client:** A3Geo

Project: IGB

●**Source of Sample:** B-6 **Depth:** 5.5'
■**Source of Sample:** B-6 **Depth:** 11.0'
▲**Source of Sample:** B-6 **Depth:** 15.0'

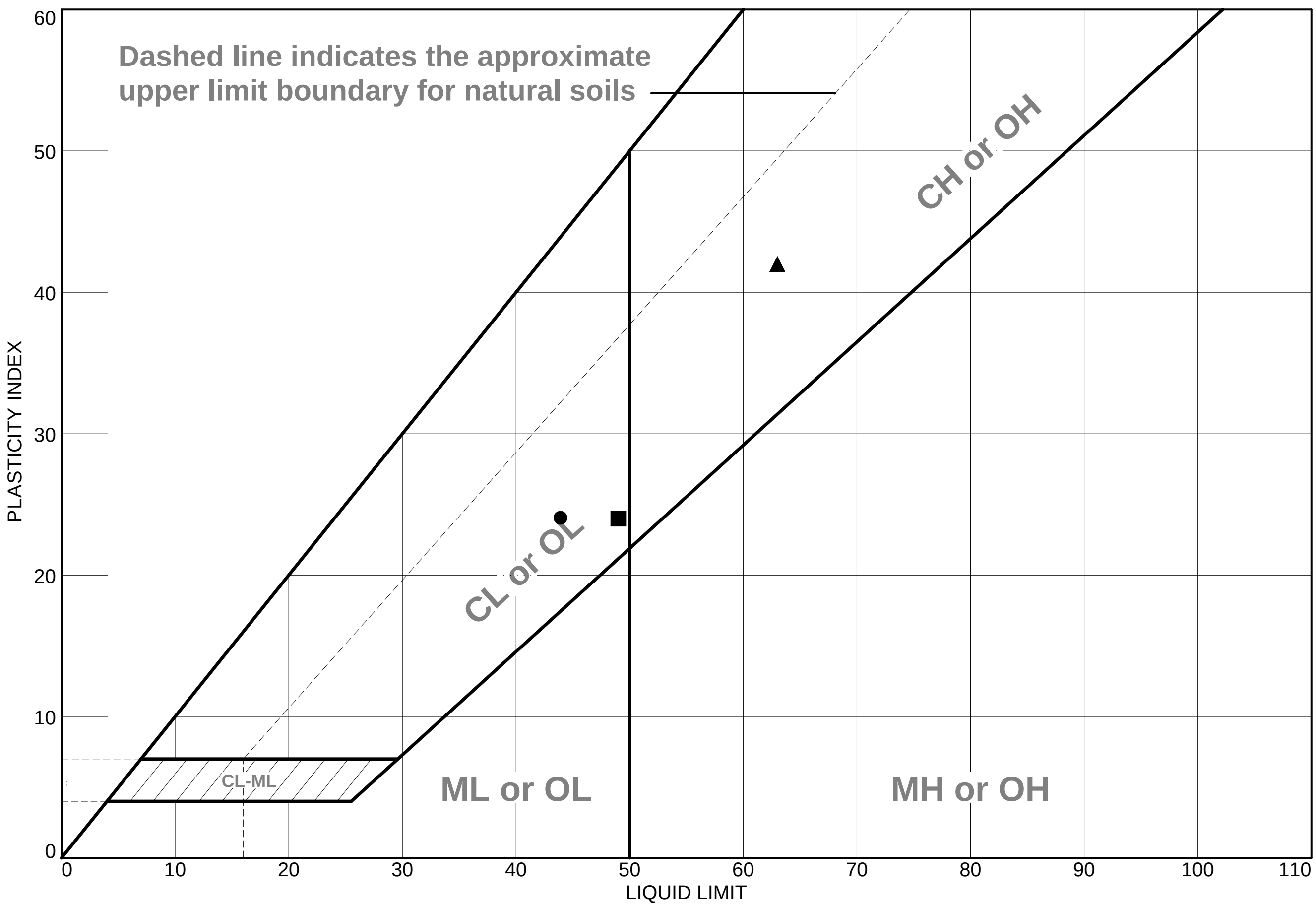
Remarks:

B. HILLEBRANDT SOILS TESTING, INC.
+1 510-409-2816
SoilTesting@aol.com

Figure

Tested By: BH

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark gray clayey GRAVEL with sand	44	20	24	52.1	37.9	GC
■	Grayish brown clayey GRAVEL with sand	49	25	24	31.3	20.8	GC
▲	Dark brown fat CLAY	63	21	42	96.1	85.1	CH

Project No. 1100-17B **Client:** A3Geo

Project: IGB

●**Source of Sample:** B-7 **Depth:** 3.0'
■**Source of Sample:** B-7 **Depth:** 10.5'
▲**Source of Sample:** B-7 **Depth:** 15.0'

Remarks:

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Figure

Tested By: BH

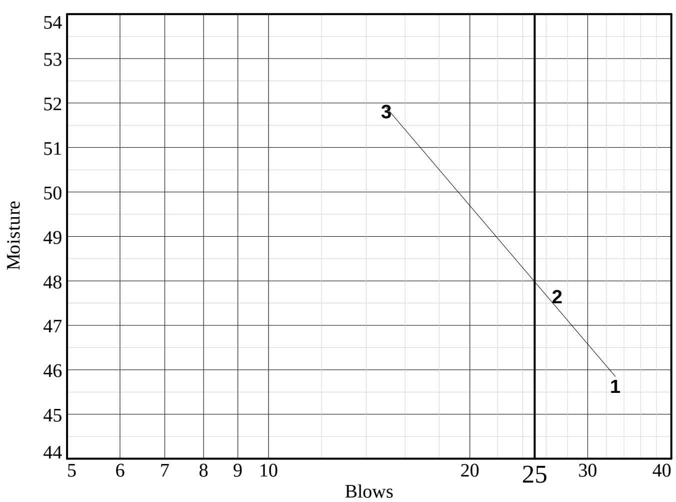
LIQUID AND PLASTIC LIMIT TEST DATA

9/28/2014

Client: A3Geo
Project: IGB
Project Number: 1100-17B
Location: B-6
Depth: 5.5'
Material Description: Dark grayish brown sandy lean CLAY with gravel
%<#40: 65.5 **%<#200:** 51.7 **USCS:** CL **AASHTO:** A-7-6(11)
Tested by: BH

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	29.84	32.15	30.83			
Dry+Tare	24.02	25.43	24.17			
Tare	11.27	11.33	11.32			
# Blows	33	27	15			
Moisture	45.6	47.7	51.8			



Liquid Limit= 48
Plastic Limit= 18
Plasticity Index= 30

Plastic Limit Data

Run No.	1	2	3	4
Wet+Tare	16.98	17.73		
Dry+Tare	16.08	16.77		
Tare	11.10	11.30		
Moisture	18.1	17.6		

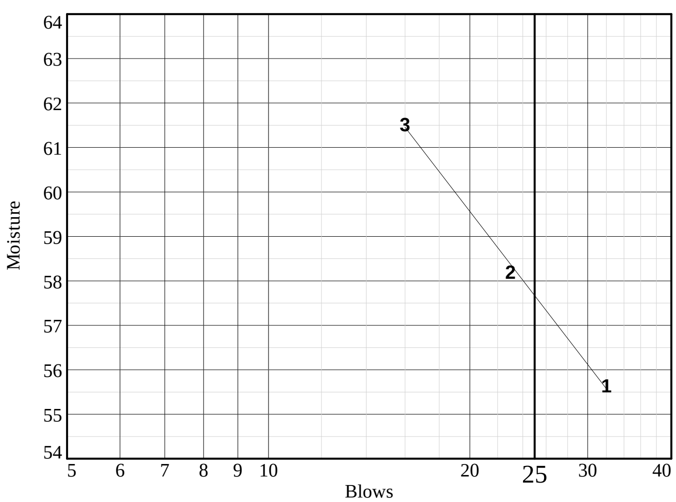
LIQUID AND PLASTIC LIMIT TEST DATA

9/28/2014

Client: A3Geo
Project: IGB
Project Number: 1100-17B
Location: B-6
Depth: 11.0'
Material Description: Grayish brown fat CLAY with sand
%<#40: 94.1 **%<#200:** 77.0 **USCS:** CH **AASHTO:** A-7-6(31)
Tested by: BH

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	30.16	29.09	27.09			
Dry+Tare	23.42	22.50	20.98			
Tare	11.31	11.18	11.05			
# Blows	32	23	16			
Moisture	55.7	58.2	61.5			



Liquid Limit= 58
Plastic Limit= 18
Plasticity Index= 40

Plastic Limit Data

Run No.	1	2	3	4
Wet+Tare	17.00	17.42		
Dry+Tare	16.10	16.53		
Tare	11.26	11.33		
Moisture	18.6	17.1		

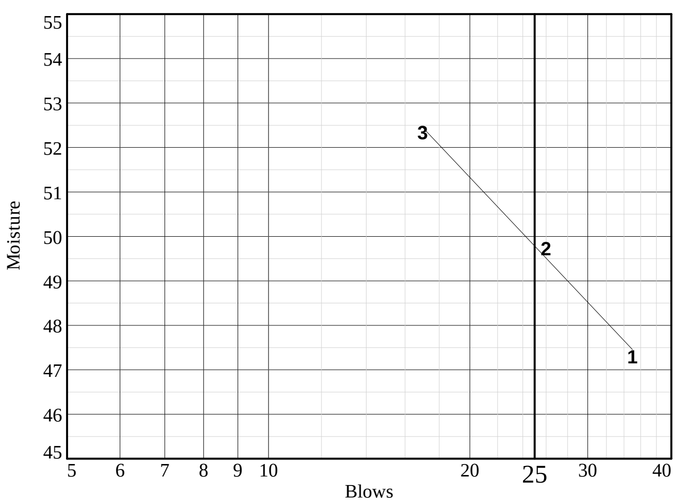
LIQUID AND PLASTIC LIMIT TEST DATA

9/28/2014

Client: A3Geo
Project: IGB
Project Number: 1100-17B
Location: B-6
Depth: 15.0'
Material Description: Dark brown to dark gray gravelly fat CLAY with sand
%<#40: 70.7 **%<#200:** 58.2 **USCS:** CH **AASHTO:** A-7-6(15)
Tested by: BH

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	26.95	26.03	29.74			
Dry+Tare	21.84	21.12	23.4			
Tare	11.04	11.25	11.29			
# Blows	35	26	17			
Moisture	47.3	49.7	52.4			



Liquid Limit= 50
Plastic Limit= 18
Plasticity Index= 32

Plastic Limit Data

Run No.	1	2	3	4
Wet+Tare	17.65	16.72		
Dry+Tare	16.67	15.86		
Tare	11.18	11.3		
Moisture	17.9	18.9		

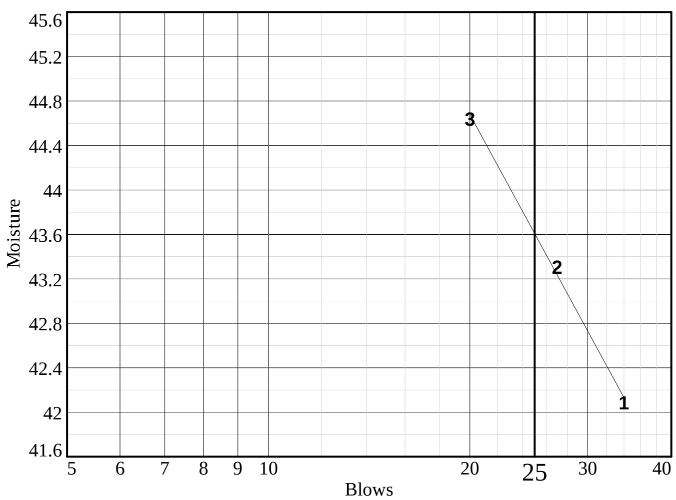
LIQUID AND PLASTIC LIMIT TEST DATA

9/28/2014

Client: A3Geo
Project: IGB
Project Number: 1100-17B
Location: B-7
Depth: 3.0'
Material Description: Dark gray clayey GRAVEL with sand
%<#40: 52.1 **%<#200:** 37.9 **USCS:** GC **AASHTO:** A-7-6(4)
Tested by: BH

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	25.97	31.52	29.36			
Dry+Tare	21.58	25.40	23.73			
Tare	11.15	11.27	11.12			
# Blows	34	27	20			
Moisture	42.1	43.3	44.6			



Liquid Limit= 44
Plastic Limit= 20
Plasticity Index= 24

Plastic Limit Data

Run No.	1	2	3	4	
Wet+Tare	17.10	16.95			
Dry+Tare	16.15	15.98			
Tare	11.19	11.27			
Moisture	19.2	20.6			

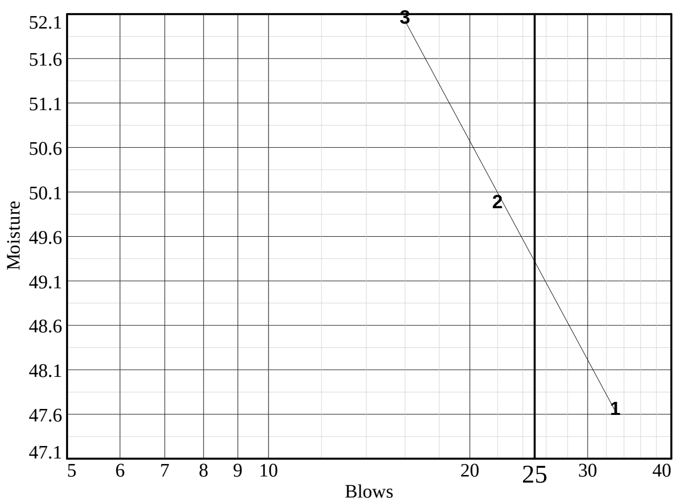
LIQUID AND PLASTIC LIMIT TEST DATA

9/28/2014

Client: A3Geo
Project: IGB
Project Number: 1100-17B
Location: B-7
Depth: 10.5'
Material Description: Grayish brown clayey GRAVEL with sand
%<#40: 31.3 **%<#200:** 20.8 **USCS:** GC **AASHTO:** A-2-7(1)
Tested by: BH

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	30.04	28.01	27.21			
Dry+Tare	23.99	22.40	21.69			
Tare	11.30	11.18	11.09			
# Blows	33	22	16			
Moisture	47.7	50.0	52.1			



Liquid Limit= 49
Plastic Limit= 25
Plasticity Index= 24

Plastic Limit Data

Run No.	1	2	3	4
Wet+Tare	17.65	17.30		
Dry+Tare	16.35	16.13		
Tare	11.30	11.29		
Moisture	25.7	24.2		

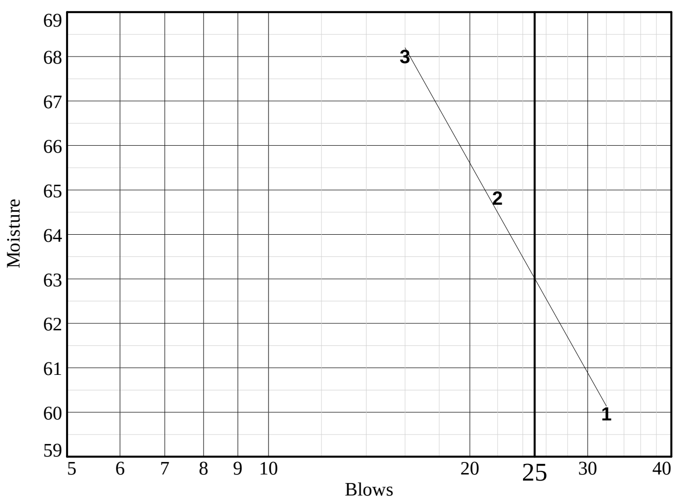
LIQUID AND PLASTIC LIMIT TEST DATA

9/28/2014

Client: A3Geo
Project: IGB
Project Number: 1100-17B
Location: B-7
Depth: 15.0'
Material Description: Dark brown fat CLAY
%<#40: 96.1 **%<#200:** 85.1 **USCS:** CH **AASHTO:** A-7-6(38)
Tested by: BH

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	30.33	28.98	27.06			
Dry+Tare	23.21	22.03	20.68			
Tare	11.34	11.31	11.30			
# Blows	32	22	16			
Moisture	60.0	64.8	68.0			

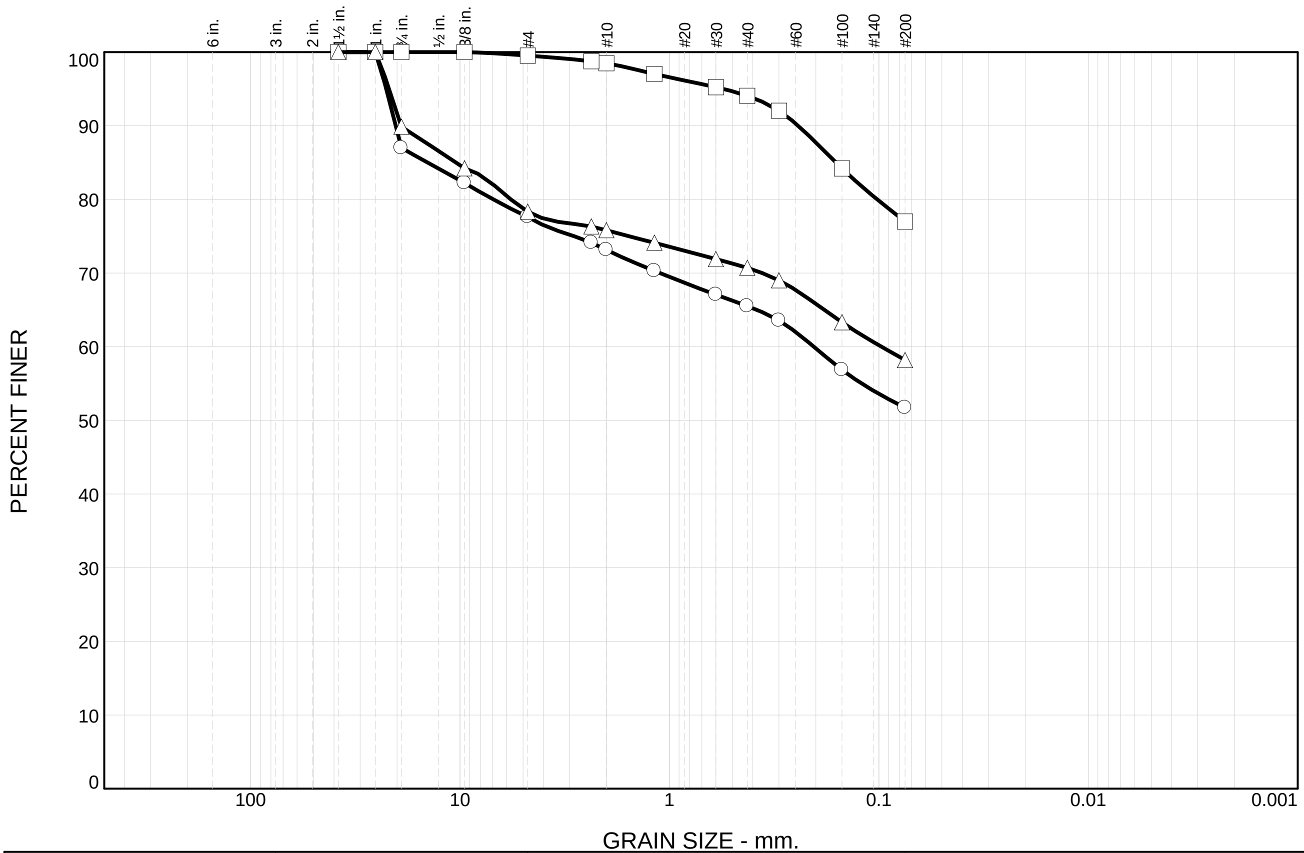


Liquid Limit= 63
Plastic Limit= 21
Plasticity Index= 42

Plastic Limit Data

Run No.	1	2	3	4
Wet+Tare	17.27	17.07		
Dry+Tare	16.21	16.06		
Tare	11.25	11.06		
Moisture	21.4	20.2		

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

MATERIAL DATA					
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-6		5.5'	Dark grayish brown sandy lean CLAY with gravel	CL
□	B-6		11.0'	Grayish brown fat CLAY with sand	CH
△	B-6		15.0'	Dark brown to dark gray gravelly fat CLAY with sand	CH

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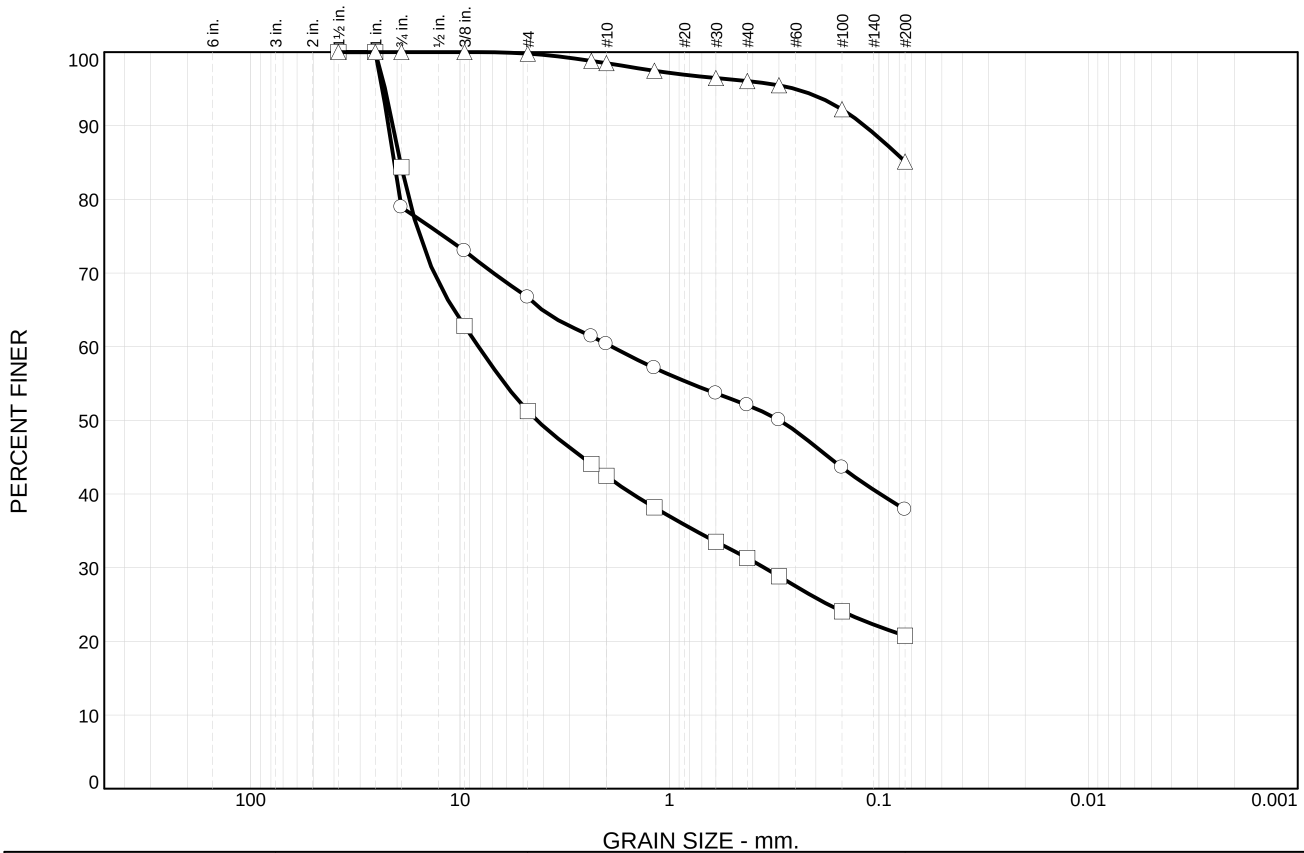
Client: A3Geo
Project: IGB

Project No.: 1100-17B **Figure**

Tested By: BH

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

MATERIAL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-7		3.0'	Dark gray clayey GRAVEL with sand	GC
□	B-7		10.5'	Grayish brown clayey GRAVEL with sand	GC
△	B-7		15.0'	Dark brown fat CLAY	CH

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Client: A3Geo
Project: IGB

Project No.: 1100-17B **Figure**

Tested By: BH

GRAIN SIZE DISTRIBUTION TEST DATA

9/29/2014

Client: A3Geo
Project: IGB
Project Number: 1100-17B
Location: B-6
Depth: 5.5'
Material Description: Dark grayish brown sandy lean CLAY with gravel
USCS: CL
Tested by: BH

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer
263.60	38.30	1.5"	0.00	0.00	100.0
		1"	0.00	0.00	100.0
		3/4"	29.30	0.00	87.0
		3/8"	10.70	0.00	82.2
		#4	10.40	0.00	77.6
		#8	7.90	0.00	74.1
		#10	2.20	0.00	73.1
		#16	6.40	0.00	70.3
		#30	7.30	0.00	67.1
		#40	3.50	0.00	65.5
		#50	4.40	0.00	63.6
		#100	15.10	0.00	56.9
		#200	11.60	0.00	51.7

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	13.0	9.4	22.4	4.5	7.6	13.8	25.9			51.7

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
					0.2044	6.9340	14.2755	20.3719	22.5262

Fineness Modulus
2.21

GRAIN SIZE DISTRIBUTION TEST DATA

9/29/2014

Client: A3Geo
Project: IGB
Project Number: 1100-17B
Location: B-6
Depth: 11.0'
Material Description: Grayish brown fat CLAY with sand
USCS: CH
Tested by: BH

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer
672.90	272.20	1.5"	0.00	0.00	100.0
		1"	0.00	0.00	100.0
		3/4"	0.00	0.00	100.0
		3/8"	0.00	0.00	100.0
		#4	1.90	0.00	99.5
		#8	3.10	0.00	98.8
		#10	1.10	0.00	98.5
		#16	5.80	0.00	97.0
		#30	7.20	0.00	95.2
		#40	4.70	0.00	94.1
		#50	8.10	0.00	92.0
		#100	31.40	0.00	84.2
		#200	28.90	0.00	77.0

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.5	0.5	1.0	4.4	17.1	22.5			77.0

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
						0.1019	0.1603	0.2428	0.5543

Fineness Modulus
0.33

GRAIN SIZE DISTRIBUTION TEST DATA

9/29/2014

Client: A3Geo
Project: IGB
Project Number: 1100-17B
Location: B-6
Depth: 15.0'
Material Description: Dark brown to dark gray gravelly fat CLAY with sand
USCS: CH
Tested by: BH

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer
686.90	275.50	1.5"	0.00	0.00	100.0
		1"	0.00	0.00	100.0
		3/4"	41.70	0.00	89.9
		3/8"	23.20	0.00	84.2
		#4	24.20	0.00	78.3
		#8	8.30	0.00	76.3
		#10	2.10	0.00	75.8
		#16	7.00	0.00	74.1
		#30	9.10	0.00	71.9
		#40	4.90	0.00	70.7
		#50	7.00	0.00	69.0
		#100	23.40	0.00	63.3
		#200	21.10	0.00	58.2

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	10.1	11.6	21.7	2.5	5.1	12.5	20.1			58.2

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
					0.0972	5.7179	10.4609	19.1304	21.8728

Fineness Modulus
1.93

GRAIN SIZE DISTRIBUTION TEST DATA

9/29/2014

Client: A3Geo
Project: IGB
Project Number: 1100-17B
Location: B-7
Depth: 3.0'
Material Description: Dark gray clayey GRAVEL with sand
USCS: GC
Tested by: BH

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer
612.00	275.50	1.5"	0.00	0.00	100.0
		1"	0.00	0.00	100.0
		3/4"	70.90	0.00	78.9
		3/8"	19.90	0.00	73.0
		#4	21.20	0.00	66.7
		#8	17.80	0.00	61.4
		#10	3.50	0.00	60.4
		#16	11.00	0.00	57.1
		#30	11.60	0.00	53.7
		#40	5.40	0.00	52.1
		#50	6.80	0.00	50.0
		#100	21.70	0.00	43.6
		#200	19.20	0.00	37.9

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	21.1	12.2	33.3	6.3	8.3	14.2	28.8			37.9

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
				0.2982	1.8832	19.3589	20.6934	22.0089	23.4670

Fineness Modulus
3.15

GRAIN SIZE DISTRIBUTION TEST DATA

9/29/2014

Client: A3Geo
Project: IGB
Project Number: 1100-17B
Location: B-7
Depth: 10.5'
Material Description: Grayish brown clayey GRAVEL with sand
USCS: GC
Tested by: BH

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer
595.90	278.50	1.5"	0.00	0.00	100.0
		1"	0.00	0.00	100.0
		3/4"	49.60	0.00	84.4
		3/8"	68.40	0.00	62.8
		#4	36.70	0.00	51.3
		#8	22.80	0.00	44.1
		#10	5.10	0.00	42.5
		#16	13.60	0.00	38.2
		#30	14.80	0.00	33.5
		#40	7.00	0.00	31.3
		#50	7.90	0.00	28.8
		#100	15.10	0.00	24.1
		#200	10.50	0.00	20.8

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	15.6	33.1	48.7	8.8	11.2	10.5	30.5			20.8

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
			0.3521	4.2930	8.1643	17.5094	19.2613	20.9397	22.7982

Fineness Modulus
4.33

GRAIN SIZE DISTRIBUTION TEST DATA

9/29/2014

Client: A3Geo
Project: IGB
Project Number: 1100-17B
Location: B-7
Depth: 15.0'
Material Description: Dark brown fat CLAY
USCS: CH
Tested by: BH

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Sieve Opening Size	Weight Retained (grams)	Sieve Weight (grams)	Percent Finer
231.20	38.00	1.5"	0.00	0.00	100.0
		1"	0.00	0.00	100.0
		3/4"	0.00	0.00	100.0
		3/8"	0.00	0.00	100.0
		#4	0.40	0.00	99.8
		#8	1.90	0.00	98.8
		#10	0.60	0.00	98.5
		#16	2.00	0.00	97.5
		#30	1.90	0.00	96.5
		#40	0.80	0.00	96.1
		#50	1.10	0.00	95.5
		#100	6.30	0.00	92.2
		#200	13.70	0.00	85.1

Fractional Components

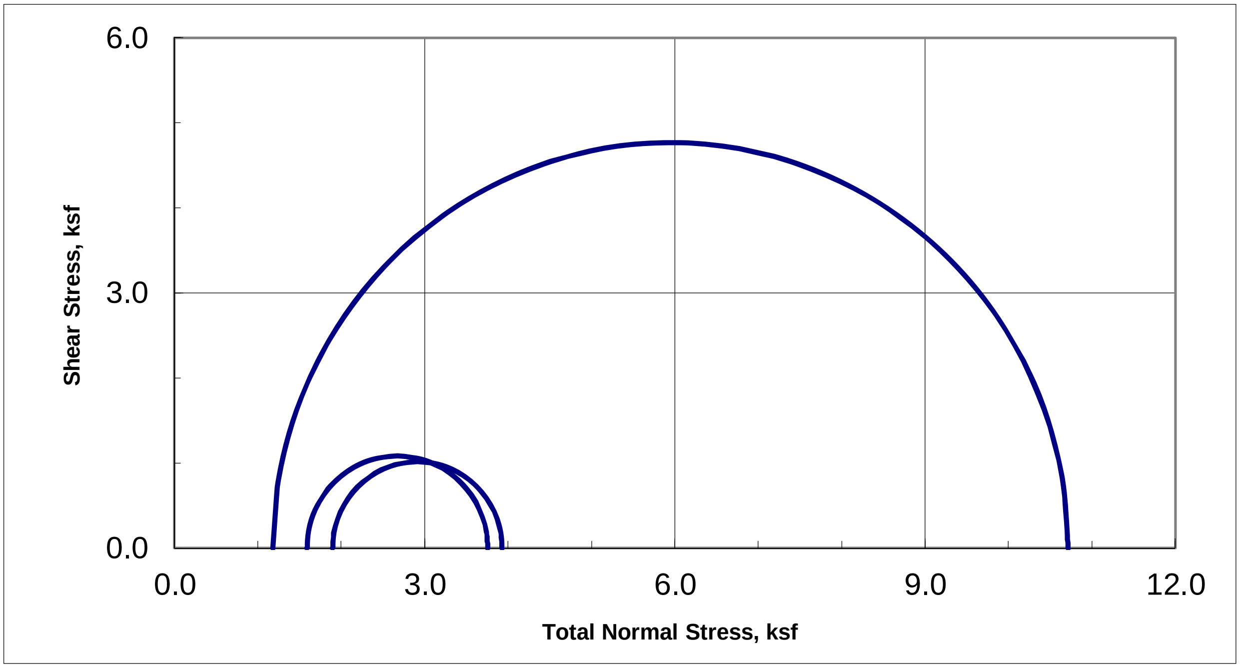
Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.2	0.2	1.3	2.4	11.0	14.7			85.1

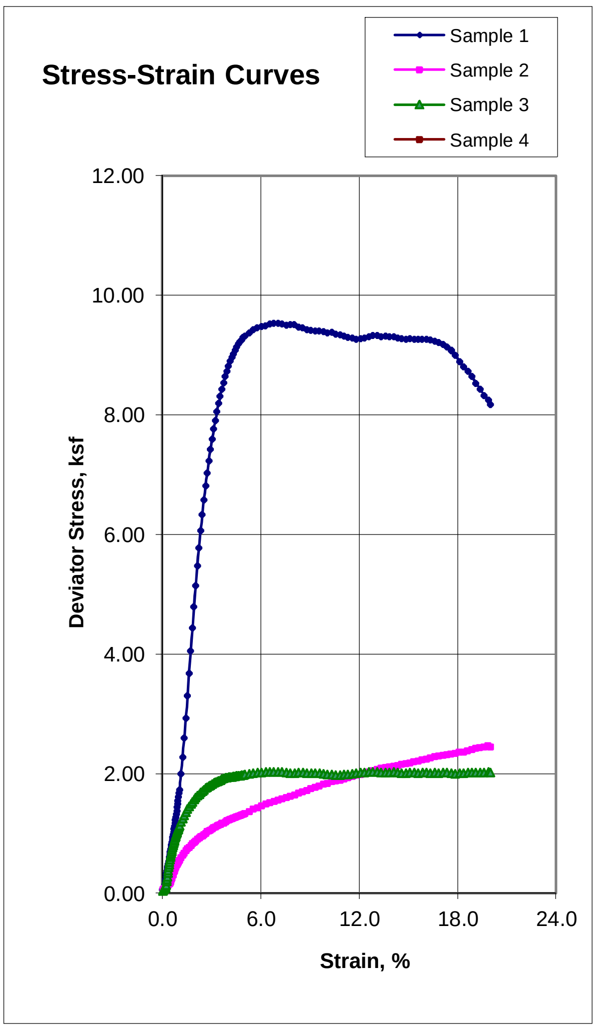
D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
								0.1168	0.2509

Fineness Modulus
0.20

Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

Unconsolidated-Undrained Triaxial Test
ASTM D2850





Sample Data				
	1	2	3	4
Moisture %	10.2	18.2	18.7	
Dry Den,pcf	127.9	111.8	113.0	
Void Ratio	0.318	0.508	0.519	
Saturation %	86.3	96.6	98.9	
Height in	6.01	6.01	5.99	
Diameter in	2.88	2.88	2.88	
Cell psi	8.2	11.0	13.2	
Strain %	6.79	15.00	6.81	
Deviator, ksf	9.536	2.168	2.033	
Rate %/min	1.00	1.00	1.00	
in/min	0.060	0.060	0.060	
Job No.:	**748-016**			
Client:	**A3Geo, Inc.**			
Project:	**LBNL-IGB - 100-17B**			
Boring:	B2	B4	B4	
Sample:				
Depth ft:	15.5-18(Tip-8")	22.5-25(Tip-14")	25-27.5(Tip-7")	

Visual Soil Description	
Sample #	
1	Olive Silty SAND (slightly plastic)
2	Bluish Gray Sandy CLAY/ near Clayey SAND, trace Gravel
3	Greenish Gray Sandy CLAY/ Change to Drk Gr CLAY w/Sa
4	

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

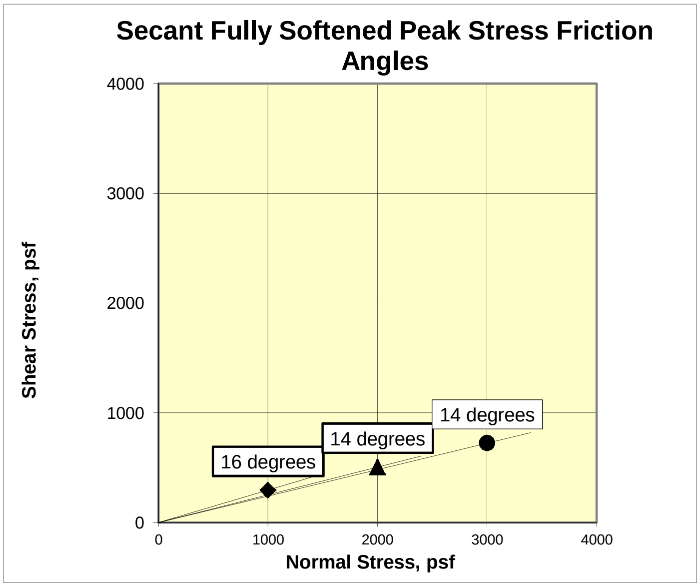
Drained Residual Torsional Shear Strength (ASTM D6467)

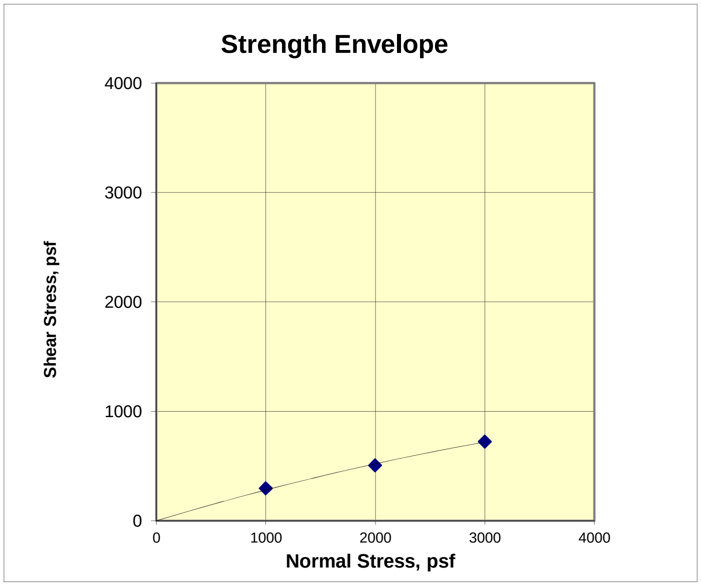
CTL Job No.:	748-016b	**Boring:**	B2	**Date:**	8/28/2014	**Clay, %:**	
Client:	A3Geo	**Sample:**		**By:**	PJ	**LL:**	
Project Name:	LBNL-IGB	**Depth (ft):**	12-12.5'	**Checked:**	DC	**PL:**	
Project Number:	1100-17B	**Test Type:**	Fully Softened Residual				

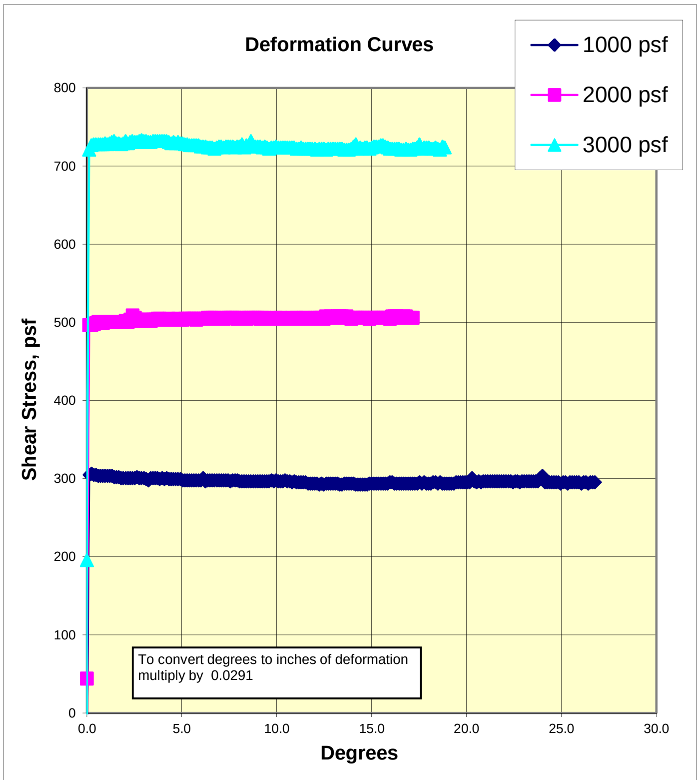
Soil Type: Dark Reddish Brown CLAY, trace sand

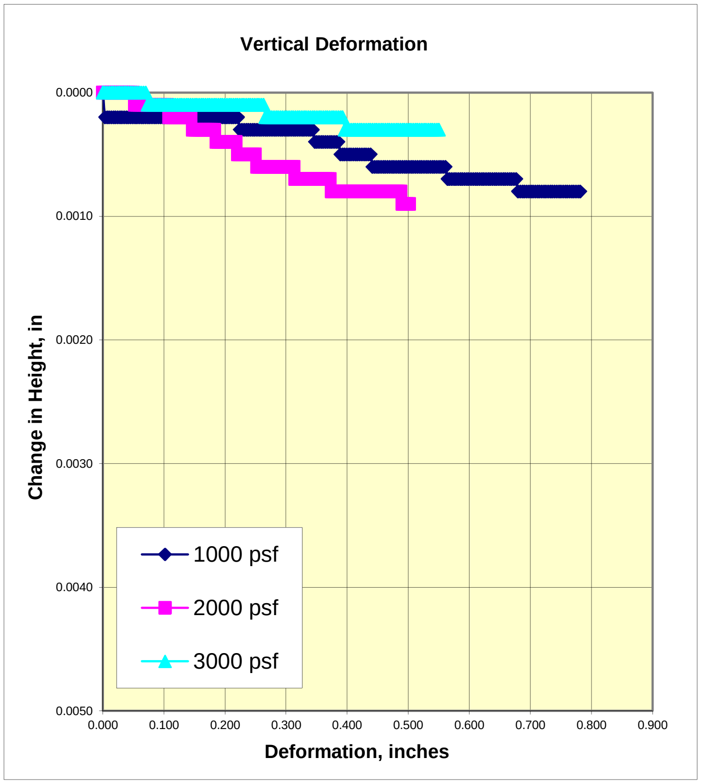
Normal Stress, psf:	1000	2000	3000
Secant Phi, deg.:	16	14	14

Remarks: A small friction correction was applied to each point.









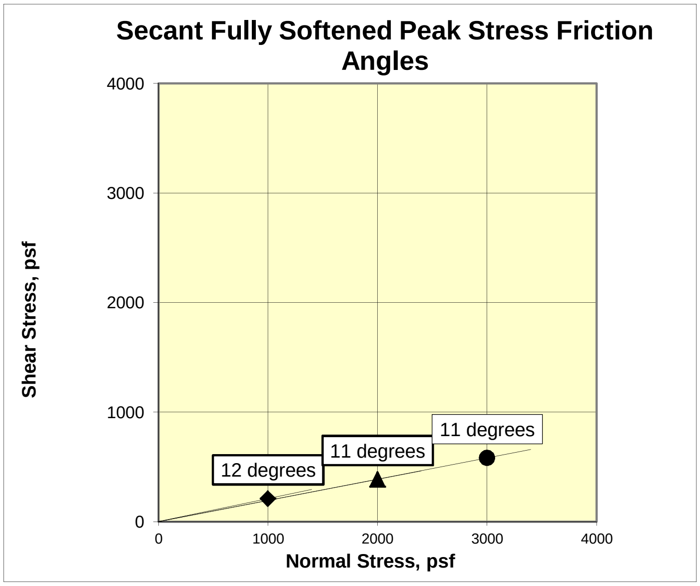
Drained Residual Torsional Shear Strength (ASTM D6467)

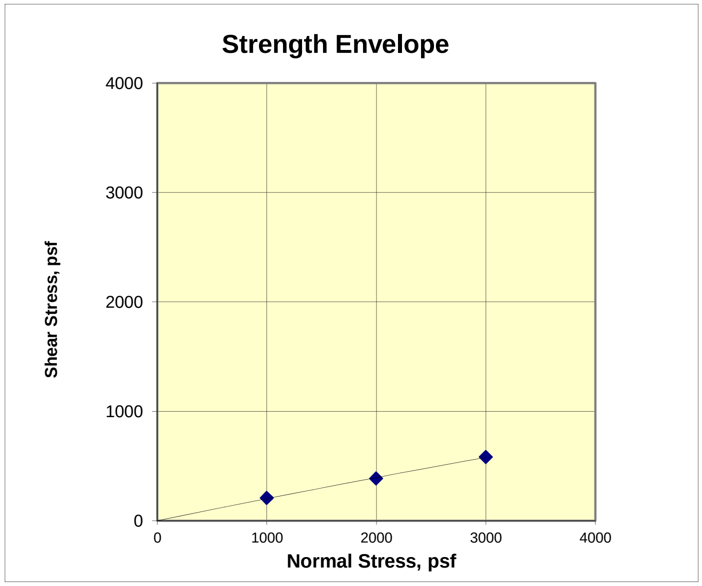
CTL Job No.:	748-016a	**Boring:**	B1	**Date:**	8/28/2014	**Clay, %:**	
Client:	A3Geo	**Sample:**		**By:**	PJ	**LL:**	
Project Name:	LBNL-IGB	**Depth (ft):**	21-21.6	**Checked:**	DC	**PL:**	
Project Number:	1100-17B	**Test Type:**	Fully Softened Residual				

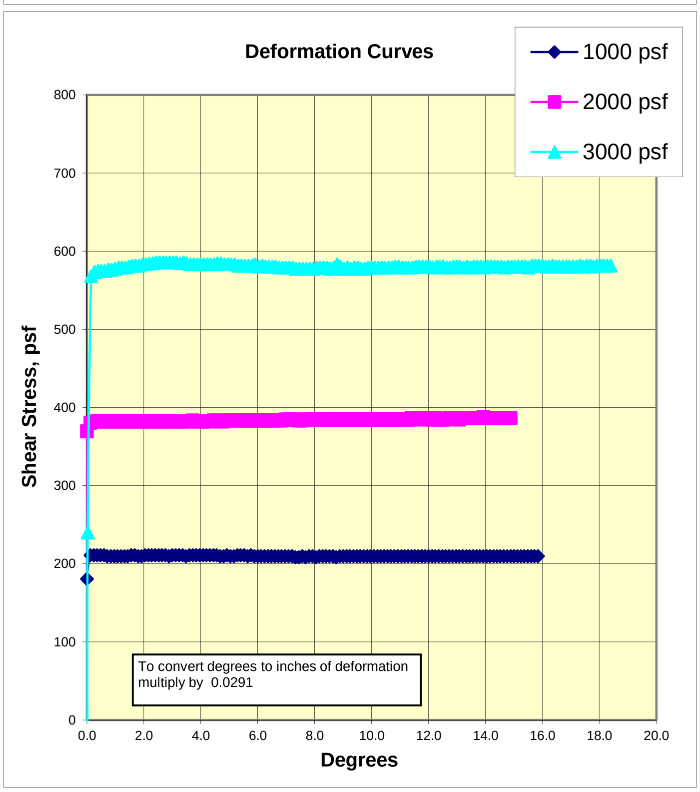
Soil Type: Reddish Brown CLAY

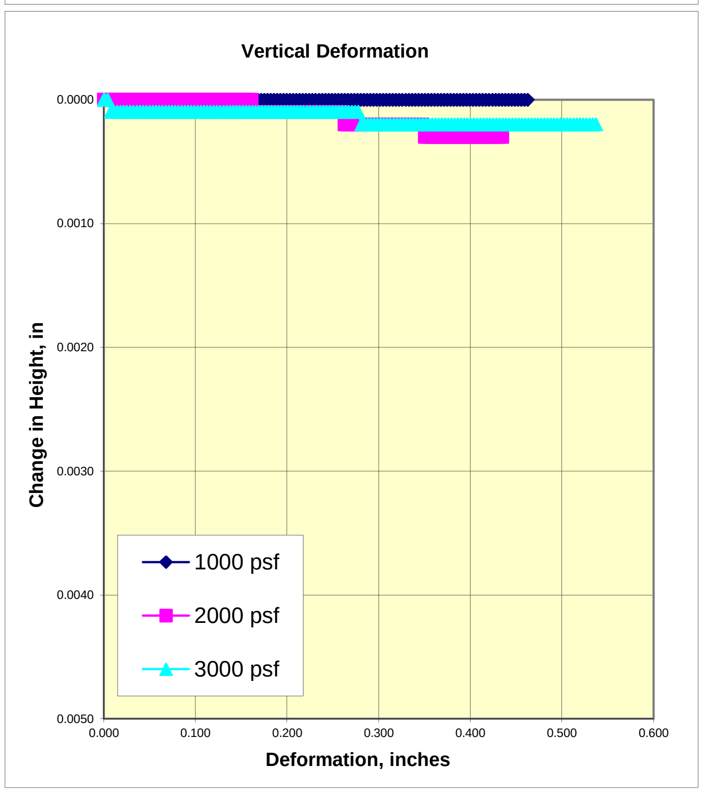
Normal Stress, psf:	1000	2000	3000
Secant Phi, deg.:	12	11	11

Remarks: A small friction correction was applied to each point.







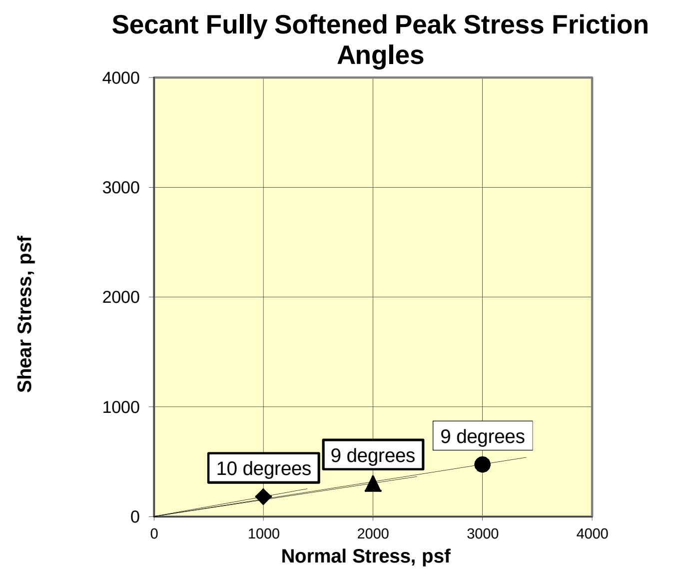


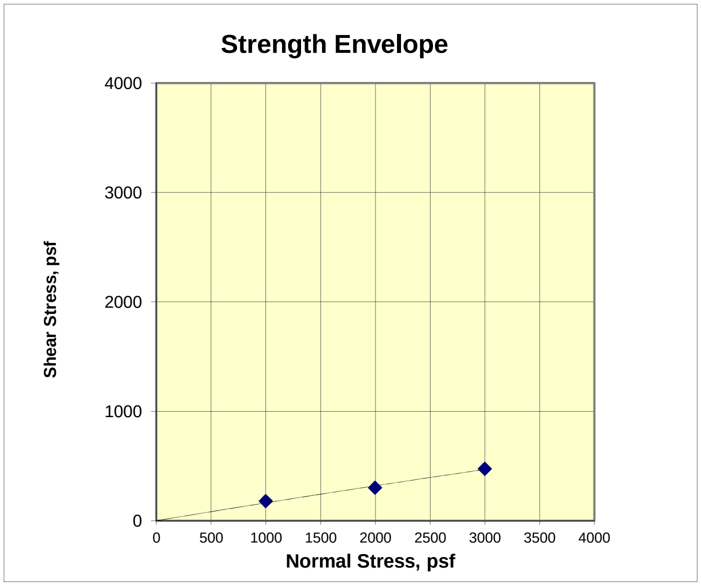
Drained Residual Torsional Shear Strength (ASTM D6467)

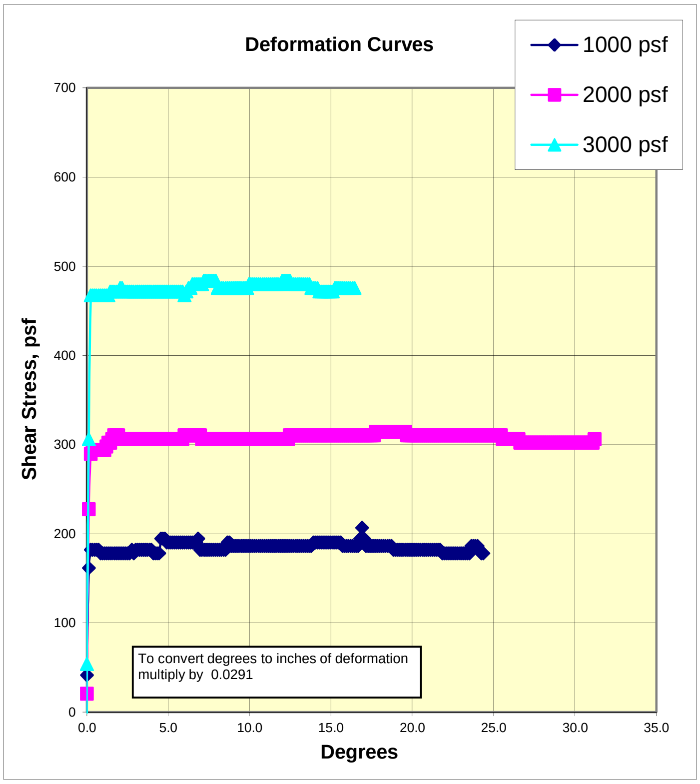
CTL Job No.:	748-016d	**Boring:**	B4	**Date:**	9/3/2014	**Clay, %:**	
Client:	A3GEO, Inc.	**Sample:**		**By:**	PJ	**LL:**	
Project Name:	LBNL - IGB	**Depth (ft):**	18-18.5	**Checked:**	DC	**PL:**	
Project Number:	1100-17B	**Test Type:**	Fully Softened Residual				

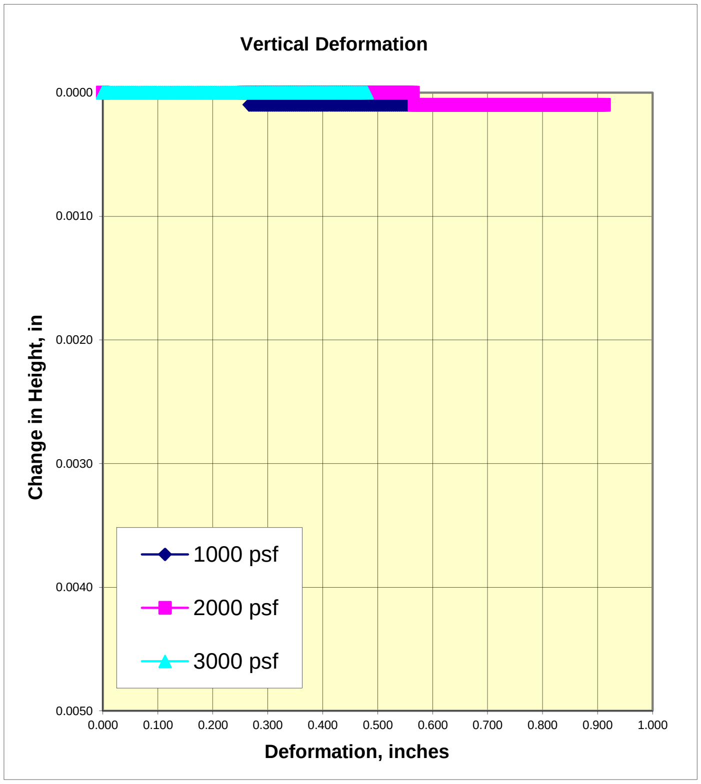
Soil Type:	Dark Brown CLAY		
Normal Stress, psf:	1000	2000	3000
Secant Phi, deg.:	10	9	9

Remarks: A small friction correction was applied to each point.









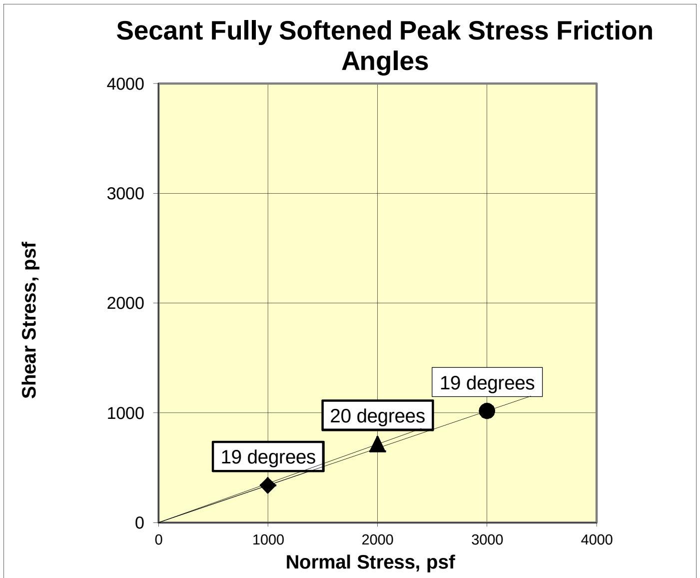
Drained Fully Softened Peak Torsional Shear Strength (ASTM D7608)

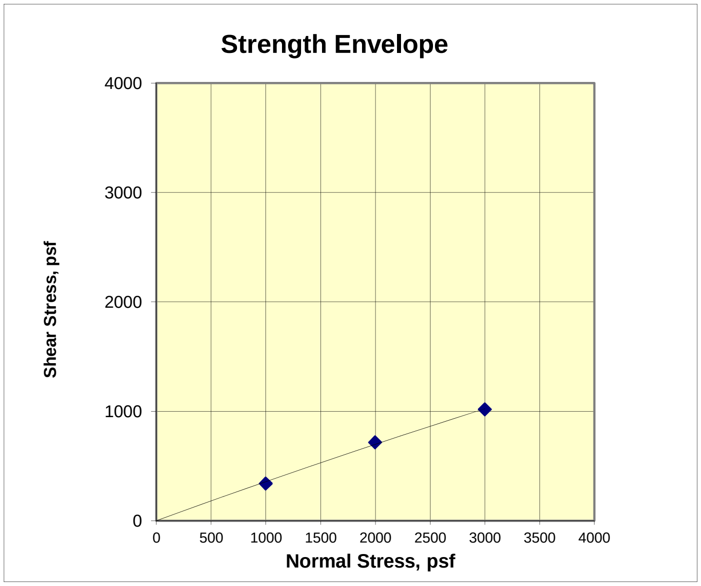
CTL Job No.:	748-016c	**Boring:**	B4	**Date:**	9/3/2014	**Clay, %:**	
Client:	A3GEO, Inc.	**Sample:**		**By:**	PJ	**LL:**	
Project Name:	LBNL - IGB	**Depth (ft):**	18-18.5	**Checked:**	DC	**PL:**	
Project Number:	1100-17B	**Test Type:**	Fully Softened Peak				

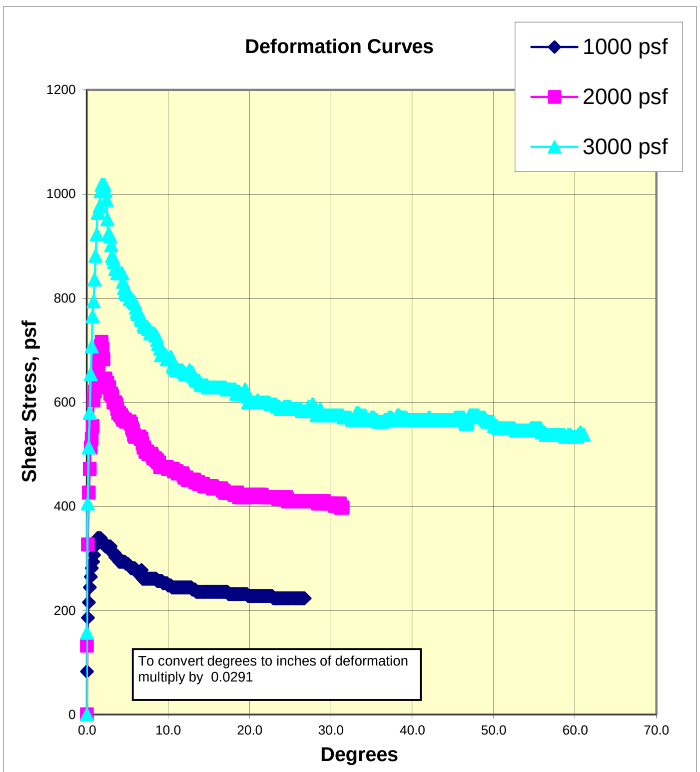
Soil Type: Dark Brown CLAY

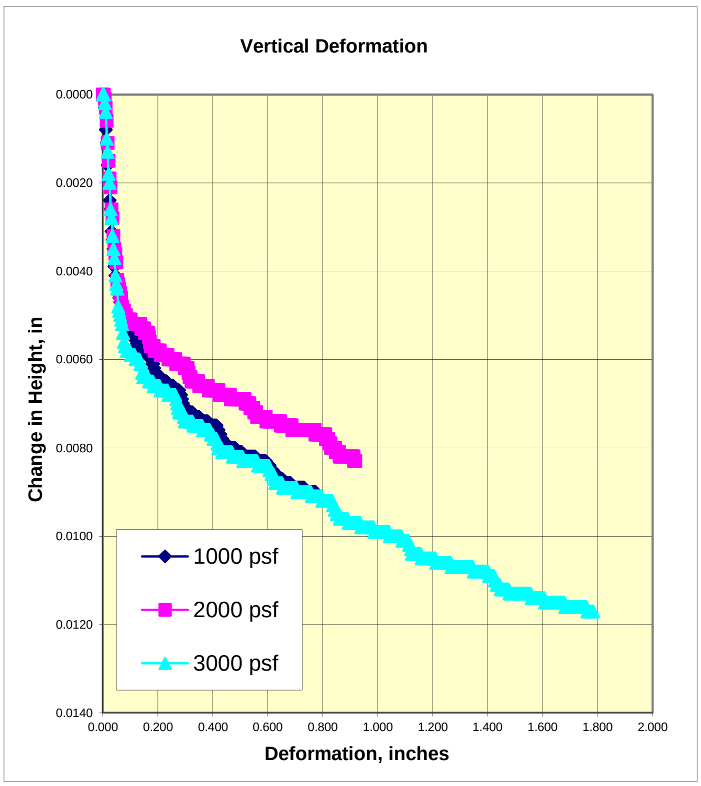
Remarks:

Normal Stress, psf:	1000	2000	3000
Secant Phi, deg.:	19	20	19





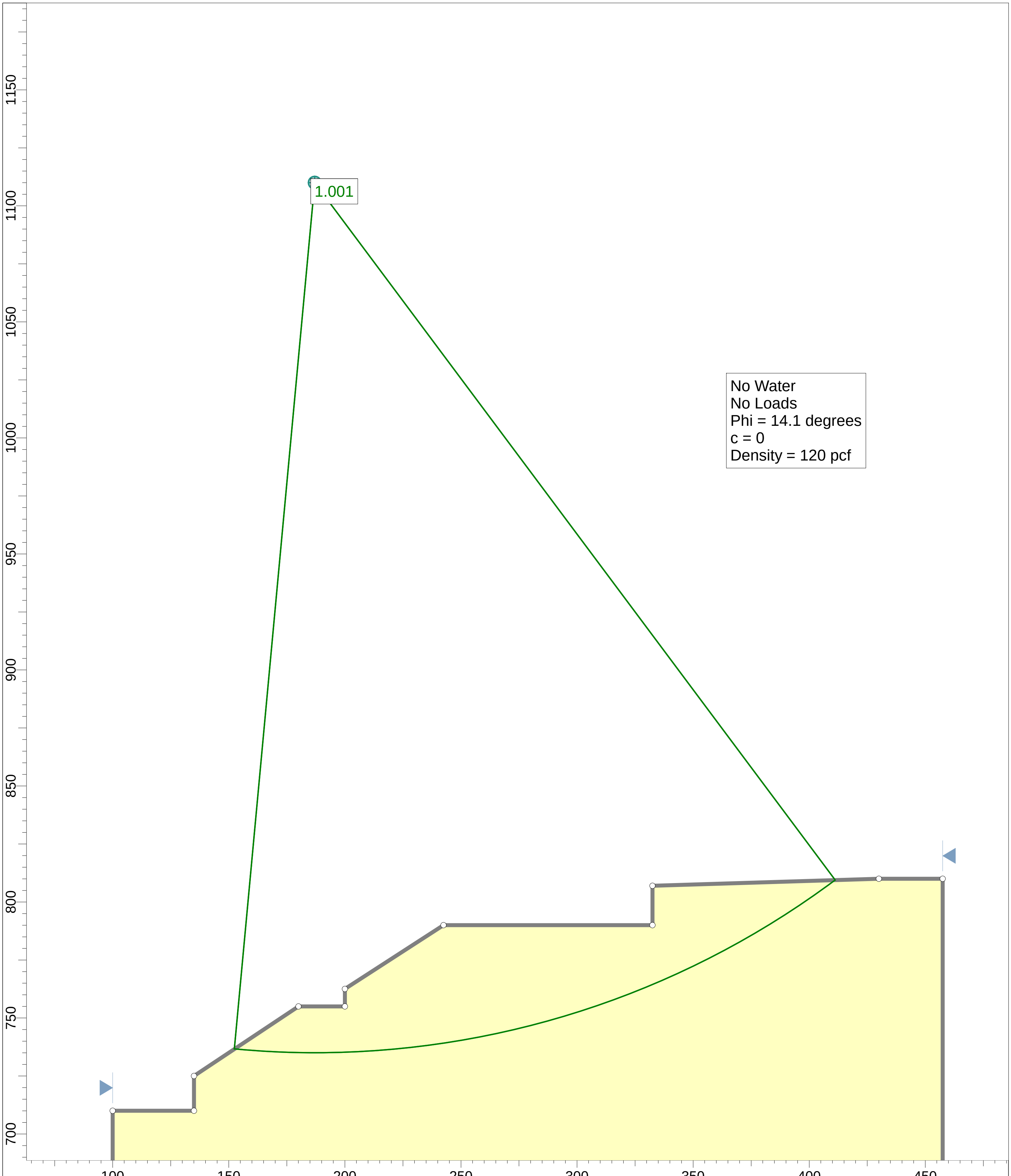




Appendix D

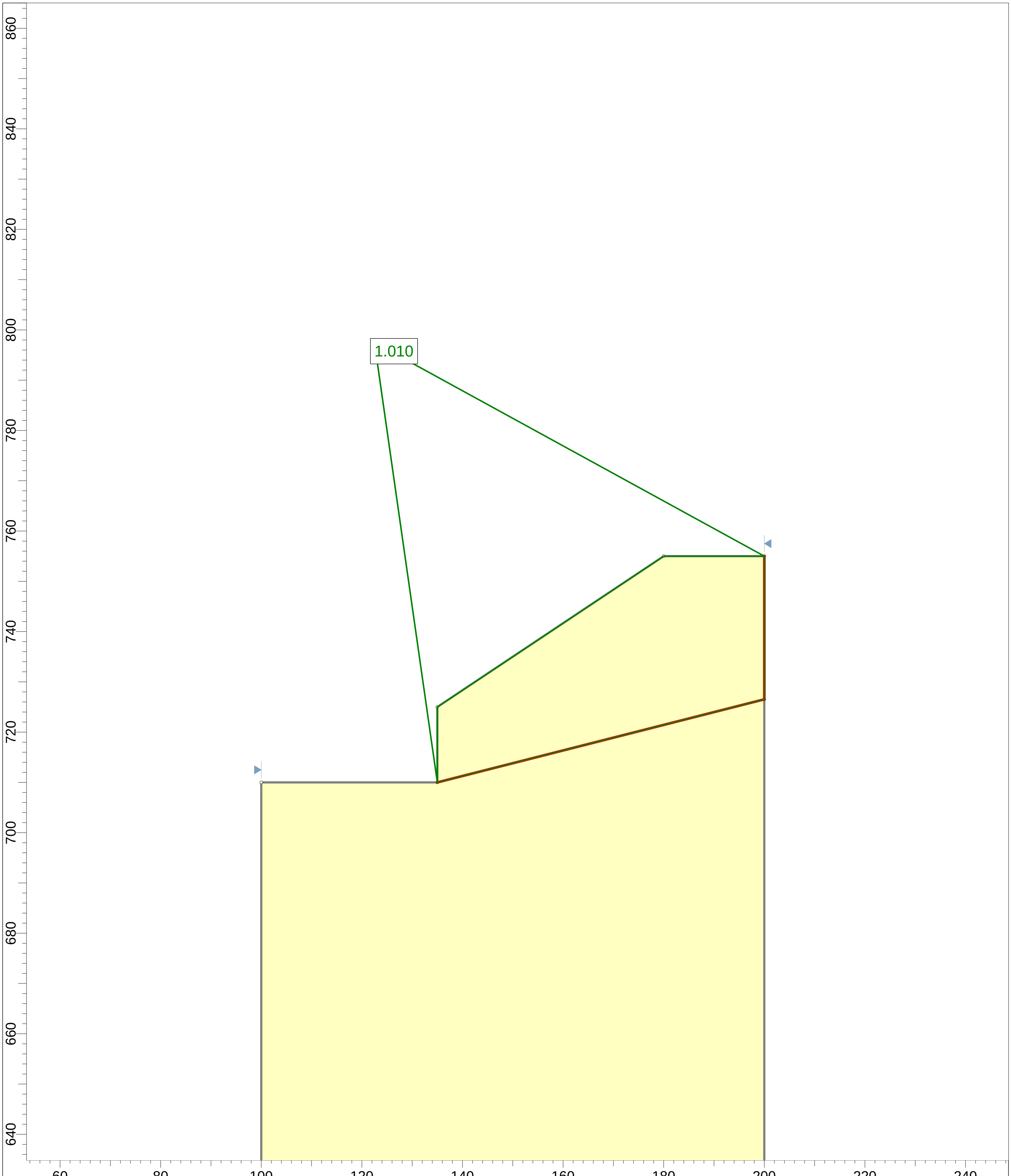
Slope Stability and Seismic Displacement Analyses

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

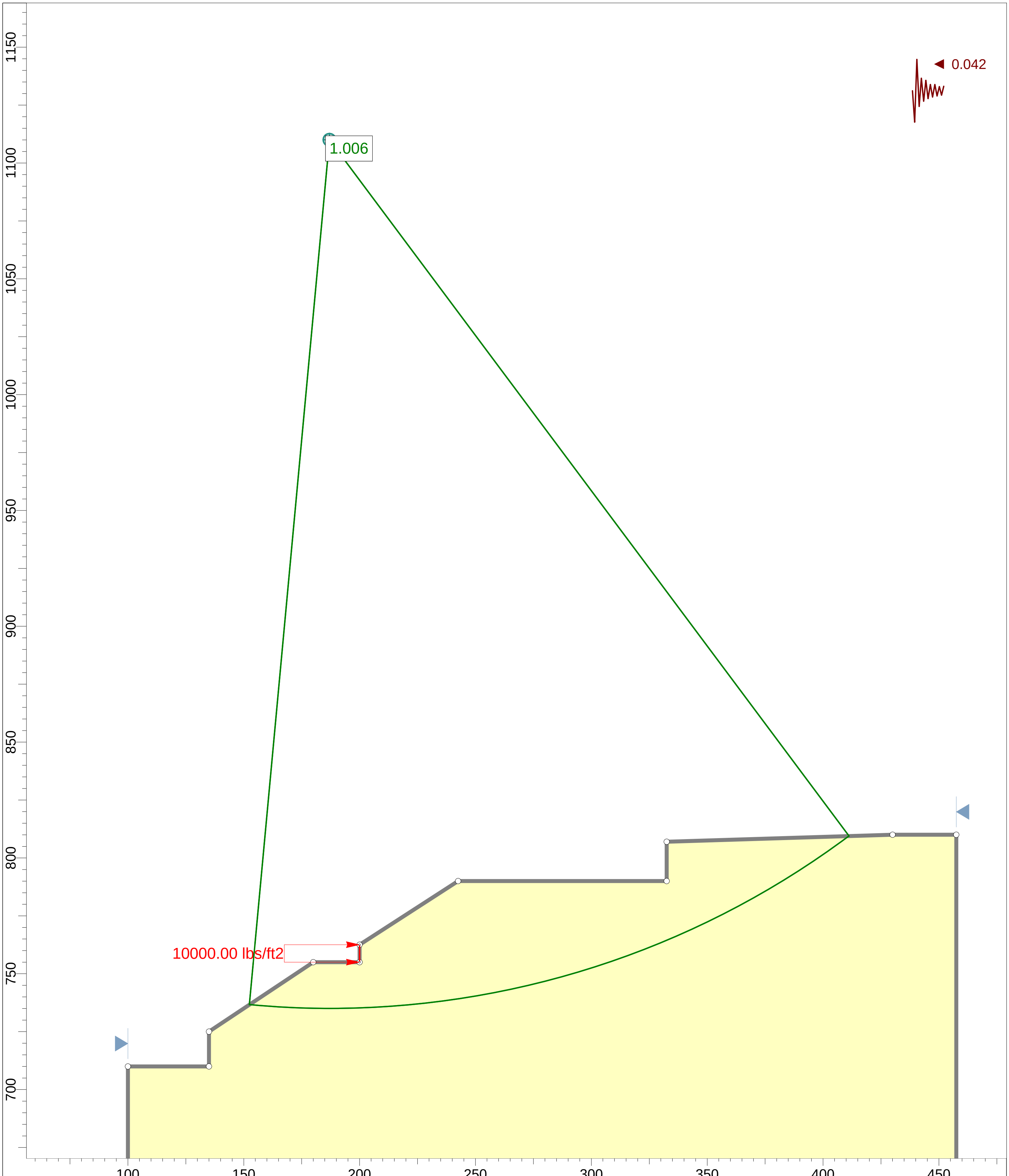


Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description			
Drawn By		Scale 1:650	Company
Date 9/2/2014, 12:16:50 PM		File Name CASE A - (FS=1, No H20, No Loads, c=0, D=120).slim	

SLIDEINTERPRET 6.029

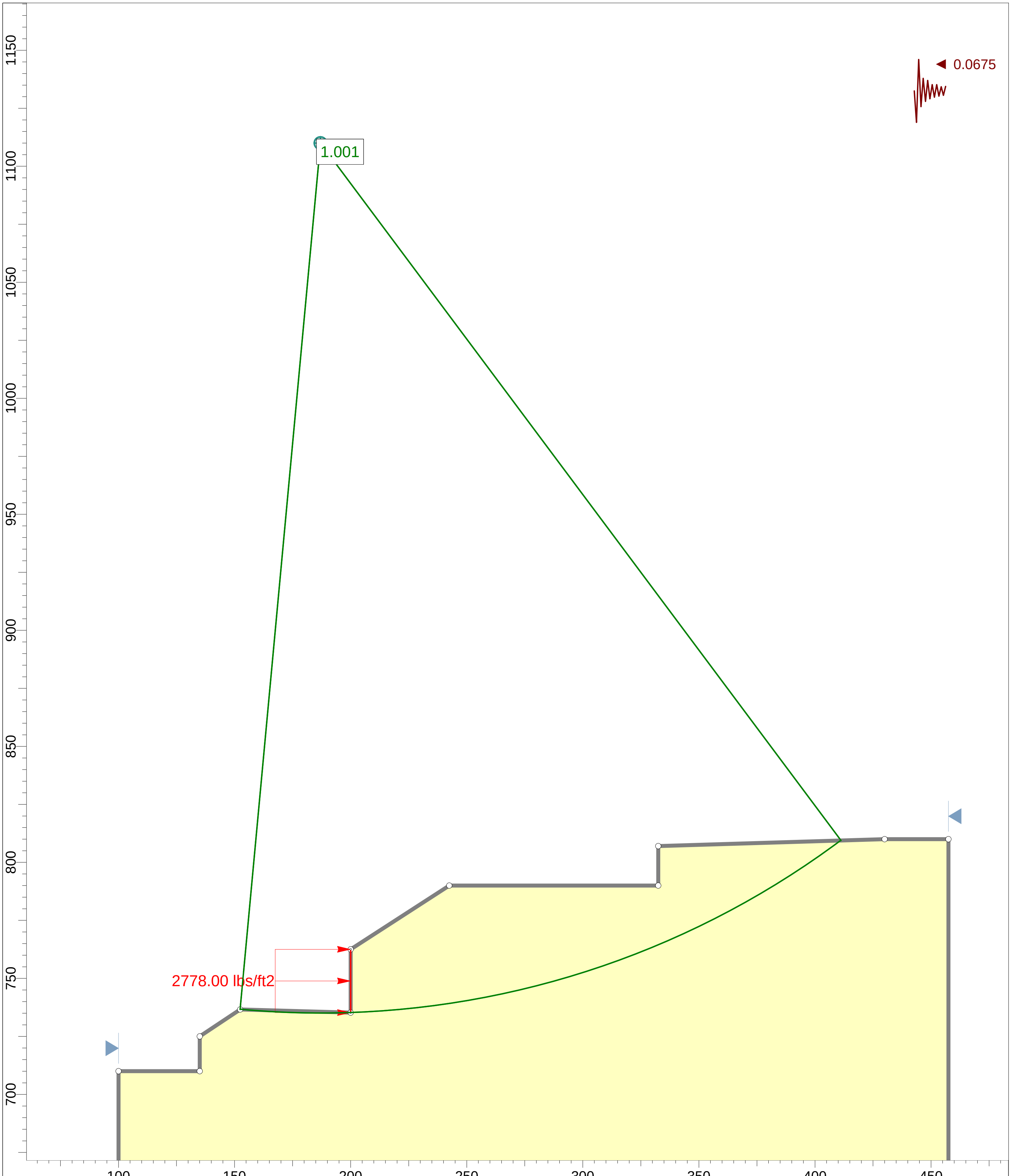


rocscience	*Project* SLIDE - An Interactive Slope Stability Program		
	Analysis Description		
	Drawn By	*Scale* 1:300	*Company*
SLIDEINTERPRET 6.029	*Date* 9/2/2014, 12:16:50 PM		*File Name* CASE B - Toe of Slope (No H20, No Loads, c=0, D=120).slim



SLIDEINTERPRET 6.029

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description			
Drawn By	*Scale* 1:650	*Company*	
Date 9/2/2014, 12:16:50 PM		*File Name* CASE C - (No H20, No Loads, c=0, D=120).slim	



Project
SLIDE - An Interactive Slope Stability Program

Analysis Description

Drawn By

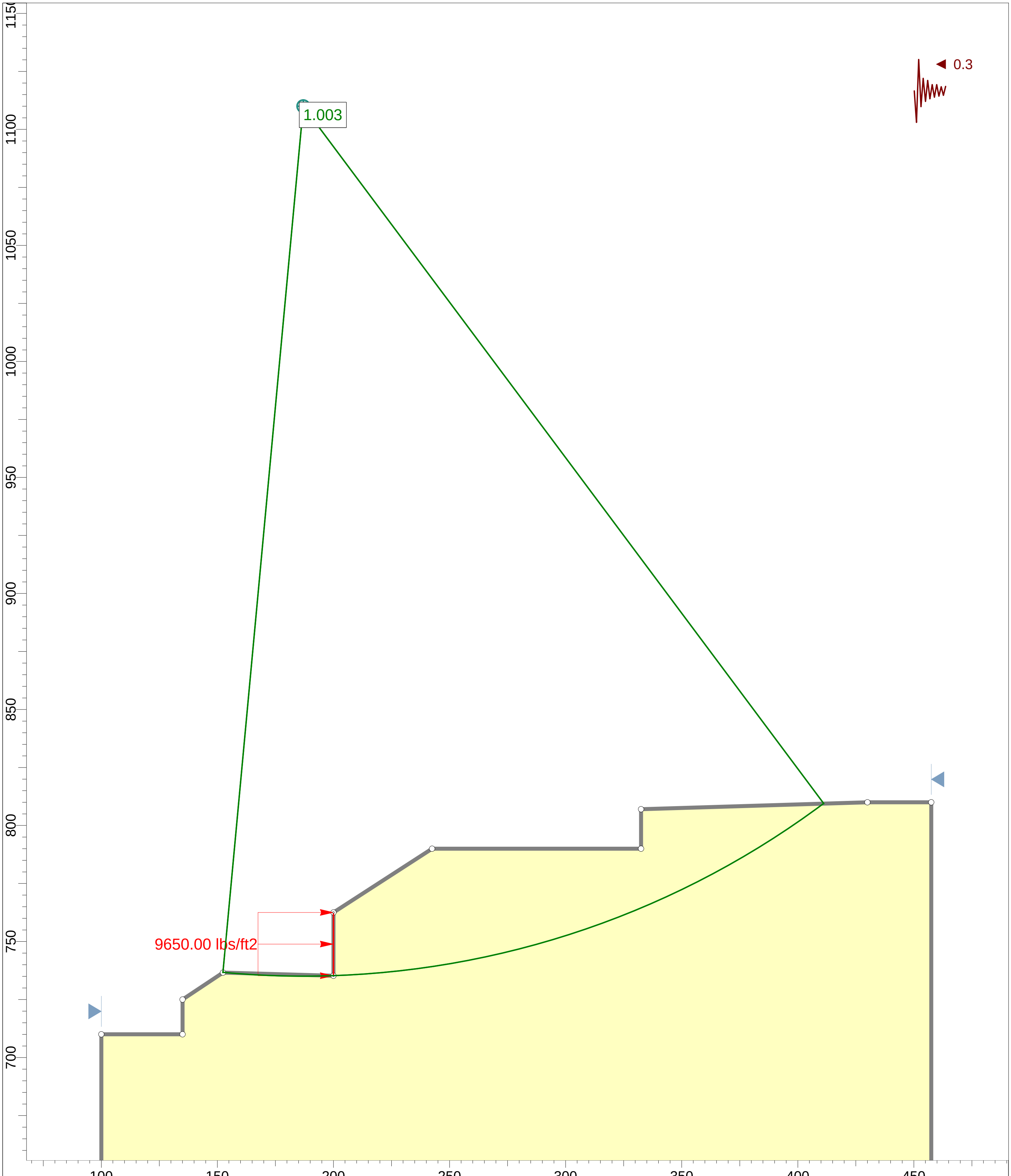
Scale 1:650

Company

Date 9/2/2014, 12:16:50 PM

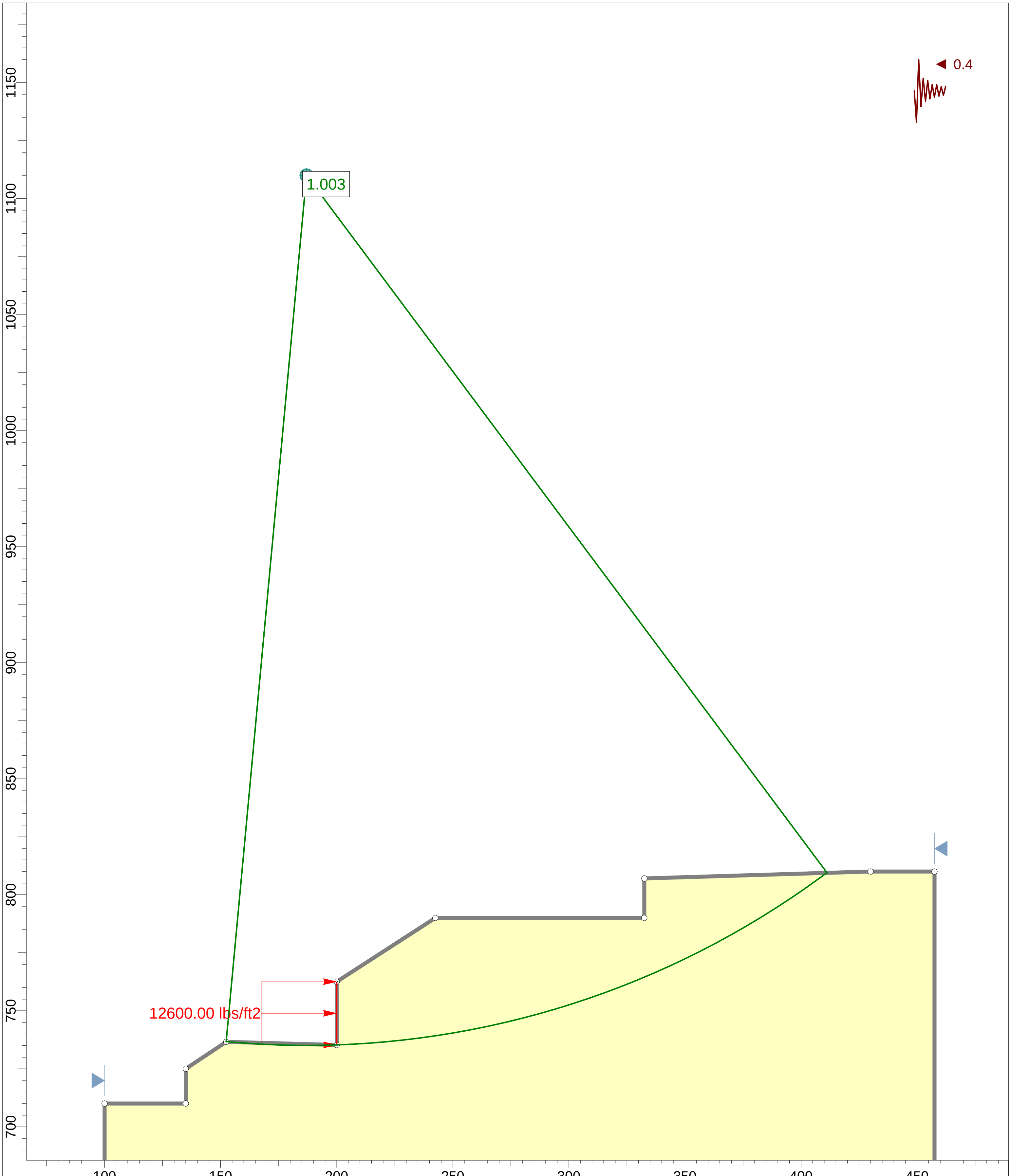
File Name CASE D - No Block 2 (No H20, Seismic=0.0675, c=0, phi=14.1, D=120).slim

SLIDEINTERPRET 6.029

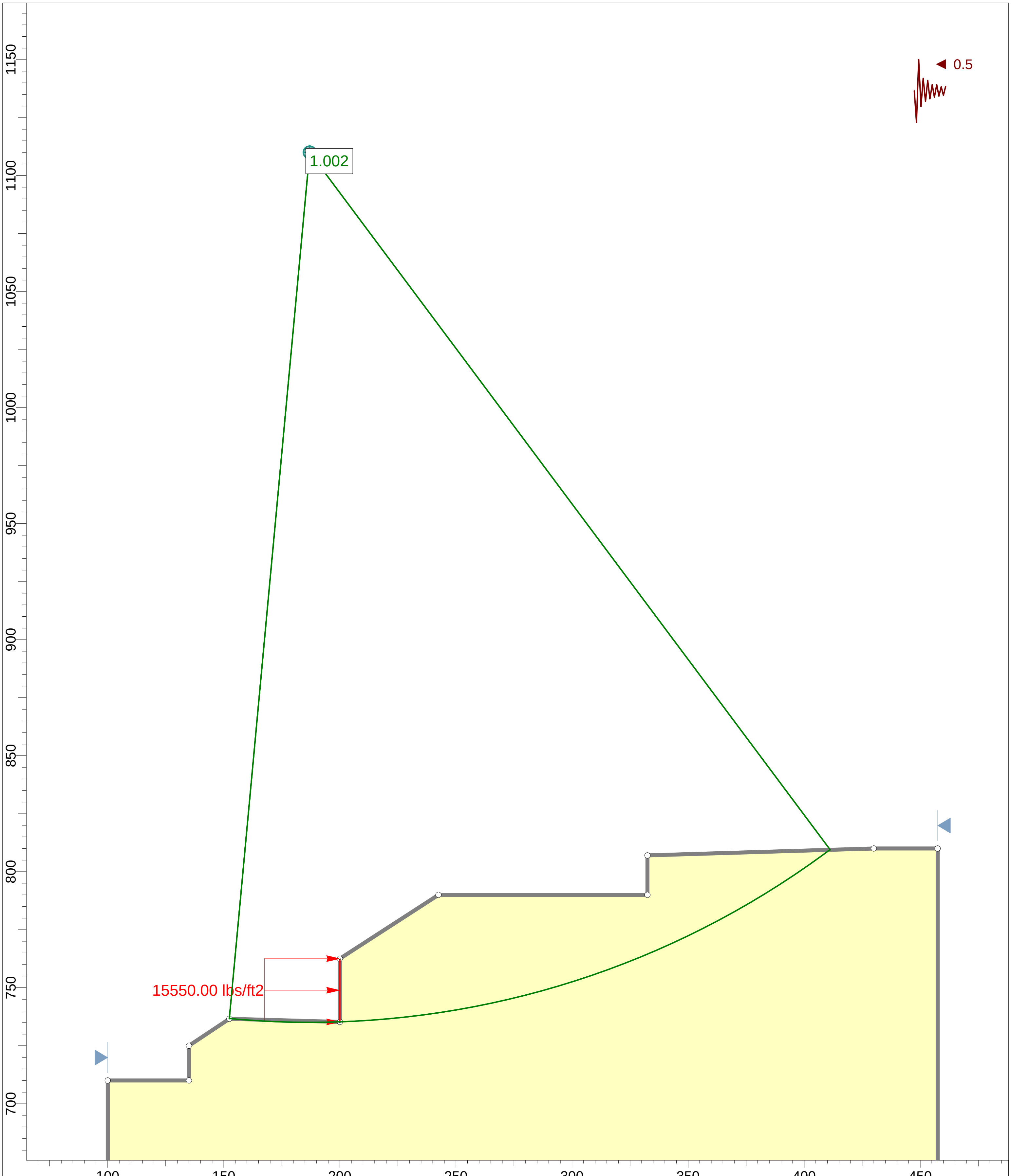


Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description			
Drawn By		Scale 1:650	Company
Date 9/2/2014, 12:16:50 PM			File Name CASE E1 - No Block 2 (No H20, Seismic=0.3 for FS=1, c=0, phi=14.1, D=120).slim

SLIDEINTERPRET 6.029

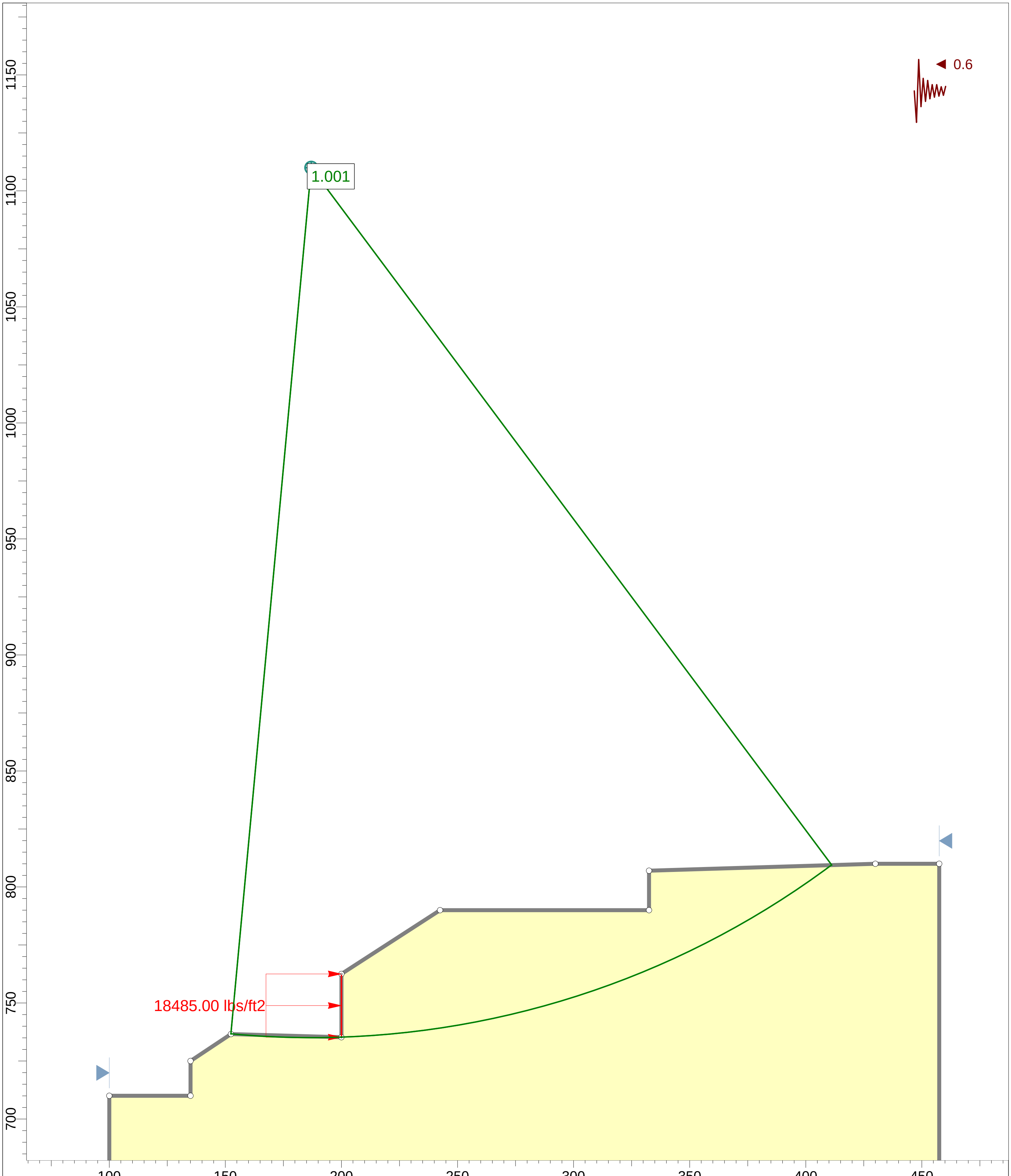


	Project SLIDE - An Interactive Slope Stability Program		
	Analysis Description		
	Drawn By	*Scale* 1:650	*Company*
SLIDEINTERPRET 6.029	*Date* 9/2/2014, 12:16:50 PM		*File Name* CASE E2 - No Block 2 (No H20, Seismic=0.4 for FS=1, c=0, phi=14.1, D=120).slim

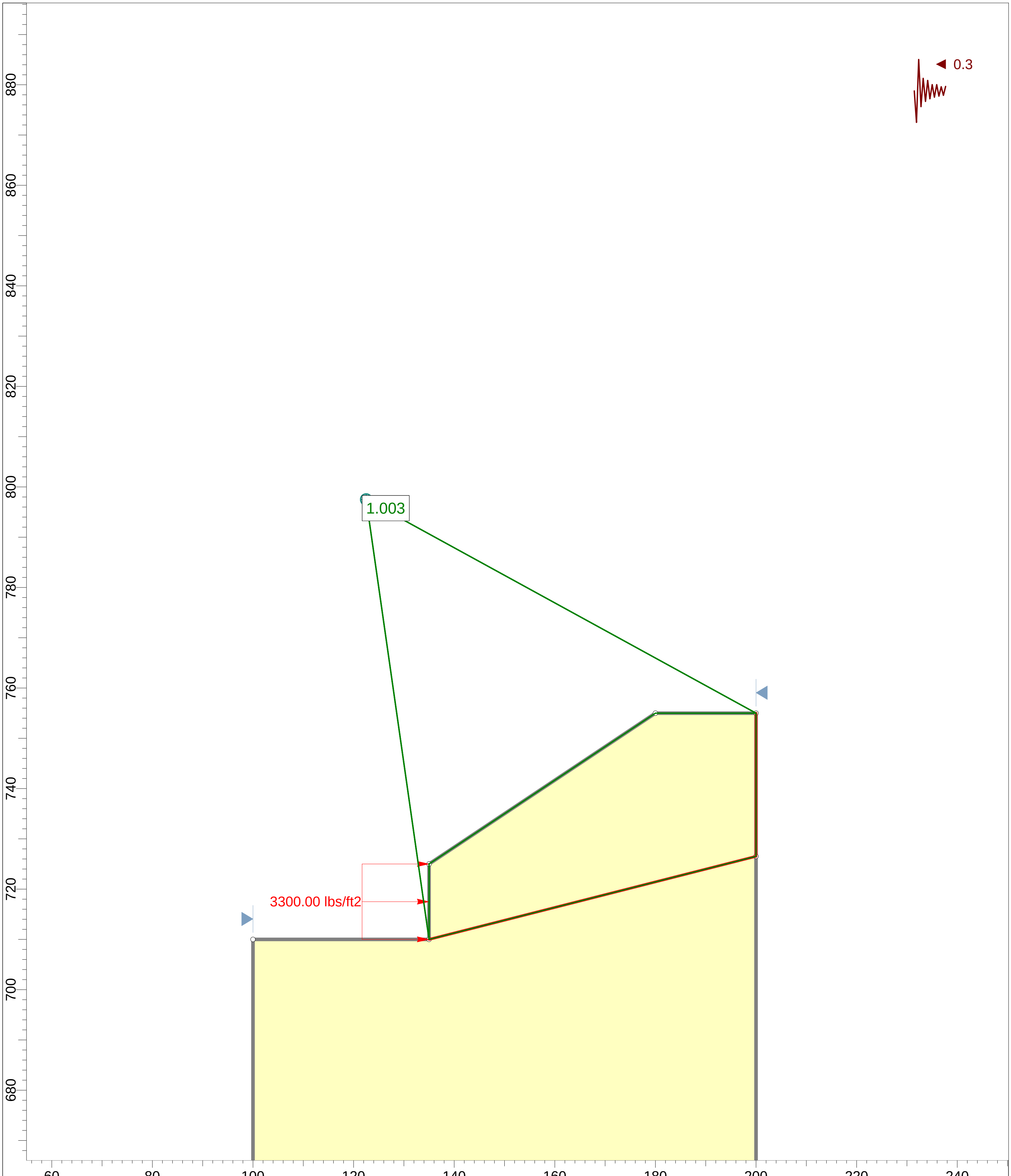


Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description			
Drawn By		*Scale* 1:650	*Company*
Date 9/2/2014, 12:16:50 PM		*File Name* CASE E3 - No Block 2 (No H20, Seismic=0.5 for FS=1, c=0, phi=14.1, D=120).slim	

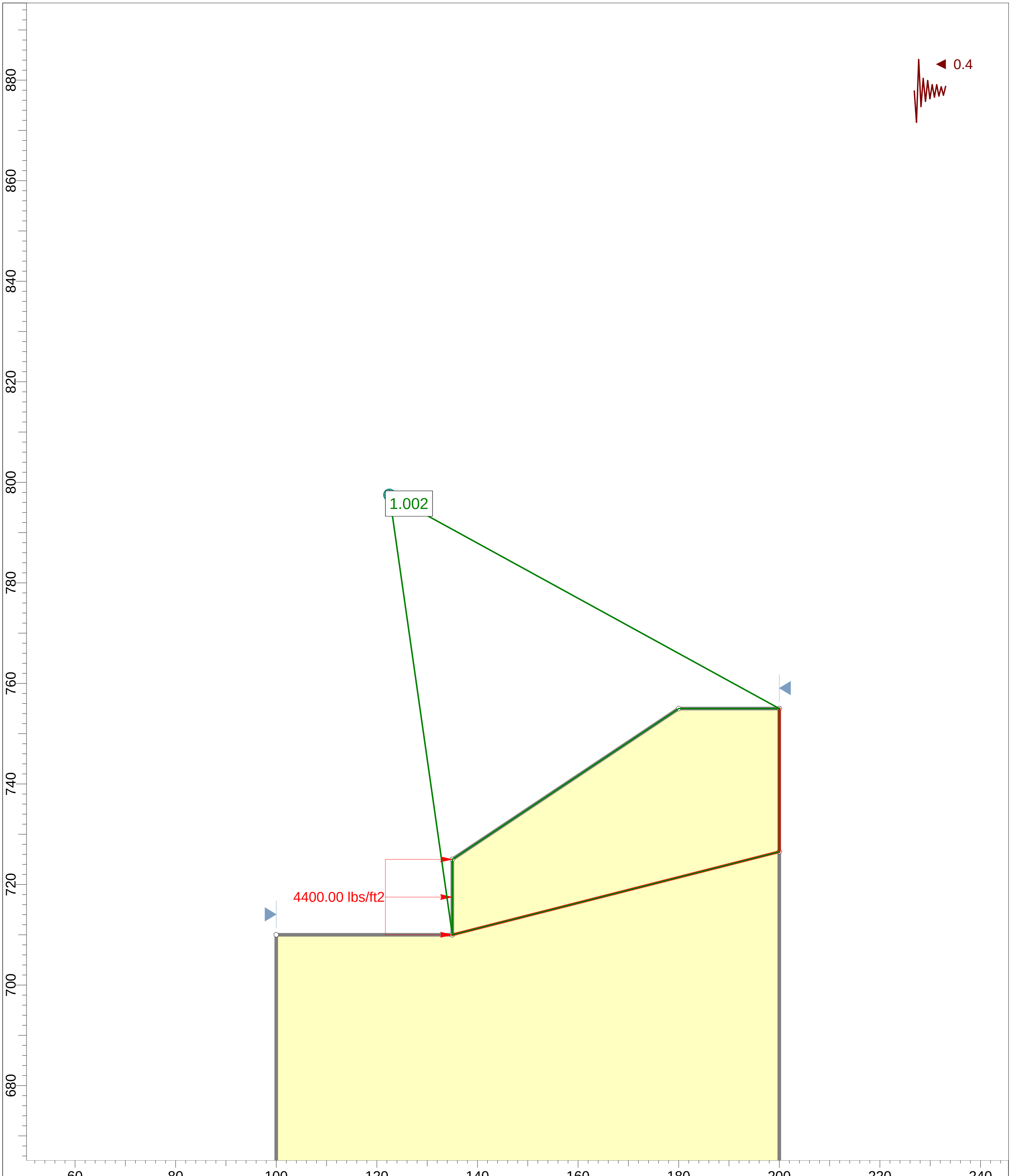
SLIDEINTERPRET 6.029



	Project SLIDE - An Interactive Slope Stability Program		
	Analysis Description		
	Drawn By	*Scale* 1:650	*Company*
SLIDEINTERPRET 6.029	*Date* 9/2/2014, 12:16:50 PM		*File Name* CASE E4 - No Block 2 (No H20, Seismic=0.6 for FS=1, c=0, phi=14.1, D=120).slim



rocscience	*Project* SLIDE - An Interactive Slope Stability Program		
	Analysis Description		
	Drawn By	*Scale* 1:300	*Company*
SLIDEINTERPRET 6.029	*Date* 9/2/2014, 12:16:50 PM		*File Name* CASE G1 - Toe of Slope (Seismic=0.3,c=0, D=120).slim



	Project SLIDE - An Interactive Slope Stability Program		
	Analysis Description		
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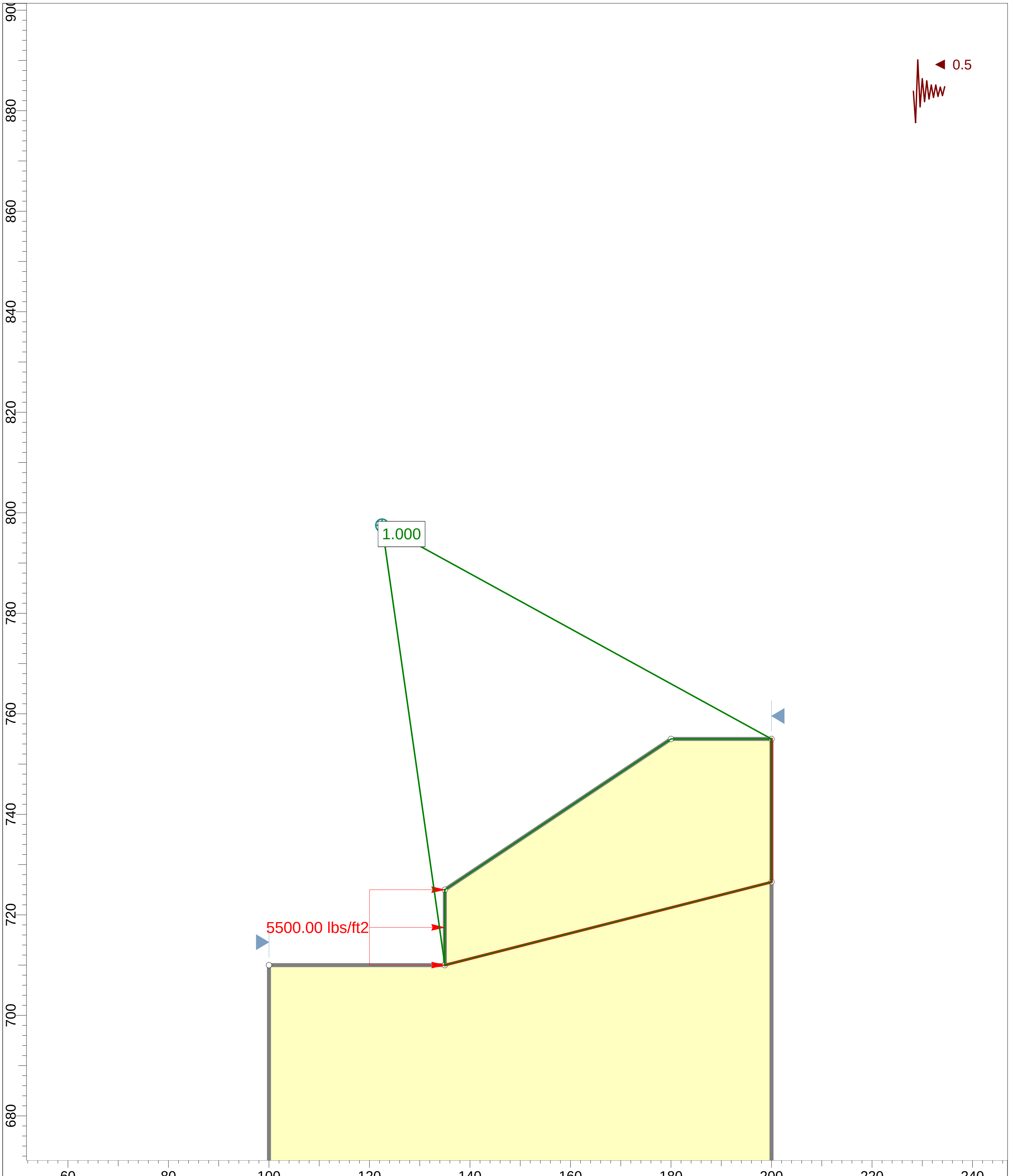
SLIDEINTERPRET 6.029	*Date* 9/2/2014, 12:16:50 PM		*File Name* CASE G2- Toe of Slope (Seismic=0.4,c=0, D=120).slim



Project
SLIDE - An Interactive Slope Stability Program

Analysis Description

Drawn By

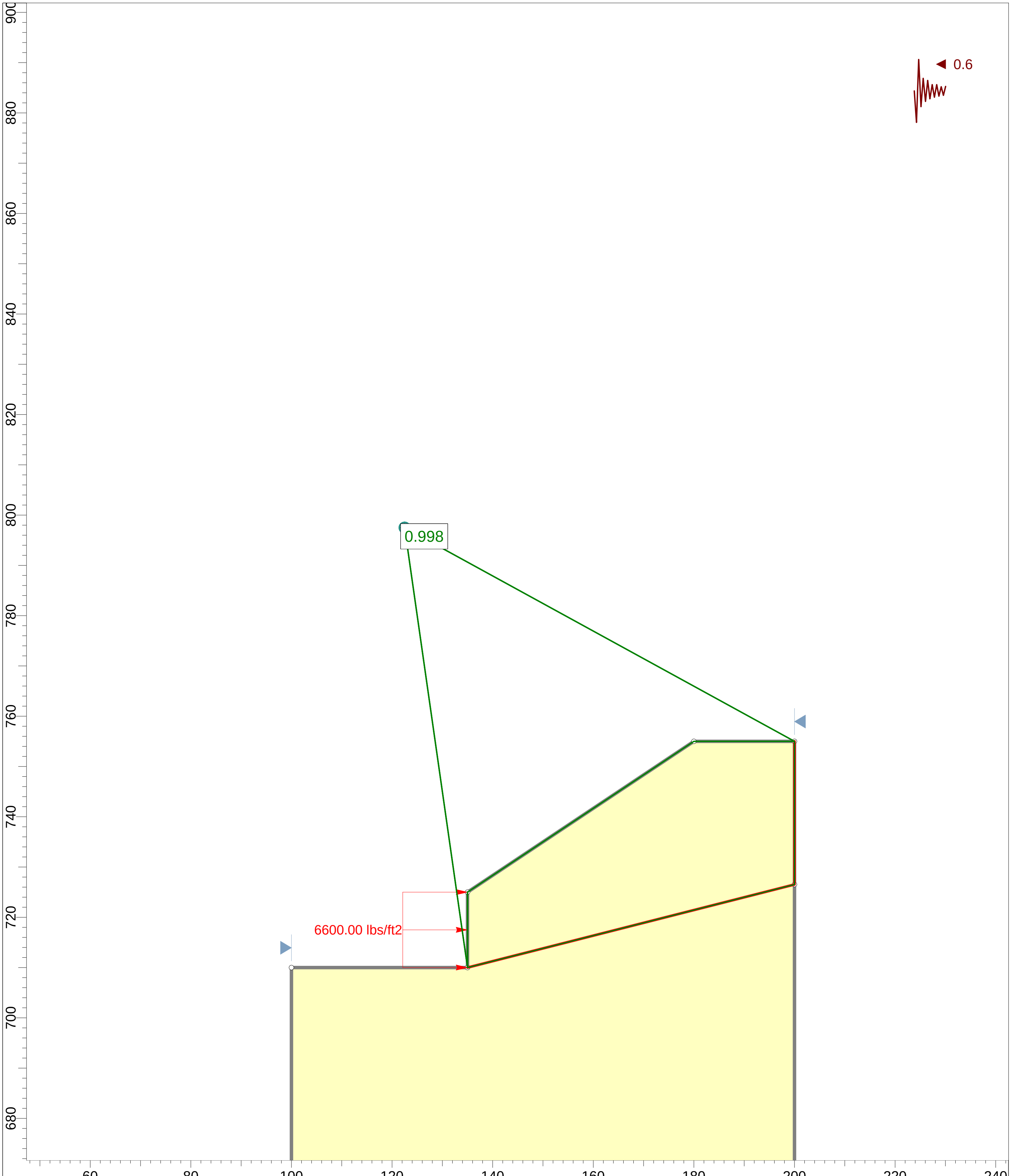
Scale 1:300

Company

Date 9/2/2014, 12:16:50 PM

File Name CASE G3 - Toe of Slope (Seismic=0.5,c=0, D=120).slim

SLIDEINTERPRET 6.029



Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description			
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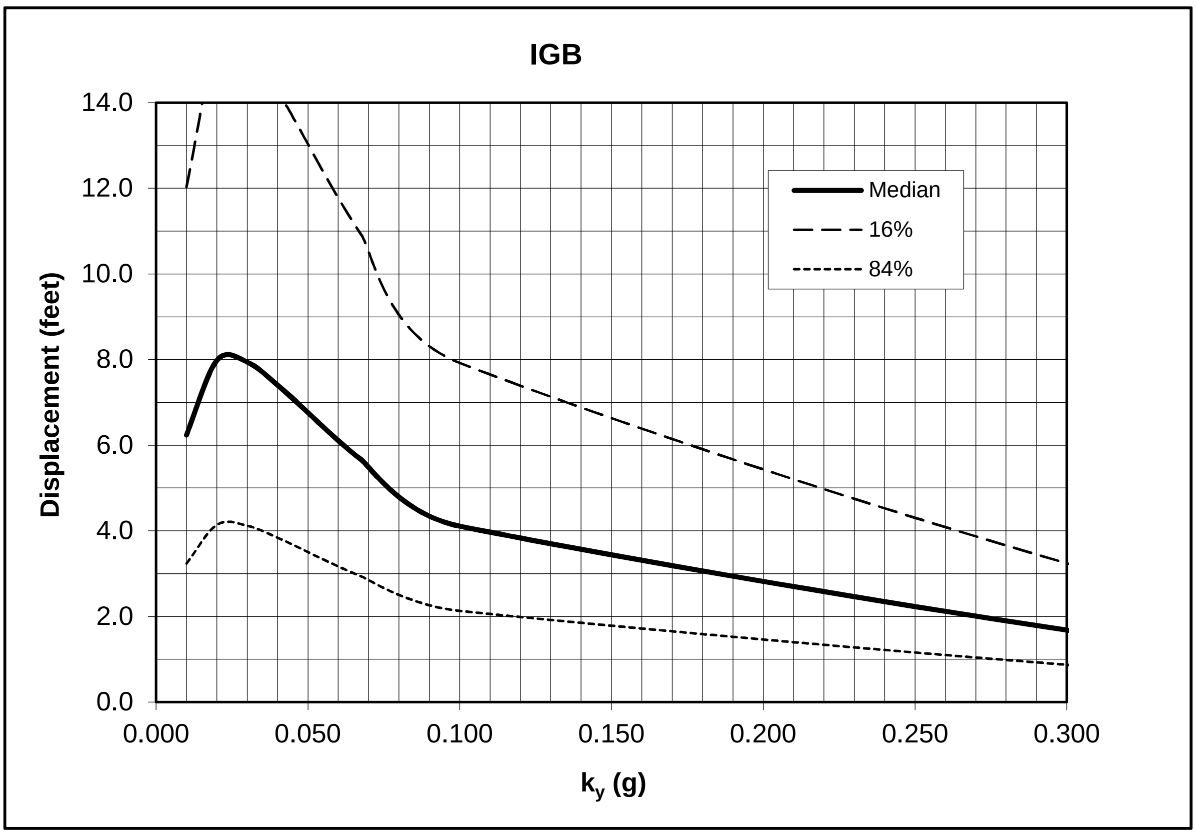
Date 9/2/2014, 12:16:50 PM		*File Name* CASE G4 - Toe of Slope (Seismic=0.6,c=0, D=120).slim	

SLIDEINTERPRET 6.029

IGB
A3GEO Project #1100-17B
Ts = 0.08s

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

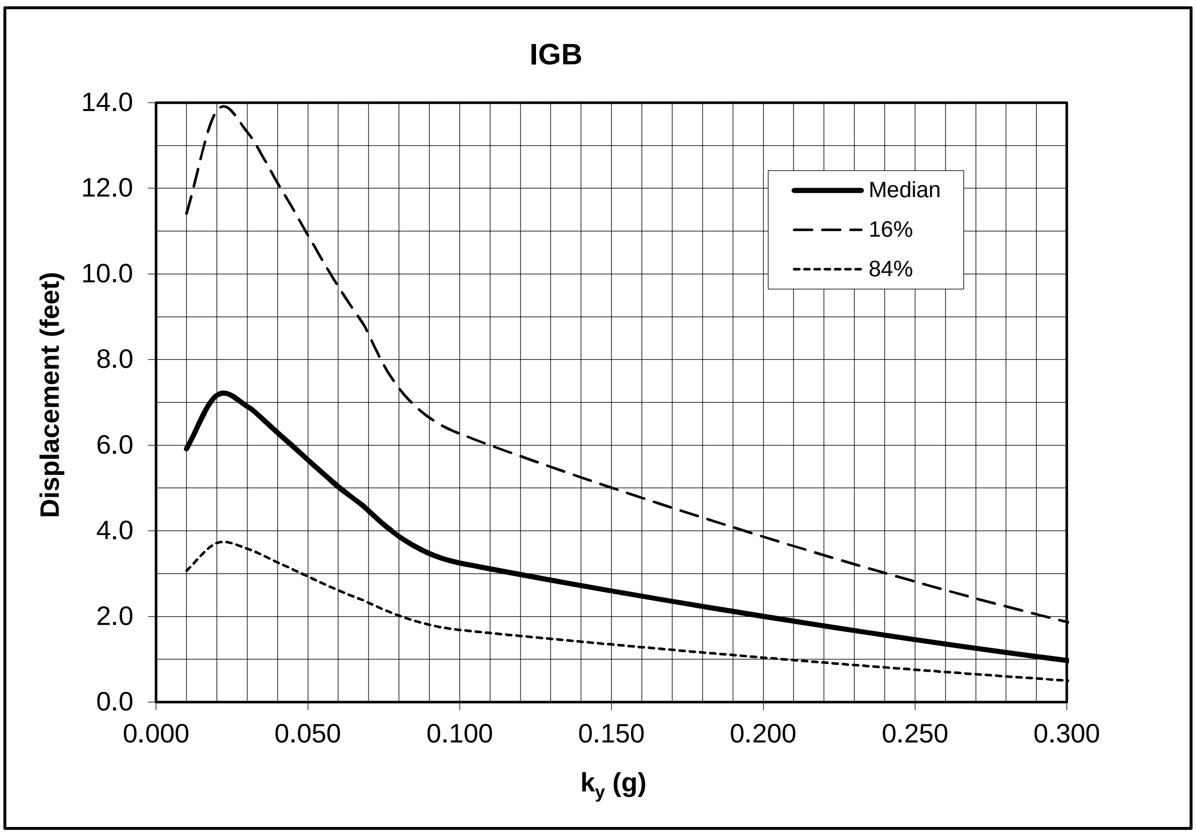
ky	Dmedian (cm)	D1 (cm)	D3 (cm)	Dmedian (ft)	D1 (ft)	D3 (f)
0.010	190.2	366.7	98.7	6.24	12.03	3.24
0.020	243.2	468.8	126.2	7.98	15.38	4.14
0.030	242.1	466.7	125.6	7.94	15.31	4.12
0.042	221.9	427.8	115.1	7.28	14.04	3.78
0.068	172.7	332.9	89.6	5.67	10.92	2.94
0.100	125.2	241.4	65.0	4.11	7.92	2.13
0.430	15.5	29.8	8.0	0.51	0.98	0.26
0.700	5.3	10.5	2.5	0.17	0.35	0.08



IGB
A3GEO Project #1100-17B
Ts = 0.06s

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

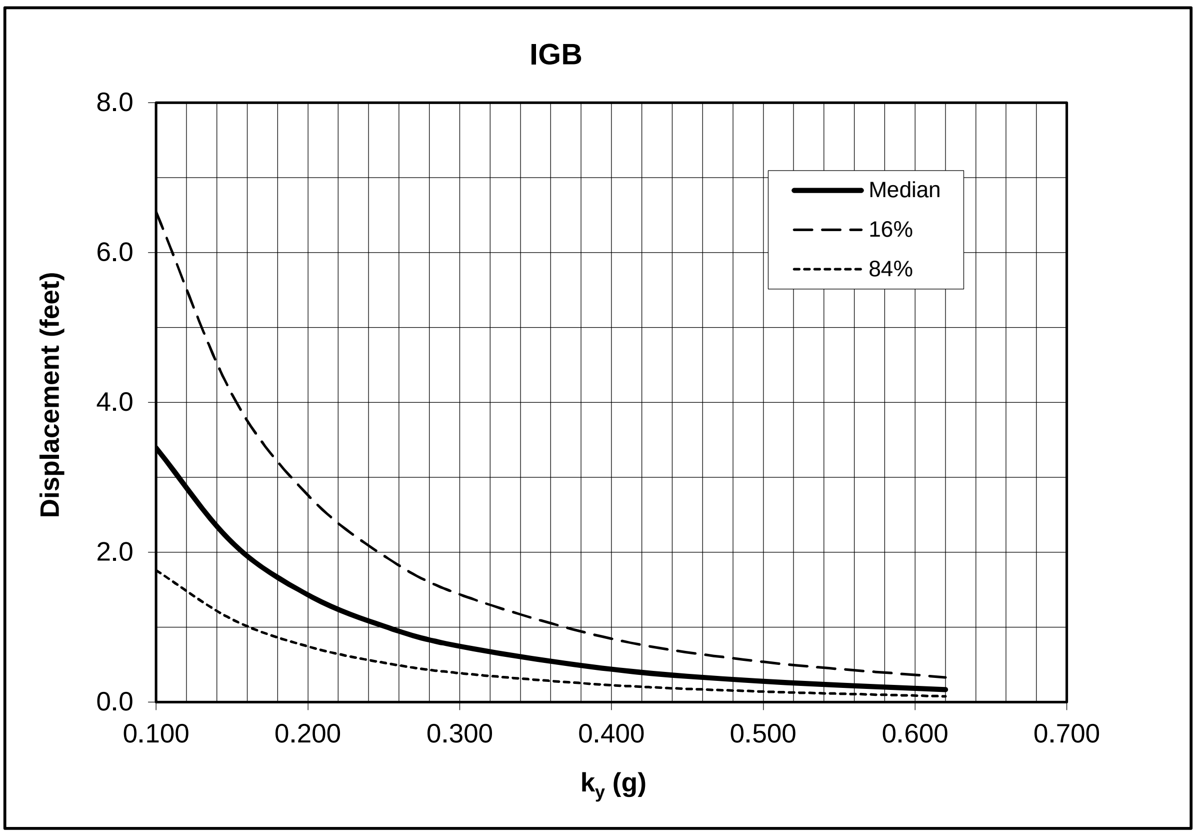
ky	Dmedian (cm)	D1 (cm)	D3 (cm)	Dmedian (ft)	D1 (ft)	D3 (f)
0.010	180.4	347.7	93.6	5.92	11.41	3.07
0.020	218.4	420.9	113.3	7.16	13.81	3.72
0.030	210.5	405.8	109.2	6.91	13.32	3.58
0.042	188.0	362.3	97.5	6.17	11.89	3.20
0.068	140.9	271.6	73.1	4.62	8.91	2.40
0.100	99.1	191.0	51.4	3.25	6.27	1.69
0.360	15.4	29.7	8.0	0.50	0.97	0.26
0.600	5.1	10.1	2.4	0.17	0.33	0.08

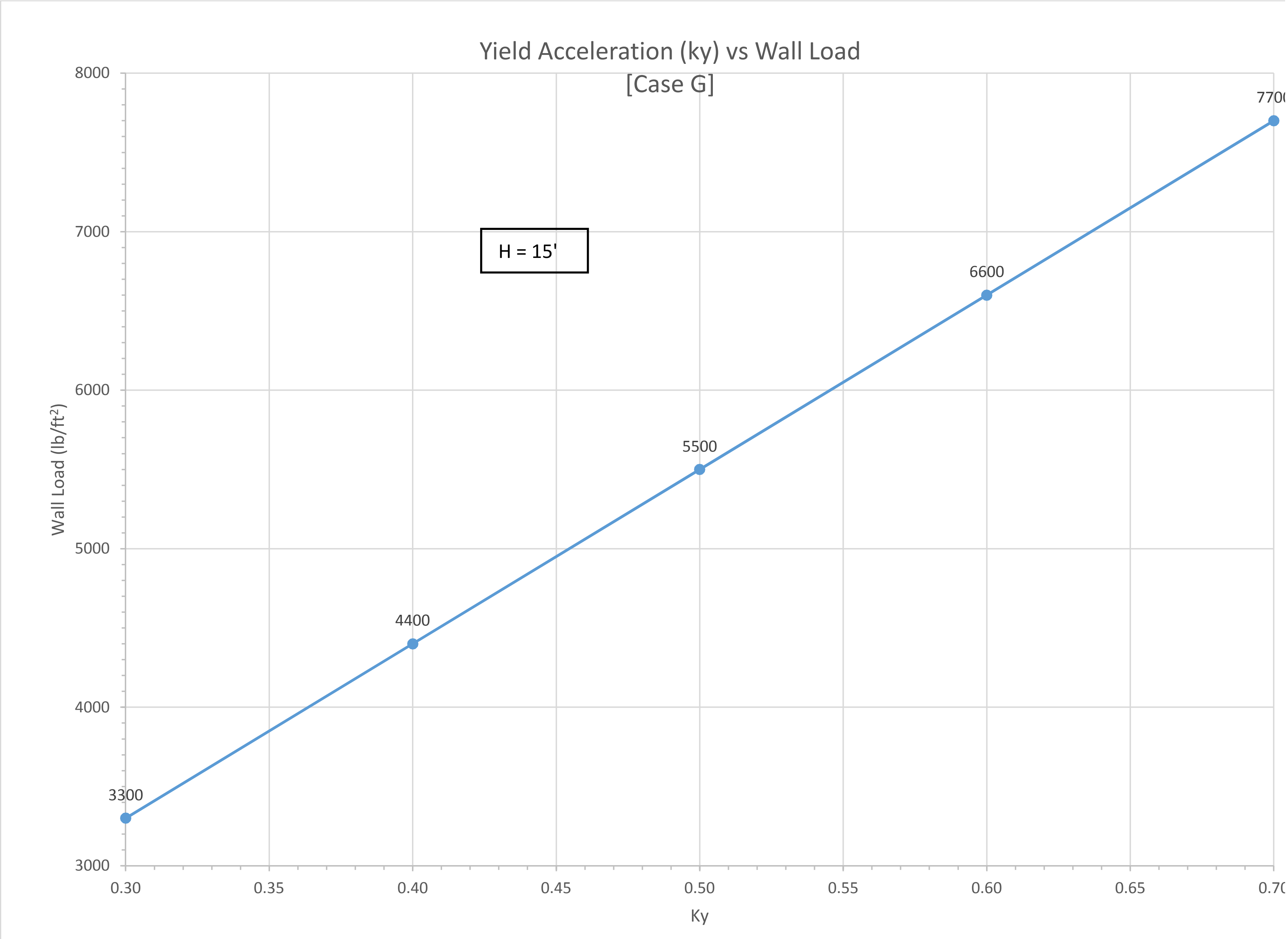


IGB
A3GEO Project #1100-17B
CASE G, Ts = 0.064s

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

ky	Dmedian (cm)	D1 (cm)	D3 (cm)	Dmedian (ft)	D1 (ft)	D3 (f)
0.100	103.4	199.3	53.6	3.39	6.54	1.76
0.150	64.9	125.2	33.7	2.13	4.11	1.11
0.200	43.7	84.2	22.7	1.43	2.76	0.74
0.250	30.9	59.6	16.0	1.01	1.95	0.53
0.300	22.7	43.8	11.8	0.75	1.44	0.39
0.400	13.4	25.8	6.9	0.44	0.85	0.23
0.500	8.5	16.4	4.3	0.28	0.54	0.14
0.620	5.1	10.1	2.4	0.17	0.33	0.08





Yield Acceleration (ky) vs Wall Load
[Case G]
H = 15'
Wall Load (lb/ft²)
Ky
8000
7000
6000
5000
4000
3000
0.30
0.35
0.40
0.45
0.50
0.55
0.60
0.65
0.70
3300
4400
5500
6600
770

A3GEO

Appendix E

USGS Design Maps Detailed Report

LBNL INTEGRATIVE GENOMICS BUILDING
BERKELEY, CALIFORNIA

USGS Design Maps Detailed Report

ASCE 7-10 Standard (37.8773°N, 122.25079°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 2.474 g
From Figure 22-2 [2]	S_1 = 1.029 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	$\boxed{1.0}$
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = C and S_S = 2.474 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	$\boxed{1.3}$
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = C and S_1 = 1.029 g, F_v = 1.300

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 2.474 = 2.474$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.300 \times 1.029 = 1.337$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

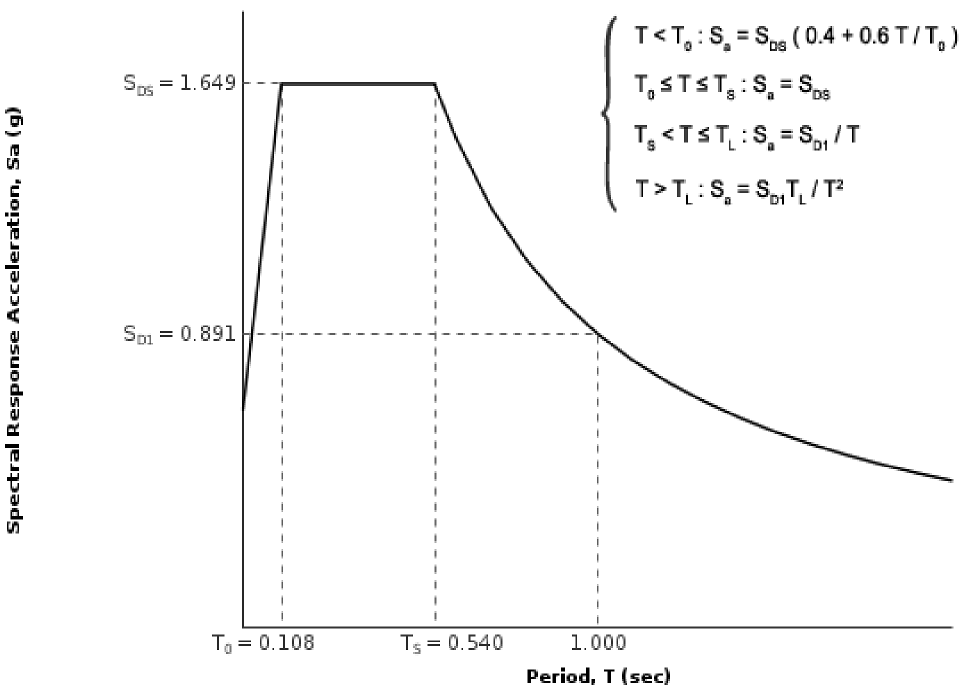
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 2.474 = 1.649$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.337 = 0.891$ g

Section 11.4.5 — Design Response Spectrum

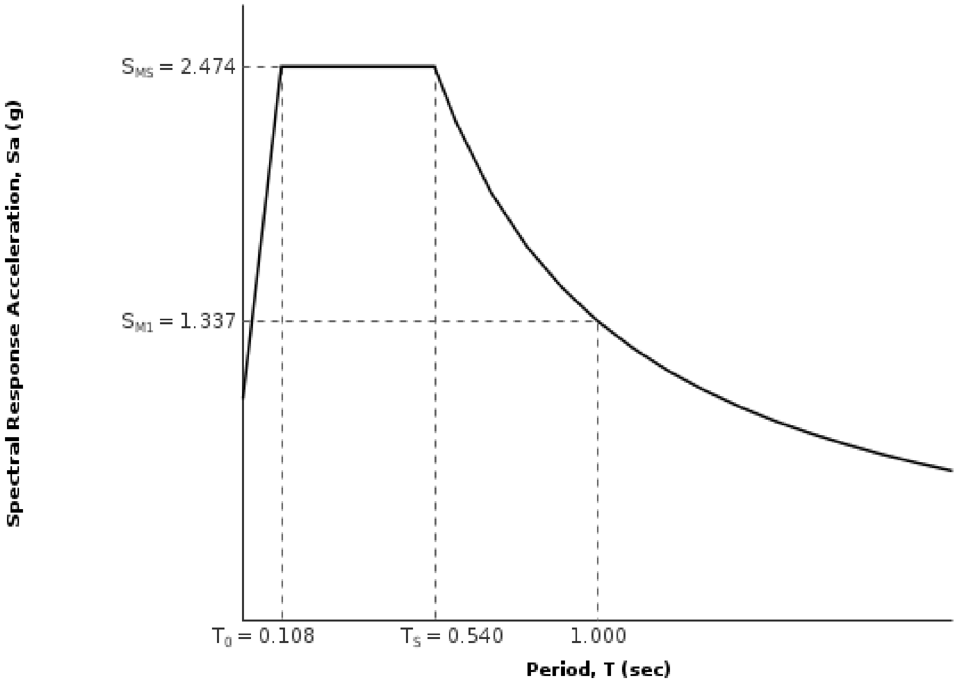
From Figure 22-12 [3] T_L = 8 seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.958

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.958 = 0.958$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = C and PGA = 0.958 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.000

From Figure 22-18 [6] C_{R1} = 0.976

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.649 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.891 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ “the more severe design category in accordance with Table 11.6-1 or 11.6-2” = E

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf