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GEOLOGIC HAZARDS STUDY
PROPOSED APARTMENT DEVELOPMENT
AKERS DRIVE AND CONSTITUTION AVENUE
COLORADO SPRINGS, COLORADO

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Project No. 21-2-147

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PURPOSE AND SCOPE OF STUDY

This report presents the results of a Geologic Hazards Study for the proposed apartment development to be located at Akers Drive and Constitution Avenue in El Paso County, Colorado. The project site is shown on Figures 1A & 1B. The purpose of the study was to evaluate the geologic conditions and assess their potential impact on the project and surrounding properties. The study was conducted in accordance with our proposal for engineering geology services to Watermark Residential, dated March 26, 2021, Proposal No. C21-197.

A reconnaissance of the project site was conducted on April 6, 2021 to obtain information on the geologic conditions of the site. Aerial photographs and published regional geologic, engineering geology, and mineral extraction maps were also reviewed. This report summarizes the data obtained during this study and the previous Kumar & Associates, Inc. geotechnical engineering report, project no. 20-2-194, dated September 10, 2020, and presents our conclusions, recommendations, and other geologic considerations based on the proposed construction and geologic conditions observed.

PROPOSED DEVELOPMENT

We understand the proposed construction will include nine separate three-story apartment buildings, a clubhouse with pool, a rental office, and 11 garage pods and parking areas. Paved access roadways and parking stalls will also be constructed throughout the site. The planned site grading would be relatively minor, with cuts and fills on the order of approximately 5 feet or less.

If conditions are significantly different from those described above or depicted in this report, we should be notified to reevaluate the recommendations contained herein.

SITE CONDITIONS

The proposed development is located within an unincorporated area of El Paso County that is surrounded by neighborhoods of eastern Colorado Springs. The property is situated in the south half of the southeast quarter of the southeast quarter of Section 32 in Township 13 South, Range 65 West, Sixth Meridian of the Public Land Survey System. The subject site consists of vacant, undeveloped land, bound by Akers Drive to the west, Constitution Avenue to the south, and Marksheffel Road to the east. Additional vacant land and commercial development is located to the north. The site slopes gently down to the southeast and there was roughly 25 feet of elevation difference across the property. The site appeared relatively undisturbed; however, review of historic aerial photographs indicates some potential site grading and unknown land use occurred in the 1970's and 80's. The site was vegetated with natural grasses, weeds, yucca and cacti.

There were some deciduous trees along the south property line. The site itself consists of 15.39 acres.

GEOLOGIC SETTING

The main geologic features in the vicinity of the project area are shown on Figure 2. This map is based on the published regional map by Madole & Thorson (2002) and our field reconnaissance on April 6, 2021.

The project site is located within the Colorado Piedmont of the Great Plains physiographic province. Structurally, this area is east of the Ute Pass and Rampart Range Faults, which bounds this portion of the Front Range. According to Robinson (1977), Trimble and Machette (1979), and Madole (2003), regional uplift east of the Front Range has exposed Upper Cretaceous-age gently northeast dipping claystone, siltstone, sandstone and thin coal beds representing a regressional sea sequence. The Upper Cretaceous and Paleocene Age Dawson Formation is stratigraphically the youngest bedrock that occurs in the area of the site and occurs at relatively shallow depths below the ground surface at this location.

Madole & Thorson (2001) indicate the entirety of the site as covered by a surficial deposit classified as older eolian sand. This late Pleistocene age unit is similar to younger eolian sand in the region, other than having a thicker, more complex soil profile than the younger eolian sand and containing more fine sediment, mainly silt. These units are described as "very pale-brown, pale-brown, and light-yellowish-brown sand. Unit is chiefly fine to very coarse sand that appears to have been deposited in sheets." The thickness of these units is estimated to range from 3 to 15 to 20 feet.

The Dawson Formation consists of sandstone arkose interbedded with claystone. "The arkose of the Dawson Formation is interpreted to have eroded from an uplifting mass of granitic rock in the Front Range during the Laramide Orogeny (Tweto, 1975, Epis et al., 1980) and deposited in a braided stream floodplain in the adjacent subsiding Denver Basin (Raynolds, 1997)" (Carroll & Crawford, 2000). The Dawson Formation overlays the Laramie Formation unconformably. The Dawson Formation is broken out into an upper and lower part. The upper part of the Dawson Formation is Upper Cretaceous to Paleocene in age and is divided into five informal members by Madole & Thorson (2002), with only facies units one and two found in the Elsmere quadrangle, while the lower part of the Dawson Formation is Upper Cretaceous in age.

Bedrock at the subject site can be described as the upper part of the Dawson Formation, facies unit two. Facies unit two is described by Madole & Thorson (2002) as a “brownish gray, yellowish gray and light yellowish brown, pebbly sandstone interbedded with yellowish gray to grayish green, fine to coarse-grained micaceous sandstone and sandy claystone, and dark gray, greenish gray, and dark brown sandy claystones that contain variable amounts of organic material. About 1,000 ft of strata… exposed in the Elsmere quadrangle.”

The upper part of the Dawson Formation lays over the lower part of the Dawson Formation, which is described by Thorson, Carroll, and Morgan (2001) as greenish-gray to olive-brown, cross-bedded or massive, very thick beds of sandstone containing andesite pebbles up to 3 inches in diameter; interbedded with grayish-green to dark-green and brown to brownish-gray siltstone and sandy claystone. Thickness may be up to 240 feet.

Underlying the Dawson formation unconformably is the Laramie Formation. Carroll & Crawford (2000) describe the Laramie Formation as a sandstone interbedded with siltstone, carbonaceous shale, and black sub-bituminous coal which was deposited in a near-shore coastal-plain environment. Coal beds are found in the lower part of the formation, developing in poorly drained swamps in overbank areas adjacent to the channel interface. Older formations underlay the above-mentioned formations, but are not relevant to this study.

FIELD EXPLORATION

The subsurface conditions at the site were explored during the preparation of the geotechnical engineering report, Kumar & Associates, Inc. Project No. 20-2-194, dated September 10, 2020 by drilling a total of 18 exploratory borings. The borings were drilled August 24 and 25, 2020, and the locations were approximated using a handheld GPS unit, and the elevations were measured using a hand level. Graphic logs of the borings are presented on Figs. 4 through 6, and the corresponding legend and notes are presented on Fig. 7.

The borings were drilled with 4-inch diameter continuous flight augers and were logged by a representative of Kumar & Associates, Inc. Samples of the soils were taken with either a 2-inch I.D. California Sampler. The samplers were driven into the various strata with blows from a 140-pound hammer falling 30 inches. Penetration resistance values, when properly evaluated, provide an indication of the relative density or consistency of the soils. Depths at which the samples were taken and the penetration resistance values are shown on the boring logs.

LABORATORY TESTING

Samples obtained from the exploratory borings were visually classified in the laboratory by the project engineer and samples were selected for laboratory testing. Laboratory testing included index property tests such as in-situ moisture content and dry unit weight, grain size analysis, and Atterberg limits. Additional testing included in-situ swell-consolidation and concentration of water-soluble sulfates. The testing was conducted in general accordance with recognized test procedures, primarily those of the American Society for Testing of Materials (ASTM). Results of the laboratory testing program are shown on Figs. 4 through 6 and are summarized on Table I.

SUBSURFACE CONDITIONS

Beneath a layer of topsoil (root zone), the generalized subsurface profile encountered in the borings consisted of a combination of granular and cohesive overburden soils, underlain by claystone and sandstone bedrock. Man-placed fill was encountered in one of the borings. Given the wide spacings of the borings drilled for this study, it is possible for existing fill to be present elsewhere on site. The following subsurface descriptions are of a generalized nature to highlight the soil and bedrock types encountered in the borings drilled for this study. The boring logs should be reviewed for more detailed information.

Existing Fill: In Boring 11, man-placed fill was encountered to an approximate depth of 7 feet. The fill consisted of a mixture of clayey sand (SC) and sandy silty clay (CL-ML), and appeared to consist of reworked on-site soils. Due to the similarity of the natural soil and fill materials, it was not possible to clearly differentiate between fill and native soils. The fill was slightly moist to moist, and light brown to brown in color. Our study did not determine the exact lateral or vertical extent of the fill. Swell-consolidation test results presented on Fig. 9 indicate the tested sample of sandy silty clay fill had a low swell potential when wetted under a 1,000 psf surcharge.

Native Granular Soils: The native granular soils encountered were grouped as follows: clayey sand (SC) with silty-clayey sand (SM-SC), and poorly to well-graded sand with silt (SP-SM, SW-SM) with silty sand (SM) and occasional gravel. These soils were encountered in 17 of the 18 borings, beginning at depths ranging from near surface (below topsoil layer) to 10 feet, and extending to depths ranging from 4 to 22 feet in 10 of the borings, and to the maximum 15 to 30-foot depths explored in seven of the borings. The native granular soils were slightly moist to very moist, and tan to brown in color. Sampler penetration blow counts indicate the granular soils are generally medium dense to very dense. The exception was Boring 9 at a depth of 9 feet, where the granular soils were very loose (blow count of 3).

Native Clay Soils: Native lean clay (CL) soil with varied amounts of sand were encountered in 15 of the 18 borings. These soils were encountered beginning at depths ranging from near surface (below topsoil layer) to 13 feet, and extending to depths ranging from 4.5 feet to 26 feet in 13 of the borings, and to the maximum 20-foot depth explored in two of the borings. The native clay soils were slightly moist to moist, and brown, dark brown, and gray in color. Sampler penetration blow counts indicate the clay soils are medium stiff to hard in consistency. Swell-consolidation test results presented on Figs. 6 thru 10 indicate the tested samples of clay varied from having a nil to high swell potential to a low potential for compression, when wetted under a 1,000 psf surcharge.

Bedrock: Sandstone and/or claystone bedrock was encountered in 9 of the borings, beginning at depths of 9 to 26 feet, and extending to the maximum 15 to 30-foot depths explored. In two of these borings, the upper few feet of claystone was weathered. The sandstone was poorly cemented, moist and brown in color. The claystone was slightly moist to moist, and brown to gray in color. Sampler penetration blow counts indicate the non-weathered bedrock is hard to very hard, and the weathered claystone is very stiff to hard. Swell-consolidation testing was not performed on the claystone due to the depth encountered, however, based on our experience in the area, we recognize that it typically has a similar potential for swell as the tested overburden clay soils.

Groundwater: Groundwater was not encountered at the time of drilling. When the borings were checked 8 to 9 days later, groundwater was encountered in Boring 8 at an approximate depth of 25.1 feet. Fluctuations in the water level may occur with time, particularly during wetter seasons and after precipitation events. The borings were backfilled with auger cuttings upon completion of water level measurements.

The subsurface conditions observed within the exploratory borings generally correlate to the regional and site geology conditions described in the "Geologic Setting" section above.

POTENTIAL MINERAL RESOURCES

According to the "El Paso County – Aggregate Resource Evaluation Maps, El Paso County – Master Plan for Mineral Extraction" (1996), the site is designated as 'Municipalities' and no aggregate resources are indicated. The United States Department of Agriculture Natural Resource Conservation Service (NRCS) Web Soil Survey indicates the site as overlain with Blakeland loamy sand on the majority of the site and the Blendon sandy loam on the east side of the site, as shown on Figure 3. The NRCS classifies both the Blakeland loamy sand and the

Blendon sandy loam as good suitability for roadfill, poor suitability as a gravel source and fair suitability as a sand source. The Blakeland loamy sand has a low runoff and is somewhat excessively drained, while the Blendon sandy loam has a low runoff and is well drained. The parent material of the Blakeland loamy sand is alluvium derived from sedimentary rock and/or eolian deposits derived from sedimentary rock. The parent material of the Blendon sandy loam is a sandy alluvium derived from arkose. The NRCS classifies the Blakeland loamy sand within Hydrologic Group A and the Blendon sandy loam within Hydrologic Group B. Evaluation of commercial feasibility of gravel, sand, or roadfill mining on the subject site is beyond the scope of this study.

GEOLOGIC SITE ASSESSMENT

The project site geology should not present major constraints or unusually high risks to the development or surrounding properties. There are, however, several conditions of a geologic nature that should be considered. These conditions, their potential risks, and suggestions to mitigate the potential risks are discussed below.

POTENTIAL FLOODING

According to the "Flood Insurance Rate Map" (FIRM), map number 08041C0756G produced by the Federal Emergency Management Agency (FEMA, 2018), the site is located in an unshaded region of Zone X (unshaded – areas of minimal flood hazard). The nearest flood hazard is located approximately 850 feet to the east of the site, and consists of a shaded region of Zone X. Shaded regions of Zone X represent areas that are subject to inundation by the 0.2% annual chance flood (500-year flood). The 500-year flood has a 0.2 percent probability of being equaled or exceeded in any given year, and during a 70-year period (the supposed useful life of many buildings) the probability of occurrence is 18 percent. This 500-year floodplain is associated with the Sand Creek East Fork Subtributary, the 100-year floodplain of which is located approximately 1,450 feet east of the project site. Based on the designation of minimal flood hazard by FEMA and the distance separating the site from the nearest flood hazard in addition to separation by infrastructure, including Marksheffel Road, the hazard of potential flooding on the subject site is considered minimal. No flood modeling was performed as a part of this study.

SEASONALLY SHALLOW GROUNDWATER

Groundwater was not encountered at the time of drilling, but was observed in a single boring 9 days following drilling. Fluctuations in the water level may occur with time, particularly after precipitation events and as a result of nearby irrigation practices after development. Boring 8 from the Kumar & Associates, Inc. geotechnical engineering report, project no. 20-2-194, was found to have groundwater at a depth of 25.1 feet, apparently perched on a layer of lean clay. Groundwater may become perched on less permeable subsurface layers when introduced through surface infiltration. Evidence of seepage was not encountered during the geotechnical exploration nor during the site visit for the preparation of this report. This area should be evaluated for seepage during a period of seasonally high flow. If seepage is encountered, it may need to be collected and diverted away from structures and pavements. The extent and amount of perched water beneath the building site as a result of irrigation and inadequate surface drainage is difficult, if not impossible, to foresee.

PRE-EXISTING MAN-PLACED FILL

Existing fill was encountered in one of the borings from the Kumar & Associates, Inc. geotechnical report, project no. 20-2-194. Sampler penetration blow counts suggest the fill is relatively compact. Kumar & Associates, Inc. is not aware of any documentation stating the manner of fill placement. Uncontrolled or inadequately compacted fill presents risks of excessive or differential settlement of foundations, floor slabs or pavements constructed on the fill. Additionally, expansive clays could present the risk of heave upon wetting. Engineering risk from uncontrolled fill is typically mitigated by removal and replacement of the material. It is our opinion that pre-existing man-placed fill should be considered unsuitable for support of the proposed development unless documentation is available stating the site fills were properly controlled to the compaction criteria presented in the geotechnical report. The suitability of the pre-existing man-placed fill was evaluated in the Kumar & Associates, Inc. geotechnical engineering report, Project No. 20-2-194. The geotechnical report states that existing on-site fill "…would be suitable for reuse as structural fill if it is processed and moisture conditioned" and is "…suitable for reuse, minus any deleterious materials." The geotechnical engineering report Existing fill should be removed and placed back, properly compacted, based on the specifications recommended in the geotechnical engineering report. Structures placed on uncontrolled fill may experience significant movement, resulting in structural distress. Floor slabs and pavements may also be distressed by movement of poorly compacted or expansive fill. Overexcavation and replacement of a portion of the fill is generally suitable if some movement can be tolerated. The vertical and lateral extent of man-placed fill on the subject site was not determined and is beyond the scope of our work for this report.

EXPANSIVE/COLLAPSIBLE SOILS & BEDROCK

Swelling soils have been found to occur on this site. Clay overburden soil found during the exploration associated with the Kumar & Associates, Inc. geotechnical engineering report varied from having a nil to high swell potential to a low potential for compression, and was encountered in several of the borings within the assumed elevation of construction. Such materials are stable at their natural moisture content but will undergo high volume changes with changes in moisture content. Expansive materials may cause distress to structures or pavement if changes in moisture content occur. Overexcavation and replacement or moisture conditioning of expansive materials are standard construction practices commonly used in this area for mitigation of moisture sensitive soils. The claystone will be expansive when placed in a compacted condition and is not suitable for use as nonexpansive fill.

SUBSURFACE MINING

The Colorado Geological Survey and the Colorado Springs Subsidence Investigation by Dames & Moore (1985) indicate several historic mines within three miles of the site. The historic Jimmy Camp Coal Mine, including Slopes No. 1 through 3, is located approximately 2.3 miles south-southeast of the subject site. The historic McFerran Mine is located approximately 2.6 miles southeast of the site. The historic Enterprise Mine and Hall Slope are located approximately 2.8 miles southwest of the site. These mines are part of the Colorado Springs Coal Field, which encompasses a southwest portion of the Denver Basin, and historically removed approximately 16 million tons of coal from the Laramie Formation (Roberts, 2007). In addition to the previously mentioned historic mines, the El Paso County – Aggregate Resource Evaluation Map (1996), indicates the presence of a coal pocket approximately 3,800 feet north of the subject site, although no historic mining is known to have occurred there. The Colorado Springs Coal Field trends northwest to southeast through Colorado Springs, from just south of the US Air Force Academy to just north of the Colorado Springs Airport, continuing southeast then east from there (El Paso County – Master Plan for Mineral Extraction, 1996). Subsidence has been an issue related to these historic mines as relatively shallow tunnels are located beneath densely populated neighborhoods through the Colorado Springs area. The subject site, however, displayed no evidence of mine subsidence at the surface, and the risk is considered minimal as the site is located outside of the limit of potential subsidence as determined by Dames & Moore (1985).

SEISMICITY

The Rampart Range Fault, a high-angle generally north-south trending reverse fault, and the Ute Pass Fault, generally characterized by several northwest-southeast trending reverse faults, are mapped approximately 10.4 miles west and 11.2 miles southwest, respectively, of the subject site. According to the “Preliminary Quaternary Fault and Fold Map and Database of Colorado” by Widmann, Kirkham and Rogers (1998), there is evidence that the Rampart Range Fault may have moved between 600,000 and 30,000 years ago, and the Ute Pass Fault may have ruptured during the last 750,000 years. The largest historic earthquake in the project region occurred in 1882. It was located in the northern Front Range and had an estimated magnitude of M6.4 ± 0.2

and a maximum intensity of VII. Historic ground shaking at the project site does not appear to have exceeded Modified Mercalli Intensity VI (Kirkham and Rogers, 2000). Modified Mercalli Intensity VI ground shaking should be expected during a reasonable exposure time for the development, but the probability of stronger ground shaking is low. Intensity VI ground shaking is felt by most people and causes general alarm, but results in negligible damage to structures of good design and construction. According to the Colorado Geological Survey (Kirkham and Rogers, 1981), Colorado Springs should be considered as Zone 2 in the Uniform Building Code (UBC) scheme of seismic zonation.

Using estimated shear wave velocities for the subgrade materials encountered based on standard penetration testing, calculations indicate that the seismic soil profile within the upper 100 feet at the subject site should be considered Class D, *stiff soil*, as described in the 2015 International Building Code, unless site specific shear wave velocity studies show otherwise. Based on the subsurface profile and the anticipated ground conditions, liquefaction is not a design consideration. Using the USGS National Earthquake Hazard Reduction Program online database, the following probabilistic ground motion values are reported for the subject site.

Intensity Measure Type	Intensity Measure Level 2 percent in 50 Years
0.2 Sec. Spectral Acceleration S_s	0.173g
1.0 Sec. Spectral Acceleration S_1	0.059g

The USGS National Earthquake Hazard Reduction Program online database also indicates a peak ground acceleration (PGA) of 0.086 at the subject site. The PGA is the lower of the deterministic or the probabilistic value with a 2% exceedance probability for a 50-year exposure time at the project site (statistical recurrence interval of 2,500 years).

RADIOACTIVE GASES

According to the Environmental Protection Agency (EPA) and the El Paso County Department of Health, elevated levels of radon gas (4pCi/L or more) have been found in buildings in El Paso County. Radon is a radioactive gas that forms from the natural breakdown of uranium in soil, rock, and water. Radon tends to accumulate in poorly ventilated areas below ground level; however, radon may accumulate inside any above or below grade construction. According to the EPA, radon levels in buildings can be reduced by several methods, including pressurization of the building using a heating, ventilation and air-conditioning system, sealing of cracks in the foundation walls and floor slabs which may allow entry of radon, and using active soil depressurization (ASD) systems. Radon risk and potential mitigation measures should be evaluated by an industry professional based on structure type and potential risk in accordance with established guidelines.

DEVELOPMENT CONSIDERATIONS

Presented below is a discussion of geologic and geotechnical engineering related development considerations, including identified geologic hazards.

Expansive Soils/Bedrock: We recommend the expansive clays and claystone bedrock be overexcavated and replaced with a nonexpansive structural fill where present within 5 feet of the bottom of spread footing foundations, floor slabs and the pool. For PT slab foundations, we recommend a 4-foot overexcavation from the lowest portion of the foundation element/rib. The overexcavation zone should extend 10 feet beyond each building where exterior flatwork is located, including sidewalks and patio areas, and where reduction of heave potential is considered critical. For pavement areas and other areas with movement sensitive exterior flatwork, we recommend a minimum 2-foot overexcavation and replacement. Placement of excavated claystone should be limited to nonstructural areas such as landscape areas to the extent practical. We should be present on site to observe test pits and to assist the contractor in determining the limits of overexcavation that will be required.

Site Grading and Surface Drainage: Proper surface drainage is very important for acceptable performance of the development during the proposed construction and after construction has been completed. Development plans should attempt to place the buildings relatively high with respect to the surrounding ground. Grading to accommodate the collection and diversion of surface drainage away from building and pavement locations is recommended. Site grading modifications should be planned to provide positive surface drainage away from all building and pavement areas and wetting of subgrade soils should be prevented. The ponding of water should not be allowed in backfill material or in a zone within 10 feet of the foundation walls of the structure, whichever is greater. We recommend a minimum slope of 6 inches in the first 10 feet in unpaved areas. Site drainage beyond the 10-foot zone should be designed to promote runoff and reduce infiltration. A minimum slope of 3 inches in the first 10 feet is recommended in paved areas. These slopes may be changed as required for handicap access points in accordance with the Americans with Disabilities Act. Surface diversion features should be provided around parking areas to prevent surface runoff from flowing across the paved surfaces. The likelihood of maintaining relatively stable foundations and floor slabs for the life of the project will be significantly increased by planning a well-drained site with little to no irrigation adjacent to structures. Drainage recommendations provided by local, state and national entities should be followed based on the intended use of the structure. The use of proper drainage will also reduce potential runoff impacts to surrounding properties.

Fill should not contain concentrations of organic matter or other deleterious substances. A geotechnical engineer should evaluate the suitability of proposed imported fill materials prior to placement.

Permanent slopes should not be steeper that 3:1 (horizontal to vertical). The risk of slope instability will be significantly increased if seepage is encountered in cuts. If seepage is encountered in permanent excavations, and investigation should be conducted to determine if the seepage will adversely affect the cut stability. Good surface drainage should be provided for all permanent cuts and fills to direct the surface runoff away from the slope faces. Cut and fill slopes and other stripped areas should be protected against erosion by revegetation or other means. Fills should be benched into hillsides that exceed 4 horizontal to 1 vertical. Site grading should be planned to provide positive surface drainage away from all building and pavement areas. No formal stability analyses were performed to evaluate the slopes recommended above. Published literature and our experience with similar cuts and fills indicate the recommended slopes should have adequate factors of safety. If a detailed stability analysis is required, we should be notified.

LIMITATIONS

This study has been conducted for exclusive use by the client for geotechnical related design and construction criteria for the project. The conclusions and preliminary recommendations submitted in this report are based upon the data obtained from the exploratory borings, the site reconnaissance, published regional geology information, the proposed type of construction and our experience in the area. Our services do not include determining the presence, prevention or possibility of mold or other biological contaminants (MOBC) developing in the future. If the client is concerned about MOBC, then a professional in this special field of practice should be consulted. This report may not reflect subsurface variations that occur, and the nature and extent of variations across the site may not become evident until site grading and excavations are performed. If during construction, fill, soil, bedrock, or water conditions appear to be different from those described herein, Kumar & Associates, Inc. should be advised at once so that a re-evaluation of the recommendations presented in this report can be made. Kumar & Associates, Inc. is not responsible for liability associated with interpretation of subsurface data by others.

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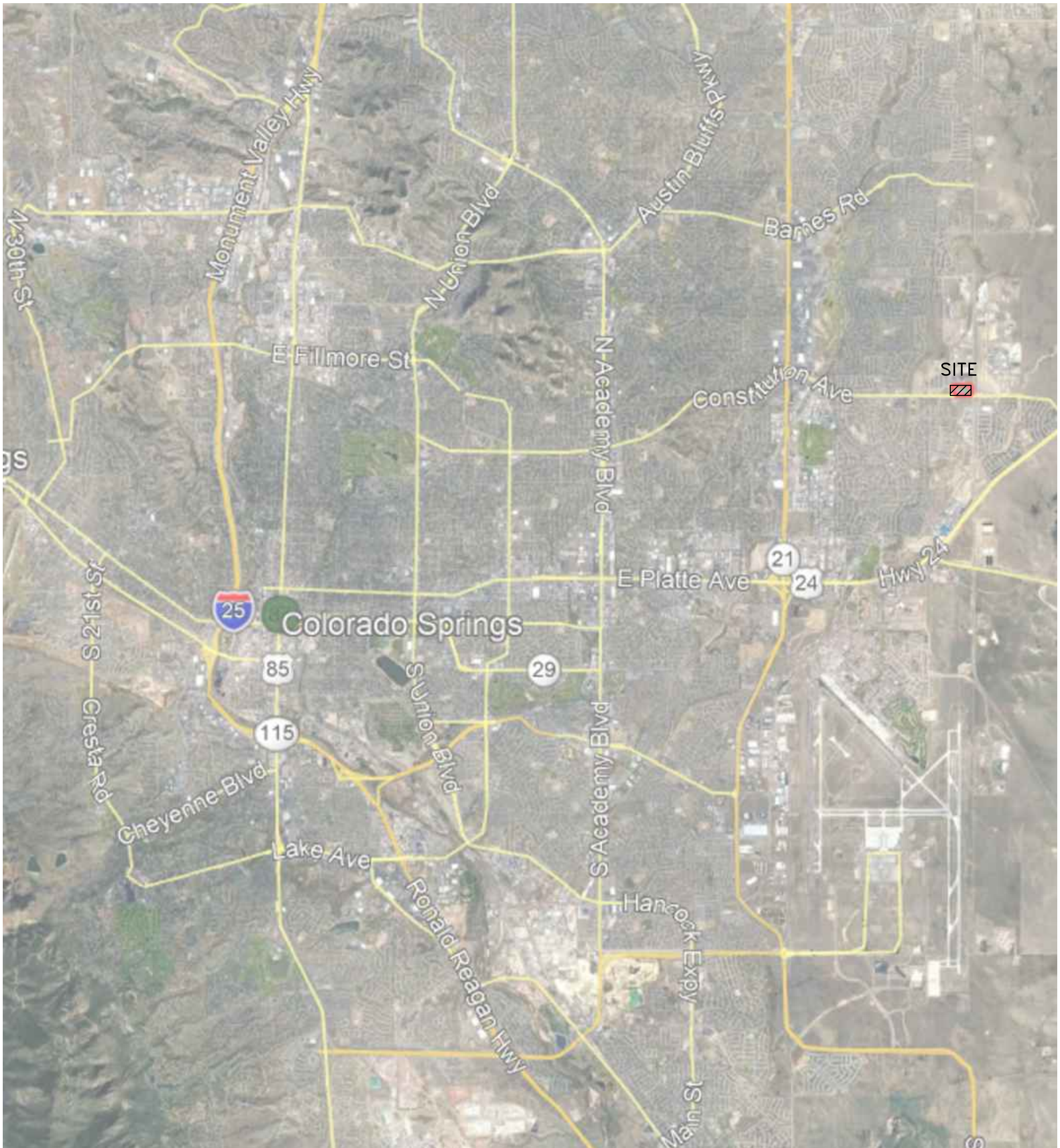

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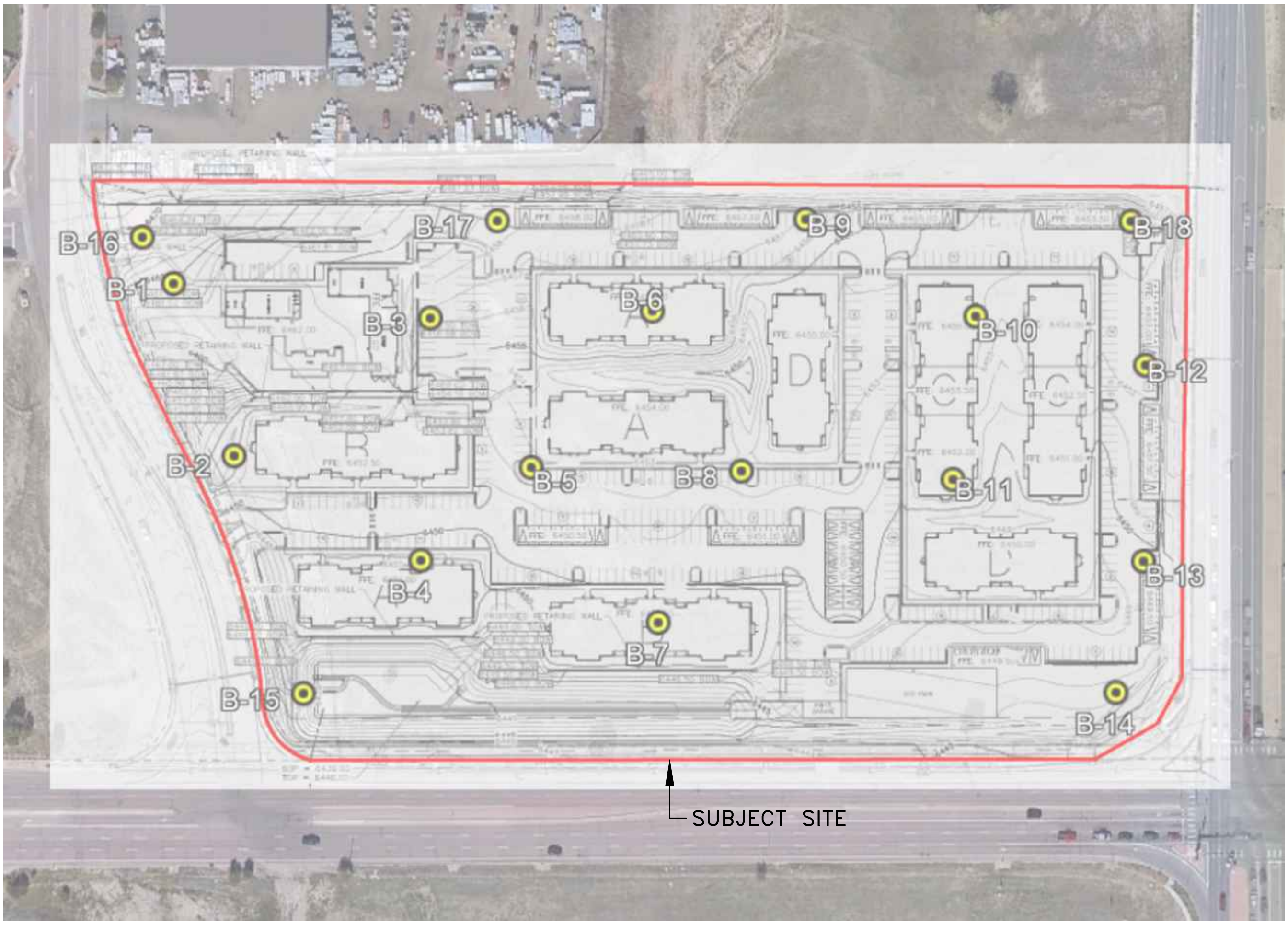
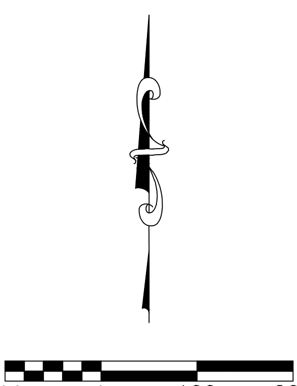
21-2-147	Kumar & Associates	VICINITY MAP	Fig. 1A





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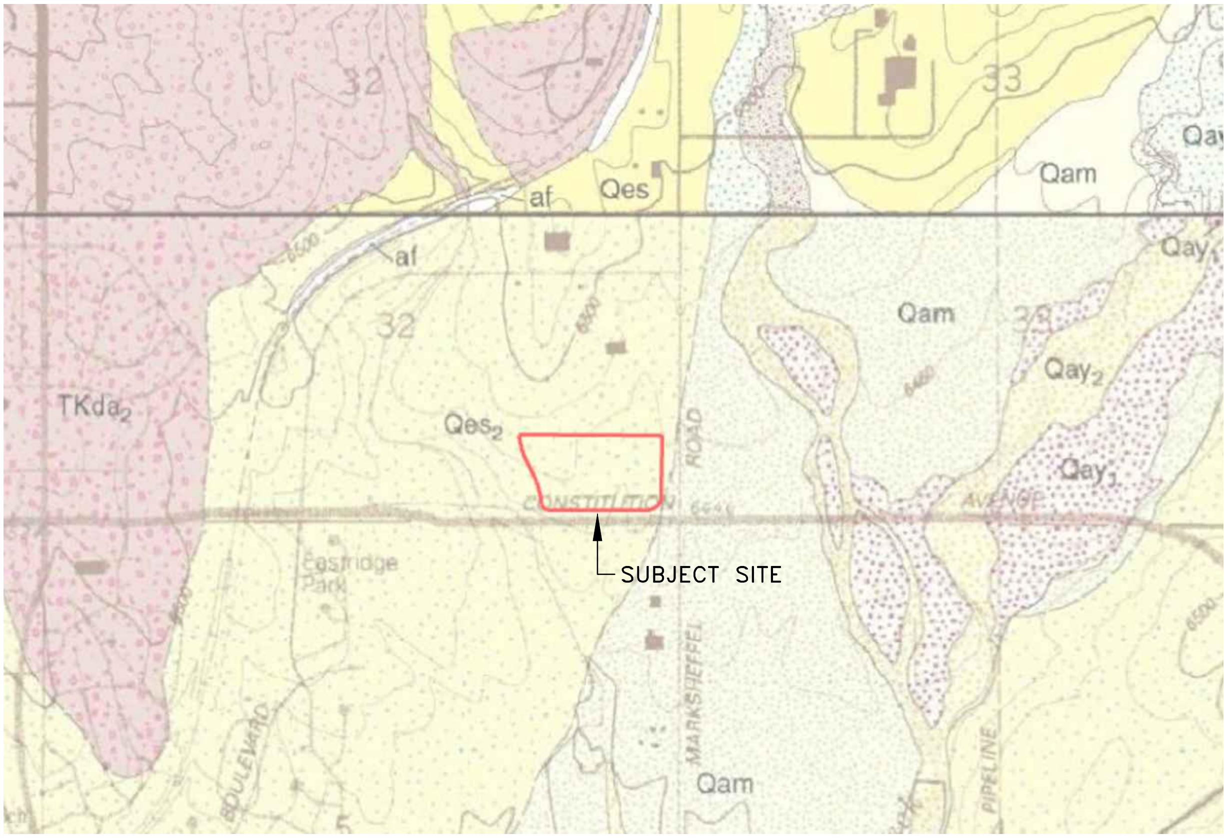
21–2–147	Kumar & Associates	SITE PLAN	Fig. 1B



LEGEND:

af	Artificial fill (late Holocene)
Qa	Alluvium (young, middle, old, & valley-side)
Qay	Young alluvium (one, two, & undivided)
Qay1	Young alluvium one (late Holocene)
Qay2	Young alluvium two (late and middle Holocene)
Qam	Middle alluvium (late Pleistocene)
Qes	Eolian sand (younger & older)
Qes1	Younger eolian sand (middle & early Holocene & late Pleistocene)
Qes2	Older eolian sand (late Pleistocene)
Tkda	Upper part of the Dawson Formation (facies unit one & two)
TKda2	Facies unit two (upper Cretaceous & Paleocene)

800 0 800 1600

APPROXIMATE SCALE–FEET

21–2–147	Kumar & Associates	GEOLOGIC MAP	Fig. 2

LEGEND:

8: Blakeland loamy sand
10: Blendon sandy loam
28: Ellicott loamy coarse sand
97: Truckton sandy loam

800 0 800 1600

APPROXIMATE SCALE–FEET

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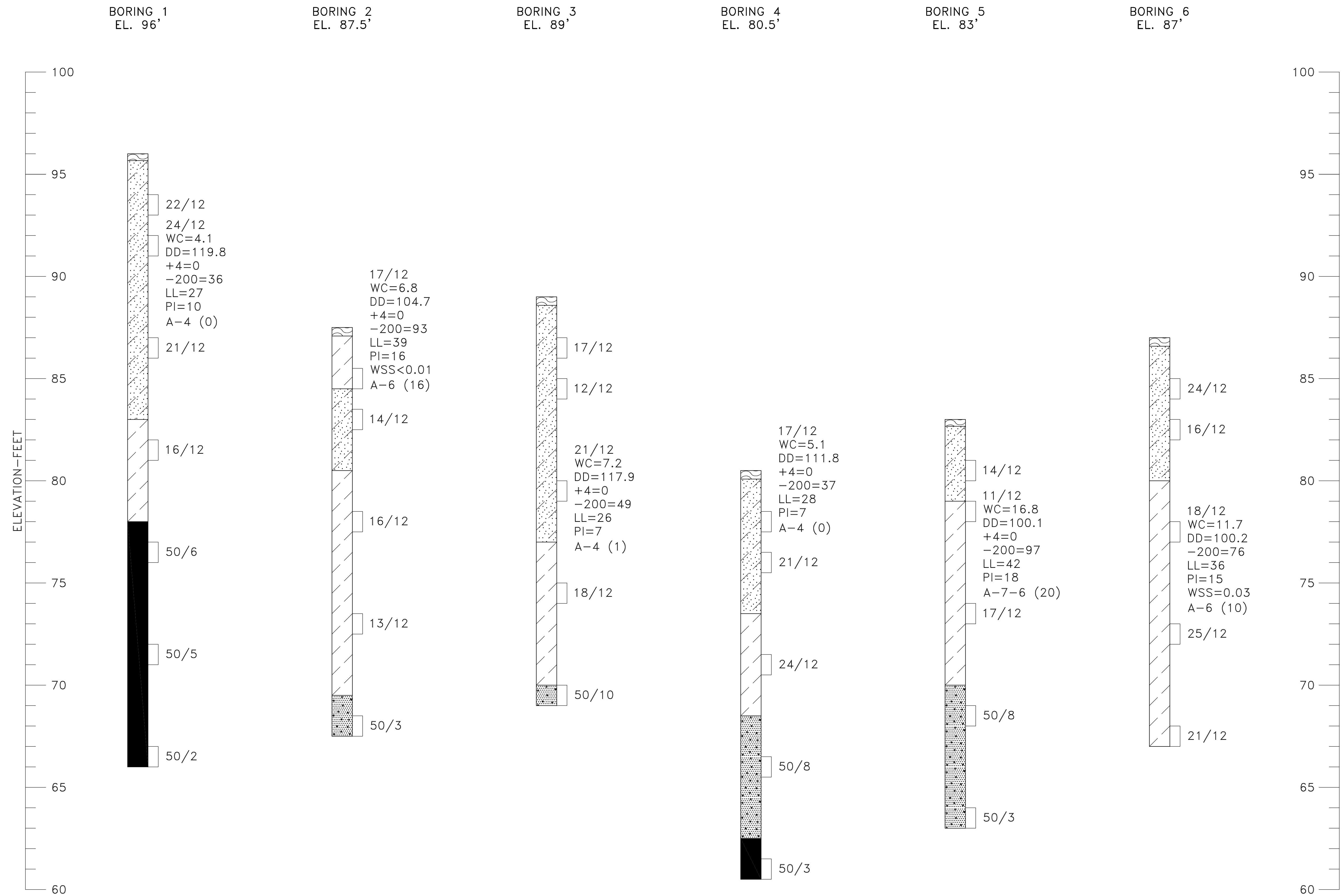
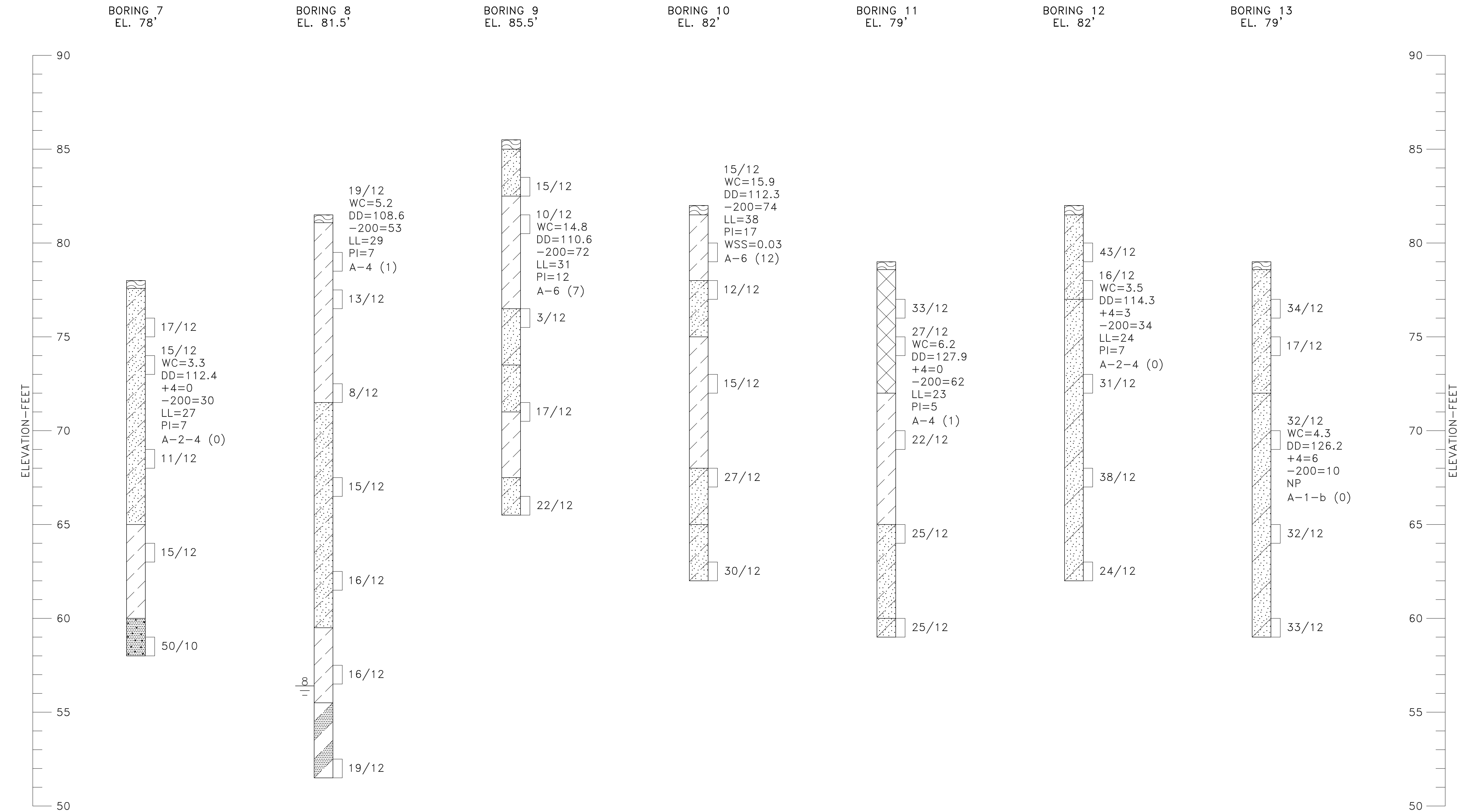
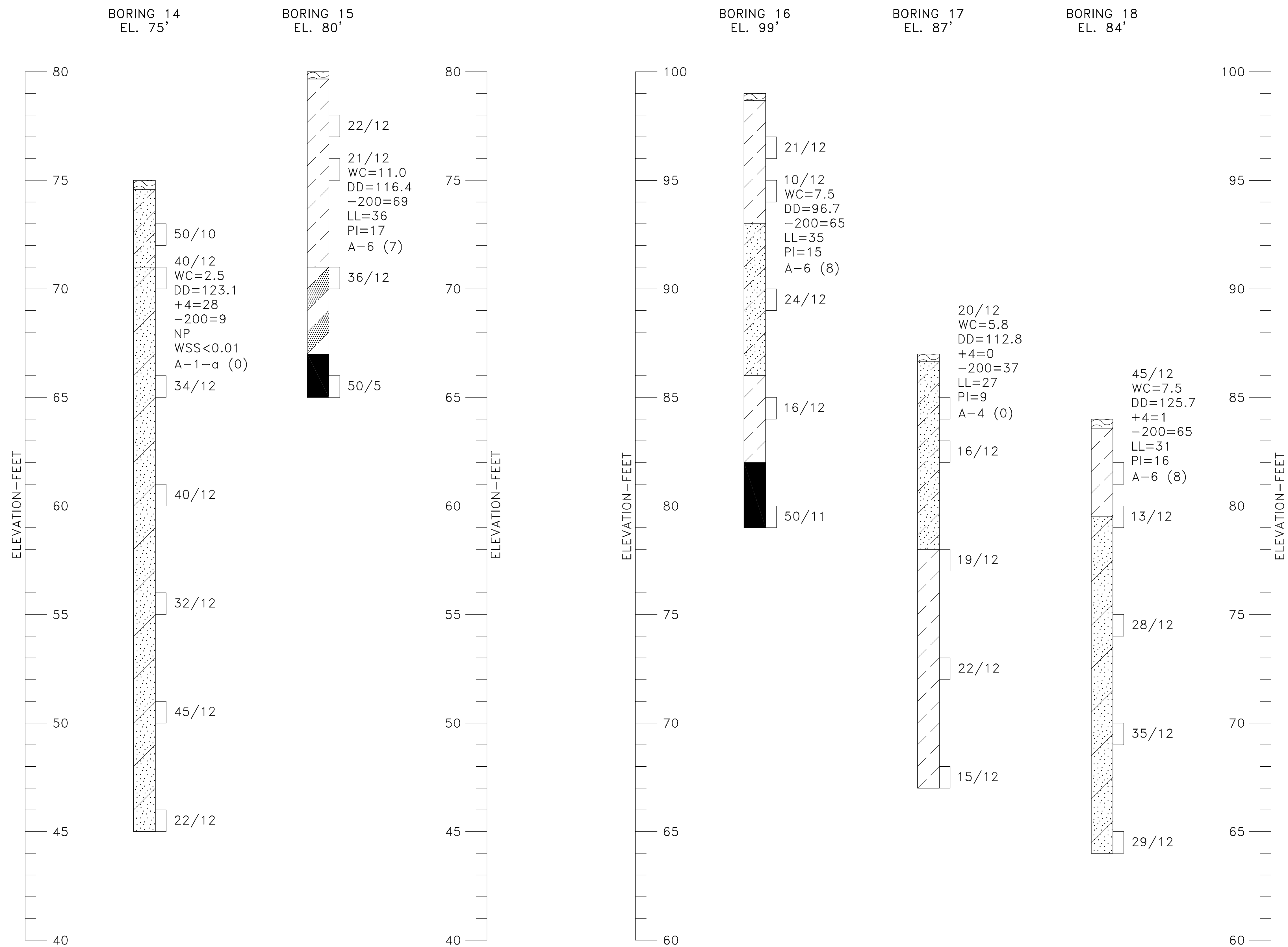
21–2–147	Kumar & Associates	NRCS SOIL SURVEY SOILS	Fig. 3



20–2–194	Kumar & Associates		LOGS OF EXPLORATORY BORINGS	Fig. 4



20−2−194	Kumar & Associates		LOGS OF EXPLORATORY BORINGS	Fig. 5



20−2−194	Kumar & Associates	LOGS OF EXPLORATORY BORINGS	Fig. 6

LEGEND

TOPSOIL.

FILL: CLAYEY SAND (SC) AND SANDY-SILTY CLAY (CL-ML), SLIGHTLY MOIST TO MOIST, LIGHT BROWN TO BROWN.

CLAYEY SAND (SC) AND SILTY-CLAYEY SAND (SC-SM), MEDIUM DENSE TO VERY DENSE, SLIGHTLY MOIST TO MOIST, TAN TO BROWN.

LEAN CLAY WITH VARIED AMOUNTS OF SAND (CL), WITH OCCASIONAL CLAYEY SAND (SC) LAYERS, MEDIUM STIFF TO HARD, SLIGHTLY MOIST TO MOIST, BROWN, DARK BROW AND GRAY.

POORLY TO WELL GRADED SAND WITH SILT (SP-SM, SW-SM), AND SILTY SAND (SM), WITH OCCASIONAL GRAVEL, VERY LOOSE TO DENSE, MOIST TO VERY MOIST, TAN TO BROWN.

SANDSTONE BEDROCK, POORLY CEMENTED, HARD TO VERY HARD, MOIST, BROWN.

WEATHERED CLAYSTONE BEDROCK, VERY STIFF TO HARD, SLIGHTLY MOIST TO MOIST, BROWN TO GRAY.

CLAYSTONE BEDROCK, SANDY, HARD TO VERY HARD, SLIGHTLY MOIST TO MOIST, BROWN TO DARK BROWN, AND GRAY TO DARK GRAY.

DRIVE SAMPLE, 2-INCH I.D. CALIFORNIA LINER SAMPLE.

DISTURBED BULK SAMPLE.

22/12 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 22 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE SAMPLER 12 INCHES.

8 DEPTH TO WATER LEVEL AND NUMBER OF DAYS AFTER DRILLING MEASUREMENT WAS MADE.

NOTES

1. THE EXPLORATORY BORINGS WERE DRILLED ON AUGUST 24 AND 25, 2020 WITH A 4-INCH-DIAMETER CONTINUOUS-FLIGHT POWER AUGER.
2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE APPROXIMATED USING A HANDHELD GPS UNIT.
3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE MEASURED BY HAND LEVEL AND REFER TO THE BENCHMARK ON FIG. 1.
4. THE EXPLORATORY BORING LOCATIONS AND ELEVATIONS SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.
5. THE LINES BETWEEN MATERIALS SHOWN ON THE EXPLORATORY BORING LOGS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN MATERIAL TYPES AND THE TRANSITIONS MAY BE GRADUAL.
6. GROUNDWATER LEVELS SHOWN ON THE LOGS WERE MEASURED AT THE TIME AND UNDER CONDITIONS INDICATED. FLUCTUATIONS IN THE WATER LEVEL MAY OCCUR WITH TIME.
7. LABORATORY TEST RESULTS:
   WC = WATER CONTENT (%) (ASTM D2216);
   DD = DRY DENSITY (pcf) (ASTM D2216);
   +4 = PERCENTAGE RETAINED ON NO. 4 SIEVE (ASTM D6913);
   -200= PERCENTAGE PASSING NO. 200 SIEVE (ASTM D1140);
   LL = LIQUID LIMIT (ASTM D4318);
   PI = PLASTICITY INDEX (ASTM D4318);
   NP = NON-PLASTIC (ASTM D 4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP-L 2103);
   A-4 (0) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M145).

Kumar and Associates, Inc.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

Project No.: 20-2-194
Project Name: Watermark Apartments - Akers Drive, Colorado Springs, CO
Date Sampled: 8/24/2020 and 8/25/2020
Date Received: 8/24/2020 and 8/25/2020

SAMPLE LOCATION		DATE TESTED	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO. 200 SIEVE	PERCENT PASSING 0.002 mm	ATTERBERG LIMITS		WATER SOLUBLE SULFATES (%)	AASHTO CLASSIFICATION (Group Index)	SOIL OR BEDROCK TYPE (Unified Soil Classification)
BORING	DEPTH (ft)				GRAVEL (%)	SAND (%)			LIQUID LIMIT	PLASTICITY INDEX			
1	4	9/2/20	4.1	119.8	0	64	36		27	10		A-4 (0)	Clayey Sand (SC)
2	2	9/2/20	6.8	104.7	0	7	93	62	39	16	<0.01	A-6 (16)	Lean Clay (CL)
3	9	9/2/20	7.2	117.9	0	51	49		26	7		A-4 (1)	Silty Clayey Sand (SC-SM)
4	2	9/2/20	5.1	111.8	0	63	37		28	7		A-4 (0)	Silty Clayey Sand (SC-SM)
5	4	9/2/20	16.6	100.1	0	3	97	66	42	18		A-7-6 (20)	Lean Clay (CL)
6	9	9/2/20	11.7	100.2			76		36	15	0.03	A-6 (10)	Lean Clay with Sand (CL)
7	4	9/2/20	3.3	112.4	0	70	30		27	7		A-2-4 (0)	Silty Clayey Sand (SC-SM)
8	2	9/2/20	5.2	108.6			53		29	7		A-4 (1)	Sandy Silty Clay (CL-ML)
9	4	9/2/20	14.8	110.6			72		31	12		A-6 (7)	Lean Clay with Sand (CL)
10	2	9/2/20	15.9	112.3			74		38	17	0.03	A-6 (12)	Lean Clay with Sand (CL)
11	4	9/2/20	6.2	127.9	0	38	62	38	23	5		A-4 (1)	Fill: Sandy Silty Clay (CL-ML)

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BORING	DEPTH (ft)				GRAVEL (%)	SAND (%)			LIQUID LIMIT	PLASTICITY INDEX			
12	4	9/2/20	3.5	114.3	3	63	34		24	7		A-2-4 (0)	Silty Clayey Sand (SC-SM)
13	9	9/2/20	4.3	126.2	6	84	10			NP		A-1-b (0)	Well Graded Sand with Silt (SW-SM)
14	4	9/2/20	2.5	123.1	28	63	9			NP	<0.01	A-1-a (0)	Well Graded Sand with Silt and Gravel (SW-SM)
15	4	9/2/20	11.0	116.4			69		36	17		A-6 (7)	Sandy Lean Clay (CL)
16	4	9/2/20	7.5	96.7			65		35	15		A-6 (8)	Sandy Lean Clay (CL)
17	2	9/2/20	5.8	112.8	0	63	37		27	9		A-4 (0)	Clayey Sand (SC)
18	2	9/2/20	7.5	125.7	1	34	65		31	16		A-6 (8)	Sandy Lean Clay (CL)