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2.5.4 STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

The U.S. EPR FSAR includes the following COL Item for Section 2.5.4:

> A COL Applicant that references the U.S. EPR design certification will present site-specific information about the properties and stability of all soils and rocks that may affect the nuclear power plant facilities, under both static and dynamic conditions including the vibratory ground motions associated with the CSDRS and the site-specific (SSE).

This COL item is addressed as follows:

{This section addresses site-specific subsurface materials and foundation conditions. It was prepared based on the guidance in relevant sections of NRC Regulatory Guide 1.206, Combined License Applications for Nuclear Power Plants (LWR Edition) (NRC, 2007).

The information presented in this section is based on results of a subsurface investigation program implemented at the Callaway Plant Unit 2 site, and evaluation of the collected data, unless otherwise indicated. The Callaway Plant Unit 1 Final Safety Analysis Report (FSAR) (AmerenUE, 2004) contains a summary of the geotechnical information collected previously for the construction of Callaway Plant Unit 1. Callaway Plant Unit 2 is located adjacent to Callaway Plant Unit 1 (AmerenUE, 2004). The geologic and geotechnical work performed for the Callaway Plant Unit 2 is a "stand-alone" investigation. Its outcome and conclusions do not rely on the existing FSAR Unit 1 information. This document provides the complete investigation data set, including both geotechnical boring logs, and results from the laboratory testing program. The body, tables, and figures in the text organize the data, providing an engineering recommendation for the use of geotechnical parameters.}

2.5.4.1 Geologic Features

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.1:

> Geologic features are site-specific and will be addressed by the COL applicant.

This COL Item is addressed as follows.

{Section 2.5.1.1 describes the regional geology, including regional physiography and geomorphology, regional geologic history, regional stratigraphy, regional tectonic and nontectonic conditions, and geologic hazards, as well as maps, cross-sections, and references. Section 2.5.1.2 addresses site-specific conditions, including site structural geology, physiography and geomorphology, site geologic history, site stratigraphy and lithology, seismic conditions, and a site geologic hazard evaluation, accompanied by figures, maps, and references. Pre-loading influences on soil deposits, including estimates of consolidation, pre-consolidation pressures, and methods used for their estimation are addressed in Section 2.5.4.2. Related maps and stratigraphic profiles are also addressed in Section 2.5.4.2.}

2.5.4.2 Properties of Subsurface Materials

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.2:

> A COL applicant that references the U.S. EPR design certification will reconcile the site-specific soil properties with those used for design of U.S. EPR Seismic Category I structures and foundations described in Section 3.8.

This COL Item is addressed as follows:

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{This subsection presents the properties of underlying materials encountered at the Callaway Plant Unit 2 Site. It is divided into five subsections, as follows.

- Section 2.5.4.2.1 provides an introduction to the soil profile and subsurface conditions,
- Section 2.5.4.2.2 provides a description of the field investigation program, including borings, sampling, and in-situ tests,
- Section 2.5.4.2.3 provides a description of the laboratory testing program,
- Section 2.5.4.2.4 provides a narrative on the origin and characteristics of the engineered fill soils,
- Section 2.5.4.2.5 provides the Callaway Plant Unit 2 recommended soil properties.

These sections are added as a supplement to the U.S. EPR FSAR.

2.5.4.2.1 Callaway Plant Unit 2 Soil Profile

The natural topography at the Callaway Plant Unit 2 site, at the time of the subsurface exploration, was gently sloping. The maximum variation in relief was about 19 ft (5.8 m) across the site. Ground surface elevations at the time of exploration ranged from approximately elevation 833 ft to elevation 852 ft (253.9 m to 259.7 m) mean sea level (msl), with an average of about elevation 846 ft (257.9 m). The ground surface elevations in the Powerblock area ranged from about elevation 839 ft to 852 ft (255.7 m to 259.7 m), with the centerline of Callaway Plant Unit 2 through the Reactor Building at an elevation of 849 ft (258.8 m). The Powerblock includes the Reactor Building, Fuel Pool Building, Nuclear Auxiliary Building, Safeguard Buildings, Radioactive Waste Processing Building, Emergency Power Generating Buildings, Essential Service Water Building, Access Building, and Turbine Building.

The upper 350 ft (107 m) of the subsurface layer was the subject of the Callaway Plant Unit 2 subsurface investigation. The site geology is comprised of glacial and postglacial soil deposits underlain by bedrock, which is, on average, 36 ft (11 m) below the ground surface. The subsurface layer is divided into the following stratigraphic units:

- Overburden Soils:
  - Modified Loess
  - Accretion-Gley
  - Glacial Till
- Rock Formations:
  - Graydon Chert Conglomerate
  - Burlington Formation, Bushberg Formation
  - Snyder Creek Formation, Callaway Formation
  - Cotter-Jefferson City Formation

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Identification of soil and rock layers was based on their physical and engineering characteristics. The characterization of the soils and rocks was based on a suite of tests performed on these soils and rocks, consisting of standard penetration tests (SPT) in soil borings including auto-hammer energy measurements, geophysical testing, pressuremeter tests (PMTs) and laboratory testing.

Figure 2.5.4-1 provides a general soil column profile. Overall, the subsurface conditions encountered throughout the site are uniform, in both depth and area extension. With the exception of Burlington and Bushberg formation, each layer was encountered at each of the boreholes during the investigation.

The loess varies in thickness from 5 ft to 22 ft (1.5 m to 6.7 m), according to the Callaway Plant Unit 2 test borings. This soil was originally deposited as windblown silt forming a loess deposit that has been altered by weathering to a mottled brown and gray, low to moderately plastic silty clay. Occasional lenses of clayey silt or silt have been encountered at the bottom of the loess. The loess is not suitable for the support of large or safety-related structures.

The Accretion-Gley underlies the loess. This material is a moderately to highly plastic gray silty clay. Its thickness varies from 5 ft to 26 ft (1.5 m to 7.9 m) at Callaway Plant Unit 2. This material becomes sticky and soft when exposed to water.

The thickness of the glacial till varies from 3 ft to 19 ft (0.9 m to 5.8 m). The till consists of over-consolidated brown or mottled brown and gray silty clay containing some mixed sand and gravel and occasional lenses of silty or clayey sand.

None of the overburden soils are adequate foundation strata for safety related structures or facilities that will impose high contact pressures. These soils are susceptible to unacceptable levels of both elastic and long-term settlements.

The Graydon Chert Conglomerate is the bedrock horizon for the Callaway Plant Unit 2 site. This unit of rock varied in thickness from 12 ft to 55 ft (3.7 m to 16.8 m). The Graydon Chert Conglomerate consists of hard clay containing irregular Chert fragments and local deposits of indurated sandstone and sandy Chert Conglomerate. The Chert fragments vary from pebble-size to boulders nearly 2 ft (0.6 m) in diameter. No open spaces or voids are expected between rock fragments in this conglomerate. This layer supports large and safety-related structures for Callaway Plant Unit 1.

The Burlington Formation underlies the Graydon chert conglomerate at a depth of about 52 ft to 80 ft (15.8 m to 24.4 m) below ground surface, with an average of about 66 ft (20.1 m). The Burlington Formation is a light tan to brownish gray, medium to massive bedded, coarse grained, cherty fossiliferous limestone. This material is discontinuous across the Site allowing the very thin Bushberg Formation or thicker Snyder Creek Formation to contact the overlying Graydon Chert Conglomerate. The upper surface of the Burlington Formation shows the effects of solution and weathering, but it is predominantly very competent and indurated.

The Bushberg Formation is a thin greenish white to yellowish brown fine to medium grained sandstone which occurs at a depth of about 62 ft to 96 ft (18.9 m to 29.3 m), with an average of about 75 ft (22.9 m) below the existing ground surface. The Bushberg varies in thickness from 0 ft to 7 ft (0 m to 2.1 m), with an average thickness throughout the site of about 2 ft (0.6 m).

The Snyder Creek Formation underlies the Bushberg Formation at a depth of about 62 ft to 99 ft (18.9 m to 30.2 m), with an average of about 77 ft (23.5 m). The top portion of this layer is light gray to brown limestone, while the bottom layer is calcareous shale/mudstone. The thickness of this layer varies from 22 ft to 32 ft (6.7 m to 9.8 m), with an average of about 29 ft (8.8 m).

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The Callaway formation was encountered in 12 borings (7 of which advanced to the next rock layer) at a depth of about 101 ft to 112 ft (30.8 m to 34.1 m), with an average of about 107 ft (32.6 m). The thickness of the Callaway overburden material, based on 7 of 12 borings varied from about 31 ft to 41 ft (9.5 m to 12.5 m), with an average thickness of about 38 ft (11.6 m). This layer is a light gray to brownish gray, fine to coarse grained, and medium to massive bedded.

The Cotter-Jefferson City Formation was encountered in 7 borings (R-1, R-3, R-9, R-13, R-20, R-25 and R-28). The thickness of this layer rock cannot be estimated from the information obtained from Callaway Plant Unit 2 subsurface investigation, as these borings did not penetrate this layer of rock in their entirety, although 3 of the 7 borings (R-1, R-3 and R-13) penetrated the Cotter-Jefferson Formation by more than 200 ft (60.1 m). The formation is encountered at a depth of about 140 ft to 150 ft, with an average of 144 ft.

The thicknesses and termination elevations of rocks are summarized in Table 2.5.4-1. The positions of the soil and rock strata are best visualized by cross section drawings and contour elevation plots. These are developed at locations where the main power block and other safety related facilities will be implemented. The following plots are presented for visualization purposes:

- Figure 2.5.4-2, Boring Layout Plan
- Figure 2.5.4-3, Geophysical Exploration Locations
- Figure 2.5.4-4, Surface Elevation Contours
- Figure 2.5.4-5, Depth to Graydon Chert Conglomerate
- Figure 2.5.4-6, Graydon Chert Conglomerate (Elevation of Top Surface)
- Figure 2.5.4-7, Graydon Chert Conglomerate (Thickness)
- Figure 2.5.4-8, Depth to Bottom of Graydon Chert Conglomerate
- Figure 2.5.4-9, Burlington Formation (Thickness)
- Figure 2.5.4-10, Bushberg Formation (Depth to Top Surface)
- Figure 2.5.4-11, Bushberg Formation (Thickness)
- Figure 2.5.4-12, Snyder Creek Formation (Depth to Top Surface)
- Figure 2.5L-10, Site Geological Cross Sections

2.5.4.2.2 Field Investigation Program

The planning of the field investigation referred to the guidance provided in NRC Regulatory Guide 1.132, "Site Investigations for Foundations of Nuclear Power Plants" (NRC, 2003a). Subsection 2.5.4.2.2.1 provides a brief summary of the field investigation conducted for Callaway Plant Unit 1, and subsection 2.5.4.2.2.2 details the field investigation program for the Callaway Plant Unit 2 site.

A thorough field investigation program was planned and implemented at the Callaway Plant Unit 2 site. The program included:

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- Boring Program,
  - Wash Rotary Drilling and Standard Penetration Test,
  - Rock Coring (NQ Wireline),
- In-Situ Pressuremeter Testing,
- Geophysical Exploration,
  - Downhole Tests,
  - PS Suspension Logging Tests,
  - Deviation Surveys,
  - Reflection Surveys,

2.5.4.2.2.1 Previous Subsurface Investigations

Based on information available from the Callaway Plant Unit 1 FSAR (AmerenUE, 2004), the subsurface investigations for Unit 1 began in March 1972 with a total of 10 C5-Series exploratory borings ranging in depth from 18 ft to 209 ft below the ground surface, with soil and rock samples obtained at various intervals for identification and testing. Supplemental investigations in support of detailed design were performed in November 1972 through February 1975 (148 P-Series borings ranging in depth from 28 ft to 402 ft) and July to August 1973 (8 R-Series borings ranging in depth from 95 ft to 208 ft). Initially, 49 piezometers were installed in selected boreholes for groundwater observation and monitoring in 1974 until construction began. In 1979, eleven permanent piezometers were installed. Additional investigations were performed in 1973 (4 borings) to obtain bulk samples of various soils and to perform plate load tests, and in1974 to 1979, ninety-nine quarry borings (66 Q-Series, 6 A-Series, and 27 H-Series) to investigate existing and potential quarry sites as sources of coarse aggregates. Various laboratory testing was also performed on selected portions of the recovered soils.

Geological descriptions in Callaway Plant Unit 1 FSAR (AmerenUE, 2004) indicate that nine primary layers existed at the site. The modified loess encountered varied in thickness from about 3 ft to 12 ft, with an average thickness of 7 ft. Accretion-gley varied from 5 ft to 24 ft, with an average thickness of 14 ft. Glacial till underlies the accretion-gley deposit with a thickness of about 3 ft to 18 ft. The Graydon chert conglomerate varied in thickness from about 12 ft to 49 ft, with the top of the layer generally at an elevation of about 812 ft. The Burlington Formation was encountered at a depth of about 50 ft to 60 ft below existing grade, and was discontinuous across the site allowing the thin Bushberg Formation to contact the overburden Graydon Chert Conglomerate. Snyder Creek, Callaway and Cotter-Jefferson City formations were competent to the maximum depth of test borings.

Comparable observations were made on these soil and rock layers from the Callaway Plant Unit 2 investigation borings. Given the reasonably parallel geologic conditions between Callaway Plant Unit 1 and Callaway Plant Unit 2, exploration and testing at Callaway Plant Unit 2 should result in enhanced characterization of the subsurface conditions. Findings from previous investigations are not discussed further, unless a differing condition is reported from the previous investigation.

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2.5.4.2.2.2 Callaway Plant Unit 2 Subsurface Exploration

The subsurface exploration was performed in accordance with the guidance outlined in Regulatory Guide 1.132 (NRC, 2003a). Deviations are identified at point of use, and alternatives and/or basis for deviations are provided.

Regulatory Guide 1.132 (NRC, 2003a) provides guidance on spacing and depth of borings, sampling procedures, in-situ testing, geophysical investigations, etc. This guidance was used in preparing a technical specification, addressing the basis for the Callaway Plant Unit 2 subsurface exploration. Per Regulation Guide 1.132 (NRC, 2003a), "the minimum required depth of borings in competent bedrock should extend to the greatest depth where discontinuities or zones of weakness or alteration can affect foundations or at least 20 ft into sound rock. For safety-related structures, one boring per 10,000 sq ft and at least one-fourth of those borings should penetrate into sound rock." In accordance with this guideline, a subsurface exploration program was developed.

In total, 70 R-Series boreholes were completed for the Callaway Plant Unit 2 site, of which 35 boreholes were located in the vicinity of the proposed Category I structures and the remainder were located in other plant locations. It was determined that 3 borings would be extended to depth of 350 ft for detailed core logging and geophysical testing at the location of the proposed reactor building. In addition, 4 borings from the reactor building would be extended to about 160 ft to verify Soil Structure Interaction analyses. Two (2) CH-Series destructive drilling boreholes were extended for geophysical testing purposes in the proposed location of the reactor building.

A team consisting of a geologist, a geotechnical engineer, and a member of the project management performed a site reconnaissance prior to start of the field investigation. The focus of this task was to observe the site and access conditions, locations of borings and wells, and identify potential test relocation areas.

According to Regulatory Guide 1.132 (NRC, 2003a), boreholes with depths greater than about 100 ft should be surveyed for deviation. At the Callaway site, rock was penetrated at an average depth of 40 ft and deviation surveys were limited to boreholes with geophysical testing. Crosshole testing boreholes should be surveyed for deviation regardless of depth. In lieu of surveying for deviation in boreholes greater than 100 ft, deviation surveys were used in the 3 crosshole boring locations (R1, CH-1, CH-2) to depths of approximately 162 ft. The results indicated that the deviation, measured in feet at the bottom of the borehole, with respect to the Plant North (rotated 225.5 degrees with respect to the global coordinate system north) is: 2.2 (R1), 0.1 (CH-1) and 0.5 (CH-2). In the East direction, the recorded deviation is: 1.1 (R1), 0.9 (CH-1) and 0.4 (CH-2).

Regulatory Guide 1.132 (NRC, 2003a) provides guidance for color photographs of all cores to be taken immediately upon removal from the borehole to document the condition of the soils and rocks at the time of drilling. Undisturbed samples were sealed in steel tubes, and could not be photographed. Otherwise, all SPT and rock core samples were photographed.

The Callaway Plant Unit 2 subsurface geotechnical field exploration was conducted from December 2006 through April 2007. Locations of the geotechnical investigation field activities are shown in Figure 2.5.4-2 (Boring Layout Plan) and in Figure 2.5.4-3 (Geophysical Exploration Locations), and the extent of the field tests are summarized in Table 2.5.4-2. Surveying was conducted in order to establish the horizontal and vertical locations of exploration points as shown in Table 2.5.4-3. Each boring location was investigated for the presence of underground utilities prior to drilling boreholes. Boring R-37 was cancelled due to difficulties in placing the drill rig at the specified location due to interference of the wall of an existing warehouse.

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However, sufficient information was available from neighboring boreholes to provide information about the spatial variability of the soils and the depth to bedrock.

Subsurface explorations were performed using geotechnical drill rigs mounted trucks or tracked vehicles. Field borings logs and other field records were maintained by a rig geologist (geologist or geotechnical engineer). A rig geologist was assigned to each rig and was responsible for maintaining the field records associated with activities conducted at a specific exploration point.

Sixty-nine (69) R-Series and two (2) P-Series borings were advanced with SPT sampling, and 19 undisturbed samples (using Shelby push tubes and two Pitcher samplers) were collected from the overburden soils. Soils were sampled using the SPT sampler in accordance with ASTM D1586 (ASTM, 1999). Disturbed soil samples were obtained using 1.5-inch inside diameter split-spoon samplers in conjunction with the SPT, as described by ASTM D1586 (ASTM, 1999). The split spoon sampler was driven a minimum of 18 inches or to refusal. The sampling interval was continuous or 2.5-feet for borings in the vicinity of Category I structures, and 5-feet in the vicinity of the proposed non-safety-related structures. At least one boring below each proposed safety-related structure was performed with continuous sampling as shown in Table 2.5.4-3. The recovered soil samples were visually described and classified by the rig geologist in accordance with ASTM D2488 (ASTM, 2006c). Pocket penetrometer tests were performed on SPT samples obtained, and are shown on boring logs in COLA Part 11F. Two representative samples of the soil recovered from each SPT were placed in glass jars with moisture-preserving lids. The sample jars were labeled, placed in boxes, and transported to the on-site storage facility.

Undisturbed samples were obtained in accordance with ASTM D1587 (ASTM, 2000b) using the push Shelby tubes, and in two instances, the rotary Pitcher sampler. Immediately upon sample retrieval, the disturbed portions at both ends of the tube were removed, both ends were trimmed square to establish an effective seal, and pocket penetrometer (PP) tests were performed on the trimmed lower end of the samples. Both ends of the sample were then sealed with hot wax, covered with plastic caps, and sealed once again using electrician tape and wax to preserve their natural moisture content and prevent soil movement. The tubes were labeled and transported in a vertical orientation to the on-site storage area. Undisturbed samples were stored in an upright position with the top side of the sample up. The locations from which the undisturbed samples were obtained are shown in Figure 2.5L-14. The Pitcher tube sampler was used to obtain undisturbed samples of boring R-2 for two samples after which this method of sampling was abandoned. Pitcher tube sampling was to be performed in fine-grained soils (modified loess, accretion-gley, and glacial till) where SPT blow counts exceeded 30 blows per foot. However, blow counts in these sediments generally did not exceed 30 blows per ft, thus Shelby tube samplers were hydraulically pushed into the soil instead of Pitcher tubes. Although not a fine-grained deposit, the Pitcher tube was used to obtain samples of the Graydon Chert Conglomerate, but this resulted in damage to the tubes because of the hardness of the chert. In addition, any sample obtained of the Graydon chert conglomerate with the Pitcher tube sampler would have been disturbed because the cutter teeth snagged on the chert fragments and rotated them within the matrix of the conglomerate. Only one of the two tube pitcher tube samples obtained from boring R-2 could be analyzed.

A better sampling approach for the Graydon Chert Conglomerate was to use rock coring techniques with NQ tools (1.875-inches core diameter). At the boreholes, SPT soil sampling was continued to the top of bedrock. Once the presumed top of bedrock was encountered, a split-spoon was attempted to verify the presence of rock. Once the presence of rock was confirmed, that is, 50 blows/6" or 10 hammer refusals, rock coring was initiated, as summarized below.

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All boreholes advanced during the field investigation program penetrated the rock layer. The top of the rock layer was identified by the refusal of the split-spoon sampler and/or by the presence of rock fragments in the sampler. Rock coring was performed using wire line core barrels and NQWL dual tube (1.875 inches core diameter), diamond-tipped rock core tools. Dual tube core barrels, 5 ft or 10 ft in length were used to collect continuous rock samples in accordance with ASTM D2113 (ASTM, 2006e). The recovered rock samples were visually described and classified by the engineer or geologist in accordance with ASTM D5878 (ASTM, 2005e). "Routine care" and "special care" rock core samples were collected during this exploration. Routine care samples were placed directly into wooden rock core boxes with a locking lid and photographed. Wood spacers were place in the core box when needed to stabilize the core laterally. Special care samples were photographed immediately upon retrieval, then wrapped tightly in a plastic film and aluminum foil, coated with wax, wrapped in a bubble wrap and stored in a polyvinyl chloride (PVC) tube to preserve the in-situ characteristics. The locations from which special care rock samples were obtained are shown in Figure 2.5L-15.

The rig geologist visually described the core and noted the presence of joints and fractures, distinguishing mechanical breaks from natural breaks where possible. The rig geologist also calculated percent recovery and Rock Quality Designation (RQD) prior to moving the core from the drill site. Field boring logs and photographs were used to document the drilling operations and recovered materials. In borings to be geophysically logged, PVC casing was grouted in place in lieu of the temporary casing.

An on-site storage facility for soil and rock samples was established prior to the initiating the boring exploration program. The site facility provided adequate temperature control conditions in accordance with Regulatory Guide 1.132 (NRC, 2003a). Each soil and rock sample obtained were logged into an inventory system. Samples removed from the facility were noted in the logbook. A chain-of-custody form was completed for all samples removed from the facility. Material storage and handling was in accordance with ASTM D4220 (ASTM, 2000a) and ASTM D5079 (ASTM, 2006f) for soil and rock samples, respectively.

2.5.4.2.2.2.1 Hammer Calibration and SPT Measurements

The depth of soil and rock penetrated by each borehole is shown in Table 2.5.4-3. Soil and rock samples retrieved are identified on the boring logs included in COLA Part 11F.

Energy measurements were made on the hammer-rod system on 3 of the 4 drilling rigs used in the subsurface investigation. One of the rigs was retired from the investigation during early stages due to mechanical failure. This rig was not calibrated, but SPT measurements were only performed at two locations underneath non-safety related facilities. The rig was a CME850 ATV used for SPT sampling and rock coring on four (4) geotechnical borings (R-15, R-45, R-46 and R-48).

A Pile Driving Analyzer (PDA) was used to acquire and process the data. A summary of measured energies is provided in Table 2.5.4-4. The adjusted average field-measured N-values are shown in Table 2.5.4-6. The total number of measurements made at each boring was five (5) for boreholes R-5 and R-33, and 6 at borehole R-55. Energy transfer to the gage locations was estimated using the Case Method, in accordance with ASTM D4633 (ASTM, 2005a). The average energy transfer efficiency measurements ranged from 81 to 90 percent, with an average of 88 percent. As shown in Figure 2.5L-10, the soil on site is relatively consistent, and the blow counts recorded with the CME850 ATV drill rig are consistent with those taken by other rigs used on-site.

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Soil samples were collected from the borings by means of Standard Penetration Test (SPT) and tube samples. Samples were collected more frequently in the borings located in the vicinity of the proposed Category I (safety-related) structures for Callaway Plant Unit 2. For other boring locations on site, samples were obtained at 5 ft intervals. SPT N-values were measured during the sampling and recorded on the boring logs as shown in COLA Part 11F. SPT N-values ranged from 3 blows/ft to 89 blows/ft, with an average measured N-value of 13 blows/ft. Most of the recordings were done in the overburden soils. It was possible to make limited amount of readings in the Graydon Chert Conglomerate. These were typically above 50 blows/ft. SPT information on each overburden soil layer is presented in Table 2.5.4-5. The variability of measured SPT N-values is presented in Figure 2.5.4-13. The figure indicates that, for the overburden soils, a majority of the SPT N-values are within a relatively uniform range of about 5 blows/ft to 20 blows/ft, with occasional higher SPT N-values occurring between a depth of about 30 ft and 40 ft below the ground surface. There are several outlier points of the loess with blow counts above 15 and even above 25. These are due to the presence of fill soils from previous construction. There is a slight increase with depth of the penetration resistance in the overburden soils. At the Glacial Till and at the Graydon Chert Conglomerate, the SPT process was stopped at 50 blows/ft or when hammer rejection was observed. The selected subsurface profiles, Figure 2.5L-10 shows the samples collected with their corresponding SPT N-values and classification symbols. The figures indicate if the sample was disturbed or undisturbed, and the number recorded field SPT blows/ft.

2.5.4.2.2.2.2 Pressuremeter Tests

Pressuremeter tests were conducted in two (2) boreholes on the Callaway Plant Unit 2 site, R-2B, and R-5, in accordance with ASTM D 4917 (ASTM, 2000d) at two depth intervals to measure the volumetric change of a pressurized cell surrounded by in-situ rock, specifically the Graydon Chert Conglomerate. The Pressuremeter test is an in situ stress-strain test performed on the wall of a borehole using a cylindrical probe that is expanded in the radial stress direction. The Pressuremeter was field-calibrated using a steel pipe as the surrounding media of the pressure cell. The pressure and displacement gages were properly calibrated and the field geologists matched the serial numbers with the calibration records documentation. Table 2.5.4-7 presents the results of the borehole pressuremeter tests. The plots showing pressure versus volumetric displacement are shown by Figure 2.5.4-14 and Figure 2.5.4-15. These results are later discussed in Section 2.5.4.2.4 within the context of soil and rock properties.

2.5.4.2.2.2.3 Geophysical Tests

Geophysical tests were conducted in the 2 CH-Series boreholes, and 4 R-Series boreholes. Geophysical logging consisted of high resolution shallow reflection surveys, P-S suspension logging surveys, downhole velocity measurements, and crosshole seismic velocity measurements. During the field operations, it was required to develop new boreholes for the installation of PVC casing and annular space grouting. Existing sampling holes had stability problems, especially in the overburden soils and the Graydon Chert Conglomerate. It was not possible to maintain the integrity of the hole for the install after of the required casing.

This section provides a summary of the geophysical surveys undertaken for the Callaway Plant Unit 2. Information obtained from these surveys was utilized in the analysis of and discussions pertaining to the site geology and characterization of geologic features as presented in Section 2.5.1.2, and surface faulting potential presented in Section 2.5.3.

The location, and depth or extent of each test is shown by Figure 2.5.4-3. Figure 2.5.4-16, Figure 2.5.4-17, Figure 2.5.4-18, Figure 2.5.4-19, Figure 2.5.4-20, and Figure 2.5.4-21, present the plots for compressional and shear wave velocities. The plots provide the results from the three different surveys performed: downhole test, PS-Suspension Logging, and Crosshole Test.

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A high resolution surface wave reflection survey was performed for the three profiles indicated by Figure 2.5.4-3. The results of the survey are provided in Figure 2.5.4-22 through Figure 2.5.4-24.

Downhole Seismic Velocity Surveys

Downhole seismic velocity surveys were conducted in borings CH-2, R-1, R-1B, R-25B, and R-28B. Installation of PVC casing was critical for acquiring good downhole data. The space between the outside of the casing had to be backfilled with low-strength grout to ensure that the casing follows the motions of the adjacent soil or rock materials. The boreholes were purged of water to a depth of 50 feet to reduce the effect of tube waves traveling down the borehole.

Downhole seismic velocity surveys are conducted by measuring the time for seismic waves (generated by an impulsive source at the surface) to travel to a sensor located at a sequence of depths in the borehole. A typical sensor consists of three orthogonal geophones. The two horizontal geophones are used to detect shear-wave (S-wave) arrivals and a vertical geophone is used to detect compression-wave (P-wave) arrivals. Various methods are used to align one of the horizontal geophones with the source polarization. At each measurement level, the sensor assembly is locked to the borehole wall using a clamping mechanism so that the geophones will couple with the seismic signals propagating in the earth.

Seismic waveforms for each depth interval are analyzed and the travel time picked from those waveforms. Interval velocities are calculated and reported as seismic velocity versus depth. This procedure is typically repeated every 2.5 ft (0.76 m) through overburden soil, and every 5 ft (1.52 m) through bedrock. The shear wave source was a wooden plank approximately 6”x 6”x 8 feet with steel end caps and cleats attached to the bottom to better couple with the ground. The plank was struck on one end to generate horizontal waves. The compressional wave (P-wave) source was a sledge hammer blow on a steel or aluminum plate adjacent to the borehole.

Crosshole Seismic Velocity Surveys

The crosshole seismic velocity data will be used as part of the geotechnical site investigation, specifically for earthquake site response modeling, foundation design for dynamic (vibratory) loads, and rock quality assessment. This field test was performed in accordance with ASTM D4428/ D4428M (ASTM, 2000c). Crosshole seismic data provides high-resolution information on stratigraphy and the engineering properties of subsurface soils and rocks that cannot be obtained with surface seismic surveys. Crosshole surveys were conducted in borings CH-1, CH-2 and R-1 shown in Figure 2.5.4-3.

Suspension and Downhole Velocity Logging

Downhole surveys were performed in five (5) boreholes. Comparison of downhole velocity measurements to the P-S suspension measurements for each borehole indicated good correlation of velocity values. P-S suspension velocity logging was performed in borings R-1, R-1B, R-25B, R-28B and CH-2 shown in Figure 2.5.4-3. The objective of the suspension and downhole logging tests was to obtain shear wave (V_S) and compressional wave (V_P) velocity measurements as a function of depth within each borehole.

At this time, an ASTM standard is not available for the P-S suspension velocity logging method; therefore, a brief description follows. P-S suspension velocity logging uses a 23 ft (7 m) probe containing a source near the bottom, and two geophone receivers spaced 3.3 ft (1 m) apart, suspended by a cable. The probe is lowered into the borehole to a specified depth where the source generates a pressure wave in the borehole fluid (drilling mud). The pressure wave is converted to seismic waves (P-wave and S-wave) at the borehole wall. At each receiver location,

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P- and S-waves are converted to pressure waves in the fluid and received by the geophones mounted in the probe, which in turn send the data to a recorder on the surface. At each measurement depth, two opposite horizontal records and one vertical record are obtained. This procedure is typically repeated every 1.6 ft (0.5 m) or 3.3 ft (1 m) as the probe is moved from the bottom of the borehole toward the ground surface. The elapsed time between arrivals of the waves at the geophone receivers is used to determine the average velocity of a 1.6 ft (0.5 m) high column of soil around the borehole.

High Resolution Seismic S-wave Reflection Survey

The purpose of the high resolution S-wave reflection survey is to identify possible anomalies within the subsurface, in particular seismic faults. The geophysical reflection seismic survey was performed in 3 selected lines as shown by Figure 2.5.4-3. Reflection seismic surveys are used to generate a cross sectional acoustic image of the subsurface strata. The survey is conducted by laying out a series of geophones (typically one to six geophones per station for high resolution surveys), connecting the geophones to the seismograph via the seismic spread cables advancing the spread along a profile as the seismic source is activated for each designated source position. The source is activated and the record is written to 4 mm tape and internal hard drive. The Oyo DAS-1 seismic recording system is advanced, the source is moved up and this procedure is repeated. After the source is advanced to the point where no new geophones are available at the end of the spread cable the seismograph and/or jumper cables must be moved to add new spread cables to the end of the profile. The geophone testing procedures and roll switch position tests are to be repeated for all new geophones added to the line. Source shots are repeated as described above. The Seismograph Operator/Seismic Observer recorded pertinent data on a seismic observer's log.

The results of the survey are provided in Figure 2.5.4-22 through Figure 2.5.4-24. The interpretation of the results was performed with borehole data in the form of contour profiles. Figure 2.5.4-5 through Figure 2.5.4-12 provide such contours and do not show any evidence of faulting or discontinuity. The thickness of the overburden to top of bedrock constructed from over 50 borings that penetrated the glacial material shows that there has been no thinning or thickening of the glacial material to suggest that this material has been faulted. Further, the surface of the bedrock (i.e., contact between the glacial material and bedrock) is flat and without relief indicating that post-Pennsylvania faulting has not occurred.

Reflection Line 1: At Reflection Line 1, both the Graydon Chert Conglomerate and the Snyder Creek formation exhibit an undulation of approximately 10 milliseconds (ms) which corresponds to a change in depth of 3-4 ft. There is an apparent offset in the Snyder Creek Reflection at Station 255 of approximately 12 ms. This feature is not interpreted as a true offset since: (1) reflections immediately above and below are continuous, (2) the feature is located close to the end of the line, where statistical redundancy (fold) required for accurate imaging is reduced, and (3) the feature does not correspond with any similar feature on any of the other seismic profiles.

Reflection Line 2: The Graydon Chert Conglomerate and Snyder Creek events undulate across the profiles displayed in Figure 2.5.4-22 through Figure 2.5.4-24 by the amount of approximately 10 ms. A feature is observed between stations 158 and 190 that exhibits an apparent abrupt offset of approximately 13 ms. This appears to be related to a lenticular feature immediately beneath this seismic event. Additionally, reflections above and below the feature are continuous.

Reflection Line 3: A structure with amplitude of approximately 10 ms is evident on the profile. An apparent offset of the Snyder Creek event is observed at about station 154. This feature is located too close to the end of profile to be given credibility due to the reduction

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of fold. Additionally, events immediately above and below the feature are continuous and do not appear to show any vertical offsetting.

The minimal vertical time shifts noted in the Reflection Survey are not interpreted as significant offsets within the Snyder Creek event that could be attributed to faulting. These offsets do not appear above or below the Snyder Creek event. Most likely, the apparent offsets are the result of lenticular facies, a depositional feature, within the Snyder Creek formation. Other apparent offsets may be a result of low fold (low statistical redundancy) that occurs at the line ends of seismic reflection data and thus reducing the confidence of the interpreted apparent offsets.

The results and interpretation of the geophysical tests are further discussed in the following Sections:

- Section 2.5.3, in the context of surface faulting and seismic wave reflection,
- Section 2.5.4.2.5, in the context of recommended soil properties for engineering design purposes,
- Section 2.5.4.4, in the context of the approach to select the best estimate soil column profile for dynamic analysis at the Callaway site.

2.5.4.2.2.2.4 Hydrogeologic Investigation

The hydrogeologic field investigation included a site specific data collection to support a comprehensive hydrogeological evaluation of the Callaway Plant Unit 2 and surrounding areas as required for Section 2.4. The data collected in the field and from other sources (i.e., Callaway Plant Unit 1 FSAR, USACE, USGS) were utilized to support the surface hydrology analysis, hydrogeological characterization, and the development of a groundwater flow model. The model has the capability to evaluate the impact of successive rain events on groundwater elevations across the facility as well as the hypothetical discharge of water from facility operations and storage structures to the ground and the resulting impact to groundwater flow and transport of radionuclides from the facility. Section 2.4 presents the detailed information related to the hydrogeological field investigation.

2.5.4.2.3 Laboratory Testing Program

The laboratory investigation of soils and rocks was performed in accordance with the guidance of the NRC Regulatory Guide 1.138, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants" (NRC, 2003b). Soil and rock samples were shipped under chain-of-custody from the on-site storage to the testing laboratories. ASTM D4220 (ASTM, 2000a) and ASTM D5079 (ASTM, 2006f) provide guidance on standard practices for preserving and transporting soil and rock core samples, respectively. These guidelines were referenced in preparing technical specifications for the Callaway Plant Unit 2 subsurface investigation, addressing sample storage and transportation, as well as other subsurface investigation and geotechnical requirements.

Laboratory testing consisted of testing soils and rocks samples obtained from the subsurface investigation program. Laboratory testing of soil samples consisted of index and engineering properties on selected SPT disturbed samples, strength, consolidation, and permeability on undisturbed samples and rock cores recovered from borings. Laboratory tests included the following: engineering classification, moisture (water) content, unit weight, specific gravity, Atterberg limits, grain size (sieve and hydrometer), percent passing #200 sieve, permeability, consolidated-undrained triaxial compression (Cū), unconfined compression (UC), consolidation, resonant column torsional shear (RCTS), free-free resonant column (FF), resistivity, chloride ion content, and sulphate ion content.

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The number and types of tests selected were consistent with the field investigation findings, and the uniform conditions encountered at the site. Overall, the SPT blow counts were very consistent both in depth and spatial distribution. The soil strata at the site were distinguishable and there was a good correlation between the in-situ soil classification and the SPT results. At the Callaway Plant Unit 2 Site, the comprehensive index testing program along with refined testing at strategically selected locations has provided the required information to adequately characterize the soil properties.

A summary of laboratory tests and specifications used for the laboratory testing program is shown in Table 2.5.4-8. The soil and rock laboratory tests listed in Regulatory Guide 1.138 (NRC, 2003b) are common tests performed in most well-equipped soil and rock testing laboratories. Additional tests that are not covered in regulatory guides are also being performed for the Callaway field exploration (i.e. RCTS, FF, and chemical tests).

Resonant Column Torsional Shear (RCTS) and Free-Free Resonant Column (FF) were performed at the laboratory of the University of Texas at Austin.

The following sections provide a summary of each test, showing the most important and relevant results.

2.5.4.2.3.1 Laboratory Index Tests

Soil samples were classified in the laboratory using the Unified Soil Classification System (USCS) in accordance with ASTM D2487 (ASTM, 2006a). Rock samples were classified in the laboratory using the Unified Rock Classification System (URCS) in accordance with ASTM D5878 (ASTM, 2005e).

2.5.4.2.3.1.1 Grain Size Analyses

Grain size analyses were performed on selected SPT samples of overburden soils and Graydon chert conglomerate. The grain size/hydrometer tests were done in accordance with ASTM D422-63 (ASTM, 2002a), and the percent passing #200 sieve was done in accordance with ASTM D1140 (ASTM, 2006e). The results of these tests were used for classification and correlation purposes.

2.5.4.2.3.1.2 Atterberg Limits and Moisture Content

SPT samples of soil and Graydon chert conglomerate were tested to evaluate their plasticity characteristics, in accordance with ASTM D4318 (ASTM, 2005b). The results of these tests were used for classification and correlation purposes.

Moisture content was determined from samples in accordance with ASTM D2216 (ASTM, 2005d). The water content and Atterberg limits are plotted versus depth by Figure 2.5.4-25. Consistently throughout the site, and down through the depth of the borings, the natural moisture content is close to the plastic limit, suggesting an over-consolidation condition. The Casagrande plasticity chart for the overburden soil is shown in Figure 2.5.4-26, using the following equation for the A-line and U-line (FHWA, 2002):

A-line: $PI = 0.73\ (LL-20)$ Eq. 2.5.4-1

U-line: $PI = 0.90\ (LL-8)$ Eq. 2.5.4-2

2.5.4.2.3.1.3 Unit Weight Determinations

Unit weight determinations were made based on a weight-volume relationship on SPT samples soil samples and Graydon chert conglomerate. The moisture and unit weight determinations

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on rock core samples were obtained in conjunction with unconfined compression tests and dynamic tests. For the case of soils, Table 2.5.4-9 lists the samples with the corresponding dry and wet unit weights. Table 2.5.4-10 provides the unit weight results for rocks samples from the static and dynamic testing program. Unit weight measurements were performed during resonant column testing tests and these are included in Table 2.5.4-10.

2.5.4.2.3.1.4 Specific Gravity

Specific gravity tests were performed on soil samples and Graydon chert conglomerate in accordance with ASTM D854 (ASTM, 2006 b). Typical values of specific gravity of most soils lie within the narrow range of 2.7 ± 0.1. In using ASTM D854, Graydon chert samples had to be crushed. For hard rocks samples, ASTM D6473 (ASTM, 2005c) was used to determine the specific gravity. Specific Gravity results are listed in Table 2.5.4-11.

2.5.4.2.3.1.5 Total Organic Content

Total organic content tests were performed on soil samples in accordance with ASTM D2974-07. Typical values of total organic content lie within the range of 1.3 percent to 2 percent.

2.5.4.2.3.1.6 Chemical Classification Tests

Chemical tests were conducted on SPT samples selected from the overburden soil and Graydon Chert Conglomerate layers that have been identified at the Callaway Plant Unit 2 Site. These tests provide quantitative information related to the aggressiveness of the soil conditions, and the potential for deterioration of a foundation material. The following chemical tests were conducted on samples from the Callaway Plant Unit 2 site: resistivity; chloride ion content; and sulphate ion content. The results of the tests are provided in Table 2.5.4-12

2.5.4.2.3.2 Laboratory Performance Tests

2.5.4.2.3.2.1 Consolidation

One-dimensional consolidation tests were conducted on undisturbed samples, in accordance with ASTM D2435 (ASTM, 2004d), to determine their compressibility properties. Table 2.5.4-13 provides a summary of the results.

2.5.4.2.3.2.2 Consolidated-Undrained Triaxial Compression with Pore Water Pressure Measurements

Consolidated-undrained triaxial compression tests with pore water pressure measurements were performed on selected undisturbed samples under confining pressures representative of their in-situ condition in order to determine total stress and effective stress strength parameters. The tests were conducted in accordance with ASTM D4767-04 (ASTM, 2004c). Table 2.5.4-14 provides a summary of the results.

2.5.4.2.3.2.3 Unconfined Compression

Unconfined Compression tests were conducted on representative rock core samples to determine their compressive strength, in accordance ASTM D7012-04 (ASTM, 2004e). Due to the nature of the Graydon Chert Conglomerate and the underlying Bushberg, Burlington, and Snyder Creek formations, some rock core samples could not be prepared to meet the ASTM tolerances. Thus, additional preparation time and effort were required as follows:

Rock core specimens were cut to a length using a diamond wet blade. The ends could not be machined flat due to the fragile nature of the rock core.

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Test specimens were measured in general accordance to ASTM D4543-07 (ASTM, 2007). The core ends could not meet the requirements of the specification due to the fragile nature of the core that could not be machined flat.

The core measurements for the average diameter, average length and average weight were recorded, and used in calculating the core unit weight and compressive unit load.

The core was then tested for unconfined compressive strength.

Table 2.5.4-15 presents a summary of the Unconfined Compression test results. The core samples of Graydon Chert Conglomerate are typically formed by the matrix of soils and gravels or boulders. Therefore, most of the samples did not present resistance to unconfined compression. This condition was expected and better criteria to define the geotechnical performance of the Graydon Chert Conglomerate are the shear wave velocity measurements and the pressuremeter tests. Section 2.5.4.2.4 discusses the recommended geotechnical performance parameters.

2.5.4.2.3.2.4 Rock Sample URCS Engineering Classification

Table 2.5.4-16 provides the URCS engineering classification of the rock core samples tested for unconfined compression. The Graydon Chert Conglomerate was the main target of the investigation, and as expected it has a classification that is typical of a highly weathered or fractured rock.

2.5.4.2.3.2.5 Permeability

Laboratory permeability tests were performed to determine the hydraulic conductivity of undisturbed samples, according to ASTM D5084 (ASTM, 2003a). Results of the permeability tests are presented in Table 2.5.4-17. The tests were performed on undisturbed samples recovered from the overburden clays. Section 2.4 presents detailed information related to the hydrogeological field investigation and additional information regarding permeability and hydraulic conductivity.

2.5.4.2.3.2.6 Resonant Column Torsional Shear

Resonant Column Torsional Shear (RCTS) tests were conducted according to the procedure developed by the University of Texas at Austin (UTA) entitled, “Technical Procedures for RCTS Tests,” (PBRCTS-1, Rev. 4, October 2004). The tests targeted the overburden clays and the Graydon Chert Conglomerate. Three tests were performed for the overburden and four tests were done on Graydon Chert Conglomerate Special Care Rock Core Samples. The clay materials were obtained using thin-walled samplers. Graydon Chert Conglomerate samples had to be specially wrapped to preserve the moisture content of the extracted core. Opposite to the soil case, the Graydon Chert Conglomerate rock samples were maintained in a horizontal position to preserve the original clay-rock matrix as intact as possible. These samples were extracted with the use of a core barrel after drillers and geologists in the field exercised extreme care to preserve the core.

The RCTS testing program included a total of six tests. The samples used for testing are listed by Table 2.5.4-18. Three tests were scheduled for the Graydon Chert Conglomerate in order to identify the sample that best represented the field conditions. It was anticipated that the RCTS tests on Graydon Chert Conglomerate would reflect the behavior of the soil matrix rather than that of the cherty fragments. Regardless, the laboratory staff invested special care and additional time to prepare representative samples with larger diameter particles. The purpose was to obtain results that account for the presence of these particles and for the contact between soil and rocks. This was successfully accomplished.

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The RCTS test is performed in a series of steps that incorporate different confining pressures and loading frequencies. The Torsional Shear portion of the test is able to capture physical properties at large strains under lower frequency loading that best resembles the seismic demand. The details of the testing methodology are documented by the procedure "UTA, 2004a - Technical Procedures for RCTS Tests, Revision 4, PBRCTS-1, University of Texas at Austin, October 2004."

Resonant Column (RC) and/or Resonant Column Torsional Shear (RCTS) testing are performed to measure two critical parameters in laboratory soil (and sometimes rock) specimens:

1. Shear modulus, which is directly related to shear wave velocity of the soil (Equation 2.5.4-3)

   Eq. 2.5.4-3

$$G = \rho V_s^2$$

2. Damping, which allows for the dissipation of the energy released during an earthquake or any given vibratory process.

Both the shear modulus and damping depend on:

- The amount of strain (or unit deformation),
- The confining pressure,
- The frequency of the motion, in this context, the frequency of the cyclic load applied during testing.

Table 2.5.4-19 presents the results for shear modulus and damping at low strains. The table highlights the medium range confining pressure applied during testing. The most distinctive item of the results shown by the Table occurs for the Graydon Chert Conglomerate. There is a substantial difference in the maximum shear modulus and damping between samples retrieved from the same formation at the same depth. A discussion of the recommended values for engineering purposes is included in Section 2.5.4.2.4. The strain dependency variation of the shear modulus and damping is shown in the form of normalized plots by Figure 2.5.4-27, Figure 2.5.4-28, Figure 2.5.4-29, Figure 2.5.4-30, Figure 2.5.4-31, and Figure 2.5.4-32, respectively for each of the samples tested. The results from the test are compared against relevant generic curves available. A discussion for recommended values is presented in Section 2.5.4.2.4 and Section 2.5.4.7.

2.5.4.2.3.2.7 "Free-Free" Resonant Column Testing

The Free-Free Resonant Column Tests (FF) was conducted according to the procedure, also developed by UTA, entitled, "Technical Procedures for URC Tests," (URC-1, Rev. 4, August 2004). The Free-Free resonant column device allows for a simpler setting that can measure small-strain shear modulus (G_{max}) and small-strain material damping (D_{min}). The term "Free-Free" is used to differentiate from the "Fixed-Free" condition of the typical RC test, meaning that one end of the sample is fixed while the other is free to rotate or displace. No confining pressure is used in this test. A total of nine "Free-Free" tests were performed on special care rock samples retrieved from various boring locations. Table 2.5.4-20 lists the samples with photographs and Table 2.5.4-21 shows the results. A discussion of how the "Free-Free" testing results are used for the analysis is presented in Section 2.5.4.2.4 and Section 2.5.4.7.

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2.5.4.2.4 Engineered Soils

Category 1 Granular Structural Fills and Backfills will originate from either the Reform or Auxvasse quarries, which were used for the development of Callaway Plant Unit 1. Structural fill is used below Category I structure basements and structural fill is used to refill around Category I structures. In case that cohesive fills are required, these may originate from the loess materials extracted during the excavation or some other nearby borrow area depending on material properties needed.

2.5.4.2.4.1 Category 1 Granular Structural Fill

Category 1 Granular Structural Fill from one of the following sources: (1) Callaway Limestone Formation obtained from Mertens, Inc. Reform MO quarry approximately six (6) miles north of the site, and/or (2) Portion of the Auxvasse Quarry, located about 30 miles North of the site close to Route 54. The fill will not include or add organic matter and material above or below the Callaway Formation. Category 1 Structural Fill should meet the gradation limits specified by Table 2.5.4-22. A Laboratory Testing Program has been implemented to fully characterize the properties of the proposed material. The tests included:

- Modified Proctor tests,
- Triaxial tests on remolded samples,
- Grain size,
- Resonant Column Torsional Shear,
- Chemical Tests

The Modified Proctor Test results, showing optimum moisture content and maximum unit weight are provided by Table 2.5.4-22. The optimum water content is about 6 percent and the material proved to be quite dense with maximum moist unit weights above 150 pcf (kN).The grain size analysis result is compared against the required specification. Table 2.5.4-22 provides the recommended properties for structural fill.

2.5.4.2.4.2 Category 1 Granular Structural Backfill

The specifications for Category I Granular Structural Backfill are provided in Table 2.5.4-22. It allows a slightly higher percentage of fines and lower compaction requirements.

2.5.4.2.4.3 Category 1 Cohesive Fill

Category I Cohesive Fill will be modified loess obtained from the site excavation or some other nearby borrow area depending on material properties needed.

2.5.4.2.5 Recommended Soil, Fill, and Rock Properties

The following paragraphs provide recommendations of soil properties for engineering analysis and design purposes. The properties are based on a combination of field measurements, laboratory testing, engineering analysis, engineering judgment, and available reference material. The properties are given for each of the geologic units found during the investigation as described by Section 2.5.4.2.1 and presented by Figure 2.5.4-1. Those units are listed below:

- Soils:
  - Modified Loess

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- Accretion-Gley
- Glacial Till

- Rock Formations:
  - Graydon Chert Conglomerate
  - Burlington Formation, Bushberg Formation
  - Snyder Creek Formation, Callaway Formation
  - Cotter-Jefferson City Formation

In addition to the existing soils, it is required to provide properties for engineered fills that will likely be placed as foundation media for safety related structures. The Nuclear Reactor Building and its adjacent facilities will be placed directly on top of the Graydon Chert Conglomerate. This will not be the case for the Essential Service Water Buildings or the Emergency Power Generating Buildings. Therefore, soil properties are also given for:

- Category I Granular Structural Fill
- Category I Granular Structural Backfill

This Section is divided in:

- Classification and Index Properties,
- Strength Properties,
- Performance Properties (Consolidation and Permeability),
- Static Elastic Properties,
- Dynamic Elastic Properties, and
- Chemical Properties.

The overburden soils consist of three soil layers: Modified Loess, Accretion-Gley, and Glacial Till. It is anticipated that this overburden soil will be removed in order for the foundation mats to bear directly on the first rock layer (Graydon Chert Conglomerate) or on engineered fill, as the soil matter is not suitable for the support of large or safety-related structures. The thickness of the overburden soils varies from about 24 ft to 47 ft, with an average of 36 ft. The thickness of the overburden soil or the depth from surface boring elevation to the Graydon chert conglomerate layer is shown in Figure 2.5.4-5 for the Powerblock. The thickness of the soil layers is based on estimating the termination elevations encountered for the layer at the boring locations from the boring logs shown in COLA Part 11F.

The Modified Loess was originally deposited as windblown silt forming a loess deposit that has been altered by weathering to a mottled brown and gray, low to moderately plastic silty clay. Occasional lenses of clayey silt or silt have been encountered at the bottom of the loess. The thickness of this surface soil varies from 5 ft to 22 ft, with an average thickness of about 9 ft. For engineering analysis purposes, modified loess was characterized, on average, as low to

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medium plasticity lean clay. The predominant USCS designation was clay of low plasticity (CL); however, several samples were classified as clay of high plasticity (CH).

The Accretion-Gley was encountered beneath the modified loess. This layer consists of moderately to highly plastic gray silty clay with its thickness varying from about 5 ft to 26 ft, with an average thickness of about 17 ft. This material becomes sticky and soft when exposed to water. It will also swell, which results in volume change and strength reduction upon saturation. Termination elevation varies from 812 ft to 842 ft, with an average of about 821 ft. For engineering analysis purposes, the Accretion-Gley is characterized, on average, as a medium to high plasticity fat clay. The predominant USCS designation was clay of high plasticity (CH); however, few samples were classified as clay of low plasticity (CL).

The Glacial Till is the final soil layer before rock is encountered which underlies the Accretion-Gley. This layer of till consists of highly compacted brown or mottled brown and gray silty clay containing some mixed sand and gravel and occasional lenses of silty or clayey sand. In some borings, a basal saturated sand layer was encountered. The thickness of this layer varies from about 3 ft to 39 ft at test borings, with an average of 11 ft. The termination elevation varies from 803 ft to 813 ft, with an average of 810 ft. For engineering analysis purposes, glacial till was characterized, on average, as low to medium plasticity lean clay with sand. The predominant USCS designation was clay of low plasticity (CL); however, several samples were classified as clay of high plasticity (CH) and clayey sands (SC).

The Graydon Chert Conglomerate restricted the penetration of the spoon sampler. Drilling techniques were modified and wire core barrel drilling was used to extract the samples. The core drilling was successful and the site geologists were able to extract continuous core of Graydon Chert Conglomerate. Coring was continued through the Bushberg, Burlington, Snyder Creek, Callaway, and into the Cotter-Jefferson City Formation. The following subsections provide the recommended soil properties for each unit.

2.5.4.2.5.1 Index Properties

Index properties are:

- USCS Classification (or URCS Classification for Rocks),
- Water Content,
- Dry Density,
- Wet Density,
- Liquid Limit,
- Plastic Index,
- Grain Size (or Fines Content),
- Index properties determined for rocks are:
- Classification
- Unit Weight,
- Specific Gravity.

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Selected samples were submitted for laboratory index tests and testing for determination of engineering properties. Section 2.5.4.3 presents the detail of the laboratory testing program. Table 2.5.4-23 provides the recommended index properties.

2.5.4.2.5.2 Strength Properties

Strength properties are obtained directly from laboratory tests or from field measurements and supplemental calculations. An example of field measurement is undrained shear strength of the Graydon Chert Conglomerate obtained from Pressuremeter Testing (Table 2.5.4-24). Strength properties include the Mohr-Coulomb parameters commonly used for many geotechnical analysis issues such as bearing capacity, slope stability, retaining walls, and foundation design. Table 2.5.4-25 provides:

- Penetration Resistance (SPT)
- Undrained Shear Strength,
- Cohesion (Total Stress Analysis),
- Friction Angle (Total Stress Analysis),
- Cohesion (Effective Stress Analysis),
- Friction Angle (Effective Stress Analysis),
- Unconfined Compression.

The recommended values for penetration resistance are also provided by Table 2.5.4-6. Although the Graydon Chert Conglomerate is not classified as a soil, it is included in the table since it was possible to drive a very limited number of spoon samplers through the upper layers. Strength properties were determined by either correlation with SPT blow counts or by statistical averaging of the laboratory tests.

The undrained shear strength, su, is best determined from empirical correlations with SPT N-values or from laboratory testing, such as Equation 2.5.4-4 (Bowles 1996).

Eq. 2.5.4-4

$$s_u = 2\frac{N}{16} \qquad [\text{ksf}]$$

Equation 2.5.4-4 yields a recommended value of 2.5 ksf, which is consistent with the Terzaghi (1948) recommended value of 2.6 ksf (N=10). The cohesion for total stress analysis (and the undrained shear strength) is recommended based on an average between the field correlation values and the average of the values of the laboratory testing program. Equation 2.5.4-4 is not applicable for the Graydon Chert Conglomerate.

The strength parameters (angle of shearing resistance or friction angle [ϕ' and ϕ], and cohesion intercept [c' and c]) evaluated from Cū Triaxial tests and are shown in Table 2.5.4-14. For the overburden soils (Loess, Accretion-Gley, and Glacial Till), the recommended values are based on the laboratory testing. For the case of the Graydon Chert Conglomerate it was not possible to obtained undisturbed samples for triaxial testing. In addition, the split spoon sampler cannot be driven into the Graydon Chert during Standard Penetration Tests and the amount of jar samples is very limited and inadequate for laboratory procedures. Undrained shear strength could be obtained from Unconfined Compressive Strength Tests. However, the small scale of

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the samples is also not representative of the behavior of the Graydon Chert Conglomerate. The following is considered to provide a recommendation for the strength parameters of the Graydon Chert Conglomerate:

1. Obtain undrained shear strength from Pressuremeter Testing (Table 2.5.4-24)

2. Results are consistent with values reported in FSAR for Unit 1. Callaway Plant Unit 2 Pressuremeter Testing value is 18.5 ksf (890 kPa).

3. Callaway Plant Unit 1 FSAR Recommendation: 14 +/- 2 ksf (670 +/- 96 kPa)

4. In case of drained conditions, assign a conservative drained friction angle of 25° with cohesion of 2.0 ksf, lower than typical values for soft weathered rocks, and consistent with the drained friction of the overburden soils. Drained conditions at the depth of the Graydon Chert Conglomerate are unlikely and not typical of the foundation setting.

Pressuremeter testing yields higher estimates of the shear strength due to the nature of the Graydon Chert Conglomerate. Only with Pressuremeter tests can the rock be tested in its in-situ setting with the original soil structure and compaction characteristics. Pressuremeter testing is the best approach to analyze the elastic and strength properties of the Graydon Chert Conglomerate.

The position of the water table is close to the top surface of the Graydon Chert Conglomerate at a depth of 37.0 ft (11.3m). The foundation grade of the EPR Nuclear Island (NI) and its adjacent facilities will be placed at the top surface of the Graydon Chert Conglomerate slightly below the water table. Given the foundation grade elevation and the size of the foundation mats, the use of average undrained shear strength is recommended. The Pressuremeter Testing confirmed the values published in the Callaway Plant Unit 1 FSAR. A conservative approach is suggested and a value of 10.5 ksf (500 kPa) is recommended as the Undrained Shear Strength of the Graydon Chert Conglomerate. This value is below any of the reported Pressuremeter strengths and it is within the admissible deformation range reported in Plate Load Testing. Furthermore, it will later be shown in Section 2.5.4.10 that this value is consistent with the specifications provided by AASHTO for very dense mixture of fine and coarse grained soil typical of glacial tills and boulder clays. Since the Graydon Chart Conglomerate will develop friction due to its structural composition, a friction of 25 degrees and a cohesion of 2.0 ksf (95 kPa) may be used instead of the undrained shear strength.

Table 2.5.4-25 provides the recommended strength parameter values.

2.5.4.2.5.3 Performance Properties

Two performance properties are provided: (1) permeability, (2) consolidation.

Permeability

Section 2.4 presents detailed information related to the hydrogeological field investigation, and additional information regarding permeability and hydraulic conductivity is available. The Laboratory Testing Program of Section 2.5 focused on specific values at the site and tests were performed on samples extracted from the geotechnical boring program. As expected, the Glacial Till presents higher permeability than its overlying formations.

Table 2.5.4-26 provides the recommended values for permeability.

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Consolidation

Consolidation properties and stress history were assessed via laboratory testing. A summary of the laboratory consolidation test results is presented in Table 2.5.4-13. Results indicate that, on average, the modified loess is pre-consolidated to about 8.6 ksf (410 kPa), with an overconsolidation ratio (OCR) of 5.0. An OCR = 4 and a pre-consolidation pressure of 8.0 ksf (380 kPa) are recommended for modified loess. The Accretion-Gley soils are pre-consolidated to about 6.8 ksf (325 kPa), with an over-consolidation ratio (OCR) of 3.3. An OCR = 4 and a pre-consolidation pressure of 6.8 ksf (325 kPa) are recommended for the Accretion-Gley. For the case of the Glacial Till, one test yielded a low Consolidation Ratio of 0.4. This result is not consistent with other tests and the observations at the field. Based on the description from the boring log (stiff silty clay with high plasticity) and the SPT blow counts from the sample above (16), it is recommended to eliminate this data point. The inconsistency in the test is likely due to the proximity of the sample to the Graydon Chert Conglomerate. This test is an indication of the need to clearly expose the Graydon Chert Conglomerate at the time of excavation. The foundation grade should be placed after the Glacial Till is removed completely and boulders, gravels, and chert are clearly exposed. The recommended values for the Glacial Till are an over-consolidation ratio of 2.0 and a pre-consolidation load of 6.3 ksf (300.0 kPa).

Table 2.5.4-26 provides the recommended values for coefficient of consolidation and void ratio.

2.5.4.2.5.4 Static Elastic Properties

The static elastic properties of interest are the elastic modulus, the shear modulus, and the Poisson's ratio. The shear modulus is derived directly from the elastic modulus and the Poisson's ratio.

Elastic Modulus

The static elastic modulus of soils and rocks is determined from data retrieved during the field investigation and laboratory testing. Several criteria and/or references are used:

1. Correlations with undrained shear strength (Bowles, 1996).
2. Correlations with plastic index (Bowles, 1996).
3. Correlations with SPT blow counts (FHWA, 2002).
4. Pressuremeter Testing (Graydon Chert Conglomerate).
5. Other references.

Each of the criteria is applied and an average or a conservative approach is used for the recommended parameter. Table 2.5.4-27 presents the values according to each criterion and the recommended elastic modulus value. For deeper rock formations, beginning with the Snyder Creek, general references are cited. It is anticipated that static loading in excess of the current in-situ overburden pressure will not occur below the Graydon Chert Conglomerate, and, therefore, the static elastic modulus is not a critical parameter.

Poisson's Ratio

The most representative value of the Poisson's ratio is obtained directly from the shear wave and compressional wave velocity measurements. Equation 2.5.4-5 is used to establish the Poisson's ratio.

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Eq. 2.5.4-5

$$\frac{V_p}{V_s} = \sqrt{\frac{2-2\nu}{1-2\nu}}$$

Static Shear Modulus

The static shear modulus is directly determined from the Elastic Modulus and the Poisson's Ratio with the use of Eq. 2.5.4-6.

Eq. 2.5.4-6

$$G = \frac{E}{2(1+\nu)}$$

Table 2.5.4-28 provides the recommendation for the static elastic properties.

2.5.4.2.5.5 Dynamic Elastic Properties

A comprehensive field geophysical investigation program and laboratory testing program were undertaken to establish the dynamic properties for Callaway Plant Unit 2. The properties are required for site amplification analysis, Soil Structure Interaction (SSI) analysis, and foundation design. The dynamic properties established are:

- Shear Wave Velocity (V_s),
- Compressional Wave Velocity (V_p),
- Density (ρ),
- Poisson's Ratio (ν),
- Maximum Dynamic Shear Modulus (G_{max}),
- Maximum Dynamic Elastic Modulus (E_{max}),
- Damping at small strain or initial damping (DS_o),
- Strain dependant shear modulus and damping (for Overburden and Graydon Chert Conglomerate).

The shear wave velocity profile was determined by means of a data interpretation analysis that incorporated the results from Downhole, Crosshole, and P-S Suspension Logging tests. The details of the analysis are described in Section 2.5.4.4. The recommended values for dynamic properties are presented in Table 2.5.4-29. More descriptions of the procedures followed to provide the recommendation are included in Section 2.5.4.7. The recommended shear wave velocity profile is plotted by Figure 2.5.4-33.

For the case of the overburden soils and the Graydon Chert Conglomerate, dynamic properties will vary depending on the level of strain present in the soil. The two strain dependant properties of interest are Shear Modulus and Damping. Figure 2.5.4-34 presents the recommended curves for the overburden soils and Figure 2.5.4-35provides the recommendation for the Graydon Chert Conglomerate. The recommended curve for the overburden soils is the generic curve that best adapts to the Torsional Shear experimental data. For Shear Modulus, this curve is generic curve (Vucetic) that corresponds to a Plastic Index of 30 or 40. For Damping, the curve with a Plastic Index of 50 is recommended. The same criterion is

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recommended for the Graydon Chert Conglomerate, in this case using the generic curve for shallow soft rock published by Pacific Engineering and Analysis (Silva, 1996).

The analysis assumed the dynamic properties of the engineered structural fill result in a variable profile: starting at the ground surface and down to a depth of 10 feet, the shear wave velocity is 600 fps (180 m/s), from 10 ft (3.0m) to 20 ft (6.1m) deep, the shear wave velocity is 800 fps (245 m/s), and from 20 ft to the contact with the Graydon Chert Conglomerate, the shear wave velocity is 1000 fps (300 m/s).

2.5.4.2.5.6 Chemical Properties of Soils

The chemical properties of the soils are given by Table 2.5.4-12.}

2.5.4.3 Foundation Interfaces

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.3:

> Foundation interfaces with underlying materials are site specific and will be addressed by the COL applicant. The COL applicant will confirm that the site soils have (1) sliding coefficient of friction equal to at least 0.7, (2) adequate shear strength to provide adequate static and dynamic bearing capacity, (3) adequate elastic and consolidation properties to satisfy the limits on settlement described in Section 2.5.4.10.2, and (4) adequate dynamic properties (i.e., shear wave velocity and strain-dependent modulus-reduction and hysteretic damping properties) to support the Seismic Category 1 structures of the U.S. EPR under earthquake loading.

This COL Item is addressed as follows:

{This section discusses the interfaces between the planned structures and other components and the subsurface characteristics. A plot plan showing the location of the borings, seismic lines, and downhole surveys is provided by Figure 2.5.4-1 and Figure 2.5.4-2. Based on the information obtained during the subsurface investigation and laboratory testing program for Callaway Plant Unit 2, it was determined that exploratory trenches were not necessary in order to characterize the soils at the Callaway Plant Unit 2 site. Cross sections showing the main geologic units are presented by Figure 2.5L-10. Contour plans of geologic unit elevations are given by Figure 2.5.4-4 through Figure 2.5.4-12.

2.5.4.3.1 Design Control Document

The U.S. EPR FSAR developed by the AREVA NP provides criteria related to various siting issues, which must be satisfied by the particular features of the Callaway Plant site. The U.S. EPR FSAR identifies the type of information that should be developed to demonstrate that the site is in compliance with the design. Generic soil profiles are listed by the U.S. EPR FSAR and these represent a broad range of foundation media characterized by shear wave velocities ranging from 700 fps to those typical of hard rock conditions. It is expected that this range captures the static and dynamic response of plant SSCs which will, in general, envelop the actual response at sites exhibiting foundation soils with shear wave velocity at foundation level greater than 1,000 fps. At Callaway, the shear wave velocity of foundation media (Graydon Chert) for the Nuclear Island (NI) facilities is 2300 fps (700 m /s). The NI basemat is a monolith that includes the Reactor Building (UJA), Safeguards Buildings (UJH- mechanical and UJK-electrical) and Fuel Building. Table 2.5.4-30 provides the soil conditions that have been considered by the EPR Foundation Interface Document proposed (RIZZO, 2007).

The seismic ground motion utilized in the design of NI structures is defined as a hypothetical free-field outcrop motion at approximately 41 ft below grade, representing the bottom

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elevation of the NI base mat. On the other hand, the design of adjacent safety related structures founded near the ground surface (Plant Grade) use the free-field soil surface ground motion for design. Section 2.5.2 presents Foundation Input Response Spectra (FIRS) for the NI at the base mat elevation as well as the soil surface (Plant Grade). The FIRS are compared with the respective Certified Design Response Spectra (CDRS) presented by the U.S. EPR FSAR. Both the Foundation Input Response Spectra as well as the seismic response of the soil structure system depends on the subsurface soil stratigraphy including the soil layering, layer thickness, layer shear wave velocities and damping and impedance mismatch.

The foundation interface analysis relates to how the foundation medium and its variability affect the bearing pressure distribution and the settlement of the NI Basemat, and other safety related structures in the vicinity, particularly for soil sites. The foundation interface analysis also determines how these same items affect the seismic response of the structures and the foundation medium. The structural design of the NI Basemat is governed by the bearing pressure and its distribution due to dead and live load and seismic forces, as well as the foundation settlements. On the other hand, the seismic loads on the plant structures and foundations are determined by the vertical and the coupled horizontal and rocking response analysis as estimated with Soil-Structure Interaction (SSI) analysis or Rock-Structure Interaction (RSI) analysis, as the case may be.

Dynamic aspects of the foundation interface are discussed in Sections 3.7 and 3.8 of the FSAR. Static aspects such as bearing capacities, settlement and horizontal variability of stiffness and subgrade reaction under the mat are discussed in Section 2.5.4.10. The subsurface soils beneath the NI base mat should have the capacity to support the bearing pressures with a factor of safety of 3.0 under static conditions and 2.0 under Safe Shutdown Earthquake conditions.

2.5.4.3.2 Site Classification

Based on the review of the subsurface conditions at existing Nuclear Power Plant sites, the potential sites for the EPR can be broadly categorized into four primary groups:

- Rock sites,
- Thin soil sites,
- Shallow soil sites, and
- Deep soil sites.

This categorization provides a framework for reporting site-specific conditions in a COLA referencing the EPR relative to the Plant Parameter Envelope (PPE) considered in the standard plant design. Based on the site groups and the parameters specified by Table 2.5.4-31, the Callaway site is classified as a Thin Soil Site over Soft Rock. As such, the Ground Motion Response Spectra (GMRS, 2.5.2) is provided at the top of bedrock or top of Graydon Chert Conglomerate. For the NI, this level corresponds to the top of rock or top of Graydon Chert Conglomerate. The static and dynamic bearing capacity is verified without the need of time dependant settlement computations. Details related to shear wave velocity are included in Sections 2.5.4.2 and 2.5.4.4.

2.5.4.3.3 Horizontal Layering

Most geotechnical analyses, including SSI analysis, settlement analysis and bearing capacity analysis, assume that the soil layers are horizontal and effects of non-horizontal layering are

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practically ignored. Given the characteristics of the subsurface conditions, these assumptions are valid for Callaway Plant Unit 2.

2.5.4.3.4 Uniform Site Conditions

The variation of the dynamic properties between distant points of the NI facilities may be represented by a Lower Bound, Best Estimate and Upper Bound for the shear wave velocity value at the center point of the facility. Subsurface conditions are uniform across the site and this assessment was verified during the geotechnical and geophysical exploration programs.

The maximum variation of the thickness of the Graydon Chert Conglomerate below the NI footprint is approximately 10 ft (3 m) over a distance of 100 ft (30 m), in other words 10 percent difference. As it may be seen by contour maps of the Graydon Chert Conglomerate thickness, there is not a clear or accentuated dipping trend. The site may be considered as uniform.

2.5.4.3.5 Interface Figures

The Interface Figures present cross sections of the site subsurface conditions with the location of the main components of the Project. Safety-related structures are shown at their planned foundation elevation and on top of the corresponding foundation material. Table 2.5.4-32 provides the depth, elevation, and foundation footprint of the Callaway Plant Unit 2 safety-related structures. Elevations and depths may vary slightly due to surface water drainage design. Such variation will be marginal and does not affect the bearing capacity or settlement estimate calculations. Figure 2.5.4-36, provides a plan view with the selected profiles showing interface contacts. Figure 2.5.4-37, Figure 2.5.4-38, Figure 2.5.4-39, and Figure 2.5.4-40, show typical profiles with the structures and the foundation strata. Groundwater levels shown correspond to Normal Groundwater Elevation and Maximum Possible Groundwater Elevation.

Section 2.5.4.5 provides the excavation concept and Section 2.5.4.10 presents bearing capacities and estimated settlements.}

2.5.4.4 Geophysical Surveys

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.4:

> Geophysical surveys are site specific and will be addressed by the COL applicant.

This COL Item is addressed as follows:

{Section 2.5.4.2.2.2.3 presents the results of the geophysical investigation surveys. Section 2.5.4.2.5.5 provides the recommended dynamic soil properties, based on the results from the field investigation and on the post-processing analysis of the retrieved data.}

2.5.4.5 Excavations and Structural Fill

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.5:

> Excavations and backfill are site specific and will be addressed by the COL applicant.

This COL Item is addressed as follows:

{Callaway Plant Unit 2 will utilize a combination of excavation slopes and temporary retaining structures to facilitate construction of below grade portions of the nuclear facility. The planned finish grade for Callaway Plant Unit 2 is at an elevation of approximately 845 ft. The location of the Callaway Plant Unit 2 structures is shown by Figure 2.5.4-36, Figure 2.5.4-37, Figure 2.5.4-38, Figure 2.5.4-39, and Figure 2.5.4-40.

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The materials excavated as part of the site grading are primarily the overburden soils belonging to the clay units. These units are inadequate for foundation purposes and will be removed from the footprint of all the facilities. No in-situ soils will be used for the preparation of granular fills. These soils are predominantly plastic and their composition consists of a wide variety of clay of medium to high plasticity, based on the USCS. The highly plastic or clay portion of these soils will not be suitable for use as structural fill, given the high percentage of fines (average 59 percent) and the average natural moisture content nearly twice the optimum value of 10 percent. Therefore, structural fill shall be obtained from off-site borrow sources.

Sections 2.5.4.5.1 through 2.5.4.5.4 are added as a supplement to the U.S. EPR FSAR.

2.5.4.5.1 Sources and Quantities of Backfill and Borrow

Structural Category 1 fill and backfill will be obtained from the Callaway Limestone Formation obtained from either the Reform MO Quarry, approximately six miles north of the site or portion of the Auxvasse, MO Quarry, about thirty miles north of the site. Each of the quarries is capable of supplying more than the required amount of borrow material. Earthwork operations will be performed to achieve the planned site grades. Excavations for foundations of the proposed Category I structures within the Power Block area, including the ESW pump house and the Essential Service Water Emergency Makeup System (ESWEMS) Pumphouse, will result in removing the overburden soils in their entirety, and will extend to top of the Graydon chert conglomerate. The maximum depth of cut in the overburden soils is estimated to be about 40 ft (12.2 m) for the plant area. The estimated upper bound of the excavation and backfill volume is 600,000 cubic yards (460 thousand cubic meters).

2.5.4.5.2 Extent of Excavations

Permanent excavation and fill slopes, created due to site grading, are addressed in Section 2.5.5. Temporary excavation slopes, such as those for foundation excavation, would be graded on an inclination of at least 1.5:1 horizontal to vertical (H:V) or flatter. The ESWEMS Retention Pond will be constructed as a dug reservoir in the natural soils. The planned dimensions for the pond are 695 ft by 400 ft with the bottom at an elevation of 818 ft, and side slopes of 3:1 H: V.

An excavation plan is provided by Figure 2.5.4-41. The approach for excavation, as recommended in the Foundation Interface Document (RIZZO, 2007), and confirmed by the slope stability analysis (Section 2.5.5), will implement 1.5H: 1V slopes, offset by 6 feet at the base of excavation. A bench (eight feet wide) should be located at the midpoint of the slope. At Callaway Plant Unit 2, structural fill will be required beneath the near-ground founded safety related structures. Figure 2.5.4-41 and Figure 2.5.4-42 show a conceptual excavation scheme for the cruciform and adjacent Nuclear Island structures (e.g., UJA, UJH, UJK, UKA, UKE and UKS), the Emergency Power Generating Buildings (UBP), the Essential Service Water (UQB and URB) structures, and the Turbine Building (UMA). A cut shown under the Reactor Building is to accommodate the tendon gallery. Note that the excavation plan discussed here may change somewhat based upon input from the plant designer and constructor.

2.5.4.5.3 Compaction Specifications

Structural fill sources were identified, as discussed in Section 2.5.4.5.1, several samples of the materials are obtained and tested for indices and engineering properties, including moisture-density relationships. For foundation support, fill beneath mats is compacted to 95 percent Modified Proctor optimum dry density, and backfill against walls is compacted to 90 percent, as determined based on the Modified Proctor compaction test procedure (ASTM, 2002c). The fill is compacted to within 3 percent of its optimum moisture content, which is between 5 and 6 percent. Fill placement and compaction control procedures are addressed in a technical specification prepared during the detailed design stage of the project. It includes

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requirements for suitable fill, sufficient testing to address potential material variations, and in-place density and moisture content testing frequency, e.g., a minimum of one test per 10,000 sq ft of fill placed. The technical specification also includes requirements for an on-site testing laboratory for quality control, especially material gradation and plasticity characteristics, the achievement of specified moisture-density criteria, fill placement/compaction, and other requirements to ensure that the fill operations conform to the earthwork specification for Callaway Plant Unit 2. The soil testing firm is required to be independent of the earthwork contractor and to have an approved quality program. A sufficient number of laboratory tests are required to be performed to ensure that variations in the fill material are accounted for. A trial fill program is normally conducted for the purposes of determining an optimum number of compactor coverages (passes), the maximum loose lift thickness, and other relevant data for optimum achievement of the specified moisture-density (compaction) criteria.

2.5.4.5.4 Dewatering and Excavation Methods

Temporary groundwater control will be required during construction. Measurement of the groundwater conditions at the site indicate that the lower portions of the site excavations will be below the groundwater level. Thus site grading and excavation plans will implement measures to divert these groundwater flows away from excavations, such as, runoff prevention measures and /or trenches. Category I foundations are planned within the upper water-bearing Graydon Chert Conglomerate. Groundwater conditions and dewatering are discussed in Section 2.5.4.12.

Given the soil conditions, excavations are expected to be performed using conventional earth-moving equipment. Excavations will not present any major difficulties. Excavations in the top of the Graydon Chert Conglomerate may require greater excavating effort, such as ripping tools and even explosives; however, the depth excavated in Graydon Chert Conglomerate, will be limited to about 8 ft (2.4m) under the Nuclear Island Facilities. Upon reaching the final excavation levels, all excavations will be cleaned of any loose materials, by either removal or compaction in place. All final subgrades will be inspected and approved prior to being covered by backfill or concrete. The inspection and approval procedure(s) will be addressed in the foundation and earthwork specifications that will be developed during the detailed design stage of the project. These specifications will include measures, such as proof-rolling, excavation and replacement of unsuitable soils, and protection of surfaces from deterioration.

2.5.4.5.5 Earth Pressure Coefficients

Active, passive, and at-rest lateral earth pressure coefficients, K_A, K_P, and K_O, respectively, were estimated assuming frictionless vertical walls and horizontal backfill by the following relationships:

<u>Active Earth Pressure Coefficient:</u>

Eq. 2.5.4-7

$$K_a = \tan^2\left(45 - \frac{\phi'}{2}\right)$$

<u>Passive Earth Pressure Coefficient:</u>

Eq. 2.5.4-8

$$K_p = \tan^2\left(45 + \frac{\phi'}{2}\right)$$

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At Rest Earth Pressure Coefficient:

The At Rest Earth pressure coefficient depends mainly on the deposition process of the formation, and its subsequent stress history. Its value is between the active and passive earth pressure coefficients and there are documented relationships to estimate its value, for example:

General Application, Jaky (1948), Eq. 2.5.4-9

$$K_o = 1 - \sin\phi'$$

In order to account for compaction effects on engineered fills, the Passive Earth Pressure Coefficient is modified with Equation 2.5.4-10 (Sherif et al. 1984).

Eq. 2.5.4-10

$$K_{oc} = K_o(5.8\sin(\phi') - 2.1)$$

Dynamic Active and Passive Earth Pressure Coefficient:

The Dynamic Active and Passive Earth Pressure Coefficients are determined with the use of the Mononobe-Okabe theory. For a vertical wall and a horizontal grade the theory provides Equations 2.5.4-11).

Eq. 2.5.4-11 (a)

$$K_{ae} = \frac{\cos^2(\phi - \theta)}{\cos\theta\cos(\delta + \theta)\left(1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta)}{\cos(\delta + \theta)}}\right)^2}$$

Eq. 2.5.4-11 (b)

$$K_{pe} = \frac{\cos^2(\phi - \theta)}{\cos\theta\cos(\delta + \theta)\left(1 - \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta)}{\cos(\delta + \theta)}}\right)^2}$$

Eq. 2.5.4-11 (c)

$$\theta = a\tan\left(\frac{F_h}{F_v}\right) = \frac{k_h}{1 \pm k_v}$$

Where:

K_{ae} → Dynamic active earth pressure coefficient,

K_{pe} → Dynamic passive earth pressure coefficient,

θ → Angle of resultant of seismic load,

ϕ → Friction angle (Drained conditions),

δ → Friction between wall and soil ($\delta = 2\phi/3$ (Das, 1993)),

F_h → Horizontal earthquake force,

F_v → Vertical earthquake force,

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k_h → Horizontal earthquake acceleration at El. 823 (0.30 g at Callaway)

k_v → Vertical earthquake acceleration at El. 823 (0.32 g at Callaway),

Table 2.5.4-33 provides the recommended earth pressure coefficients.

2.5.4.6 Groundwater Conditions

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.6:

> The COL applicant will address site-specific ground water conditions.

This COL Item is addressed as follows:

{The groundwater data collection and monitoring program was conducted from March 2007 through February 2008. Groundwater monitoring will continue as required for the construction and operation of Callaway Plant Unit 2. Details of the groundwater investigation, evaluation, and results are provided in Section 2.4.12. A brief summary is as follows.

Across the plateau, the shallow aquifer is defined primarily by the Graydon Chert. There are localized areas where the overlying material may be apart of this aquifer, but on the whole it was found that the saturated groundwater zone is confined within the Graydon Chert Conglomerate. During the Callaway Plant Unit 2 field investigation, field personnel identified the Graydon Chert Conglomerate as the major water-bearing unit with the glacial till acting as the confining unit above and the Burlington Limestone acting as the confining unit and top of the aquitard beneath it. The Graydon Chert Conglomerate lies unconformably atop the Burlington Limestone and unconformably below the glacial till so its elevation and thickness vary. Across the plateau, the depth of the Graydon Chert ranges from 15 ft to 39 ft (4.6 m to 11.9 m) below ground surface (bgs) and averages approximately 27 ft (8.2 m) bgs. Its thickness ranges from 16 ft to 61 ft (4.9-18.6 m) and averages approximately 38 ft (11.6 m). At the centrally located MW-18 well (beneath the proposed power block area), the depth to the Graydon Chert Conglomerate is approximately 30 ft (9.1 m) bgs and its thickness is approximately 40 ft (12.2 m).

Due to confined groundwater conditions, piezometric levels measured at the monitoring wells (installed within the Graydon Chert Conglomerate) rise above the top of the Graydon Chert Conglomerate to within approximately 7 to 15 ft (2.1 to 4.6 m) of the ground surface in the central portion of the plateau. Overall, the groundwater elevation, which is not the same as the piezometric level, does not vary much through the year, typically by less than 1 ft to 2 ft (0.3 m to 0.6 m) across the central part of the plateau and several feet at the shallow wells around the perimeter of the plateau. Normal Groundwater Elevation is present just above the top Surface of the Graydon Chert Conglomerate. A Maximum Groundwater Elevation of 828 ft msl is considered for design purposes. This level is below the measured piezometric line of El. 832.0 ft msl, but it is still very conservative since the groundwater table will not rise to the piezometric levels obtained from the Graydon Chert Conglomerate aquifer.

Beneath the shallow aquifer, there is a leaky, confining aquitard. The depth to the top of the aquitard averages 68 ft bgs (20.7 m) across the plateau, and its thickness is approximately 290 ft (88 m) in the central portion of the plateau.

Beneath the aquitard is the Cotter-Jefferson City (CJC) aquifer. The depth to the CJC aquifer is approximately 350 ft (107 m) bgs in the central portion of the plateau. The thickness of the CJC aquifer was not determined; however, based on the well logs for the three AmerenUE industrial wells, the thickness beneath the plateau is approximately 300 ft (92 m). Regionally, the CJC

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aquifer is considered to be a minor aquifer and represents the top of the Cambrian-Ordovician aquifer system, which consists of intervals of minor aquifers and major aquifers with intermittent aquitards to depths up to 2000 ft (610 m) bgs.

Groundwater levels for the deeper CJC wells beneath the plateau are also confined such that measured groundwater levels rise approximately 50 ft (15.2 m) above the top of the CJC aquifer to an approximate elevation of between 550 ft and 560 ft (168 m to 171 m) mean sea level. Although groundwater elevations appear to respond to seasonal changes in precipitation, they vary only by approximately 1 ft (0.3 m).

From the Callaway Plant Unit 1 investigation, it was estimated that the well yield for the Graydon Chert Conglomerate aquifer is less than 1 gpm (3.8 lpm) and for the CJC aquifer is approximately 5 gpm to 10 gpm (19 lpm to 38 lpm). Pumping test results for the Callaway Plant Unit 2 investigation confirm these estimates. Two pumping tests were performed successfully in the CJC aquifer, and the relatively low estimates of storativity are consistent with mildly fractured bedrock aquifers where the small size of fractures and low degree of interconnectedness limits the amount of water in storage and the amount of water to potentially yield to a well. A step-drawdown test at a pumping well in the chert aquifer resulted in a dry well after a short period of time, which made the pumping test unviable. It is expected that the storativity of the chert aquifer is even less than the deeper aquifer.

Sections 2.5.4.6.1 through 2.5.4.6.4 are added as a supplement to U.S. EPR FSAR.

2.5.4.6.1 Dewatering During Construction

Temporary dewatering will be required for groundwater management during excavation and construction of Callaway Plant Unit 2 foundations. During the Callaway Plant Unit 1 construction, the low yield of the glacial and postglacial soil deposits and older sediments resulted in minimal seepage into excavations during construction. The maximum depth of excavations for Unit 1 was below the base of glacial till, extending approximately 15 ft (4.6 m) into the Graydon Chert conglomerate. Even though the highest water table in the site area was about 10 ft to 15 ft (3.0 m to 4.6 m) above the top of the chert conglomerate, neither the postglacial and glacial soils nor the chert conglomerate layer required dewatering.

Observations of the groundwater conditions during construction did not reveal any seepage into the excavations through the cohesive materials. Isolated saturated silt lenses at the bottom of the modified loess and sand lenses in the glacial till did yield seepage when exposed by excavations, but the small seepage did not hinder construction or affect construction quality. Sump pumps located in the excavations were adequate to remove seepage and any runoff occurring after periods of rainfall.

It is anticipated that seepage and subsequent control of seepage will be similar to that encountered during Unit 1 construction. Groundwater control associated with seepage in the shallow units will be controlled through site grading and/or a system of drains and ditches. Water will be diverted to on-site stormwater and sediment control ponds. Temporary dewatering is required for the excavation of the ESWEMS Retention Pond. During the Callaway Plant Unit 1 construction, the Unit 1 Retention Pond was constructed above the Graydon Chert Conglomerate unit. Due to the low permeability of the glacial and postglacial materials, it was considered unnecessary to seal the pond side slopes and bottom with an impervious blanket. The pond side slopes and bottom were inspected during construction. Any sand or silt lenses encountered were removed and replaced with Category I Cohesive Fill.

For Callaway Plant Unit 2 construction, de-watering and stability control will be designed to maintain a relatively dry excavation during construction. As a minimum, sumps will be installed

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to control and/or lower the groundwater level inside the excavation. Full details of the dewatering requirements will be developed during the detailed design stage of the project.

2.5.4.6.2 Analysis and Interpretation of Seepage

Analysis of the groundwater conditions at the site is described in Section 2.4.12. A groundwater model, based on information currently available, has been prepared for the overall groundwater conditions at the site and is addressed in detail in Section 2.4.12.

2.5.4.6.3 Permeability Testing

Evaluation of permeability of the site soils was performed with lab testing of Shelby-tube samples obtained in shallow soils above the Graydon Chert aquifer. Slug and pumping tests were performed on monitoring wells screened in the Graydon Chert and CJC aquifers. A detailed description of the tests and results are provided in Section 2.4.12.

2.5.4.6.4 History of Groundwater Fluctuations

A detailed discussion of the groundwater conditions is provided in Section 2.4.12.}

2.5.4.7 Response of Soil and Rock to Dynamic Loading

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.7:

> The COL applicant will address site-specific response of soil and rock to dynamic loading, including the determination of strain-dependent modulus-reduction and hysteretic damping properties.

This COL Item is addressed as follows:

{The Safe Shutdown Earthquake (SSE) spectra and its specific location at a free ground surface reflect the seismic hazard in terms of Probabilistic Seismic Hazard Analysis (PSHA) and geologic characteristics of the site and represent the site-specific ground motion response spectrum. These spectra would be expected to be modified as appropriate to develop ground motion for design considerations. Detailed descriptions on response of site soils and rocks to dynamic loading are addressed in Section 2.5.2.

Sections 2.5.4.7.1 through 2.5.4.7.6 are added as a supplement to the U.S. EPR FSAR.

2.5.4.7.1 Seismic History

The seismic history of the area and the site, including any prior history of seismicity, evidence of liquefaction or boils, is addressed in Sections 2.5.1.1 and 2.5.1.2.

2.5.4.7.2 Field Dynamic Measurements

The following techniques were used to measure field dynamic properties:

- Crosshole seismic velocity surveys in 3 borings ranging in depth from about 5 ft to 155 ft below ground surface and including overburden soil and rock.
- P-S suspension logging and downhole seismic velocity surveys in 5 borings ranging in depth from about 2 ft to 350 ft below ground surface, including overburden soil and rock.
- High resolution seismic s-wave reflection survey was conducted in 3 borings.

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- Geophysical testing borehole locations are shown on Figure 2.5.4-2.

Three crosshole seismic surveys were completed in boreholes in the vicinity of the proposed reactor at the Callaway Plant Unit 2 site. Data obtained from borehole survey techniques were integrated for development of the site velocity profiles. Each borehole velocity profile was evaluated and compared against the stratigraphic logging and laboratory test data of borehole samples to correlate velocities with soil and rock types by elevation and corresponding depth below ground surface. After each individual borehole velocity data set was evaluated, borehole profiles were grouped based on site-specific location and were compiled using a common reference point (elevation or depth below ground surface).

2.5.4.7.3 Dynamic Laboratory Testing

Dynamic testing, consisting of RCTS and FF tests, to obtain data on shear modulus and damping characteristics of rocks, is described by Section 2.5.4.2.3.2.6.

2.5.4.7.4 Recommended Soil Profile

The Uniform Hazard Spectra (UHS) described in Section 2.5.2 are defined on hard rock, which is located two thousand feet or deeper below the ground surface at the Callaway Plant site. To determine the dynamic motion at the ground surface, it was necessary to adjust the UHS for amplification or de-amplification as the vibratory ground motion propagated through the rock and soil media. The adjustment was made by conducting Site Response Analyses following Approach 2B described in NUREG-6728 (McGuire et al., 2001). These analyses consist of defining the shear wave velocity and material damping characteristics in the soil and rock profile between the ground surface and the depth of hard rock, and then conducting site response studies using a one-dimensional, equivalent linear computer code: SHAKE (Schnabel et al., 1972).

The NI foundation material is the Graydon Chert Conglomerate which has a shear-wave velocity of approximately 2,300 fps. Consequently, the site amplification to define the GMRS for the Callaway Site is computed at the top of the Graydon Chert Conglomerate layer. For the Nuclear Island Facility, the GMRS corresponds to its Foundation Input Response Spectra (FIRS). The site amplification corresponding to the FIRS for other structures is computed at two elevations: 5.0 ft and 23.0 ft below the ground surface.

The Subsurface Investigation at Callaway included extensive Boring and Geophysical Exploration Programs. The field data available are divided into three sets:

1. Shear-wave velocities,
2. Compressional wave velocities, and
3. Layer thickness.

The compressional and shear-wave velocities are taken from geophysical field tests using three different techniques:

1. Five sets of downhole tests,
2. Five sets of suspension logging tests, and
3. Three sets of crosshole tests (from one crosshole array).

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Nine geophysical surveys are concentrated near the center of the Nuclear Island Reactor Building. The spatial variability of geotechnical properties along the site was investigated through four geophysical surveys located under two Emergency Power Generating Buildings. The stratigraphy, layer notation and layer thickness are taken from the geotechnical boring logs R1 to R70. The results from the geophysical investigations are provided in Section 2.5.4.2.2.2.3.

Another source of data for Site Response Analyses was the Missouri Department of Natural Resources (MoDNR MEGA 2007). This source provided data from two wells drilled for their Cambrian System correlation effort (27795 and 28076) that went down to a depth of 1,510 ft (460.3 m). Besides these two stratigraphic records, seven other records were used from the same MEGA 2007 in neighboring Boone and Audrain counties.

The following steps have been used to develop the best estimate of the compression and shear-wave velocity profiles for Callaway Plant Unit 2:

1. The best estimate of the layer thickness was calculated using the average of the individual thicknesses identified for Borings R1 to R7, which are located at the center and on the perimeter of the Reactor Building. Further layer subdivision has been considered when significant changes in either the shear or compressional wave velocities are identified.

2. The statistical scattering of layer thickness was calculated using the individual thicknesses identified for Borings R1 to R52, which are spread over the investigation area, without considering the ESWEMS Retention Pond and the Natural Draft Cooling Towers.

3. One base case for shear-wave velocities and one base case for compressional wave velocities were developed for each field measurement technique: (a) downhole, (b) suspension logging, and (c) crosshole. The base cases for the downhole technique are calculated using the individual surveys of field tests measured for the following Borings: R1, CH2, R1B, R-25 and R-28. The base cases for suspension logging were calculated using the individual surveys of field tests measured for the following Borings: R1, CH2, R1B, R-25B and R-28B. The base cases for crosshole technique were calculated using the individual surveys of field tests measured for the following Borings: R1, CH1 and CH2.

4. Two different weighting schemes were considered. One of them is based on the quality of each individual test (borehole), and the other is based on the applicability of each testing technique.

5. According to the quality of the data, field observations and engineering judgment, a first set of weighting coefficients for shear-wave velocities were defined for each layer and for each boring. The same criterion was applied to compressional wave velocities. Using the stratigraphy defined in Step 1, each base case was defined as the average of all individual values of body wave velocities that corresponds to the same formation, according to the information given in the geophysical surveys and in the boring logs.

6. A second set of weighting coefficients for shear-wave velocities were defined for each layer and for each testing technique.

7. The best estimate of shear-wave velocities for a given layer was calculated as the weighted average of the shear-wave velocities from the three base cases. The same logic applies to the compression wave velocities. Table 2.5.4-29 and Figure 2.5.4-33

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shows the best estimate shear and compression wave velocity profiles, together with the three base cases for downhole, suspension logging and crosshole measurements.

8. The standard deviation of body wave velocities was calculated using the individual surveys from the six borings: R1, CH1, CH2, R1B, R25 and R28 and the three field measurement techniques: downhole, suspension logging and crosshole. Using the stratigraphy defined in Step 2, the standard deviation (sigma) of the shear-wave velocity for every layer was determined using the individual values of shear-wave velocities between two consecutive layer boundaries. The coefficient of variation of Vs is about 0.2 for shallow layers, decreasing to less than 0.1 for the deep unweathered rock.

Structural Fill and Backfill Parameters

For the Callaway Nuclear Power Plant Site, the in-situ soils above the Graydon Chert Conglomerate have shear-wave velocities between 600 ft/s and 700 ft/s (see Table 2.5.4-30), which do not qualify as competent material as defined by RG-1.208. Therefore, these layers will be excavated and replaced by Category I Structural Fill and Backfill, characterized by granular soil compacted to a minimum wet unit weight and shear-wave velocity, as specified in section 2.5.4.5.

Compliant base

RG-1.208 defines "Hard Rock" as materials with a shear-wave velocity of 9,200 fps (2,800 m/s) or higher. Site amplification models need to include the soil and soft rock materials down to a rock formation with a minimum shear-wave velocity of 9,200 fps (2,800 m/s). The Callaway Plant Unit 2 Subsurface Exploration Program included deep borings of 350 ft (107 m). Although some individual surveys reported shear-wave velocities higher than 9,200 ft (2,800 m/s) for the Callaway formation, the shear-wave velocities of the underlying Cotter-Jefferson City formation are of the order of 8,300 ft/s (2,530 m/s) down to the deepest portion of the survey. Section 2.5.2.5.1 provides the methodology developed to establish the depth of the Hard Rock Horizon or base of amplification model.

Strain Dependant and Linear Properties

Resonant Column, Torsional Shear, Combined Resonant Column Torsional Shear, and Unconfined Resonant Column ("Free-Free") Laboratory Tests were performed on soil and rock samples. The complete set of results from these tests is reported in Section 2.5.4.2. To account for variations in shear-wave velocity across the site, 60 artificial profiles were generated. The procedures and methodologies to incorporate uncertainties of strain dependant properties are described in Section 2.5.2.5.2.

2.5.4.7.5 Amplification Functions

A site amplification analysis is performed to obtain the ground motion response parameters. Section 2.5.2.5.3 provides the amplification functions.

2.5.4.7.6 Acceleration Time History for Soil-Structure Interaction Analysis

A spectrum-compatible acceleration-time history was developed for use with the velocity profile described in Section 2.5.4.7.4. This acceleration-time history was chosen based on the probabilistic seismic hazard de-aggregation information described in Section 2.5.2.

The development of the single horizontal component spectrum-compatible time history is based on the mean 1E-4 uniform hazard target spectrum described in Section 2.5.2. The spectrum compatible time history was developed for the frequency range of 100 Hz to 0.5 Hz.

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Using the site-specific soil column extended to the ground surface including the amplification factor, and the performance-based hazard methodology utilized to develop the SSE (refer to Sections 2.5.2.5 and 2.5.2.6), a Ground Motion Response Spectra (GMRS) peak ground acceleration at the top of the Graydon Chert Conglomerate was computed. These parameters apply to analysis of interaction of soils with structures. For reconciliation of site specific design parameters affecting the SSE analysis results, refer to Sections 3.7.1 and 3.7.2.}

2.5.4.8 Liquefaction Potential

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.8:

> The COL applicant will address site-specific liquefaction potential. As stated in Section 2.5.2, the evaluation of liquefaction is performed for the seismic level of the site specific SSE.

This COL Item is addressed as follows:

{The potential for soil liquefaction at the Callaway Plant Unit 2 site was evaluated following NRC Regulatory Guide 1.198 (NRC, 2003c). The soil properties and profiles utilized are those described in Section 2.5.4.2.

Sections 2.5.4.8.1 through 2.5.4.8.3 are added as a supplement to the U.S. EPR FSAR

2.5.4.8.1 Previous Liquefaction Studies

The liquefaction analyses for the Callaway Plant Unit 1 plant was of structural fill compacted to 95 percent of the maximum dry density beneath the Category I structures. The previous evaluation was based on a maximum horizontal ground surface acceleration of 0.20g during a Safe Shutdown Earthquake (SSE) for the site. Results from this liquefaction study indicate that compaction of the structural fill to 85 percent relative density will prohibit the occurrence of liquefaction. No other liquefaction analyses were conducted for Unit 1 because all other Category I structures were founded on the Graydon Chert Conglomerate, and the Category I ESWS retention pond was excavated into the in-situ cohesive soils and the Category I cohesive fill. The Graydon Chert Conglomerate and the cohesive materials are not susceptible to liquefaction when subjected to earthquake loading.

A new evaluation of liquefaction potential is performed for Callaway Plant Unit 2.

2.5.4.8.2 Regulatory Guide 1.198

Under "Screening Techniques for Evaluation of Liquefaction Potential," NRC Regulatory Guide 1.198 (NRC, 2003c) lists the most commonly observed liquefiable soils as fluvial-alluvial deposits, eolian sands and silts, beach sands, reclaimed land, and uncompacted hydraulic fills. The liquefaction evaluation included all soils at the Callaway Nuclear Power Plant Site. NRC Regulatory Guide 1.198 (NRC, 2003c) indicates that clay to silt, silty clay to clayey sand, or silty gravel to clayey gravel soils can be considered potentially liquefiable. The geology at the Callaway Nuclear Power Plant Site includes fluvial overburden soils that consist of wind deposited and glacial transported clays. These soils will not be used for foundation purposes and are classified as non-liquefiable due to the high amounts of fines and plasticity levels (Section 2.5.4.8.3). In addition, Regulatory Guide 1.198 (NRC, 2003c) confirms that potentially liquefiable soils that are currently above the groundwater table, are above the historic high groundwater table, and cannot reasonably be expected to become saturated, pose no potential liquefaction hazard. The liquefaction analysis is performed assuming that the overburden soils and engineered fills are below the water level. However, it is postulated that the normal groundwater level at the site is at El. 810, which is below the overburden soils and

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below the future location of structural fill. Groundwater levels at the site are not expected to rise in the future given the relief and topography of the site, promoting drainage.

NRC Regulatory Guide 1.198 (NRC, 2003c) indicates that if the geologic site evaluation indicates the presence of potentially liquefiable soils, the resistance of these soils to liquefaction or significant strength loss to cyclic pore pressure generation should be evaluated. The liquefaction evaluation (Section 2.5.4.8.3) indicates that the soils at the Callaway Plant Unit 2 site are not susceptible to liquefaction.

2.5.4.8.3 Callaway Plant Unit 2 Liquefaction Analysis

The in-situ, overburden soils (Modified Loess, Accretion Gley, and Glacial Till) will not be directly used for the foundation of Safety Related Facilities. A liquefaction potential evaluation of the overburden materials is performed since these soils are adjacent to the plant facility.

Assessments of liquefaction for the Callaway Plant Unit 2 site were based on observations and conclusions contained within Callaway Plant Unit 1 FSAR (AmerenUE, 2004). Structural fill compacted to at least 85 percent relative density, Graydon Chert Conglomerate and cohesive materials did not possess the potential to liquefy. Like Unit 1, the Category I Nuclear Island structures for Callaway Plant Unit 2 are expected to be constructed on the Graydon Chert Conglomerate. Shallow foundations for other Category I and non-Category I Power Block structures are to be founded on structural fill over Graydon Chert Conglomerate.

2.5.4.8.3.1 In-Situ Soils

Based on the information obtained during the investigations of the underlying soil encountered at the Callaway Plant Unit 2 site, an evaluation of the soil liquefaction potential was performed using the screening techniques proposed in the Regulatory Guide 1.198 (NRC, 2003c).

A total of 243 samples of the 70 borings at different depths, from 1.5 ft to 52 ft, were used in the evaluation. These borings include samples for the first four subsurface layers of the site: Loess, Accretion-Gley, Glacial Till and Graydon Chert conglomerate.

The guidelines proposed by Seed (Boulanger, 2004) were followed in the evaluation, wherein the potential for liquefaction of soils with significant fines content is described in three zones on the Atterberg Limits Chart: Zone A, with soils that are considered potentially susceptible to "classic cyclic induced liquefaction" if the water content is greater than 80 percent of the Liquid Limit (LL); Zone B, with soils that are considered susceptible to liquefaction with detailed laboratory testing recommended if water content is greater than 85 percent of the LL; and Zone C, with soils (outside Zones A and B) that are considered not susceptible to classic cyclic liquefaction (Figure 2.5.4-42).

Cohesive soils with fines content (FC) greater than 30 percent and fines that have a Plasticity Index (PI) greater than 30 percent, are included in Zone C and therefore not be considered susceptible to liquefaction. Most of the samples from the Callaway Plant Unit 2 geotechnical investigation are within Zone C and therefore not susceptible to liquefaction. Some of the samples correspond to Zones A or B of the Atterberg Chart as shown in Figure 2.5.4-34. The samples have water contents that are less than 80% of the Liquid Limit and therefore not susceptible to liquefaction.

Only one sample from Boring R-23 of the Glacial Till, located in Zone A, was the only case in which the water content was higher than the 80 percent of the LL.

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The borings located nearby boring R-23 present higher values of Plasticity Index (PI) and LL showing that this isolated case at Boring R-23 is not consistent with the conclusions obtained from the soil above, below, and around.

Based on the screening techniques for evaluating liquefaction potential used in the boring samples of the underlying soil encountered at the Callaway Plant Unit 2 Site, even if there is an unlikely surge of the groundwater elevation, there is no evidence of existence of potentially liquefiable soils.

2.5.4.8.3.2 Structural Fill and backfill

Section 2.5.4.5 describes material specifications and compaction for structural fill and backfill. The term "fill" is used for material placed beneath foundation mats and the term "backfill" for material placed around structures and against foundation walls. For foundation support backfill, compaction will be done to 90 percent of Modified Proctor optimum dry density. For structural fill, a 95 percent Modified Proctor level will be set. The fill will be compacted to within 3 percent of its optimum moisture content.}

2.5.4.9 Earthquake Site Characteristics

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.9:

> Site-specific earthquake site characteristics will be described by the COL applicant.

This COL Item is addressed as follows:

{Section 2.5.2.6 describes the development of the ground motion for the Callaway Plant Unit 2 site. The selected ground motion is based on the risk-consistent/performance-based approach of NRC Regulatory Guide 1.208, "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion" with reference to NUREG/CR-6728 and ASCE/SEI 43-05. Any deviation from the guidance provided in Regulatory Guide 1.208 is discussed in Section 2.5.2. Horizontal ground motion amplification factors are developed in Section 2.5.2.5 using site-specific data and estimates of near-surface soil and rock properties presented in Sections 2.5.4.2, 2.5.4.4 and 2.5.4.7. These amplification factors are then used to scale the hard rock spectra, presented in Section 2.5.2.4, to develop Uniform Hazard Spectra (UHS), accounting for site-specific conditions using Approach 2B of NUREG/CR-6769. Horizontal SSE spectra are developed from these soil UHS, using the performance-based approach of ASCE/SEI 43-05, accepted by Regulatory Guide 1.208. The Ground Motion Response Spectra (GMRS) is defined at the free ground surface of a hypothetical outcrop at the base of the foundation. Section 2.5.2.6 also describes vertical ground motion, which was developed by scaling the horizontal spectrum by a frequency-dependent vertical-to-horizontal (V: H) factor.}

2.5.4.10 Static Stability

{The area of planned Callaway Plant Unit 2 is graded to establish the final site elevation, which is to be at elevation 845 ft msl (258 m msl) at the center of the reactor building unit. The Reactor, Safeguard, and Fuel Buildings are seismic Category I structures and are supported on a common basemat. The common basemat has an irregular shape, estimated to be approximately 81,000 sq ft, (7530 m^2) in plan. All Category I structures' size and depth ranges are summarized in Table 2.5.4-32.

Structure locations and designations are shown in Figure 2.5.4-2. Other major structures in the power block area are the Nuclear Auxiliary Building, RadWaste Building, and the Turbine Building, which are non-Category I structures.

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Construction of the Nuclear Island Basemat requires an excavation of about 41 ft (from approximately elevation 845 ft). The resulting rebound (heave) in the ground due to the removal of the soils is expected to primarily take place in Graydon Chert Conglomerate and it will be minor. A rebound of less than 2 inches (5 cm) is estimated due to excavation for the Nuclear Island Basemat, and is expected to take place concurrent with the excavation. Ground rebound should be monitored during excavation. The heave estimate was made based on the elastic properties of the Graydon Chert Conglomerate soils and the response to the unloading of the ground by about 41 ft of excavation. The magnitude and rate of ground heave is a function of, among other factors, excavation speed and duration that the excavation remains open. Other factors remaining unchanged, shorter durations culminate in smaller values of ground heave. The excavation shall remain open for a period sufficiently long such that ground heave fully develops.}

2.5.4.10.1 Bearing Capacity

The U.S. EPR FSAR includes the following COL item in Section 2.5.4.10.1:

> A COL applicant that references the U.S. EPR design certification will verify that site-specific foundation soils beneath the foundation basemats of Seismic Category I structures have the capacity to support the bearing pressure with a factor of safety of 3.0 under static conditions.

This COL item is addressed in the following section:

{Sections 2.5.4.10.1.1 is added as a supplement to the U.S. EPR FSAR.

2.5.4.10.1.1 Bearing Capacity

The bearing capacity of the subsurface materials depends on the (1) properties of the foundation soils or rocks, including dimensions of bearing strata and geotechnical strength parameters, (2) on the geometry of the building foundations, (3) on the foundation depth, and (4) on the position of the water table, in case drained conditions are assumed for the calculations. Geotechnical properties and soil profiles are detailed in Section 2.5.4.2. The foundation depth and building geometry are provided in Section 2.5.4.3, and particularly in Table 2.5.4-32. The bearing capacity estimates for the Graydon Chert Conglomerate are determined with the use of undrained shear strength. Other facilities are placed directly on top of structural engineered fill. In this case, the bearing capacity estimate is determined using drained conditions and the high water level as shown by the interface figures (Figure 2.5.4-37 through Figure 2.5.4-40).

The Nuclear Island (NI) Common Basemat Structure is designed for an average static soil bearing pressure of 14,500 pounds per square foot and a maximum static bearing pressure of 22,000 pounds per square foot. Accordingly, Seismic Category I foundations are sized and reinforced to accommodate these bearing pressure values.

The bearing capacity for the NI is estimated assuming that the foundation stratum is a homogenous layer with the properties of the Graydon Chert Conglomerate. The backfill unit weight of 140 pcf (22.0 kN/m^3) is used to calculate the embedment contribution to a depth of 41.0 ft (12.5 m). The position of the water table is placed 1 ft (0.3 m) above the foundation contact.

Other Seismic Category I facilities have their foundation base resting on structural fill. A friction angle of 35o is assumed with no cohesion and the groundwater level is taken at 17 ft (5.2 m) below surface elevation, which corresponds to the maximum groundwater level. Estimated bearing capacities using these parameters yield much higher values than the resistance

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provided by the Graydon Chert Conglomerate and therefore a recommendation is capped by the capacity offered by such formation.

The ultimate (gross) bearing capacity is estimated by the Terzaghi theory using the Vesic capacity factors (Bowles, 1996).

Eq. 2.5.4-12

$$q_{ult} = cN_c s_c + \gamma' D_f N_q s_q + \frac{1}{2}\gamma B N_\gamma s_\gamma$$

In Eq. 2.5.4-12:

c	→	Undrained shear strength of the Graydon Chert Conglomerate,
γ'	→	Effective unit weight of soil,
D_f	→	Depth to calculate effective overburden pressure at base of foundation,
B	→	Width of foundation,
N_c, N_q, N_γ	→	Bearing capacity factors (defined in Vesic, 1975)
s_c, s_q, s_γ	→	Shape factors (defined in Vesic, 1975).

For the case of NI and the bearing capacity of the Graydon Chert Conglomerate, Eq. 2.5.4-12 reduces to Eq. 2.5.4-13.

Eq. 2.5.4-13

$$q_{ult} = cN_c s_c + \gamma' D_f N_q s_q$$

For the case of other Safety Class Structures founded on top of the structural fill, Equation 2.5.4-12 reduces to Eq. 2.5.4-14.

Eq. 2.5.4-14

$$q_{ult} = \gamma' D_f N_q s_q + \frac{1}{2}\gamma B N_\gamma s_\gamma$$

The subsurface conditions and material properties were described in Section 2.5.4.2. Material properties, conservatively designated for the various strata, were used for foundation evaluation, as shown by the recommended strength parameters shown on Table 2.5.4-25. A summary of the estimated allowable bearing pressures are presented in Table 2.5.4-34. A factor of safety of 3.0 was applied to obtain the allowable values.

Design values of foundation pressures for the Category I structures were estimated based on project knowledge and the specifications provided by the Design Control Document for the EPR Nuclear Power Plant Facilities. The recommended allowable bearing pressures meet the Design Control Document specifications and a Factor of Safety of 3.0 is used in the calculation. The site-specific foundation soils beneath the NI basemat and other safety class facilities have been verified to have the capacity to support the bearing pressures with a factor of safety of 3.0 under static conditions and a Factor of Safety of 2.0 under dynamic conditions.

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2.5.4.10.2 Settlement

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.10.2:

> A COL applicant that references the U.S. EPR design certification will verify that the differential settlement value of ½ inch per 50 ft in any direction across the foundation basemat of a Seismic Category I structure is not exceeded. Settlement values larger than this may be demonstrated acceptable by performing additional site specific evaluations.

This COL Item is addressed in the following section:

{The safety-related Category I facilities at the Callaway Plant Unit 2 Site will bear either on top of the Graydon Chert Conglomerate or on top of structural fill which in turn bears directly on top of the Graydon Chert Conglomerate. The overburden clay deposits will not be used for foundation purposes as these have an inherent risk for long term settlements due to consolidation. Elastic short term settlements will occur in either the Graydon Chert Conglomerate or the structural fill.

2.5.4.10.2.1 Settlement Estimation by Simplified Methods

Three of the ASCE recommended methods of analysis were used to estimate settlements:

Improved Janbu Approximation – Provides an average estimate of the settlement beneath the foundation. Settlement is estimated by Eq. 2.5.4-15.

Eq. 2.5.4-15

$$\rho_i = \mu_o \cdot \mu_1 \cdot \frac{q \cdot B}{E_s^*}$$

Where:

ρ_i	→	Settlement,
μ_o	→	Embedment adjustment coefficient,
μ_o	→	Shape Adjustment Coefficient,
q	→	Service pressure load,
B	→	Foundation width,
E_s*	→	Equivalent Young's Modulus.

Perloff Approximation – Provides settlement at both center and edge of the foundation. This methodology is useful to analyze differential displacements. Settlement is estimated by Eq. 2.5.4-16.

Eq. 2.5.4-16

$$\rho_i = I \cdot q \cdot B \left(\frac{1 - \nu^2}{E_s} \right)$$

Where:

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ρ_i → Settlement,

I → Stress influence factor at either depth or edge of foundation

q → Service pressure load,

B → Foundation width,

ν → Poisson Ratio,

E_s → Young's Modulus.

Kay and Cavagnaro Approach – This approach can adapt to layers of variable elastic properties, and provides settlements at both center and edge of the foundation. It is based on same elastic principles, as it is shown by Eq. 2.5.4-16.

Settlement estimates utilizing the noted simplified methods have been calculated for each of the facilities listed in Table 2.5.4-35. The adopted elastic modulus is the lesser of the one recommended for the Graydon Chert Conglomerate and the one recommended for the engineered fill. This recommendation is provided by Table 2.5.4-27. The estimates assume that displacements will end before reaching the deeper Burlington, Bushberg, or Snyder formations.

Table 2.5.4-35 provides the settlement estimates at the Callaway Plant Unit 2 site, obtained from the three noted methodologies. Overall, the Jambu approach produces the lower estimates and the Perloff approximation the most conservative. The load used for the calculation is a U.S. EPR FSAR recommended service pressure of 14 ksf (670 kPa). On average the immediate settlement will be approximately two inches (5 cm). This estimate is considered conservative, given the relatively low elastic modulus of 2000 ksf (100 MPa) assigned to the Graydon Chert Conglomerate. A more detailed analysis follows.

2.5.4.10.2.2 Settlement Estimates for the NI from Detailed Analysis

A detailed analysis was used to evaluate the foundation mat displacements and rotations for the NI Buildings (Figure 2.5.4-44) due to potential elastic settlements of the subgrade soils at the Callaway Plant site. The loads assigned are different for each area of the NI, according to the specifications described in the U.S. EPR FSAR. The calculations are performed assuming a 40 ft (12.2 m) compressible layer resting on top of a rigid base, resembling the setting of the Graydon Chert Conglomerate and building foundation. Settlements are expected to occur within the Graydon Chert Conglomerate. The soil profile considers a single 40-ft Graydon Chert Conglomerate layer. The methodology utilized here quantifies the settlements, foundation mat displacements and rotation associated with the proposed site specific subsurface profile.

The first part of the evaluation estimates elastic settlements due to the total construction loads of the Nuclear Island Buildings: Reactor (UJA), Fuel Building (UFA) and four Safeguard Buildings (UJH). The 40-ft height soil column weight that was replaced with engineering backfill is not considered so the results are conservative.

Settlements that are reported correspond to the top of the soft rock layer (interface between the foundation mat and the sublaying soft rock). In the second part of the evaluation, the surface settlements are used to compute equivalent Winkler springs representing the deformation characteristics of the subsurface.

The Winkler springs are incorporated into a three-dimensional structural finite element analysis which assumes a rigid foundation mat for each of the buildings included in the analysis. This

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analysis uses applied loads on the foundation and calculates the foundation mat deformations. Because the settlements estimated in the first step and, consequently, the equivalent springs vary over the foundation area (soft springs at locations of large settlements and stiff springs at locations of small settlements), the structural analysis redistributes the applied loads on the foundation mat consistent with the stiffness of the mat relative to the subsurface. For example, the elastic settlements are larger near the centers of the loaded disks. Consequently, the soil springs near the center are softer than those at locations closer to the outer edge of the mat, assuming that the mat carries uniformly distributed load. Consistent with this distribution, the structural analysis redistributes the applied loads away from the center of the disks.

In an iterative procedure, the bearing pressures resulting from the structural analysis are used to recalculate the elastic settlements and the corresponding soil springs, and the structural analysis is repeated until a satisfactory agreement is obtained for average settlements.

Surface loads are applied as uniformly loaded flexible disks and the resulting stresses in the soil medium are calculated using an axisymmetric analysis. At-depth stresses are calculated for each of the profile points where settlement is desired due to each of the loaded disks, consistent with the locations of the profile points relative to the center of the disks. The stresses from each of the disks are then superimposed to compute the stresses due to the entire loaded foundation. The at-depth stresses are updated each time the cumulative load is specified.

The analysis reported herein uses a three dimensional finite element model representing the soil-structure system. The model considers a rigid plate element for the Nuclear Island base mat. The foundation medium is represented by supports with stiffness of the Winkler Soil Springs. Calculations are performed using computer codes specifically designed to calculate soil settlements (Dapset), and structural response (SAP).

Immediate elastic settlements resulting from the applied loads at different locations across the Nuclear Island are presented in Figure 2.5.4-45 and Figure 2.5.4-46.

As a result of a different load distribution among the buildings of the Nuclear Island (the Fuel Building has a bigger load than the Safeguard Buildings 2 and 3) the north-south direction of the foundation mat presents a tilt of about 0.81 inches (2 cm) in a total length of approximately 350 feet. No tilt was observed on the east-west direction since the applied loads are practically symmetrical.

2.5.4.10.2.3 Settlement Analysis Conclusions

Displacements will be immediate during and after construction of civil works. Differential settlements estimated using simplified methodologies, opposed to the detailed analyses as described in the previous section, are less than two inches. These have an additional degree of conservatism since they do not account for the stiffness of the foundation mat.

Settlements will take place concurrent with construction and these will have taken place prior to placing the equipment, piping, and the final finishes. Hence, post-construction total and differential settlements are expected to be lower than the values noted herein, particularly after accounting for foundation mat rigidity.

The results from the detailed analysis are reported in Table 2.5.4-36. This table provides the best estimate for settlement at the NI and other Safety Related Facilities.

Settlements are within tolerable thresholds and all foundations will be able to safely tolerate the anticipated total and differential settlements. Additionally, engineering measures are incorporated into design for control of differential movements between adjacent structures,

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piping, and appurtenances sensitive to movement, consistent with settlement estimates. This includes the development and implementation of a monitoring plan that supplies and requires evaluation of information throughout construction and post-construction on ground heave, settlement, pore water pressure, foundation pressure, building tilt, and other necessary data. This information provides a basis for comparison with design conditions and for projections of future performance.

Detailed analysis shows good agreement with the simplified methods. The settlements for the Power Block Category I structures (i.e. ESW Buildings, Emergency Power Generating Buildings) are within acceptable thresholds.

Analysis indicates favorable conditions for total and differential settling. In order to monitor and verify settling, Callaway Plant Unit 2 major structure foundations will be monitored for any settling movement after construction.}

2.5.4.10.3 Uniformity and Variability of Foundation Support Media

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.10.3:

> A COL applicant that references the U.S. EPR design certification will investigate and determine the uniformity of the underlying layers of site specific soil conditions beneath the foundation basemats. The classification of uniformity or non-uniformity will be established by a geotechnical engineer.

These COL Item is addressed as follows:

{Three criteria are identified in the U.S. EPR FSAR for establishing uniformity in foundation support media, namely, 1) presence of soil and rock, 2) dip angle of soil and rock layers, and 3) shear wave velocity. Each is addressed below:

1. Foundations of Seismic Category I structures, mainly the Nuclear Island, at the Callaway Plant Unit 2 site are supported on the Graydon Chert Conglomerate. The remaining Category I structures are supported on compacted structural fill which is in turn supported on the Graydon Chert Conglomerate. These structures are illustrated on cross sections in Figure 2.5L-10. Bedrock at the site, the Graydon Chert Conglomerate, is encountered, on average, at 40 ft below ground surface. The horizontally bedded overburden along with horizontal contact of the Graydon Chert Conglomerate with the overburden provides uniform foundation conditions resulting from combined soil-rock support under the Category I structures at the Callaway Plant Unit 2 site.

2. The Callaway Plant Unit 2 site is situated on a level plateau. Stratigraphic profiles (Presented in Figure 2.5L-10 of the referenced section) indicate that the stratigraphic lines delineating various soil units are slightly irregular due to erosional surfaces. However, the contact between the overburden soils and the Graydon Chert Conglomerate is nearly horizontal in all borings. The basal contact of the Graydon Chert Conglomerate and the underlying bedrock layers, mainly the Burlington and Bushberg Formations, which are not present at all locations at the site, create a undulating and irregular surface, due to differential erosion. Regardless, these steeper angles are less than the dip angle of 20 degrees from the horizontal identified in the U.S. EPR FSAR as the criterion for determining levelness of layers. On this basis, the soil and rock layers at the Callaway Plant Unit 2 site are considered horizontal.

3. Classification of uniformity (or non-uniformity) in foundation support media resides with the geotechnical engineer, per the U.S. EPR FSAR. Shear wave velocity (Vs)

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measurements are used for this determination because they are a) in-situ measurements reflecting the natural ground conditions and b) important input to the safety evaluation of structures such as in soil-structure interaction and seismic analyses. The Vs values were evaluated to a depth of 320 ft below the Nuclear Island (NI) foundation basemat, corresponding to El. 500 ft. The 320 ft value was selected based on the three U.S. EPR FSAR criteria of: 1) 1.5 times an equivalent radius of foundation basemat, 2) 1.0 times the maximum foundation basemat dimension, or 3) no less than 200 ft below the bottom of the foundation basemat.

Detailed Vs data are discussed in detail in Section 2.5.4.2.2.2.3. An evaluation of Vs values to 320 ft below the NI foundation basemat (from El. 820 ft. to El. 500 ft.) is located in Table 2.5.4-29.

From the values calculated in Table 2.5.4-29, the lowest standard deviation and coefficient of variation (therefore, the lowest variability- or the highest uniformity) is noted for the Graydon Chert Conglomerate with a coefficient of variation of 0.15.

Larger variations are noted in the Burlington and Bushberg Formations. These rock layers are very thin and not present at all locations.

These naturally-occurring variations were confirmed from the site investigation results via three different measurement techniques for shear wave velocity (by suspension P-S velocity logging, downhole seismic velocity surveys, and crosshole seismic velocity surveys), indicating similar velocity profiles, as shown in Figure 2.5.4-16 through Figure 2.5.4-21. These variations have been accounted for in developing the site-specific horizontal and vertical ground motion response spectra (GMRS) at the locations shown in Figure 2.5.4-37 through Figure 2.5.4-40. The GMRS are defined at a depth of 36 ft which is the foundation level for the U.S EPR Nuclear Island (NI). Therefore, the GMRS coincide with the foundation input response spectra (FIRS) for the NI. Since the GMRS (FIRS for the NI) are enveloped by the certified seismic design response spectra (CSDRS) for the NI, the noted natural variations in Vs, minor as they are relative to the overall 320 ft soil and rock column, are accounted for in the development of the site-specific input motion.}

2.5.4.10.3.1 Earth Pressures

Static and seismic lateral earth pressures are addressed for plant below-ground walls. Seismic earth pressure diagrams are structure-specific and are, therefore, only addressed generically herein. Specific earth pressure diagrams are developed for specific structures based upon each structure's final configuration. Passive earth pressures are not addressed; they are ignored for conservatism for general purpose applications. The following soil properties were assumed for the backfill; an angle of shearing resistance of 35 degrees and a total unit weight of 140 pcf (22.4 Kn/m^3). Structural backfill material is verified to meet the design requirements prior to use during construction.

2.5.4.10.3.2 Static Lateral Earth Pressures

The static active earth pressure, pAS, is estimated using the following expression:

Eq. 2.5.4-17

$$p = K \cdot \gamma \cdot z$$

Where:

p → Active (p_a), Passive(p_p), or At Rest (p_o) Pressure

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K→ Active (K_a), Passive (K_p), or At Rest (K_o) Pressure Coefficient as defined by Section 2.5.4.5.6.

γ → Unit weight of backfill

z → Depth below ground surface

2.5.4.10.3.3 Dynamic Earth Pressures

The following symbols apply:

P_{ae} → Dynamic active earth force,

P_{pe} → Dynamic passive earth force,

K_{ae} → Dynamic active earth pressure coefficient,

K_{pe} → Dynamic passive earth pressure coefficient,

k_h → Horizontal earthquake acceleration at El. 823 (0.30 g at Callaway)

k_v → Vertical earthquake acceleration (0.32 g at Callaway),

γ_{sat} → Saturated Unit Weight,

γ' → Effective Unit Weight,

F_r → Resultant force associated with dynamic soil pressure distribution,

M_r → Resultant overturning moment about base of retaining structure,

H → Embedment Height,

α_h → Horizontal earthquake acceleration (g),

ν → Poisson's ratio,

C_v,D_v → Empirical Coefficients as a function of Poisson's ratio.

The active/passive earth force on a wall (cohesionless and dry backfill) is estimated as follows:

Eq. 2.5.4-18

$$P_{ae} = \frac{1}{2}\gamma K_{ae} H^2 \left(1 - k_v\right)$$
$$P_{pe} = \frac{1}{2}\gamma K_{pe} H^2 \left(1 - k_v\right)$$

The total active/passive thrust, P_{ae}, can be divided into a static component, P_a, and a dynamic component, ΔP_{ae}:

Eq. 2.5.4-19

$$P_{ae} = P_a + \Delta P_{ae}$$
$$P_{pe} = P_a + \Delta P_{pe}$$

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The static component is known to act at H/3 above the base of the wall. Seed and Whitman (1970) recommended that the dynamic component be taken to act at approximately 0.6H. On this basis, the total active thrust will act at a height given by:

Eq. 2.5.4-20

$$h = \frac{P_a H/3 + \Delta P_{ae}(0.6H)}{P_{ae}}$$

When there is no significant structure-structure interaction, a conservative estimate of dynamic soil pressures may be obtained from a parabolic distribution with a maximum value at a height of 60 to 70 percent of the total fill height (ASCE 4-98, 1998). The corresponding resultant force and overturning moment may be obtained by Eq. 2.5.4-21.

Eq. 2.5.4-21

$$F_r = \alpha_h C_v \gamma H^2$$
$$Mr = \alpha_h D_v \gamma H^3$$

2.5.4.10.3.4 Sample Earth Pressure Diagrams

Using the relationship outlined above and assumed backfill properties, sample earth pressures were estimated. Sample earth pressure diagrams are provided in Figure 2.5.4-47 for a wall height of 40 ft (12.2 m), level ground surface, and, for conservative purposes, with groundwater level at 17 ft (5.2 m) below the surface. The backfill is taken as granular soils, with an angle of friction of 35 degrees and a unit weight of140 pcf (22.4 Kn/m^3). The horizontal ground acceleration is taken as 0.30g, which is the peak ground Acceleration at El. 823. The validity of assumptions regarding surcharge loads, structural fill and backfill properties, and structural configurations is confirmed during the detailed design stage. Actual earth pressure evaluations are performed at that time for the design of below-grade walls, based on actual project conditions. Figure 2.5.4-47 provides an example and it is up to the designer to adjust the calculation assumption accordingly.

2.5.4.10.3.5 Selected Design Parameters

The field and laboratory test results are discussed in Section 2.5.4.2. The parameters employed for the bearing capacity, settlement, and earth pressure evaluations are based on the material characterization addressed in Section 2.5.4.2. Normal Groundwater Elevation is present just above the top Surface of the Graydon Chert Conglomerate. The Maximum Groundwater Elevation is shown at El. 828, 17 ft (5.2 m) below grade by the Interface Cross Section Figures at El. 845. A value of 3.0 is commonly used as the factor of safety when determining the bearing capacity of soils. An angle of shearing resistance of 35 degrees was used for characterization of a structural fill and backfill for earth pressure evaluations.}

2.5.4.10.4 Site Investigation for Uniform Sites

No departures or supplements.

2.5.4.10.5 Site Investigation for Non-uniform Sites

No departures or supplements.

2.5.4.11 Design Criteria

{Section 3.8.5 provides criteria, references, and design methods used in static and seismic analysis and design of foundations, including an explanation of computer programs used in the analyses and a description of soil loads on subsurface facilities.}

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2.5.4.12 Techniques to Improve Subsurface Conditions

The U.S. EPR FSAR includes the following COL Items in Section 2.5.4.12:

> Techniques used for improving subsurface conditions are site specific and will be addressed by the COL applicant.

This COL Item is addressed as follows:

{Major structures will derive support from the Graydon Chert Conglomerate. Ground improvement will be limited to excavation of unsuitable soils, such as suspected fill or loose/soft soils, and their replacement with structural fill. It will also include proof-rolling of foundation subgrade for the purpose of identifying any unsuitable soils for further excavation and replacement, which further densifies the upper portions of the subgrade. In absence of subsurface conditions at the site that require ground improvement, ground control, that is, maintaining the integrity of existing dense or stiff foundation soils, will be the primary focus of earthworks during foundation preparation. These measures will include such steps as groundwater control, use of appropriate measures and equipment for excavation and compaction, subgrade protection, and other similar measures.

2.5.4.13 References

This section is added as a supplement to the U.S. EPR FSAR.

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Table 2.5.4-1—{Summary of Thicknesses and Termination Elevations for Various Strata (English Units)}

Stratum		Thickness (feet)			Termination Elevation (feet msl)		
		Min	Max	Avg	Min	Max	Avg
Soil	Modified Loess	5	22	9	821	846	838
	Accretion-Gley	0	26	16	812	842	821
	Glacial Till	3	39	11	803	813	810
Rock	Graydon Chert Conglomerate	12	55	32	764	794	780
	Burlington Formation	0	20	6	756	785	772
	Bushberg Formation	0	7	2	750	784	770
	Snyder Creek Formation	22	32	29	736	747	742
	Callaway Formation	31	41	38	698	710	705
	Cotter-Jefferson City Formation	>10	>210	>110	—	—	—

Table 2.5.4-1—{Summary of Thicknesses and Termination Elevations for Various Strata (SI Units)}

Stratum		Thickness (m)			Termination Elevation (m msl)		
		Min	Max	Avg	Min	Max	Avg
Soil	Modified Loess	1.5	6.7	2.7	250.3	257.9	255.5
	Accretion-Gley	0.0	7.9	4.9	247.6	256.7	250.3
	Glacial Till	0.9	11.9	3.4	244.8	247.9	247.0
Rock	Graydon Chert Conglomerate	3.7	16.8	9.8	232.9	242.1	237.8
	Burlington Formation	0.0	6.1	1.8	230.5	239.3	235.4
	Bushberg Formation	0.0	2.1	0.6	228.7	239.0	234.8
	Snyder Creek Formation	6.7	9.8	8.8	224.4	227.7	226.2
	Callaway Formation	9.5	12.5	11.6	212.8	216.5	214.9
	Cotter-Jefferson City Formation	>3.1	>64.0	>33.5	—	—	—

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Table 2.5.4-2—{Summary Of Callaway Plant Unit 2 Geotechnical Field Tests}

Test	Specification	Quantity
Soil and Rock Borings	ASTM D1586/1587	71
Seismic S-wave Reflection Survey	NA	3
P-S Suspension Logging Surveys	NA	5
Downhole Velocity Measurements	NA	5
Crosshole Velocity Measurements	ASTM D4428/4428M	3
Pressuremeter Test (PMT)	ASTM D4719-00	5
SPT Hammer Energy Measurements	ASTM D4633	3

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Table 2.5.4-3—{Boring Locations and Surface Elevations}

(Page 1 of 2)

Structure (Foundation)			Boring	Horizontal Coordinates Plant N	Horizontal Coordinates Plant E	El. [ft]	Boring Depth [ft] Soil	Boring Depth [ft] Rock	Boring Depth [ft] Total	Type of Test
SAFETY RELATED FACILITIES	Reactor Building	UJA	R-1	100,000.00	101,350.00	848.96	36.3	313.7	350.0	Continuous SPT/ Coring
			R-2	100,000.00	101,259.29	849.41	38.0	72.0	110.0	SS 2.5ft – SPT / Coring
			R-3	100,080.24	101,304.34	849.92	38.5	311.5	350.0	Continuous SPT/ Coring
			R-4	100,080.24	101,395.25	849.77	38.0	72.5	110.5	SS 2.5ft - SPT / Coring
			R-5	100,000.00	101,440.50	848.28	36.0	49.0	85.0	SS 2.5ft - SPT / Coring
			R-6	99,923.49	101,395.25	848.60	37.5	78.0	115.5	SS 2.5ft - SPT / Coring
			R-7	99,923.49	101,304.75	848.99	38.0	42.0	80.0	SS 2.5ft - SPT / Coring
	Fuel Pool Building/Vent Stack	UFA/ UKH	R-8	99,854.48	101,287.59	848.01	36.3	53.7	90.0	SS 2.5ft - SPT / Coring
			R-9	99,854.56	101,350.02	847.28	36.0	124.8	160.8	Continuous SPT/ Coring
			R-10	99,824.47	101,415.78	846.11	34.5	75.6	110.1	SS 2.5ft - SPT / Coring
	Reactor Auxiliary Building	UKA	R-11	99,918.16	101,467.70	848.13	36.2	53.7	89.9	SS 2.5ft - SPT / Coring
			R-12	99,855.08	101,509.42	846.98	37.0	64.0	101.0	Continuous SPT/ Coring
	Mechanical/Electrical Safeguard	UJH/ UJK	R-13	99,961.70	101,515.54	847.47	35.1	314.9	350.0	Continuous SPT/ Coring
			R-14	100,038.30	101,516.03	846.87	37.0	58.0	95.0	SS 2.5ft - SPT / Coring
			R-16	100,160.26	101,427.83	850.12	39.5	79.2	118.7	SS 2.5ft - SPT / Coring
			R-17	100,160.26	101,349.96	850.74	40.0	55.0	95.0	SS 2.5ft - SPT / Coring
			R-18	100,158.41	101,279.50	850.15	39.5	56.0	95.5	SS 2.5ft - SPT / Coring
			R-19	100,039.26	101,197.52	850.11	39.5	61.1	100.6	SS 2.5ft - SPT / Coring
			R-20	99,963.94	101,197.52	850.03	39.0	121.0	160.0	Continuous SPT/ Coring
	Radioactive Waste Processing Building	UKS	R-21	99,936.52	101,574.24	847.23	35.5	59.5	95.0	SS 2.5ft - SPT / Coring
			R-22	99,834.30	101,574.24	846.50	34.8	65.8	100.6	SS 2.5ft - SPT / Coring
			R-23	99,838.31	101,678.41	845.79	35.0	50.0	85.0	SS 2.5ft - SPT / Coring
			R-24	99,938.35	101,676.68	844.11	32.2	71.8	104.0	Continuous SPT/ Coring
	Emergency Diesel Generator Building	UBP	R-25	100,108.50	101,077.22	851.53	40.0	120.0	160.0	Continuous SPT/ Coring
			R-26	100,053.94	101,077.20	851.64	42.5	42.3	84.8	SS 2.5ft - SPT / Coring
			R-27	100,053.94	101,622.80	843.83	32.0	58.0	90.0	SS 2.5ft - SPT / Coring
			R-28	100,108.50	101,622.80	843.87	34.5	126.6	161.1	Continuous SPT/ Coring
	Ultimate Heat Sink	URB	R-29	99,868.40	101,157.77	848.65	37.5	37.5	75.0	Continuous SPT/ Coring
			R-30	99,795.55	101,157.76	848.65	37.5	37.5	75.0	Continuous SPT/ Coring
			R-31	99,868.40	101,010.36	847.08	36.5	53.7	90.2	Continuous SPT/ Coring
			R-32	99,805.90	101,010.36	846.26	34.5	61.1	95.6	Continuous SPT/ Coring
			R-33	100,338.75	101,802.91	840.08	28.9	61.6	90.5	Continuous SPT/ Coring
			R-34	100,276.25	101,802.91	838.82	28.0	76.3	104.3	Continuous SPT/ Coring
			R-35	100,338.75	101,655.10	842.30	31.5	58.6	90.1	Continuous SPT/ Coring
			R-36	100,276.25	101,655.10	842.67	31.0	42.3	73.3	Continuous SPT/ Coring

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Table 2.5.4-3—{Boring Locations and Surface Elevations}

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Structure (Foundation)			Boring	Horizontal Coordinates Plant N	Horizontal Coordinates Plant E	El. [ft]	Boring Depth [ft] Soil	Boring Depth [ft] Rock	Boring Depth [ft] Total	Type of Test
OTHER STRUCTURES	Access Building	UKE	R-15	100,126.25	101,516.65	847.43	36.3	53.7	90.0	SS 5ft - SPT / Coring
	Turbine Building	UMA	R-37	R-37 Cancelled due to building interference.						
			R-38	100,223.00	101.421.08	851.39	39.5	49.7	89.2	SS 5ft - SPT / Coring
			R-39	100,457.04	101,421.10	851.26	42.0	58.8	100.8	SS 5ft - SPT / Coring
			R-40	100,457.03	101,305.65	851.24	42.0	36.5	78.5	SS 5ft - SPT / Coring
			R-41	100,312.37	101,369.03	851.36	47.0	58.3	105.3	SS 5ft - SPT / Coring
			R-42	100,401.11	101,366.94	851.39	41.5	48.7	90.2	SS 5ft - SPT / Coring
	Demineralized Water Tanks	UGX	R-43	100,284.45	101,159.08	851.32	42.0	48.0	90.0	SS 5ft - SPT / Coring
	Generator Transformers	UBF	R-44	100,565.22	101,353.73	847.46	37.5	50.5	88.0	SS 5ft - SPT / Coring
	MV Auxiliary Power Transformer	UBE	R-45	100,564.82	101,493.06	846.87	37.5	16.5	54.0	SS 5ft - SPT / Coring
	Switchgear Building	UBA	R-46	100,276.16	101,532.57	850.70	39.9	40.6	80.5	SS 5ft - SPT / Coring
	Central Gas Supply Systems	UTG	R-47	100,277.63	100,999.93	851.29	40.5	69.8	110.3	SS 5ft - SPT / Coring
	Station Blackout Diesel	UXJ	R-48	100,075.06	101,843.18	837.71	28.0	17.6	45.6	SS 5ft - SPT / Coring
	Cold Machine Workshop	UST	R-49	100,423.84	101,236.64	851.39	40.0	55.0	95.0	SS 5ft - SPT / Coring
	Water Treatment	UPQ	R-50	99,609.82	102,101.72	833.16	24.0	56.5	80.5	SS 5ft - SPT / Coring
	Central Gas Supply Systems	USG	R-51	99,679.44	101,833.82	840.53	29.7	50.7	80.4	SS 5ft - SPT / Coring
			R-52	99,661.04	101,350.03	845.03	32.8	40.4	73.2	SS 5ft - SPT / Coring
	ESWEMS Pumphouse	UOF	R-53	100,297.43	102,143.90	842.28	34.0	51.0	85.0	SS 5ft - SPT / Coring
	ESWEMS Retention Pond & Ground Surface		R-54	99,837.21	102,139.13	832.70	24.5	46.0	70.5	SS 5ft - SPT / Coring
			R-55	99,829.43	102,350.91	838.10	30.0	55.0	85.0	SS 5ft - SPT / Coring
			R-56	100,297.43	102,350.91	841.61	32.3	42.5	74.8	SS 5ft - SPT / Coring
			R-57	99,463.06	101,350.04	844.03	32.0	36.7	68.7	SS 5ft - SPT / Coring
			R-58	99,267.87	101,350.62	836.20	25.6	45.5	71.1	SS 5ft - SPT / Coring
	Cooling Tower Structure 1	1-URA	R-59	98,801.08	100,765.80	846.06	36.5	63.5	100.0	SS 5ft - SPT / Coring
			R-60	98,653.16	100,813.76	845.67	37.0	53.3	90.3	SS 5ft - SPT / Coring
			R-61	98,801.04	100,921.30	845.98	36.0	43.5	79.5	SS 5ft - SPT / Coring
			R-62	98,948.92	100,813.99	845.95	36.0	44.0	80.0	SS 5ft - SPT / Coring
			R-63	98,892.44	100,639.97	846.23	37.8	50.7	88.5	SS 5ft - SPT / Coring
			R-64	98,709.67	100,640.00	846.21	38.0	46.5	84.5	SS 5ft - SPT / Coring
	Cooling Tower Structure 2	2-URA	R-65	98,478.11	101,277.52	846.24	38.0	47.0	85.0	SS 5ft - SPT / Coring
			R-66	99,330.19	101,325.36	846.58	41.5	60.2	101.7	SS 5ft - SPT / Coring
			R-67	98,478.07	101,433.01	845.72	38.7	51.3	90.0	SS 5ft - SPT / Coring
			R-68	98,626.00	101,325.57	846.20	38.0	37.0	75.0	SS 5ft - SPT / Coring
			R-69	98,569.47	101,151.73	846.27	38.0	47.0	85.0	SS 5ft - SPT / Coring
			R-70	98,386.76	101,151.73	846.57	41.5	43.6	85.1	SS 5ft - SPT / Coring
	Additional Borings		P-6	100,161.67	101,571.83	845.32	33.6	66.4	100.0	SS 5ft - SPT / Coring
			P-10	100,092.33	101,571.83	845.48	34.0	66.2	100.2	SS 5ft - SPT / Coring

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FSAR: Tables 2.5.4

Table 2.5.4-4—{Summary of Hammer-Rod Energy Measurements}

Drill Rig	Measurement in Boring No.	ETR Range [%]	Average ETR [%]	Energy Adjustment [ETR%/60%]
CME Truck Mount	R-5	78-83	81	1.34
CME 750 ATV	R-55	85-87	86	1.43
CME 55 300 Carrier	R-33	83-95	90	1.50

ETR = Percentage of theoretical hammer energy measured in the field

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Table 2.5.4-5—{Summary of Field-Measured Standard Penetration Test (SPT) N-Values}

Stratum	SPT N-values [blows / feet]		
	Minimum	Maximum	Average
Modified Loess	3	25	10
Accretion-Gley	5	19	10
Glacial Till	3	89	20
Graydon Chert Conglomerate	31	100	57

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Table 2.5.4-6—{Summary of Adjusted SPT N-Values Based on Energy Measurements}

Stratum	Adjusted Minimum N-Value [blows/feet]	Adjusted Maximum N-Value [blows/feet]	Adjusted Average N-Value [blows/feet]	Recommended N-Value for Engineering Purposes [blows/ft]
Modified Loess	4	34	15	10
Accretion-Gley	6	29	14	10
Glacial Till	4	106	23	20
Graydon Chert Conglomerate	44	134	80	60

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Table 2.5.4-7—{Summary of Borehole Pressuremeter Test Results (English Units)}

Boring	Depth [feet]	p_l [ksf]	E_s [ksf]	G [ksf]	s_u [ksf]
R-2B	48.4 - 50.4	514	5520	1,917	138
R-2B	63.4 - 65.4	531	4858	1,687	150
R-5	43.5 - 45.5	141	1148	399	39
R-5	58.5 - 60.5	598	4662	1,619	184

Table 2.5.4-7—{Summary of Borehole Pressuremeter Test Results (SI Units)}

Boring	Depth [feet]	p_l [kPa]	E_s [kPa]	G [kPa]	s_u [kPa]
R-2B	48.4 - 50.4	24610	264300	91,790	6,610
R-2B	63.4 - 65.4	25420	232600	80,770	7,180
R-5	43.5 - 45.5	6750	54970	19,100	1,870
R-5	58.5 - 60.5	28630	223220	77,520	8,810

Note:

p_l - Pressuremeter Limit Pressure

E_s - elastic modulus

G - shear modulus

s_u - Undrained Shear Strength

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Table 2.5.4-8—{Summary of Laboratory Tests}

Test	Specification	Quantity
Engineering Classification	ASTM D2487-06 / ASTM D5878-05	281
Moisture Content	ASTM D2216-05	256
Unit Weight	From weight-volume relationship	38
Specific Gravity	ASTM D854-06 / ASTM D6473-99(2005)	73
Atterberg Limits	ASTM D4318-05	258
Grain Size - Hydrometer	ASTM D422-63(2002)E1	25
Percent Passing #200 Sieve	ASTM D1140-00(2006)	237
Permeability	ASTM D5084-03	4
$C\bar{u}$ Triaxial	ASTM D4767-04	17
Unconfined Compression	ASTM D7012-04	21
Consolidation	ASTM D2435-04	7
Resonant Column Torsional Shear	Technical Procedures for RCTS Tests [1]	7
Free-Free Test	Technical Procedures for URC Tests [2]	6
Organic Content	ASTM D2974-07	9
Isotope Absorption Coefficient (KD)	ASTM D4646-03	45
pH	ASTM G51-95(2005)	15
Resistivity	ASTM G187-05	15
Chloride ion content	AASHTO T291	15
Sulphate ion content	AASHTO T290	15

[1] PBRCTS-1 Rev. 4, October 2004, University of Texas at Austin
[2] URC-1 Rev. 1, August 2004, University of Texas at Austin

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Table 2.5.4-9—{Summary of Unit Weights (Special Care Rock Samples and Undisturbed Samples) (English Units)}

(Page 1 of 2)

Formation	Boring	Depth [ft]	Elevation [ft] msl	Dry Unit Weight [pcf]		Wet Unit Weight [pcf]	
	R-1	12.5	836.5	108	108	129	129
Modified Loess	R-50	13.0	820.2	107		128	
	R-1	19.0	830.0	104	102	125	125
	R-34	11.7	827.1	98		123	
	R-38	18.0	933.4	102		125	
	R-41	19.0	832.4	103		125	
	R-42	18.0	833.4	104		127	
Accretion-Gley	R-56	13.5	828.1	102		125	
	R-1	27.0	822.0	109	111	129	131
	R-1	33.5	815.5	110		131	
	R-2	35.0	814.0	113		133	
	R-38	33.0	818.4	114		133	
	R-41	35.5	815.9	112		131	
	R-42	38.0	813.4	111		131	
	R-56	24.0	817.6	106		127	
	R-59	20.0	826.1	108		129	
Glacial Till	R-59	33.0	813.1	113		133	
	R-9	46.4	800.9	126	127	143	142
		58.8	788.5	123		139	
	R-17	44.8	805.9	133		146	
		62.2	788.5	131		144	
	R-23	42.3	803.5	127		143	
		60.1	785.7	119		135	
	R-25	48.1	803.4	123		137	
		67.9	783.6	147		161	
	R-31	46.8	800.3	123		138	
		66.3	780.8	116		134	
	R-42	46.9	163	124		142	
		77.2	774.2	122		139	
	R-59	51.0	795.1	136		145	
Graydon Chert Conglomerate		72.3	773.8	129		144	
Burlington	R-12	67.3	779.7	162	147	162	162
Snyder Creek	R-3	99.7	750.2	168	154	168	168
Callaway	R-3	116.2	733.7	166	158	166	166
	R-3	167.4	682.5	153	155	160	162
		212.8	637.1	163		168	
		258.2	591.7	152		158	
Cotter-Jefferson City		346.4	503.5	153		164	

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Table 2.5.4-9—{Summary of Unit Weights (Special Care Rock Samples and Undisturbed Samples) (SI Units)}

(Page 2 of 2)

Formation	Boring	Depth [m]	Elevation [m] msl	Dry Unit Weight [Pa]		Wet Unit Weight [Pa]	
	R-1	3.8	255.0	16890	17005	20170	20270
Modified Loess	R-50	4.0	250.0	17120		20370	
	R-1	5.8	253.0	16650	16138	19730	19713
	R-34	3.6	252.2	15720		19540	
	R-38	5.5	284.6	16020		19640	
	R-41	5.8	253.8	16130		19710	
	R-42	5.5	254.1	16270		19980	
Accretion-Gley	R-56	4.1	252.5	16040		19680	
	R-1	8.2	250.6	17170	17507	20280	20620
	R-1	10.2	248.6	17280		20580	
	R-2	10.7	248.2	17830		20830	
	R-38	10.1	249.5	18110		21030	
	R-41	10.8	248.7	17560		20530	
	R-42	11.6	248.0	18190		21110	
	R-56	7.3	249.3	16700		20000	
	R-59	6.1	251.8	16780		20190	
Glacial Till	R-59	10.1	247.9	17940		21030	
	R-9	14.1	244.2	19760	19959	22450	22319
		17.9	240.4	19250		21880	
	R-17	13.7	245.7	20950		22860	
		19.0	240.4	20590		22650	
	R-23	12.9	245.0	19930		22380	
		18.3	239.5	18680		21140	
	R-25	14.7	244.9	19300		21520	
		20.7	238.9	23060		25280	
	R-31	14.3	244.0	19360		21730	
		20.2	238.0	18200		20970	
	R-42	14.3	245.3	19500		22370	
		23.5	236.0	19200		21870	
Graydon Chert	R-59	15.5	242.4	21300		22730	
Conglomerate		22.0	235.9	20340		22640	
Burlington	R-12	20.5	237.7	23150	23150	25510	25510
Snyder Creek	R-3	30.4	228.7	24200	24200	26370	26370
Callaway	R-3	35.4	223.7	24880	24880	26000	26000
	R-3	51.0	208.1	24080	24408	25090	25498
		64.9	194.2	25610		26410	
		78.7	180.4	23860		24740	
Cotter-Jefferson City		105.6	153.5	24080		25750	

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Table 2.5.4-10—{Unit Weight of Rock Samples (English Units)}

(Page 1 of 2)

	Sample	Location		Depth	Formation	γ
		Boring	ID	[ft]		[pcf]
Static Testing Program	Run 10	R-3	Run 10	101.0	Snyder Creek	167.9
	Run 12	R-3	Run 12	117.7	Callaway	165.5
	Run 17	R-3	Run 17	168.8	Cotter-Jefferson City	159.7
	Run 21	R-3	Run 21	213.9	Cotter-Jefferson City	168.1
	Run 26	R-3	Run 26	259.5	Cotter-Jefferson City	157.5
	Run 34	R-3	Run 34	347.8	Cotter-Jefferson City	163.9
	Run 3	R-9	Run 3	47.4	Graydon Chert Conglomerate	142.9
	Run 5	R-9	Run 5	59.6	Graydon Chert Conglomerate	139.3
	Run 8	R-12	Run 8	68.8	Burlington	162.4
	Run 1	R-17	Run 1	45.8	Graydon Chert Conglomerate	145.5
	Run 4	R-17	Run 4	63.1	Graydon Chert Conglomerate	144.2
	Run 2	R-23	Run 2	43.2	Graydon Chert Conglomerate	142.5
	Run 5	R-23	Run 5	60.9	Graydon Chert Conglomerate	134.6
	Run 3	R-25	Run 3	49.1	Graydon Chert Conglomerate	137.0
	Run 7	R-25	Run 7	69.1	Graydon Chert Conglomerate	160.9
	Run 3	R-31	Run 3	48.3	Graydon Chert Conglomerate	138.3
	Run 7	R-31	Run 7	67.6	Graydon Chert Conglomerate	133.5
	Run 2	R-42	Run 2	47.9	Graydon Chert Conglomerate	142.4
	Run 8	R-42	Run 8	78.1	Graydon Chert Conglomerate	139.2
	Run 2	R-59	Run 2	52.9	Graydon Chert Conglomerate	144.7
	Run 7	R-59	Run 7	74.2	Graydon Chert Conglomerate	144.1
Dynamic Testing Program	UTA-56-C-3C	R-3	Run 1	42.5	Graydon Chert Conglomerate	140.0
	UTA-56-D-3C	R-3	Run 5	68.7	Graydon Chert Conglomerate	143.5
	UTA-56-D-4C	R-2B	AS-1	45.6	Graydon Chert Conglomerate	141.6
	UTA-56-FR1	R-3	Run 6	70.3	Burlington / Bushberg	148.5
	UTA-56-FR2	R-3	Run 6	73.9	Burlington / Bushberg	174.0
	UTA-56-FR3	R-3	Run 7	76.1	Snyder Creek	160.5
	UTA-56-FR4	R-3	Run 12	120.5	Callaway	166.6
	UTA-56-FR5	R-3	Run 17	171.4	Cotter-Jefferson City	158.3
	UTA-56-FR6	R-3	Run 32	324	Cotter-Jefferson City	154.7
	UTA-56-FR7	R-3	Run 6	70.8	Burlington / Bushberg	162.8
	UTA-56-FR8	R-3	Run 6	73.4	Burlington / Bushberg	154.8
	UTA-56-FR9	R-67	Run 5	59.4	Graydon Chert Conglomerate	165.9

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Table 2.5.4-10—{Unit Weight of Rock Samples (SI Units)}

(Page 2 of 2)

Sample		Location		Depth	Formation	γ
		Boring	ID	[m]		[kN/m³]
Static Testing Program	Run 10	R-3	Run 10	30.8	Snyder Creek	26.4
	Run 12	R-3	Run 12	35.9	Callaway	26.0
	Run 17	R-3	Run 17	51.4	Cotter-Jefferson City	25.1
	Run 21	R-3	Run 21	65.2	Cotter-Jefferson City	26.4
	Run 26	R-3	Run 26	79.1	Cotter-Jefferson City	24.7
	Run 34	R-3	Run 34	106.0	Cotter-Jefferson City	25.7
	Run 3	R-9	Run 3	14.5	Graydon Chert Conglomerate	22.4
	Run 5	R-9	Run 5	18.2	Graydon Chert Conglomerate	21.9
	Run 8	R-12	Run 8	21.0	Burlington	25.5
	Run 1	R-17	Run 1	14.0	Graydon Chert Conglomerate	22.9
	Run 4	R-17	Run 4	19.2	Graydon Chert Conglomerate	22.7
	Run 2	R-23	Run 2	13.2	Graydon Chert Conglomerate	22.4
	Run 5	R-23	Run 5	18.6	Graydon Chert Conglomerate	21.1
	Run 3	R-25	Run 3	15.0	Graydon Chert Conglomerate	21.5
	Run 7	R-25	Run 7	21.1	Graydon Chert Conglomerate	25.3
	Run 3	R-31	Run 3	14.7	Graydon Chert Conglomerate	21.7
	Run 7	R-31	Run 7	20.6	Graydon Chert Conglomerate	21.0
	Run 2	R-42	Run 2	14.6	Graydon Chert Conglomerate	22.4
	Run 8	R-42	Run 8	23.8	Graydon Chert Conglomerate	21.9
	Run 2	R-59	Run 2	16.1	Graydon Chert Conglomerate	22.7
	Run 7	R-59	Run 7	22.6	Graydon Chert Conglomerate	22.6
Dynamic Testing Program	UTA-56-C-3C	R-3	Run 1	13.0	Graydon Chert Conglomerate	22.3
	UTA-56-D-3C	R-3	Run 5	20.9	Graydon Chert Conglomerate	22.6
	UTA-56-D-4C	R-2B	AS-1	13.9	Graydon Chert Conglomerate	22.3
	UTA-56-FR1	R-3	Run 6	21.4	Burlington / Bushberg	23.3
	UTA-56-FR2	R-3	Run 6	22.5	Burlington / Bushberg	27.3
	UTA-56-FR3	R-3	Run 7	23.2	Snyder Creek	25.2
	UTA-56-FR4	R-3	Run 12	36.7	Callaway	26.2
	UTA-56-FR5	R-3	Run 17	52.3	Cotter-Jefferson City	24.9
	UTA-56-FR6	R-3	Run 32	98.8	Cotter-Jefferson City	24.3
	UTA-56-FR7	R-3	Run 6	21.6	Burlington / Bushberg	25.6
	UTA-56-FR8	R-3	Run 6	22.4	Burlington / Bushberg	24.3
	UTA-56-FR9	R-67	Run 5	18.1	Graydon Chert Conglomerate	26.1

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Table 2.5.4-11—{Specific Gravity of Rock Samples (English Units)}

Location		Depth		
Boring	ID	[ft]	Formation	SG
R-3	Run 21	213.9	Cotter-Jefferson City	2.704
R-3	Run 26	259.5	Cotter-Jefferson City	2.745
R-3	Run 34	347.8	Cotter-Jefferson City	2.745
R-17	Run 1	45.8	Graydon Chert Conglomerate	2.673
R-17	Run 4	63.1	Graydon Chert Conglomerate	2.713
R-42	Run 2	47.9	Graydon Chert Conglomerate	2.658
R-42	Run 8	78.1	Graydon Chert Conglomerate	2.684
R-59	Run 2	52.9	Graydon Chert Conglomerate	2.663
R-59	Run 7	74.2	Graydon Chert Conglomerate	2.678

Table 2.5.4-11—{Specific Gravity of Rock Samples (SI Units)}

Location		Depth		
Boring	ID	[m]	Formation	SG
R-3	Run 21	65.2	Cotter-Jefferson City	2.704
R-3	Run 26	79.1	Cotter-Jefferson City	2.745
R-3	Run 34	106.0	Cotter-Jefferson City	2.745
R-17	Run 1	14.0	Graydon Chert Conglomerate	2.673
R-17	Run 4	19.2	Graydon Chert Conglomerate	2.713
R-42	Run 2	14.6	Graydon Chert Conglomerate	2.658
R-42	Run 8	23.8	Graydon Chert Conglomerate	2.684
R-59	Run 2	16.1	Graydon Chert Conglomerate	2.663
R-59	Run 7	22.6	Graydon Chert Conglomerate	2.678

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Table 2.5.4-12—{Chemical Test Results (English and SI Units)}

Location		Depth				Resistivity [Ω - cm]			
Boring	ID	[ft]	[m]	Formation	pH	As Received	100% Saturated	Sulfates* [ppm]	Chlorides* [ppm]
R-7	S-11	27.5	8.4	Glacial Till	7.1	1115	1004	131	< 10
R-9	S-22	33.0	10.1	Glacial Till	7.7	1316	1160	189	20
R-14	S-17	42.5	13.0	Graydon Chert Conglomerate	8.3	3791	3345	142	159
R-16	S-16	40.0	12.2	Graydon Chert Conglomerate	7.7	6913	4906	132	< 10
R-16	S-17	42.5	13.0	Graydon Chert Conglomerate	7.6	4683	4237	125	< 10
R-26	S-16	40.0	12.2	Graydon Chert Conglomerate[1]	7.0	35680	2029	147	59
R-31	S-21	35.5	10.8	Glacial Till	8.7	2029	2163	184	20
R-32	S-2	3.0	0.9	Modified Loess	7.4	937	1093	237	< 10
R-32	S-14	21.0	6.4	Glacial till	7.3	558	580	400	< 10
R-33	S-2	3.0	0.9	Modified Loess	5.8	1405	1583	201	< 10
R-36	S-11	16.5	5.0	Glacial Till	7.9	892	1004	162	603
R-59	S-2	8.5	2.6	Modified Loess	6.2	825	825	159	<10
R-61	S-8	38.0	11.6	Graydon Chert Conglomerate	6.9	1784	1985	188	73
R-68	S-4	18.0	5.5	Accretion Gley	6.2	870	781	337	13
Aux-7	7	NA	NA	Auxvasse Quarry	7.9	322300	10255	1262	20
Aux-7b	7	NA	NA	Auxvasse Quarry	7.1	NT	NT	1343	10

(*) Water Soluble Ion Content

(1) Sample received in very dry conditions. Jar was not sealed

NT - Not Tested

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FSAR: Tables 2.5.4

Table 2.5.4-13—{Consolidation Properties (Undisturbed Samples) (English Units)}

Stratum	Boring	Depth [ft]	El. [ft] msl	C_c (1)	C_r (1)	e_o	P_c [ksf]	OCR
Modified Loess	R-1	12.5	836.6	0.110	0.011	0.56	10.40	6.0
	R-50	13.0	820.3	0.226	0.064	0.76	6.90	4.0
Accretion-Gley	R-34	11.7	827.3	0.259	0.046	0.88	4.30	2.9
	R-1	19.0	830.0	0.123	0.031	0.70	9.30	3.7
Glacial Till	R-59	20.0	826.1	0.123	0.026	0.60	7.60	3.6
	R-59	33.0	813.2	0.143	0.038	0.56	9.30	2.1
	R-42	38.0	813.5	0.179	0.039	0.70	1.90	0.4

Table 2.5.4-13—{Consolidation Properties (Undisturbed Samples) (SI Units)}

Stratum	Boring	Depth [m]	El. [m] msl	C_c (1)	C_r (1)	e_o	P_c [kPa]	OCR
Modified Loess	R-1	3.8	255.1	0.110	0.011	0.56	498	6.0
	R-50	13.0	250.1	0.226	0.064	0.76	330	4.0
Accretion-Gley	R-34	11.7	252.2	0.259	0.046	0.88	206	2.9
	R-1	19.0	253.0	0.123	0.031	0.70	445	3.7
Glacial Till	R-59	20.0	251.8	0.123	0.026	0.60	364	3.6
	R-59	33.0	247.9	0.143	0.038	0.56	445	2.1
	R-42	38.0	248.0	0.179	0.039	0.70	91	0.4

Notes:

C_c - compression index
e_O - void ratio
P_c - pre-consolidation pressure
OCR- over-consolidation ratio
(1) Values are based on void ratio

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Table 2.5.4-14—{Strength Parameters Based on Consolidated-Undrained Triaxial Test Results With Pore Pressure Measurements (Undisturbed Samples) (English Units)}

(Page 1 of 2)

Stratum	Boring	Depth [ft]	El. [ft] msl	Effective		Total	
				ϕ' [o]	c' [psf]	ϕ [o]	c [psf]
Modified Loess	R-1	12.5	836.46	18.8	881.0	7.1	1541.0
	R-50	13.0	820.16	28.3	402.0	13.2	917.0
Accretion-Gley	R-34	11.7	827.12	21.2	232.0	8.9	452.0
	R-56	13.5	828.11	20.8	380.0	7.7	733.0
	R-38	18.0	933.39	24.4	292.0	9.3	766.0
	R-42	18.0	833.39	17.1	641.0	1.5	1714.0
	R-1	19.0	829.96	23.2	465.0	12.5	760.0
	R-41	19.0	832.36	15.1	1070.0	0.0	1872.0
Glacial Till	R-59	20.0	826.06	20.5	782.0	2.7	1886.0
	R-56	24.0	817.61	4.6	1302.0	0.0	1800.0
	R-1	27.0	821.96	31.1	366.0	21.1	220.0
	R-38	33.0	818.39	17.8	1584.0	6.2	2218.0
	R-59	33.0	813.06	45.8	282.0	19.5	1041.0
	R-1	33.5	815.46	32.6	232.0	17.1	241.0
	R-2	35.0	813.96	27.7	668.0	14.2	805.0
	R-41	35.5	815.86	31.2	318.0	13.7	3053.0
	R-42	38.0	813.39	3.7	2549.0	0.0	3240.0

Note:

Effective strength parameters are based on maximum obliquity
Total strength parameters are based on maximum shear

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Table 2.5.4-14—{Strength Parameters Based on Consolidated-Undrained Triaxial Test Results With Pore Pressure Measurements (Undisturbed Samples) (SI Units)}

(Page 2 of 2)

Stratum	Boring	Depth [m]	El. [m] msl	Effective		Total	
				ϕ' [o]	c' [kPa]	ϕ' [o]	c [kPa]
	R-1	3.8	255.02	18.8	42	7.1	74
Modified Loess	R-50	4.0	250.05	28.3	19	13.2	44
	R-34	3.6	252.17	21.2	11	8.9	22
	R-56	4.1	252.47	20.8	18	7.7	35
	R-38	5.5	284.57	24.4	14	9.3	37
	R-42	5.5	254.08	17.1	31	1.5	82
	R-1	5.8	253.04	23.2	22	12.5	36
Accretion-Gley	R-41	5.8	253.77	15.1	51	0.0	90
	R-59	6.1	251.85	20.5	37	2.7	90
	R-56	7.3	249.27	4.6	62	0.0	86
	R-1	8.2	250.60	31.1	18	21.1	11
	R-38	10.1	249.51	17.8	76	6.2	106
	R-59	10.1	247.88	45.8	14	19.5	50
	R-1	10.2	248.62	32.6	11	17.1	12
	R-2	10.7	248.16	27.7	32	14.2	39
	R-41	10.8	248.74	31.2	15	13.7	146
Glacial Till	R-42	11.6	247.98	3.7	122	0.0	155

Note:

Effective strength parameters are based on maximum obliquity
Total strength parameters are based on maximum shear

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Table 2.5.4-15—{Unconfined Compression of Rock Samples (English Units)}

(Page 1 of 2)

Location		Depth		q
Boring	ID	[ft]	Formation	[ksf]
R-3	Run 10	101.0	Snyder Creek	2730
R-3	Run 12	117.7	Callaway	1339
R-3	Run 17	168.8	Cotter-Jefferson City	2533
R-3	Run 21	213.9	Cotter-Jefferson City	2748
R-3	Run 26	259.5	Cotter-Jefferson City	1198
R-3	Run 34	347.8	Cotter-Jefferson City	1803
R-9	Run 3	47.4	Graydon Chert Conglomerate	<15
R-9	Run 5	59.6	Graydon Chert Conglomerate	<15
R-12	Run 8	68.8	Burlington	1590
R-17	Run 1	45.8	Graydon Chert Conglomerate	<15
R-17	Run 4	63.1	Graydon Chert Conglomerate	<15
R-23	Run 2	43.2	Graydon Chert Conglomerate	<15
R-23	Run 5	60.9	Graydon Chert Conglomerate	<15
R-25	Run 3	49.1	Graydon Chert Conglomerate	<15
R-25	Run 7	69.1	Graydon Chert Conglomerate	227
R-31	Run 3	48.3	Graydon Chert Conglomerate	<15
R-31	Run 7	67.6	Graydon Chert Conglomerate	<15
R-42	Run 2	47.9	Graydon Chert Conglomerate	<15
R-42	Run 8	78.1	Graydon Chert Conglomerate	<15
R-59	Run 2	52.9	Graydon Chert Conglomerate	111
R-59	Run 7	74.2	Graydon Chert Conglomerate	<15

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Table 2.5.4-15—{Unconfined Compression of Rock Samples (SI Units)}

(Page 2 of 2)

Location		Depth		q
Boring	ID	[m]	Formation	[kPa]
R-3	Run 10	30.8	Snyder Creek	130724
R-3	Run 12	35.9	Callaway	64121
R-3	Run 17	51.4	Cotter-Jefferson City	121278
R-3	Run 21	65.2	Cotter-Jefferson City	131551
R-3	Run 26	79.1	Cotter-Jefferson City	57364
R-3	Run 34	106.0	Cotter-Jefferson City	86322
R-9	Run 3	14.5	Graydon Chert Conglomerate	<700
R-9	Run 5	18.2	Graydon Chert Conglomerate	<700
R-12	Run 8	21.0	Burlington	76118
R-17	Run 1	14.0	Graydon Chert Conglomerate	<700
R-17	Run 4	19.2	Graydon Chert Conglomerate	<700
R-23	Run 2	13.2	Graydon Chert Conglomerate	<700
R-23	Run 5	18.6	Graydon Chert Conglomerate	<700
R-25	Run 3	15.0	Graydon Chert Conglomerate	<700
R-25	Run 7	21.1	Graydon Chert Conglomerate	10880
R-31	Run 3	14.7	Graydon Chert Conglomerate	<700
R-31	Run 7	20.6	Graydon Chert Conglomerate	<700
R-42	Run 2	14.6	Graydon Chert Conglomerate	<700
R-42	Run 8	23.8	Graydon Chert Conglomerate	<700
R-59	Run 2	16.1	Graydon Chert Conglomerate	5337
R-59	Run 7	22.6	Graydon Chert Conglomerate	<700

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Table 2.5.4-16—{URCS Classification of Rock Samples (English Units)}

(Page 1 of 2)

Location		Depth [ft]	Formation	Weathering	Strength	Discontinuity	Weight
Boring	ID						
R-3	Run 10	101.0	Snyder Creek	B	A	A	A
R-3	Run 12	117.7	Callaway	B	B	B	A
R-3	Run 17	168.8	Cotter-Jefferson City	B	A	A	B
R-3	Run 21	213.9	Cotter-Jefferson City	B	A	A	A
R-3	Run 26	259.5	Cotter-Jefferson City	B	B	A	B
R-3	Run 34	347.8	Cotter-Jefferson City	B	B	A	A
R-9	Run 3	47.4	Graydon Chert Conglomerate	E	E	E	C
R-9	Run 5	59.6	Graydon Chert Conglomerate	E	E	E	D
R-12	Run 8	68.8	Burlington	B	B	A	A
R-17	Run 1	45.8	Graydon Chert Conglomerate	E	E	E	C
R-17	Run 4	63.1	Graydon Chert Conglomerate	E	E	E	C
R-23	Run 2	43.2	Graydon Chert Conglomerate	E	E	E	C
R-23	Run 5	60.9	Graydon Chert Conglomerate	E	E	E	D
R-25	Run 3	49.1	Graydon Chert Conglomerate	E	E	E	D
R-25	Run 7	69.1	Graydon Chert Conglomerate	C	D	E	A
R-31	Run 3	48.3	Graydon Chert Conglomerate	E	E	E	D
R-31	Run 7	67.6	Graydon Chert Conglomerate	E	E	E	D
R-42	Run 2	47.9	Graydon Chert Conglomerate	E	E	E	C
R-42	Run 8	78.1	Graydon Chert Conglomerate	E	E	E	D
R-59	Run 2	52.9	Graydon Chert Conglomerate	E	E	D	C
R-59	Run 7	74.2	Graydon Chert Conglomerate	E	E	D	C

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Table 2.5.4-16—{URCS Classification of Rock Samples (SI Units)}

(Page 2 of 2)

Location		Depth [m]	Formation	Weathering	Strength	Discontinuity	Weight
Boring	ID						
R-3	Run 10	30.8	Snyder Creek	B	A	A	A
R-3	Run 12	35.9	Callaway	B	B	B	A
R-3	Run 17	51.4	Cotter-Jefferson City	B	A	A	B
R-3	Run 21	65.2	Cotter-Jefferson City	B	A	A	A
R-3	Run 26	79.1	Cotter-Jefferson City	B	B	A	B
R-3	Run 34	106.0	Cotter-Jefferson City	B	B	A	A
R-9	Run 3	14.5	Graydon Chert Conglomerate	E	E	E	C
R-9	Run 5	18.2	Graydon Chert Conglomerate	E	E	E	D
R-12	Run 8	21.0	Burlington	B	B	A	A
R-17	Run 1	14.0	Graydon Chert Conglomerate	E	E	E	C
R-17	Run 4	19.2	Graydon Chert Conglomerate	E	E	E	C
R-23	Run 2	13.2	Graydon Chert Conglomerate	E	E	E	C
R-23	Run 5	18.6	Graydon Chert Conglomerate	E	E	E	D
R-25	Run 3	15.0	Graydon Chert Conglomerate	E	E	E	D
R-25	Run 7	21.1	Graydon Chert Conglomerate	C	D	E	A
R-31	Run 3	14.7	Graydon Chert Conglomerate	E	E	E	D
R-31	Run 7	20.6	Graydon Chert Conglomerate	E	E	E	D
R-42	Run 2	14.6	Graydon Chert Conglomerate	E	E	E	C
R-42	Run 8	23.8	Graydon Chert Conglomerate	E	E	E	D
R-59	Run 2	16.1	Graydon Chert Conglomerate	E	E	D	C
R-59	Run 7	22.6	Graydon Chert Conglomerate	E	E	D	C

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Table 2.5.4-17—{Permeability Test Results (English Units)}

Soil Type	Boring	Depth [ft]	Elevation [feet] msl	Eff. Confining Pressure [psf]	Hydraulic Conductivity [fps]
Modified Loess	R-1	12.5	836.46	900	7.87E-11
Accretion-Gley	R-1	19.0	829.96	1320	8.20E-11
Glacial Till	R-2	35.0	813.96	2320	1.44E-10
Glacial Till	R-38	33.0	818.39	1220	8.20E-11

Table 2.5.4-17—{Permeability Test Results (SI Units)}

Soil Type	Boring	Depth [m]	Elevation [m] msl	Eff. Confining Pressure [kPa]	Hydraulic Conductivity [cm/s]
Modified Loess	R-1	3.8	255.02	43	2.40E-09
Accretion-Gley	R-1	5.8	253.04	63	2.50E-09
Glacial Till	R-2	10.7	248.16	111	4.40E-09
Glacial Till	R-38	10.1	249.51	58	2.50E-09

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Table 2.5.4-18—{Resonant Column Torsional Shear Test Samples (English and SI Units)}

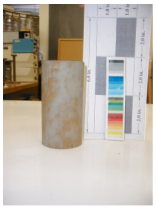
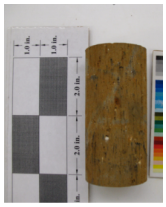
Sample	Location		Depth [ft (m)]	Photograph	Classification
	Boring	ID			
UTA-56-A (1E)	R-31	ST-1	10.2 (3.11)		Loess/Accretion Gley (CH)
UTA-56-B (2C)	R-31	ST-2	33.8 (10.30)		Glacial Till (CL)
UTA-56-C (3C)	R-3	Run 1	42.5 (12.95)		Graydon Chert Conglomerate (Mixed Rock/Soil Matrix)
UTA-56-D (4C)	R-3	Run 5	68.7 (20.94)		Graydon Chert Conglomerate (Fragmented Weathered Chert)
UTA-56-E (5C)	R-2B	AS-1	45.6 (13.90)		Graydon Chert Conglomerate (Cored Boulder)
UTA-56-B (2G)	R-31	ST-2	33.2 (10.12)		Glacial Till (CL)

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Table 2.5.4-19—{Resonant Column Low Strain Results (English Units)}

Sample	Location Boring	Location ID	Depth [ft]	Classification	Confining Pressures [psf]		γ [pcf]	G_{max} [ksf]	V_s [fps]	D [%]
UTA-56-A (1E)	R-31	ST-1	10.2	Loess/Accretion Gley (CH)	$\sigma_{o(min)}$	216	126	1008	507	8.21
					$\sigma_{o(site)}$	864	127	1114	532	7.94
					$\sigma_{o(max)}$	3456	128	1282	569	7.95
UTA-56-B (2C)	R-31	ST-2	33.8	Glacial Till (CL)	$\sigma_{o(min)}$	720	133	2096	712	7.13
					$\sigma_{o(site)}$	3024	133	2322	750	6.78
					$\sigma_{o(max)}$	11952	135	2937	837	6.41
UTA-56-C (3C)	R-3	Run 1	42.5	Graydon Chert Conglomerate (Mixed Rock/Soil Matrix)	$\sigma_{o(min)}$	432	140	2855	810	6.89
					$\sigma_{o(site)}$	3312	142	4585	1020	5.64
					$\sigma_{o(max)}$	13248	146	9773	1468	4.78
UTA-56-D (4C)	R-3	Run 5	68.7	Graydon Chert Conglomerate (Fragmented Weathered Chert)	$\sigma_{o(min)}$	1152	144	6966	1248	7.96
					$\sigma_{o(site)}$	4608	144	8244	1358	7.71
					$\sigma_{o(max)}$	18288	145	10863	1553	7.65
UTA-56-D (5C)	R-2B	AS-1	45.6	Graydon Chert Conglomerate (Cored Boulder)	$\sigma_{o(min)}$	864	142	111050	5023	1.66
					$\sigma_{o(site)}$	3456	142	111040	5023	1.83
					$\sigma_{o(max)}$	27648	142	114190	5092	1.78
UTA-56-B (2G)	R-31	ST-2	33.2	Glacial Till (CL)	$\sigma_{o(min)}$	720	132	2448	774	6.76
					$\sigma_{o(site)}$	2880	132	2754	820	6.75
					$\sigma_{o(max)}$	11520	133	3421	912	6.32

Table 2.5.4-19—{Resonant Column Low Strain Results (SI Units)}

Sample	Location Boring	Location ID	Depth [m]	Classification	Confining Pressures [kPa]		γ [kN/m³]	G_{max} [MPa]	V_s [m/s]	D [%]
UTA-56-A (1E)	R-31	ST-1	3.11	Loess/Accretion Gley (CH)	10	20	48	155	8.21	10
					41	20	53	162	7.94	41
					165	20	61	173	7.95	165
UTA-56-B (2C)	R-31	ST-2	10.30	Glacial Till (CL)	34	21	100	217	7.13	34
					145	21	111	229	6.78	145
					572	21	141	255	6.41	572
UTA-56-C (3C)	R-3	Run 1	12.96	Graydon Chert Conglomerate (Mixed Rock/Soil Matrix)	21	22	137	247	6.89	21
					159	22	220	311	5.64	159
					634	23	468	448	4.78	634
UTA-56-D (4C)	R-3	Run 5	20.95	Graydon Chert Conglomerate (Fragmented Weathered Chert)	55	23	334	381	7.96	55
					221	23	395	414	7.71	221
					876	23	520	474	7.65	876
UTA-56-D (5C)	R-2B	AS-1	13.90	Graydon Chert Conglomerate (Cored Boulder)	41	22	5317	1532	1.66	41
					165	22	5317	1531	1.83	165
					1324	22	5467	1552	1.78	1324
UTA-56-B (2G)	R-31	ST-2	10.12	Glacial Till (CL)	34	21	117	236	6.76	34
					138	21	132	250	6.75	138
					552	21	164	278	6.32	552

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Table 2.5.4-20—{"Free-Free" Test Samples (English and SI Units)}

(Page 1 of 2)

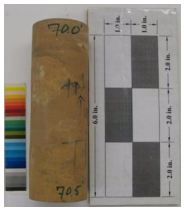
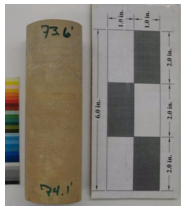
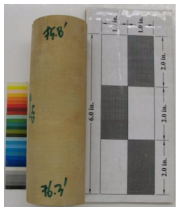
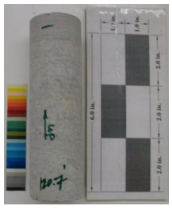
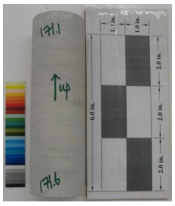
Sample	Location		Depth [ft (m)]	Photograph	Formation
	Boring	ID			
UTA-56-FR1	R-3	Run 6	70.3 (21.42)		Burlington / Bushberg
UTA-56-FR2	R-3	Run 6	73.9 (22.52)		Burlington / Bushberg
UTA-56-FR3	R-3	Run 7	76.1 (23.19)		Snyder Creek
UTA-56-FR4	R-3	Run 12	120.5 (36.72)		Callaway
UTA-56-FR5	R-3	Run 17	171.4 (52.24)		Cotter-Jefferson City
UTA-56-FR6	R-3	Run 32	324.0 (98.77)		Cotter-Jefferson City (w/Shale Laminae)
UTA-56-FR7	R-3	Run 6	70.8 (21.57)		Burlington / Bushberg

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Table 2.5.4-20—{"Free-Free" Test Samples (English and SI Units)}

(Page 2 of 2)

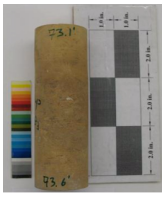
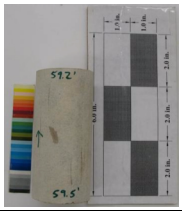
Sample	Location		Depth [ft (m)]	Photograph	Formation
	Boring	ID			
UTA-56-FR8	R-3	Run 6	73.4 (22.36)		Burlington / Bushberg
UTA-56-FR9	R-67	Run 5	59.4 (18.09)		Graydon Chert Conglomerate

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Table 2.5.4-21—{"Free-Free" Test Results (English Units)}

Sample	Location		Depth	Formation	γ	V_s	D_s	V_c [1]	D_c	V_p [2]
	Boring	ID	[ft]		[pcf]	[fps]	[%]	[fps]	[%]	[fps]
UTA-56-FR1	R-3	Run 6	70.3	Burlington / Bushberg	148.5	7420	0.41	11300	0.53	12190
UTA-56-FR2	R-3	Run 6	73.9	Burlington / Bushberg	174.0	11180	0.43	17760	1.30	19130
UTA-56-FR3	R-3	Run 7	76.1	Snyder Creek	160.5	9210	0.48	14150	1.00	14710
UTA-56-FR4	R-3	Run 12	120.5	Callaway	166.6	10510	0.20	16820	0.36	18610
UTA-56-FR5	R-3	Run 17	171.4	Cotter-Jefferson City	158.3	8800	1.08	14200	0.76	14880
UTA-56-FR6	R-3	Run 32	324	Cotter-Jefferson City (w/Shale Laminae)	154.7	8070	1.74	11820	4.87	12410
UTA-56-FR7	R-3	Run 6	70.8	Burlington / Bushberg	162.8	8980	0.84	13110	0.95	14020
UTA-56-FR8	R-3	Run 6	73.4	Burlington / Bushberg	154.8	9430	0.25	14330	0.62	15250
UTA-56-FR9	R-67	Run 5	59.4	Graydon Chert Conglomerate	165.9	10770	0.96	17230	0.59	19050

(1) Compression wave velocity from unconstrained test

(2) Compression wave velocity from constrained test

Table 2.5.4-21—{"Free-Free" Test Results (SI Units)}

Sample	Location		Depth	Formation	γ	V_s	D_s	V_c [1]	D_c	V_p [2]
	Boring	ID	[m]		[kN/m^3]	[m/s]	[%]	[m/s]	[%]	[m/s]
UTA-56-FR1	R-3	Run 6	21.43	Burlington / Bushberg	23.3	2262	0.41	3445	0.53	3716
UTA-56-FR2	R-3	Run 6	22.53	Burlington / Bushberg	27.3	3409	0.43	5415	1.30	5832
UTA-56-FR3	R-3	Run 7	23.201	Snyder Creek	25.2	2808	0.48	4314	1.00	4485
UTA-56-FR4	R-3	Run 12	36.738	Callaway	26.2	3204	0.20	5128	0.36	5674
UTA-56-FR5	R-3	Run 17	52.256	Cotter-Jefferson City	24.9	2683	1.08	4329	0.76	4537
UTA-56-FR6	R-3	Run 32	98.78	Cotter-Jefferson City (w/Shale Laminae)	24.3	2460	1.74	3604	4.87	3784
UTA-56-FR7	R-3	Run 6	21.585	Burlington / Bushberg	25.6	2738	0.84	3997	0.95	4274
UTA-56-FR8	R-3	Run 6	22.378	Burlington / Bushberg	24.3	2875	0.25	4369	0.62	4649
UTA-56-FR9	R-67	Run 5	18.11	Graydon Chert Conglomerate	26.1	3284	0.96	5253	0.59	5808

(1) Compression wave velocity from unconstrained test

(2) Compression wave velocity from constrained test

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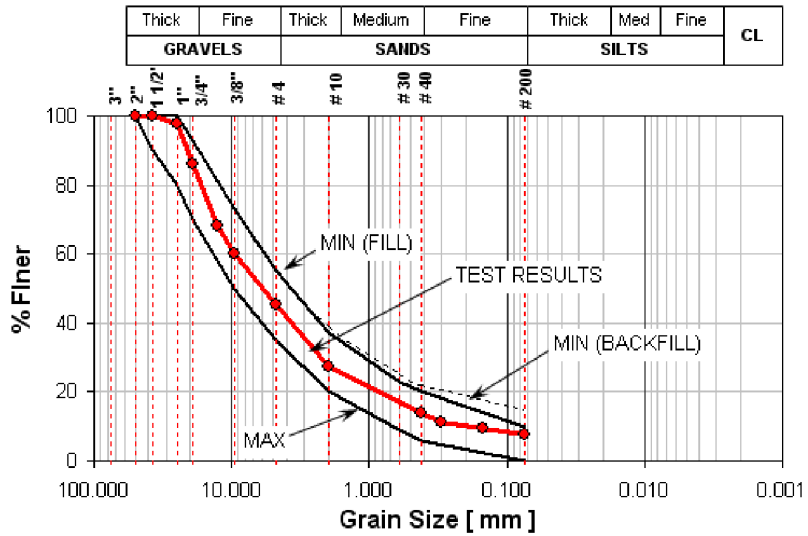
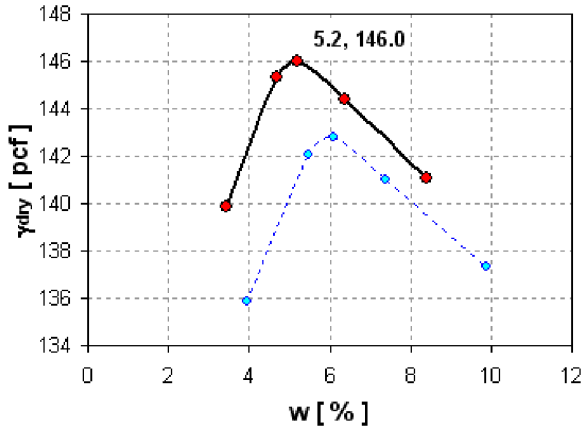
Table 2.5.4-22—{Category 1 Structural Fill and Backfill Properties (English Units)}

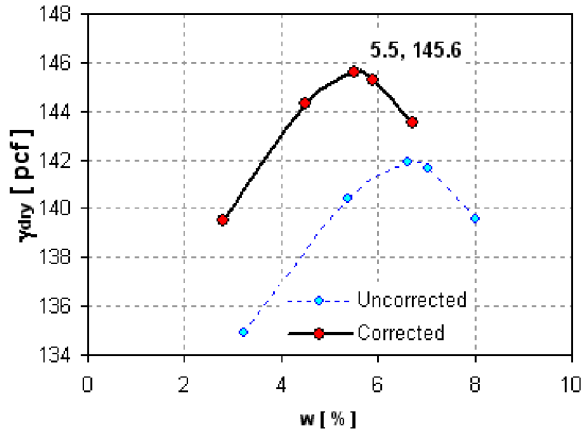
(Page 1 of 2)

Parameter				Fill[1]	Backfill[2]
Grain Size Analysis	% Gravel		[%]	57.0	57.0
	% Sands		[%]	37.7	37.7
	% Fines		[%]	7.6	7.6
Mass (in 1 ft^3)	Solids	m_s	[lbm]	139	131
	Water	m_w	[lbm]	7.4	7.0
	Total	m_t	[lbm]	146	138
Comb	Moisture Content	w	[%]	5.4	5.4
	Saturation	S	[%]	57.0	45.0
	Specific Gravity	SG	[]	2.8	2.8
Unit Weight	Dry	γ_{dry}	[pcf]	139	131
	Moist	γ_{moist}	[pcf]	146	138
	Wet	γ_{wet}	[pcf]	151	147



Modified Proctor





(1) 95% Modified Proctor; (2) 90% Modified Proctor

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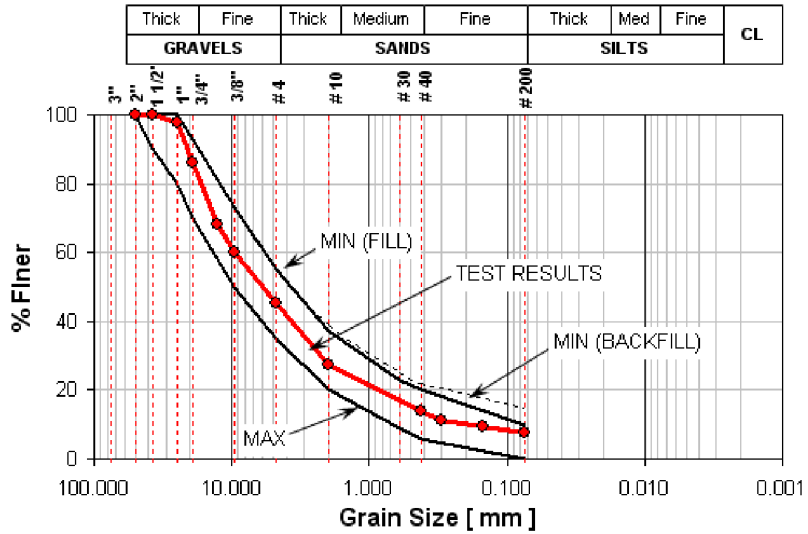
FSAR: Tables 2.5.4

Table 2.5.4-22—{Category 1 Structural Fill and Backfill Properties (SI Units)}
(Page 2 of 2)

Parameter				Fill[1]	Backfill[2]
Grain Size Analysis	% Gravel		[%]	57.0	57.0
	% Sands		[%]	37.7	37.7
	% Fines		[%]	7.6	7.6
Mass (in 1 ft^3)	Solids	m_s	[kg]	2220	2100
	Water	m_w	[kg]	120.0	110.0
	Total	m_t	[kg]	2340	2210
Comb	Moisture Content	w	[%]	5.4	5.4
	Saturation	S	[%]	57.0	45.0
	Specific Gravity	SG	[]	2.8	2.8
Unit Weight	Dry	γ_{dry}	[kN]	21.8	20.6
	Moist	γ_{moist}	[kN]	22.9	21.7
	Wet	γ_{wet}	[kN]	23.8	23.1



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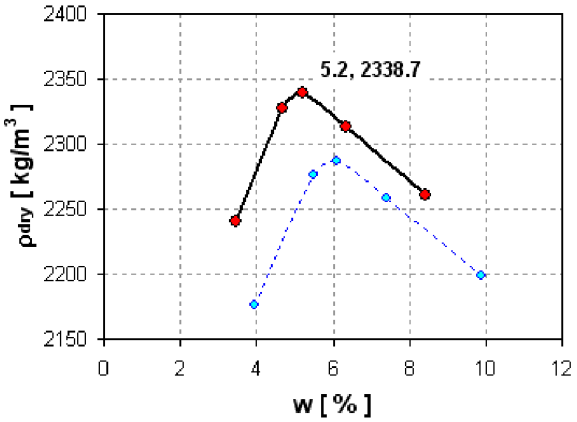

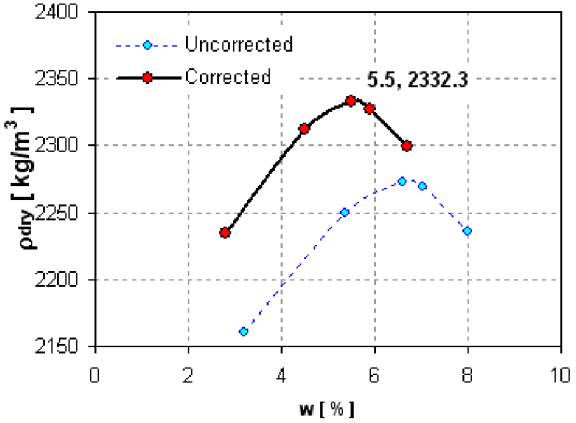

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FSAR: Tables 2.5.4

Table 2.5.4-23—{Recommended Values of Index Properties (English Units)}

(Page 1 of 2)

	Unit	USCS or URCS	Water Content [%]	Liquid Limit [%]	Plastic Index [%]	Unit Weight [pcf]	Grain Size	Observations
OVERBURDEN	Modified Loess	CL	23	47	30	130	95% > No. 200	Values are determined from observation and statistical analysis of field and laboratory tests. For more information refer to Section 2.5.4.1 and Section 2.5.4.2
	Accretion-Gley	CH	23	52	40	130	90% > No. 200	
	Glacial Till	CL	19	41	30	130	70% > No. 200	
ROCK FORMATIONS	Graydon Chert Conglomerate	EEEC	NA	NP	NP	142	NA	- URCS Classification: (Weathering, Strength, Discontinuity, Weight) - Bushberg Formation Classification provided from Burlington data - NA: Not Applicable - NP: Non Plastic
	Burlington Formation	BBAA	NA	NP	NP	160	NA	
	Bushberg Formation	BBAA	NA	NP	NP	160	NA	
	Snyder Creek Formation	BAAA	NA	NP	NP	161	NA	
	Callaway Formation	BBBA	NA	NP	NP	167	NA	
	Cotter-Jefferson City Formation	BBAB	NA	NP	NP	157	NA	
FILLS	Category 1 Granular Fill	SW	5.2	NP	NP	140	(*)	(*) Refer to Table 2.5.4-22
	Category 1 Granular Backfill	SW	5.2	NP	NP	140	(*)	

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FSAR: Tables 2.5.4

Table 2.5.4-23—{Recommended Values of Index Properties (SI Units)}

(Page 2 of 2)

	Unit	USCS or URCS	Water Content [%]	Liquid Limit [%]	Plastic Index [%]	Unit Weight [kN/m³]	Grain Size	Observations
OVERBURDEN	Modified Loess	CL	23	47	30	20.4	95% > No. 200	Values are determined from observation and statistical analysis of field and laboratory tests. For more information refer to Section 2.5.4.1 and Section 2.5.4.2
	Accretion-Gley	CH	23	52	40	20.4	90% > No. 200	
	Glacial Till	CL	19	41	30	20.4	70% > No. 200	
ROCK FORMATIONS	Graydon Chert Conglomerate	EEEC	NA	NP	NP	22.3	NA	-URCS Classification: (Weathering, Strength, Discontinuity, Weight) - Bushberg Formation Classification provided from Burlington data - NA: Not Applicable - NP: Non Plastic
	Burlington Formation	BBAA	NA	NP	NP	25.1	NA	
	Bushberg Formation	BBAA	NA	NP	NP	25.1	NA	
	Snyder Creek Formation	BAAA	NA	NP	NP	25.3	NA	
	Callaway Formation	BBBA	NA	NP	NP	26.2	NA	
	Cotter-Jefferson City Formation	BBAB	NA	NP	NP	24.7	NA	
FILLS	Category 1 Granular Fill	SW	5.2	NP	NP	22.0	(*)	(*) Refer to Table 2.5.4-22
	Category 1 Granular Backfill	SW	5.2	NP	NP	22.0	(*)	

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FSAR: Tables 2.5.4

Table 2.5.4-24—{Undrained Shear Strength From Pressuremeter (English Units)}

Boring	Depth [feet]	E_s [ksf]	G [ksf]	s_u [ksf]	s_u Average [ksf]	s_u Avg - σ [ksf] [1]
R-5B	43.5 - 45.5	1,148	400	39	88.5	18.5
R-5B	58.5 - 60.5	4,662	1,624	184		
R-2B	48.4 - 50.4	5,520	1,922	138		
R-2B	63.4 - 65.4	4,858	1,692	150		

Table 2.5.4-24—{Undrained Shear Strength from Pressuremeter (SI Units)}

Boring	Depth [feet]	E_s [kPa]	G [kPa]	s_u [kPa]	s_u Average [kPa]	s_u Avg - σ [ksf] [1]
R-5B	13.3 - 13.9	54,970	19,140	1870	4240	888
R-5B	17.8 - 18.4	223,220	77,740	8810		
R-2B	14.8 - 15.4	264,300	92,050	6610		
R-2B	19.3 - 19.9	232,600	81,010	7180		

Note:

(1) Average of shallower samples minus 1 Standard Deviation
E_s - elastic modulus
G - shear modulus

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Table 2.5.4-25—{Recommended Values for Strength Properties (English Units)}

	Unit	SPT	c [ksf]	ϕ [°]	c' [ksf]	ϕ' [°]	s_u [ksf]	q_u [ksf]	Observations
OVERBURDEN	Modified Loess	10	1.3	10.2	0.6	23.6	1.3	2.6	- 'c and 's_u' recommendation based on average of triaxial test results and SPT correlation - 'c', ϕ, ϕ', and q_u recommendation based on Laboratory Testing Program
	Accretion-Gley	10	1.2	6.7	0.5	20.3	1.2	2.4	
	Glacial Till	20	2.1	10.5	0.9	23.9	2.1	4.2	
ROCK FORMATIONS	Graydon Chert Conglomerate	> 50	10.5	0.0	2.0	25.0	10.5	< 15.0	Unconfined compression for Graydon Chert Conglomerate is not possible to determine from Laboratory Tests. Value based on Pressuremeter Tests and existing references. Refer to Table 2.5.4-24 NA: Not applicable.
	Burlington Formation	NA	795	NA	NA	NA	795	1590	
	Bushberg Formation	NA	795	NA	NA	NA	795	1590	
	Snyder Creek Formation	NA	1365	NA	NA	NA	1365	2730	
	Callaway Formation	NA	670	NA	NA	NA	670	1340	
	Cotter-Jefferson City Formation	NA	1035	NA	NA	NA	1035	2070	
FILLS	Category 1 Granular Fill	NA	NA	NA	0.0	35.0	NA	NA	NA: Not Applicable Estimated Angle of Internal Friction
	Category 1 Granular Backfill	NA	NA	NA	0.0	35.0	NA	NA	

Table 2.5.4-25—{Recommended Values for Strength Properties (SI Units)}

	Unit	SPT	c [kPa]	ϕ [°]	c' [kPa]	ϕ' [°]	s_u [kPa]	q_u [kPa]	Observations
OVERBURDEN	Modified Loess	10	62	10.2	29	23.6	62	124	- 'c and 's_u' recommendation based on average of triaxial test results and SPT correlation - 'c', ϕ, ϕ', and q_u recommendation based on Laboratory Testing Program
	Accretion-Gley	10	57	6.7	24	20.3	57	114.9	
	Glacial Till	20	101	10.5	43	23.9	101	201.1	
ROCK FORMATIONS	Graydon Chert Conglomerate	> 50	500	0	95.76	25	500	< 720	Unconfined compression for Graydon Chert Conglomerate is not possible to determine from Laboratory Tests. Value based on Pressuremeter Tests and existing references. Refer to Table 2.5.4-24. NA: Not applicable.
	Burlington Formation	NA	38060	NA	NA	NA	38060	76130	
	Bushberg Formation	NA	38060	NA	NA	NA	38060	76130	
	Snyder Creek Formation	NA	65360	NA	NA	NA	65360	130710	
	Callaway Formation	NA	32080	NA	NA	NA	32080	64160	
	Cotter-Jefferson City Formation	NA	49560	NA	NA	NA	49560	99110	
FILLS	Category 1 Granular Fill	NA	NA	NA	0.0	35.0	NA	NA	NA: Not Applicable Estimated Angle of Internal Friction
	Category 1 Granular Backfill	NA	NA	NA	0.0	35.0	NA	NA	

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Table 2.5.4-26—{Recommended Values For Permeability And Consolidation (English Units)}

Unit		K [fps]	Pc [ksf]	OCR	$C_c^{(*)}$	e_o	Observations
OVERBURDEN	Modified Loess	7.9E-11	8.0	5	0.17	0.66	Values based on Laboratory Testing Program. (*) C_c - Compression Index
	Accretion-Gley	8.2E-11	6.8	3	0.19	0.79	
	Glacial Till	1.1E-10	6.3	2	0.15	0.62	
ROCK FORMATIONS	Graydon Chert Conglomerate	(1)	NA	NA	NA	NA	(1) Not measured, refer to Section 2.4 for additional information NA: Not Applicable
	Burlington Formation	NA	NA	NA	NA	NA	
	Bushberg Formation	NA	NA	NA	NA	NA	
	Snyder Creek Formation	NA	NA	NA	NA	NA	
	Callaway Formation	NA	NA	NA	NA	NA	
	Cotter-Jefferson City Formation	NA	NA	NA	NA	NA	
FILLS	Category 1 Granular Fill	1.6E-02	NA	NA	NA	NA	Permeability recommended based on typical well graded sands and gravels NA: Not Applicable
	Category 1 Granular Backfill	1.6E-02	NA	NA	NA	NA	

Table 2.5.4-26—{Recommended Values For Permeability And Consolidation (SI Units)}

Unit		K [cm/s]	Pc [kPa]	OCR	$C_c^{(*)}$	e_o	Observations
OVERBURDEN	Modified Loess	2.40E-09	380	5	0.17	0.66	Values based on Laboratory Testing Program. $C_c^{(*)}$ Compression Index
	Accretion-Gley	2.50E-09	330	3	0.19	0.79	
	Glacial Till	3.45E-09	300	2	0.15	0.62	
ROCK FORMATIONS	Graydon Chert Conglomerate	(1)	NA	NA	NA	NA	(1) Not measured, refer to Section 2.4 for additional information NA: Not Applicable
	Burlington Formation	NA	NA	NA	NA	NA	
	Bushberg Formation	NA	NA	NA	NA	NA	
	Snyder Creek Formation	NA	NA	NA	NA	NA	
	Callaway Formation	NA	NA	NA	NA	NA	
	Cotter-Jefferson City Formation	NA	NA	NA	NA	NA	
FILLS	Category 1 Granular Fill	5.0E-01	NA	NA	NA	NA	Permeability recommended based on typical well graded sands and gravels NA: Not Applicable
	Category 1 Granular Backfill	5.0E-01	NA	NA	NA	NA	

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Table 2.5.4-27—{Recommended Values For Elastic Modulus (English Units)}

(Page 1 of 2)

Unit		E [ksf]						Observations
		Method 1	Method 2	Method 3	Method 4	Method 5	Rec.	
OVERBURDEN	Modified Loess	650	1676	1258	-	-	650	Method 1: $E = 500\ s_u$ (Bowles 1996) Method 2: $E = K\ s_{u\ (Bowles\ 1996)}$ $K = 4200 - 142.54I_p + 1.73I_p^2 - 0.0071I_p^3$ Method 3: FWHA (2002), SPT Correlation
	Accretion-Gley	600	974	1267	-	-	600	
	Glacial Till	1050	2707	2015	-	-	1050	
ROCK FORMATIONS	Graydon Chert Conglomerate	-	-	-	4050	2000	2000	For Graydon Chert Conglomerate: - Method 4: Pressuremeter Testing - Method 5: Callaway Plant Unit 1 FSAR Plate Load and Pressuremeter - Dynamic Modulus is 70000 ksf Largest static value recommended Localized tests are too conservative Method 5 for other Rocks - "Free-Free" Testing
	Burlington Formation	-	-	-	-	1036250	1036250	
	Bushberg Formation	-	-	-	-	1036250	1036250	
	Snyder Creek Formation	-	-	-	-	998000	998000	
	Callaway Formation	-	-	-	-	1460000	1460000	
	Cotter-Jefferson City Formation	-	-	-	-	831500	831500	
FILLS	Category 1 Granular Fill	-	-	-	-	2500	2500	Recommended values (Callaway Plant Unit 1 FSAR)
	Category 1 Granular Backfill	-	-	-	-	2500	2500	

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FSAR: Tables 2.5.4

Table 2.5.4-27—{Recommended Values For Elastic Modulus (SI Units)}

(Page 2 of 2)

Unit		E [MPa]						Observations
		Method 1	Method 2	Method 3	Method 4	Method 5	Rec.	
OVERBURDEN	Modified Loess	31	80	60	-	-	31	Method 1: $E = 500\ s_u$ (Bowles 1996) Method 2: $E = K\ s_{u\ (Bowles\ 1996)}$ $K = 4200 - 142.54I_p + 1.73I_p^2 - 0.0071I_p^3$ Method 3: FWHA (2002), SPT Correlation
	Accretion-Gley	29	47	61	-	-	29	
	Glacial Till	50	130	96	-	-	50	
ROCK FORMATIONS	Graydon Chert Conglomerate	-	-	-	194	96	96	For Graydon Chert Conglomerate: - Method 4: Pressuremeter Testing - Method 5: Callaway Plant Unit 1 FSAR Plate Load and Pressuremeter - Dynamic Modulus is 3350 MPa Largest static value recommended Localized tests are too conservative Method 5 for other Rocks - "Free-Free" Testing
	Burlington Formation	-	-	-	-	49620	49620	
	Bushberg Formation	-	-	-	-	49620	49620	
	Snyder Creek Formation	-	-	-	-	47780	47780	
	Callaway Formation	-	-	-	-	69900	69900	
	Cotter-Jefferson City Formation	-	-	-	-	39810	39810	
FILLS	Category 1 Granular Fill	-	-	-	-	120	120	Recommended values (Callaway Plant Unit 1 FSAR)
	Category 1 Granular Backfill	-	-	-	-	120	120	

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FSAR: Tables 2.5.4

Table 2.5.4-28—{Recommended Values For Static Elastic Properties (English Units)}

Unit		E [ksf]	ν	G [ksf]	Observations
OVER BURDEN	Modified Loess	650	0.44	230	See Table 2.5.4-24 for Elastic Modulus determination basis
	Accretion-Gley	600	0.45	210	
	Glacial Till	1050	0.44	360	
ROCK FORMATIONS	Graydon Chert Conglomerate	2000	0.44	700	
	Burlington Formation	1036250	0.39	372510	
	Bushberg Formation	1036250	0.35	384370	
	Snyder Creek Formation	998000	0.42	352510	
	Callaway Formation	1460000	0.37	531450	
	Cotter-Jefferson City Formation	831500	0.33	313040	
FILL	Category 1 Granular Fill	2500	0.35	930	
	Category 1 Granular Backfill	2500	0.35	930	

Table 2.5.4-28—{Recommended Values For Static Elastic Properties (SI Units)}

Unit		E [MPa]	ν	G [MPa]	Observations
OVER BURDEN	Modified Loess	31	0.44	11	See Table 2.5.4-24 for Elastic Modulus determination basis
	Accretion-Gley	29	0.45	10	
	Glacial Till	50	0.44	17	
ROCK FORMATIONS	Graydon Chert Conglomerate	96	0.44	34	
	Burlington Formation	49620	0.39	17840	
	Bushberg Formation	49620	0.35	18400	
	Snyder Creek Formation	47780	0.42	16880	
	Callaway Formation	69900	0.37	25450	
	Cotter-Jefferson City Formation	39810	0.33	14990	
FILL	Category 1 Granular Fill	120	0.35	45	
	Category 1 Granular Backfill	120	0.35	45	

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Table 2.5.4-29—{Recommended Values For Dynamic Elastic Properties (English Units)}

(Page 1 of 2)

	Unit	Depth [ft]		V_s [fps]	V_p [fps]	ρ [$p_m$cf]	ν	G_{max} [ksf]	E_{max} [ksf]	DS_o [%]	DP_o [%]	Observations
OVERBURDEN	Modified Loess	0.0	10.4	610	3000	130	0.44	1510	4350	1.90	(1)	- Velocities determined from best estimate soil profile - For rocks, Poisson's Ratio determined from velocity ratio squared - Initial Shear Damping (DSo): determined from "Free-Free" Testing for Rocks, and RCTS Testing for Overburden and Graydon Chert Conglomerate - Initial Compressional Damping (Dpo): determined from "Free-Free" Testing for Rocks, and RCTS Testing and Poisson's Ratio for Overburden and Graydon Chert Conglomerate - (1) Not measured, not applicable for site response analysis $G_{max} = \rho V_s^2$ $E_{max} = 2(1+\nu)G_s$
	Accretion-Gley	10.4	28.3	670	4370	130	0.45	1790	5190	1.90	(1)	
	Glacial Till	28.3	37.5	820	5260	130	0.44	2690	7750	1.90	(1)	
ROCK FORMATIONS	Graydon Chert Conglomerate	37.5	68.9	2340	6930	142	0.44	24150	69340	3.50	(1)	
	Burlington/Bushberg Formation	68.9	75.6	4050	9570	160	0.39	81580	226940	0.48	0.85	
	Snyder Creek Formation 1	75.6	86.0	6050	12530	161	0.35	183010	493390	0.48	1.00	
	Snyder Creek Formation 2	86.0	105.8	3710	9760	161	0.42	68970	195260	0.48	1.00	
	Callaway Formation	105.8	143.6	8360	18610	167	0.37	362300	995300	0.20	0.36	
	Cotter-Jefferson City Formation 1	143.6	176.4	6950	13740	157	0.33	235510	625560	1.41	2.82	
	Cotter-Jefferson City Formation 2	176.4	2218.0	8320	14690	157	0.26	337510	853130	1.41	2.82	
FILLS	Category 1 Granular Fill/Backfill	0.0	10.0	600	1250	140	0.35	1570	4240	1.50	(1)	Suggested values, RCTS and Proctor Tests Pending NA: Not Applicable
		10.0	20.0	800	1670	140	0.35	2780	7510	1.50	(1)	
		> 20.0		1000	2100	140	0.35	4350	11750	1.50	(1)	

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FSAR: Tables 2.5.4

Table 2.5.4-29—{RECOMMENDED VALUES FOR DYNAMIC ELASTIC PROPERTIES (SI Units)}

(Page 2 of 2)

	Unit	Depth [m]		V_s [m/s]	V_p [m/s]	ρ [kg/m^3]	ν	G_{max} [MPa]	E_{max} [Mpa]	DS_o [%]	DP_o [%]	Observations
OVERBURDEN	Modified Loess	0.0	3.2	190	910	2080	0.44	75	220	1.90	(1)	- Velocities determined from best estimate soil profile - For rocks, Poisson's Ratio determined from velocity ratio squared - Initial Shear Damping (DSo): determined from "Free-Free" Testing for Rocks, and RCTS Testing for Overburden and Graydon Chert Conglomerate - Initial Compressional Damping (Dpo): determined from "Free-Free" Testing for Rocks, and RCTS Testing and Poisson's Ratio for Overburden and Graydon Chert Conglomerate - (1) Not measured, not applicable for site response analysis. $G_{max} = \rho V_s^2$ $E_{max} = 2(1+\nu)G_s$
	Accretion-Gley	3.2	8.6	200	1330	2080	0.45	83	240	1.90	(1)	
	Glacial Till	8.6	11.4	250	1600	2080	0.44	130	370	1.90	(1)	
ROCK FORMATIONS	Graydon Chert Conglomerate	11.4	21.0	710	2110	2270	0.44	1144	3280	3.50	(1)	
	Burlington/Bushberg Formation	21.0	23.0	1240	2920	2560	0.39	3936	10950	0.48	0.85	
	Snyder Creek Formation 1	23.0	26.2	1840	3820	2580	0.35	8735	23550	0.48	1.00	
	Snyder Creek Formation 2	26.2	32.3	1130	2980	2580	0.42	3294	9330	0.48	1.00	
	Callaway Formation	32.3	43.8	2550	5670	2670	0.37	17362	47700	0.20	0.36	
	Cotter-Jefferson City Formation 1	43.8	53.8	2120	4190	2510	0.33	11281	29960	1.41	2.82	
	Cotter-Jefferson City Formation 2	53.8	676.2	2540	4480	2510	0.26	16194	40930	1.41	2.82	
FILLS	Category 1 Granular Fill/Backfill	0.0	3.0	180	380	2240	0.35	73	200	1.50	(1)	Suggested values, RCTS and Proctor Tests Pending NA: Not Applicable
		3.0	6.1	240	510	2240	0.35	129	350	1.50	(1)	
		> 6.1		300	640	2240	0.35	202	550	1.50	(1)	

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FSAR: Tables 2.5.4

Table 2.5.4-30—{Soil Conditions For the U.S. EPR Standard Plant (English Units)}

Soil Case No.	Seismic Control Motion Applied	Soil Profile (Half Space or Layered)	Shear-wave Velocity of Soil(1) [fps]
1u	EUR Soft	Half-space	700
2u (A and B)	EUR Soft and Medium	Half-space	1,640
3u	EUR Medium	Half-space	2,625
4u (A and B)	EUR Medium and Hard	Half-space	3,937
5u	EUR Hard	Half-space	5,249
5a	EUR Hard	Half-space	13,123
1n2u	EUR Soft	Linear gradient within a 100 ft layer over a half-space	820 to 1,640
2sn4u	EUR Medium	49 ft uniform layer over a half-space	1,640/3,937
2n3u	EUR Medium	Linear gradient within a 200 ft layer over a half-space	1,640 to 2,625
3r3u	EUR Medium	20 ft uniform layer over 46 ft stiffer layer followed by soil half-space	2,625/5,249/2,625

(1) Shear wave velocities of generic soil profiles are taken as strain-compatible properties.

Table 2.5.4-30—{Soil Conditions for the U.S. EPR Standard Plant (SI Units)}

Soil Case No.	Seismic Control Motion Applied	Soil Profile (Half Space or Layered)	Shear-wave Velocity of Soil(1) [fps]
1u	EUR Soft	Half-space	215
2u (A and B)	EUR Soft and Medium	Half-space	500
3u	EUR Medium	Half-space	800
4u (A and B)	EUR Medium and Hard	Half-space	1,200
5u	EUR Hard	Half-space	1,600
5a	EUR Hard	Half-space	4,000
1n2u	EUR Soft	Linear gradient within a 30 m layer over a half-space	250 to 500
2sn4u	EUR Medium	15 m uniform layer over a half-space	500/1,200
2n3u	EUR Medium	Linear gradient within a 60 m layer over a half-space	500 to 800
3r3u	EUR Medium	6 m uniform layer over 14 m stiffer layer followed by soil half-space	800/1,600/800

(1) Shear wave velocities of generic soil profiles are taken as strain-compatible properties.

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Table 2.5.4-31—{Site Classification (English Units)}

Site Class		Shear Wave Velocity (mean) [fps]	Location of GMRS	Geotechnical Analysis Requirements
Rock Site (Rock is at the ground surface at the site)	**Hard and Firm Rock Site**	Vs>3,500	At top of Rock	Static and dynamic bearing capacity to be verified; no time-dependent settlement analysis required.
	Soft Rock Site	2,400<Vs<3,500	At top of Rock	Static and dynamic bearing capacity to be verified; no time-dependent settlement analysis required.
Thin Soil Site (Rock is generally within 40 to 60 feet of the ground surface and the EPR Nuclear Island is founded on rock)	**Thin Soil Site over Hard or Firm Rock**	At depth below Nuclear Island Basemat, Vs > 3,500	At Top of Outcropping Rock	Static and dynamic bearing capacity to be verified; no time-dependent settlement analysis required.
	Thin Soil Site over Soft Rock	At depth below Nuclear Island Basemat, Vs < 3,500	At Grade Elevation	Static and dynamic bearing capacity to be verified; no time-dependent settlement analysis required.
Soil Sites (Foundation underlain by < 200′ of soil for Shallow and > 200′ feet for Deep)	**Shallow Soil and Deep Soil Sites**	1,000<Vs<3,500 with soil below Nuclear Island Basemat of unlimited thickness	At free-field soil surface	Static and dynamic bearing capacity analysis required; requires verification that time-dependent settlement falls within EPR envelope.

Table 2.5.4-31—{Site Classification (SI Units)}

Site Class		Shear Wave Velocity (mean) [m/s]	Location of GMRS	Geotechnical Analysis Requirements
Rock Site (Rock is at the ground surface at the site)	**Hard and Firm Rock Site**	Vs>1,070	At top of Rock	Static and dynamic bearing capacity to be verified; no time-dependent settlement analysis required.
	Soft Rock Site	730<Vs<1,070	At top of Rock	Static and dynamic bearing capacity to be verified; no time-dependent settlement analysis required.
Thin Soil Site (Rock is generally within 40 to 60 feet of the ground surface and the EPR Nuclear Island is founded on rock)	**Thin Soil Site over Hard or Firm Rock**	At depth below Nuclear Island Basemat, Vs > 1,070	At top of Outcropping Rock	Static and dynamic bearing capacity to be verified; no time-dependent settlement analysis required.
	Thin Soil Site over Soft Rock	At depth below Nuclear Island Basemat, Vs < 1,070	At Grade Elevation	Static and dynamic bearing capacity to be verified; no time-dependent settlement analysis required.
Soil Sites (Foundation underlain by < 60 m of soil for Shallow and > 60 m feet for Deep)	**Shallow Soil and Deep Soil Sites**	305<Vs<1,070 with soil below Nuclear Island Basemat of unlimited thickness	At free-field soil surface	Static and dynamic bearing capacity analysis required; requires verification that time-dependent settlement falls within EPR envelope.

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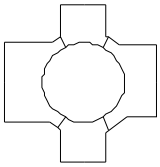
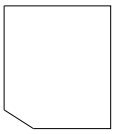
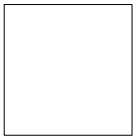
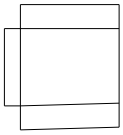
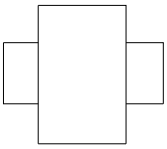
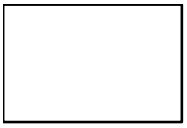
Table 2.5.4-32—{Foundation Elevations (English Units)}

(Page 1 of 2)

STRUCTURE		FOUNDATION DEPTH [ft]	FOUNDATION El.(1) [ft] msl	FOOTPRINT SHAPE	APPROXIMATE AREA [ft²]	Contact Pressure [ksf](2) Average Static	Contact Pressure [ksf](2) Maximum Static	Contact Pressure [ksf](2) Dynamic
Nuclear Island(3)	NI	36.0	809.0		82,120	14.7	22.0	25.0
Nuclear Auxiliary Building	UKA	41.5	803.5		12,580	9.8	22.0	25.0
Radioactive Waste Processing Building	UKS	36.0	809.0		17,210	4.3	22.0	25.0
Ultimate Heat Sink	URB	22.0	823.0		22,120	5.4	22.0	25.0
Emergency Power Generating Building	UBP	5.0	840.0		12,650	3.2	22.0	25.0
Turbine Building	UMA	33.0	812.0		100,000	4.2	10.0	12.0

(1) Plant Grade El. [ft msl] 845
(2) As prescribed by the U.S. EPR FSAR and Foundation Interface Document
(3) Depth shown for UJA (Containment Building). Depth is 41.3 feet around containment

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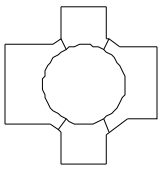
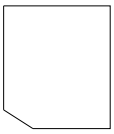
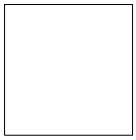
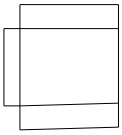
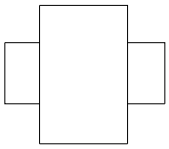
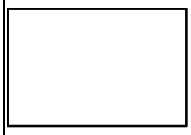
Table 2.5.4-32—{Foundation Elevations (SI Units)}

(Page 2 of 2)

STRUCTURE		FOUNDATION DEPTH [m]	FOUNDATION El.(1) [m] msl	FOOTPRINT SHAPE	APPROXIMATE AREA [m^2]	Contact Pressure [kPa](2) Average Static	Maximum Static	Dynamic
Nuclear Island(3)	NI	11.0	246.6		7,633	700	1,050	1,200
Nuclear Auxiliary Building	UKA	12.7	245.0		1,169	470	1,050	1,200
Radioactive Waste Processing Building	UKS	11.0	246.6		1,600	210	1,050	1,200
Ultimate Heat Sink	URB	6.7	250.9		2,056	260	1,050	1,200
Emergency Power Generation Building	UBP	1.5	256.1		1,176	150	1,050	1,200
Turbine Building	UMA	10.1	247.6		9,295	200	480	570

(1) Plant Grade El. [m msl] 258
(2) As prescribed by the U.S. EPR FSAR and Foundation Interface Document
(3) Depth shown for UJA (Containment Building). Depth is 12.6 m around containment

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Table 2.5.4-33—{Recommended Earth Pressure Coefficients}

	Unit	ϕ' [o]	k_a	k_p	k_o	k_{AE}	k_{PE}	Observations
OVERBURDEN	Modified Loess	23.6	0.43	2.34	0.57	1.42	2.82	NA - Not Applicable k_a - Active Earth Pressure Coefficient k_p - Passive Earth Pressure Coefficient k_o - At Rest Earth Pressure Coefficient k_{AE} - Active Earthquake k_{AP} - Passive Pressure
	Accretion-Gley	20.3	0.48	2.06	0.61	1.37	2.20	
	Glacial Till	23.9	0.42	2.36	0.57	1.32	2.88	
ROCK FORMATIONS	Graydon Chert Conglomerate	NA	NA	NA	NA	NA	NA	
	Burlington Formation	NA	NA	NA	NA	NA	NA	
	Bushberg Formation	NA	NA	NA	NA	NA	NA	
	Snyder Creek Formation	NA	NA	NA	NA	NA	NA	
	Callaway Formation	NA	NA	NA	NA	NA	NA	
	Cotter-Jefferson City Formation	NA	NA	NA	NA	NA	NA	
FILLS	Category 1 Granular Fill	35.0	0.27	3.69	0.43	0.69	7.99	
	Category 1 Granular Backfill	35.0	0.27	3.69	0.43	0.69	7.99	

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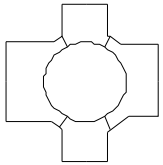
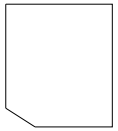
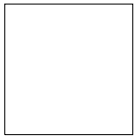
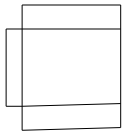
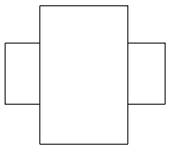
Table 2.5.4-34—{Bearing Capacity (Failure Controlled)}

(Page 1 of 2)

STRUCTURE		FOUNDATION DEPTH [ft]	FOUNDATION El.[1] [ft msl]	FOOTPRINT SHAPE	Simplified B [ft]	Simplified L [ft]	Contact Surface[2]	Bearing Capacity [ksf] Ultimate	Bearing Capacity [ksf] Allowable FS = 3.0
		[m]	[m msl]		B [m]	L [m]			
Nuclear Island[3]	NI	36.0	809.0		286	286	GCC	67.4	22.5
		11.0	246.6		87	87		3,230	1,080
Nuclear Auxiliary Building	UKA	41.5	803.5		105	120	GCC	66.9	22.3
		12.7	245.0		32	37		3,200	1,070
Radioactive Waste Processing Building	UKS	36.0	809.0		130	130	GCC	67.8	22.6
		11.0	246.6		40	40		3,250	1,080
Ultimate Heat Sink	URB	22.0	823.0		124	180	F	66.9	22.3
		6.7	250.9		38	55		3,200	1,070
Emergency Power Generating Building	UBP	5.0	840.0		90	140	F	66.9	22.3
		1.5	256.1		27	43		3,200	1,070

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FSAR: Tables 2.5.4

Table 2.5.4-34—{Bearing Capacity (Failure Controlled)}

(Page 2 of 2)

STRUCTURE		FOUNDATION DEPTH [ft]	FOUNDATION El.(1) [ft msl]	FOOTPRINT SHAPE	Simplified B [ft]	Simplified L [ft]	Contact Surface(2)	Bearing Capacity [ksf] Ultimate	Bearing Capacity [ksf] Allowable FS = 3.0
		[m]	[m msl]		B [m]	L [m]			
Turbine Building	UMA	33.0	812.0		300	330	F	66.2	22.1
		10.1	247.6		91	101		3,170	1,060
10 x 10' Footing on Fill (3' Deep)		3.0	842.0		10	10	F	42.6	14.2
		0.9	256.7		3	3		2,040	680
10 x 10' Footing on In-Situ Clays (Undrained Condition)		3.0	842.0		10	10	S	11.7	3.9
		0.9	256.7		3	3		560	190

(1) Plant Grade El. 845 ft 258 m msl
(2) GCC = Graydon Chert Conglomerate, F = Fill, S = In-Situ Overburden Clay
(3) Depth shown for UJA (Containment Building). Depth is 41.3 ft (12.7) around containment

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Table 2.5.4-35—{Elastic Settlement Analysis By Simplified Approximations (English Units)}

STRUCTURE		FOUNDATION DEPTH [ft]	FOUNDATION El.[1] [ft] msl	Contact Surface[2]	Service Load [ksf]	Settlement Approximation [in] Janbu Center	Janbu Edge	Perloff Center	Perloff Edge	Kay & Cav. Center	Kay & Cav. Edge
Nuclear Island[3]	NI	36.0	809.0	GCC	14.7	1.2	-	2.0	0.8	1.2	0.6
Nuclear Auxiliary Building	UKA	41.5	803.5	GCC	9.8	0.7	-	2.0	0.8	1.1	0.5
Radioactive Waste Processing Building	UKS	36.0	809.0	GCC	4.3	0.3	-	0.8	0.3	0.4	0.2
Ultimate Heat Sink	URB	22.0	823.0	F	5.4	0.4	-	1.1	0.5	0.5	0.2
Emergency Power Generating Building	UBP	5.0	840.0	F	3.2	0.4	-	0.8	0.4	0.6	0.2
Turbine Building	UMA	33.0	812.0	F	5.0	0.3	-	0.5	0.2	0.3	0.2
10 x 10' Footing on Fill (3' Deep)		3.0	842.0	F	3.0	0.0	-	0.1	0.1	0.1	0.0
10 x 10' Footing on In-Situ Clays [4]		3.0	842.0	S	2.0	9.7	-	9.9	9.7	9.8	9.7

1 Plant Grade El. [ft msl] 845.0
2 GCC = Graydon Chert Conglomerate, F = Fill, S = In-Situ Overburden Clay
3 Depth shown for UJA (Containment Building). Depth is 41.3 ft around containment
4 Includes long term settlements

Table 2.5.4-35—{Elastic Settlement Analysis By Simplified Approximations (SI Units)}

STRUCTURE		FOUNDATION DEPTH [m]	FOUNDATION El.[1] [m] msl	Contact Surface[2]	Service Load [kPa]	Settlement Approximation [cm] Janbu Center	Janbu Edge	Perloff Center	Perloff Edge	Kay & Cav. Center	Kay & Cav. Edge
Nuclear Island[3]	NI	11.0	246.6	GCC	700	3.1	-	5.2	2.1	3.0	1.6
Nuclear Auxiliary Building	UKA	12.7	245.0	GCC	470	1.9	-	5.1	2.0	2.7	1.2
Radioactive Waste Processing Building	UKS	11.0	246.6	GCC	210	0.8	-	2.1	0.8	1.1	0.5
Ultimate Heat Sink	URB	6.7	250.9	F	260	1.0	-	2.9	1.1	1.4	0.6
Emergency Power Generating Building	UBP	1.5	256.1	F	150	0.9	-	1.9	1.0	1.5	0.6
Turbine Building	UMA	10.1	247.6	F	240	0.8	-	1.3	0.5	0.7	0.4
3 x 3 m Footing on Fill (3 m Deep)		0.9	256.7	F	140	0.0	-	0.2	0.1	0.2	0.1
3 x 3 m Footing on In-Situ Clays [4]		0.9	256.7	S	100	24.8	-	25.1	24.8	25.0	24.6

1 Plant Grade El. [m msl] 258
2 GCC = Graydon Chert Conglomerate, F = Fill, S = In-Situ Overburden Clay
3 Depth shown for UJA (Containment Building). Depth is 41.3 ft around containment
4 Includes long term settlements

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Table 2.5.4-36—{Detailed Elastic Settlement Analysis (English Units)}

STRUCTURE		D [ft]	El.[1] [ft] msl	Contact Surface[2]	Service Load [ksf]	Max Settlement and Tilt	
						Δ [in]	Tilt [in/50ft]
Nuclear Island[3]	NI	36.0	809.0	GCC	14.7	2.3	0.116
Nuclear Aux. Build.	UKA	41.5	803.5	GCC	9.8	2.2	0.096
Ultimate Heat Sink	URB	22.0	823.0	F	5.4	1.8	0.049
Emerg. Power Build.	UBP	5.0	840.0	F	3.2	1.5	0.023

[1] Plant Grade El. [ft msl] 845.0
[2] GCC = Graydon Chert Conglomerate, F = Fill, S = In-Situ Overburden Clay
[3] Depth shown for UJA (Containment Building). Depth is 41.3 feet around containment

Table 2.5.4-36—{Detailed Elastic Settlement Analysis (SI Units)}

STRUCTURE		D [m]	El.[1] [m] msl	Contact Surface[2]	Service Load [kPa]	Max Settlement and Tilt	
						Δ [cm]	Tilt [cm/100m]
Nuclear Island[3]	NI	11.0	246.6	GCC	700.0	5.8	1.932
Nuclear Aux. Build.	UKA	12.7	245.0	GCC	470.0	5.6	1.6
Ultimate Heat Sink	URB	6.7	250.9	F	260.0	4.6	0.816
Emerg. Power Build.	UBP	1.5	256.1	F	150.0	3.8	0.383

[1] Plant Grade El. [m msl] 257.6
[2] GCC = Graydon Chert Conglomerate, F = Fill, S = In-Situ Overburden Clay
[3] Depth shown for UJA (Containment Building). Depth is 12.7 m around containment

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Figure 2.5.4-1—{Generic Soil Profile}

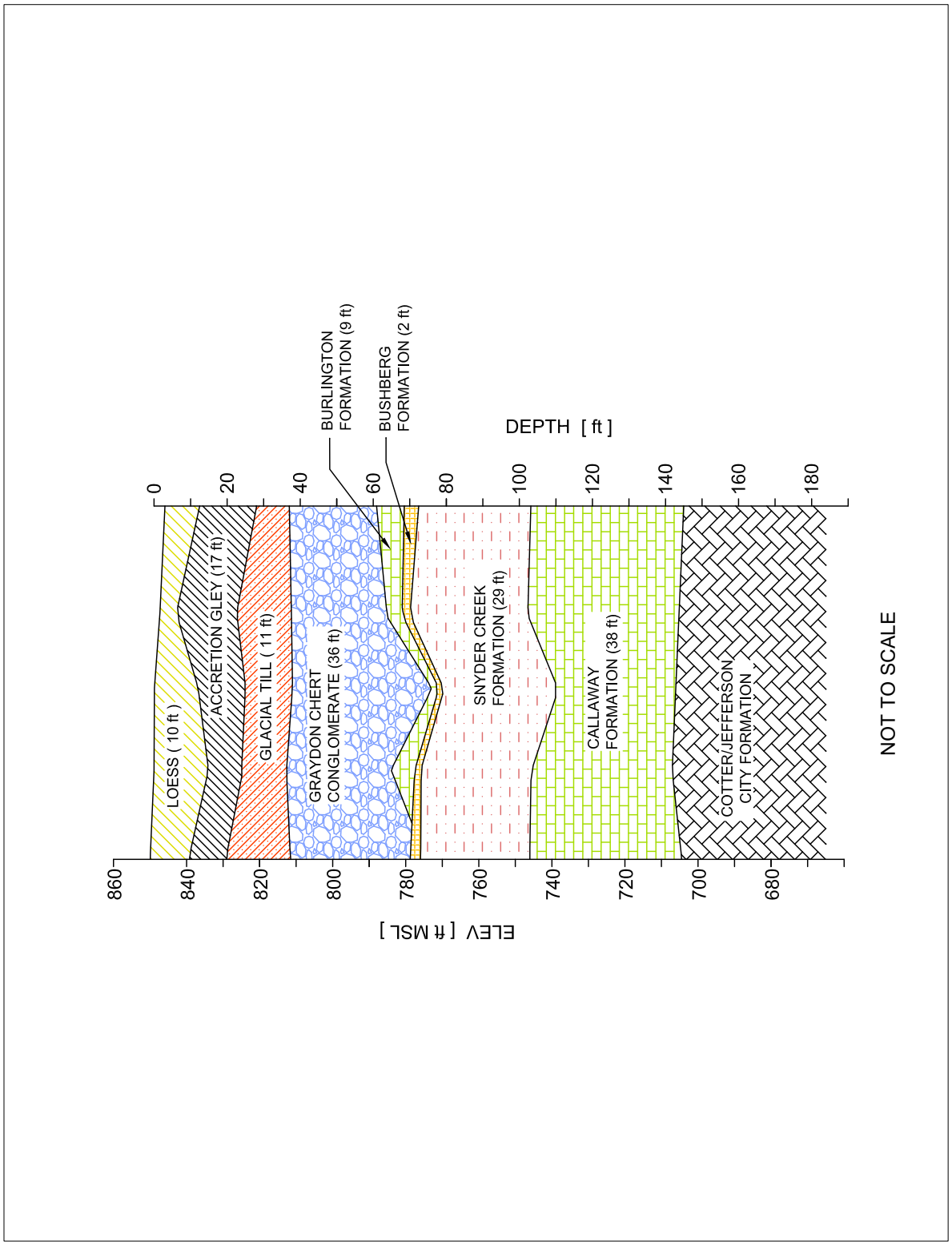

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Figure 2.5.4-2—{Boring Layout Plan}

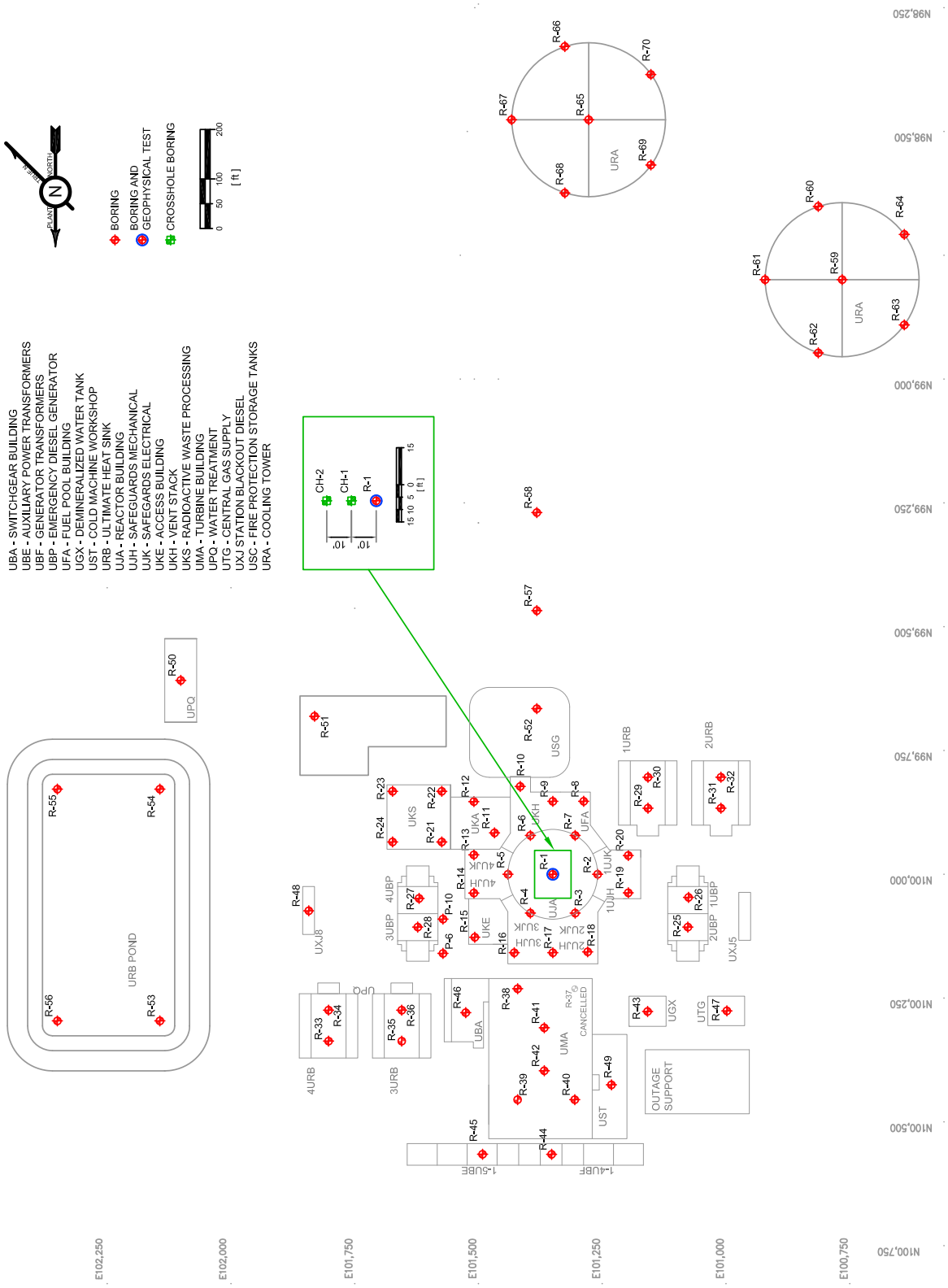

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Figure 2.5.4-3—{Geophysical Exploration Locations}

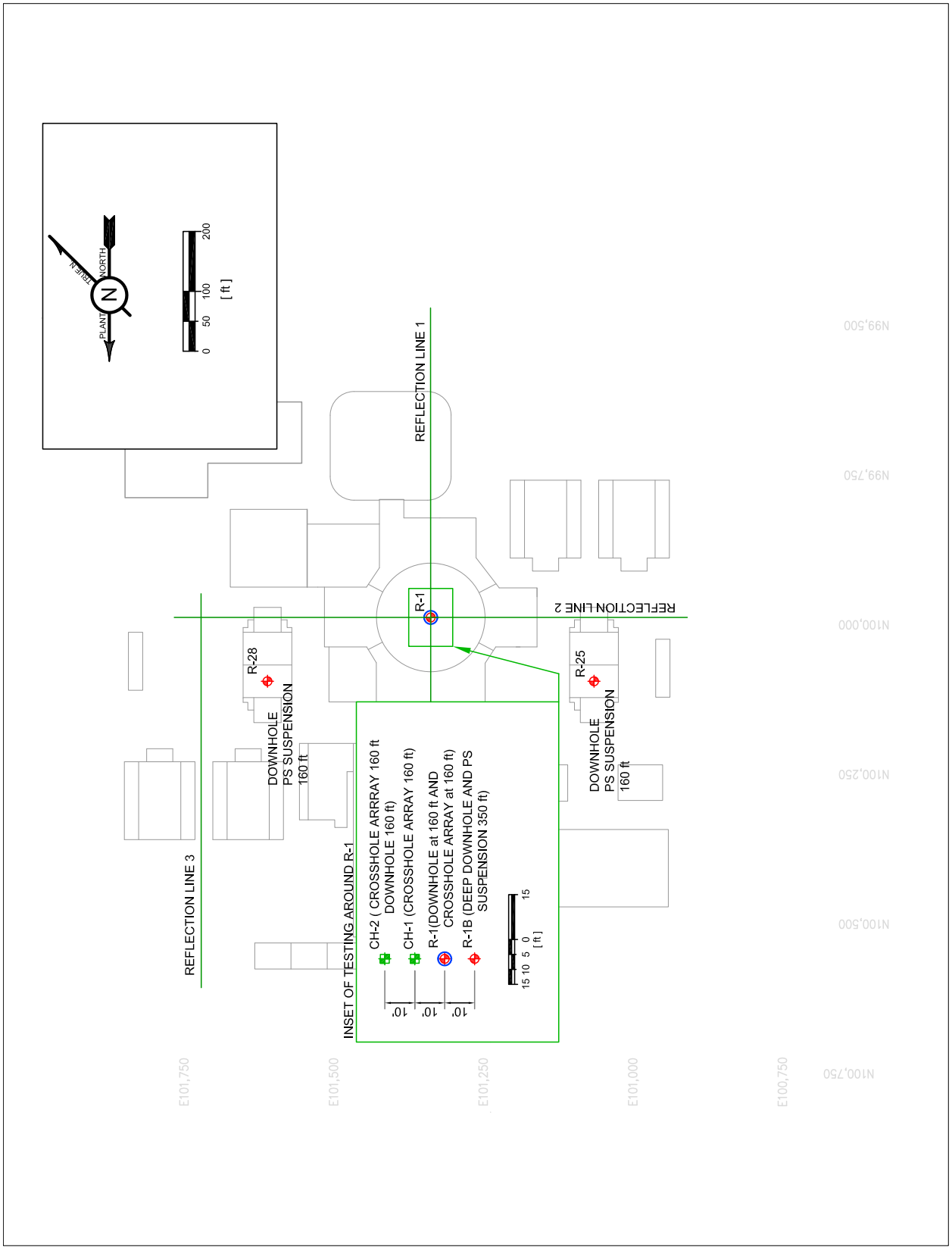

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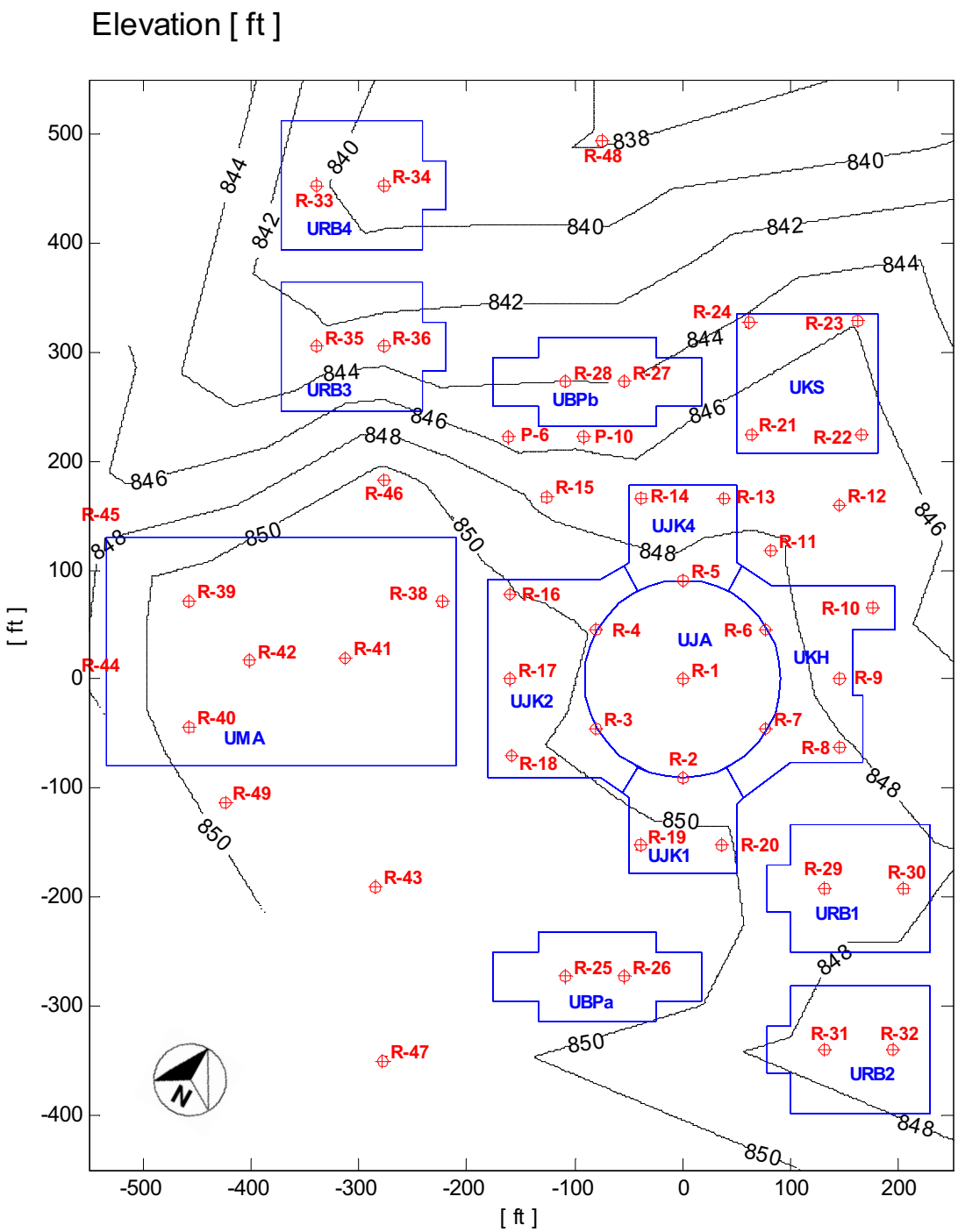
Figure 2.5.4-4—{Surface Elevation Contours}



Callaway Plant Unit 2 Facilities:

UBP Emegency Diesel Generator Building
URB Essential Service Water Cooling Tower Structure
UJA Reactor Building
UJK Safeguard Building
UKH Vent Stack
UKS Radioactive Waste Processing Building
UMA Turbine Building

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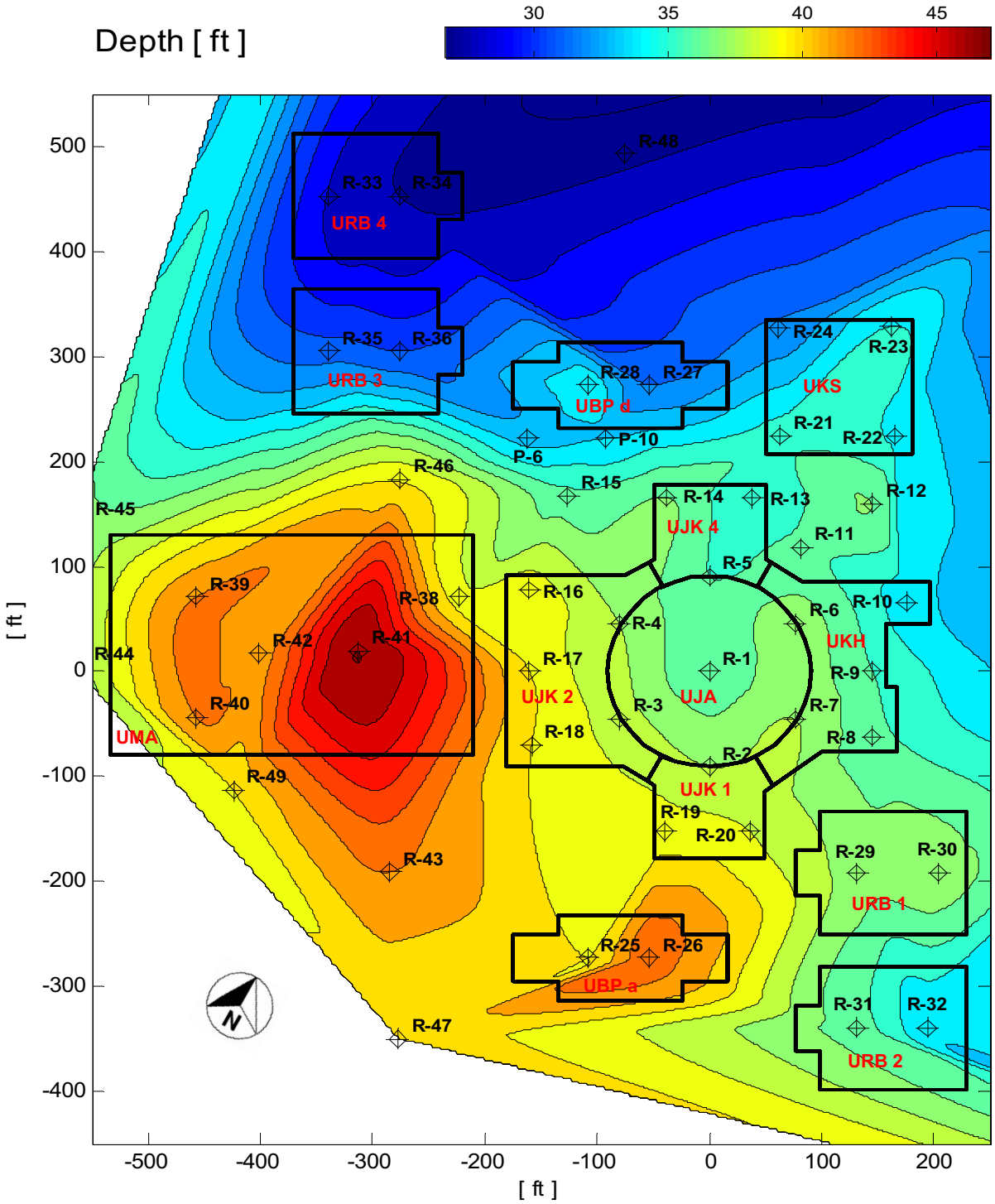
Figure 2.5.4-5—{Depth to Graydon Chert Conglomerate}



Callaway Plant Unit 2 Facilities:

UBP	*Emegency Diesel Generator Building*
URB	*Essential Service Water Cooling Tower Structure*
UJA	*Reactor Building*
UJK	*Safeguard Buildings Electrical*
UKH	*Vent Stack*
UKS	*Radioactive Waste Processing Building*
UMA	*Turbine Building*

Note: Contours have a 1-ft resolution. Values between borings are obtained by linear interpolation

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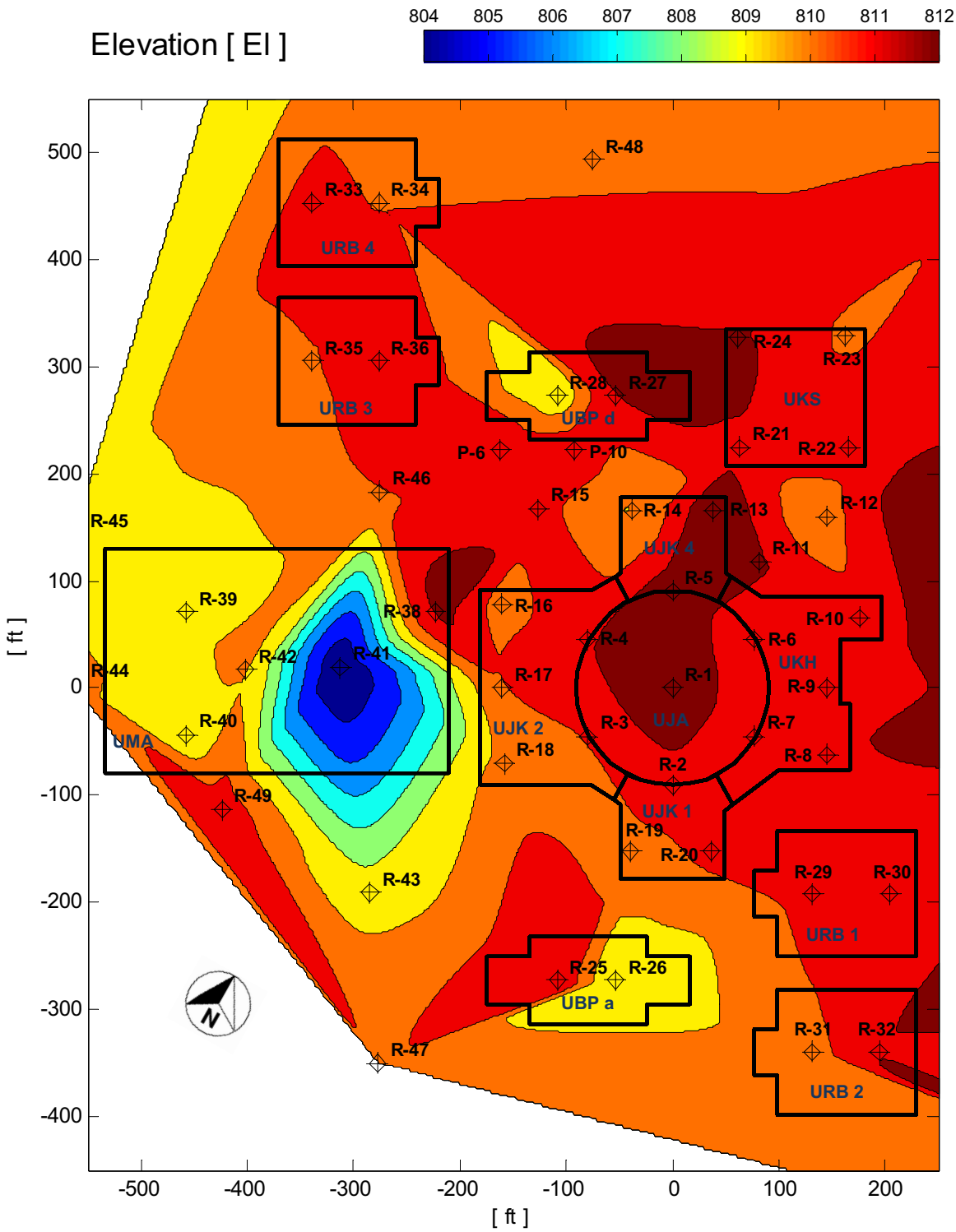
Figure 2.5.4-6—{Graydon Chert Conglomerate (Elevation of Top Surface)}



Note: Contours have a 1-ft resolution. Values between borings are obtained by linear interpolation

Callaway Plant Unit 2 Facilities:

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UJK Safeguard Buildings Electrical
UKH Vent Stack
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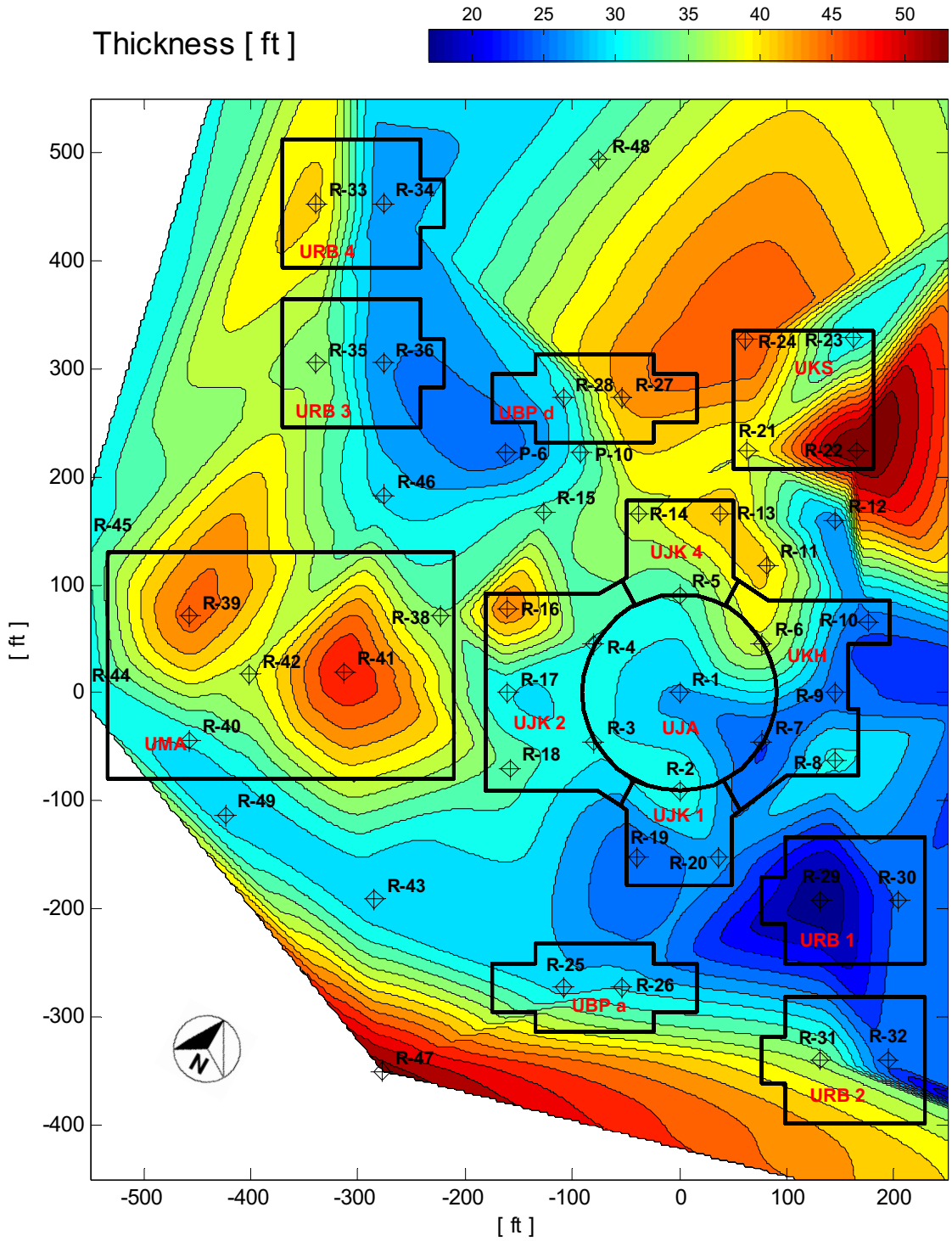
Figure 2.5.4-7—{Graydon Chert Conglomerate (Thickness)}



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UJK Safeguard Buildings Electrical
UKH Vent Stack
UKS Radioactive Waste Processing Building
UMA Turbine Building

Note: Contours have a 2-ft resolution. Values between borings are obtained by linear interpolation

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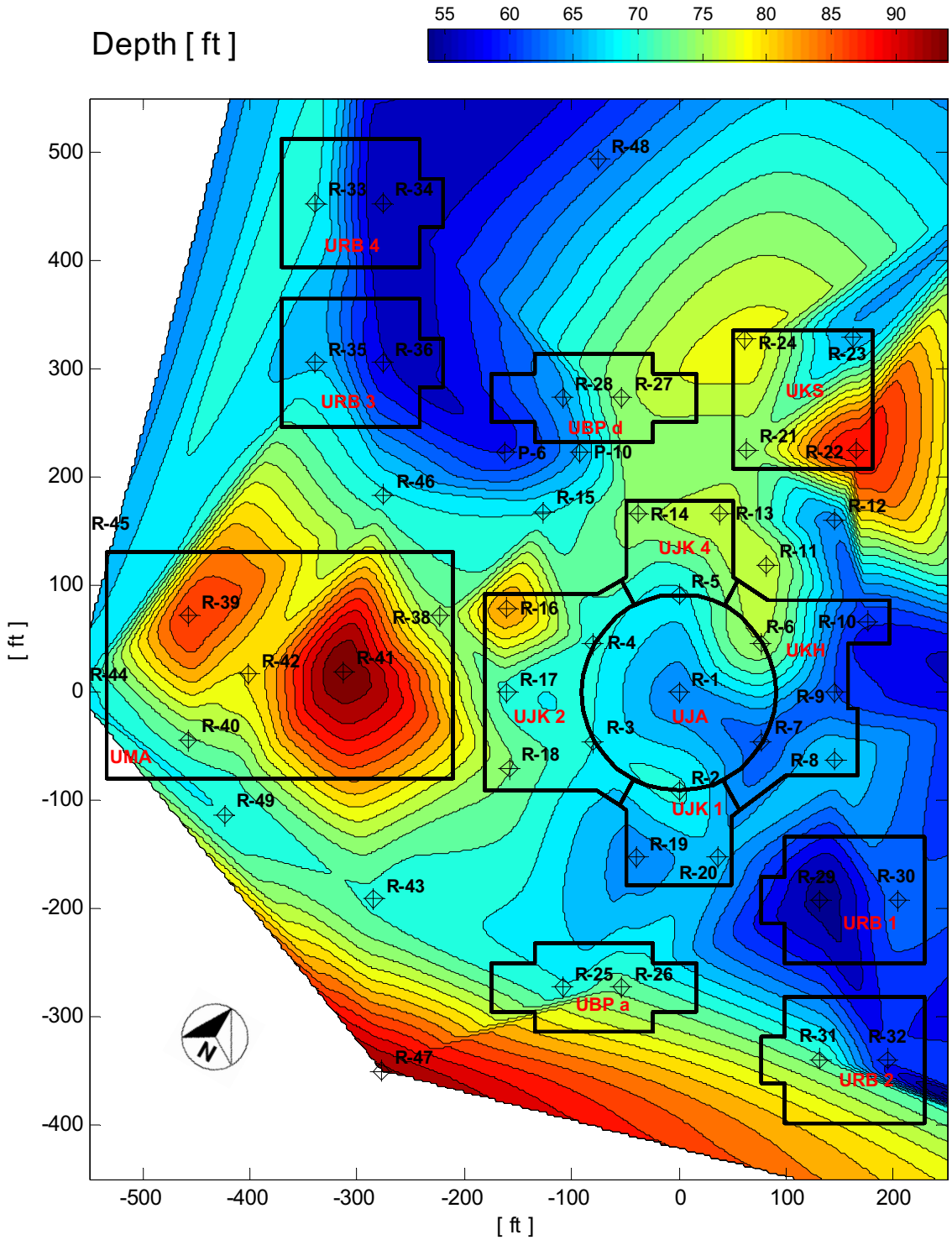
Figure 2.5.4-8—{Depth to Bottom of Graydon Chert Conglomerate}



Note: Contours have a 2-ft resolution. Values between borings are obtained by interpolation

Callaway Plant Unit 2 Facilities:

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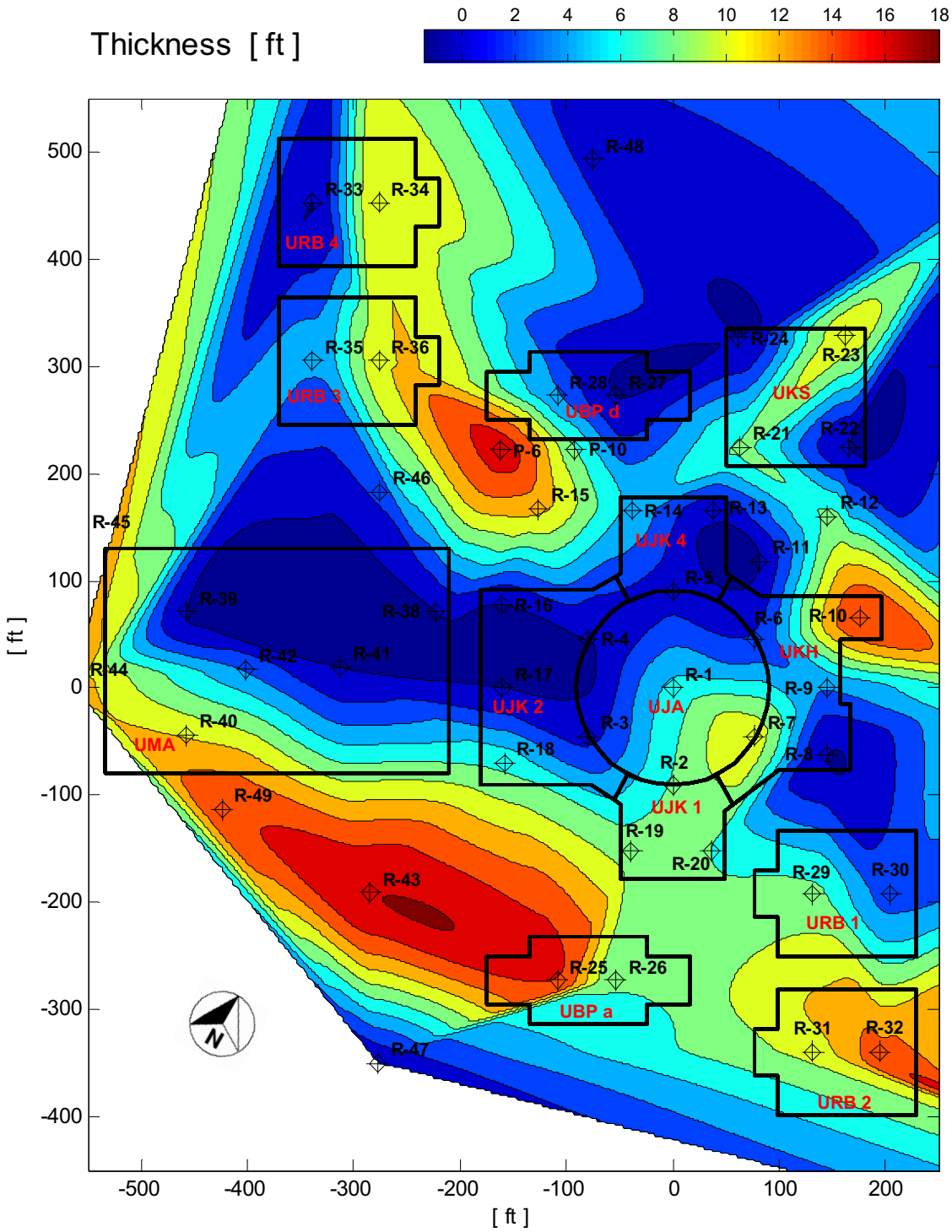
Figure 2.5.4-9—{Burlington Formation (Thickness)}



Note: Contours have a 0.5 ft resolution. Values between borings are obtained by interpolation

Callaway Plant Unit 2 Facilities:

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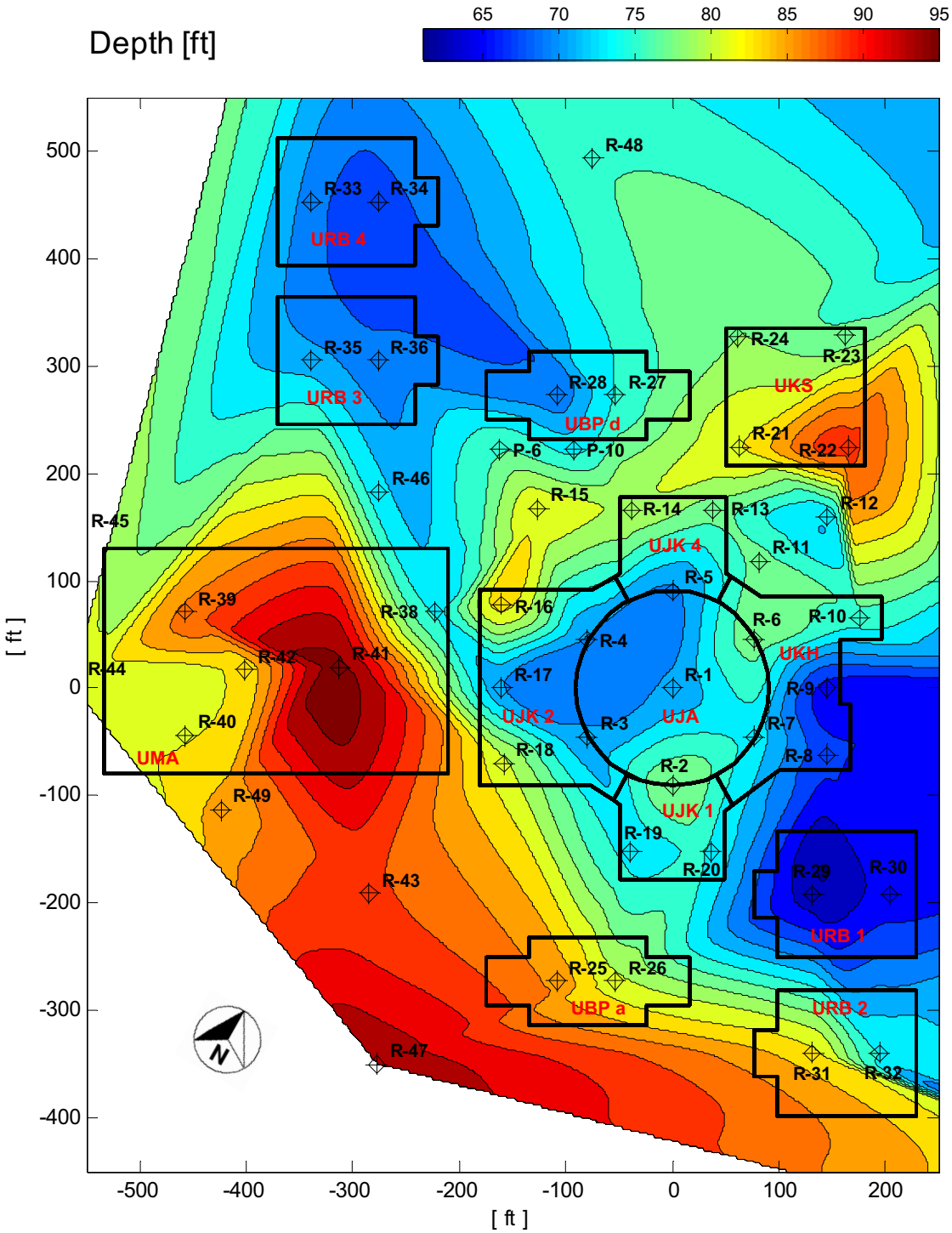
Figure 2.5.4-10—{Bushberg Formation (Depth to Top Surface)}



Note: Contours have a 2 ft resolution. Values between borings are obtained by interpolation

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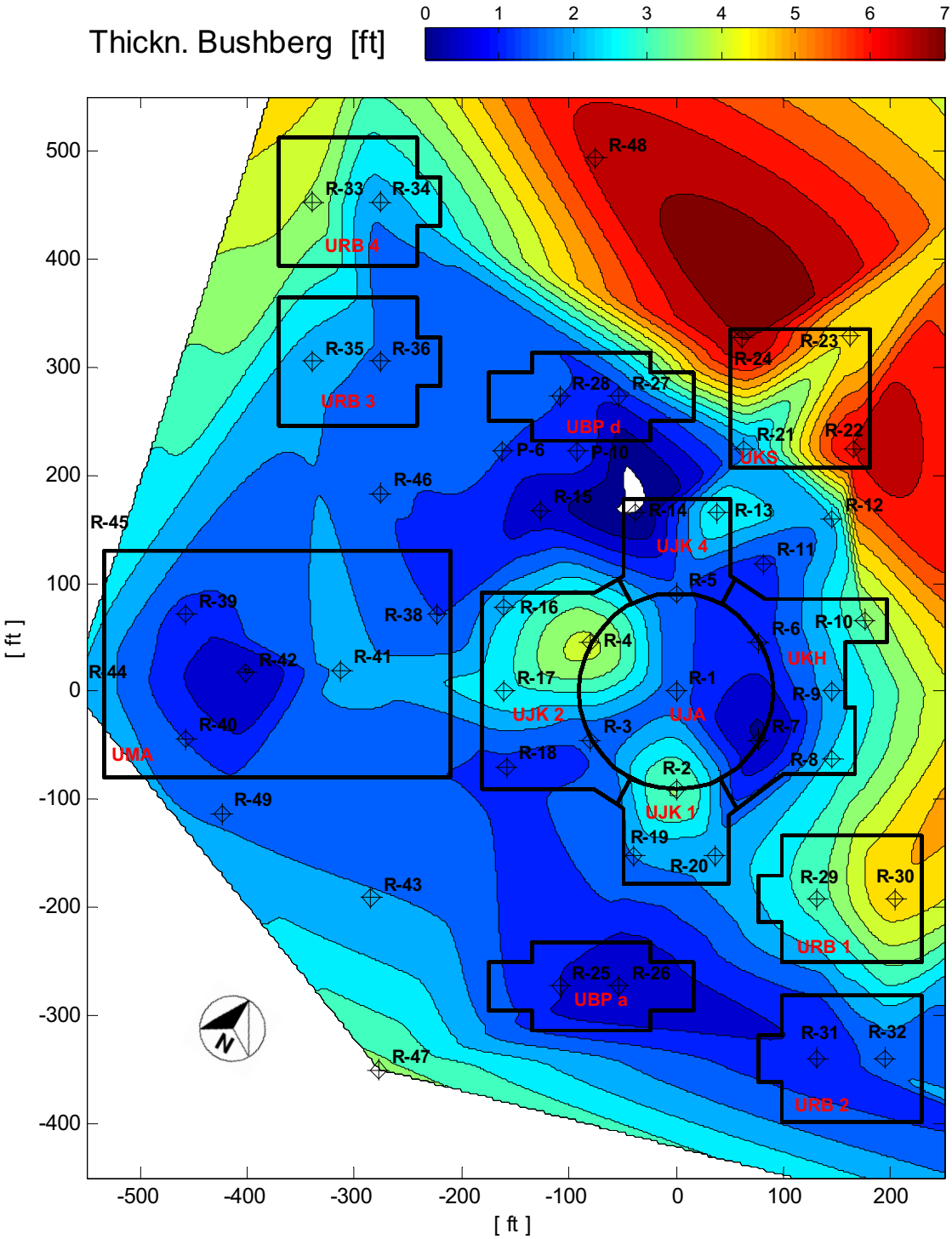
Figure 2.5.4-11—{Bushberg Formation (Thickness)}



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Note: Contours have a 0.5 ft resolution. Values between borings are obtained by interpolation

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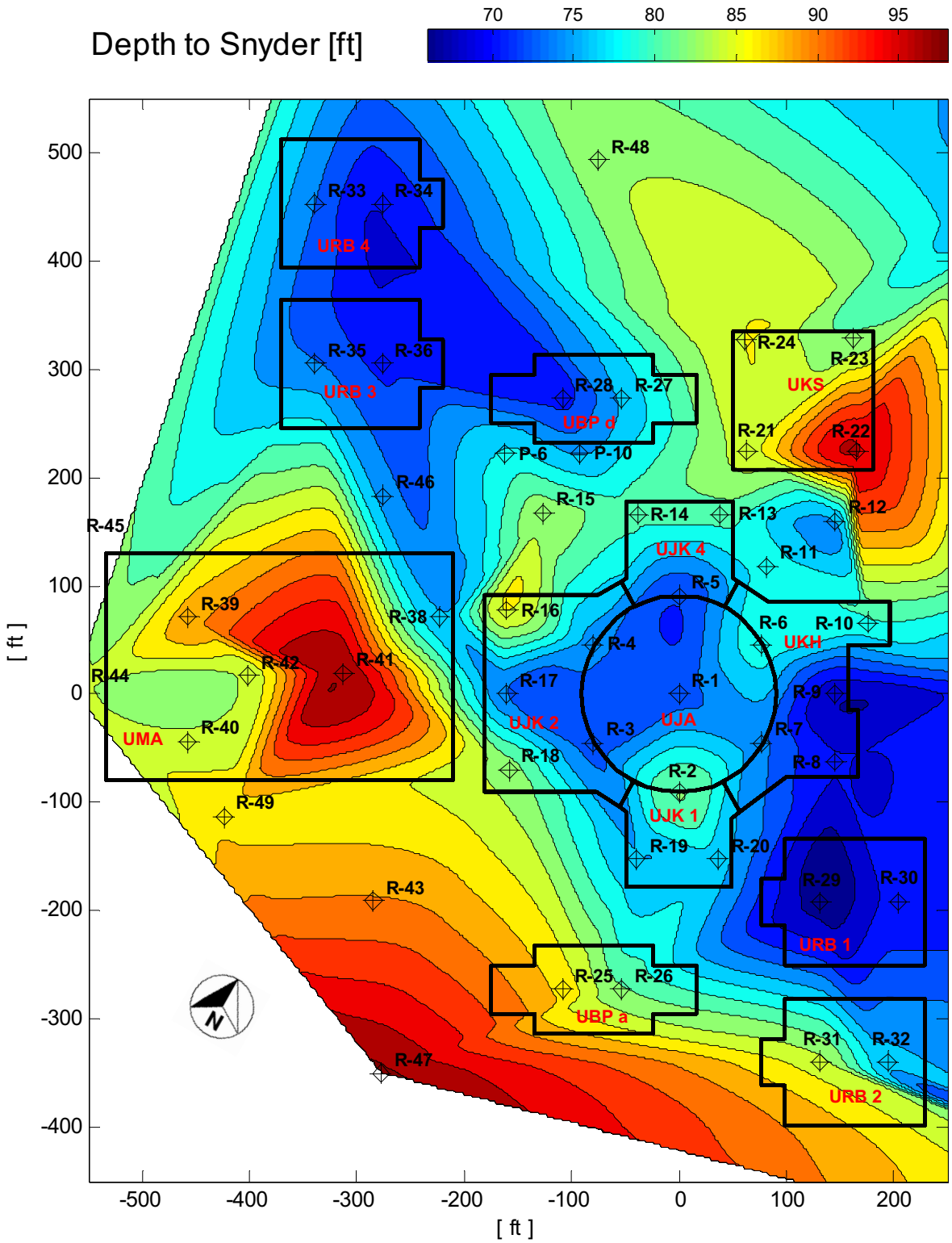
Figure 2.5.4-12—{Snyder Creek Formation (Depth to Top Surface)}



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Note: Contours have a 2 ft resolution. Values between borings are obtained by interpolation

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Figure 2.5.4-13—{SPT Variability}

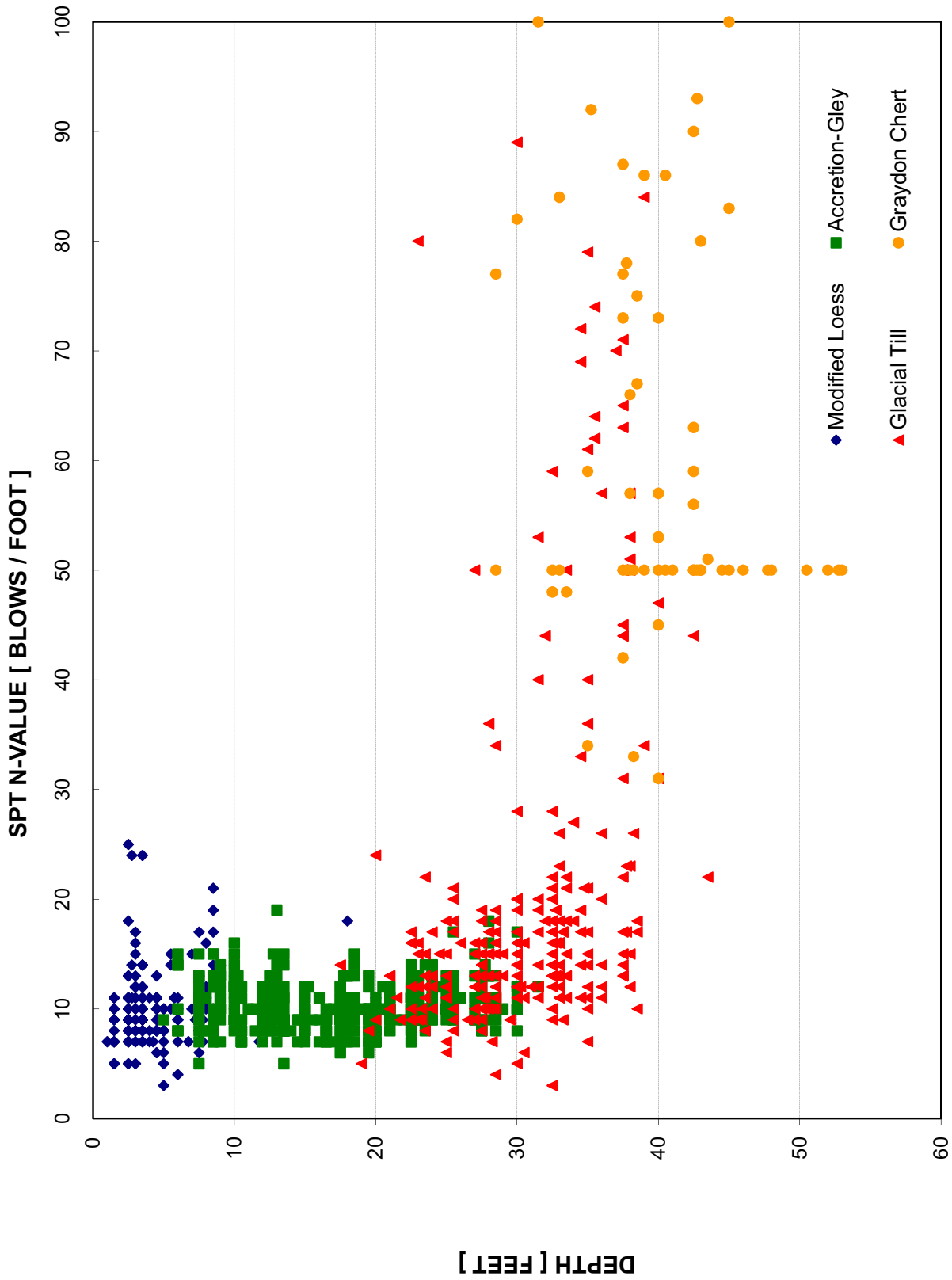

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Figure 2.5.4-14—{Pressuremeter Test Results}

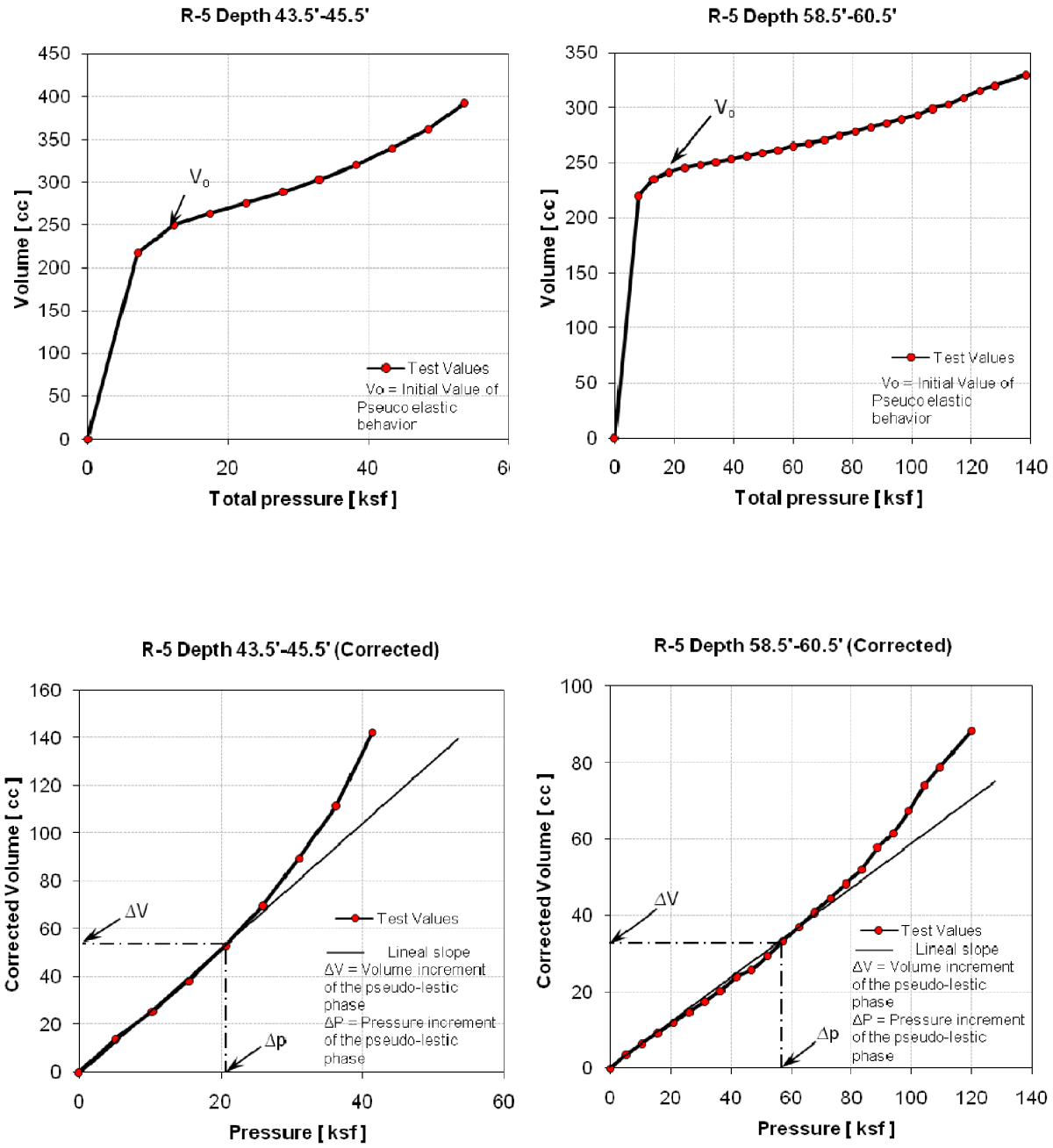

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Figure 2.5.4-15—{Pressuremeter Test Results}

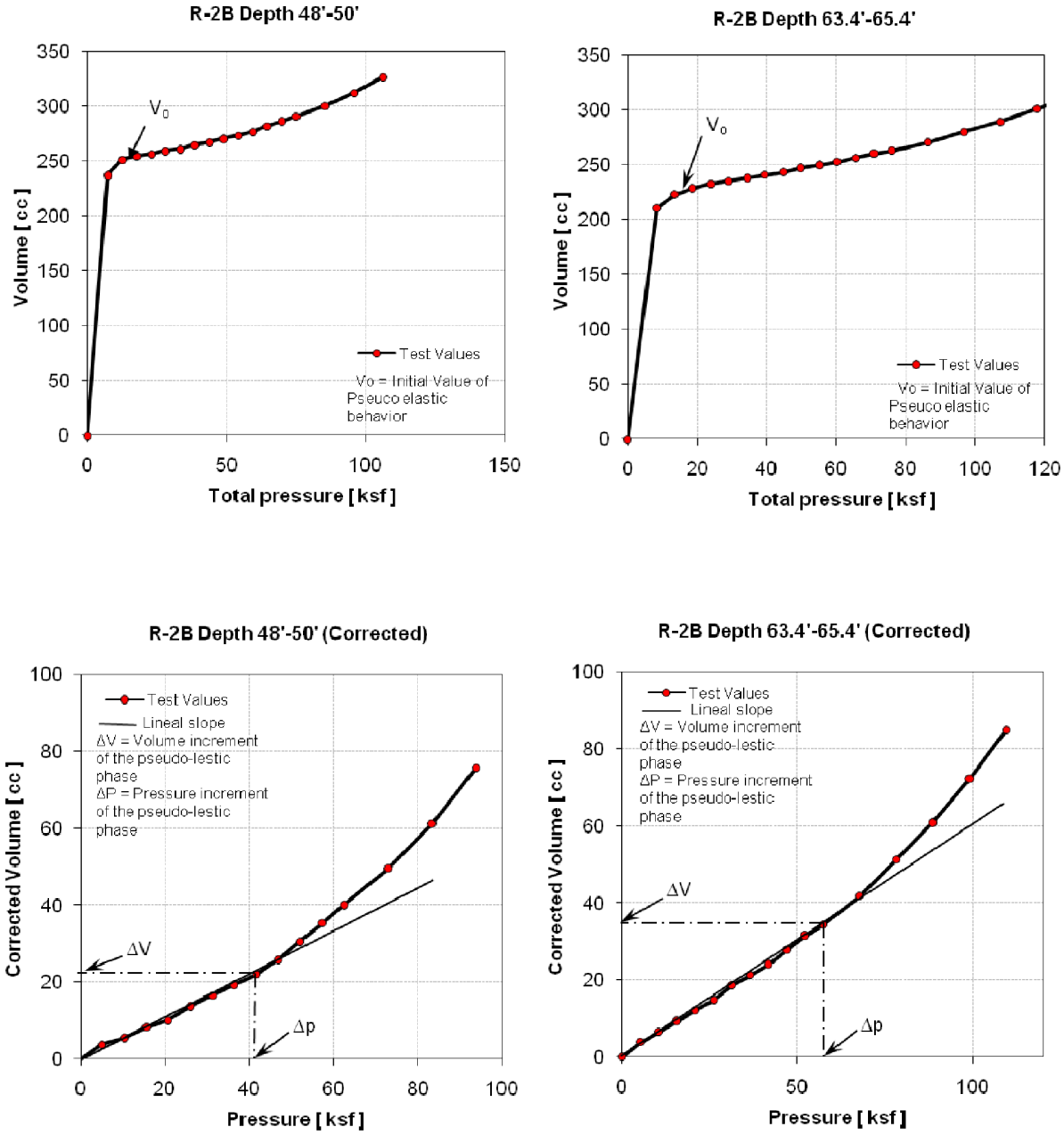

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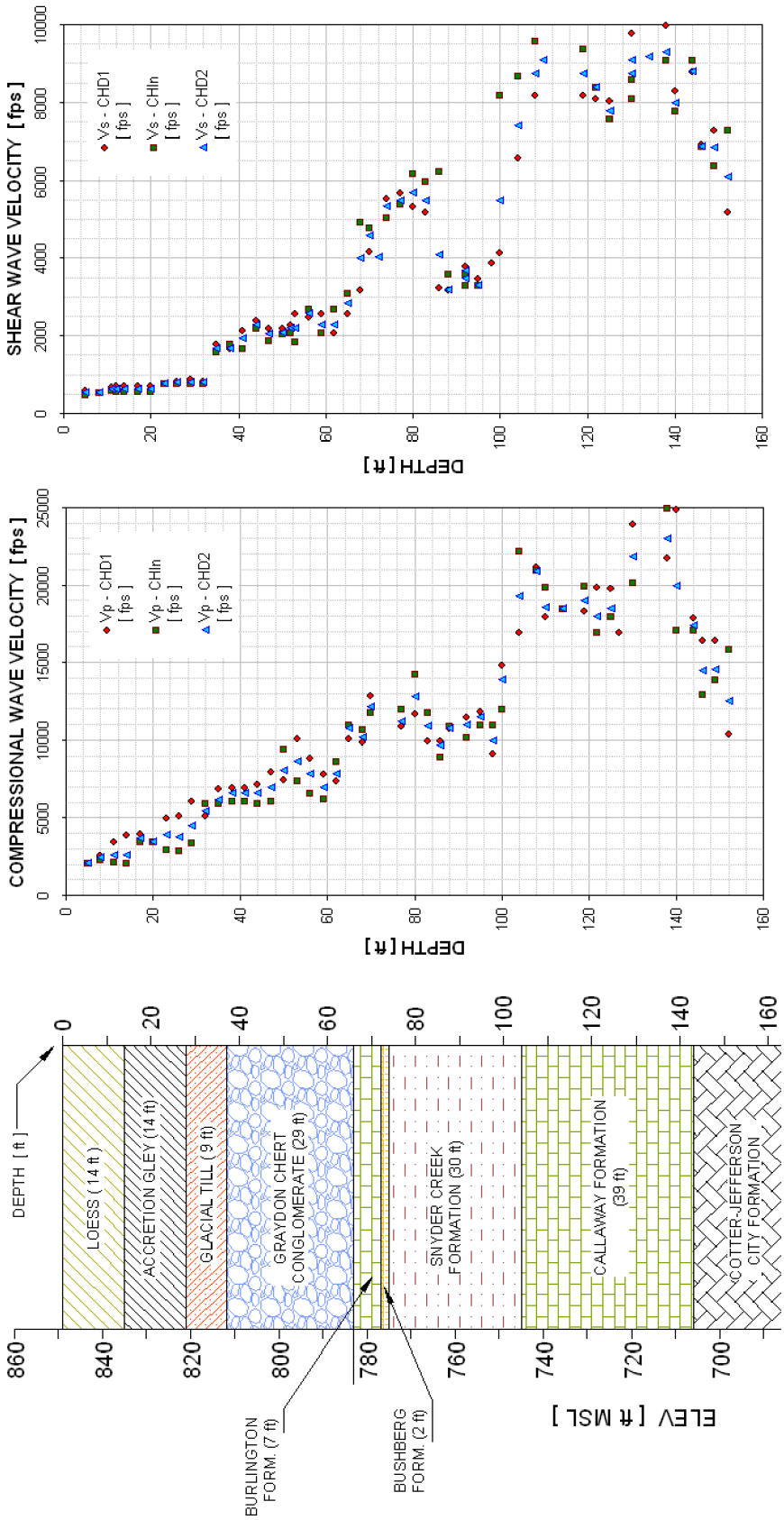
Figure 2.5.4-16—{Geophysical Testing (Cross-Hole)}



Note: Soil profile not to scale

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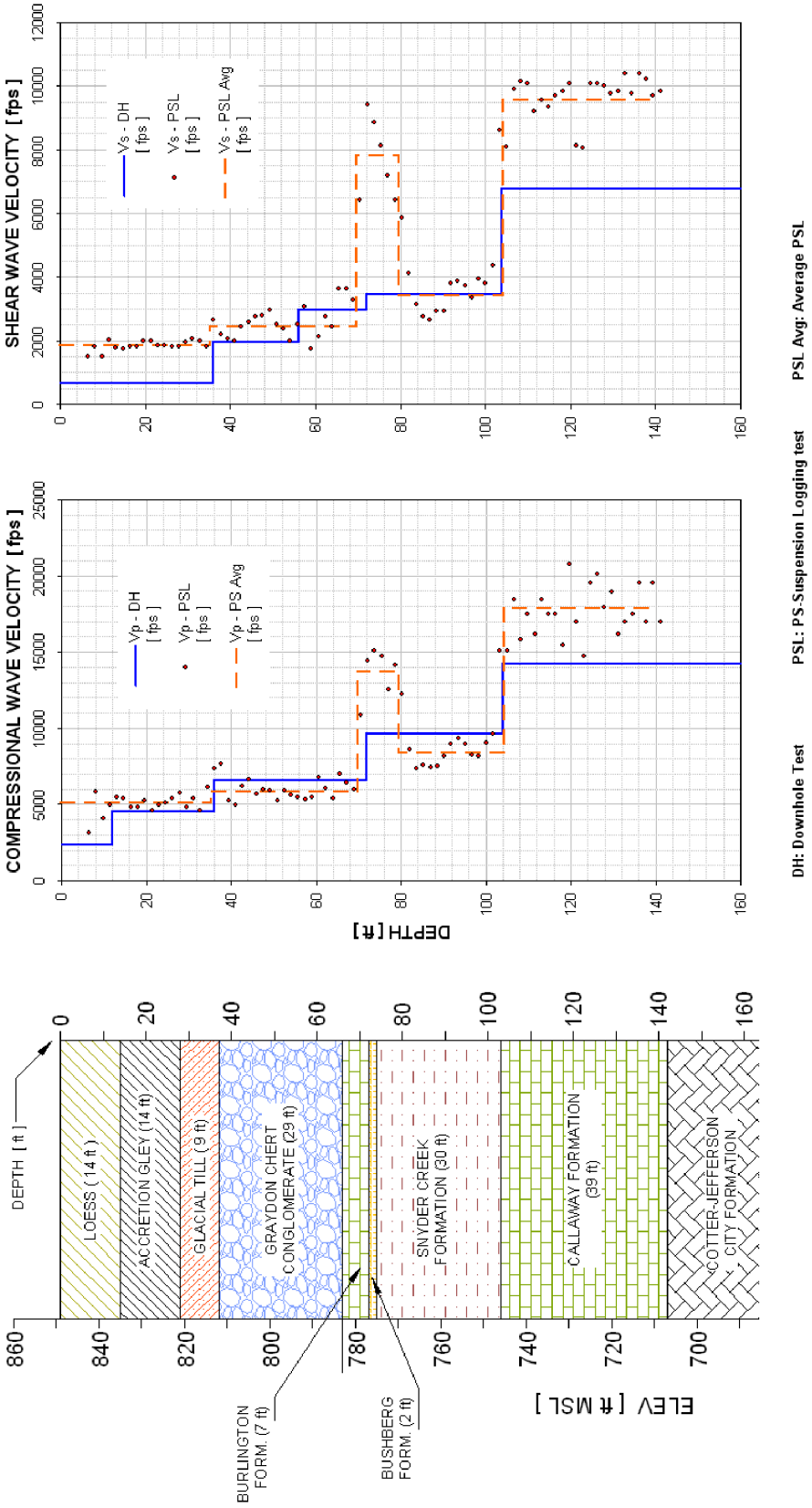
Figure 2.5.4-17—{Geophysical Testing at CH-2}



DH: Downhole Test

PSL: PS-Suspension Logging test

PSL Avg: Average PSL

Note: Soil profile not to scale

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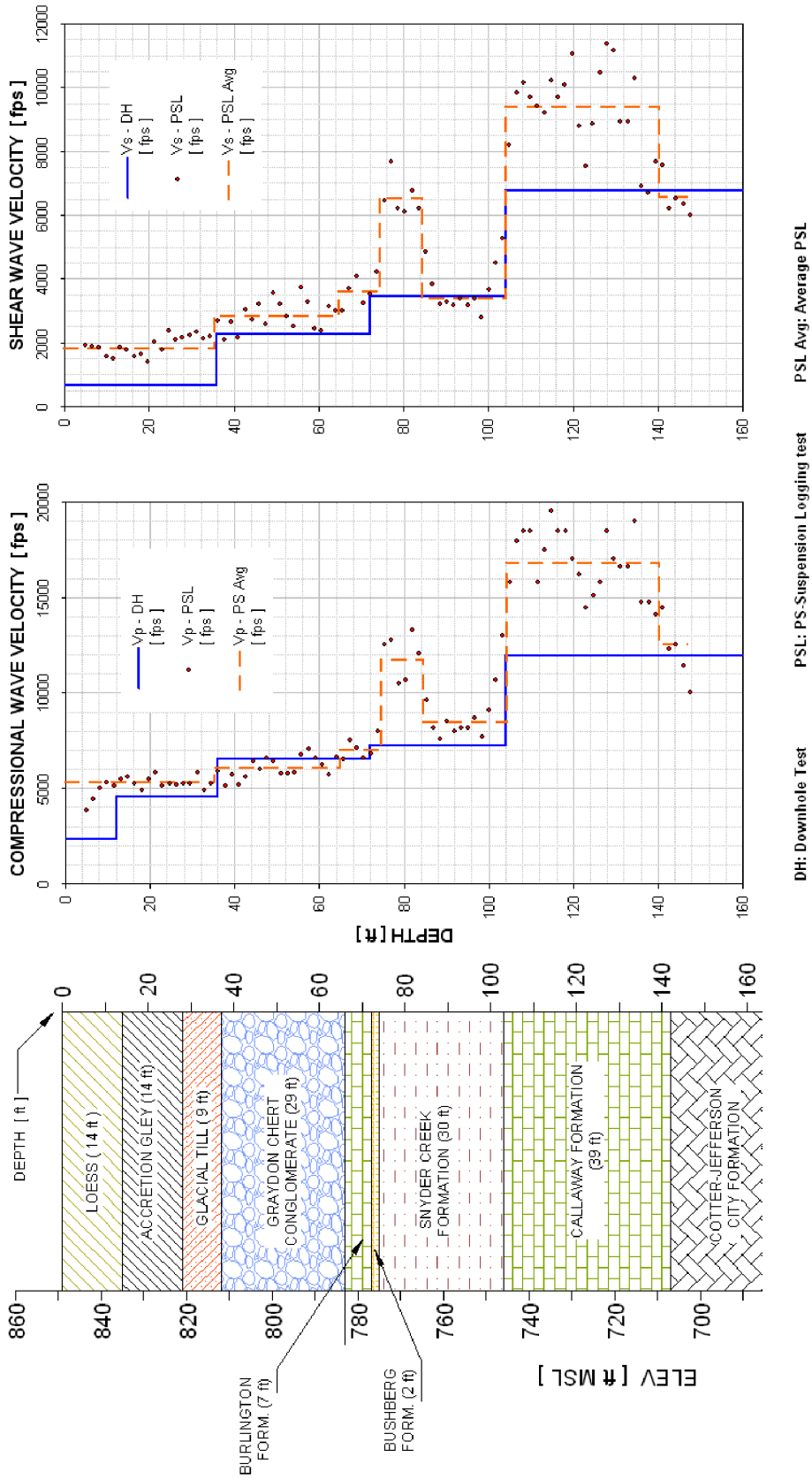
Figure 2.5.4-18—{Geophysical Testing at R1}



DH: Downhole Test

PSL: PS-Suspension Logging test

PSL Avg: Average PSL

Note: Soil profile not to scale

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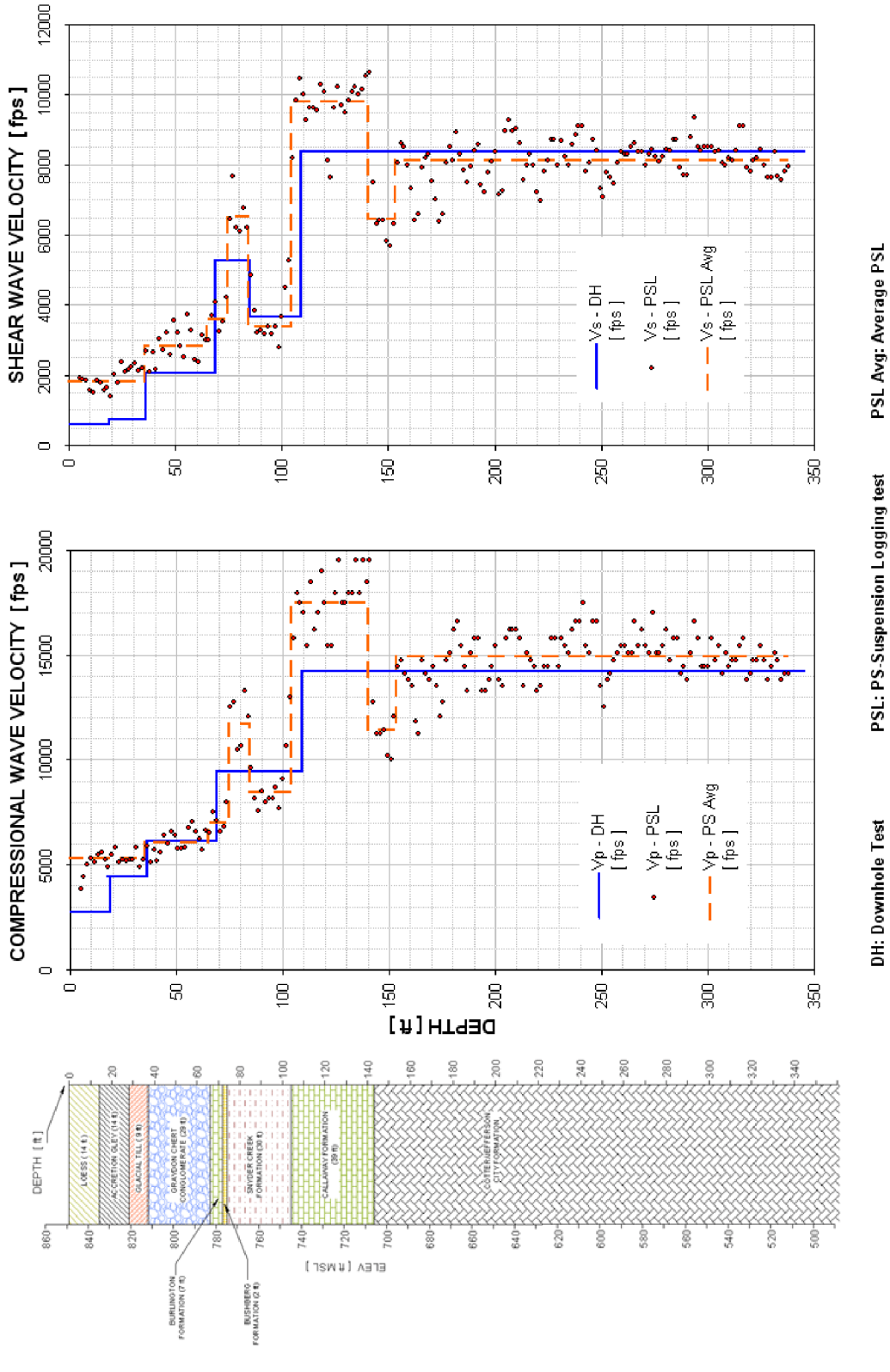
Figure 2.5.4-19—{Geophysical Testing at R1-B}



DH: Downhole Test PSL: PS-Suspension Logging test PSL Avg: Average PSL

Note: Soil profile not to scale

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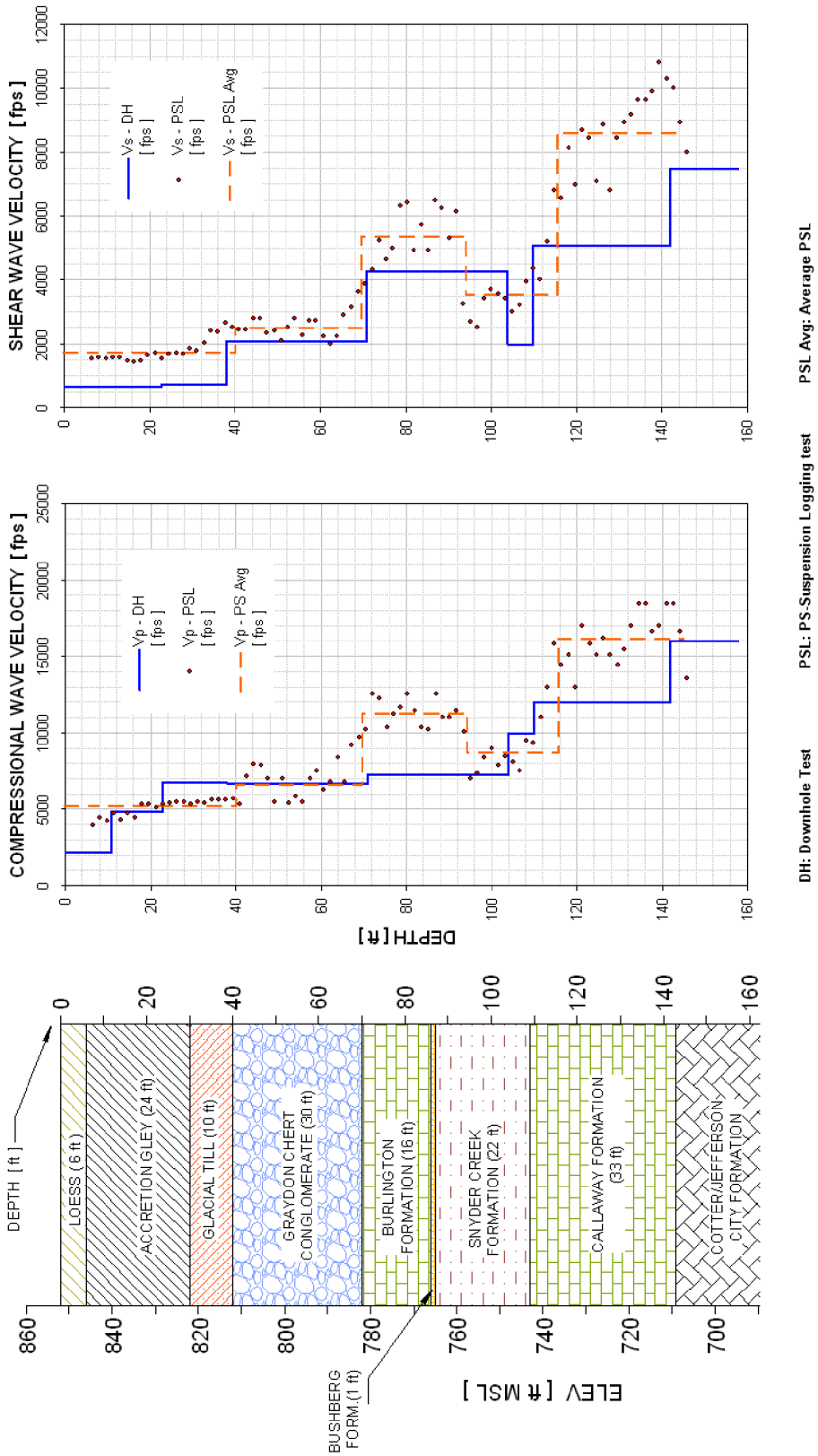
Figure 2.5.4-20—{Geophysical Testing at R-25}



Note: Soil profile not to scale

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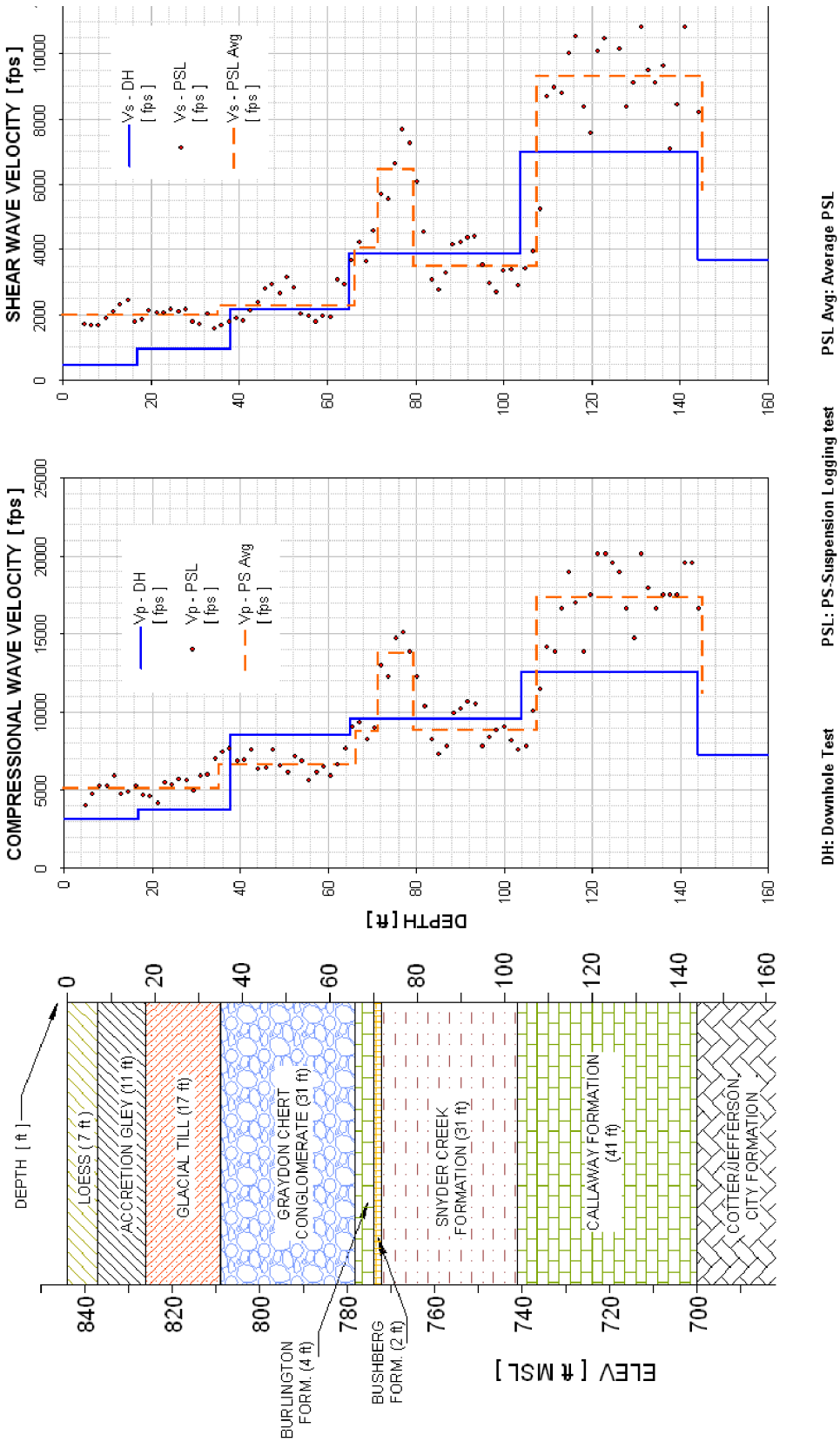
Figure 2.5.4-21—{Geophysical Testing at R-28}



DH: Downhole Test

PSL: PS-Suspension Logging test

PSL Avg: Average PSL

Note: Soil profile not to scale

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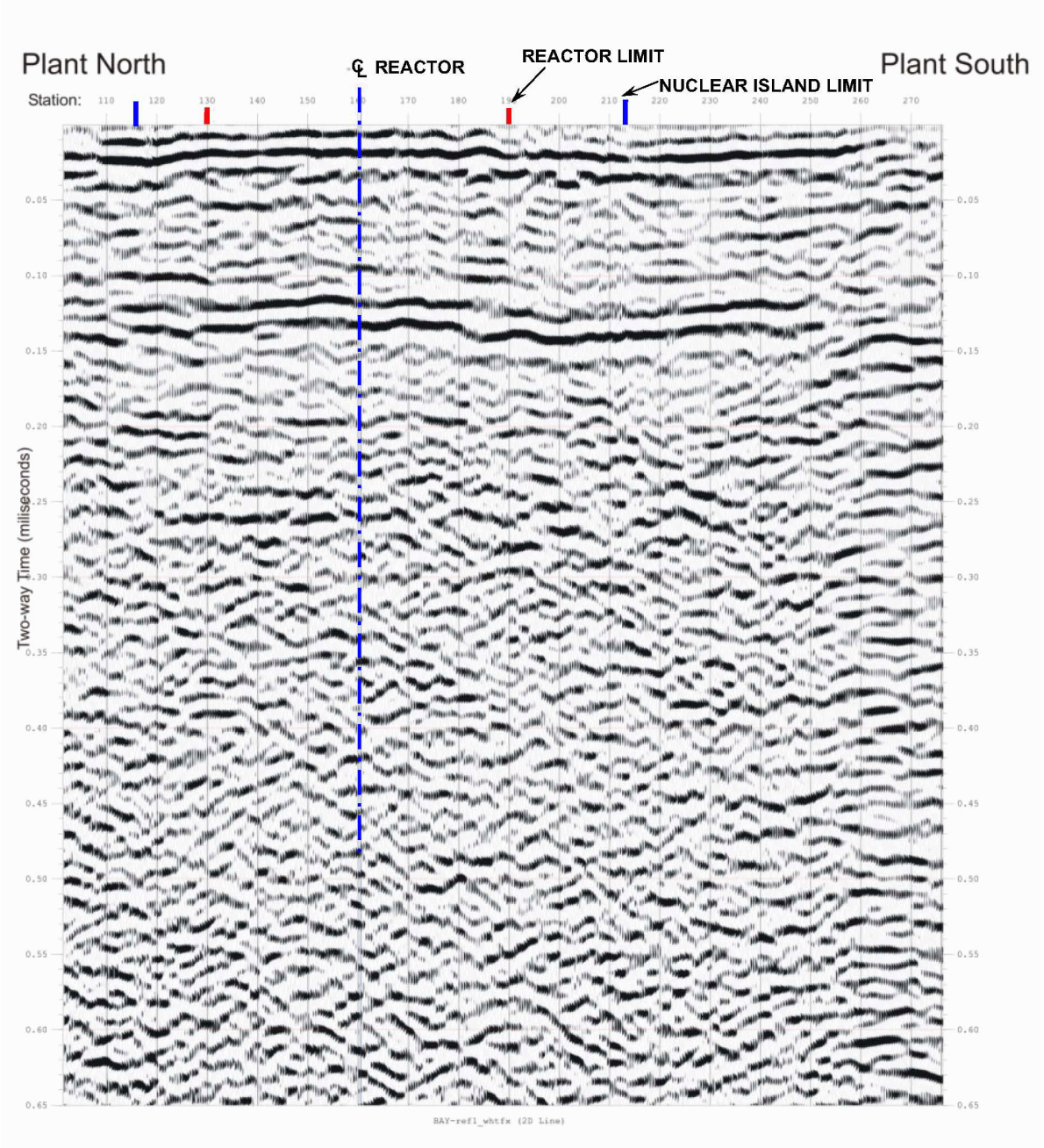
Figure 2.5.4-22—{Reflection Survey (1 of 3)}



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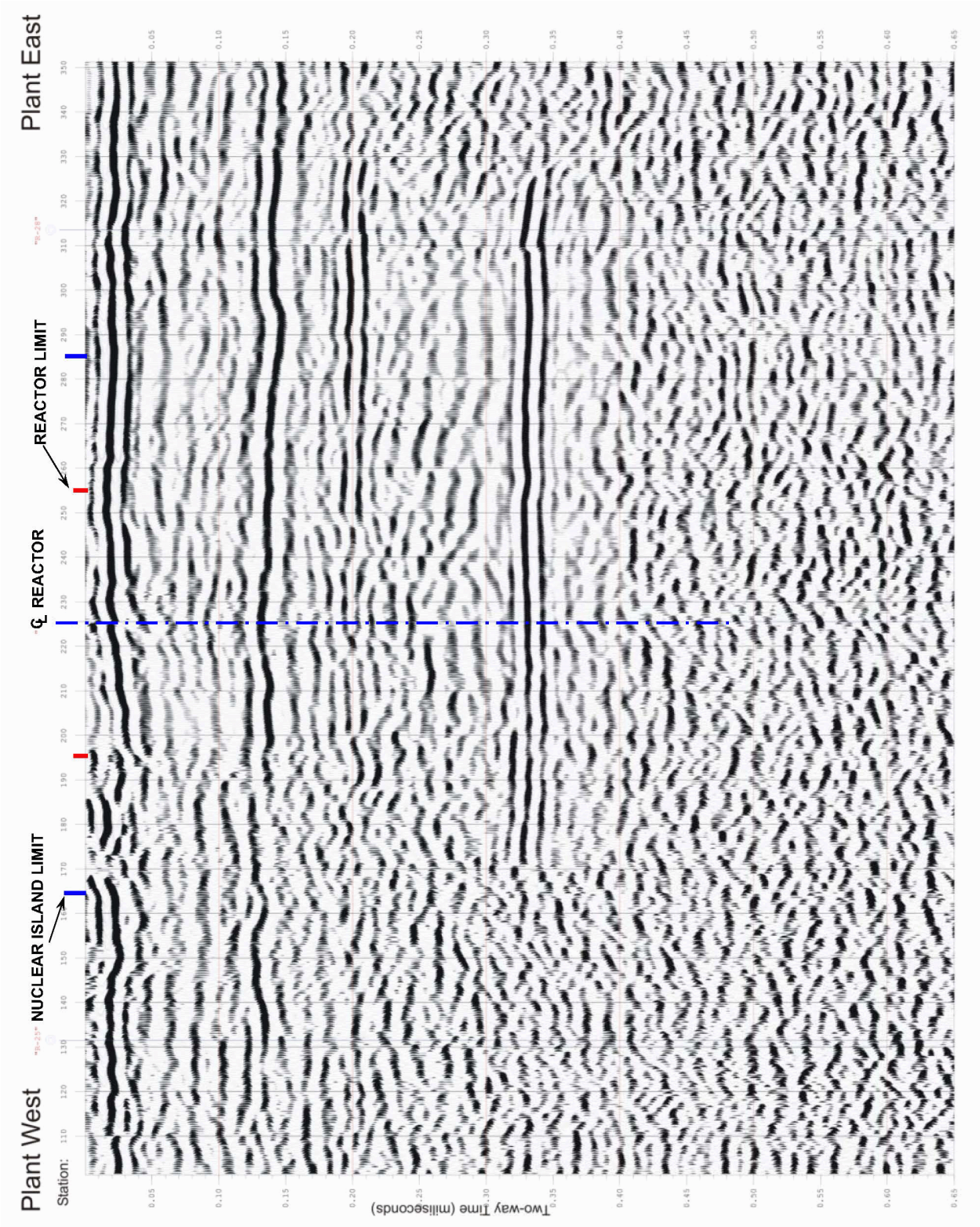

Figure 2.5.4-23—{Reflection Survey (2 of 3)}

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Figure 2.5.4-24—{Reflection Survey (3 of 3)}

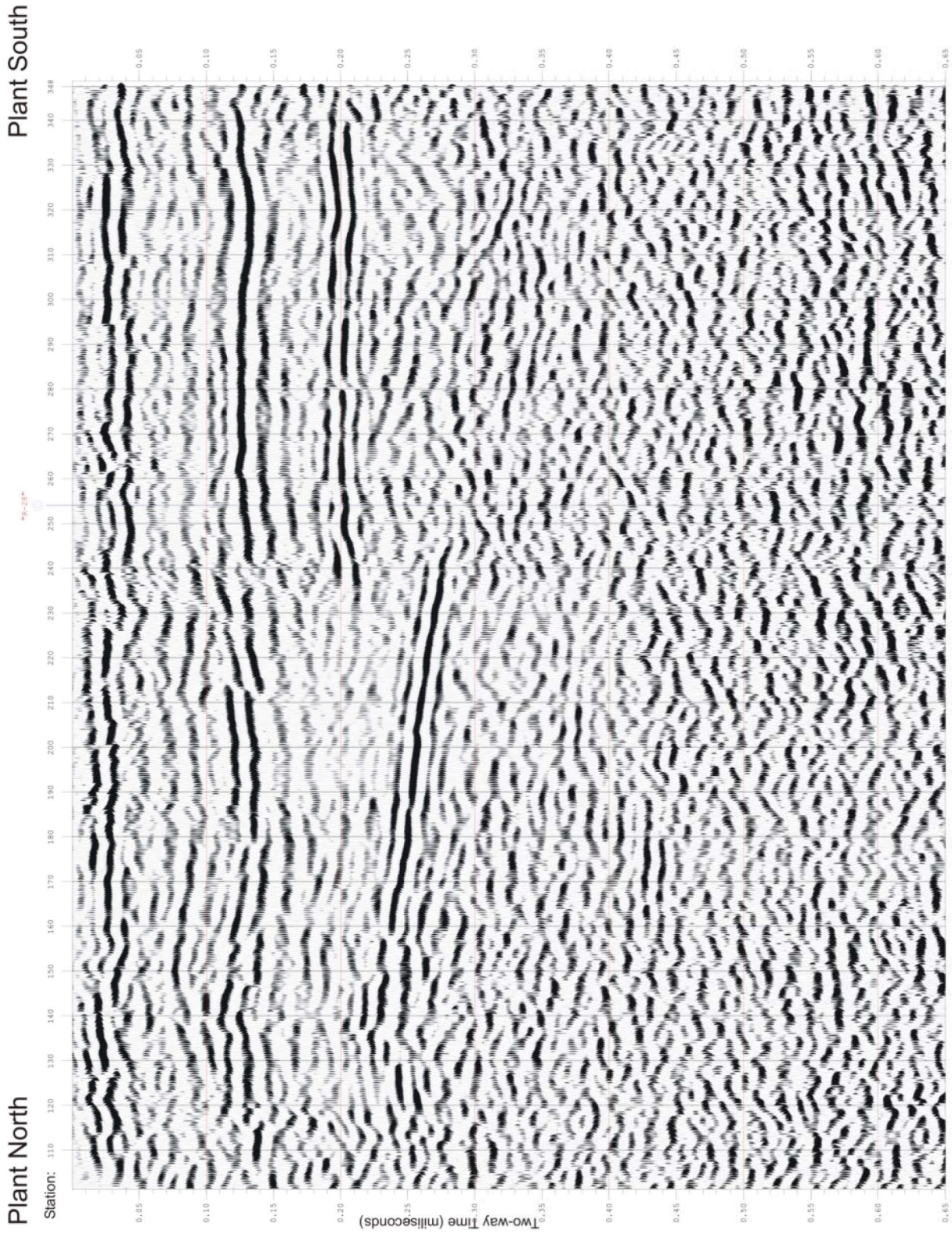

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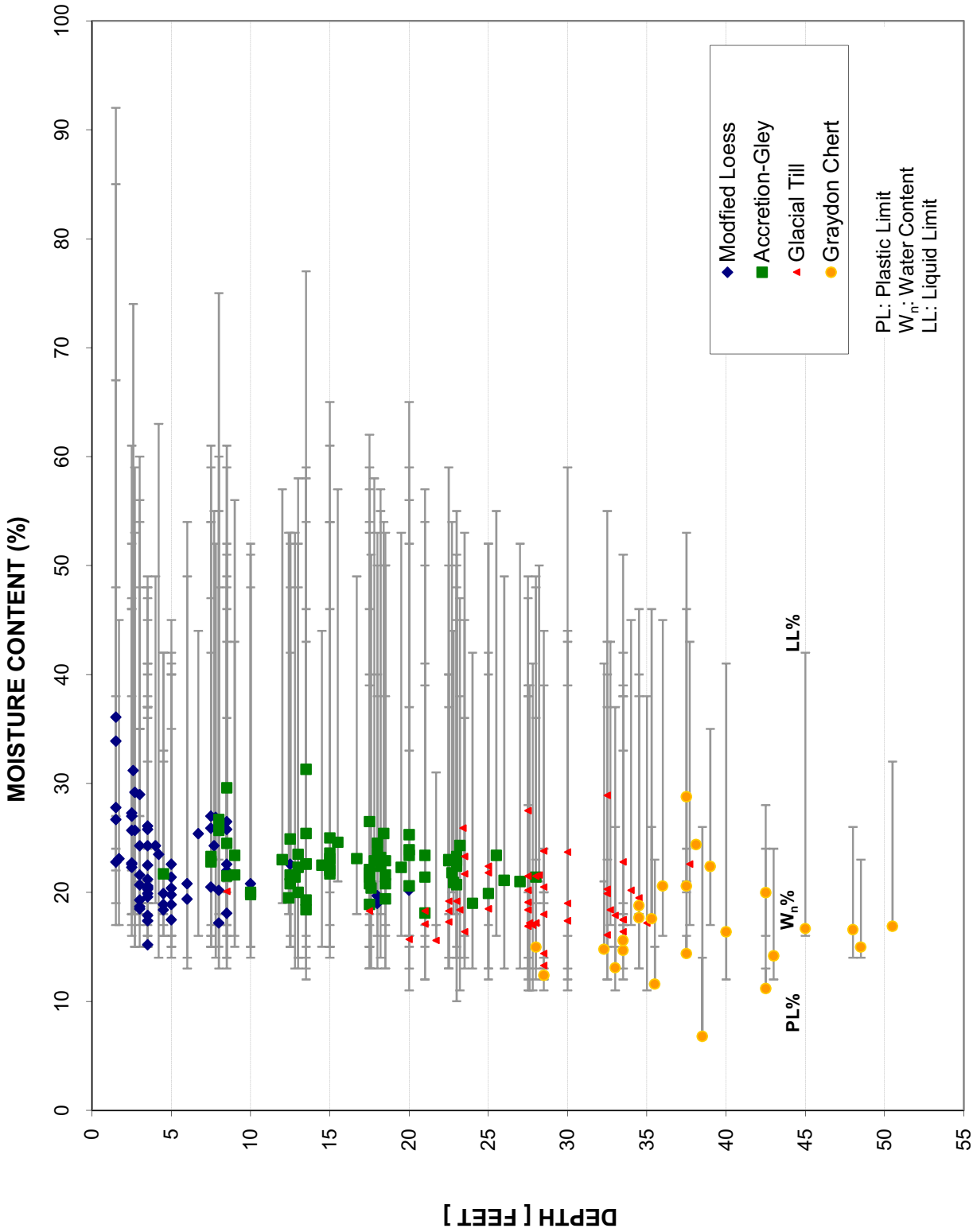
Figure 2.5.4-25—{Atterberg Limits and Moisture Content}



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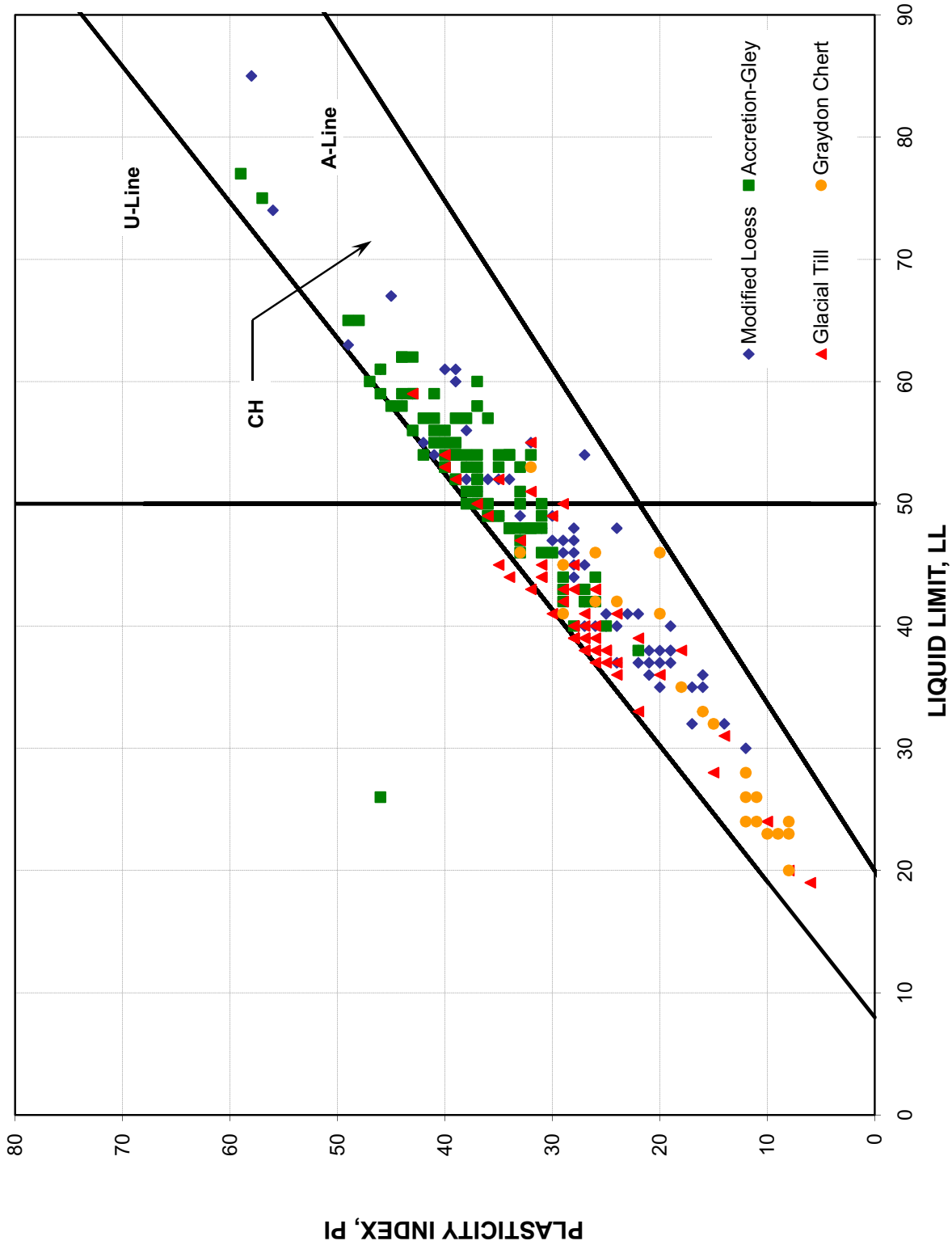

Figure 2.5.4-26—{Casagrande Plasticity Chart}

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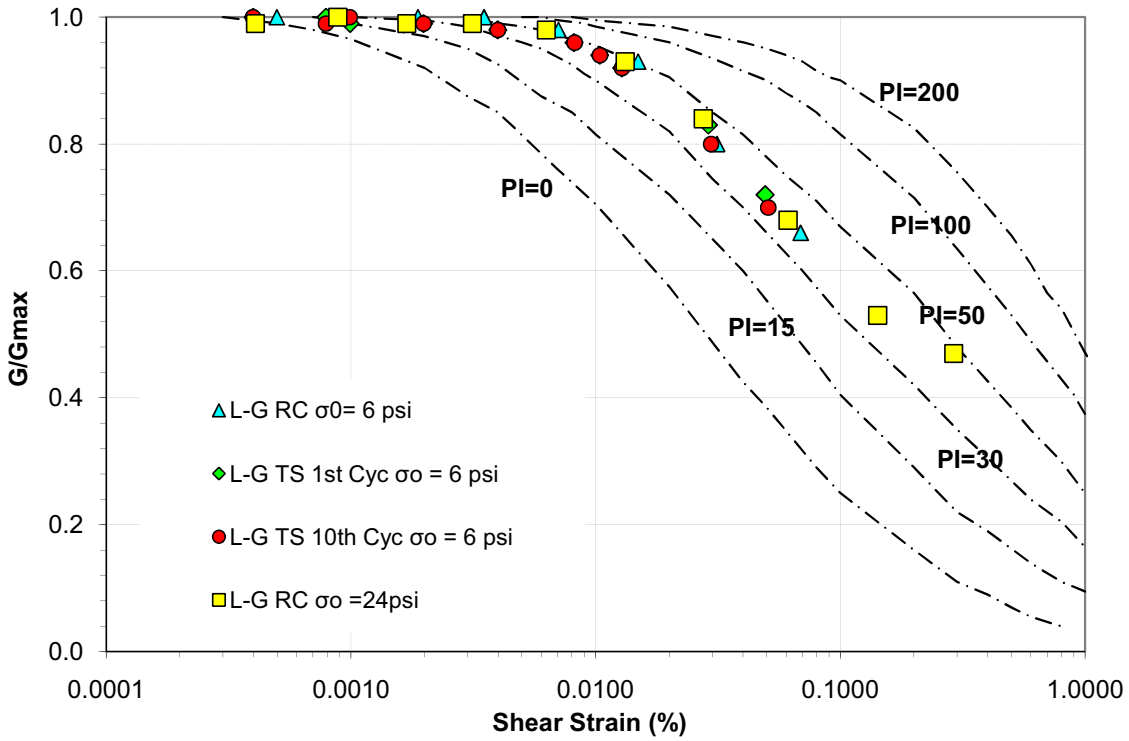
Figure 2.5.4-27—{Resonant Column Test, UTA-56-A (1E) Loess / Acretion Gley}



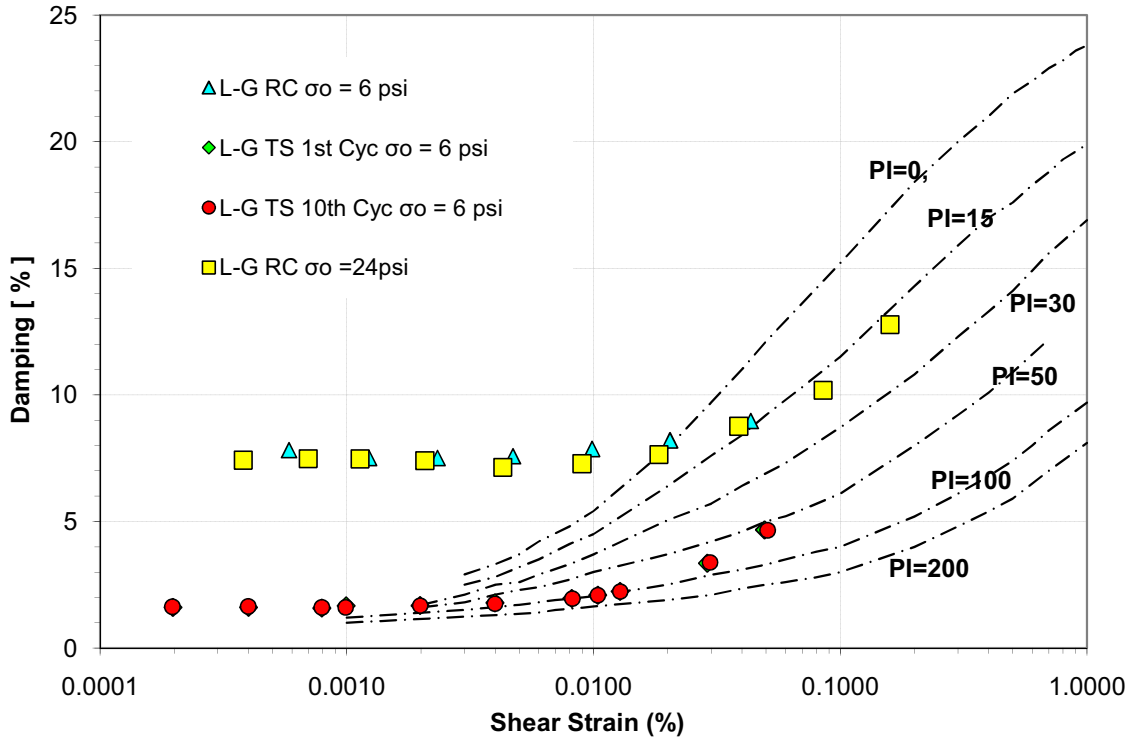


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Figure 2.5.4-28—{Resonant Column Test, UTA-56-B (2C) Glacial Till}

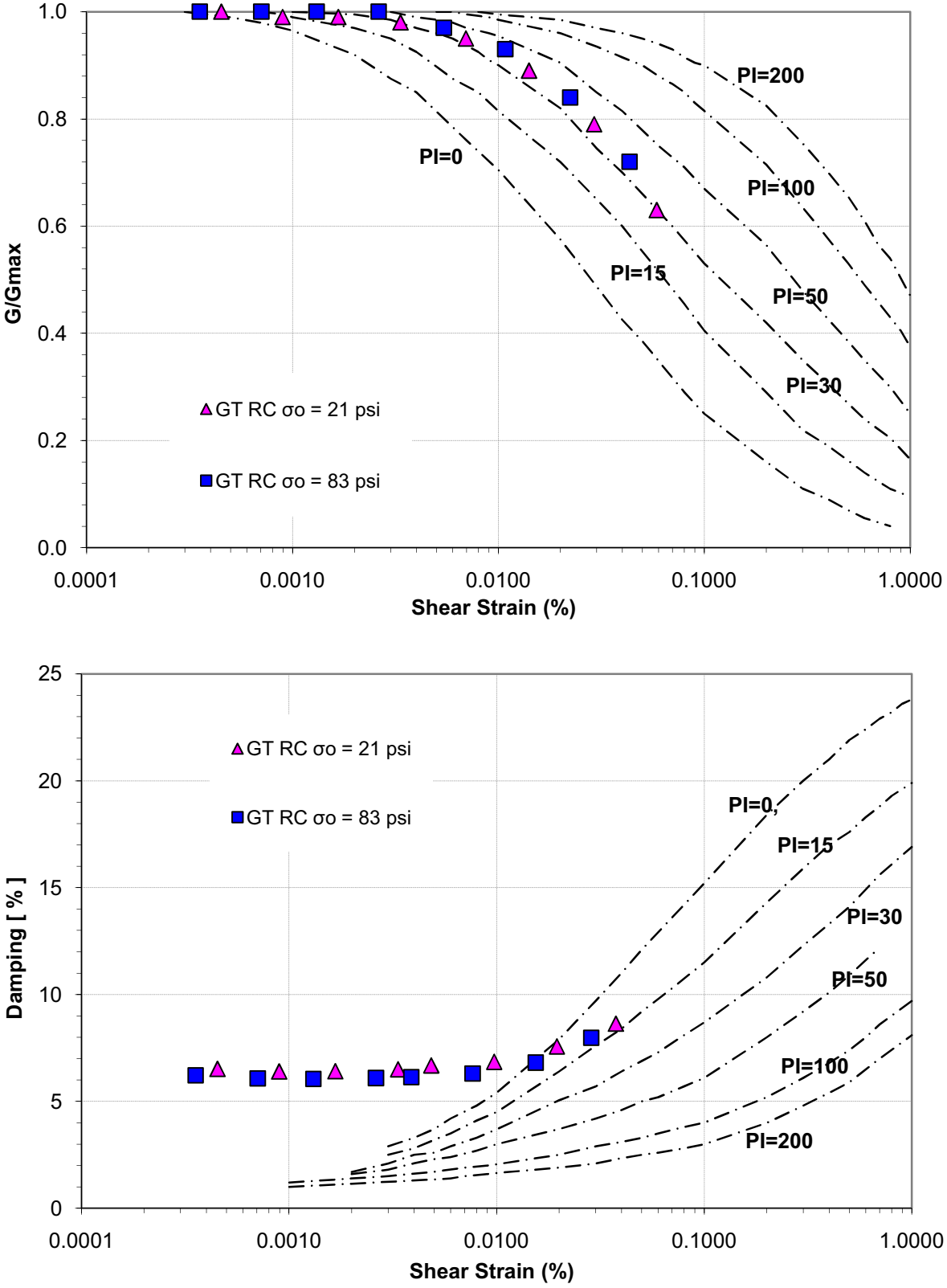

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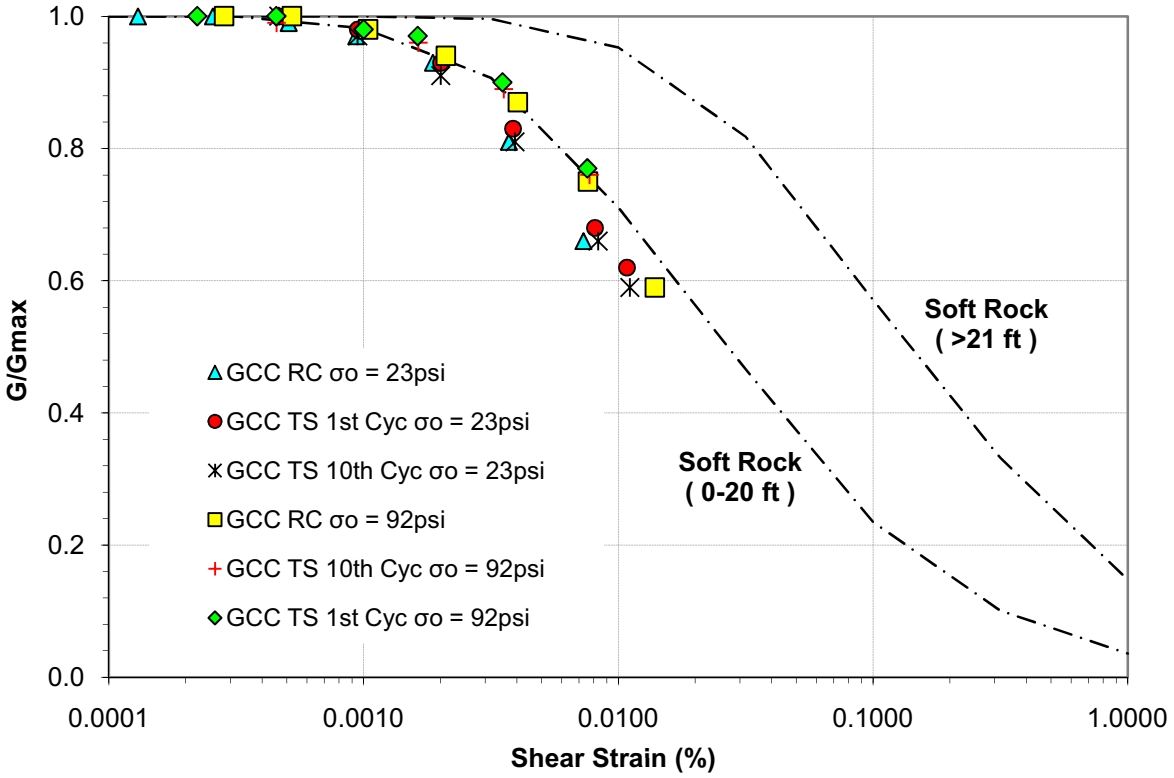
Figure 2.5.4-29—{Resonant Column Test, UTA-56-C (3C) Graydon Chert Conglomerate}



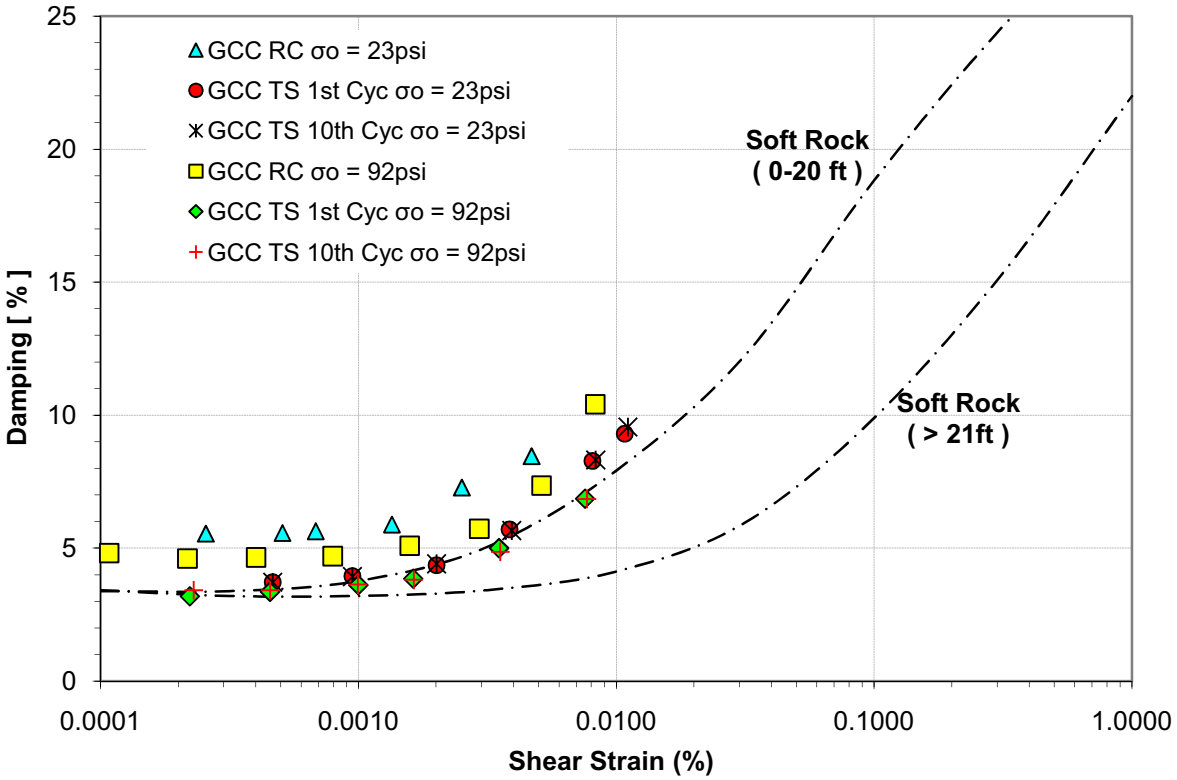


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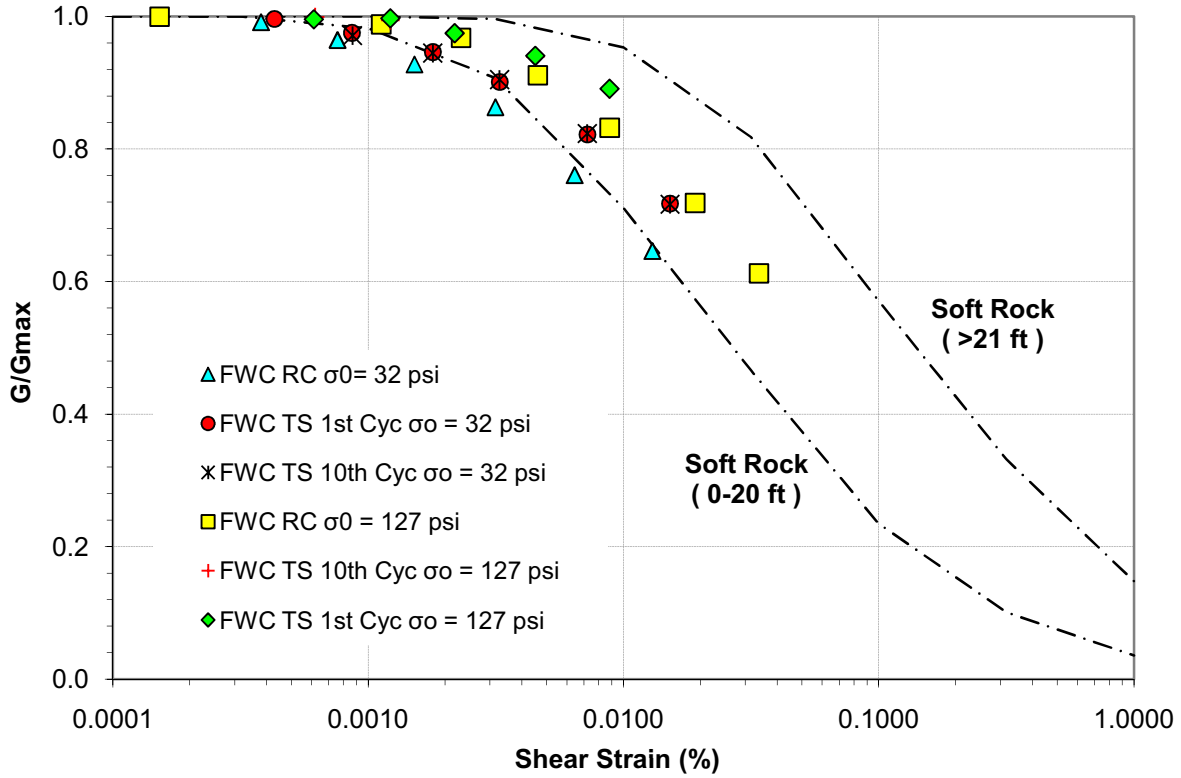
Figure 2.5.4-30—{Resonant Column Test, UTA-56-D (4C) Graydon Chert Conglomerate (FWC)}



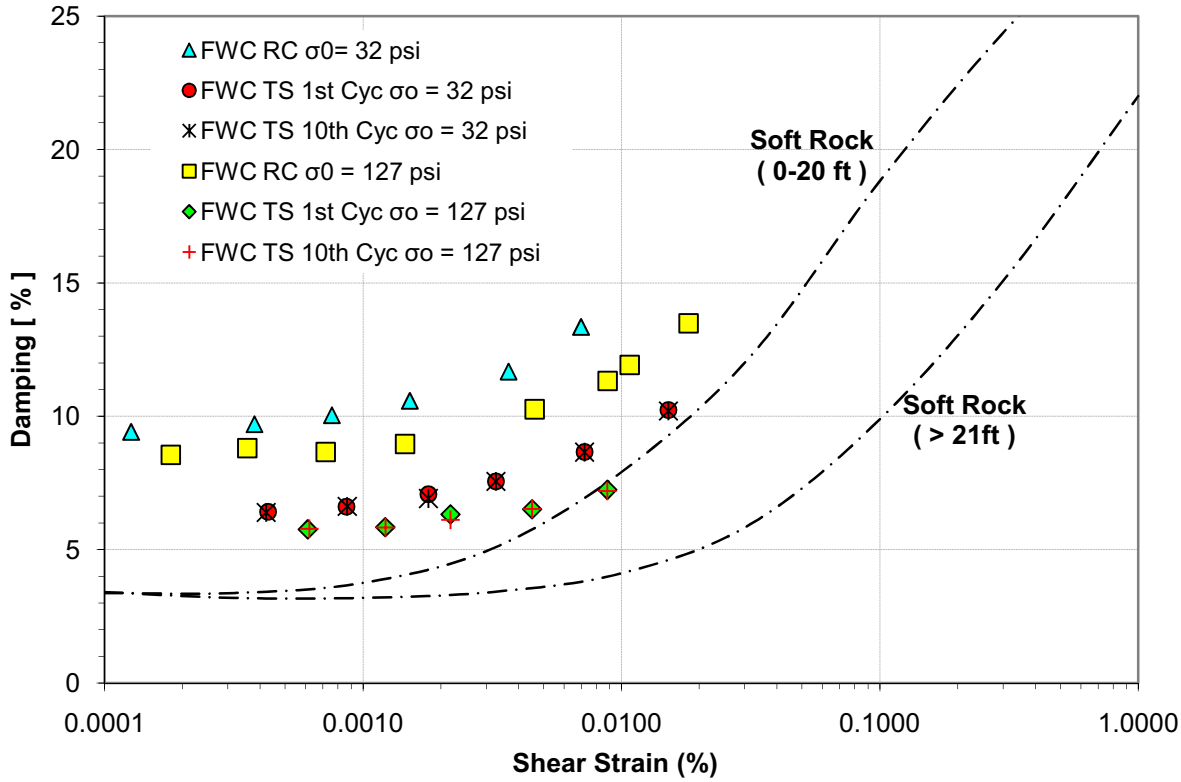


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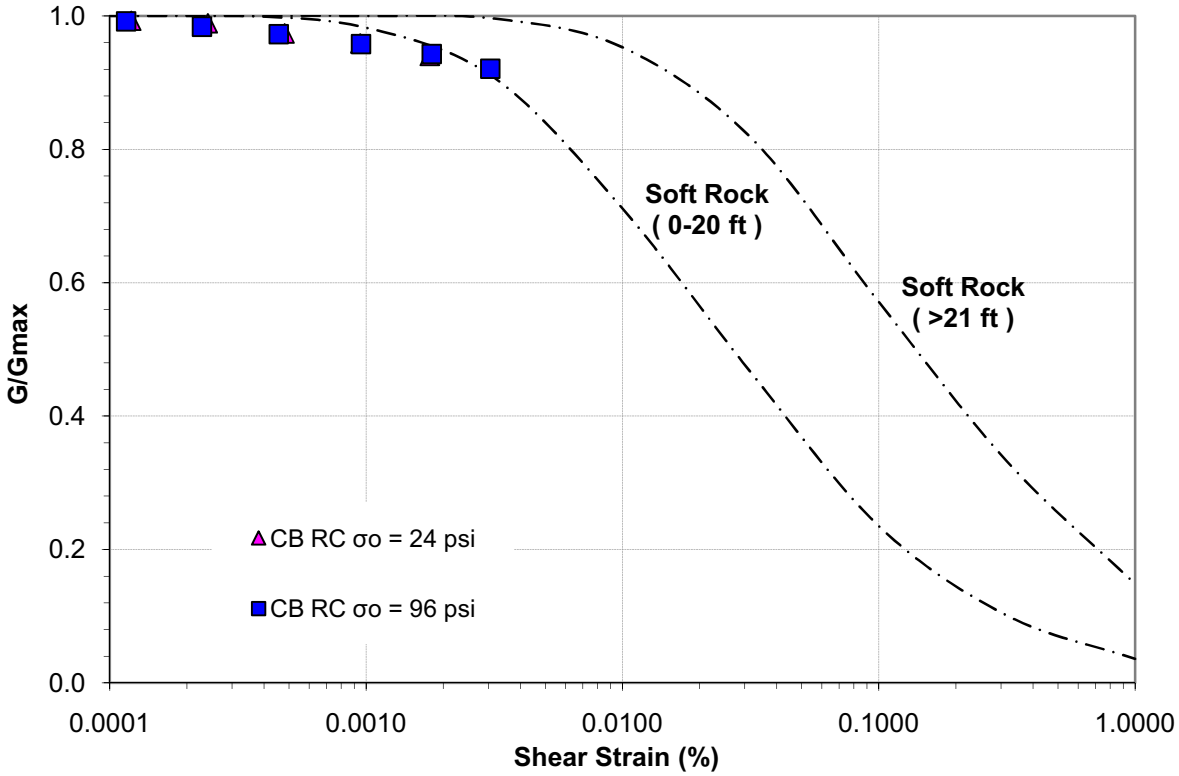
Figure 2.5.4-31—{Resonant Column Test, UTA-56-E (5C) Graydon Chert Conglomerate (CB)}



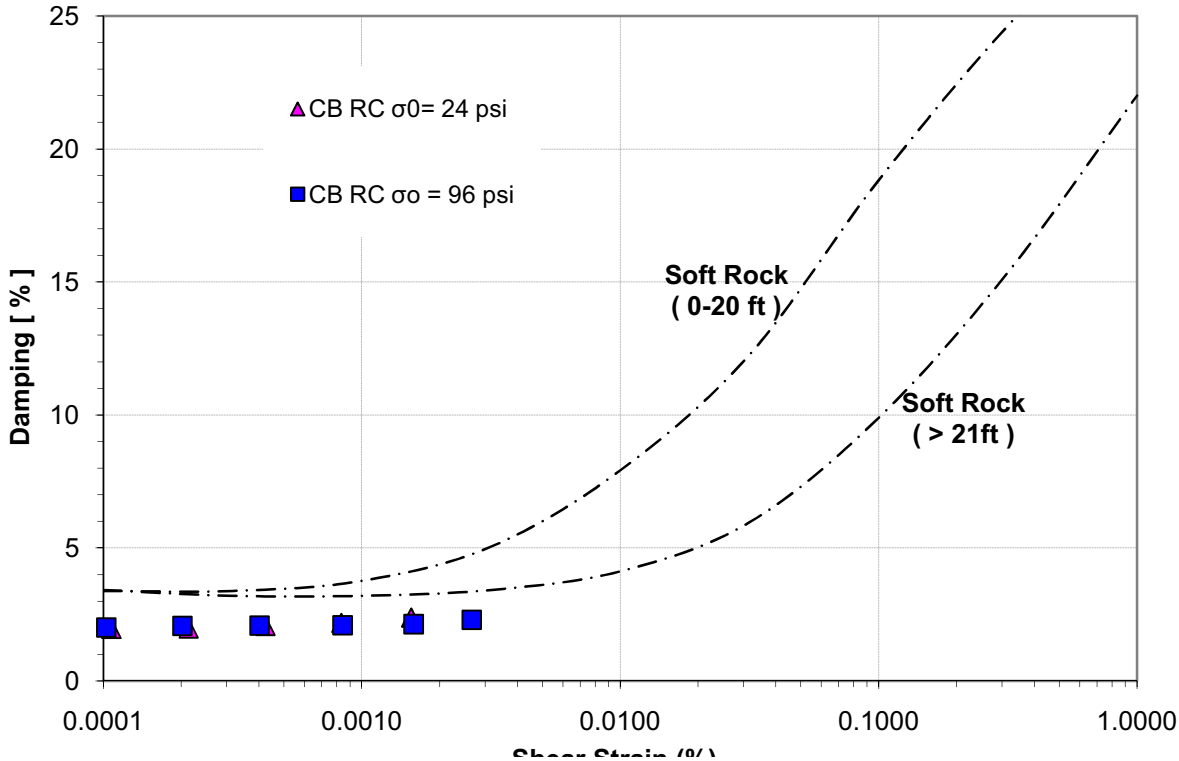


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Figure 2.5.4-32—{Resonant Column Test, UTA-56-B (2G) Glacial Till}

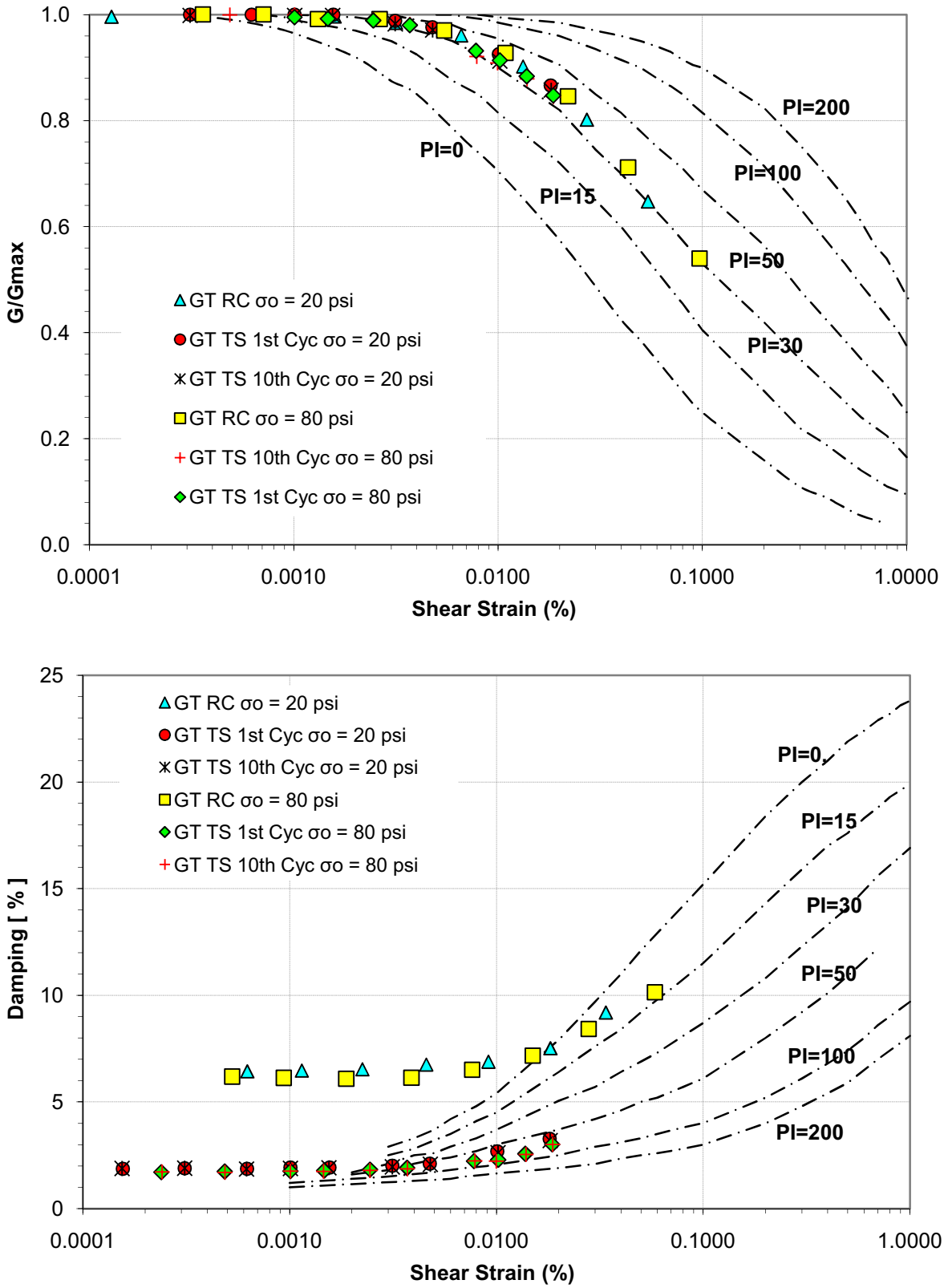

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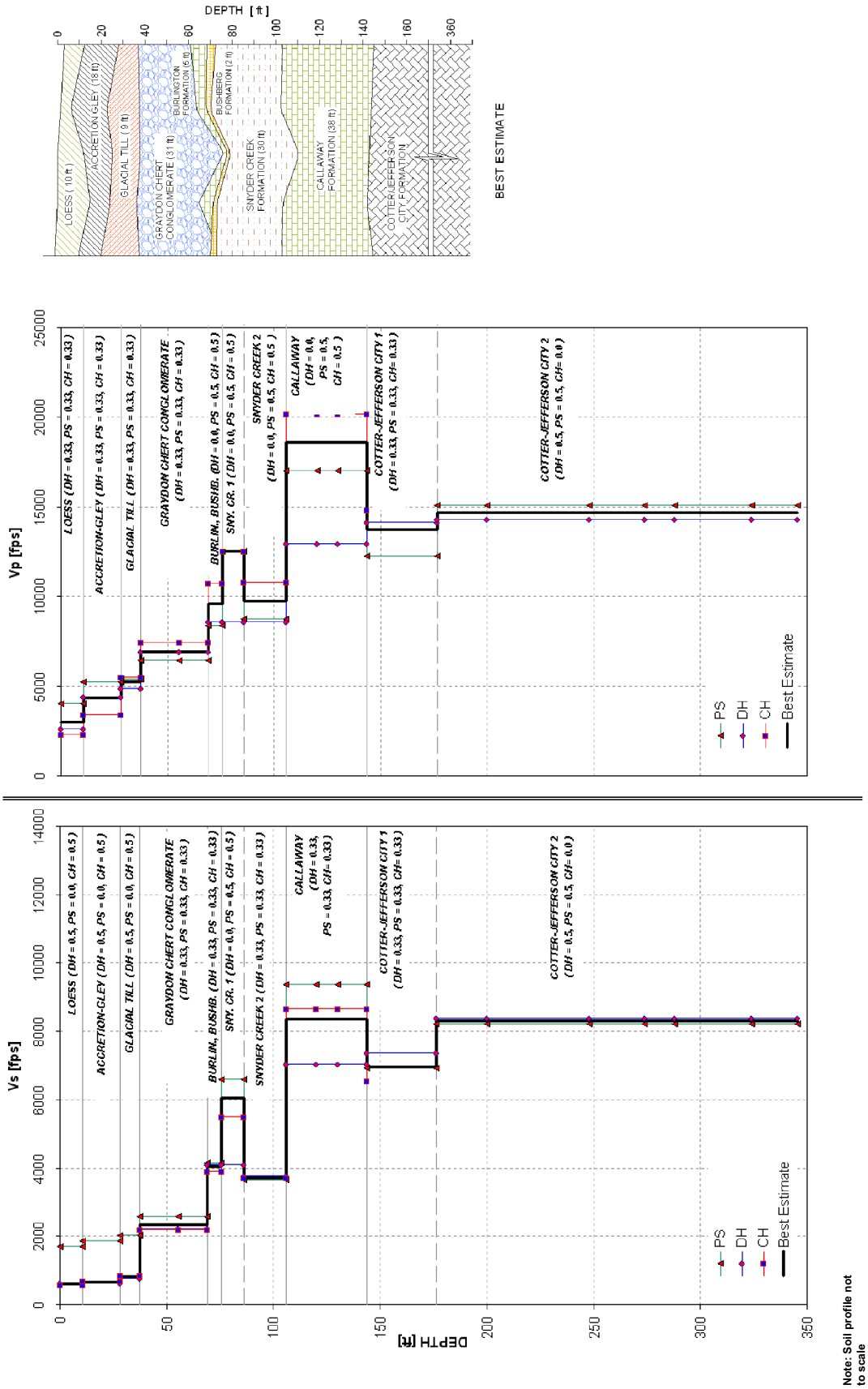
Figure 2.5.4-33—{Seismic Wave Velocity Profiles Soil Velocity Profile}



Note: Soil profile not to scale

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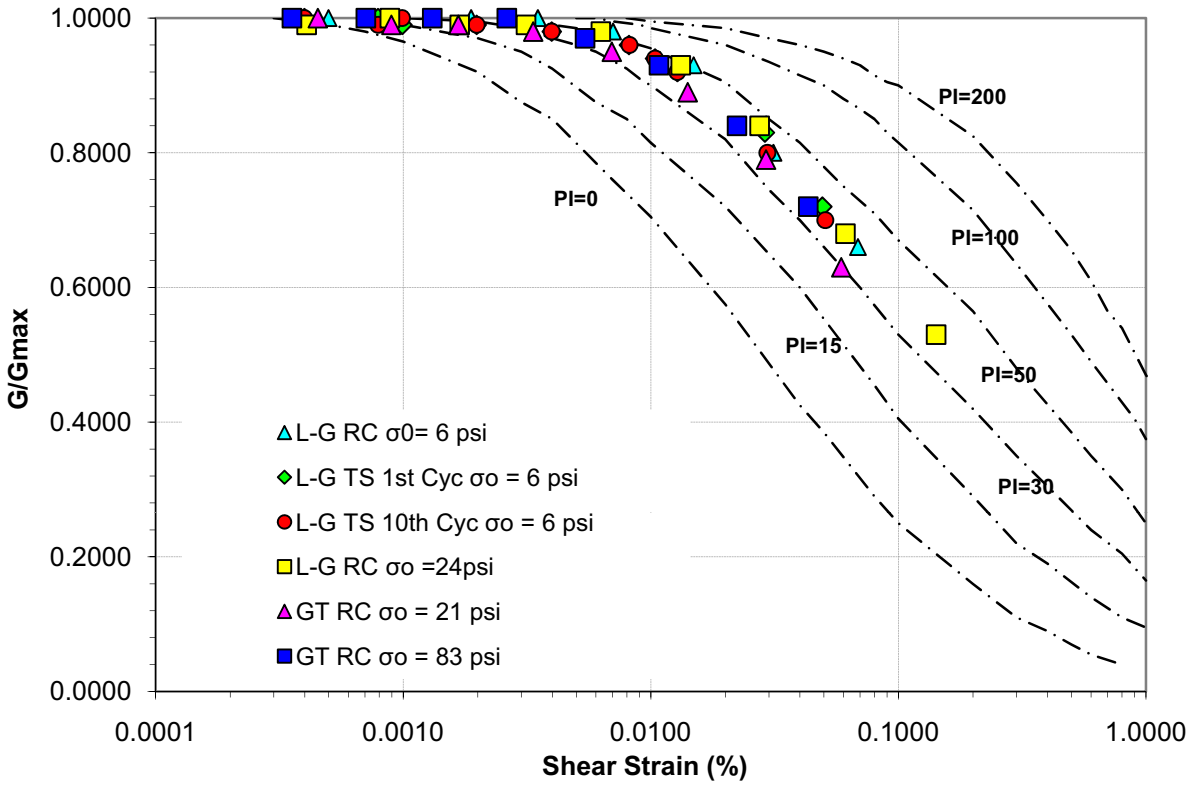
Figure 2.5.4-34—{Strain Dependent Properties for the Overburden Soils}



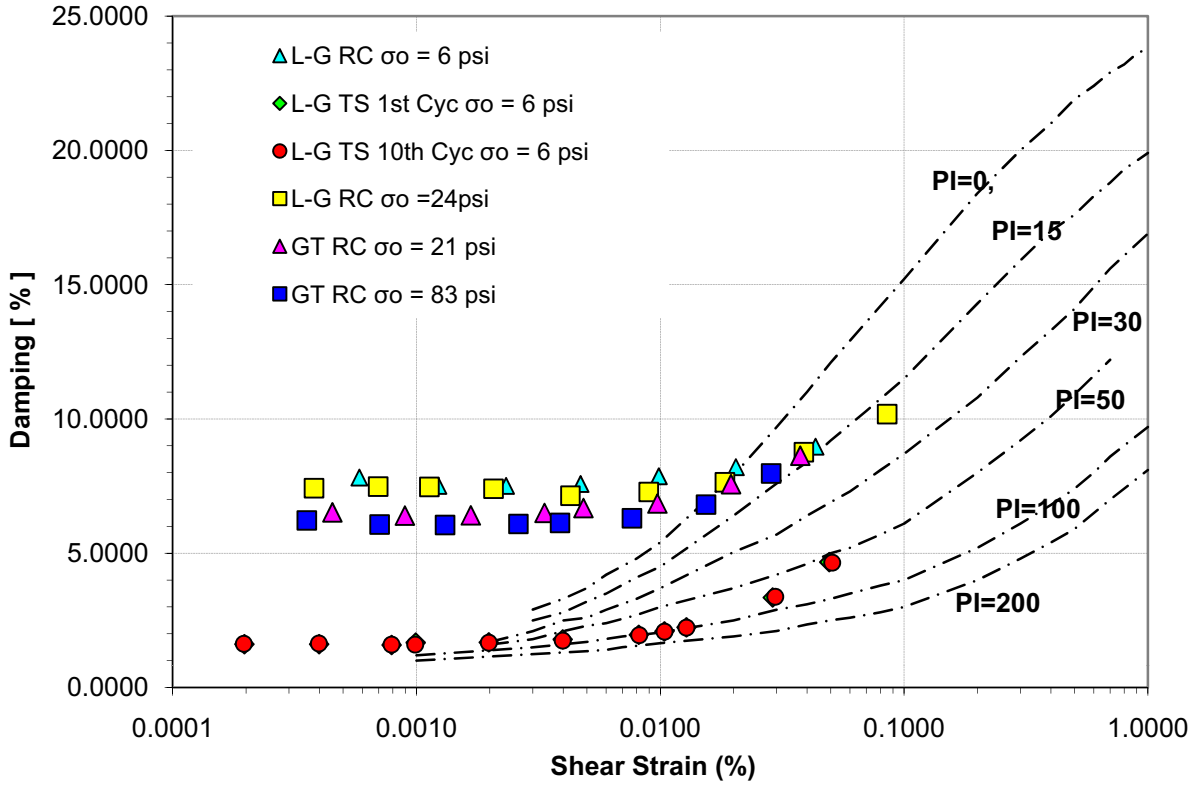


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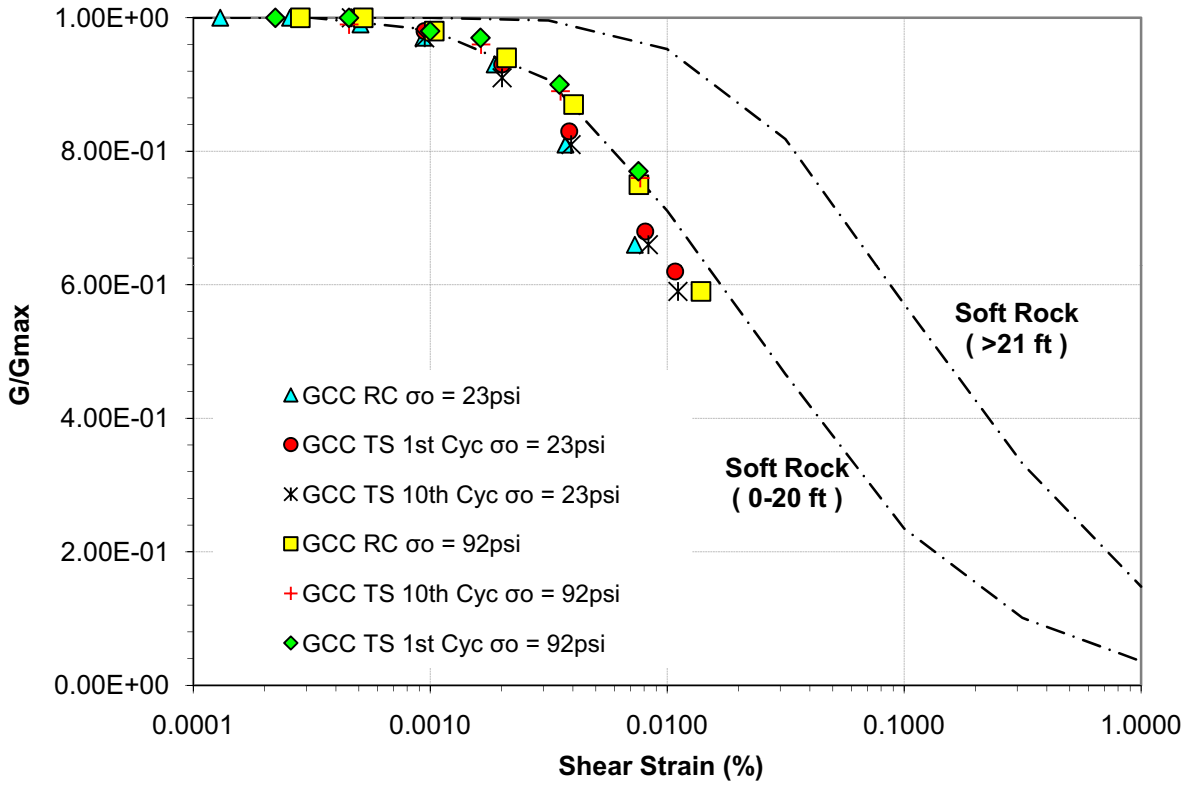
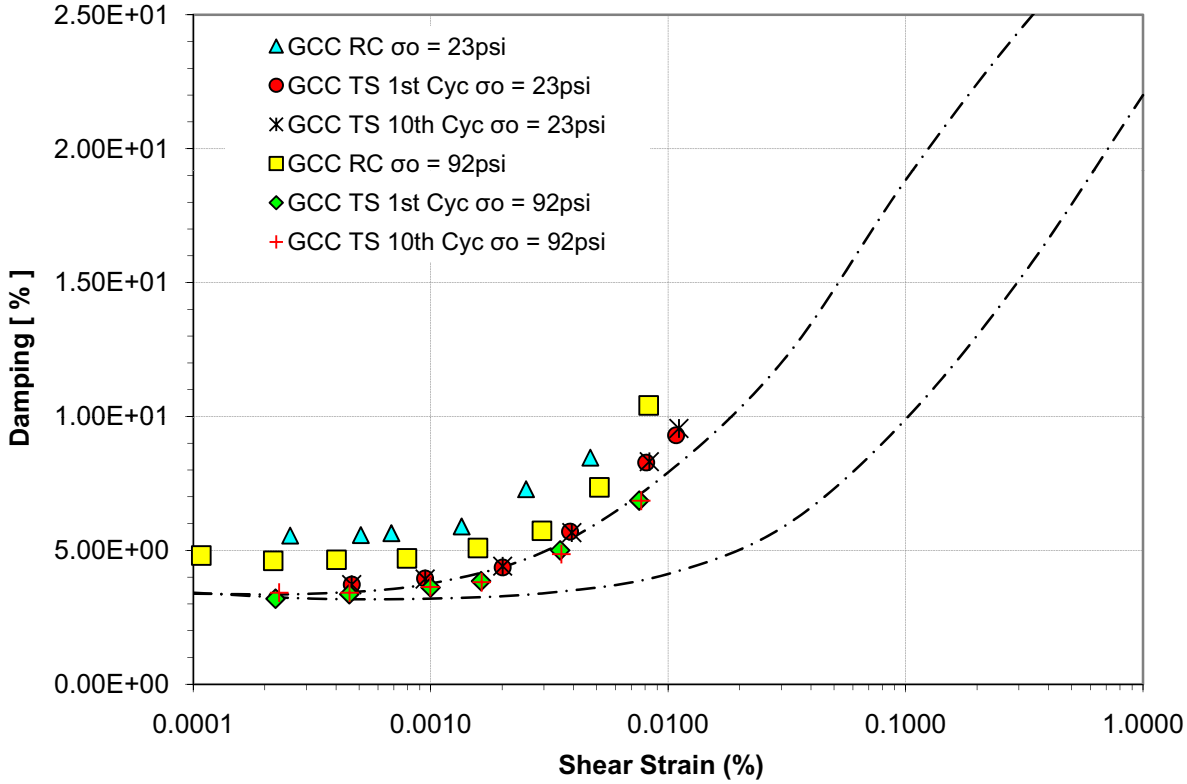
Figure 2.5.4-35—{Recommended Strain Dependent Properties for the Graydon Chert Conglomerate}





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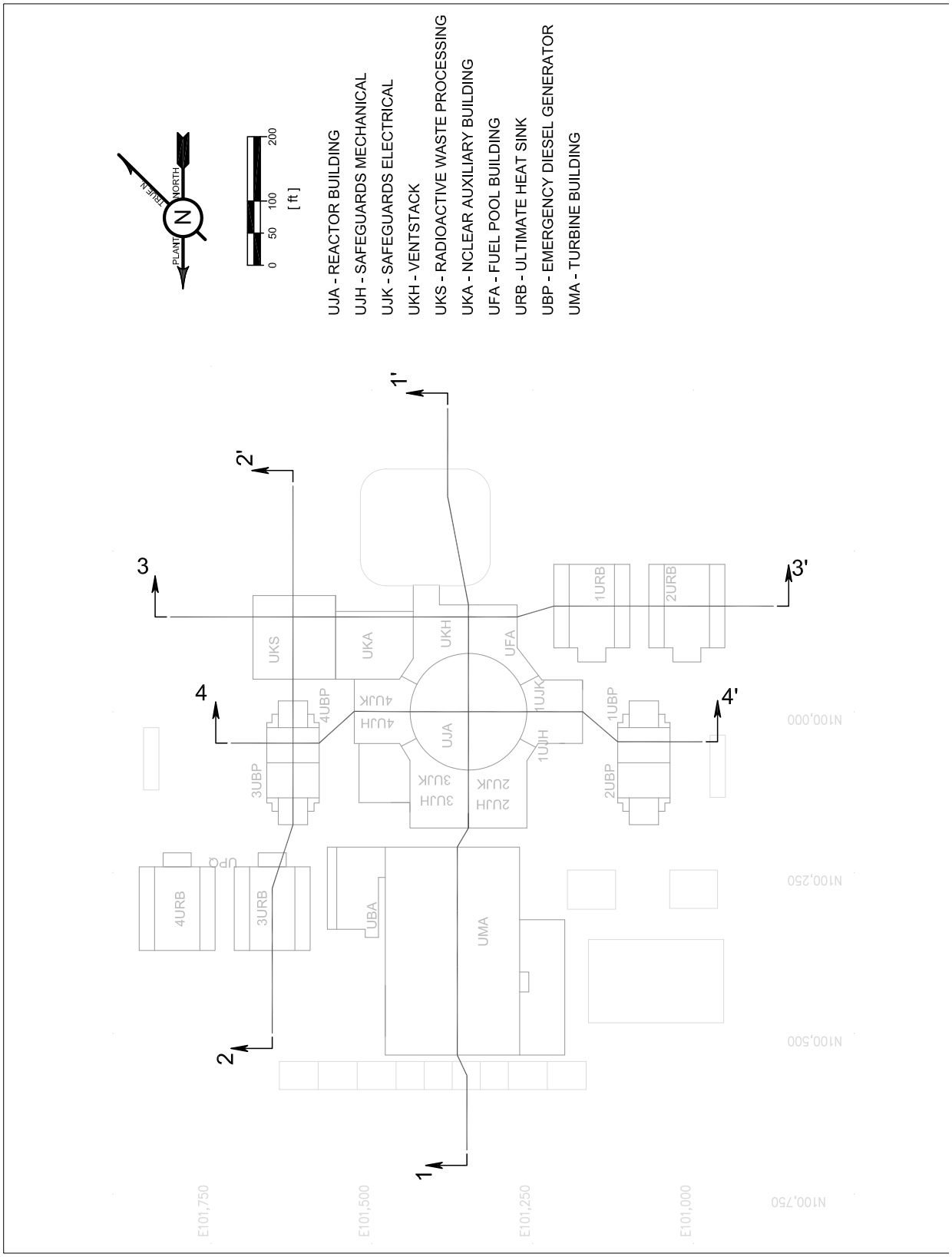

Figure 2.5.4-36—{Foundation Interface Profiles}

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Figure 2.5.4-37—{Foundation Interface Cross Section 1-1'}

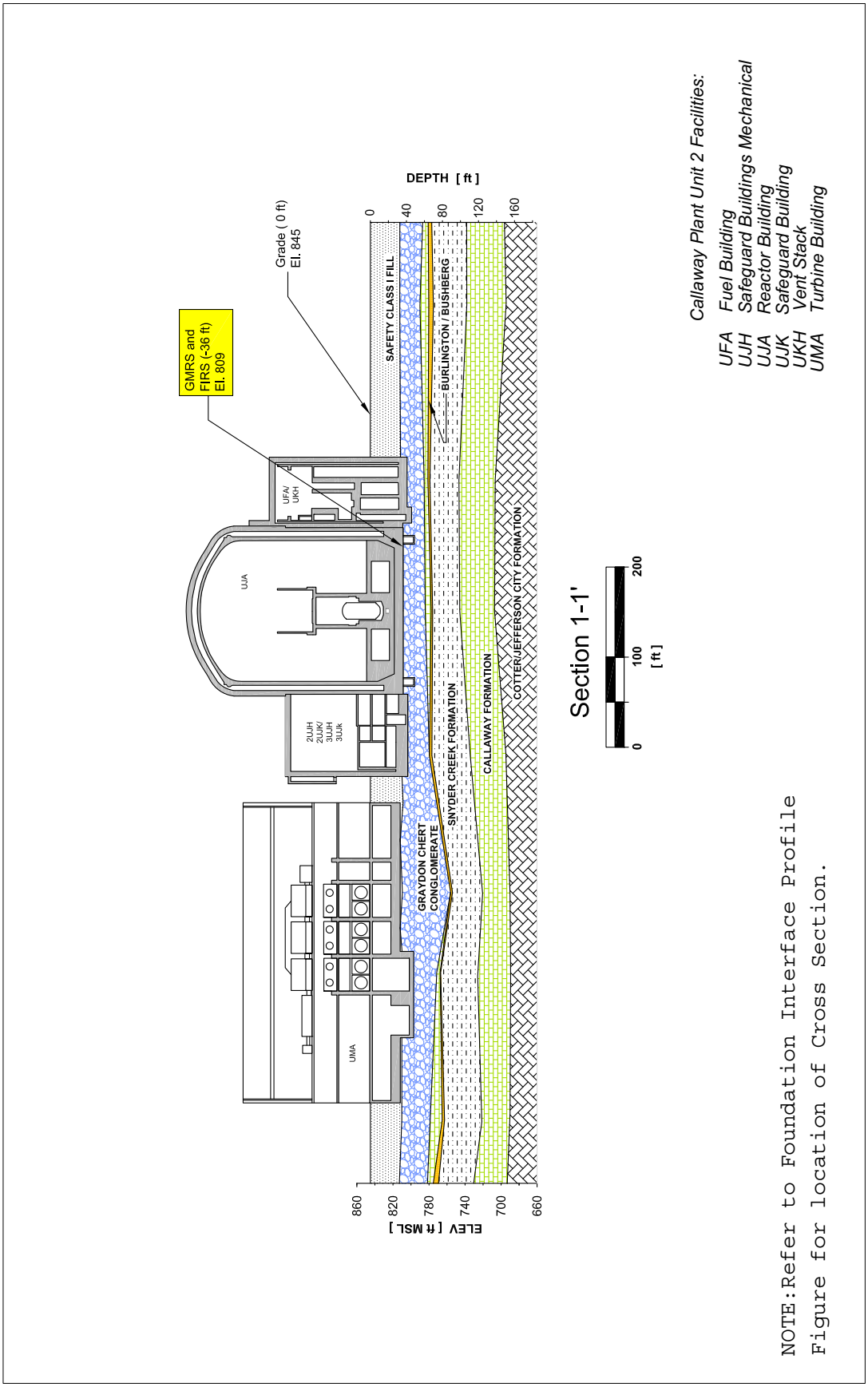

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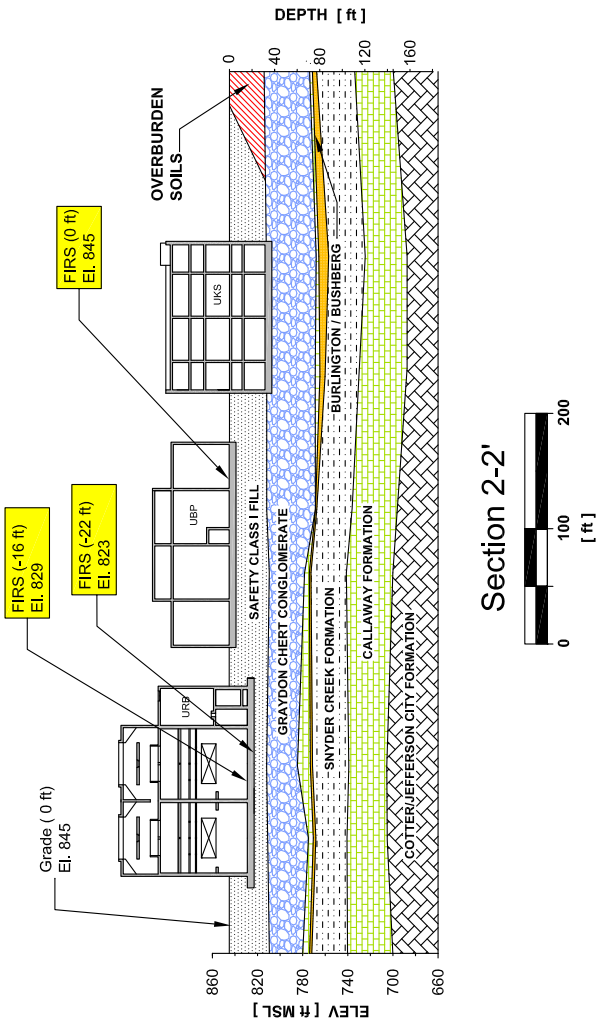
Figure 2.5.4-38—{Foundation Interface Cross Section 2-2'}



Callaway Plant Unit 2 Facilities:

UBP Emegency Diesel Generator Building
URB Essential Service Water Cooling Tower Structure
UKS Radioactive Waste Processing Building

NOTE:Refer to Foundation Interface Profile
Figure for location of Cross Section.

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Figure 2.5.4-39—{Foundation Interface Cross Section 3-3'}

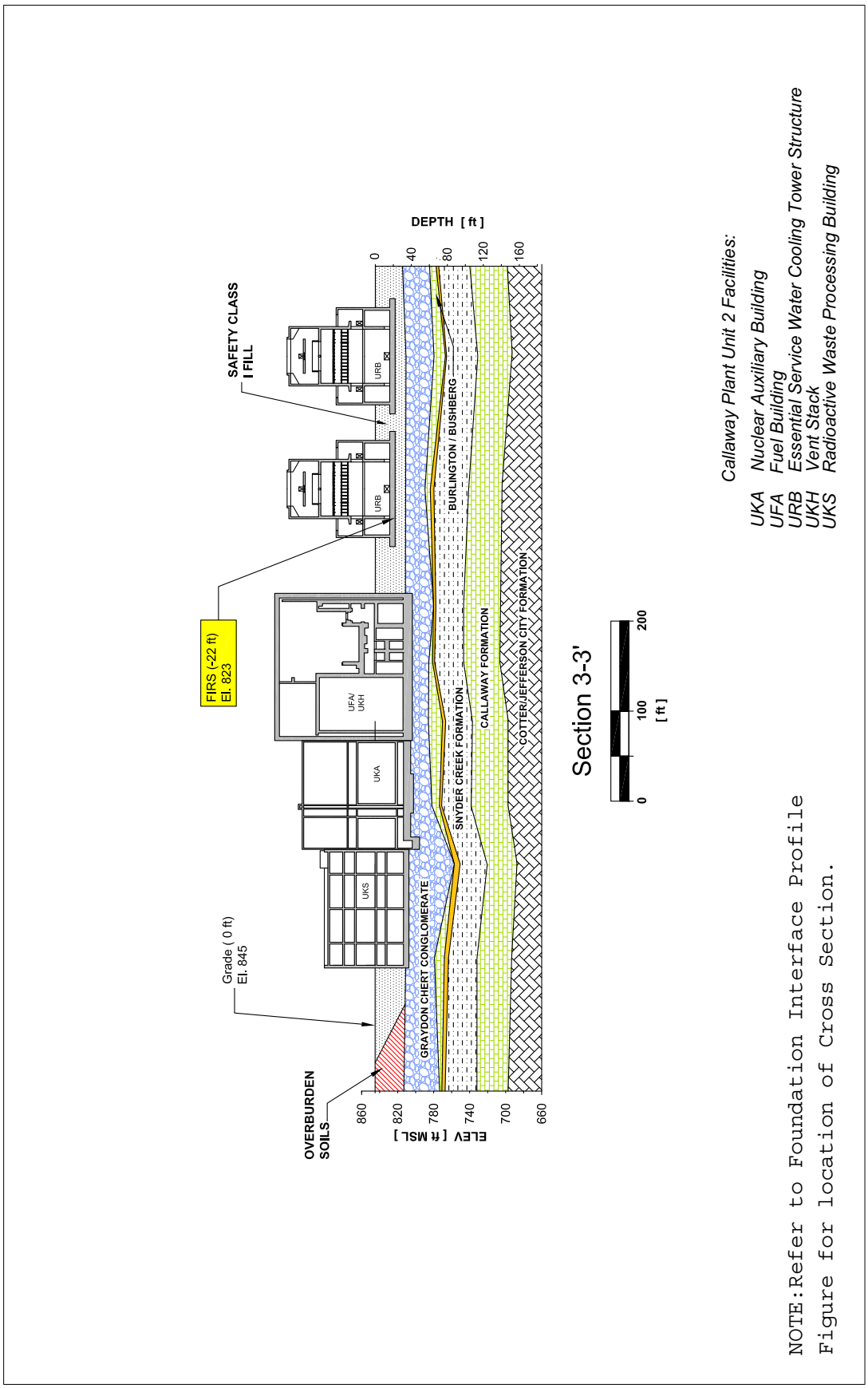

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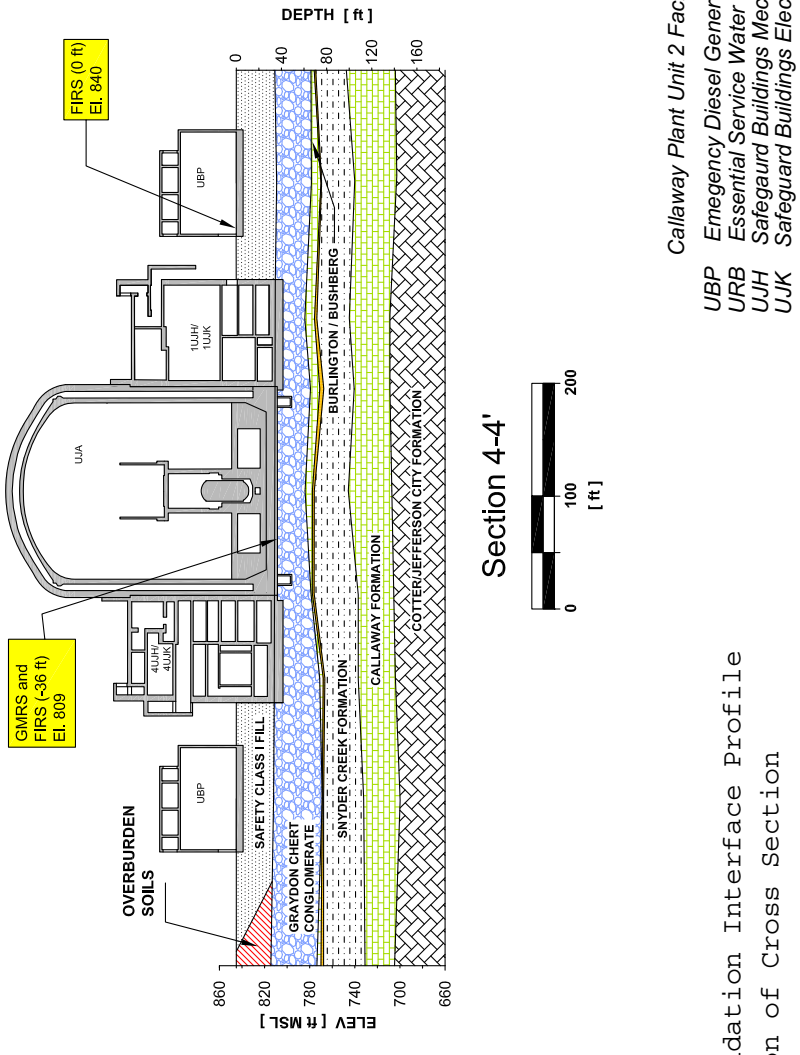
Figure 2.5.4-40—{Foundation Interface Cross Section 4-4'}



NOTE:Refer to Foundation Interface Profile
Figure for location of Cross Section

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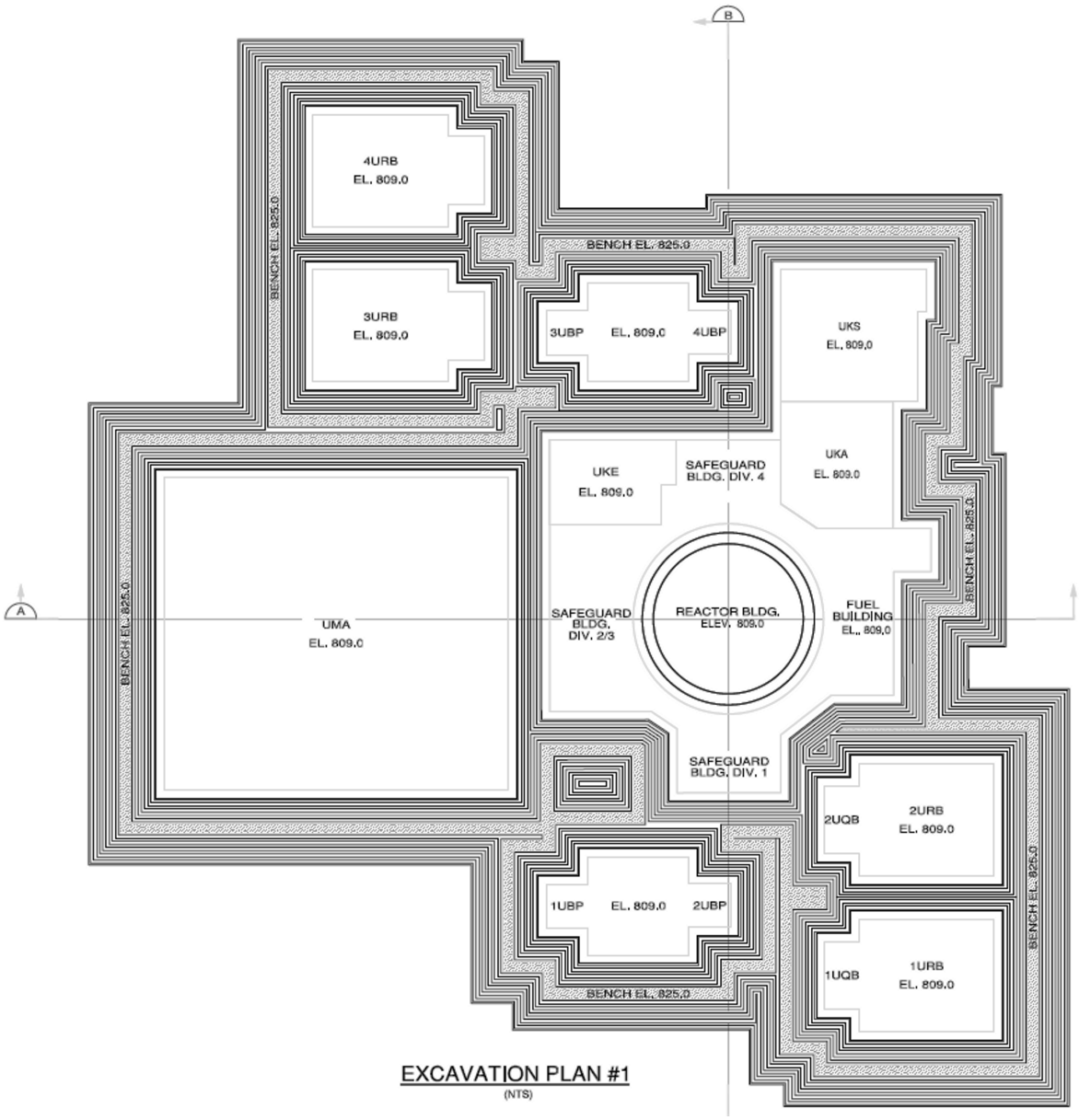
Figure 2.5.4-41—{Excavation Plan}



Bottom of Excavation: El. 809
Plant Grade: El. 845
Estimated Excavation Volume: 750 yd^3
Refer Excavation Cross Sections A and B shown on next page.

Callaway Plant Unit 2 Facilities:

UBP	*Emegency Diesel Generator Building*
URB	*Essential Service Water Cooling Tower Structure*
UKA	*Nuclear Auxiliary Building*
UKE	*Access Building*
UKS	*Radioactive Waste Processing Building*
UMA	*Turbine Building*

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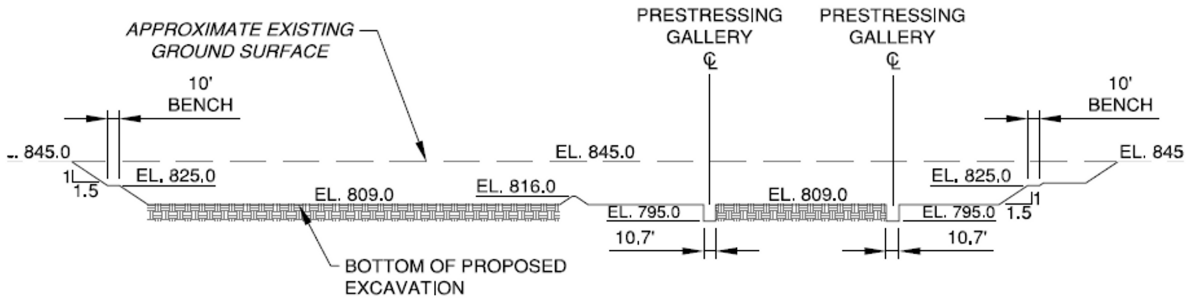
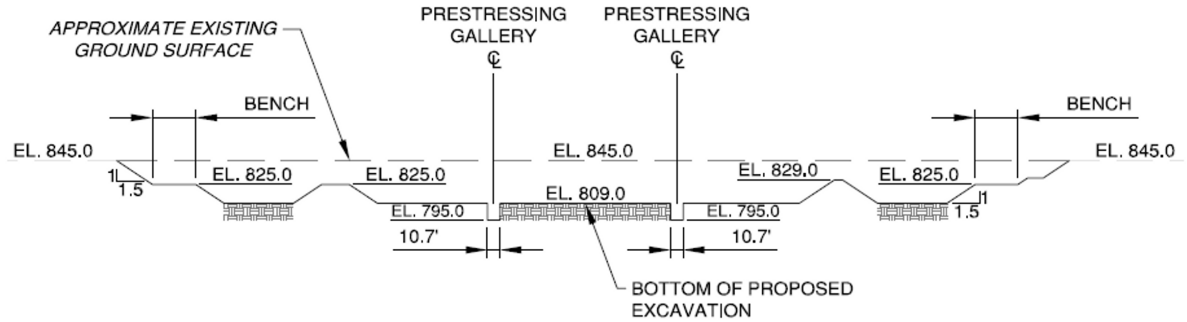
Figure 2.5.4-42—{Excavation Cross Sections}



SECTION A-A
(NTS)



SECTION B-B
(NTS)

Bottom of Excavation: El. 809
Plant Grade: El. 845
Estimated Excavation Volume: 600,000 yd^3
Refer to Excavation Plan on previous page
for location of Cross Sections A and B.

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Figure 2.5.4-43—{Liquefaction Potential}

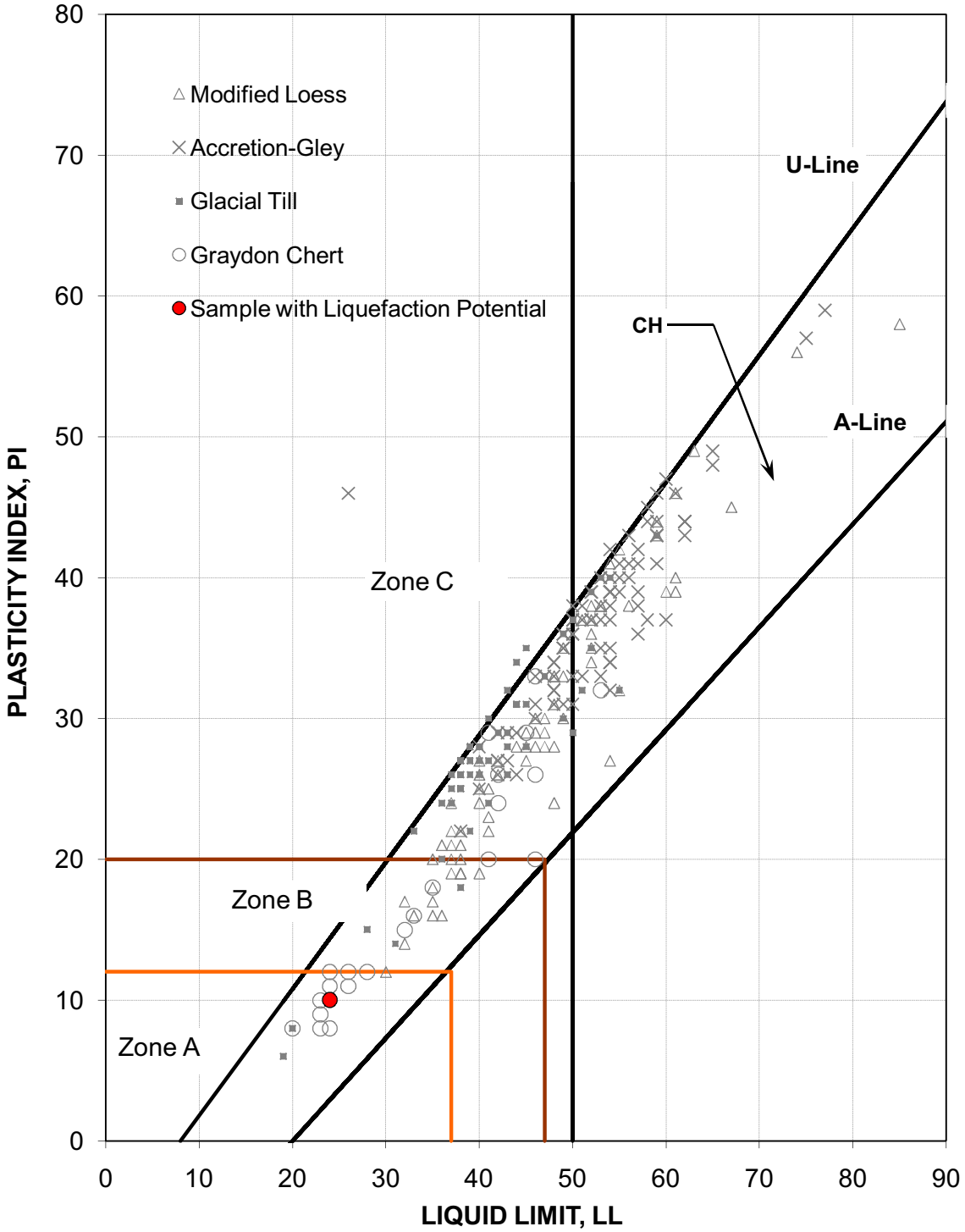

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Figure 2.5.4-44—{NI Settlement Analysis Service Loads}

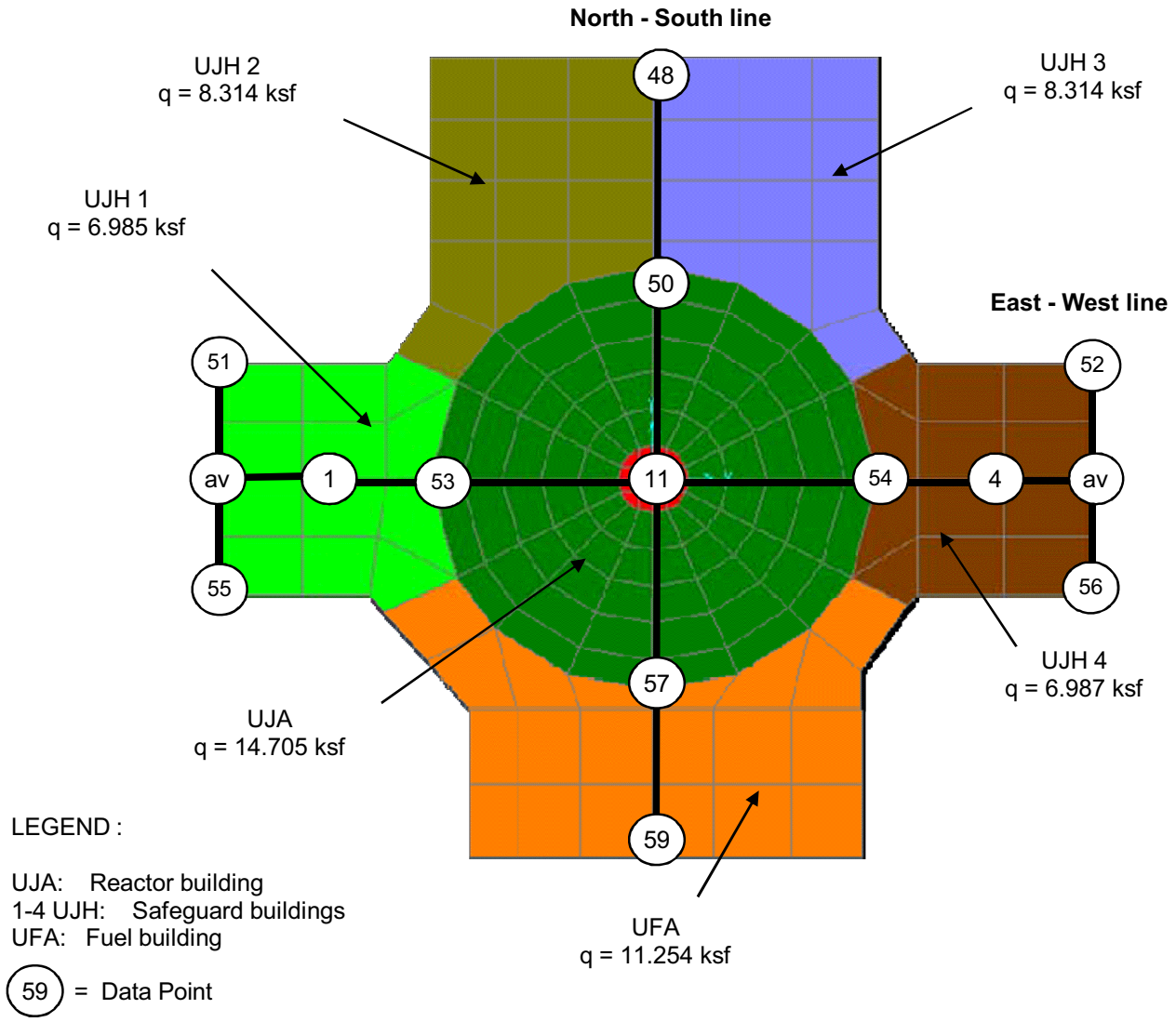

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Figure 2.5.4-45—{NI Settlement Analysis East West Section}

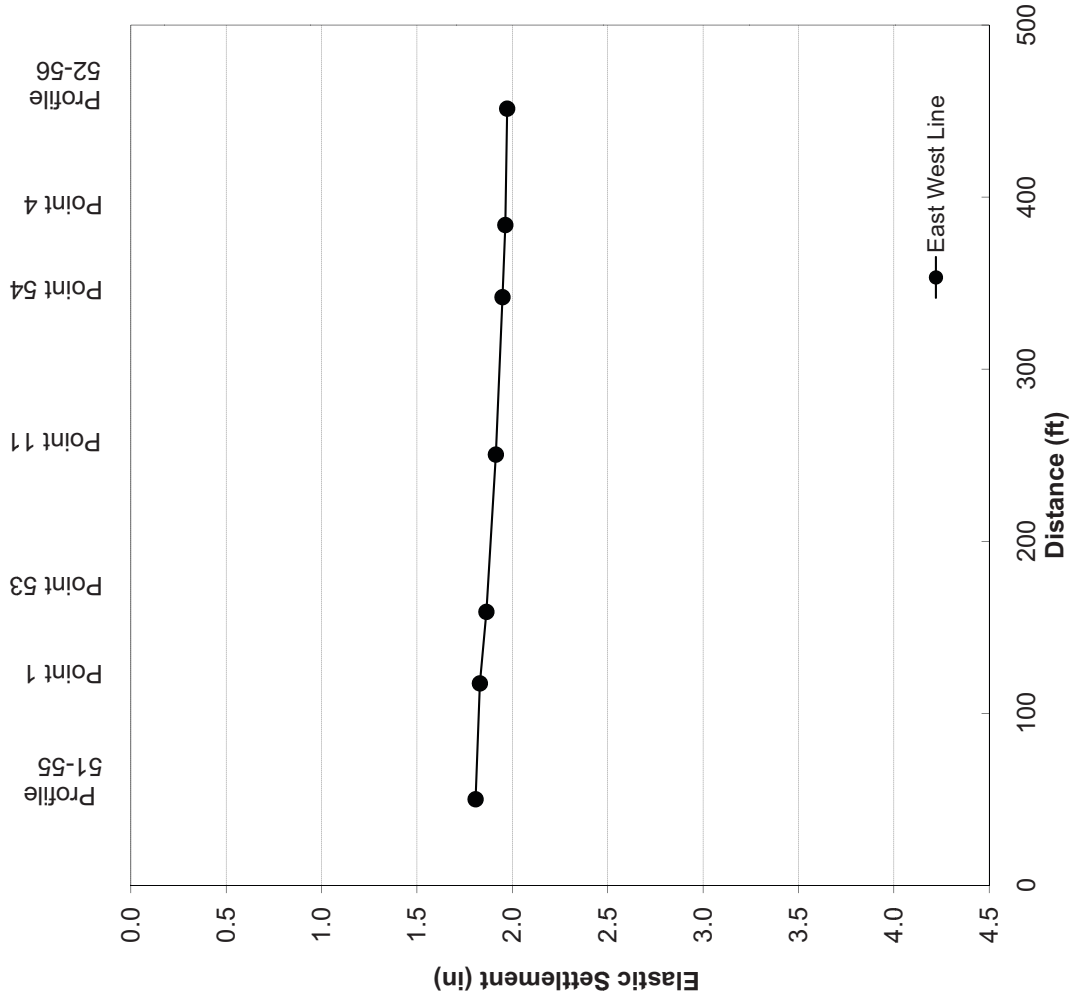

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Figure 2.5.4-46—{NI Settlement Analysis North South Section}

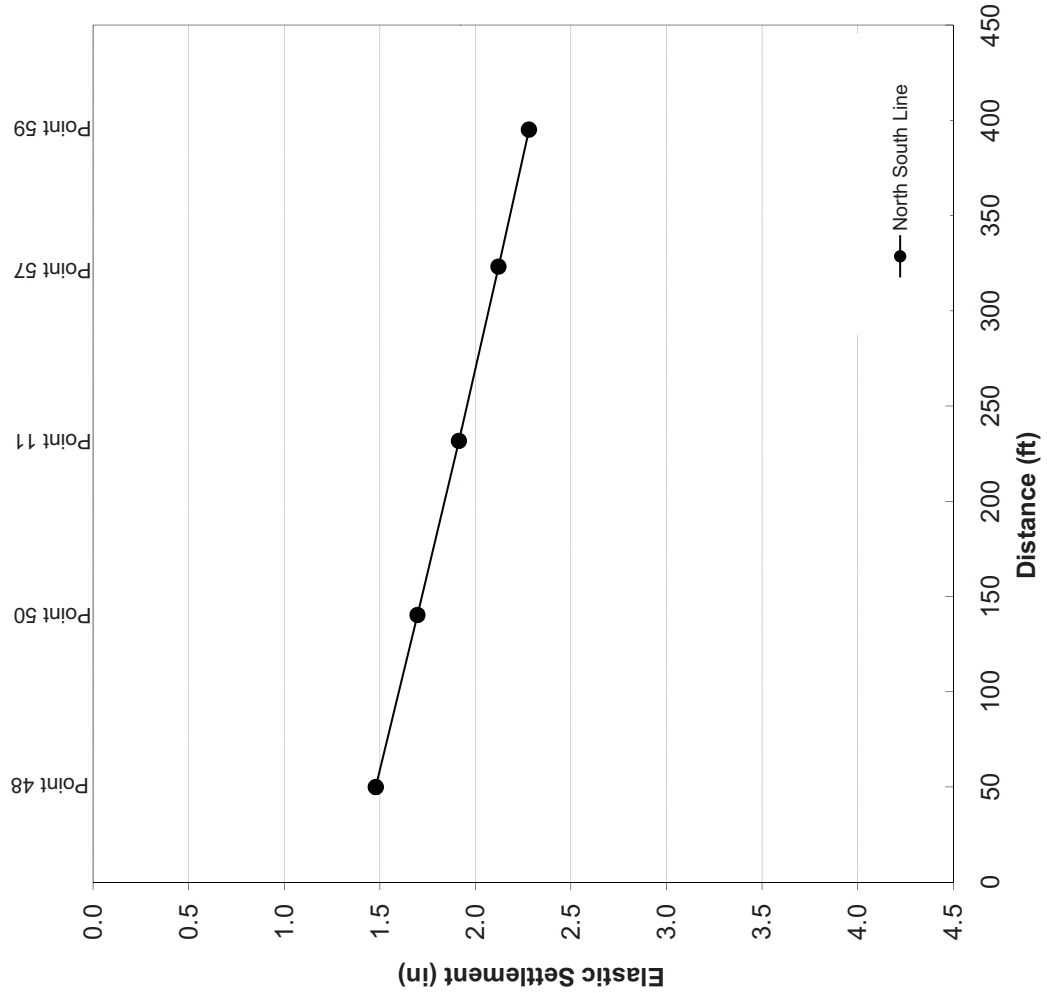

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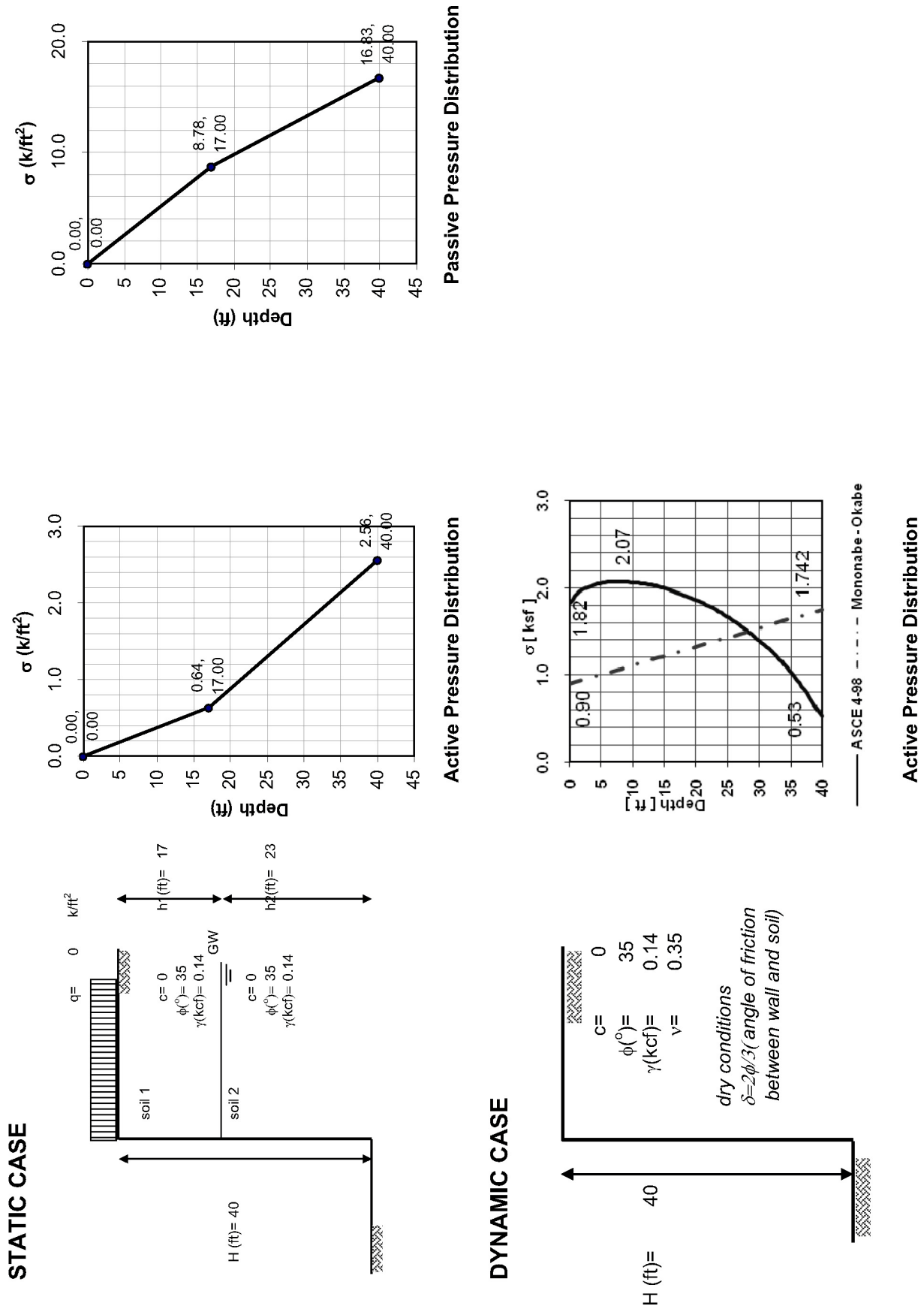
Figure 2.5.4-47—{Examples of Earth Pressure Diagrams}



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