2.5.5 STABILITY OF SLOPES

The U.S. EPR FSAR includes the following COL Item for Section 2.5.5.

> A COL applicant that references the U.S. EPR design certification will evaluate site-specific information concerning the stability of earth and rock slopes, both natural and manmade (e.g., cuts, fill, embankments, dams, etc.), of which failure could adversely affect the safety of the plant.

This COL Item is addressed as follows:

{This Section addresses the stability of constructed and natural slopes. It was prepared based on the guidance in relevant Section of NRC Regulatory Guide 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)," (NRC, 2007). Constructed slopes evolve as part of the overall site development.

The site of the Callaway Plant Unit 2 is comprised of generally flat topography in the vicinity of the primary structures and components. The site is planned to be graded in order to establish the final grade for the project, resulting in minor cuts and fills, as well as slopes. The stability of these slopes and their potential impact on safety-related structures are evaluated herein. In the vicinity of the primary structures and components there are no significant natural slopes at the site or any steep slopes, undergoing continuous erosion.

Sections 2.5.5.1 through 2.5.5.5 are added as a supplement to the U.S. EPR FSAR.

2.5.5.1 Slope Characteristics

The characteristics of constructed and natural slopes are described below.

2.5.5.1.1 Characteristics of Constructed Slopes

Site grading areas for Callaway Plant Unit 2 includes such structures as the power block, switchyard, cooling towers, and Essential Service Water Emergency Makeup System (ESWEMS) Building and Retention Pond. The power block includes the Reactor Building, Fuel Building, Safeguards Building, Emergency Power Generating Building, Nuclear Auxiliary Building, Access Building, Radioactive Waste Building, and Turbine Building. The centerline of the Callaway Plant Unit 2 power block is graded to approximately Elevation 845 ft (258 m). The finished grade in the area of each major structure is approximately:

- Power block: Elevation 845 ft (258 m)
- Switchyard: Elevation 852 ft (260 m)
- Cooling Towers: Elevation 845 ft (258 m)
- ESWEMS Retention Pond: Elevation 840 ft (256 m)

Locations of these structures, and a schematic of the overall grading configuration, are shown in Figure 2.5L-11.

2.5.5.1.1.1 Temporary Slopes

The site grading will require the excavation of the natural overburden soils, currently estimated at approximately 40 ft (12.2 m) maximum depth. The natural soils are inadequate for foundation purposes. These will be replaced by granular fill and backfill of crushed rock composition. Category 1 Granular Structural Fills and Backfills will most likely originate from the

same sources used for the development of Callaway Plant Unit 1. Category 1 Granular Structural Fill will likely come 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 to the site close to Route 54. The cut/fill operations will not result in permanent slopes in and around the power block and around Category I structures outside the immediate power block area.

Temporary construction slopes will be implemented in cuts with a minimum 1.5:1 (horizontal to vertical) slope. The maximum possible height is approximately 40 ft (12.2 m), from the surface to the topsoil horizon of the Graydon Chert Conglomerate. There are three classifications of the overburden soils, as depicted by Figure 2.5.5-3.

2.5.5.1.1.2 Permanent Slopes

The only permanent slopes are the cut and fill slopes of the ESWEMS Retention Pond. The design of the retention pond is addressed in Sections 2.4 and 9.2.5 of the FSAR. Based on the findings of the drilling program, the ground surface elevations near the retention pond corners are 842.3 ft (256.8 m), 837.1 ft (255.2 m), 832.1 ft (253.7 m) and 841.6 ft (256.6 m). The finished grade is 840.0 ft (256.1 m). The excavation line and resulting pond side surface has a 3:1 (horizontal to vertical) slope. The depth of the excavation is 22.0 ft (6.7 m) down to El. 818.0 ft (249.4 m).

2.5.5.1.2 Characteristics of Natural Slopes

The finish grade elevation along the center line of the Power Block is 845 ft msl (258 m). Within the area of the Power Block area the natural grade changes elevation by less than 5 ft (1.5 m) over distances of approximately 500 ft (150 m). The same variation extends to the ESWEMS Retention Pond areas. Overall, the natural grade has a variation of less than 2 percent. This is the result of extended periods of wind deposition of the surface loess. There are no natural slope instability concerns in the plant vicinity. The closest slopes of consideration are located about 0.6 miles (1 km) away from the primary structures and components at the fringe of the Callaway Plant Unit 2 Plateau. Figure 2.5.5-1 and Figure 2.5.5-2 provide the location of the plateau.

2.5.5.1.3 Exploration Program and Geotechnical Conditions

The geotechnical exploration program, groundwater conditions, sampling, materials and properties, liquefaction potential, and other geotechnical parameters are addressed in Section 2.5.4. A summary relevant to the slope stability evaluation is presented below.

A geotechnical subsurface investigation was performed to characterize the upper 350 ft (107 m) of soil and rock materials at the Callaway Plant Unit 2 site. The site geology, based on geotechnical borings beneath the power block area, is comprised of glacial and postglacial soil deposits underlain by the Graydon Chert Conglomerate (bedrock), which is, on average, approximately 40 ft (12.2 m) below the ground surface. Only the deposits above bedrock are of interest for the slope stability case. The subsurface, in the upper formations, is divided into the following stratigraphic units:

- Overburden Soils:
  - Modified Loess
  - Accretion-Gley
  - Glacial Till

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

Figure 2.5.5-3 provides a general geologic 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 to 19 ft (0.9 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 overburden soils will be removed in the power block area. No permanent slopes will be implemented except for the slopes of the ESWEMS Retention Pond.

The overburden soils rest on top of the Graydon Chert Conglomerate which 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 currently supports large and safety-related structures for Callaway Plant Unit 1.

For the purpose of slope stability analyses, the shallow aquifer is of main interest. Across the plateau, the shallow aquifer is defined primarily by the Graydon Chert Conglomerate. There are localized areas where the overlying material may be a part of this aquifer, but on the whole it was found that groundwater 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 aquitard beneath it. Beneath the powerblock area the thickness of the Graydon Chert ranges from 12 ft to 55 ft (3.7 m to 16.8 m) and averages approximately 32 ft (9.8 m). At the centrally located boring R-1 (beneath the proposed power block area), the depth to the Graydon Chert Conglomerate is approximately 40 ft (11.0 m) below ground surface (bgs) and its thickness is approximately 30 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 15 ft (4.6 m) of the ground surface in the central portion of the plateau. Overall, groundwater elevations do not vary much through the

year, typically by less than 1 to 2 ft (0.3-0.6 m) across the central part of the plateau and several feet at the shallow wells around the perimeter of the plateau.

Temporary slopes will exist above the groundwater level.

2.5.5.2 Design Criteria and Analysis

The stability of constructed slopes was assessed using limit equilibrium methods, which generally consider moment or force equilibrium of a potential sliding mass by discretizing the mass into vertical slices. This approach results in a Factor of Safety (FOS) that can be defined as (Duncan, 2005):

Eq. 2.5.5-1

$$\text{FOS} = \frac{\text{Shear Strength of Soil}}{\text{Shear Stress Required for Equilibrium}}$$

Factor of Safety is defined as the ratio of the available strength of the cross section versus the forces placed upon it, such as water or seismic force. A Factor of Safety greater than one indicates that the available strength is greater than the stresses being placed upon the cross section and implies that under these particular circumstances there should be no measurable damage or permanent displacements of the cross section.

Various limit equilibrium methods are available for slope stability evaluation, including the Ordinary method (Fellenius, 1936), Bishop's simplified method (Bishop, 1955), Janbu's simplified method (Janbu, 1968), and the Morgenstern-Price method (Morgenstern, 1965), among others. These methods were selected for evaluation of slopes for they are routinely used, and their limitations, and advantages, are well documented. The main differences are:

1. Equations of statics that are included and satisfied
2. Interslice forces that are included in the analysis
3. Assumed relationship between the interslice shear and normal forces

The Ordinary (Fellenius, 1936) method is one of the earliest methods developed. It ignores all interslice forces and satisfies only moment equilibrium. Both Bishop's (Bishop, 1955) simplified method and Janbu's (Janbu, 1968) simplified method include the interslice normal force, E, but ignore the interslice shear force. Bishop's (Bishop, 1955) and Janbu's (Janbu, 1968) simplified methods satisfy only moment equilibrium and horizontal force equilibrium, respectively.

The slope stability analysis is performed using the latest version of Computer Program GSTABL7 with STEDwin (Gregory 2003). This program was originally developed by Purdue University for the Indiana State Highway Commission in 1986 and later revised and marketed by Geotechnical Engineering Software Company. The program calculates the factor of safety against slope failure utilizing a two-dimensional limit equilibrium method. The calculation of the factor of safety against slope instability is performed using the Simplified Bishop method of slices, which is applicable to circular shaped failure surfaces, the Simplified Janbu method of slices, which is applicable to failure surfaces of a general shape, or Spencer's method of slices which is applicable to surfaces having a circular or general shape.

Dynamic analysis of the slopes can be performed using a pseudo-static approach, which represents the effects of seismic shaking by accelerations that create inertial forces. These forces act in the horizontal and vertical directions at the centroid of each slice, and are defined as:

$$F_h = (a_h / g)W = k_h W \quad \text{Eq. 2.5.5-2}$$

$$F_v = (a_v / g)W = k_v W \quad \text{Eq. 2.5.5-3}$$

Where a_h and a_v are horizontal and vertical ground accelerations, respectively, W is the slice weight, and g is the gravitational acceleration constant. The inertial effect is specified by k_h and k_a coefficients, based on site seismic considerations.

Typical minimum acceptable values of FOS are 1.5 for normal long-term loading conditions and 1.0 to 1.2 for infrequent loading conditions (Duncan, 2005), e.g., during earthquakes.

2.5.5.2.1 Stability of Permanent Slopes

The ESWEMS Retention Pond at the Callaway Plant Unit 2 is constructed primarily via excavation below existing grade with earthen embankment sides. Fill is utilized in some areas for the purpose of the embankment meeting grade.

The excavation is cut through the first three layers of the site subsurface materials. These layers comprise Modified Loess, Accretion-Gley, and Glacial Till. The Graydon Chert Conglomerate underlies these layers. The soil profile was verified with four borings placed directly within the area near and surrounding the ESWEMS Retention Pond.

Four separate sections are analyzed to represent the various design differences of slopes for the ESWEMS Retention Pond. These are shown by Figure 2.5.5-5. Two sections include embankment construction on the land side to meet grade elevation. The third section is without the embankment construction consistent with the existing grade. The fourth is a typical spillway cross section. Figure 2.5.5-6 shows typical dry, construction cross-sections. The permanent section locations can be found on the plan view in Figure 2.5.5-7

Since the ESWEMS Retention Pond is constructed by excavating from grade there is no downstream slope of significance to require a stability analysis. The upstream sections are all more critical, and govern the analysis.

The analyses are performed for steady state loading conditions as well as earthquake loading conditions. The total stress strength parameters of the soils, as specified by Table 2.5.5-2, are utilized in the analyses.

Both circular and wedge analyses are performed on the cross sections. The circular failure analysis uses the Simplified Bishop Method. Wedge analysis utilizes the Simplified Janbu method.

Phreatic surface is defined as the boundary between the saturated and unsaturated zones in the cross sectional profile. Normally, Casagrande's solution for seepage through an earthen dam can be used to calculate the expected phreatic surface. Under the conditions presented for the ESWEMS Retention Pond there is no significant downstream slope thereby making Casagrande's equation non-applicable to this situation. Instead engineering judgment based on previous similar applications was used in order to approximate a conservative phreatic surface for the profile. The phreatic surface is realistic and conservative for a gradual drop in water surface from El 835 (the normal water level) as it passes through the embankment

The Safe Shutdown Earthquake (SSE) load consists of one horizontal load applied pseudo-statically to the model. A peak horizontal seismic acceleration of a_h = 0.3 g was used in the analysis. The a_h = 0.3 g corresponds to the peak ground acceleration of the Foundation Input Response Spectra (FIRS) at an elevation of 823 ft. The FIRS study also found the vertical

seismic coefficient to be approximately a_v = 0.3 g. The FIRS study is described in Section 2.5.2. A Pseudo-Static stability analysis has been performed. Total stress parameters for the soil properties are being used for the earthquake loading analysis.

In order to find the worst case slope failure, the program GSTABL7 allows the calculation of numerous iterations and provides the corresponding location and Factor of Safety of the worst case scenario. The worst case is the location exhibiting the lowest Factor of Safety.

The slope stability analyses results are summarized in Table 2.5.5-3 and the critical failure surfaces are shown in Figure 2.5.5-6. These results indicate that the Callaway Plant Unit 2 ESWEMS Retention Pond side slopes have Factor of Safety values ranging from 1.9 to 9.1, depending upon the slope configuration and analysis method. Therefore it can be stated that the current design is safe.

The ESWEMS Retention Pond slopes are the only permanent slopes planned for Callaway Unit 2.

2.5.5.2.2 Stability of Temporary Fill Slopes

Temporary cut and fill slopes will exist in dry conditions during construction. The slope stability calculations assume that the groundwater level is at the Graydon Chert Conglomerate top horizon. The soil properties used are as recommended by Table 2.5.5-1. Temporary slopes should not have a horizontal to vertical ratio steeper than 1.5:1. The height of the slope should be limited to 20 ft (6.1 m), providing horizontal benches of at least 10 ft (3.3 m) whenever the height reaches 20 ft (6.1 m). Figure 2.5.5-4 provides an example of a temporary slope on fill.

The slope stability analyses are performed for the sections shown by Figure 2.5.5-6. Results are summarized in Table 2.5.5-3 and the critical failure surfaces are shown in the sections of Figure 2.5.5-6. These results indicate that the temporary side slopes have Factor of Safety values ranging from 2.5 to 3.7 depending upon the slope configuration and analysis method. Therefore it can be stated that the current design is safe.

2.5.5.2.3 Concluding Remarks

Based on analyses provided in this Section, it is concluded that the planned constructed and natural slopes at the site are sufficiently stable, exhibit Factor of Safety values in excess of the typical minimum acceptable values noted in 2.5.5.2, and present no failure potential.

2.5.5.3 Logs of Borings

Log of borings, and associated references, are provided in Part 11F.

2.5.5.4 Compacted Fill

Compacted fill, and associated references, are addressed in Section 2.5.4.5.

2.5.5.5 References

Bishop, A.W., 1955. "The Use of Slip Surface Circle in Stability Analysis of Slopes", Geotechnique Vol. 5 No.1, pp 7-17, 1955.

Das, B., 2002. "Principles of Geotechnical Engineering, Fifth Edition", Brooks/Cole, 2002.

Duncan, J.M. and, Wright, S. G., 2005. "Soil Strength and Slope Stability", Wiley & Sons, 2005.

Fellenius, W., 1936. "Calculation of the Stability of Earth Dams. Proceedings of the Second Congress of Large Dams, Vol. 4, pp. 445-463, 1936.

Gregory, Garry H., 2003. " GSTABL7 with STEDwin: Slope Stability Analysis System Program Manual", 2003.

Janbu, N., 1968. "Slope Stability Computations" Soil Mechanics and Foundation Engineering, The Technical University of Norway, 1968.

Morgenstern, N.R., and Price, V.E. 1965. "The Analysis of the Stability of General Slip Surfaces", Geotechnique, Vol. 15, pp. 79-93, 1965}

2.5.6 REFERENCES

No departures or supplements.

{References are provided for each subsection}

Table 2.5.5-1—{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 2.5.5-1—{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

Table 2.5.5-2—{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 2.5.5-1 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	

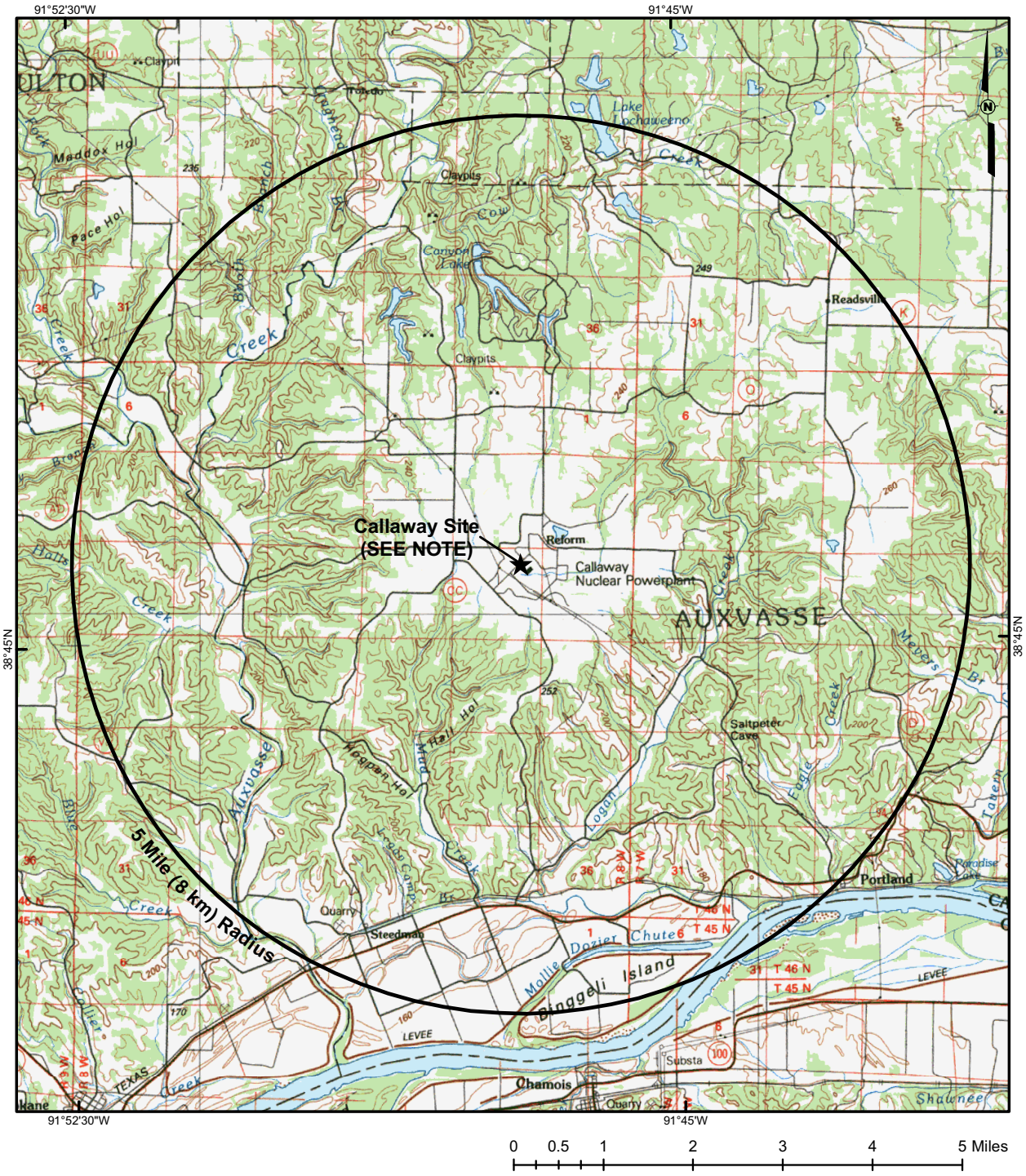
Table 2.5.5-2—{Recommended Values for Strength Properties (SI Units)}

	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 2.5.5-1. 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	

Table 2.5.5-3—{Factor of Safety Against Sliding}

SECTION		Circular Failure		Wedge Failure		Observations
		Static	Dynamic	Static	Dynamic	
PERMANENT ESWEMS SLOPES	Section 1	7.9	2.5	9.1	3.2	
	Section 2	6.7	2.1	6.8	2.5	
	Section 3	5.8	1.9	6.8	2.1	
	Section 4	7.5	2.3	6.9	2.4	
TEMPORARY SLOPES	Section 1	3.2	N/A	3.7	N/A	Note: Dynamic case not applicable for temporary slopes
	Section 2	2.5	N/A	2.5	N/A	

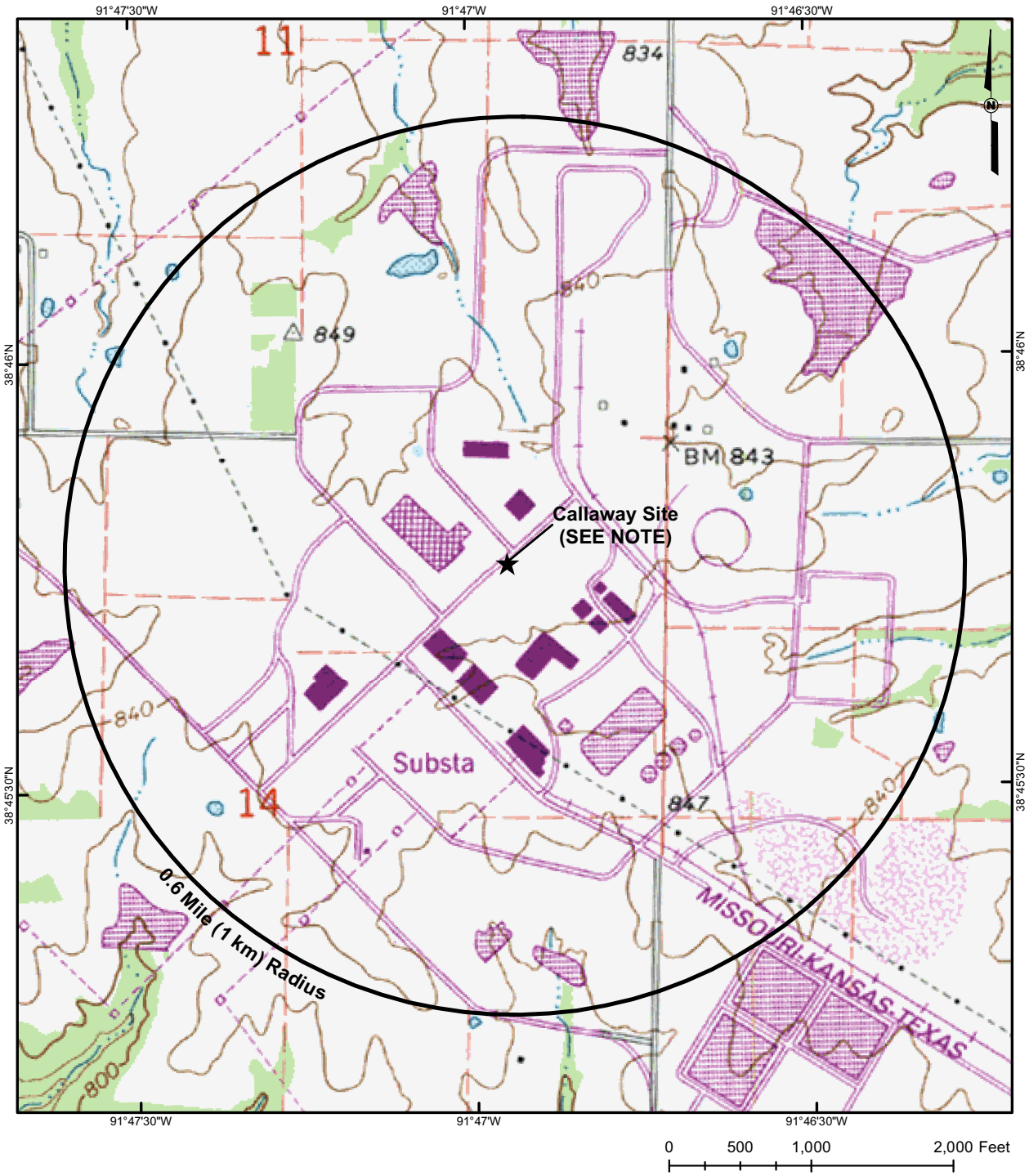
Figure 2.5.5-1—{Site Topographic Map 5 Mile (8 km) Radius}



NOTE:
REFERENCE CENTER POINT OF PLANT SITE
IS DEFINED AS THE MIDPOINT BETWEEN EXISTING
REACTOR FOR CALLAWAY PLANT UNIT 1 AND
REACTOR FOR CALLAWAY PLANT UNIT 2.

REFERENCE:
USGS,1985a.

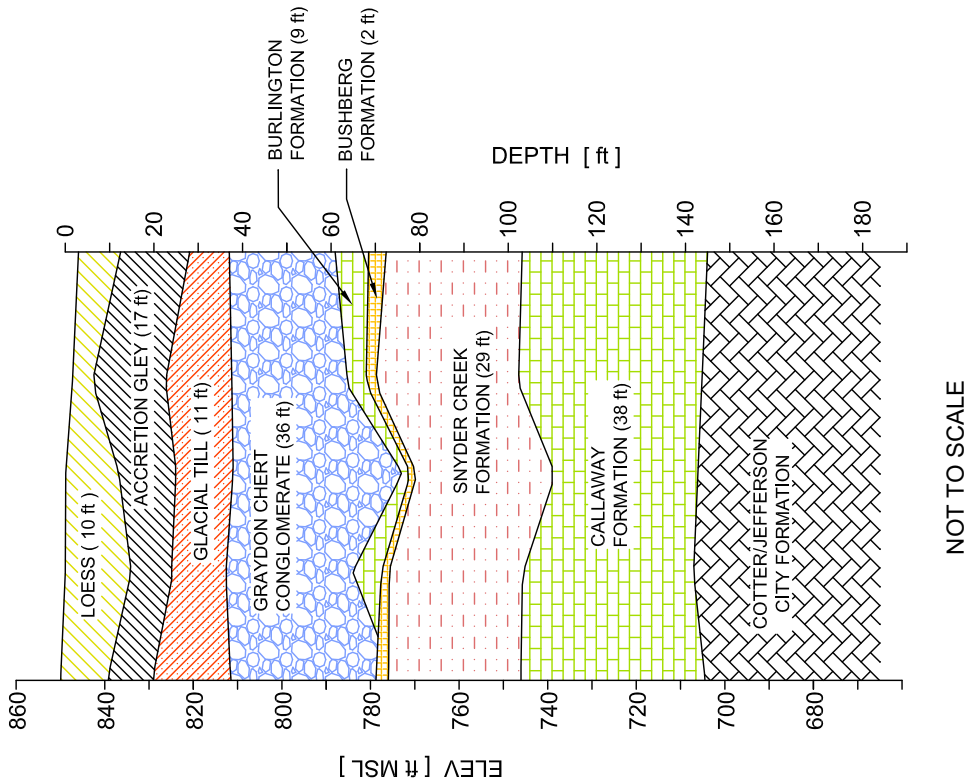
Figure 2.5.5-2—{Site Area Topographic Map 0.6 Mile (1 km) Radius}



NOTE:
REFERENCE CENTER POINT OF PLANT SITE IS DEFINED AS THE MIDPOINT BETWEEN EXISTING REACTOR FOR CALLAWAY PLANT UNIT 1 AND REACTOR FOR CALLAWAY PLANT UNIT 2.

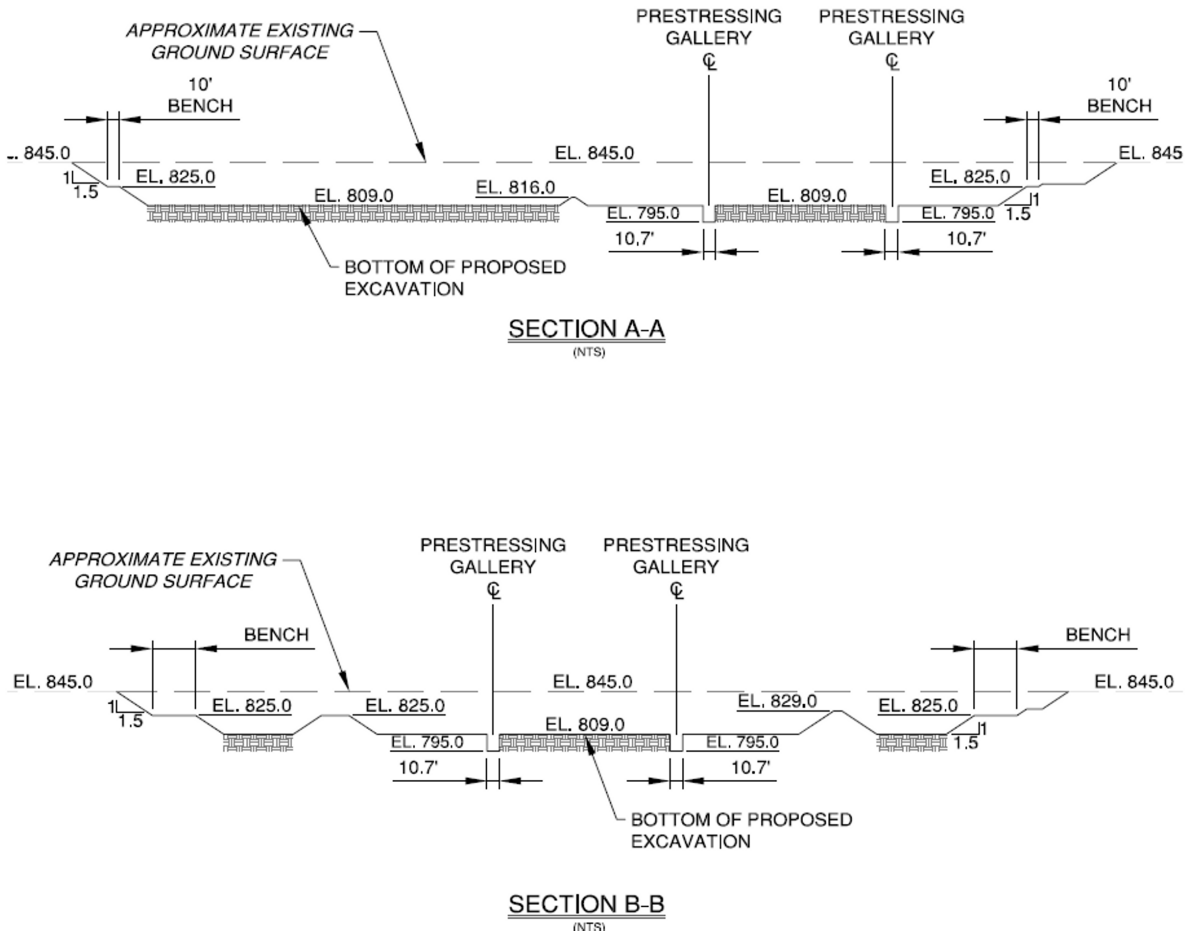
REFERENCE:
USGS,1985b.

Figure 2.5.5-3—{Generic Soil Profile}



FSAR: Figures 2.5.5

Figure 2.5.5-4—{Excavation Cross Sections}



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.

Figure 2.5.5-5—{Permanent Slope Cross Sections and Failure Surfaces}
(Sheet 1 of 2)

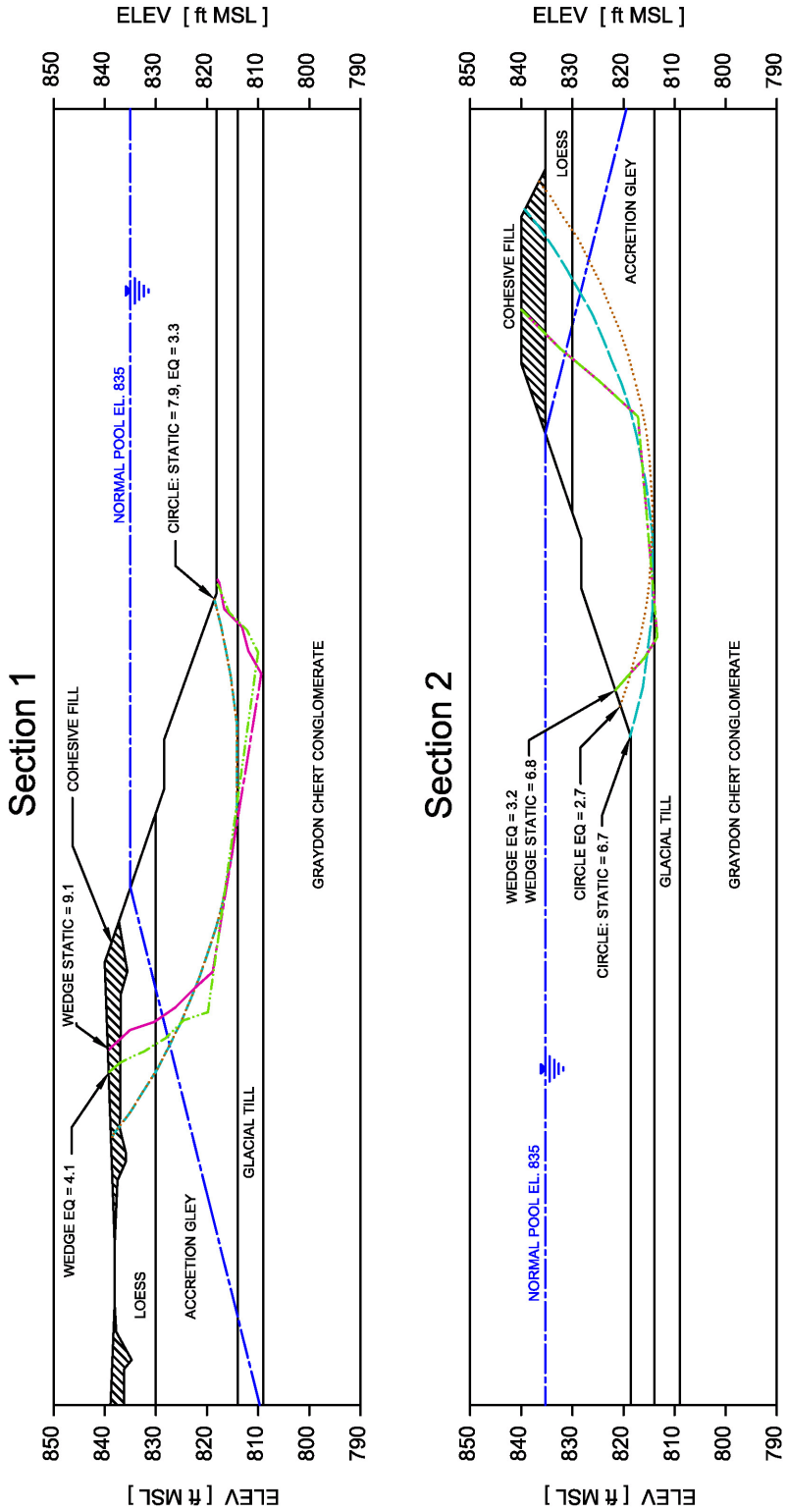
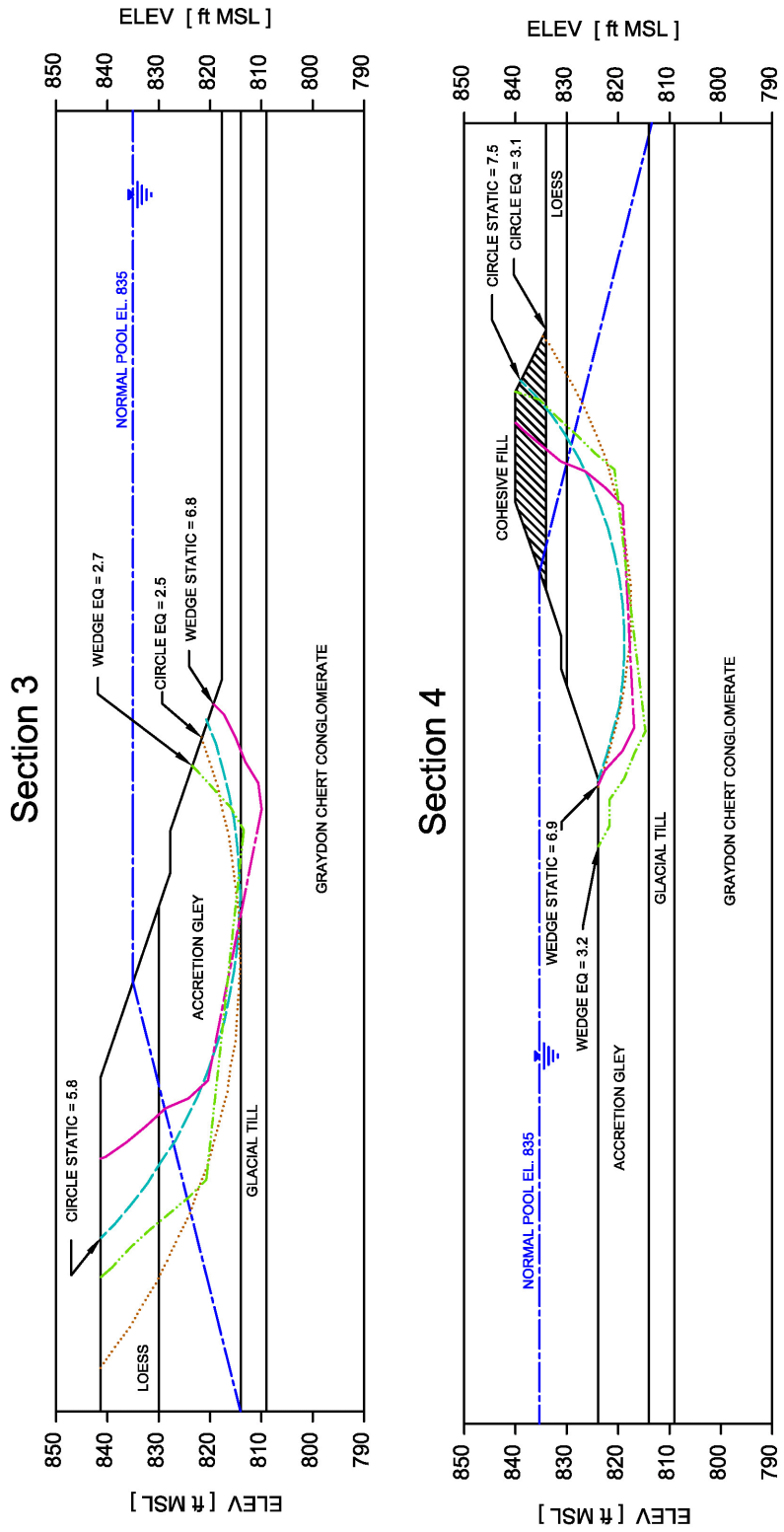


Figure 2.5.5-5—{Permanent Slope Cross Sections and Failure Surfaces}
(Sheet 2 of 2)



FSAR: Figures 2.5.5

Figure 2.5.5-6—{Temporary Slope Cross Sections and Failure Surfaces}

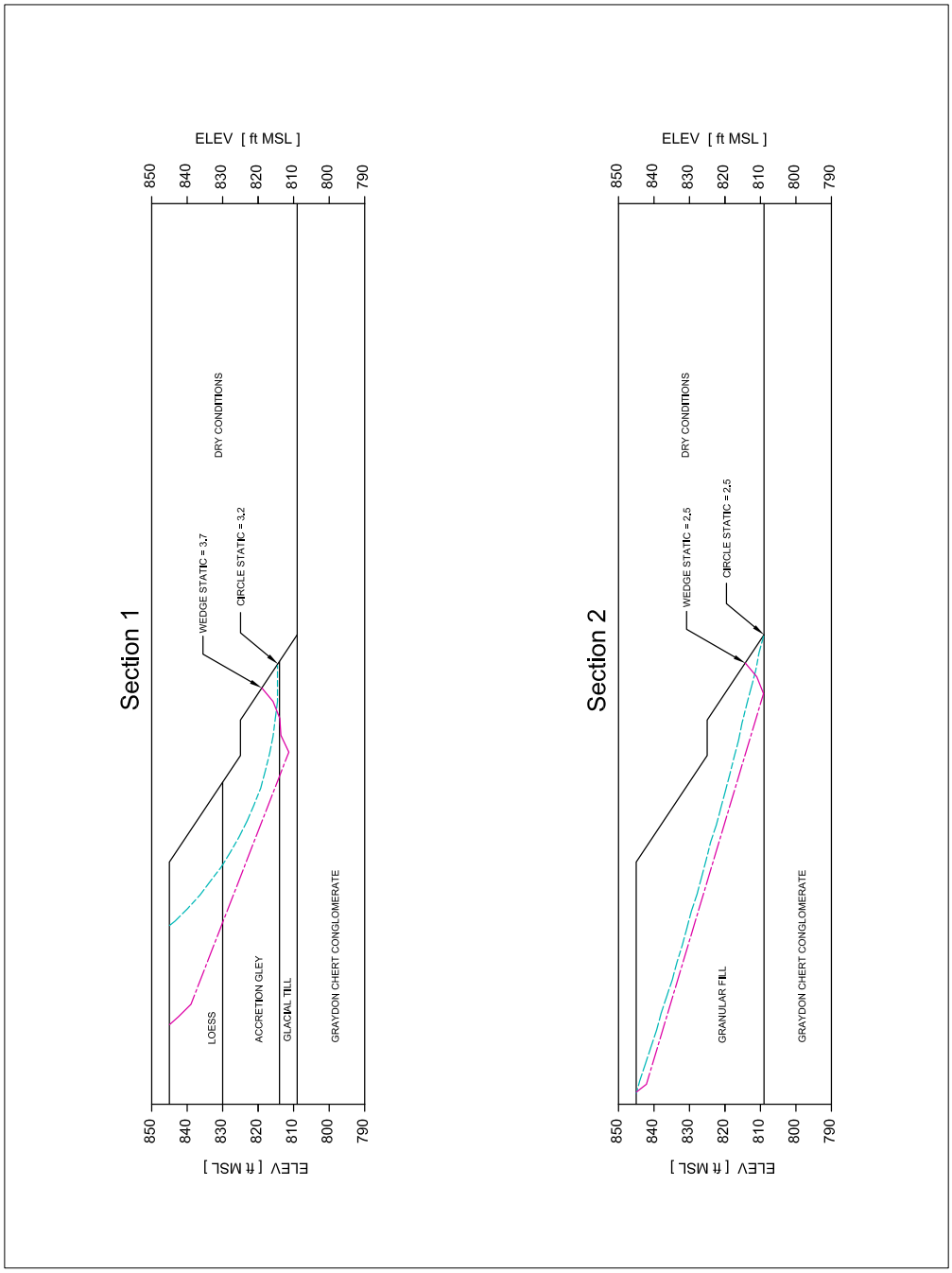


Figure 2.5.5-7—{ESWEMS Pond Plan View}