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CHAPTER 2.0

SITE CHARACTERISTICS

This chapter of the Site Addendum provides detailed information on the geological, seismological, hydrological and meteorological characteristics of the site and vicinity. Also discussed therein, are site activities and controls, population distribution and land use.

The discussions provided in Chapter 2.0 of the power block FSAR are those considered necessary to augment the Site Addendum discussions as they pertain to the power block envelopes or the design and analyses of power block structures, systems and components.

During the PSAR stage, when the power block envelopes were being developed, there were four sites (Callaway, Wolf Creek, Sterling and Tyrone) upon which five plants were to be built. Now, there are two sites upon which two plants are to be built.

The SNUPPS design envelopes were developed by use of the most restrictive site conditions imposed by any one of the four original sites or by generic design criteria which are conservative for each of the sites. With the cancellation of the Tyrone plant, however, the four site enveloping approach was modified in the seismic design area (development of spectra et. al) for work not yet completed to include only the three remaining sites. Refer to Sections 2.5 and 3.7(B) for details. The design envelopes were not revised to reflect the cancellation of Sterling.

The elevations given are based on the 1929 mean sea level (msl) datum. The geographic (latitude and longitude) and Universal Transverse Mercator coordinates given are based on the North American Datum of 1927. The Missouri State Plane coordinates given are based on the Missouri Coordinate System of 1927 - Central Zone.

2.1 GEOGRAPHY AND DEMOGRAPHY

Refer to the Site Addendum

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

2.2.1 LOCATIONS AND ROUTES

Discussions and maps indicating the locations of all military bases, missile sites, manufacturing plants, chemical plants and storage facilities, airports, transportation routes, oil and gas pipelines, military firing ranges, and air traffic patterns are provided in Section 2.2.1 of the Site Addendum.

2.2.2 DESCRIPTIONS

A description of products manufactured, stored, or transported offsite is provided in Section 2.2.1 of the Site Addendum. Onsite hazards are discussed in Section 2.2.3. Evaluations of onsite and offsite hazards of consequence are also presented in Section 2.2.3 of the Site Addendum.

2.2.3 EVALUATION OF POTENTIAL ACCIDENTS

For this section, the term "significant hazard" is defined as any hazard against which design provisions must be considered to protect the plant or which must be assessed in detail for consequences serious enough to affect the safety of the plant.

There are no onsite or offsite hazards which are expected to have an adverse effect on the plant structures. Refer to Section 2.2.3 of the Site Addendum for the evaluation of potential accidents.

2.3 METEOROLOGY

2.3.4 SHORT-TERM (ACCIDENT) DIFFUSION ESTIMATES

2.3.4.1 Objective

The objective of this section is to provide short-term atmospheric dispersion factors (χ/Qs) for the postulated accident analyses presented in Chapter 15.0.

2.3.4.2 Calculations

2.3.4.2.1 Site Boundary and LPZ

The short-term atmospheric dispersion factors (χ/Qs) are based on onsite meteorological data for the Callaway Plant site. The diffusion equations and assumptions used in the calculations were those outlined in NRC Regulatory Guide 1.145, "Atmospheric Dispersion Models for Potential Accident Assessment at Nuclear Power Plants." Table 2.3-1 lists the limiting χ/Qs for the Callaway site. The detailed procedures used in the calculations are given in Section 2.3.4.2 of the Site Addendum.

2.3.4.2.2 Control Room Intake

The basic model employed for the distribution of relative concentrations (χ/Qs) within a building wake at the Callaway control room intakes following an accident is given by Reference 1 to be:

$$\chi / Q = \frac{K_C}{AV} \qquad (1)$$

Where A = reference cross-sectional building area, m^2

V = reference wind speed, m/sec

K_C = nondimensional concentration coefficient

K_C is a function of nondimensional space coordinates x/L, y/L, and z/L, building configuration, wind direction, and source configuration. The K_C field for a given building configuration, source configuration, and wind configuration is considered to be invariant. Accordingly, K_C values determined by wind tunnel tests with a model structure are expected to be the same as those that would be obtained with a geometrically similar building in the full-scale atmosphere in the same wind direction, with a similar leak. The Callaway Plant contiguous building arrangement is shown in Figure 2.3-1. The K_C data used in the analysis for low level release are presented in Figure 2.3-2 and were derived from two sets of tests. One used rectangular prisms (Ref. 2), the other used a model of the EBR-II complex (Ref. 1). Both tests were described and portions of the data presented in Reference 3. The K_C data for the unit vent release from the top of the containment were extracted from Figure 10 of Reference 1 and are presented in Table 2.3-2. The value of A used in conjunction with K_C in Figure 2.3-2 and Table 2.3-2 is the Callaway Plant equivalent of the EBR-II area, $A = 1.12\ D^2 = 2280\ m^2$ with the diameter of the reactor D = 45.1 m.

The value of V used in conjunction with Figure 2.3-2 is the mean velocity of the approach flow at an elevation corresponding to the anemometer elevation of the EBR-II model tests. Reference 3 reports this elevation to be 62 feet or 0.77D above the top of the dome. The Callaway Plant equivalent height becomes 63.4 + 0.77 x 45.1 = 98.1m above ground. The V values were obtained by extrapolating wind speeds at anemometer elevations equivalent to 98.1 meters by the power law.

$$V = u_1(98.1/z_1)^n \qquad (2)$$

Where u_1 = mean speed at elevation z_1, m/sec

z_1 = anemometer elevation at a given site, m

n = atmospheric stability exponent

Values of n were arbitrarily assumed for the various stability classes as follows:

Pasquill Stability Class	A	B	C	D	E	F	G
n	0.20	0.25	0.29	0.33	0.40	0.50	0.60

A cumulative frequency distribution was constructed for the χ/Q values calculated by equations 1 and 2 above, using 3 years combined onsite meteorological data. The corresponding highest 5 percent, 10 percent, 20 percent, and 40 percent χ/Q values are given in Table 2.3-3.

2.3.5 REFERENCES

1. Halitsky, J., Golden, J., Halpern, P., (1963): "Wind Tunnel Tests of Gas Diffusion From a Leak in the Shell of a Nuclear Power Reactor and from a Nearby Stack," N. Y. University Department of Met. & Ocean, GSL Rep. 63-2 under USWB Contract Cwb-10321

2. Halitsky, J. (1963): "Gas Diffusion Near Buildings," ASHRAE Trans. 69: pp. 464-484

3. Slade, D. H., ed. (1968): "Meteorology and Atomic Energy," U. S. AEC Division of Technical Information TID-24190

TABLE 2.3-1 LIMITING ATMOSPHERIC DISPERSION FACTOR, $\chi/Q(sec/m^3)$

<u>Site Boundary</u>	χ/Q
0-2 hr.	2.OE-4
<u>Low Population Zone</u>	
0-8 hr.	2.6E-5
8-24 hr.	1.7E-5
24-96 hr.	7.2E-6
96-720 hr.	2.0E-6

TABLE 2.3-2 VARIATION OF INTAKE K_C WITH WIND DIRECTION UNIT VENT RELEASE

Wind Direction	K_c
N	1.5
NNE	0.5
NE	0
ENE	0
E	0
ESE	0
SE	0
SSE	0
S	0
SSW	0
SW	0
WSW	0
W	0
WNW	0.5
NW	1.5
NNW	2.5

TABLE 2.3-3 RELATIVE CONCENTRATION (χ/Q) AT CONTROL BUILDING AIR INTAKE*

From Low Level Release

Percentage	(χ/Q)
5	7.18
10	5.28
20	1.66
40	0

For Unit Vent Release

Percentage	(χ/Q)
5	1.33
10	0.90
20	0.41
40	0

* Units for χ/Qs are 10^{-4} sec/m^3

2.4 HYDROLOGIC ENGINEERING

This section of the Site Addendum describes the location and the hydrologic features of the site and the method of analysis adopted in evaluating the hydrologic conditions at safety-related structures.

2.4.2 FLOODS

2.4.2.3 Effects of Local Intense Precipitation

2.4.2.3.1 Site Drainage

The storm drainage system at the site is designed in accordance with the parameters discussed in Section 2.4.2.3 of the Site Addendum. Site drainage is designed to convey local flooding resulting from rainfall on the roofs and the site area without endangering safety-related structures.

The design basis for the roof drainage system is a rainfall intensity of 7.4 inches per hour with a recurrence interval of 100 years. Any rainfall in excess of this design intensity will overflow the roof curb and the building walls to the site drainage system.

During a local probable maximum precipitation, the storm drainage system carries runoff up to the design capacity. Runoff in excess of the design capacity flows outside the system to the natural drainage outlet of the site. Provision is made in the design of the plant yard grading to prevent backwater from endangering safety-related structures. Refer to the Site Addendum for a discussion of the site drainage system.

2.4.2.3.2 Ice and Snow

Estimation of the snow load on the roofs of the safety-related structures is based on a frequency analysis of snowpack on the ground combined with a local winter probable maximum precipitation (PMP), which is assumed to occur in the form of snow. The assumption of PMP to occur in the form of snow is very conservative, since snowfall depends on latitude, altitude, and temperature. By combining the antecedent snowpack and the PMP in the form of snow, additional conservatism of the snowpack load is obtained.

Historical snowpack depth data at stations near the SNUPPS sites are analyzed statistically for the months of December through March. Frequency analysis of the maximum monthly snowpack is performed on the data, and a 100-year frequency snowpack depth for each month is selected from the analysis. Based on the analyses presented in the Site Addendum, the months of February and March exhibit the highest combined snowpack and winter PMP load.

In estimating the snow load on the roofs of the safety-related structures, the effects of wind action on the snowpack and snowfall are considered. This effect tends to decrease the load on elevated buildings and could increase the load on adjacent low roofs due to snow drifts.

Site drainage and plant yard grading are designed to handle the runoff from local winter PMP without endangering safety-related structures. In establishing the required grading and outlets, clogging of inlets and certain size culverts by ice is assumed in the design.

The maximum postulated ground snow loads have been developed based on frequency analyses of the maximum snowpack on the ground, combined with the local probable maximum winter precipitation in the form of snow. The roof snow load used in the design of the safety-related structures is determined by multiplying the ground snow load by the appropriate coefficient C_S given in Figure 2.4-1. The minimum roof snow load used in design is taken as 0.8 times the ground snow load and is increased on the lower levels of multilevel roofs and on roof areas adjacent to projections to account for wind action and drifting. The maximum drifted snow load is taken as 3 times the enveloping ground snow load.

Two snow loading conditions are analyzed in the design of safety-related structures for the standard plant, as described in Section 3.8.4.3. The maximum 100-year-recurrence snowpack from each of the SNUPPS sites is analyzed, in combination with other live loads. The enveloping 100-year-recurrence ground snowpack load for the SNUPPS sites is 91 psf, as shown in Table 2.4-1. This load is increased or decreased when applied to roofs, in accordance with the coefficients given in Figure 2.4-1.

In addition, the probable maximum winter precipitation, PMP (winter), in the form of snow, coincident with the 100-year-recurrence snowpack, is analyzed in combination with other normal operating live loads. The enveloping 100-year-recurrence ground snowpack plus PMP (winter) for the SNUPPS sites is 153 psf. This load is increased or decreased when applied to the roofs in accordance with the coefficients given in Figure 2.4-1.

The maximum postulated ground snow load for the site is presented in Section 2.4.2.3 of the Site Addendum. These values are shown in Table 2.4-1. The snow load is combined with other loads, in accordance with the loading combinations presented in Section 3.8.4.3.

TABLE 2.4-1 DESIGN GROUND SNOW LOAD

	100-Year Recurrence Snowpack Load psf	PMP (Winter) Snow Load With 100-Year Recurrence Snowpack psf
Standard plant facilities	91(1)	153
Safety-related site facilities	21	123

(1) The 91 psf load is based on data from the Sterling site and has been retained, even though the Sterling unit has been cancelled.

2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

This section of the Site Addendum provides detailed information on the geological and seismological characteristics of the site. The Site Addendum also provides the methods, criteria, and findings of the investigations. Based on the results of those investigations, it is concluded that there are no geological, seismological or foundation support conditions that adversely affect the design, construction and operation of the nuclear plant at the site.

During the PSAR stage, when the SNUPPS power block structures, systems and components were first being designed, there were four sites (Callaway, Wolf Creek, Sterling, and Tyrone) upon which five plants were to be built. Now there are only two sites (Callaway and Wolf Creek) upon which two plants are to be built.

The final geological and seismological design of the power block structures, systems and components is based on three sites (Callaway, Wolf Creek and Sterling) to ensure conservatism in the seismic design envelope. Certain items, whose final design was completed prior to the cancellation of Tyrone (the fourth site), are within the envelope for the four original sites.

The discussions provided below are considered necessary to augment the Site Addendum discussions as they pertain to the power block design envelopes and the design and analyses of power block structures, systems and components.

2.5.2 VIBRATORY GROUND MOTION

2.5.2.8 Response Spectra

Response spectra for the SSE and OBE are presented in Section 2.5.2 of the Site Addenda. Figures 2.5-1 through 2.5-6 present the response spectra for the standard plant facilities and demonstrate that the selected spectra for the standard plant SSE and OBE envelope the bounds of the spectra for each site.

The response spectra are scaled or normalized to the maximum horizontal ground acceleration for the SSE of 0.20 g and for the OBE of 0.12 g in accordance with Regulatory Guide 1.60.

Seismic Category I structures other than the standard plant facilities listed in Section 1.2.2.1 are designed using the applicable response spectra for the site.

2.5.4 STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

2.5.4.9 Earthquake Design Basis

All seismic Category I standard plant structures are designed for an SSE of 0.20 g maximum horizontal and vertical ground acceleration. The OBE for the site is a minimum of one-half the ground acceleration of the selected SSE. All seismic Category I standard plant structures are designed for an OBE of 0.12 g maximum horizontal and vertical ground acceleration. Individual systems and structures for individual sites other than the standard plant structures are designed for the SSE and OBE values developed for each site.

2.5.4.10 Static Stability

All standard plant seismic Category I structures are supported on reinforced concrete mat foundations. These structures, and the maximum static design loads for each, are listed below.

Structure	Applied Static Load (psf)
Reactor building	7,500
Auxiliary building	7,900
Fuel building	10,600
Control building	7,900
Diesel generators building	5,300
Refueling water storage tank	3,700
Emergency fuel oil storage tanks (Buried)	4,400

The relationships of the foundations of the various plant buildings to each other and to the subsurface materials are shown in the Site Addendum. Generalized subsurface conditions in the plant area for the site are summarized on Figure 3.7(B)-11A for ease of review. The foundations of the standard plant structures and systems are designed for the subsurface conditions that result in the most conservative foundation thickness and reinforcing steel. The soil and rock parameters used for design are given in the Site Addendum. Their derivation from field and laboratory tests is discussed.

2.5.4.10.1 Bearing Capacity

The methods and results of the determination of static and dynamic bearing capacity are presented in Section 2.5.4 of the Site Addendum. The minimum factor of safety against bearing capacity failure is 3 for static loading and 2 for dynamic loading. Computed safety factors exceed these values.

2.5.4.10.2 Settlement

The methods and results of the determination of settlement for major plant structures are presented in Section 2.5.4 of each Site Addendum.

2.5.4.10.3 Lateral Earth Pressures

Materials available at the site location are used for backfill around the structures, as described in the Site Addendum. Compaction criteria are established for the site to result in sound, homogeneous backfills. The parameters used to develop the pressures for each static at-rest condition are given in Section 2.5.4 of the Site Addendum. The dynamic lateral earth pressures are computed based on the theory developed by Mononabe-Okabe as simplified by Seed and Whitman (Ref. 1).

The subsurface walls for the seismic Category I standard plant structures are designed as rigid, restrained walls to resist static at-rest and dynamic pressures. The lateral earth pressures used in the design of these walls are based on the maximum pressures developed at any site. The equations developed for each site, as shown in Figure 2.5-7, are used in conjunction with the respective site dependent soil parameters and the enveloping earthquake loads to compute the lateral pressures at the top and bottom of the subsurface walls at each site. The maximum earth pressures thus computed are taken as the enveloping pressures and are used in design. In addition, a minimum surcharge of 250 pounds per square foot is assumed to act over the backfill surface, and the resulting pressures on the subsurface walls are included in the design loads. Similarly, the surcharge loads of foundations located near the subsurface walls are included in the design of the walls.

2.5.7 REFERENCES

1. Seed, H. B., and Whitman, R. V., "Design of Earth Retaining Structures for Dynamic Loads," Proceedings of the Specialty Conference on Lateral Stresses in the Ground and Design of Earth-Retaining Structures, Cornell University, Soil Mechanics and Foundation Division, ASCE, June 1970, pp. 103-147.