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LASALLE COUNTY NUCLEAR GENERATING STATION

Final foundation levels, dimensions, and static loads for the plant structures are shown in Table 2.5-30 and Figure 2.5-57. The settlement due to construction is computed based on the increase in effective stress equal to the gross foundation pressure minus the uplift pressure.

The settlement analyses were performed using Janbu's tangent modulus method (Reference 297, Janbu, 1967). The consolidation parameters of the foundation subsurface materials used in the analysis are evaluated from the results of laboratory consolidation tests (summarized in Table 2.5-40). The actual computations were made using the computer program SETTLE developed by Sargent & Lundy. The description of the SETTLE program is presented in Appendix F of the FSAR.

The foundation settlement has been investigated by assuming the structural foundation system to be either completely rigid or completely flexible. The actual field settlement behavior of the main plant is bounded by these two extreme cases.

For both foundation cases, the effective foundation pressure is placed directly at the base of the structure. For the completely flexible case, the rigidity of foundation and superstructure system is neglected. For the completely rigid foundation case, the distribution of contact pressure due to the effect of the foundation rigidity is taken into account by considering linear settlement. An iterative procedure is used so as to make the settlement pattern of the foundation and the subsoil compatible. The iterative procedure is illustrated by a flowchart shown in Figure 2.5-91. This iterative procedure has been included in the computer program SETTLE.

The computed maximum rebound values range from 0.84 to 2.76 inches. Based on the estimated time-rate of consolidation data (Table 2.5-40), the rebounds occur quickly with the excavation operations. These excavations also remove all the rebounded soil. Because foundations are placed at their design elevation, these rebounds will not affect the subsequent settlement due to the construction of plant structures.

The final settlement contours due to construction for the major structures at the plant site are shown in Figure 2.5-58, Sheets 1 and 2. The predicted maximum and minimum final settlements are 0.88 inches to 2.46 inches for a rigid foundation and 0.24 inches to 3.37 inches for a flexible foundation.

The settlements are monitored by the subsurface instrumentation program described in Subsection 2.5.4.13. The settlement readings for all monument points are presented in Figure 2.5-67. The comparison of theoretical final settlements and latest measured settlements at the monument points are presented in Table 2.5-41. The theoretical settlement agree reasonably with the measured values.

Theoretical time-settlement curves have been plotted for measurement point TR2 within the turbine building mat and measurement point R1 within the reactor

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A settlement analysis was performed to estimate the probable settlement due to the consolidation of the soil beneath the peripheral dikes and the embankment material itself. In this study, the subsoil conditions along the dike alignment and the variable heights of the dike were considered.

Embankment consolidation was determined to be 0.6% of the height of the proposed dike. This conservative value is consistent with published data that is based on past experience, as described in the design manual (Reference 306, U.S. Department of the Navy Naval Facilities Engineering Command, 1971). This manual suggests a range of 0.3% to 0.6% of the dike height based on embankments with crest widths less than 20% of the embankment height. A value of 0.6% corresponds to approximately 3 inches of settlement for the maximum dike height of 40 feet.

The foundation consolidation has been determined from representative laboratory consolidation test data from the Wedron silty clay till. The upper portion of the Wedron silty clay till was found to consolidate a maximum of 5 inches during the plant life of 40 years. This settlement calculation is based on a total dike height of 40 feet, which represents only 2% of the total dike length.

The lower portion of the Wedron silty clay till has been heavily overconsolidated which, when loaded with the light loads due to the embankment, will produce negligible settlement. Computation of this settlement is based on time settlement data which corresponds to 68% of the total consolidation for the 40-year plant life. The percentage of consolidation at 10, 20, and 30 years is 35%, 50%, and 59% respectively. The settlements occur over a very long period of time and do not endanger the structural stability of the dike. As the heights of the peripheral dike are variable, the settlements along the dike alignment vary. Combined settlement estimates for the foundation and the embankment were made for different dike heights, and a camber (as shown in Table 2.5-35 and Figure 2.5-41) was provided along the longitudinal direction of the peripheral dikes to compensate for the variable settlements. This was to ensure that there was the required freeboard above the maximum water level in the lake and to maintain an adequate crest elevation at all times. Settlement monuments and foundation settlement measuring devices were established along the entire dike system to monitor the vertical movements of the soil mass in the dike and in the base, as described in Subsection 2.5.6.8.

2.5.6.4.3 Slope Protection

The upstream slope of the peripheral dike was protected against wave action by means of riprap 18 inches in thickness measured perpendicular to the slope over a crushed stone bedding course extending from elevation 694 feet MSL to the crest, as shown in Figure 2.5-70. The riprap was designed, using the United States Army Corps of Engineers publication, Shore Protection Manual (1973), to withstand wave action due to 40-mph winds coincident with the probable maximum water level in

RIVER BEND NUCLEAR POWER STATION

TABLE 2.5-19

TOTAL SETTLEMENTS OF MAJOR STRUCTURES

	Settlement Marker No.	Predicted (inches) Unit 2 Backfilled	Predicted (inches) Unit 2 Excavated	Measured (inches) As of January 1985
Diesel Generator Building	1	3.5	3.4	2.4
	2	3.8	3.7	1.9
	3	3.8	3.6	2.3
	4	4.0	3.8	2.5
Control Building	5	3.8	3.7	2.1
	6	3.4	3.3	1.9
	7	4.0	3.7	2.2
	8	4.0	3.7	2.0
BF Tunnel	9	2.4	2.1	0.6
	10	3.0	2.5	0.8
Fuel Building	11	4.1	3.7	2.3
	12	4.5	4.0	2.6
	13	4.4	3.5	1.7
	14	4.7	3.8	2.2
Reactor Building	15	4.6	4.0	2.5
	16	4.5	4.0	2.6
	17	4.9	4.0	2.6
Auxiliary Building	18	4.2	3.8	2.5
	19	4.1	3.6	2.2
	20	4.9	3.9	2.7
	21	4.7	3.7	2.5
Main Steam Tunnel	22	4.5	3.8	2.1
	23	4.4	3.8	1.9
Turbine Building	24	4.3	3.7	1.9
	25	3.6	3.1	1.7
	26	4.4	3.7	2.0
	27	3.7	3.1	1.8
•→1 E Tunnel	28	4.4	3.3	1.9
	29	3.8	2.8	1.6

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SHOREHAM NUCLEAR
POWER STATION

TABLE 2.5.4-5

ANALYSIS OF FOUNDATION SYSTEMS

Structure	Approx Contact Dimensions (ft)	Approximate Founding El MLW (ft)	Approx Static Foundation Pressures (psf)	Factor of Safety for Bearing Pressure	Estimated Max Settlement (in.)	Bearing Material
Reactor Building	174 diam	-2	8000	17	2.9	Compacted backfill
Control Bldg	58 x 108	+10	7000	3.9	0.6	Compacted backfill
Radwaste Building	130 x 170	+7.5	3500	30	1.5	Compacted backfill
Screenwell	86 x 95	-29	3800	7.6	0.75	Pleistocene sand
Diesel Fuel Oil Storage Tanks	12 x 50	+5	580	not applicable*	<0.1	Pleistocene sand

*Tanks weigh less than the soil they replaced.