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Dalwyn R. Davidson
VICE PRESIDENT
SYSTEM ENGINEERING AND CONSTRUCTION

August 13, 1981

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U.S. NUCLEAR REGULATORY COMMISSION

Mr. Robert L. Tedesco
Assistant Director of Licensing
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Perry Nuclear Power Plant
Docket Nos. 50-440; 50-441
Response to Request for
Additional Information -
Geotechnical Engineering

Dear Mr. Tedesco:

This letter and its attachment is submitted to respond to your letter dated June 30, 1981, concerning geotechnical engineering.

The requested information is provided as draft responses for discussion with your staff in a meeting scheduled for September 2 and 3 at the Perry site. It is our intention to incorporate these responses in a subsequent amendment to our Final Safety Analysis Report.

Sincerely,

Dalwyn R. Davidson

Dalwyn R. Davidson
Vice President
System Engineering and Construction

DRD:dlp

Attachment

cc: D. Houston
G. Charnoff
NRC Resident Inspector

B001
S
1/1

241.1 (2.4.5.5.1.4) (2.5.5.2) (RSP)

In Section 2.4.5.5.1.4, you have stated that widespread slumping of the upper bluff materials (which includes the lacustrine deposits) is caused by groundwater seepage and frost action. Demonstrate how this fact has been incorporated in the stability analysis (Section 2.5.5.2) conducted to determine the amount of bluff recession which can occur before the emergency service water pumphouse becomes endangered. In this connection, indicate, in Figure 2.5-174, the location of the groundwater table used in the analysis. The staff requires that the stability of sliding wedges, for both the static and seismic case, within the lacustrine deposits, be investigated, utilizing methods such as the Morgenstern-Price method of analysis.

Response

The response to this question is provided in revised Sections 2.5.5.1, 2.5.5.2, Table 2.5-49, Figure 2.5-174, and new Figure 2.5-175.

Based upon the results of the testing and analyses, the following conclusions evolved. The infiltration of silt which occurred in localized areas of the then existing portions of the porous concrete blanket would have a negligible effect on the future performance of the underdrain/pressure relief system. Laboratory testing confirmed that significant pore pressures cannot build up in even highly contaminated porous concrete.

2.5.5 STABILITY OF SLOPES

The plant is constructed on an essentially level site and the final grades are similar to the preconstruction grades. All excavations for Seismic Category I plant structures have been backfilled and, hence, there are no man-made slopes which could fail and adversely affect the safety of the plant. The only natural slopes which could affect the safety of the plant are a bluff along Lake Erie which is described in the following sections.

2.5.5.1 Slope Characteristics

A steep bluff which forms the shoreline of Lake Erie is located approximately 305 feet north of the emergency service water pumphouse. The lower portion of this slope is periodically subjected to erosion due to wave action. In addition, some slumping of the upper bluff materials due to groundwater seepage and frost action has been observed. The resulting estimated average recession rate is two feet per year, as described in Section 2.4.5.5. The bluff is about 45 feet in height and has an average slope inclination of about 2 horizontal to 1 vertical, as shown in Figure 2.5-174. 241.1

2.5.5.2 Design Criteria and Analyses

Stability analyses have been conducted to determine the amount of bluff recession which can occur before the emergency service water pumphouse would become endangered. The subsurface stratigraphy of the bluff was determined from observations of the exposed bluff slope and from nearby test borings. The soil properties used in the analyses are described in Section 2.5.4.2. The stability analyses were conducted using the LEASE-I and LEASE-II computer 241.1

programs, which utilize the simplified Bishop circular arc method[193,194] and the Morgenstern-Price method[223,224], respectively. For the seismic condition, a seismic coefficient of 0.15 was used for pseudostatic analyses. The groundwater level was taken to be elevation 615' near the emergency service water pumphouse, exiting the bluff slope at elevation 590'.

To stablize the bluff slope against wave action and against slumping in the zone of groundwater emergence, a flattened slope with rip-rap slope protection is required. The results of Bishop method stability analyses with bluff slope inclinations ranging from 1:1 (horizontal:vertical) to 3:1 are shown on Figure 2.5-174. It was determined in this analysis that a 3:1 slope was required for the minimum desired factors of safety. However, the presence of the rock rip-rap slope protection materials were not considered in this analysis, which would add to the overall stability of the slope.

A morgenstern-Price stability analysis was also conducted on the 3:1 slope. The results of this analysis are shown in Figure 2.5-175 in comparison to the Bishop method results for the same slope. The Morgenstern-Price analysis yielded somewhat higher factors of safety than the Bishop method for failure surfaces passing through the upper till (note that only the most critical failure surfaces are shown out of many trial surfaces). Failure surfaces passing only through the lacustrine stratum were also evaluated, and resulted in considerably higher factors of safety than those also passing through the upper till.

Additional stability analyses have been conducted on the final slope protection design configuration, which is shown in detail in Figure 2.4-39. The results of this analysis, which incorporated strength characteristics of the rip-rap, indicate that the stability of the final design is satisfactory, with a minimum factor of safety of 1.45 for SSE conditions.

The results of the various stability analyses determined that the toe of the bluff could recede about 220 feet before a potential failure arc of the bluff would approach within 40 feet of the emergency service water pumphouse.

However, as discussed in Section 2.4.5.5, if the shoreline recedes approximately 130 feet, protective measures will be initiated. 241.1

A monitoring program has been established to measure the bluff recession. This program is described in Section 2.4.5.5.

2.5.5.3 Logs of Borings

Boring logs are presented in Appendix 2E. Figure 2.5-53 shows the locations of the borings.

2.5.5.4 Compacted Fill

There is no compacted fill associated with the Lake Erie bluff.

2.5.6 EMBANKMENTS AND DAMS

There are no Seismic Category I embankments or dams associated with the Perry Nuclear Power Plant.

REFERENCES

223. Dawson, A.W., 1972, LEASE II, A Computerized System for the Analysis of Slope Stability, Thesis, Department of Civil Engineering, Massachusetts Institute of Technology.

224. Morgerstern, N. and Price, V.E., 1965, The Analysis of the Stability of General Slip Surface, Geotechnique, Vol. 15, pp. 79-93.

2.4.1

TABLE 2.5-49

MATERIAL PROPERTIES ADOPTED FOR DESIGN

Stratigraphic Unit	Saturated Unit Weight γ_{sat} (pcf)	Shear Strength[1] τ (tsf)	Undrained Shear Strength S_u (tsf)
Lacustrine	131	$0.12 + \bar{\sigma}_n \tan 33.5°$ $0.12 + \bar{\sigma}_n \tan 31°$ (2)	0.75
Upper Till	130	$0 + \bar{\sigma}_n \tan 35°$ $0.12 + \bar{\sigma}_n \tan 31°$ (2)	1.0
Lower Till	142	$0.60 + \bar{\sigma}_n \tan 35°$	5.5
Chagrin Shale	152	-	130

241.1

NOTES:

1. Effective stress basis; $\tau = \bar{c} + \bar{\sigma}_n \tan \bar{\phi}$

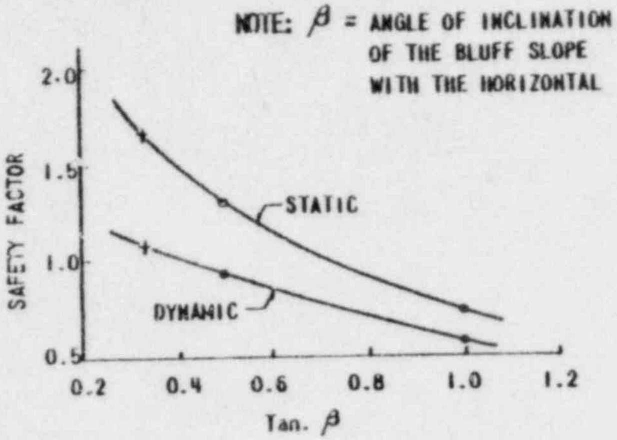
   where: $\bar{c}$ = Effective cohesion, tsf

   $\bar{\sigma}_n$ = Effective normal stress, tsf

   $\bar{\phi}$ = Effective friction angle, degrees

2. Strength paramters used for the Lake Erie Bluff Stability Analysis are shown on Figures 2.5-174 and 2.5-175. 241.1

FIGURE 2.5-174



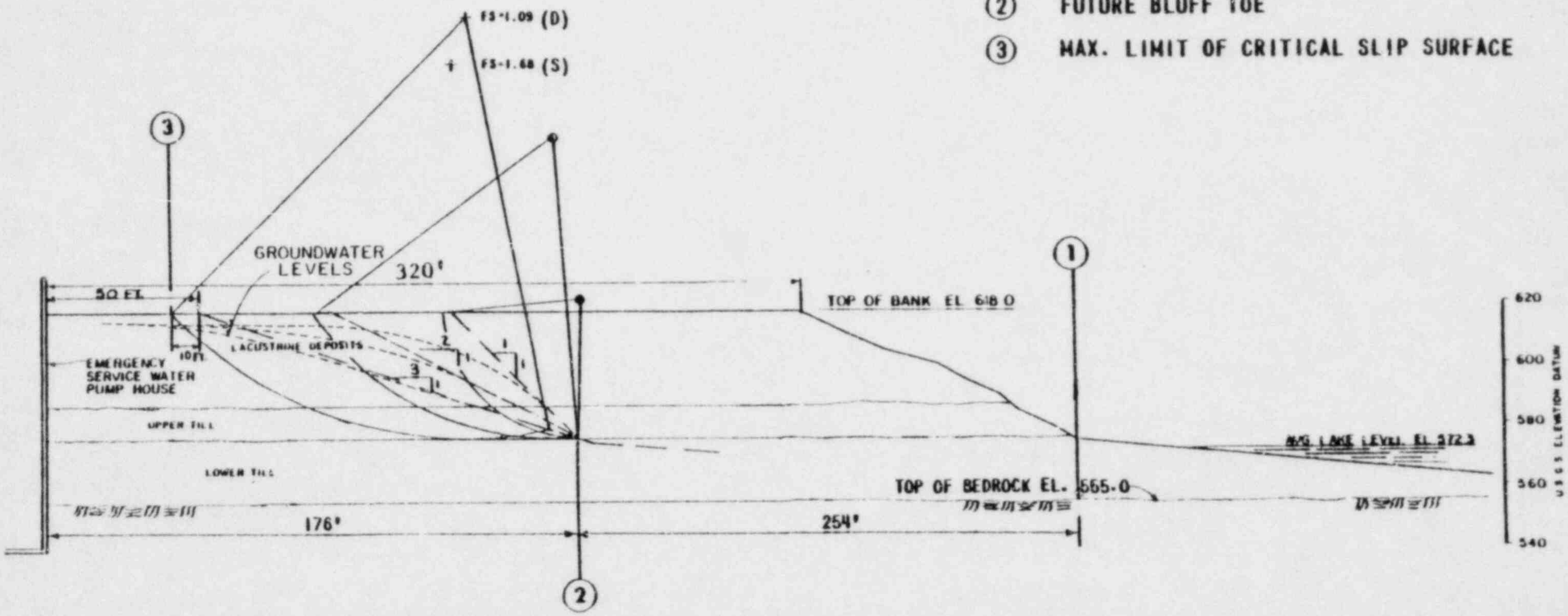
SYMBOL	AVERAGE SLOPE	FACTOR of SAFETY		MAX. ②-③ DISTANCE (Ft.)
		STATIC	DYNAMIC*	
●	1:1	0.73	0.56	46
o	2:1	1.32	0.94	86
+	3:1	1.68	1.09	136

* k_s = 0.15

① EXISTING TOE OF BLUFF SLOPE

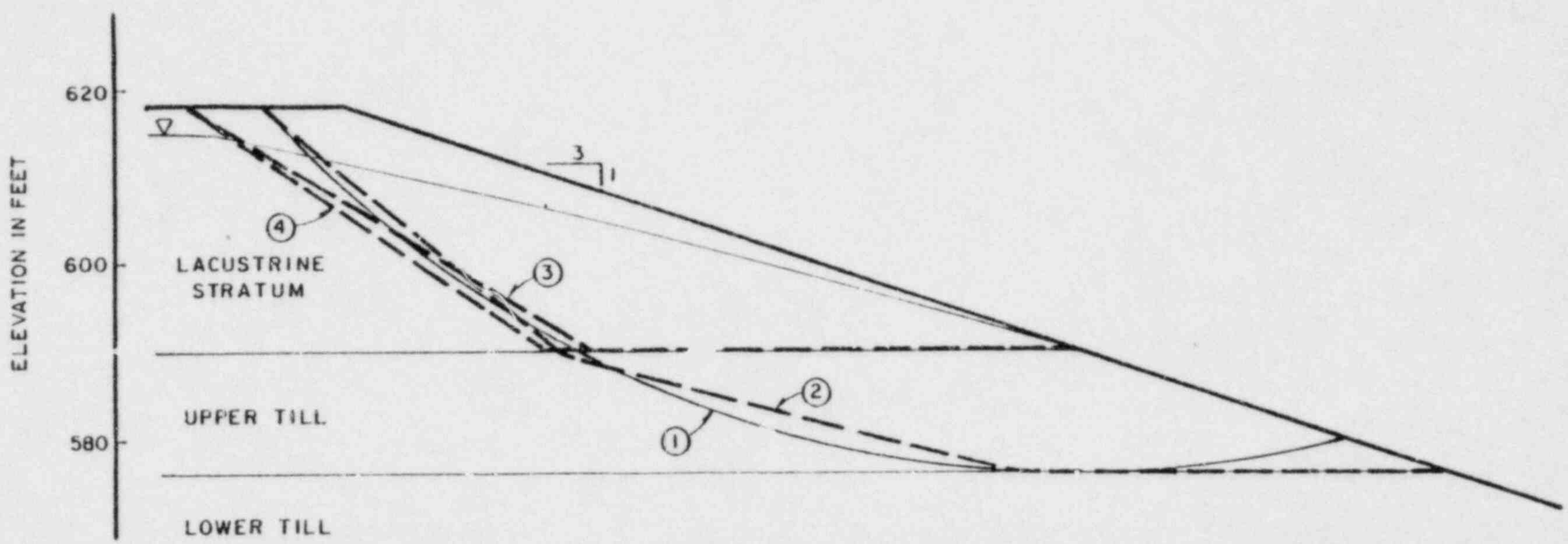
② FUTURE BLUFF TOE

③ MAX. LIMIT OF CRITICAL SLIP SURFACE



FAILURE SURFACE NO.	METHOD	SEISMIC COEFFICIENT	FACTOR OF SAFETY
1	B	0.00	1.68
1	B	0.15g	.09
2	M-P	0.00	1.69
2	M-P	0.15g	1.18
3	M-P	0.00	2.16
4	M-P	0.15g	1.45

NOTE: B = BISHOP METHOD
M-P = MORGENSTERN-PRICE METHOD



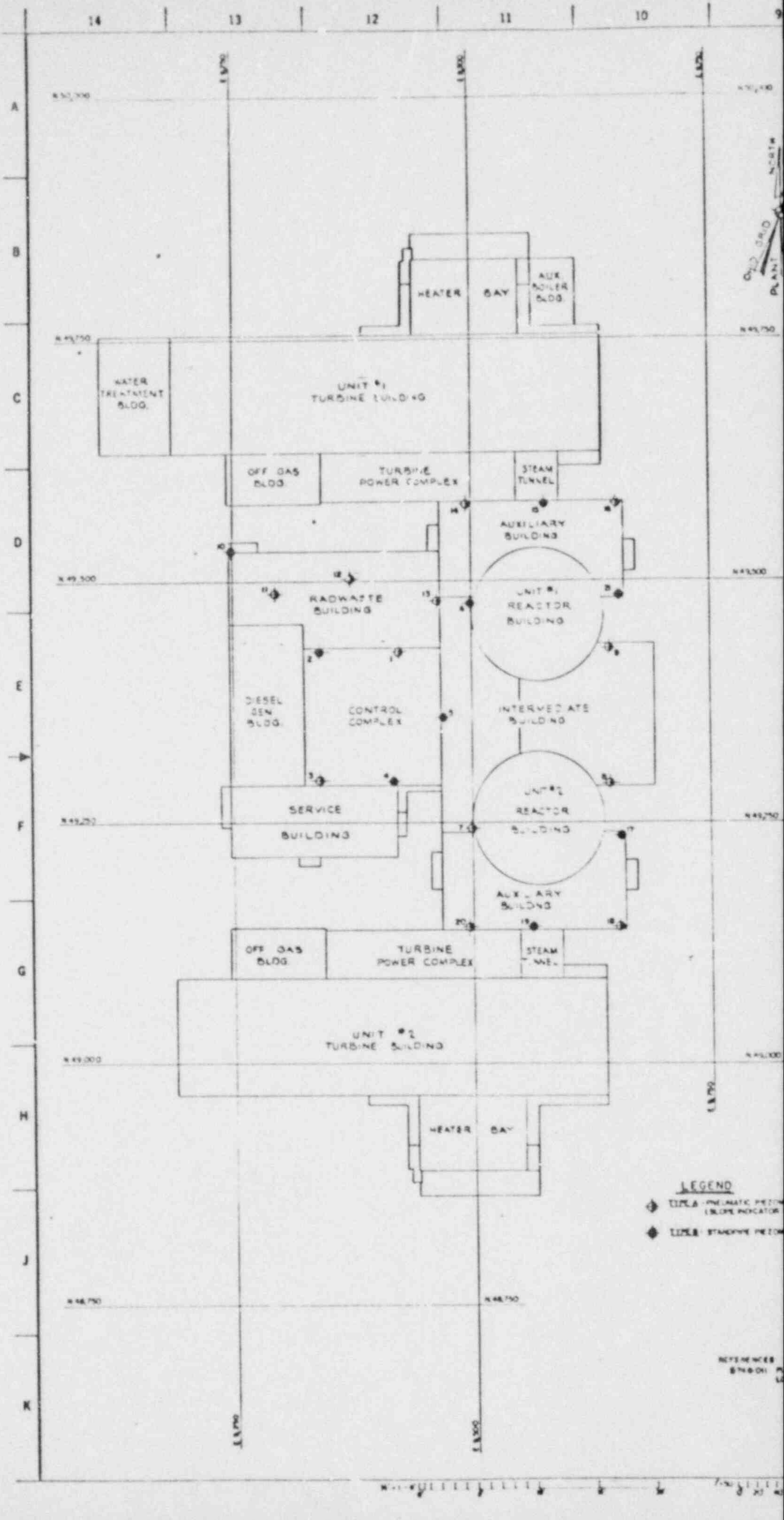
STABILITY ANALYSIS OF
LAKE ERIE BLUFF

FIGURE 2.5-175

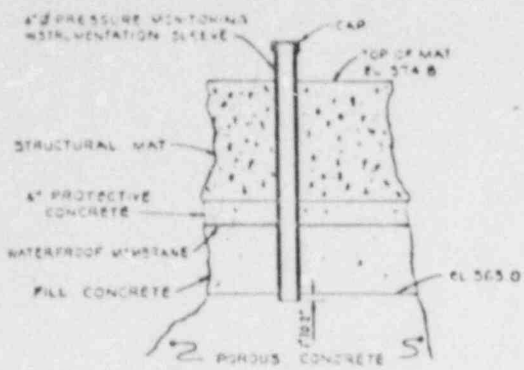
2.4.1.2 (2.4.13.5.3.d) Figure 2.4-72 is purported t show piezometric devices installed through each of the building mats of the auxiliary buildings, the control complex, the intermediate building and the radwaste building, to measure the hydrostatic uplift pressure acting under these structures. However, the tip of the only piezometer shown on the figure is founded in relatively impermeable Class B fill. Is this an error in the figure? If not, discuss the function of the piezometer in the context of measuring the hydrostatic uplift pressures acting under the structures during the life of the plant, or otherwise define its purpose.

Response

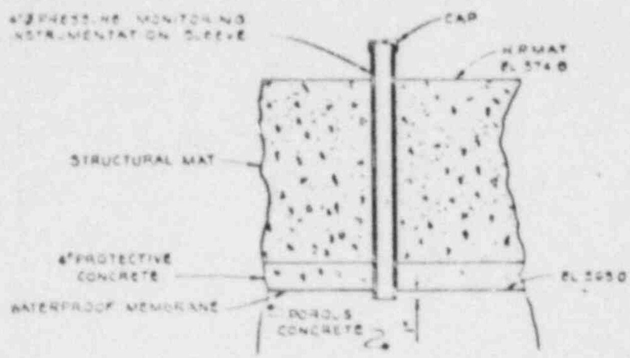
Yes, the wrong figure was referenced. A new Figure 2.4-76 will be added to the FSAR and will be referenced in Section 2.4.13.5.3.d in lieu of Figure 2.4-72.



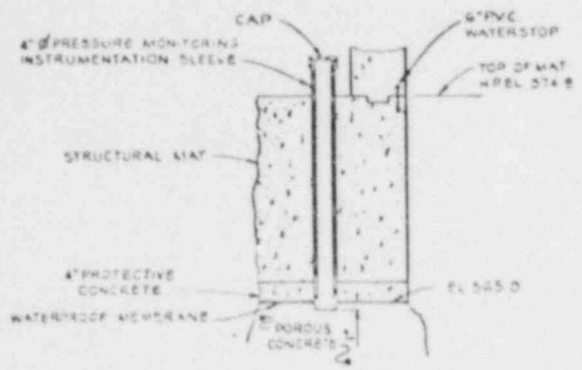
14
13
12
11
10
A
B
C
D
E
F
G
H
J
K
HEATER BAY
AUX BOILER BLDG.
WATER TREATMENT BLDG.
UNIT #1 TURBINE BUILDING
OFF GAS BLDG.
TURBINE POWER COMPLEX
STEAM TUNNEL
AUXILIARY BUILDING
UNIT #1 REACTOR BUILDING
RADWASTE BUILDING
DIESEL GEN. BLDG.
CONTROL COMPLEX
INTERMEDIATE BUILDING
UNIT #2 REACTOR BUILDING
SERVICE BUILDING
AUXILIARY BUILDING
OFF GAS BLDG.
TURBINE POWER COMPLEX
STEAM TUNNEL
UNIT #2 TURBINE BUILDING
HEATER BAY
LEGEND
N 49,500
N 49,250
N 49,000
N 48,750



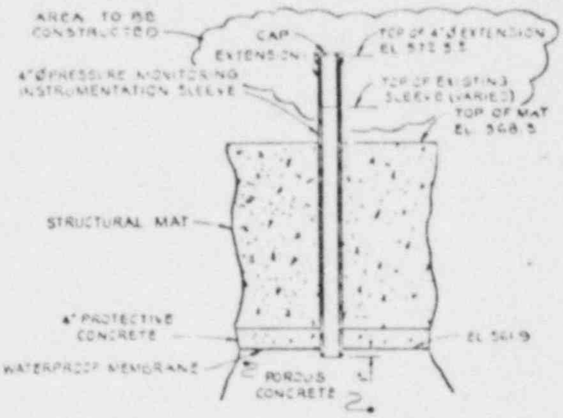
TYPICAL SECTION
MONITORING INSTRUMENTATION
SLEEVES - 1, 2, 3, 4, 11, 12 & 13
NO SCALE



TYPICAL SECTION
MONITORING INSTRUMENTATION
SLEEVES - 5, 8 & 9
NO SCALE



TYPICAL SECTION
MONITORING INSTRUMENTATION
SLEEVES - 6 & 7
NO SCALE



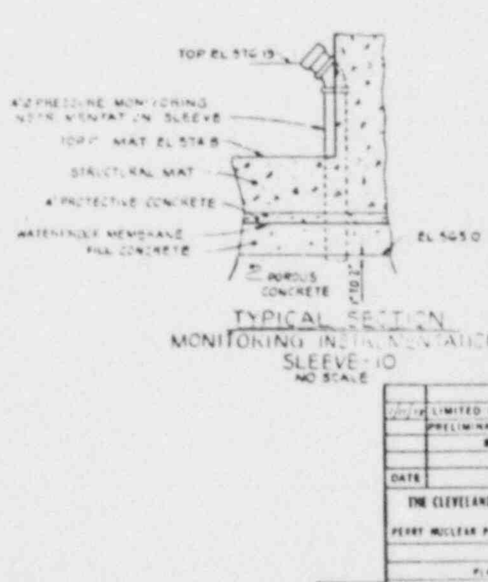
TYPICAL SECTION
MONITORING INSTRUMENTATION
SLEEVES - 14, 15, 16, 17, 18, 19, 20 & 21
NO SCALE

SEE LEGEND

PIEZOMETER NUMBER	TYPE	LENGTH 4" Ø SLEEVE	TOP ELEV OF EXIST SLEEVE	TOP BASEMENT FLOOR ELEV	NORTH COORDINATE	EAST COORDINATE	BUILDING
1	A	11'-0"	[illegible]	574.8	[illegible]	[illegible]	CONTROL COMPLEX
2	B	11'-0"	[illegible]	574.8	[illegible]	[illegible]	CONTROL COMPLEX
3	A	11'-0"	[illegible]	574.8	[illegible]	[illegible]	CONTROL COMPLEX
4	B	11'-0"	[illegible]	574.8	[illegible]	[illegible]	CONTROL COMPLEX
5	B	11'-0"	[illegible]	574.8	[illegible]	[illegible]	INTERMEDIATE BUILDING
6	B	11'-0"	[illegible]	574.8	[illegible]	[illegible]	INTERMEDIATE BUILDING
7	A	11'-0"	[illegible]	574.8	[illegible]	[illegible]	INTERMEDIATE BUILDING
8	A	11'-0"	[illegible]	574.8	[illegible]	[illegible]	INTERMEDIATE BUILDING
9	A	11'-0"	[illegible]	574.8	[illegible]	[illegible]	INTERMEDIATE BUILDING
10	B	11'-0"	[illegible] *	574.8	[illegible]	[illegible]	RADWASTE BUILDING
11	A	11'-0"	[illegible]	574.8	[illegible]	[illegible]	RADWASTE BUILDING
12	B	11'-0"	[illegible]	574.8	[illegible]	[illegible]	RADWASTE BUILDING
13	A	11'-0"	[illegible]	574.8	[illegible]	[illegible]	RADWASTE BUILDING
14	A	7'-3"	[illegible]	568.3	[illegible]	[illegible]	AUXILIARY BLDG [illegible]
15	B	7'-3"	[illegible]	568.3	[illegible]	[illegible]	AUXILIARY BLDG [illegible]
16	A	7'-3"	[illegible]	568.3	[illegible]	[illegible]	AUXILIARY BLDG [illegible]
17	B	7'-3"	[illegible]	568.3	[illegible]	[illegible]	AUXILIARY BLDG [illegible]
18	A	7'-3"	[illegible]	568.3	[illegible]	[illegible]	AUXILIARY BLDG [illegible]
19	B	7'-3"	[illegible]	568.3	[illegible]	[illegible]	AUXILIARY BLDG [illegible]
20	A	7'-3"	[illegible]	568.3	[illegible]	[illegible]	AUXILIARY BLDG [illegible]
21	B	7'-3"	[illegible]	568.3	[illegible]	[illegible]	AUXILIARY BLDG [illegible]

NOTES

1. ELEVATION FOR THE TOP OF SLEEVES ARE TO THE TOP OF SLEEVE WITHOUT THE CAP.
* 2. THE STANDPIPE AT PIEZOMETER NO. 10 WAS CONSTRUCTED INSIDE THE RADWASTE BUILDING WALL AND PENETRATES THE WALL WITH A [illegible] ELBOW. THE ELEVATION GIVEN IS THE LOWER LIP OF THE SLANTING SLEEVE OPENING.
3. ALL TOP OF SLEEVE ELEVATIONS AND COORDINATE LOCATIONS WERE RECEIVED FROM THE FIELD AS CONSTRUCTED.



TYPICAL SECTION
MONITORING INSTRUMENTATION
SLEEVE - 10
NO SCALE

...ER BY SINCO
...OF SEATTLE OR EQUAL

...PLAN PIEZOMETER
...ATIONS

REVISIONS

CONSTRUCTION	
LIMITED CONSTRUCTION AS NOTED	
PRELIMINARY NOT FOR CONSTRUCTION	
BIDDING PURPOSES	
DATE	RELEASED FOR

THE CLEVELAND ELECTRIC ILLUMINATING COMPANY
PERRY NUCLEAR POWER PLANT
CIVIL
PLANT UNDERDRAIN SYSTEM
NUCLEAR ISLAND PIEZOMETER LOCATIONS
GILBERT ASSOCIATES, INC.
04 4549 D-746

Figure 2.4-76

241.3 (2.4.13.5.5.C .4.b) (2.5.4.5.2)

Provide single grain size distribution plots for each of the following materials, showing the lower and upper bounds (range) of the distribution.

a) Class A fill during construction (one plot each for construction materials from the Bestone and R. W. Sidley Quarries).

b) Class B fill during construction.

c) Upper till.

d) Lower till.

Response

The response to this question is provided in revised Sections 2.5.4.2.1.1, 2.5.4.5.2, 2.5.4.5.3, and new Figures 2.5-179 through 2.5-183.

was overexcavated and backfilled with lean concrete having a 28-day compressive strength of at least 1500 psi. Figure 2.5-55 delineates the areas overexcavated and treated as such.

2.5.4.1.4 Residual Stress

Refer to Section 2.5.1.2.5.3.

2.5.4.1.5 Unstable Rock and Soil Composition

The lower till and the Chagrin shale units supporting plant foundations are not susceptible to detrimental consolidation, densification or liquefaction under either static or dynamic loading.

2.5.4.2 Properties of Subsurface Materials

As described in detail in Section 2.5.4.3.3, four stratigraphic units were encountered by subsurface exploration at the site. In descending order, these units are identified as lacustrine sediments, two distinct glacial ground moraine deposits which are denoted as upper till and lower till, and finally, an Upper Devonian shale identified as the Chagrin shale. The properties of these materials are described in the following sections. The test methods used to determine the properties are summarized in Table 2.5-16.

2.5.4.2.1 Properties of Soil Materials

2.5.4.2.1.1 Physical Properties

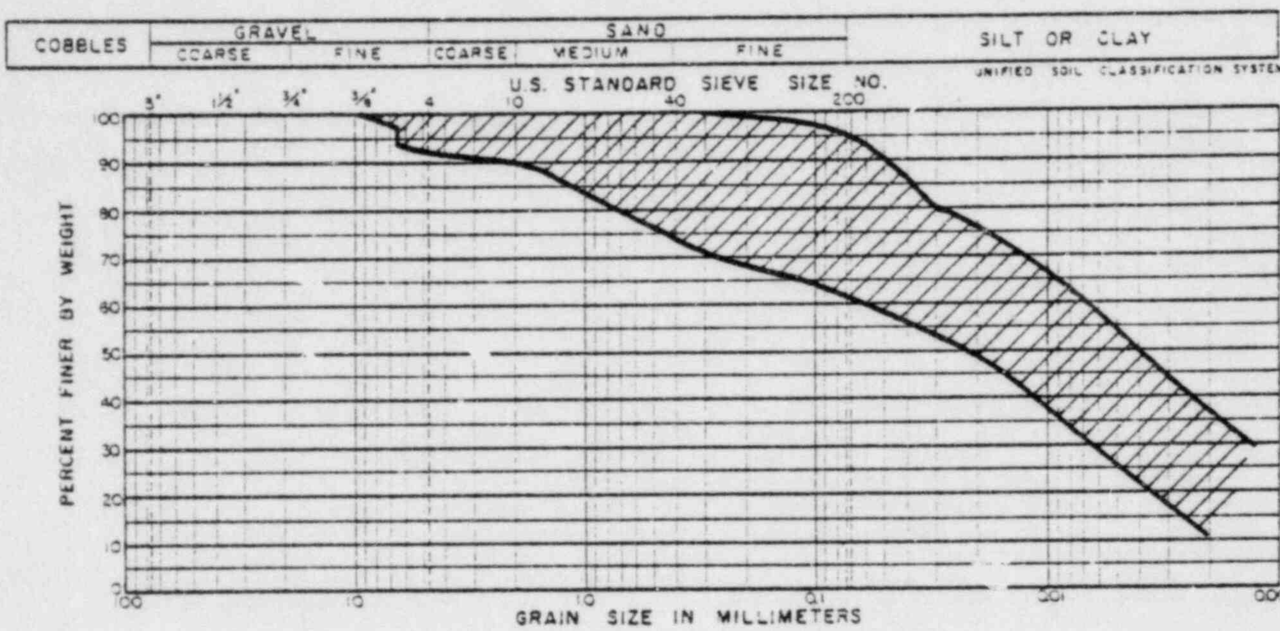
Physical property tests conducted on representative samples of the soils at the site include natural water content, Atterberg (liquid and plastic) limits, unit weight, specific gravity and grain size distribution. The results of these tests are presented in Tables 2.5-17 through 2.5-19 and are summarized in Table 2.5-20. Grain size distribution curves are presented in Figures 2.5-94 through 2.5-96. The range of gradations of the upper till and lower till are shown in Figures 2.5-179 and 2.5-180. 241.3

areas outside of building lines. Minimum and maximum density tests were performed for each 4,500 cubic yards of fill placed, and in-place density and grain size distribution tests for each 150 cubic yards or once per lift, whichever was more frequent. However, in confined areas, where the volume of each lift was less than 50 cubic yards, in-place density tests were performed once every third lift or every 50 cubic yards, whichever was more frequent. Through the end of July, 1981, approximately 437,000 cubic yards of safety-related Class A fill have been placed, and approximately 6,170 in-place density tests and grain size distribution tests have been performed. The gradation range of the Class A fill which has been placed is shown in Figures 2.5-181 and 2.5-182. A summary of field density tests obtained for quality control during the placement of Class A fill is shown in Figure 2.5-184. 241.3, 241.5

A total of 181 laboratory constant-head permeability tests have been performed on material removed from the fill with the lowest coefficient of permeability obtained being 2.16×10^{-3} cm/sec and the average 1.69×10^{-2} cm/sec. Also, 51 in-place falling head permeability tests have been performed with the lowest coefficient of permeability obtained being 9.45×10^{-3} cm/sec and the average 3.77×10^{-2} cm/sec. The minimum required coefficient of permeability is 2×10^{-4} cm/sec.

2.5.4.5.3 Class B Fill

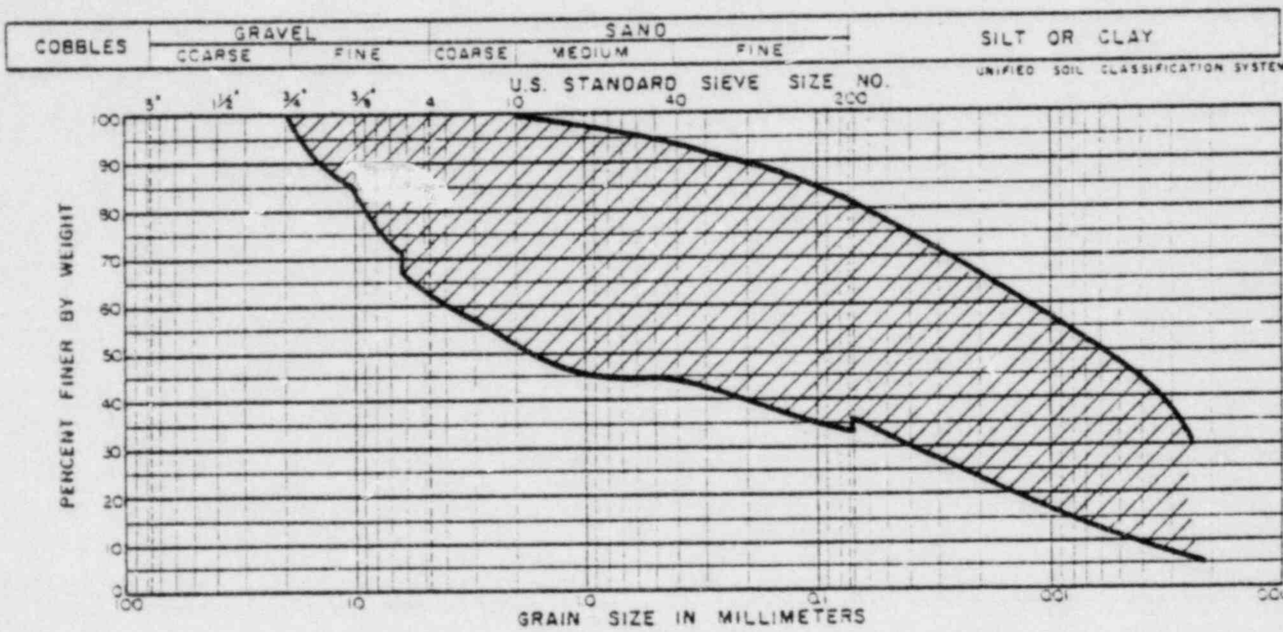
Class B fill was used for nonload bearing backfill around Seismic Category I structures as shown in Figure 2.5-151, and consists of lower till soil which was removed and stockpiled during plant excavation. A typical c[illegible]action curve is shown in Figure 2.5-156. The maximum dry density (AS[illegible] 1557) has been found to range from 128.6 to 137.5 pounds per cubic foot and the optimum moisture content from 7.4 to 13.0 percent. Class B fill is compacted to not less than 92 percent of the maximum dry density, at a moisture content not less than three percentage points below nor four percentage points above the optimum moisture content. Through the end of July, 1981, approximately 286,000 cubic yards of Class B fill have been placed and approximately 380 241.3, 241.5



NOTE: RANGE REPRESENTS THE RESULTS OF 44 TESTS

RANGE OF GRAIN SIZE DISTRIBUTION
TE[illegible] RESULTS FOR UPPER TILL

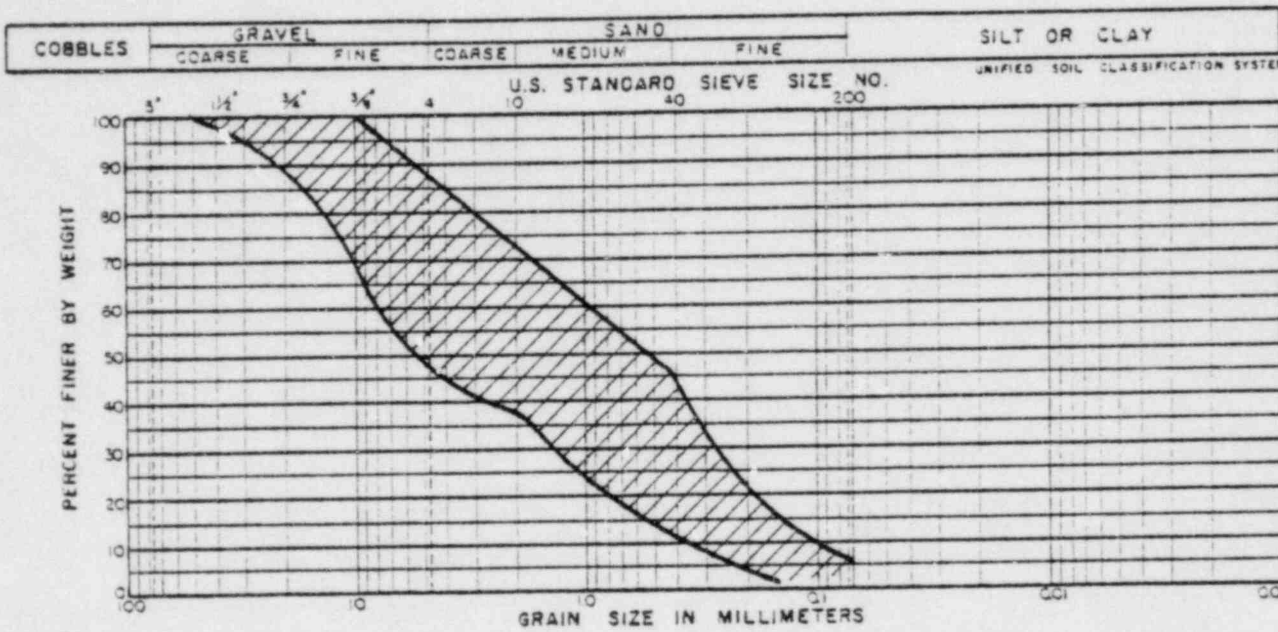
FIGURE 2.5-[illegible]79



NOTE: RANGE REPRESENTS THE RESULTS OF 56 TESTS

RANGE OF GRAIN SIZE DISTRIBUTION
TEST RESULTS FOR LOWER TILL

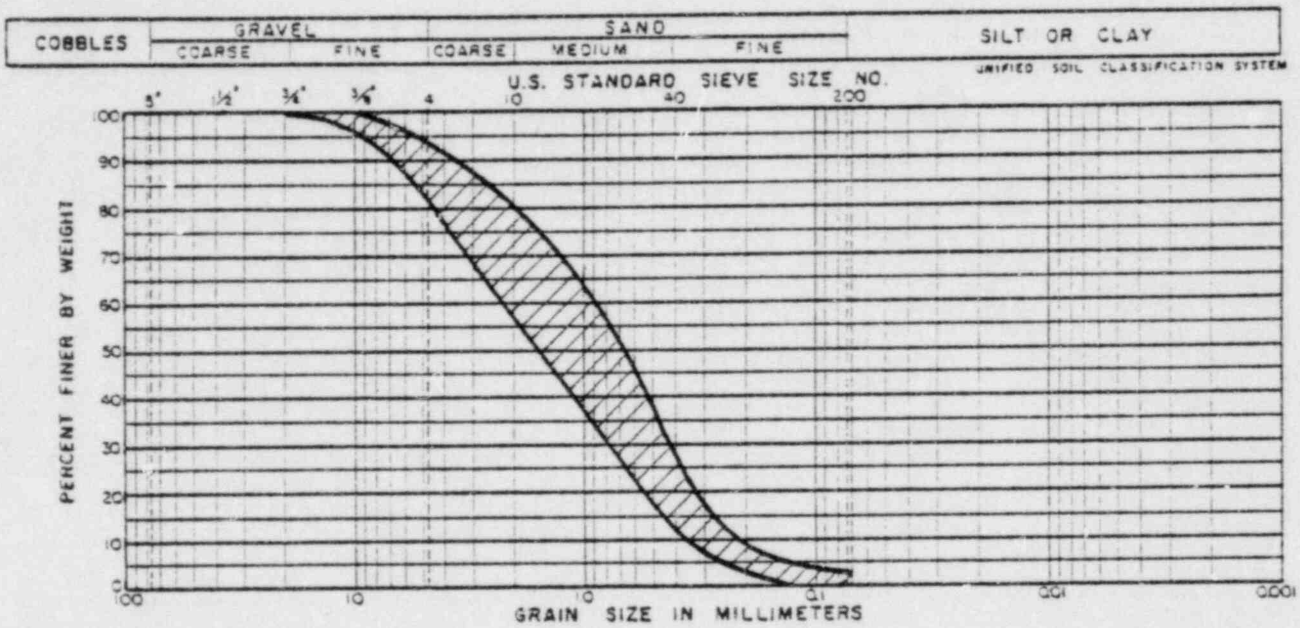
FIGURE 2.5-180



NOTE: RANGE IS ESTIMATED BASED ON A RANDOM SAMPLING
OF APPROXIMATELY 1500 TESTS

RANGE OF GRAIN SIZE DISTRIBUTION
TEST RESULTS FOR CLASS A FILL
(BESTONE QUARRY)

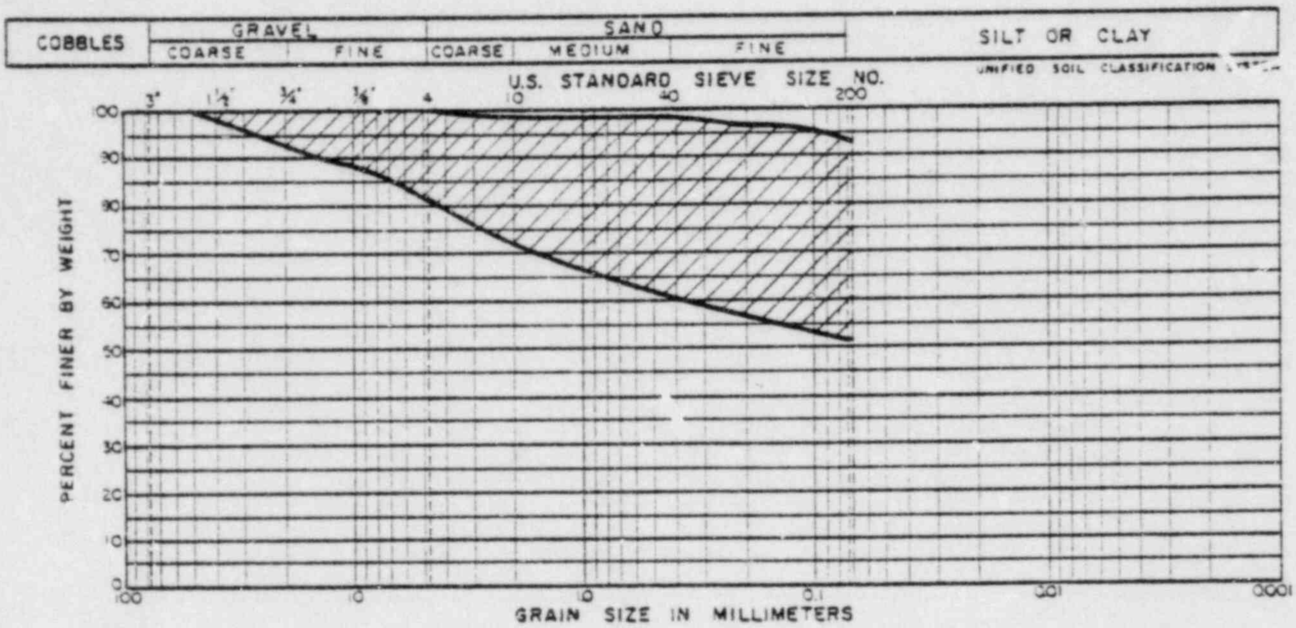
FIGURE 2.5-181



NOTE: RANGE IS ESTIMATED BASED ON A RANDOM SAMPLING OF APPROXIMATELY 4700 TESTS

RANGE OF GRAIN SIZE DISTRIBUTION
TEST RESULTS FOR CLASS A FILL
(SIDLEY QUARRY)

FIGURE 2.5-182



NOTE: RANGE IS ESTIMATED BASED ON A RANDOM SAMPLING
OF APPROXIMATELY 200 TESTS

RANGE OF GRAIN SIZE DISTRIBUTION
TEST RESULTS FOR CLASS B FILL

FIGURE 2.5-183

241.4 (2.4.13.5.5. c.7.d)

You state in section 2.4.13.5.5.c.7.d that "Falling head permeability testing conducted at the Perry Site reported in Section 2.5.4, further shows that the velocity and transport capability of groundwater to erode either the Chagrin shale or lower till are negligible." However, you have not addressed this aspect in the mentioned section; please do so in quantitative detail. The Staff is concerned that possible erosion of the till and shale by flowing water could contaminate and/or clog the Class A filter. This phenomenon could seriously impair the intended use of the filter.

Response

The response to this question is provided in revised Section 2.5.4.6.3.

During construction, the seepage estimate described above was confirmed. The seepage collected in the peripheral ditch from the lacustrine stratum was estimated to be less than ten gallons per minute. No seepage was detected in the till strata or shale, and the excavation bottom was dry. Seepage into the plant underdrain system after the excavation is backfilled will be essentially the same as that experienced during construction. 231.02

As described above, no seepage was detected in the lower till stratum of shale and the plant excavation bottom was dry. The estimated mean coefficients of permeability for these materials are 2.0×10^{-7} cm/sec and 8.0×10^{-8} cm/sec, respectively. The corresponding seepage velocity in these materials is less than 4 feet per year, for gradients as large as 4. This amount, which is consistent with the lack of observable seepage, is far too small to cause erosion which could contaminate and/or clog the Class A filter. 241.4

2.5.4.6.4 Radius of Groundwater Drawdown

In order to monitor the long-term effect of the plant dewatering system on the local groundwater levels, four lines of well-point piezometers were installed as shown in Figure 2.5-157. The piezometer lines extend 1000 feet from the plant in the east, west and south directions and 550 feet in the north direction. The average monthly readings for each piezometer are shown in Figure 2.5-158. Groundwater drawdown profiles along the piezometer lines are shown on Figure 2.5-159. (Some of the piezometers were removed and replaced at various times due to construction activity conflicts.) It is concluded that the groundwater level within the lacustrine stratum is not affected beyond a radius on the order of 500 feet from the plant, as anticipated. In most piezometers, the groundwater drawdown appears to have already (March 1979) stabilized to a steady-state condition.

The piezometers sealed within the lower till and shale (see Figure 2.5-159) generally indicate piezometric levels within about three feet, above or below, the lacustrine level. However, in piezometers E-3B and N-4B, both in shale, the piezometric levels (March 1979) are 7.0 and 4.3 feet below the lacustrine

241.5 (2.5.4.5.2) Provide a summary of field density and moisture tests obtained for quality control during construction of Class A and Class B fill. The results may b. presented as a statistical distribution plot showing low, high and average values. This would help the staff to conclude that suitable compaction has been obtained.

Response

The response to this question is provided in revised Sections 2.5.4.5.2, 2.5.4.5.3, and new Figures 2.5-184 through 2.5.-186.

areas outside of building lines. Minimum and maximum density tests were performed for each 4,500 cubic yards of fill placed, and in-place density and grain size distribution tests for each 150 cubic yards or once per lift, whichever was more frequent. However, in confined areas, where the volume of each lift was less than 50 cubic yards, in-place density tests were performed once every third lift or every 50 cubic yards, whichever was more frequent. Through the end of July, 1981, approximately 437,000 cubic yards of safety-related Class A fill have been placed, and approximately 6,170 in-place density tests and grain size distribution tests have been performed. The gradation range of the Class A fill which has been placed is shown in Figures 2.5-181 and 2.5-182. A summary of field density tests obtained for quality control during the placement of Class A fill is shown in Figure 2.5-184.

241.3, 241.5

A total of 181 laboratory constant-head permeability tests have been performed on material removed from the fill with the lowest coefficient of permeability obtained being 2.16×10^{-3} cm/sec and the average 1.69×10^{-2} cm/sec. Also, 51 in-place falling head permeability tests have been performed with the lowest coefficient of permeability obtained being 9.45×10^{-3} cm/sec and the average 3.77×10^{-2} cm/sec. The minimum required coefficient of permeability is 2×10^{-4} cm/sec.

2.5.4.5.3 Class B Fill

Class B fill was used for nonload bearing backfill around Seismic Category I structures as shown in Figure 2.5-151, and consists of lower till soil which was removed and stockpiled during plant excavation. A typical compaction curve is shown in Figure 2.5-156. The maximum dry density (ASTM D 1557) has been found to range from 128.6 to 137.5 pounds per cubic foot and the optimum moisture content from 7.4 to 13.0 percent. Class B fill is compacted to not less than 92 percent of the maximum dry density, at a moisture content not less than three percentage points below nor four percentage points above the optimum moisture content. Through the end of July, 1981, approximately 286,000 cubic yards of Class B fill have been placed and approximately 380

241.3, 241.5

in-place density tests have been performed. The gradation range of the Class B fill which has been placed is shown in Figure 2.5-183. A summary of field density and moisture tests taken for quality control during placement of the Class B fill is shown in Figures 2.5-185 and 2.5-186. 241.5

2.5.4.5.4 Field Testing of Backfill

An onsite testing laboratory was established to perform all field testing. A defined Quality Assurance Program and approved procedures were implemented to assure that proper testing methods, procedures and equipment were used in field testing.

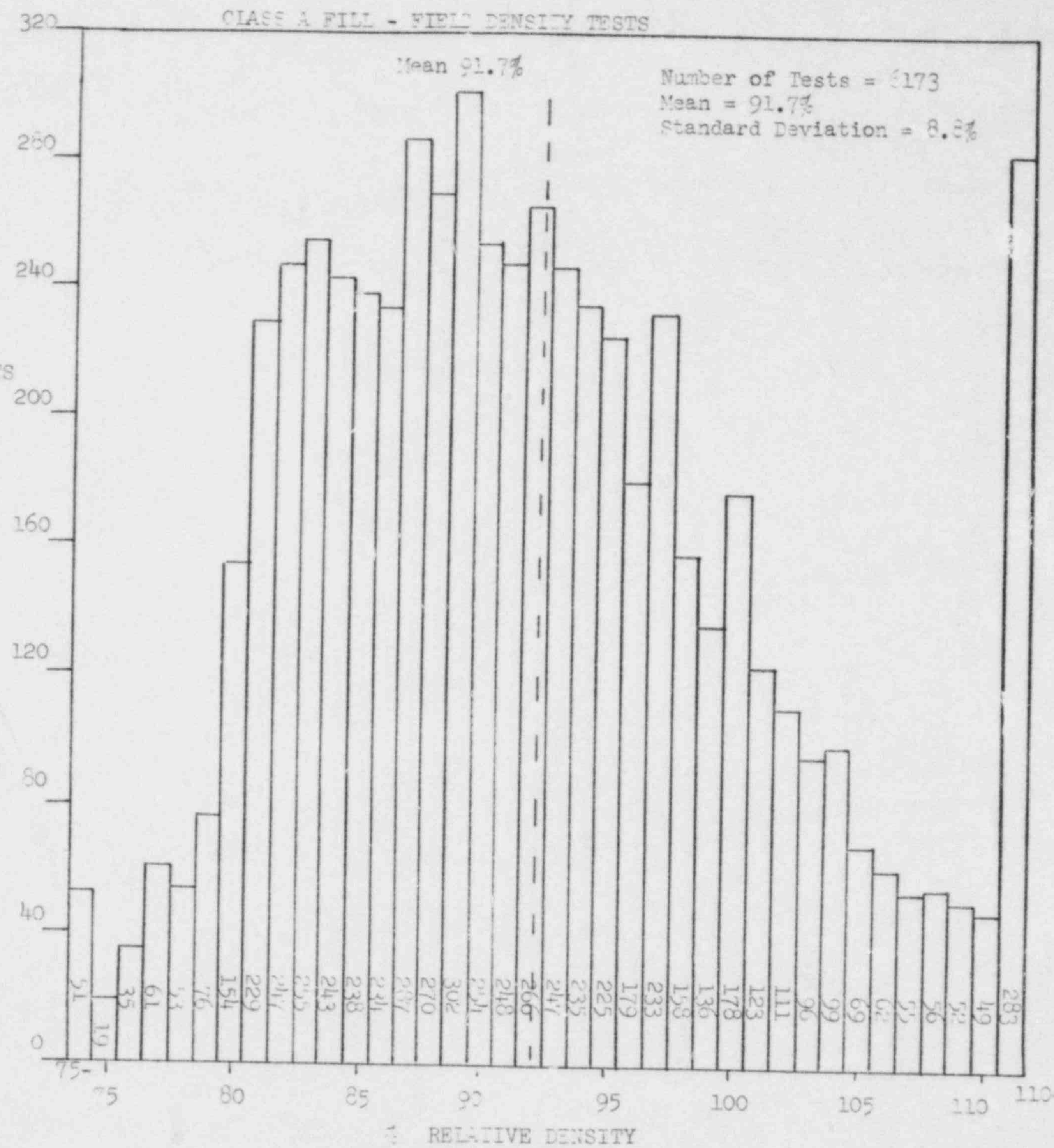


Figure 2.5-184

CLASS B FILL-FIELD DENSITY TESTS

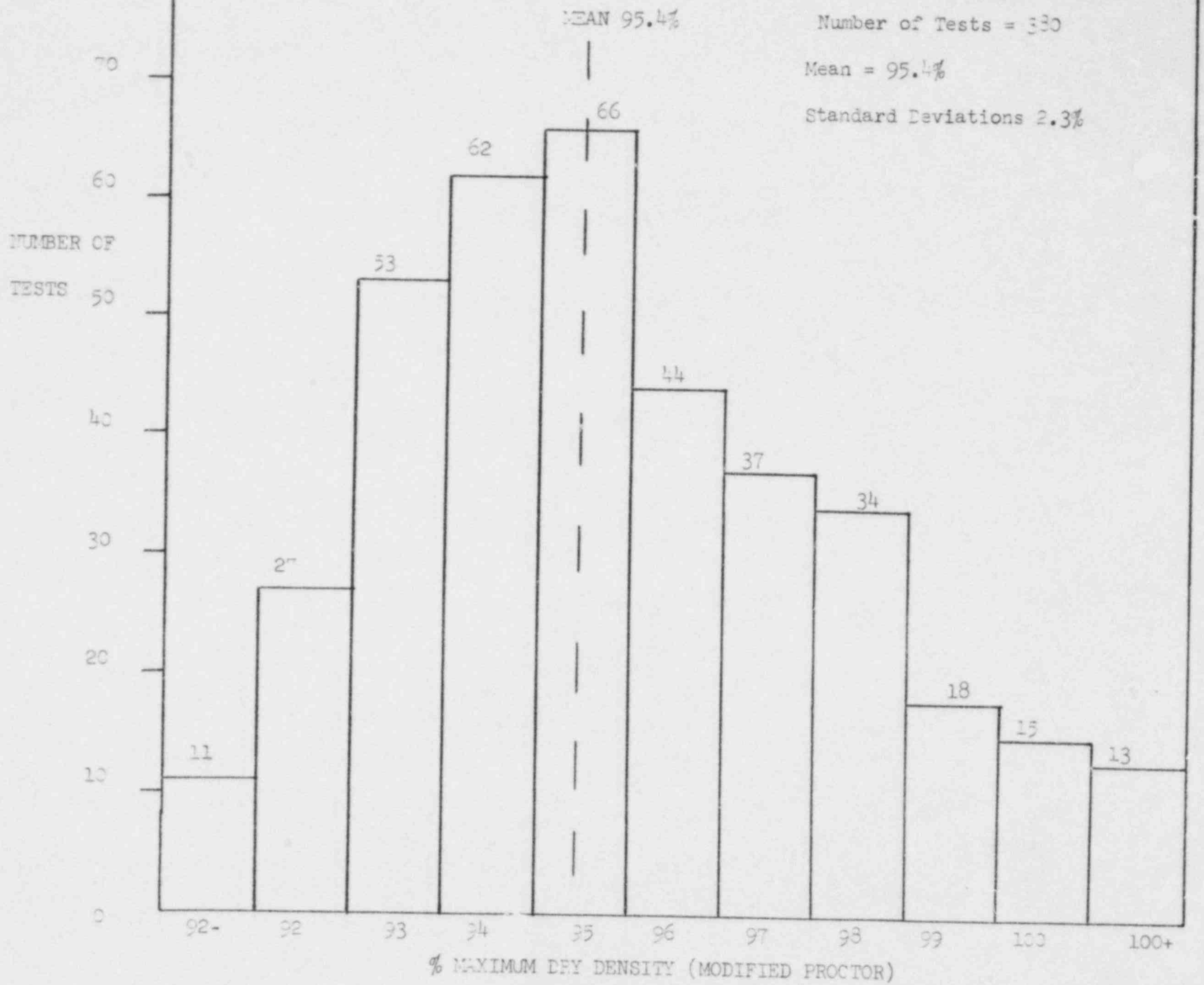


FIGURE 2.5-185

CLASS B FILL-FIELD MOISTURE TESTS

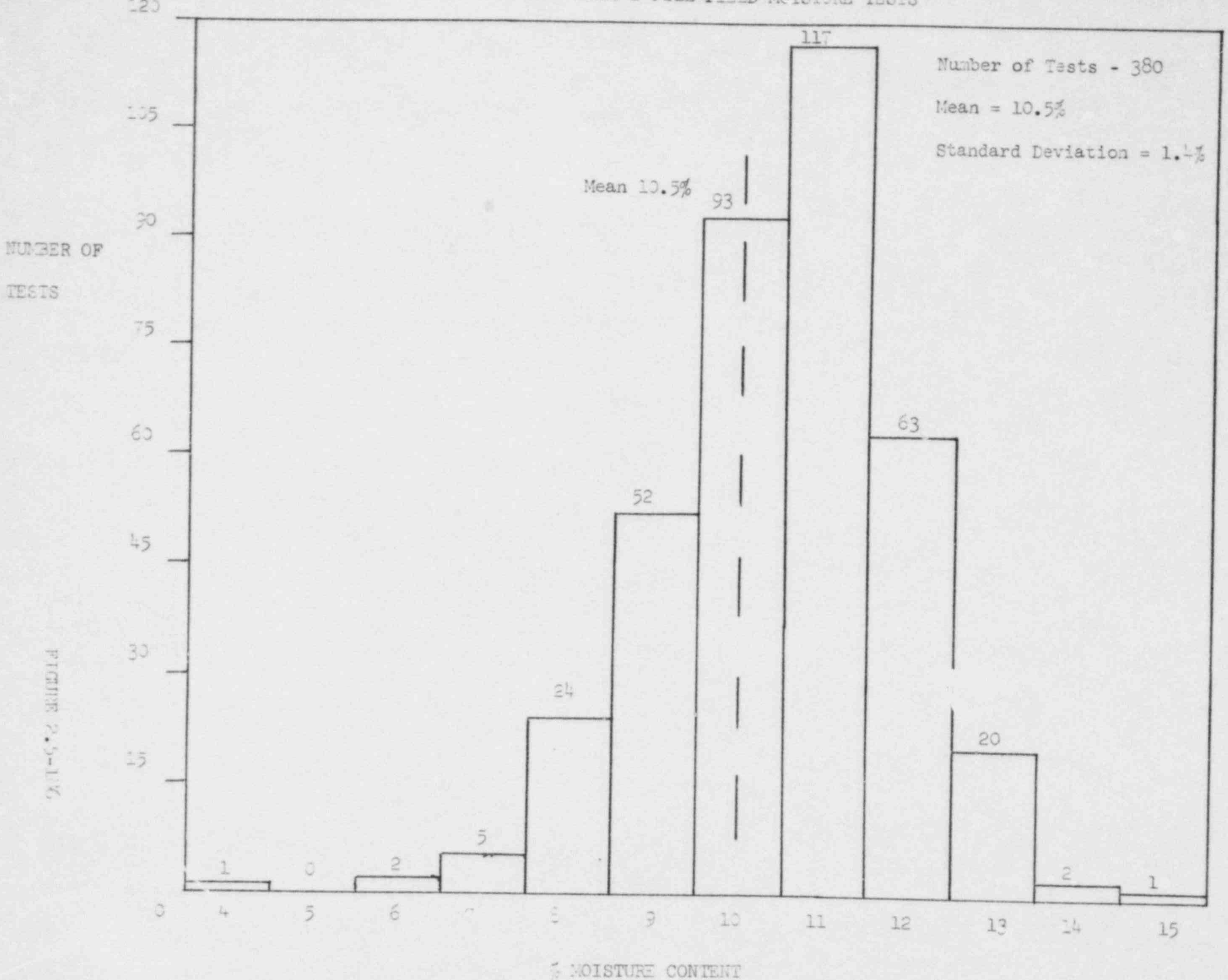


FIGURE 2.5-13

241.6 (2.5.4.10.4.1) In Section 2.5.4.10.4.1, justify the use of 0.54 as the value of the coefficient of earth pressure at rest. Provide a plot of maximum earth pressure vs. depth used to design subsurface walls under static and dynamic loads. Include and distinguish the added lateral pressures due to compaction of the backfill and the effect of compaction equipment.

Response

The response to this question is provided in revised Section 2.5.10.4.1.

For both the combined one-dimensional plus elastic and the plane strain, finite element analyses of foundation deformation, the time dependent compression and swell characteristics of the shale were estimated using the results of oedometer tests, presented in Section 2.5.4.2, and the records of monitored excavations in stiff clays and shales, reported by Moorhouse[188]. Selection of the amount of swell occurring before backfilling of the emergency service water pumphouse was very conservatively chosen to be one-third of the predicted ultimate swell of the excavation (the maximum possible) by assuming an interval of only 1 year between excavation and backfilling. Both theoretical and case history considerations predict from 1/2 to 7/8 of this ultimate swell would be expected within the 1-year period. This was confirmed by monitoring of shale movements during construction, as described in Section 2.5.4.13.

The drained and undrained volume change characteristics of the shale chosen for the deformation analyses were conservatively weighted towards the properties of the surficial shale zone. Because unsuitable shale has been excavated and mat foundations have been used, no attempt was made to model any localized variations in the properties of the competent shale which might be attributed to random differential weathering effects.

2.5.4.10.4 Lateral Earth Pressures

The magnitude and distribution of lateral earth pressures were formulated for application to the design of both nonyielding and yielding walls, the former typified by restrained substructure walls and the latter by cantilever retaining walls. The typically massive foundation walls of the Seismic Category I structures indicate that the nonyielding assumption is appropriate for these elements. The formulations in the following sections are conservative because the properties of Class B rather than Class A fill have been used throughout.

2.5.4.10.4.1 At-Rest Earth Pressure

Earth pressure, such as would be imposed against nonyielding walls, can be conservatively expressed above the groundwater by Equation 2.5-16. 241.6

$$p_o = 69.1\ Z + 0.54\ p_s \qquad (2.5\text{-}16)$$

Where: p_o = Lateral earth pressure at rest, psf

Z = Depth below horizontal backfill surface, ft

p_s = Surface surcharge loading, psf

The value of 0.54 used for the coefficient of earth pressure at rest was determined from the formula:

$$K_o = 1.0 - \sin \Phi'$$

Where : K_o = Coefficient of earth pressure at rest

Φ' = Effective angle of internal friction

The value of Φ' was assumed to be 27 degrees, which is a conversative value for the effective friction angle of Class B fill and totally ignores any contribution of effective cohesion for the fine-grained Class B materials. The 27 degrees friction angle results in a higher at rest earth pressure coefficient (0.54) than would be determined for Class A fill, which has a minimum design Φ' value of 35 degrees and an equivalent at rest earth pressure coefficient of 0.43.

241.6

The groundwater level is elevation 590.0' for normal operation and 618.0' for massive spill conditions for all structures except the Emergency Service Water Pumphouse; for this structure the groundwater level is elevation 557.0'. A surface surcharge loading of 100 psf was used for the construction loading condition to account for pressures due to construction equipment.

Below groundwater level, the effective weight of the backfill soil is reduced by byoyancy and a hydrostatic pressure component also acts on the wall. The two components of wall pressure are calculated in accordance with equations 2.5-17 and 2.5-18.

$$p_o = 69.1\ Z_w + 35.4\ Z_o + 0.54\ p_s \qquad (2.5\text{-}17)$$

$$p_w = 62.4\ Z_o$$

where: Z_o = Depth below groundwater level, ft

Z_w = Depth from surface to groundwater level, ft

p_w = Water pressure, psf

It is likely that compaction of the backfill adjacent to walls imposed pressures on the walls which were initially somewhat greater than the at-rest condition. However, the additional pressure would be expected to have dissipated within a relatively short time and, thus, is not a design condition. The conservatism in the design soil parameters and the various combinations of temporary loadings, as discribed in the next paragraph, would provide adequate reserve for any residual long-term compaction induced earth pressures.

Plots of the maximum earth pressure vs. depth used to design rigid subsurface walls for static and dynamic loads are provided in Figures 2.5-176 and 2.5-177. Diagrams in Figure 2.5-176 are applicable to all Category I structures, except for the emergency service water pumphouse which is shown in Figure 2.5-177. Each structure was analyzed to determine the maximum design stresses resulting from the following earth pressure loading conditions: 241.6

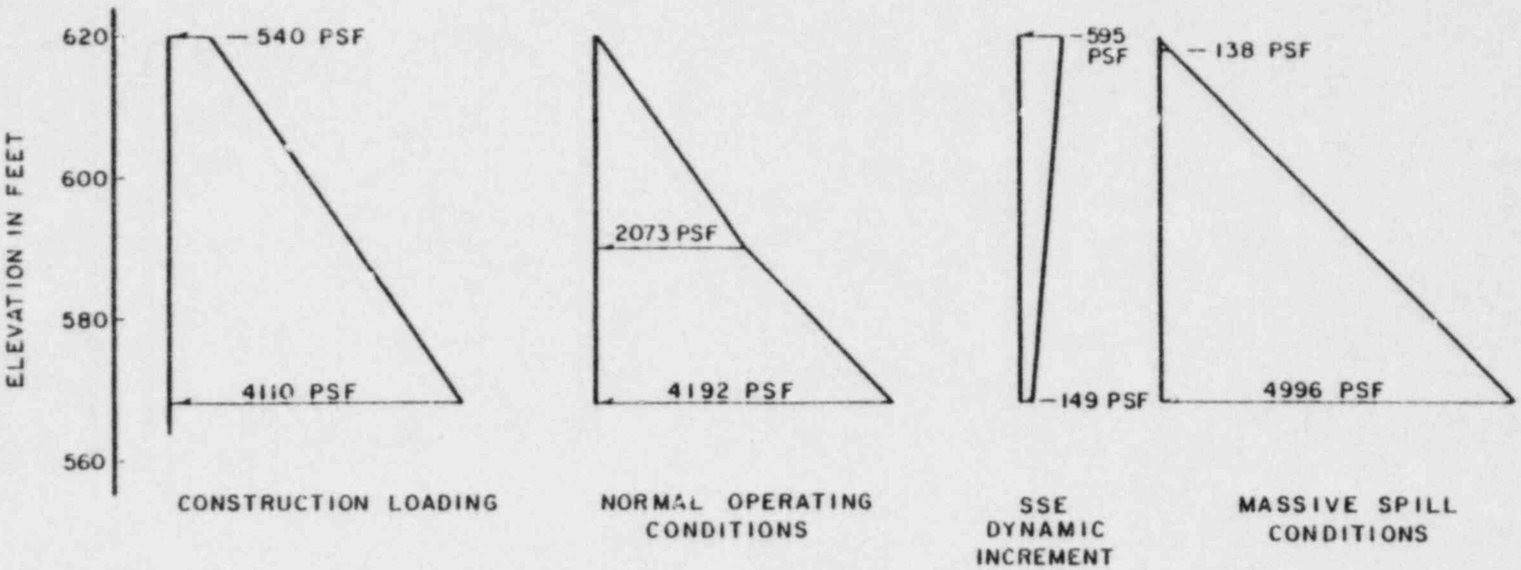
(1) construction loading

(2) normal operating conditions,

(3) normal operating conditions plus the SSE event increment, and

(4) massive spill conditions.

Additional loadings due to surcharge from such items as cranes, railroads and adjecent foundations were added as necessary, on a case-by-case basis.



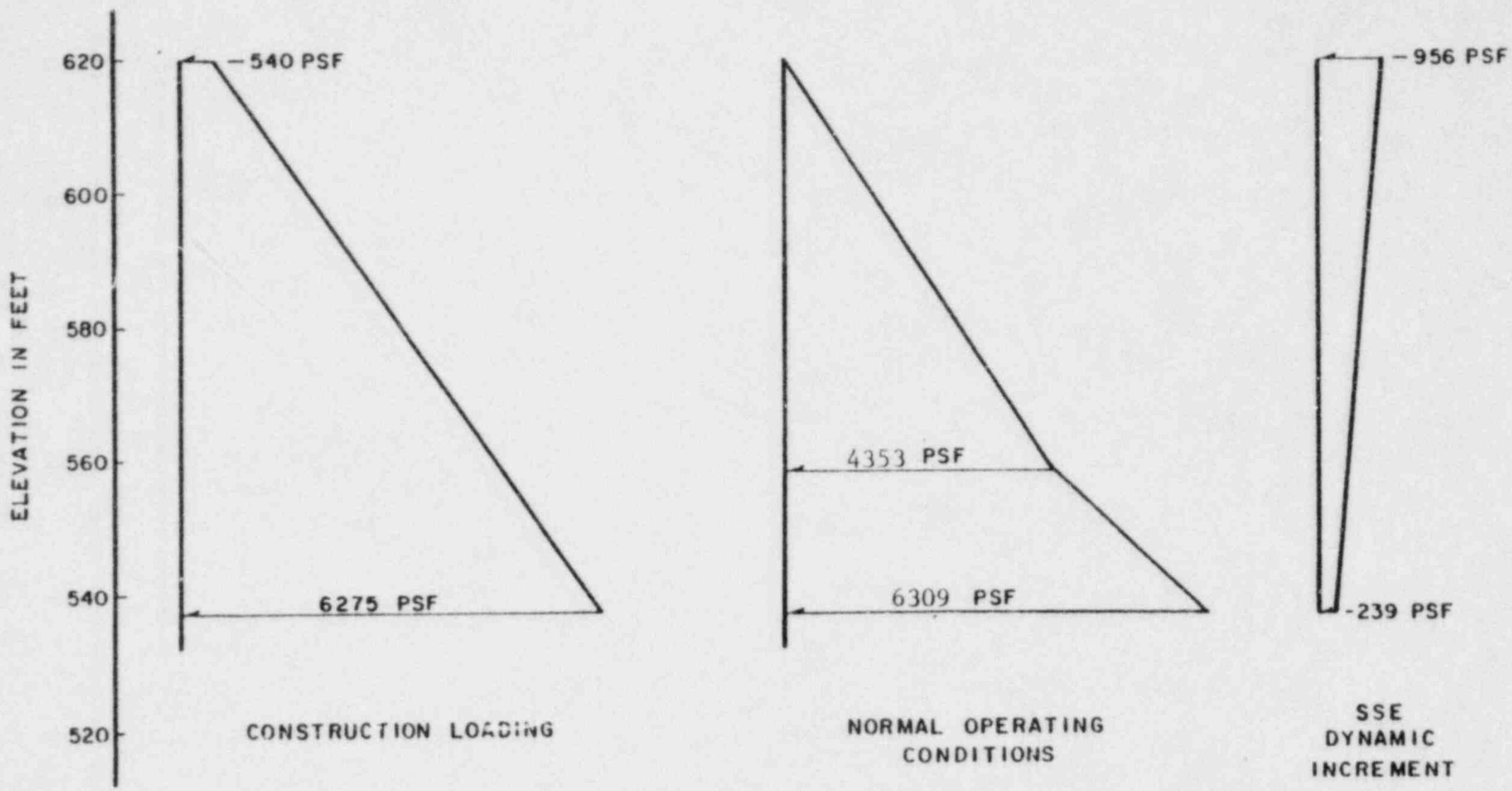
NOTE: 1. DYNAMIC INCREMENT ADDED TO NORMAL OPERATING CONDITIONS FOR SSE EVENT

2. ADDITIONAL LOADINGS DUE TO SURCHARGE FROM CRANES, RAILROADS OR ADJACENT FOUNDATIONS ADDED AS REQUIRED

3. FOR LOADS APPLICABLE TO EMERGENCY SERVICE WATER PUMPHOUSE SEE FIGURE 241.6-2

EARTH PRESSURE DIAGRAMS FOR RIGID SUBSURFACE WALLS

FIGURE 2.5-176



NOTE: DYNAMIC INCREMENTS ADDED TO NORMAL OPERATIONS CONDITIONS FOR SSE EVENT

EARTH PRESSURE DIAGRAMS
FOR EMERGENCY SERVICE WATER
PUMPHOUSE SUBSURFACE WALLS

FIGURE 2.5-177

241.7 Provide construction details of the settlement monitoring points.

(2.5.4.13.4) Update the time vs. settlement plots for all Category 1 structures where settlements are being monitored. Discuss any deviations from anticipated settlements assumed in the analysis and design of these structures and components. Evaluate all deviations for their impact on the design and construction of these structures and components.

Response

The response to this question is provided in revised Section 2.5.4.13.4 and new Section 2.5.4.13.5. Also, see revised Figures 2.5-162, 2.5-171, 2.5-172(2 sheets), 2.5-173(5 sheets), and new Figures 2.5-173(Sheet 6), 2.5-173a and b, and 2.5-178.

2.6 and 1.2 inches of immediate rebound, followed by 1.2 and 0.7 inch of time-dependent heave, respectively. These two gauges were located within a bedrock deformation zone consisting of an anticlinal fold traversing Unit 2 reactor building, striking northwesterly and bounded vertically on competent rock. Heave of fractured rock in the deformation zone, exposed to extreme climatic conditions, has been attributed to post-excavation stress reduction and swell associated with shale weathering. Heave gauge HG-5 was destroyed during excavation; hence, no data was acquired for this gauge.

2.5.4.13.3 Shale Extensometers

Six extensometers were installed in the sidewalls of the emergency service water pumphouse, as shown in Figure 2.5-168, to monitor horizontal movements of the shale. A typical installed detail of an extensometer is shown in Figure 2.5-169. Monitoring results are shown in Figure 2.5-170.

The shale movements measured by the extensometers ranged from essentially 0.0 to 0.1 inch and were judged to be essentially completed about 10 months after the completion of excavation. Although some later movement was detected in extensometers EX-2 and EX-5 during the last 4 months of monitoring, it is likely that at least some of that movement can be attributed to vibrations or other disturbance relating to an increased level of construction activity in the ESW pumphouse excavation during that period.

2.5.4.13.4 Settlement Monitoring

Settlement monitoring points have been established in the interior of the reactor buildings, diesel generator building and off-gass buildings, and on the exterior walls of various other Seismic Category I structures. The settlement monitoring points were typically designated by pencil or paint marks on poured concrete or steel frame structural elements. The locations of the exterior points are shown on Figure 2.5-171. The interior reactor points are located near the outer circumference of each reactor building, with eight points in each building spaced 45 degrees apart. 241.7

Figure 2.5-172 shows the recorded movements of the settlement points within the reactor buildings, together with the approximate time history of the percentage of structural concrete which was placed in these structures. It is noted, however, that these recorded movements are with reference to a monument within the control complex which experienced a settlement of 0.64 inch during the period from November 1976 through February 1981. Therefore, the average actual settlement for Unit 1 reactor is about 0.53 inch and that for Unit 2 reactor is about 0.67 inch, through December 1980. Monitoring of the interior of the reactor buildings was discontinued after December 1980, due to inaccessibility. However, monitoring of the exterior reactor points will continue.

Figure 2.5-173 shows the results of settlement measurements at the six points shown on Figure 2.5-171 on the exterior walls of various structures. Settlement points were initially established at low elevations when the lower portions of the walls were cast (elevations ranging from about 563' to 574'). As the walls were raised and backfill placed around the structures, the settlement points were also raised to higher elevations. The settlement data obtained is conservative because the settlement of the higher points includes the elastic deformation of the underlying concrete walls. Occasionally, 241.7
settlement points were covered by construction activities before the next higher corresponding point was established. Gaps in the settlement records occur at these times and the settlements which occurred during these periods have been estimated, as shown in Figure 2.5-173.

The maximum settlement recorded to date is about 0.9 inch. It should be noted, however, that in some cases substantial amounts of structural concrete was placed prior to the start of monitoring. Permanent brass settlement markers are installed at each location as the walls are extended above finished exterior grade (about elevation 620'), and continuous post-construction settlement records will be obtained from these markers. Construction details of the settlement monitoring points are shown on Figure 2.5-178.

Six settlement monitoring points were extablished on the diesel generator building in June 1979, shortly after the structural mat was cast. Seven new points were established at a slightly higher elevation in June 1980, and the old points were subsequently abandoned. The monitoring results, shown on Figure 2.5-173a, indicate that the average settlement through February 1981 has been slightly less than one-half inch.

At the request of NRC, settlement points were established on the off-gas buildings after the structural concrete for these structures had been completed. Four points were established within the Unit 1 structure and three points within the Unit 2 structure. As shown in Figure 2.5-173b, the maximum average settlement of these structures during the period from June 1980 through February 1981, has been about 0.04 inches and 0.12 inches, respectively. The maximum settlement of any individual monitoring point has been 0.06 inches for Unit 1 and 0.16 inches for Unit 2. 241.7

2.5.4.13.5 Comparsion of Actual and Predicted Deformations

Figure 2.5-162 shows the anticipated deformation behavior of the Unit 1 reactor building, as discussed in Section 2.5.4.10.3. The deformation sonsists of three phases: heave of the shale bearing surface during the following excavation, rapid compression during construction and backfill of the structures and, finally, long-term post-construction consolidation at a very slow rate. The calculated deformation behavior for the reactor building is typical of all of the structures on the site.

The computed heave of the shale within the main plant excavation ranged from about 1/2 to 3/4 inch. As discussed in Section 2.5.4.13.2, the actual heave was only about 1/4 to 1/2 inch, except within the area of a bedrock deformation zone which was subsequently excavated.

241.7

The computed immediate settlement for the auxiliary buildings, radwaste building and control complex was about 1/2 inch in the interior and about 1/4 inch along the edges of the buildings adjacent to the toe of the plant excavation. The analysis method, however, did not account for structural rigidity of the foundation mats which would tend to decrease the interior settlement and increase the edge settlement. The actual immediate settlement of these structures, as measured at settlement points SP-1, SP-4, and SP-6, plus the disk in the control complex, has been about 1/4 to 3/4 inch, averaging about 1/2 inch, through February, 1981. Long-term settlement after completion of construction is expected to be on the order of 1/10 inch.

The calculated immediate settlement of the reactor buildings was about 3/4 inch in the interior and 1/3 to 1/2 inch along the edges. Again, the structural rigidity of the mat would tend to increase the settlement of the edges. The actual settlement, as measured at the 16 interior points on the reactor mat, as well as settlement points SP-2 and SP-3, has been about 1/2 to 1 inch through February 1981. Long-term settlement, after completion of construction, is expected to be on the order of an additional 1/10 inch.

It is concluded that settlement of the Seismic Category I structures is very small and of the magnitude anticipated. POst-construction differential settlement is expected to be negligible. 241.7

2.5.4.14 Construction Notes

2.5.4.14.1 Shale Deformation

The onshore geologic structures mapped and investigated at the site had little impact on the plant foundation as designed. To preclude the possibility of fines infiltrating porous concrete, the following additional construction measures were taken. Degraded material was overexcavated and replaced with lean concrete having a 28-day compressive strength of at least 1500 psi. Exposed joints, open or filled, were cleaned and filled with slush grout.

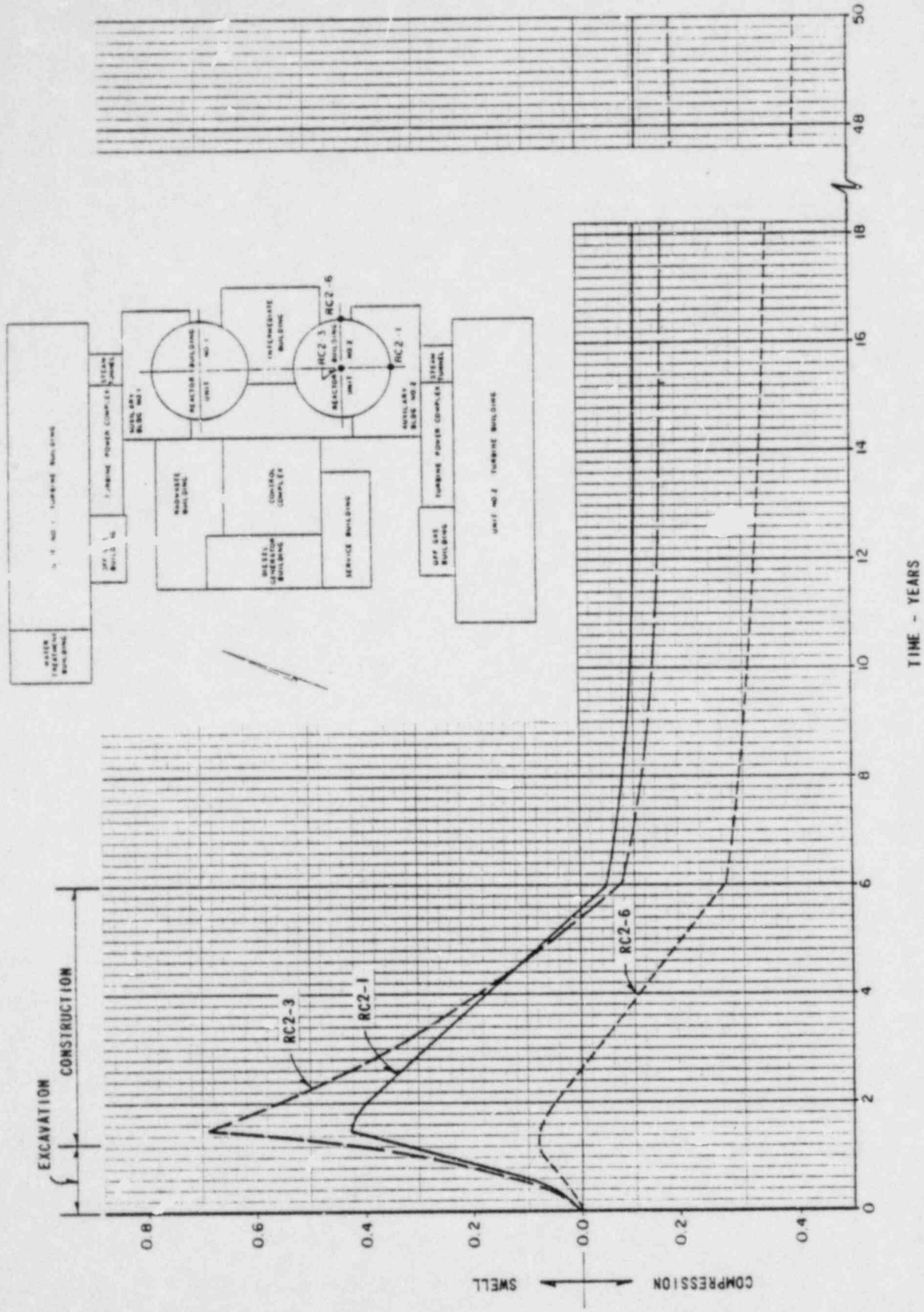


FIGURE 2.5-162

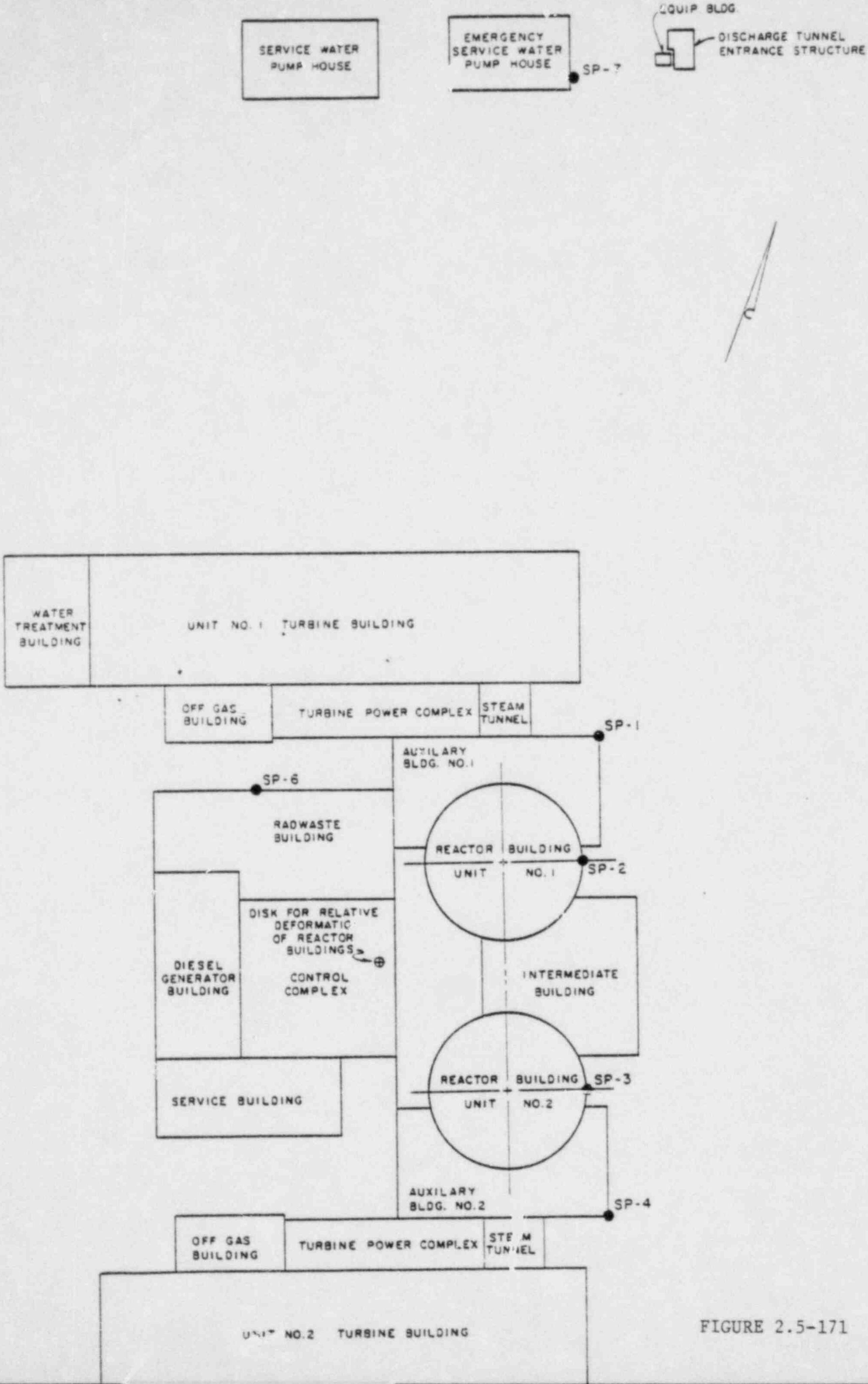
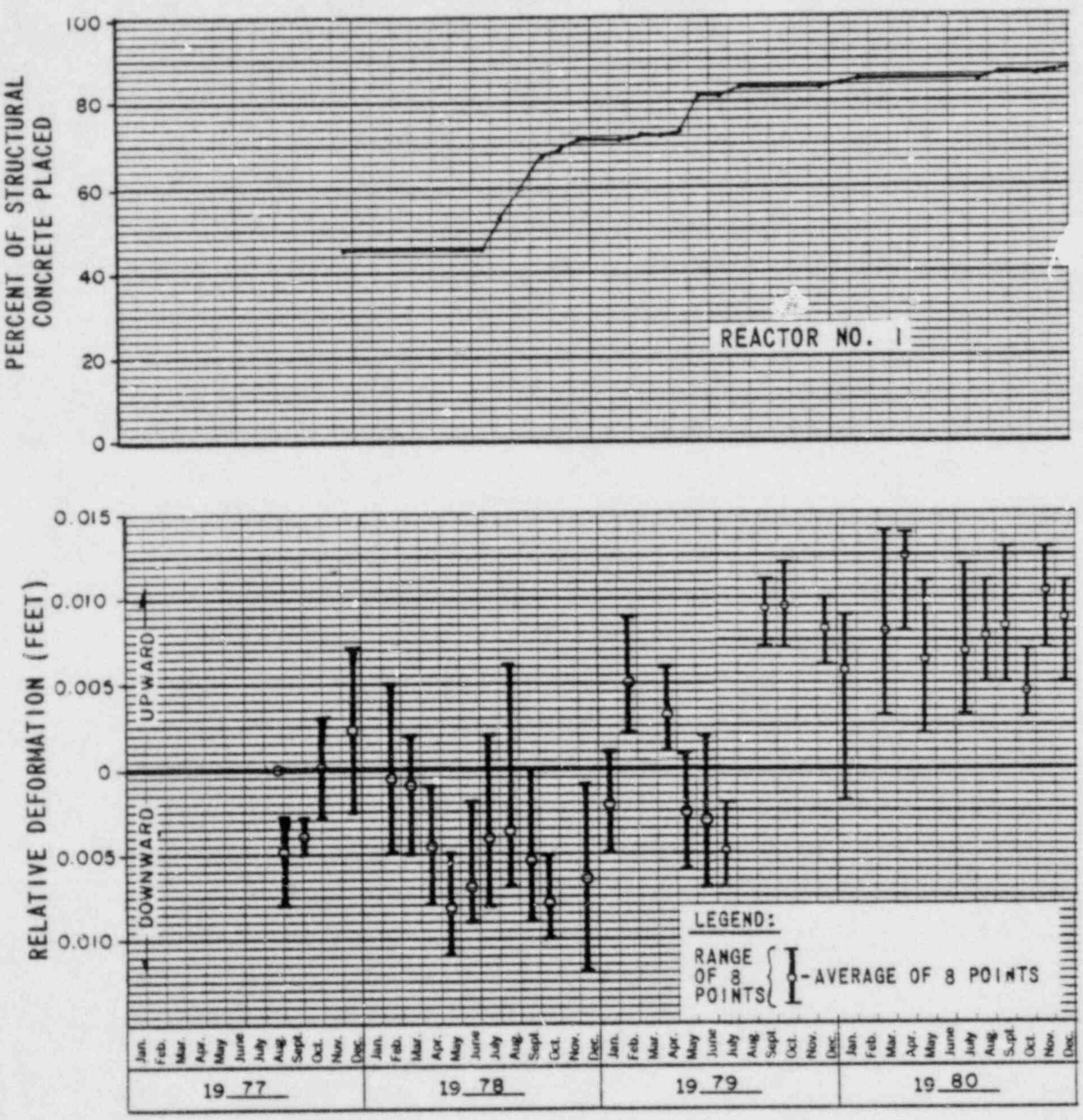
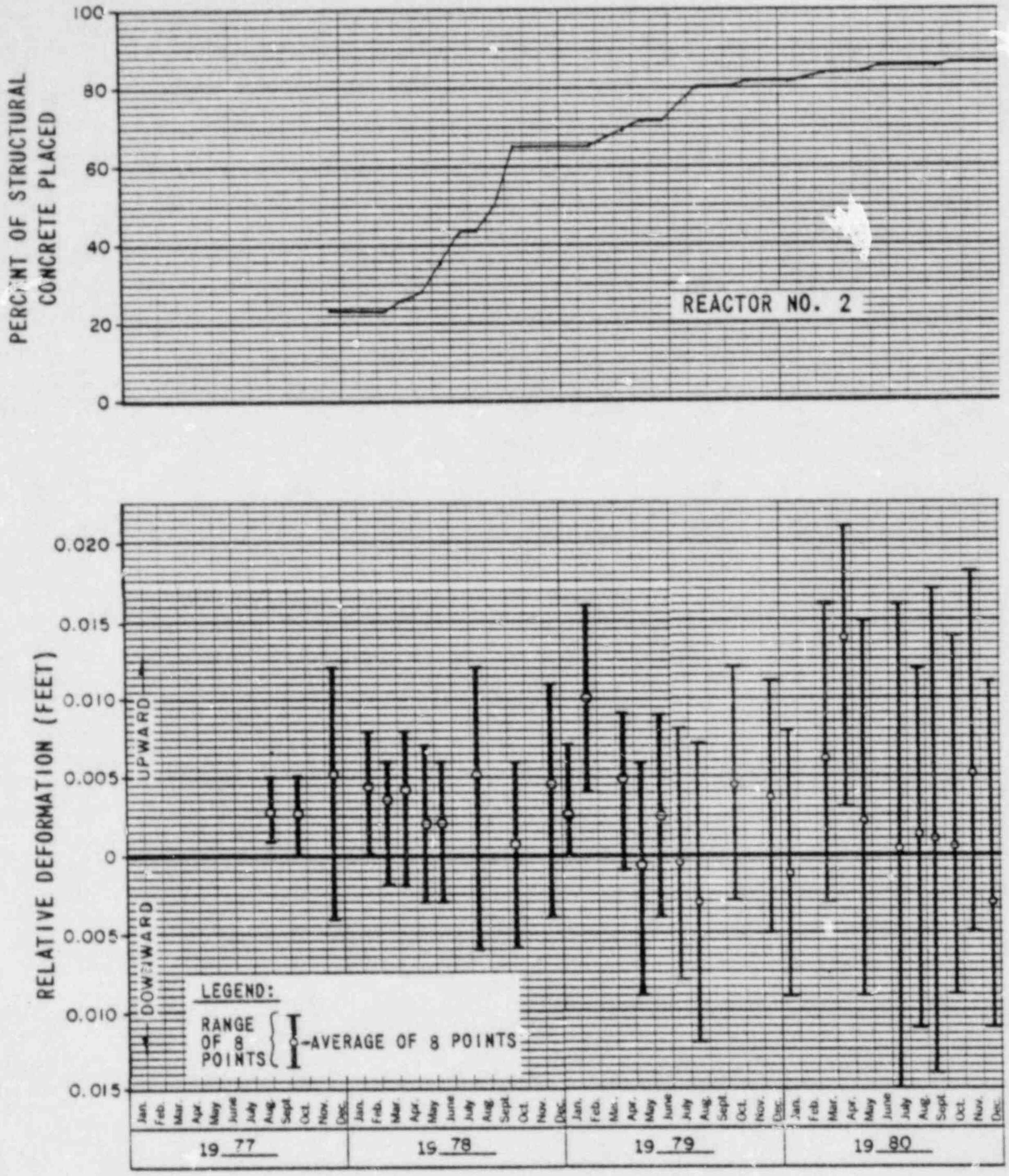


FIGURE 2.5-171



NOTE: MEASURED DEFORMATION IS RELATIVE TO MONUMENT WITHIN CONTROL COMPLEX

FIGURE 2.5-172 (1 of 2)



NOTE: MEASURED DEFORMATION IS RELATIVE TO MONUMENT WITHIN CONTROL COMPLEX

FIGURE 2.5-172 (2 of 2)

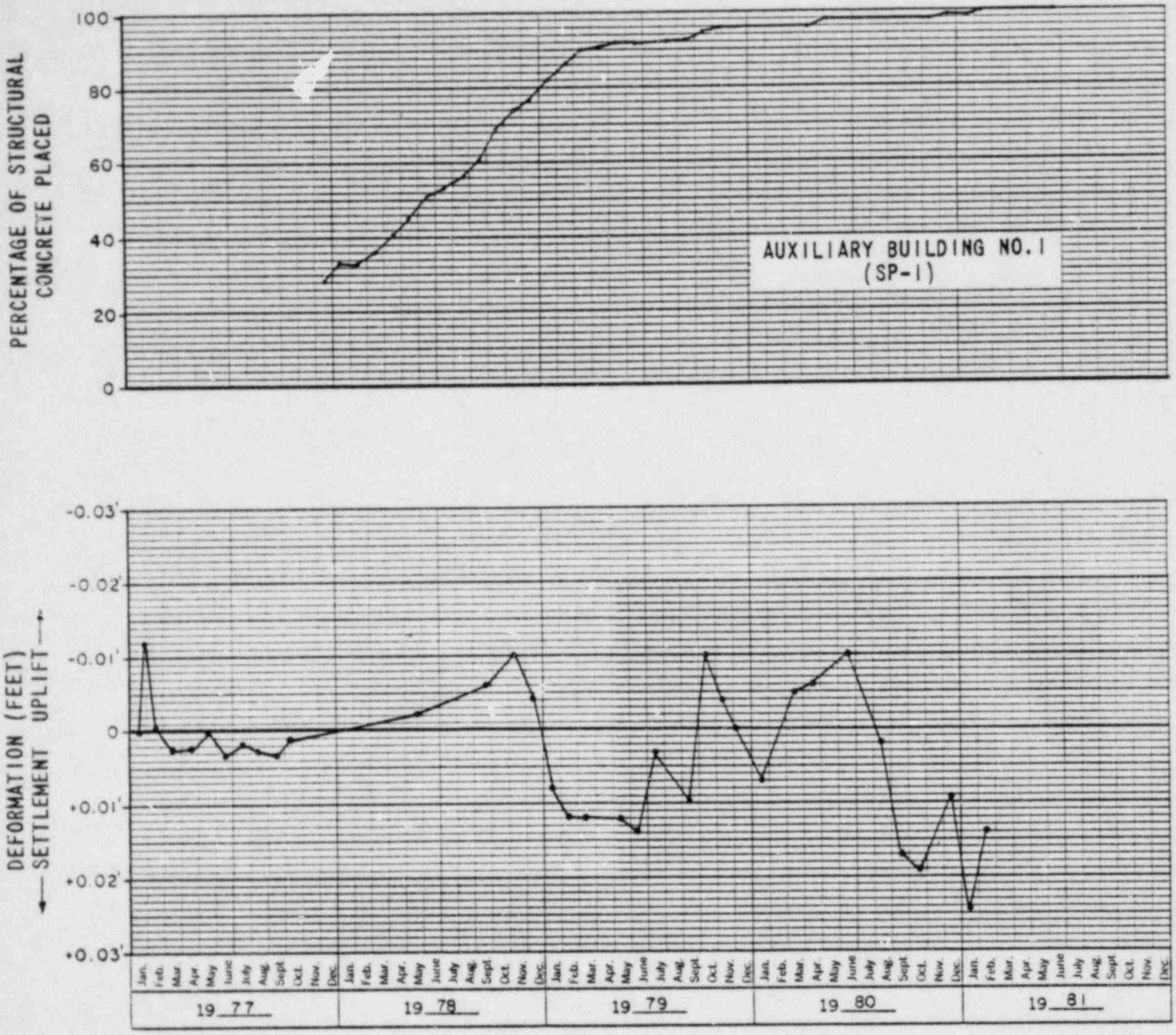


FIGURE 2.5-173 (1 of 6)

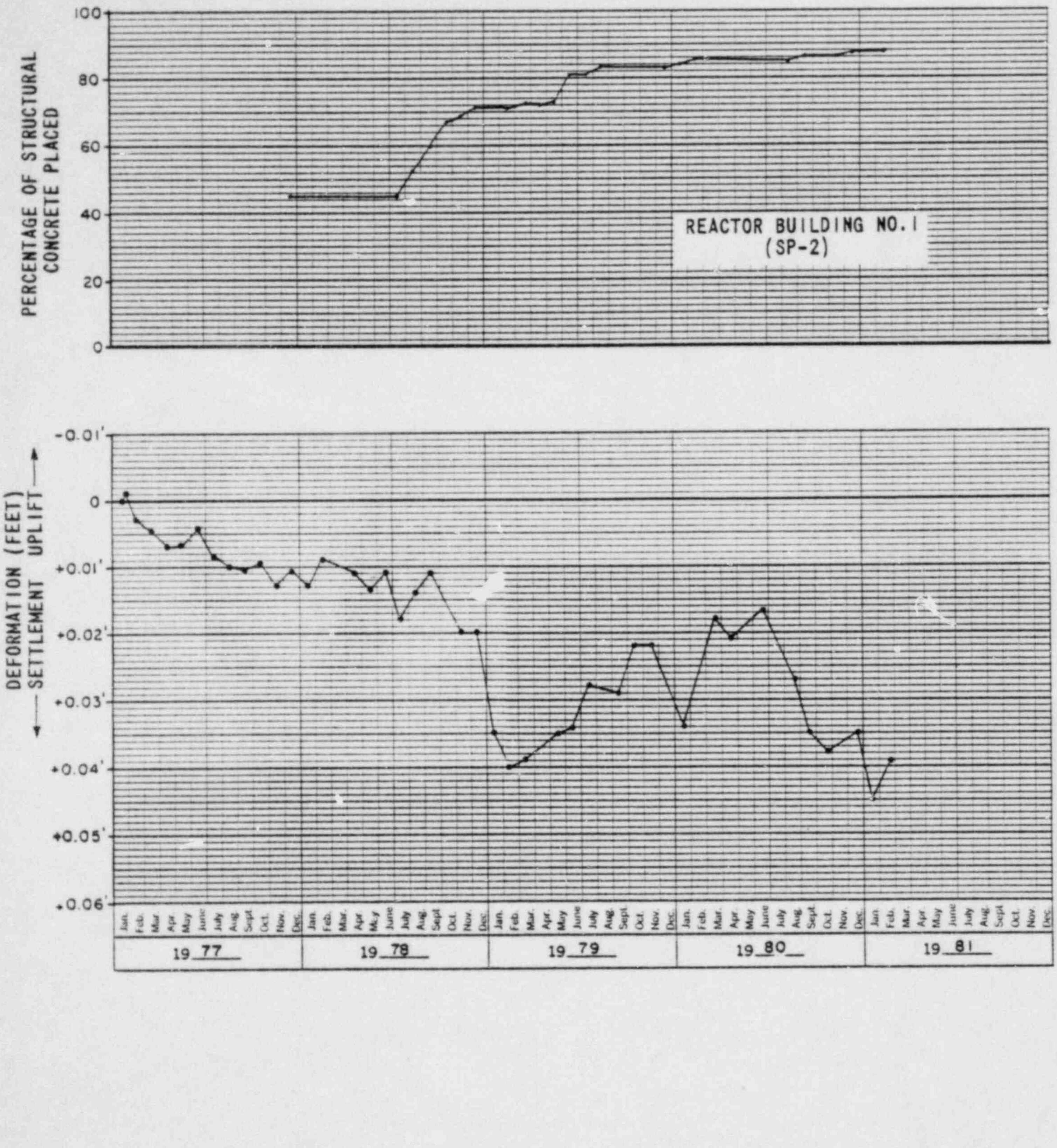


FIGURE 2.5-173 (2 of 6)

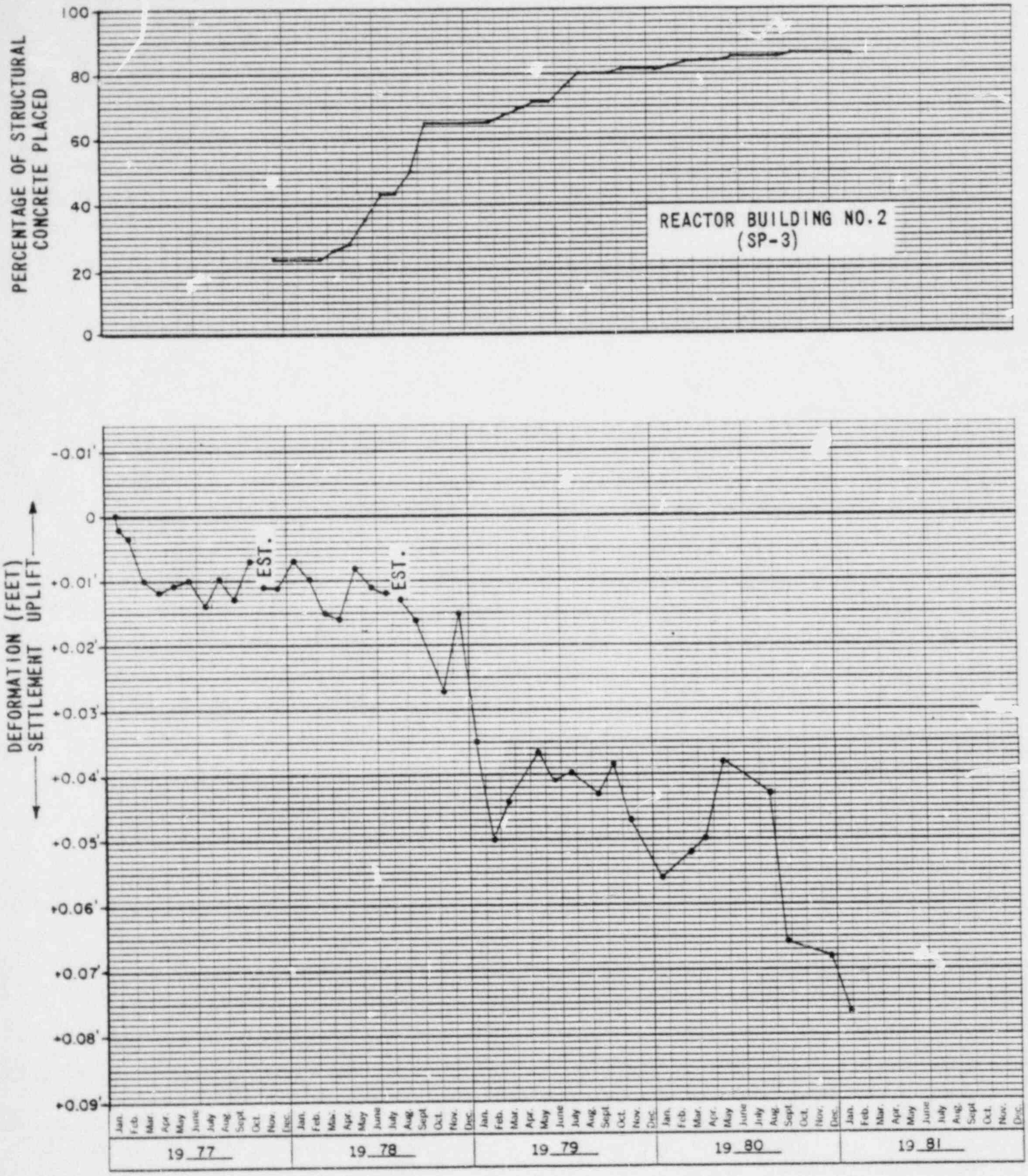


FIGURE 2.5-173 (3 of 6)

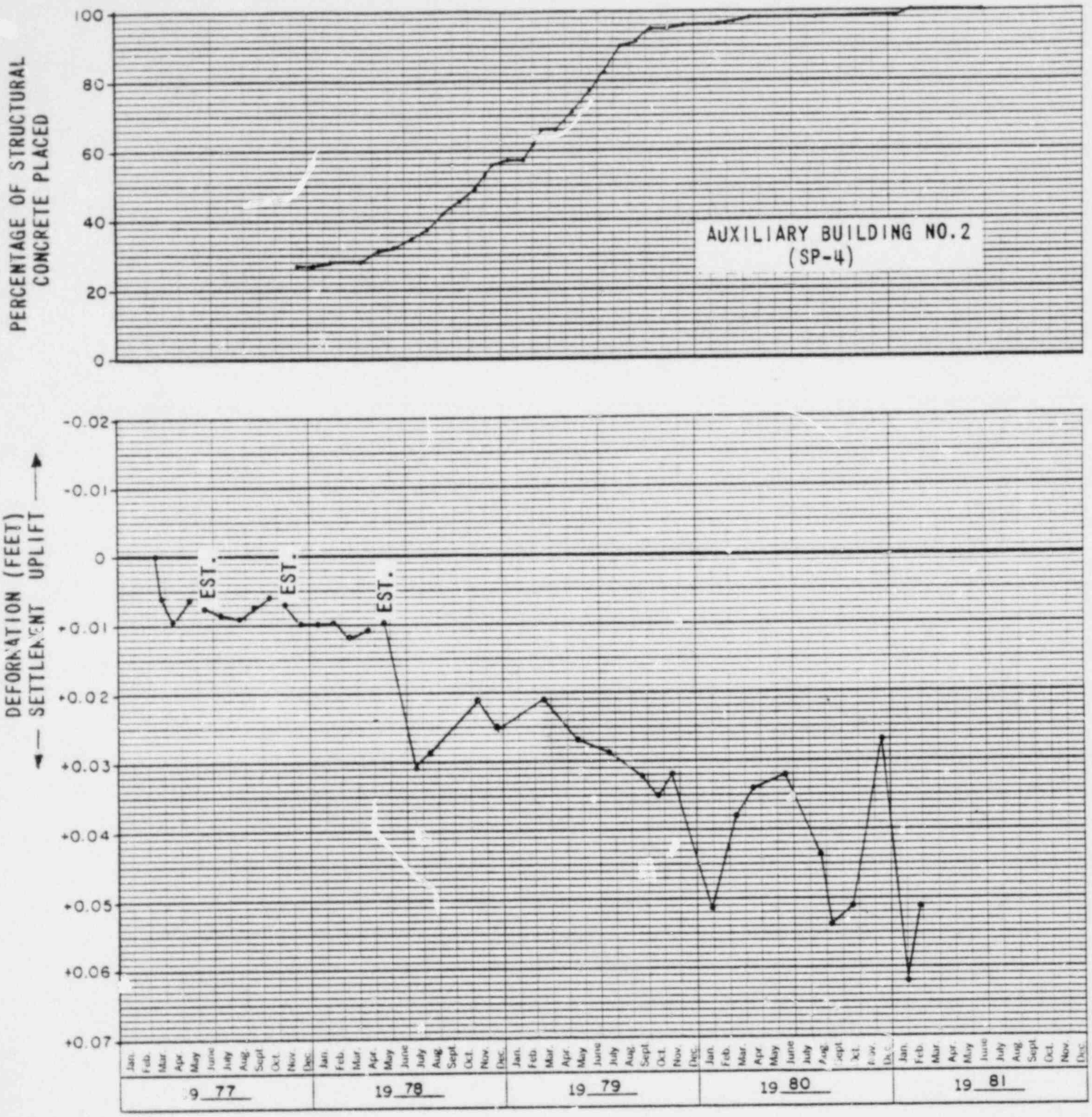


FIGURE 2.5-173 (4 of 6)

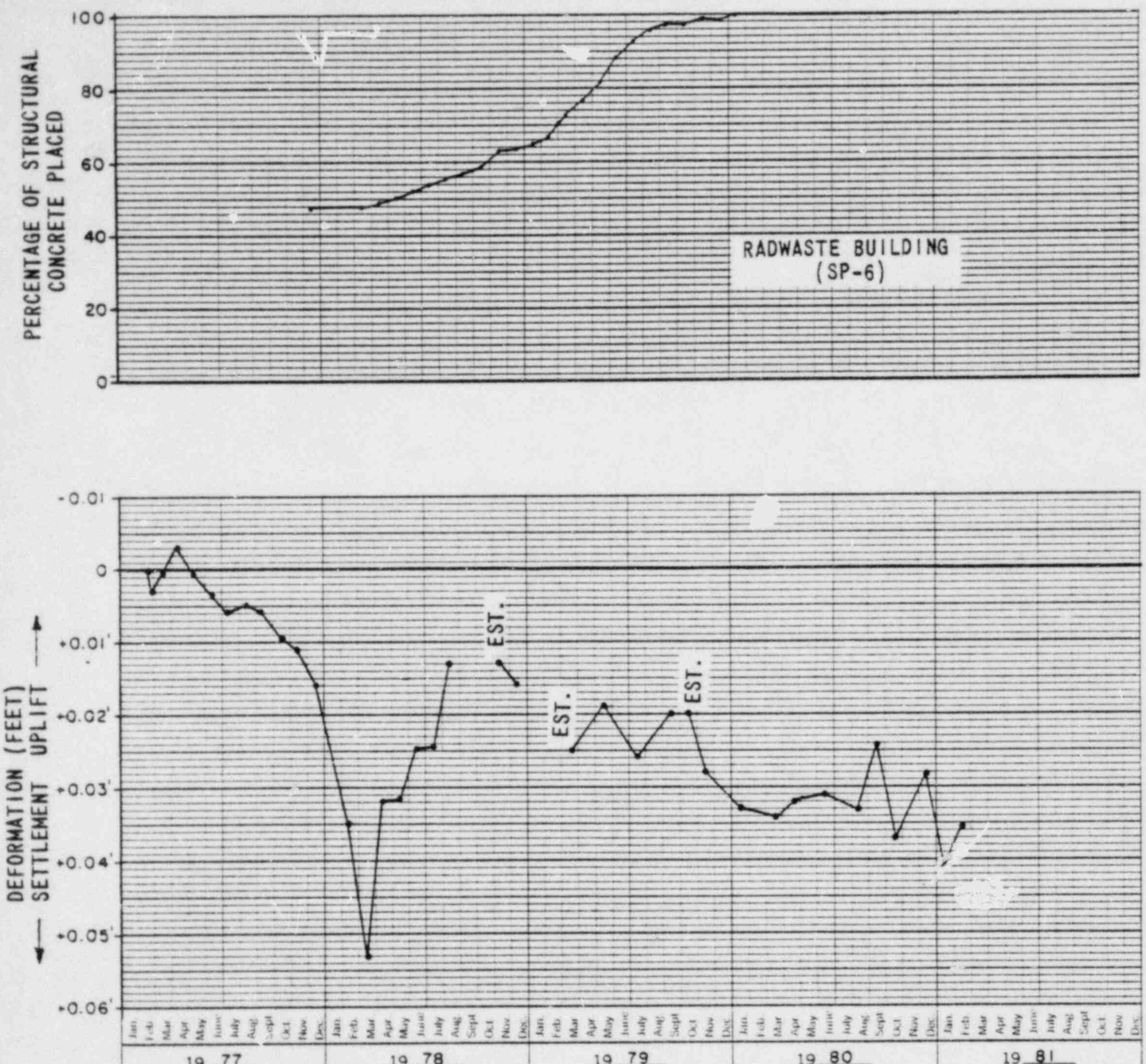


FIGURE 2.5-173 (5 of 6)

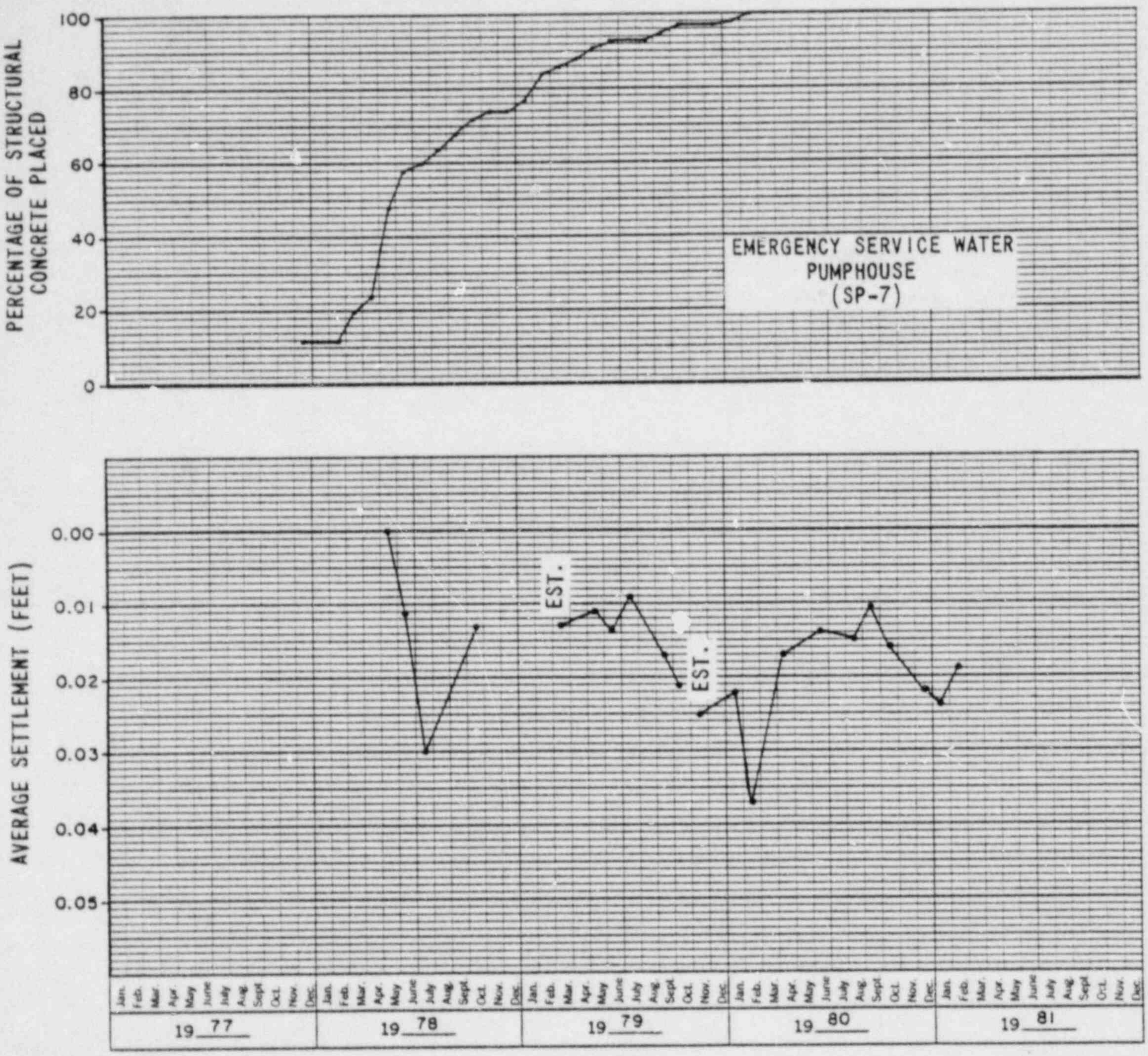


FIGURE 2.5-173 (6 of 6)

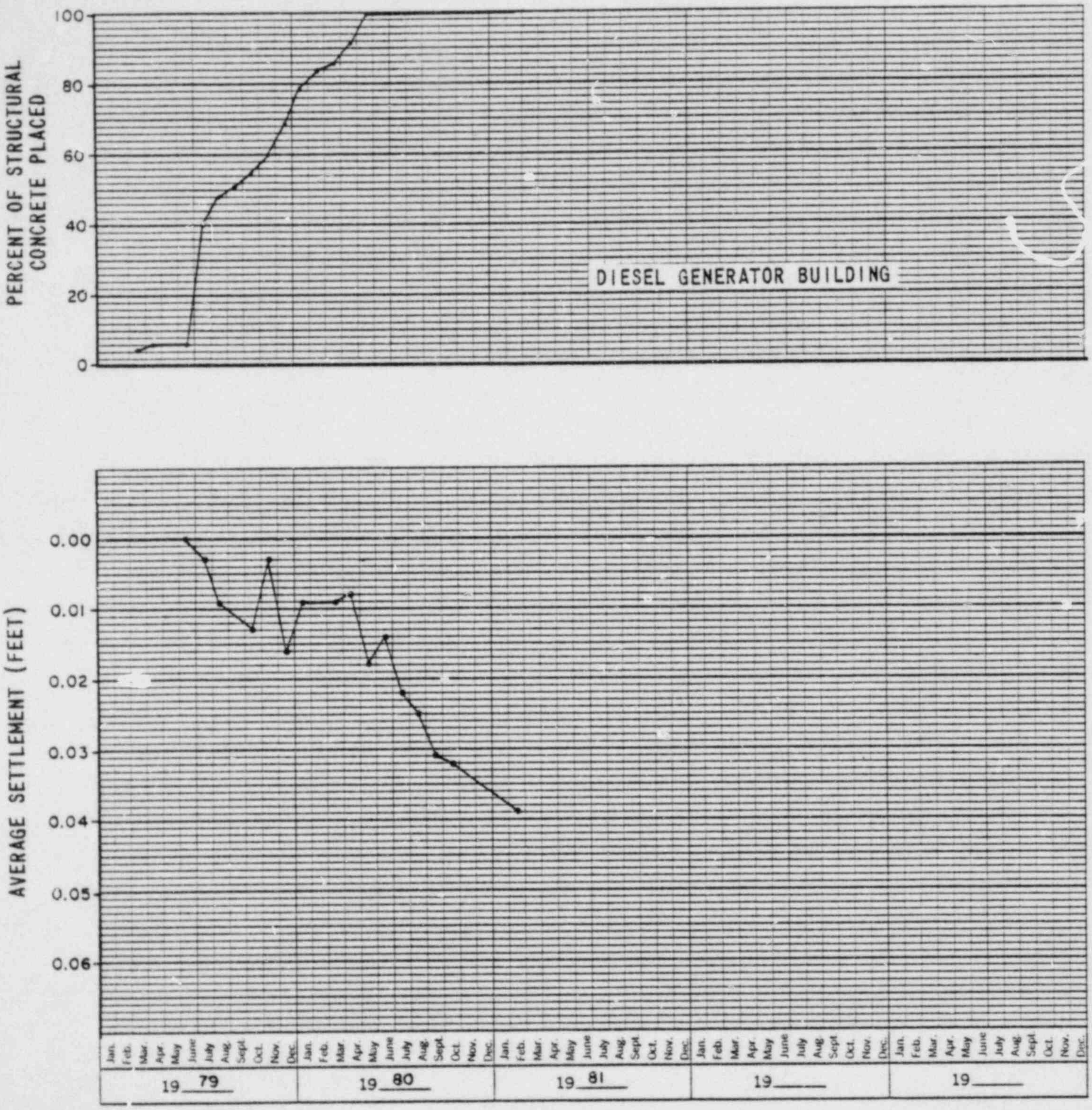
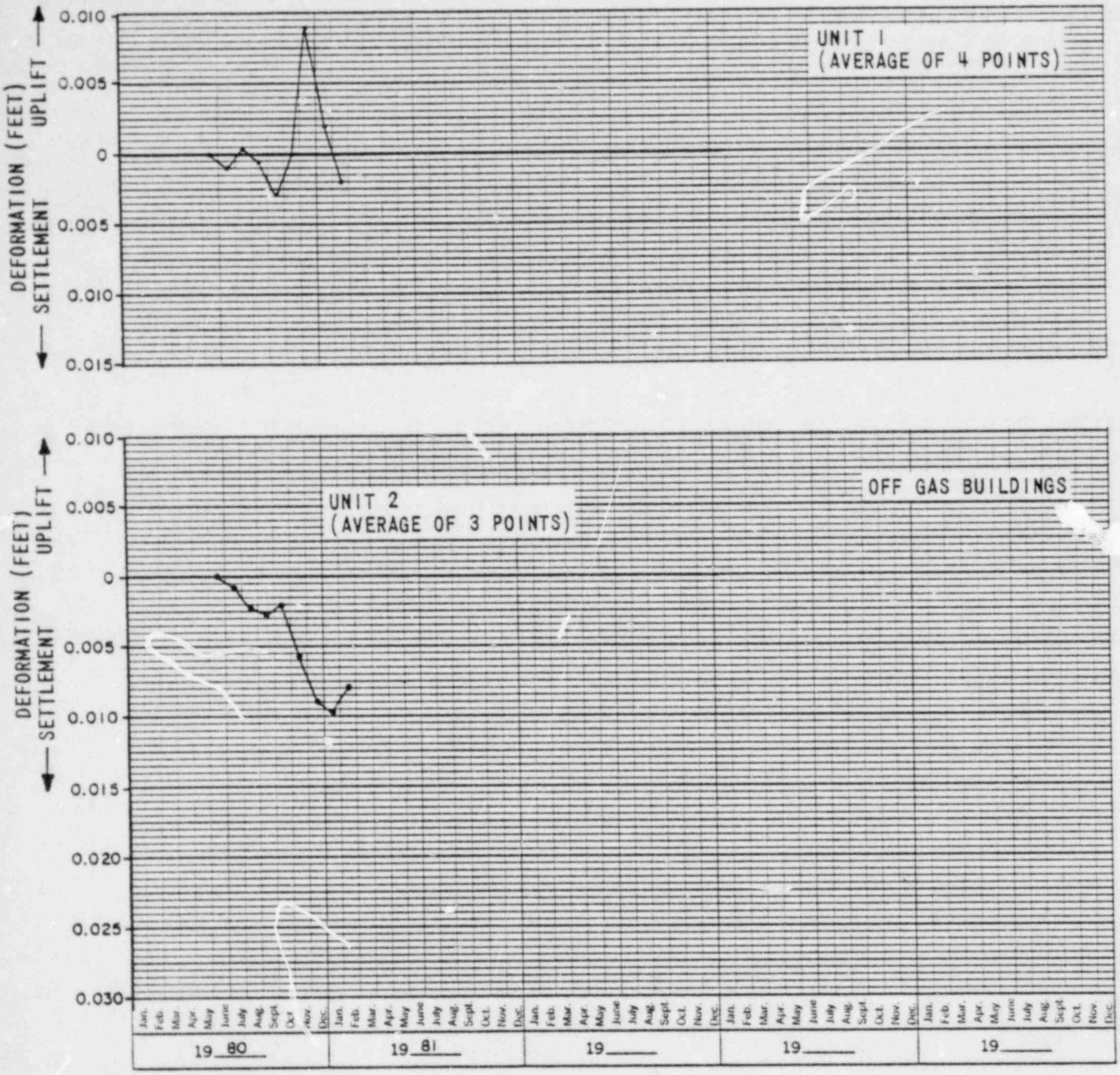
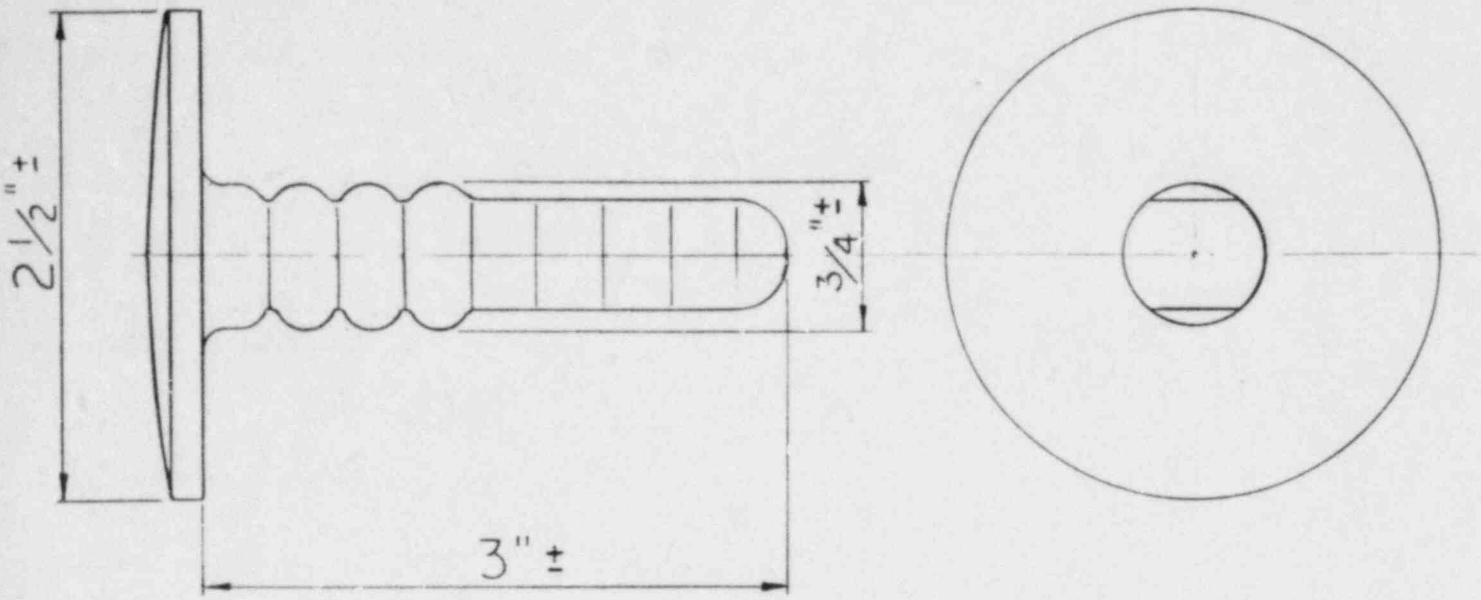


FIGURE 2.5-173a

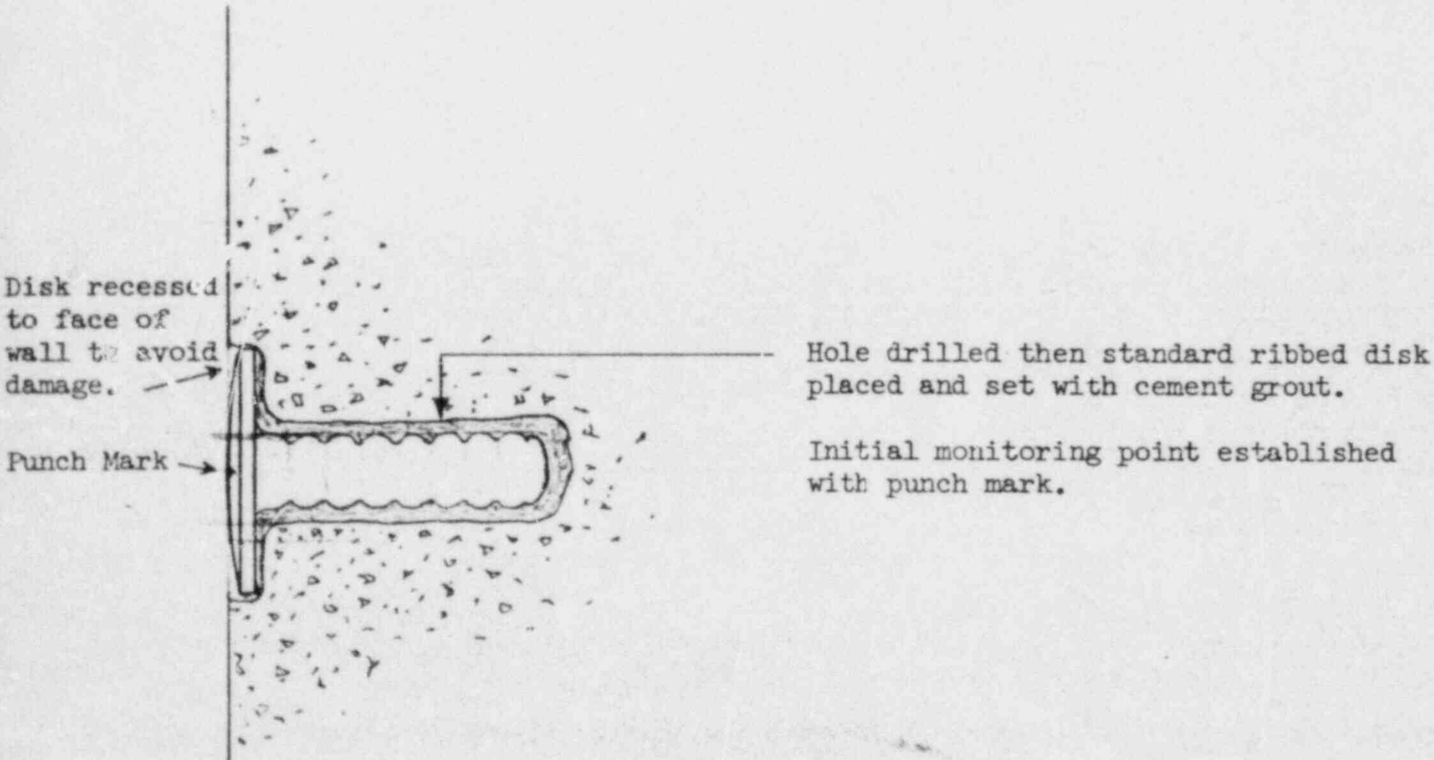


NOTE: STRUCTURAL CONCRETE PLACEMENT COMPLETED PRIOR TO SETTLEMENT MONITORING.

FIGURE 2.5-173b



BRASS DISK DIMENSIONS



TYPICAL DISK INSTALLATION

Building Settlement
Monitoring Disks
Fig. 2.5 - 178

241.8 The Staff requires construction details of the remedial work done in connection with the installation of 15 double Class B fill waterstops, inadvertently omitted by the contractor (refer to letter from D. R. Davidson, Vice President, System Engineering and Construction, Cleveland Electric Illuminating Co., to Mr. James Keppler, Director, Region III, Office of I&E, USNRC, Glen Ellyn, Illinois, dated February 29, 1980). Were there other similar omissions? If so, discuss any corrective measures undertaken to resolve the situation.

Response

Where Class B fill waterstops were inadvertently omitted by the contractor from pipelines penetrating the Class A fill surrounding the main plant buildings, the contractor had to excavate down to these pipes and ensure that two minimum 3 ft wide zones of Class B fill were installed completely around the pipe. Where possible, the one waterstop was installed relatively close to the buildings and the other installed approximately 12 feet away along the pipe. These waterstops were installed in accordance with the original contract documents and under the direction of the Resident Geotechnical Engineer.

No other waterstops were inadvertently omitted.