Enclosure 1

TECHNICAL REPORT

STRUCTURAL STRESSES INDUCED BY DIFFERENTIAL SETTLEMENT OF THE DIESEL GENERATOR BUILDING

CONSUMERS POWER COMPANY

MIDLAND PLANT UNITS 1 AND 2

MIDLAND PLANT UNITS 1 AND 2
TECHNICAL REPORT
STRUCTURAL STRESSES INDUCED BY
DIFFERENTIAL SETTLEMENT OF THE
DIESEL GENERATOR BUILDING

TABLE OF CONTENTS

APPENDIX

TABLES

Table of Contents (Continued)

FIGURES

MIDLAND PLANT UNITS 1 AND 2
TECHNICAL REPORT
STRUCTURAL STRESSES INDUCED BY
DIFFERENTIAL SETTLEMENT OF THE
DIESEL GENERATOR BUILDING

1.0 STRUCTURAL REANALYSIS

To account for the effect of the observed and predicted settlement on the diesel generator building, a structural reanalysis was performed. This reanalysis proceeded by defining the acceptance criteria for the structure (see Subsection 1.1). These acceptance criteria differ from the acceptance criteria used in the original design and analysis of the structure and set forth in the Final Safety Analysis Report (FSAR) only in the addition of four load combinations that include the effect of settlement. These additional load combinations are described in Subsection 1.1.2.

To investigate the effects of the load combinations on the structure, the structural reanalysis uses two different mathematical models of the diesel generator building: a dynamic, lumped mass model and a static, finite element model. The dynamic, lumped mass model (described in Subsection 2.1.6 and illustrated in Figure I-1) is used to generate seismic forces in the building, given the input ground motion from the operating basis earthquake and safe shutdown earthquake (SSE) specified in the FSAR.

The finite element model illustrated in Figure I-2 is a more complex mathematical model that reduces the diesel generator building to an interrelated system of plate, beam, and boundary elements representing the walls, slabs, foundation, and soil. The finite element model is used to assess the effect on individual elements of various load combinations applied to the structure as a whole. (These load combinations include seismic forces generated with the dynamic, lumped mass model.) The finite element model thereby allows the identification of those sections of the diesel generator building that will experience the greatest forces due to the postulated load combinations. The allowable stress is then calculated and compared to the actual stress level in these sections based on the forces derived from the finite element model. This comparison shows that even those sections of the building experiencing the highest forces meet the acceptance criteria.

1.1 STRUCTURAL ACCEPTANCE CRITERIA

Because of the settlement problem, a structural reanalysis of the diesel generator building was performed in accordance with the structural acceptance criteria which are consistent with FSAR Subsection 3.8.6.3, with settlement effects included as outlined in the response to NRC Requests Regarding Plant Fill, Question 15 (Revision 3, September 1979).

1.1.1 Load Cases

The following loads are considered in the reanalysis:

a. Dead loads (D)

b. Effects of settlement combined with creep, shrinkage, and temperature (T)

c. Live loads (L)

d. Wind loads (W)

e. Tornado loads (W')

f. OBE loads (E)

g. SSE loads (E')

h. Thermal effects (T_0)

Thermal effects appear twice in this list (Items b and h). For load combinations committed to in the response to Question 15 of the NRC Requests Regarding Plant Fill, thermal effects are contained within the settlement effects term, T. For load combinations committed to in FSAR Subsection 3.8.6.3, thermal effects are contained in the thermal term, T_0 (Refer to Table I-1).

All other load cases appearing in the load combinations for Seismic Category I structures listed in FSAR Subsection 3.8.6.3 (e.g., rupture of pipe lines) do not occur in the diesel generator building and are not addressed.

1.1.2 Load Combinations

The load combinations employed for the original analysis and design of the diesel generator building are provided in FSAR Subsection 3.8.6.3. The original FSAR load combinations did not contain a settlement effects term (T). For the structural reanalysis performed in response to Question 15 of the NRC Requests Regarding Plant Fill (September 1979), four additional load combinations were established and committed to be considered. These additional combinations consider the effects of differential settlement in combination with long-term operating conditions and with either wind load or OBE. Table I-1 provides the load combinations listed in FSAR Subsection 3.8.6.3 and the four additional load combinations. These load combinations comprise the acceptance criteria for the diesel generator building and are hereinafter referred to as the Midland acceptance criteria.

By requiring combination of differential settlement with wind loads and OBE, the Midland acceptance criteria are more stringent than the requirements of American Concrete Institute (ACI) 318. ACI 318 only requires combining the effects of differential settlement with the dead loads and live loads. The Midland acceptance criteria are less stringent than ACI 349, because ACI 349 (as supplemented by Regulatory Guide 1.142) includes load combinations that combine the effects of differential settlement with extreme loads such as tornados and SSEs. In the response to Question 26 of NRC Requests Regarding Plant Fill, a commitment was made to do a separate structural reanalysis of the diesel generator building in accordance with ACI 349, as supplemented by Regulatory Guide 1.142, for comparative purposes only. Table I-2 provides the load combinations of ACI 349 as supplemented by Regulatory Guide 1.142.

It is unnecessary to use all Table I-1 load combinations in the structural reanalysis. A number of combinations can be eliminated from the analysis after comparison with more severe loads or load equations. For example, Equations 6 and 10 from Table I-1 are:

a. $U = 1.25\ (D + L + H\ \ + E) + 1.0T_o$ (6)

b. $U = 1.4\ (D + L + E) + 1.0T_o + 1.25H$ (10)

Because there are no significant forces on the structure due to thermal expansion of pipes (H_o), these two expressions can be rewritten in simpler forms:

a. $U = 1.25\ (D + L + E) + 1.0T_o$ (6)

b. $U = 1.4\ (D + L + E) + 1.0T_o$ (10)

The second expression is more critical than the first. Therefore, Equation 10 is used in the analysis and is considered to envelop the lower force components resulting from an analysis using Equation 6. Utilizing this approach with the entire set of load combinations eliminates the less critical equations and condenses the list to nine load combinations.

Load Combinations	Table I-1 Equation No.
a. $1.05D + 1.28L + 1.05T$	(1)
b. $1.4D + 1.4T$	(2)
c. $1.0D + 1.0L + 1.0W + 1.0T_o$	(3)
d. $1.0D + 1.0L + 1.0E + 1.0T_o$	(4)

e. $1.4D + 1.7L$ (5)

f. $1.25 (D + L + W) + 1.0T_o$ (7)

g. $1.4 (D + L + E) + 1.0T_o$ (10)

h. $1.0 (D + L + E') + 1.0T_o$ (15)

i. $1.0 (D + L + W') + 1.0T_o$ (18)

1.1.3 Allowable Material Limits

In accordance with regulatory requirements and the recommendations of the American Concrete Institute (ACI 318 and ACI 349), the maximum rebar tensile stress allowed in the diesel generator building rebar equals 0.90 f_y (where f_y equals yield stress) for computation of section capacities. Because the diesel generator building rebar has an f_y value of 60 ksi, the maximum allowable tensile rebar stress due to flexural and axial loads is 54.0 ksi. In similar fashion, the ultimate compressive strength of concrete is based on a strain of 0.003 in./in. Rebar stress values subsequently calculated for critical, reinforced concrete sections of the diesel generator building were based on this maximum allowable rebar stress value (54 ksi) and a maximum allowable concrete strain level of 0.003.

2.0 DIESEL GENERATOR BUILDING ANALYTICAL MODEL

The structural reanalysis of the diesel generator building uses a finite element model. The required load combinations were applied to this model and the resulting forces were investigated for compliance with the structural acceptance criteria. The diesel generator building was modeled as an assemblage of plate, beam, and boundary elements. The structure is defined by a set of 853 nodal points and 1,294 elements. Of these elements, 901 are plate elements representing walls and slabs, 141 are beam elements, and 252 are boundary elements (translational springs, in both the vertical and horizontal directions) representing varying soil pressures. Certain items, such as steel platforms and lightly reinforced interior secondary structural walls, have not been included in the model for the reasons listed in subsequent sections. Figure I-2 illustrates an isometric view of the finite element model.

2.1 APPLICATION OF LOADS TO THE BUILDING MODEL

The following loads have been applied to the model in the manner noted.

2.1.1 Dead Loads

The dead load of the structure was simulated by specifying a mass acceleration value equaling that of gravity (32.2 ft/s^2). Secondary structural walls and platforms were not included in the model because their contribution to the gross weight of the structure is minimal (less than an estimated 5 percent) relative to the sum of the other loads considered. Their exclusion does not significantly affect the magnitude or distribution of stresses. The louvers on both the north wall and south wall, along with the doors on the north and south walls of the building, were modeled simply as penetrations, with dimensions equivalent to those of the doors and louvers. This is acceptable because the doors and louvers contribute insignificantly to the building stiffness and total building weight. The diesel generator pedestals and the ground floor slabs were omitted from the finite element model because they were not constructed monolithically with the remainder of the structure. Consequently, they do not add stiffness to the structure.

2.1.2 Settlement Loads

The civil engineering group modeled settlement effects into the structure by representing varying soil conditions as boundary elements comprised of translational (vertical and horizontal) springs. At 84 locations along the building footing, a set of various spring values (one vertical spring and at least one horizontal spring) was applied to represent the nonhomogenous nature of soil conditions existing beneath the diesel generator building.

Spring values were developed for two general cases: those springs calculated for long-term loading and those springs calculated for short-term loading, e.g., tornados and earthquakes. For long-term loading, a set of springs was calculated for the determination of structural stresses caused by the settlement of the diesel generator building after 40 years. These springs were calculated at each nodal point along the foundation by dividing the total load represented at the selected point by the predicted settlement at that point, so that the spring constant was expressed in terms of force/unit displacement.

The estimated secondary compression settlement values from August 15, 1979, to December 31, 2025, are shown in Figure I-3 and are explained in Dr. Peck's testimony (Figure I-3 is the sum of settlement from August 15, 1979, to December 31, 1981, and from December 31, 1981, to December 31, 2025, as shown in Figures 27-12 and Figure 27-13, respectively, of the Responses to NRC Requests Regarding Plant Fill.) These estimates are based on the conservative assumption that the surcharge remains in place over the 40-year life of the plant, thus exceeding actual settlement predictions. Figure I-4 compares these settlement values with those settlement values resulting from the finite element analysis of the diesel generator building model. The comparison shows a close correlation between values resulting from the finite element model and estimated settlement values generated by Dr. Peck and Bechtel soil engineers. Because the estimates of the soils engineers are based on the conservative assumption of the surcharge remaining in place over the 40-year life of the plant, the model overestimates the settlement loads on the structure considered in the structural reanalysis and is therefore conservative.

Figure I-4 also indicates the settlement and differential settlement occurring in the building subsequent to August 1979 (when the surcharge material was removed). As Figure I-4 shows, the settlement and differential settlement which has occurred since the removal of the surcharge are very small compared with the settlement and differential settlement conservatively estimated for the purpose of the structural reanalysis.

The other set of springs was developed for short-term loading, in which it was assumed that the structural movement was small enough to assume the soil was linearly elastic. The modulus of elasticity was estimated using the results of laboratory and field investigations. Springs were developed for the vertical and horizontal modes. These springs were calculated by determining the amount of force required to produce a unit displacement in the direction indicated by the particular mode. The footings of the diesel generator building were assumed to be resting on a large mass of elastic soil for the vertical mode and embedded within the mass of soil for the horizontal mode.

The settlement due to seismic shakedown was also identified as a possible occurrence during a seismic event. The maximum differential settlement due to seismic shakedown, as stated in Question 15 of the NRC Requests Regarding Plant Fill is approximately one-half inch. The effects of seismic shakedown settlement will act to reduce the effects of differential settlement and for this reason was not the governing case in the structural reanalysis of the diesel generator building.

2.1.3 Live Loads

Live loads were applied to the modeled structure by applying pressure loads on the plate elements which represent the floor slab at el 664'-0" and the roof at el 680'-0". During the plant life, a maximum live load of 100 psf is predicted to occur on the roof slab, whereas for the floor at el 664'-0", a maximum live load of 250 psf is postulated. One hundred percent of the live load was used in the design of individual structural members, such as floor slab at el 664'-0" and roof slab at el 680'-0". For overall building response, however, the live loads considered were limited to 25 percent of the above maximum loads. This 25 percent value represents the live load expected to be present when the plant is in operation, i.e., 100 percent of the live load will not act simultaneously on every square foot of the floor space.

2.1.4 Wind Loads

Loads resulting from the design wind (100-year recurrence with a velocity of 85 mph) were applied to the modeled structure as a pressure load on the plate elements that represent the exposed walls. Wind loads on the roof and south wall hatch covers were determined assuming the hatch covers were in place. These loads were then distributed to the nodal points which define the perimeter of the respective hatches.

2.1.5 Tornado Loads

As specified in BC-TOP-3-A (Reference 1), various combinations of velocity wind pressure, differential pressure, and local pressures were applied to the modeled structure. The maximum wind velocity of the tornado was 360 mph.

The original structural analysis performed in accordance with the FSAR considered various tornado-generated missiles. The analysis considered missiles equivalent to a 4" by 12" by 12' wooden plank (108 pounds) traveling end-on at 300 mph at any height; a 4,000 pound automobile with a velocity of 72 mph no higher than 30 feet above the ground with a contact area of 20 square feet; a 1-inch diameter, 3-foot long, 8-pound steel bar traveling at 216 mph at any height in any direction, and a 35-foot long utility pole, 13-1/2 inches in diameter, weighing 1,490 pounds, traveling at 144 mph, and striking the structure not more than 30 feet above the ground. For tornado-generated missile loads, the structure was allowed to locally exceed the yield strain.

The results of the original tornado-generated missile load analysis showed the diesel generator building was acceptable. Results of missile impact tests conducted over the last 6 years indicate that reinforced concrete walls, thinner than the exterior walls of the diesel generator building, have a considerable margin against local damage. The tests indicate that a wall thickness of 12 inches would sufficiently preclude unacceptable local damage (spalling) from these missiles. (The thinnest exterior wall of the diesel generator building is 30 inches thick.)

2.1.6 Seismic Loads

The seismic response of a structure depends on the stiffness properties and mass of the structure, the input seismic motion at the structure location, and the soil properties of the foundation medium. Of these parameters, only soil properties are affected by insufficient compaction of backfill. The following paragraphs describe how the effects of insufficient compaction and eventual surcharging were accounted for in the revised diesel generator building seismic analysis.

The analytical models used for the original seismic analysis and for the seismic reanalyses described in this report are one-dimensional, stick-type, lumped mass models using beam elements to represent the structural stiffness and impedence functions of the foundation medium (see Figure I-1). The models were analyzed by the modal superposition method, a conventional method used in dynamic analysis. Design responses are calculated by modal superposition in conjunction with the site ground spectra. The site ground spectra are those associated with ground acceleration set forth in FSAR Section 3.7 and approved by the NRC at the construction permit stage. The floor response spectra in the building are calculated by the modal superposition method using the design time history. The design time history is a modification of the N21E (north, 21° east) ground motion component recorded during the July 21, 1952, earthquake at Taft, California.

The effect of soil-structural interaction is accounted for by coupling the structural model with the foundation media. The foundation media are represented by impedance functions which represent the equivalent spring stiffness and radiation damping coefficients as specified in BC-TOP-4-A (Reference 2).

The structural portion of the lumped mass model was not revised in the new dynamic analysis. The difference in the new model was confined to the treatment of the soil-structural interface. The revised analysis developed the impedance functions based on the building's foundation dimensions and the modification in the soil properties described below. In addition, the weight of the soil and the concrete pedestals and diesel generator pedestals within the building were included in this revised model.

The original (presettlement) diesel generator building seismic analysis was based on the underlying till material, which has a shear wave velocity value of 1,359 ft/s (see Table I-2). This value was not adjusted for the 30 feet of plant fill between the till and building foundation elevation. The seismic reanalysis accounted for the soil properties of the fill by averaging the low strain shear wave velocity of the fill and underlying till (Figure I-5) over a depth of 75 feet, which is the smallest dimension of the building. This resulted in the value of 796 ft/s, which was used in the seismic reanalysis. However, the effect of decreasing shear wave velocity to a lower bound estimate of 500 ft/s was also analyzed. Both the low strain shear wave velocity value of 796 ft/s and the lower bound shear wave velocity value of 500 ft/s were supplied by soil consultants.

The floor spectra at all elevations of the diesel generator building were generated using a shear wave velocity value of 796 ft/s. The resulting floor response spectra were combined in an enveloping fashion with the spectra developed in the original analysis which used a shear wave velocity value of 1,359 ft/s. The floor response spectra were further broadened to account for a lower bound shear wave velocity of 500 ft/s. Thus, conservative floor response spectra were generated.

The results of the seismic reanalysis indicated that the seismic forces at all elevations of the diesel generator building were somewhat higher than the forces determined in the original analysis. This increased seismic load was conservatively simulated by applying the maximum structural acceleration occurring in the dynamic model to the finite element model in north-south, east-west, and vertical directions. The combined effect of the three directional responses was assessed using the square-root-of-the-sum-of-the-squares method recommended in NRC Regulatory Guide 1.92.

The ability of the structure to withstand these increased seismic forces in combination with the other loads is described in Section 3.0.

2.1.7 Thermal Loads

Thermal effects were included in the analysis as a linear variation of temperature across the thickness of an element. The thermal effect due to linear variation of temperature across the thickness of an element (also called gradient) results in bending moments being applied to the element.

In general, the temperature gradient which is of most concern for the diesel generator building is that anticipated to occur in the winter. In accordance with the Handbook of Concrete Engineering (Reference 3) and FSAR meteorological data, the equivalent steady-state exterior winter temperature of 14.6F was calculated. The corresponding maximum interior ambient air temperature was 75F. For information on how thermal effects were applied to the model, see Section 3.0.

3.0 STRUCTURAL ADEQUACY COMPUTATIONS

The computations necessary to verify structural adequacy were performed using a computer analysis program (OPTCON) capable of analyzing reinforced concrete sections. This reinforced concrete analysis program models a portion of the diesel generator building and analyzes it for forces that resulted from the BSAP finite element model analysis. Refer to Appendix A for additional information concerning OPTCON.

To determine the structural adequacy of the diesel generator building, the modeled structure was partitioned into structural categories (i.e., north wall, center wall, roof, etc). Critical elements from each category were then selected for further investigation based on their axial force, moment, and in-plane shear force. Using OPTCON, rebar stress values were then calculated in these critical elements to verify that the allowable rebar tensile stress value was not exceeded. To facilitate the calculation process, a computer program was specifically written for selecting critical elements that would undergo OPTCON investigation. This program was written so that its selection of critical elements was based on a comparison of the axial force, bending moment, and in-plane shear force of each separate element within a structural category with all other elements of the same structural category.

Once these critical elements were selected, a thermal gradient was assigned to each element based on the location of that element within the building. The gradient is assigned on a temperature basis, and is converted by OPTCON into a thermal moment.

Based upon the procedure discussed above, all structural categories of the diesel generator building were investigated and found to meet the structural acceptance criteria. Table I-4 shows the results of the analysis. The left-hand column of Table I-4 shows the element with the highest rebar stress value for each structural category. The second column shows the load combination which produces the highest stress. In other words, this is the load combination which is critical for this category. The next three columns show the axial, flexural, and in-plane shear force calculated by BSAP for this element's critical load combination. The sixth column presents the rebar stress value computed by OPTCON for each critical element within each structural category. The highest rebar stress value (reflecting the combined effects of flexural, axial, and in-plane shear loads) exists in the south wall where the rebar stress value is 42.5 ksi. The last three columns compare maximum separate force component allowables in all structural categories (axial, flexural, and in-plane) against the corresponding critical loads generated by BSAP. This comparison of separate force components is provided for information only. The interactive method used by OPTCON to calculate actual rebar stress values more accurately depicts how close an element is to the maximum allowable stress value of 54 ksi as it considers the combined effect of flexural, axial, and in-plane loads.

The final structural reanalysis of the diesel generator building showed that the critical load combinations (Table I-1) are those which include either the tornado load case (W') or the SSE load case (E'), specifically:

a. $1.0D + 1.0L + 1.0W' + 1.0T_o$ (18)

b. $1.0D + 1.0L + 1.0E' + 1.0T$ (10)

In approximately 70 percent of the diesel generator building, the tornado load combinations produce the stress levels.

4.0 CONCLUSIONS

The diesel generator building is a massive, reinforced concrete structure with extensive reserve strength. The structural reanalysis performed on the diesel generator building verifies that the integrity of the structure will not be violated even under the most critical load combinations. Based on the analysis performed, it can be stated that the settlement has had minimal effect on the structure, and there is reasonable assurance that the diesel generator building will safely perform its intended function over the operating life of the Midland plant.

REFERENCES

1. Bechtel Power Corporation, Tornado and Extreme Wind Design Criteria for Nuclear Power Plants, Revision 3, August 1974 (BC-TOP-3-A)

2. Bechtel Power Corporation, Seismic Analyses of Structures and Equipment for Nuclear Power Plants, Revision 3, November 1974 (BC-TOP-4-A)

3. M. Fintel, Handbook of Concrete Engineering, Van Nostrand Reinhold Company, September 1974

TABLE I-1

LOADS AND LOAD COMBINATIONS FOR CONCRETE
STRUCTURES OTHER THAN THE CONTAINMENT BUILDING
FROM THE FSAR AND QUESTION 15 OF RESPONSES TO
NRC REQUESTS REGARDING PLANT FILL

Responses to NRC Requests Regarding Plant Fill, Question 15

a. Service Load Condition

$U = 1.05D + 1.28L + 1.05T$ (1)

$U = 1.4D + 1.4T$ (2)

b. Severe Environmental Condition

$U = 1.0D + 1.0L + 1.0W + 1.0T$ (3)

$U = 1.0D + 1.0L + 1.0E + 1.0T$ (4)

FSAR Subsection 3.8.6.3

a. Normal Load Condition

$U = 1.4D + 1.7L$ (5)

b. Severe Environmental Condition

$U = 1.25\ (D + L + H_0 + E) + 1.0T_0$ (6)

$U = 1.25\ (D + L + H_0 + W) + 1.0T_0$ (7)

$U = 0.9D + 1.25\ (H_0 + E) + 1.0T_0$ (8)

$U = 0.9D + 1.25\ (H_0 + W) + 1.0T_0$ (9)

c. Shear Walls and Moment Resisting Frames

$U = 1.4\ (D + L + E) + 1.0T_0 + 1.25H_0$ (10)

$U = 0.9D + 1.25E + 1.0T_0 + 1.25H_0$ (11)

d. Structural elements carrying mainly earthquake forces, such as equipment supports

$U = 1.0D + 1.0L + 1.8E + 1.0T_0 + 1.25H_0$ (12)

Table I-1 (continued)

e. Extreme Environmental and Accident Conditions

$$U = 1.05D + 1.05L + 1.25E + 1.0T_A + 1.0H_A + 1.0R \quad (13)$$

$$U = 0.95D + 1.25E + 1.0T_A + 1.0H_A + 1.0R \quad (14)$$

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_0 + 1.25H_0 + 1.0R \quad (15)$$

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_A + 1.0H_A + 1.0R \quad (16)$$

$$U = 1.0D + 1.0L + 1.0B + 1.0T_0 + 1.25H_0 \quad (17)$$

$$U = 1.0D + 1.0L + 1.0T_0 + 1.25H_0 + 1.0W' \quad (18)$$

where

B = hydrostatic forces due to the postulated maximum flood

D = dead loads of structures and equipment and other permanent load contributing stress

E = operating basis earthquake (OBE)

E' = safe shutdown earthquake load (SSE)

H_0 = force on structure caused by thermal expansion of pipes under operating conditions.

H_A = force on structure caused by thermal expansion of pipes under accident conditions

L = conventional floor and roof live loads (includes moveable equipment loads or other loads which very in intensity)

R = local force, pressure on structure, or penetration caused by rupture of pipe

T = effects of differential settlement, creep, shrinkage, and temperature

T_0 = thermal effects during normal operating conditions, including linear expansion of equipment and temperature gradients

T_A = total thermal effects which may occur during a design accident

U = required strength to resist design loads or their related internal moments and forces

Table I-1 (continued)

W = design wind load

W' = tornado wind loads, excluding missile effects, if applicable (refer to Subsection 2.2.3.5)

TABLE I-2

LOADS AND LOAD COMBINATIONS FOR
COMPARISON ANALYSIS REQUESTED IN
QUESTION 26 OF NRC REQUESTS
REGARDING PLANT FILL

ACI 349 as Supplemented by Regulatory Guide 1.142

a. Normal Load Condition:

$U = 1.4\ (D + T) + 1.7L + 1.7R_o$

$U = 0.75\ [1.4\ (D + T) + 1.7L + 1.7T_o + 1.7R_o]$

b. Severe Environmental Condition:

$U = 1.4\ (D + T) + 1.4F + 1.7L + 1.7H + 1.9E_o + 1.7R_o$

$U = 1.4\ (D + T) + 1.4F + 1.7L + 1.7H + 1.7W + 1.7R_o$

$U = 0.75\ [1.4\ (D + T) + 1.4F + 1.7L + 1.7H + 1.9E_o + 1.7T_o + 1.7R_o]$

$U = 0.75\ [1.4\ (D + T) + 1.4F + 1.7L + 1.7H + 1.7W + 1.7T_o + 1.7R_o]$

c. Extreme Environmental Conditions:

$U = (D + T) + F + L + H + T_o + R_o + W_t$

$U = (D + T) + F + L + H + T_o + R_o + E_{ss}$

d. Abnormal Load Conditions:

$U = (D + T) + F + L + H + T_a + R_a + 1.5P_a$

$U = (D + T) + F + L + H + T_a + R_a + 1.25P_a + 1.0(Y_r + Y_j + Y_m) + 1.25E_o$

$U = (D + T) + F + L + H + T_a + R_a + 1.0P_a + 1.0(Y_r + Y_j + Y_m) + 1.0E_{ss}$

Table I-2 (Continued)

where

Normal loads are those loads encountered during normal plant operation and shutdown, and include:

T = settlement loads

D = dead loads or their related internal moments and forces

L = applicable live loads or their related internal moments and forces

F = lateral and vertical pressure of liquids or their related internal moments and forces

H = lateral earth pressure or its related internal moments and forces

T_0 = thermal effects and loads during normal operating or shutdown conditions, based on the most critical transient or steady-state condition

R_0 = maximum pipe and equipment reactions if not included in the above loads

Severe environmental loads are those loads that could infrequently be encountered during the plant life and include:

E_0 = loads generated by the operating basis earthquake (BOE)

W = loads generated by the operating basis wind (OBW) specified for the plant

Extreme environmental loads are those loads which are credible but highly improbable, and include:

E_{ss} = loads generated by the safe shutdown earthquake (SSE)

W_t = loads generated by the design tornado specified for the plant

Abnormal loads are those loads generated by a postulated high-energy pipe break accident and include:

P_a = maximum differential pressure load generated by a postulated break

T_a = thermal loads under accident conditions generated by a postulated break and including T_0

Table I-2 (Continued)

R_a = pipe and equipment reactions under accident conditions generated by a postulated break and including R_0

U = required strength to resist design loads or their related internal moments and forces

Y_r = loads on the structure generated by the reaction on the broken high-energy pipe during a postulated break

Y_j = jet impingement load on a structure generated by a postulated break

Y_m = missile impact load on a structure generated by or during a postulated break, such as pipe whipping

TABLE I-3

SOIL PROPERTIES USED IN

THE SEISMIC ANALYSIS

	Original Analysis	First Revised[1] Analysis	Second Revised[1] Analysis
Modulus of Elasticity (E)	22,000 ksf	6,598 ksf	2,609 ksf
Poisson's Ratio	0.42	0.45	0.40
Unit Weight (w)	135 pcf	116 pcf	120 pc/s
Shear Wave Velocity (V_S)	1,359 ft/s	796 ft/s	500 ft/s
Shear Modulus	7,746 ksf	2,275 ksf	971 ksf

[1] Note different shear wave velocity values.

TABLE I-4

REBAR STRESS VALUES FOR THE DIESEL GENERATOR BUILDING STRUCTURAL MEMBERS (ACCORDING TO FSAR AND THE RESPONSES TO NRC REQUESTS REGARDING PLANT FILL, QUESTION 15)

Description of Members/Location	Load[1] Combination	Maximum Calculated Loads (BSAP) Axial Tension (k/ft)	Maximum Calculated Loads (BSAP) Flexural (k-ft/ft)	Maximum Calculated Loads (BSAP) In-Plane Shear[2] (k/ft)	Rebar Stress Value (ksi) (Allowable = 54 ksi)	Maximum Allowable Loads Axial Tension (k/ft)	Maximum Allowable Loads Flexural (k-ft/ft)	Maximum Allowable Loads In-Plane Shear[3] (k/ft)	Gradient (°F)	Concrete[4] Stress (ksi) (Allowable = 3.400 ksi)
Exterior - West										
2'-6" thick wall horizontal reinforcement, plate element 44	Tornado	9.73	27.17	5.36	22.17	85.3	95.7	34.2	0	0.354
Exterior - South										
2'-6" thick wall horizontal reinforcement, plate element 287	Seismic	27.30	1.33	67.58	42.46	85.3	95.7	34.2	60.4	0.000[5]
Elevation - 664'-0"										
2'-0" floor slab E-W reinforcement, plate element 167	Tornado	17.70	13.67	5.34	39.15	47.5	44.7	26.7	24	0.068
Elevation - 680'-0"										
1'-9" floor slab N-S reinforcement, plate element 788	Tornado	3.5[illegible]	26.62	1.77	36.06	85.3	63.7	22.7	24	0.834
South										
2'-0" missile shield wall south, horizontal reinforcement, plate element 631	Seismic	15.81	12.50	14.34	32.84	64.8	55.5	26.7	60.4	0.[illegible]72
Interior										
2'-0" interior missile shield wall, vertical reinforcement, plate element 824	Tornado	18.99	3.78	1.35	28.06	47.5	55.5	26.7	24	0.000[5]

Table I-4 (continued)

Description of Members/Location	Load(1) Combination	Maximum Calculated Loads (BSAP) Axial Tension (k/ft)	Flexural (k-ft/ft)	In-Plane Shear(2) (k/ft)	Rebar Stress Value (ksi) (Allowable = 54 ksi)	Maximum Allowable Loads Axial Tension (k/ft)	Flexural (k-ft/ft)	In-Plane Shear(3) (k/ft)	Gradient (°F)	Concrete(4) Stress (ksi) (Allowable = 3.400 ksi)
North										
2'-0" missile shield wall north, horizontal reinforcement, plate element 839	Tornado	23.46	9.14	7.76	13.85	137.2	114.7	26.7	0	0.000(5)
Exterior - North										
2'-6" thick wall horizontal reinforcement, plate element 767	Tornado	11.17	20.27	5.19	21.90	85.3	95.7	34.2	24.0	0.313
Exterior - East										
2'-6" thick wall horizontal reinforcement, plate element 896	Tornado	9.15	25.44	8.11	23.64	85.3	95.7	34.2	24.0	0.403
Interior										
1'-6" thick wall horizontal reinforcement, plate element 683	Tornado	17.0	0.65	0.31	16.66	64.8	41.2	20.6	24.0	0.000(5)
South										
2'-0" thick box missile shield/south, horizontal reinforcement, plate element 732	Tornado	4.93	1.62	1.45	8.02	48.6	39.2	26.7	0	0.00(5)

Table I-4 (continued)

Description of Members/Location	Load[1] Combination	Maximum Calculated Loads (BSAP): Axial Tension (k/ft)	Maximum Calculated Loads (BSAP): Flexural (k-ft/ft)	Maximum Calculated Loads (BSAP): In-Plane Shear[2] (k/ft)	Rebar Stress Value (ksi) (Allowable = 54 ksi)	Maximum Allowable Loads: Axial Tension (k/ft)	Maximum Allowable Loads: Flexural (k-ft/ft)	Maximum Allowable Loads: In-Plane Shear[3] (k/ft)	Gradient (°F)	Concrete[4] Stress (ksi) (Allowable = 3.400 ksi)
Footing										
2'-6" thick footing, beam element 87	Seismic	-	75.0	-	46.04[6]	N/A	92.14	34.20	-	-

NOTES:

[1] The tornado load combination is $1.0\ (D + L) + 1.0\ (W_T) + 1.0\ (T_0)$.
The seismic load combination is $1.0\ (D + T) + 1.0\ (E') + 1.0\ (T_0)$.

[2] Out-of-plane shear loads were investigated independently from axial, flexural, and in-plane loads. This investigation indicated that the maximum allowable out-of-plane shear force was never exceeded.

[3] Shear capacity of concrete only with no tension load on the section

[4] Stresses are in compressive sense. Concrete stresses shown are associated with maximum rebar tensile stresses shown in this table.

[5] Section is cracked.

[6] This value is conservatively high and will be reduced.

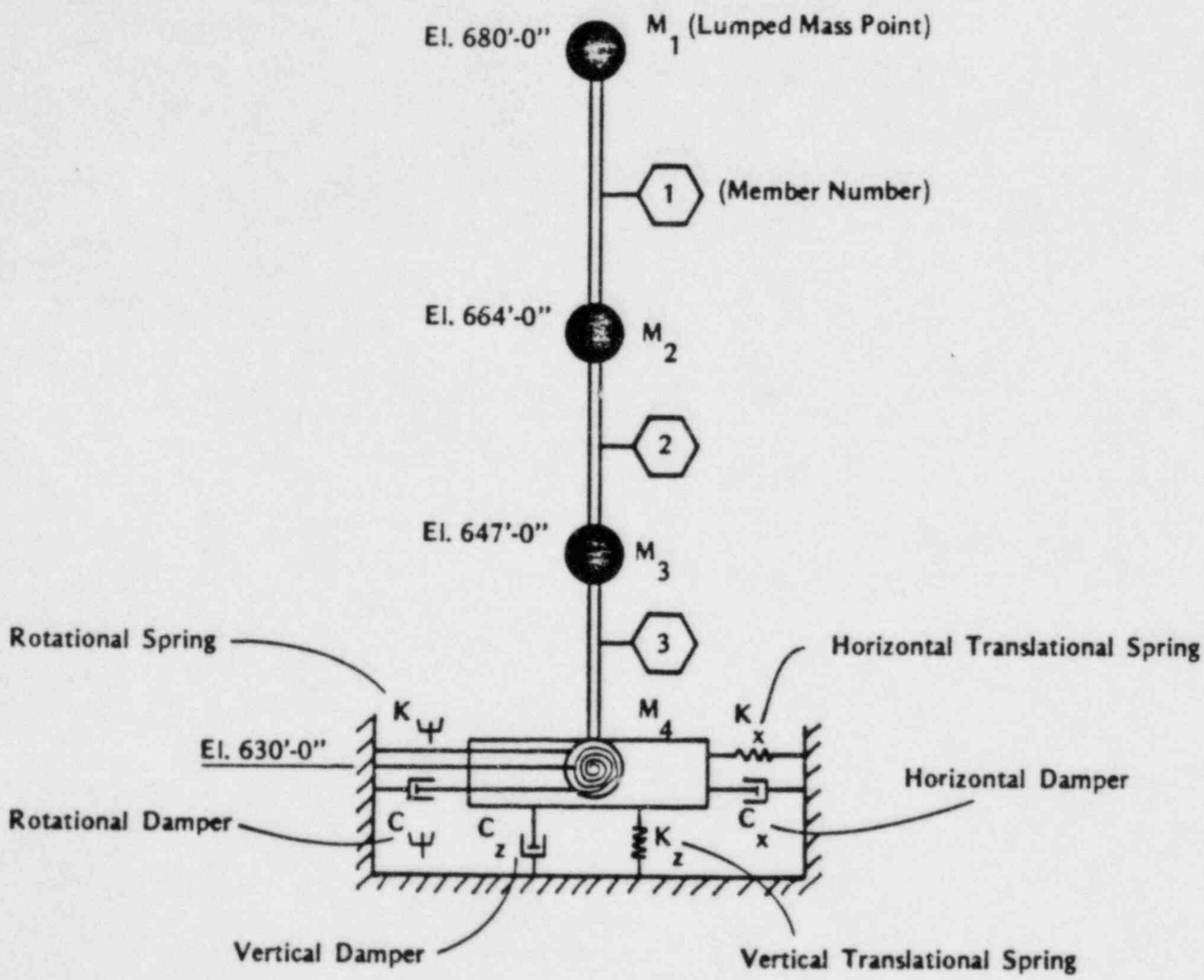


FIGURE I-1
DIESEL GENERATOR BUILDING DYNAMIC LUMPED MASS MODEL
FOR SEISMIC ANALYSIS

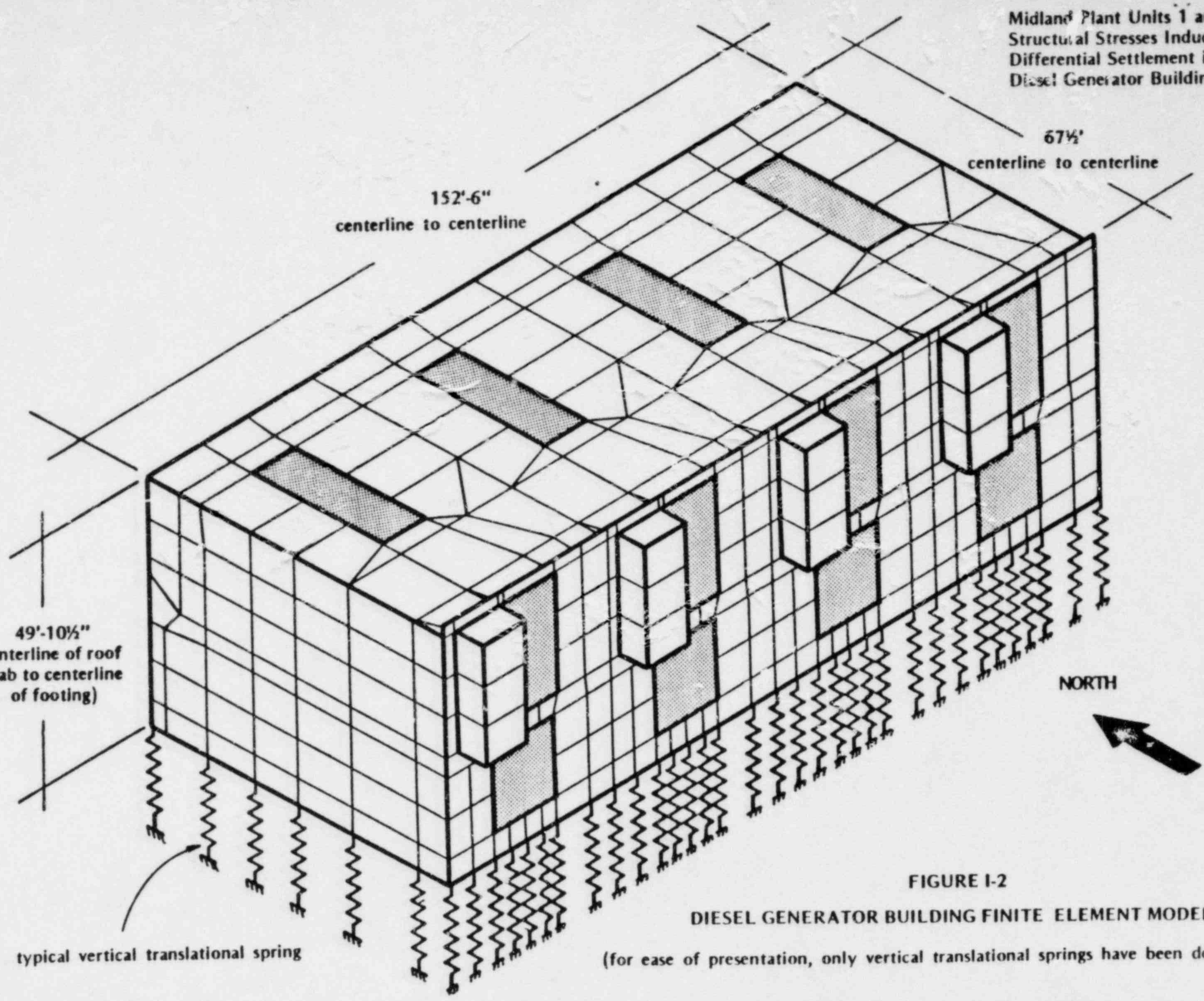
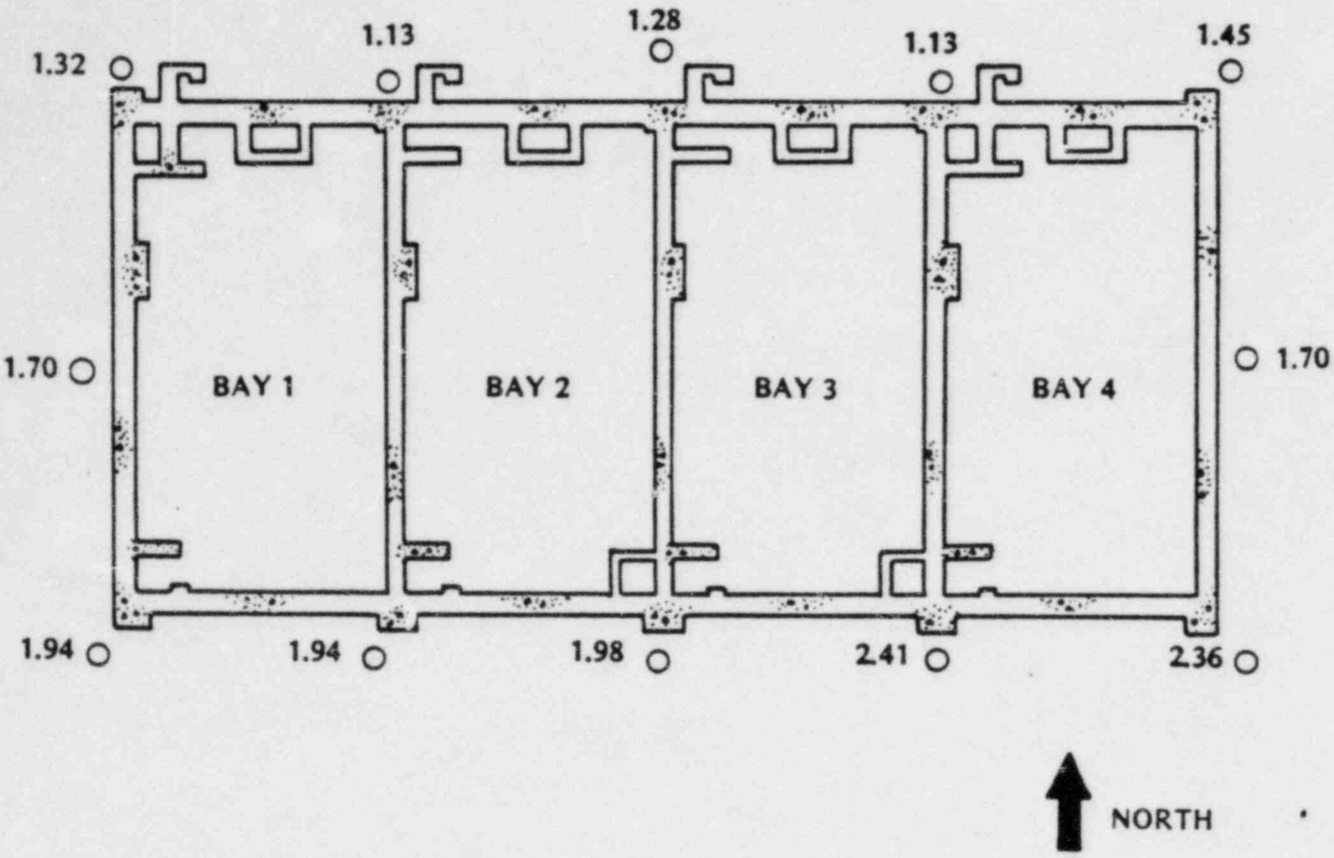


FIGURE I-2

DIESEL GENERATOR BUILDING FINITE ELEMENT MODEL

(for ease of presentation, only vertical translational springs have been depicted)

DIESEL GENERATOR BUILDING



LEGEND

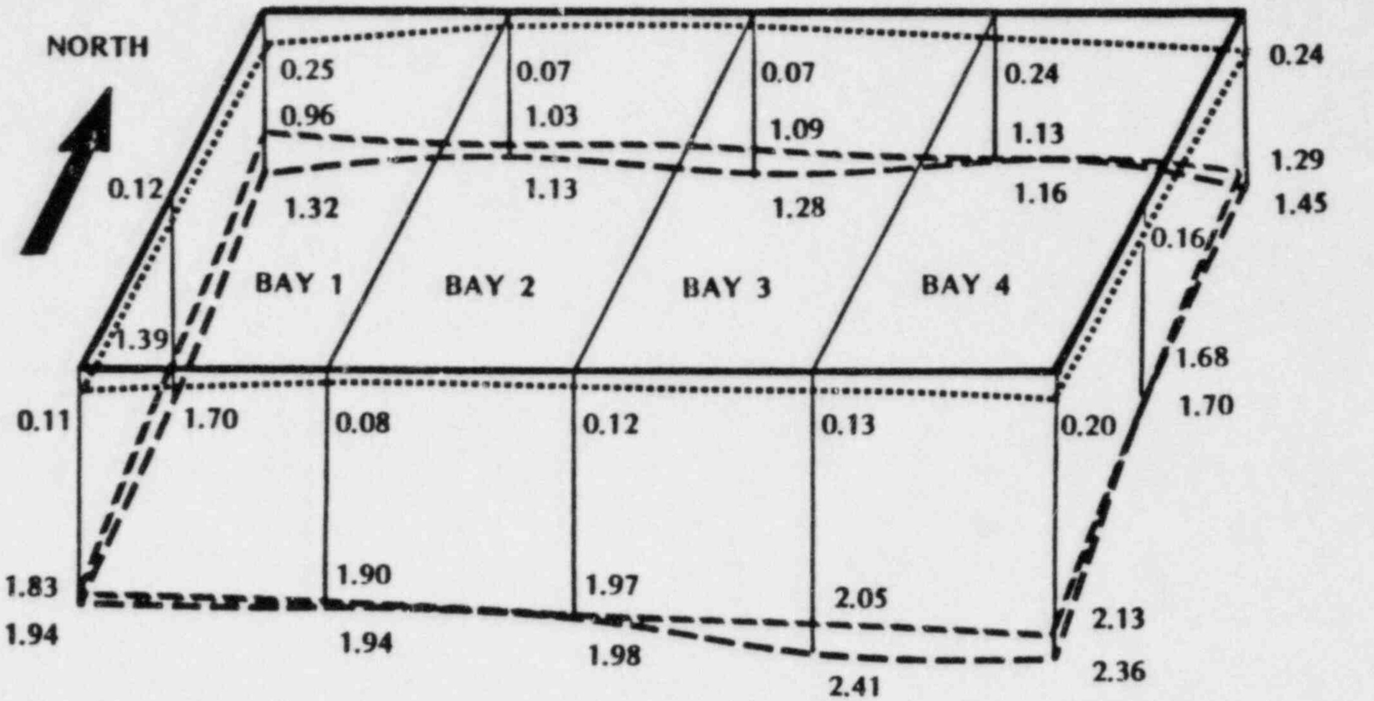
○ —— BUILDING SETTLEMENT MARKER

2.36 —— SETTLEMENT IN INCHES

(THIS DRAWING AND THE INFORMATION CONTAINED ON IT WERE OBTAINED FROM FIGURES 27-12 AND 27-13 OF THE RESPONSES TO NRC QUESTIONS REGARDING PLANT FILL)

FIGURE I-3

ESTIMATED SECONDARY COMPRESSION SETTLEMENTS
FROM 8-15-79 to 12-31-2025
ASSUMING SURCHARGE REMAINS



SURFACE A ———————— REFERENCE (AS OF AUGUST 15, 1979)

SURFACE B .... ACTUAL SETTLEMENT VALUES FROM SEPTEMBER 14, 1979 TO JULY 9, 1981

SURFACE C – – – – – – SECONDARY COMPRESSION SETTLEMENT VALUES CALCULATED BY FINITE ELEMENT ANALYSIS

SURFACE D — — — — — ESTIMATED SECONDARY COMPRESSION SETTLEMENT VALUES FROM AUGUST 15, 1979 TO DECEMBER 31, 2025 ASSUMING SURCHARGE REMAINS IN PLACE.

FIGURE I-4

COMPARISON OF ESTIMATED SECONDARY COMPRESSION SETTLEMENT VALUES WITH SETTLEMENT VALUES RESULTING FROM A FINITE ELEMENT ANALYSIS OF THE DIESEL GENERATOR BUILDING

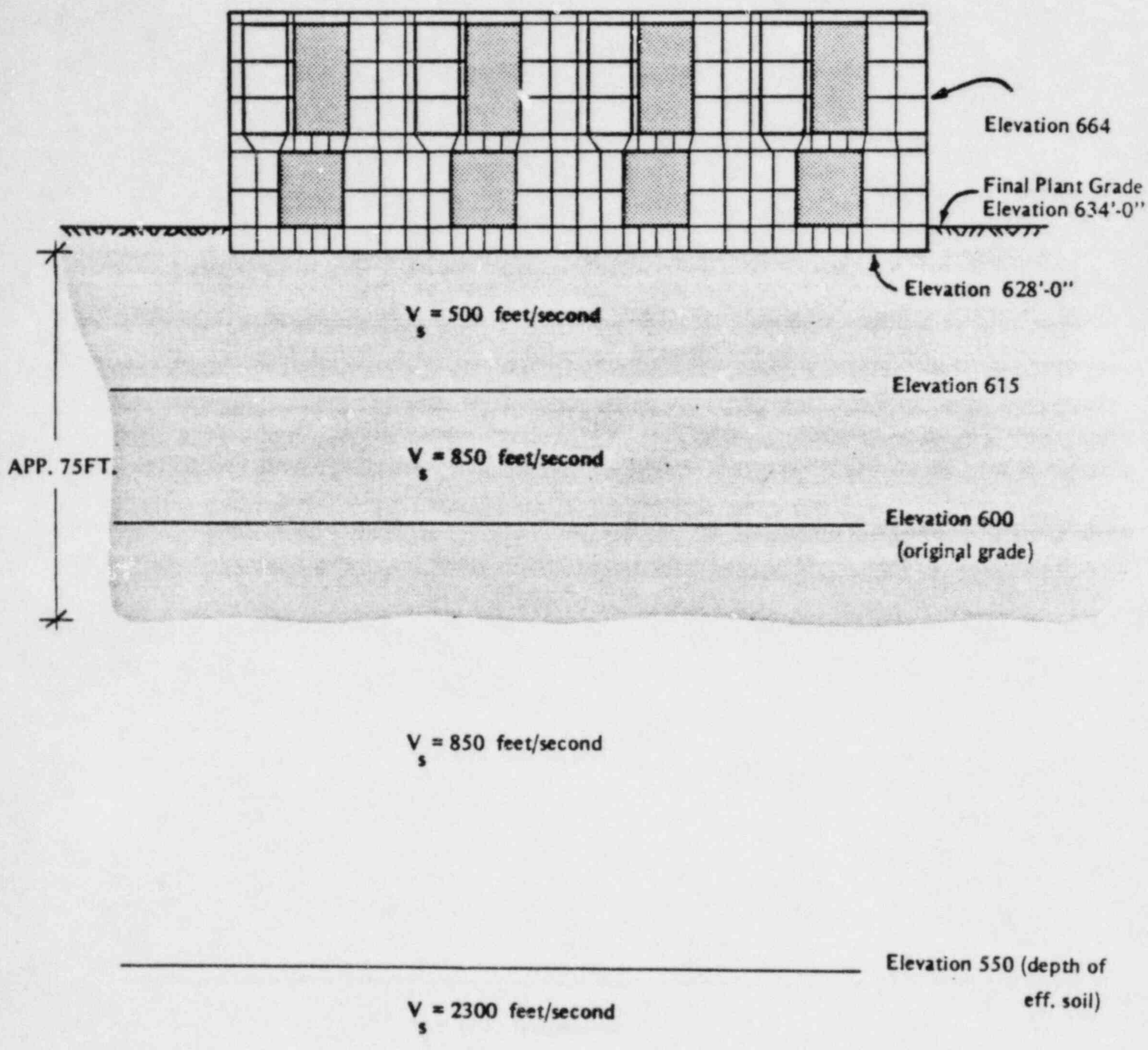


FIGURE I-5

BASIS FOR CALCULATION OF

EQUIVALENT SHEAR WAVE VELOCITY VALUES (V_s)

(Shaded region represents the area over which low-strain shear wave velocity values (V_s) were averaged, resulting in a V_s value of 796 ft/sec.)

APPENDIX A

OPTCON

The OPTCON computer code is a versatile and complete design and analysis program for reinforced concrete structures. The program may be used for the investigation of an existing reinforced concrete section where the reinforcing steel area is predetermined. Alternatively, it can be used for obtaining an optimum design by allowing the program to determine the minimum reinforcement required.

The computer program operates on the axial force/moment interaction diagram (IAD) of a section, where an IAD is a plot of the maximum allowable resistance of a section for given stress and strain limitations. Combinations of moment (M) and axial load (P) falling within this area are acceptable. Figure IA-1 depicts the appropriate IAD for a symmetrically reinforced, symmetrically shaped section subjected to a combination of flexural and axial loads.

The OPTCON program handles loads consisting of axial forces and corresponding bending moments due to different types of loads. Special subroutines are provided to incorporate the thermal effects into the design and/or investigation. The cracking effect of the concrete and the yielding effects of the reinforcement (as allowed by the appropriate stress/strain yielding criteria) are considered in the calculation of the thermal loads and moments computed by the program.

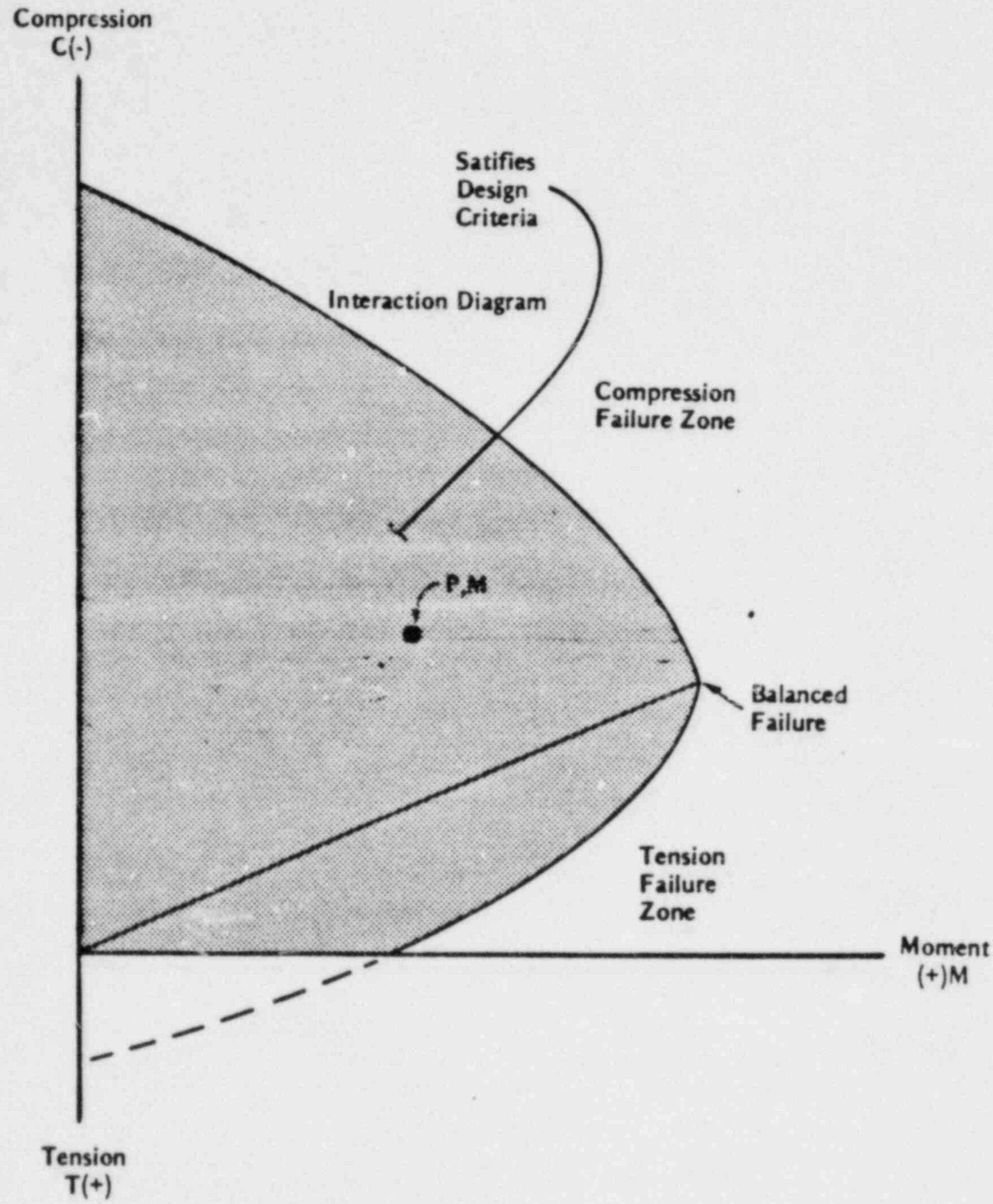


Figure IA-1

TYPICAL INTERACTION DIAGRAM
(for single axis bending on a section
with symmetrical reinforcement)

ENCLOSURE 2

SUBGRADE MODULUS AND SPRING CONSTANT VALUES

FOR DIESEL GENERATOR BUILDING STRUCTURAL ANALYSIS

Issue:

Basis and actual numerical values of adopted spring constants - both static and dynamic (reconcile with settlement predictions and observed behavior)

Response:

Introduction

This describes the methods used to develop the subgrade modulus and spring constant values intended for use in the structural and seismic analysis of the diesel generator building. These values are indicated in Appendix A.

Springs for Dynamic (Short-Term Static) Condition

Springs were developed for movements which were assumed small enough to cause a negligible decrease in the low-strain shear modulus which was determined from field geophysical testing. Vertical translation, horizontal translation, and rocking mode springs were developed. The shear wave velocity profile assumed for use in the spring development consisted of a shear wave velocity of 500 fps from elevation 634 to 615 feet and 850 fps from elevation 615 to 550 feet. The diesel generator building foundation elevation is 628 feet. In all cases, it was assumed that the structure foundation and walls were rigid. The spring constant values were developed for the best estimate of the existing soil shear wave velocity. However, as discussed later, a parametric study was performed and the effect of varying shear wave velocity profiles on the spring constants was determined.

The spring constant for the vertical translation mode is 9.6×10^5 k/ft (subgrade modulus, 163 kcf). This value was developed using the following formula developed by Timoshenko and Goodier which is described in Reference 2, Pages 347 and 350:

$$K_z = \frac{4Gr_o}{1 - \mu} \qquad r_o = \sqrt{\frac{4cd}{\pi}}$$

This formula represents the ratio of force to deflection for a rigid circular plate resting on an elastic half-space with shear modulus, G. The shear modulus used was a weighted average of the shear moduli profile for a depth below the foundation equal to the width (78 feet) of the building. The subgrade modulus indicated, 163 kcf, was determined by dividing the spring constant by the area of the diesel generator building wall footings.

The spring constant for the horizontal translation mode was developed as separate components considering the effects of:

1) The horizontal shear force of the base of the building and soil within the building perimeter on the elastic half-space

2) The horizontal shear force of the buried portion of the exterior side walls on the elastic half-space

3) The horizontal force of the walls perpendicular to the translation direction on the half-space. The base shear component, 3.1×10^5k/f, was developed using the following formula developed by Bycroft which is described in Richart, Hall, and Woods (1970), Pages 347 and 350:

$$K_x = \frac{32\,(1-\mu)\,Gr_o}{7-8\mu} \qquad r_o = \sqrt{\frac{4cd}{\pi}}$$

This formula represents the ratio of horizontal force to horizontal deflection for a rigid circular plate resting on an elastic half-space with shear modulus, G. The shear modulus used was based on the shear wave velocity of 500 fps.

The side shear components, 1.48×10^5 and 2.97×10^5 k/f, were developed using the following formula developed by Groth and Chapman which is described in Reference 1, Pages 99 through 102:

$$\rho = \frac{q\,a\,I}{E}$$

This formula represents the horizontal deflection of the top or bottom corner (depending on the value of I) of a flexible rectangle buried in an elastic half-space with Young's modulus, E. The spring constant was determined by rearranging terms and multiplying by the area of the rectangle (wall). Because the diesel generator building walls are assumed rigid, the spring constants developed for the top and bottom corners were averaged. The Young's modulus used was based on a shear wave velocity of 500 fps.

The components represented by the wall pushing against the soil, 3.[illegible] x 10 and 2.0 x 10 k/f were developed using the following formulas developed by Douglas and Davis which are described in Reference 1, Pages 97 and 98:

(Upper Corners) $\rho_x = \frac{pb}{32\pi G(1-\mu)} \; (3-4\mu)F_1 + F_4 + 4(1-2\mu)(1-\mu)F_5$

(Lower Corners) $\rho_x = \frac{pb}{32\pi G(1-\mu)} \; (3-4\mu)F_1 + F_2 + 4(1-2\mu)(1-\mu)F_3$

These formulas represent the horizontal deflection of the upper and lower corners of a flexible rectangle buried in an elastic half-space with shear modulus, G. The spring constant was determined by rearranging terms and multiplying by the area of the rectangle (wall). Because the diesel generator building walls are assumed rigid, the spring constants developed for the top and bottom corners were averaged. The Young's modulus used was based on a shear wave velocity of 500 fps.

The spring constants for the rocking mode are 1.85×10^9 and 4.65×10^9 k-f/rad. These values were developed using the following formula developed by Gorbunov-Possadov which is described in Reference 2, Page 350:

$$K_\psi = \frac{G}{1-\mu} \beta_\psi \; c d^2$$

This formula represents the ratio of moment to angular rotation for a rigid rectangular plate resting on an elastic half-space with shear modulus, G. The shear modulus used was the same weighted average value that was used for the vertical translation mode.

Finally, it was necessary to determine the variation in the vertical translation and rocking mode spring constants for a variation of backfill properties consisting of 1) fill below foundation level (el 628 to 600 feet) with a constant shear wave velocity of 500 fps and, 2) fill below foundation level with a constant shear wave velocity of 1,350 fps. This was done by substituting the weighted average shear moduli for these cases for the weighted averages used earlier in the calculations for a shear wave velocity profile for fill (el 628 to 600 feet) varying from 500 fps to 850 fps. Because the vertical translations and rocking mode spring constants, K , and K_ψ, respectively, are linearly proportional to the shear modulus, G', the above can also be accomplished by multiplying the original spring constants by 0.85 and 1.8 for fill shear wave velocities of 500 fps and 1,359 fps, respectively.

Springs for Long-Term Static Condition

The subgrade moduli for the long-term settlement condition of the diesel generator building were developed from the settlement of the structure after the surcharge was removed neglecting the immediate heave which occurred following load removal, September 14, 1979, to December 31, 2025. Figure 2 contains the contact pressures used to determine these subgrade moduli.

The vertical subgrade moduli were determined by dividing the contact pressures by the corresponding measured and estimated settlements. The settlement used is the sum of 1) the measured settlement which occurred from September 14, 1979, to January 16, 1980, neglecting the immediate heave occurring after surcharge removal on August 15, 1979, 2) the estimated settlement from January 16, 1980, to December 31, 2025, extrapolated from settlement versus log (time) plots of the building settlement markers which were plotted for the time period during surcharge loading, and 3) the estimated dewatering settlement which had an estimated range of 0 inch to 0.25 inch. The estimated values were then proportioned within this range according to the settlement predicted from August 15, 1979, to December 31, 1981, by extrapolation of settlement versus log (time) plots described in (2) above. The effect of seismic shakedown settlements has been omitted as discussed in Subsection 2.1.2 of Enclosure 1.

The horizontal spring constant value to be used is the same as the value computed for the short-term static case.

Springs for Seismic Analysis

The seismic analysis for the diesel generator building was done by using the half-space lumped spring and mass representation approach presented in BC-TOP-4-A, Revision 3.

Differences may be noted between the spring values used for the seismic analysis and the springs used for the dynamic (short-term static) analysis. This is primarily due to the consideration of only local effects for the short-term static analysis, whereas the seismic analysis must consider global effects due to ground motion. Minor differences arise also because of the use of alternate formulas to calculate equivalent area, variations in Poisson's ratio, and graphical interpolation.

REFERENCES

1. Poulos, H.G., and Davis, E.H., Elastic Solutions for Soil and Rock Mechanics, John Wiley and Sons, Inc., 1974, (New York), 411 pp.

2. Richart, F.E., Jr., Hall, J.R., Jr., and Woods, R.D., Vibrations of Soils and Foundations, Prentice-Hall, Inc. 1970, (Englewood Cliffs, N.J.), 414 pp.

APPENDIX A

SUBGRADE MODULUS AND SPRING CONSTANT

VALUES FOR STRUCTURAL AND SEISMIC

ANALYSIS OF THE DIESEL GENERATOR BUILDING

APPENDIX A

I. Dynamic (Short-Term Static) Condition

The vertical subgrade modulus values and the horizontal and rocking spring constant values for the dynamic (short-term static) analyses of the diesel generator building are listed below.

Mode	Subgrade Modulus	Spring Constants N-S Direction (k/ft)	Spring Constants E-W Direction (k/ft)
Vertical	163 kcf		
Horizontal			
1. Base shear		3.1×10^5	3.1×10^5
2. Side friction between soil and wall		1.48×10^5	2.97×10^5
3. Wall pushing against soil		3.56×10^5	2.0×10^5
TOTAL OF 1, 2, and 3		8.14×10^5	8.07×10^5
Rocking		Rocking Axis E-W (k-ft/rad)	Rocking Axis N-S (k-ft/rad)
		1.85×10^9	4.65×10^9

II. Long-Term Static Condition

The vertical subgrade modulus values and the horizontal spring constants values for the long-term static analysis of the diesel generator building are given below.

1. Vertical subgrade modulus - The subgrade moduli of the selected points along the exterior wall footings are shown in Figure 1.

2. Horizontal spring constants - The horizontal spring constants given previously for the short-term static or dynamic case can also be used for the long-term static case. This is because horizontal displacements are small and in the elastic range.

III. Seismic Analysis

Soil springs for the seismic analysis of the diesel generator building were determined for two conditions to envelope the anticipated range of soil properties beneath the building. These springs are used for both the safe shutdown earthquake and the operating basis earthquake. Springs for the $E = 22 \times 10^3$ ksf are as follows:

	North-South	East-West	Vertical
K_x (K/ft)	1.5×10^6	1.5×10^6	-
K_z (K/ft)	-	-	2.0×10^6
K_ψ (K-ft/rad)	3.30×10^9	8.10×10^9	-

Soil springs based on revised soil properties with $E = 6.6 \times 10^3$ ksf and $V_s = 796$ ft/s as follows:

	North-South	East-West	Vertical
K_x (k/ft)	6.9×10^5	7.0×10^5	-
K_z (k/ft)	-	-	9.3×10^5
K_ψ (k-ft/rad)	1.70×10^9	3.50×10^9	-

The shear wave velocity of 796 ft/s is a low strain value. Its derivation is discussed in Subsection 2.1.6 of Enclosure 1. To account for strain effects on the modulus of elasticity, this low strain value is degraded to 660 ft/s. The modulus of elasticity corresponding to this reduced value is then varied by ±50% to account for variations in soil properties. The effect of lowering the shear wave velocity to 500 ft/s was also analyzed to ensure that the most conservative values were used for the structural analysis.

FIGURE 1

LONG-TERM STATIC SUBGRADE MODULI IN KCF

DIESEL GENERATOR BUILDING

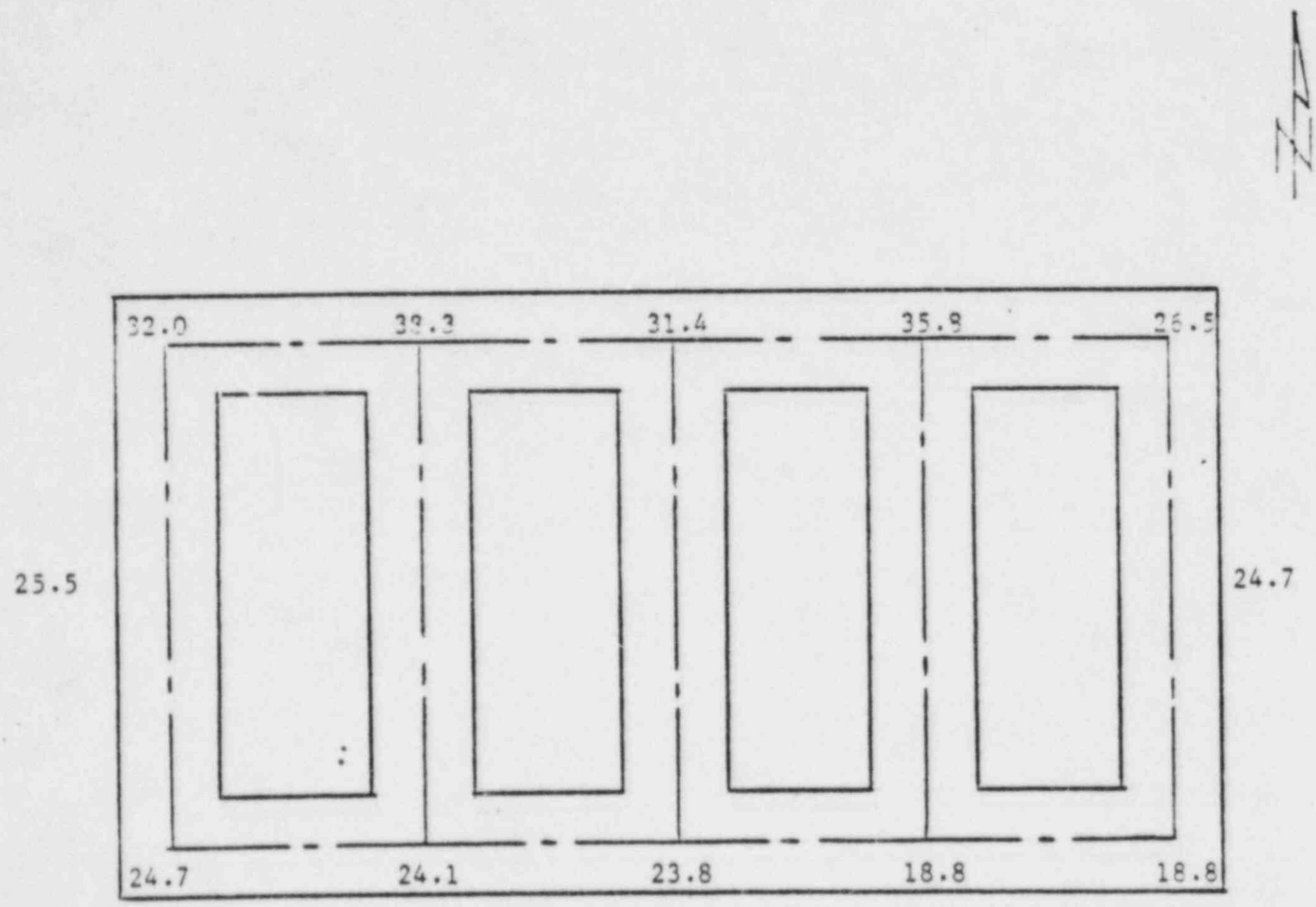
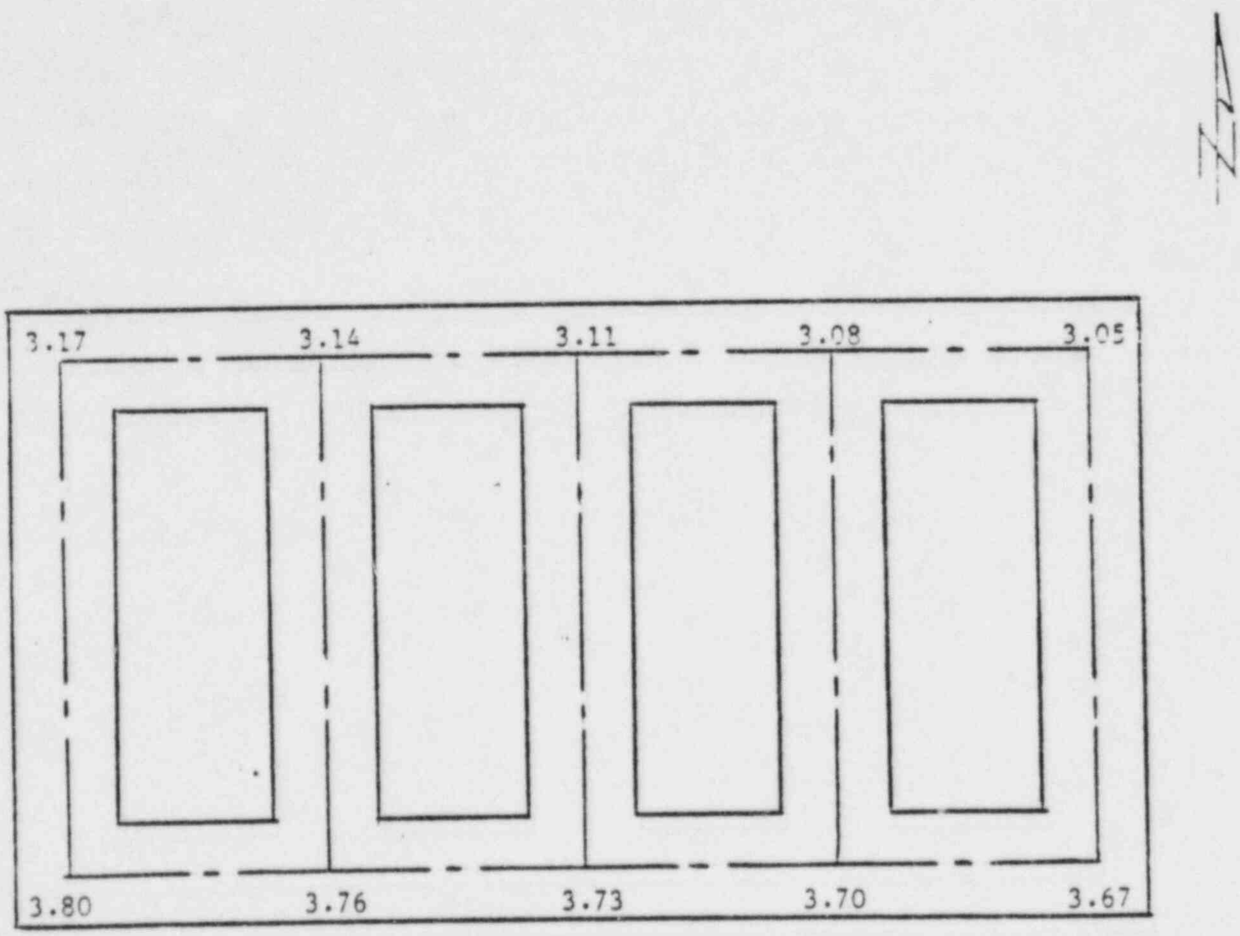


FIGURE 2

SUMMARY OF BEARING STRESS (KSF) DUE TO DEAD LOAD



<u>ENCLOSURE 3</u>

BEARING CAPACITY EVALUATION OF DIESEL GENERATOR BUILDING FOUNDATION

INTRODUCTION

Analyses were carried out to evaluate the factor of safety against bearing capacity failure of the diesel generator building footings when subjected to static and combined static and earthquake loading.

<u>Factors of Safety</u>

Factors of safety against bearing capacity failure were calculated for two different conditions:

1. A long time after the permanent structural load has been applied and the induced pore water pressures below the footing have dissipated. Effective shear strength parameters of the soil were used to calculate the factors of safety under these conditions.

   Laboratory testing of the fill materials indicated the effective strength parameters to be $c' = 0$ psf and $\phi' = 30°$. Factors of safety under drained conditions were calculated for a high water level at elevation 628' (at the bottom of the footing) and a low water table below elevation 603'. Factors of safety equal to 5.3 and 7.2 were calculated for these two conditions, respectively. These values exceed the value of 3 required in the Midland FSAR for dead and live load conditions.

2. Factor of safety against bearing capacity failure were calculated for a second condition corresponding to an earthquake occurrence sometime after the structure has been built. In the analysis it was assumed that during the time span before the earthquake occurs the excess pore water pressure in the ground, induced by the static loads on the footing, have dissipated. The undrained shear strength of the soil corresponding to consolidation under the 6 feet of surcharge plus the static footing load was used in estimating the factor of safety. The effective confining stresses along the failure surface was estimated by utilizing the method of slices to determine the mobilized friction angle required to maintain static equilibrium.

   The effective normal stress and shear stress at the base of each slice were then used to estimate the absolute values of the major and minor principle stresses $\overline{\sigma}_1$ and $\overline{\sigma}_3$ at the base of each slice due to the long term static loads. These initial consolidation stresses were used by Woodward-Clyde Consultants for determining the range of consolidation pressures and the appropriate anisotropic consolidation stress ratio, $\overline{\sigma}_1/\overline{\sigma}_3$, to use in anisotropically consolidated undrained triaxial tests. The results of these triaxial tests were summarized by plotting the undrained shear strength, τ_{ff}, along the failure surface for each sample as defined by Loewe and Karafiath (1959), and the initial effective confining stress, $\overline{\sigma}_n$, on that surface for both anisotropically and isotropically consolidated samples. It was found that the anisotropically consolidated samples gave slightly higher undrained strengths than the isotropically consolidated samples.

The factors of safety for the combined earthquake and static loading were obtained by using the method of slices. The undrained shear strength, τ_{ff}, at the base of each slice was determined from the plots of τ_{ff} versus $\overline{\sigma}_n$, referenced above, by entering the plots at the value of the initial static confining stress, $\overline{\sigma}_n$, at the base of each slice.

Factors of safety were calculated for a high water table at elevation 628' for the anisotropic and isotropic shear strength relationships which yielded factors of safety of 2.8 and 2.4, respectively. For the dewatered case when the water table is below the failure surfaces considered, a factor of safety of 3.1 was calculated from the anisotropic shear strength relationship.

In current engineering practice which is consistent with the Midland FSAR, a factor of safety of 2 is considered adequate when considering combined static and earthquake loads; under these conditions it is concluded that the present diesel generator building footings are adequate for the static and earthquake loads considered above.

ENCLOSURE 4

LONG-TERM MONITORING OF SETTLEMENT FOR DIESEL GENERATOR BUILDING

Issue:

Plans for Long-Term Monitoring of Settlement for Diesel Generator Building (Includes Technical Specifications, etc)

Response:

The settlement monitoring program for the diesel generator building has been established and is being implemented. Settlement monitoring points have been located around the building and on the machine pedestals to identify any tilting or warpage.

These points are surveyed every 60 days during construction, and every 90 days during the first year of plant operation. It is currently planned to evaluate the settlement data during the first year of plant operation and develop an appropriate monitoring interval for the remaining plant operating life. As a minimum, the building would be monitored annually for the next 5 years of operation and then at 5-year intervals thereafter. At least 6 points on the building will be monitored for the operating life of the plant: one point at each building corner and a point at the center of each east-west wall. Each corner of each machine pedestal will also be monitored as discussed above.

If the rate of settlement increases at any time during the monitoring program to a value greater than predicted for that monitoring point (see the response to NRC 10 CFR 50.54(f) Question 27), the monitoring interval will be increased to every 60 days to permit evaluation of the change in settlement. The allowable limit of absolute settlement of any point and relative settlement between points will be provided as part of the technical specifications in the Final Safety Analysis Report. These values for the pedestals are given in the response to NRC 10 CFR 50.54(f) Question 8. The building values due to fill consolidation are given below.

	SW Corner DG 1	South DG 21	SE Corner DG 3	NE Corner DG 28	North DG 26	NW Corner DG 24
Allowable limit of settlement after August 15, 1979 (inches)	1.85	1.89	2.34	1.38	1.19	1.18

ENCLOSURE 5

RELATIVE DENSITY AND SHAKEDOWN SETTLEMENT OF SAND UNDER THE DIESEL GENERATOR BUILDING

Comparison of Relative Densities

The relative densities of the sand fill in the area of the diesel generator building were compared based on information from the DG series borings conducted before surcharge and the COE series borings conducted after surcharge. The relative densities from the DG series bc ings were determined from standard penetration blow count data using Gibbs and Holtz (1957) relationships. The relative densities from the COE series borings were determined by Woodward-Clyde Consultants (WCC) from in-situ densities of tube samples and grain size data as described in the report entitled "Estimates of Relative Density of Granular Fill Materials, Diesel Generator Building, Midland Plant, Units 1 & 2, Midland, Michigan" dated July 24, 1981.

For the purpose of presentation of relative density data the building was divided into four quadrants as shown in Figure 1. The same Figure 1 also shows the location of DG series borings and COE series borings. The relative densities are compared below.

Relative Density Comparison (%)

	DG Series Borings		COE Series Borings	
Quadrant	Range	Average	Range	Average
Northwest	15-100	62	42-100	82
Northeast	15-100	72	28-100	74
Southwest	45	45	57-100	88
Southeast	10-100	69	56-100	88

It is seen from the above comparison that the relative density obtained from the COE series borings after surcharge are higher than the relative densities obtained from the DG borings before surcharge.

Shakedown Settlement

The settlement of the granular fill materials under the diesel generator building due to ground shaking (SSE = 0.12g) caused by earthquakes was calculated based on tests performed on soil samples obtained after the surcharge progarm during the recent (1981) soil investigation program by Woodward-Clyde Consultants (WCC). The details of the tests and results obtained are presented in the WCC report entitled, "Estimates of Relative Density of Granular Fill Materials, Diesel Generator Building, Midland Plant Units 1 and 2, Midland, Michigan" dated July 24, 1981. The settlements were estimated based on the approach described by Seed and Silver (1969) and the recommendations on multidirectional shaking by Pike, Seed and Chen (1975) at the location of Borings COE 8, COE 10, COE 11, COE 12 and COE 13. For the COE borings the relative density used in the analyses was the average relative density reported by WCC in the report referenced above. The settlement values based on DS series borings prior to surcharge were also calculated using the above approach. However, the relative density values were obtained using the standard penetration blow count and the Gibbs and Holtz relationship. The settlement estimate based on borings prior to and after surcharge program are compared below.

Location	DG Series Borings (Inch)	COE Series Borings (Inch)
Northwest Quarter	0.02-0.36	0.07
Northeast Quarter	0.00-0.25	0.07
Southwest Quarter	0.00-0.02	0.02-0.11
Southeast Quarter	0.00-0.14	0.02-0.11

It can be seen from the above comparison that the maximum settlement calculated based on the COE series borings (after surcharge) are lower than that calculated based on the DG series borings (before surcharge). Therefore, the design values of settlement and differential settlement of 1/2 inch provided in the response to 10CFR50.54(f) Question 27 are conservative.

10/15/81

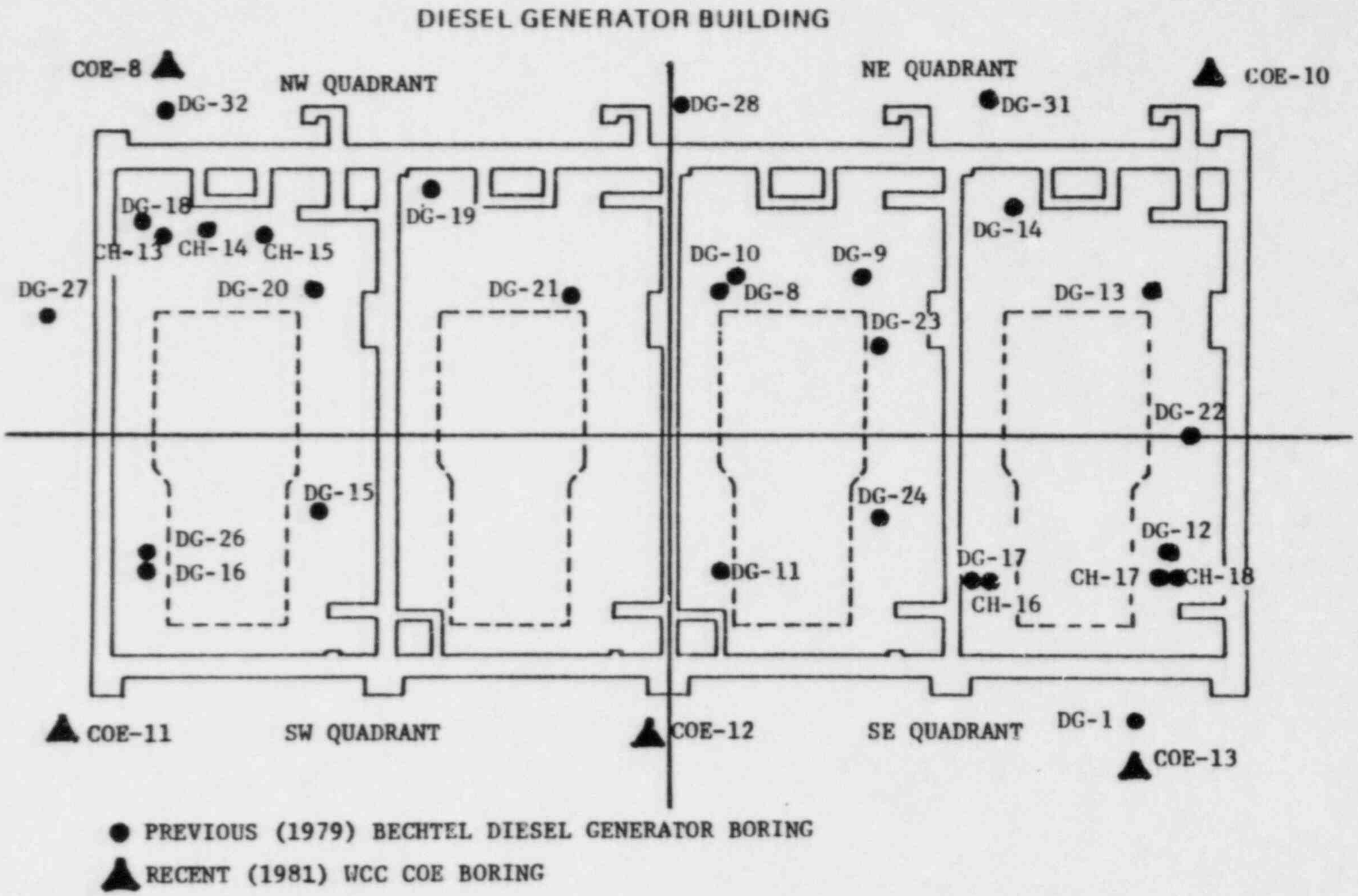


FIGURE 1
BORING LOCATION PLAN IN AREA OF
DIESEL GENERATOR BUILDING

ENCLOSURE 6

ESTIMATES OF RELATIVE DENSITY
OF GRANULAR FILL MATERIALS
DIESEL GENERATOR BUILDING
MIDLAND PLANT - UNIT 1 and 2
MIDLAND, MICHIGAN

11 East Adams Street
Suite 1500
Chicago, Illinois 60603
312-939-1000
Telex 253875 (WOODWARD CGO)

Woodward-Clyde Consultants

ESTIMATES OF RELATIVE DENSITY OF GRANULAR FILL MATERIALS DIESEL GENERATOR BUILDING MIDLAND PLANT - UNIT 1 and 2 MIDLAND, MICHIGAN

For

Consumers Power Company
1945 West Parnall Road
Jackson, Michigan

By

James W. Stenborg — and — Larry M. Campbell

James W. Stenborg and Larry M. Campbell

24 July 1981
81C217

Consulting Engineers Geologists
and Environmental Scientists

Offices in Other Principal Cities

~~8111020149~~ 61pp

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1. INTRODUCTION

This report presents estimates of relative density of granular fill materials obtained at the Diesel Generator Building (DGB) at Consumers Power Company (CPCo) Midland Plant - Units 1 and 2. These estimates of relative density are provided in direct response to a request from (and in the manner suggested by) Dr. Paul Hadala of the Corps of Engineers Waterways Experiment Station. This request was made during a meeting with the Nuclear Regulatory Commission in Bethesday, MD on 6 May 1981.

Estimates of relative density were made using densities of granular fill samples obtained from the vicinity of the DGB during a Soil Boring and Testing Program conducted at the Midland Plant by Woodward-Clyde Consultants (WCC) during the spring of 1981. Maximum and minimum densities of these granular fill samples were estimated from various empirical relationships based on the gradation of the samples as determined from particle-size distribution analyses performed during the referenced testing program.

2. DATA SOURCES

All "test" data for the granular fill samples were obtained from a report to CPCo prepared by WCC (1981) concerning test results of the Soil Boring and Testing Program for the DGB. These data had been obtained from measurements and tests on granular fill samples obtained above elev. 600 from five borings drilled outside of the DGB, (but within the crest of the previously applied surcharge) as shown on Fig. 1. Densities of granular fill materials had been determined during the referenced testing program based on measurements of the entire thin-walled tube samples and some 6-in.-long sections extruded from the tubes.

These total tube (wet) densities, section (dry) densities, and water contents were obtained from the tables in Appendix B of WCC (1981). Particle-size distribution data were obtained from Appendix C of the same report.

Copies of 38 particle-size distribution curves for the granular fill samples considered are presented in boring/sample-number order in Appendix A of this report. Multiple particle-size distribution analyses were performed on some tube samples because the tubes contained materials of differing gradations.

Maximum and minimum densities of the granular fill samples were estimated from empirical relationships between density and gradation as described by Burmister (1962), Alpan (1976), and Johnston (1973). Similarly, maximum and minimum void ratios were estimated from an empirical relationship described by Youd (1973). Brief descriptions of each of the methods and illustrations of the relationships are presented in Appendix B.

3. ANALYTICAL PROCEDURE

3.1 In-situ Density

Values of in-situ dry density were calculated for each of 27 tube samples by dividing the reported total tube density by the quantity one plus the averaged tube sample water content (expressed in decimal form). The average tube water content was calculated as the average of the water content determinations done on specimens from the tube. In nine cases, in-situ dry densities had been determined based on measurements of sections extruded from the tubes. Because most of the densities were calculated for an entire tube sample, the density correction factor suggested by Marcuson and Franklin (1979) in Fig. 5d (for location within the sample tube) was not applied to the available data.

3.2 Maximum and Minimum Densities (Void Ratios)

For each of the 38 particle-size distribution curves presented in Appendix A, maximum and minimum densities (or void ratios) were estimated using each of the four empirical relationships illustrated in Appendix B. Three of these relationships (Alpan, Johnston, and Youd) utilize the coefficient of uniformity (C_u) as the parameter to represent the gradation characteristics of the samples. For the Burmister maximum density relationship, a shape designation of the particle-size distribution curve and the effective grain size range (C_r) are required; the minimum density relationship requires C_r and the effective grain size (D_{50}).

These required parameters were determined for each particle-size distribution curve. Using these parameters, maximum and minimum densities were estimated for the relationships developed by Burmister, Alpan, and Johnston. Using Youd's relationship maximum and minimum void ratios were estimated. These void ratios were converted to densities assuming that the specific gravity of the granular fill materials was 2.68.

3.3 Relative Density

Relative density (D_r) was calculated using the following equation:

$$D_r \ (\%) = \frac{\gamma_{dmax}}{\gamma_d} \times \frac{\gamma_d - \gamma_{dmin}}{\gamma_{dmax} - \gamma_{dmin}} \times 100$$

where: γ_d, γ_{dmax}, and γ_{dmin} are the in-situ, maximum, and minimum dry densities, respectively. In-situ dry densities and maximum and minimum dry densities were obtained as previously described in Sections 3.1 and 3.2, respectively.

Estimates of relative density were calculated for applicable combinations of tube and section densities and multiple gradations from a single tube. A total of 47 relative densities, for each of the four different relationships previously described, were calculated; 38 were based on tube densities and 9 were based on section densities for each relationship.

4. DATA PRESENTATION

The data determined and/or calculated as described in Section 3 are presented graphically in Appendix C. The densities (γ_d, γ_{max} and γ_{min}) and relative density values (D_r) are plotted by elevation in Figures C-1 through

C-5 for each of the five borings from which density and gradation data were obtained. For each boring, the results based on the Burmister Alpan, Johnston, and Youd relationships are plotted and presented as Figures -a, -b, -c, and -d, respectively.

The left-hand portion of each figure depicts the tube and section dry densities and the estimates of maximum and minimum dry densities. The calculated relative densities are plotted on the right-portion of each figure. These calculated relative densities are also summarized by boring in Tables 1 through 5. The data are tabulated in order of decreasing elevation and include 1) the relative densities based on the four relationships, 2) the range of these calculated values, and 3) the average for each sample. The range and average of these estimated relative densities are also plotted by elevation for all 38 tube densities as Fig. 2 and for all 9 section densities on Fig. 3.

A histogram showing the distribution of the average estimated relative densities based on the four relationships is presented on Fig. 4. The average estimated relative density is greater than 70% for 37 of the 47 densities (79%), greater than 60% for 94% of the densities, and greater than 50% for 100% of the densities.

5. REFERENCES

Alpan, I. (1976). Personal communications with D.M. Hendron of Woodward-Clyde Consultants.

Burmister, D.M. (1962). Physical, Stress-Strain, and Strength Responses of Granular Soils. In Symposium on Field Testing of Soils, ASTM Special Technical Publication No. 322, pp 67-97.

Johnston, M.M. (1973). Laboratory Studies of Maximum and Minimum Dry Densities of Cohesionless Soils. In Evaluation of Relative Density and Its Role in Geotechnical Projects Involving Cohesionless Soils, ASTM Special Technical Publication No. 523, pp 133-140.

Marcuson, III, W.F., and Franklin, A. G. (1979). State of the Art of Undisturbed Sampling of Cohesionless Soils. In State of the Art on Current Practice of Soil Sampling, Proceedings of the International Symposium of Soil Sampling, Singapore, pp 57-71.

Woodward-Clyde Consultants (1981). Test Results, Diesel Generator Building, Soil Boring and Testing Program, Midland Plant-Units 1 and 2, Midland, Michigan. Report preared for Consumers Power Company, Jackson, MI, by Woodward-Clyde Consultants, Chicago, IL, 8 July 1981, p 9, 8 appendices.

Youd, T.L. (1973). Factors Controlling Maximum and Minimum Densities of Sand. In Evaluation of Relative Density and Its Role in Geotechnical Projects Involving Cohesionless Soils, ASTM Special Technical Publication No. 523, pp 98-112.

TABLE 1

SUMMARY OF RELATIVE DENSITY VALUES
BASED ON PARTICLE-SIZE DISTRIBUTION

Boring COE-8

Sample Number	Depth (ft)	Elev. (ft)	D_r, Relative Density (%) Calculated Based On: Burmister	Youd	Alpan	Johnston	Average	Range
S-3A	6.6	627.6	74	79	88	93	83.5	74-93
S-4A	8.4	625.8	67	53	66	42	57.0	42-67
S-5B	10.4	623.8	65	59	75	66	66.2	59-75
S-7A	12.4	621.8	82	93	100	103	94.5	82-103
S-7B	13.0	621.2	80	73	81	60	73.5	60-81
S-8A	13.5	620.7	95	83	93	90	90.2	83-95
S-8B	14.2	620.0	94	90	97	99	95.0	90-99
S-9A	14.6	619.6	86	76	86	77	81.2	76-86
S-10B	15.6	618.6	85	84	92	94	88.8	84-94
S-11A	17.6	616.6	74	103	110	127	103.5	74-127
S-11D	19.4	614.8	79	78	88	88	83.2	78-88
S-12A	20.1	614.1	85	77	86	80	82.0	77-86
S-12C	21.0	613.2	85	82	92	92	87.8	82-92
S-13A	22.8	611.4	67	55	71	57	62.5	55-71
S-14A	25.1	609.1	67	59	74	61	65.2	59-74
S-14D	26.6	607.6	74	80	91	97	85.5	74-97
S-15A	27.5	606.7	86	98	101	129	103.5	86-129
S-15B	28.0	606.2	46	57	69	77	62.2	46-77
S-16B	30.6	603.6	89	89	99	115	98.0	89-115
S-16C	31.7	603.1	65	65	80	80	72.5	65-80
S-17A	32.7	601.5	49	62	75	88	68.5	49-88
S-17A	32.7	601.5	(82)	(96)	(106)	(121)	(101.2)	(82-121)
S-17C	33.6	600.6	81	89	91	116	94.2	81-116
S-17C	33.6	600.6	(57)	(71)	(74)	(100)	(75.5)	(57-100)

Note: All relative densities were calculated using tube densities except those shown in parentheses which were calculated using section densities.

81C217

TABLE 2

SUMMARY OF RELATIVE DENSITY VALUES
BASED ON PARTICLE-SIZE DISTRIBUTION

Boring COE-10

Sample Number	Depth (ft)	Elev. (ft)	D_r, Relative Density (%) Calculated Based On: Burmister	Youd	Alpan	Johnston	Average	Range
S-1B	6.8	627.1	28	49	59	72	52.0	28-72
S-1B	6.8	627.1	(51)	(70)	(77)	(92)	(72.5)	(51-92)
S-2A	8.3	625.6	64	81	88	103	84.0	64-103
S-2A	8.3	625.6	(53)	(71)	(78)	(93)	(73.8)	(53-93)
S-8B	24.8	609.1	44	55	57	88	61.0	44-88
S-9B	26.3	607.6	98	102	106	128	108.5	98-128
S-9B	26.3	607.6	(91)	(96)	(100)	(122)	(102.2)	(91-122)
S-10A	28.3	605.6	66	62	72	92	73.0	62-92
S-10A	28.3	605.6	(58)	(55)	(65)	(85)	(65.8)	(55-85)
S-10C	29.0	604.9	49	53	67	63	58.0	49-63
S-11B	31.2	602.7	57	72	82	93	76.0	57-93

Note: All relative densities were calculated using tube densities except those shown in parentheses which were calculated using section densities.

TABLE 3

SUMMARY OF RELATIVE DENSITY VALUES
BASED ON PARTICLE-SIZE DISTRIBUTION

Boring COE-11

			D_r, Relative Density (%)					
			Calculated Based On:					
Sample Number	Depth (ft)	Elev. (ft)	Burmister	Youd	Alpan	Johnston	Average	Range
S-2B	8.6	624.9	123	110	112	116	115.2	110-123
S-10A	21.1	612.4	83	80	84	111	89.5	80-111
S-10A	21.1	612.4	(72)	(73)	(76)	(104)	(81.3)	(72-104)
S-11B	24.6	608.9	74	88	90	119	92.8	74-119
S-14A	30.7	602.8	94	69	76	96	83.8	69-96
S-14A	30.7	602.8	(78)	(57)	(85)	(85)	(76.2)	(57-85)
S-14C	31.7	601.8	94	82	74	110	90.0	74-110

Note: All relative densities were calculated using tube densities except those shown in parentheses which were calculated using section densities.

TABLE 4

SUMMARY OF RELATIVE DENSITY VALUES
BASED ON PARTICLE-SIZE DISTRIBUTION

Boring COE-12

			D_r, Relative Density (%)					
			Calculated Based On:					
Sample Number	Depth (ft)	Elev. (ft)	Burmister	Youd	Alpan	Johnston	Average	Range
S-5A	11.4	622.2	74	73	85	86	79.5	73-86

Note: All relative densities were calculated using tube densities except those shown in parentheses which were calculated using section densities.

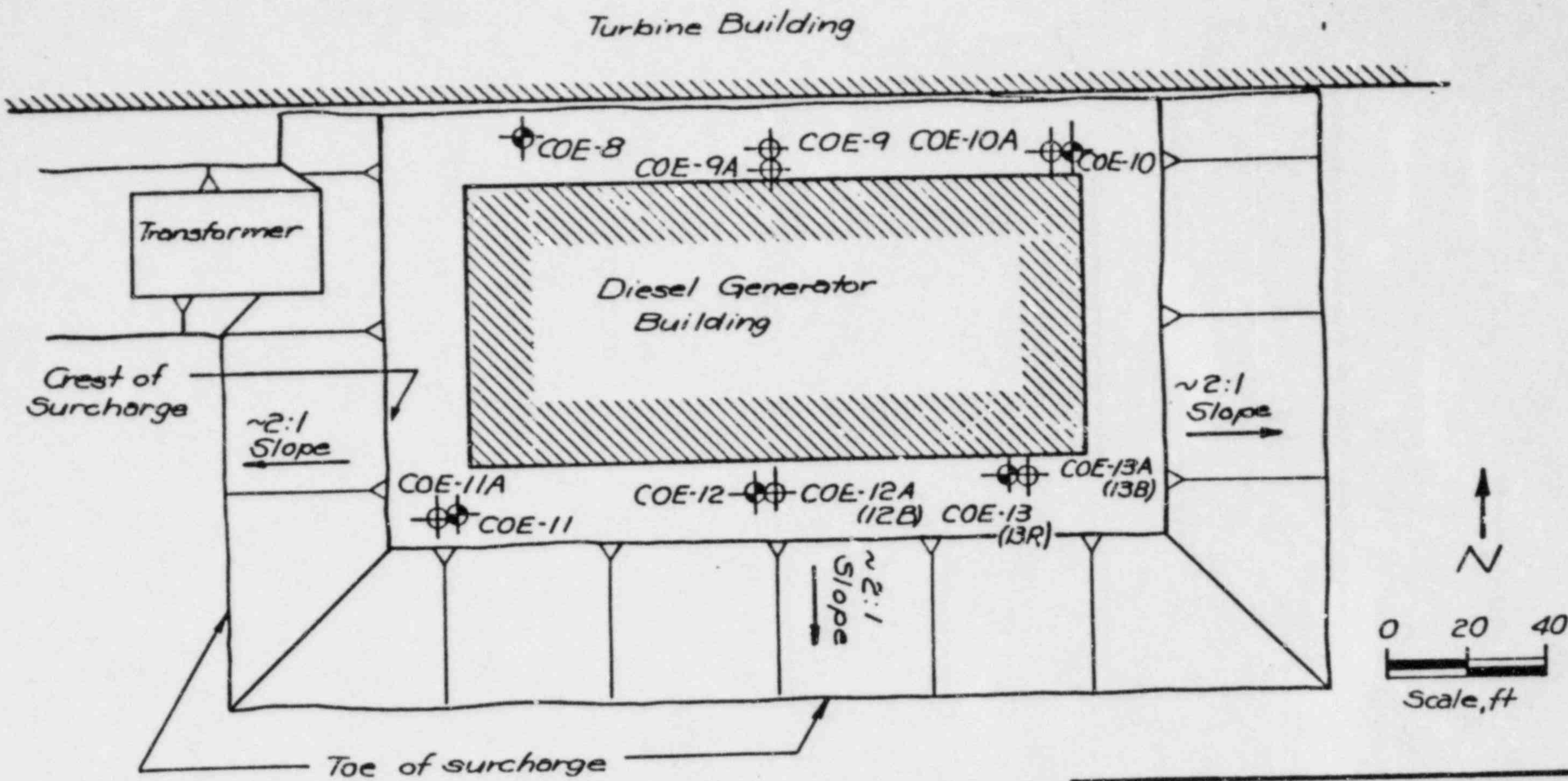
TABLE 5

SUMMARY OF RELATIVE DENSITY VALUES
BASED ON PARTICLE-SIZE DISTRIBUTION

Boring COE-13R

Sample Number	Depth (ft)	Elev. (ft)	D_r, Relative Density (%)					
			Calculated Based On:					
			Burmister	Youd	Alpan	Johnston	Average	Range
S-16B	30.0	603.6	56	68	76	110	77.5	56-110
S-17A	31.3	602.3	63	82	85	113	85.8	63-113
S-17C	32.4	601.2	86	97	95	125	100.8	86-125
S-17C	32.4	601.2	(66)	(85)	(82)	(114)	(86.8)	66-114

Note: All relative densities were calculated using tube densities except those shown in parentheses which were calculated using section densities.



COE-11A ⊕ Boring Location and number

COE-8 ⊕ Boring from which in-situ density and particle-size analysis specimen(s) were obtained

(13R) Indicates first attempt to drill Boring

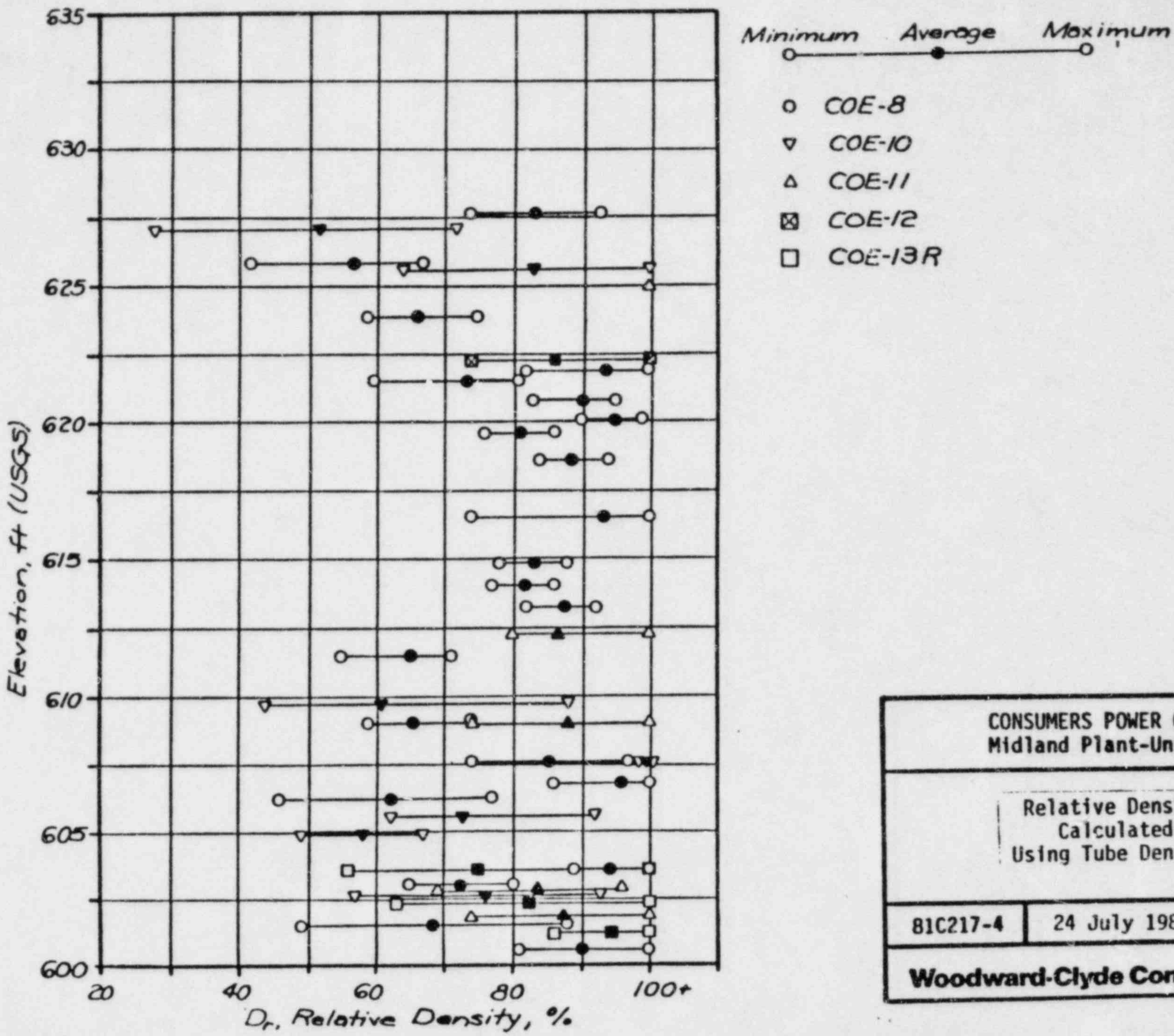
Source: Bechtel drawings C3 and C-1040Q

CONSUMERS POWER COMPANY
Midland Plant-Units 1&2

Boring Location Plan
Borings Drilled in 1981

81C217-4	24 July 1981	Fig. 1

Woodward-Clyde Consultants

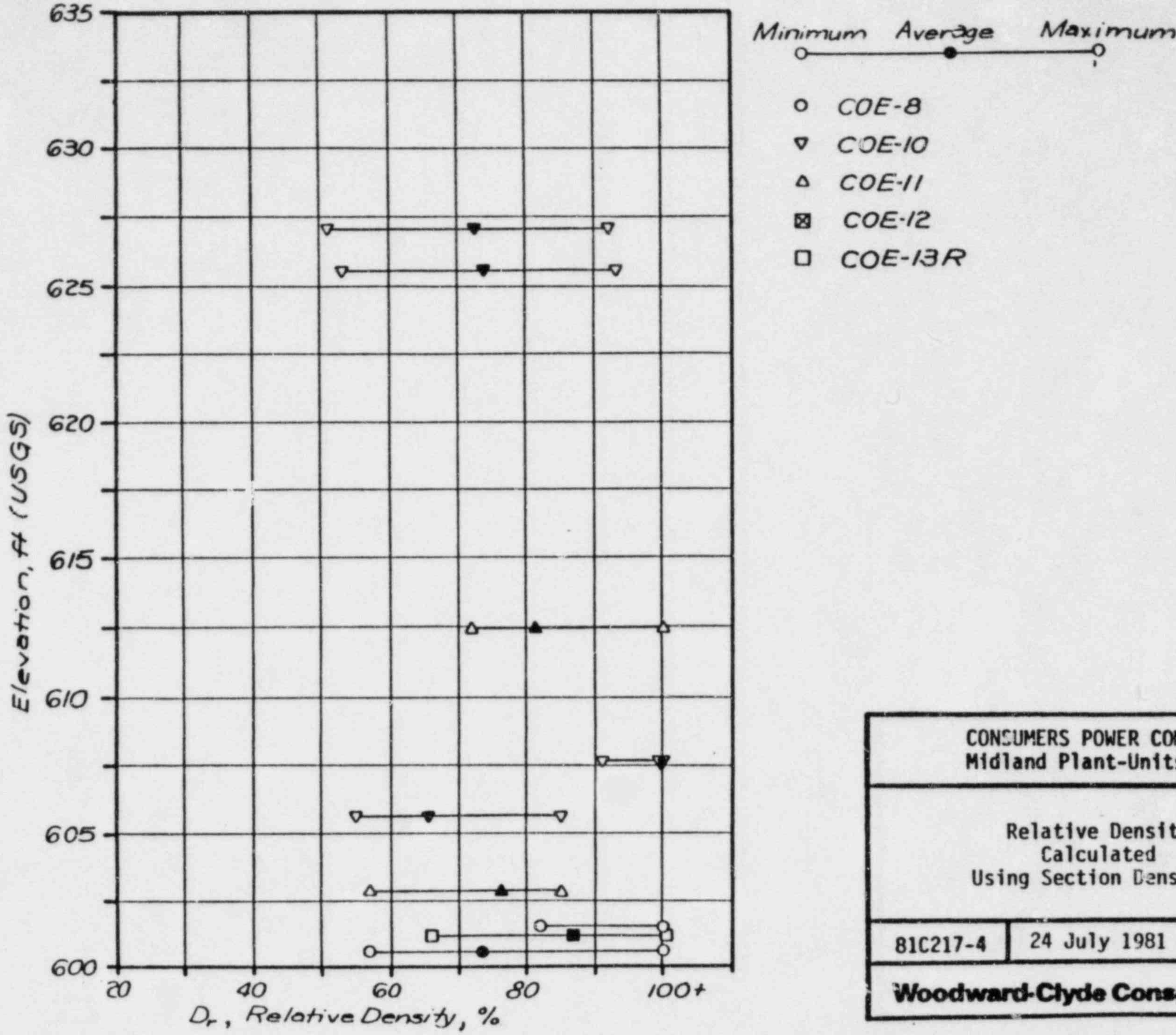


CONSUMERS POWER COMPANY
Midland Plant-Units 1&2

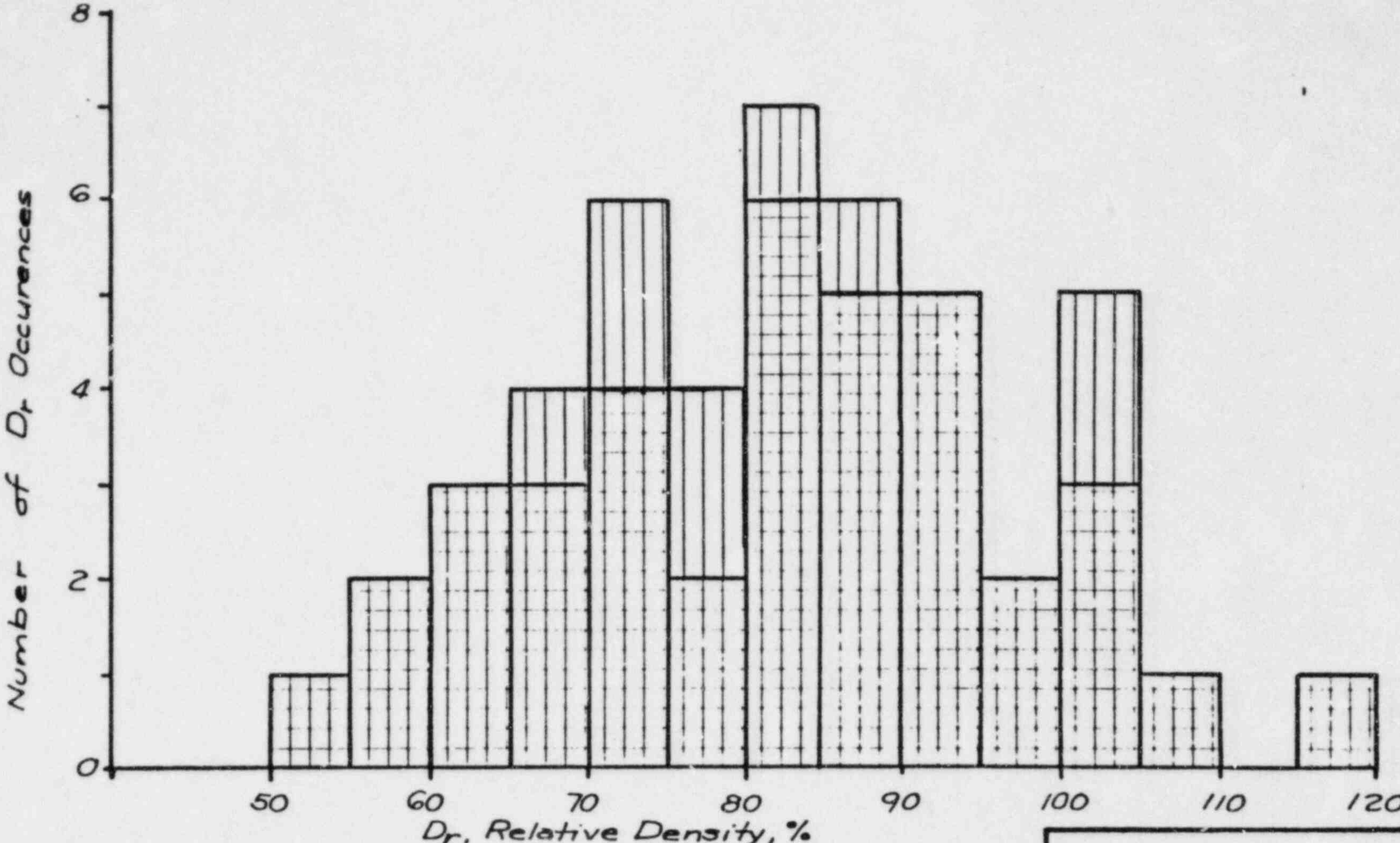
Relative Densities
Calculated
Using Tube Densities

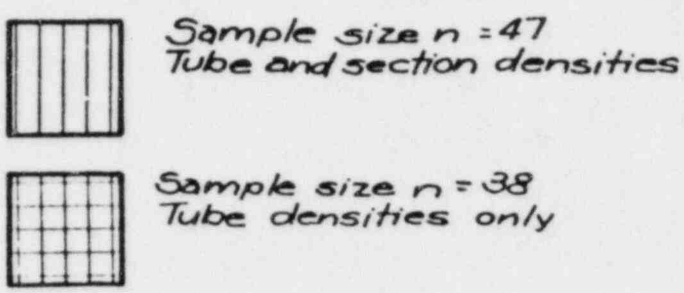
81C217-4	24 July 1981	Fig. 2

Woodward-Clyde Consultants



CONSUMERS POWER COMPANY Midland Plant-Units 1&2		
Relative Densities Calculated Using Section Densities		
81C217-4	24 July 1981	Fig. 3
Woodward-Clyde Consultants		





CONSUMERS POWER COMPANY
Midland Plant-Units 1&2

Histogram of Relative Densities

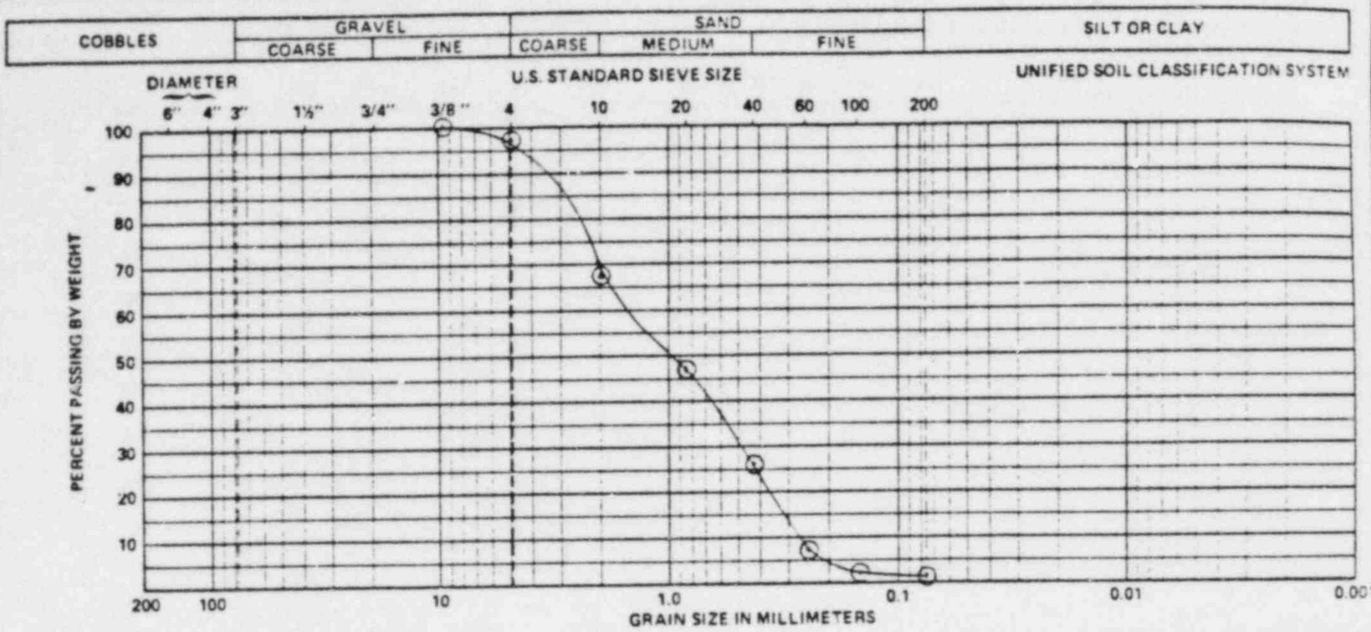
81C217-4	24 July 1981	Fig. 4

Woodward-Clyde Consultants

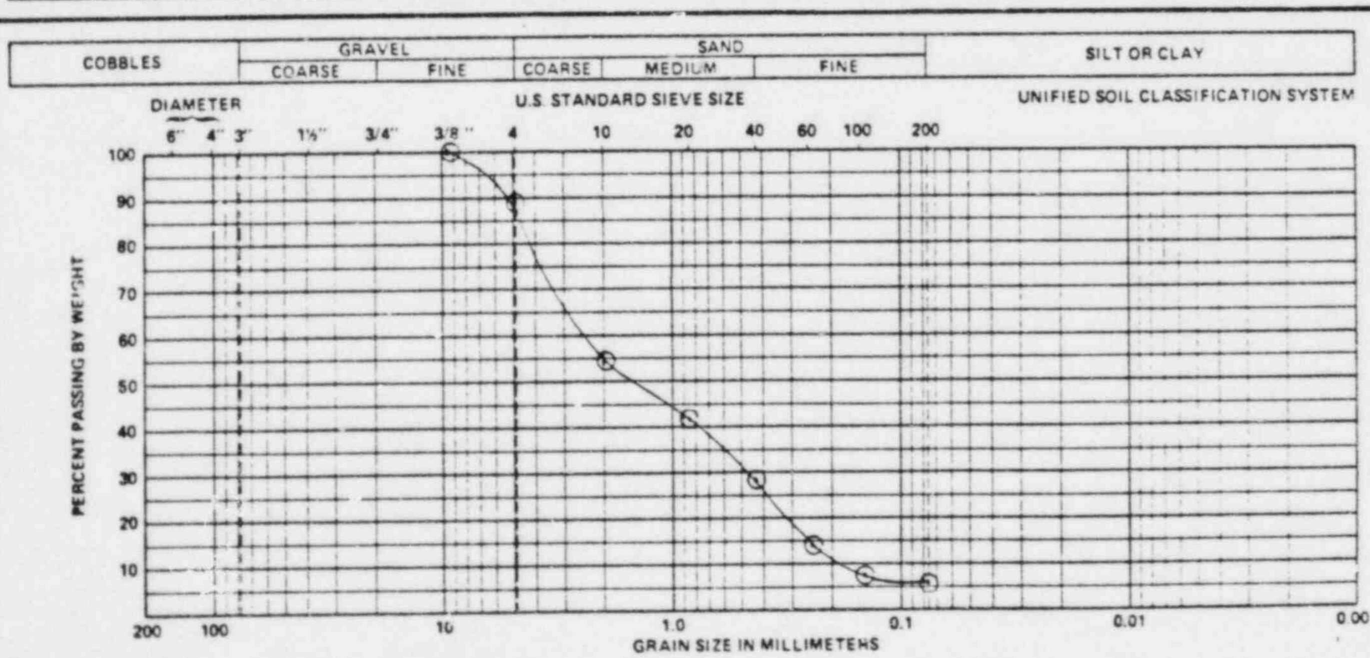
APPENDIX A

Particle-Size Distribution Curves

PARTICLE-SIZE DISTRIBUTION



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE B	S-3A	6.6	⊙	SP, brown and gray, c. to f. SAND, tr. f. gravel, tr. silt	3.5		



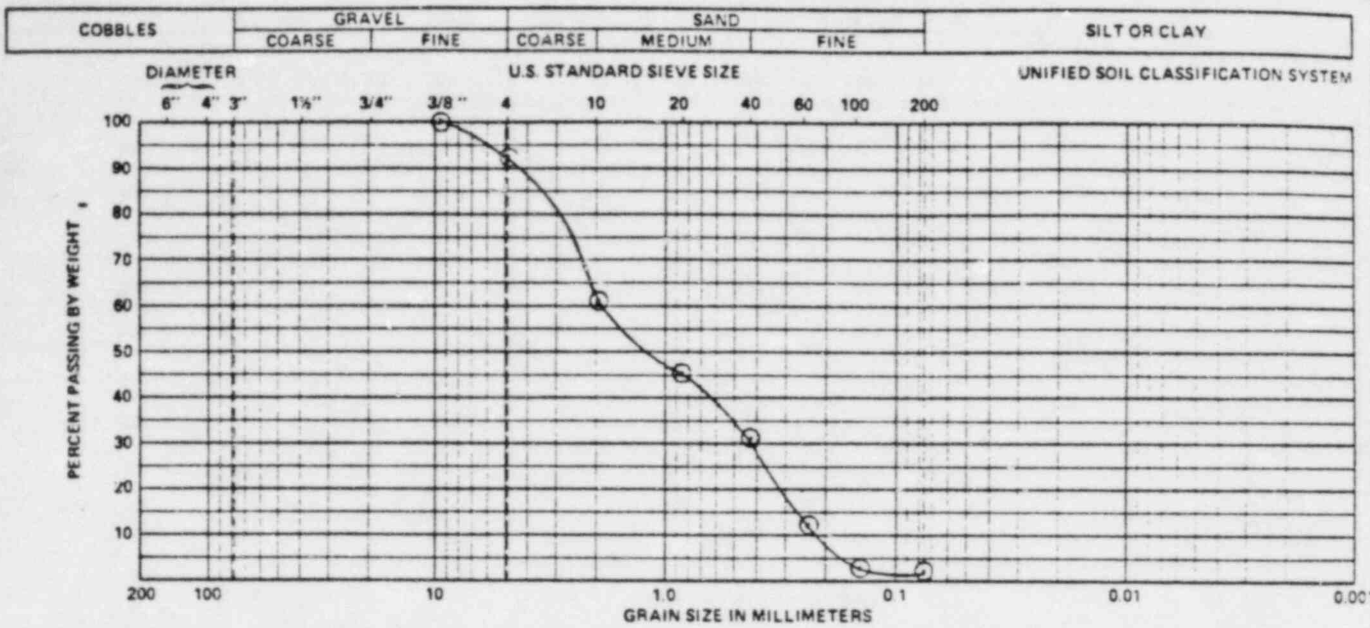
BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE B	S-4A	8.4'	⊙	SP-SM, brown, c. to f. SAND, tr. f. gravel, tr. silt	10.0		

Fig. A-1 (1/7)

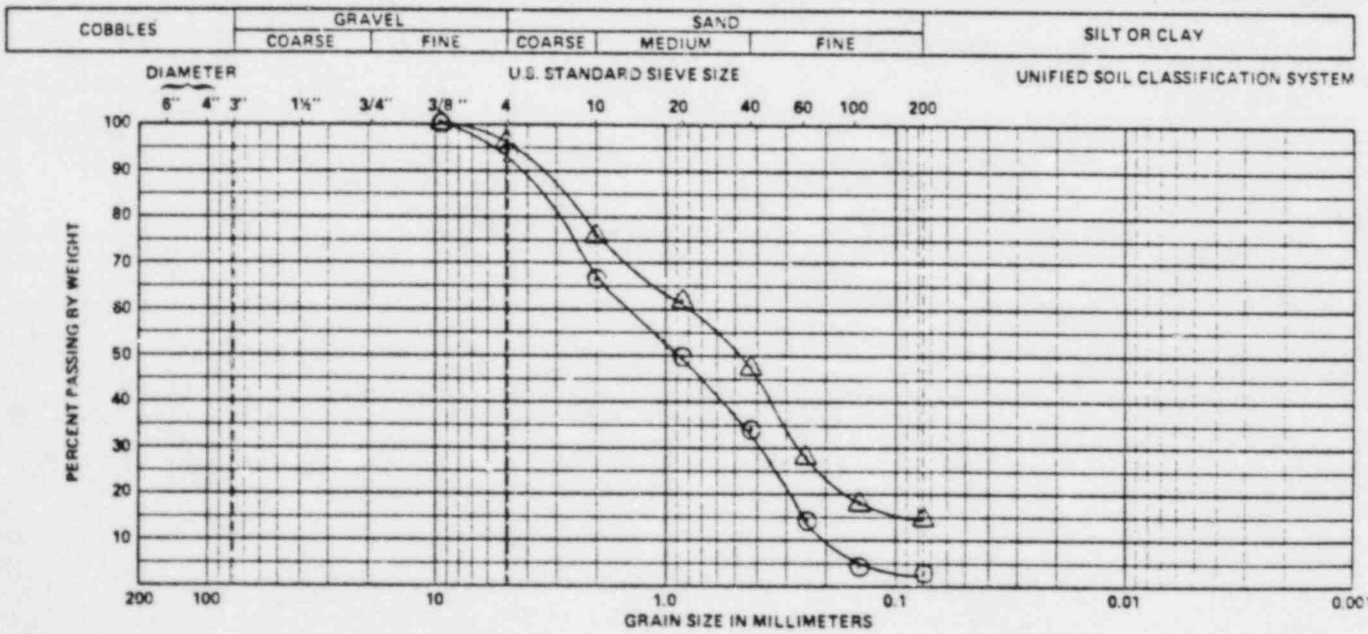
PROJECT NO. 81C4055T3 DRAWN BY: PRF REVIEWED BY: WCC LR-101 (9/80)

CHECKED BY MTD 30 JUNE 81

PARTICLE-SIZE DISTRIBUTION



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE 8	S-5-B	10.4	⊙	SP, brown, c. to f. SAND, tr. f. gravel, tr. silt	9.5		



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE 8	S-7-A	12.4	⊙	SP, tan, c. to f. SAND, tr. f. gravel, tr. silt	7.4		
COE 8	S-7-B	12.8	△	SP-SM, tan, c. to f. SAND, tr. f. gravel, some silt with occ. pockets gray sandy silty CLAY (CL)	9.5		

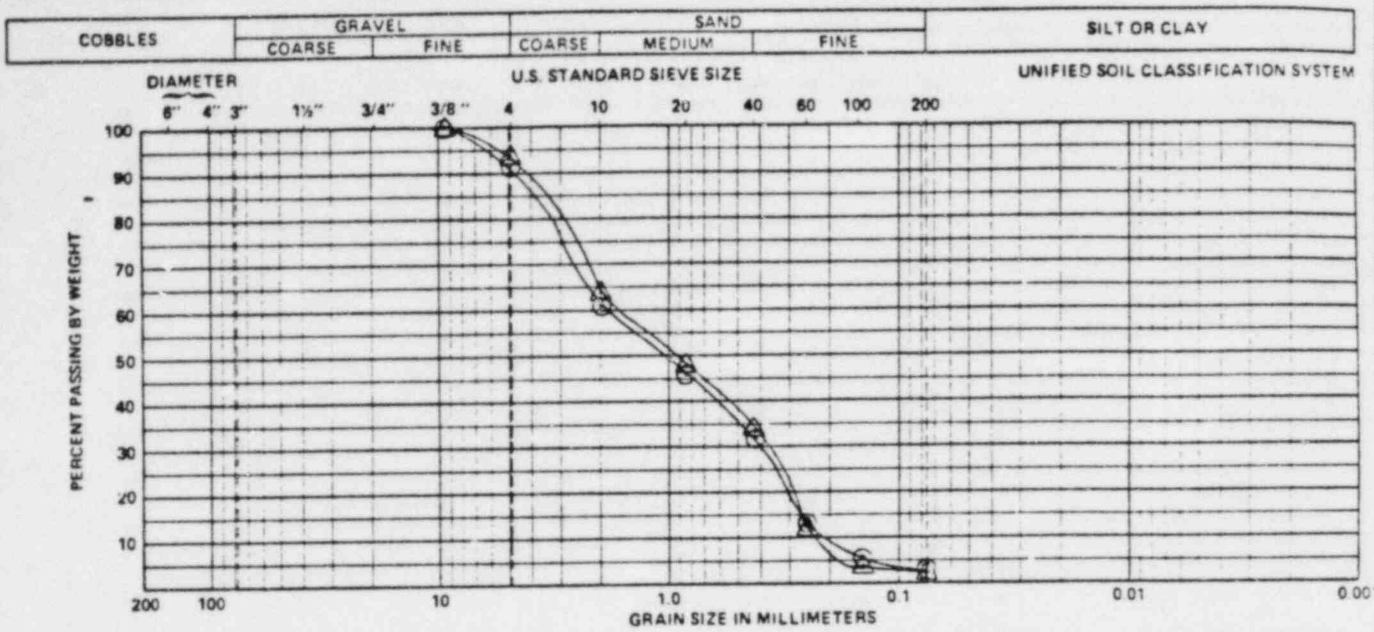
Fig. A-1 (2/7)

CHECKED BY MTB 30 JUNE 81

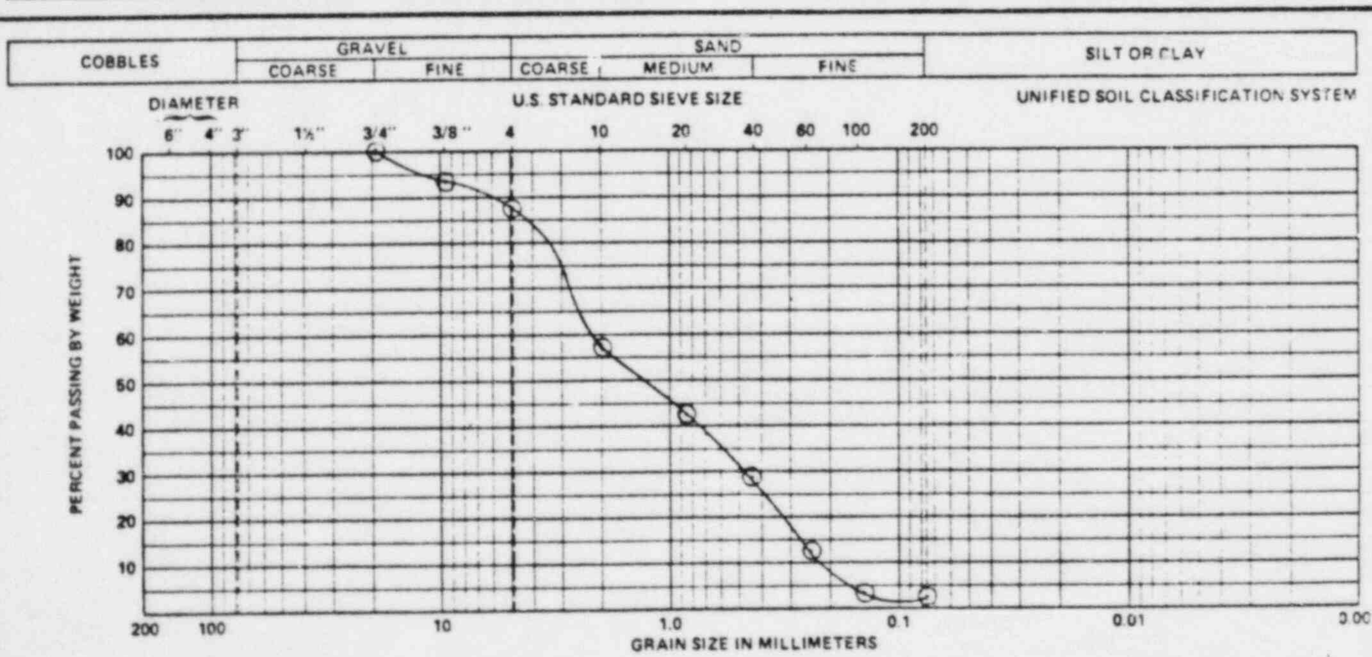
PROJECT NO: 81C 4055 T3 DRAWN BY: PRF

REVIEWED BY WCC LR-101 (9/80)

PARTICLE-SIZE DISTRIBUTION



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE B	S-8-A	13.5	⊙	SP, brown, c. to f. SAND, tr. f. gravel, tr. silt	8.9		
COE B	S-8-B	14.2	△	SP, brown, c. to f. SAND, tr. f. gravel, tr. silt	6.1		



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE B	S-9-A	14.65	⊙	SP, brown, c. to f. SAND, tr. f. gravel, tr. silt	9.6		

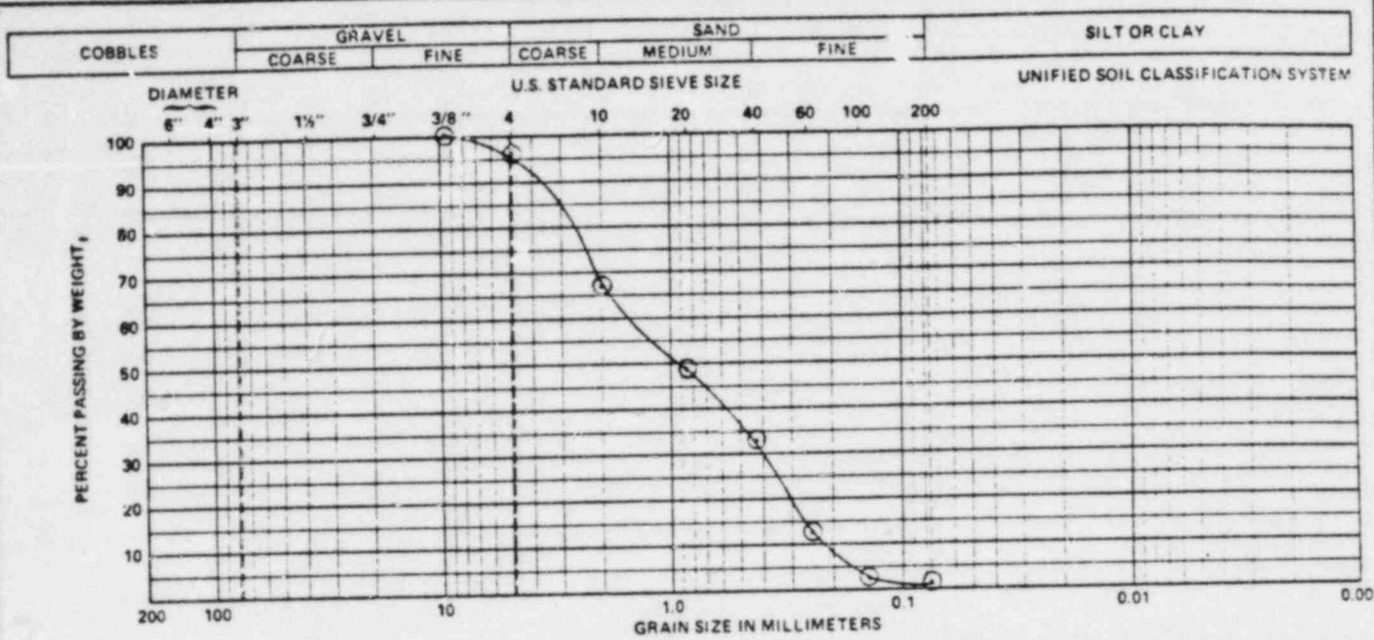
Fig. A-1 (3/7)

CHECKED BY MB 20 JUNE 81

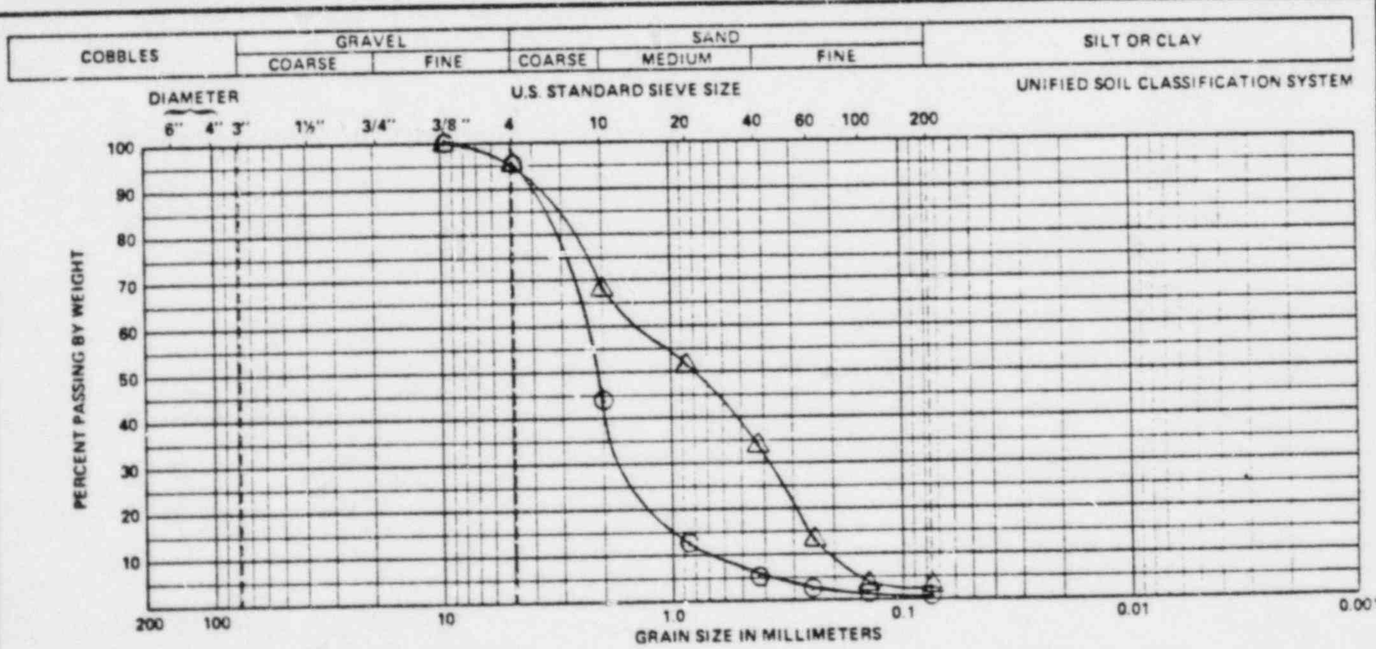
REVIEWED BY WCC LR-101 (9/80)

PROJECT NO. BIC line 73 DRAWN BY: PDP

PARTICLE-SIZE DISTRIBUTION



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE 8	S-10-B	15.65	⊙	SP, tan, c. to f. SAND, tr. f. gravel, tr. silt	5.8		



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE 8	S-11-A	17.7	⊙	SP, tan, c. to f. SAND, tr. f. gravel, tr silt	4.5		
COE 8	S-11-D	19.4	△	SP, tan, c. to f. SAND, tr. f. gravel, tr silt	11.9		

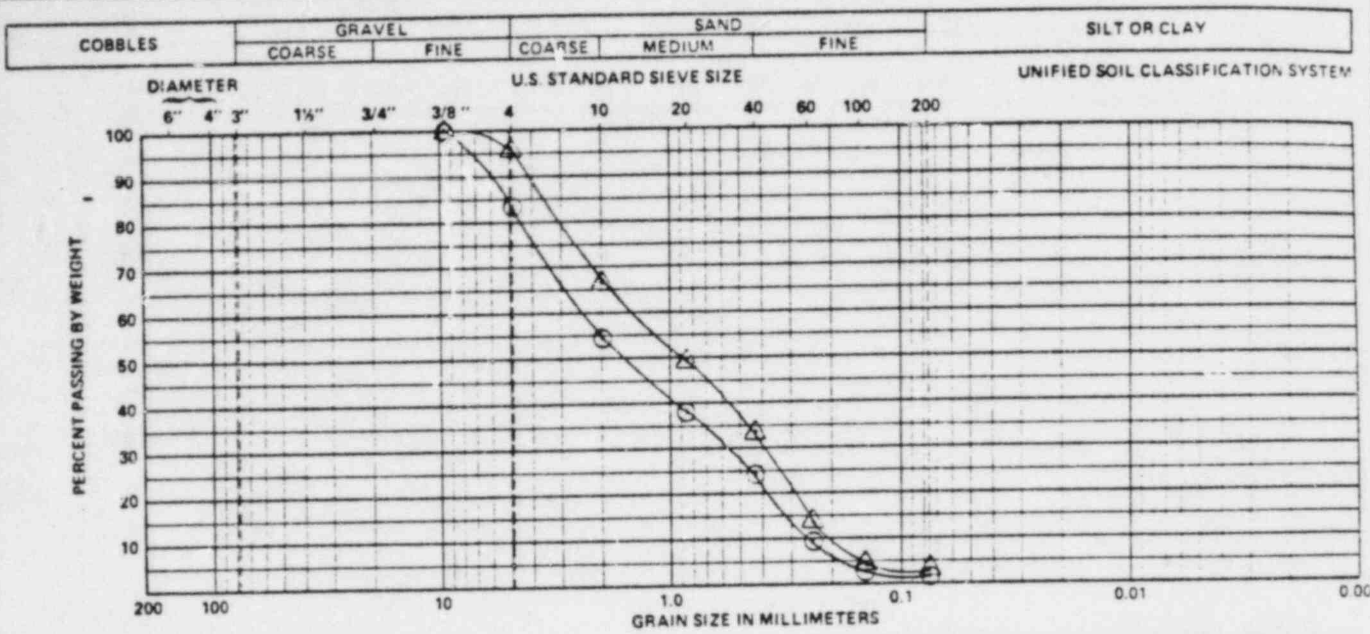
Fig. A-1 (4/7)

CHECKED BY [illegible] 30 JUNE 81

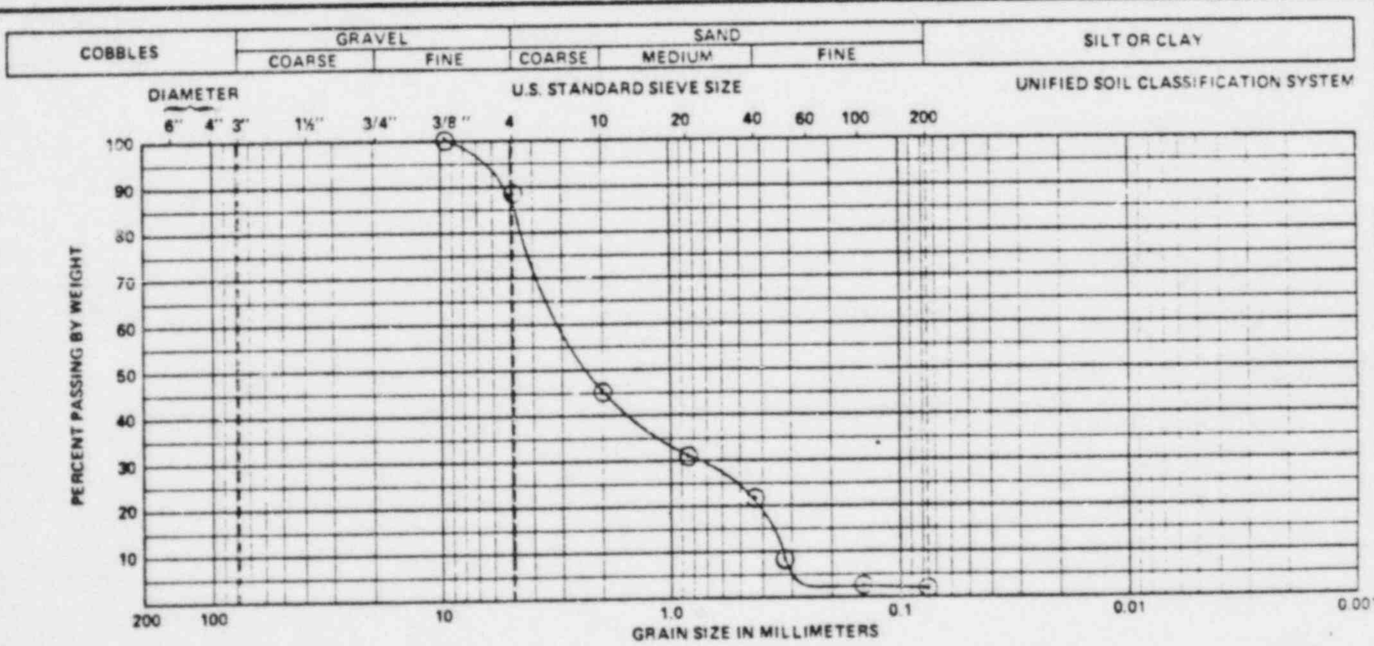
REVIEWED BY WCC [illegible] (9/80)

PREPARED BY BIRCHSTEIN [illegible] Jim

PARTICLE-SIZE DISTRIBUTION



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE B	S-12-A	20.1	⊙	SP, tan, c. to f. SAND, some f. gravel, tr. silt	6.0		
COE B	S-12-C	21.0	△	SP, tan, c. to f. SAND, tr. f. gravel, tr silt	10.9		



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE B	S-13	22.8	⊙	SP, brown, c. to m SAND, some f. gravel, some f. sand, tr. silt	10.2		

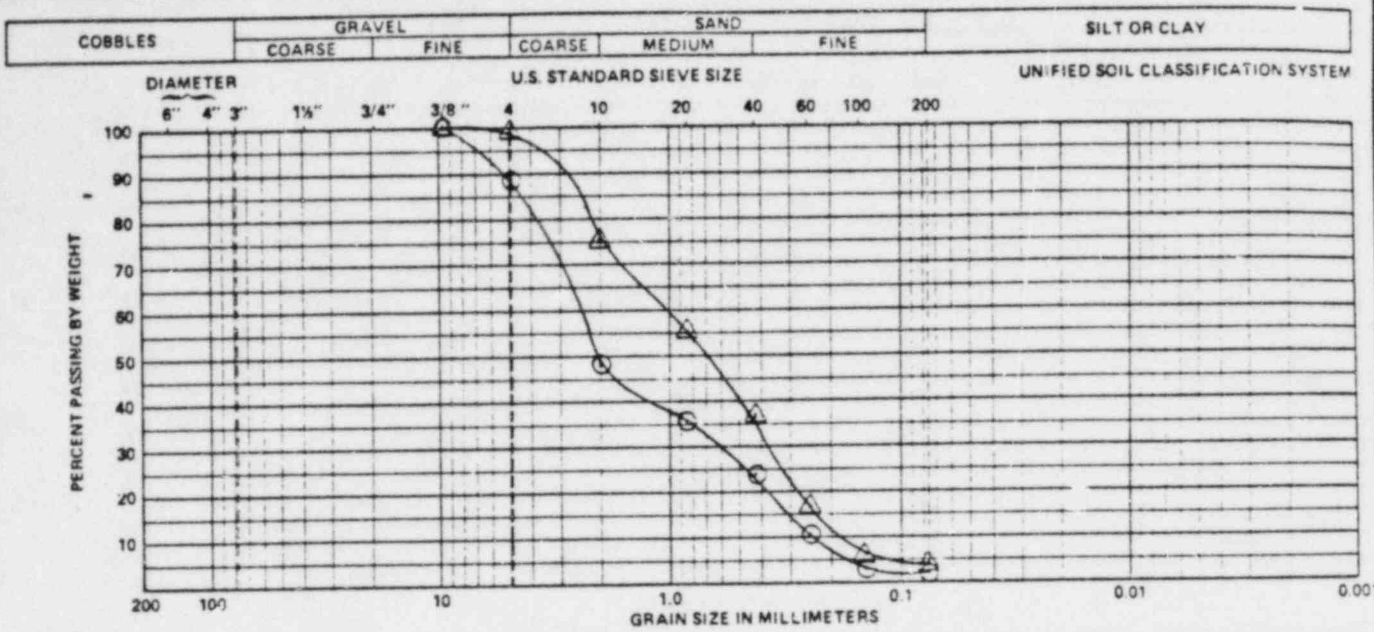
Fig. A-1 (5/7)

CHECKED BY [illegible] 30 JUNE 81

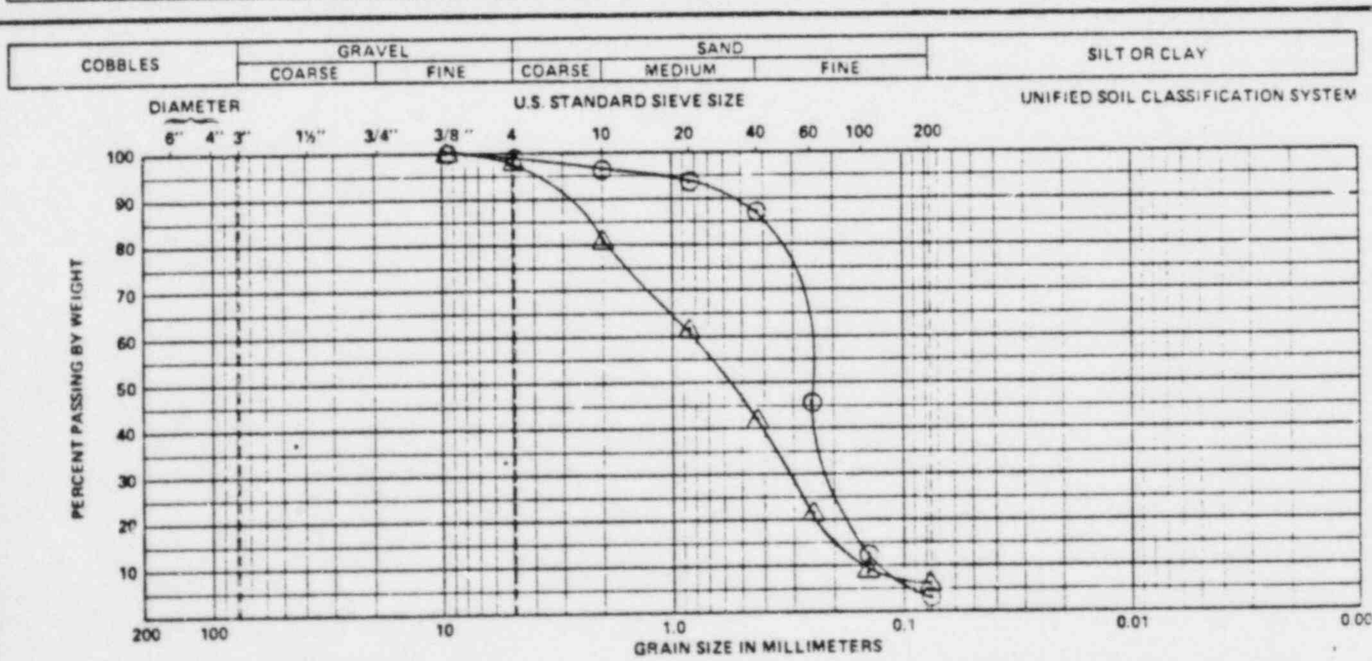
REVIEWED BY: WCC LR-101 (9/80)

PROJECT NO. 31C[illegible] T3 DRAWN BY: PDE

PARTICLE-SIZE DISTRIBUTION



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE 8	S-14-A	22.15	⊙	SP, tan, c. to m. SAND, some f. gravel some f. sand, tr. silt	8.2		
COE 8	S-14-D	26.6	△	SP, tan, c. to f. SAND, tr. f. gravel tr. silt	14.9		



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE 8	S-15-A	27.5	⊙	SP, tan, f. SAND, some f. gravel to m. sand, tr. silt	13.4		
COE 8	S-15-B	28.0	△	SP-SM, tan, c. to f. SAND, tr. f. gravel, tr. silt	10.2		

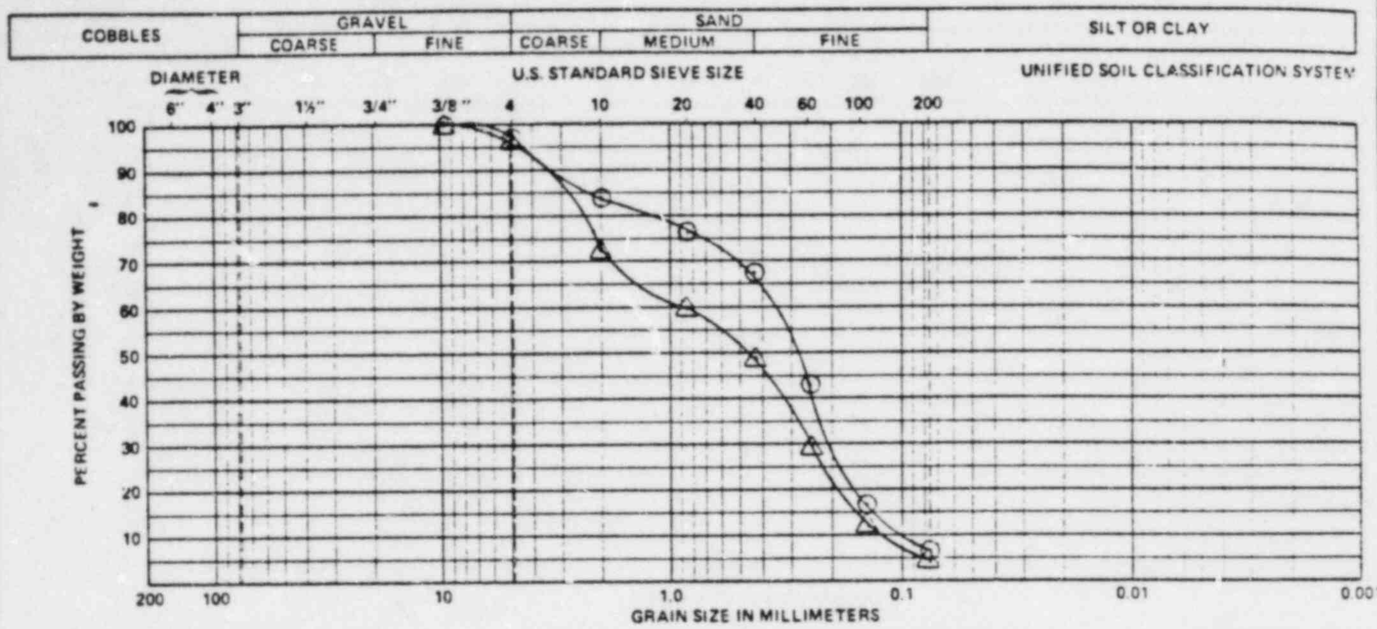
Fig. A-1 (6/7)

CHECKED BY MB 30 June 81

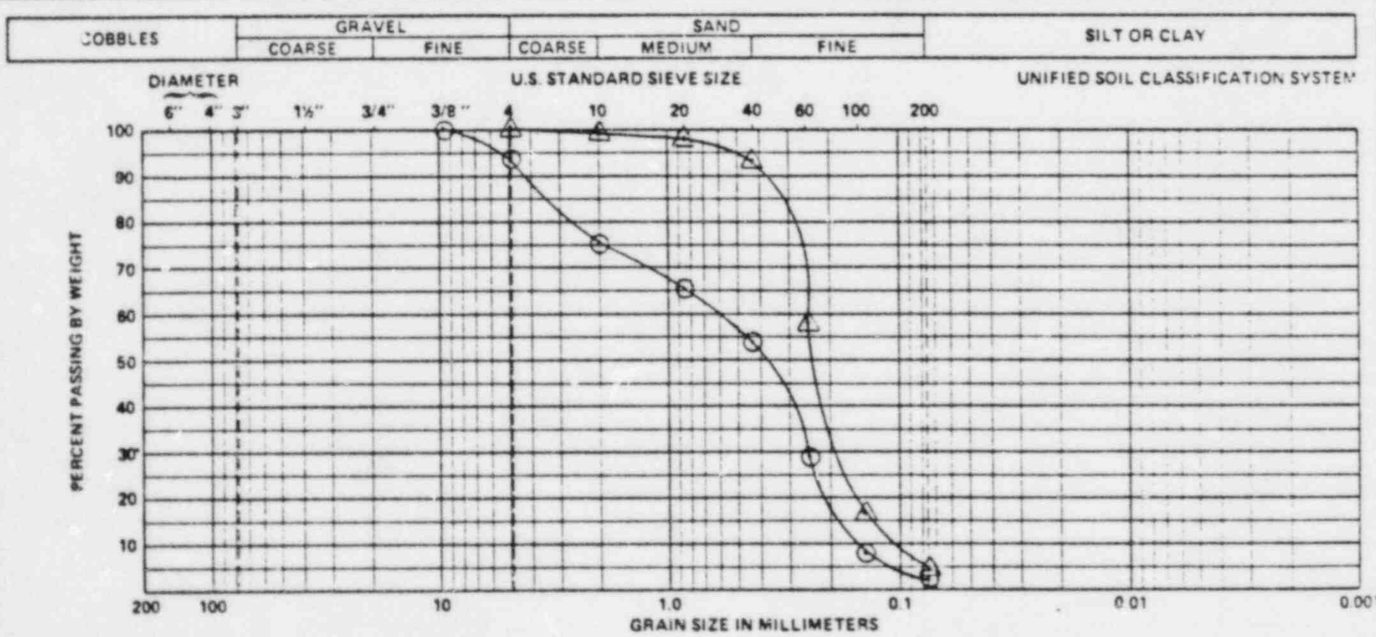
REVIEWED BY WCC LR-101 (9/80)

PROJECT NO. 81C0055 73 DRAWN BY: PMC

PARTICLE-SIZE DISTRIBUTION



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE B	S-16-B	30.6	⊙	SP-SM, tan, c. to f. SAND, tr. f. gravel tr. silt	12.3		
COE B	S-16-C	31.1	△	SP, tan, c. to f. SAND, tr. f. gravel, tr. silt	8.6		

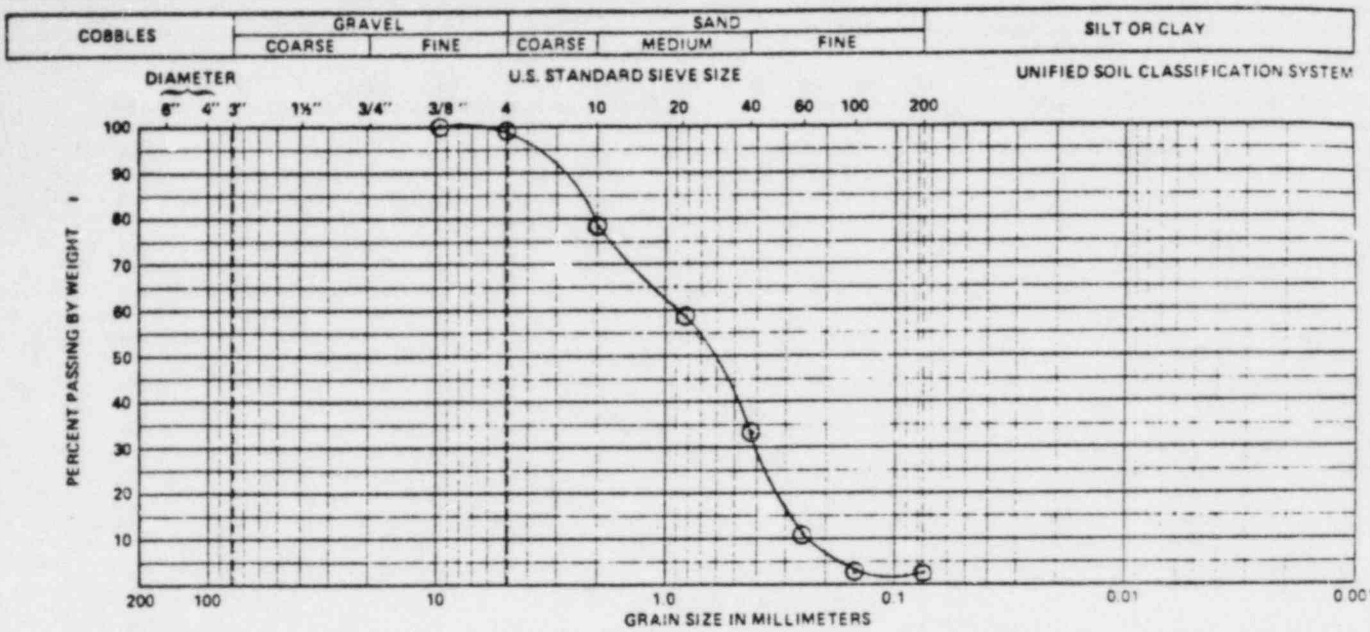


BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE B	S-17-A	32.7	⊙	SP, tan, c. to f. SAND, tr. f. gravel, tr. silt	9.0		
COE B	S-17-C	33.6	△	SP, tan, f. SAND, tr. f. gravel to m. sand, tr. silt	16.8		

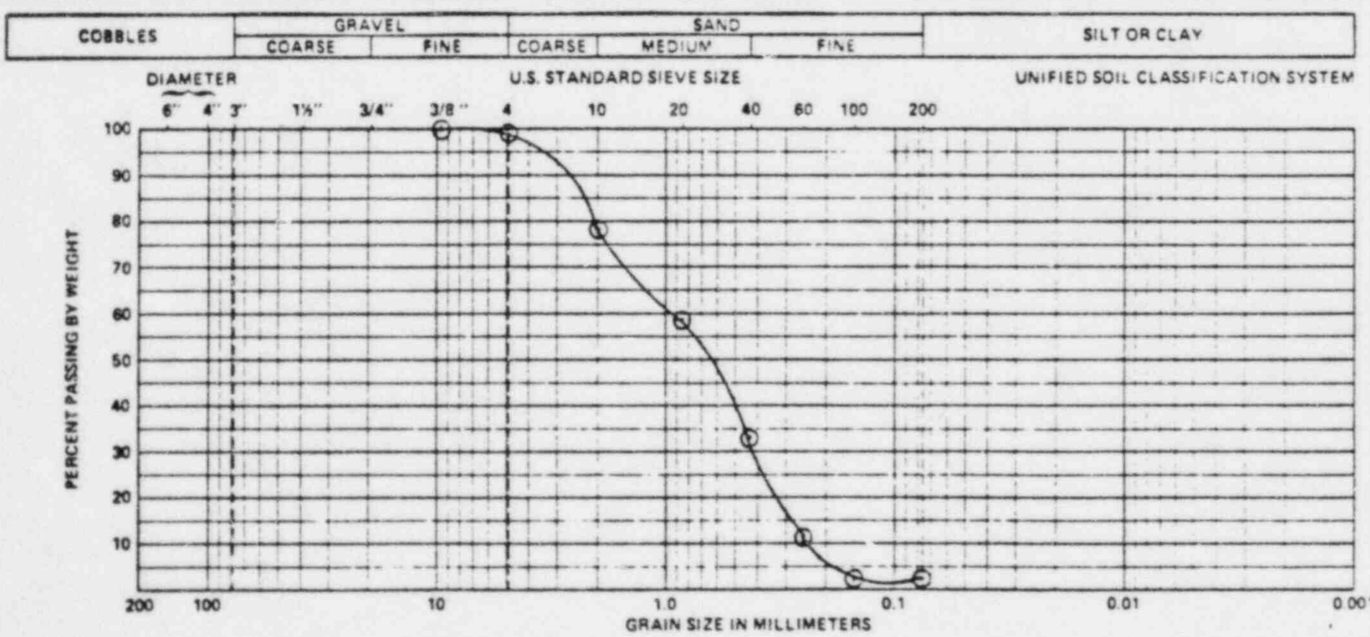
Fig. A-1 (7/7)

PROJECT NO. 81C4055 73 DRAWN BY: PRF REVIEWED BY: WCC LR-101 (9/80) CHECKED BY MB 30 JUNE 81

PARTICLE-SIZE DISTRIBUTION



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE 10	S-1B	6.8	⊙	SP, tan c. to f. SAND, tr. f gravel, tr. silt	3.7		



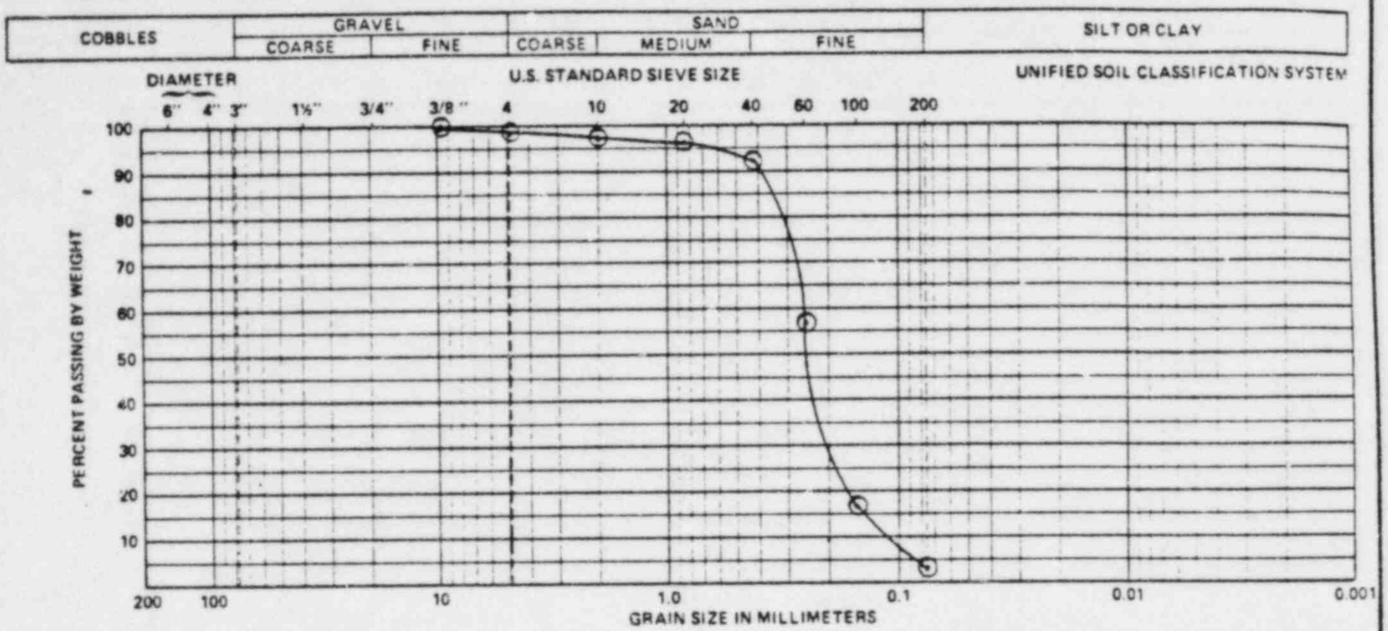
BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE10	S-2-A	8.3	⊙	SP, tan, c. to f. SAND, tr f gravel, tr. silt	11.2		

Fig. A-2 (1/3)

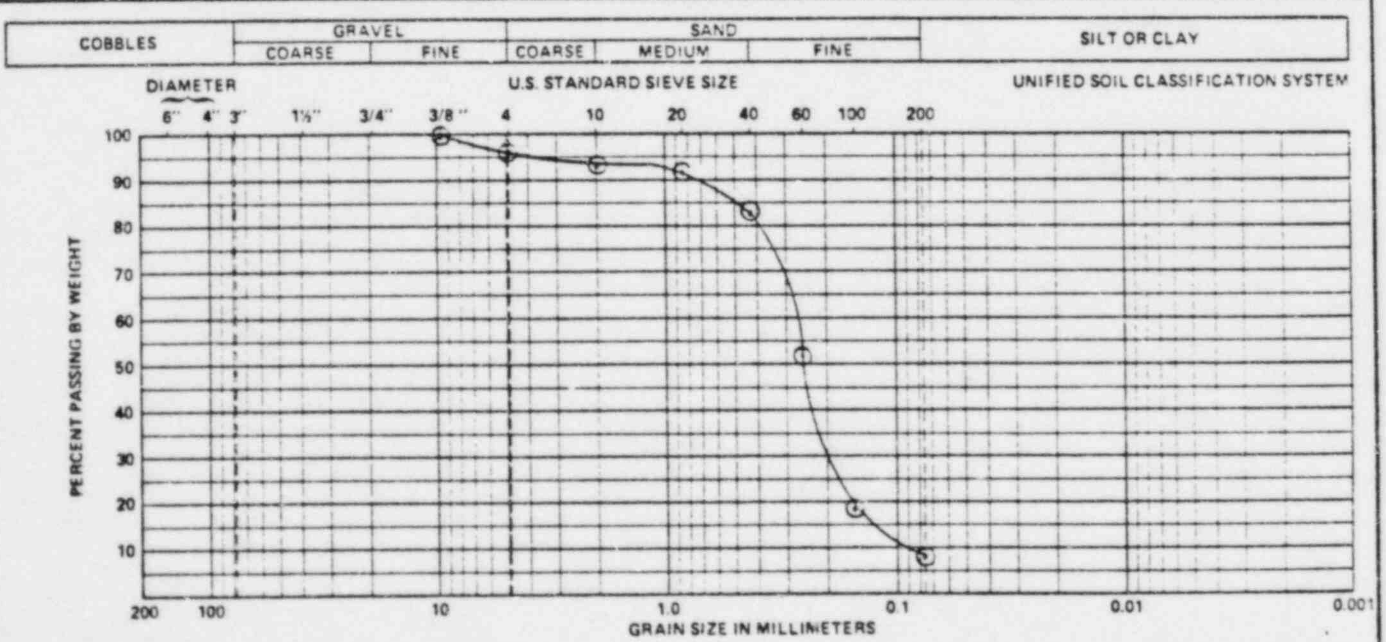
CHK'D BY RU [illegible] 20 JUNE 81

PROJECT NO: 81C4055T3 DRAWN BY: RER

PARTICLE-SIZE DISTRIBUTION



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE-10	S-8-B	24.2	⊙	SP, tan, f SAND, tr. f gravel to m. sand, tr silt, with occ gray sandy silty CLAY (CL)	18.3		

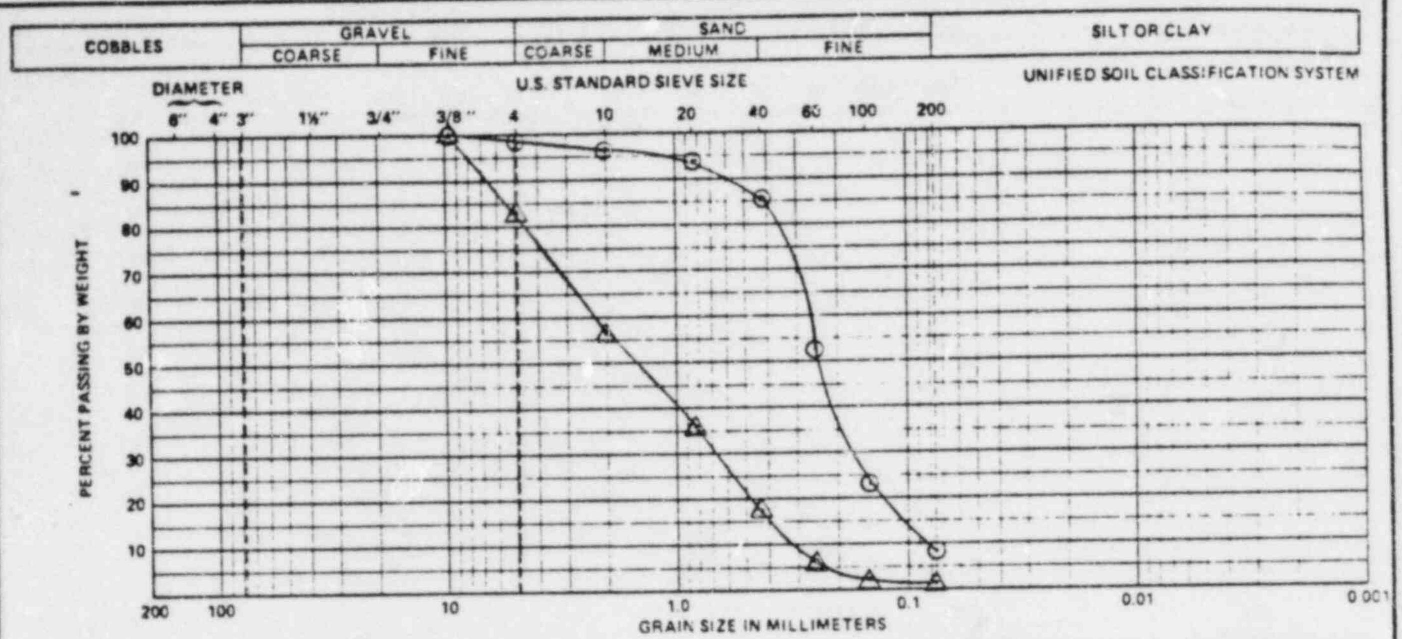


BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE10	S-9-B	26.3	⊙	SP-SM, tan, m. to f. SAND, tr. f gravel to c sand, tr. silt	10.7		

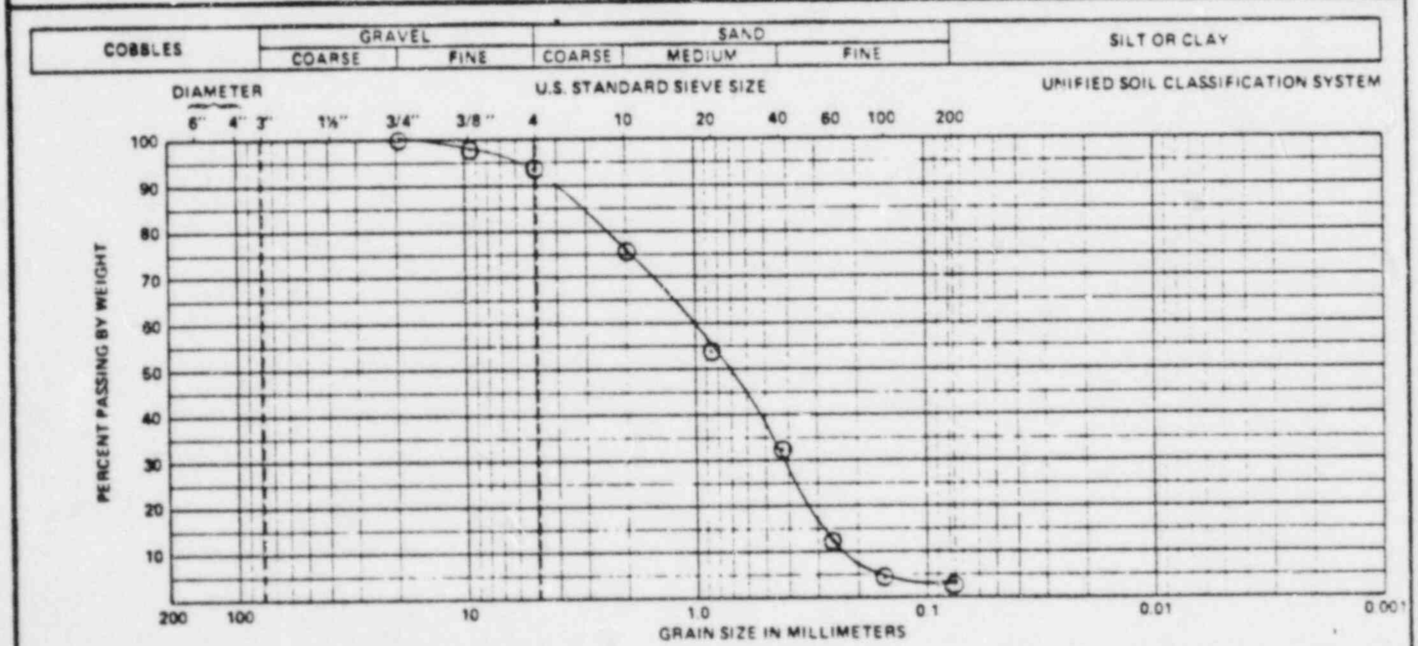
Fig. A-2 (2/3)

PROJECT NO. 81C… DRAWN BY: … REVIEWED BY WCC LR-101 (9/80) Checked by: DPE 7/6/81

PARTICLE-SIZE DISTRIBUTION



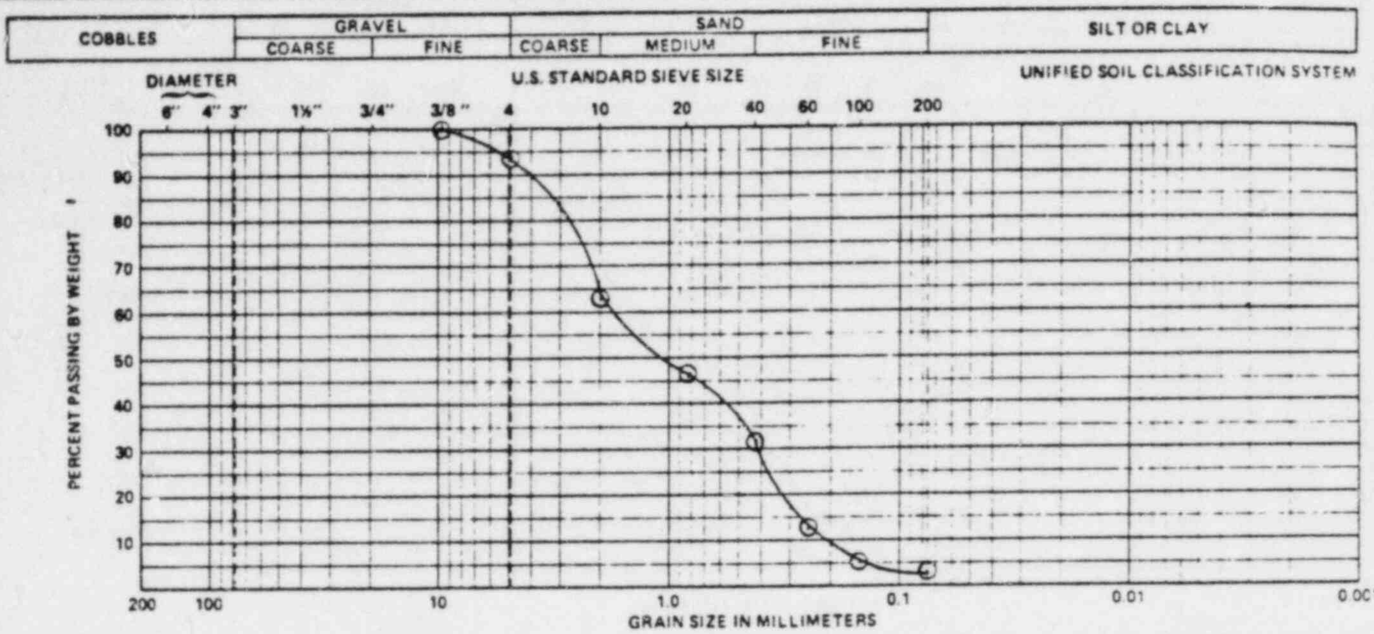
BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE 10	S-10-A	28.8	⊙	SP-SM brown, c. to f. SAND, tr. f. gravel, tr. silt, with occ. c. gravel	15.7		
COE 10	S-10-C	29.0	△	SP brown, c. to f. SAND, tr. f. gravel, tr. silt	9.1		



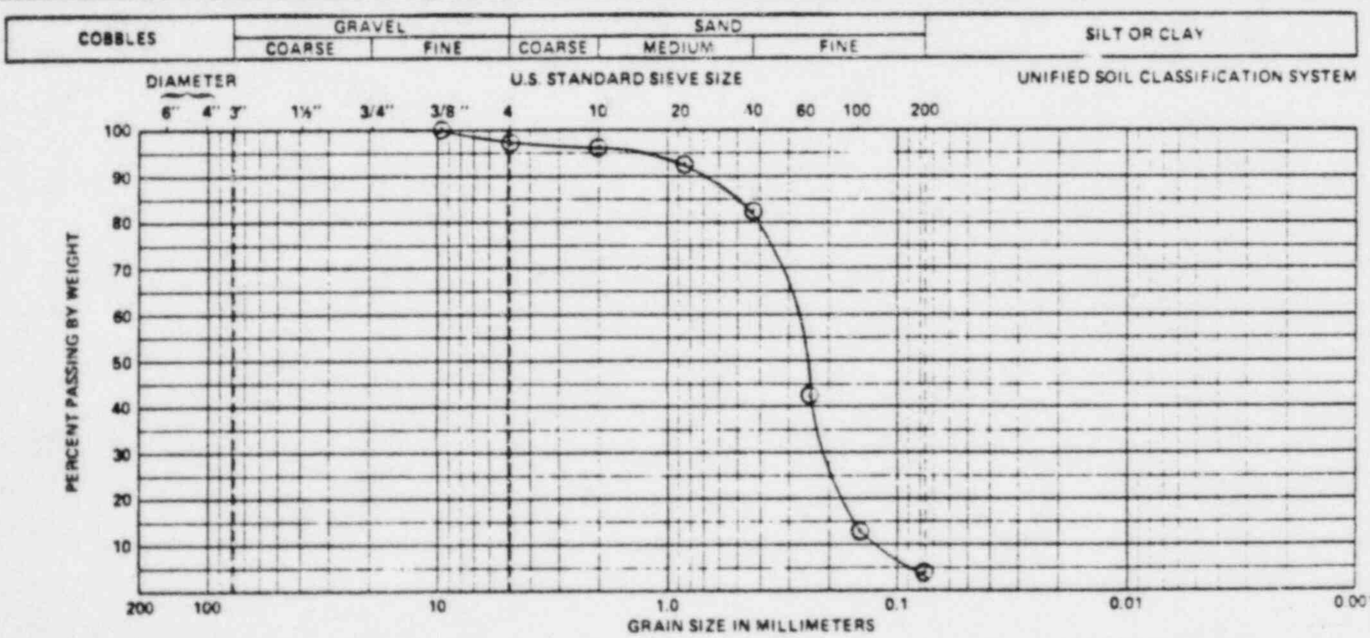
BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE 10	S-11E	31.2	⊙	SP, gray-brown, c. to f. SAND, tr. f. gravel, tr. silt			

Fig. A-2 (3/3)

PARTICLE-SIZE DISTRIBUTION



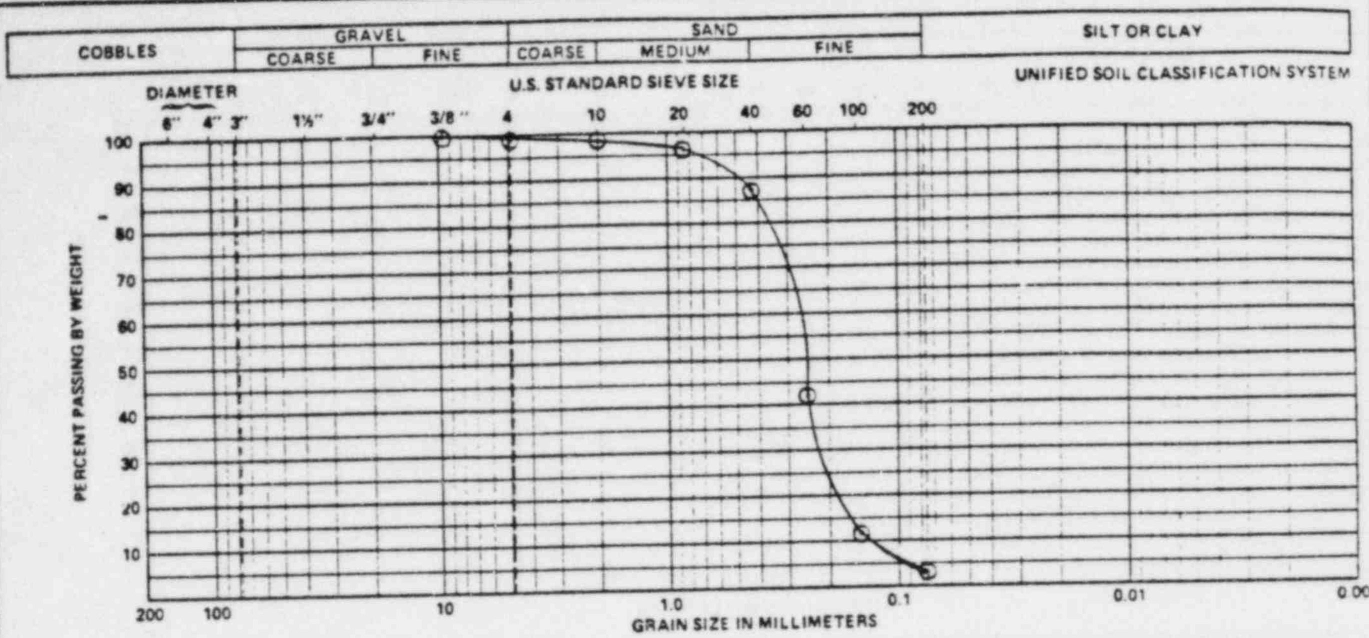
BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE-11	S2-B	8.6	⊙	SP, gray-brown, c. to f. SAND, tr. f. gravel, tr. silt	10.4		



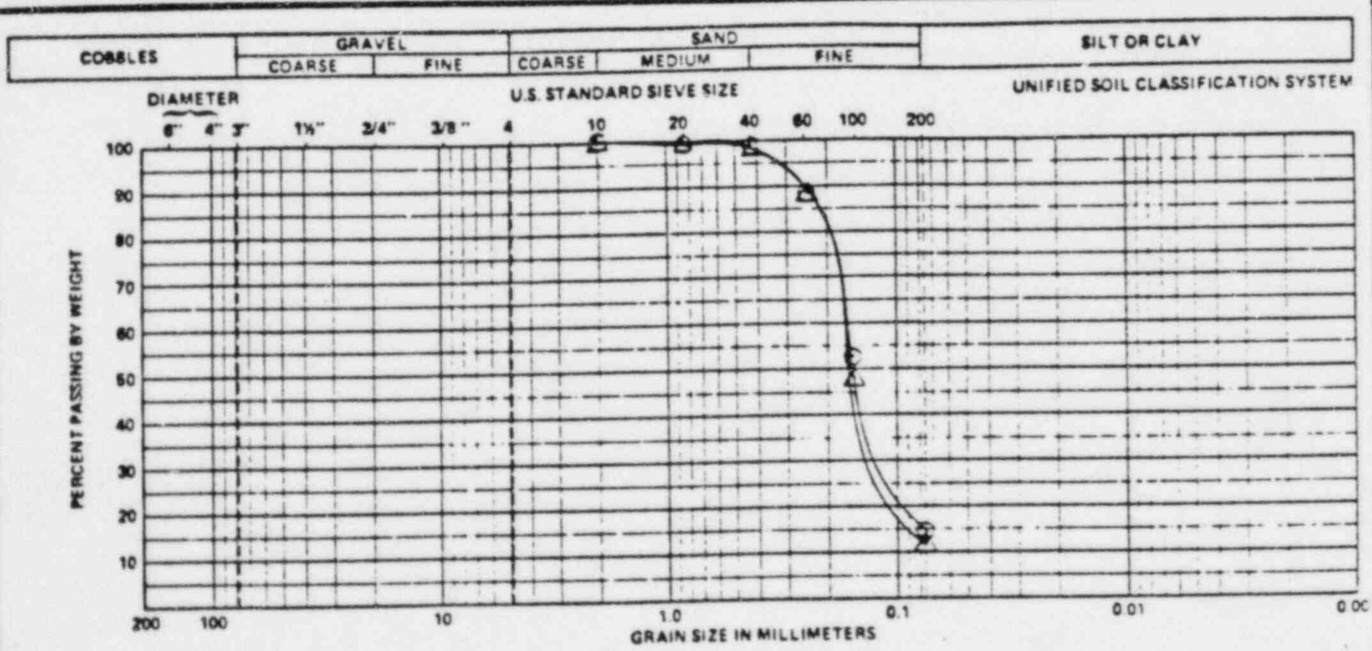
BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE 11	S-10-A	21.1	⊙	SP-SM, tan, m. to f. SAND, tr. f. gravel to c. sand, tr. silt	12.3		

Fig. A-3 (1/2)

PARTICLE-SIZE DISTRIBUTION



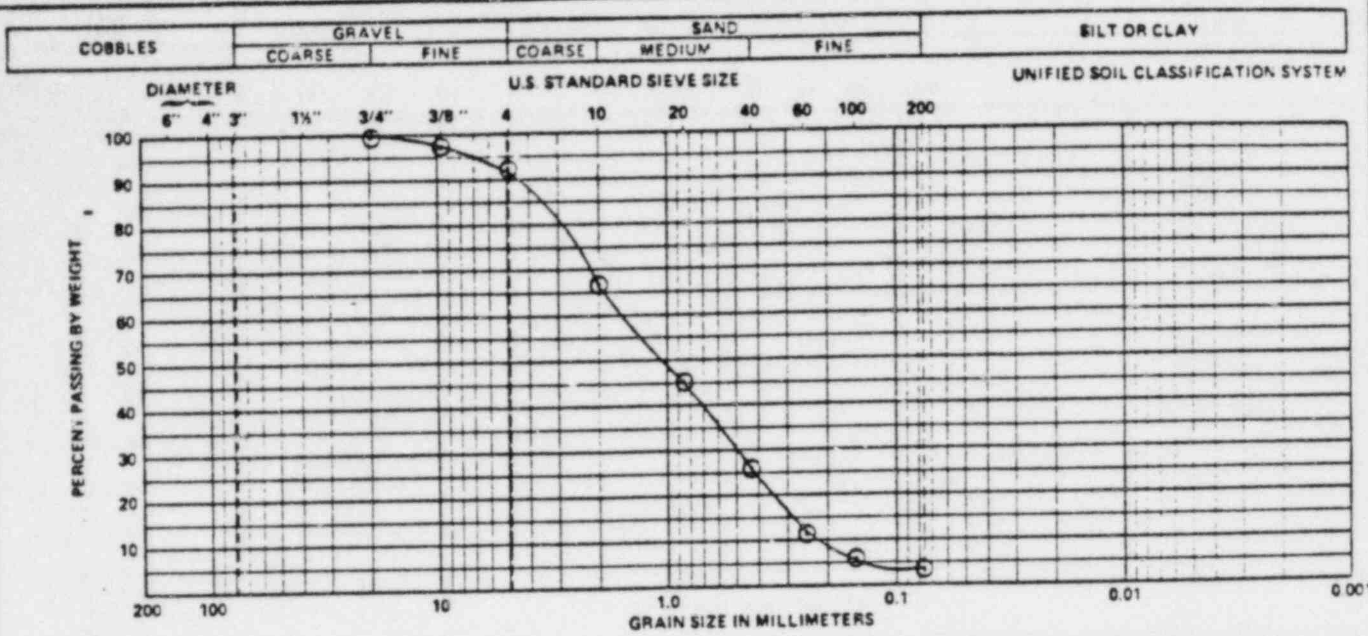
BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE 11	S-11-B	24.6	⊙	SP, tan, f. SAND, some f. gravel to m. sand, tr. silt	8.7		



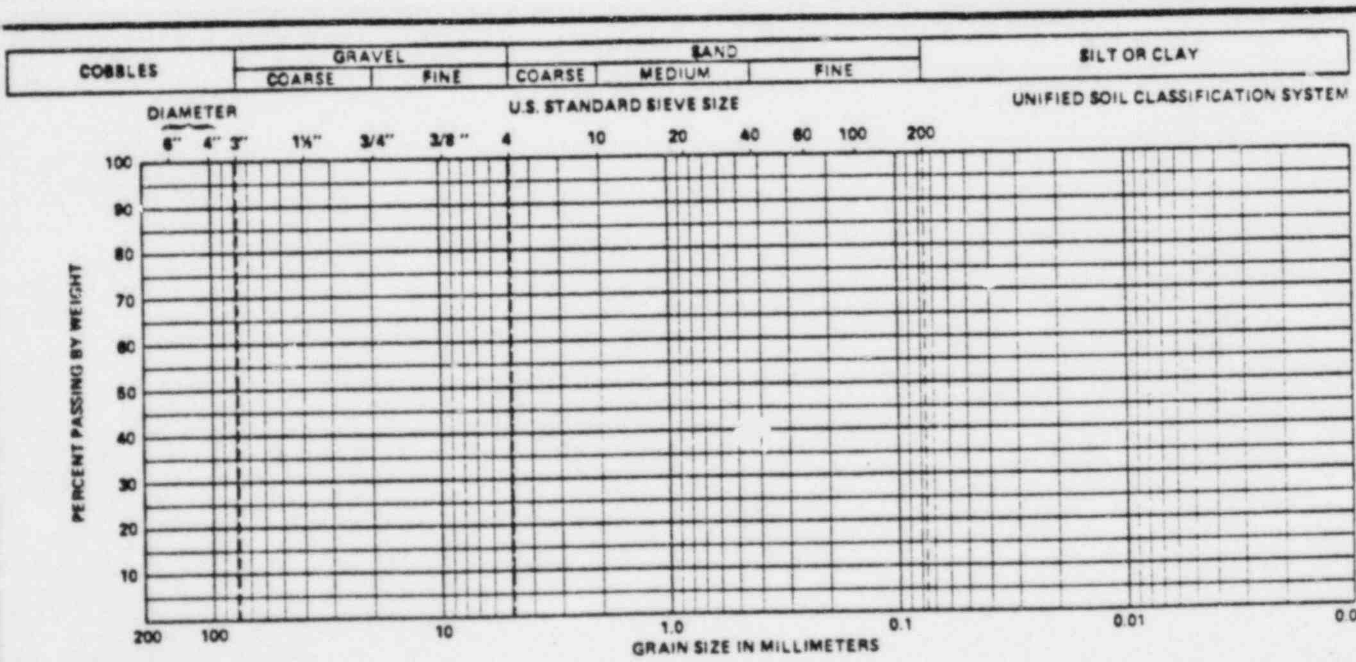
BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE 11	S-14A	30.7	⊙	SM, tan, f. SAND, tr. m. sand, some silt	16.0		
COE 11	S-14C	31.7	△	SP-SM, tan, f. SAND, tr. m. sand, some silt	18.6		

Fig. A-3 (2/2)

PARTICLE-SIZE DISTRIBUTION



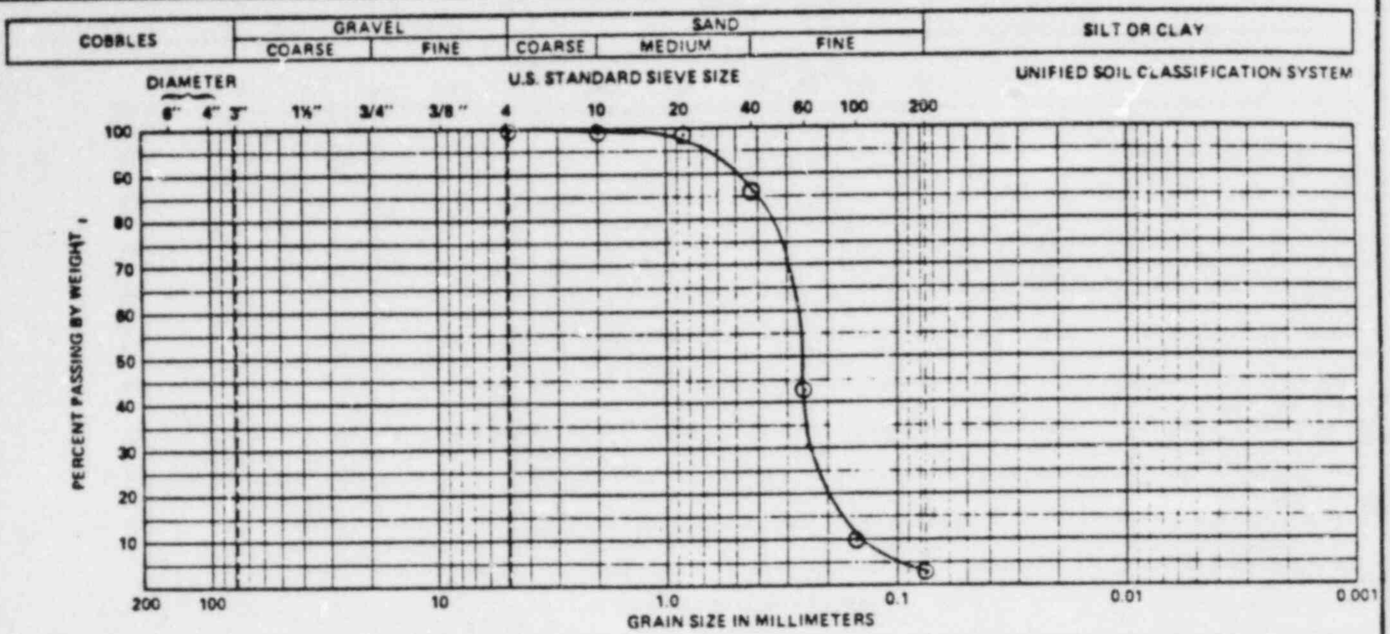
BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE-12	S-5A	11.4	⊙	SP, tan, c to f SAND, tr. f gravel, tr. silt	11.3		



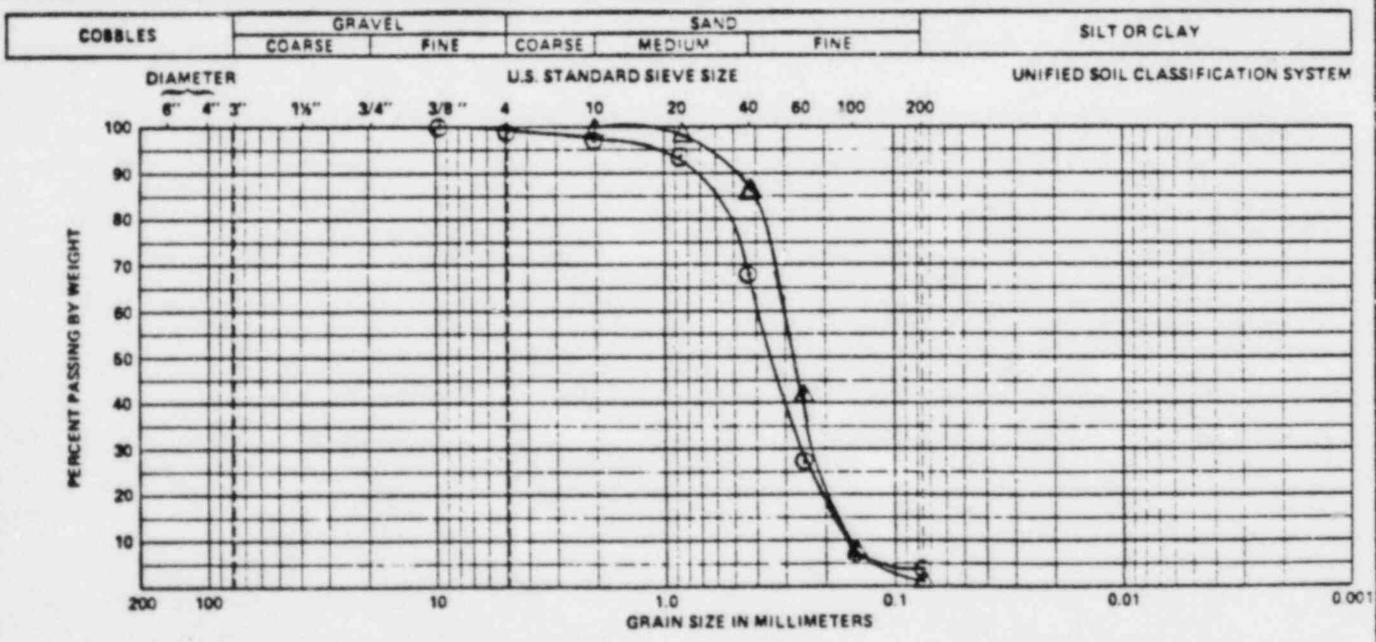
BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)

Fig. A-4 (1/1)

PARTICLE-SIZE DISTRIBUTION



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE13R	S16C	30.0	⊙	SP, tan, fine SAND, tr m. sand, tr. silt	18.4		



BORING	SAMPLE	DEPTH (ft)	SYMBOL	CLASSIFICATION	w (%)	w_L (%)	w_P (%)
COE13R	S-17-A	31.3	⊙	SP, brown, m. to f. SAND, tr. f. gravel to c sand, tr. silt	7.0		
COE13R	S-17-C	32.4	△	SP, brown, f. SAND, some m. sand, tr. silt	16.5		

Fig. A-5 (1/1)

PROJECT NO. 81C4085T3 DRAWN BY: RPA

APPENDIX B

Empirical Relationships for Estimating Densities (Void Ratios)

APPENDIX B
EMPIRICAL RELATIONSHIPS FOR ESTIMATING DENSITIES (VOID RATIOS)

B.1 INTRODUCTION

Maximum density (γ_{max}) and minimum density of (γ_{min}) of granular soils may be estimated from empirical relationships between density and gradation from Burmister (1962), Alpan (1976), and Johnston (1973). Similarly, maximum void ratio (e_{max}) and minimum void ratio (e_{min}) may be estimated from empirical relationships by Youd (1973). Brief descriptions of these methods and illustrations of these relationships, are presented herein.

B.2 COEFFICIENT OF UNIFORMITY

The empirical relationships of Alpan, Johnston, and Youd utilize the coefficient of uniformity (C_u) as the parameter to correlate γ_{max} and γ_{min} or e_{max} and e_{min} with the gradation characteristics of the soil. C_u is calculated from the following equation:

$$C_u = \frac{D_{60}}{D_{10}}$$

where:

D_{10} = Particle-size diameter in millimeters corresponding to 10% passing on the cumulative particle-size distribution curve;

D_{60} = Particle-size diameter in millimeters corresponding to 60% passing on the cumulative particle-size distribution curve.

B.3 DETERMINATION OF DENSITIES

B.3.1 Burmister (1962)

For a given particle-size distribution curve, classify the shape of the curve as S ("s" shaped), L (linear), C (concave upward), E (convex upward), D (irregular), or a combination of these symbols. Fit a mean slope line to the particle-size distribution curve and calculate the effective grain size range (C_r) using the following equation:

$$C_r = 2 \text{ Log}_{10} (D_{100}/D_o)$$

where: D_{100} and D_o are particle-size diameters at the terminal points of the mean slope line corresponding to 100% and 0% passing on the particle-size distribution, respectively.

To determine γ_{max}, locate the value of C_r on the horizontal axis of Fig. B-1(a) and project it vertically to intersect the appropriate curve corresponding to the shape classification of the particle-size distribution curve. Project this intersection horizontally and read the value of γ_{max} from the vertical axis.

To determine γ_{min}, first determine the effective grain size (D_{50}) of the particle-size distribution curve (D_{50} is the particle-size diameter in millimeters corresponding to 50% passing on the cumulative particle-size distribution curve). Then locate the value of D_{50} on the horizontal axis of Fig. B-1(b) and project it vertically to intersect the curve corresponding to the appropriate value of C_r. Project this intersection horizontally and read the value of γ_{min} from the vertical axis.

B.3.2 Alpan (1976)

For a given particle-size distribution curve, locate the value of C_u on the horizontal axis of Fig. B-2 and project it vertically to intersect the lines D_r = 0% and D_r = 100%, corresponding to γ_{min} and γ_{max}, respectively. Project these intersections horizontally to the left and read the values of γ_{max} and γ_{min} from the vertical axis. These values are expressed in t/m^3 (metric tons per cubic meter) and can be converted to lb/ft^3 (pounds per cubic foot) by multiplying by 62.4.

B.3.3 Johnston (1973)

For a given particle-size distribution curve, locate the value of C_u on the horizontal axis of Fig. B-3 and project it vertically to intersect the lines marked "maximum" and "minimum". Project these intersections horizontally and read the values of γ_{max} and γ_{min} from the vertical axis.

B.4 DETERMINATION OF VOID RATIOS

B.4.1 Youd (1973)

For a given particle-size distribution curve, locate the value of C_u on the horizontal axis of Fig. B-4 and project it vertically to intersect the curve for the appropriate mean roundness (R) of the soil. Project these intersections horizontally and read the values of e_{max} and e_{min} from the vertical axis. The void ratios determined in this study were converted to densities using an assumed specific gravity of 2.68.

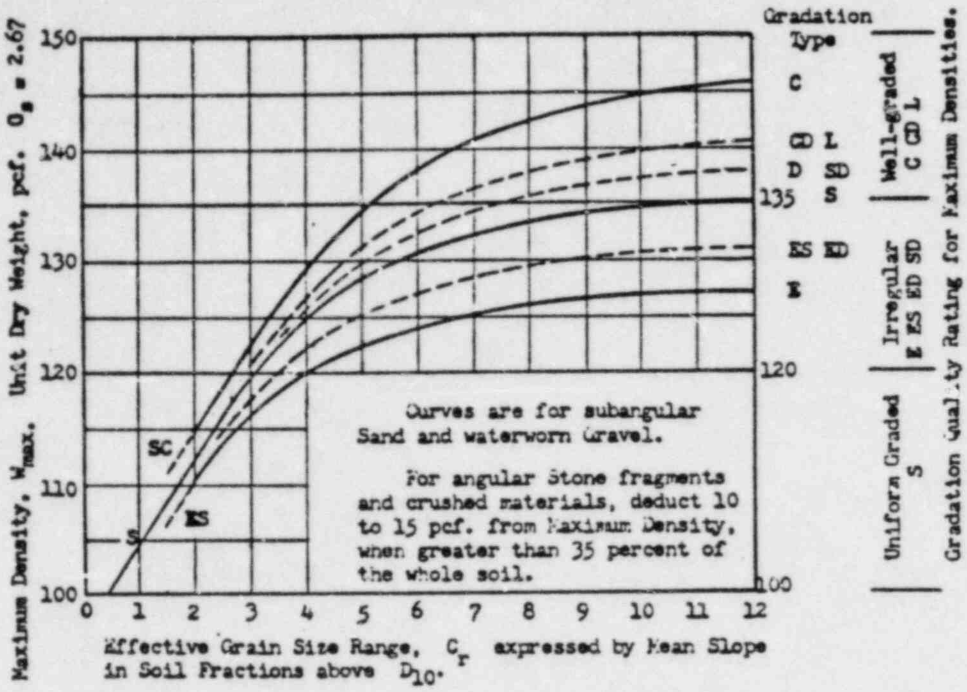


Fig. A

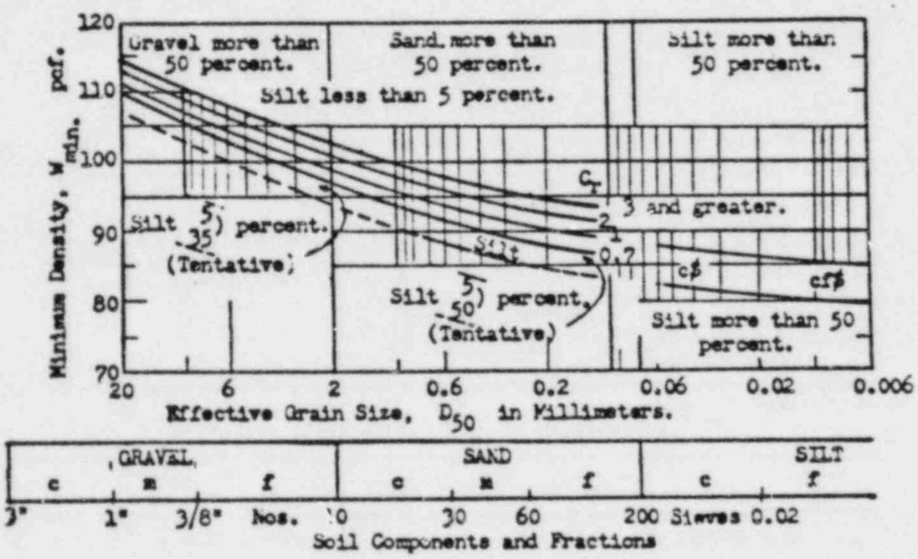
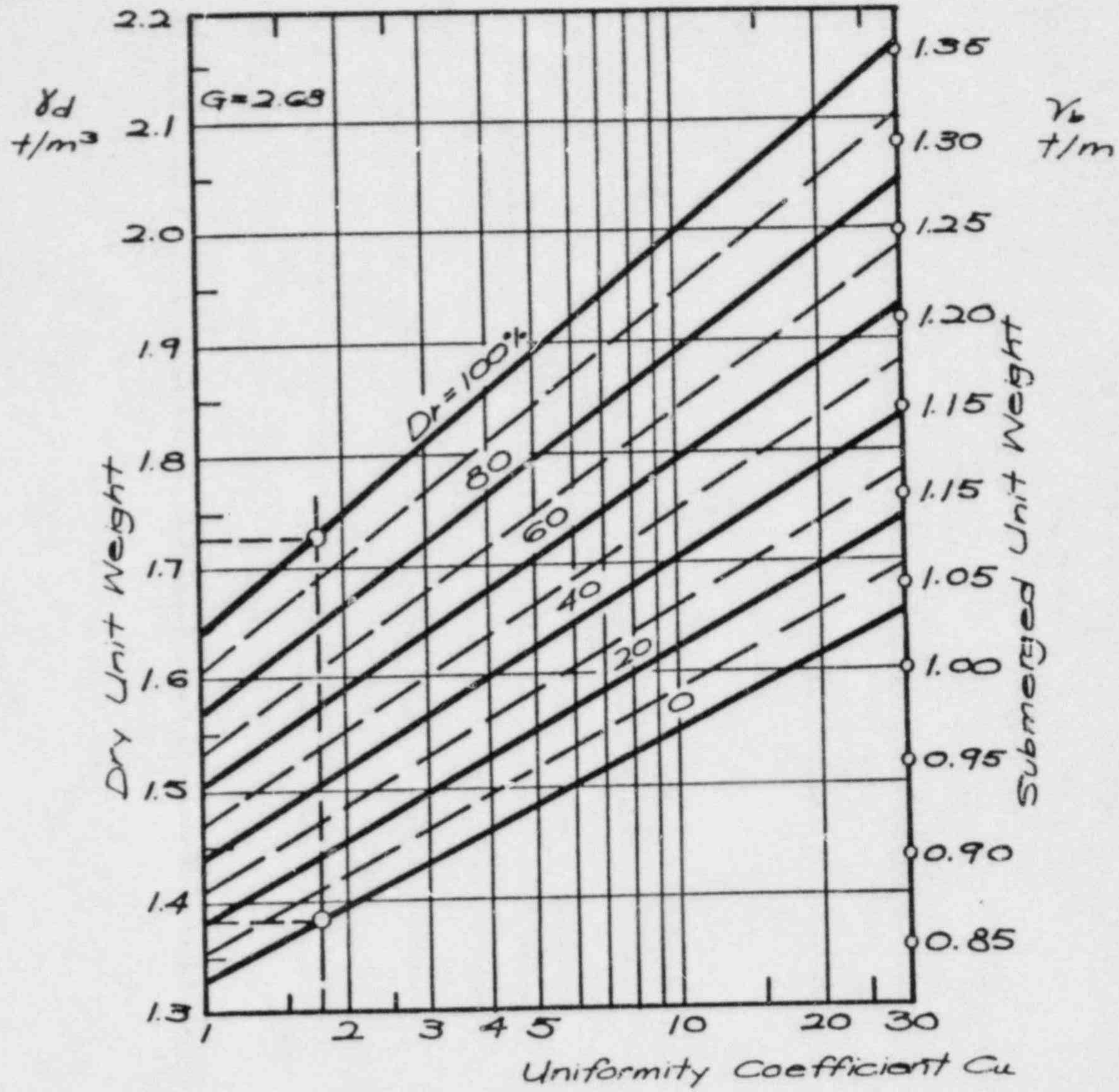
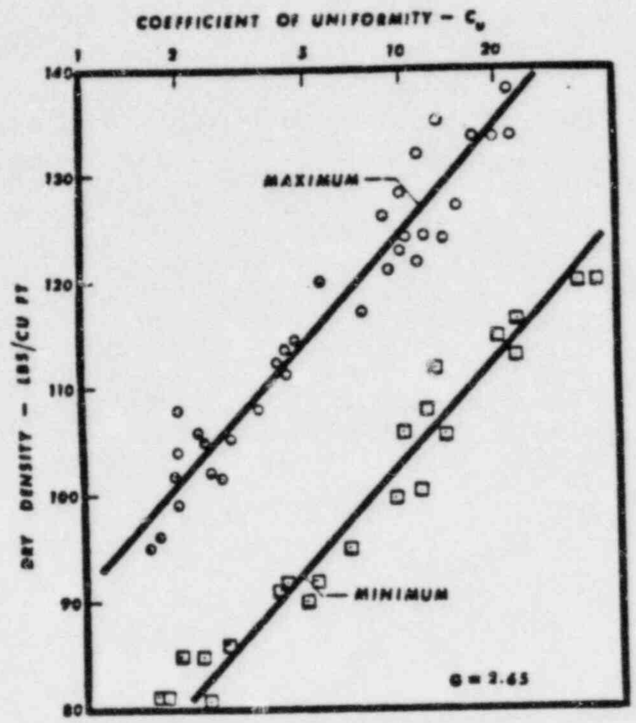


Fig. B

CONSUMERS POWER COMPANY Midland Plant-Units 1&2		
Empirical Relationships for Estimating Densities from Burmister		
81C217-4	24 July 1981	Fig. B-1
Woodward-Clyde Consultants		



CONSUMERS POWER COMPANY Midland Plant-Units 1&2		
Empirical Relationships for Estimating Densities from Alpan		
81C217-4	24 July 1981	Fig. B-2
Woodward-Clyde Consultants		

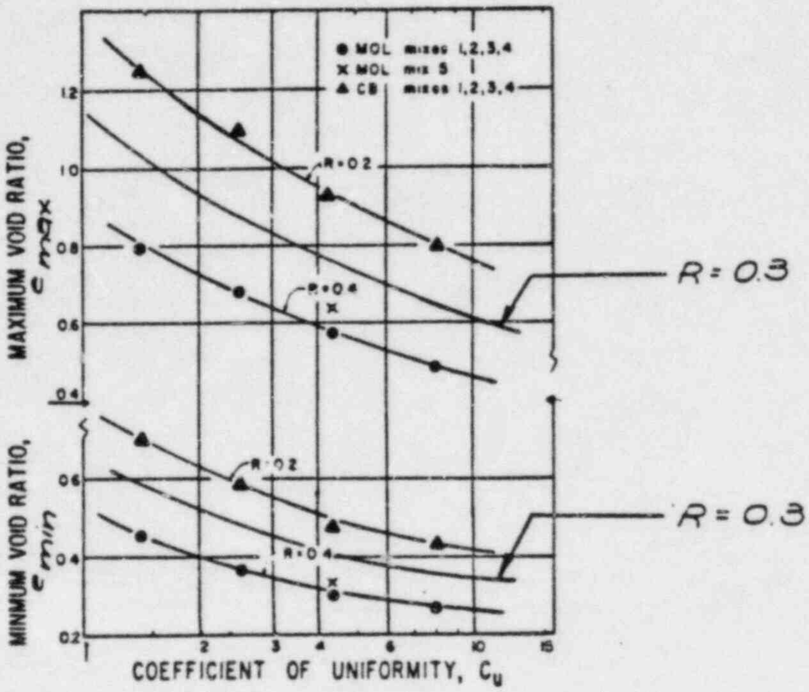


CONSUMERS POWER COMPANY
Midland Plant-Units 1&2

Empirical Relationships for Estimating Densities from Johnston

81C217-4	24 July 1981	Fig. B-3

Woodward-Clyde Consultants



CONSUMERS POWER COMPANY Midland Plant-Units 1&2		
Empirical Relationships for Estimating Void Ratios from Youd		
81C217-4	24 July 1981	Fig. B-4
Woodward-Clyde Consultants		

APPENDIX C

Density and Relative Density Plots

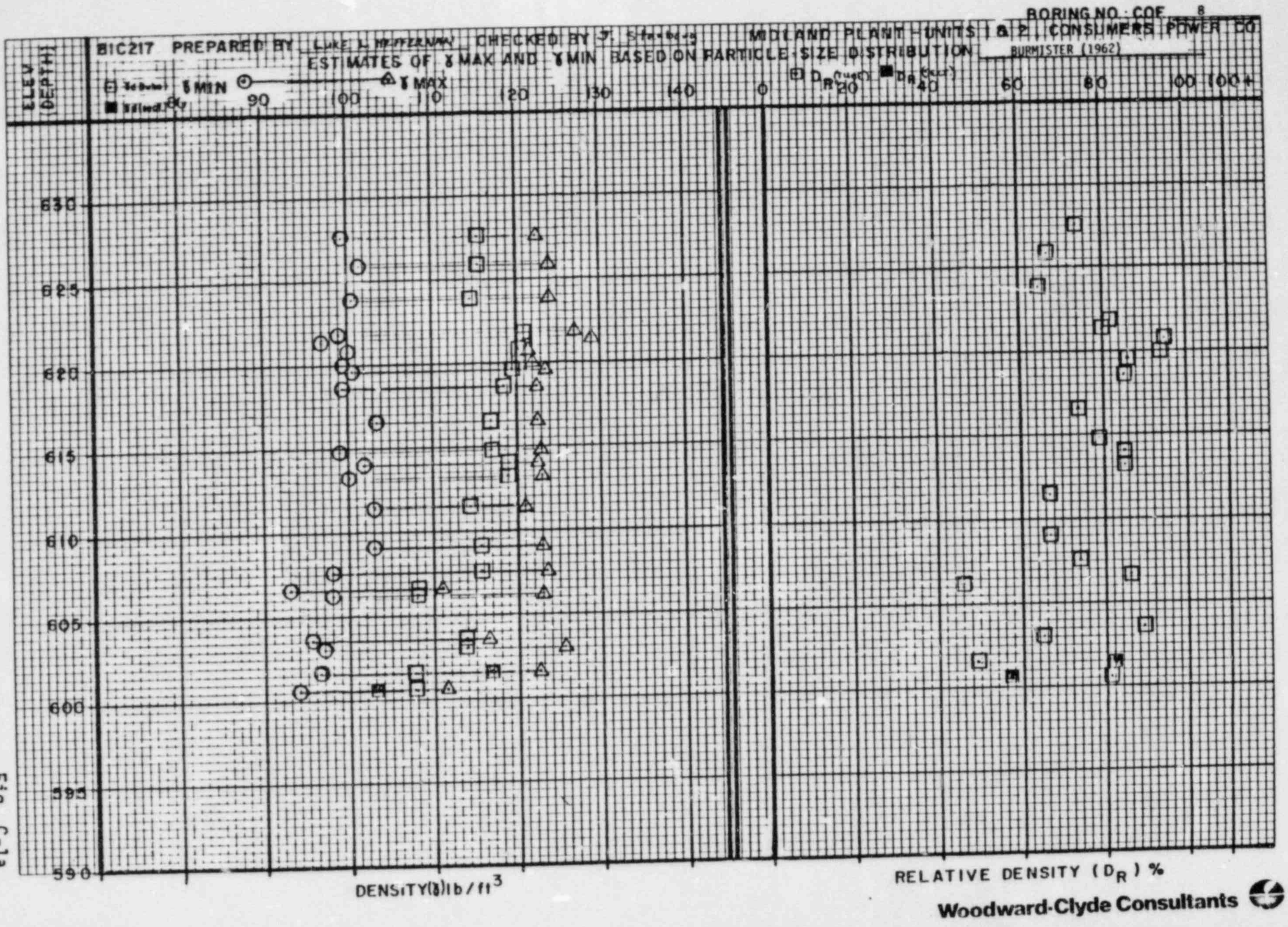

BORING NO. COF 8
BIC217 PREPARED BY Luke L. Huitzenga CHECKED BY J. Strasberg
MIDLAND PLANT - UNITS 1 & 2, CONSUMERS POWER CO.
ESTIMATES OF γ MAX AND γ MIN BASED ON PARTICLE SIZE DISTRIBUTION
BURMISTER (1962)
γ MIN
γ MAX
ELEV (DEPTH)
630
625
620
615
610
605
600
595
590
90
100
110
120
130
140
0
20
40
60
80
100
100+
DENSITY (γ) lb/ft³
RELATIVE DENSITY (D_R) %
Woodward-Clyde Consultants

Fig. C-1a

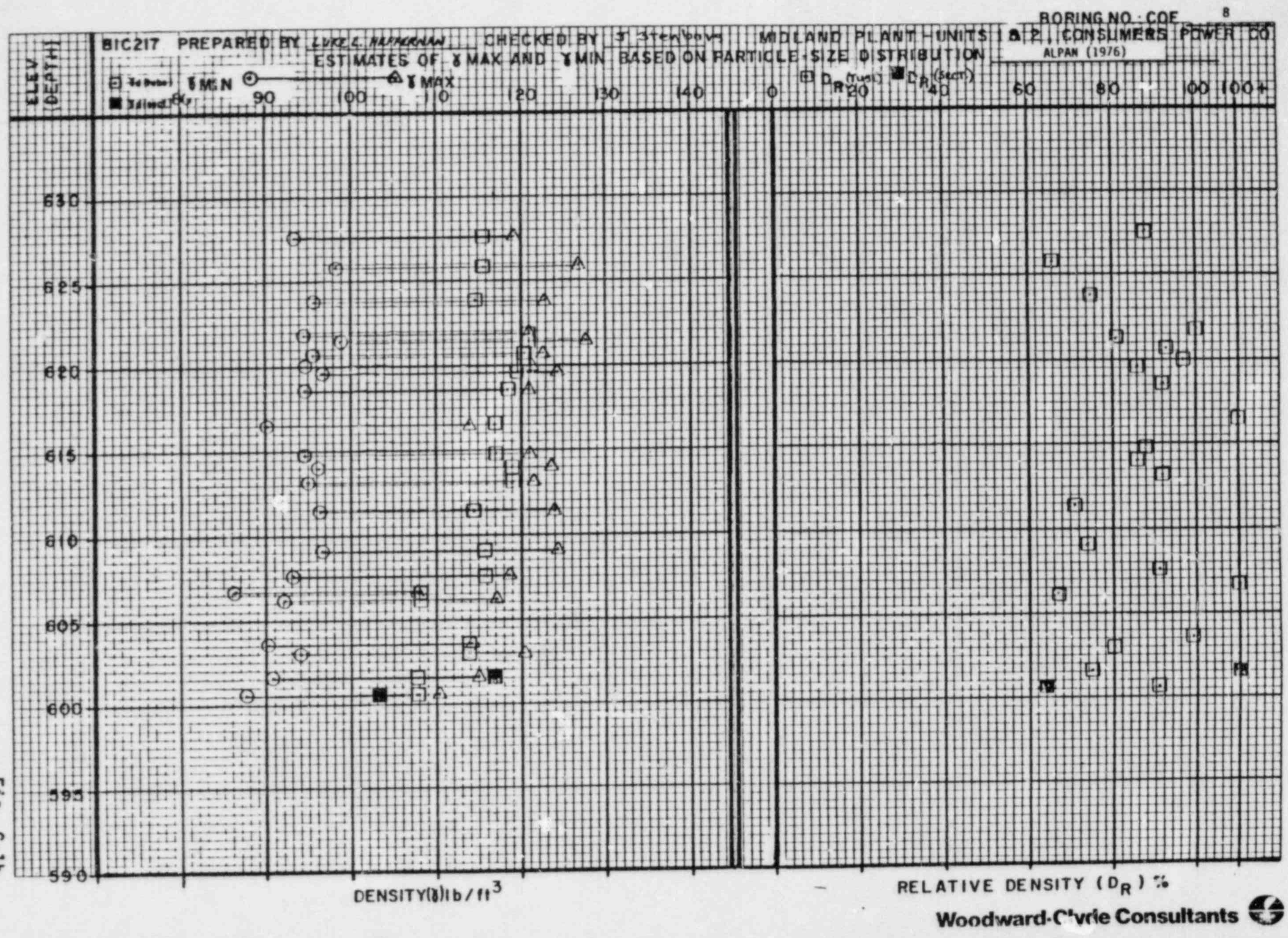

BORING NO.: COE 8
BIC217 PREPARED BY LUKE L. HAFFERMAN CHECKED BY J. Stenbom MIDLAND PLANT - UNITS 1 & 2, CONSUMERS POWER CO.
ESTIMATES OF γMAX AND γMIN BASED ON PARTICLE-SIZE DISTRIBUTION
ALPAN (1976)
γMIN
γMAX
ELEV (DEPTH)
630
625
620
615
610
605
600
595
590
90
100
110
120
130
140
0
20
40
60
80
100
100+
DENSITY(γ) lb/ft^3
RELATIVE DENSITY (D_R) %
Woodward-Clyde Consultants

Fig. C-1b

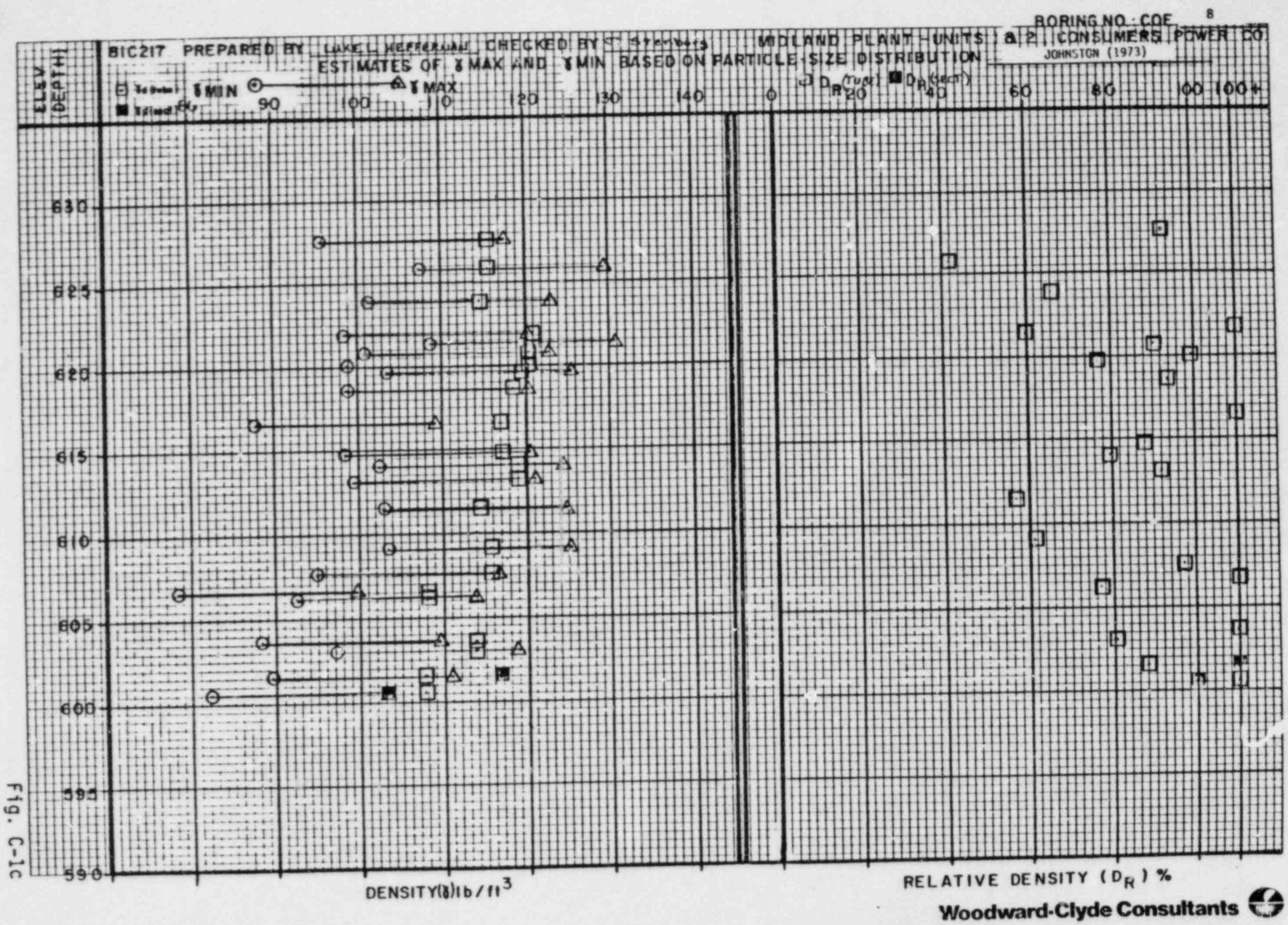

BORING NO. COE 8
BIC217 PREPARED BY Luke L. Hefferman CHECKED BY S. Sternberg MIDLAND PLANT - UNITS 1 & 2, CONSUMERS POWER CO.
ESTIMATES OF γ MAX AND γ MIN BASED ON PARTICLE-SIZE DISTRIBUTION
JOHNSTON (1973)
γ MIN
γ MAX
ELEV (DEPTH)
630
625
620
615
610
605
600
595
590
90
100
110
120
130
140
0
20
40
60
80
100
100+
DENSITY (γ) lb/ft³
RELATIVE DENSITY (D_R) %
Woodward-Clyde Consultants

Fig. C-1c

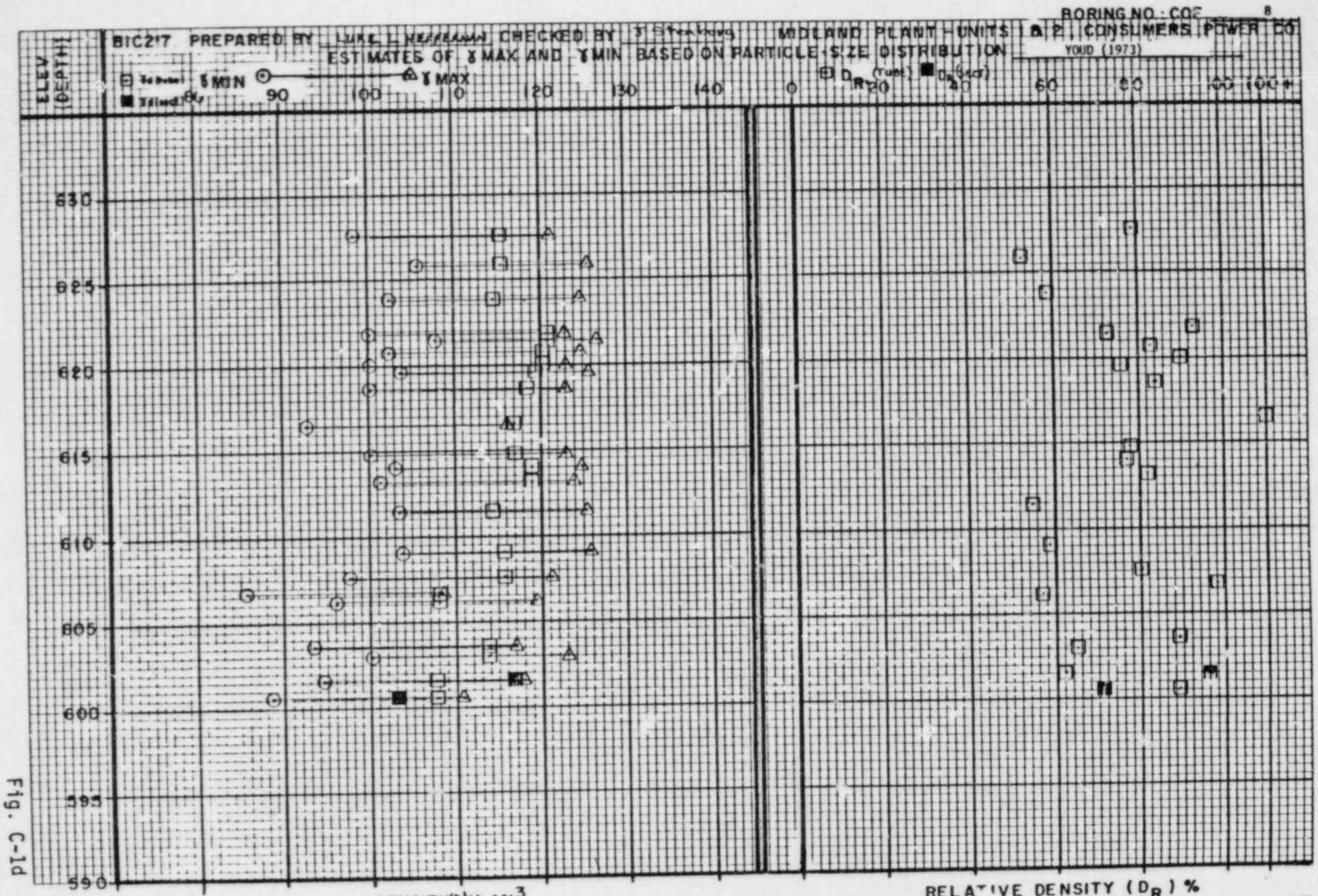
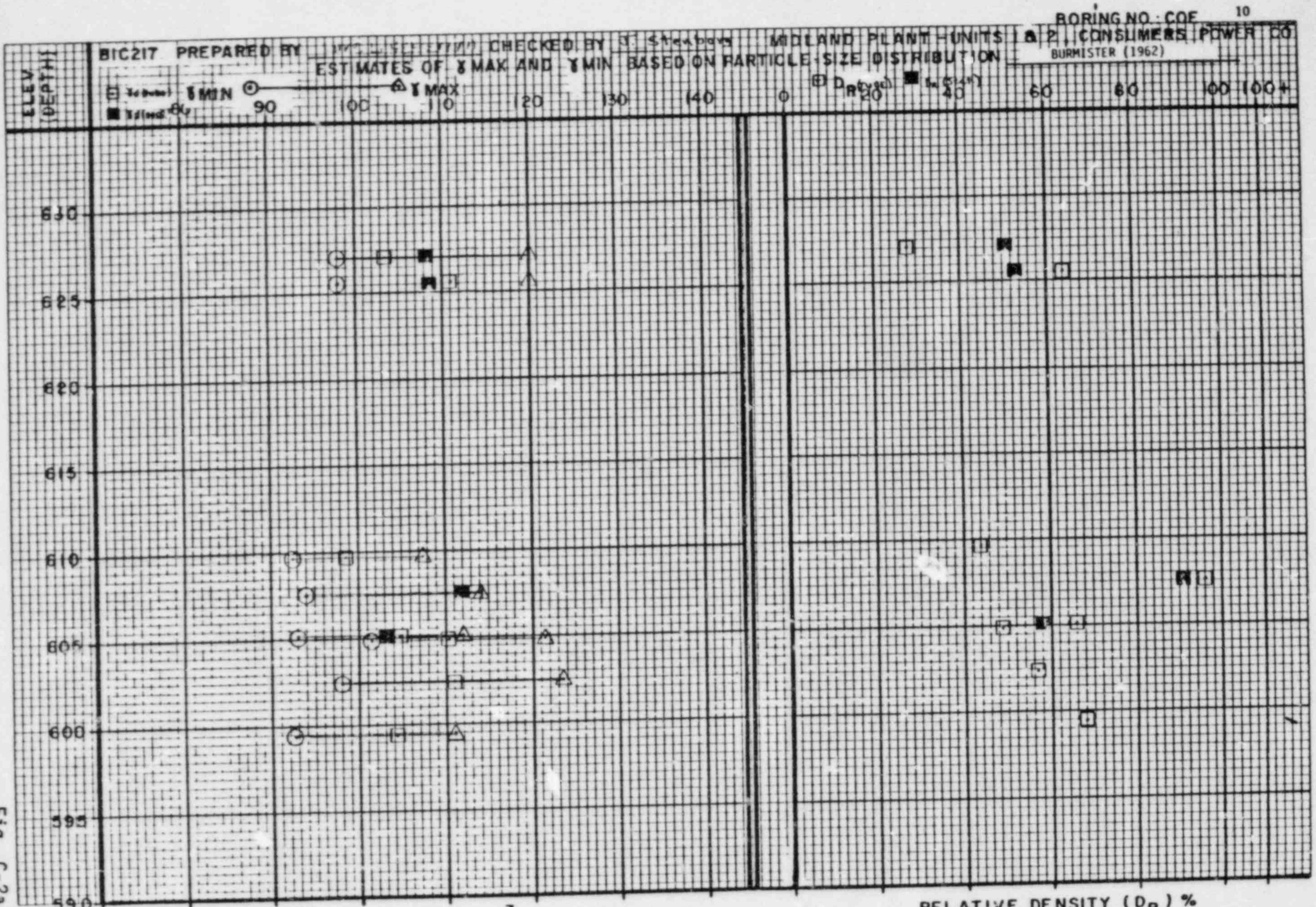


Fig. C-1d

Woodward-Clyde Consultants



Woodward-Clyde Consultants

Fig. C-2a

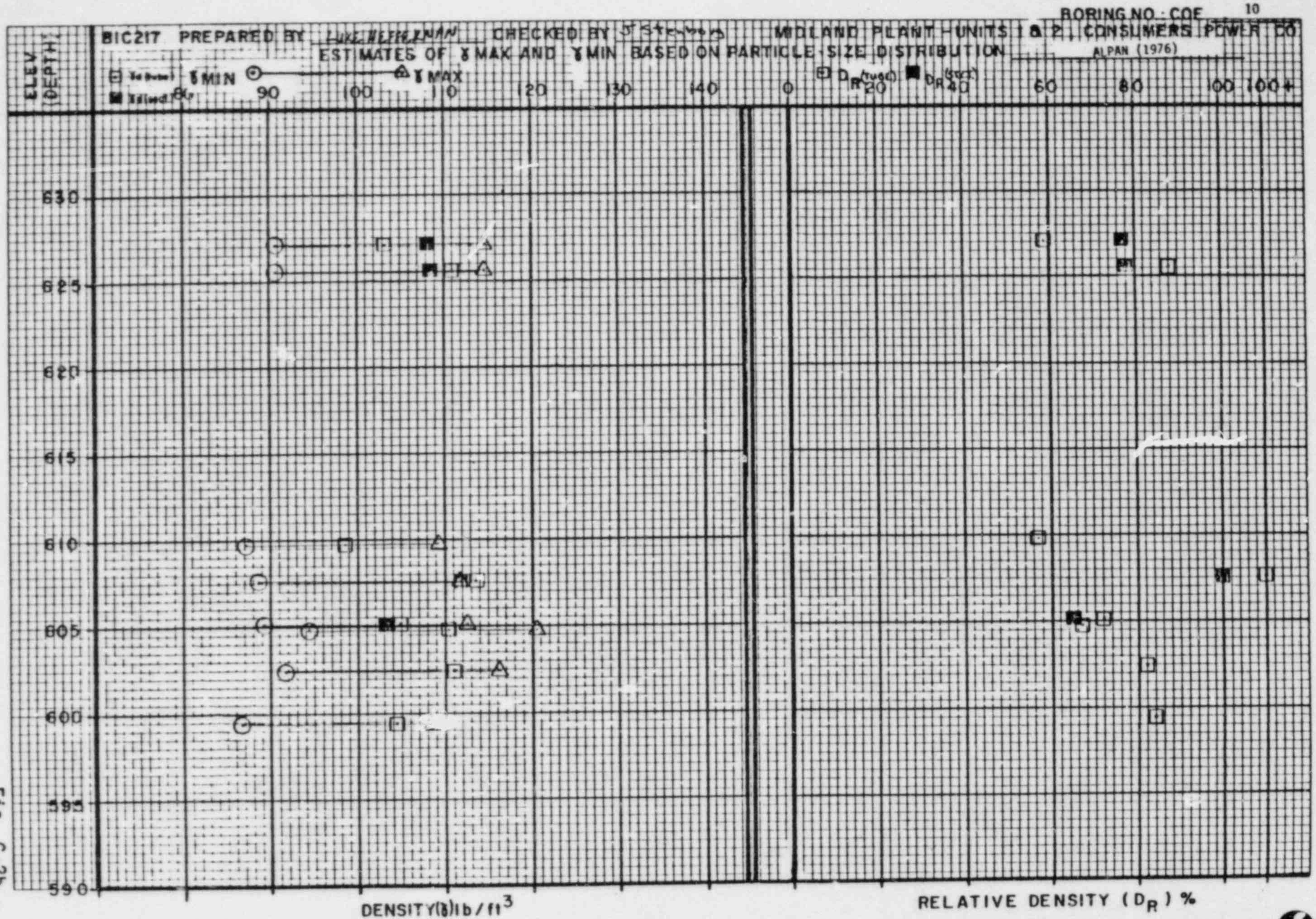


Fig. C-2b

Woodward-Clyde Consultants

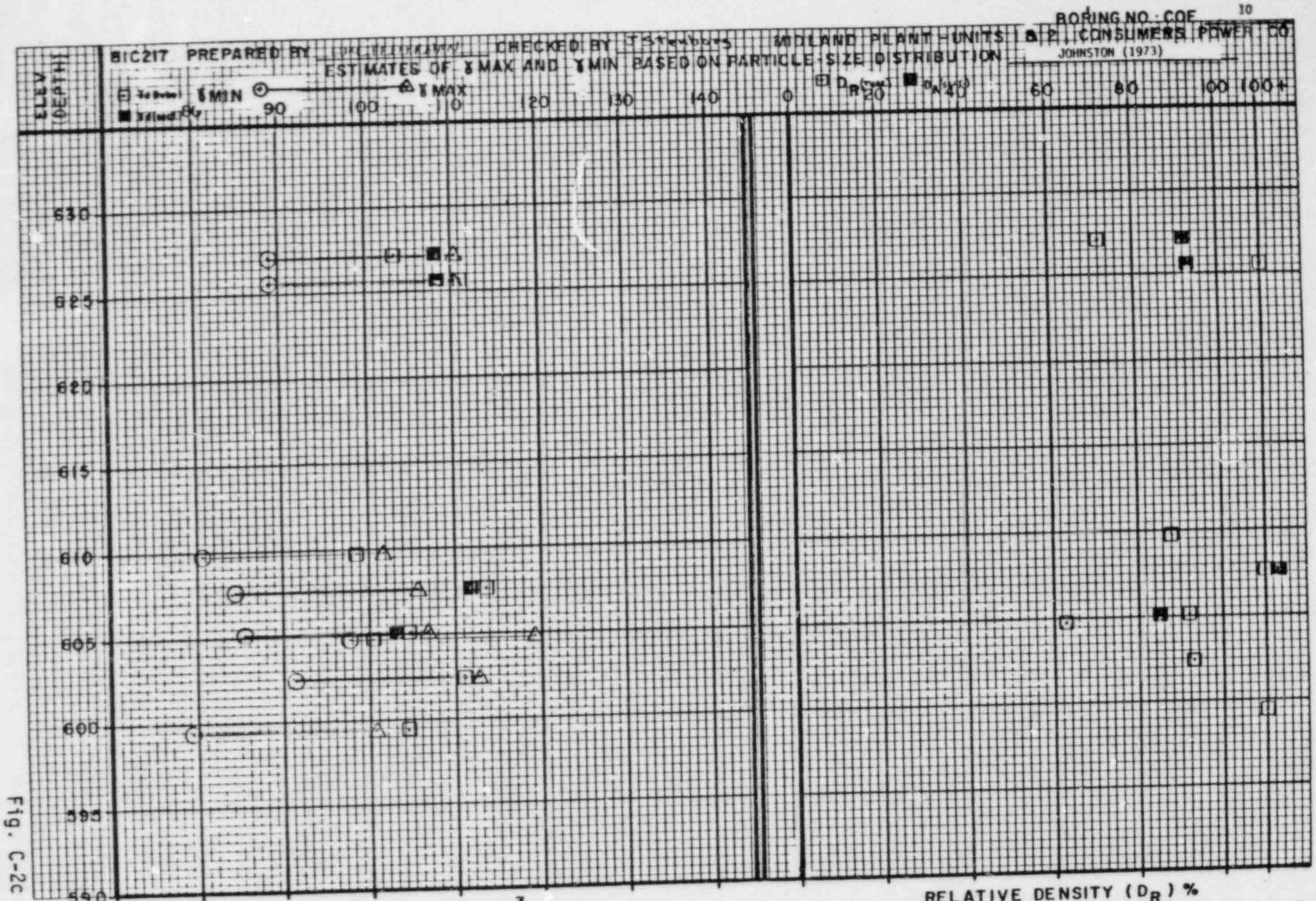


Fig. C-2c

Woodward-Clyde Consultants

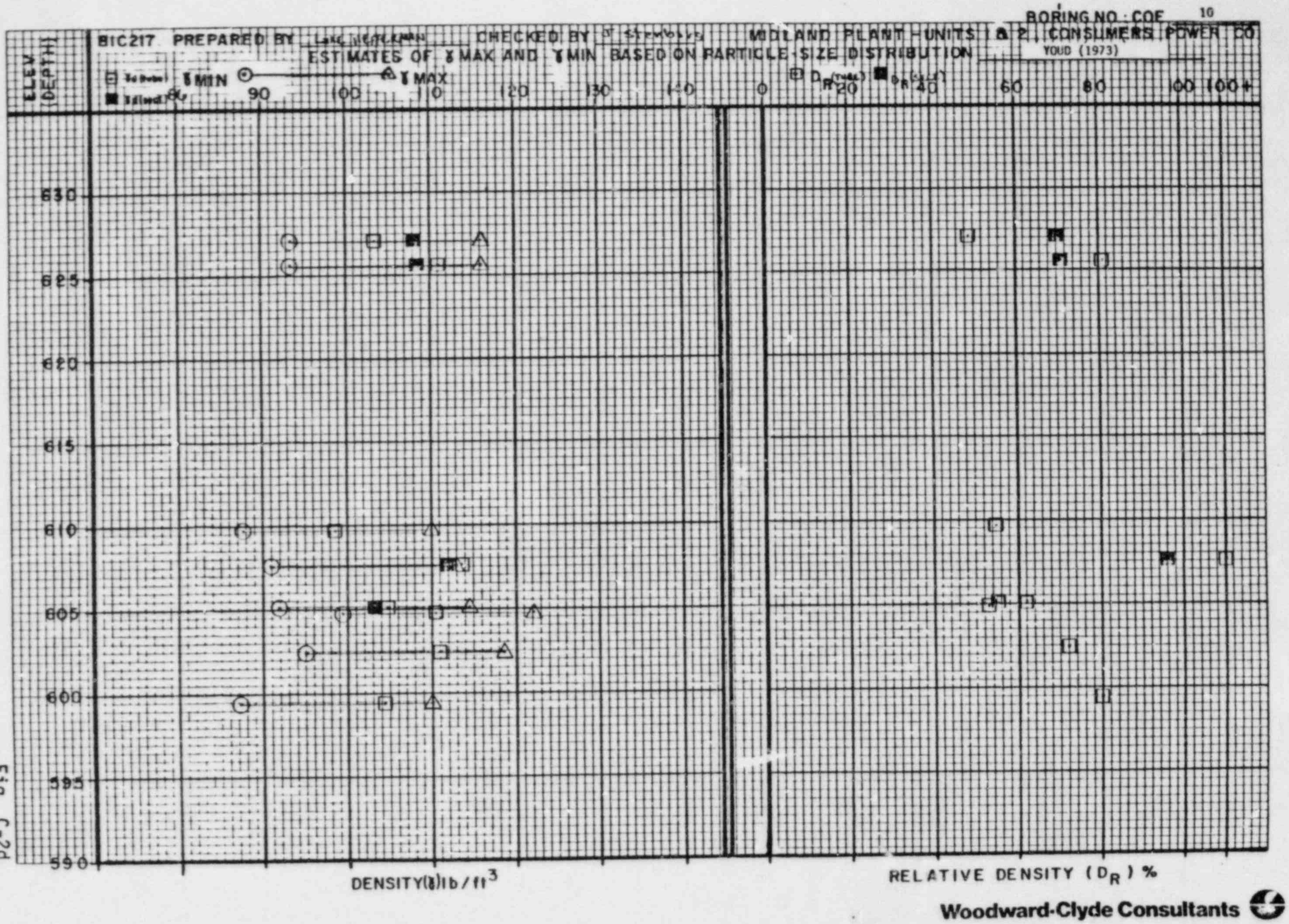
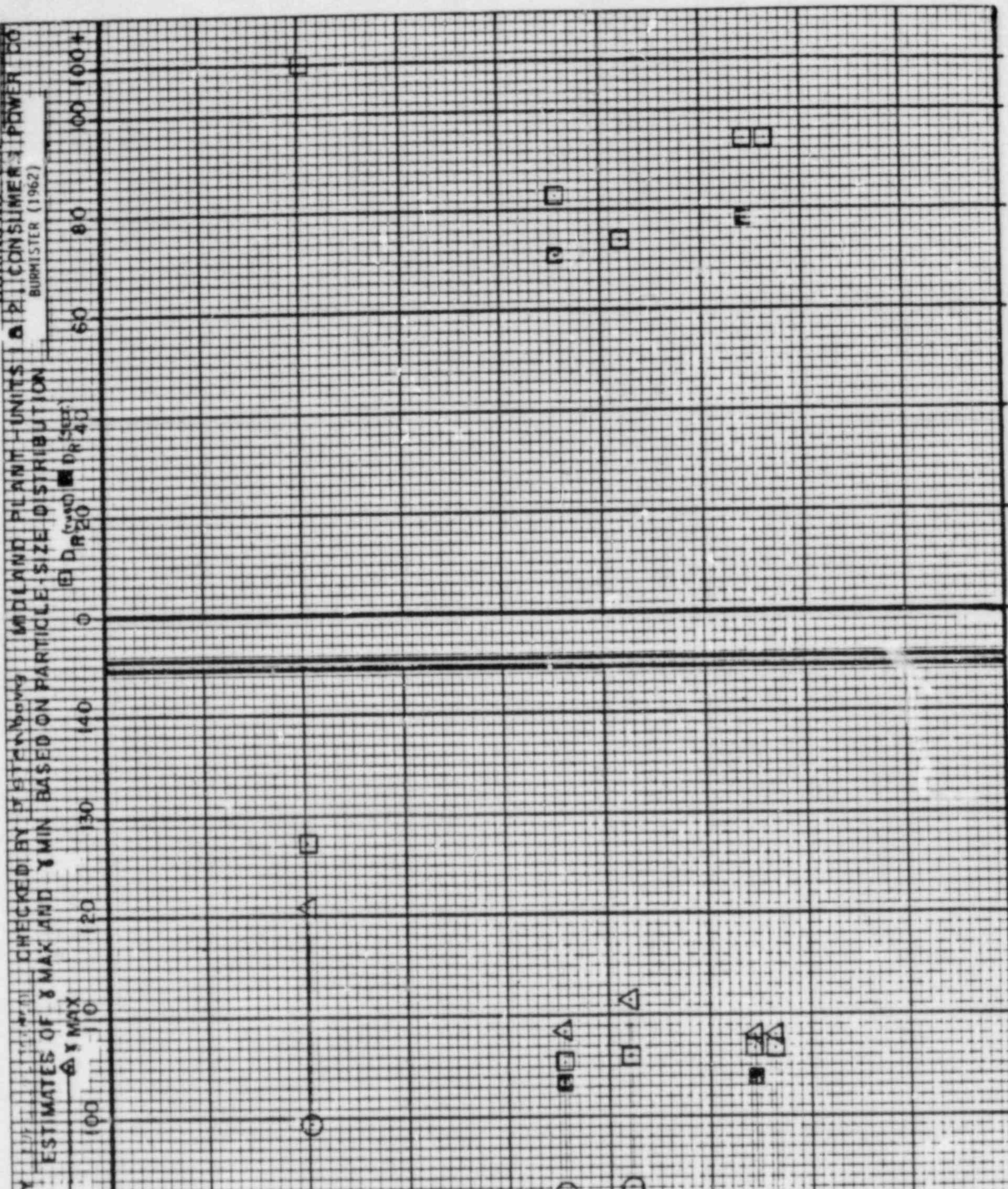

BORING NO.: COE 10
BIC217 PREPARED BY
CHECKED BY
MIDLAND PLANT - UNITS 1 & 2, CONSUMERS POWER CO
ESTIMATES OF γ MAX AND γ MIN BASED ON PARTICLE SIZE DISTRIBUTION
YOUD (1973)
γ MIN
γ MAX
D_R
ELEV (DEPTH)
630
625
620
615
610
605
600
595
590
90
100
110
120
130
140
0
20
40
60
80
100
100+
DENSITY (γ) lb/ft³
RELATIVE DENSITY (D_R) %
Woodward-Clyde Consultants

Fig. C-2d



Woodward-Clyde Consultants

Fig. C-3a

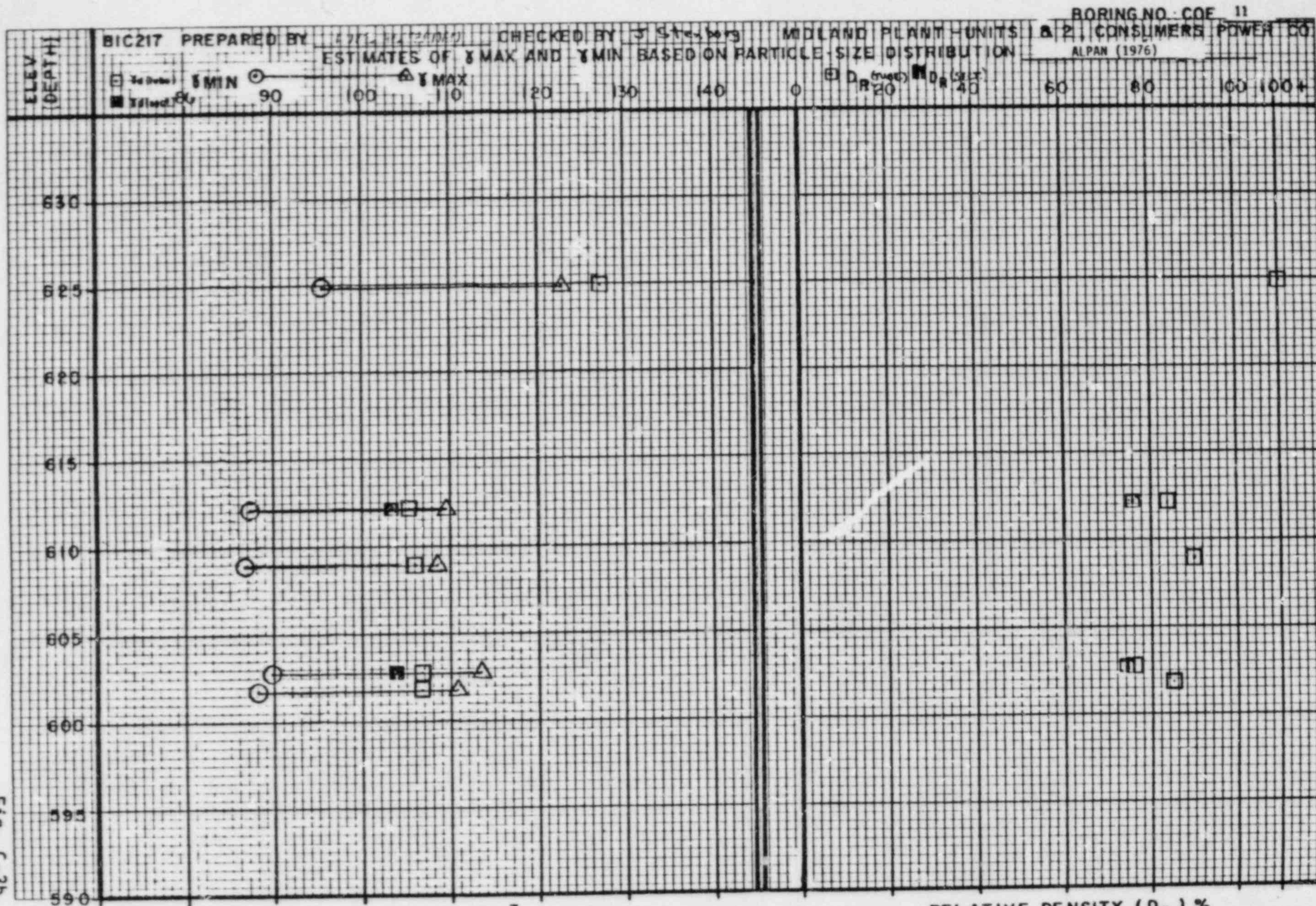
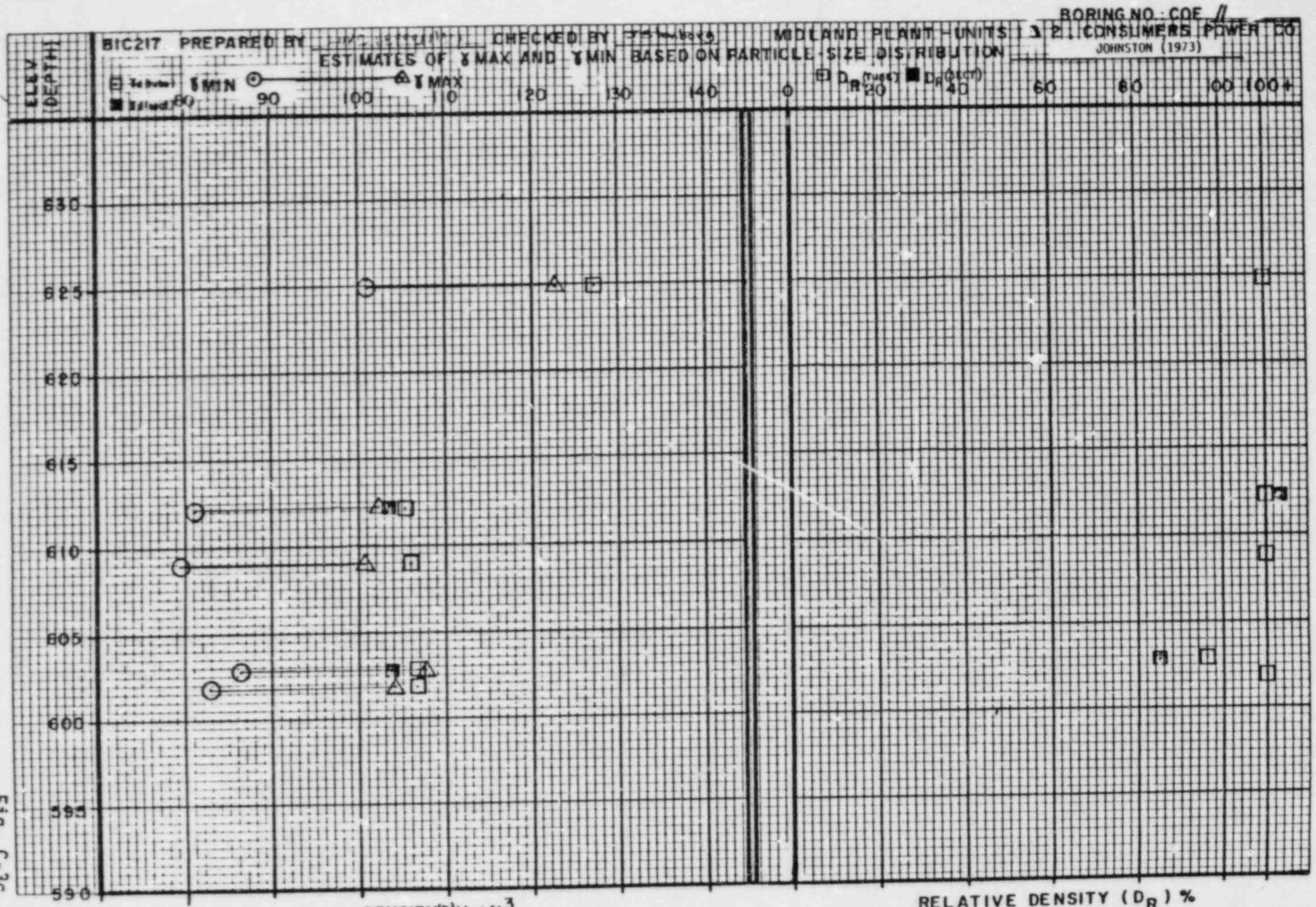


Fig. C-3b

Woodward-Clyde Consultants



Woodward-Clyde Consultants

Fig. C-3c

BORING NO. COE 11

BIC217 PREPARED BY ______ CHECKED BY ______ MIDLAND PLANT - UNITS 1 & 2, CONSUMERS POWER CO.

ESTIMATES OF γMAX AND γMIN BASED ON PARTICLE SIZE DISTRIBUTION YOUD (1973)

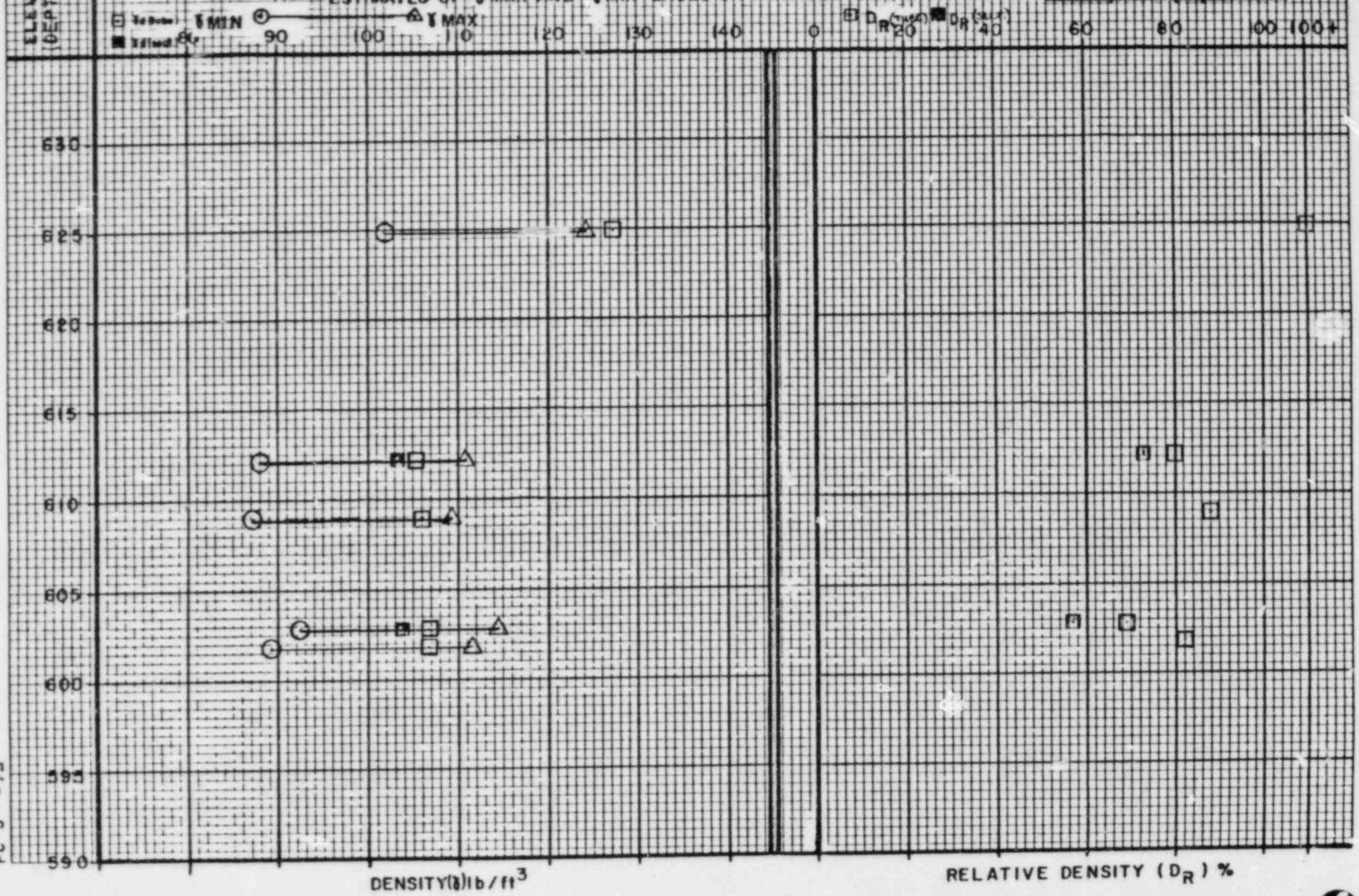


Fig. C-3d

Woodward-Clyde Consultants

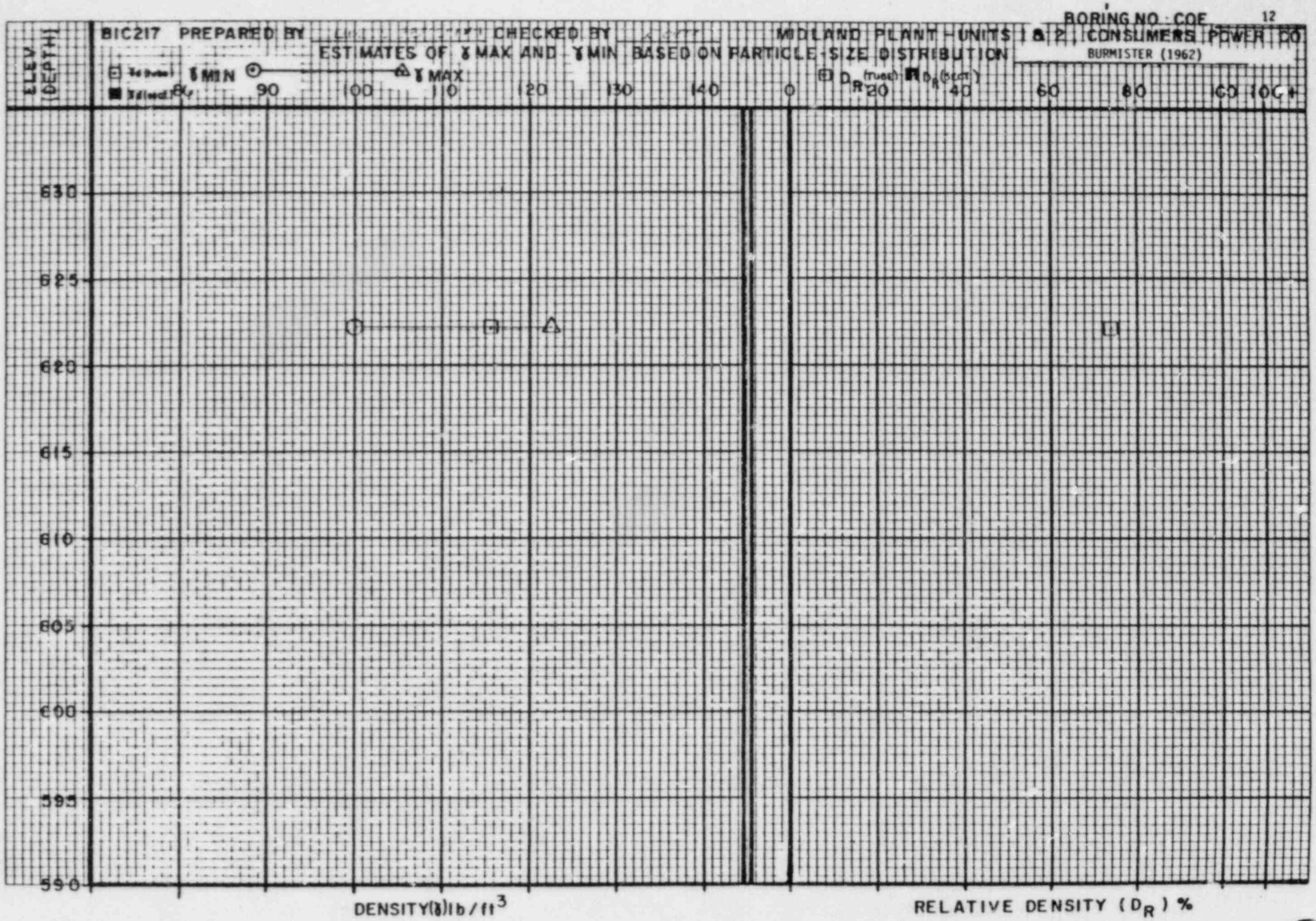


Fig. C-4a

Woodward-Clyde Consultants

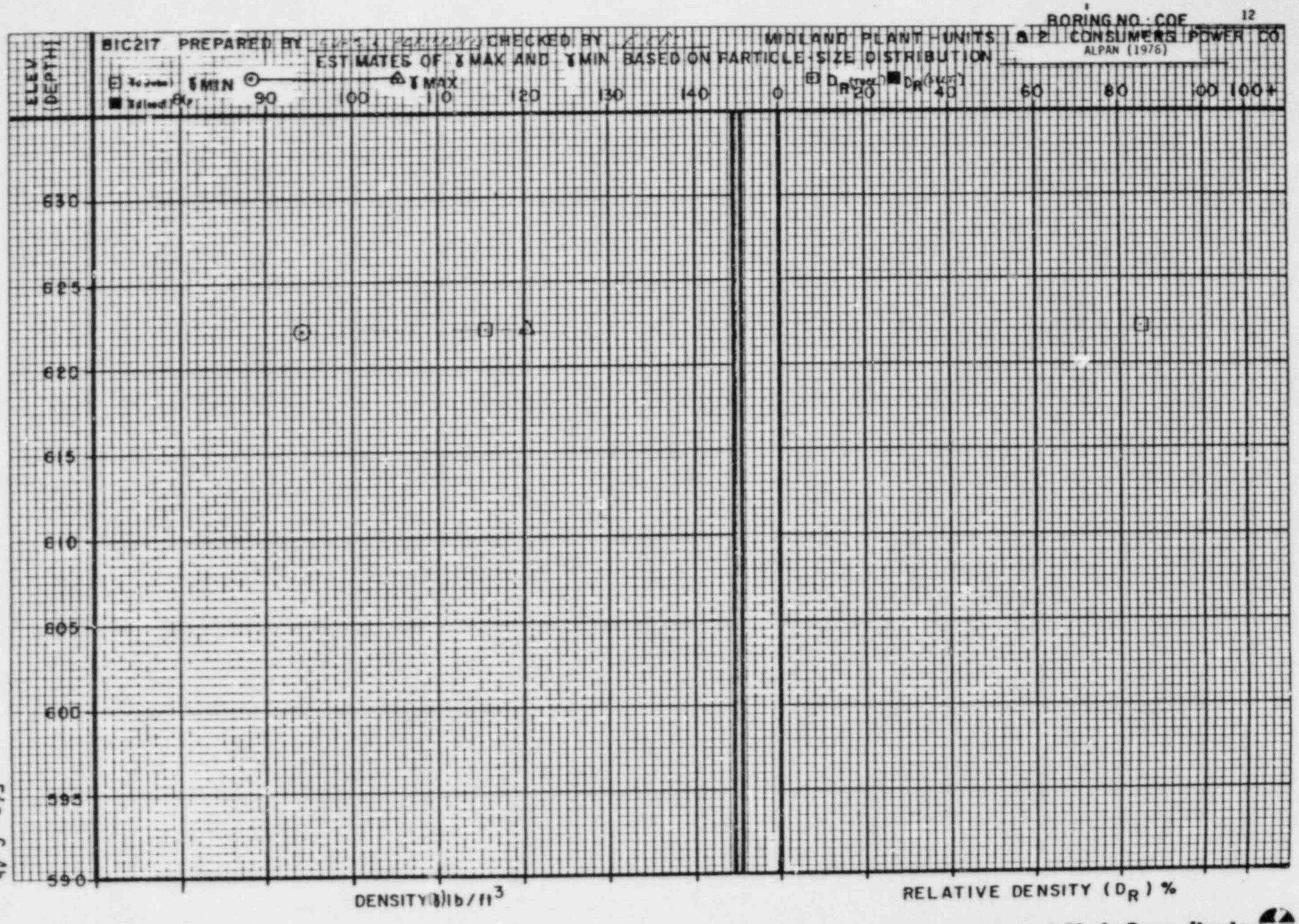
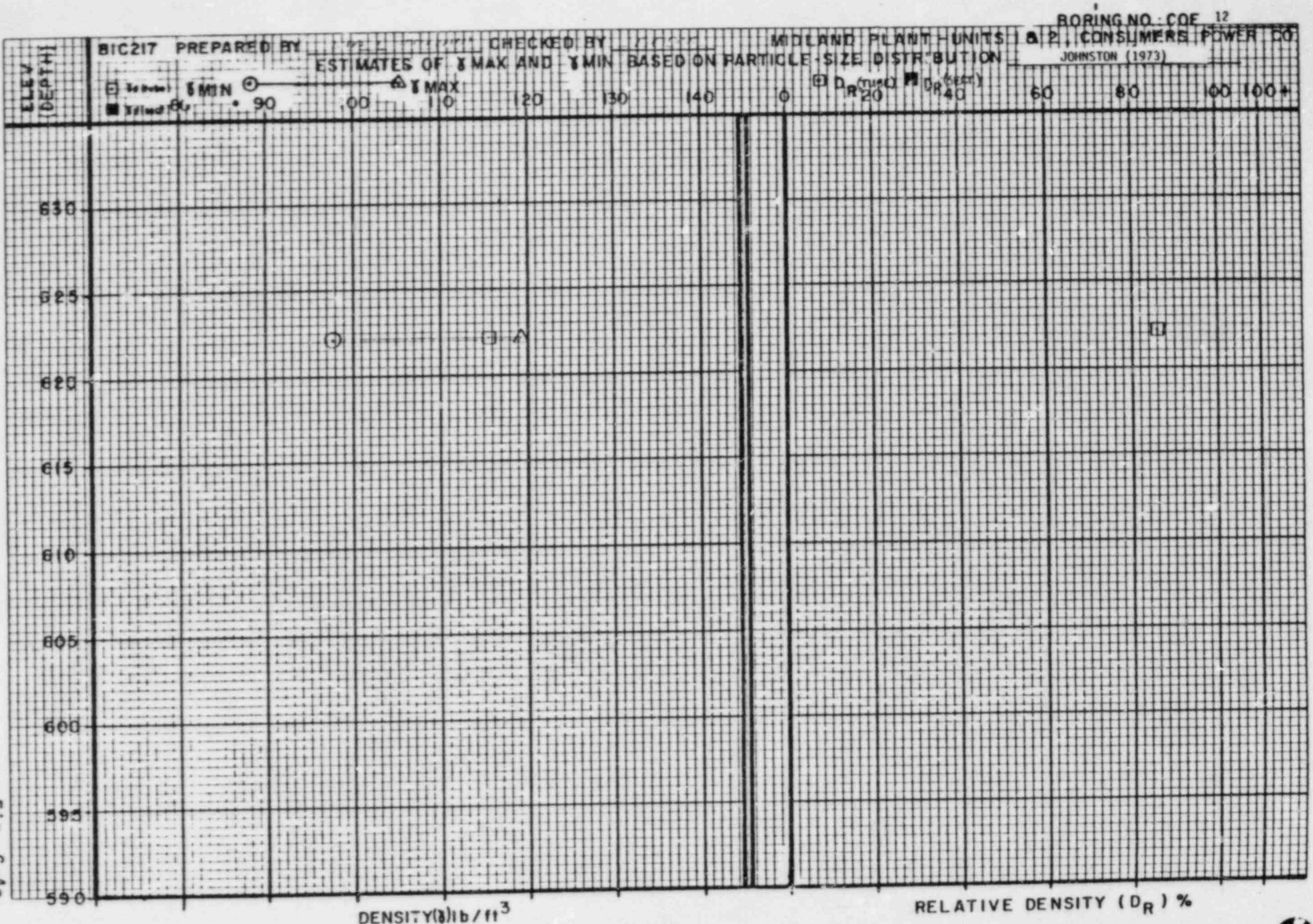


Fig. C-4b

Woodward-Clyde Consultants



Woodward-Clyde Consultants

Fig. C-4c

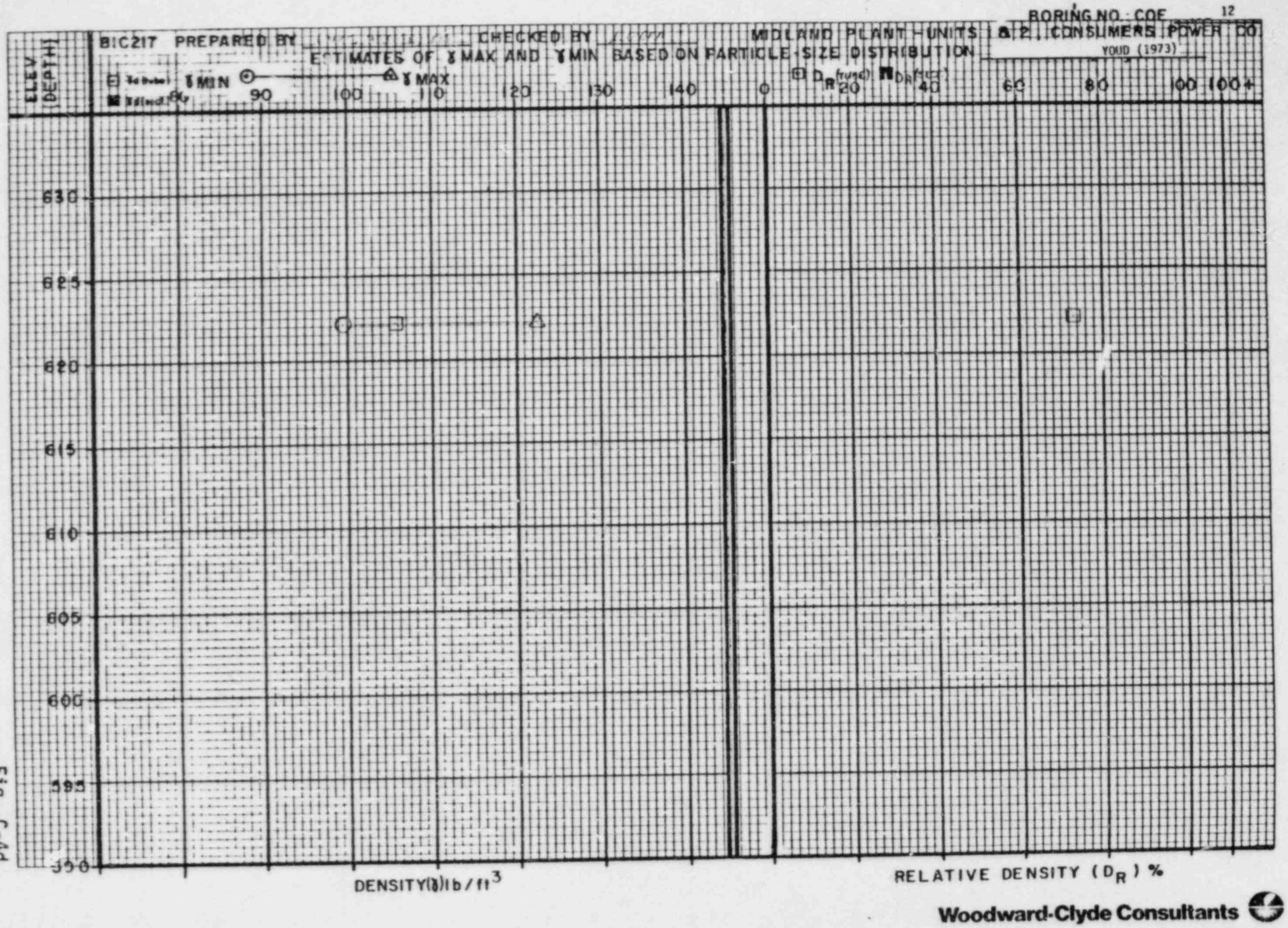

BORING NO. COE 12
BIC217 PREPARED BY
CHECKED BY
MIDLAND PLANT - UNITS 1 & 2, CONSUMERS POWER CO.
ESTIMATES OF γMAX AND γMIN BASED ON PARTICLE-SIZE DISTRIBUTION
YOUD (1973)
γMIN
γMAX
ELEV (DEPTH)
80
90
100
110
120
130
140
0
20
40
60
80
100
100+
630
625
620
615
610
605
600
595
590
DENSITY(γ) lb/ft³
RELATIVE DENSITY (D_R) %
Woodward-Clyde Consultants

Fig. C-4d

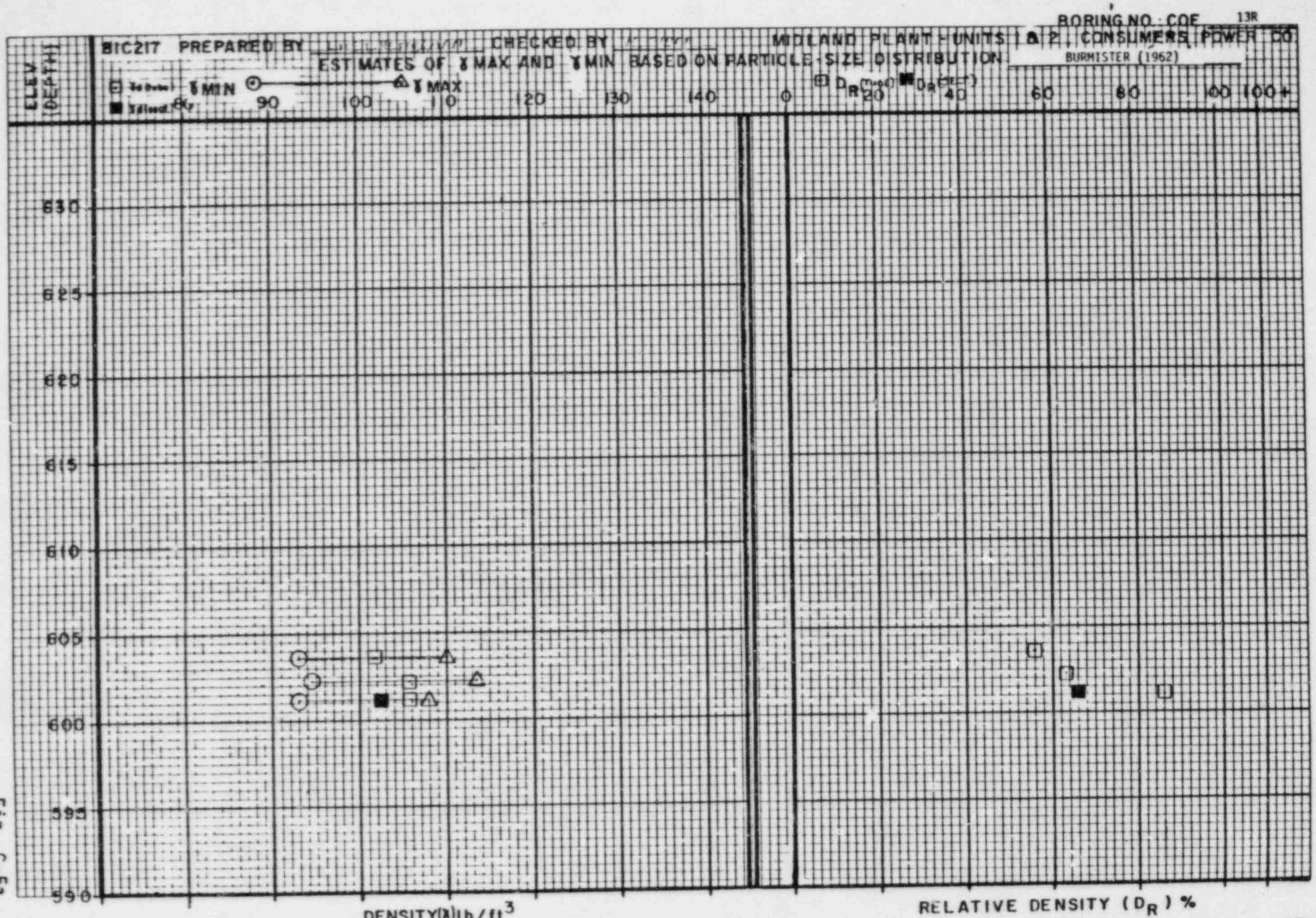
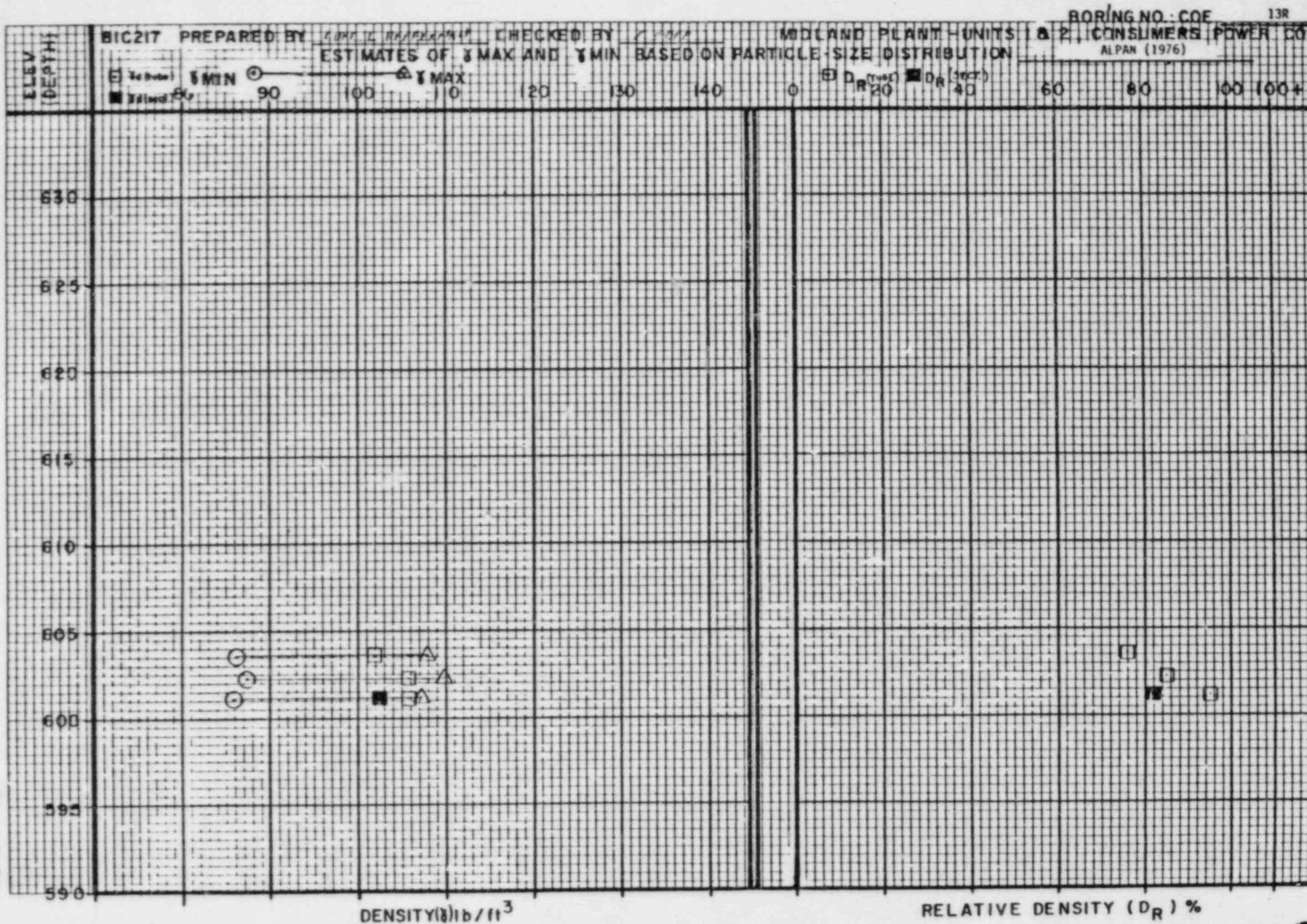


Fig. C-5a

Woodward-Clyde Consultants



Woodward-Clyde Consultants

Fig. C-5b

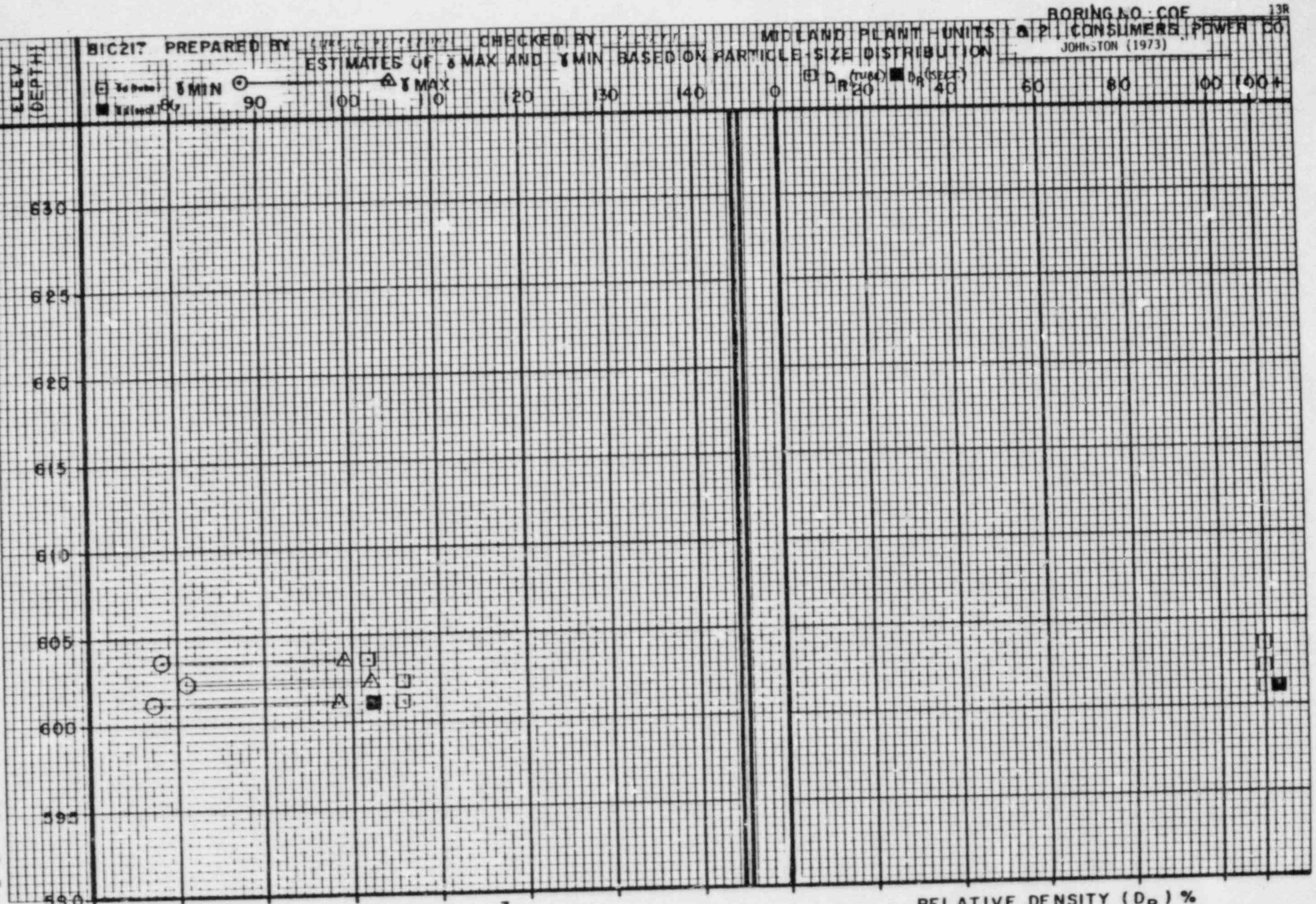


Fig. C-5c

Woodward-Clyde Consultants

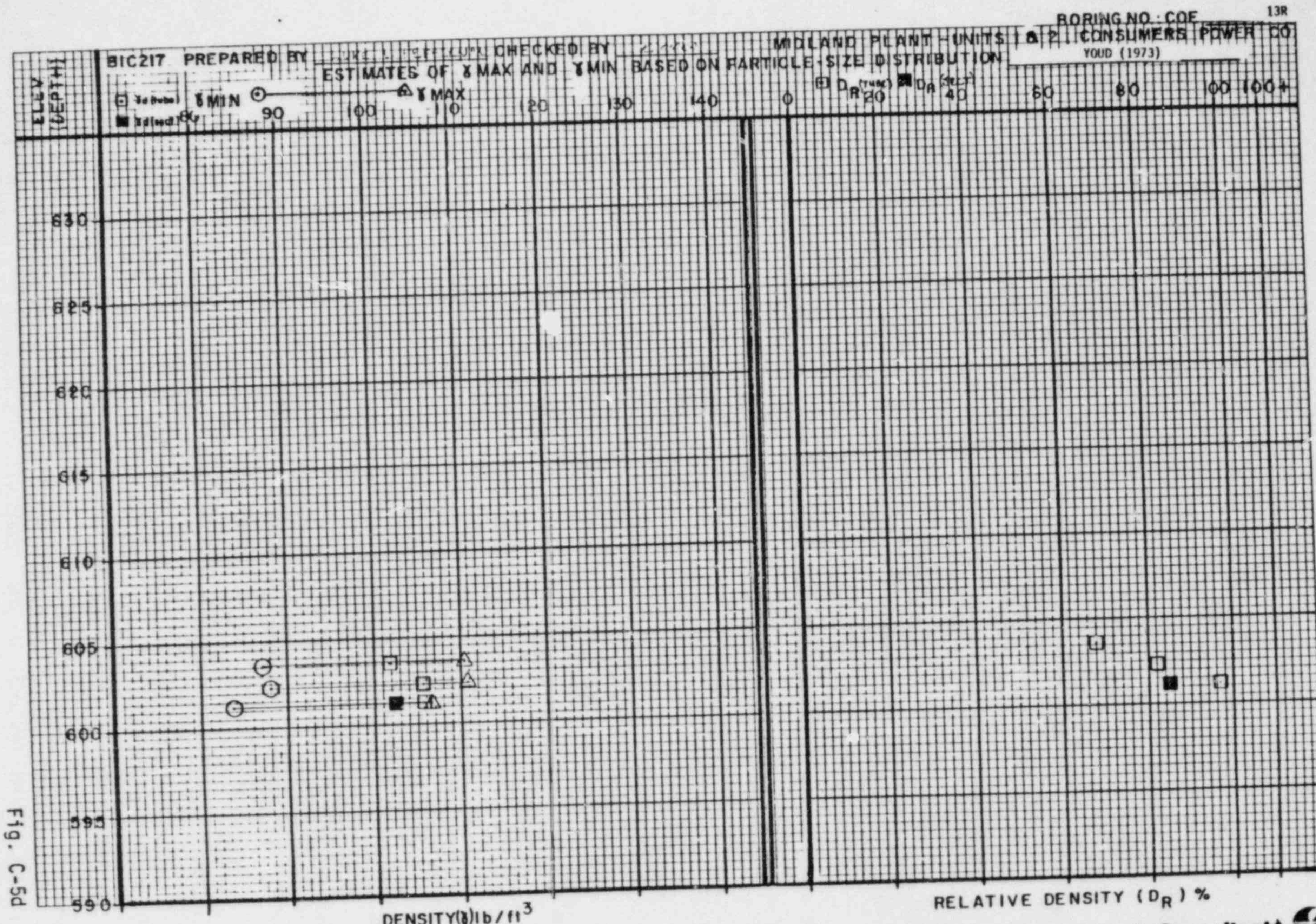


Fig. C-5d

Woodward-Clyde Consultants

ENCLOSURE 7

REVIEW AND CONTROL OF FACILITY CHANGES TO THE DIESEL GENERATOR BUILDING

Facility changes, ie, changes in structures or changes and additions in equipment and bulk commodities, to the Midland Plant safety-related structures are subjected to the reviews of the organizations identified in the Administrative Controls Section of the Midland Technical Specifications, FSAR Subsection 16.6.5.

The two major organizations responsible for the reviews of facility changes are the Plant Review Committee (PRC), composed of plant staff members, and the independent Safety and Audit Review Board (SARB), which reports directly to the Vice President of Nuclear Operations. The functions, responsibilities and authority of these two review groups are identified in the Midland FSAR Subsections 16.6.5.1 and 16.6.5.2.

Facility change reviews will be conducted in accordance with the requirements of 10CFR50.59. Facility change reviews will include a review of the Midland FSAR to determine whether the change affects the safety of the structure or system and whether an unreviewed safety question is involved. This review is performed by a Consumers Power employee or a consultant. The subsequent evaluation is reviewed by the onsite Plant Review Committee prior to implementation of the change. The SARB also conducts an independent review of facility changes.

10/15/81

ENCLOSURE 8

DIESEL GENERATOR BUILDING
BEARING PRESSURE DUE TO EQUIPMENT AND COMMODITIES

A detailed weight summary has been made for equipment and commodities (piping, cable tray, wire, etc) which were included as live loads in the bearing pressure calculations for the diesel generator building.

The total weight of equipment is estimated to be 1,474 kips of which 1,217 kips is on the pedestals and 257 kips is distributed throughout the building. The weight of commodities distributed throughout the building is estimated to be 614 kips.

The weight of equipment was determined from vendor drawings for each piece of equipment. The majority of equipment is directly mounted on the diesel generator pedestal and contributes to the bearing pressure below the pedestals. The remaining pieces of equipment are mounted on walls and elevated slabs and contribute to the bearing pressure below the building spread footings.

The weight of commodities was determined by performing a take-off of the lineal footage or square footage of the various commodities. These values are then multiplied by an appropriate unit weight for each commodity to determine the total weight of the commodity.

The commodities are attached to the walls and elevated slabs of the building and contribute to the bearing pressure below the building spread footings.

The contact area of each pedestal is approximately 745 square feet and the contact area of the building spread footings is approximately 6,425 square feet. Thus, the load intensity due to equipment and commodities is 408 psf under each pedestal and 136 psf under the building footings.

Because the building is normally unoccupied, occupancy loads contributing to settlement will be negligible. The specified live load for the building floors represents the maximum estimated load on the floor during construction and maintenance. This load is used for design of the floor slab. Hence, the assumption of a 5 psf allowance for the occupancy load at the building footings is considered conservative. A 5 psf allowance for occupancy live loads on the grade slab would also be conservative.

10/15/81

mi1081-0459a100

Consumers Power Company

James W Cook
Vice President - Projects, Engineering and Construction

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0453

September 30, 1981

Harold R Denton, Director
Office of Nuclear Reactor Regulation
US Nuclear Regulatory Commission
Washington, DC 20555

MIDLAND PROJECT
DOCKET NOS 50-329, 50-330
SUBMITTAL OF THE AUXILIARY BUILDING
DYNAMIC MODEL, SERVICE WATER PUMP
STRUCTURE DYNAMIC MODEL AND DESCRIPTION OF SOILS
SETTLEMENT REMEDIAL FIX FOR THE AUXILIARY BUILDING
FILE 0485.16, B3.0.1 SERIAL 14110
REFERENCES: (1) JWCook Letter to HRDenton, Serial 11625 Dated March 23, 1981
(2) JWCook Letter to HRDenton, Serial 13738 Dated August 26, 1981
ENCLOSURES: (1) Service Water Pump Structure Seismic Model
(2) Auxiliary Building Seismic Model
(3) Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits

In our previous correspondence of August 26, 1981 (Reference 2) construction permit level design information relating to the remedial actions for the service water pump structure was provided to the staff. Enclosed are twenty-five (25) copies of the report (Enclosure 1) entitled "Service Water Pump Structure Seismic Model" which is based upon the design information already forwarded to the NRC. In addition, we are providing twenty-five (25) copies each of two reports, Enclosures 2 and 3. Enclosure 2 describes the seismic model for the auxiliary building for computing the building response to seismic loading as well as to generate instructure response spectra. Enclosure 3 represents the construction permit level of design information for the auxiliary building remedial actions. All three of the enclosed documents are provided to complete commitments contained in the "Statement of Agreement" from the ASLB Prehearing Conference Order of May 5, 1981.

The seismic model reports for the service water pump structure and the auxiliary building include the following information: (1) model description, (2) soil-structure interaction considerations; (3) the dynamic model properties; and (4) fundamental frequencies and mode shapes. The auxiliary building model includes full underpinning of the control tower and electrical penetration areas, integrally tied to the main auxiliary building at Column Line H. The service water pump structure model includes full underpinning of the northern portion of the building originally supported by the fill. The models reflect the underpinning currently planned and, therefore, are subject to possible revision after the final building structural analysis and NRC staff review is completed. We believe that the enclosed reports combined with our scheduled meeting with the staff during the week of September 30, 1981 provides sufficient information to permit the NRC to review and provide its concurrence with the proposed remedial actions. Your expeditious review and approval would be most appreciated to support the hearings and construction of the remedial work.

for JW Cook

JWC/RLT/bh

CC Atomic Safety and Licensing Appeal Board, w/o
CBechhoefer, ASLB, w/o
MMCherry, Esq, w/o
RJCook, Midland Resident Inspector, w/o
RSDecker, ASLB, w/o
DHood, NRC, w/a (2)
DFJudd, B&W, w/o
JDKane, NRC, w/a
FJKelley, Esq, w/o
RBLandsman, NRC Region III, w/a
WHMarshall, w/o
WOtto, US Army Corps of Engineers, w/a
WDPaton, Esq, w/o
FRinaldi, NRC, w/a
HSingh, Army Corps of Engineers, w/a
BStamiris, w/o
FPCowan, w/o

Ross Landsman
I E - RIII

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

SEP 25 1981

Docket Nos. 50-329 OM, OL
and 50-330 OM, OL

Mr. J. W. Cook
Vice President
Consumers Power Company
1945 West Parnall Road
Jackson, Michigan 49201

Dear Mr. Cook:

Subject: Staff Concurrence on Surcharging of Valve Pits for Borated Water Storage Tank Foundations

References: (1) Interim 50.55(e) Reports 81-03 dated February 20, 1981, April 3, 1981, June 12, 1981, June 26, 1981 July 21, 1981 and August 28, 1981
(2) Structural Design Audit by NRC Staff of Midland Plant, April 20-24, 1981, AnnArbor, Michigan
(3) Meetings of May 4-6, 1981, to discuss Soils Remediation, Bethesda, Maryland
(4) Meeting of August 25, 1981, Midland, Michigan
(5) Telephone conference on July 30, 1981, August 12 & 14, 1981, September 10 & 11, 1981

By several interim 50.55(e) reports, meetings and telephone conversations (Reference 1-5), you have informed us of the status of the cracks in the concrete foundations of the Borated Water Storage Tanks (BWST) for Midland Plant, Units 1 and 2 and your preliminary plans for remediation. Your plans for remediation include, in part, surcharging a portion of the BWST valve pits and the surrounding area in order to consolidate the fill beneath the pits, reduce residual settlement during plant life, reduce distortion of the ring wall foundation, partially close existing cracks, and reduce tank shell deformation. As noted in Mr. J. Keppler's letter of July 13, 1981, you have agreed that the surcharging would not begin until conferences with the NRR staff were completed. Your letter of August 28, 1981, states your belief that resolution of NRC concerns has been achieved and requests NRR concurrence of the proposed surcharge program.

Your plans call for daily visual inspection of cracks in the BWST ring walls and valve pits during the surcharge period. You have also committed to stop further loading if a maximum 1/2-inch settlement is reached prior to full surcharge loading to provide for engineering evaluations. We find these plans to be acceptable, but not sufficient. Our approval recognizes your adoption of two further conditions:

~~8110200142~~

SEP 29 1981

1. You state that, while it is not anticipated that existing cracks will widen or that significant new cracks will form, any new or existing cracks in excess of 10 mils during the surcharge program will be monitored and the results reported to the NRC upon removal of the surcharge. The absence of any immediate actions to assure that cracks approaching acceptable limits during the program will be terminated in a timely manner was unacceptable to the NRC staff. You, therefore, have also committed that in the event that the monitoring program should indicate a crack reaching or exceeding 16 mils, then the last increment of surcharge which was added prior to start of crack growth will be removed immediately and any further surcharge addition will be prohibited pending engineering evaluation and further NRC staff concurrence. This, of course, excludes the existing 20 mil crack already known to exist at the ring-pit interface of Unit 1.

2. You state that propagation of a crack from the tension zone of the wall is not expected to occur because the ultimate moment capacity of the valve pit wall is governed by the yielding of the reinforcement steel. Your expected limit of new crack propagation is 18 inches from the top of the valve pit roof slab. However, your plans provide no immediate action in the event a crack should propagate above this 18 inch value. You, therefore, have committed that in the event a crack should propagate to within 18 inches from the top of the valve pit roof slab, then the last increment of surcharge which was added prior to start of crack propagation will be removed immediately and any further surcharge addition will be prohibited pending engineering evaluation and further NRC staff concurrence.

On the basis the above two additional committments, the NRC staff concurs with your plans to commence surcharging of the BWST valve pits.

Our concurrence to begin this activity does not address the adequacy of the proposed remedy to achieve its intended purpose nor does it have any effect on any other remedial action that may be required as a result of the staff's OL review or as a result of the OL-OM hearing. Rather, the staff's review at this point has been limited to assurance that proper precautions are or will be in place to preclude potential detrimental effects due to surcharging.

Sincerely,

Robert L. Tedesco, Assistant Director
for Licensing
Division of Licensing

cc: See next page

ALBoos

040611

James W Cook
Vice President - Projects, Engineering and Construction

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0453

August 26, 1981

Harold R Denton, Director
Office of Nuclear Reactor Regulation
US Nuclear Regulatory Commission
Washington, DC 20555

MIDLAND PROJECT
DOCKET NOS 50-329, 50-330
SOILS SETTLEMENT REMEDIAL ACTION
FOR THE SERVICE WATER PUMP STRUCTURE (SWPS)
FILE: 0485.16, B3.0.8 SERIAL: 13738
REFERENCES: JWCOOK TO HRDENTON, SERIAL 11625 DATED MARCH 23, 1981.
ENCLOSURES: MIDLAND UNITS 1 AND 2 - TECHNICAL REPORT
ON UNDERPINNING THE SERVICE WATER PUMP STRUCTURE.

In the referenced correspondence of March 23, 1981 we advised the NRC of the underpinning concept for the overhanging portion of the service water pump structure which is a full length wall extending into the natural till material. This full length wall concept was adopted to replace the original remedial action, a driven pile support concept, as a result of the increased seismic requirements imposed by the staff. We are forwarding thirty (30) copies of the enclosed report entitled "Technical Report on Underpinning the Service Water Pump Structure" which describes the design and construction requirements of this SWPS remedial action.

The design and construction criteria contained in the attached report has been written to provide the NRC with information which substantially exceeds the construction permit level of detail. Included in this report are the following types of information: (1) drawings showing the underpinning scheme and a description of the construction sequence for this scheme; (2) dewatering for construction; (3) the design and acceptance criteria for the underpinning scheme, including load combinations, bearing pressures, structural stresses, and seismic loads; (4) applicable codes; and (5) scope of the quality assurance requirements.

The proposed service water structure remedial underpinning is approximately a 4-foot thick, reinforced concrete wall that is approximately 30 feet high with a flared base at the north wall and is constructed to act as a continuous member under the perimeter of that portion of the structure founded on backfill material. In addition, a predetermined jacking force will be applied to the full perimeter of the SWPS overhang during construction to provide adequate load transfer from the structure to the underpinning wall.

oc0881-0407a100

~~8109030306~~

040611

While we believe that the enclosed report provides sufficient information to permit the NRC to review and provide its concurrence with the proposed underpinning scheme, we suggest that a technical review meeting be held during the week of August 31, 1981 to respond to any outstanding NRC concerns. Please contact us to establish a mutually agreeable day for this meeting.

Your expeditious review and approval would be most appreciated to support the hearings and construction of the remedial work.

James W. Cook

JWC/RLT/cr

CC Atomic Safety & Licensing Appeal Board, w/o
Atomic Safety & Licensing Board Panel, w/o
Charles Bechhoefer, Esq, w/o
MMCherry, Esq, w/o
RJCook, Midland Resident Inspector, w/o
Dr FPCowan, w/o
RSDecker, w/o
NRC Docketing Service Section, w/a
SGadler, w/o
RWHuston, Washington, w/a
JDKane, NRC w/a
FJKelley, Esq, w/o
WHMarshall, w/o
MIMiller, Esq, w/a
WOtto, US Army Corps of Engineers, w/a
WDPaton, Esq, w/o
MSinclair, w/o
BStamiris, w/o
HSingh, US Army Corps of Engineers, w/a

oc0881-0407a100

04061i

BCC RCBauman/TRThiruvengadam, P-14-400, w/o
WRBird, P-14-418A, w/a
JEBrunner, M-1079, w/a
GSKeeley, P-14-113B, w/a
DBMiller, Midland, w/a
NRamanujam, P-14-100, w/a
TJSullivan/DMBudzik, P-24-517, w/o
RLTeuteberg, P-24-513, w/a
ALBoos, Bechtel, w/a
Dr AJHendron, Bechtel Consultant, w/a
DFJudd, B&W, w/o
Dr Ralph B Peck, Becthel Consultant, w/a
SSAfifi, Becthel, w/a
JARutgers, Bechtel, w/a
WJCloutier, P-24-611, w/a
KLRazdan, P-13-220, w/a

oc0881-0407a100

040611

TECHNICAL REPORT ON UNDERPINNING THE SERVICE WATER PUMP STRUCTURE FOR MIDLAND PLANT UNITS 1 AND 2 CONSUMERS POWER COMPANY DOCKET NUMBERS 50-329 AND 50-330

AUGUST 25, 1981

~~8109030308~~

040611

TECHNICAL REPORT ON UNDERPINNING THE SERVICE WATER PUMP STRUCTURE

TABLE OF CONTENTS

TABLES

040611

FIGURES

040611

MIDLAND PLANT UNITS 1 AND 2

TECHNICAL REPORT ON UNDERPINNING

THE SERVICE WATER PUMP STRUCTURE

1.0 INTRODUCTION

This report describes the design and construction requirements of the remedial action for the service water pump structure (SWPS) necessitated by the settlement potential of the plant fill underlying the structure.

2.0 PRESENT CONDITION

The SWPS is a two level, rectangular, reinforced concrete structure. Below el 617', it measures 86 feet by 71 feet 11 inches; above el 617' it measures 106 feet by 86 feet. The maximum overall height is 69 feet [See Figures 1 and 2 (FSAR Figures 3.8-56 and 3.8-57)].

The structure was designed to be supported by the two foundation slabs, one at el 587'-0" and the other at el 617'-0". The lower slab rests on undisturbed natural material and the upper slab rests on fill material placed during construction in 1977.

After discovering settlement of the fill under the diesel generator building, an investigation of the plant fill revealed some questionable areas under the upper base slab, el 617'-0", of the SWPS.

3.0 REMEDIAL ACTION

For the part of the structure resting on plant fill, a continuous underpinning wall, resting on undisturbed natural material, is provided to support the structure adequately under all design load conditions. The underpinning wall provides the necessary vertical and horizontal support to the affected part of the structure. To ensure adequate load transfer, the underpinned structure is jacked from the underpinning walls (Refer to Figure 3).

4.0 DESIGN FEATURES

The proposed underpinning is a 4-foot thick, reinforced concrete wall that is 30 feet high and is constructed to act as a continuous member under the perimeter of the structure overhang. The entire wall is founded on undisturbed natural material. The base of the north underpinning wall is belled out to a 6-foot thickness to limit bearing pressures to the allowable values, whereas the bases of the east and west side walls are 4 feet wide. The allowable bearing pressures

040611

for the undisturbed natural material are based on safety factors of 2 for dynamic loading and 3 for static loading.

A predetermined jacking force is applied to the overhang perimeter to provide adequate load transfer from the structure to the underpinning.

The connection between the underpinning wall and the existing structure is made by 2-inch diameter rock bolts at the vertical interfaces and 2-3/4-inch diameter anchor bolt assemblies at the horizontal interfaces (Refer to Figures 4 and 5). The connectors are designed to transfer shear and tension forces to the underpinned wall. The connectors are not subject to stresses during the jacking procedures because the rock bolts have not yet been installed and the anchor bolts have not been tightened (Refer to Subsection 5.3.2). After the underpinning wall is connected to the existing structure, the connectors are stressed by loads applied to the underpinned structure.

5.0 CONSTRUCTION

The construction procedures discussed in this report are recommended for underpinning the SWPS. If subcontractor recommendations result in improved procedures, they will be incorporated. For details of construction and the construction procedures, refer to Figures 4 and 5.

5.1 DEWATERING

To construct the underpinning, the SWPS site is dewatered: The groundwater level is lowered to el 587 (approximately) by using temporary dewatering wells. These wells will be sealed after the underpinning wall is completed. The acceptance criteria for the dewatering system require that the system produces an effluent that has less than 10 parts per million of soil particles larger than 0.05 millimeters.

5.2 BUILDING POST-TENSIONING

Construction site dewatering removes the buoyancy force on the overhang portion of the structure, resulting in additional loading on the overhang. To compensate for this additional loading of the overhang, a temporary post-tensioning system applies a compressive force to the upper part of the building along each north-south wall. This post-tensioning allows the additional force to be transferred from the overhang by beam action to the adjoining walls which rest on undisturbed natural material (Refer to Figure 6). The post-tensioning system is removed after the initial jacking loads are applied.

040611

5.3 CONSTRUCTION PROCEDURES

The underpinning is constructed as individual piers tied together by threaded reinforcing bar couplers and shear keys to form a continuous wall. Refer to details and procedures in Figures 4 and 5.

5.3.1 Initial Construction Activities

To preserve the structural integrity of the building, the underpinning wall is constructed in small sections (piers) from tunnels which are advanced simultaneously from access shafts located at the northeast and northwest corners of the building. The tunnels initially extend only far enough to construct an approximately 30-foot deep, 5 foot by 4 foot, sheeted pit at each corner of the overhang. The pit is hand dug. The shear strength of the subgrade soil is assessed with a Corps of Engineers cone penetrometer, model CN-973. Under a maximum force of 150 pounds, the cone should not penetrate the surface more than 1/2 inch. After the subgrade is inspected and approved by a geotechnical engineer, reinforcement, subgrade settlement monitoring instrumentation, and anchor bolt assemblies to tie the pier to the underside of the slab, are installed. The pier is then cast with concrete pumped from the access shaft. After at least 48 hours of curing, an initial jacking load is applied to the overhang from jacks placed on the pier top. To ensure adequate support to the building, the tunnel is not advanced to the next stage until the pier is jacked.

Simultaneously with applying the jacking force, the tunnels are advanced to the location of the next pier, which is constructed in a similar manner to the first pier. The piers are tied together with threaded reinforcing bar couplers and shear keys to form a continuous underpinning wall. The threaded reinforcing bar couplers (see Detail 1, Figure 5) conform to the requirements of Section III, Division 2 of the American Society of Mechanical Engineers Boiler and Pressure Vessel Code, 1980 Edition, 1980 and 1981 Summer Addenda. The tensile strength of the splice system is not less than 125% of the specified minimum yield strength of the spliced bar.

A settlement monitoring program for the top and base of each pier begins immediately after pier construction. Instruments accurate to 0.001 inch are installed before the initial jacking is applied. The information from the monitoring program is used to evaluate the time required to dissipate shrinkage and creep of the concrete and creep of the undisturbed natural material. The rate of settlement decreases with time. At the proper point on the settlement-time curve (as determined by the geotechnical engineer), the final jacking operations (as described below) begins.

5.3.2 Final Jacking Stage

After Piers 10 (Figure 4) are constructed, the underpinning wall has progressed to within 6 feet of the vertical interfaces with the existing structure, and the final jacking load is applied. Settlements caused by this load are monitored. When the geotechnical engineer judges that the settlement rate has decreased to a proper value, the load is transferred from the jacks to wedges positioned between the top of the piers and the underside of the overhang, and the jacks are removed. Piers 11 are poured, encasing rock anchors that were previously drilled into the vertical face of the existing structure and thereby connecting the underpinning wall to the vertical face of the existing stucture (Refer to Detail 5, Figure 5). The space between the top of the underpinning wall and the underside of the base slab is filled with nonshrink grout, and previously placed anchor bolt assemblies (which tie the top of the piers to the foundation slab) are tightened (Refer to Detail 7, Figure 4). The underpinning wall is connected to the structure at both the vertical and horizontal interfaces.

5.3.3 Completion of the Underpinning Wall

The tunnel is backfilled with lean concrete beginning at the vertical interface and at the north wall. The completion of the tunnel backfilling terminates at the locations of Piers 12. These piers are then constructed, completing the underpinning wall.

6.0 MONITORING REQUIREMENTS

During construction, the underpinning of the existing structure is monitored for settlement and crack propogation. The long-term surveillance program of the building after the construction of the underpinning is being evaluated.

6.1 SETTLEMENTS

The elevations of settlement markers attached to the structure are measured in accordance with a schedule based on construction procedures. Expected building movements during underpinning operations are small. These movements are recorded, and those exceeding 1/4 inch will be evaluated and reported to the NRC.

6.2 CRACKS

Monitoring of existing or new cracks appearing during the underpinning construction is scheduled. Because of the sequencing of construction procedures, it is not anticipated that existing cracks will significantly widen or new cracks will appear. However, any new structural cracks or changes in existing structural crack widths exceeding 0.010 inch will be evaluated and reported to the NRC.

7.0 ANALYSIS AND DESIGN

The SWPS was originally designed in accordance with FSAR requirements for Seismic Category I structures. A preliminary analysis of the underpinned structure was made which complied with these FSAR requirements, and added a jacking load to the load combinations. The seismic loads used in this analysis were extrapolated from the seismic loading from a previous underpinning design based on piles. When the final seismic loads become available, they will be incorporated in the final design.

In the final design, seismically induced forces and instructure response spectra of the structure are generated in accordance with FSAR Section 3.7. The revised model portrays the structural behavior including the effects of the underpinning and associated foundation modification.

The mathematical seismic model and a description of the soil-structure interaction coefficients to be used in the seismic analysis will be submitted to the NRC in September 1981.

The static structural analysis uses an analytical model capable of representing the structure behavior. The interface between the existing structure and the underpinning wall is modeled to transfer direct loads without providing rotational restraint. The soil media are represented by springs of appropriate stiffness at the base of the structure. The detailed analysis will be performed by conventional methods such as beam theory and/or plate theory or by using the computer program Bechtel Structural Analysis Program (BSAP). For details of the BSAP computer program see FSAR Subsection 3.8.3.4.

7.1 STRUCTURE BEHAVIOR

The vertical loads of the structure are transmitted to the foundation medium through the existing base slab at el 587'-0" and the underpinning wall bearing area. The lateral forces due to seismic and tornado loads are resisted by the shear walls in the structure. These lateral loads are transferred to the foundation medium by the combined action of the base slab at el 587'-0" and the underpinning wall bearing area. To ensure this action, the underpinning walls are connected to the existing structure by rock anchors and anchor bolts capable of transferring all direct loads. This connection is a pinned connection that is consistent with the analysis method.

040611

7.2 DESIGN CRITERIA AND APPLICABLE CODES

The underpinned structure is designed as a Seismic Category I structure. The design complies with the requirements of ACI 318-71 and the 1969 edition of the AISC.

7.3 LOADS AND LOAD COMBINATIONS

The underpinning structure rests entirely on undisturbed natural material. The preliminary analysis of the underpinned structure utilizes the same load combinations used in the original design. However, each load combination is modified by adding the jacking load (P_L). For each loading combination, the jacking load was evaluated with two load factors: a value of 1.0, and the load factor associated with the dead load for that load combination.

For the design of the underpinning and the connections to the existing structure, the safe shutdown earthquake (SSE) forces were increased by 50% to provide for a possible future increase in this loading. The 50% increase was applied to the seismic response of the structure corresponding to the analytical model with the mean soil properties. The existing structure was checked for a 0.12g SSE.

The long-term settlement of the underpinning wall after it is connected to the existing structure will be calculated. The calculation is based on properties of the supporting soil. The long-term settlement effects will be considered in the final analysis of the structure. To provide for these effects, the final analysis is governed by four additional load combinations. These load combinations are discussed in the response to Question 15 of the NRC Requests Regarding Plant Fill (September 1979) and were used in the diesel generator building reanalysis. The load combinations are modified by the addition of the jacking load.

Table 1 lists 26 load combinations, modified for jacking loads. For the preliminary analysis of the underpinned SWPS, the following load combination was most critical:

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_o + 1.25H_o + 1.0R + P_L$$

where

D = dead loads

L = live loads

E' = safe shutdown earthquake

0406||

T_o = thermal effects during normal operating conditions

H_o = force on structure due to thermal expansion of pipes under operating conditions

R = local force or pressure on structure or penetration caused by rupture of any one pipe

P_L = load on structure due to jacking preload

In addition to this load combination, the underpinned structure was checked for stability using the load combinations specified in FSAR Subsection 3.8.6.3.4.

A complete analysis of the underpinned structure, using all applicable load combinations, will be made when the final seismic loads become available.

7.4 STRUCTURAL ACCEPTANCE CRITERIA

The acceptance criterion for analyzing the underpinned structure is in accordance with FSAR Subsection 3.8.6.5.

8.0 QUALITY ASSURANCE REQUIREMENT

This project work is a combination of Q- and non-Q-listed work. The construction of the permanent structures such as the underpinning wall and the connectors are Q-listed, as well as any other activity or structure necessary to protect the SWPS. Construction of temporary structures such as the access shafts and tunnels is non-Q-listed. A detailed quality plan shall be prepared by the subcontractor to identify those specific activities which are required to have a safety "Q" quality program applied along with the major quality program elements for these activities. This quality plan shall be approved by Bechtel and Consumers Power Company prior to the start of any Q-listed work.

9.0 ADDITIONAL NRC REQUIREMENTS

For information purposes, an analysis of the critical sections of the underpinned structure will be made conforming to the provisions of ACI 349-76 as supplemented by NRC Regulatory Guide 1.142.

040611

TABLE 1

LOAD EQUATIONS FOR THE SERVICE WATER PUMP STRUCTURE
MODIFIED TO INCLUDE PRELOAD

Responses to NRC Requests Regarding Plant Fill, Question 15

a. Normal Operating Condition

$$U = 1.05D + 1.28L + 1.05T + P_L \quad (1)$$

$$U = 1.4D + 1.4T + P_L \quad (2)$$

b. Severe Environmental Condition

$$U = 1.0D + 1.0L + 1.0W + 1.0T + P_L \quad (3)$$

$$U = 1.0D + 1.0L + 1.0E + 1.0T + P_L \quad (4)$$

Loading Under Normal Conditions

a. Concrete

$$U = 1.4D + 1.7L + P_L \quad (5)$$

$$U = 1.25\ (D + L + H_o + E) + 1.0T_o + P_L \quad (6)$$

$$U = 1.25\ (D + L + H + W) + 1.0T_o + P_L \quad (7)$$

$$U = 0.9D + 1.25\ (H + E) + 1.0T_o + P_L \quad (8)$$

$$U = 0.9D + 1.25\ (H + W) + 1.0T_o + P_L \quad (9)$$

For ductile moment resisting concrete frames and for shear walls

$$U = 1.4\ (D + L + E) + 1.0T_o + 1.25H_o + P_L \quad (10)$$

$$U = 0.9D + 1.25E + 1.0T_o + 1.25H_o + P_L \quad (11)$$

Structural Elements Carrying Mainly Earthquake Forces, Such as Equipment Supports

$$U = 1.0D + 1.0L + 1.8E + 1.0T_o + 1.25H_o + P_L \quad (12)$$

b. Structural Steel

$$D + L + P_L \text{ (stress limit} = f_s) \quad (13)$$

$$D + L + T_o + H_o + E + P_L \text{ (stress limit} = 1.25f_s) \quad (14)$$

Table 1 (Continued)

0406Lj

$$D + L + T_o + H_o + W + P_L \text{ (stress limit} = 1.33f_s) \tag{15}$$

In addition, for structural elements carrying mainly earthquake forces, such as struts and bracing:

$$D + L + T_o + H_o + E + P_L \text{ (stress limit} = f_s) \tag{16}$$

Loading Under Accident Conditions

a. Concrete

$$U = 1.05D + 1.05L + 1.25E + 1.0T_A + 1.0H_A + 1.0R + P_L \tag{17}$$

$$U = 0.95D + 1.25E + 1.0T_A + 1.0H_A + 1.0R + P_L \tag{18}$$

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_o + 1.25H_o + 1.0R + P_L \tag{19}$$

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_A + 1.0H_A + 1.0R + P_L \tag{20}$$

$$U = 1.0D + 1.0L + 1.0B + 1.0T_o + 1.25H_o + P_L \tag{21}$$

$$U = 1.0D + 1.0L + 1.0T_o + 1.25H_o + 1.0W' + P_L \tag{22}$$

b. Structural Steel

$$D + L + R + T_o + H_o + E' + P_L \text{ (stress limit} = 1.5f_s) \tag{23}$$

$$D + L + R + T_A + H_A + E' + P_L \text{ (stress limit} = 1.5f_s) \tag{24}$$

$$D + L + B + T_o + H_o + P_L \text{ (stress limit} = 1.5f_s) \tag{25}$$

$$D + L + T_o + H_o + W' + P_L \text{ (stress limit} = 1.5f_s) \tag{26}$$

where

U = required strength to resist design loads or their related internal moments and forces

For the ultimate load capacity of a concrete section, U is calculated in accordance with ACI 318-71.

F_y = specified minimum yield strength for structural steel

f_s = allowable stress for structural steel; f_s is calculated in accordance with the AISC Code, 1963 Edition for design calculations initiated prior to February 1, 1973.

f_s is calculated in accordance with the AISC Code, 1969 Edition, with Supplements, 1, 2, and 3 for design calculations initiated after February 1, 1973.

Table 1 (Continued)

040611

D - dead loads

L = live loads

P_L = load on structure due to jacking preload

R = local force or pressure on structure or penetration caused by rupture of any one pipe

T_o = thermal effects during normal operating conditons

H_o = force on structure due to thermal expansion of pipes under operating conditions

T_A = total thermal effects which may occur during a design accident other than H_A

H_A = force on structure due to thermal expansion of pipes under accident condition

E = operating basis earthquake (OBE)

E' = safe shutdown earthquake load (SSE)

B = hydrostatic forces due to the postulated maximum flood (PMF) elevation of 635.5 feet

W = design wind load

W' = tornado wind loads, including missile effects and differential pressure

Ø = capacity reduction factor

The capacity reduction factor (Ø) provides for the possibility that small adverse variations in material strengths, workmanship, dimensions, control, and degree of supervision, although individually within required tolerances and the limits of good practice, occasionally may combine to result in undercapacity.

NOTES:

1. In the load equations, the following factors are used:

   Ø = 0.90 for reinforced concrete in flexure

   Ø = 0.75 for spirally reinforced concrete compression members

   Ø = 0.70 for tied compression members

   Ø = 0.90 for fabricated structural steel

Table 1 (Continued)

040611

ϕ = 0.90 for reinforced steel in direct tension

ϕ = 0.90 for welded or mechanical splices of reinforcing steel

2. Unity load factor is shown for P_L. An alternative load factor to be considered in all load combinations is the load factor associated with dead load (D) in that loading combination.

For load combinations 23-26:

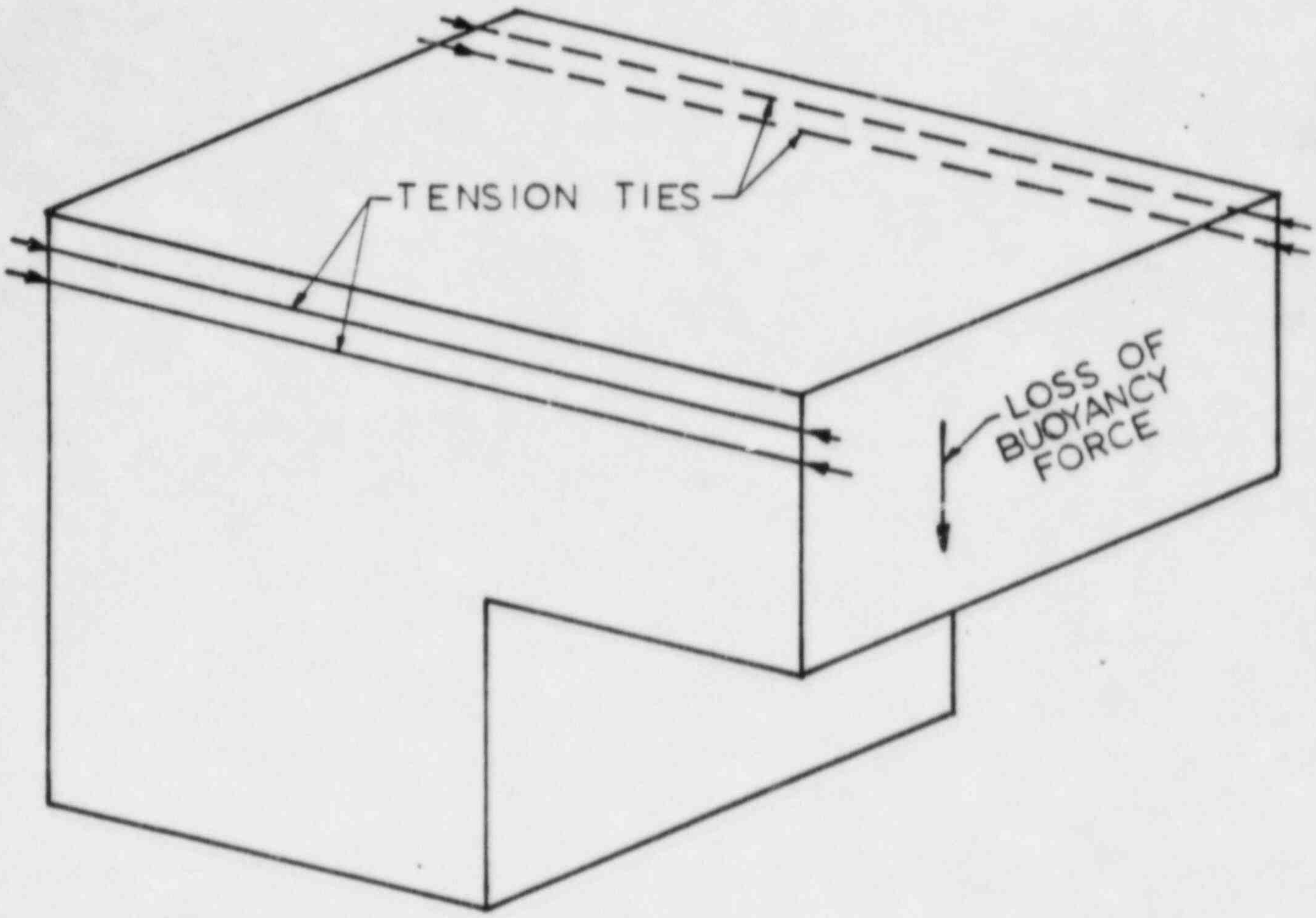
Maximum allowable stress in bending and tension is 0.9 F_y. Maximum allowable stress in shear is 0.5 F_y.

For these load combinations, the maximum allowable stress except for local areas that do not affect overall stability is limited to 0.9 F_y for bending, bearing, and tension and 0.5 F_y for shear. The time phasing between loadings is used where applicable to satisfy the above equations.

Structural components subjected to postulated impulse loads and/or impact effects are designed in accordance with BC-TOP-9-A, Rev 2, using ductility ratios not exceeding 10.

Structural members subjected to missile and pipe break loads are designed in accordance with Bechtel's BC-TOP-9-A, Rev 2, and Bechtel's BN-TOP-2, Rev 2.

040611



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

SERVICE WATER PUMP STRUCTURE
TENSION TIES

Figure 6

040611

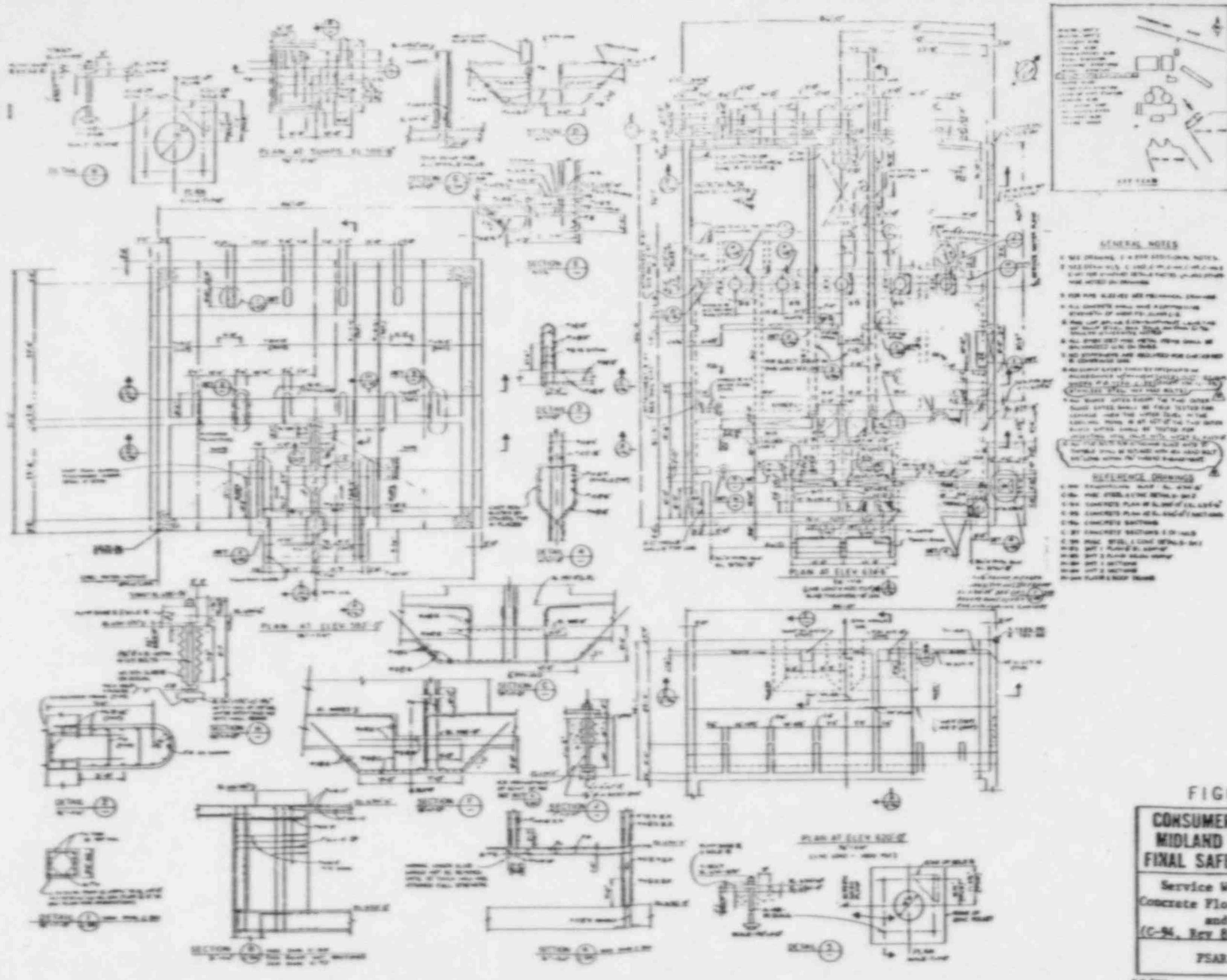


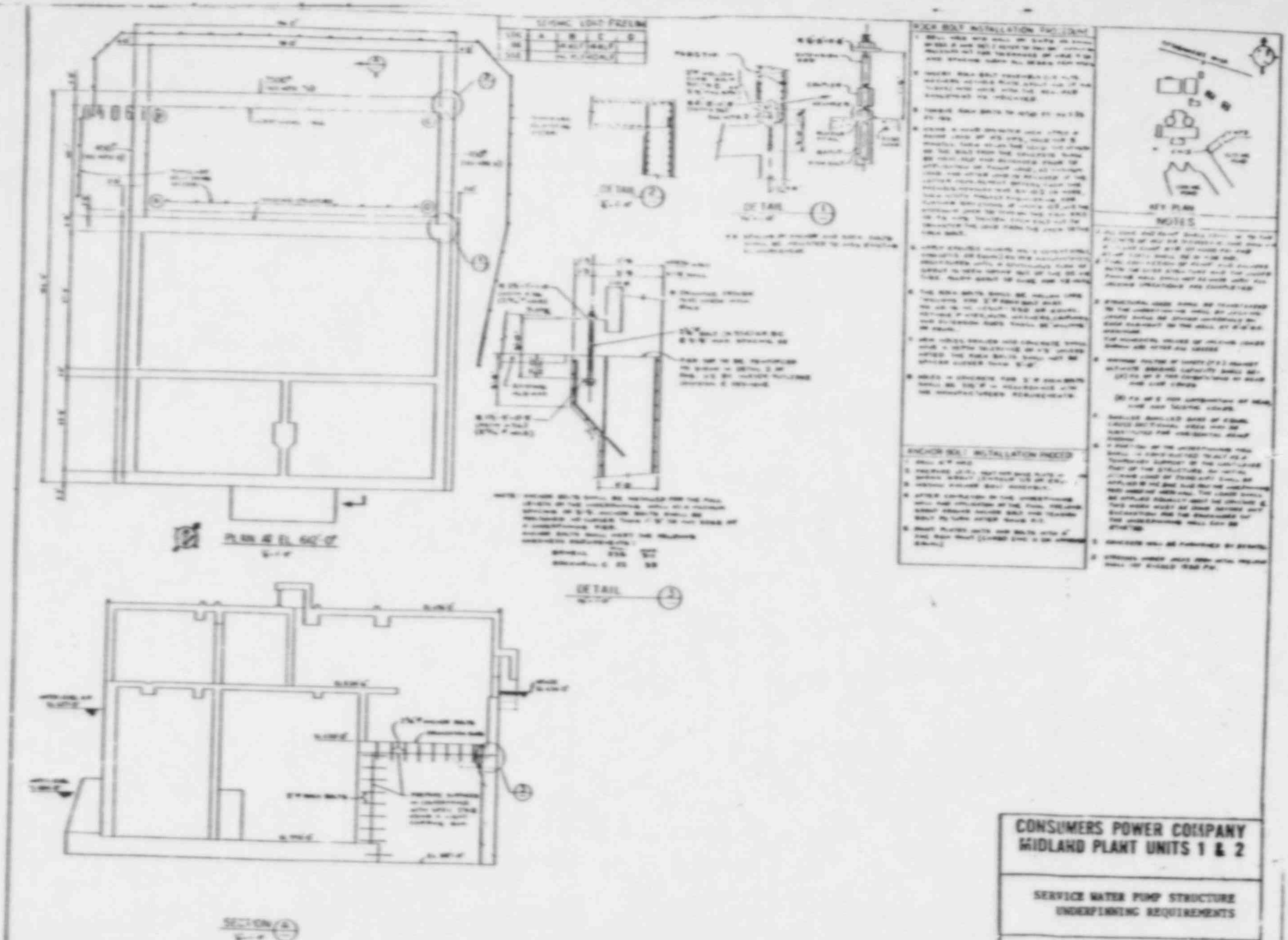
FIGURE 1

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Service Water Pump Structure
Concrete Floor Plans at El. 592'-0" and El. 634'-6"
(C-94, Rev 8)

FSAR Figure 3.8-56

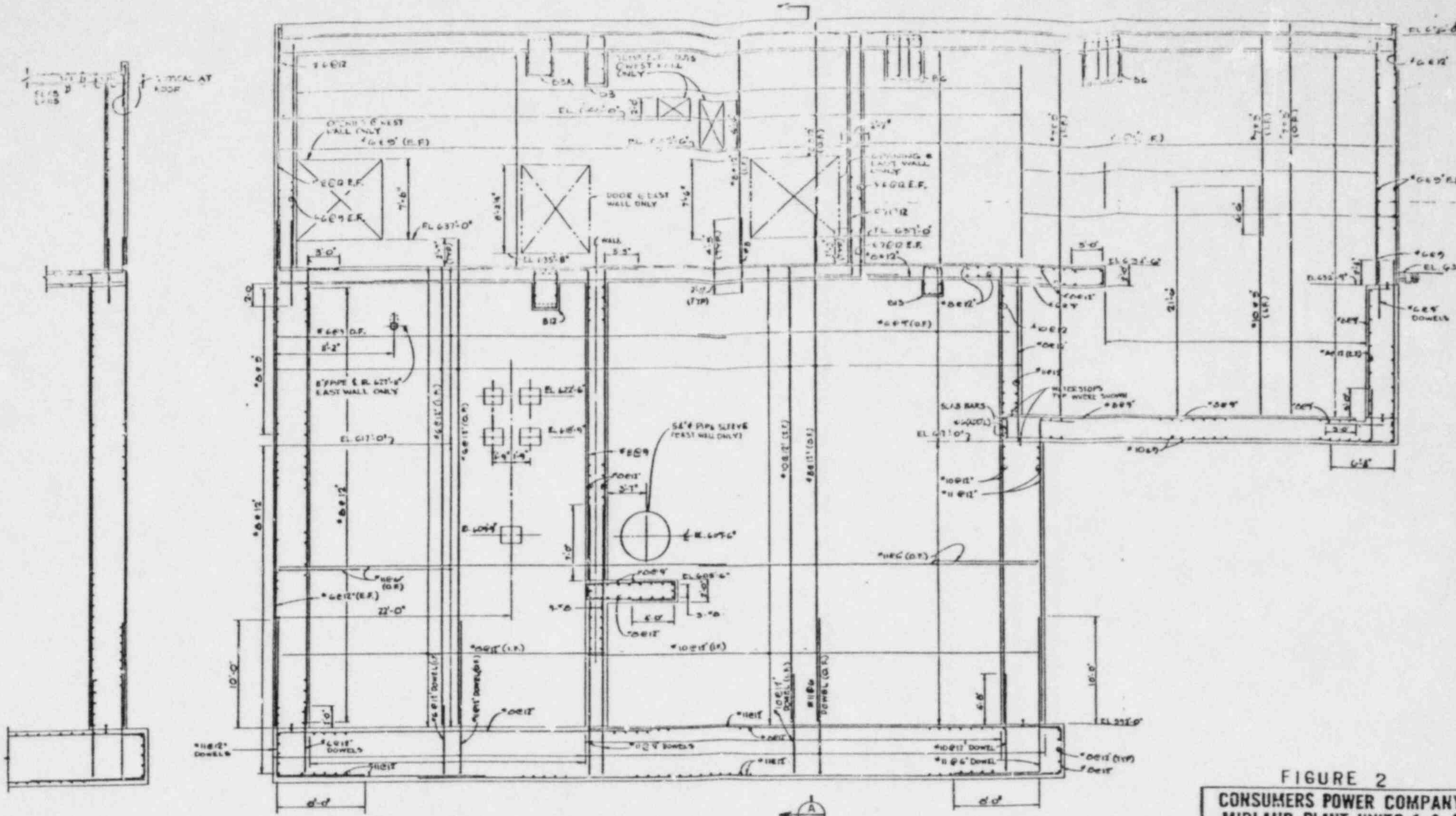
10/78 Revision 14



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

SERVICE WATER PUMP STRUCTURE
UNDERPINNING REQUIREMENTS

Figure 3



SECTION A

ELEVATION - WEST WALL
ELEVATION EAST WALL (OPP. HAND)

FIGURE 2

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Service Water Pump
Structure Section

(C-97, Rev 2)

FSAR Figure 3.8-57

* II
See Note on * I

~~Hoop~~
~~Kny~~
~~Jones~~
~~Landsman~~

James W Cook
Vice President - Projects, Engineering and Construction

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0453

June 19, 1981

PRINCIPAL STAFF			
DIR		F&IS	
D/D		PAO	
A/D		SLO	
DRSPI			
DE&TI			
DEP&OS		File	X

Mr Harold R Denton, Director
Office of Nuclear Reactor Regulation
US Nuclear Regulatory Commission
7920 Norfolk Ave
Bethesda, MD 20014

MIDLAND PROJECT
DOCKET NOS 50-329, 50-330
PRELIMINARY RESULTS OF SOIL BORING
AND TESTING PROGRAM FOR PERIMETER AND BAFFLE DIKE AREAS
FILE: 0485.16 UFI: 70*01*/00234(S) SERIAL: 12244
ENCLOSURES: (1) COMPARISON OF SOIL PROPERTIES USED IN THE FSAR WITH DATA RECEIVED FROM WOODWARD-CLYDE CONSULTANTS, DATED JUNE 1981
(2) COMPARISON OF SOIL PROPERTIES USED IN THE RESPONSE 50.54(f) (NEWMARK ANALYSIS) WITH DATA RECEIVED FROM WOODWARD-CLYDE CONSULTANTS, JUNE 1981
(3) PRELIMINARY TEST RESULTS, SOIL BORING AND TESTING PROGRAM, PERIMETER AND BAFFLE DIKE AREAS FOR MIDLAND - UNITS 1 AND 2

In accordance with our discussion with the NRC on May 13, 1981, we are providing the enclosed Woodward-Clyde Consultants' report dated June 10, 1981 which documents the preliminary soil boring and laboratory testing data for the fill and natural foundation materials in the perimeter and baffle dike areas. The report is marked "preliminary" because a few index property and undrained triaxial compression test results are not yet available. The Soil Boring and Testing Program was performed by Woodward-Clyde Consultants who have been retained by Consumers Power as an independent contractor.

The data presented on the dike areas can be considered essentially complete and representative of the materials from the dike areas. The soil properties obtained from the laboratory tests (Boring Nos COE-1 through COE-7 and COE-7A) and the property values used in the design of the dikes are provided in Tables 1 and 2. Table 1 indicates that the shear strength values of the fill and the natural foundation materials of the perimeter dike are higher than the conservative values used in the design of the dikes. The slope stability analysis of the perimeter dike with the new shear strength properties will result in a higher factor of safety than the factor of safety values reported in the FSAR Table 2.5-20.

B001
s
1/40

Table 2 indicates that the shear strength values of fill material and foundation clay for the baffle dike to an Elevation 598 are higher than the values used in the 50.54(f) response (Response to Question 45, Part 1f, Table

oc0681-0327a100

A
~~810623~~ ~~280~~

JUN 2 ? 1981

45-2) while the shear strength of foundation clay below Elevation 598 is slightly lower than the values used in the 50.54(f) response. The analysis done in the 50.54(f) response showed a high margin for 1.0g seismic acceleration and the acceleration values assumed for the Midland site are at least a factor of 5 less than 1.0g. Therefore the slightly less shear strength values do not in any way alter our conclusions in the 50.54(f) response.

We will continue to provide the NRC with the soil boring and laboratory test data and analysis of this data from other areas, such as the diesel generator building, when these results become available.

James W. Cook

JWC/NR/RLT/cr

oc0681-0327a100

CC Atomic Safety & Licensing Appeal Board w/o
Atomic Safety & Licensing Board Panel w/o
Charles Bechhoefer, Esq w/o
Myron M Cherry, Esq, w/o
RJCook, Midland Resident Inspector w/o
Dr Frederick P Cowan, w/o
Ralph S Decker w/o
NRC Docketing and Service Section w/o
Steve Gadler, w/o
RWHuston, w/o
Frank J Kelley, Esq, w/o
Wendell H Marshall, w/o
Michael I Miller, Esq w/a
W Otto, US Army Corps of Engineers, w/a
William D Paton, w/o
Mary Sinclair, w/o
Barbara Stamiris, w/o

oc0681-0327a100

TABLE - 1

Comparison of Soil Properties Used in the FSAR
with Data Received from
Woodward-Clyde Consultants, June 1981

PERIMETER DIKE

		Effective Stresses						Total Stresses					
		FSAR			Woodward-Clyde			FSAR			Woodward-Clyde		
Zone	Description	γ (pcf)	φ' (deg.)	c' (psf)	γ (pcf)	φ' (deg.)	c' (psf)	γ (pcf)	φ (deg.)	c (psf)	γ (pcf)	φ (deg.)	c (psf)
1	Impervious fill	135(1)	29	0	135	30.9	140		–		135	0	1600-6300(2)
2	Random fill	135(1)	29	0		–(3)			–			–(2)	
7	Foundation sand, silt, and firm clay	110(1)	32	0		–(4)			–			–	
8	Foundation glacial till	140(1)	35(5)	0	140	33.4	1810		–		140	0	11,000-25,000(6)
9	Foundation glacial till		–			–		125	0	7000(7)	140	0	11,000-25,000(6)

(1) See FSAR Table 2.5-22 for ranges of dry density for various zones. Values shown above were used in stability analysis.

(2) Values used in analysis June 1981 are c=2500 psf and φ =0.

(3) Samples taken from the Baffle Dike random fill-cohesive gave c'=190 psf and φ' =28.6° and random fill-granular gave c'=0 and φ'=35.1°.

(4) Samples taken from the Baffle Dike foundation clay gave an effective angle of internal friction of φ=25.3° and c'=780 psf however, borings COE 2, 3, 4, 5 and 6 drilled in the Perimeter Dike showed only foundation glacial till.

(5) FSAR Table 2.5-22 shows 37° while actual value used was 35°.

(6) Values used in analysis June, 1981 are c=11,000 psf and φ =0.

(7) c' used only in earthquake analysis of emergency cooling water reservoir slope (Section Z-Z').

TABLE - 2

Comparison of Soil Properties Used in the Response 50.54(f) (Newmark Analysis) with Data Received from Woodward-Clyde Consultants, June 1981

BAFFLE DIKE (SECTION Y-Y')

		Total Stresses					
		Response 50.54(f)			Woodward-Clyde		
Zone	Description	γ (pcf)	ϕ (deg.)	c (psf)	γ (psf)	ϕ (deg.)	c (psf)
2	Random fill	130	0	1000-2000	130	0	2100-4200
	" "	130	25[1]	0		-[1]	
7	Foundation clay El 598 to El 604	132	0	3500	140	0	3200-5600[2]
8A	Foundation clay El 590 to El 598	132	0	6000	140	0	5300-6300[2]
8B	Foundation clay Below El 590	140	0	7000	140	0	6500[2]

(1) The random fill materials were assumed to have effective angle of internal friction of 29° but for the total stress analysis, an angle of 25° was used. The tests from Woodward-Clyde showed an effective angle of internal friction of ϕ '=35.1° and c'=0 for random fill-granular and an effective angle of internal friction of ϕ '=28.6° and c'=190 psf for random fill-cohesive.

(2) For the Perimeter Dike (section T-T'), glacial till with shear strength ranging from 11,000 to 25,000 psf was encountered in borings COE 2, 3, 4, 5 and 6.

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

APR 16 1981

Docket Nos.: 50-329/330 OM, OL

MEMORANDUM FOR: F. J. Miraglia, Acting Chief, Licensing Branch No. 3, DL

FROM: D. S. Hood, Project Manager, Licensing Branch No. 3, DL

SUBJECT: NOTICE OF MEETING - MIDLAND PLANT, UNITS 1 AND 2

DATE & TIME: May 5-7, 1981
8:30 a.m.

LOCATION: Rm. P-114 (5th), Rm. P-114 (6th), Rm. P-422 (7th)
Phillips Building
Bethesda, Maryland

PURPOSE: TO DISCUSS (1) REMEDIAL ACTIONS AND UPDATE OF SOILS DEFICIENCIES, AND (2) REVIEW STATUS OF AMENDMENT 85 OF APPLICATION

PARTICIPANTS[1/]:

NRC
G. Lear
F. Schauer
R. Bosnak, et.al.

NRC Consultants

U. S. Army Corps of Engineers

Naval Surface Weapons Center

Energy Technology Engineering Center

Consumers Power Company
G. Keeley, et.al.

Bechtel
L. Curtis, et.al.

Darl Hood

Darl S. Hood, Project Manager
Licensing Branch No. 3
Division of Licensing

~~8105060085~~

Enclosures:
1. Background and Agenda
2. 3/23/81 Letter

cc: See next page.

[1/] Meetings between NRC technical staff and applicants for licenses are open for interested members of the public, petitioners, intervenors, or other parties to attend as observers pursuant to "Open Meeting and Statement of NRC Staff Policy", 43 Federal Register 28058, 6/28/78.

BACKGROUND

The meeting is in response to the applicant's letter of March 23, 1981 (attached) to further discuss the matters therein. However, the matter of seismic design criteria is the subject of separate meetings 1/.

Amendment 85 of the Midland application, which responds to several requests for additional information (Questions 39 through 53) regarding soils deficiencies, will also be discussed, with particular emphasis upon the identification and resolution of open items 2/ from the review of that amendment by the NRR staff and its consultants.

AGENDA

1. Dewatering - installation of intercepter back-up wells and permanent dewatering system design.
2. Underground Piping - Planned activities and results of investigation to date.
3. Additional Soil Borings - Update on borings.
4. BWST - Causes of ring beam foundation cracks and remedial activities.
5. Review of Amendment 85 (Soils, Structural)
6. Remedial Activities
   (a) Service Water Building
   (b) Auxiliary Bldg. Electrical Penetration area and FW Valve Pit.

1/ A previous Notice of Meeting has scheduled April 16, 1981 for discussion of the first two of three reports on Midland seismic criteria; scheduling of a subsequent meeting for the third report is pending receipt of that report by NRC.

2/ See Transcript for Oral Deposition of Joseph Kane, NRC, on 3/26,27/81.

ACTION
Schedule Meetings

James W Cook
Vice President - Projects, Engineering and Construction

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0453

March 23, 1981

RECEIVED
MAR 25 1981
U.S. NUCLEAR REGULATORY COMMISSION

Harold R Denton, Director
Office of Nuclear Reactor Regulation
US Nuclear Regulatory Commission
Washington, DC 20555

MIDLAND PROJECT
ACTIVITIES FOR THE RESOLUTION OF OUTSTANDING ISSUES REGARDING THE MIDLAND SOILS HEARINGS
FILE: 0485.16, B2.5.2 UFI: 42*05*22*01, 00234S, 71*01 SERIAL: 11625

This letter is submitted to document the telephone conversation of February 27, 1981 between myself, R H Vollmer, and members of both our staffs. The call addressed several matters relating to the Midland soils hearings. In order to put the items discussed in context, a brief background summary is presented below.

On August 22, 1978 Consumers Power Company verbally notified the Region III Resident Inspector that the partially completed diesel generator building was experiencing more settlement than had been postulated. This was later determined to be due to inadequate compaction of backfill. A 50.55(e) report was initially issued on September 29, 1978 and further interim 50.55(e) reports were issued until the last report of February 7, 1980, after which subsequent information was supplied by 50.54(f) responses.

On March 21, 1979 the NRC issued the initial 50.54(f) request regarding plant fill and subsequent requests were issued. Answers to most of these 50.54(f) questions have been forwarded with the latest being Amendment 88 (Rev 11 to the 50.54(f) responses) dated March 16, 1981. On December 6, 1979 an Order Modifying Construction Permits No CPPR-81 and CPPR-82 was issued by the NRC. A principal reason this order was issued was due to the Staff's erroneous assumption that remedial actions, other than the surcharging of the diesel generator building, were proceeding. On December 26, 1979 Consumers Power Company requested a hearing in accordance with Part V of the Order.

On October 14, 1980 a letter from R L Tedesco to us indicated that one of the open items associated with the review of our operating licenses for Midland Units 1 and 2 was the establishment of additional seismological input parameters against which to review the plant structures and equipment. The letter stated that consideration of this open item would also be introduced into the review of the remedial actions associated with the soils settlement matter which was the subject of the December 6, 1979 Order Modifying Construction Permits.

oc0381-0258a100

B001
S 1/1

8103260700

Since our initial notification to the NRC about the soils problem, there have been many meetings and telephone conversations to discuss the proposed remedial actions and the responses to questions by the NRC and their consultants. In addition, depositions have been taken by Consumers and the NRC.

During all the above activities it has become apparent that there are some areas of disagreement between the Staff and its consultants and Consumers and its consultants. In addition, the October 14, 1980 letter from R L Tedesco on a seismic response spectra has placed us in the position of having to evaluate our remedial fixes against an unknown, but possibly higher margin, since the site specific response spectra issue could result in structural loads larger than those resulting from an SSE zero period acceleration of 0.12g and modified Housner spectra which are the PSAR and FSAR design basis.

On February 27, 1981 I initiated the referenced call to R H Vollmer and other Staff personnel to inform the Staff of new developments. We hope these actions will help resolve certain issues that to date have been in contention with the Staff. We also hope that the Staff will look favorably on our requests to pursue with your concurrence certain activities which if not undertaken shortly will have a significant adverse schedule impact.

1. BORINGS

   While we still disagree with the need to take additional borings and run tests, we will take borings as specified in the January 8, 1981 letter to us from R L Tedesco. Consolidation tests by an independent laboratory will be run on soils samples taken near the diesel generator area to obtain pre-consolidation pressures, and comparisons will be made to the calculated stresses to which the soils in the areas of the samples were subjected during the surcharge program. An evaluation of these tests and results will be undertaken to assess the level of uncertainty inherent in these data. Shear strength tests will be run on soils samples taken in the power block area to determine factors of safety for bearing capacity. Shear strength tests will also be run on soils samples taken from dike borings to substantiate slope stability. We will keep the Staff informed of all the above activities so that they can witness the activities, if desired. The results of the test program will be submitted and reviewed with the Staff.

2. SERVICE WATER BUILDING AND ELECTRICAL PENETRATION AREAS

   The October 14, 1980 R L Tedesco letter on seismic, accelerated the completion of a margin analysis of the underpinning systems proposed as remedial actions. While all structural analyses have shown the fixes to be adequate for the plant seismic design basis of .12g, there was concern that a seismic margin analysis based on the currently undefined site specific response spectra would introduce new potential areas of contention with the Staff. As a result we have decided to change from a pile support design for the overhang portion of the service water building to an underpinning concept involving a full length wall under the overhang portion with the wall extending into the till. We are confident that this will provide sufficient margin for any reasonable resolution of the site specific response spectra issue. The conceptual design for this approach will be available to allow discussions with the Staff in April 1981.

The remedial action under the electrical penetration area will remain essentially the same as has been described both in discussions and in answers to Staff questions, except that more caissons and some enlargement of the base of the pier under the valve pit will be utilized in order to obtain additional margin.

3. PERMANENT PLANT DEWATERING

Although it is our legal opinion that we can implement remedial actions at our own risk without Staff concurrence, we have chosen not to proceed without their knowledge and concurrence. The single pacing activity for the entire sequence of installing the remedial underpinnings is the completion of the permanent plant dewatering system. The first phase of this activty is the installation of a few back-up wells commencing in May of 1981. A large amount of information on the permanent plant dewatering system has been provided to the Staff. Installation of back-up wells along the service water and circulating water buildings will facilitate draw down and recharge rate tests, verify the design of the remainder of the permanent plant dewatering system, provide dewatering settlement data, and facilitate preparation for installation of the wall under the overhang portion of the service water structure.

Since the wells can be abandoned and grouted, we do not believe it is necessary to consider the installation of wells as an irrevocable committment.

We request that the Staff concur with our position and that we so notify the Soils Licensing Board.

4. SITE SPECIFIC SEISMIC CRITERIA

We have had several discussions with the Staff on this subject, and as previously requested by them we supplied them with the Final Report Part I "Response Spectra - Original Ground Surface" and Part III "Seismic Hazard Analysis". Part II entitled "Response Spectra For Top of Fill and Theoretical studies on possible Ground Motion Amplification Through Fill under the Diesel Generator Building" will be furnished by April 1981. As already scheduled, we will be meeting with the Staff on these issues on April 16, 1981.

Our objective is not only to resolve the site specific response spectra with the Staff but also to recognize and schedule with the Staff management the total sequence of seismic margin analysis activities that are currently required in the operating licensing process.

We are also petitioning the soils hearing Licensing Board to remove the seismic issue from that hearing and urge the Staff to consider our motion and join with us if possible.

In prior conversations with Mr Vollmer on the general topic of resolution of issues, it was anticipated that the Staff could support an expedited review of the underpinning designs. Based on the scheduled submittals of Attachment 1, we are hopeful that as much staff review of these materials as possible can be accomplished prior to the hearing while still reflecting the limitation of Staff resources. We will be in contact with the NRC's Midland project manager to pursue in detail the additional submittals and meetings referenced in this letter.

James W. Cook

JWC/GSK/cr

CC: RJCook, Midland Resident Inspector
Atomic Safety & Licensing Appeal Board
Atomic Safety & Licensing Board Panel
Charles Bechhoefer, Esq
James E Brunner, Esq
Myron M Cherry, Esq
Dr Frederick P Cowan
Mr Steve Gadler
D F Judd, Sr Project Manager
Frank J Kelley, Esq
Ralph S Decker
Mr Wendell H Marshall
Michael Miller, Esq
William D Paton, Esq
Ms Mary Sinclair
Barbara Stamiris
Mr C R Stephens
Chief, Docketing & Service Section

oc0381-0258a100

ATTACHMENT 1
NEAR TERM SCHEDULE MILESTONES FOR
ACTIVITIES RELATED TO SOILS HEARINGS

A. SEISMIC DESIGN CRITERIA

1. Submit Weston Report Part I and III to NRC. Completed and provided to NRC on 3/3/81.

2. Submit Part II in April 1981.

3. Meet with NRC Staff in April to discuss resolution of issues. Also discuss schedule for resolution of these issues with respect to Operating License.

B. PERMANENT PLANT DEWATERING

1. Drill and develop back-up wells along service water and circulating water pump house 5/1/81 start.

2. Drill and development remainder of permanent plant dewatering wells. 11/1/81 start.

C. AUXILIARY BUILDING

1. Meet with NRC on conceptual design April 1981.

2. Complete conceptual design 6/1/81.

3. Complete final design 8/1/81.

NOTE: Construction activities are scheduled to the following milestones:

Award subcontract 1/1/82; Mobilize 4/1/82; Start Excavation and installation 6/1/82; Complete April 1983.

D. SERVICE WATER PUMP STRUCTURE

1. Meet with NRC on conceptual design April 1981.

2. Complete conceptual design April 1981.

3. Complete design 6/15/81.

NOTE: Construction activities are scheduled to the following milestones:

Award subcontract 1/1/82; Mobilize 9/1/82; Start Excavation and installation 11/1/82; Complete May 1983.

E. UNDERGROUND UTILITIES

Meet with NRC in April 1981 on results of discussions with Consultants and discuss schedule for completion of investigation.

oc0381-0258b100

F. BORINGS

1. Issue specification and retain subcontractor 3/23/81.
2. Commence Borings week of 3/23/81.
3. Commence Lab Testing week of 3/30/81.
4. Complete Borings 5/1/81.
5. Complete Lab Testing 6/8/81.
6. Periodically review results of detailed program 3/23/81 to 6/8/81, with NRC.

G. BORATED WATER STORAGE TANKS

1. Meet with NRC on remedial actions April 1981.