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August 31, 1984

United States Nuclear Regulatory Commission
Washington, DC 20555

ATTENTION: Mr. George W. Knighton, Chief
Licensing Branch 3
Office of Nuclear Reactor Regulation

SUBJECT: Beaver Valley Power Station - Unit No. 2
Docket No. 50-412
Response to Draft SER Open Item No. 172

Gentlemen:

The response to the NRC Geotechnical Engineering Section's Draft SER Open Item No. 172 is provided in Attachment 1. The associated revisions to FSAR Section 2.5.4 are provided in Attachment 2.

DUQUESNE LIGHT COMPANY

By E. J. Woolever
E. J. Woolever
Vice President

JDO/wjs
Attachments
cc: Ms. M. Ley, Project Manager (w/a)
Mr. E. A. Licitra, Project Manager (w/a)
Mr. G. Walton, NRC Resident Inspector (w/a)

COMMONWEALTH OF PENNSYLVANIA)
) SS:
COUNTY OF ALLEGHENY)

On this 31st day of August, 1984, before me, a Notary Public in and for said Commonwealth and County, personally appeared E. J. Woolever, who being duly sworn, deposed and said that (1) he is Vice President of Duquesne Light, (2) he is duly authorized to execute and file the foregoing Submittal on behalf of said Company, and (3) the statements set forth in the Submittal are true and correct to the best of his knowledge.

Anita Elaine Reiter
Notary Public

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ANITA ELAINE REITER, NOTARY PUBLIC
ROBINSON TOWNSHIP, ALLEGHENY COUNTY
MY COMMISSION EXPIRES OCTOBER 20, 1986

ATTACHMENT 1

Draft SER Open Item No. 172 (Sections 2.5.4.1.2 and 2.5.4.5) - Longitudinal Sections and Parameters of Category I Buried Pipelines:

Section 2.5.4.1.2:

The applicant has agreed to furnish longitudinal sections of all Category I pipelines (1) from the Valve Pit No. 1 to the main plant structures, and (2) from the main plant area to the Emergency Outfall structure. These sections should show the soil profile and the static and dynamic soil properties used in the pipe stress analysis, such as the subgrade modulus, shear wave velocity, shear modulus, etc.

Section 2.5.4.5:

The major items that need to be addressed by the applicant in the forthcoming amendment of the FSAR are the following:

1. Furnish longitudinal sections of Category I pipelines and ducts not already provided showing therein the soil profile and the elevations at which the pipes are laid. Locations of manholes and their foundation configuration should also be shown in these longitudinal sections.

2. Provide the actual values of the geotechnical parameters such as subgrade modulus, shear wave velocity and soil modulus, etc. used in the static and dynamic analysis of buried pipes.

Response:

Longitudinal sections along Category I pipelines and locations of duct-lines on soil profiles were provided in response to FSAR Question 241.2 in FSAR Amendment 6.

Static and dynamic properties of in situ soils and compacted fill are provided in FSAR Section 2.5.4.4 and revised FSAR Sections 2.5.4.5 and 2.5.4.7. The revised FSAR sections are provided in Attachment 2 and will be incorporated into FSAR Amendment 8. The effect of these revisions on FSAR Section 3.7B.3.12 will be addressed in FSAR Amendment 9.

Soil Properties

The dry unit weight of compacted structural fill was taken as 130 pcf, corresponding to 95 percent of the mean maximum dry density from 115 moisture density tests.

The specific gravity was taken as 2.65.

The void ratio was computed to be 0.27.

The saturated unit weight below the ground-water table was taken as 144 pcf from the equation:

$$\gamma_T = \frac{G + Se}{1+e} \cdot \gamma_w \qquad (2.5.4\text{-}4)$$

where:

γ_T = total unit weight (pcf)
G = specific gravity
S = degree of saturation, decimal (100%)
e = void ratio
γ_w = unit weight of water = 62.4 pcf

Above the ground-water table, the total unit weight was taken as 136 pcf assuming an average water content of 5 percent.

The angle of internal friction of compacted structural fill was conservatively assumed to be 36 degrees.

Low strain shear moduli were estimated using Equation 2.5.4-5 as follows (Hardin and Drnevich 1972):

$$G = \frac{1{,}230\ (2.97-e)^2\ (\bar{\sigma}_0)^{0.5}}{1+e} \qquad (2.5.4\text{-}5)$$

where:

G = shear modulus (psi)
$\bar{\sigma}_o$ = effective octahedral stress (psi)

2.5.4.6 Ground-water Conditions

Regional and local aquifer characteristics are described in detail in Section 2.4.13.

Insert 2.5.4.5A
(pages 2.5.4.9a and 2.5.4-9b)

Insert 2.5.4.5A

The vertical coefficient of subgrade reaction for buried pipe was computed according to the following equation (Vesic 1961, 1961a):

$$k = \frac{0.65}{D_o} \sqrt[12]{\frac{E_s \; D_o^4}{E_p \; I_p}} \left[\frac{E_s}{1-\nu^2}\right] \quad (2.5.4\text{-}5a)$$

where: k_v = vertical coefficient of subgrade reaction (lb/in^3)
D_o = outside diameter of pipe (in)
E_s = Young's modulus of soil (lb/in^2)
E_p = Young's modulus of pipe (lb/in^2)
I_p = moment of inertia of pipe section (in^4)
ν = Poisson's ratio of soil

An average, low strain value of shear modulus, G, was estimated using equation 2.5.4-5 for two ranges of pipe embedment depth, H_e:

$$H_e < 15 \text{ ft}; \; G = 2250 \text{ ksf}$$

$$15 \text{ ft} \leq H_e < 30 \text{ ft}; \; G = 4350 \text{ ksf}$$

Using these values of shear modulus, Young's modulus, with a reduction to account for strain, was estimated as:

$$E_s = 2(1 + \nu)\frac{G}{3} \quad (2.5.4\text{-}5b)$$

Vertical coefficient of subgrade reaction is shown in Figure 2.5.4-62 as a function of depth of embedment and pipe diameter.

The horizontal coefficient of subgrade reaction for buried pipe was determined according to the empirical procedure described by Audibert and Nyman (1977). An analytical procedure was developed to determine the horizontal load-displacement (p-y) curve for any size pipe embedded at any given depth. Considering the horizontal coefficient of subgrade reaction as the amount of soil pressure reaction generated by a given amount of horizontal displacement (that is, as a secant to the p-y curve), the coefficient of horizontal subgrade reaction can be expressed by:

$$k_h = \frac{p}{y} = \frac{1}{A' + B'y} \quad (2.5.4\text{-}5c)$$

where: k_h = horizontal coefficient of subgrade reaction (lb/in^3)
p = pressure (lb/in^2)
y = displacement (in)
$A' = \frac{0.145 \; y}{q_u}$ (in^3/lb)

$$B' = \frac{0.855}{q_u} \ (in^2/lb)$$

y_u = ultimate displacement (in)

q_u = ultimate soil resistance (lb/in^2)

Considering the buried pipe as a horizontal footing, the ultimate soil resistance, q_u, is computed as:

$$q_u = \gamma Z N_q \quad (2.5.4\text{-}5d)$$

where: q_u = ultimate soil resistance (lb/in^2)

γ = unit weight of soil around pipe (lb/in^3)

Z = depth to center of pipe (in)

N_q = bearing capacity factor

The bearing capacity factor is given on Figure 2.5.4-63. The ultimate displacement, y_u, was evaluated from Figure 2.5.4-63. The iterative procedure used to calculate displacements assumes an inital value of displacement in order to compute an initial value of k_h. Then, using this initial value of k_h, an actual displacement is computed. This procedure continues until the iterative values converge at a final displacement.

resistance to liquefaction of the in situ sands and gravels at the site was investigated by two methods:

1. Based on dynamic triaxial tests on sands susceptible to liquefaction (DLC 1972e), and

2. Based on the observed behavior of sand deposits in previous earthquakes (DLC 1976).

The results of dynamic triaxial tests upon Sacramento River sand, considered to be extremely susceptible to liquefaction, are presented on Figure 2.5.4-28 (DLC 1972e). The figure shows the relationship between shearing stresses, expressed as a ratio of shear stress to effective stress, to the number of cycles necessary to cause initial liquefaction for this sand at several relative densities. It was used to evaluate the liquefaction potential of the soils within the main plant area as described in Section 2.6.5.2 of the BVPS-2 PSAR (DLC 1972e). This approach was conservative since the Sacramento River sand was considered especially susceptible to liquefaction in comparison to the sands and gravels at the site.

After the discovery of the loose zone in the main plant area and its subsequent densification, a liquefaction analysis was performed for soils within the densified zone (DLC 1976). The shear stress required to cause liquefaction of the in situ sands and gravels was evaluated using Figure 2.5.4-29. This figure presents a lower bound envelope for sites where liquefaction has occurred during earthquakes of Richter Magnitude 5.5 or less, correlated with corrected standard penetration resistance, N_1, of the sand deposit. This figure was used to evaluate the resistance to liquefaction of the soils in the vicinity of the intake structure as well. Further discussion is presented in Section 2.5.4.8.

2.5.4.7.4 Relative Displacements

The procedure used to evaluate the relative displacements between two structures during a seismic event is discussed in this section. It was assumed that the relative displacement will result from the horizontal propagation of seismic waves with little or no change in wave form. It was further assumed that the maximum particle motions produced by each wave will occur simultaneously. The procedure only determines the magnitude of displacement without consideration for direction.

For soil sites such as BVPS, relative displacements are caused by Rayleigh waves and Love waves. The particle motion for the Rayleigh wave occurs in the vertical plane and is elliptical and retrograde with respect to the direction of propagation. By their nature, Rayleigh waves cause horizontal push-pull (R_x) and vertical (R_z) displacements. The particle motion of Love waves is transverse to the direction of propagation and as a result, they are the cause of translational (R_y) displacements.

Insert 2.5.4.7A
(page 2.5.4-13a)

Insert 2.5.4.7A

The maximum Rayleigh wave velocity used in the analysis of buried pipe was determined to be 3000 ft/sec, using the procedure described below:

Ewing et al. (1957) presented data, produced on Figure 2.5.4-31, which showed that Rayleigh wave velocity in a layered system was a complicated function of the depth of soil, the shear wave velocity of soil and rock, and the frequency/wave length of the Rayleigh wave. Using Figure 2.5.4-31, for $C_2/C_1 = 4.5$, the variation of Rayleigh wave velocity with frequency for the in situ soil conditions in the main plant area was determined and is shown on Figure 2.5.4-64. Rayleigh wave velocity is seen to vary widely depending on frequency.

Since an earthquake is likely to produce Rayleigh waves of many frequencies, the selection of a control value of Rayleigh wave velocity was based upon a consideration of the predominant frequency likely to be produced by an earthquake occurring near the site.

The peaks of Fourier spectra for earthquake time histories represent frequencies at which large amounts of energy are released by the earthquake. Housner (1970) compared Fourier spectra with velocity response spectra and found that the peaks occurred at about the same frequencies. Accordingly, a predominant frequency of 2-3 H_Z was determined from response spectra presented in SWEC (1984). These response spectra were computed for real earthquake time histories, with magnitudes corresponding to the BVPS-2 SSE, that were amplified through the BVPS-2 soil profile. From Figure 2.5.4-64, a frequency of 2-3 H_Z corresponds to a Rayleigh wave velocity of about 3000 ft/sec.

profiles begin to level out, the period between readings will be increased.

Leveling loops run for settlement monitoring must close to one of the permanent bench marks with a maximum error of ±0.005 foot.

2.5.4.13.4 Data Processing

Data processing is accomplished using a SWEC computerized data storage system entitled Settlement Monitoring System (IS-233). The settlement marker elevations are input into the computer storage files and a computer printout providing the complete settlement record of each marker is produced. A specimen page of output is given on Figure 2.5.4-49.

For each settlement marker, settlement versus time plots have been prepared using arithmetic and log time scales. These plots are not included herein but are provided in the report on Settlement Monitoring Program (DLC 1980). A summary of the observed settlements to date is provided on Figure 2.5.4-46.

The Ohio River elevation and piezometer data is included in Appendix 2.5A.

2.5.4.14 Construction Notes

The removal of uncontrolled fill placed during the construction of SAPS and BVPS-1 is discussed in Section 2.5.4.5. The removal of a lens of stiff silty clay found during the reactor containment excavation is also discussed in Section 2.5.4.5.

A zone of loose granular material was discovered in the BVPS-2 area during the excavation for the reactor containment excavation. It was densified using the pressure injected footing technique. The densification program and its evaluation are fully described in the Report on Soil Densification Program, (DLC 1976).

2.5.4.15 References for Section 2.5.4

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Insert "A" (page 2.5.4-35a)

Insert "B"
(page 2.5.4-35a)

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Insert "D" (page 2.5.4-35a)

Insert "C" (page 2.5.4-35a)

Insert "A"

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Insert "B"

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Insert "C"

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Insert "D"

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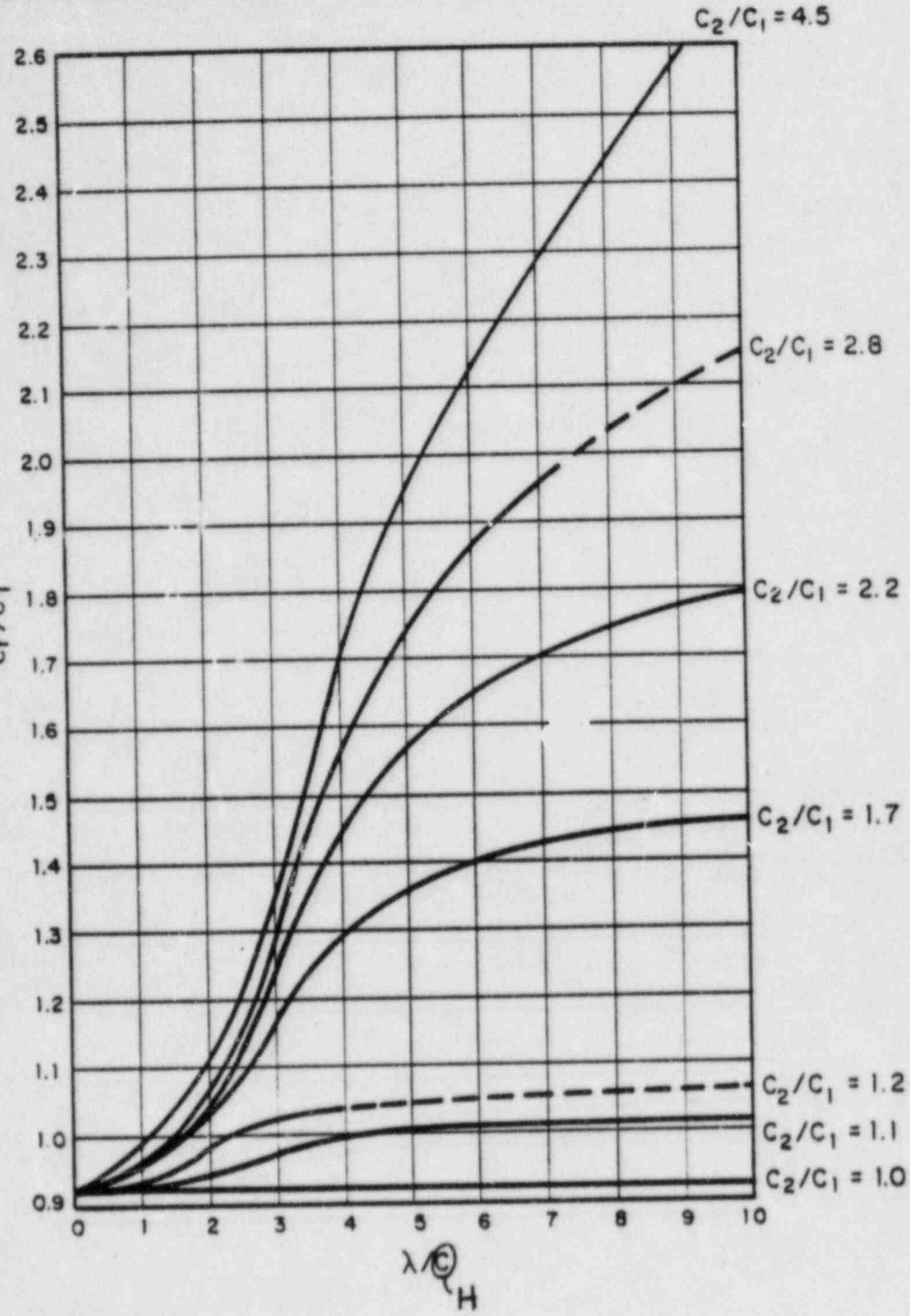


LEGEND:

C_r = RAYLEIGH WAVE VELOCITY

C_1 = SHEAR WAVE VELOCITY OF UPPER LAYER

C_2 = SHEAR WAVE VELOCITY OF LOWER DENSER LAYER

H = THICKNESS OF UPPER LAYER

λ = WAVELENGTH; 2b FOR PARALLEL WAVES; 2b SIN 45° FOR OBLIQUE WAVES.

b = CENTROIDAL DISTANCE BETWEEN STRUCTURES

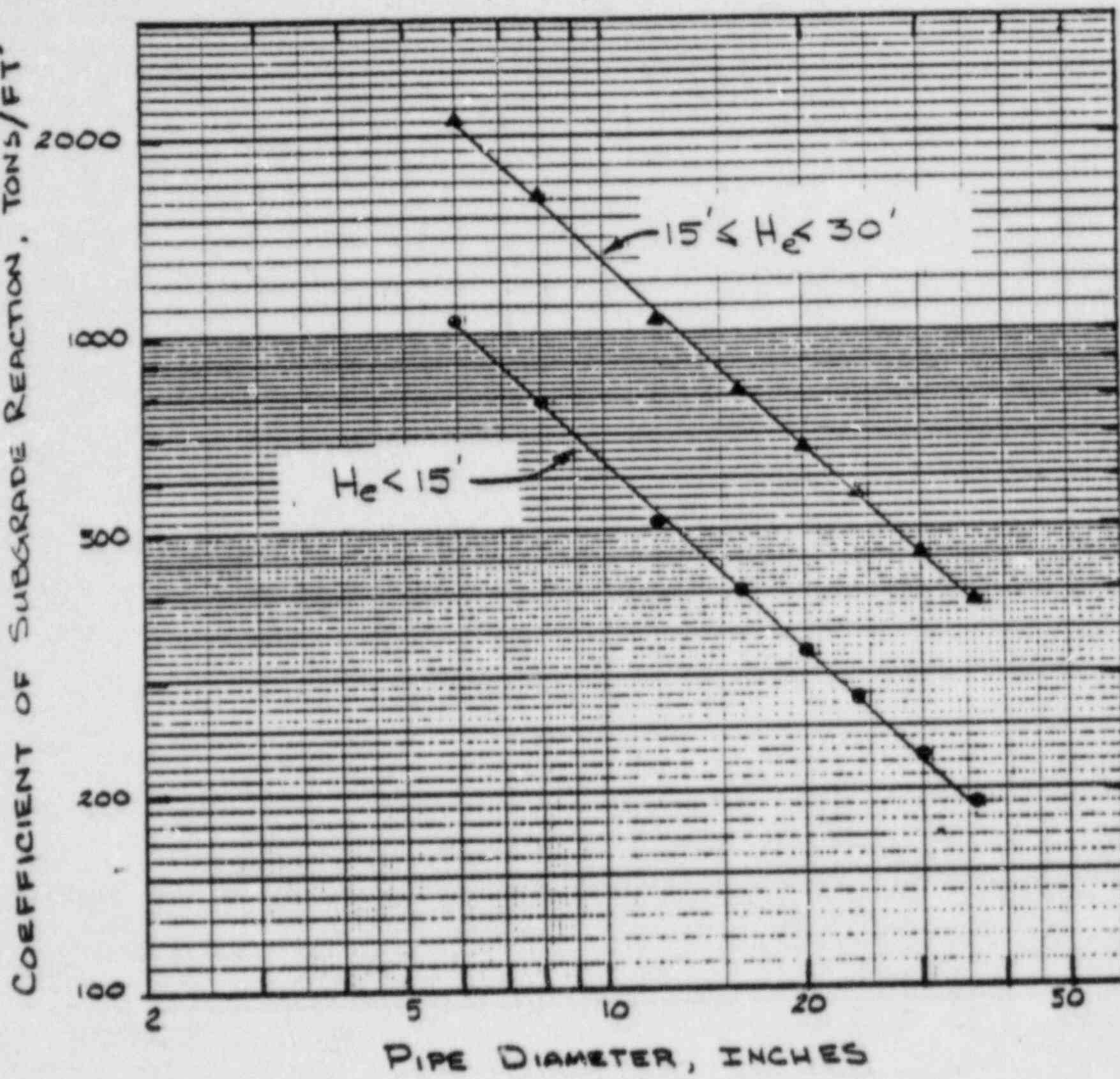
Ref.: Ewing et al. 1957

FIGURE 2.5.4-31

RELATIONSHIP BETWEEN RAYLEIGH WAVE VELOCITY AND SOIL PARAMETERS

BEAVER VALLEY POWER STATION-UNIT 2

FINAL SAFETY ANALYSIS REPORT



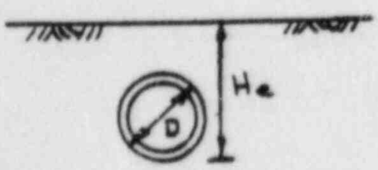


FIGURE 2.5.4-62

		TITLE VERTICAL COEFFICIENT OF SUBGRADE REACTION FOR BURIED PIPE	SCALE:
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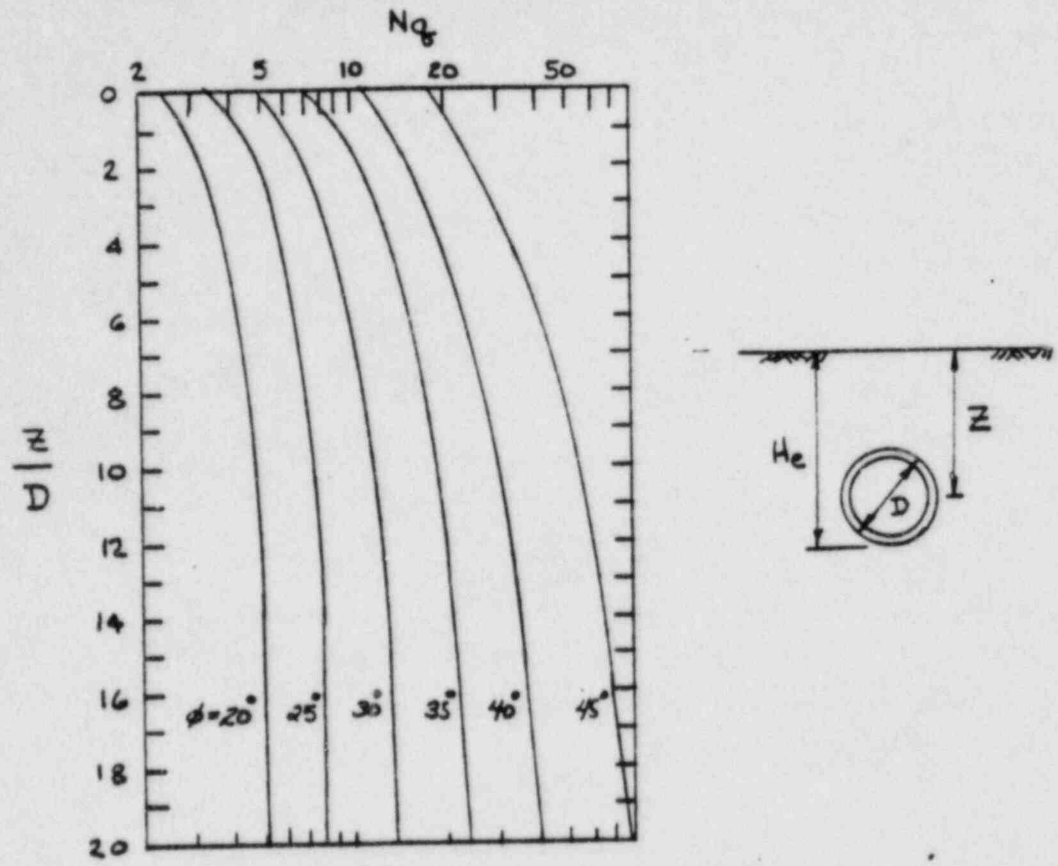
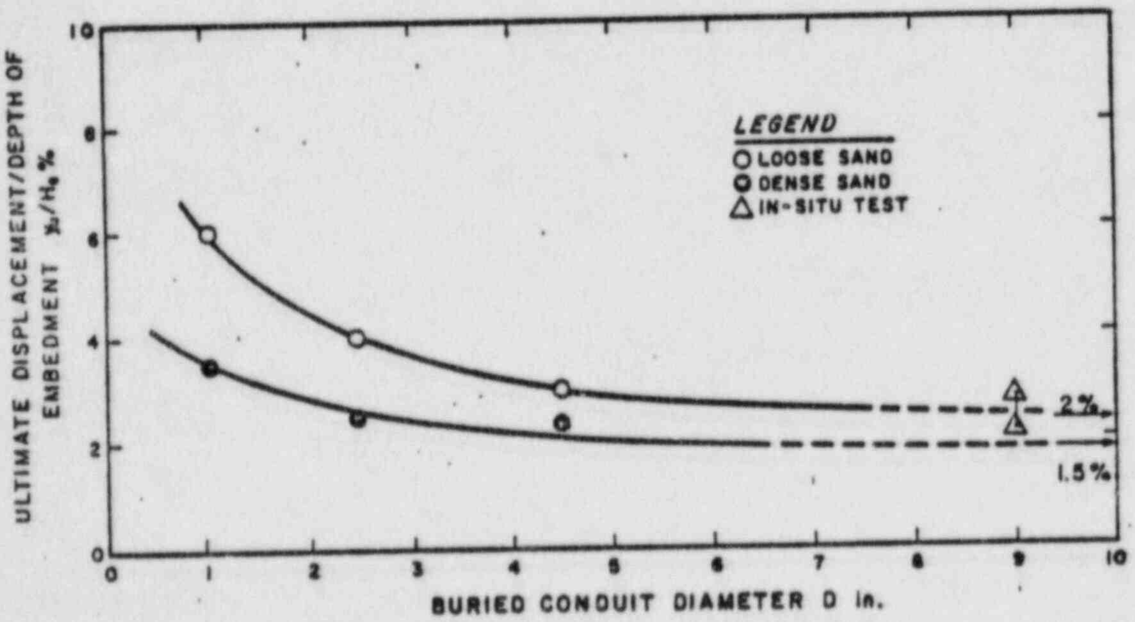
CORRECT			SKETCH NUMBER
APPROVED			
REVISIONS	②	③ ④ ⑤	



a) BEARING CAPACITY FACTOR, Nq



b) ULTIMATE DISPLACEMENT, y_u

CHECKED	TITLE FIGURE 2.5.4-63 HORIZONTAL BEARING CAPACITY FACTOR AND ULTIMATE DISPLACEMENT FOR BURIED PIPE	SCALE:
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APPROVED		SKETCH NUMBER
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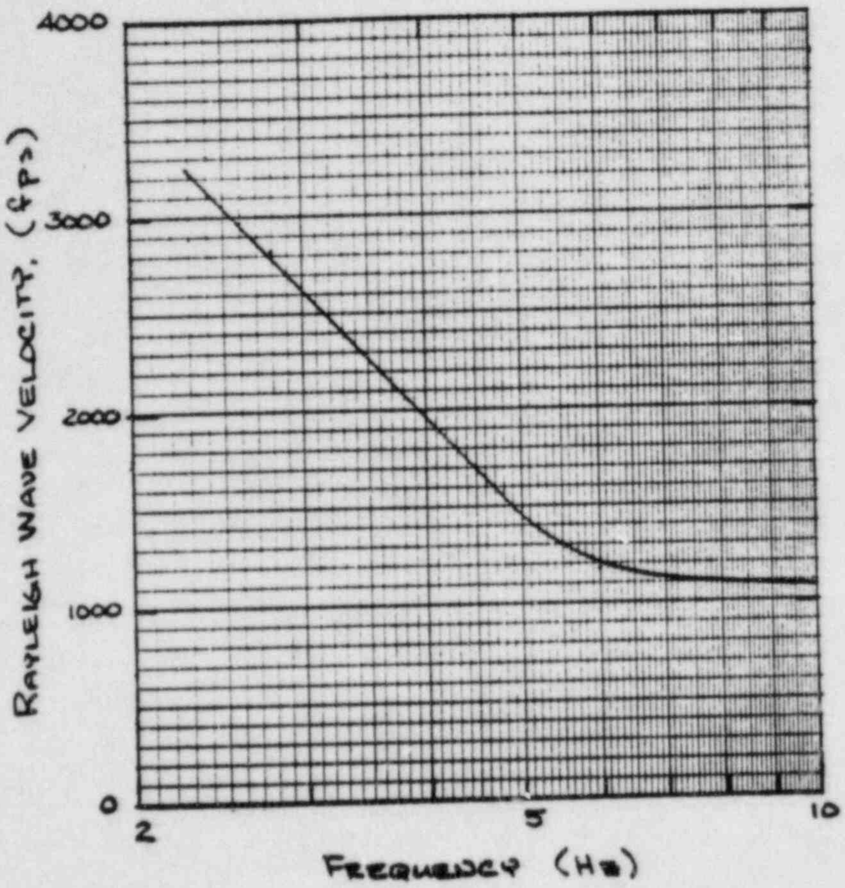

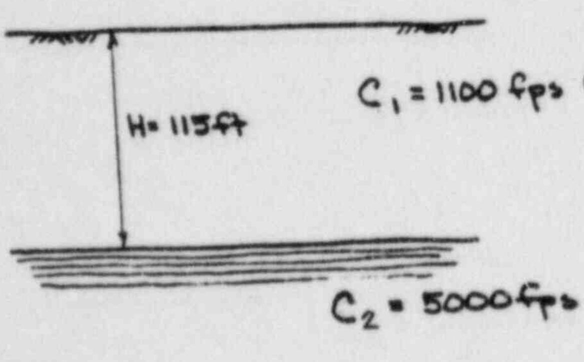

$\frac{C_2}{C_1} = 4.5$

C = shear wave velocity

	TITLE FIGURE 2.5.4-64 RAYLEIGH WAVE VELOCITY	SCALE:
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